Criteria to Estimate and Use Peak and Residual Shear Strength of Interfaces in Heap Leach Pads Design

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Abstract

One of the essential geotechnical aspects of heap leach pads design is related to the interface shear strength of the underlying geomembrane liner system, which isolates the leachate solution and ore from the natural ground. Conventional liner system design considers a double contention formed by a geomembrane liner (GM) over a compacted soil liner (SL) layer. The overliner (OL) and solution collection pipe systems are placed over the GM. The OL is generally granular soil that protects the GM and allows the solution drainage.

The interface shear strength between GM and SL is usually estimated through a large-scale direct shear (LSDS) test, which typically provides the peak and post-peak shear (or residual) strength parameters. The slope stability analyses are run conservatively, with the lowest interface shear strength corresponding to the residual strength. Usually, the LSDS test results show the GM-SL interfaces as having the weakest post-peak and residual shear strengths. However, this assumption could be the opposite because the recommended minimum asperity height for textured geomembrane is 0.4 mm according to GRI-GM 17 (GRI, 2021a).

The limit equilibrium method (LEM) is commonly used to evaluate the stability of a heap leach pad. In this method the interface is modeled as a continuum element (like a thin soil layer). 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 strengths are usually considered in stability analyses. However, this criterion can be very conservative in estimating the factor of safety (FS).

This paper proposes empirical formulas to estimate the GM-SL interface peak and post-peak shear strengths based on the asperity height of GM and SL classification (USSC). Also, empirical formulas for the OL-GM interface are proposed based on the gravel content of the OL. In addition, this research suggests using peak shear strength for flat zones and post-peak shear strength for stepped zones in statics slope analysis of heap leach pads.

Introduction

Among the most significant mining structures are leach piles or heaps, in which ore is dumped and irrigated with a solution that dissolves the mineral impregnated in the rocks. The solution with mineral is called the pregnant leach solution (PLS). The PLS passes through the ore in the heap via the collection and liner system located on the bottom of the heap. The liner is usually composed of clayey soil, or geosynthetic clay liner (GCL), a geomembrane (GM), granular drainage, and a protection material (overliner) that is supplemented with drainpipes at controlled spacing (Thiel and Smith, 2004).

Critical aspects of the geotechnical design of leach pads are associated with the height of the ore heap, liner system, ore saturation, site location topography, and site geology. In practice, a conservative approach is usually incorporated into slope stability analysis by using post-peak shear strength values.

Liner system design

The liner system (see Figure 1 for an illustrative example) is a containment that isolates the leaching solution from the natural terrain. The characteristics of liner systems depend on environmental regulations, site conditions, risks associated with solution leaks, operation characteristics, operating equipment, height of the heap ore, slope stability, etc. (Romo, 2015). The design of liner systems often includes a composite containment system composed of a GM over a low permeability layer (compacted clay or GCL).

In Peru, most of the leach pads are in the highlands, where underground water or lagoons are often very close to mining projects, so the current standard for liner systems design includes composite containment.

The liner system design for a leach pad depends on earthworks for grading (cut and fill), selection and characterization of geosynthetics, low permeability soil, and overliner materials. Also, the liner system design must address the operating and environmental conditions (Lupo, 2010).

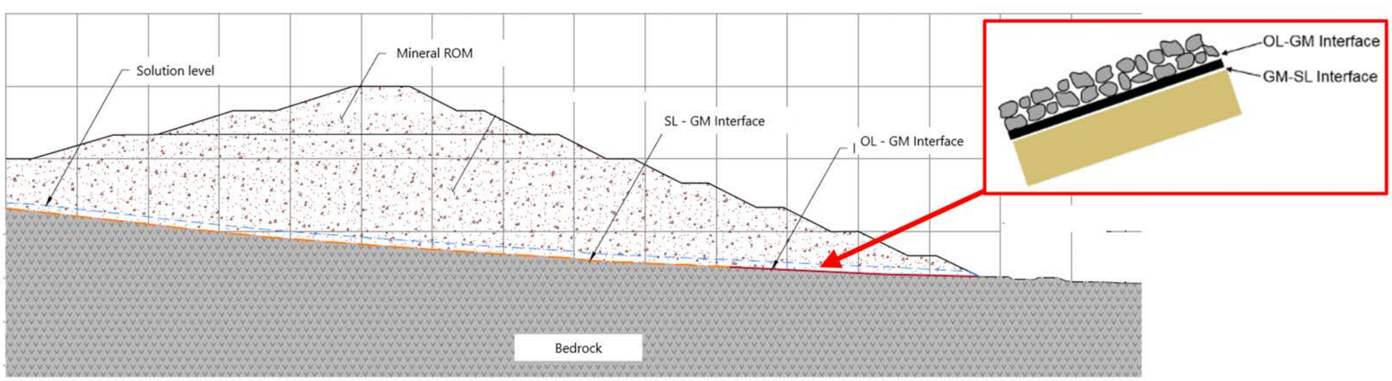


Figure 1: Heap leach pad and composite liner system

Geomembrane selection

The selection of a suitable geosynthetic liner is crucial for leach pad design because the ore is stacked over the composite liner, and the heap height is continuously increasing. The thickness of the geomembrane (GM) is generally selected based on its puncture resistance.

The extensive use of linear low density polyethylene (LLDPE) GM in leach pad operations has shown that it is suitable for containing corrosive acid drainage and metal leaching products for at least 20 years. However, there is not enough data regarding its long-term behaviour (50 to 100 years) (Renken et al., 2007). The behaviour of the GM under loaded conditions is strongly influenced by its interaction and compatibility with the components of the composite liner, so the design requires a thorough understanding of the interaction between these components, the load applied, the type of solution, and the solution level.

For mining projects, GM manufacturing is generally carried out according to the relevant Geosynthetic Research Institute (GRI) standard specifications, in addition to the recommendations of engineers and consultants specialized in heap leach facilities. GRI's standard specifications GM13 (GRI, 2021b) and GM17 (GRI, 2021a) apply to high density polyethylene (HDPE) and LLDPE geomembranes, respectively; however, some engineers may recommend more rigorous specifications.

The main restriction of the geotechnical design of heap leach pads is related to the strength of the liner system, which usually provides at least two interfaces with low shear strengths due to contact with the GM. These interfaces control the stability conditions. Some efforts have been made to adequately model and predict this kind of shear strength (e.g., Ivy, 2003; Yesiller, 2005; Blond and Elie, 2006). In previous research (Ayala and Huallanca, 2014), the authors noted that GM-SL interface shear strengths increase with GM asperity height (a), normal stress (σ_n) increment, and the increment of granular material content in the soil liner.

This paper proposes empirical formulas to estimate the shear strength of GM-SL and OL-GM interfaces.

Large-scale direct shear tests (LSDS)

The shear strength of OL-GM and GM-SL interfaces is measured by large-scale direct shear tests (LSDS) following the ASTM D5321 standard. Figure 2a shows the typical behaviour of shear strength vs. displacement. The shear stress gradually increases until the peak shear stress ($\tau_{p,1}$); it should be noted that beyond the maximum displacement of the LSDS there is no additional reduction of the shear strength, indicating that in this case, as many other observed testing results, the post-peak shear strength is the same as the residual strength. After the peak, the shear stress falls, as demonstrated in the $\tau_{p,1} - \tau_{r,1}$ path in Figure 2a. Generally, several test specimens with soil and GM are tested, varying σ_n; for each normal stress, the shear stress vs. displacement curve is obtained, as shown in Figure 2 (a). The peak or residual shear stress can be plotted against the corresponding σ_n, as shown in Figure 2 (b). This envelope could be defined by the Mohr-Coulomb model (angle of friction and adhesion); however, interface shear strength envelopes often show non-linear behaviour, as noted by Stark and Choi (2004).

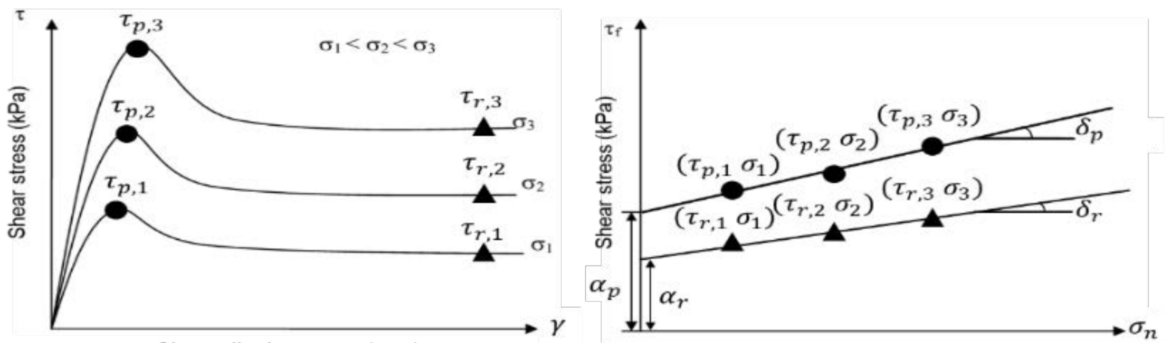


Figure 2: Typical plots from LSDS. Left (a) shear strength vs. displacement; right (b) shear vs. normal stress

Data selection

The collected data shows the typical shear stress-displacement behaviour of OL-GM and GM-SL interfaces. This research has compiled 64 LSDS tests on OL-GM interfaces with smooth GM and 388 LSDS tests on GM-SL interfaces mainly with side textured GM. Compiled LSDS tests were carried out with site-specific materials (OL and SL) of different mining projects and with GM, principally LLDPE, at each σ_n (100, 200, 400, and 800 kPa). The test data were divided into five groups based on interface type, soil classification according to the Unified System of Soil Classification (USSC), and fines content. The main features of soil samples and GM type of each group are summarized in Table 1, where the average (μ) of particle size content also is indicated.

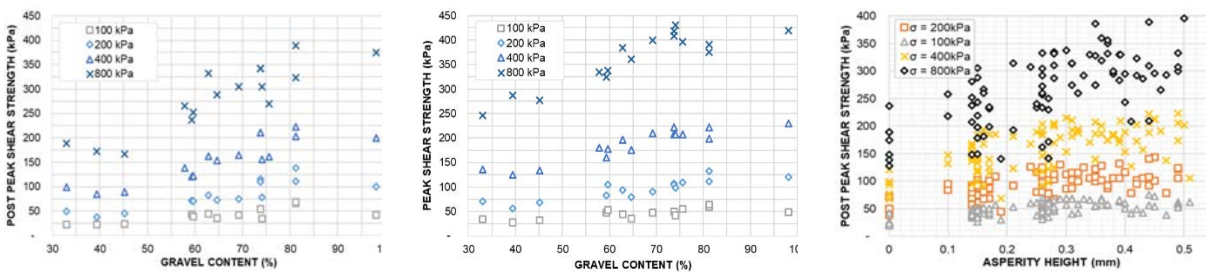
Table 1: Summary of interface types

Interface type	Interface condition	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	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 data regarding peak and post-peak shear strengths of the OL-GM interfaces are shown in Figures 3 (a) and 3 (b), respectively, separated by each σ_n (in different colours). It should be noted that in most of the cases the post-peak shear strength at approximately 70 to 80 mm displacement corresponds to the residual strength, because there is no further strength reduction after the maximum displacement of the LSDS. These figures also show the influence of gravel content of OL in their interface shear strength.

The data regarding the post-peak shear strength of the GM-SL interfaces vs. GM asperity height are shown in Figure 3 (c). This figure does not present the influence of soil classification or fines content. There is no clear tendency for peak shear strength of the GM-SL interface vs. asperity height.

The Mohr-Coulomb model (a failure envelope characterized by an angle of friction and cohesion or adhesion) as applied to OL-GM interface shear strengths is shown in Figure 4, in which three average lines are proposed for different gravel contents. To achieve a better relationship to estimate the OL-GM interface shear strength, expressions based on gravel content (G) are proposed in this paper.



(a) (b) (c)

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

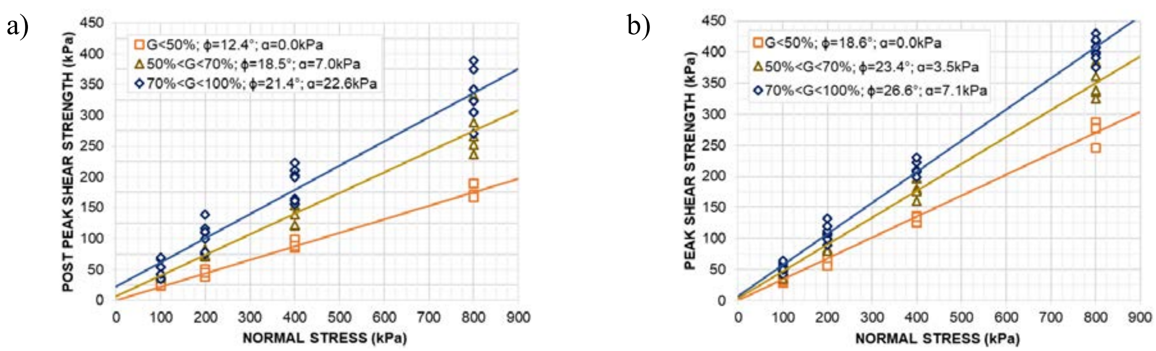


Figure 4: Average failure envelopes for different gravel content ranges:
(a) post-peak shear strength vs. normal stress; (b) peak shear strength vs. normal stress

Proposed relationships for interface shear strength

The average shear strength envelope was obtained by linear and non-linear (polynomial) regressions. The "goodness of fit" was verified with the correlation coefficient (R^2); R^2 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). The fit of the average shear strength envelopes produces R^2 values greater than 0.50, which means that empirical formulas can be defined.

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. Therefore, it is recommended that shear strength is defined with confidence levels, in which there is confidence with a probability (P) that the interface shear strength will be at least the estimated shear strength. Hence, a statistical treatment of data is proposed for envelopes with confidence levels. Probabilities associated with ± 1S (16 and 84% confidence) were considered. The "goodness of fit" test (Anderson and Darling, 1952) was applied to verify if the difference between the estimated and the observed values has a normal distribution.

OL – GM interface shear strength relationship

The shear strength relationship for the overliner-smooth geomembrane (OL-GM) interfaces for each σ_n was defined based on the influence of overliner gravel content. Regarding the data for this study, the following observations are noted: most of the data were obtained from LSDS tests on interfaces with LLDPE geomembrane; GM thickness has not been considered; most of the overliner corresponds to sub-angular and sub-rounded gravels; the maximum particle size of overliner was 50 mm; and the maximum σ_n 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. Figure 6 shows the same relationships with both lower and upper confidence boundaries for one standard deviation (S). 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 GM would be lower, which reduces the interface shear strength. The evaluation of the liner system must also consider these aspects in heap leach pad design.

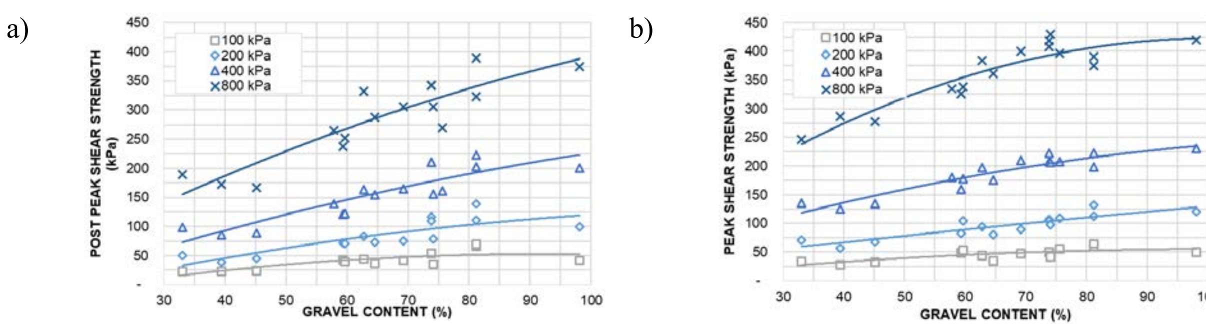
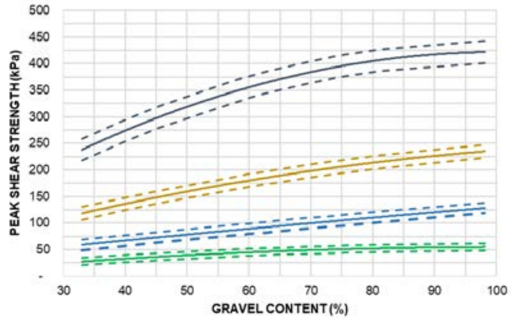


Figure 5: Proposed relationships between gravel content and (a) post-peak shear strength and (b) peak shear strength

a)



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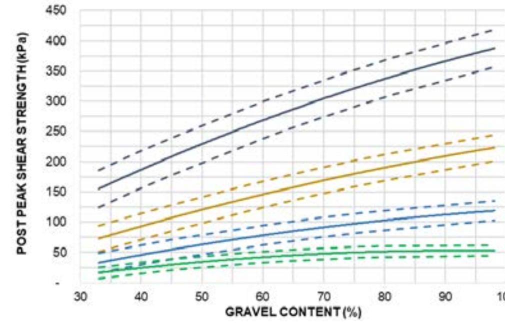

Figure 6: With confidence intervals (±1S) based on relationships between gravel content and (a) peak shear strength and (b) shear strength (for σ_n: 100, 200, 400 and 800 kPa)

OL-GM interface shear strength discussion:

There is a clear nonlinear tendency of shear strength increment for post-peak and peak values. The standard deviation of the 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 S value increases for high normal stresses, as noted in Figure 6 (b).

The proposed empirical formulas based on gravel content (G) are summarized in Table 2 for post-peak (τ_r) and peak shear (τ_p) strength for different normal stresses. These relationships could be used in stability analysis where the failure occurs at the interface. The S and R^2 values obtained for each relationship are also indicated in Table 2.

Table 2: Empirical formulas for OL-GM interface shear strength based on gravel content

σ_n (kPa)	τ_r (kPa)	S (kPa)	R^2	τ_p (kPa)	S (kPa)	R^2
100	τ_r = -0.01G^2 + 1.92G – 35.85	9.1	0.61	τ_p = -0.01G2 + 1.17G – 4.60	6.7	0.58
200	τ_r = -0.01G^2 + 2.57G – 41.70	16.3	0.69	τ_p = -0.001G2 + 1.23G + 20.05	10.0	0.77
400	τ_r= -0.01G ^{2} + 3.68G – 37.16	21.6	0.77	τ_p = -0.01G2 + 3.48G + 17.57	12.0	0.87
800	τ_r = -0.02 G^2 + 5.72G – 15.82	30.6	0.80	τ_p = -0.04G2 + 8.07G + 15.55	20.4	0.87

GM – SL interface shear strength relationship

The empirical formula for textured geomembrane–soil liner (GM-SL) interface was obtained by the influence of asperity height and the type of soil; like the OL-GM relationship, these results could be used to run a stability analysis of heap leach pads for block failure, or maybe the combination of both formulas.

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 geomembrane; geomembrane thickness has not been considered; and the maximum σ_n was 800 kPa for all tests and 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 for different soil types.

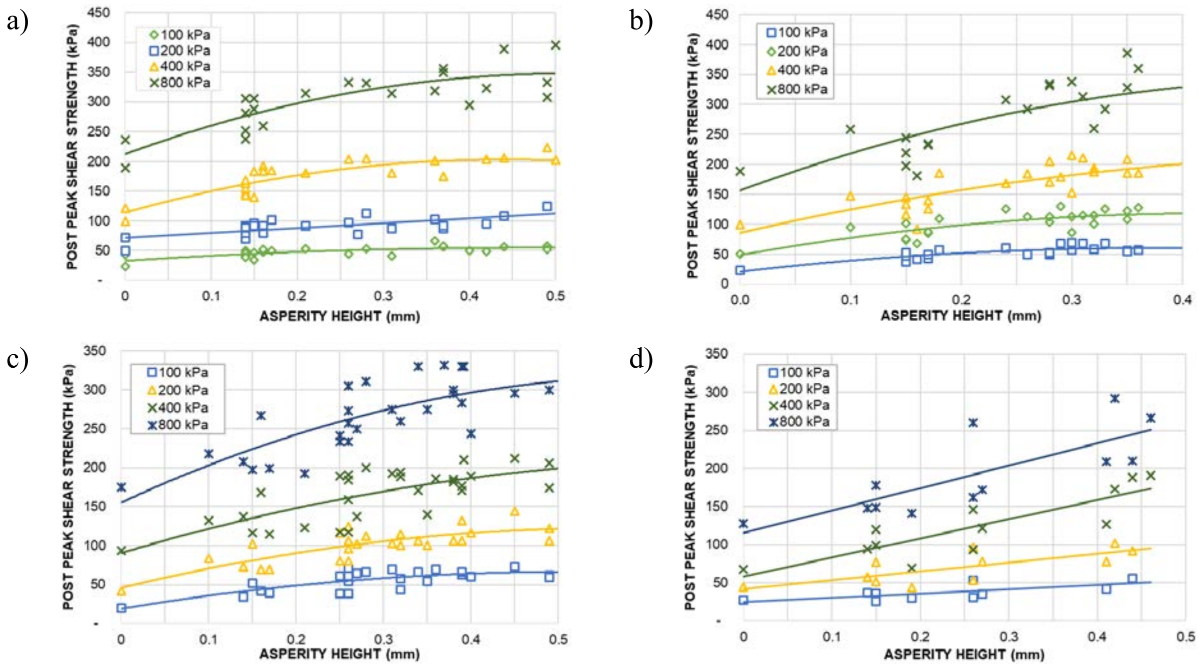

Figure 7: Relationship between asperity height and interface post-peak shear strength for the following soil types: (a) GC; (b) SC; (c) CL (F < 65 %) and (d) CL (F > 65 %)

Figure 8 shows relationships with lower and upper boundaries considering ±1S. The influence of gravel content is noted in shear strength; it can be for friction generated between gravel particles and GM.

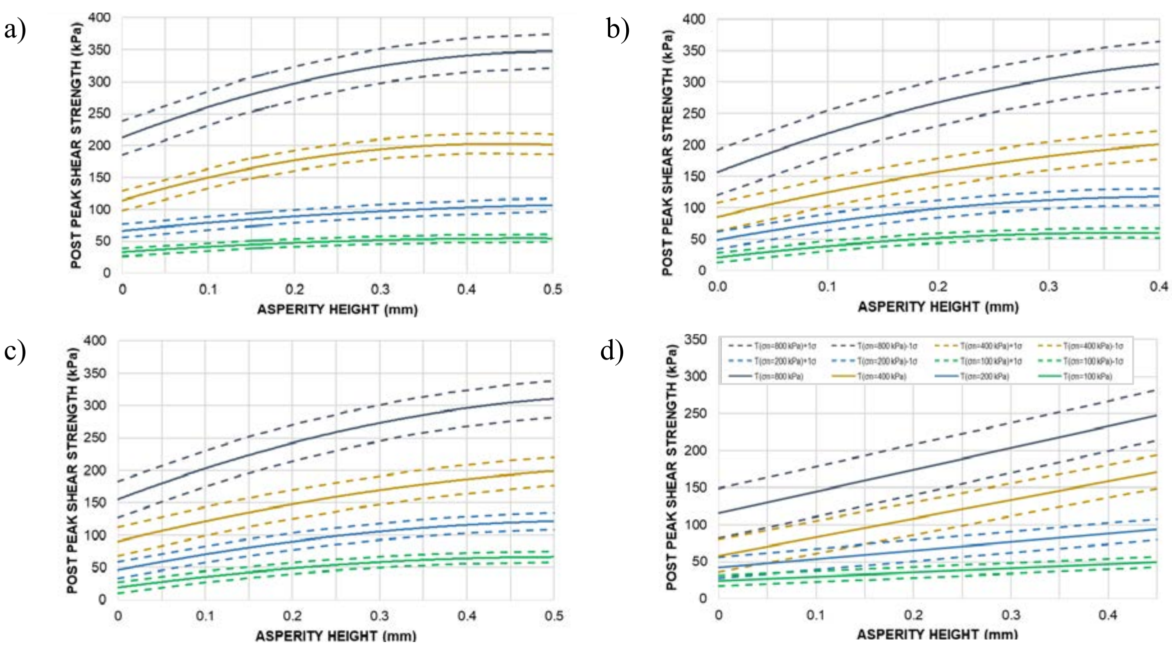

Figure 8: Graphs with confidence intervals of ±1S based on relationships between asperity height and interface post-peak shear strength for the following soil types: (a) GC; (b) SC; (c) CL (F < 65 %) and (d) CL (F > 65 %)

GM-SL interface shear strength discussion:

There is a clear nonlinear tendency of shear strength increment for post-peak and peak values, where the behaviour tends to be asymptotic at 0.4 mm asperity height. Empirical formulas for peak shear strength were not defined due to low R^2 values and high standard deviations. The standard deviation of post-peak shear strength increases for higher normal stresses for all GM-SL types, as noted in Figure 8. The proposed empirical formulas are summarized in Table 3 for post-peak shear strength based on asperity height and considering different normal stresses. The S and R^2 values estimated also are indicated.

Table 3: Empirical formulas for GM-SL interface post-peak shear strength based on asperity height

Interface	σ_n (kPa)	τ_r (kPa)	S (kPa)	R^2
GM-SL (CL, F < 65 %)	100	τ_r = -179.3a² + 184.39 + 19.02	8.5	0.59
	200	τ_r = -234.42a² + 269.02 + 45.88	12.9	0.65
	400	τ_r = -232.17a² + 333.15a + 90.5	22.2	0.55
	800	τ_r = -414.45a² + 518.37a + 155.42	28.0	0.61
GM-SL (CL, F > 65 %)	100	τ_r = 55.789a + 24.634	7.2	0.51
	200	τ_r = 115a + 42.195	14.3	0.55
	400	τ_r = 252.07a + 57.99	22.2	0.73
	800	τ_r = 294.52a + 115.57	33.9	0.61
GM-SL (SC)	100	τ_r = -287.26a² + 212.09a + 20.82	7.6	0.56
	200	τ_r = -383.19a² + 326.66a + 48.43	13.6	0.58
	400	τ_r = -344.73a² + 425.5a + 85.54	22.4	0.62
	800	τ_r = -623.37a² + 680.41a + 156.34	36.1	0.59
GM-SL (GC)	100	τ_r = -100.57a² + 94.059a + 32.85	6.2	0.51
	200	τ_r = -103.03a² + 132.17a + 66.74	10.4	0.52
	400	τ_r = -459.25a² + 405.22a + 114.05	15.5	0.73
	800	τ_r = -517.56a² + 528.9a + 212.46	26.6	0.70

Influence of the use of peak or post-peak shear strengths

Application of interface shear strength

Gilbert and Byrne (1996) indicated that the peak shear strength mobilizes between 1 and 15 mm of displacement; also, the post-peak shear strength can be as low as 30% of the peak strength. About these findings, the current investigation found the following:

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

It is logical to use post-peak shear strength of the interface in the liner system design because progressive failures across the liner mobilize the shear stress. The suggestions of different authors for the use of peak or post-peak shear strength in the design are summarized below, listed from the most to the least conservative:

- Use residual post-peak shear strength for all conditions (Stark and Poeppel, 1994).
- Use post-peak shear strength of the interface with the lowest peak shear strength (Koerner and Bowman, 2003).
- Use 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 peak strength at the top of the slope and post-peak strength at the base (Koerner and Bowman, 2003).
- Use peak strength for all static (non-seismic) conditions (Koerner and Bowman, 2003).

This paper evaluates the use of peak shear strength for flat zones and post-peak shear strength for stepped zones of heap leach pads, and the findings compare with the results of numerical analysis.

Stability analyses of heap leach facilities

The limit equilibrium method (LEM) is commonly used to evaluate the stability of slopes by calculating a representative factor of safety (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 may produce conservative estimates of the FS when post peak shear strength is used.

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

$$\tau_{rel} = \frac{\tau_{mob}}{\tau_{máx}} \qquad \text{(Equation 1)}$$

Three case studies were performed using finite element method (FEM) and LEM analyses. One of the cases considers a 90-m deep heap, which is presented in Figure 9 (a). The interface shear strength parameters were obtained from 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 9 (b).

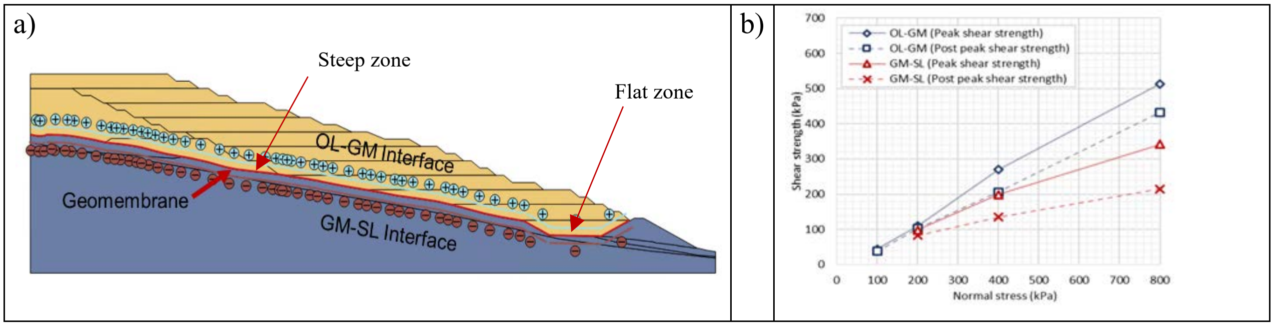

Figure 9: (a) Heap leach facility model considered in the numerical analysis with finite elements. (b) Results of LSDS tests used for stability analysis

To obtain more accurate FS, the FEM analysis were performed with the 2-D Plaxis program. The interfaces of the heap leach liner (OL-GM and SL-GM) were modeled with interface elements using the shear strength envelope calibrated with the peak shear strength. The FEM analyses verified that the shear stresses are equal in both interfaces. However, the relative shear stresses have significant differences, as seen in Figure 10. The FEM results demonstrate that interfaces at step zones are particularly more susceptible to reaching the fault limit (τ_{rel}=1,0) but the peak shear strength is not mobilized. Additional failure mechanisms could cause stresses to overcome the peak strength ($\tau_{máx}$), mobilizing post-peak strengths and causing a progressive failure of the structure.

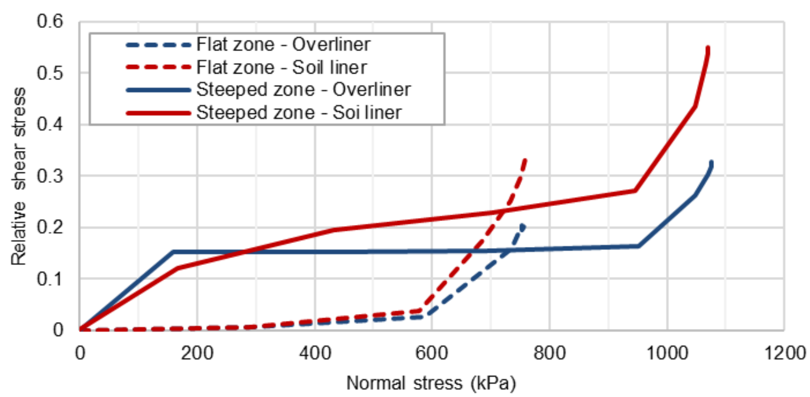

Figure 10: Relative shear stress vs. normal stress in flat and steep zones of a heap leach pad

The stability analyses using the LEM method were performed for four scenarios, considering block failures along OL-GM and GM-SL interfaces; peak and post-peak shear strength were considered in each interface. The interface has been modeled as a continuum element.

Table 5 shows that the FS obtained by the FEM is different from the FS obtained by LEM in each scenario due to different shear strength values. Likewise, post-peak GM-SL scenario trends to be more conservative that FEM results because FEM consider both interfaces (OL-GM and GM-SL) in the model, which estimate the shear strength mobilization along to the interfaces. The analysis shows that peak shear strength is not reached to mobilize it, thus the use of the post peak shear strength along to the interfaces (flat and steep zone) could be overconservative.

Table 5: FS results with FEM and LEM

	Analysis cases	Peak shear strength			Post peak shear strength	
		FEM	**Interface GM-SL** τ_p	**Interface OL-GM** τ_p	**Interface GM-SL** τ_r	**Interface OL-GM** τ_r
Factor of safety	Generic case	1.96	1.94	2.16	1.17	1.91
	Case 1	2.05	1.98	2.84	1.35	2.53
	Case 2	2.16	2.08	2.28	1.39	2.06

Conclusions

The GM-SL interface shear strength increases based on the granular nature of the soil liner, and the asperity height is a crucial parameter for interface shear strength determination. The OL-GM interface shear strength increases based on the gravel content of the overliner with little influence by asperity height.

Analyzing progressive failures is a good approach for designing heap leach facilities. It allows for evaluating the shear stresses mobilized along both OL-GM and GM-SL interfaces. The cases analyzed show that mobilized shear stresses are less than the peak shear strength along the interface. This analysis allows the identification of the problems in the interface most likely to mobilize post-peak strength, which require greater attention in the design.

In heap leach pad design, both OL-GM and GM-SL interface shear strengths must be estimated based on laboratory tests. It is not advisable to assume that the weak interface will be above or below the GM 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 FS obtained by LEM are more conservative than those obtained by FEM. The FEM analysis is more rigorous for stability analysis because it allows the estimation of the relative shear stresses along interfaces (under and above the liner system) and predicts zones of possible progressive failures.

Peak shear strength for static conditions for flat zones could be used in stability analyses if numerical analysis shows mobilized shear stresses are less than the peak shear strength, as long as there are data and models of higher confidence to support the FEM analysis.

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