

Laboratory Study of Interface Characteristics of Landfill Liners

Faisal Hj Ali¹, Masashi Kamon², Takeshi Katsumi³, Tomoyuki Akai⁴,
Toru Inui⁵, Akira Matsumoto⁴ and Mariappan Saravanan⁶

¹Professor, Department of Civil Engineering, University Malaya, Kuala Lumpur, Malaysia.

²Professor, Graduate School of Global Environmental Studies, Kyoto University, Kyoto, Japan.

³Associate Professor, Graduate School of Global Environmental Studies, Kyoto University, Kyoto, Japan.

⁴Senior Research Scientist, Technology Research Institute of Osaka Prefecture, Osaka, Japan

⁵Research Associate, Graduate School of Global Environmental Studies, Kyoto University, Kyoto, Japan.

⁶Graduate Student, Graduate School of Global Environmental Studies, Kyoto University, Kyoto, Japan.

ABSTRACT:

Slope stability at landfill site is slowly becoming a critical factor in designing a landfill site especially in sloping terrain. Geotechnical and environmental engineers have shown much interest in recent years, as the conscious of safe guarding the environment become a social responsibility. Based on experience gained from past landfill failures, such as Kettleman and Cincinnati (Koerner and Soong, 2000) interface parameters between soil and geomembrane in landfill liner system was identified as the most weakest and sensitive point within the landfill configuration. Hence many engineers and researchers used various methods of parameter evaluation to evaluate the interface shear strength of various configuration of composite liners in landfill design. However there is no specific testing methodology and apparatus adopted till today. The current testing procedures are based on ASTM testing guidelines and basic fundamental engineering testing philosophies. This paper discusses the laboratory tests conducted for various composite liner systems for interface shear strength. The tests conducted include 1) interface shear strength evaluation for geosynthetic clay liners, 2) interface shear strength between soil and soil, 3) geomembrane and soil, 4) geosynthetic / compacted clay liners and soil, 5) geomembrane and geotextile, 6) geotextile and soil, 7) geotextile and geosynthetic / compacted clay liners, and 8) geomembrane and geosynthetic / compacted clay liners. The tests were performed at optimum moisture content and at saturated conditions. This paper also addresses the testing guidelines as per ASTM for landfill liner parameter evaluations. As such large scale shear box apparatus was adopted for the research works. Some interface test results are also presented herewith.

Keywords ; Interface, Shear Strength, Land Fill Liner System, Modified Large Scale Shear Box, Internal Shear Strength.

1 INTRODUCTION

In the case of Malaysia the volume of waste generation increased as a result of industrialization and population growth of 2.5 % per annum which generates 0.7 kg / day per capita waste. Solid waste collection in Malaysia stands at 15,000 tons daily as estimated by Ministry of Housing and Local Government (MHLG). This generates about 5 million tons of solid in 1994. By the year 2010, the collection is estimated to reach 9 million tons.

It is estimated that at least 5 % of the Malaysian population (approximately 1 million people) are living within 1 km radius from closed landfills and existing dumpsite. The recorded 300 to 400 dumpsites located around the country have severe social and health implications to Malaysian population (Salim et al., 2003)

The estimated life spend of landfill site is about 6 to 7 years of operational. The vast range of toxic material, constitute of Municipal Solid Waste need to be disposed systematically. Modern and well constructed landfill can be characterized as an engineered structure that consists primarily of a composite liner, leachate collection and removal system, gas collection and control system and final cover.

1.1 Basic landfill design

An engineered landfill site must be geologically, hydrologically and environmentally suitable. Landfills are not an open dump site. Nuisance conditions such as smoke, odor, unsightliness, insect, rodent, and seagull are not present in a properly designed, operated and maintained sanitary landfill. As such landfill site need to be carefully design to envelope the waste and prevent escape of leachate into the environment. Most important requirement of a landfill site is that it does not pollute or degrade the surrounding environment.

An engineered Municipal Solid Waste landfills

consist of the following (Qian et al., 2002).

- i. Bottom and lateral side liners system
- ii. Leachate collection and removal system
- iii. Gas collection and control system
- iv. Final cover system
- v. Storm water management system
- vi. Ground water monitoring system
- vii. Gas monitoring system

During construction or design of a landfill site, the engineers required to perform detail engineering evaluation on :

- i. Landfill foot print layout
- ii. Subsoil grading
- iii. Cell layout and filling
- iv. Temporary cover selection
- v. Final cover grading
- vi. Final cover selection

The above are directly relate to geotechnical engineering works which involves the use of ground improvement and slope stabilization technology. Every geotechnical engineers are required to engage in the environmental engineering problems with the motto of “Think Globally, Act Locally” (Kamon 2001).

1.2 Environmental aspect of landfill

The basic environmental guidelines have contributed in developing suitable liners or hydraulic barriers for the landfill site. Early liners consisted primarily of a single liner composed of a clay layer or a synthetic polymeric membrane. During the past few decades the trend is to use composite liner systems comprising both clay and synthetic geomembranes together with interspersed drainage layers.

The following are the approximate chronology showing the introduction date for each of these approaches.

- | | |
|------------|--|
| Pre – 1982 | Single clay liner |
| 1982 | Single geomembrane liner |
| 1983 | Double geomembrane liner |
| 1984 | Single composite liner |
| 1985 | Double composite liner with primary and secondary leachate collection system |

Double composite liners with both primary and secondary leachate collection system have been widely adopted in solid waste landfills in the United States. This type of liner system is mandated by Federal and State regulations for hazardous waste, in United States. Figure 1, shows the typical details of double composite liner system.

Progressively many other countries have imposed their own guidelines in bottom composite liners system. Figure 2 shows the various type of bottom lining system used in many countries.

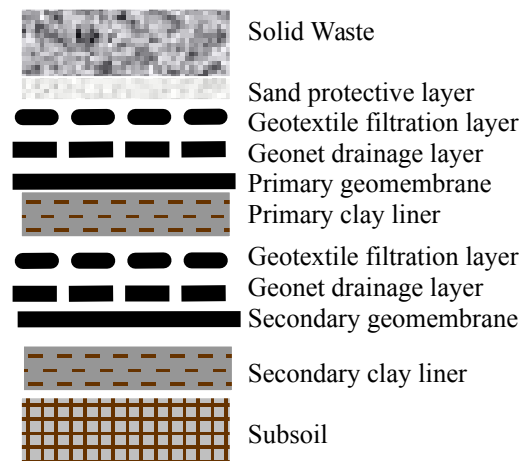


Figure 1 : Double Composite Liner System

1.3 Geotechnical engineering aspect of landfill

Geotechnical aspects of landfill involves the assessment of engineering properties of landfill components and design a stable landfill site against any mode of failure and avoid contamination to environment.

Some recent landfill failures have indicated failures taking place along low friction angle zone between subsoil and geosynthetic or geosynthetic layers, clay liners, landfill cover slopes in static state or under seismic condition. This has lead to various researches to be carried on the shear strength and interface properties of subsoils, clay liners, geosynthetic and waste material.

Most of the researches suggest the importance of geotechnical design in a landfill to prevent failures cause by low interface coefficient. The weakest interface identified, is generally lower between woven geotextile component of composite clay liner and the adjacent materials (Daniel et al., 1998). As the interface shear strength are dependent on many factors such as product type, hydration, shearing conditions and the specification of the equipment used to perform the tests (Triplett et al., 2001).

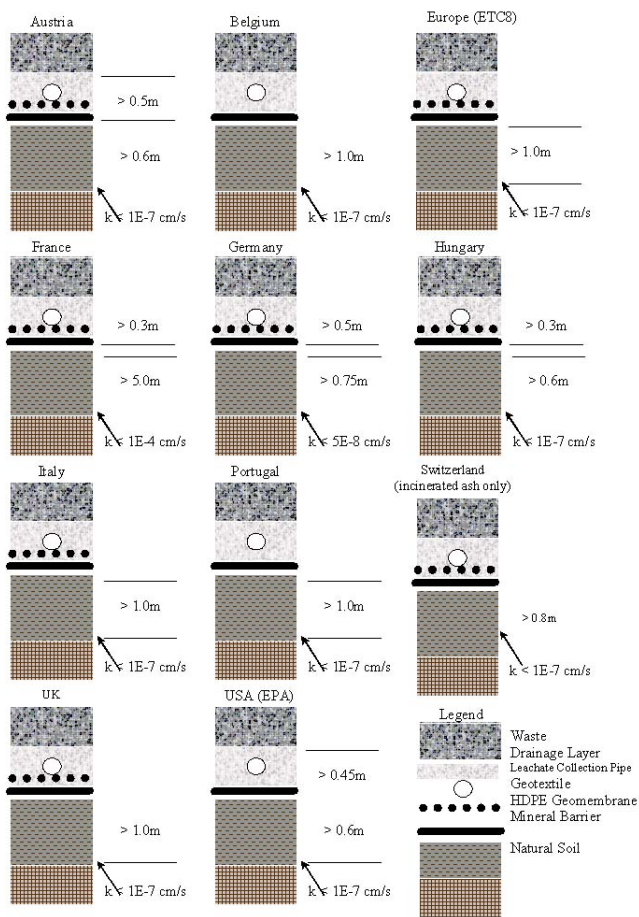


Figure 2 : Bottom lining systems used in many countries (Kamon, 2001)

Engineers are required to be careful in not designing slope that exceeds the safe slope angle for the clay liners or their respective interface within the system. For example, an infinite slope consisting of cohesionless interfaces with no seepage, the factor of safety (F) is (Daniel et al., 1998) :

$$F = \tan \phi / \tan \beta$$

Where ϕ = angle of internal friction;
 β = slope angle

During progressive failure in native soil, the peak strength of the MSW would be mobilized at a time when the shear strength of the native soil had declined to a value significantly below peak.

This condition takes place cause by stain incompatibility between native soil and MSW. Similar condition is also applied for geosynthetic interface and foundation soils because of their strain incompatibility with the adjacent materials in stability analysis (Hisham, 2000). Strain incompatibility could suggest the use of residual shear strength in stability analysis instate of peak shear strength.

Potential failure mode include the following ;

- Sliding failure along the leachate collection system
- Rotational failure along sidewall slope and base
- Rotational failure through waste, liner and foundation subsoil
- Rotational failure within the waste mass
- Translational failure by movement along the underlying liner system

The failures through liner system beneath the waste mass are common, due to by multiple layer components consisting of clay, soil and geosynthetic materials. Double-lined system can consist of as many as 6 to 10 individual components. As such the interfaces resistance of the individual components against shear stress could be low and cause potential failure plane. Figure 3 and 4 shows the type of potential failure along the liner system.

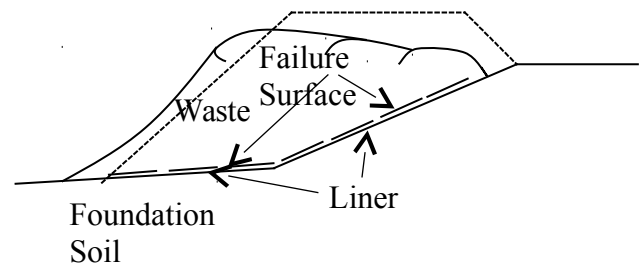


Fig. 3 : Failure Completely Along (or Within) Liner System (Xuede Qian, 2003)

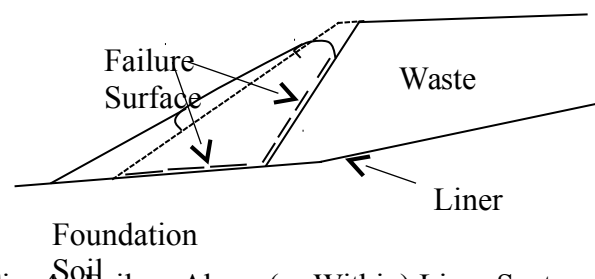


Fig. 4 : Failure Along (or Within) Liner System and Solid Waste (Xuede Qian, 2003)

The liners and closure cover system of a modern municipal solid waste (MSW) landfill are constructed with layers of material having dissimilar properties, such as compacted clay or geosynthetic clay liner, geomembrane (liquid barrier), geonet (drainage layer), geotextile (filter) and geogrid (reinforcement). Typical detail of such system is shown in Figure 5. While compacted clay or geosynthetic clay and geomembranes function effectively as flow barriers to leachate infiltration. However their interface peak and residual friction angles are lower than those of the soil alone. Such lower friction angle between a geomembrane and other geosynthetics could trigger much rapid failure during seismic loading conditions.

The soil-geomembrane interface acts as a possible plane of potential instability of the system under both static and seismic loading (Hoe I. Ling, 1997). Hence environmental geotechnical engineers are very concern about the potential instability caused by the waste containment liner system.

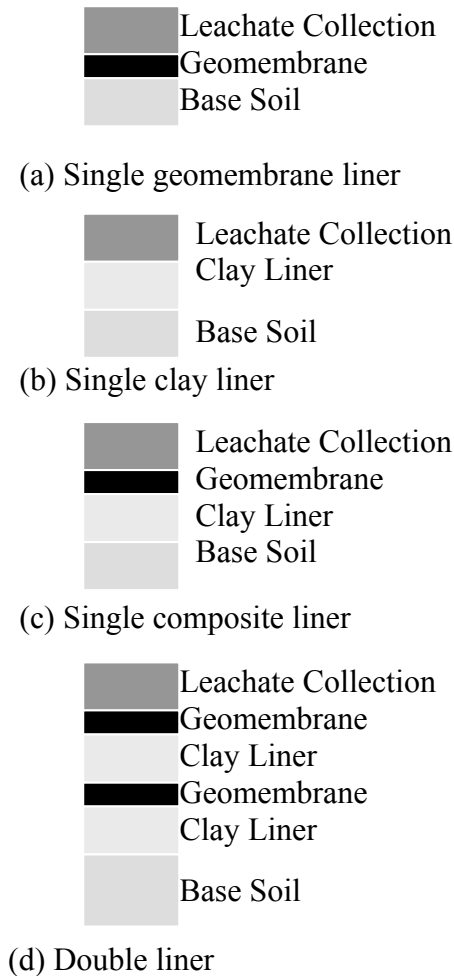


Fig. 5 : Cross section of typical bottom liner systems (Kamon, 2001)

Attention to slope stability of municipal solid waste during static and seismic loading has increased following report of Kettleman Hills waste landfill failure. The cause of failure was due to low friction angle between the soil and geosynthetic or geosynthetic layers in the liner system. This failure however was not attributed to seismic loading. Seismic performance of landfills has been reported from the 1989 Loma Prieta Earthquake and the 1994 Northridge Earthquake.

Seismic design of landfill systems should include response analysis, liquefaction analysis, deformation analysis and slope stability analysis. Shear failure involving liner system can occur at three possible location :

- i. The external interface between top of liner system and the overlying material
- ii. Internally within the liner system

- iii. Interface between clay liner and geosynthetic layer
- iv. The external interface between the bottom of the liner system and the underlying subsoil material

Current engineering design practice is to establish appropriate internal and interface shear strength parameters for design using direct shear test on test specimens and employing traditional limit equilibrium techniques for analyzing the landfill slope stability (David E. Daniel, 1998). As such simplified Janbu analysis procedure is recommended as it often gives factor of safety that is significantly less than those calculated by Spencer's procedure (Robert B. Gilbert, 1998).

2 TESTING APPARATUS DESIGN GUIDE

The modified large scale shear box for the interface shear strength evaluation for landfill liner system was developed based on the guideline of

- i. American Standard – ASTM D3080 – 98 – Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.
- ii. American Standard – ASTM D5321 – 02 – Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method.
- iii. American Standard – ASTM D6243 – 98 – Standard Test Method for Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liner by the Direct Shear Method.

As per the above guideline and testing requirement the apparatus design is subdivided into three categories, namely

- i. Soil and soil internal and interface testing apparatus to perform test on
 - Interface shear strength between native soil and compacted clay liner
 - Internal shear strength of native soil and compacted clay liner
- ii. Geosynthetic and geosynthetic internal and interface testing apparatus to perform test on
 - Internal shear strength evaluation of geosynthetic clay liners
 - Geomembrane and geotextile
 - Geotextile and geosynthetic clay liners
 - Geomembrane and geosynthetic clay liners

- iii. Geosynthetic and soil interface testing apparatus to perform test on
 - Geomembrane and native soil / compacted clay liner
 - Geosynthetic clay liners and native soil
 - Geotextile and native soil / compacted clay liner

All the above specified experiment are required to be conducted under both saturated and at optimum moisture content. Hence the equipment should meet the necessary guideline on sample saturation procedure. Following are the design guide adopted to modify the large scale shear box

i. Shear Box Design Guide

- a. The shear box size shall have a minimum size of 300mm x 300mm or 15 times the d_{85} of the coarse soil sample used, or 5 times the maximum opening size (in plan) of the geosynthetic to be tested. The adopted shear box size is 250mm x 500mm for top box and 350mm x 600mm for bottom box.
- b. The shear box height shall have a minimum height of 50mm or 6 times the maximum particle size of the coarse soil used. The adopted box height ranges between 85mm to 100mm.
- c. Test failure is defined as shear stress at 15 % to 20 % of relative lateral displacement. The shear box is designed to have maximum displacement of 100mm which is 20 % of 500mm of shear box length.
- d. The top and bottom box opening shall be $\frac{1}{2}$ of d_{85} or 1mm.

ii. Geosynthetic Clay Liner Hydration Process Guide (Patrick J. Fox, 1998)

- a. Determine the received geosynthetic clay liner water content as whole
- b. Add sufficient water in shallow pan and allow the geosynthetic clay liner for 2 days hydration with 1 kPa normal aerial load.
- c. Determine water content of geosynthetic clay liner as whole before and after shearing process.

iii. Shearing Process Guide

- a. The shearing machine is required to have a range of displacement rate of 0.025mm/min to 6.35mm/min however the proposed testing procedure will adopt a displacement rate of 1mm/min due to machine constrains.
- b. The normal loading plate shall have 0.2 to 0.5mm lesser dimension than the inner box dimension.
- c. The load cell or proving ring shall have an accuracy of 2.5N the record or monitor the shearing forces.
- d. Horizontal displacement measuring device shall have an accuracy of 0.02mm with maximum displacement of 120 ~ 150mm.
- e. LVDT – Linear Variable Differential Transformer is proposed to be use to measure displacements.

The above listed is the summary of interface and internal shear strength requirement base on the guideline in , ASTM D3080-98, ASTM D5321-02 and ASTM D6343-98. With such stringent guide and testing complexity, much attention was required to modify the conventional shear box to meet the standard guideline. Figure 6a,b,c, 7a,b,c and 8a,b,c shows one of the typical modifications of large scale shear box adopted for the research work for three different test conditions. Namely A) Case 1 – Interface testing between Geosynthetic and Geosynthetic, B) Case 2 - Interface testing between Geosynthetic and Soil, and C) Case 3 - Interface testing between Soil and Soil. Bottom box size of 350 x 600mm and the top box size of 250 x 500mm are used. Larger 100mm bottom box is used to define test failure of 15 % to 20% to relative lateral displacement of the top box dimension. However, shearing surface contact areas are made same for both top and bottom box of 250 x 500mm in size. Hence height adjustable bottom box base plate with spacer blocks are required to cater of variation in sample thickness and allowance for settlement or sample deformation during normal load loading prior to shearing. The method also minimize plowing kind of effect during shearing process, occurring when two different material hardness are in contact and sheared. Hence area correction method is adopted to obtain shear stresses. Constant shearing speed of 1 mm/min is used for test normal loads of 100, 200 and 300 kPa to obtain the interface parameters

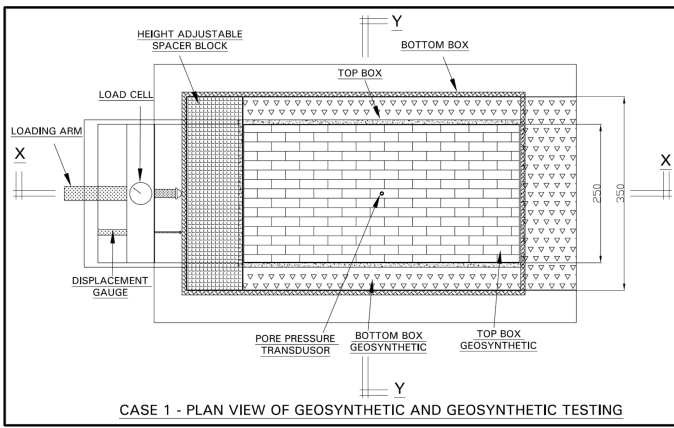


Fig. 6a : Case 1 - Plan View

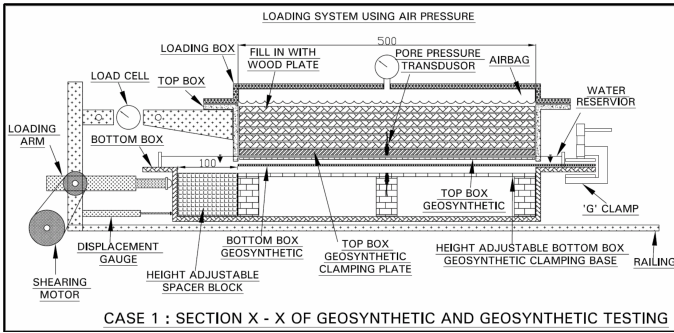


Fig. 6b : Case 1 – Section X - X

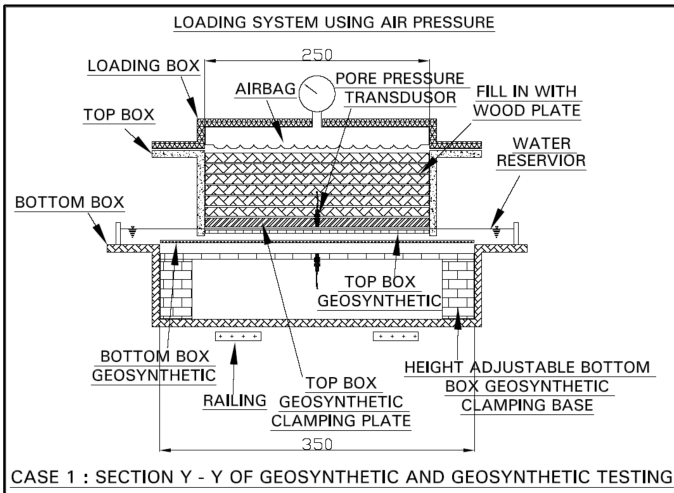


Fig. 6c : Case 1 – Section Y - Y

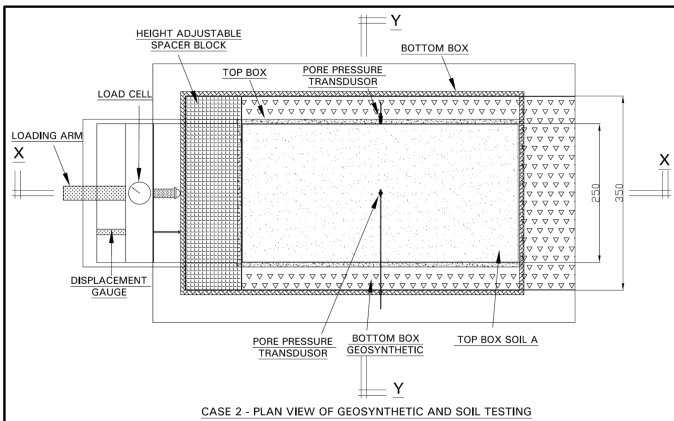


Fig. 7a : Case 2 - Plan View

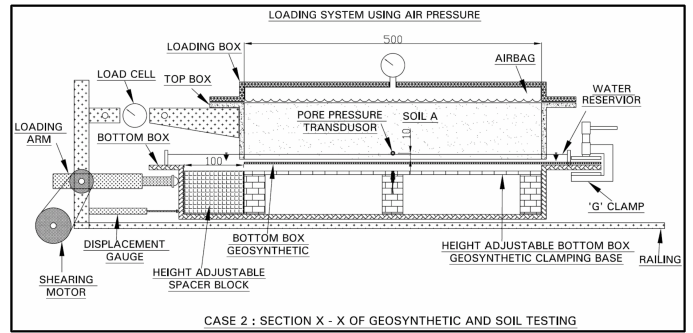


Fig. 7b : Case 2 – Section X - X

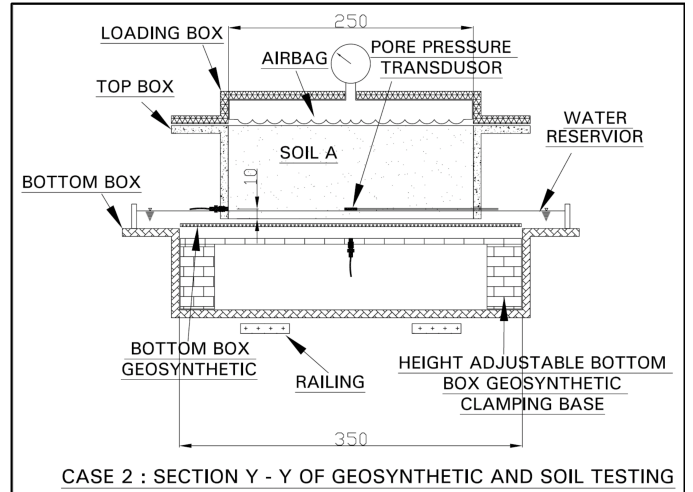


Fig. 7c : Case 2 – Section Y - Y

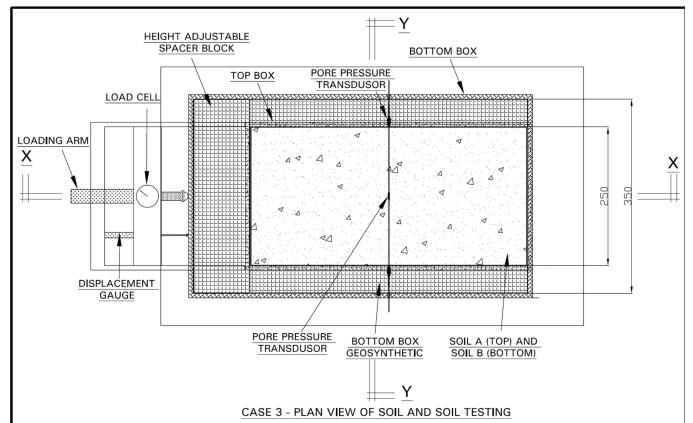


Fig. 8a : Case 3 - Plan View

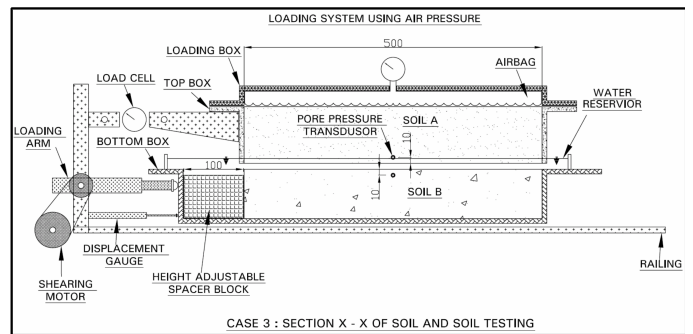


Fig. 8b : Case 3 – Section X - X

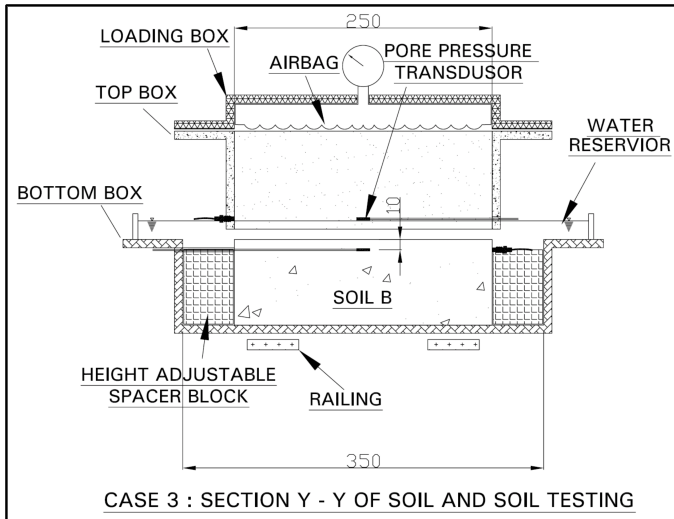


Fig. 8b : Case 3 – Section Y – Y

3 INTERFACE PARAMETER STUDY

The above discussion calls for detail and compressive study of landfill stability on the following :

- 1 Study landfill liner component, their internal shear strength and external interface properties
- 2 Liner geosynthetic material and physical properties.
- 3 Study the compacted clay liner (CCL) internal shear strength and external interface properties with geomembrane and geosynthetic clay liners
- 4 Study the interface property of compacted clay liners (CCL) and geosynthetic clay liner (GCL) with native soils
- 5 Study the interface property between CCL, GCL, non woven geotextile and geomembrane.
- 6 Study the suitable configuration of composite liner system which could improve the liner stability without neglecting the hydraulic conductivity requirement
7. Conduct detail stability analysis study of various configurations of landfill liner using the data from laboratory study, using limit equilibrium method.
8. Prepare a manual for landfill stability design and installation guide for landfill liner and cover soil to improve overall stability of landfill site by providing sufficient strain compatibility within the component members

3.1 Landfill liner configuration for research

The list of testing conducted will be dependent on the configuration and the material used for landfill liner system, adopted for research.

Following Figure 9 shows the configuration used for research

Figure 9 shows the typical configuration of landfill liner system and material component which will be studied in this research work. The configuration consists of both single and double composite liner system. However this paper discusses interface shear stress of single composite liner system at as installed condition. The research is still under progress to study the interface performance under saturated condition for both single and double composite liner system

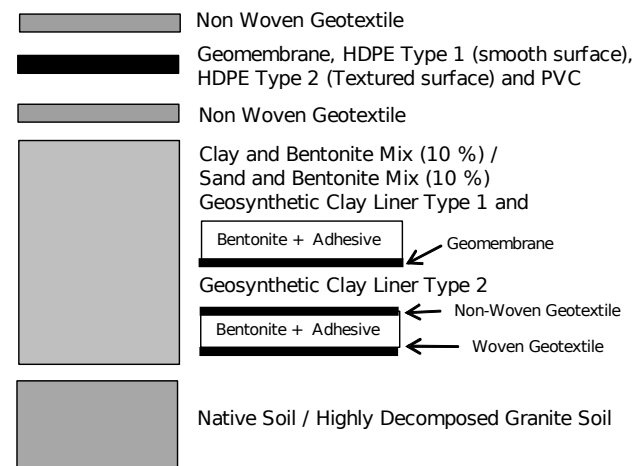


Fig. 9 : Details of Landfill Liner Configuration for Research

5 TEST RESULTS AND DISCUSSIONS

Figure 10 shows one of the commonly used configuration of single composite liner for landfill, which consist of a layer of HDPE type 1 geomembrane and a layer of Geosynthetic Clay Liner on top of a native soil which is highly decomposed granitic soil. Table 1 shows the test configurations.

The interface shear stress for the configuration is studied under as installed condition and the results are presented in Figures 11a,b, 12a,b, 13a,b and 14a,b respectively. Figure 15 shows the summary of interface shear stress for the said tests. Interface between geotextile and Geosynthetic Clay Liner (Test 4A) is higher as compared to interface between geotextile and HDPE type 1 (Test 1A). Similarly interface between native soil and HDPE type 1 (Test 27A) is much higher than Geosynthetic Clay Liner and HDPE type 1 (Test 6A). As for the design, the lower most interface parameters should be considered for analysis. In the case of stain incompatibility approach, HDPE type 1 reaches the peak shear stress within displacement of 5 to 15mm.

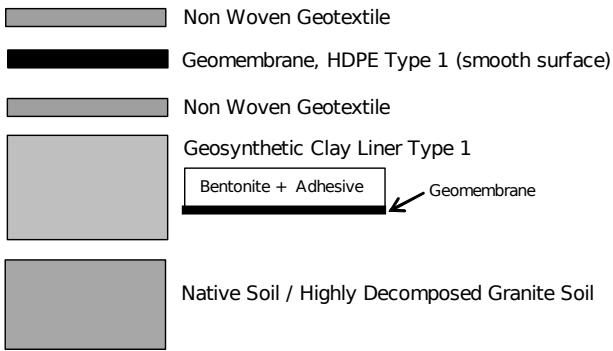


Fig. 10 : One of the commonly used configurations of single composite liner

Table 1 : Interface of Testing for Fig. 9 configurations

No	Primary Material	Secondary Material	Material Type	Test Condition	Series
1	Non Woven Geotextile Type 1	HDPE	Type 1	As Installed Condition	3 (A)
		Geomembrane	Type 1	Saturated Condition	3 (B)
4		Geomembrane	Type 1	As Installed Condition (Front Side)	3 (A)
		Clay Liner (GCL)	Type 1	Saturated Condition (Front Side)	3 (B)
6	HDPE Geomembrane Type 1	Geomembrane	Type 1	As Installed Condition (Front Side)	3 (A)
		Clay Liner (GCL)	Type 1	Saturated Condition (Front Side)	3 (B)
27	Native Soil	HDPE	Type 1	As Installed Condition	3 (A)
		Geomembrane	Type 1	Saturated Condition	3 (B)

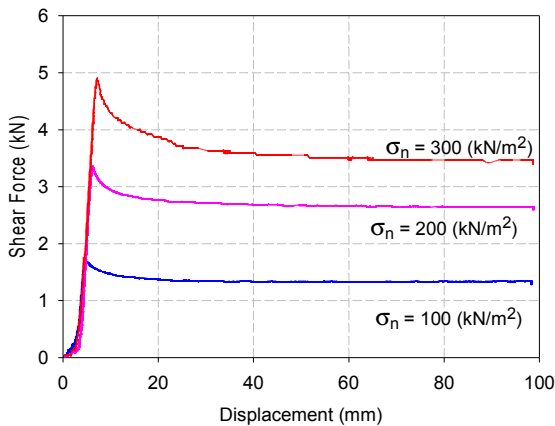


Fig. 11a : Test 1A Geotextile & HDPE Type 1, Shear Force (kN) Vrs Displacement (mm)

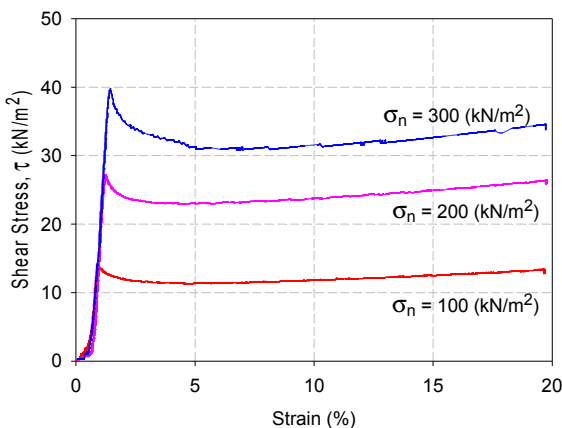


Fig. 11b : Test 1A Geotextile & HDPE Type 1, Shear Stress τ (kN/m²) Vrs Strain (%)

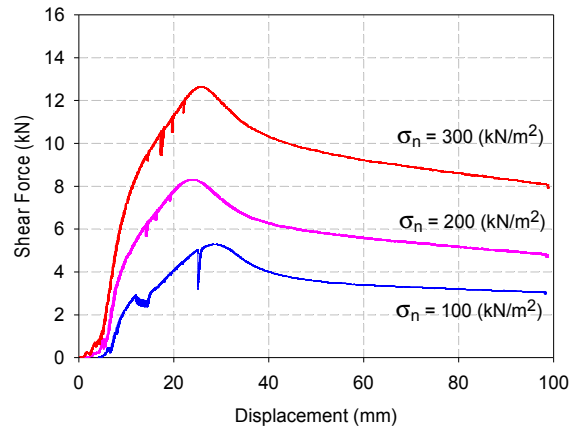


Fig. 12a : Test 4A Geotextile & GCL Type 1, Shear Force (kN) Vrs Displacement (mm)

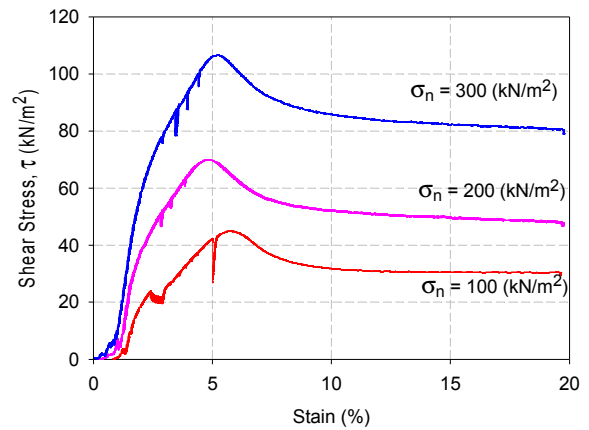


Fig. 12b : Test 4A Geotextile & GCL Type 1, Shear Stress τ (kN/m²) Vrs Strain (%)

However HDPE type 1, retain much constant residual shear stress as compared to geotextile. This could be due to the property of HDPE type 1, which required much higher displacement or strain before ultimate tensile strength is reached. As for geotextile peak shear stress is reached with displacement between 20 to 30mm. Geotextile residual shear stress tends to constantly reduce with displacement. As such the strain incompatibility between HDPE type 1 and geotextile could suggest the use of different selection approach of interface parameters for stability analysis.

Hence the interface test results presented under Figure 15 was based on maximum shear stresses obtained within 5 ~ 8 % of specific constrain on strain. This approach was adopted cause not in all cases the residual shear stresses are lower as compared to peak shear stresses. Example in the case of test 6A (Figure 13a,b) interface between HDPE Type 1 and GCL Type 1 the residual shear stresses are higher as compared to peak shear stresses. This findings are not consistent with the mode of failure obtained, in the case of test 4A (Figure 12a,b) interface between Geotextile & GCL Type 1. The higher residual shear stresses could not be considered for interface parameter selections.

Hence the approach of selecting residual shear stresses for stability analysis, in the case of interface parameters would not be appropriate. These shows that the shear stresses behavior at interfaces are much different as compared to internal shear stress failures of soils during shearing using shear box tests. Hence this indicates the complex behavior of interface shear stresses during failure due to material physical properties and strain incompatibility.

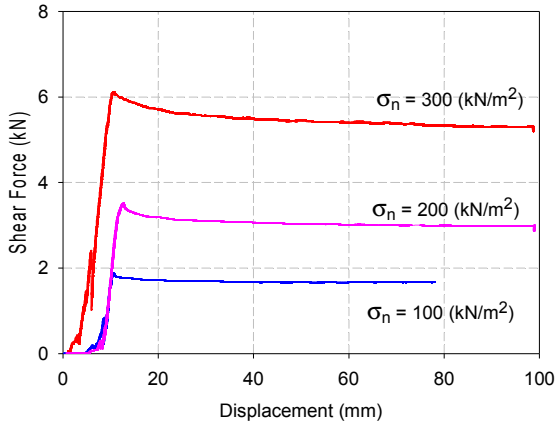


Fig. 13a : Test 6A HDPE Type 1 & GCL Type 1, Shear Force (kN) Vrs Displacement (mm)

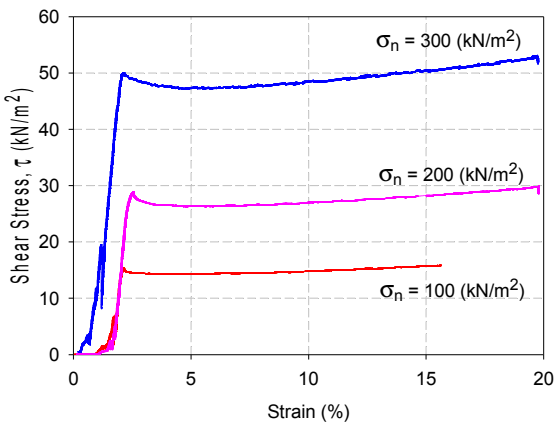


Fig. 13b : Test 6A HDPE Type 1 & GCL Type 1, Shear Stress τ (kN/m²) Vrs Strain (%)

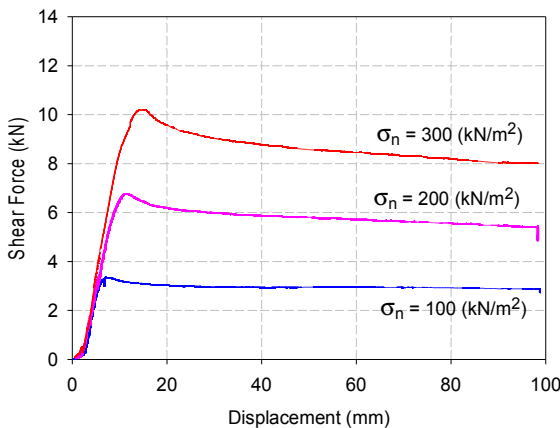


Fig. 14a : Test 27A Native Soil & HDPE Type 1, Shear Force (kN) Vrs Displacement (mm)

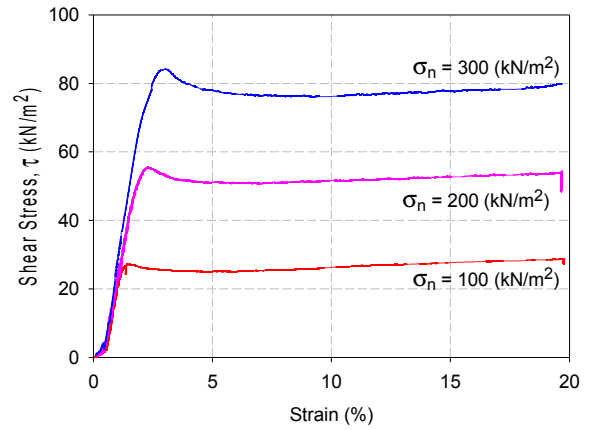


Fig. 14b : Test 27A Native Soil & HDPE Type 1, Shear Stress τ (kN/m²) Vrs Strain (%)

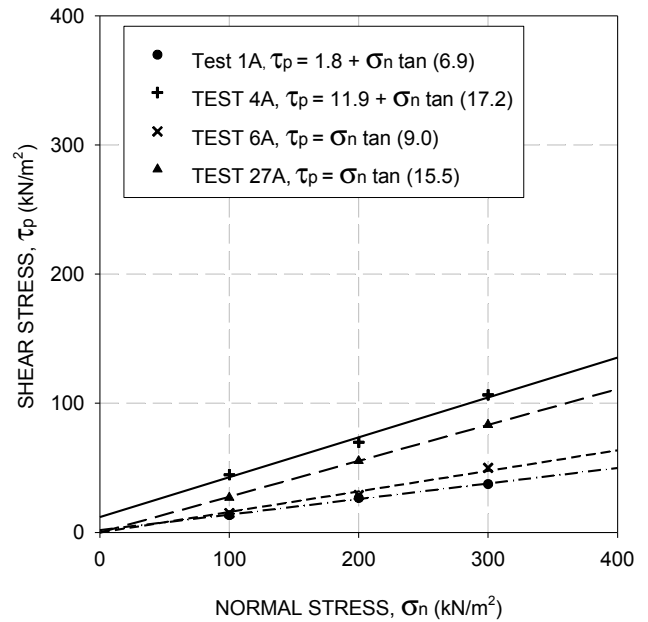


Fig. 15 : Interface shear stress results for Test 1A, Test 4A, Test 6A & Test 27A

6 CONCLUSION

The interface test results are much lower than anticipated. The mode of failure for various interface test combinations shows that there is no specific trend of failures. However the residual shear stresses are not lower for all the test cases within the defined 20% strain failure or 100mm shear displacement. Hence the adoption of using residual shear stresses to evaluate interface stability might not be appropriate. In this study the maximum shear stresses were computed within specific strain of 5 ~ 8% as redefined failure strain. Based on this method the interface parameters obtained in Figure 15 is much reliable to be used for stability analysis. With the information presented in Figure 15, the selection of appropriate and cost effective landfill configuration can be obtained prior to stability analysis for detail designs. Example the use of geosynthetic locking method can be decided based on data presented in Figure 14.

The data presented in Figure 15 will be updated further to make it as an immediate and quick reference guide for engineers in selecting the landfill liner materials. Data of interface test results under saturated condition will be included in the future.

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