Interface performance of landfill liners

Masashi Kamon[1], Faisal Hj Ali[2], Takeshi Katsumi[3], Tomoyuki Akai[4], Toru Inui[5], Akira Matsumoto[4] and Mariappan Saravanan[6]

[1]*Professor, Graduate School of Global Environmental Studies, Kyoto University, Kyoto, Japan.*
[2]*Professor, Department of Civil Engineering, University Malaya, Kuala Lumpur, Malaysia.*
[3]*Associate Professor, Graduate School of Global Environmental Studies, Kyoto University, Kyoto, Japan.*
[4]*Senior Research Scientist, Technology Research Institute of Osaka Prefecture, Osaka, Japan*
[5]*Research Associate, Graduate School of Global Environmental Studies, Kyoto University, Kyoto, Japan.*
[6] *Graduate Student, Graduate School of Global Environmental Studies, Kyoto University, Kyoto, Japan.*

ABSTRACT: Slope stability at landfill sites are gradually becoming a critical factor in designing a landfill site especially in sloping terrains. Geotechnical and environmental engineers have shown much interest in recent years, as the conscious of safe guarding the environment becoming a social responsibility. Based on experience gained from the past landfill failures, such as Kettleman and Cincinnati (Koerner and Soong, 2000b) 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 liner in landfill design. However there is no specific testing methodology and apparatus adopted till today. The current testing procedures are based on ASTM testing guideline and basic fundamental engineering testing philosophies. This paper discusses the interface tests conducted for various composite liner systems for interface shear strength. The tests were performed at optimum moisture content and at saturated condition. This paper also addresses the testing guideline 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.

1 INTRODUCTION

In Japan the material flow is about 2100 million tones annually. This consumption of resources generates waste of 600 million tones, which consist of 400 million tons of industrial waste and 50 million tons of municipal waste. Out of this 220 million tons are recycled and reused, 324 million tons are pre-treated waste for disposal and recycling. 56 million tons are used for landfill in Japan in year 2000.

The estimated life spend of landfill site in Japan is about 6 to 7 years of operational. It is becoming impossible to build new sites in Japan cause of the syndrome of “Not In My Back Yard”. The cost of new site in Tokyo could cost up to 500 million US dollors. The running cost of existing landfill site in Tokyo is at 300 USD / m^3 or 250 USD / tons

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 (Xuede Qian (2002):

- Bottom and lateral side liner system
- Leachate collection and removal system
- Gas collection and control system
- Final cover system
- Strom water management system
- Ground water monitoring system
- 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. Although the issue of landfill and environmental stability is part of global environmental problem, it is essential to solve them one by one. Every geotechnical engineers are required to engage in the environmental engineering problems with the motto of “Think Globally, Act Locally” (Kamon 2001).

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 is an 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 a typical details of double composite liner system.

2 LANDFILL STABILITY

Stability of landfills has been a major concern of the present environmental geotechnical engineering community. Failures at landfill sites can be minor, however the cost of rectification is huge. As landfill sites generally used to contain solid waste of various kinds, which some cases can contaminate and harm the environment. Hence landfill failures could lead to serious environment pollutions. However, stability is an issue that has be sometimes overlooked for the need of maximization of waste storage per unit area during continuous filling exceeding the initially design. In general majority of landfill sites are overfilled.

Hence 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 (David E. Daniel, 1998) :

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

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

Strain incompatibility with MSW could be another cause of stability failures. Example when failure occurs for the first, in the native soil, only a fraction of the MSW peak strength will be mobilized. As progressive failure occurs in the 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. Higher displacement is required before residual shear strength is mobilized and it is lower then peak shear strength which can be mobilized with relatively minor displacement. In relation to this, the geosynthetic material should able to withstand such high displacement with continuous strength contribution for stability prior to tearing before native soil failures completely.

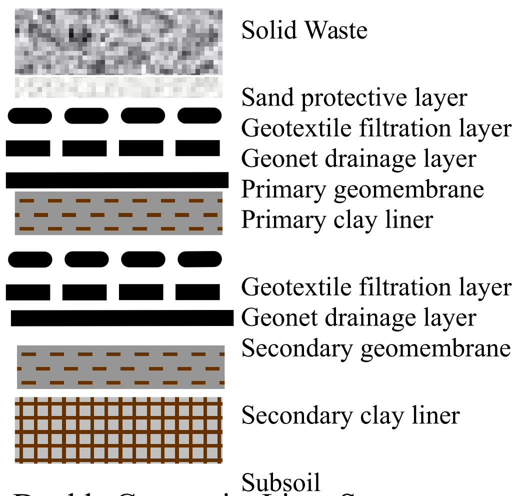

Fig. 1 : Double Composite Liner System

Potential failure mode include the following ;

i. Sliding failure along the leachate collection system
ii. Rotational failure along sidewall slope and base

iii. Rotational failure through waste, liner and foundation subsoil
iv. Rotational failure within the waste mass
v. 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 2 and 3 shows the type of potential failure along the liner system.

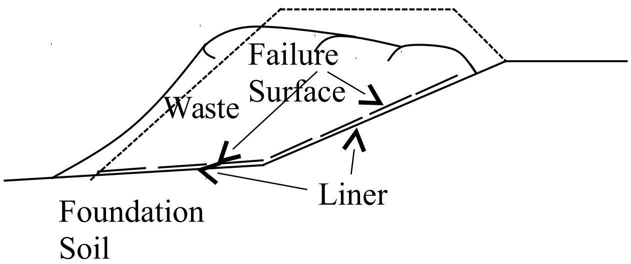

Fig. 2 : Failure Completely Along (or Within) Liner System (Xuede Qian, 2003)

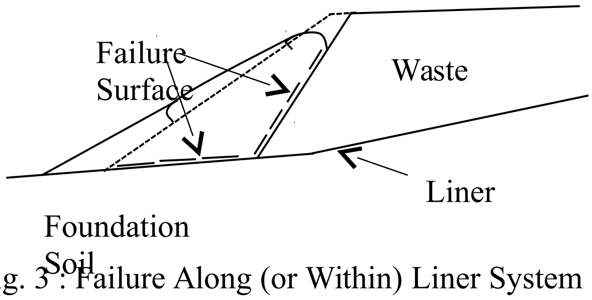

Fig. 3 : Failure Along (or Within) Liner System and Solid Waste (Xuede Qian, 2003)

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 4. 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.

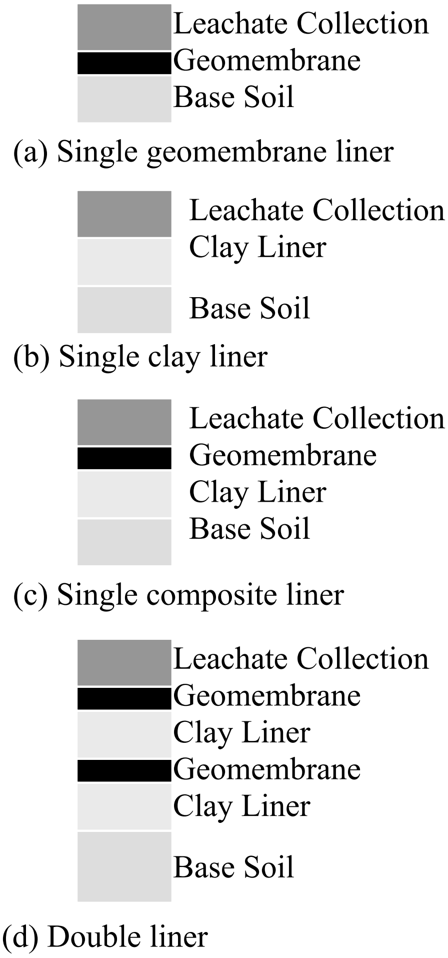

Fig. 4 : Cross section of typical bottom liner systems (Kamon, 2001)

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).

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 5 shows the configuration used for research

4 TESTING APPARATUS

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.

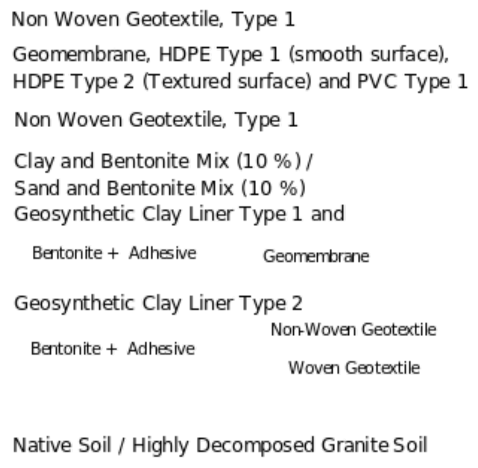

Fig. 5 : Details of Landfill Liner Configuration for Research

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

Figure 5 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

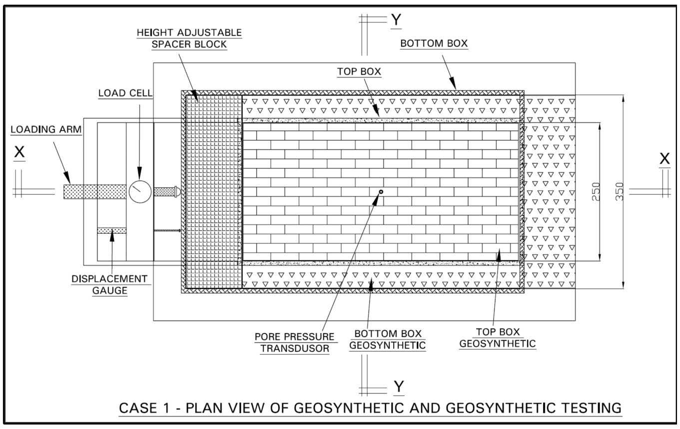

Fig. 6a : Case 1 - Plan View

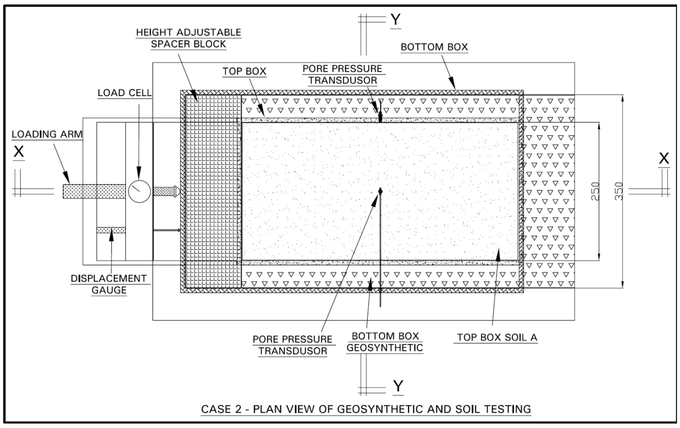

Fig. 7a : Case 2 - Plan View

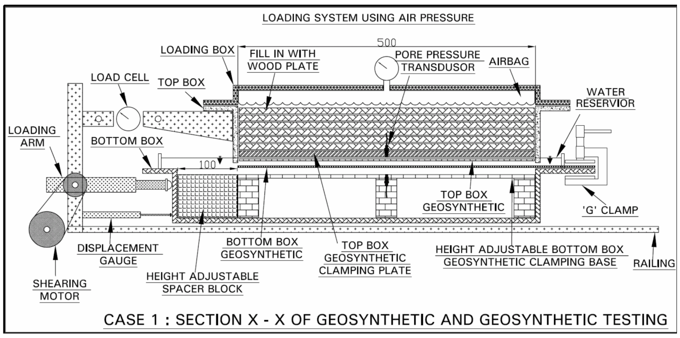

Fig. 6b : Case 1 – Section X - X

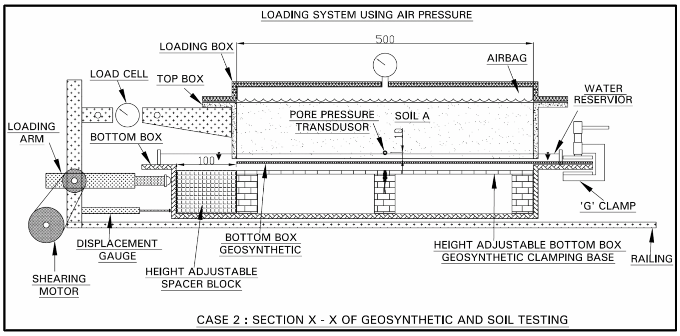

Fig. 7b : Case 2 – Section X - X

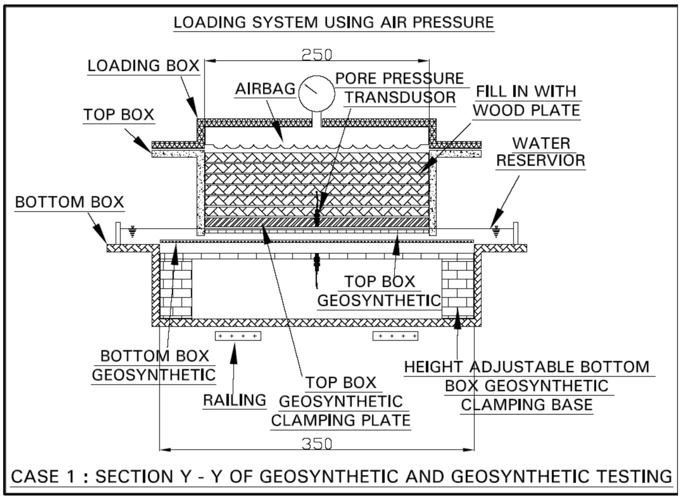

Fig. 6c : Case 1 – Section Y - Y

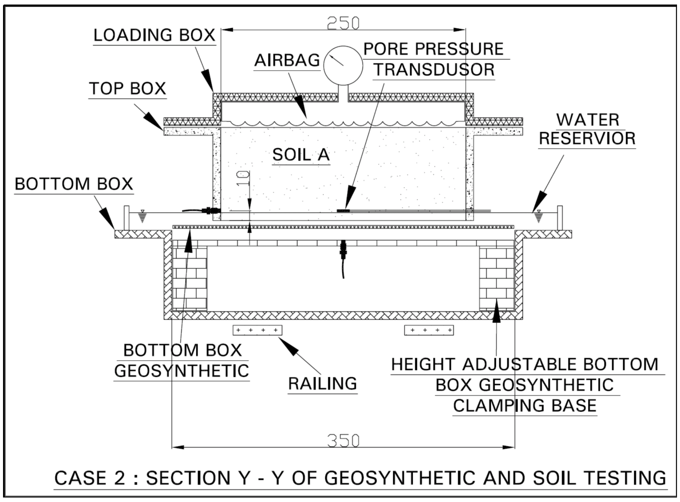

Fig. 7c : Case 2 – Section Y - Y

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

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 eliminates 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

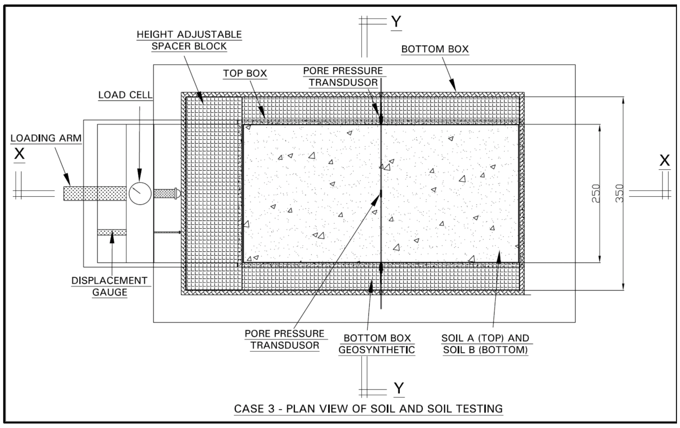

Fig. 8a : Case 3 - Plan View

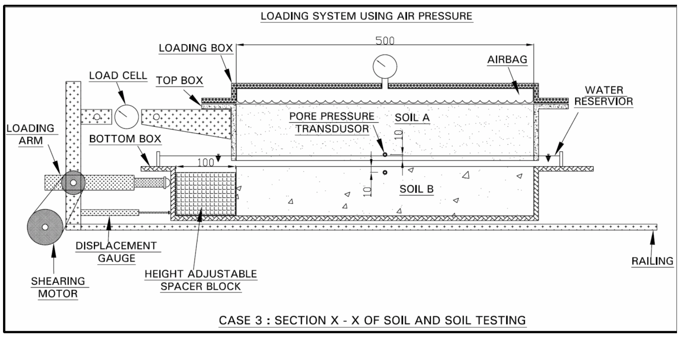

Fig. 8b : Case 3 – Section X – X

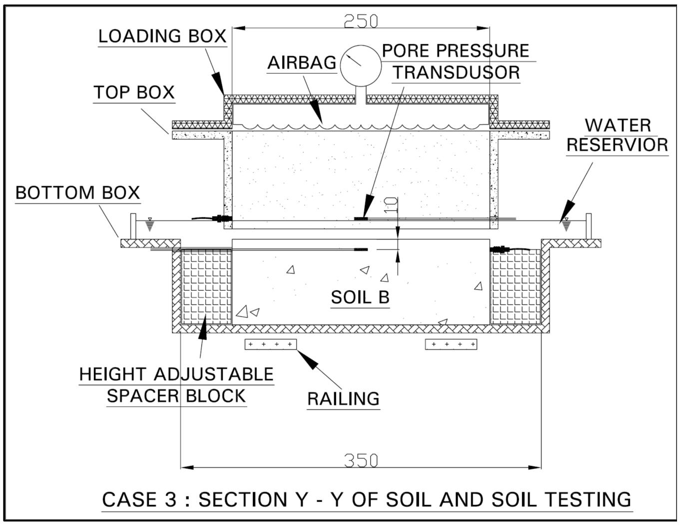

Fig. 8b : Case 3 – Section Y – Y

5 TEST RESULTS AND DISCUSSIONS

Figure 9 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 10a,b, 11a,b, 12a,b and 13a,b respectively. Figure 14 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

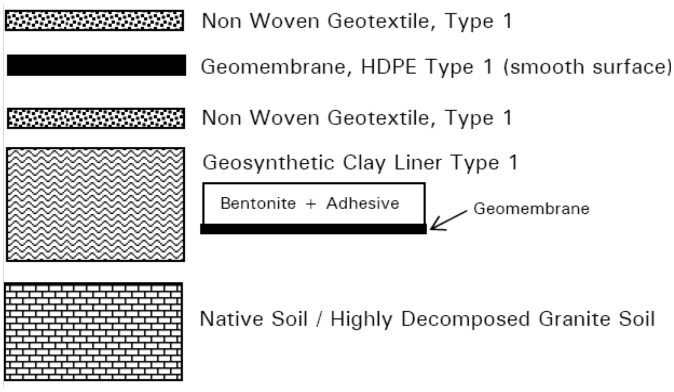

Fig. 9 : One of the commonly used configurations of single composite liner

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 Geomembrane	Type 1	As Installed Condition	3 (A)
				Saturated Condition	3 (B)
4		Geomembrane Clay Liner (GCL)	Type 1	As Installed Condition (Front Side)	3 (A)
				Saturated Condition (Front Side)	3 (B)
6	HDPE Geomembrane Type 1	Geomembrane Clay Liner (GCL)	Type 1	As Installed Condition (Front Side)	3 (A)
				Saturated Condition (Front Side)	3 (B)
27	Native Soil	HDPE Geomembrane	Type 1	As Installed Condition	3 (A)
				Saturated Condition	3 (B)

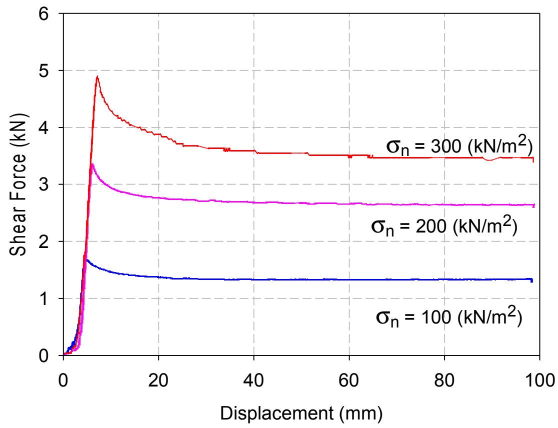

Fig. 10a : Test 1A Geotextile & HDPE Type 1, Shear Force (kN) Vrs Displacement (mm)

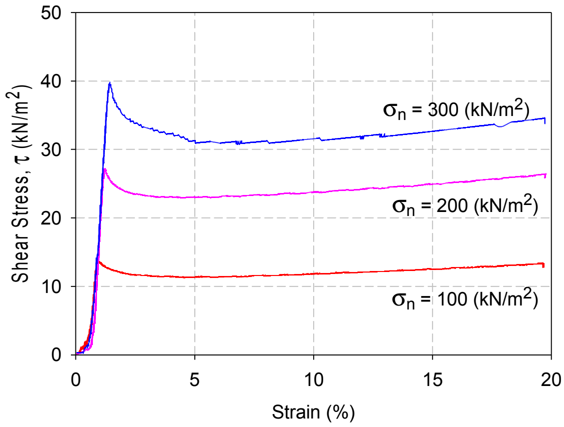

Fig. 10b : Test 1A Geotextile & HDPE Type 1, Shear Stress τ (kN/m^2) Vrs Strain (%)

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

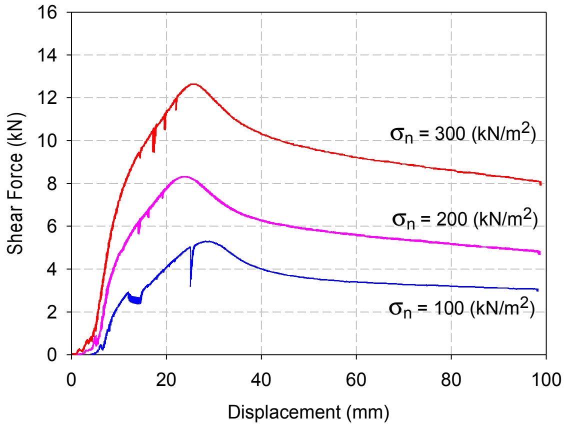

Fig. 11a : Test 4A Geotextile & GCL Type 1, Shear Force (kN) Vrs Displacement (mm)

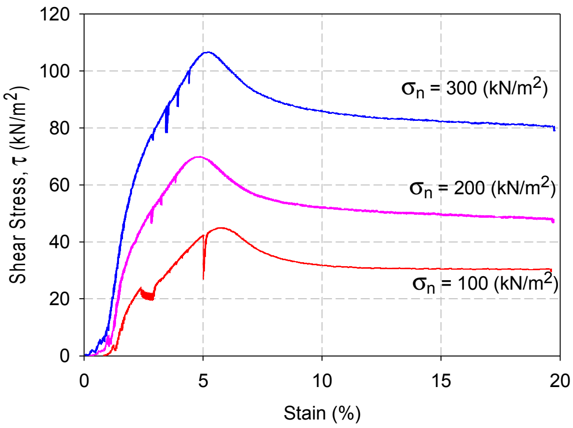

Fig. 11b : Test 4A Geotextile & GCL Type 1, Shear Stress τ (kN/m^2) Vrs Strain (%)

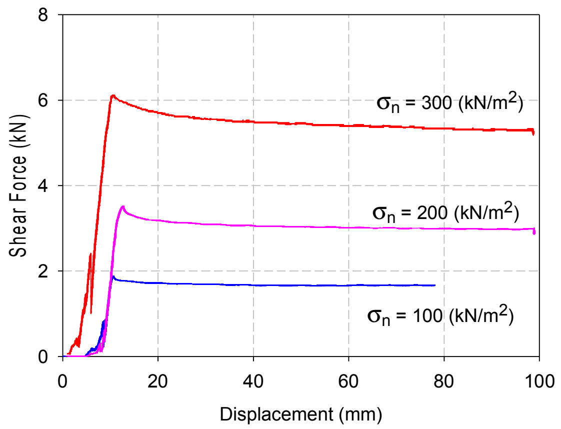

Fig. 12a : Test 6A HDPE Type 1 & GCL Type 1, Shear Force (kN) Vrs Displacement (mm)

peak shear stress within displacement of 5 to 15mm. 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 stain 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.

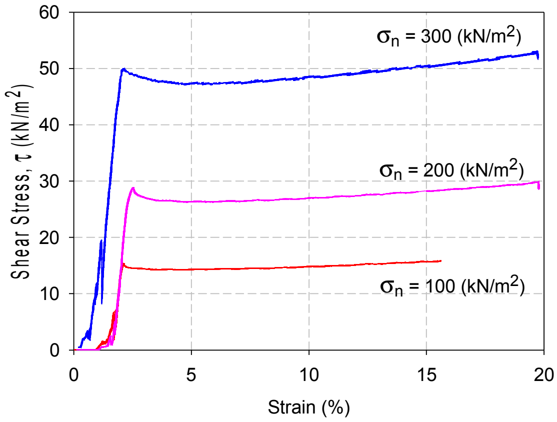

Fig. 12b : Test 6A HDPE Type 1 & GCL Type 1, Shear Stress τ (kN/m^2) Vrs Strain (%)

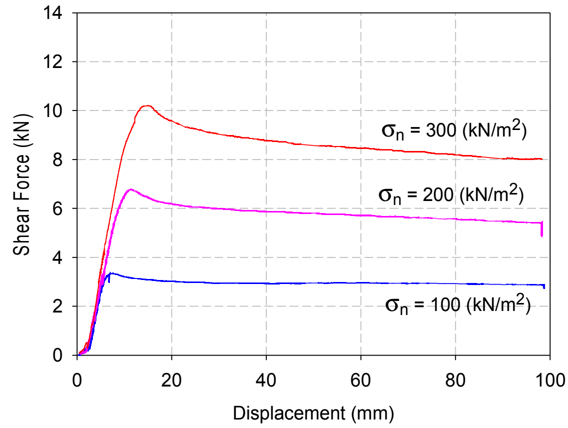

Fig. 13a : Test 27A Native Soil & HDPE Type 1, Shear Force (kN) Vrs Displacement (mm)

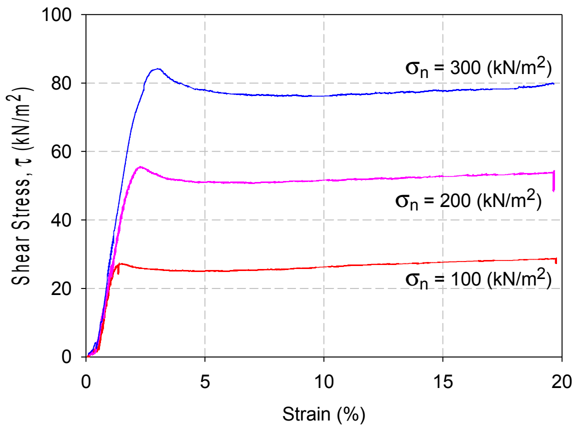

Fig. 13b : Test 27A Native Soil & HDPE Type 1, Shear Stress τ (kN/m^2) Vrs Strain (%)

Hence the interface test results presented under Figure 14 was based on maximum shear stresses obtained within 5 ~ 8 % of specific constrain on strain. This approach was adopted because not in all cases the residual shear stresses are lower as compared to the peak shear stresses. Example in the case of test 6A (Figure 12a,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 11a,b) interface between Geotextile & GCL Type 1. The higher residual shear stresses could not

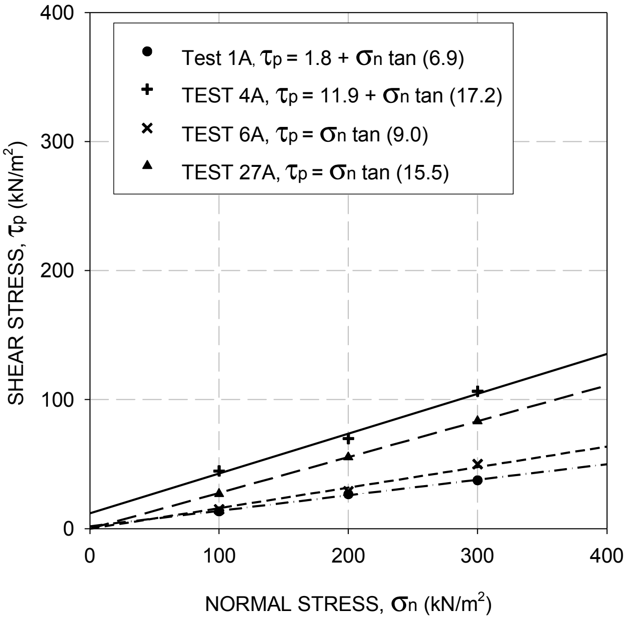

Fig. 14 : Interface shear stress results for Test 1A, Test 4A, Test 6A & Test 27A

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.

6 CONCLUSION

The interface test results are much lower then 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 14 is much reliable to be used for stability analysis. With the information presented in Figure 14, 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 14 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.

REFERENCES

ASTM D3080–98 "Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions". Annual Book of ASTM Standards, Vol 04.08. pp 347 – 352.

ASTM D5321–02 "Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method". Annual Book of ASTM Standards, Vol 04.13. pp 123-129.

ASTM D6243–98 "Standard Test Method for Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liner by the Direct Shear Method". Annual Book of ASTM Standards, Vol 04.13. pp 287-293.

David E. Daniel, Robert M. Koerner, Rudolph Bonaparte, Robert E. Landreth, David A. Carson and Heather B. Scranton (July 1998) "Slope Stability Of Geosynthetic Clay Liner test Plots", Journal of Geotechnical and Geoenvironmental Engineering, pp 628-637.

Hisham T. Eid, Timothy D. Stark, W. Douglas Evans, and Paul E. Sherry (May 2000) "Municipal Solid Waste Slope Failure. I Waste and Foundation Soil Properties", Journal of Geotechnical and Geoenvironmental Engineering, pp 397-407.

Hoe I. Ling and Dov Leshchinsky, (February 1997) "Seismic Stability And Permanent Displacement of Landfill Cover Systems", Journal of Geotechnical and Geoenvironmental Engineering, pp 113-122.

Kamon, M and Jang, Y.-S. (2001) "Solution Scenarios of Geo-Environmental Problems", Eleventh Asian Regional Conference on Soil Mechanics and Geotechnical Engineering". S. W. Hong et al., Swets & Zeitlinger, Lisse, pp. 833-852.

Kamon, M. and Katsumi, T. (2001): Clay liners for waste landfill, Clay Science for Engineering", K. Adachi and M. Fukue, eds., Balkema, Rotterdam, pp. 29-45.

Koerner, R. M. and Soong, T. (2000b). "Stability assessment of ten large landfill failures", Advances in Transportation and Geoenvironment Systems Using Geosynthetics, Geotechnical Special Publication No. 103, J.G. Zornberg and B.R. Christopher (Eds.), ASCE, pp 1-38.

Robert B. Gilbert, Stephen G Wright, and Eric Liedtke (Dec 1998) "Uncertainty in Back Analysis of Slopes : Kettleman Hills Case History", Journal of Geotechnical and Geoenvironmental Engineering, pp 1167-1176.