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Shear Behaviour of Multiple Layer Interface of Typical Components in a Landfill Liner System

S. Muluti[1], D. Kalumba[1], L. Sobhee-Beetul[1]

[1]Department of Civil Engineering, University of Cape Town, Cape Town, Western Cape
mltsha004@myuct.ac.za, denis.kalumba@uct.ac.za, laxmee.sobhee-beetul@uct.ac.za

Abstract

This paper presents the interface shear strength behaviour of a multiple layer interface shear test for a typical landfill liner system. The components of the liner system consisted of three geosynthetics: Geotextile (GTX), Geomembrane (GMB) and Geosynthetic Clay Liner (GCL), which were sandwiched between two soils: river sand and compacted clay. The test configuration and procedures for accurately representing field conditions, as well as the choice of normal stresses for determining shear strengths were discussed. Findings showed that at higher stresses of 200 and 400 kPa, strain softening was observed to achieve a steady-state strength beyond the peak strength, whereas at lower stresses of 50 and 100 kPa, little to no post-peak shear strength softening or reduction was observed. This discrepancy could be attributed to the tension within the geosynthetic components. Moreover, there was a significant difference in the peak and LD interface friction angles determined from the respective failure envelopes.

***Keywords**: Geosynthetics • Interface shear behaviour • Landfill liner system • Multiple layer interface test • Single interface test*

1. Introduction

Multiple interfaces exist in geotechnical structures such as composite liner systems in landfills, which consist of compacted soil with low permeability and geosynthetic materials, (Feng and Cheng, 2014; Guler, 2017). As a consequence, several interface planes are introduced into the system, potentially causing instability along the slope, and eventually leading to failure. Shear strength between soils and geosynthetics has been identified as a significant challenge in landfill designs. The shear strength of each material layer and the different interfaces between contact layers in the system influences the stability of these liner systems, (Stark and Choi, 2004; Stark *et al.*, 2012; Feng and Cheng, 2014). The shear strength at the interface between the different materials determines both the overall slope stability and the integrity of the geosynthetics. The interface shear behaviour of these components has been studied using a variety of laboratory test techniques by previous researchers, (i.e., Stark and Choi, 2004; Feng and Cheng, 2014; Bacas *et al.*, 2015; Buthelezi *et al.*, 2016; Chai and Saito, 2016). Using a Large Direct Shear Apparatus (LDSA), two methods of determining the interface shear strengths can be used, namely, single and multiple layer interface shear testing, (Stark *et al.*, 2015; Shenthan *et al.*, 2019; Swan and Stark, 2019).

A landfill liner system is made up of several interfaces that must be tested as a single system in order to fully comprehend the shear strength characteristics of the entire composite liner system. Researchers such as Stark *et al.* (2015); Khilnani *et al.* (2017); Shenthan *et al.* (2019); Sikwanda *et al.* (2019) and Swan and Stark (2019) have proposed multiple layer interface shear tests. This test method has an advantage over single interface tests in that it can directly determine the set of interface shear strength design values used in the slope stability analysis of the composite landfill liner system.

Therefore, a study of the interface shear strength behaviour of components of a typical liner system (see Figure 1) is presented in this paper. This study carried out multiple layer interface shear tests using the LDSA under various normal stresses to explore the actual shear stress of the interfaces in engineered landfills. The effects of this interface shear behaviour at different normal stress levels are also discussed.

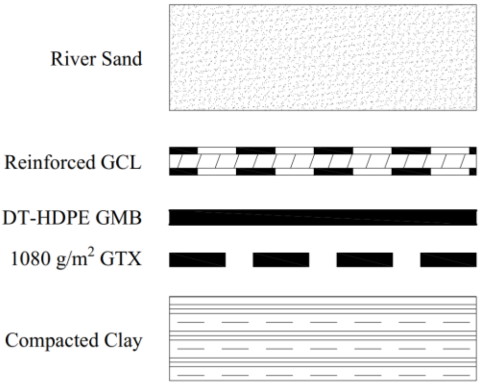

Figure 1 Components of a typical multiple interface landfill liner system.

2. Materials and Laboratory Testing

2.1. Soil Material

The two soils used in this study were river sand and clay (see Figure 2), which tend to be predominant in Durban, South Africa. Both soils were consistent, and simple to work with, allowing the results obtained to be replicated. The river sand was classified as a poorly graded uniform sand, while the clay was classified as a lean clay, based on the Unified Soil Classification System (USCS). The engineering properties of both soils are shown in Table 1.

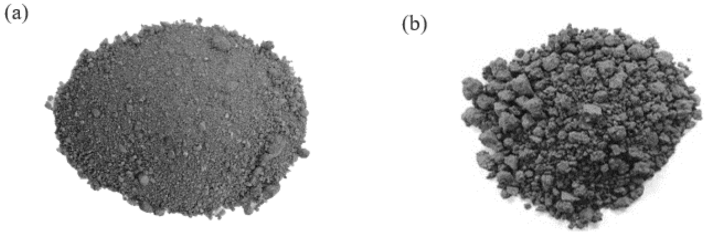

Figure 2 Photographs of soil materials used: (a) river sand and (b) clay.

Table 1 Engineering properties of the soils

Soil Properties	River sand	Clay
Specific Gravity, G_s	2.6	2.8
Cohesion, c' *(kPa)*	0.0	20.9
Angle of Internal Friction, ϕ' (°)	43.0	38.7
Optimum Moisture Content, *OMC* (*%*)	11.5	24.3
Maximum Dry Density, *MDD* (Mg/m^3)	1.7	1.6
Coefficient of uniformity, C_u	2.4	-
Coefficient of curvature, C_c	1.2	-
Plasticity Index, *PI* (*%*)	-	30.2

2.2. Geosynthetics

The study used the following geosynthetics, which are commonly used in landfill liner systems, to replicate the envisaged landfill interface conditions.

2.2.1. Geotextile

A 6.4 mm thick Geotextile (GTX) was used in this study. The GTX had a mass per unit area of 1080 g/m^2, a static puncture strength of 11.7 kN, a permeability of 0.0026 m/s and 50 – 80 % of elongation at break.

2.2.2. Geomembrane

The Geomembrane (GMB) used in this study was a 2 mm thick black Double Textured (DT) High Density Polyethylene (HDPE) with carbon black content of 2.5 % and a formulated density of 0.94 g/cm^3. The GMB was able to withstand up to 249 N of tear and 534 N of puncture resistance, respectively.

2.2.3. Geosynthetic Clay Liner

The reinforced Geosynthetic Clay Liner (GCL) was used in this study. This product was made up of, from top to bottom, a white polypropylene non-woven GTX cover, a light brown dry sodium bentonite powder layer in the middle with 0 % moisture content, and a polypropylene slit film woven GTX carrier layer. For the cover layer, bentonite layer, and carrier layer, the Minimum Average Roll Value (MARV) was 200, 3700, and 110 g/m^2, respectively. The bentonite layer also had a 4210 g/m^2 mass per unit area and a MARV swelling index of 12 ml/g.

2.3. Test Equipment

The automated *ShearTrac-III* Large Direct Shear Apparatus (LDSA) was used in this study. The equipment consisted of a top (static) shear box with plane dimensions of 305 × 305 mm and 100 mm depth, and a lower (moving) shear box with plane dimensions of 460 × 355 mm and a depth of 100 mm. A water tank (for hydration), two load cells (horizontal and vertical), two displacement transducers (horizontal and vertical), and a computer-controlled unit are also included in the system, as shown in Figure 3. The device could apply a constant shear rate of up to 15 mm/min, a maximum normal stress of 450 kPa and had a maximum load capacity of 44.5 kN.

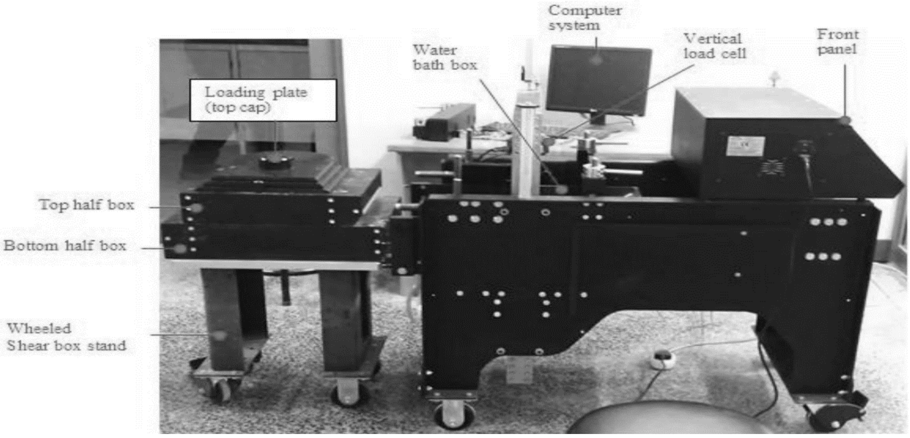

Figure 3 General configuration of Large Direct Shear Apparatus (LDSA).

2.4. Test Procedures

The preparation of the geosynthetic specimens was done in accordance with the guidelines specified in the ASTM D5321/D5321M - 17 and ASTM D6243/D6243 – 18 testing standards. The geosynthetics were acquired from different suppliers and they were cut either in the Machine Direction (MD) or at right angles to it. The specimens were cut into rectangles with sizes of 300 × 420 mm and 300 × 520 mm to fit the top and bottom shear box of the LDSA, respectively. The tests were conducted at increasing normal stresses of 50, 100, 200, and 400 kPa. This was done to simulate the increasing load of waste during the landfill design life. The test configuration for the multiple layer interface test was briefly summarized below. More detailed information on the test procedures followed can be found in Muluti (2021).

Four multiple layer interface shear tests were conducted. The two soil specimens were each placed and compacted into the respective shear boxes, with the river sand and clay placed in the bottom and top shear boxes, respectively. The geosynthetic specimens were placed unclamped between the top and bottom shear boxes, to allow failure to occur at the weakest interface during shearing. The configuration for the multiple layer interface testing is shown in Figure 4.

A Shearing Displacement Rate (SDR) of 0.1 mm/min was used for all interface shear tests due to the presence of GCL or clay specimens. This was done to ensure that during the experiments there were no excessive pore pressures built up at the interface. In landfills, GCLs are expected to hydrate soon after installation due to the liquids from the stored waste and/or the rains. Hence, hydration is required, which is why all test samples were immersed in water before testing to replicate the worst possible condition in a landfill, (Fox and Stark, 2004). Due to the presence of the GCL and clay specimens, hydration and consolidation periods of 24 hours was used, (Stark *et al.*, 2015; Buthelezi *et al.*, 2016; Sikwanda *et al.*, 2018; Swan and Stark, 2019; Adeleke *et al.*, 2021). During shearing, the bottom shear box was allowed to move relative to the upper shear box at a constant shear rate to achieve a shear displacement of 75 mm in order to capture both the peak and Large Displacement (LD) behaviour of the test samples.

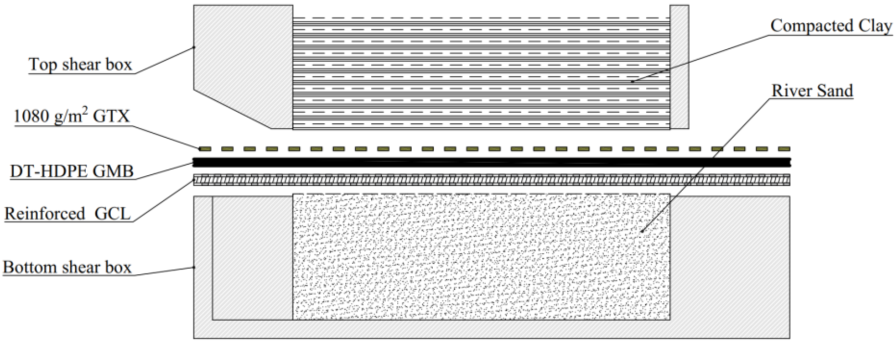

Figure 4 Multiple layer interface shear test configuration.

3. Results and Discussion

3.1. Interface shear curves

Figure 5 displays the shear stress versus horizontal displacement results from the multiple layer interface test performed at 50, 100, 200, and 400 kPa applied normal stresses. It can be observed that the measured shear stress responses were non-linear for all normal stress applied. At 50 kPa, no strain softening was observed, however at 100 kPa, slight strain-softening was recorded. A rapid reduction in shear stresses (strain-softening) was observed before reaching LD strength at normal stresses of 200 and 400 kPa. As such, the relative strain softening increased as the applied normal stress increased. The strain softening difference observed may have been caused by dislocation movements within the crystal structure of the tested soil and geosynthetic specimens, which were related to strain softening behaviour.

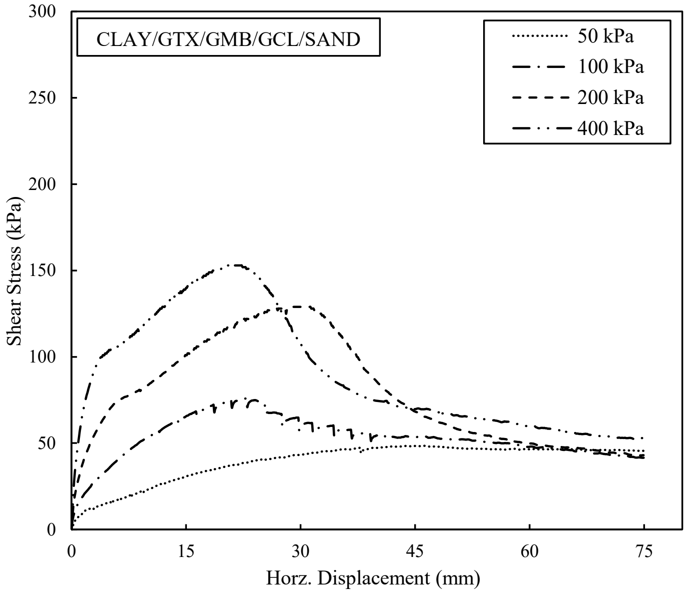

Figure 5 Shear stress versus horizontal displacement results for multiple layer interface test.

Furthermore, as a result of shearing possibly occurring along the CLAY/GTX interface at lower normal stresses of 50 and 100 kPa, the plots displayed "smooth" shear stress development curves similar to the stress curves that were obtained by Chai and Saito (2016), which was of a hyperbolic nature. The "pre-peak" stage at higher normal stresses of 200 and 400 kPa, on the other hand, displayed "*skewing*" behaviour at horizontal displacements of approximately 3 mm and 7 mm, respectively, as a result of shearing possibly occurring along the GCL/SAND. A phenomenon known as "*stick-slip*" behaviour was observed at a normal stress of 100 kPa. According to Soleimanian (2016), this may have possibly been due to the lack of any gripping system used, which could have resulted in slippage between the test specimens. Table 2 shows a quantitative summary of the peak and LD shear strength values obtained from the multiple layer interface tests.

Table 2 Summary of peak and LD shear strengths for multiple layer interface test.

Normal Stress (kPa)	Multiple layer interface	
	CLAY/GTX/GMB/GCL/SAND	
	Peak (kPa)	LD (kPa)
50	48.4	45.3
100	76.0	41.5
200	129.0	43.0
400	154.0	52.7

3.2. Interface shear strength envelopes

Figure 6 shows a failure envelope for the peak and LD shear stresses, plotted using a line of best fit for the multiple layer interface using data from Table 2. It can be seen that when peak shear stresses are plotted against normal stresses for each individual test, the plotted points lie on a slightly curved failure envelope. Although linear envelopes are the simplest relationships, multi-linear envelopes made up of two or more line segments, or non-linear envelopes, according to Fox and Stark (2015), provide an abrupt change in the angle of interface friction at the intersection point(s). This may, in some cases, reflect true shear strength behaviour. Therefore, in addition to the linear failure envelope, a curvilinear failure envelope (dashed lines) was used to represent the failure envelope for the peak shear stress, as shown in Figure 6.

3.3. Shear strength parameters

For further comparison and discussion, the peak and LD shear strength parameters for the multiple layer interface failure envelopes were obtained, and they are summarised in Table 3. The determined peak and LD interface friction angles from the multiple layer interface envelopes indicated a significant difference in the results. The peak strength failure envelope had an interface friction angle of 16.3°, while the LD strength envelope had an interface friction angle of 1.5°. This was a percentage difference of approximately 91 %. The apparent adhesion obtained for both the peak and LD failure envelopes, on the other hand, was very comparable. A peak apparent adhesion value of 47.0 kPa was obtained, compared to 40.7 kPa for LD apparent adhesion, a difference of approximately 13 %.

Table 3 Shear strength parameters of multiple layer interface failure envelopes.

Interface failure envelopes	**Peak shear strength parameters**		**LD shear strength parameters**	
	δ_p (°)	$c_{\alpha\text{-}p}$ (kPa)	δ_{LD} (°)	$c_{\alpha\text{-}LD}$ (kPa)
Multiple layer interface envelope	16.3	47.0	1.5	40.7

δ_p & δ_{LD} are Peak and LD Interface friction angle, and $c_{\alpha\text{-}p}$ & $c_{\alpha\text{-}LD}$ are Peak and LD apparent adhesion.

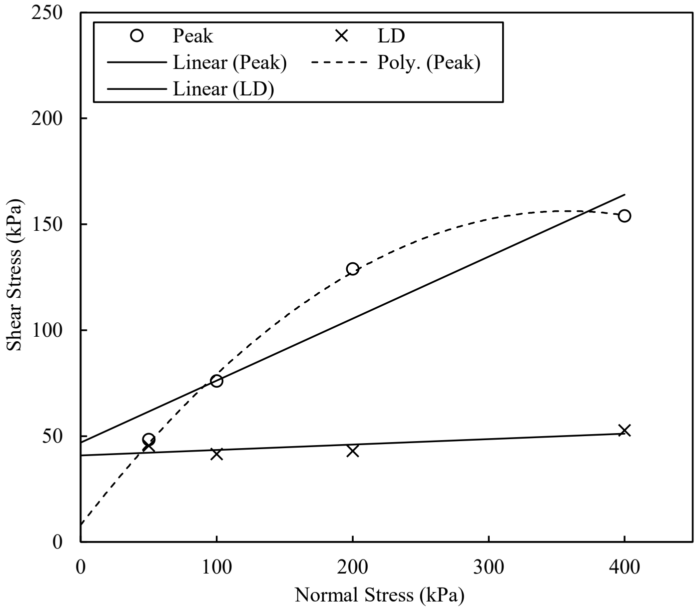

Figure 6 Failure envelopes multiple layer interface: (a) peak and (b) LD shear stress.

4. Conclusions

This study illustrated the interface shear strength behaviour of the components typically utilised in a landfill liner system. To investigate the actual shear stress of interfaces in engineered landfills, multiple layer interface was prepared and tested using a Large Direct Shear Apparatus. The findings of this paper can be summarised as follows:

- For the multiple layer interface test, the shear stress versus horizontal displacement curves were non-linear for all the normal stresses applied.
- Strain softening was observed to reach a steady-state strength beyond the peak strength at higher normal stresses, while little to no post-peak shear strength softening or reduction was observed at lower normal stresses. This may have been caused by dislocation movements within the crystal structure of the tested soil and geosynthetic specimens.
- The peak and LD interface friction angle of 16.3° and 1.5° were determined, respectively. This was a percentage difference of approximately 91 %. On the other hand, peak and LD apparent adhesion values of 47.0 and 40.7 kPa was obtained, respectively. This was a percentage difference of approximately 13 %, which was very comparable.

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