Analysis of Physical Stability Conditions for Cover Systems with Geosynthetics for Mine Closure

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RESUMEN
This paper presents the physical stability analysis of three types of cover systems with geomembrane and geosynthetic clay liner that could be placed in the closure of tailings and mine waste storage facilities and heap leach pads. The physical stability of these cover systems was analyzed considering the reinforcement with geogrids and geocells and the effects of seismic and seepage forces. The reinforcement force was estimated considering the strain compatibility in a composite column model. The shear strength parameters of interfaces between geosynthetics and adjacent materials were estimated by large scale direct shear tests carried out at low normal stress (<50 kPa).

A parametric study was performed previously to illustrate the sensitivity of the factor of safety to the most important design parameters. Subsequently, multiple scenarios have been analyzed considering different geometric configurations, seismic and seepage buildup conditions, types of reinforcement, failure surfaces above and up the liner, etc. Then, a matrix based on this analysis is proposed. The necessary conditions for the stability of the cover systems analyzed are indicated in this matrix. This study was complemented with a probabilistic analysis based on reliability theory using the Monte Carlo simulation method to take into account the variability in the design parameters. Finally, design charts with reliability indexes are also proposed which will be useful for preliminary designs of cover systems in mining projects.

1. INTRODUCTION

There is great concern in the closure of mines due to the negative impacts generated by mining in recent years. The regulations of many countries indicate that mining owners have the obligation to plan and execute the closure activities that allow the rehabilitation of the affected areas, as well as to prevent and mitigate any negative impact on the environment. One of the most important activities is the closure of mining waste, waste rock and leached ore storage facilities. These materials represent a potential risk to the health and safety of the ecosystem and surrounding communities, mainly because they can generate acid drainage flows. For this reason, the closure of these deposits must ensure the chemical stability of these materials.

The closure of storage facilities traditionally consists of placing a cover system composed only of layers of soils (e.g., organic soil, drainage material and low permeability soil). The low permeability soil layer generates a hydraulic barrier that reduces water infiltration and prevents the generation of acid drainage. However, the performance of this layer is affected due to the desiccation and erosion that occurs in the long term that increases its hydraulic conductivity. This configuration may be inadequate or very expensive when low-permeability soils quarries are in remote areas or have reduced amounts of material and when the compaction become a difficult process due to steep or very steep slopes.

The limitations of traditional cover systems have increased the use of liner geosynthetics (e.g., geomembrane or GCL) to complement or replace the low permeability soil layer. These geosynthetics are waterproof and have good durability, thus, provide more advantages tan traditional cover systems. Although cover systems with geosynthetics liner have advantages over traditional soil cover systems, stability problems can happen such as soil cover slide down over the geosynthetic which could be caused by own weight and seismic or seepage forces. This stability problems increases the repair and maintenance costs. For this reason, the use of reinforcement geosynthetics such as geogrids and geocells is being considered more frequently in the design of cover systems.

The physical stability of cover systems is usually analyzed with analytical methods based on a deterministic approach where a safety factor is calculated. This approach can be complemented with the reliability-based design approach (RBD) that considers the influence of parameter variability and evaluates the design with a probability of failure or reliability index. These values are defined based on qualitative variables (risk, performance level, consequences of the failure, etc.) or quantitative (repair costs, failure rates, expected acceptable damage, etc.) depending on the specific conditions of each case. Application of both approaches is a more appropriate design criterion than application only of deterministic approach. In this article, both approaches were considered in the present study to propose criteria for the preliminary design of cover systems considering different heights and slopes.

2. COVER SYSTEMS

Three types of cover systems have been considered in the present study. The liner of the type I cover system (Figure 1a) is a geomembrane placed on a low permeability soil (SBP) layer. If deterioration or rupture of the geomembrane occurs, the SBP layer prevents water infiltration. This cover system has the GM-SG interface (between the geomembrane and the granular soil of the drainage layer) and the GM-SBP interface (between the geomembrane and the SBP layer), both interfaces are potential failure surfaces. The liner of the type II cover system is a GCL placed directly on the surface (Figure 1b). The bentonite in the GCL reduces water filtration due to its low hydraulic conductivity. The GCL is useful when there is not enough availability of SBP quarries and when the SBP layer is placed on slopes more steep than 1V: 2H because the compaction of the clay layer is a difficult and expensive process and requires rigorous safety standards. This cover system has the GCL-SG interface (between the GCL and the granular soil of the drainage layer) and the GCL-SR interface (between the GCL and the subgrade).

The configuration of the type III cover system is similar to type I, the only difference is that the SBP layer is replaced by GCL liner (Figure 1c). The GCL and the geomembrane constitute a composite liner, where the GCL is a secondary hydraulic barrier. This composite liner is more effective, reduces the possibility of ion exchange in the bentonite and avoids the possible increase in the hydraulic conductivity of the GCL due to the drying and cracking of the bentonite. However, this cover system is not usual in closure of mines due to the cost of both geosynthetics.

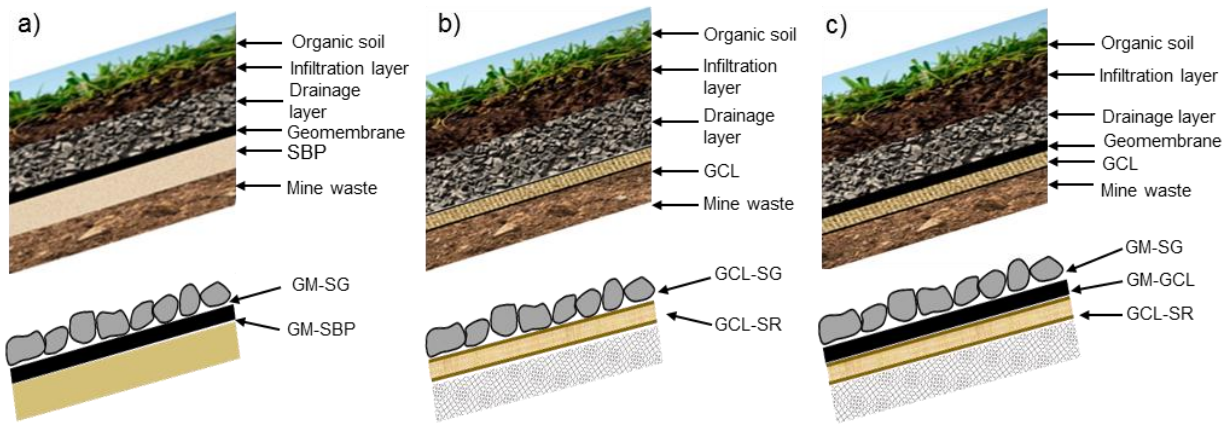

Figure 1. (a) Configuration of cover system with geomembrane; (b) configuration of cover system with GCL; (c) configuration of cover system with geomembrane and GCL

3. LARGE SCALE DIRECT SHEAR TESTS (LSDS)

The shear strength parameters of GM-SG, GM-SBP, GCL-SG and GM-GCL interfaces were estimated by large scale direct shear tests (LSDS) in accordance to ASTM D5321. The testing programme consisted of 17 tests carried out at each normal stress of 10, 30 y 50 kPa. The LSDS tests were carried out with 300x300 mm^2 constant shearing area. Table 1 summarizes the test conditions and the residual shear strengths obtained for each group of tests. LSDS tests in GCL-SR interface were not conducted because its shear strength is variable and depends of mine waste type.

Table 1. Summary of large direct shear tests

Cover system	Interfaces	Test conditions	Mean residual shear strength		Maximum residual shear strength	
Types I y III	GM-SG	Saturated	1,2	18,4	2,0	25,5
Type I	GM-SBP	Moist soil (95 % MDS)	1,9	19,3	-	33,8
Type II	GCL (woven)-SG	Saturated	9,7	30,9	10,5	36,5
Type III	GM-GCL (woven)	Moist bentonite	2,9	13,4	-	-

4. STABILITY ANALYSIS OF COVER SYSTEMS

The physical stability analysis was performed using methods based on deterministic and probabilistic design approaches. In the first case, the results of the LSDS tests were used to analyze possible failures above and below the geosynthetic liner. Previously, a parametric analysis was carried out in order to know the influence of the most important parameters on the factor of safety. After the deterministic analysis, a matrix is proposed, in this matrix stability conditions are indicated for the three cover systems evaluated. As results of the probabilistic analysis, design charts are proposed considering seepage and seismic conditions, these charts may be applicable in the preliminary design of cover systems.

4.1 Analytical Method for Stability Analysis of Cover Systems

The analytical method proposed by Koshand *et al.* (2018) was used in stability analysis. This method is based on limit equilibrium method. The free body diagram of a tapered cover system and forces involved en active and passive wedges are presented in Figures 2a y 2b. This is a more rigorous method than others traditional methods (e. g., Giroud et al., 1995; Koerner & Soong, 2005) because the equilibrium of forces is analyzed considering the simultaneous effects of seismic and seepage forces. Also, the tensile load of the geosynthetic reinforcement is estimated based on the strain compatibility between the individual soil and geosynthetic components (Figure 2c), this criterion allows a more accurate estimation of tensile loads but is not yet widely used.

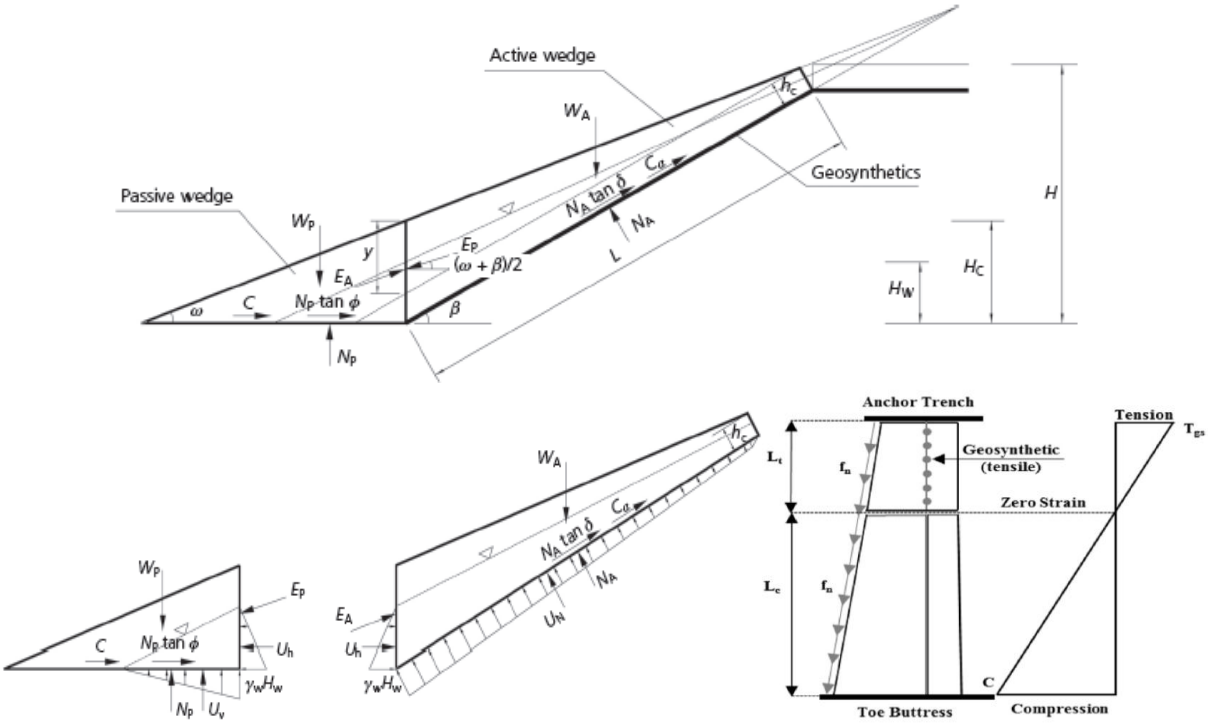

Figura 2. (a) Free body diagram of cover system with phreatic surface, seismic forces and geosynthetic tension force; (b) Forces involved in wedges; (c) Distribution of axial loads and displacements along composite column model.

The definition of each parameter is:

δ	: Interface friction angle (°).
β, ω	: Soil slope angle beneath the geomembrane and final cover soil slope angle (°).
C_α, C_p	: Total interface adhesion and total cohesion of the cover soil (N/m).
E_A, E_P	: Resultant forces from passive and active wedges (N/m).
h_c,h_w	: Thickness of cover soil and thickness of saturated cover soil at crest of the slope (m).
H_w, H_c	: Vertical height of the cover soil and vertical height of the free water surface measured from the toe (m).
k_h, k_v	: Horizontal and vertical seismic coefficients.
L	: Length of the cover system (m).
N_A, N_P	: Effective forces normal to the failure plane of the active and pasive wedges (N/m).
T_{gs}	: Geosynthetic tension force (N/m).
U_h, U_n	: Resultants of the pore pressures acting on the interwedge surfaces and perpendicular to the slope (N/m).
U_v	: Resultant of the vertical pore pressures acting on the passive wedge (N/m).
W_P, W_A	: Total weight of the pasive and active wedges (N/m).
γ_d, γ_{sat}, γ_w	: Unit weight and saturated unit weight of the cover soil and unit weight of water (N/m^3).

The FS value is obtained from the solution of the quadratic equation:

$$\mathrm{FS} = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} \quad [1]$$

where

$$a = AH + CF \quad [2]$$

$$b = FD - (BH + AG + EC) \quad [3]$$

$$c = BG - ED \quad [4]$$

$$A = W_A + k_v W_A - \left((U_h - K_h W_A)\text{sen}\beta + (1 + k_v)W_A \cos\beta\right)\cos\beta - \frac{F\text{sen}\beta}{1+\sqrt{K_C/K_T}} \quad [5]$$

$$B = \left[1 - \frac{1}{1+\sqrt{K_C/K_T}}\right]\text{sen}\beta[C_\alpha + \left((U_h - K_h W_A)\text{sen}\beta - U_N + (1 + k_v)W_A \cos\beta\right)\tan\delta] \quad [6]$$

$$C = \text{sen}\left(\frac{\beta+\omega}{2}\right) + \text{sen}\left(\frac{\beta-\omega}{2}\right)\cos\beta \quad [7]$$

$$D = \left[1 - \frac{1}{1+\sqrt{K_C/K_T}}\right]\left[\text{sen}\beta \tan\delta \,\text{sen}\left(\frac{\beta-\omega}{2}\right)\right] \quad [8]$$

$$E = C_p + (W_P + K_v W_P - U_v)\tan\phi \quad [9]$$

$$F = K_h W_P + U_h \quad [10]$$

$$G = \text{sen}\left(\frac{\beta+\omega}{2}\right)\tan\phi \quad [11]$$

$$H = \cos\left(\frac{\beta+\omega}{2}\right) \quad [12]$$

4.2 Parametric Analysis

The procedure consists in carrying out multiple stability analyzes considering different values within the typical range of the parameter that will be evaluated, while the values of the other parameters remain constant. Previously, a base case has been defined. The slopes considered are typical slopes in tailings and mine waste storage facilities. The ranges of values for the interface shear strength parameters were defined considering the maximum and minimum values obtained in the LSDS tests. Table 2 presents the values considered for all design parameters.

Tabla 2. Range of values and base values for the design parameters in the analysis

Parameters	Symbol	Units	Typical range	Base value
Interface adhesion	α	kPa	0 - 10	2,0
Interface friction angle	δ	(°)	5 - 40	24
Unit weight of the cover soil	γ_d	kN/m^3	15 - 22	18
Saturated unit weight of the cover soil	γ_{sat}	kN/m^3	16 - 23	20
Internal cohesion of the cover soil	c_p	kPa	-	-
Friction angle of the cover soil	φ	(°)	20 - 34	30
Thickness of the cover soil	h_C	m	0,3 - 1,5	0,6
Height of the slope	H	m	5 - 50	10
Slope	$H:V$	-	1,5H:1V - 3,5H:1V	2,5H:1V
Parallel submergence ratio	PSR	-	0,0 - 1,0	0,5
Horizontal seismic coefficient	k_h	-	0 - 0,25	0,1
Stiffness of geogrid	K_T	kN/m	0 - 3000	-
Stiffness of cover soil	K_C	kN/m	100 - 2500	-
Geosynthetic tension force	T_{gs}	kN/m	0 - 150	-

The results obtained in the parametric analysis have been plotted in trend curves of the safety factor (FS) versus the variable analyzed in each case. The influence of the seismic coefficient (k_h) and the hydraulic load level (h/h_w) in FS can be seen in Figures 3 and 4, respectively. The graphs show that the increase in k_h causes a significant decrease in FS, due to the fact that the components of the seismic load increase the resulting sliding force and reduce the effective forces normal (N_A, N_P). Also, it has been verified that the reduction of FS is more significant in extended slopes than in steep slopes, this reduction may be greater than 50% of the factor of safety calculated in static condition (k_h=0).

The seepage buildup also significantly reduces FS because it reduces effective stresses and shear strength of the interface. An increase of 0.1 in h/h_w causes a reduction between 0.04 and 0.09 in the value of FS, this reduction is significant in extended slopes because the pore pressure will be reduced as the inclination of the slope increase.

The apparent adhesion (α) and the friction angle (δ) are the main factors that contribute to the physical stability of the cover system because they significantly increase the value of FS (see Figures 5 and 6). The increase of 1º in δ and 1 kPa in α can increase the value of FS in a range between 0.02 to 0.10 and 0.15 to 0.34, respectively. This increase in FS is less significant in steep slopes because the increase in inclination of the surface causes the reduction of σ_n and therefore also the reduction of the shear resistance at the interface. According to the results obtained, if the parameters of shear strength are not estimated correctly, the value of FS could be significantly underestimated or overestimated. This aspect highlights the importance of conducting LSDS tests in order to have a reliable estimate of the stability of the cover systems.

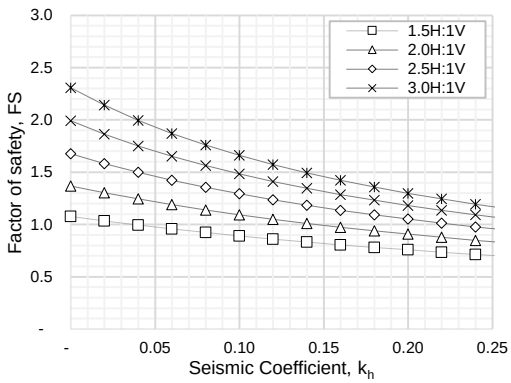

Figure 3. Relationships between k_h and FS

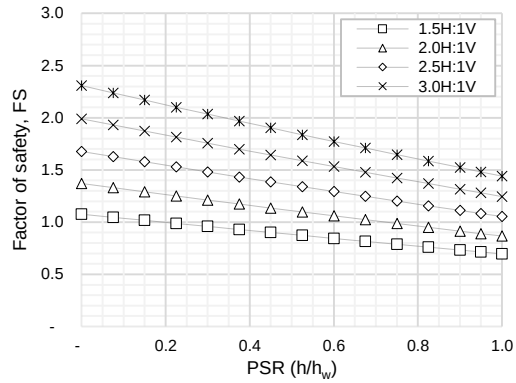

Figure 4. Relationships between PSR (h/h_w) and FS

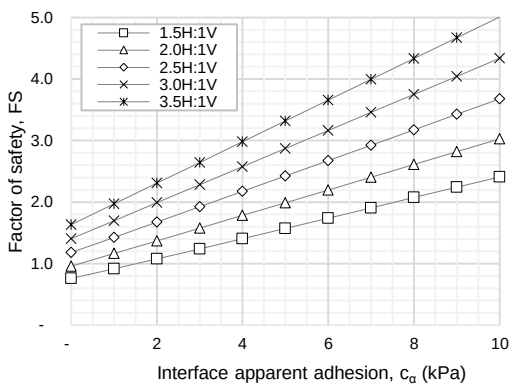

Figure 5. Relationships between c_α en FS.

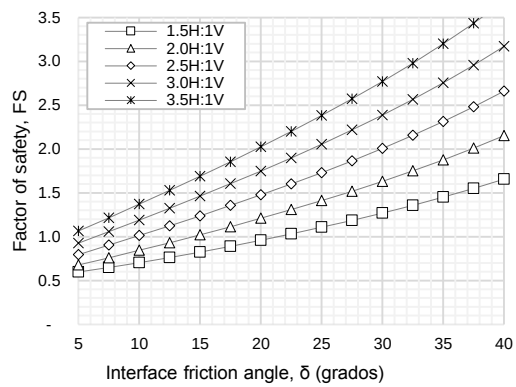

Figure 6. Relationships between δ and FS.

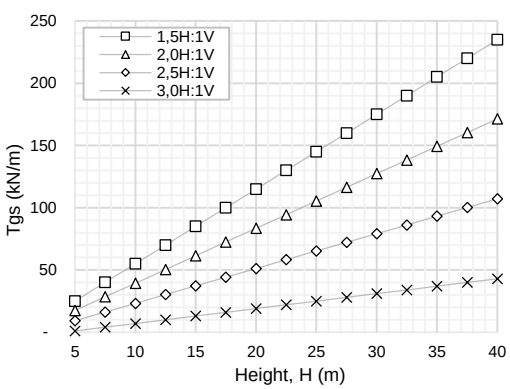

Figure 7. Reinforcement tension necessary for $FS = 1{,}5$ considering different heights and slopes

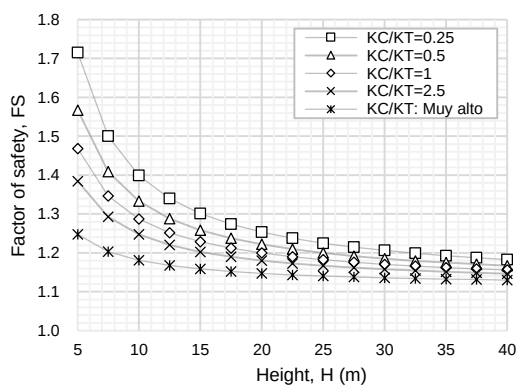

Figure 8. Relationships between K_C/K_T and FS considering different heights.

The tensile load in geosynthetics contributes to the physical stability of the cover system because sliding forces are counteracted. The reinforcement evaluation is usually performed estimating the minimum value of T_{gs} necessary for the required FS. Based on this criterion, Figure 7 shows the value of T_{gs} needed to achieve an FS equal to 1.5 for scenarios with different heights and slopes. However, this criterion is not sufficient in the evaluation of the reinforcement, it is also necessary to assess whether the necessary tension occurs in the geogrid, because the behavior of this material is passive. The tension that will occur in the geosynthetic depends on the compressive stiffness of the soil (K_C) and the tensile stiffness of the geosynthetic (K_T). The Figure 8 shows the influence of the K_C/K_T ratio in FS. It has been verified that the increase in FS begins to be significant when K_C/K_T < 1,0. In order to ensure that the reinforcement will be effective it is recommended that the tension length in the cover is greater than 75% of the total length, which occurs when K_C/K_T < 0,1.

4.3 Stability analysis of cover systems

The deterministic analysis has been carried out considering multiple scenarios where the failure can occur above or below the geosynthetic liner. In these scenarios, different service conditions, heights, slopes and types of reinforcement (with geogrid and geocell) were analyzed. Figure 9 summarizes all the scenarios that were evaluated. The objective of considering as many scenarios as possible is to identify the stability conditions for the three cover systems analyzed, in which the minimum safety factors in the analyzed service conditions are met.

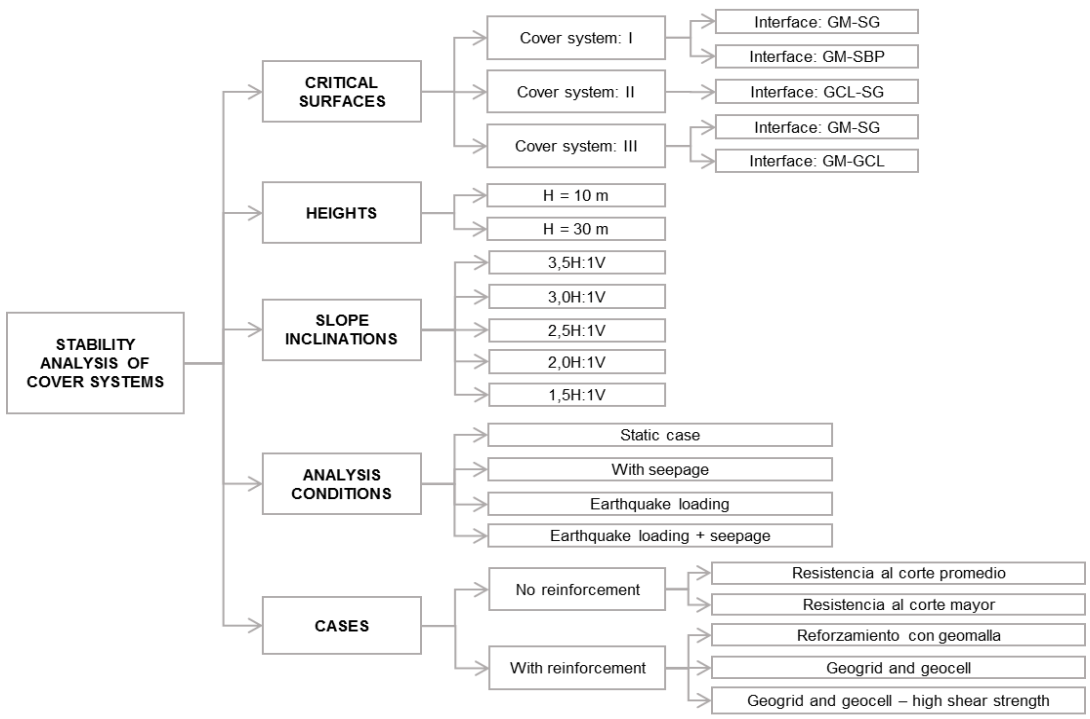

Figure 9. Scenarios considered in the stability analysis of cover systems with geomembrane and GCL

The stability analysis has been carried out considering the cases: static, with seepage buildup, with earthquake loading and with earthquake and seepage buildup. The static case is the most likely condition during the life of the cover system, in which the slip is generated mainly by its own weight. On the other hand, cases with seepage or seismic loadings have a shorter duration and probability of occurrence, however, they significantly affect the stability of the cover system. The factors of safety obtained in this analysis have been contrasted with the required safety factors indicated in Table 5.
The analysis has been conducted in both cover systems with and without reinforcement. In cover systems without reinforcement, the mean and maximum shear strength parameters obtained of LSDS tests were considered (see Table 3). The residual shear strength were considered in this analysis because cover systems are permanents structures and different mechanisms to both short or long term could mobilize the post peak shear strength in the interface.

On the other hand, reinforced cover systems with geogrids and/or geocells also were analyzed in order to identify cases in which its application is necessary. Three different cases were considered. The first case considers the use of geogrids, in which the cover system has both tension and compression loads. The second case consists in adding geocells to the geogrid so that the axial behavior is mainly in tension. The mean shear strength parameters of the interfaces were considered in both cases. The third case is similar to the second one, the only difference between them is that the maximum

shear strength parameters have been considered. Both second and third analysis cases were conducted considering a K_C/K_T ratio of 0,10 so that the tension length is greater than 75 % of the total length. This criterion is conservative because this ratio could be less, therefore, the reinforcement tension will be greater than the estimated in this analysis.

Table 3. Analysis cases, range of values y required factors of safety.

Analysis cases	Range of values	$FS_{mín}$
Static case (to long term)	-	1,5
Seepage buildup, during a rain (short duration)	h_w/h = 0,0 a 1,0	1,3
Pseudoestatic, during a seismic event (very short duration)	k_h = 0,0 a 0,3	1,1
Pseudoestatic + seepage buildup (most critical case)	h_w/h = 0,5 y k_h = 0,1	1,0

A matrix based on the results of the deterministic analysis is proposed in Table 4. The necessary conditions for the physical stability of the three cover systems are indicated in this matrix. The required factors of safety are met in this conditions. The recommendations are based on the interface shear strength and types of reinforcement necessary to ensure stability of these cover systems. The purpose of the matrix proposed in this study is to be a useful guide for the preliminary design of these components. The recommendations in this matrix are not only applicable for cover systems with the thickness (0.60 m) and slope heights (10 and 30 m) considered in this study, they are also useful for cover systems of smaller thickness or lower height, In these cases the recommendations indicated are conservative.

Table 4. Matrix with recommended stability conditions for cover systems with geomembrane and GCL.

Slope	GM-SG interface (above liner). Cover system: type I y III	GM-SBP interface (below liner). Cover systems: types I	GCL-SG interface (above liner). Cover system: type II	GM-GCL interface (below liner). Cover system: type III
1,5H:1V 2,0H:1V	The application of geogrids and geocells is recommended and the critical interface has a high cut resistance at heights of 10 m. At heights of 30 m, the geogrid must have a high resistance	The cover system is generally unstable. The anchor must be with concrete blocks in order to prevent a failure below the geomembrane.	The failure will not occur at the interface with the geosynthetic. An internal fault in the ground cover is more likely, therefore, the application of geogrids and geocells is also recommended.	The cover system is generally unstable. The anchor must be with concrete blocks in order to prevent a failure below the geomembrane.
2,5H:1V 3,0H:1V	If the interface shear strength is high, reinforcement is not necessary even under severe service conditions (p. eg., k_h<0,15 o PSR<1,0). If the shear strength is similar to the mean obtained in this study, geogrids should be use on heights of 10 m and geocells with geogrids on heights less than 30 m.	The cover system is stable even without reinforcement, therefore, the failure will not ocurr on this interface. In cover systems of 10m high, the geomembrane can be anchored through burial in a trench, but in this case a likely failure in the interface GM-SR must also be analyzed.	Reinforcement is not necessary due the high interface shear strength. The geomembrane can be anchored through burial in a trench. but in this case a likely failure in the interface GCL-SR (woven side) must also be analyzed. .	The cover system without reinforcement is stable when k_h<0,10. If k_h>0,10, the use of geomembranes and geogrids is necessary at heights of 30 m, but only geogrids are necessary at heights of 10 m. The anchor must be with concrete blocks in both cases.
3,5H:1V	If the interface has an average shear resistance, the cover system is stable when k_h<0,15 o PSR<0,5. But, if these conditions are more severe, the application of geogrids or high shear resistance at the interface should be considered.			Reinforcement is not necessary. The failure will not ocurr in this interface. In cover systems of 10m high, the geomembrane can be anchored through burial in a trench, but in this case a likely failure in the interface GM-SR must also be analyzed.

4.4 Probabilistic Analysis

The probabilistic analysis methods based on the theory of reliability evaluate the performance of a structure with probability of failure (Pf) or the reliability index (β_N), which are obtained by simulations or by applying numerical methods, both based on a method analytical (Phoon, 2016). The probabilistic method applied in the present study was the Monte Carlo simulation method (MSMC), the flow chart of this method is detailed in Figure 10. This method considers the variability of the parameters, which are considered as random variables wich have a characteristic probability distribution. The parameters considered in this analysis were the properties of the cover soil, interface shear strength, the thickness and height of the cover soil. The average and the coefficient of variation considered for these parameters are presented in Table 5.

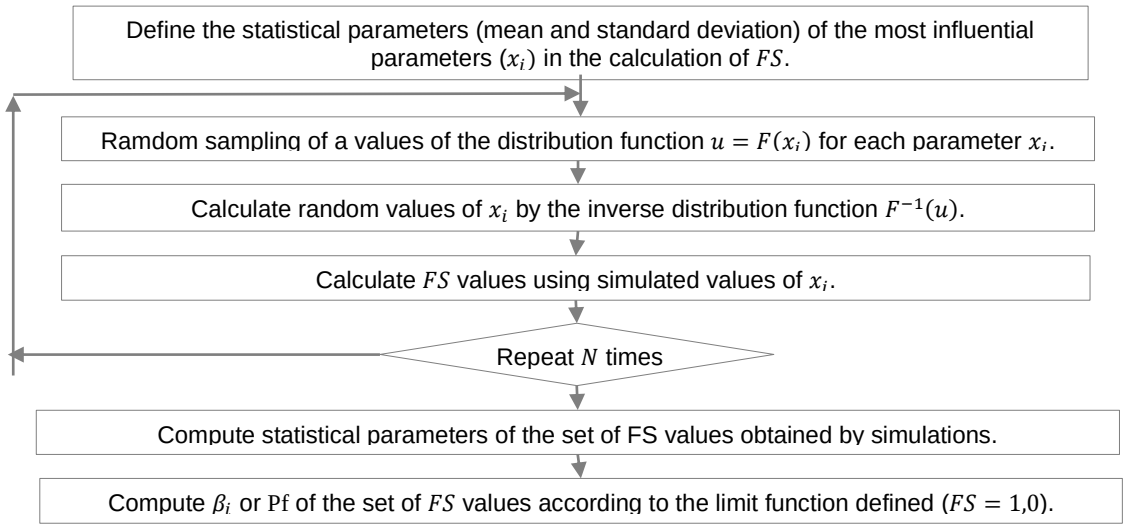

Figure 10. Flow chart of Monte Carlo simulation method for the evaluation of FS.

Table 5. Statistical parameters considered in the probabilistic analysis.

Random variable	δ (º)	γ_d (kN/m3)	γ_{sat} (kN/m3)	φ (º)	h_C (m)	H (m)
Mean value (μ)	< 40	18	20	30	0,6	30
Coefficient of variation (%)	10	5	5	10	5	5

The main purpose of performing a probabilistic analysis has been to propose design charts with reliability indexes (β_N), this parameter has an easy to interpret measurement scale and allows a better analysis of the probability distribution of FS with respect to the limit state function defined (FS = 1.0). The proposed design charts are for cover systems both with and without reinforcement, the values of β_N are associated with a friction angle, a slope and conditions with earthquake or seepage loadings. The target values of β_N considered in the analysis were: 3.0, 2.0, 1.0 and 0.5, associated with average failure probabilities of 0.03%, 1.6%, 15% and 30%, respectively.

In cover systems without reinforcement, the design charts with reliability indexes (β_N) for conditions with both seismic and seepage loadings are presented in Figures 11 (a) y (b), respectively. In the trend curves, the difference in the required friction angle (δ) to achieve a specific β_N between cover systems at heights of 10 and 30 m is not significant, this difference is only 1º in most cases. On the other hand, it has been verified that the reduction in design reliability, measured in terms of β_N, is more susceptible to the variation of δ when β_N < 1.0.

In cover systems reinforced with geogrids and geocells, the design charts with reliability indexes (β_N) for heights of 10 y 30 m are presented in the Figures 12 (a) and (b) for condition with seismic loadings and in the Figures 13 (a) and (b) for condition with seepage loadings. In the design charts presented, the difference in the friction angle required to achieve a specific β_N for cover systems in heights of 10 and 30 m, is approximately 4º and 8º for slopes of 3.5H:1V and 1.5H:1V, respectively, which are greater than those obtained for cover systems without reinforcement. These results also verify that by reinforcement with geogrids and geocells the friction angles required to achieve a specific β_N value have been reduced, which in this case are 2º to 9º less than those required in cover systems without reinforcement.

The RBD approach also allows a logical and justified selection of the acceptable FS. For this reason, the relationship between FS and β_N has also been analyzed in the proposed design charts. From this, it has been verified that a specific value of β_N can be associated with different safety factors, which reinforces the criterion that the definition of the acceptable FS value for the design must be specific for each case of analysis. In the conditions of service with earthquake and filtration, it is verified that for the same β_N the variation of FS as a function of δ is insignificant when β_N is equal to 0.5 and 1.0, in these cases FS tends to be equal or greater to 1.05 and 1.10, respectively. On the other hand, this variation is more significant when β_N> 2.0, where the trend is linear and FS is in the range of 1.2 to 1.4; if β_N> 3.0 the trend is practically exponential and it is verified that FS> 1.35. Therefore, if in the cover systems design the acceptable FS value considered in the design is equal to 1.1 in seismic condition and 1.3 in filtration condition, these values would be associated with reliability indices of 1.0 (Pf≅ 15%) and 2.0 (Pf≅ 1.6%), respectively. It is important to indicate that under critical stability conditions where the slope slope is more steeped, higher FS values are required to met a specific target β_N o P_f .

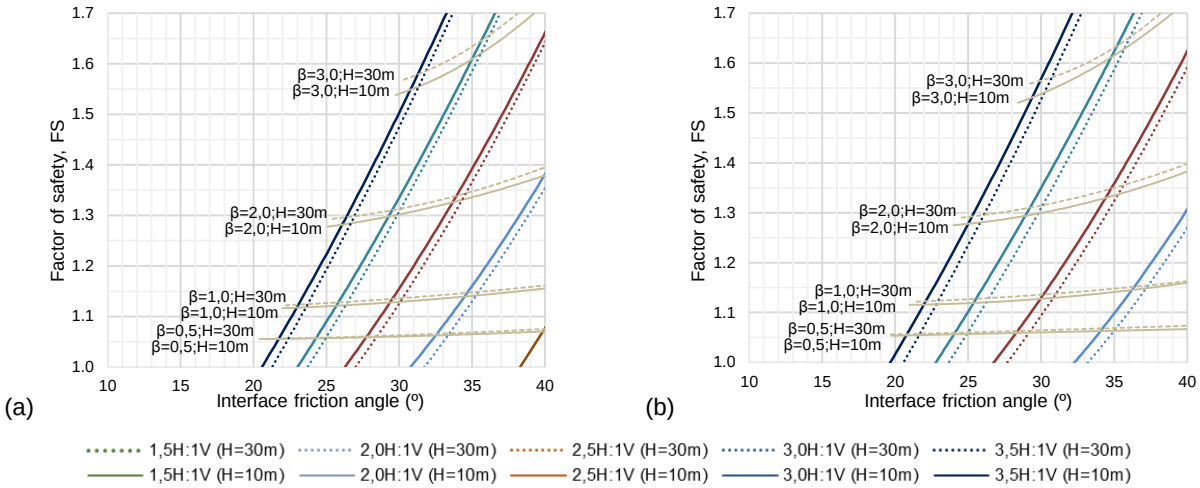

Figure 11. Design charts with reliability indexes for cover systems without reinforcement in conditions (a) with seismic loadings and (b) with seepage loadings.

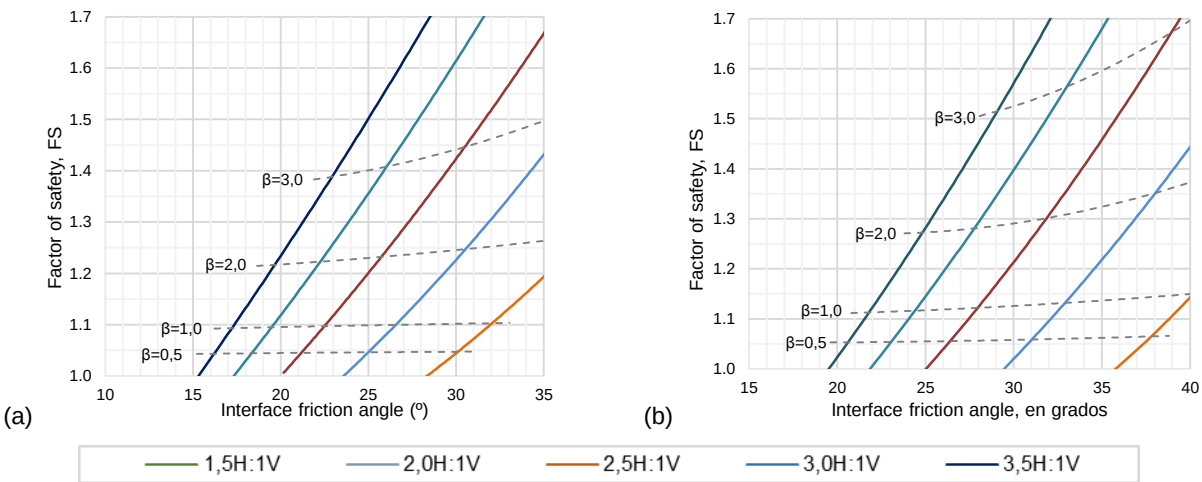

Figure 12. Design charts with reliability indexes for cover systems reinforced with geocells and geogrids in conditions with seismic loadings for (a) H=10 m y (b) H=30 m.

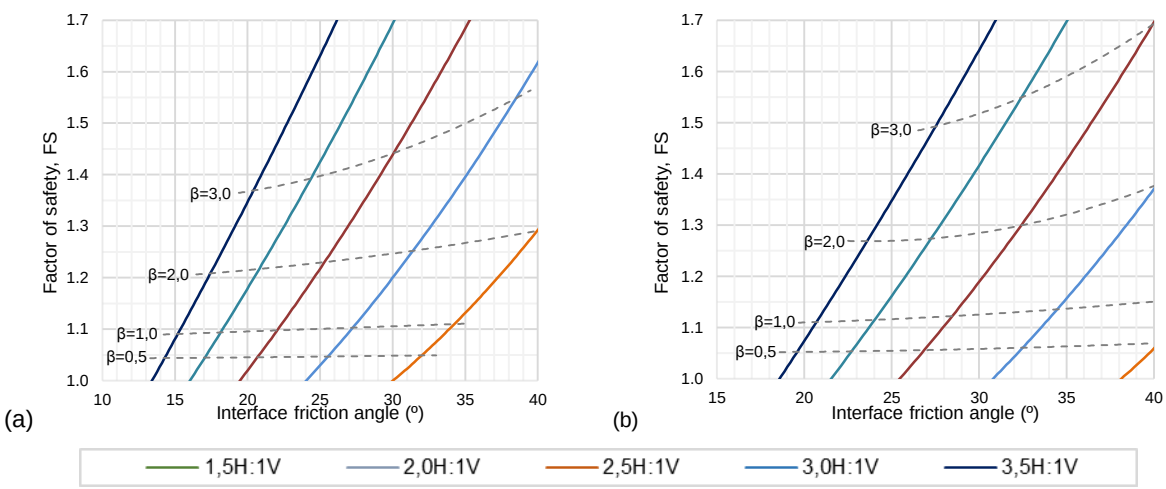

Figure 13. Design charts with reliability indexes for cover systems reinforced with geocells and geogrids in conditions with seepage loadings for (a) H=10 m y (b) H=30 m.

5. CONCLUSIONS

The analysis method carried out in the present study is more rigorous than other traditional stability analyzes that do not take into account the compatibility of axial deformations in tension elements (geosynthetics) and compression elements (ground) in the cover system. By incorporating this criterion in the stability analysis, it has been verified that the tension developed in the geosynthetics could be insignificant or, on the contrary, could exceed the ultimate tension of this material. Therefore, the design of coverage should not be carried out with methods of analysis that do not consider this criterion because there would be uncertainty as to whether the design is unsafe or conservative.

The parameters with the greatest influence on the stability of the cover system are the slope, the seismic and seepage forces. The reduction of FS by the increase in inclination is variable depending on other parameters, this reduction is in the range of 0.1 to 0.33. In seismic condition, the reduction of FS may be greater than 50% of the value calculated in static condition. In condition with seepage loadings, an increase of 0.1 in PSR can reduce 0.1 the value of FS.

According to the stability conditions indicated in the matrix proposed in this study, the use of geogrids and geocells is necessary in cover systems on slopes more steep than 2.0: 1V, even in coverage with GCL. In this case, although the GCL-SG interface has the highest shear strength compared to other interfaces, failure could occur inside the cover soil. Therefore, the proposed matrix also provides different criteria for the reinforcement of cover systems in slopes of different inclinations. Likewise, in this matrix, failure surfaces under the liner are also considered, this analysis is often not evaluated in the design of covers, the type of anchor applicable in these cases is also indicated. These recommendations are expected to be considered in future cover systems designs with geosynthetics in mining projects.

Reliability-based design should be used in the analysis of geotechnical structures. Its use allows the minimum safety factor to be justifiably defined through a reliability index that considers the variability and uncertainty inherent in the analysis parameters. This safety factor must be defined or verified in each case study because in some cases even an FS of 1.5 could lead to an insecure design. The present study is a step so that the design based on the reliability is a more common practice in the analysis of stability of geotechnical structures for its importance and the advantages that it offers.

6. REFERENCES

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