

# A Case Study: Shear Interface Testing of a Constructed Geosynthetic Barrier

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**Abstract.** Since geosynthetic interfaces often serve as a weak plane on which sliding may occur, shear strengths of these interfaces need to be carefully assessed. In this case study, geosynthetic and soil samples were exhumed from the base and side slope of a constructed facility to generate site specific data. The importance of using representative materials and performing the shear interface testing under conditions similar to those expected in the field are discussed. Once the test data was available, they were used in a stability analysis software package to determine the Factor of Safety (FoS) of the barrier system. Lastly, to illustrate the risk of using literature shear strengths parameters without confirming them during construction, the FoS using actual shear strengths will be compared to the design FoS which used literature shear strengths.

**Keywords.** Geosynthetics, Critical Shear Interface, Site Specific, Shear Strength, Stability, Factor of Safety.

## 1 Introduction

Not only waste containment systems but for many other civil engineering projects, the shear strength of soil-geosynthetic interfaces and geosynthetic-geosynthetic interfaces is a critical design parameter as it often serves as a weak plane on which sliding may occur. Hence, site specific shear testing using representative materials under conditions similar to those expected in the field is recommended for final design. In this case study, geosynthetic and soil samples were exhumed from the base and side slope of a constructed facility to generate site specific data.

## 2 Site description

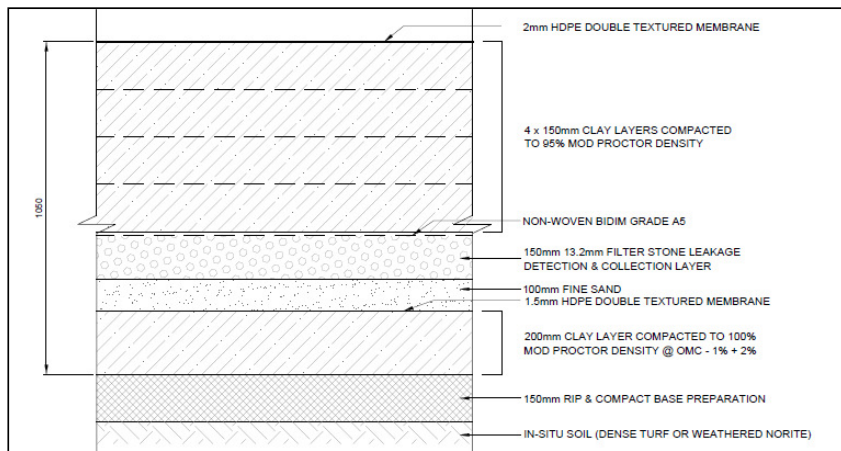
The ferrochrome smelter is located in the North West Province of South Africa and tailings in the form of a slurry remain from the beneficiation processes and is deposited onto a tailings storage facility (TSF). The TSF was designed in 2015 and construction was completed in 2018, yet the facility is still to be commissioned. Some concerns were raised by the Department of Water and Sanitation (DWS) regarding the design and expected performance of the facility during the application for an amended water use licence. This necessitated the smelter to have the original design reviewed in light of the DWS concerns to ensure that any omissions to the National Norms and Standards are identified and addressed.

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The NEMWA (National Environmental Management Waste Act 59 of 2008) and the NWA (National Water Act 36 of 1998) – Section 21(g) specify the requirements for waste disposal sites. The tailings are classified as Type 1 Waste (hazardous) and according to legislation must be contained in a Class A lined facility. The liner configuration in the basin of the TSF, as can be seen in Figure 1, from the top downward is as follows:

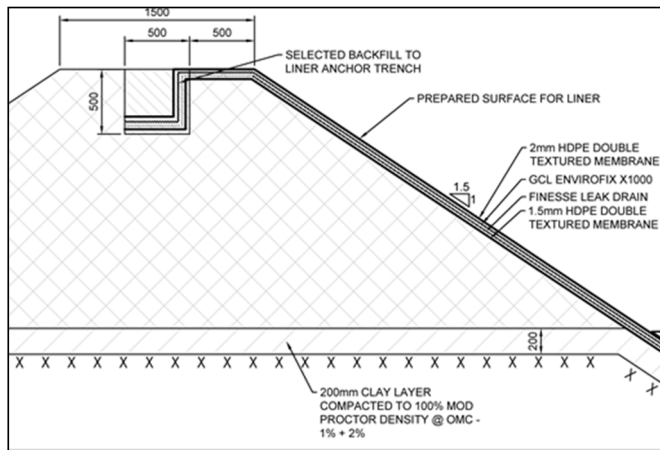
- 2 mm double textured HDPE geomembrane (primary liner)
- 4 x 150 mm thick layers of compacted clay (black turf)
- Separation geotextile
- 150 mm thick layer of 13 mm filter stone material, including a leakage detection pipe system
- 100 mm thick fine sand cushion layer
- 1.5 mm double textured HDPE geomembrane (secondary liner)
- 200 mm thick layer of compacted clay (black turf)
- Base layer ripped and re-compacted to 95% Mod AASHTO



**Fig. 1.** Liner package in the basin of the TSF according to as-built drawings.

The embankment walls of the TSF were constructed of selected clay (black turf) material compacted to 100% Proctor density at +1 to +2% of OMC. The liner configuration on the side slopes of the TSF are significantly different to that of the basin being quite geosynthetic intensive, as can be seen in Figure 2, and is as follows (from top downwards):

- 2 mm double textured HDPE geomembrane (primary liner)
- Geosynthetic Clay Liner (GCL)
- Geosynthetic Drainage Core (GDC) – cusped sheet
- 1.5 mm double textured HDPE geomembrane (secondary liner)



**Fig. 2.** Liner package on the side slopes of the TSF according to as-built drawings.

## 2.1 Sampling

On Tuesday, 3 August 2021, the exhumation of samples from the TSF was undertaken. On the basin, the 2 mm double textured HDPE liner was cut open to the dimensions of 1 m by 3 m. This was to allow sufficient space for taking clay (black turf) material samples from the underlying compacted clay liner. From this location, the 2 mm double textured HDPE liner and 50 kg of clay was retrieved. On the side slope, the 2 mm double textured HDPE liner was cut open for the retrieval of the geosynthetic clay liner (GCL), 1.5 mm double textured HDPE liner and geosynthetic drainage core (cusped sheet).

Although the primary liner has been exposed for around three years (at the time of retrieving the samples since initial installation) there are less wrinkles than one would expect. At one location on the basin, the liner has some mechanical damage to the textured surface. Once the primary liner was removed it was evident that the compacted clay was dry and desiccated. There were areas where the imprint of the texturing of the liner was visible and had striations which is expected from thermal expansion and contraction of the liner over the clay. After removing the top 150 mm of compacted clay, a large crack in the second clay layer was observed which extended the entire layer thickness (150 mm) to the top of the third layer.

Once the primary liner was removed on the side slope and the GCL was exposed and after cutting the GCL it seems as though the needle punching was still intact. Also, there was no trace of bentonite powder on the underlying GDC – this is specifically noted because a concern with the use of GCLs is that the bentonite might wash out from in between the geotextiles and clog the GDC.

## 3 Laboratory testing and results

A portion of the clay (black turf) sample was sent to a local accredited laboratory for a grading analysis and to verify the maximum dry density and optimum moisture content that was recorded in the quality control documentation during construction. More than 55% of the

material passed the 0.002 mm sieve and the resulting plasticity index was a value of 36 which confirmed that this a typical “black turf” clay [1] which has high potential expansiveness.

The remainder of the clay sample was sent to an internationally accredited laboratory where a consolidated-undrained triaxial test was undertaken to determine the internal shear strength parameters. The sample was remoulded to simulate as-built conditions which was 95% Modified Proctor (1514 kg/m<sup>3</sup>) at OMC (22.5%). However, in the design report slope stability section it was stated, “The strengths parameters of the various material layers and substrate have been selected for typical values obtained from tests performed in the past for similar soils in the same area that is part of the Bushveld Complex. It was decided to rather use average values allowing for the expected variance in the various properties.” These typical values are detailed in Table 1 below, as well as the results of the triaxial testing.

**Table 1.** Clay (black turf) material parameters.

	<b>Saturated Unit Weight (kN/m<sup>3</sup>)</b>	<b>Friction Angle (degrees)</b>	<b>Cohesion (kPa)</b>
Design report	18	25	15
Laboratory results	15	20.3	22.3

As per GRI GM13 note 6 [2], it is clear that “Shear strength associated with geomembranes is both site-specific and product-specific and should be determined by direct shear testing...”. Hence, at the same internationally accredited laboratory, shear interface testing was also performed. The four interfaces of concern are:

1. Shear interface testing of the 2 mm double textured geomembrane with clay (on the basin of the TSF). The clay was remoulded to 95% Modified Proctor and optimum moisture content.
2. Shear interface testing of the non-woven side of the GCL with 2 mm double textured geomembrane (on the side slopes of the TSF)
3. Shear interface testing of the GDC with the cuspatations facing the 1.5 mm double textured geomembrane (on the side slopes of the TSF)
4. Shear interface testing of the flat side of the GDC with the woven side of the GCL (on the side slopes of the TSF)

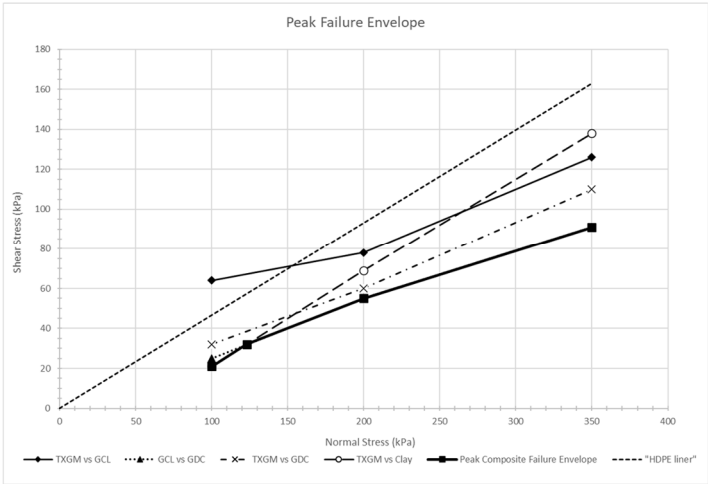
To generate test results which are representative of the site specific conditions, it is crucial for the conditions under which the shear interface test takes place be correctly specified. Laboratories generally require the following information to be detailed:

- Components, and if soil specifications for sample preparation
- Normal stresses
- Conditioning (moisture requirements)
- Consolidation (seating time)
- Shearing rate

Interfaces 1 and 3 were done according to ASTM D5321 [3] while tests 2 and 4 were according to ASTM D6243 [4] (Interface Shear Strength of Geosynthetic Clay Liner by Direct Shear). Since Interfaces 1, 2 and 4 contained clay and GCL they were tested under wet conditions (soaked) and the loading was applied for a minimum of 24 hours prior to shear. The interfaces that involve GCLs (2 and 4) were sheared at a rate of 0.1 mm/minute. Interface 3 was soaked and loading applied for 1 hour prior to shear and was sheared at 1 mm/minute.

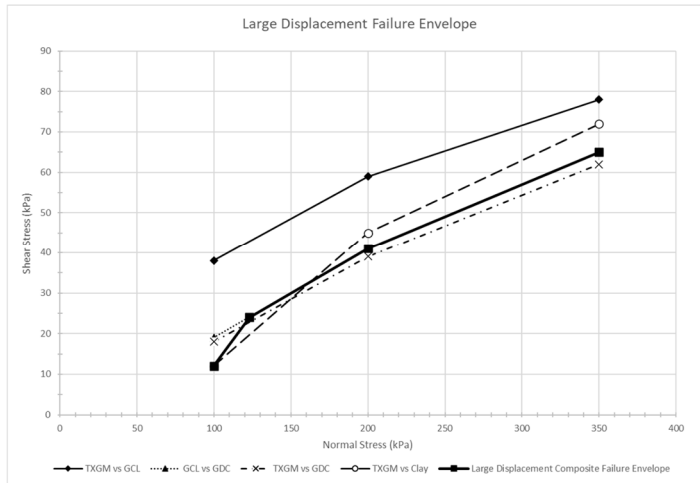
Specific consideration had to be taken for the shearing rate of Interface 1 due to the presence of the highly expansive clay (black turf). As stated in ASTM D5321, the appropriate

rate of shearing depends on several factors, including the geosynthetic, the materials on both sides of the geosynthetic, the soil, the normal stress level, the hydrating conditions, and the drainage conditions. Interface 1 was sheared at a rate of 0.01 mm/minute – this was to ensure that it was sheared at a sufficiently slow enough rate so as to not induce excess pore pressures. The normal stress range over which the test is performed should cover up to the maximum expected load at the end of the life of the facility. For this facility, the test was performed at normal stresses of 100, 200 and 350 kPa. Figure 3 presents the peak shear stress envelopes for all four interfaces which can develop within the barrier system of the TSF as well as the original design critical shear interface which been labelled “HDPE liner” (detailed further below).



**Fig. 3.** Peak shear stress envelopes.

For the liner package in the basin, there is only one critical shear interface which is of concern being the textured geomembrane and the compacted clay liner (TXGM vs Clay) interface. But since the liner package on the side slope is fairly geosynthetic intensive, the critical shear interface is not so straightforward. At low normal stress, the critical shear interface is dictated by the textured geomembrane and the compacted clay liner (TXGM vs Clay) interface. However, after 123 kPa normal stress, the interface between the GCL and the GDC becomes the main critical interface. Therefore, the critical peak shear interface is a composite envelope, consisting of more than one particular interface. The corresponding large displacement shear stress envelopes are presented in Figure 4. At first review of this graph, one would conclude that the critical large displacement shear interface should be the textured geomembrane and the geosynthetic drainage core (TXGM vs GDC) interface because it has the lowest values. However, it is explained by Stark & Poeppel [5] that the critical peak failure envelope needs to be carried through as the critical large displacement failure envelope. Therefore, the critical large displacement failure envelope starts along the textured geomembrane and the compacted clay liner (TXGM vs Clay) interface and at 123 kPa normal stress, the failure envelope of the GCL and GDC interface becomes the weakest interface, even though there are lower data points below.



**Fig. 4.** Residual shear stress envelopes.

## 4 Slope stability

The slope stability analyses were replicated to determine whether the original design FoS was overestimated and calculate what the most realistic FoS will be at the final proposed height. The original design had accounted for water pressures by using a probable phreatic line (based on the position of the toe drain and position of the pool at final elevation) – this same phreatic level was simulated in the replicated analyses.

### 4.1 Material Parameters

In the original design stability analysis, the critical shear interface was labelled “HDPE liner” and was modelled with Mohr-Coulomb parameters – unit weight of  $9.2 \text{ kN/m}^3$ , friction angle of  $25^\circ$ , and zero cohesion. Since laboratory testing had been undertaken, the stability analysis could now be updated with the site-specific critical shear interface. It is also imperative that this critical shear interface be included as a normal-shear stress function.

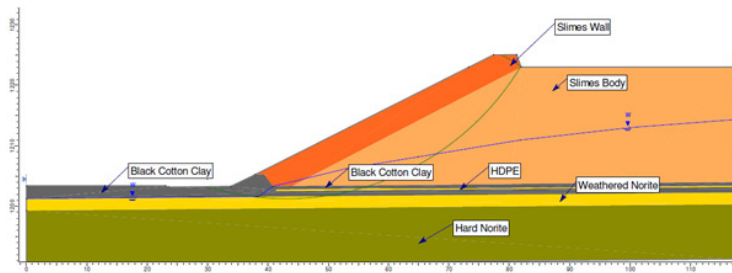
Although it is recommended by Stark & Poepfel (1994) that the large displacement interface strength be assigned to all side slopes, due to the short length of the side slope (as the embankment walls are only 2 m in height) it was considered more realistic to use the peak shear interface strengths on the side slopes. This assumption was checked by undertaking a slope stability analysis with each of the critical shear stress envelopes (peak and large displacement) on the side slope to confirm that the FoS did not differ.

### 4.2 Model geometry

In Figure 5, the typical cross section from the original design can be seen. It is evident from the tailings level down, the layers were detailed as follows:

- 600 mm clay layer
- 250 mm norite layer (presumed to be leakage detection layers)

- HDPE liner
- 1.5 m clay (presumed to be insitu black turf)
- Weathered and Hard Norite



**Fig. 5.** Original design slope stability cross section.

However, the accuracy of liner package being modelled into the software can also have an impact on the FoS results. Considering the liner package installed in the facility, it was reasoned that the most realistic manner to represent the site would be to detail the typical cross section as follows:

- Site specific critical shear interface
- 600 mm clay layer
- Impenetrable bedrock

### 4.3 Results

Three slope stability analyses were run using the cross-sectional geometry shown above in Figure 5. The analyses are as follows:

- All the same original parameters were used but change the method of slices from Bishop's modified method (design) to Morgenstern & Price method.
- Update the black clay material parameters to triaxial laboratory results.
- Update the clay material parameters to triaxial laboratory results and HDPE interface to site specific critical shear interface as normal-shear stress function.
- Most realistic site representation: Change the typical cross section layers as per the order specified above.

The results are summarized in Table 2. The FoS result for the original design as illustrated in Figure 5 was 1.41 – the critical failure surface passes through the HDPE (yellow layer) and fails in the weathered norite material. This factor drops by a decimal when changing the methods of slices to Morgenstern & Price – this time the critical failure surface fails along the HDPE (the original Mohr-Coulomb design parameters). This method is recommended as limit equilibrium method satisfies equilibrium of both the force and moment conditions under either constant or variable ratios of horizontal to vertical inter-slice forces. The FoS also drops by one decimal to a value of 1.3 when implementing the triaxial laboratory results for the clay and the critical failure surface remains unchanged from the previous analysis. The FoS then decreases by 12% when further implementing the site specific critical shear interface as a normal-shear stress function. Lastly, after changing the typical cross section of the model to the more realistic reflection of the site and making use of all the site specific laboratory results, the FoS ended up 3 decimals lower than the original design FoS. For both the last two analyses, the critical failure surface occurs along the shear interface of the

textured geomembrane and the compacted clay liner. In effect, the stability of the barrier system has gone from being adequate to unacceptable thus illustrating the risk related to making use of literature shear strength parameters in slope stability analysis.

**Table 2.** FoS against slip failure.

Configuration	Factor of Safety	Critical failure surface
Original Design	1.41	Weathered norite
i.	1.33	HDPE
ii.	1.30	HDPE
iii.	1.16	Critical shear interface (TXGM vs Clay)
iv.	1.08	Critical shear interface (TXGM vs Clay)

5 Conclusions

The importance of using representative materials and performing the shear interface testing under conditions similar to those expected in the field has been illustrated – the various components to detail for shear interface testing was mentioned. From this case study, it is clear that the designer cannot assume that the critical shear interface will only be one interface but could be a composite envelope consisting of more than one particular interface.

Also, after updating the shear strength parameters for the clay in the stability analysis software, the FoS reduced by one decimal when compared to using literature shear strengths parameters. Furthermore, after including the critical shear interface as a normal-shear stress function into the analysis, the FoS reduced by another decimal.

Hence, there is a moderate risk in using literature shear strength parameters as this can in turn have implications for eventual assessment of a barrier system, altering the FoS from acceptable to being insufficient. It is recommended that site materials be tested for actual shear strength parameters during the design stage and that geosynthetics supplied during construction are tested for confirmation of assumed shear strength parameters.

References

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