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Focus to Assess the Shear Strength of a Liner System for Block Failure in High Heap Leach Pads

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ABSTRACT
One of the most important aspects in the geotechnical heap leach pads design is related to the interface strength of the underlying geomembrane liner system which isolates the leach solutions and mineral from the natural ground. The liner system characteristics depends of the environmental regulations, site conditions, risk associates to the solution for leaks, operation characteristic, equipment of operation above the liner, normal stress for ore heaping, adequate liner strengths for slope stability, etc. In the world, the tradition liner system design considers a double contention system formed by a geomembrane (GM) liner over a compacted layer of soil liner (SL), the over liner (OL) and pipes collection system are collocated over the GM. The OL generally is a granular soil that reduces the damage in the GM and allows to drainage the leach solutions.

The interface shear strength between GM and SL is estimated through the large-scale direct shear (LSDS) test which provides the peak and post-peak shear strengths. In the practice, the slope stability analysis for heap leach pads is running with the lowest post-peak shear strength in the liner system. Usually, the executed LSDS tests show GM-SL interface has the lowest post-peak shear strength, likewise these LSDS tests reported that shear strength of OL-GM interface is higher than SL-GM interface. For this reason, OL-GM interface shear strength is not used in the slope stability analysis, however this focus could be changed because the tendency of the lowest shear strength changed to the OL-GM interface due to a minimum request for asperity height of the geomembrane of 0,4 mm according to GRI–GM 17 (2019).

This paper proposes a focus and methodology for the estimation of the shear strength of a liner system based on the analysis of 453 LSDS tests. In addition, this research proposes correlations to estimate the SL-GM interface post-peak shear strength based on asperity height of GM and SL classification (USSC), also correlations for OL-GM interface is proposed based on gravel content of OL. For slope stability analyses, the use of peak shear strength for flat zones and post-peak shear strength for stepped zones of heap leach pads are evaluated, the findings are compared with results of numerical analysis.

1. INTRODUCTION

One of the biggest mining structures is the heap leach pad, where piled ore is irrigated with solution which dissolves the mineral impregnated in the rocks obtained from the open-pit mining, the solution with mineral is called PLS solution (pregnant leach solution) which passes through the piled ore and is recovered by the collection and liner system located on the bottom of the pad, the liner is usually composed of a clayey soil or GCL (geosynthetic clay liner), LLDPE geomembranes, granular drainage and protection material are called overliner which is supplemented with drain pipes at controlled spacing (Thiel & Smith, 2003).

Critical aspects of the geotechnical design of leach pads are associated with the height of the ore pile, resistance of the interfaces produced in the liner system, mineral saturation, site location topography and site geology.

In practice, a conservative approach is usually added on heap leach pad slope stability analysis by using post-peak values of LSDS tests.

2. LINER SYSTEMS DESIGN

The liner system (Figure 1) is a containment system that isolates the leaching solution with the natural terrain. The characteristics of the liner systems will depend on environmental regulations, site conditions, risks associated with solution leaks, operation characteristics, operating equipment, compressive stresses (normal) due to mineral loads and height of the piles, physical slope stability, etc. (Romo, 2015). The traditional design of liner systems in many pads around the world includes a double containment system composed by a geomembrane over a compacted clay layer or a geosynthetic clay liner (GCL). This practice responds to the impacts of leakage solutions in the ecosystem and communities near the mining project.
In Perú, the most of pads are located in highlands where the underground water bodies or lagoons are very close to mining projects, so the current standard for liner systems design includes a double containment system. Likewise, the topographic

conditions as the valleys or stepper slopes, condition the design of the liner system to the use of GCL in terrains with slopes greater than 0.5 (V/H) and compacted soil on flat terrain or with slopes less than 0.5 (V/H).

The liner system design for a leach pad is a delicate balance between leveling and filling (earthmoving), selection and characterization of geosynthetics, low permeability soil, and over liner material. Besides, the liner system design must be addressed under the operating and environmental conditions (Lupo, 2010).

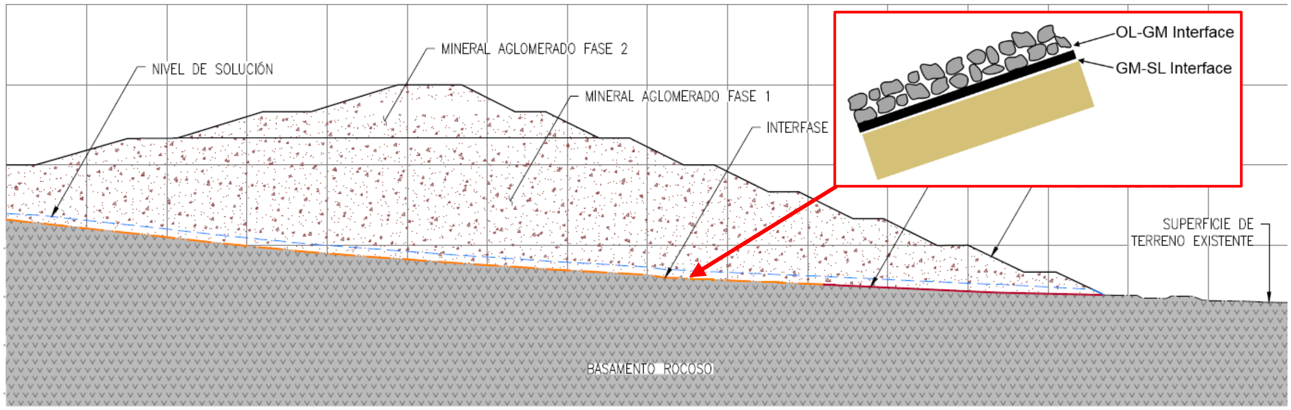


Figure 1. Heap leach pad and liner system.

2.1 Geomembrane selection

The selection of a suitable geosynthetic liner is crucial to the success of a leach pad design, due to the ore is stacked over the liner system, and the pile height is continuously growing. The thickness of LLDPE geomembrane (GM) is generally selected based on its puncture resistance. For this, it is recommended that puncture tests are conducted with the site materials (SL and OL) that will be placed in contact with each side of the geomembrane.

The extensive use of LLDPE geomembranes in leaching pad operations has shown that they are suitable for containing corrosive acid drainage and metal leaching products for periods of at least 20 years. However, there is not enough data regarding its long-term behavior (50 to 100 years) (Renken et al., 2007). The behavior of the geomembrane under load conditions is strongly influenced by its interaction and compatibility with the components of the liner system, so the design requires a thorough understanding of the interaction between these components, the normal load applied, the type of solution and the pore pressure (solution level). For mining projects, geomembrane manufacturing is carried out according to suggestions of the Geosynthetic Research Institute (GRI) in addition to suggestions of consultants specialized in heap leach pads design. The GM13 (2019) and GM17 (2015) standard specifications apply to HDPE and LLDPE geomembranes, respectively.

The main restriction of geotechnical design and analysis of heap leach pads is related to liner system, which usually provides a couple of interfaces with low shear strength due to contact of OL with GM and GM with SL. These interfaces control the stability conditions of such facilities by a block failure mode. Some efforts have been made to model and predict the shear strength of this kind of liner adequately (e.g., Ivy, 2003; Yesiller, 2005; Blond & Elie, 2006). In a previous research (Ayala & Huallanca, 2014), the author noted that GM-SL interface shear strength increases with increment of geomembrane asperity height, normal stress increment (σ_n) and the increment of granular material content in soil liner. This research updates the expressions of Ayala & Huallanca (2014) and proposes new relationships to estimate the shear strength of GM-SL and OL-GM interfaces.

3. LARGE-SCALE DIRECT SHEAR TESTS (LSDS)

Shear strength of OL-GM and GM-SL interfaces are usually estimated by large-scale direct shear tests (LSDS) in accordance with ASTM D5321 standard. Figure 2a shows the typical behavior of shear strength vs. displacement; the shear strength gradually increases until the peak shear stress ($\tau_{p,1}$). After the peak, the shear stress falls, as it is demonstrated in the $\tau_{p,1} - \tau_{r,1}$ section. Generally, several test specimens with soil and geomembrane are tested varying the normal stress (σ_n); for each normal stress, the shear stress vs. displacement curve is obtained, as shown in Figure 2a. The peak or residual shear stress can be plotted against the corresponding normal (σ_n) stress, as shown in Figure 2b. This envelope could be defined by Mohr-Coulomb model (angle of friction and adhesion); however, the interface shear strength envelope has a no linear behavior, as noted by Stark & Choi (2004).

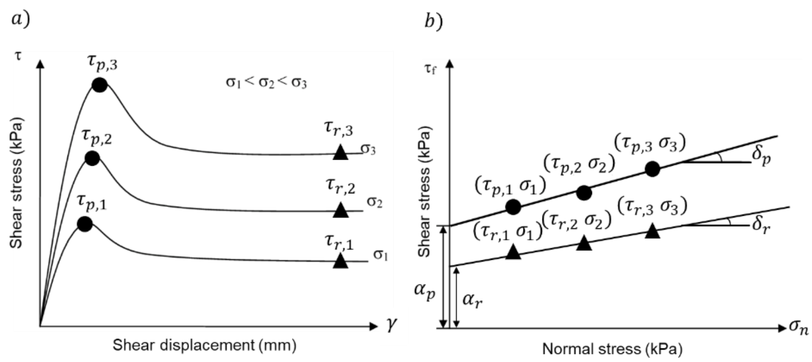


Figure 2. Typical plots obtained from LSDS tests: a) typical behavior of shear strength vs. displacement; b) plot of shear stress vs. normal stress.

3.1 Data selection

The collected data shows the typical shear stress-displacement behavior of OL-GM and GM-SL interfaces. In this research, 64 LSDS tests on OL-GM interfaces with smooth geomembranes and 388 LSDS tests on GM-SL interfaces with smooth and textured geomembranes have been compiled. Compiled LSDS tests were carried out with site-specific materials of different mining projects and using HDPE and LLDPE geomembranes, at each normal stress of 100, 200, 400, and 800 kPa, a constant displacement rate and a constant shearing area of about 300 x 300 mm. The test data were divided into five groups based on interface type and soil classification (according to the Unified System of Soil Classification, USSC) and fines content. The main features of soil samples and geomembrane type of each group are summarized in Table 1, where the average (μ) of particles content also is indicated.

Table 1: Geomembranes type and soil samples characteristics summary.

Interface type	Geomembrane type	Soil classification	Gravel content (%)	Sand content (%)	Fines content (%)	Liquid limit (%)	Plastic index (%)	Number of tests
OL-GM	Smooth	GP, GM, GM, GW and others	33 - 98 μ=65.5	0 - 29 μ=20.6	0 - 38 μ=13.9	34.5	12.1	64
GM-SL	Smooth and textured (asperity height between 0 and 0.6 mm)	GC	30 - 60 μ=41.2	9 - 39 μ=29.4	15 - 46 μ=33.3	32.7	14.7	48
		SC	0 - 34 μ=21.7	29 - 67 μ=39.3	19 - 50 μ=38.8	34.9	16.0	128
		CL & CH (fines content < 65 %)	0 - 28 μ=12.1	15 - 55 μ=32.1	46 - 69 μ=55.8	35.8	16.9	104
		CL & CH (fines content > 65 %)	0 - 20 μ=6.3	3 - 40 μ=17.0	54 - 97 μ=76.8	48.1	24.6	108

The collected data regarding peak and post-peak shear strengths of OL-GM interface are shown in Figures 3a and 3b, respectively, separated by specific normal stresses (in different colors). The influence of gravel content (material retained on a #4 sieve) of OL in interface shear strength can be distinguished in both figures. Likewise, the collected data regarding the post-peak shear strength of SL-GM interface vs. geomembrane asperity height is shown in Figure 3c for each normal stress. The influence of soil classification or fines content is not presented in this figure. The peak shear strength of SL-GM interface vs. asperity height is not shown because the behavior tendency is not clear.

The Mohr-Coulomb model (angle of friction and adhesion) applied to OL-GM interface shear strength is showed in Figure 4 in which three average curves are proposed for different gravel content ranges. However, these relationships could have a considerable error or difference due to the influence of various factors in shear strength. In order to achieve a better relationship to estimate the OL-GM shear strength, expressions based on gravel content (G) are proposed in the current study.

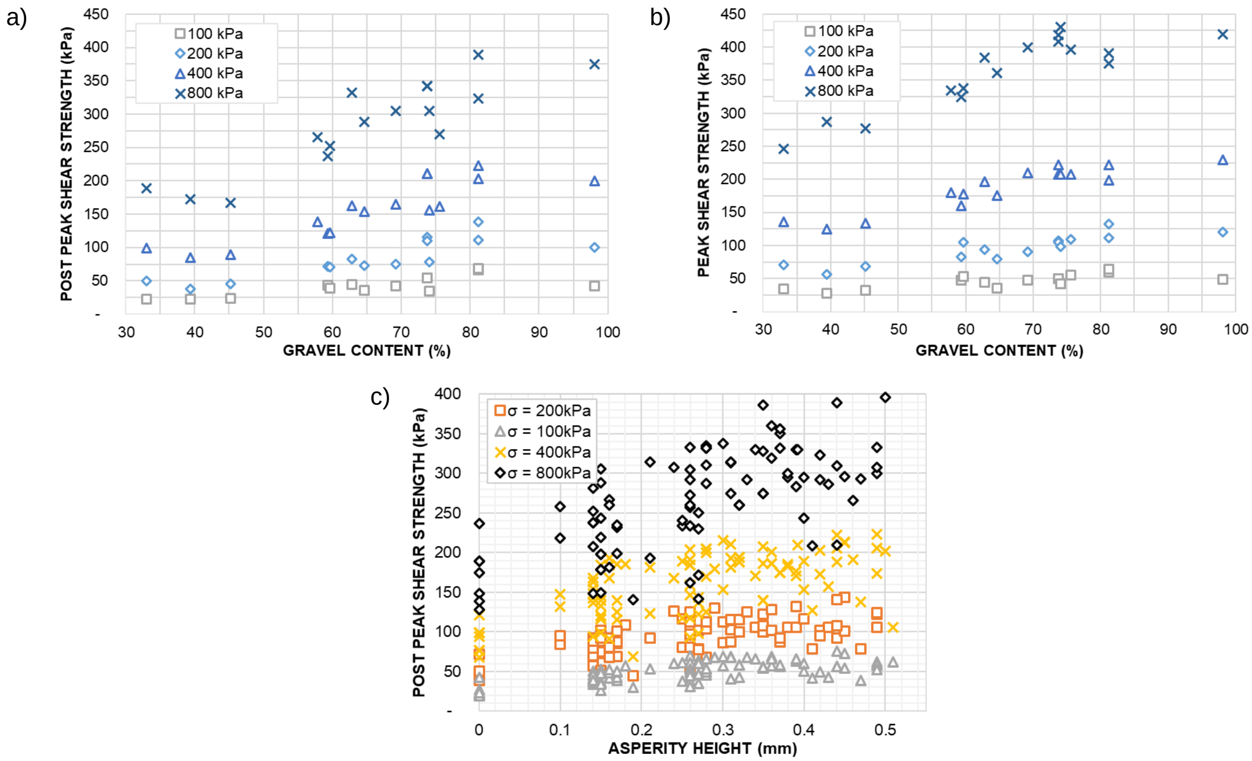


Figure 3. (a) Post-peak shear strength vs. gravel content in GM-OL interface; (b) peak shear strength vs. gravel content in GM-OL interface; (c) post-peak shear strength vs. asperity height in SL-GM interface.

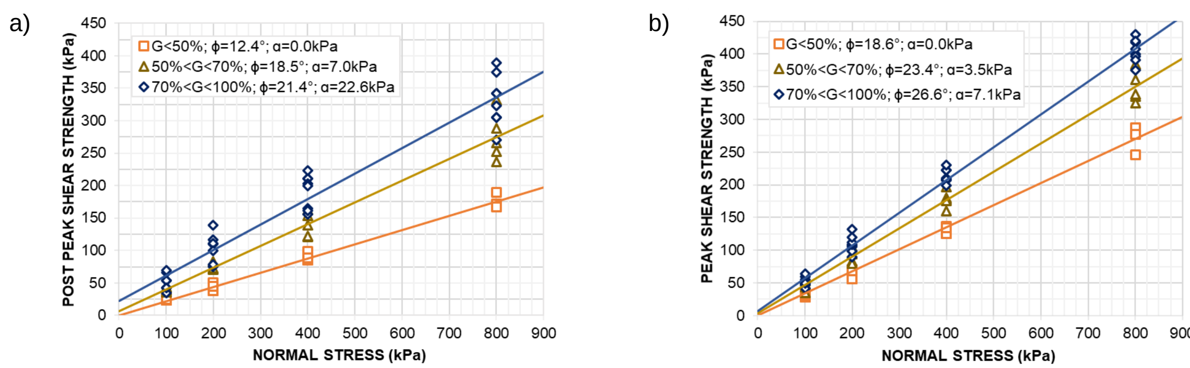


Figure 4. Average failure envelopes for different gravel content ranges based on Mohr-Coulomb model: (a) post-peak shear strength vs. normal stress; (b) peak shear strength vs. normal stress.

4. PROPOSED RELATIONSHIPS FOR INTERFACE SHEAR STRENGTH

The average shear strength envelope was obtained by both linear and nonlinear (polynomial) regressions. Goodness of fit was verified with the coefficient of determination (R^2). If R^2 is close to 1.0 means that most of the variability in shear strength can be attributed to the considered variable (asperity height or gravel content), while a value of R^2 close to 0 means that this variable does not have significant influence. The fit of the average shear strength envelopes shows that R^2 is greater than 0.50, which means that a correlation can be defined between the analyzed variables.

The interface shear strength could be greater or less than the estimated average. An overestimation of the interface shear strength can lead to inadequate physical stability analysis of the heap leach pad. Therefore, it is recommended that shear strength is defined with confidence levels, in which there is a confidence with a probability (P) that the interface shear strength will be at least the estimated shear strength. Based on this, envelopes with confidence levels have been proposed through a statistical treatment of data. Probabilities (P) associated with ± 1σ (16 and 84 %) have been considered. The

Anderson-Darling goodness of fit test was applied successfully to verify if the *Error* (the difference between the estimated value and the observed value) has a normal distribution.

4.1 OL-GM interface shear strength relationship

The shear strength relationship for overliner-smooth geomembrane (OL-GM) interface for each normal stress was defined based on the influence of overliner gravel content. Regarding the data for this study, the following observations are mentioned:

- Most of the data were obtained from LSDS tests on interfaces with LLDPE SST (single side textured) geomembrane liner and a few amounts of tests with HDPE geomembrane liner.
- The influence of geomembrane thickness has not been considered; the thickness range is from 1.5 to 2.0 mm.
- Most of the overliner corresponds to sub-angulous and sub-rounded gravels.
- The maximum particle size of overliner is 2".
- The summary of OL characteristics is indicated in Table 1.
- The maximum normal stress was 800 kPa for all tests.

Figure 5 presents the proposed relationships between gravel content and (a) post-peak shear strength and (b) peak shear strength, and Figure 6 shows the same relationships with both lower and upper confidence boundaries for one standard deviation. Other factors, such as the angularity of gravel particles, can also influence interface shear strength. Otherwise, the highest shear strength is not guaranteed with high gravel content. If an adequate gradation of the soil is not evaluated, the effective contact area between soil particles and geomembrane would be lower, which reduces the interface shear strength. The evaluation of the liner system must also consider these aspects in heap leach design.

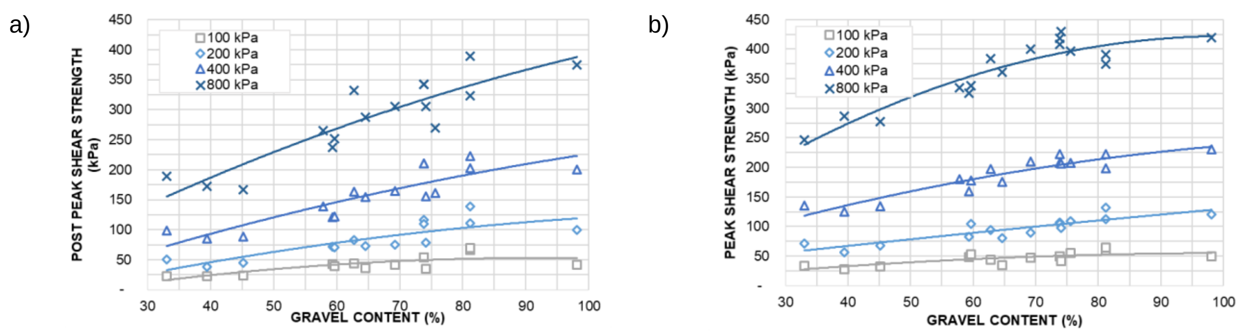


Figure 5. Proposed relationships between gravel content and (a) post-peak shear strength and (b) peak shear strength.

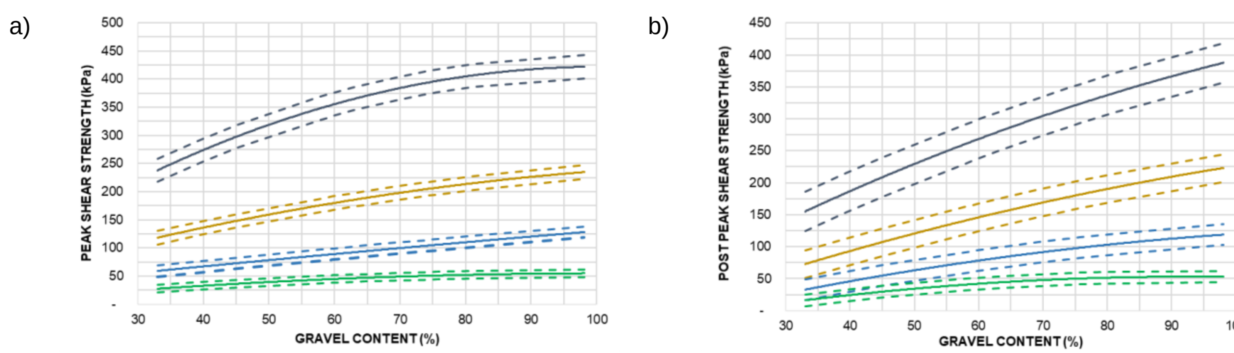


Figure 6. Graphs with confidence intervals (±1σ) based on relationships between gravel content and (a) peak shear strength and (b) post peak shear strength (normal stresses: 100, 200, 400 and 800 kPa).

OL-GM interface shear strength discussion:

- There is a clear nonlinear tendency of shear strength increment for post-peak and peak values. The behavior tends to be asymptotic at 70 % gravel content for 100 kPa normal stress.
- The standard deviation of peak shear strength relationship is lower than the post-peak shear strength relationship; thus, there is a better correlation between peak shear strength and gravel content.
- For post-peak values, the standard deviation increases for high normal stresses, as noted in Figure 6b.

- It is also observed that for high normal stresses, the rate of shear strength increment is increase and the behavior does not tend to be asymptotic.

The proposed correlations based on gravel content (G) are summarized in Table 2 for post-peak shear strength and in Table 3 for peak shear strength for different normal stresses. The standard deviation (σ) and correlation coefficient (R^2) obtained for each relationship also are indicated. These relationships could be used in stability analysis of block failure of heap leach pads.

Table 2: Expressions proposed for estimation of OL-GM interface post-peak shear strength based on gravel content (G).

Interface	Normal Stress, σ_n (kPa)	Post-peak shear strength, τ_r (kPa)	Standard deviation, σ (kPa)	Correlation coefficient, R^2
OL-GM	100	τ_r = -0.0103G^2 + 1.9185G - 35.851	9.1	0.61
	200	τ_r = -0.0095G^2 + 2.5697G - 41.702	16.3	0.69
	400	τ_r= -0.0104G^2 + 3.6755G - 37.161	21.6	0.77
	800	τ_r = -0.0164 G^2 + 5.7243G - 15.824	30.6	0.80

Table 3: Expressions proposed for estimation of OL-GM peak interface shear strength based on gravel content (G).

Interface	Normal stress, σ_n (kPa)	Peak shear strength, τ_p (kPa)	Standard deviation, σ (kPa)	Correlation coefficient, R^2
OL-GM	100	τ_p= -0.0057G^2 + 1.1666G - 4.6074	6.7	0.58
	200	τ_p = -0.0013G^2 + 1.2337G + 20.051	10.0	0.77
	400	τ_p = -0.0129G^2 + 3.4823G + 17.569	12.0	0.87
	800	τ_p = -0.04G^2 + 8.066G + 15.545	20.4	0.87

4.2 GM-SL shear strength relationship

The shear strength relationships for textured geomembrane – soil liner (GM-SL) interface were obtained by the influence of asperity height and the type of soil; similar to OL-GM relationship, these results could be used to run stability analysis of heap leach pads for block failure or maybe the combination of them. Regarding the data for this study, the following observations are mentioned:

- Most of the data were obtained from LSDS tests on interfaces with LLDPE SST (single side textured) geomembrane liner and a few amounts of tests with HDPE geomembrane liner.
- The influence of geomembrane thickness has not been considered; the thickness range is from 1.5 to 2.0 mm.
- The types of GM-SL interfaces were classified according to soil liner as noted in Table 1.
- The maximum normal stress was 800 kPa for all tests.
- Silts (ML or MH) were not evaluated due to a lack of correlation and low data availability.

Figure 7 presents the proposed relationships between asperity height and post-peak shear strength (a) GC: clayey gravel; (b) SC: clayey sand; (c) CL clay with sand (fines content below 65 %) and (d) CL & CH: clay with sand and high plasticity clay (fines content over 65 %). Figure 8 shows the same relationships with lower an upper boundary considering one standard deviation (σ). The influence of gravel content is noted in shear strength; it can be for friction generated between gravel particles and geomembrane. As the normal stress increases, OL-GM shear strength tend to be greater than GM-SL shear strength, especially in CL soils. The proposed correlations are summarized in Table 4 for post-peak shear strength based on asperity height and considering different normal stresses. The standard deviation (σ) and correlation coefficient (R^2) obtained for each relationship also are indicated.

GM-SL interface shear strength discussion:

- There is a clear nonlinear tendency of shear strength increment for post-peak and peak values, where the behavior tends to be asymptotic at 0,4 mm asperity height.
- Peak shear strength correlations were not defined due to low R^2 values and high standard deviations.
- The standard deviation of post-peak values increases for higher normal stresses for all GM-SL types, as noted in Figure 8.
- The fines content influence in interface shear strength is clearly noted for normal stresses under 400 kPa, where the GM-SL interface could have a higher shear strength than the OL-GM interface.

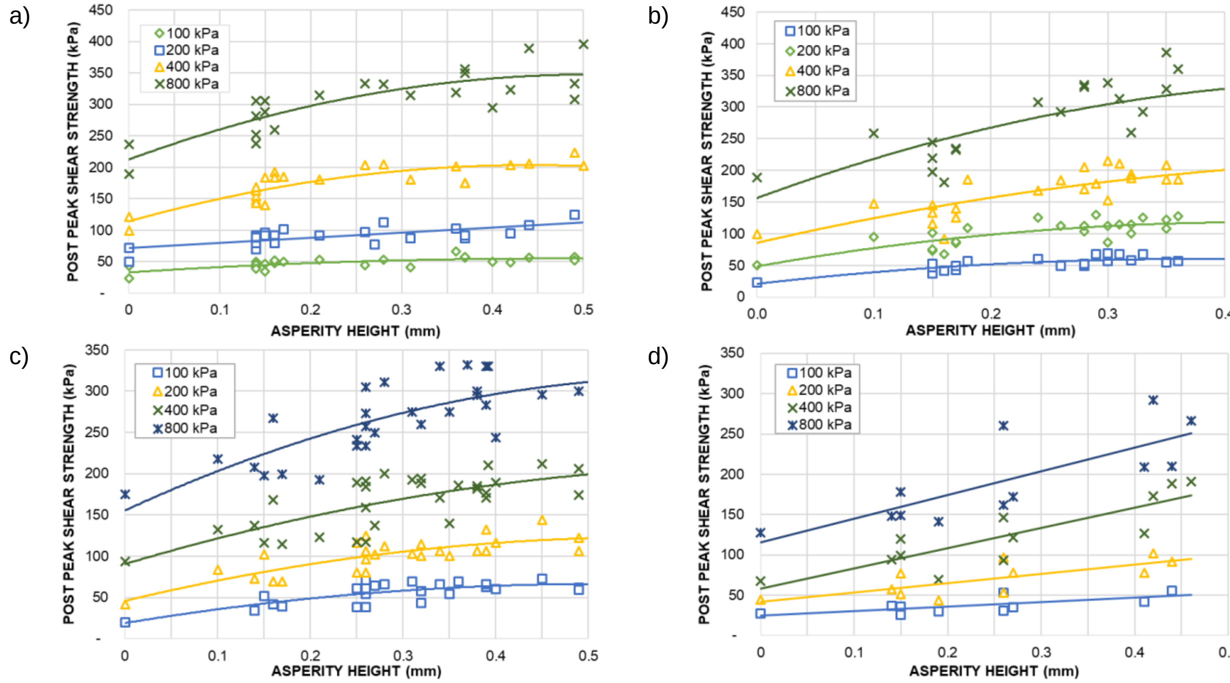


Figure 7. Relationship between asperity height and interface post -peak shear strength for the following soil types: (a) GC; (b) SC; (c) CL (F < 65 %) and (d) CL (F > 65 %).

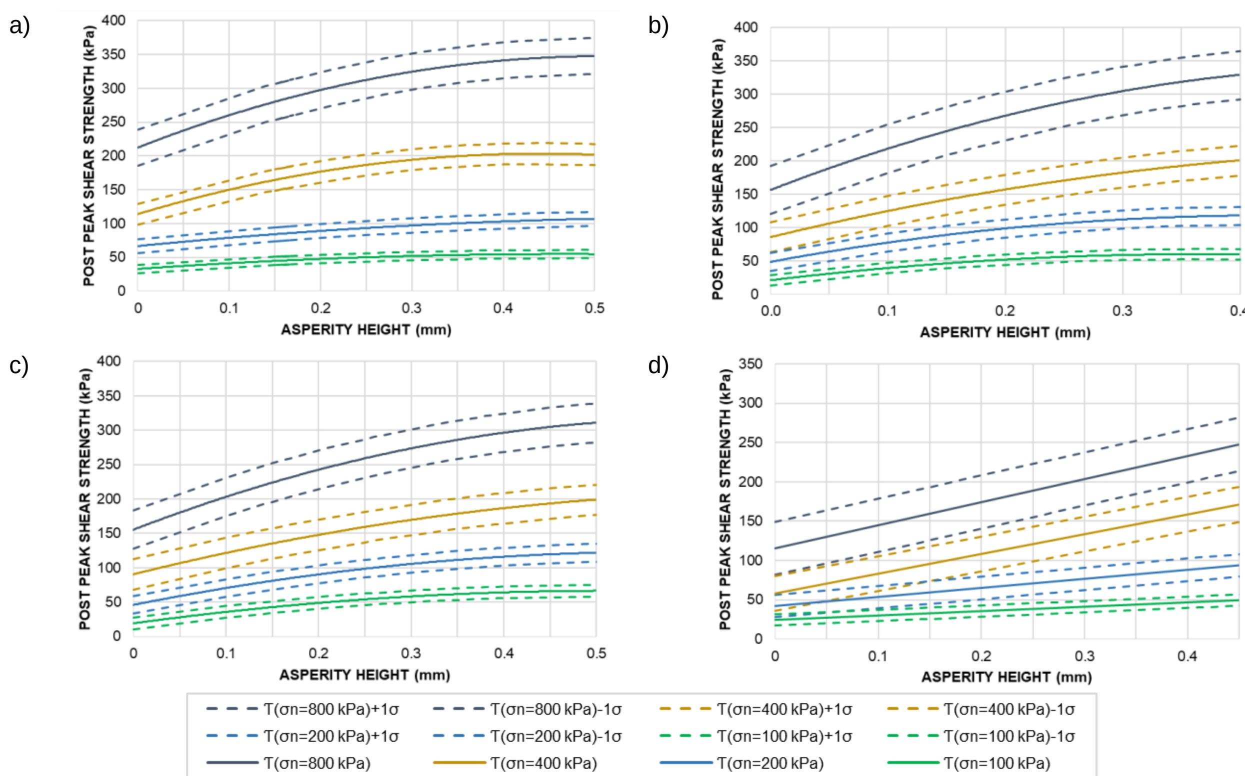


Figure 8. Graphs with confidence intervals of ±1σ based on relationships between asperity height and interface post-peak shear strength for the following soil types: (a) GC; (b) SC; (c) CL (F < 65 %) and (d) CL (F > 65 %).

Table 4: Proposed expressions for the estimation of GM-SL interface post-peak shear strength based on asperity height (a).

Interface	Normal stress, σ_n (kPa)	Post-peak shear strength, τ_r (kPa)	Standard deviation, σ (kPa)	Correlation coefficient, R^2
GM-SL (CL, F < 65 %)	100	τ_r = -179.3a^2 + 184.39 + 19.015	8.5	0.59
	200	τ_r = -234.42a^2 + 269.02 + 45.882	12.9	0.65
	400	τ_r = -232.17a^2 + 333.15a + 90.534	22.2	0.55
	800	τ_r = -414.45a^2 + 518.37a + 155.42	28.0	0.61
GM-SL (CL, F > 65 %)	100	τ_r = 55.789a + 24.636	7.2	0.51
	200	τ_r = 115a + 42.195	14.3	0.55
	400	τ_r = 252.07a + 57.999	22.2	0.73
	800	τ_r = 294.52a + 115.57	33.9	0.61
GM-SL (SC)	100	τ_r = -287.26a^2 + 212.09a + 20.822	7.6	0.56
	200	τ_r = -383.19a^2 + 326.66a + 48.429	13.6	0.58
	400	τ_r = -344.73a^2 + 425.5a + 85.542	22.4	0.62
	800	τ_r = -623.37a^2 + 680.41a + 156.34	36.1	0.59
GM-SL (GC)	100	τ_r = -100.57a^2 + 94.059a + 32.849	6.2	0.51
	200	τ_r = -103.03a^2 + 132.17a + 66.743	10.4	0.52
	400	τ_r = -459.25a^2 + 405.22a + 114.05	15.5	0.73
	800	τ_r = -517.56a^2 + 528.9a + 212.46	26.6	0.70

5. INFLUENCE OF THE USE OF PEAK OR POST-PEAK SHEAR STRENGTH

5.1 Application of interface shear strength

Gilbert & Byrne (1996) indicated that the peak shear strength is mobilized between 1 and 15 mm of displacement; also the post-peak shear strength resistance can be as less as 30 % of the peak resistance values, about these findings the current investigation found the following remarks:

- While OL-GM peak shear strength is between 12 and 22 mm of shear displacement, GM –SL peak shear strength is mobilized between 7 and 32 mm.
- While OL-GM post-peak shear strength resistance can be as less as 40 % of the peak values, GM-SL post-peak shear strength resistance can be 40 % of the peak values.

For the liner system design is logic to use post-peak values, which means a conservative criterion due to progressive failures could be occurred in geosynthetic reinforced structures (Jogi, 2005). The suggestions of different authors for the use of peak or post-peak shear strength resistance values in the design are summarized below, listed from the most to the least conservative:

- Use of post-peak shear strength for all conditions (Stark & Peoppel, 1994).
- Use of post-peak shear strength of the lowest peak resistance interface, this concept applies to multiple geosynthetic interfaces (after Koerner & Bowman, 2003).
- Use the peak shear strength at the base of the slope (flat areas) and post-peak shear strength along the steeper side-slope (Jones et al., 2000).
- Use the peak strength at the top of the slope and the post-peak strength at the base of the slope (after Koerner & Bowman, 2003).
- Use peak strength for all static (non-seismic) conditions (Koerner & Bowman, 2003).

In this paper, the use of peak shear strength for flat zones and post-peak shear strength for stepped zones of heap leach pads is evaluated, the findings have been compared with the results of numerical analysis.

5.2 Stability analysis of heap leach facilities

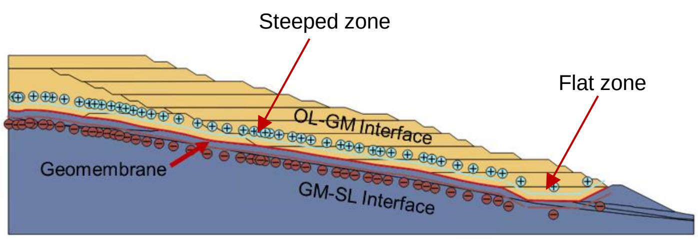
The limit equilibrium method (LEM) is usually applied for an approximated evaluation of the physical stability of heap leach pads by calculating a representative safety factor (FS). In this analysis, the interface is modeled as a continuum element (like a thin layer of soil). The analysis criterion that is most widespread and accepted in practice usually assumes that mobilized shear strength reaches or exceeds the peak shear strength of the materials; thus, post-peak shear strength is usually considered in stability analysis. However, this criterion can be considered as very conservative when the factor of safety (FS) is estimated.

In the present study, the mobilization of shear stresses along interfaces both below and above the geomembrane has been evaluated by numerical analysis with finite elements, by estimating the relative shear stresses (τ_{rel}) in each interface. The relative shear stress indicates the proximity of the mobilized shear stress (τ_{mob}) to the shear strength of the failure envelope ($\tau_{máx}$) for a specific effective normal stress (see Equation 1).

$$\tau_{rel} = \frac{\tau_{mob}}{\tau_{máx}} \quad [1]$$

3 case studies were performed with FEM and LEM analysis, one of them is a 90 m high heap leach facility which is presented in the Figure 9a. The interfaces shear strength parameters were obtained of large-scale direct shear (LSDS) tests conducted for both OL-GM and GM-SL interfaces. The envelopes of post-peak and peak shear strength vs. normal stress are presented in Figure 9b.

a)



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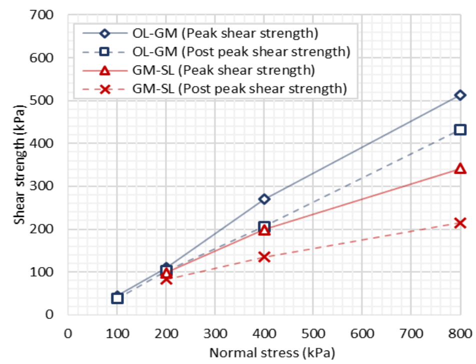

Figure 9. (a) Heap leach facility model considered in the numerical analysis with finite elements.
(b) Results of LSDS tests used for stability analysis.

The interfaces were modeled using 2D Plaxis program by interface elements, which have greater advantages compared to modeling with continuum elements. The modeling of the interface with continuum elements does not allow calculate directly stresses and strains, due to the stress and strain deformations mobilized in these continuum elements have directions that are not parallel to the interface, which is not real. Therefore, OL-GM and GM-SL interfaces should be modeled as interface elements. The analysis has been verified that the shear stresses are equal in both interfaces, however, the relative shear stresses have significant differences, as can be seen in Figure 10. It is verified that the GM-SL interface being the weakest is more susceptible to reaching the fault limit (τ_{rel}=1,0). Unprovided loads in the design or additional failure mechanisms could cause the stress to overcome the peak strength ($\tau_{máx}$) and post-peak strength is mobilized which could cause a progressive failure of the structure when shear stress is transferring to other areas of the interface.

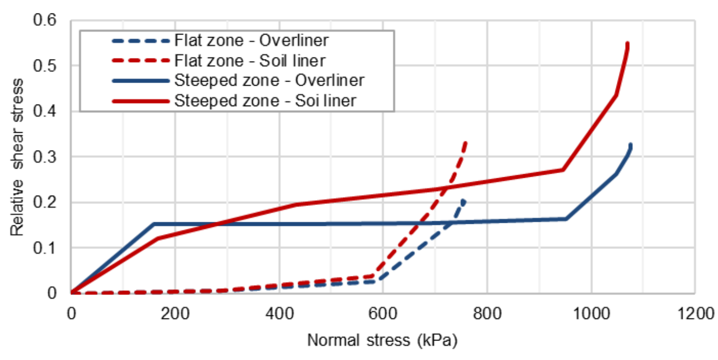

Figure 10. Relative shear stress vs. normal stress in flat zones and steeped zones of a heap leach pad.

The results of finite-element method (FEM) were compared with the results of limit equilibrium method (LEM) performed with Slide software which is the more used method in the design in contrast with other methods. The static condition (in a long term) was analyzed in both methods. The stability analyses using LEM method were performed considering block failures along both OL-GM and GM-SL interfaces and considering both peak and post-peak shear strength. The results obtained is presented in Table 5. It has been verified that the safety factors obtained in LEM method are more conservatives than more rigorous methods like FEM.

Table 5: Results obtained in slope stability analyses with both finite element method and limit equilibrium method.

	Analysis cases	**Finite element method (FEM)**	**Limit equilibrium method (LEM)**			
			Interface above geomembrane (OL-GM)		Interface under geomembrane (GM-SL)	
			Peak shear strength	Post-peak shear strength	Peak shear strength	Post-peak shear strength
Obtained factor of safety (FS)	Generic case	1.96	2.16	1.91	1.94	1.17
	Case 1	2.05	2.84	2.53	1.98	1.35
	Case 2	2.16	2.28	2.06	2.08	1.39

6. CONCLUSIONS

The GM-SL interface shear strength increases based on the granular nature of the soil liner (the more granular the soil liner), the greater the shear strength, likewise the asperity height are very important parameters for interface shear strength determination. The OL-GM interface shear strength increases based on the gravel content of over liner (the more gravel in the overliner), the greater the shear strength.

The analysis of progressive failures is an adequately approach for designing a heap leach facility. It allows to evaluate the shear stress mobilizing along both OL-GM and GM-SL interfaces. In the analysis cases, it has been verified that mobilized shear stresses are less than the peak shear strength along the interface. This analysis allows to identify the problems of the area most likely to develop post-peak efforts that require greater attention in the design.

In heap leach pads design, both OL-GM and GM-SL interfaces shear strength must be estimated with laboratory tests. It is not advisable to assume that the weak interface will be above or below the geomembrane without performing laboratory tests. Alternatively, the stability could be evaluated with a composite failure envelope, which is defined based on the lowest shear strength of both interfaces for the full range of normal stresses.

The factors of safety obtained by LEM analysis are more conservative than those obtained by more rigorous methods like FEM. The FEM analysis is a more appropriate method for stability analysis because allows us to estimate relative shear stresses along interfaces and to predict sensible zones to progressive failures.

Peak shear strength for static conditions (non-seismic) for flat zones could be used in stability analysis even in steeped zones if numerical analysis shows mobilized shear stresses are less than the peak shear strength along the weak interface.

REFERENCES

ASTM D5321. Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method, *American Society for Testing and Materials*, Pennsylvania, USA.

Jogi, M. (2005). Method for measuring smooth geomembrane / soil interface shear behavior under unsaturated condition (master thesis), *University of Saskatchewan,* Saskatton, Canada.

Ayala, R. and Huallanca, W. (2014). Interface shear strength non linearity and its effects on heap leach pad stability analysis *Proceedings of the Heap leach Solutions 2014 Conference*, Lima, Peru.

Blond E. and Elie G. (2006). Interface shear-strength properties of textured polyethylene geomembranes, viewed 15 December 2013, http://www.solmax.com/wp-content/uploads/2012/08/1_ShearStrenghtproperties-PEmembrane.pdf

Filz, G., Esterhuizen, J. and Duncan, J. (2001). Progressive Failure of Lined Waste Impoundments. *Journal of geotechnical and Geoenvironmental Engineering*, ASCE, 127, No. 10, 841-848.

GRI – GM13 (2019). Standard Specification for Test Methods, Test Properties and Testing Frequency for High Density Polyenthylene (HDPE) Smooth and Textured Geomembranes, *Geosynthetic Institute*, Pennsylvania, USA

GRI – GM17 (2019). Standard Specification for Test Methods, Test Properties and Testing Frequency for Linear Low Density Polyethylene (LLDPE) Smooth and Textured Geomembranes, *Geosynthetic Institute*, Pennsylvania, USA

Ivy N. (2003). Asperity height variability and effects, GFR Engineering Solutions, viewed 15 December 2013,, http://www.geosynthetica.net/Uploads/GFRAsperity.pdf

Jones, D. & Dixon, N. (1998). The stability of geosynthetic landfill lining systems. In Geotechnical engineering of landfills. Edited by N. Dixon, E.J. Murray and D.R.V. Jones. Thomas Telford, London, UK. pp. 99-117.

Koerner, R. and Boman, H. (2003). A recommendation to use peak shear strengths for geosynthetic interface design. *Geosynthetics Fabrics Report*.

Stark T. and Choi H. (2004), Peak versus residual interface strengths for landifll liner and cover design, *Geosynthetics International Journal*, 11(6), 491-498.

Stark, T. and Poeppel, A. (1994). Landfill liner interface strengths from torsional-ring shear tests. *American Society of Civil Engineers Journal of Geotechnical Engineering*, 120 (3): 597-617.

Yesiller N. (2005). Core Thickness and asperity height of texture geomembranes: a critical review, GFR Engineering Solutions, 23(4), viewed 15 December 2013 http://www.geosynthetica.net/resources/core-thickness-and-asperity-height-of-textured-geomembranes-a-critical-review/