Interface shear stress parameter evaluation for landfill liner using modified large scale shear box

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ABSTRACT: Interface shear stress parameter evaluations for landfill liner systems have been a tedious testing process. Various testing methods and guidelines have been proposed by engineers and researchers over the years. 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. Hence there is a need for much improved testing equipment which can perform the entire test series required for landfill liner interface shear stress parameter evaluations. As such the equipment is required to perform interface shear stress between 1) soil and soil, 2) geomembrane and soil, 3) geosynthetic (GCLs) / compacted (CCLs) clay liners and soil, 4) geomembrane and geotextile, 5) geotextile and soil, 6) geotextile and geosynthetic (GCLs) / compacted (CCLs) clay liners, 7) geomembrane and geosynthetic (GCLs) / compacted (CCLs) clay liners. The equipment is also required to perform the tests under fully saturated condition. The paper also discusses about the modifications and test results obtained by modifying conventional large scale shear box, to perform interface testing as listed above. Strain incompatibility study between geosynthetic interface and foundation soil for single composite liner system will also be studied and presented herewith. In this paper interface shear stress of single composite liner system at as installed condition and saturated condition are discussed and presented. The research is still under progress to study the interface performance under saturated condition for both single and double composite liner system.

1 INTRODUCTION

The world consumption of natural resources has been increasing exponentially. In Japan the consumption of resource is at 1900 million tones annually. This consumption 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. 56 million tons are disposed to landfill in Japan in year 2000. The estimated life spend of landfill site in Japan is about 6 to 7 years of operational. It becomes very difficult to build new sites in Japan cause of the syndrome of “Not In My Back Yard”. The cost of a new site in Tokyo could cost up to 500 million US dollars. The running cost of existing landfill site in Tokyo is at 300 USD / m^3.

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. Hence engineers are required to be careful in not designing slope that exceeds the safe slope angle for liner components, internal properties and their respective interface parameters 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) :

$$F = \tan \phi \; / \tan \beta \qquad (1)$$

Where , ϕ is angle of internal friction and β is slope angle. Strain incompatibility with municipal solid waste (MSW) could be another cause of stability failures. Example when failure occurs for the first, in native soil, only a fraction of the MSW peak strength will be mobilized. Similar condition is also applied for geosynthetic interface and foundation soils because of their strain incompatibility with the adjacent materials in stability analysis (Hisham et al. 2000). Strain incompatibility could suggest the use of residual shear strength in stability analysis instead of peak shear strength. The soil-geomembrane interface acts as a possible plane of potential instability of the system under both static and seismic loading (Hoe I. L et al. 1997). Hence environmental geotechnical engineers have strong concern about the potential instability caused by the waste containment liner system.

3 INTERFACE PARAMETER EVALUATION

The study of landfill liner interface parameters for stability calls for detail and compressive study of the following :

i. Study landfill liner components and their interface properties
ii. Study geosynthetic liner materials and their physical properties.
iii. Study the compacted clay liner (CCLs) interface properties with geomembrane and geosynthetic clay liners (GCLs).
iv. Study the interface properties of compacted clay liners (CCL) and geosynthetic clay liner (GCL) with native soils
v. Study the interface properties between CCL, GCL, non woven geotextile and geomembrane.
vi. Study the suitable configuration of composite liner system which could improve the liner stability without neglecting the hydraulic conductivity requirement
vii. Conduct detail stability analysis study of various configurations of landfill liner using laboratory data by limit equilibrium method.
viii. Propose recommendation for landfill stability design and installation guide for landfill liner and landfill cover to improve overall stability of landfill site by providing sufficient strain compatibility within the component members

The list of testing interface conducted will be dependent on the configuration and the material used for landfill liner system, adopted for research. The liner configuration used for research is shown in Figure 1. Figure 2 shows the commonly used configuration of single composite liner which was studied and presented herewith. The research configuration consists of both single and double composite liner system. However this paper discusses interface shear stress of single composite liner system. The research is still under progress to study the interface performance under saturated condition for both single and double composite liner system.

4 TESTING APPARATUS

Figures 3, 4 and 5 shows some of the typical modifications of large scale shear box adopted for the research work for three different test conditions. Namely i) Case 1 – Interface testing between geosynthetic and geosynthetic, ii) Case 2 - Interface testing between geosynthetic and soil, and iii) Case 3 - Interface testing between soil and soil. Bottom shear box size of 350 x 600mm and the top box size of 250 x 500mm were used for the test. Larger 100mm bottom box was 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 for variation in sample thickness and allowance for settlement or sample deformation during normal 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 was adopted to obtain shear stresses. Constant shearing speed of 1 mm/min was used for test normal loads of 100, 200 and 300 kPa for the interface tests. ASTM D3080 - 98, ASTM D5321-02 and ASTM D6243-98 was referred for the modifications.

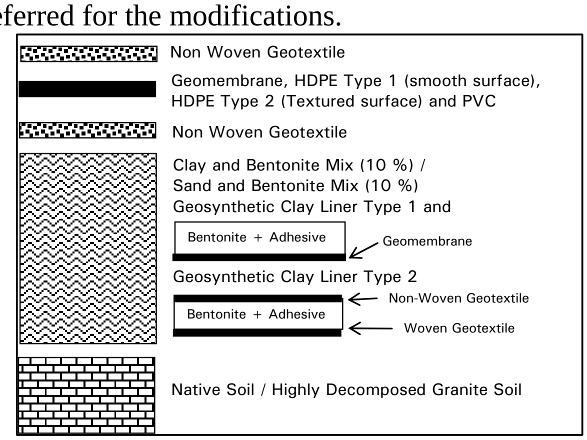

Fig. 1 : Landfill liner configuration used for the research

5 TEST RESULTS AND DISCUSSIONS

Figure 2 shows one of the commonly used configuration of single composite liner for landfill, which consist of a layer of HDPE type 1 geomembrane and compacted clay liner of sand bentonite mix (100 : 10) on top of native soil which is highly decomposed granitic soil. Table 1 shows the test configurations and interface test results. The interface shear stress for the configuration is studied under as installed condition and the results are presented in Figures 6, 7, 9 and 10 respectively. Figure 8 shows the pore pressure measurements within geotextile for Test 1B, interface between geotextile and HDPE type 1. Figure 11 shows the summary of interface shear stresses for the said tests in Table 1. Interface shear strength between sand bentonite mixture (100:10) and geotextile (Test 19A) is higher compared to interface between geotextile and HDPE type 1 (Test 1A). Similarly interface shear strength between native soil and geotextile (Test 26A) is higher as compared to Test 1A. In the case of saturated condition, there is not much variation between Test 1B for interface of goetextile and HDPE type 1 under saturated condition with Test 1A of as installed condition. As for stability and liner design lower interface parameters should be considered for analysis. In the case of strain incompatibility approach geotextile interfacing with geomembrane reaches peak shear stresses within 2 ~ 3 % strain.

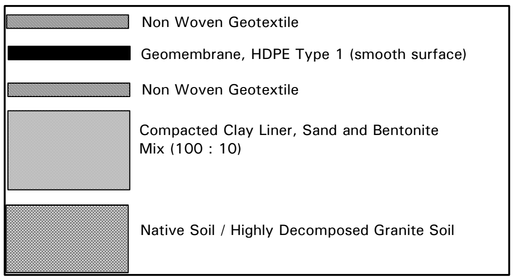

Fig. 2 : Typical configuration of single composite liner

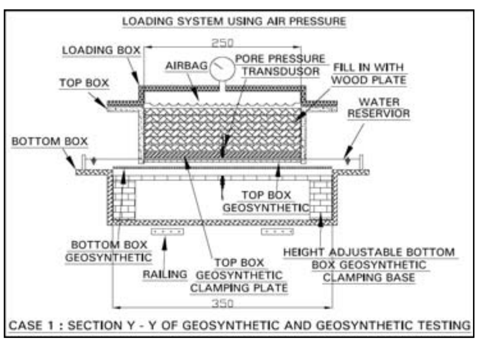

Fig. 3 : Case 1 – Geosynthetic and geosynthetic testing

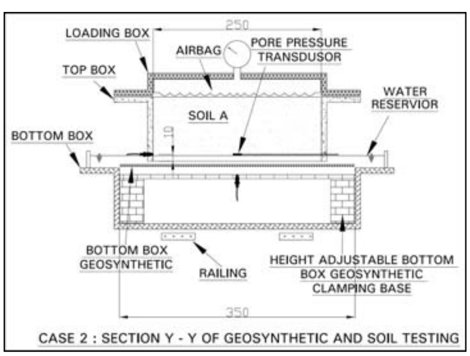

Fig. 4 : Case 2 – Geosynthetic and soil testing

Geotextile interfacing with compacted clay liner (Test 19A) and native soil (Test 26A) retains much higher strain before peak stresses are reached. As for geotextile peak shear stresses are reached with strain of 8 ~ 10 % . 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 11 was based on maximum shear stresses obtained within 5 ~ 8 % of specific constrain on strain.

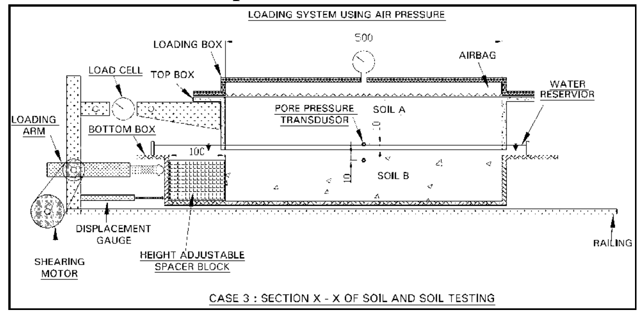

Fig. 5 : Case 3 – Soil and soil testing

Table 1 : List of the test configurations and interface test results

Test	Description	Cohesion (kN/m²)	Friction Angle (°)	Unit Weight (Mg /m³)
	Interface Parameters			
Test 1A	Interface Between Geotextile and Geomembrane HDPE Smooth Surface (Type 1) - As Installed Condition	1.8	6.9	-
Test 1B	Interface Between Geotextile and Geomembrane HDPE Smooth Surface (Type 1) - Saturated Condition	0	7.3	-
Test 19A	Interface Between Compacted Clay Liner – Sand Bentonite mix (100 : 10) and Geotextile – As Installed Condition	0	15.8	-
Test 23A	Interface Between Compacted Clay Liner – Sand Bentonite mix (100 : 10) and Native Soil – As Installed Condition	10*	15*	-
Test 26A	Interface Between Native Soil and Geotextile – As Installed Condition	0	17.8	-
	Soil Parameters			
1	Highly Weathered Granite Soil (Native soil)	25.0	46.7	2.1
2	Compacted Clay Liner – Sand Bentonite mix (100 : 10)	51.9	37.1	1.9
3	Waste (MSW) - Qian X, 2002	10.0	18.0	1.5

* is estimated values – experiment still in progress

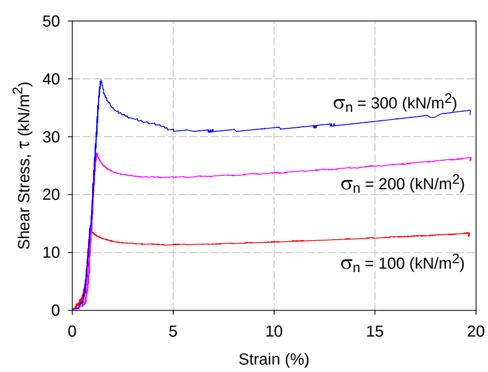

Fig. 6 : Test 1A - Geotextile and HDPE smooth surface (Type 1), shear stress τ (kN/m^2) versus strain (%) – as installed condition

This approach was adopted because not in all cases the residual shear stresses are lower as compared to peak shear stresses. For example in the case of interface test 19A (geotextile and sand bentonite mixture (100 : 10), CCL) and test 26A (geotextile and native soil) of Figures 9 and 10, 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 1A and 1B (Figures 6 and 7) interface between Geotextile and HDPE Type 1. The higher residual shear stresses could not be considered for interface parameter selections.

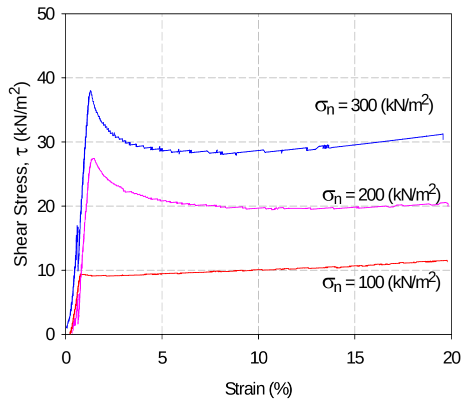

Fig. 7 : Test 1B – Geotextile and HDPE smooth surface (Type 1), shear stress τ (kN/m^2) versus strain (%) – saturated condition

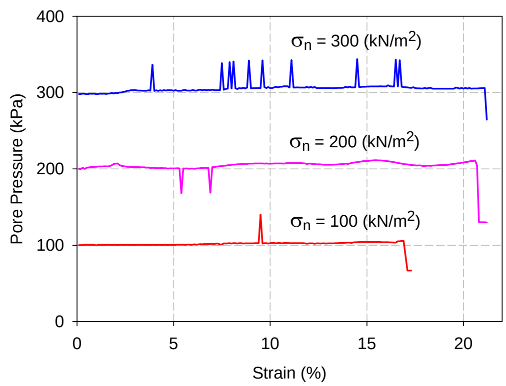

Fig. 8 : Test 1B - Geotextile and HDPE smooth surface (Type 1), pore pressure (kN/m^2) versus strain (%) – saturated condition

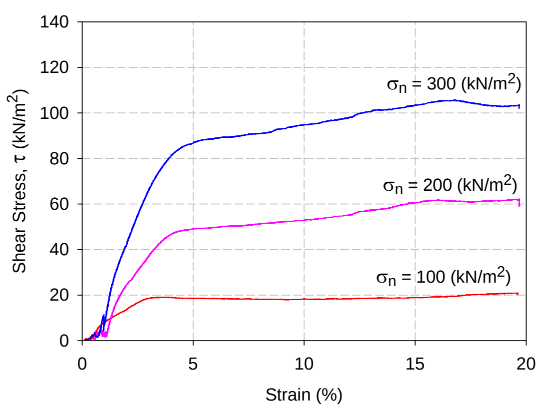

Fig. 9 : Test 19A - Sand bentonite mix (100 : 10) and geotextile shear stress τ (kN/m^2) versus strain (%) – as installed condition

6 ANALYSIS

Figure 12 shows a typical section of landfill which was used to study the liner interface performances. As for stability analysis, compatible software is required to model the landfill slope with relevant input parameters obtained from laboratory test data. Limit equilibrium based software was used to analysis both static and seismic loading conditions. Following are the list of cases considered for analysis i) Interface failure within bottom liner, ii) Internal failure within bottom liner, iii) Interface failure within liner cover, iv) Internal failure within liner cover. Table 2 lists out the analysis cases considered. All cases are analyzed for as installed condition only.

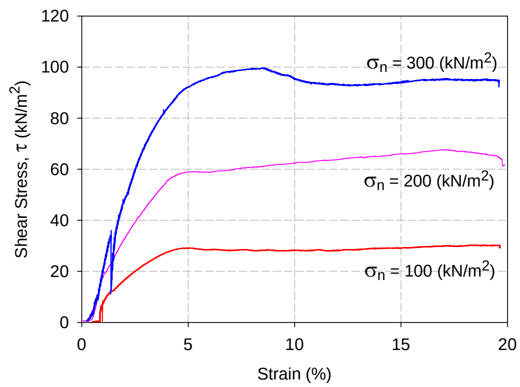

Fig. 10 : Test 26A - Native soil and geotextile shear stress τ (kN/m^2) versus strain (%) – as installed condition

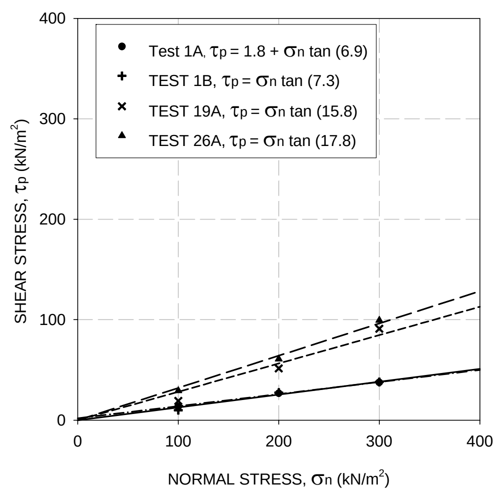

Fig. 11 : Interface shear stress results for Test 1A, Test 1B, Test 19A and Test 26A

Table 2 : Analyzing cases considered

Case	Description
Case 1	Interface Falure Between Compacted Clay Liner – Sand Bentonite mix (100 : 10) and Native Soil
Case 2	Internal Failure of Compacted Clay Liner – Sand Bentonite mix (100 : 10)
Case 3	Interface Failure Between Compacted Clay Liner – Sand Bentonite mix (100 : 10) and Geotextile
Case 4	Interface Between Geotextile and Geomembrane HDPE Smooth Surface (Type 1) - Bottom
Case 5	Interface Between Geotextile and Geomembrane HDPE Smooth Surface (Type 1) - Top
Case 6	Interface Between Geotextile and Cover Soil (Highly Weathered Granitic Soil – Native Soil)– Top
Case 7	Interface Between Geotextile and Geomembrane HDPE Smooth Surface (Type 1) - Top
Case 8	Interface Between Geotextile and Geomembrane HDPE Smooth Surface (Type 1) - Bottom
Case 9	Internal Failure of Cover Soil (Highly Weathered Granitic Soil – Native Soil)
Case 10	Toe Failure of Waste
Case 11	Overall Landfill Failure
Case 12	Overall Landfill Base Failure

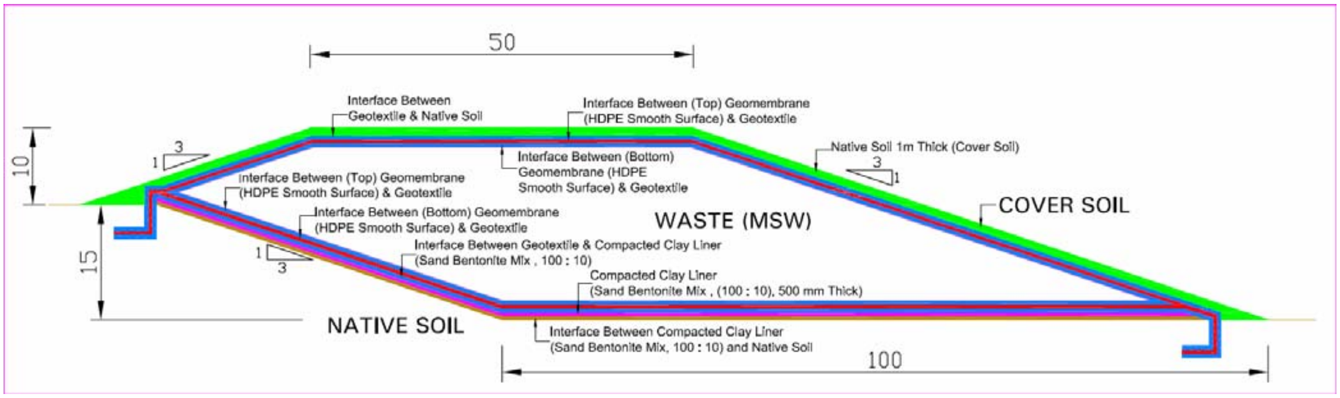

Fig. 12 : Typical section of landfill which was used for stability analysis

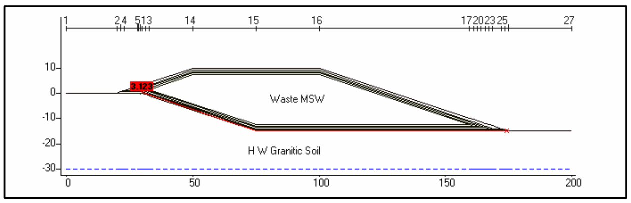

Fig. 13 : Typical failure section within bottom liner for Case 1 to 5

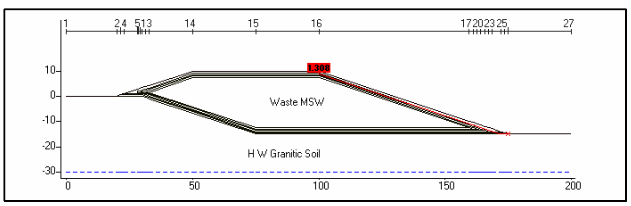

Fig. 14 : Typical failure section within landfill cover for Case 6 to 9

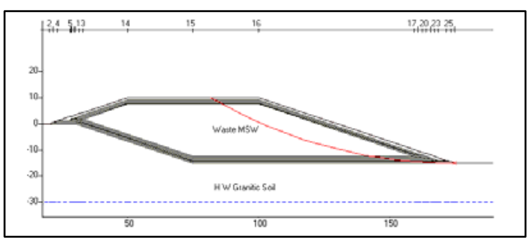

Fig. 15 : Toe failure of waste - Case 10

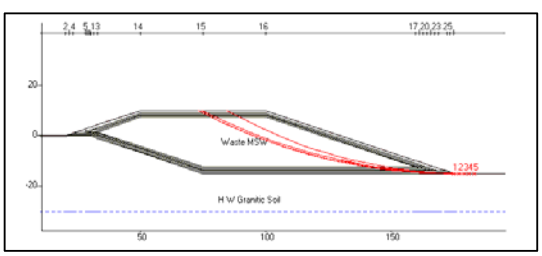

Fig. 16 : Overall landfill failure - Case 11

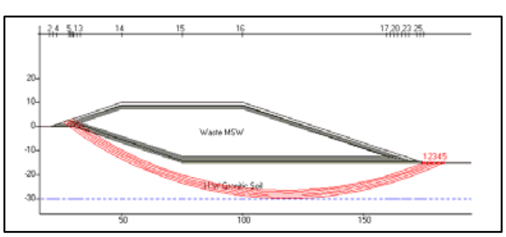

Fig. 17 : Overall landfill base failure - Case 12

Figures 13, 14, 15, 16 and 17 shows the typical analysis results for the cases listed in Table 2. Seismic horizontal coefficient of 0.1, 0.15, 0.2 and 0.25 were introduced in the analysis to study the trend of liner interface performance under earthquake loading. Based on the analysis results presented in Figures 18, 19 and 20, critical cases are 5, 7 and 8, which shows the interface between HDPE Type 1 (smooth surface) and geotextile. This is however consistent with interface Test 1A and 1B which have the lowest coefficient of friction as shown in Figure 11. However in the case of landfill cover, interface between goetextile and coversoil (Case 6), has high potential of failure during seismic loading. Similar condition of Case 3 in bottom liner is much stable as compared to Case 6 of liner cover. In the case of internal and overall stability of landfill, the factor of safety (FOS) obtained are relatively stable under both static and seismic loading.

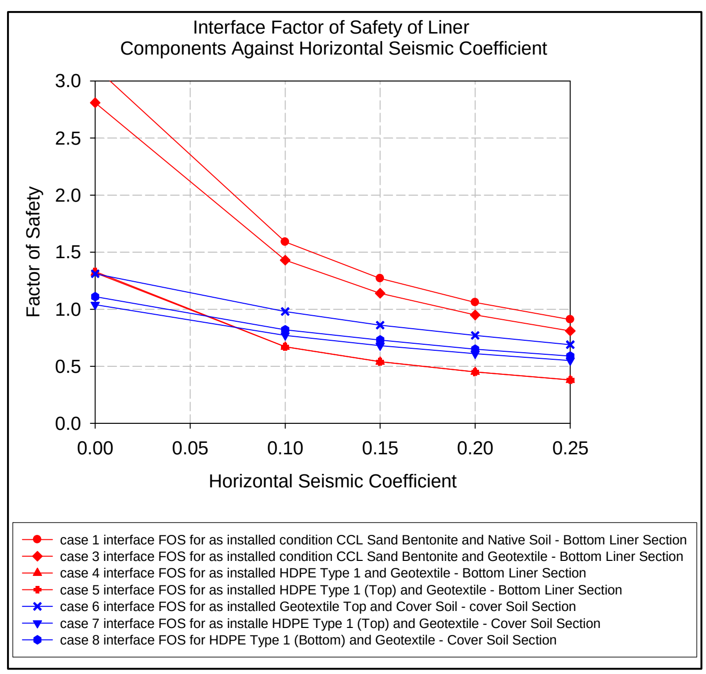

Fig. 18 : FOS performance for interface failure under seismic influence

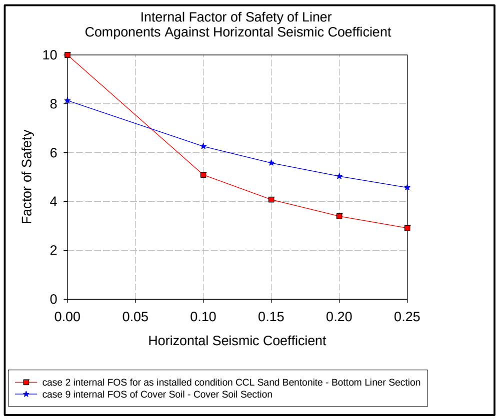

Fig. 19 : FOS performance for internal failure under seismic influence

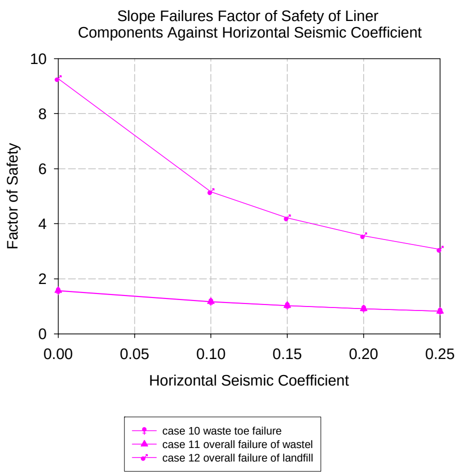

Fig. 20 : FOS performance for overall stability under seismic influence

7 CONCLUSION

Interface shear strength parameters obtained are much lower then anticipated. The mode of failure for various interface test combinations, shows that there is no specific trend of failure. 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 listed in Table 1 are much reliable to be used for stability analysis. The information are summarized and presented in Figure 11, and it can be used for selection of appropriate and cost effective landfill configuration prior to stability analysis for detail design. Example the use of suitable geosynthetic locking method can be decided based on data presented in Figure 11. As for stability analysis, interface between HDPE type 1 and geotextile is critical in both bottom liner and liner cover under seismic condition. However interface between geotextile and cover soil is also critical for liner cover. Similar condition of Case 3 for bottom liner is much stable as compared to liner cover condition. This shows the influence of vertical loads (fill height) is essential during seismic loading. Hence there is a need to investigate an alternative and design much improved interface material to be used when normal loads (fill height) are low. The data presented in Figure 11 will be updated further to make it as an immediate and quick reference guide for engineers in selecting the landfill liner materials. More data of interface test results under saturated condition will be included in the future.

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