Mine Waste Storage Facility Liner Design and Testing – A Case Study from the Cerro Corona Mine

Raquel Borja, Stantec, Peru
Jorge Dueñas, Gold Fields La Cima, Peru
Richard Ordonez, Gold Fields La Cima, Peru
Josh Rogers, Stantec, Canada

Abstract

This paper presents a case study of Waste Storage Facilities (WSFs), which include a composite liner system placed in a shingle configuration to manage seepage and mitigate stability impacts. The composite liner system consists of a textured high-density polyethylene (HDPE) geomembrane with an underlying clay layer and overlying sand layer. Engineering literature includes limited information on geomembrane interface shear strengths under high normal loads. To support the WSFs designs, a liner interface shear strength testing program was performed using the direct shear apparatus. The program included testing to verify the interface shear strength parameters selected for design, evaluating interface shear strengths at high normal loads (up to 3,200 kPa), and evaluating the impacts of the various testing conditions on interface shear strength. This paper presents the results of the laboratory testing, a discussion on the impacts of loading conditions on interface shear strength, and a discussion on various regression techniques used to verify residual shear strengths.

Introduction

Modern mining operations include structures designed to store mine waste materials while meeting high engineering and environmental standards. These include tailings storage facilities (TSFs), which are designed to store tailings produced from mine processing, and waste storage facilities (WSFs), which are designed to store mine waste such as waste rock that is typically produced from open pit mining.

Gold Fields La Cima S.A. (GFLC) owns and operates the Cerro Corona mine, a copper mine with significant copper and gold content located in the Cajamarca district of Peru. Mine waste is stored in WSFs at the site. These WSFs include a composite liner system placed in a shingle configuration to manage seepage and allow collection of mine contact water while mitigating stability impacts. The Cerro Corona WSF composite liner system consists of a textured high-density polyethylene (HDPE) geomembrane with an underlying clay layer and overlying sand layer as a drainage and liner protection layer. Both the underlying clay and the overlying sand have a 0.3 m perpendicular design thickness. The liner uses a shingle configuration to mitigate the relatively steep slopes at the site by separating the relatively low strength liner interface surface into shorter segments. Figure 1 presents a typical section of a WSF at the site, demonstrating the shingle liner concept.

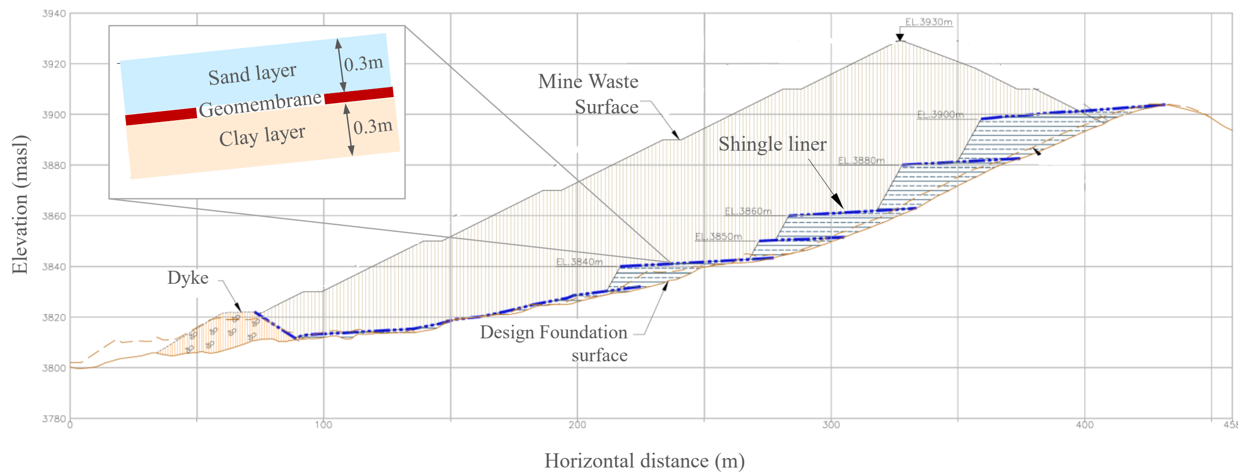

Figure 1: Typical schematic section of a WSF at the site, showing the shingle liner concept

A key component of the WSF stability is the geomembrane interface shear strength. Accurately characterizing the shear strength of this interface is critical to verify the stability of the WSFs and requires selecting appropriate strength parameters for both the peak and residual shear strengths between the textured geomembrane liner and adjacent clay and sand materials.

Literature and laboratory testing indicate that geomembrane liners placed on cohesive soil may achieve peak strengths at relatively small displacements (Bacas et al., 2015). However, it is anticipated that uncompacted mine waste materials, such as those stored in the WSFs, would reach peak strengths at significantly higher strain levels. This is described as strain incompatibility and can result in progressive failure along the liner interface where the liner interface has reached residual strength conditions. The potential for progressive failure increases as (i) the mine waste stiffness decreases relative to the initial stiffness of the liner interface; (ii) the length of the slip surface increases; and (iii) the rate of strain softening with displacement increases (Gilbert and Byrne, 1996).

Case studies of geomembrane lined landfill failures have demonstrated that progressive failure is an important mechanism, and the use of residual strength parameters can approximate the failure surface (Filz et al., 2001). Many authors (Thiel, 2001; Gilbert and Byrne, 1996) recommend the inclusion of an analysis case that characterizes the liner interface strength using residual interface strengths. Incremental placement of mine waste in the WSFs above the liner surface, which is commonly performed in the filling of WSFs, may encourage progressive failure and/or directly result in deformations that exceed the peak strength of the geomembrane interface resulting in residual strengths.

The normal loads acting on the liner of Cerro Corona WSFs range from 50 kPa for initial stages of WSF construction to 3,200 kPa under the maximum design heights. The selection of the appropriate peak and residual strengths for design should be based on considerations of the expected normal stresses applied on the liner system. However, the information included in engineering literature on geomembrane interface shear strengths under high normal loads are limited. At high normal loads, failure transitions from sliding of soil particles along the geomembrane to soil particles becoming embedded into the geomembrane and removing geomembrane texture, which results in a plowing effect (where soil particles are pushed into the geomembrane and pulled through it) and a subsequent increase in interface shear strength (Fleming et al., 2006).

For the preliminary designs of the WSFs at the site, peak friction angles and residual friction angles were selected based on engineering literature. A subsequent liner interface shear strength testing program was implemented to support the WSF design. The testing was performed using the direct shear apparatus (DSA) to test the interfaces between the clay and adjacent textured HDPE geomembrane (geomembrane/clay interface), and the interface between the sand and adjacent textured HDPE geomembrane interface (geomembrane/sand interface). These selected values are presented in Table 1.

Table 1: Design interface shear strength friction angles

Interface	Peak friction angle (°)	Residual friction angle (°)
Geomembrane/Sand	26	19
Geomembrane/Clay	23	16

As a part of the testing program, additional testing was performed using DSA to validate the design interface shear strength friction angles for both the geomembrane/clay and the geomembrane/sand interfaces. This testing was part of an ongoing testing program to verify the design shear strength of the interfaces. The program was designed to evaluate the interface shear strength at high normal loads and included testing, with various loading conditions and test procedures, the impacts of lateral loading rates, saturation conditions at the interface, normal load seating time, and variability inherent in material samples and testing. The objective of including these varied testing loading conditions was to better understand which factors most significantly impact interface shear strengths. This information was then considered when selecting interface shear strength parameters for design.

Laboratory testing program

Materials

The textured HDPE geomembrane used in the WSF and obtained for testing have a nominal thickness of 2.0 mm and are double textured (i.e., textured on both sides). The clay and sand samples tested were obtained from the material stockpiles at the site. The sand material used in interface shear testing was compacted to the average dry density and moisture content measured during Construction Quality Assurance (CQA) testing at the site. The average dry density for the sand material was 1.7 g/cm^3 and the average moisture content was 7%. For the clay samples, material coarser than 25 mm was removed before interface testing began, and then remolded to a dry density of 2.1 g/cm^3 at a moisture content of 10%.

Testing conditions

A total of nine interface shear tests were performed under various testing conditions which varied the normal stress, the saturation of the specimen, the consolidation time, and the shearing rate, as shown in Table 2. The interface shear tests were conducted at TRI Laboratories (TRI) in Austin, Texas using the direct shear apparatus (DSA) dimensions 305 mm (width) × 305 mm (length) × 102 mm (height). The geomembrane/sand interface and the geomembrane/clay interface were tested separately in the DSA. The geomembrane was placed in the lower shear box and the soil (either clay or sand material) was placed in the upper box. A total of four interface shear tests were performed on each interface, at normal loads of 50, 500, 1,000 and 3,200 kPa. The normal load of 3,200 kPa was the maximum allowable normal load for the test, which was limited by the shearing capacity of the equipment.

The following variations in testing procedures were incorporated into the testing program to evaluate the impact of variations in loading conditions on the interface shear strengths:

- Wet and dry conditions – Tests were performed under "dry" conditions, where the specimen was sheared at the remolded water content with no additional water added, and under "wet" conditions where the specimen was saturated by fully immersing it in water before shearing.
- Shearing Rate – Testing was performed under varying lateral loading (i.e., shearing) rates to evaluate the impact on the interface shear strength.
- Consolidation times – Prior to shearing, the specimens were seated in the DSA and loaded at the defined vertical load for a set time period (either 15 minutes or 16 hours) prior to shearing.

These conditions (wet/dry, shearing rate and consolidation time) were evaluated as engineering literature indicates that there can be notable variability in interface shear strength results due to variations in these conditions (Dixon, 2010). Additionally, the repeatability of the interface shear strengths was evaluated by performing a "duplicate" test on the same geomembrane/sand interface under consistent loading conditions (T1.a and T1.b).

Table 2: Summary of interface testing conditions

Test N°	Shearing rate	Condition	Consolidation time following the completion of loading
Interface shear: Geomembrane/Overliner Sand			
T1a	0.004 ipm (0.1 mm/min)	Dry	16 hrs
T1b	0.004 ipm (0.1 mm/min)	Dry	16 hrs
T1c	0.04 ipm (1 mm/min)	Dry	16 hrs
Interface shear: Geomembrane/Underliner Clay			
T2.a	0.2 ipm (5 mm/min)	Dry	16 hrs
T2.b	0.004 ipm (0.1 mm/min)	Dry	16 hrs
T3.a	0.2 ipm (5 mm/min)	Wet/Flooded	16 hrs
T3.b	0.004 ipm (0.1 mm/min)	Wet/Flooded	16 hrs
T4.a	0.2 ipm (5 mm/min)	Wet/Flooded	15 min
T4.b	0.004 ipm (0.1 mm/min)	Wet/Flooded	15 min

Test results and discussion

The peak and large displacement interface shear strengths from the DSA for the geomembrane/sand interface and the geomembrane/clay interface are presented in Figure 2. Peak and large displacement friction angles were estimated for each set of tests using the linear regression method and assuming no adhesion, as presented in Table 3. Adhesion is anticipated to be zero for the geomembrane/sand interface and was conservatively assumed to be zero for the geomembrane/clay interface (Markou and Evangelou, 2018). These friction angles were estimated for initial comparison purposes to better understand the effect of the varying loading conditions and procedures used in the testing program.

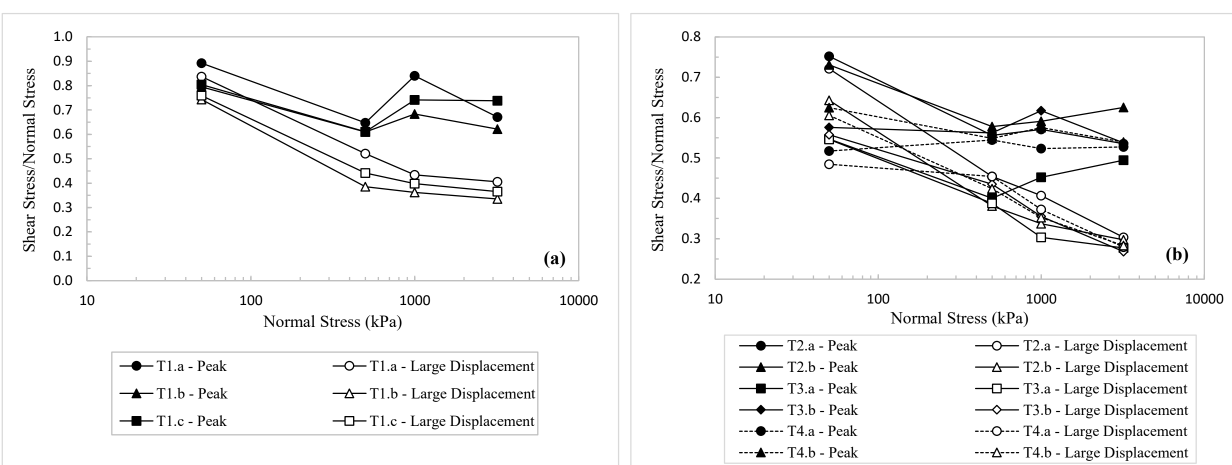

Figure 2: Test results for geomembrane/sand interface (a), and geomembrane/clay interface (b)

Table 3: Estimated peak and large displacement friction angle from each test

Test No.	Interface	Calculated peak friction angle (°)	Calculated large displacement friction angle (°)
T1.a	Geomembrane/Sand	33.9	22.1
T1.b	Geomembrane/Sand	31.9	18.5
T1.c	Geomembrane/Sand	36.4	20.1
T2.a	Geomembrane/Clay	28.2	16.9
T2.b	Geomembrane/Clay	31.0	16.6
T3.a	Geomembrane/Clay	26.3	15.5
T3.b	Geomembrane/Clay	28.3	15.0
T4.a	Geomembrane/Clay	27.8	15.6
T4.b	Geomembrane/Clay	28.4	15.8

Key observations from the test results are as follows:

- The peak and residual interface shear strength results from the geomembrane/clay interface are lower than those from the geomembrane/sand interface.
- At high normal loads, the peak shear strengths are significantly larger than the large displacement shear strengths.
- The difference in interface shear strength between the two duplicate tests (T1.a and T1.b) is about 2°. This provides some measure of the impact of variations in soil and geosynthetic samples.
- In general, the "dry" interface strengths are greater than the "wet" interface strengths (approximately 2 to 3° for peak strengths and 1.5° for large displacement strengths based on a comparison of T2 and T3) which is consistent with literature (Ivšić et al., 2005). Varying the saturation conditions of the specimens appears to have the most significant impact on the interface strength of all of the variations considered in the testing program.
- Based on a comparison of T3 and T4, the peak and large displacement interface shear strengths are slightly higher (1.5° or less) for tests performed with a more rapid consolidation time (15 minutes versus 16 hours).
- For the geomembrane/clay interface, slower shearing rates result in higher peak shear strengths (3° or less) and lower large displacement shear strength (less than 1°). However, past research (Triplett and Fox, 2001) based on DSA testing performed on Geomembrane/GCL interface, indicates that shearing rate has a limited influence on the interface shear strength. This occurs because our interpreted peak strengths are governed by the last data point at a normal stress of 3,200 kPa, which

is at higher levels than typical reported tests. The high stress could be a potential factor that contribute to the unusual dependency on the shearing rate.

- The testing includes a testing at a maximum normal stress of 3,200 kPa, which is notably higher than that included in the reviewed literature. This may influence comparisons between the test results and literature.

Figure 3 and Figure 4 present the peak and residual interface shear strength friction angles previously selected for design along with the results of DSA presented above for the geomembrane/clay and geomembrane/clay interfaces, respectively.

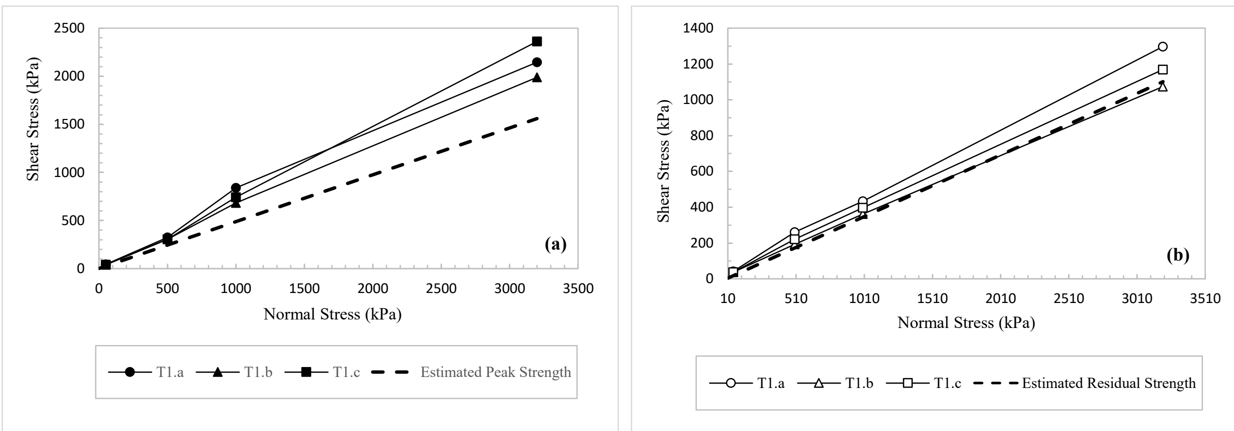

Figure 3: Peak shear strength (a) and residual shear strength (b) for geomembrane/sand interface

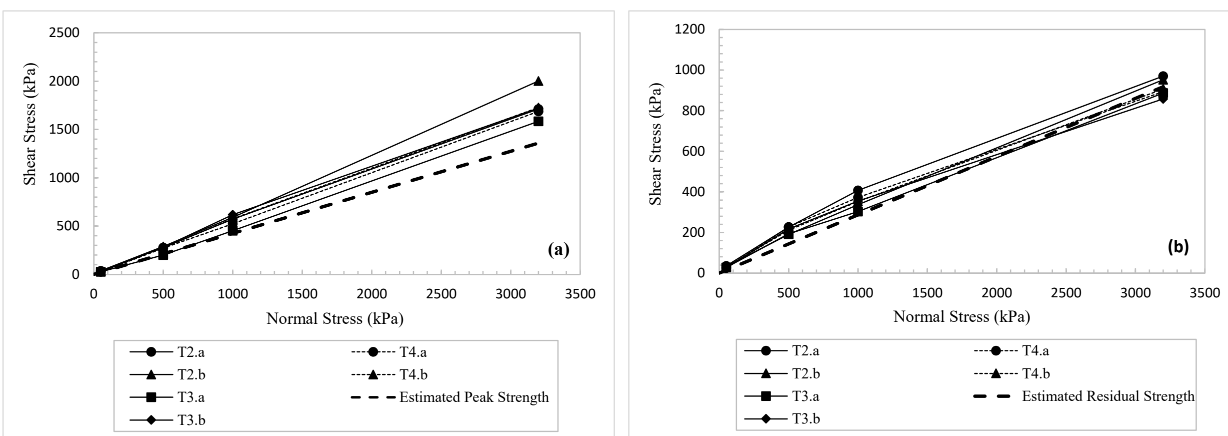

Figure 4: Peak shear strength (a) and residual shear strength (b) for geomembrane/clay interface

As presented in the figures above, the design interface shear strength friction angles are generally along the lower bound of the newly obtained interface shear strength test data. This supports the use of the values selected for design.

Residual strength verification using regression techniques

The DSA measures the shear strength at a large displacement which is not necessarily the “true” residual interface shear strength. In this testing program, a maximum displacement of about 3 inches was achieved for most of the tests and a number of the stress-displacement curves ended before reaching the 3-inch displacement when the testing operators observed that the strength became independent of the displacement. It is reported in literature that the ring shear apparatus (RSA) can be more representative of residual strength conditions than the DSA due to the ability to achieve much larger displacements with the RSA (Dixon, 2010). However, RSA testing equipment remains relatively rare in commercial laboratories and is generally limited to universities and RSA results are not commonly used in engineering practice. Thus, the DSA was utilized to obtain peak shear strengths and to obtain shear strength data at the largest displacements achievable in the DSA.

To evaluate if the large displacement test results from the DSA were representative of residual strengths, Stantec applied two plotting techniques to interpret the stress versus displacement data: the semi-logarithmic method and the inverse of the square root method.

Semi-logarithmic

Literature discusses the use of semi-logarithmic plotting to evaluate if residual clay shear strengths are reached during direct shear testing (LaGatta, 1970). In this method, the ratio between the shear and normal stresses were plotted against the logarithm of the displacement and residual strength conditions (i.e., the condition where shear strengths have reached a near steady state) will plot as an approximately flat line. The plots of the test results using the semi-logarithmic method are presented in Figure 5 and Figure 6. As presented in the figures, the test results at 50 kPa achieve a relatively flat slope at approximately 15 to 30 mm of displacement (indicating that residual strength conditions may have been reached) while the test results at higher normal stresses (500, 1,000 and 3,200 kPa) indicate decreasing slopes at higher displacements but remain with a slightly negative slope up to approximately 80 mm of displacement. These results are interpreted to indicate that the DSA testing reached residual strength conditions at 50 kPa and was nearing residual strength conditions at the end of testing for higher normal stresses.

Among the four normal stress levels, the end of test stress ratios was lowest for tests under a normal stress of 3,200 kPa. This indicates that a linear approximation of the residual strength parameter would likely be governed by the strength under the high normal stress condition. Thus, the inverse square root method described below was used only for the normal stress of 3,200 kPa.

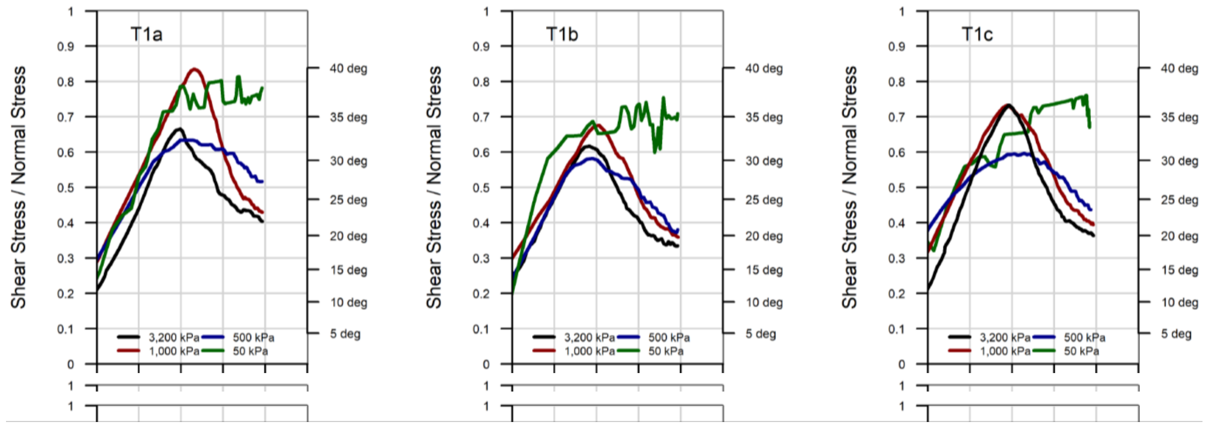

Figure 5: Semi-logarithmic plots of test results for geomembrane/sand interface

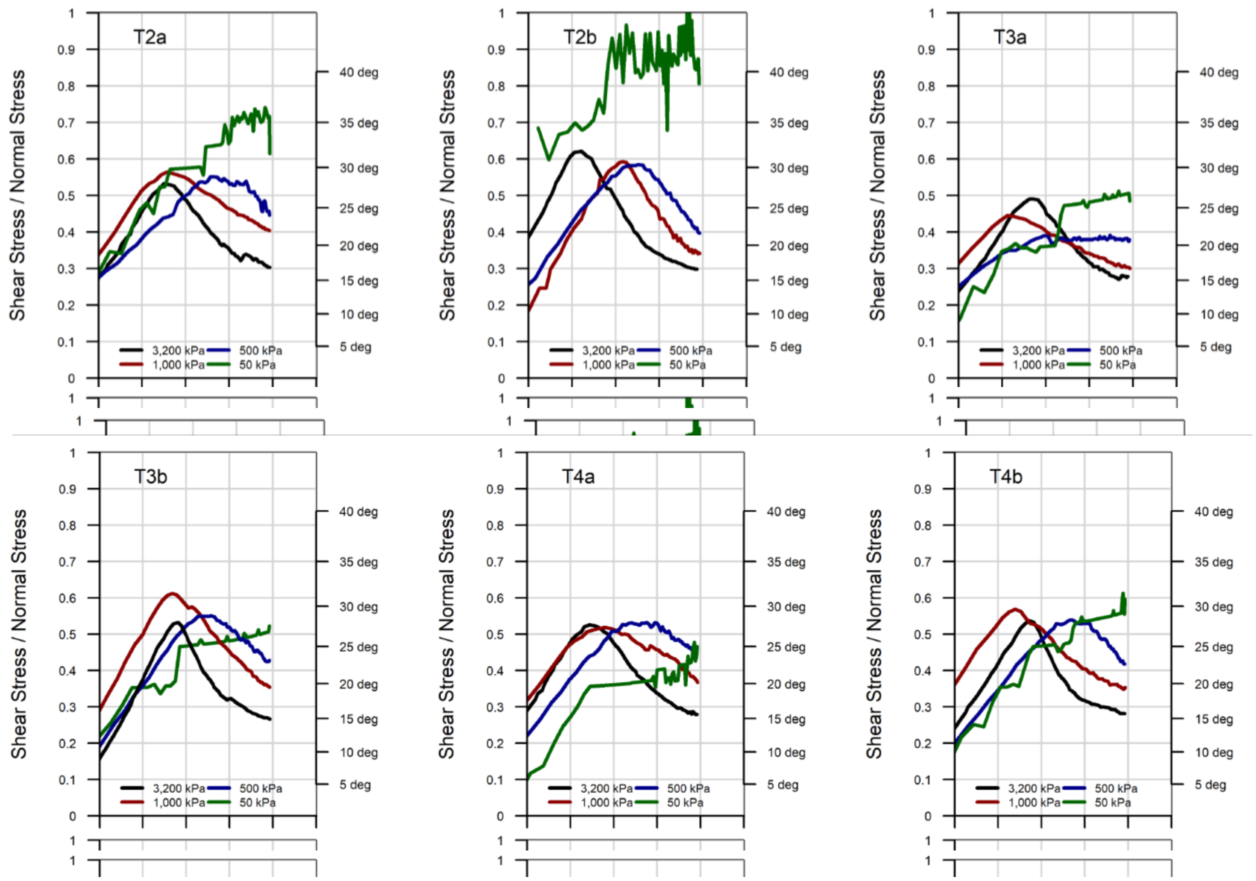

Figure 6: Semi-logarithmic plots of test results for geomembrane/clay interface

Inverse square root

The inverse square root method (Liu et al., 2014) was used to estimate the large displacement shear strength. This approach was proposed to determine the equilibrium pore pressures from incomplete cone penetration test pore pressure dissipation testing by extrapolating the last segment of measured pore pressure versus the inverse square root of time scale and is based on data from piezocone penetration test (CPTU). This method proposes to use the last segment of measured data that typically represents a linear relationship to extrapolate the incomplete data. In the authors' experience, these approaches are not commonly used in engineering practice for geosynthetic interface testing. However, they were applied in this case as another approach to evaluate if residual interface shear strengths had been obtained with the DSA.

The plots of the test results using the inverse of square-root method are presented in Figure 7 and Figure 8. The gray shared area presented in the figures was selected as the last data segment for extrapolation purposes. The end portion was selected as the final 2.5 mm to 8.0 mm of displacement in the test or visually selected to obtain the general trend of the behaviour near the end of the test while accounting for noise in the data. The "end portion" data points were then used to perform linear regression and extrapolate the data to a displacement of 600 mm. The displacement of 600 mm was selected as Stark and Poeppel (1994) suggests that a shear displacement of 400 to 600 mm is typically required to mobilize a residual interface strength in RSA tests.

For the geomembrane/sand interface, the extrapolated residual friction angles at 600 mm of displacement (14.5° to 18.5°) are significantly lower than the large displacement interface shear strengths obtained from the DSA (18.5° to 22.1°) and are lower than most of the values obtained from the extrapolations performed on the data from the geomembrane/clay interface. This indicates that the displacements achieved with the DSA used for this interface may not be sufficient to achieve residual strength conditions. This also indicates that the residual strengths along the geomembrane/sand interface are lower than that of the geomembrane/clay interface.

For the geomembrane/clay interface, the extrapolated residual friction angles from inverse square root of time plotting range from 15.1° to 16.6° (with one value at 12.8°). In general, the extrapolated lines are relatively flat, indicating that the large displacement DSA results may approximate residual strength conditions. The residual friction angle from Test T3.b (12.8°) was significantly lower than the other five tests, and may indicate test variability. However, overall, these results support the design residual shear strength (16°) selected from DSA testing.

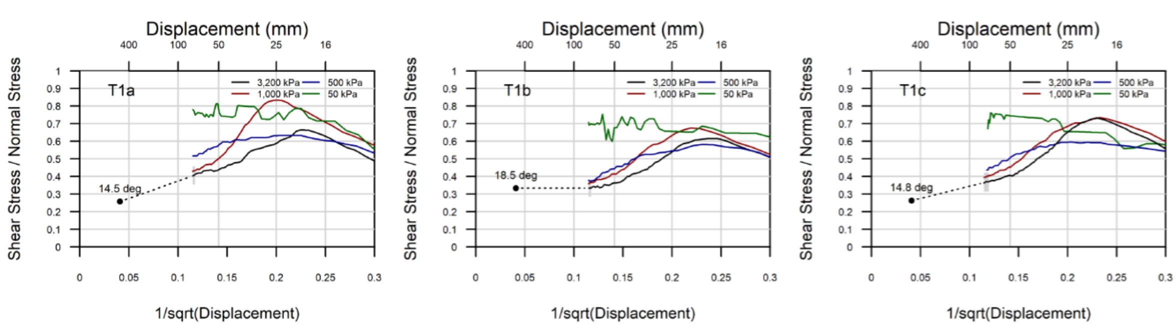

Figure 7: Inverse of square-root plots of test results for geomembrane/sand interface

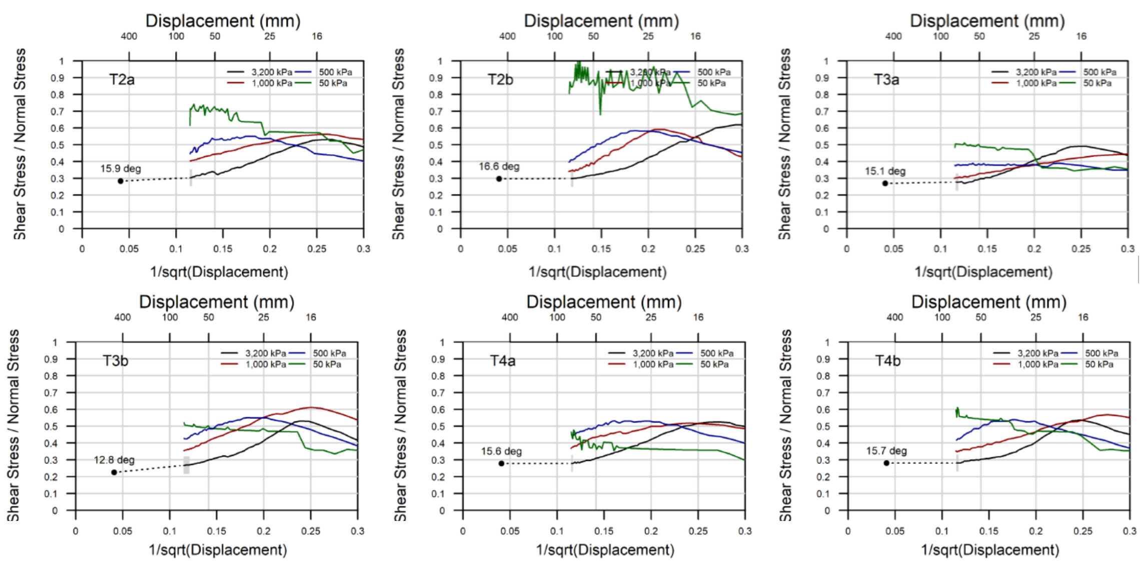

Figure 8: Inverse of square-root plots of test results for geomembrane/clay interface

Conclusions and recommendations

Generally, the interface shear strength along the geomembrane/clay interface is lower than the geomembrane/sand interface at a given level of displacement. This indicates that the clay/geomembrane interface is the critical interface and that the largest deformations would occur along this interface. Thus, the peak and residual strengths for the geomembrane/clay interface were selected to characterize the textured HDPE geomembrane interface shear strength for the design of the Cerro Corona WSFs.

For the geomembrane/clay interface, the saturation conditions of the specimen have the most significant impact on the interface shear strength of all the variations in loading conditions considered in the testing program. The testing also indicates that slower shearing rates result in higher peak shear strengths, which varies from literature, and that consolidation time (seating time prior to lateral loading) has a relatively minor impact on the interface shear strength.

The inverse square root method indicates that sand/geomembrane interface testing using the DSA may not have achieved residual strength conditions and the large displacement interface strengths may overestimate the actual residual strengths for this interface. However, this plotting method indicates that the DSA testing may have approximated residual strength conditions for the clay/geomembrane interface, and the large displacement DSA results may be reasonable estimates of residual strengths.

The design interface shear strength friction angles are generally along the lower bound of the newly obtained interface shear strength test data, which supports the values selected for design.

Overall, the testing results provide additional confidence in the interface shear strength parameters selected for design by including measurements of the interface shear strength at high normal loads and providing an understanding of the impact of variations in loading conditions on interface shear strength.

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