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Interface shear strength of compacted clay liner with parent foundation soil of Turamdih dam site and some geo-textile materials in composite liner system

The generation of large quantity of tailing from different ore mines needs an environmentally friendly manner and safe way of disposal. In the case of uranium mineralization the concentration of uranium is very low as compared to other ores deposits and hence the requirement of comparatively larger space for storage of uranium tailing is inevitable. Intense public concern regarding the environmental and health effects of uranium tailings has forced a re-evaluation of past disposal practice. Seepage from bottom and sides need to be protected to avoid underground water pollution to the nearby surrounding areas.

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 geo-membranes together with interspersed drainage layer. The basic environmental guide lines have contributed in developing suitable liner and hydraulic barriers for landfill site.

The present study involves selection of alternatives between locally available soil of Turamdih dam site with different % of bentonite and fly ash. The choice among them is presented by assessing geotechnical properties of different alternatives. This work addresses the study conducted on performance of landfill liner interface parameters. The stability of composite liner system is based on the evaluation of interface shear strength between (i) Soil and CCL (ii) Geo membrane (HDPE or PVC) and soil (iii) GCL/CCL and soil (iv) Geo-textile and soil (v) GCL/CCL (vi) Geo-membrane and GCL/CCL. The current testing procedures are based on ASTM testing guidelines, where large scale shear box apparatus is proposed to be used for the tests with certain modification.

Keywords: *Landfill liner interface, composite liner system, shear strength, optimum moisture content, large scale shear box.*

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I. Introduction

In present day situation, the single liner system made up of compacted clay liner (CCL) is not sufficient due to various reasons as outlined by various workers [20] and [6]. As a result the researchers started thinking about the use of composite liner system to avoid failure of tailing pond. In case of a composite liner system different interfaces are developed in between different layers and the overall stability of liner system depends upon interface shear strength between layers. Composite liner system consists primarily of compacted clay liner placed on parent/natural soil with layers of various geo-synthetics with their purposes are as follows:

GCL/GM	:	Hydraulic barrier
GN	:	Drainage layer
GT	:	Filter layer
GG	:	Reinforcement

The overall performance of composite liner system depends on interface parameters as their interface peak and residual friction angles are lower than those of soil alone although they function effectively as hydraulic barrier against leachate and infiltration. The soil-geo-synthetic interface acts as a possible plane of failure of the system under both static and seismic leading [15]. This is the main reason to discuss the interface shear strength of landfill liner materials [24], [9], [5], [8], [10].

Initially landfill liner system were constructed under dry/OMC but due to the contact of CCL with leachate/groundwater, it is subjected to saturated/wet condition (SWC). Thus, effect of water content of the lining material on the interface shear strength parameters should be considered carefully. The objectives of this study is to obtain the interface shear strengths between geo-synthetics with CCL and CCL with local soils using the laboratory large direct shear tests under both OMC and SWC. The results from interface were utilized to evaluate the stability of the landfill liners with different methods. Based on the test results, effect of the water content of the liner material on the interface shear performance has been discussed and summarized.

TABLE 1 SUMMARY OF THE GEO-SYNTHETIC PHYSICAL PROPERTIES (SOURCE: ALIET ALL, 2012)

Materials	Geo-textile	Geo-membrane	
Description	(non-woven type)	HDPE type 1 (smooth surface)	HDPE type 2 (blown film textured type)
Man index (gm/m^2)	≥ 1070 (JIS-L-1908)	≥ 1400 (ASTMD-1505)	≥ 1550 (JIS-L-1908)
Thickness (mm)	≥ 10 (JIS-L-1908)	≥ 1.5 (ASTMD-5199)	≥ 1.5 (JIS-K-6250)
Tensile Strength (N/mm)	≥ 16 (Weft CD)-1908) ≥ 8 (Wrap MD)-1908)	≥ 43 (ASTMD-638) both weft and wrap	≥ 544 (JIS-K-6251) both weft and wrap
Elongation at break (%)	≥ 55%(Weft CD) -1908) ≥ 70% (Wrap MD) - 1908)	≥ 700% (ASTMD-638) both weft and wrap	≥ 790 (JIS-K-6251) both weft and wrap
Tear Strength (N)	≥ 200 (Weft CD)-1908) ≥ JIS – L - 1096	≥ 187 (ASTMD-1004) both weft and wrap	≥ 289 (JIS-K-6251)
Penetration (N)	≥ 1000N ≥ (ASTMD-4833-88)	≥ 530 (ASTMD-4833)	≥ 539 (ASTMD- 4833-88)

Weft – Cross Direction, Wrap – Machine Direction

II. Geotechnical and environmental aspect of liner system

The assessment of engineering properties of landfill component to design a stable landfill site against various mode of failure along with the contamination to environment is main concern of a geo-technical aspect. The failure takes place along low friction angle zone between different interfaces under any conditions static or seismic (Daniel et al 1998). The weakest interface identified, is generally lower between woven geo-textile component of CCL and the adjacent materials. For composite liner system consisting of cohesion less interfaces with no seepage on infinite slop, the factor of safety is given by

$$F = \frac{\tan \phi}{\tan \beta} \qquad \phi = \text{Angle of internal friction}, \quad \beta = \text{Angle of slope}$$

Strain incompatibility between geo-synthetic interfaces with adjacent material suggests the use of residual shear strength instead of peak strength in stability analysis of liner system.

Proposed landfill liner system for interface testing:-

Protective sand layer

Non-woven geo-textile

Geo-membrane (HDPE/PVC)

Geo-textile (500 gm/sqm)

Compacted clay liner

Existing soil recompacted to 90% standard (native soil)

III. Test materials

A. FOR DESIGNING CCL

(i) Local soil: The soil was collected from open excavation from a depth of 1m from natural ground level near physics dept. of Turamdih dam site. The soil was air dried and used after sieving through IS: 425 micron sieve. It contains non-expansive clay mineral as chief component.

(ii) Fly ash: The ash was obtained directly from open dumped at the thermal power plant of Bokaro, Jharkhand state. The chemical composition of fly ash is very close to class c-fly ash.

(iii) Bentonite: Commercial bentonite from open market was used for this study. It contains montmorillonite clay mineral shown by the x-ray diffraction. The CEC of bentonite was 108 meq/100gm which was constituted by both sodium and calcium almost equally.

B. FOR INTERFACE SHEAR STRENGTH DETERMINATION

(i) One type of non-woven geo-textile

(ii) Geo-membrane

(a) HDPE geo-membrane

- Type 1 – Smooth non textured
- Type 2 – Textured membrane

(b) PVC geo-membrane – Smooth non textured.

(iii) Clay liners

(a) Local soil + 10 % Fly ash + 10% Bentonite.

(b) Local soil + 20 % Fly ash + 10% Bentonite.

(c) Local soil + 20 % Fly ash + 20% Bentonite.

Physical properties of geo-synthetic materials are presented in Table 1 where as physical properties of local soil and CCL's alternative are presented in Table 2.

(A) 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 guidelines of ASTM D 3080 – 98 i.e. standard test method for direct shear test of soils under consolidated drained condition, ASTM D 5321–02 i.e. standard test method for determining the co-efficient of soil and geo-synthetic or geo-synthetic and geo-synthetic friction by direct shear method and ASTM D 6243–98 i.e. standard test method for determining the internal and interface shear resistance of geo-synthetic clay liner by the direct shear method.

TABLE 2 SUMMARY OF THE PHYSICAL PROPERTIES OF NATIVE BASE SOIL AND CCL'S TESTING PARAMETERS

Soil combinations	Local soil	F	G	H
L L (W_L %)	28.86	36.28	35.23	40.38
P L (W_P %)	20.32	24.21	24.12	26.28
P I (I_P %)	8.54	12.07	11.11	14.1
S L (W_S %)	16.91	17.47	16.25	19.34
Specific gravity (G)	2.47	2.51	2.51	2.55
Dry density (γ_d)(KN/m^2)	18.75	21.057	19.67	21.5
OMC (%)	15.25	14.31	14.63	14.2
Classification (as per USCS)	SW-SM	SC	SC	SC
Hydraulic conductivity (cm/sec)	$1.45X10^{-5}$	$8.6X10^{-7}$	$7.3X10^{-7}$	$2.15X10^{-8}$
Total cohesion (Cu) KPa	43.921	58.75	54.21	64.38
Total friction angle (ϕ^o)	23°7'	18°31'	31°3'	16°2'

F = Local soil + 10% Fly ash + 10% bentonite
G = Local soil + 20% Fly ash + 10% bentonite
H = Local soil + 20% Fly ash + 20% bentonite

As per the above guideline and testing requirement, the apparatus design has been subdivided into three categories. First is the soil and soil internal and interface testing apparatus to perform test on interface shear strength between native soil and CCL as well as internal shear strength of native soil and CCL. The second is the geo-synthetic and geo-synthetic internal and interface testing apparatus to perform test on internal shear strength evaluation of GCL, geo-membrane and geo-textile, geo-textile and geo-synthetic clay liner (GCL) and geo-membrane and GCL). The third one is the geo synthetic and soil interface testing apparatus to perform test on geo-membrane and native soil /CCL, GCL and native soil and geo-textile and nature soil/CCL.

(B) TEST APPARATUS

The large shear box referred to here is a machine designed for testing specimen having bottom and top shear box of different sizes. The size of opening between two halves of direct shear box apparatus is maintained approximately 10-20 times the mean particle diameter of the tested soil particle in order to produce shear bands of thickness observed in plain strain compression tests and to subject the thin soil element of the middle height to a simple shear mode of deformation. Here the opening gap of about 1 mm between the two halves was considered as acceptable to minimize the impact of the apparatus on the interface shear strength. [19].

The size of bottom shear box was 350 × 600 mm and top box size is 250 × 500mm. The difference of 100 mm in size of bottom box was set to allow 20% lateral displacement relative to top box length (500 mm) during the shearing with the constant contact area of 250 × 500mm. The rate of shearing employed was 1 mm/min with the normal vertical loads of 100, 200 and 300 KPa which is univalent to up to 20m-height landfilling based on the assumption that the wet density of the reclaimed waste is 15 KN/m^3. Geo-synthetic is clamped by flat jaw like clamping device at three sides of bottom box to prevent any sliding of geo-synthetic during shearing. The displacement rate is controlled by a speed controller with a multiple speed gear box. Normal load up to 300 KPa could be applied by a pressure plate using a hydraulic system. The value in pressure gauge was checked from time to time during shearing to ensure that the pressure remained constant. A load cell of capacity 90 KN with accuracy of +2.5 N was fitted to the tail stock, which is attached to the top box, to measure the shear resistance of the interface. In addition, a displacement transducer of 110 mm travel with accuracy of + 0.02mm was used to monitor horizontal displacement of top box while a displacement transducer of 50 mm travel with accuracy of + .005 mm was used to monitor vertical displacement of specimen. To enable a continuous record of the test, a data logger was used to lag the data in every 15 sec until the test was terminated at horizontal displacement of 100 mm.

(C) EVALUATION OF TESTING CASES

The interface test results indicate different kind of failures at different levels of relative displacement. The maximum shear stress was either maximum shear stress or the maximum shear stress reached within 8% of relative displacement [17]. Based on the selection criteria, the use of peak or residual interface strength is proposed to be assessed within the prescribed horizontal strain value of 8%. This is due to some of the test results showing higher residual interface strength caused by horizontal strain hardening effect. Hence selection is purely based on peak or residual interface strength in some cases, which could over or under estimate the interface resistance. Thus the selection of maximum shear stress within 8% horizontal strain was used as criteria of land fill liner failure limit, were potential geo-membrane tearing and may lead to leachate pollution to environment horizontal strain was used to identify shear stresses in place of displacement, as the test results can be compared to the test. Various other shear box sizes has been reported by [12]. The selected shear stresses obtained were plotted against normal stresses to compute the failure envelope. To determine the total cohesion and total interface friction angle, best fit liner plots were developed. List of the test cases conducted and the interface shear strength parameters obtained are presented in Table 3. The effect of water content (OMC&SWC) on the interface shear performance between geo-synthetics and CCL/ Foundation soil and CCL and foundation local soil has also been evaluated. The interface test result obtained are proposed to be grouped into following strength categories.

TABLE 3

Friction (0)degree	Cohesion (KPa)	Strength
0 - 10^o	0 - 10	Low
10^o - 20^o	10 - 20	Medium
> 20^o	> 20	High

4 (D) HYDRATION METHODOLOGY

In order to saturate the native soil and CCL, vacuum pressure was used. The compacted soil samples were placed in a vacuum chamber. The compacted soil samples were placed in a vacuum chamber with maximum negative pressure between 50 and 60 KPa for 48 h. In the case of GCL it was carried out in shallow pan full of water for 48 h with 1KPa normal aerial load [8].

V. Test results and discussion

The performance of CCLs with geo-textile depends mainly on frictional contribution with little on cohesion. The performance of geo-textile produced frictional angle from 130 to 290 degree for all combinations of CCL. The peak shear stresses were reached within horizontal strain of around 5 to 6%. There were spots of tearing and total internal failure of geo-textile, which took place for higher normal loads of 200 and 300 KPa. Continuous reduction in the shear stress was observed until constant residual shear stresses obtained beyond 10% strain. In all normal stresses there are no pre peak, slippage or plowing effect taking place before peak stresses reached. Continuous increment in shear stresses was observed beyond peak stresses into residual region. Sometimes the geo-textile splits into two during the test. The frictional contribution from the interfaces is best for alternative H. The test results are not as predicted due to the presence of bentonite and fly ash with higher damages and created on interfacing member during shearing. The results are tabulated in Figs.1 to 3.

The performance of CCLs with geo-textile under saturated or wet condition is similar to OMC, having higher frictional contribution and cohesions. The peak shear stresses are produced at horizontal strain of 8%. There were no sports of tearing and internal failure of geo-textile took place for all normal loads. Continuous increment in the shear stresses was observed for low normal stresses of 100 KPa, Beyond horizontal strain of 10-15%, the reduction in shear stresses occurred at higher loads. The results are presented in Figs.4 to 6.

As for the total stress readings from transducers installed at center of CCL and at the perimeter wall of the shear box, it was observed that at about 3-13% horizontal strain, sudden increment in shear stresses were observed at normal loads of 200 and 300 KPa. The transducer readings represent the behaviour and internal shear stresses together with pore pressures, which represent total stress within the CCL during interface shearing. This indicates the occurrence of complete failure in internal strength of CCL. As for 100 KPa normal stress the internal and perimeter total stress had no clear indication of CCL failure. To record the total stress, 4 transducers were installed in the shear box at top and bottom of box. Two numbers were installed at the box wall perimeter (P) and two numbers were installed at centre (I) of the interface to record the trend of total stress during interface shearing.

Interface between native soil and CCL were covered in wide range of friction angles with cohesion and frictional contribution. Details of test results are presented in Table 4 and Figs.7 to 9. The peak stresses were reached within horizontal strain of 7.8-8%. Constant residual shear stresses were observed in the residual region for all normal loads, beyond 6% horizontal strain. No plowing kind of effect under constant increment in residual shear stresses was observed in the residual region.

The peak shear stresses were limited within horizontal strain of 8%. Consolidation was done for all normal loads prior to interface shearing to disperse the initial pore water pressure built up. During interface test horizontal strain hardening effect was observed for all normal loads.

For the total stress reading at centre of native soil and CCLs, and at the perimeter it was observed that at all normal stresses an increment in total stress was observed except for native soil. The drop in interval total stress for native soil could be due to internal failure of native soil, which pushes out the soil mass away from centre.

Figs 13 to 15 show the behaviour of CCLs with smooth geo-membrane under normal stress of 100 to 300 kPa. There is an increase in shear stress as soon as displacement starts, with large horizontal displacement required mobilizing peak stresses, followed by loss of shear stress with further deformations. Peak shear stress were reached at horizontal strain of around 4% with shear stress reducing by 20 to 30% at horizontal strain of 8% except for combination (f). The peak shear stresses were more evident in all normal stresses, which were taken as the clearly defined peak interface shear stress, which shows the strain softening interface failure shear stresses behavior in all normal stresses.

Depending on the applied normal stress, mobilized shear stress then increased slowly in further large deformations. It was found that the higher the applied normal stress on to the testing specimen interface, higher the degree of reduction in shear stress at early stage while lower degree of increment of shear stress at later stage. Plowing effects of soil onto geo-membrane surfaces were observed in all normal stresses.

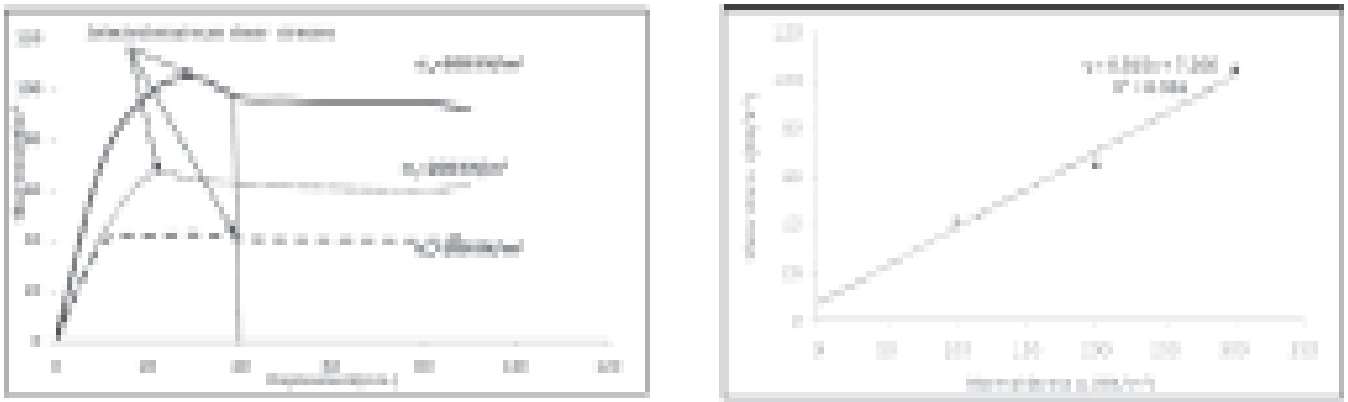
Geo-textile interfacing with compacted clay liners (CCLs) under dry or optimum moisture content (OMC):

Geo-textile interfacing with compacted clay liners (CCLs) under saturated or wet condition.

Native soil interfacing with compacted clay liners (CCLs) under dry or optimum moisture condition (OMC).

Native soil interfacing with CCLs under saturated or wet condition

$HDPET_1$ (smooth surface) interfacing with compacted clay liners (CCLs) under dry or optimum moisture content (OMC).

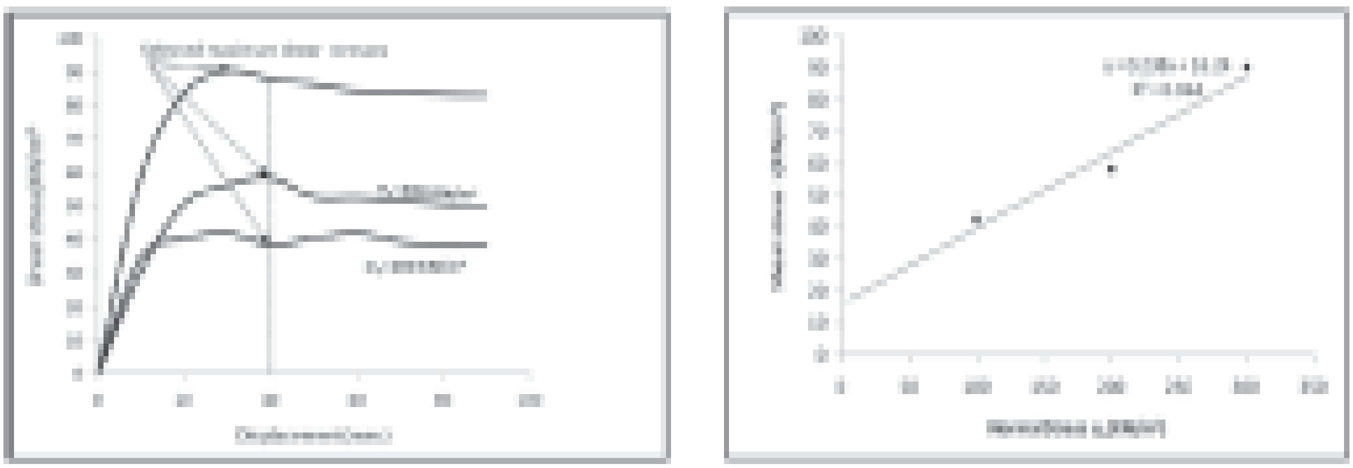


(a) Combination F (b)

Fig.1

(a) Combination G (b)

Fig.2



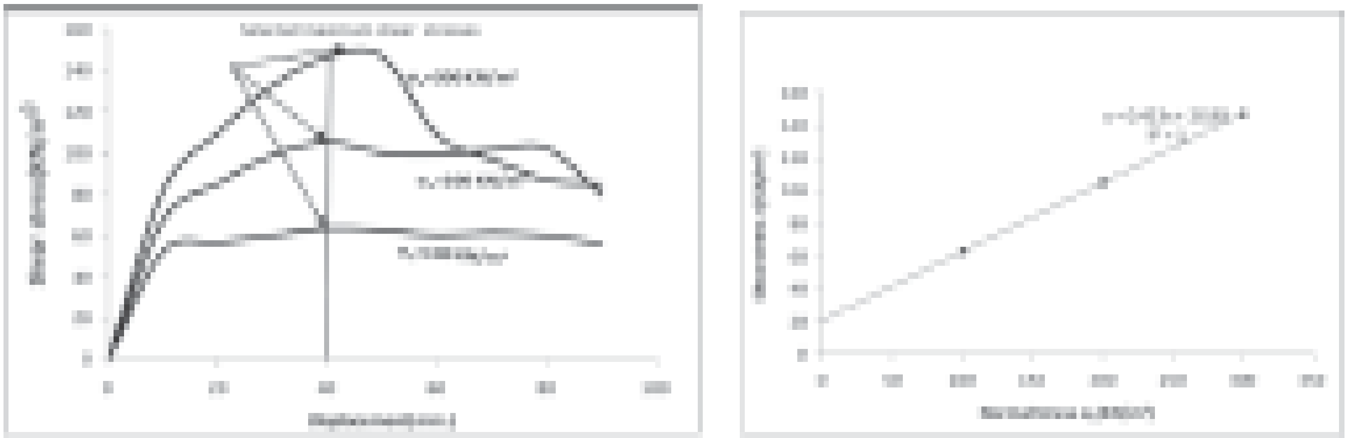
(a) Combination H (b)

Fig.3

HDPET$_1$ (smooth surface) interfacing with compacted clay liners (CCLs) under saturated or wet condition.

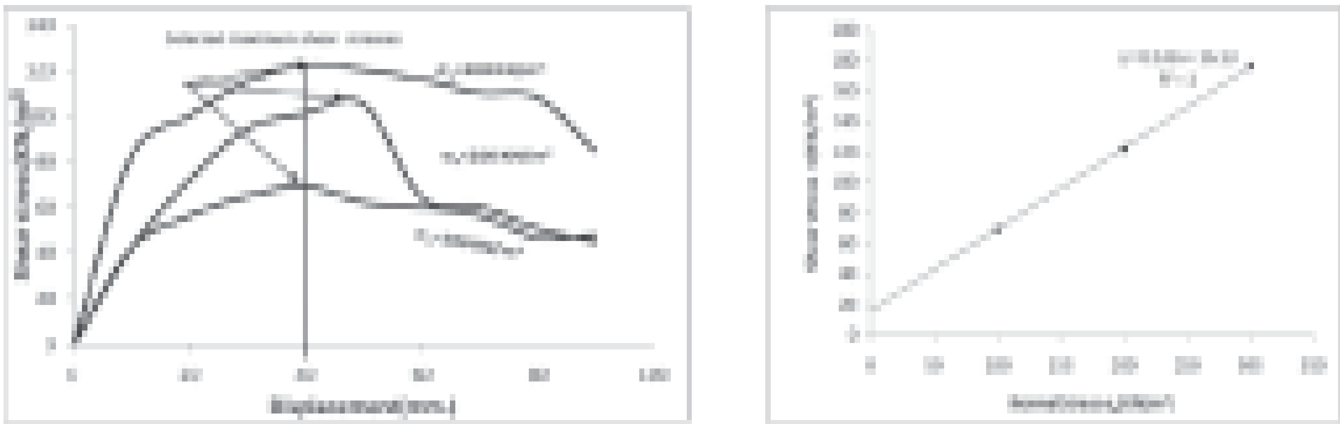
Figs.13(b) to 15(b) shows the coulomb's failure envelope. The interface gives friction angle of around 250 with little cohesion.

5.6..Figs 16 to 18 shows the interface shear stress displacement curves for interface of CCLs with smooth geomembrane under normal stress of 100 KPa to 300 KPa in saturated wet condition. There is no peak shear stress and interface failure shear stresses were taken at horizontal strain



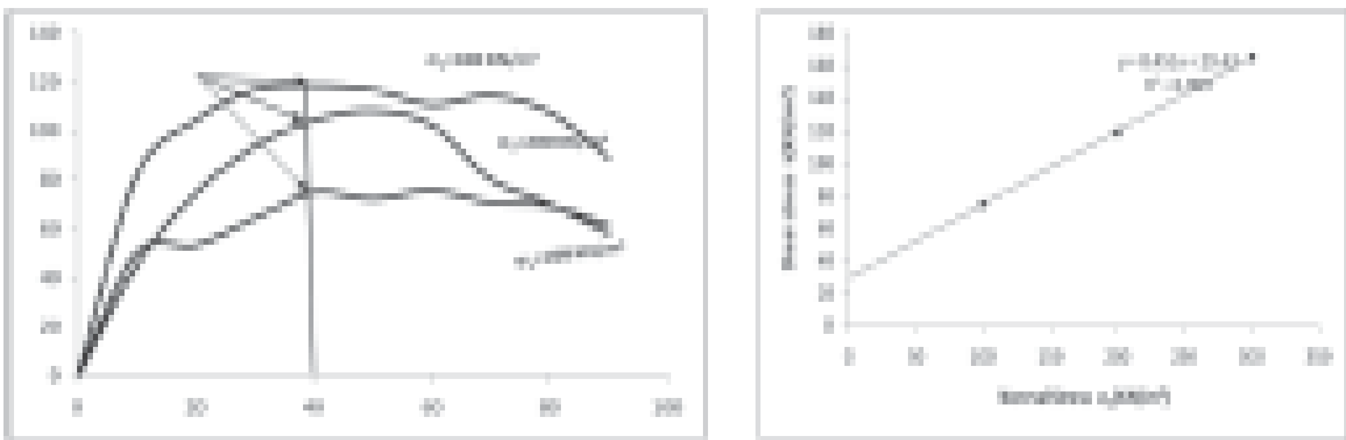
(a) Combination F (b)

Fig.4



(a) Combination G (b)

Fig.5

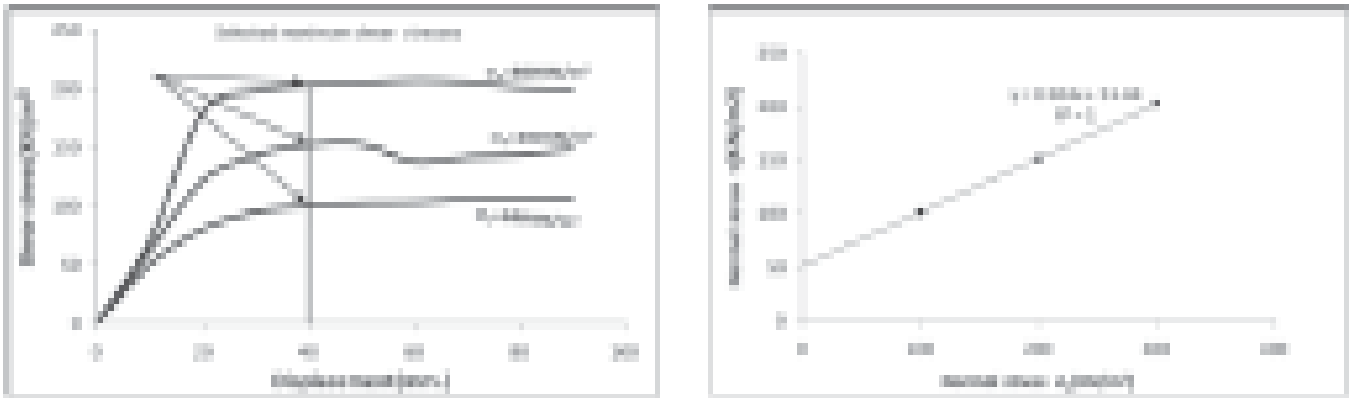


(a) Combination H (b)

Fig.6

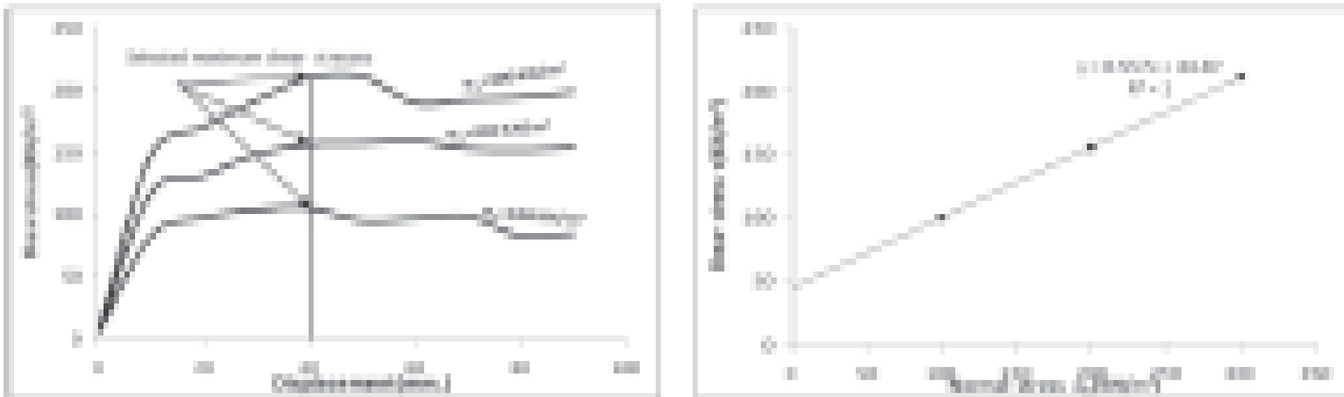
of 8% except for combination F. For normal stress of 200 KPa and 300 KPa, continuous increment of shear stress was observed from the beginning until a constant value of shear stress is reached. For the normal stress of 100 KPa, the interface exhibits strain hardening effect i.e. continuous increment of shear stress with horizontal displacement until the end of the test.

Significant stretching effect was observed at all normal stresses where geo-membrane surfaces became wavy as well as tearing of geo-membrane occurred at normal stress of 300



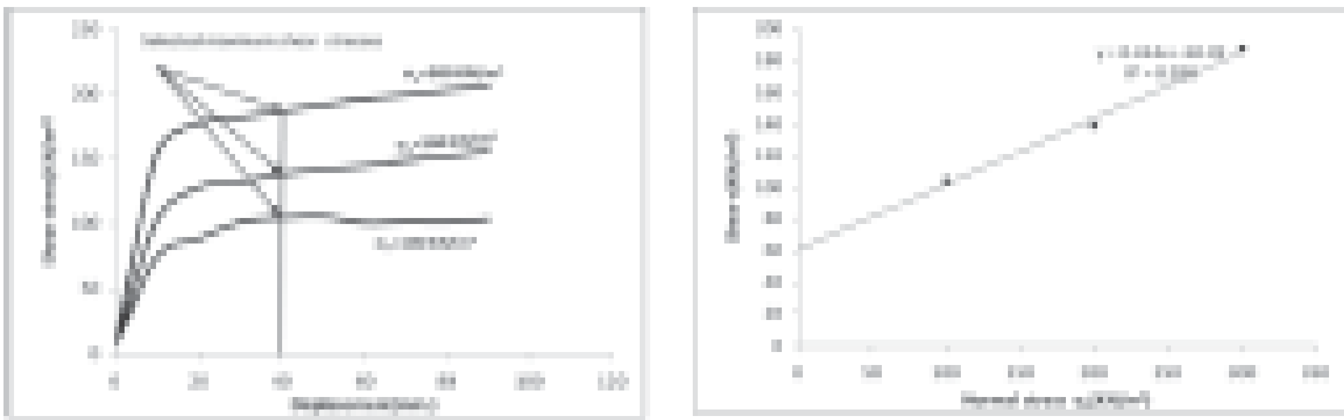
(a) Combination F (b)

Fig.7



(a) Combination G (b)

Fig.8



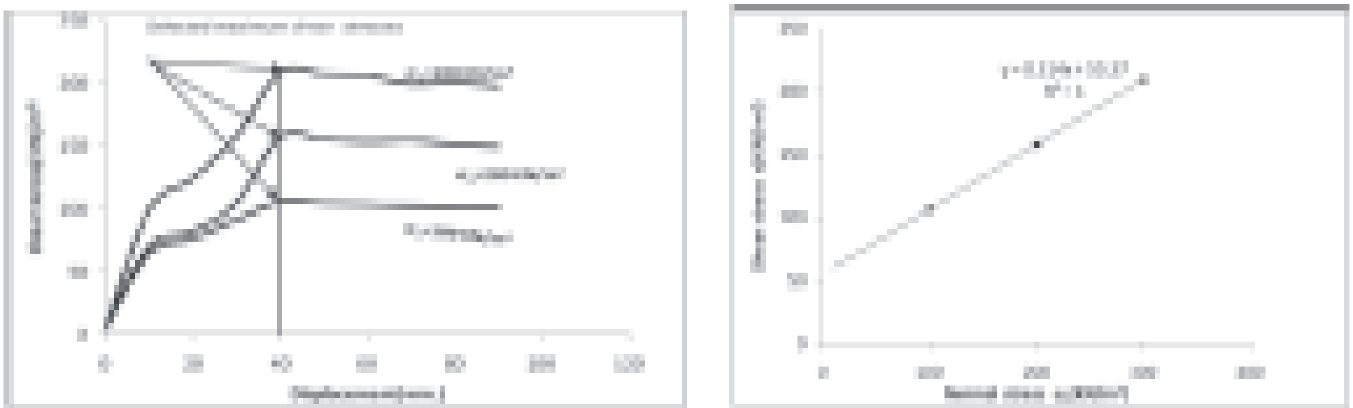
(a) Combinaton H (b)

Fig.9

KPa. This showed that good interface resistance between soil and geo-membrane was obtained. No plowing or slippage of geo-membrane occurred in all normal stresses and moderate soil mass attached on to the geo-membrane surface after tests. The interface gives friction angle of around 350 with little cohesion. Figs 16(b) to 18 (b) show the coulomb's failure envelope for the interfaces.

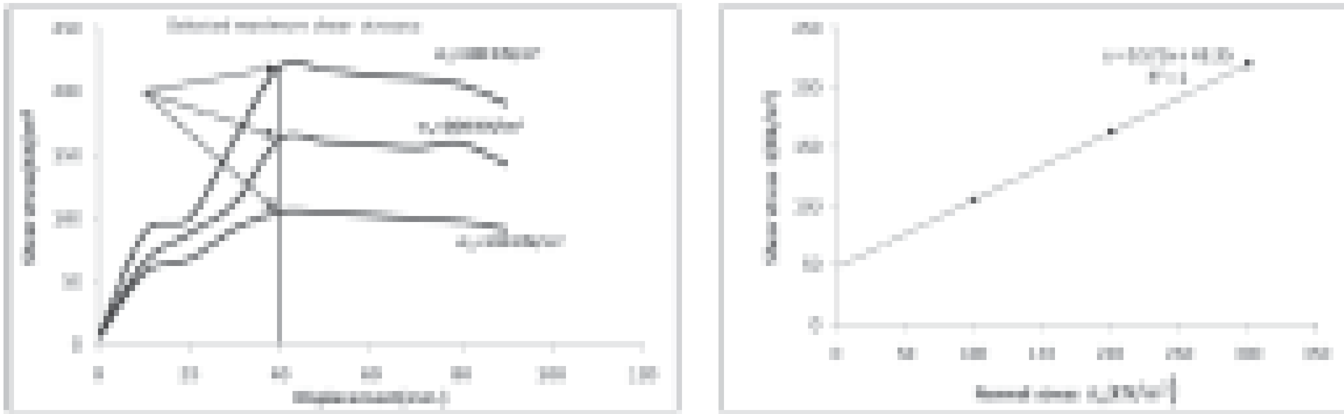
VI. Conclusions

The interface shear performance of landfill liner components under installed (optimum moisture content) condition and saturated/wet condition is based on the test results of the modified large scale shear test. The inferences can be summarized as follows.



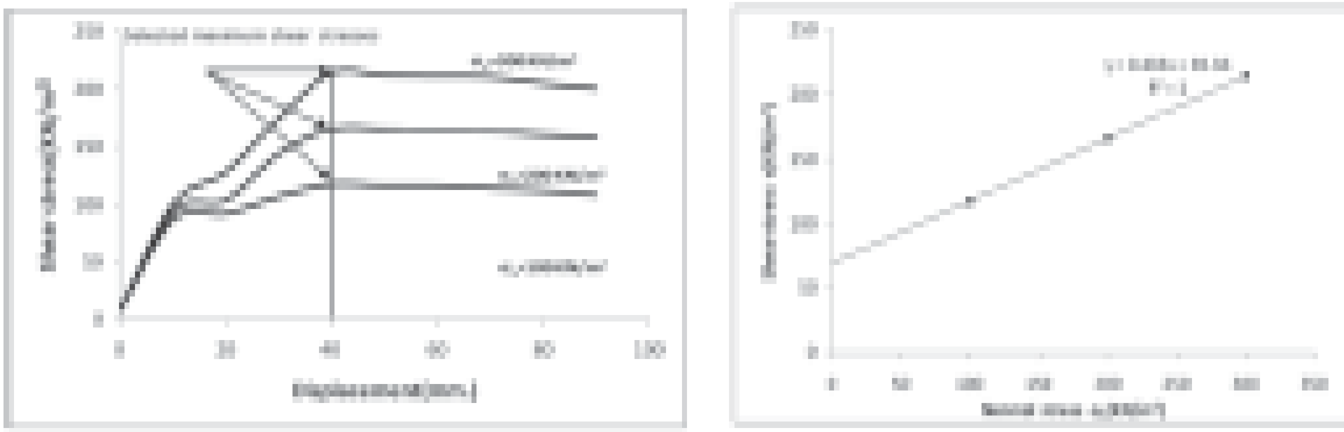
(a) Combiation F (b)

Fig.10



(a) Combination G (b)

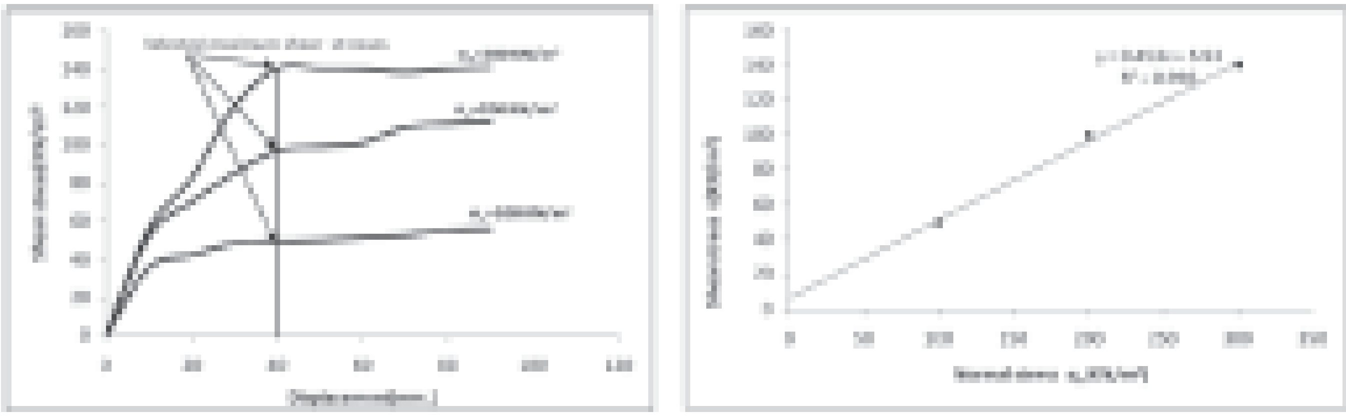
Fig.11



(a) Combination H (b)

Fig.12

1. The saturated/wet CCL – GM interface has higher shear strength compared to the interface under OMC. The peak shear stresses are not clear and horizontal strain hardening effect is observed under SWC. Especially, the frictional resistance of smooth HDDE – 2 geomembrane under SWC has some higher value from the value under OMC.
2. The saturated/wet CCL – GT interface has higher shear strength compared to the interface under OMC. The peak shear stresses are not clear and horizontal strain

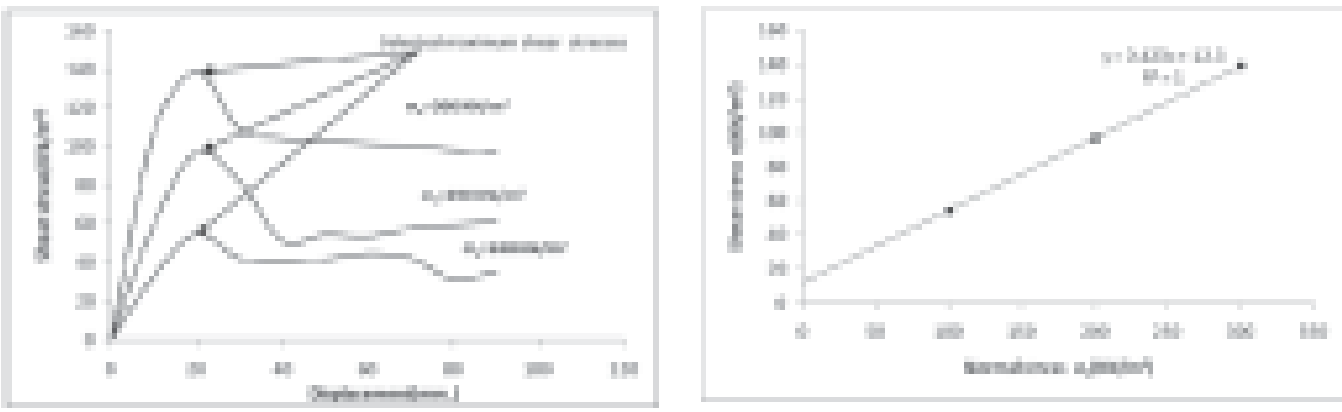


(a) Combination F (b)

Fig.13

(a) Combination G (b)

Fig.14



(a) Combination H (b)

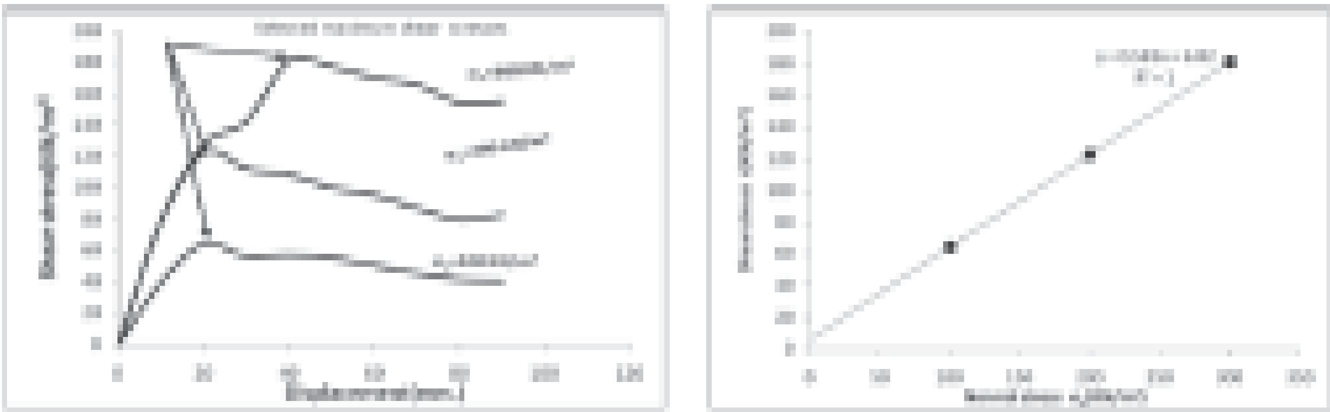
Fig.15

hardening effect is observed under SWC. The CCL – GT interface have improved value of cohesion and angle of internal friction in both condition than the CCL-GM interface.

3. The CCL combination of soil + 20% fly ash + 20% bentonite have marginally higher frictional contribution than other combination.

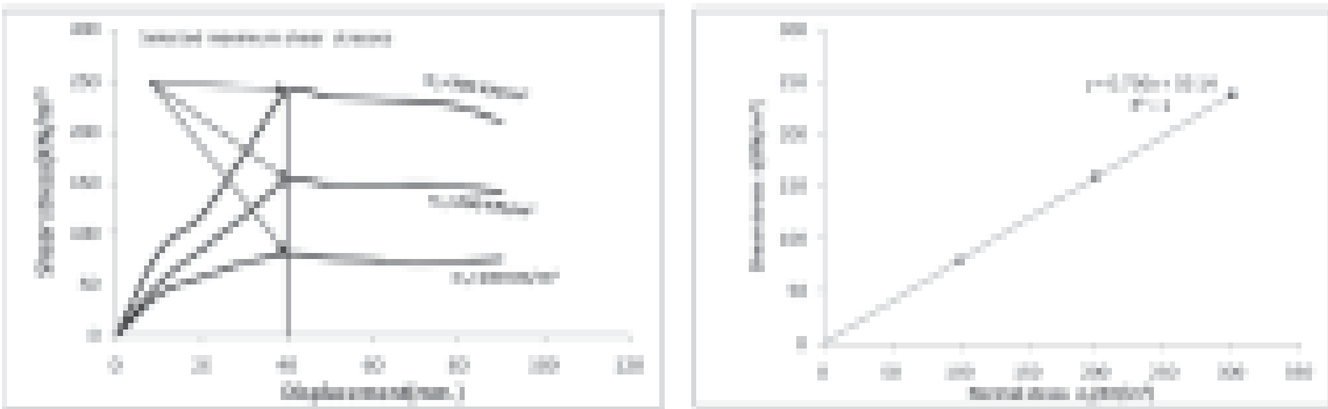
4. Compared to CCLs, foundation native soil is subjected to less significant influence on the interface performance under SWC. This is probably because the presence of bentonite in CCLs affects the interface property a lot under SWC.

5. Non-woven geo-textile maintains or enhance, the interface shear performance with all three CCLs and under



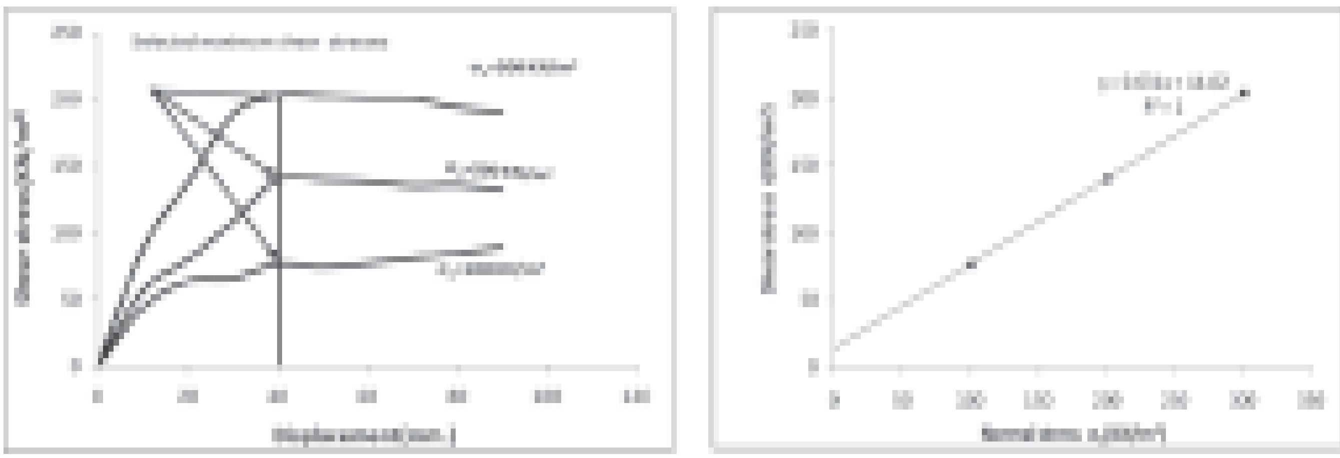
(a) Combination F (b)

Fig.16



(a) Combination G (b)

Fig.17



(a) Combination G (b)

Fig.18

saturated/wet condition (SWC).

6. Soil/geo-membrane interface frictional resistance could be improved by modifying the surface condition i.e. from smooth to textured surface.
7. The percentage of bentonite content (in low range) in the soil mixture had little effects on the interface shear stress – displacement performance.
8. The interface test result in saturated condition are lower for geo-membrane compared to geo-textile with CCLs. The C value for HDPE is lower whereas ϕ value increases by 35-28% than the geo-txtile interface with CCL.
9. Interface of native soil with CCLs at OMC and SWC have

following findings:

- Higher value of C with other interface.
- Higher value of C in SWC than OMC.
- Frictional resistance more or less same in OMC and SWC.

10. Geo-textile have higher frictional resistance under SWC as compared to OMC.
11. The combination H is the best keeping in mind the interfaces with any other combination.

Abbreviations:

GCL:	Geo-synthetics clay liner (hydraulic barrier)
CCL:	Compacted clay liner
GM:	Geo-membrane (hydraulic barrier)
GN:	Geo-net (drainage layer)
GT:	Geo-textile (filter)
HDPE:	High density polyethylene
ASTM:	American society for testing materials
SWC:	Saturated wet condition
OMC:	Optimum moisture content
PVC:	Polyvinyl chloride

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