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Cost analysis of block failure stabilization in heap leach pads

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Abstract

In designing heap leach pads it is very common to use liners composed of a geomembrane over a low permeability soil or soil liner. A granular overliner is used on top of the geomembrane in order to prevent damage from the impact of the oversize ore. In general, the upper interface, geomembrane versus overliner or ore, will usually provide higher shear strength than the clayey soil in contact with a textured geomembrane used as a soil liner due to its granular nature. Therefore, in order to prevent any heap leach pad instability at the interface, the shear strength of the lower interface should be improved, or the block failure surface may be modified by cuts or fills.

The research performed by Ayala et al. (2014) is updated by including more results of large direct shear tests of interfaces composed of textured LLDPE geomembrane and soil liner. The curves related to the variation of the interface shear strength due to geomembrane asperity height, soil liner classification, and normal stress are used as a way to perform a comparative analysis of the stability of heap leach pads in design or in construction stages.

The stabilization measurements of two heap leach pads were studied and contrasted in order to find which was the most effective action for maintaining an acceptable static factor safety for the block failure stability analysis. This included an assessment related to the best cost-effective alternative for stabilization between geometry modifications and interface improvements.

2-D limit equilibrium method slope stability analysis was employed in both cases for assessing different stabilization solutions on heap leach pad block failure slope stability; a cost estimate of each action proposed was obtained and compared to the other. The criteria for the stabilization works were related to the following: grading geometry modifications (without including heap geometry, thus conservation of the heap capacity); interface shear strength enhancement based on the updated information; and the use of both options. Key design issues related to the cost benefits of stabilization work are recommended when low factors of safety for block failure stability are encountered during a heap leach pad project.

Introduction

During the design of a heap leach pad in the mining industry it is very common to use liners composed of a geomembrane over a low permeability soil or soil liner, as a way to prevent or decrease leakage of the pregnant solution into the heap foundation, which may cause environmental damage and economical losses.

It is usual in practice for heap leach pad projects in Peru to use mainly LLDPE textured geomembrane over a clayey soil as a liner system. In Peruvian mining projects the construction of such systems has followed the GRI recommendations provided on the GM12 (2014) and GM17 (2015) specifications. Recently, however, the change to the GM17 has led to a change in the previous common design, since the update has increased the minimum asperity height for texture LLDPE geomembrane from 0.25 to 0.40 mm (10 to 16 mils).

During the geotechnical design and analysis of a heap leach pad, the main restriction is related to the liner system, which usually provides an interface with low shear strength due to the relatively high fines content of the soil liner. Hence this interface controls the stability conditions of such facilities in the event of a block failure. Some efforts have been made to model or predict the shear strength of this kind of liner (Reddy and Butul, 1999; Ivy, 2003; Yesiller, 2005; Blond and Elie, 2006).

As noted by Ayala and Huallanca (2014) the interface shear strength increases with the increment of the geomembrane asperity height, normal stress increment (σ'_N) and the increment of granular material in the soil liner. Another important aspect is that the normalization of interface shear strength behavior results in its decay as much more than normal stress is applied, which requires an extra effort to predict the interface shear strength values for very high heap leach pads.

This research updates the relationship described by Ayala et al. (2014), and focuses on the implications for block failure stability by changing the interface parameters, performing geometrical modifications, or by doing both, for two practical cases. The first one is about to close operations at the central southern Peruvian Andes, and the second one is an ongoing project at northern Peruvian Andes. The costs implications of stabilization measures when dealing with low factors of safety are also discussed.

Data preparation

The interface shear strength has a clear nonlinear behavior as discussed by Stark et al. (1996), Stark and Choi (2004), Parra et al. (2011), Ayala et al. (2014) and Ayala and Huallanca (2014). That is why this research has represented the shear strength as a function of the normal stress instead of using the Mohr-Coulomb approach. Ayala et al. (2014) reviewed a total of 190 large-scale direct shear (LSDS) tests, and finally used 82 to derive their correlation (the discarded tests lacked soil classification or asperity height measurement). In this revision, 18 tests have been added to update the correlation. The tests used in this update provided the following information: type of geomembrane (LLDPE or HDPE, most of the data corresponds to LLDPE geomembrane), nominal geomembrane thickness (1.5 mm or 2 mm), asperity height measured on laboratory, soil liner classification, Atterberg limits, peak shear stress (taken as 2.5 cm of deformation), residual shear stress (taken at 7 cm of deformation) and their corresponding normal stress.

The residual shear strength of the former (black dots) and new tests (red dots) is shown in Figure 1. It should be noted that some tests were performed for only 3 normal stresses or other normal stresses different from the ones shown in Figure 1 (50 kPa or 600 kPa as an example). The shear strength for each soil sample has been separated by typical normal stress values applied during the LSDS test (100, 200, 400 and 800 kPa). No differentiation of soil classification or fines content is presented.

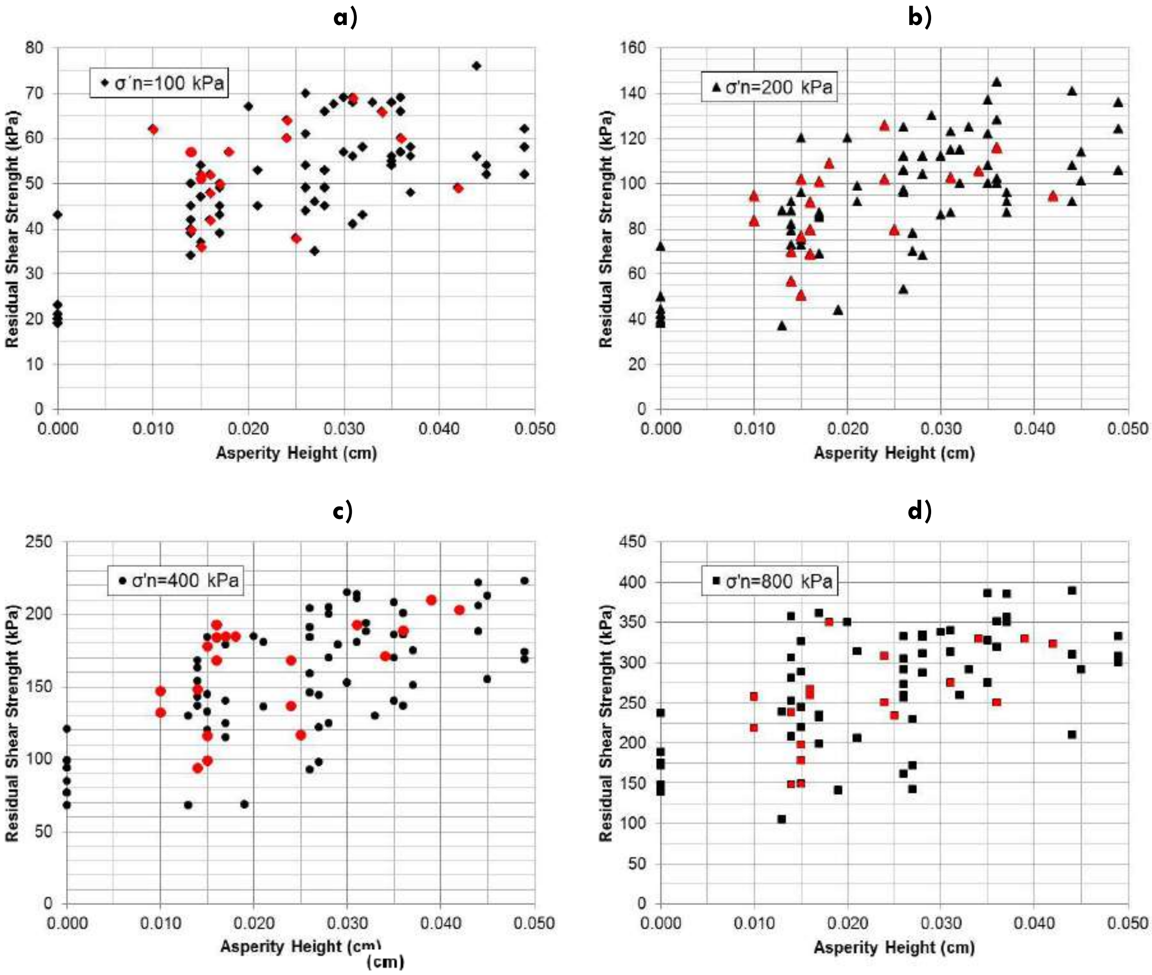

Figure 1: Residual shear strength behavior of all the testing data for different normal stresses

The peak shear strength was not taken into account in this research due to the following reasons: it is not commonly used in practice since in most high leach pads an interphase deformation larger than 2.5 cm is expected; the tendency is not as clear as in residual shear strength; and the displacement value used to calculate the peak shear strength is not always 2.5 cm but depends on the soil and geomembrane features.

As shown in Figure 1, the interface residual shear strength has a clearer tendency to increase with the increment of the asperity height. However, some scattering is observed, which may be caused by the soil properties or fines content. The samples were separated based on their classification according to the Unified System of Soil Classification (USSC) and fines content. For the purpose of this research only the updated average of the soils used are summarized in Table 1.

Table 1: Soil classification summary

Soil type	Gravel content (%)	Sand content (%)	Fines content (%)	Liquid limit (%)	Plastic Index (%)
Average GC	42.2	27.1	30.7	34.3	14.0
Average SC	23.3	37.8	38.6	33.8	14.6
Average CL and CH with fines content below 65%	11.7	33.4	54.9	37.3	19.0
Average CL & CH with fines content above 65%	3.5	11.9	84.7	58.7	32.2

There were other features that may impact interface shear strength behavior, such as the type of geomembrane (LLDPE or HDPE), thickness of the geomembrane, and geomembrane asperity distribution. However, no clear tendency was observed when the data was classified by such parameters, and there was not enough data that focuses on them to make a clear correlation – primarily the one based on the asperity distribution.

There were a total of 26 samples classified as GC (clayey gravel), 30 samples as SC (clayey sand), 24 samples as CL (low compressibility clay) and CH (high compressibility clay) with fines content (FC) below 65%, 10 samples as CL (low compressibility clay), CH (high compressibility clay) with fines content (FC) above 65%. One sample was added for the soils ML (silt with sand) and MH (high plasticity silt with sand (MH) with fines content below 75%, making a total of 10 samples for these soils. However, they were not used in this research since they are not recommended to be used as soil liner in heap leach pads.

Interphase shear strength updated correlation

Based on the new set of information, the data of shear strength based on different soil liner classifications, geomembrane asperity heights and normal stresses, a new correlation was developed as shown in Figure 2.

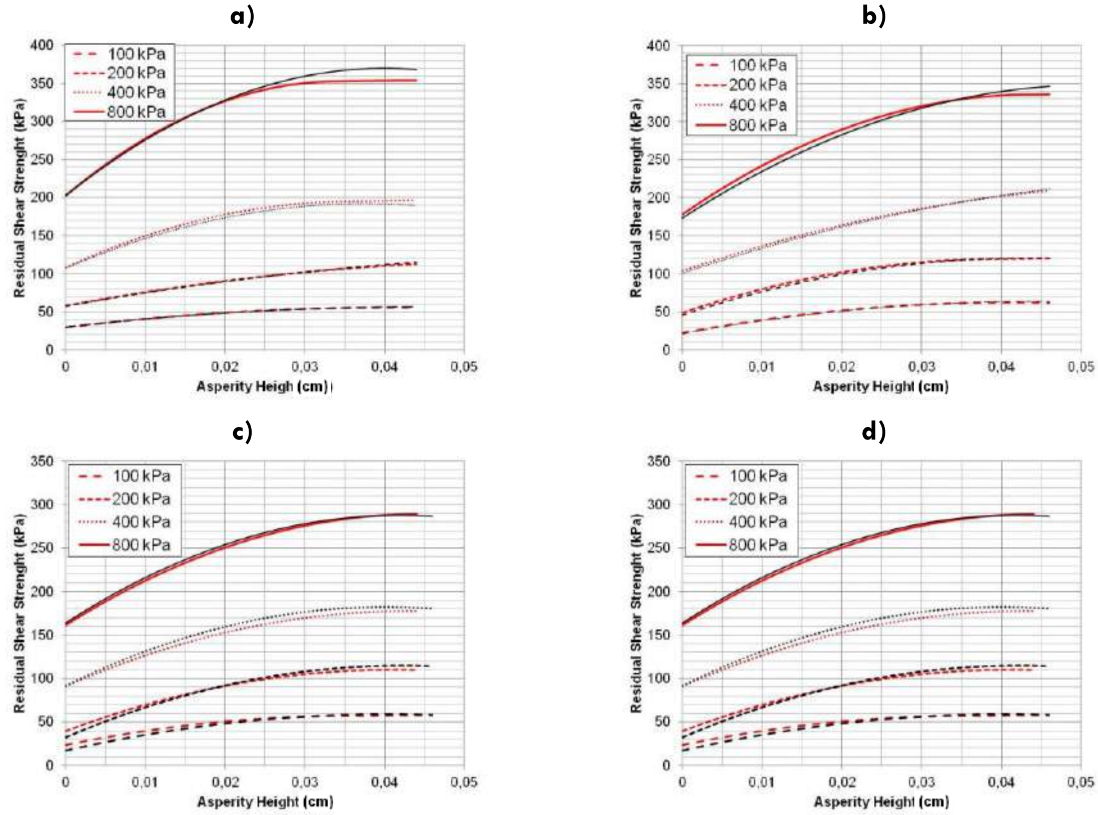

Figure 2: Summary of the previous (black lines) and updated (red lines) correlations for interface shear strength tendencies for (a) GC, (b) SC, (c) CL & CH with FC ≤ 65% and (d) CL & CH with FC > 65%

The updated curves (red lines in Figure 2) show that the previous correlations obtained by Ayala et al. (2014) have not been altered at all and are even improved by reducing the scattering, thus the conclusions presented by Ayala et al. (2014) are supported. The updated correlations show a reliable nonlinear tendency of shear strength increment for all kinds of soil, in which the behavior tends to be asymptotic at asperity heights of 0.04 cm, which agrees with the conclusion provided by Blond and Elie (2006) for low normal stresses (up to 140 kPa). These conclusions agree with the latest modification of the GM17 (2015), in the sense that the asperity height of 0.04 cm is set as the lowest to be used, since the interface shear strength below that value decreases dramatically, as shown by Ayala et al. (2014).

Normalized shear strength data processing based on soil classification

By using the red curves shown in Figure 2, shear strength normalization analysis performed by Ayala and Huallanca (2014) was updated. These results are shown in Figure 3.

In Figure 3, the rate of increment of the interface shear strength based on the increment of the normal stress is decreasing; this is related to the nonlinear behavior of the interface shear strength, which is similar to typical granular soil behavior. Therefore, a low rate of increment of interface shear strength at high normal stresses (above 800 kPa) is expected. Furthermore, information can be obtained about the interface shear strength by extrapolating the normal stresses of the curves in Figure 2, by controlling the rate of shear and normal stress. The criteria for shear strength extrapolation is taken from Parra et al. (2012), where a control point in function of an expected normal stress to be experienced in heap leach pads is chosen.

Ayala and Huallanca (2014) developed a methodology to predict the interface shear strength for high normal stresses based on a small number of large normal stress LSDS tests and the current tendency suggested in Figure 3. An example of this prediction is shown in Figure 4, in which an extrapolation of the shear strength has been done for two kinds of interfaces based on the criteria explained in Parra et al. (2012) and Ayala and Huallanca (2014). Those calculations are compared to real LSDS tests that reached a normal stress of 2,000 kPa, as a way of verifying the extrapolation methodology used as an input for the interface shear strength in slope stability analyses. It should be noted that more information should be gathered for LSDS tests at large normal stresses; however, from the trends obtained in Figure 3 for normal stresses ranging from 0 to 800 kPa, these estimates could be a reasonable approach.

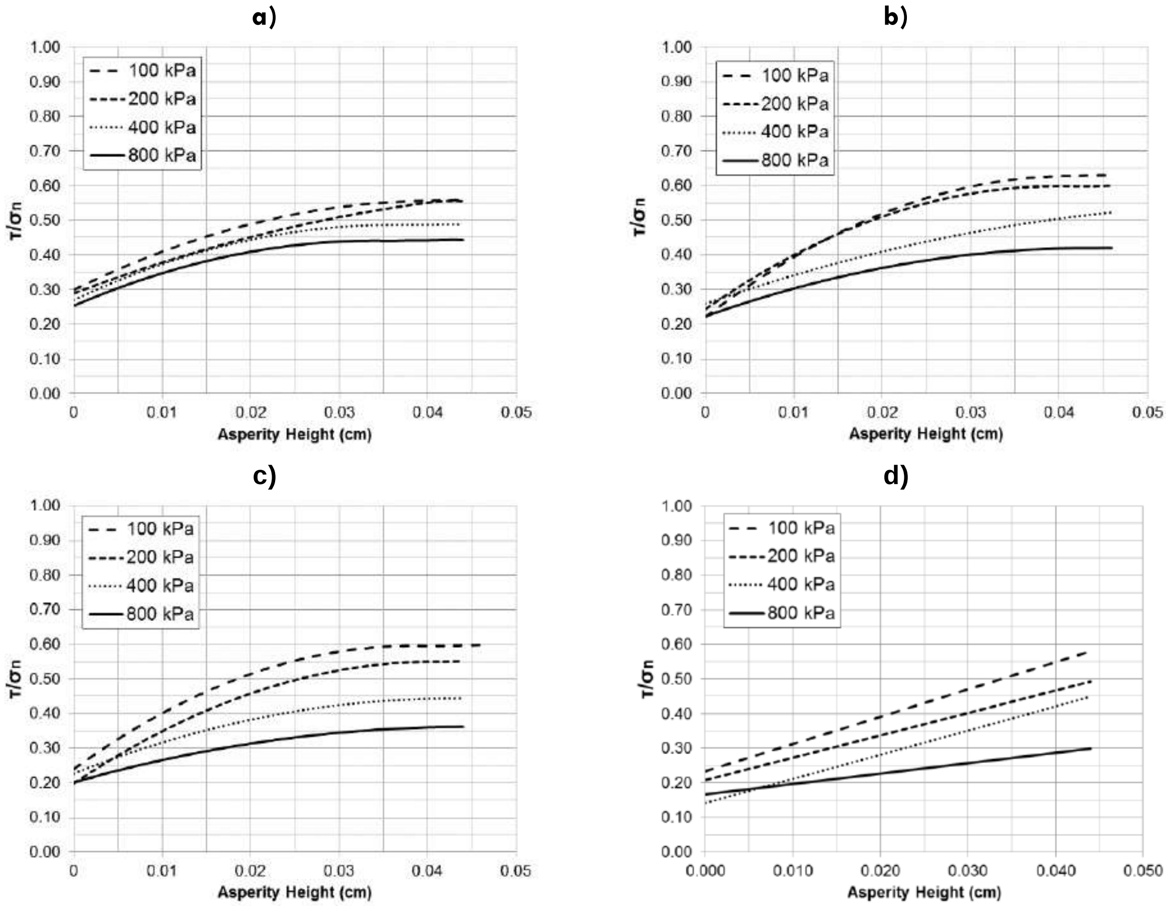

Figure 3: Summary of the normalized shear strength tendency for (a) GC, (b) SC, (c) CL & CH with FC ≤ 65% and (d) CL & CH with FC > 65%

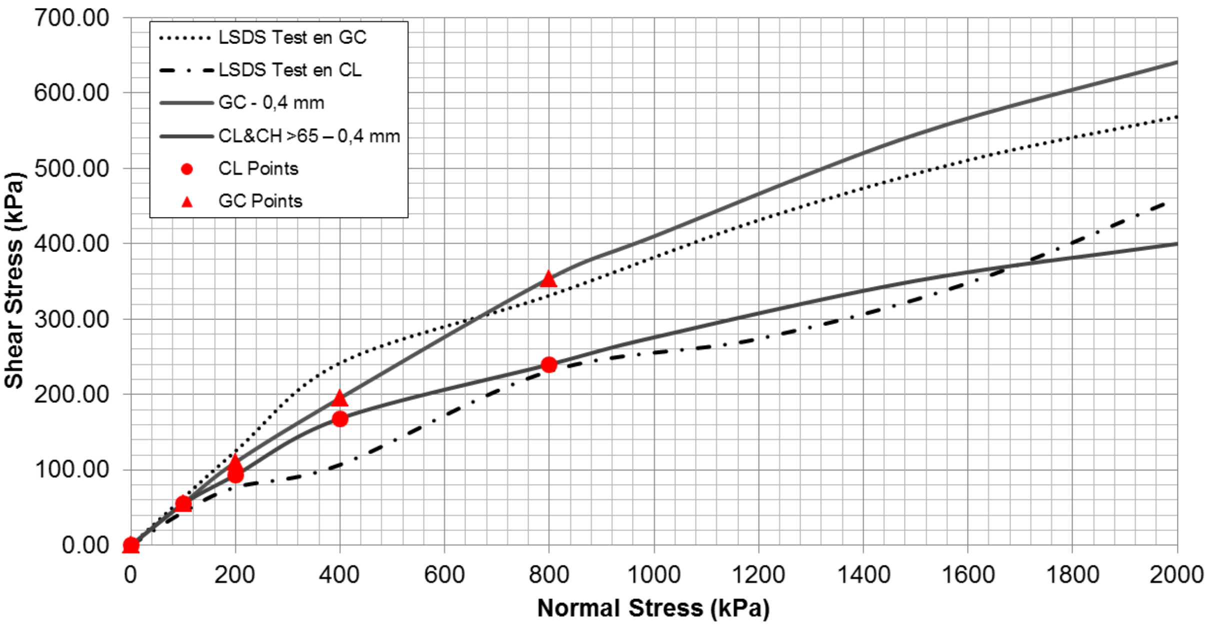

Figure 4: Normal and shear stress extrapolation for interfaces GC and CL soils with a 0.04 cm geomembrane asperity height, compared to LSDS tests performed on the same interfaces

Based on the correlation and methodology above, shear versus normal stress curves were created for typical geomembrane soil interfaces. Shear versus normal stress curves with an asperity height of 0.02 and 0.04 cm respectively are shown in Figure 5.a and Figure 5.b. A 0.02 cm asperity height is a typical feature found in the field, since the asperity is reduced by the installation process and it ends up being below the value recommended (0.025 cm) by the GRI on previous versions of the GM17. On the other hand, a 0.04 cm asperity height represents the current lowest minimum asperity recommended by the latest version of the GM17. In Table 2, the rates of shear and normal stress are shown, to prove that the increment of shear strength at high normal stresses is decreasing, as suggested by the tendencies shown in Figure 3.

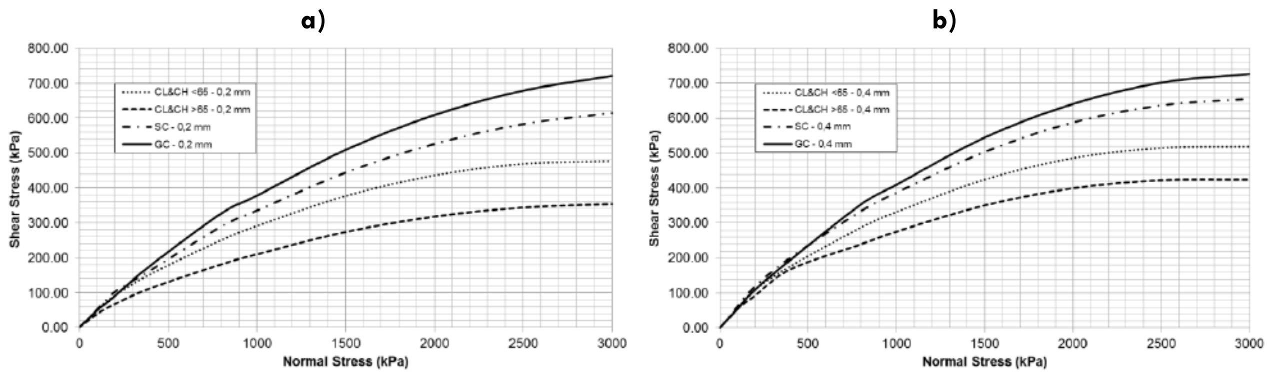

Figure 5: (a) Shear versus normal stress curves for soil liners with a textured geomembrane with a 0.02 cm asperity height (b) Shear versus normal stress curves for soil liners with a textured geomembrane with a 0.04 cm asperity height

Table 2: Normal stress versus rate of shear and normal stress

Normal stress (kPa)	Shear and normal stress rate at asperity height of 0.02 cm (kPa)				Shear and normal stress rate at asperity height of 0.04 cm (kPa)			
	Soil liner classifications				Soil liner classifications			
	CL&CH with FC > 65%	CL&CH with FC ≤ 65%	SC	GC	CL&CH with FC > 65%	CL&CH with FC ≤ 65%	SC	GC
100	0.391	0.508	0.519	0.489	0.549	0.576	0.628	0.556
200	0.337	0.458	0.511	0.451	0.467	0.550	0.598	0.551
400	0.281	0.382	0.409	0.444	0.421	0.443	0.505	0.489
800	0.227	0.313	0.362	0.409	0.3000	0.360	0.418	0.443
1,000	0.210	0.291	0.335	0.378	0.276	0.332	0.387	0.410
1,500	0.183	0.251	0.296	0.339	0.234	0.283	0.336	0.363
2,000	0.159	0.218	0.263	0.305	0.200	0.243	0.294	0.321
2,500	0.138	0.188	0.233	0.272	0.169	0.206	0.255	0.281
3,000	0.118	0.159	0.205	0.240	0.142	0.173	0.218	0.242

Case Histories 1 and 2 slope stability analyses

The main objective of this research was to determine the stabilization cost sensitivity for two heap leach pad operational cases.

Case 1 is a small-scale operation of 8.5 Mton ore heap leach pad and is shown in Figure 6. It was constructed in three stages. Initially, it was designed with a soil liner composed of around 30% gravel, 30% sand and 40% fines, with a textured geomembrane of 0.025 cm. However, due to the long distance needed to bring the previous soil liner borrow area, it was changed to a borrow area with an average of 20% sand and 80% fines. Therefore, to enhance the interface shear strength, a coarse sand frictional layer was applied. Later, during the operation of stage 2, a stability verification was performed, which found that the geomembrane only had 0.015 cm of asperity height on average; as a result the interface provided less shear resistance than the previous one calculated. Since the heap was already operating the heap needed to be stabilized, so the construction of a compacted ore buttress was proposed. Finally, the scope of stabilization for Case 1 is to offer alternatives that could have provided the best cost effective alternative, if it had been implemented initially, compared to the original one constructed at the end of its operation.

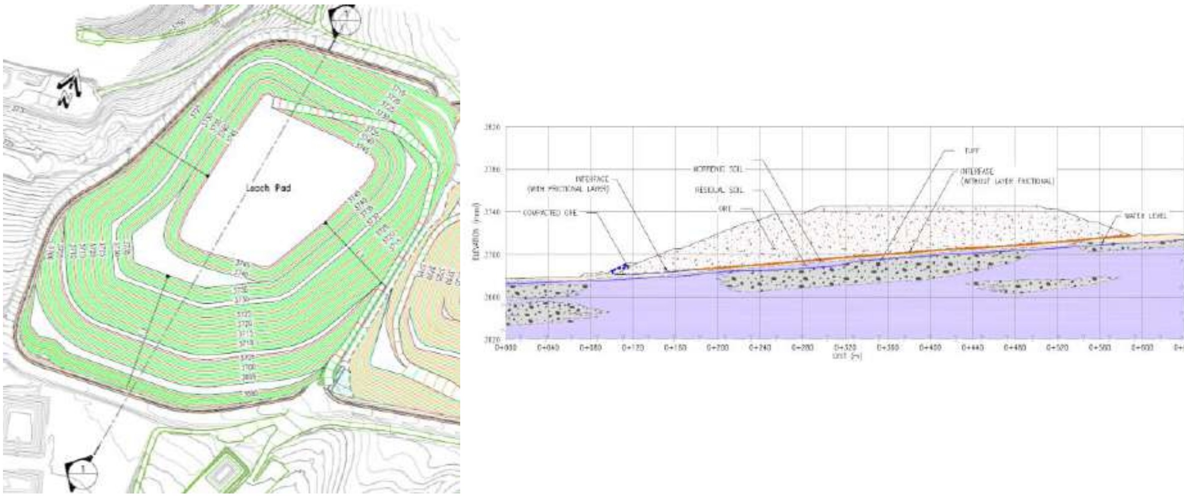

Figure 6: Case 1, layout and cross section

Case 2 is a medium size operation of a 56,6 Mton ore heap leach pad, as shown in Figure 7. It was also constructed in three stages. As the first approach for its geotechnical design, the less expensive alternative was considered, i.e., a nearby soil liner borrow area of 10% gravel, 30% sand and 60% fines, and a typical textured geomembrane with an asperity height of 0.025 cm. However, this first approach provided a very low FS (see Table 4). Therefore, other alternatives were explored to find the best cost effective stabilization solution.

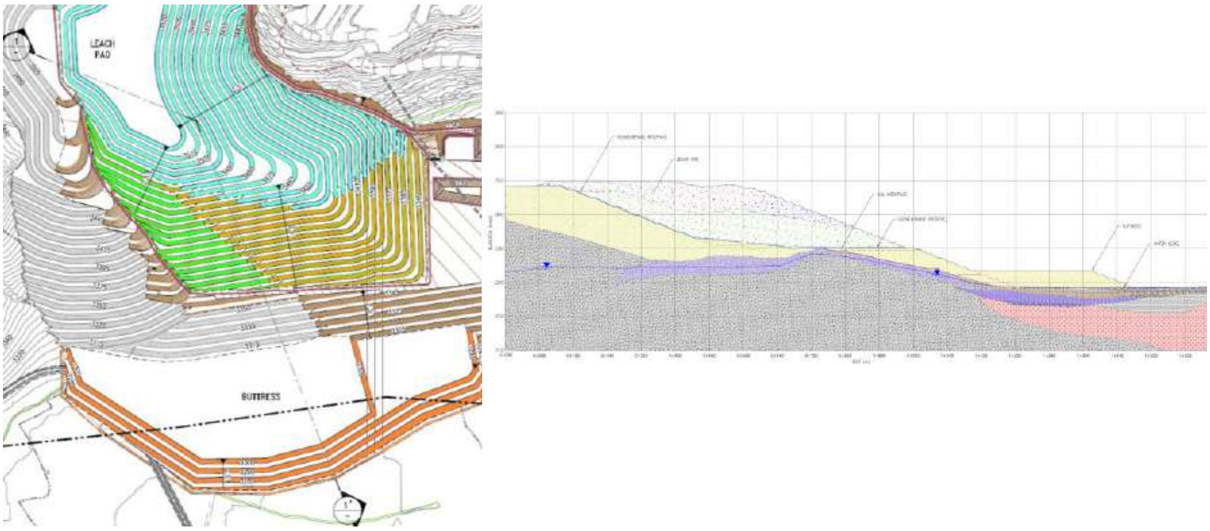

Figure 7: Case 2, layout and cross section

It should be noted that all the alternatives provided a static FS ranging from 1.53 to 1.57, which is a bit higher than the FS recommended by current state of practice (FS = 1.5). Pseudo static conditions were not analyzed, but in high seismicity regions it would be another factor to consider when dealing with the best cost effective stabilization solution. In addition, the shear strength of overliner versus smooth geomembrane interface was considered to be more important than the soil liner versus textured geomembrane interface.

In the case of leached ore properties a Mohr-Coulomb model was implemented. For Case 1 an effective stress friction angle of 35° and no cohesion was considered. For Case 2, an effective stress friction angle of 37° and no cohesion was considered.

Case 1 results are shown in Table 3 and the costs for each solution are shown in Figure 8.a. It can be concluded that if the design had considered Alternative 2 in the beginning, the operation would have saved around US$150,000. This high additional cost is due to some "savings" during construction (borrow area transportation and geomembrane asperity height increment) – but in the long run, these analyses show how effective is to consider high asperity heights in initial design to raise FS, compared to other alternatives for stabilization or post construction alternatives, such as the one shown in Alternative 1. On the other hand, it can be observed how the FS is raised by the use of an asperity height of 0.04 cm for the liner system, and how the cost may be impacted. As for other alternatives that imply the use of 0.04 cm asperity height and soil liner mixture or both, their application is not as effective as Alternative 2 in terms of cost; however, they remain much lower, even when the soil liner is mixed in wet conditions, than both Alternative 1 and Alternative 6, which apply a geometrical modification or a buttress construction.

Table 3: Solution of slope failure stability from Case 1

Alternative	Description	FS	Solution cost in dry condition (US$)	Solution cost in dry condition (US$)
Original		1.27	–	–
Alternative 1	Construction of a buttress with compacted ore and addition of a 50 m frictional layer (coarse sand)	1.53	176,800	–
Alternative 2	Additional cost of textured geomembrane (0,04 cm) for the first 85 m in stability platform	1.53	12,700	–
Alternative 3	Additional cost of textured geomembrane (0,04 cm) for all liner system	1.66	48,750	–
Alternative 4	Additional cost of textured geomembrane (0,04 cm) and borrow area mixture (GC) for the first 60 m	1.53	31,000	40,500
Alternative 5	Using similar geomembrane (0,02 cm) and borrow area mixture (GC) for the first 85 m	1.55	32,000	44,000
Alternative 6	Additional cost of textured geomembrane (0,04 cm) and 1 stability trench	1.55	129,500	–

Case 2 results are shown in Table 4, and the costs for each solution are shown in Figure 8.b. Alternative 2 results in the best cost effective stabilization solution, as long as the mixture of gravel with the original soil liner is done in wet conditions. However, it should be noted that when the solutions of asperity height increment and soil liner mixtures are compared to soil liner mixtures only, the costs are similar. However, the first one may require less preparation time to mix than the one provided by soil mixture only; such time costs have not been considered in the cost. It also should be noted that the trench excavation proposed in Alternative 2 may demand more time to be developed and require more construction complexity; such conditions have not been accounted for either. Finally, the alternative that provides for only the excavation of three trenches is the most expensive one, followed by alternatives that combine soil liner mixture with one stability trench construction.

Table 4: Solution of slope failure stability from case two

Alternative	Description	FS	Solution cost in dry condition (US$)	Solution cost in wet condition (US$)
Original		1,28	–	–
Alternative 1	Construction of 3 stability trenches	1,54	451,000	
Alternative 2	Additional cost of textured geomembrane (0,04 mm) for all liner system and 1 stability trench	1,54	270,000	
Alternative 3	Additional cost of textured geomembrane (0,04 mm) and borrow area mixture (SC) for the first 455 m	1,57	325,000	405,000
Alternative 4	Additional cost of textured geomembrane (0,04 mm) and borrow area mixture (GC) for the first 360 m	1,57	275,000	342,000
Alternative 5	Using original geomembrane (0,02 mm) and borrow area mixture (SC) for the first 640 m	1,56	387,000	516,000
Alternative 6	Using original geomembrane (0,02 mm) and borrow area mixture (GC) for the first 455 m	1,57	243,000	324,000
Alternative 7	Using original geomembrane (0,02 mm), borrow area mixture (SC) for the first 360 m and 1 stability trench	1,55	330,000	398,000
Alternative 8	Using original geomembrane (0,02 mm), borrow area mixture (GC) for the first 270 m and 1 stability trench	1,55	309,000	370,000

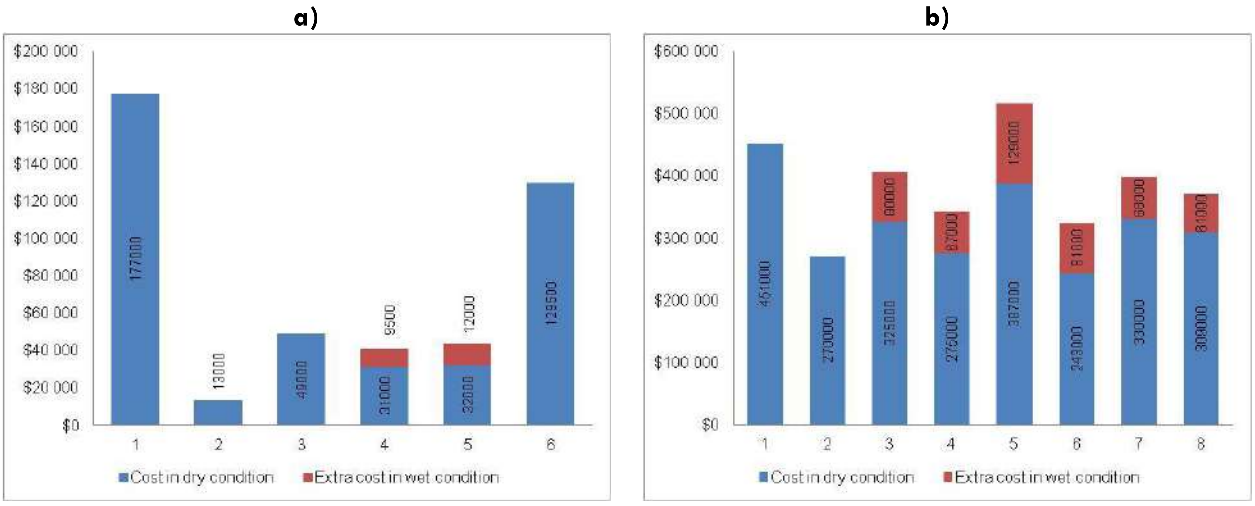

Figure 8: (a) Case 1 costs per stabilization alternative
(b) Case 2 costs per stabilization alternative

Conclusions

- The current research has updated the Ayala et al. (2014) curves by adding 18 LSDS tests to provide a data base of 100 tests to develop these curves.
- The shear strength behavior for soils used in practice as soil liner in heap leach pad projects is quite nonlinear depending on the asperity height and the normal stress; thus it must be considered when performing slope stability analysis and its implications for the cost must be considered when developing stabilization solutions.
- The methodology proposed by Ayala and Huallanca (2014) to extrapolate interface shear strength for high normal stresses has been used to develop four typical soil liner/textured geomembrane interface shear strengths for a 0.02 cm asperity height, which is bit lower than the minimum asperity height recommend by previous versions of the GM17, and to develop another four soil liner/textured geomembrane interface shear strengths for a 0.04 cm asperity height, as per the actual GM17 recommendation. The extrapolation calculations have been verified with LSDS tests performed at high normal stresses and shear strength normal stress ratio calculations that decrease as the normal stress increases. These interfaces were created in order to perform a stability and cost effective sensitivity analysis.
- A stability and cost effective sensitivity analysis was developed for 2 historical cases. Case 1 is a small-scale operation where a post construction stabilization solution was applied; Case 2 provides different stabilization solutions conceived during the design stage to provide the best cost effective alternative.
- For both cases the application of a solution that involves a high asperity height (0.04 cm) provides the best cost effective alternative. If high factors of safety are needed, as is the norm in high seismic regions, the alternative should add soil mixture, since trenches or buttresses solutions require the highest costs of the solutions explored in this paper, therefore, they should be applied as the last resorts.
- The increase of the asperity height to 0.04 cm proposed by the GM17 does not provide a high cost for operators, and it even improves the stability of heap leach pads. However, as it did not happen in the past, due to low interface shear strength because of the use of an asperity height of a low value, overliner versus smooth geomembrane interface should be assessed to ensure safe conditions for both overliner and soil liner interfaces, and to prevent failures.
- Another component to be added to the alternatives given and increase their cost efficiency would be the application of a 3-D slope stability analysis (Reyes et al., 2014), which should be a small investment compared to the savings when optimizing the cost of the stabilization solution.

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