Behaviour of Geosynthetics Clay Liner under Direct Shear Test

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Abstract. Geosynthetics clay liner system is a key component of the engineered landfill. Both the internal and interface strengths of geosynthetics clay liner are very important for evaluating landfill stability. This paper presents a study on interface shear strength behaviour between geosynthetics clay liner and sand; and geosynthetics clay liner and Powai soil making use of a direct shear test. The experiments were carried out using dry state and submerged state of sand, and optimum moisture content and submerged condition of Powai soil. The interface shear strength of geosynthetics clay liner and sand; and geosynthetics clay liner and Powai soil found to be lower than the corresponding shear strength of sand and Powai soil. The apparent adhesion was increased and interface friction angle was reduced during submerged condition.

Keywords: Geosynthetics clay liner, Interface shear strength, Direct shear test.

1 Introduction

Liner system is one of the most essential parts of the engineered landfill. The liner served as a hydraulic barrier to entrap the leachate within the landfill, with an aim to save ground water from being polluted [1-2]. Geosynthetic clay liners (GCLs) are one of the most prominent products in modern era to serve as liner, which is made up of bentonite, sandwiched between two geotextiles [3]. A land fill liner generally sandwiched between native/in-situ soil and leachate collection system. In-situ soil compacted to form subbase soil of land fill. Rounded stone and sand are commonly used as drainage materials in leachate collection system due to their high hydraulic conductivity [4]. Many landfills fail due to failure of liner system; hence many research scholars have investigated the interface shear strength [5-12]. But there is limited literature available on effect of submergence on interface shear strength [13]. Therefore, it is necessary to refined the knowledge of interface strength between geosynthetics clay liner (GCL) and different type of soils (i.e. sand (S) and Powai soil

(PS)) in submerged condition. This paper presents a study on effect of submergence on interface shear strength behaviour between geosynthetics clay liner (GCL) and sand (S) (i.e. GCL/S); and geosynthetics clay liner (GCL) and Powai soil (PS) (i.e. GCL/PS).

2 Testing materials

2.1 Powai soil

locally available silty soil commonly known as Powai soil, obtained from IIT Bombay campus in Maharashtra, was used in this study. The specific gravity of Powai soil was 2.46 as per IS:2720 (Part III/Sec 1) 1980 [14]. The grain size distribution curve is depicted in Fig. 1. It contains 3% coarse sand size particles, 16% medium sand size particles, 26% fine sand size particles, 38% silt size particles and 17% clay size particles [15]. Its liquid limit, plastic limit, and plasticity index was found out to be 39%, 27% and 12%, respectively [16]. The mean particle size (D_{50}), effective particle size (D_{10}), particle size finer than 30% (D_{30}) and particle size finer than 60% (D_{60}) found out to be 0.065mm, 0.0015mm, 0.01mm and 0.095mm, respectively. The coefficient of curvature (C_c) was 70.17 and the coefficient of uniformity (C_u) was 63.33. The Powai soil was classified as inorganic silt (ML) [17]. The Proctor compaction characteristics was Carried out as per IS:2720 (Part VII) 1980 [18]. The maximum dry density was 1.69 g/cm^3, with an optimum moisture content of 18%.

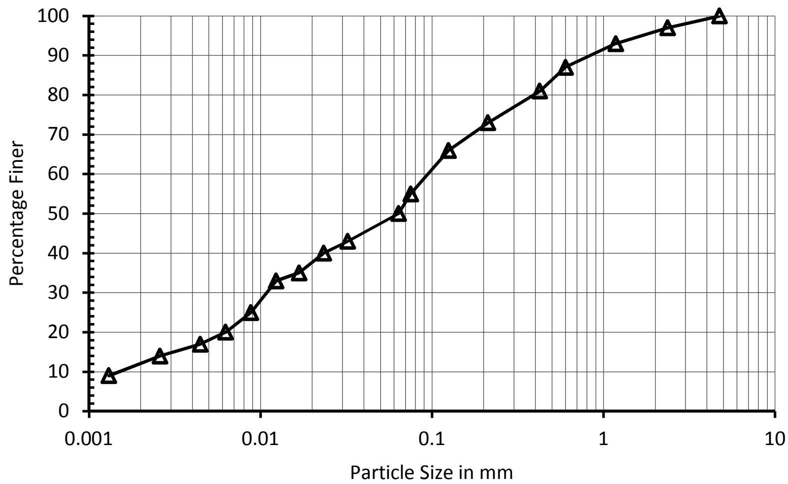


Fig .1. Grain size distribution curve of Powai soil

2.2 Sand

In the present study commercially available sand from market was used. The specific gravity of the sand was 2.64 as per IS:2720 (Part III/Sec 2) 1980 [19]. The particle size distribution curve is depicted in Fig. 2. It contains about 98% medium sand size particles and 2% silt size particles [15]. The mean particle size (D_{50}), effective particle size (D_{10}), particle size finer than 30% (D_{30}) and particle size finer than 60% (D_{60}) found out to be 1.00 mm, 0.67 mm, 0.80 mm and 1.20 mm, respectively. The coefficient of curvature (C_c) was 0.99 and the coefficient of uniformity (C_u) was 1.79. The sand was classified as poorly graded sand (SP) [17]. The Density index was Carried out as per IS:2720 (Part 14) 1983 [20]. The maximum density, minimum density, maximum void ratio and minimum void ratio was found out to be 1.53 g/cm^3, 1.32 g/cm^3, 0.96 and 0.69, respectively.

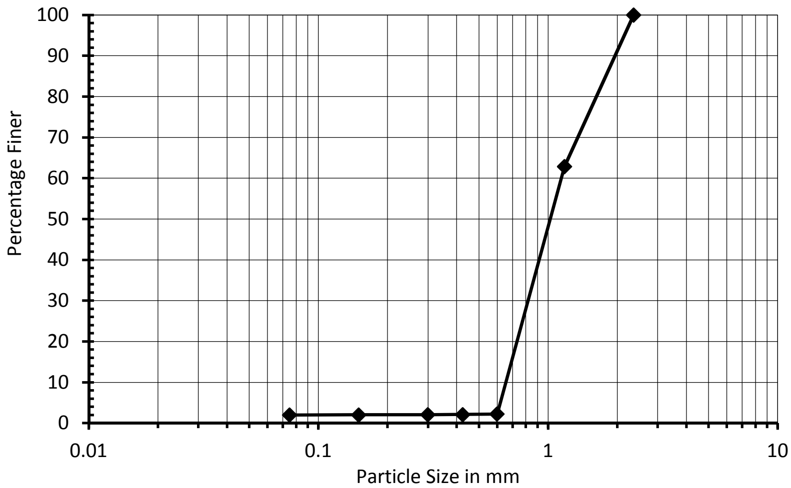


Fig .2. Grain size distribution curve of sand

2.3 Geosynthetic clay liners (GCLs)

Geosynthetic clay liners (GCLs) with Needle punched Non-woven Geotextile filled with granular sodium-bentonite were used in this study. The mass per unit area of GCL was 3800 g/m^2 as per ASTM D 5993-18 [21]. Its Thickness was about 5.95 mm [22]. It contains granular sodium-bentonite, and the Free swell Index of sodium-bentonite was 24 ml/2 gm [23]. The Tensile Properties was Carried out as per ASTM D 4595-17 [24]. The Tensile strength in machine direction and cross machine direction was found out to be 11.0 kN/m and 9.0 kN/m, respectively.

3 Testing methods

A direct shear device of size 60mm x 60mm, was used to measure the shear strength parameters of the soil to soil, and geosynthetics clay liners to soil interface [25]. Fig.3, shows the cross-sectional schematic view of sand to sand (or Powai soil to Powai soil), GCL/S (or GCL/PS) sample in direct shear box.

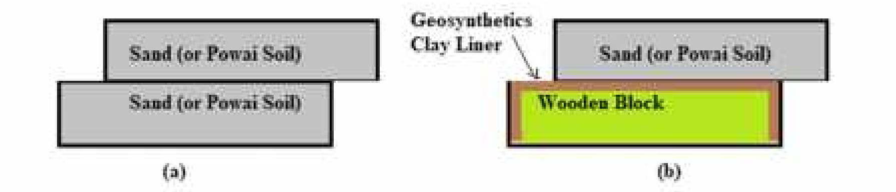
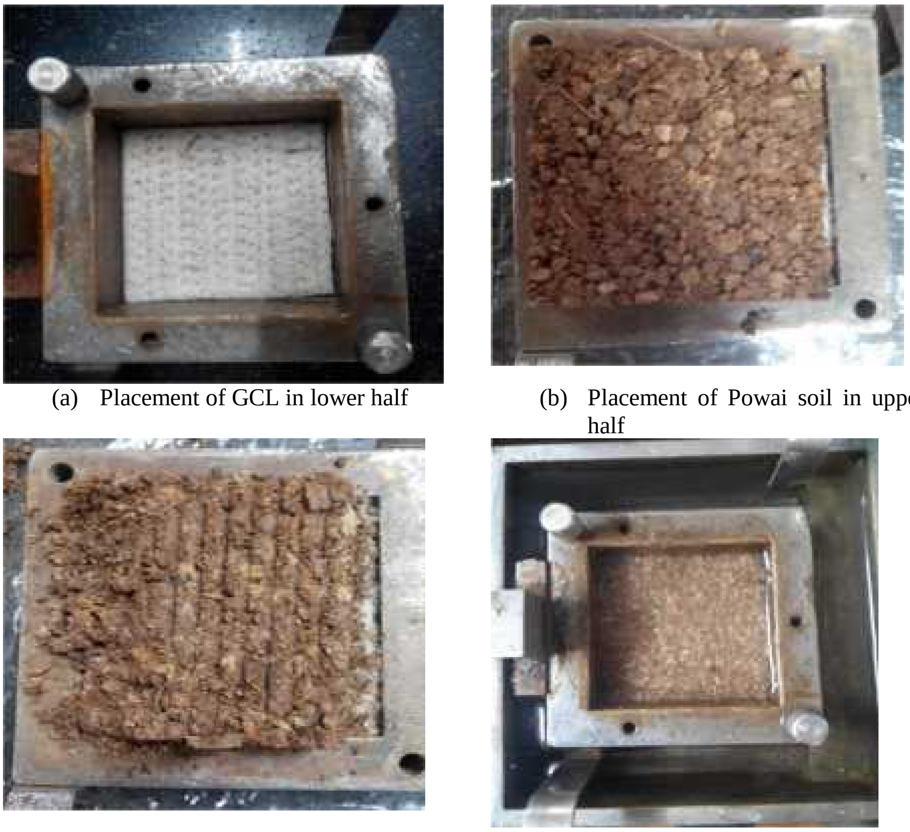


Fig .3. Schematic diagrams for (a) sand to sand [or Powai soil to Powai soil] and (b) geosynthetics clay liner to sand [or geosynthetics clay liner to Powai soil] direct shear tests

The samples were prepared in two batch. The aim of the first batch was to find the angle of internal friction of sand to sand and Powai soil to Powai soil. In case of angle of internal friction of sand to sand both the lower half and upper half of the direct shear box were filed with sand with 56% relative density (1.43 g/cm^3) (see Fig.3(a)). Similarly, for the case of angle of internal friction of Powai soil to Powai soil both the lower half and upper half of the direct shear box were filed with Powai soil with a dry density of 1.52 g/cm^3 and 15% moisture content (i.e. 90% of the maximum dry density) (see Fig.3 (a)). These (both sand to sand and Powai soil to Powai soil) samples were tested under two condition i.e. dry and submerged condition. In case of dry condition, the sample were sheared just after preparation of sample. But, in submerged condition, the soil sample were kept submerged for 24 hours in water jacket before shearing. For every sand to sand (or Powai soil to Powai soil), three tests were conducted by applying normal stresses of 50 kPa, 100 kPa, and 150 kPa. Tests were identified so that a sample tested in dry condition with a normal stress of 50 kPa was represented as 50(D) and the other sample tested in submerged condition was denoted as 50(W).

The second batches of the samples two interface systems were considered i.e. GCL/Sand and GCL/Powai soil. In second batch of samples a wooden box warped with GCL on its top was placed in the lower half of shear box (see Fig. 3(b) and 4(a)). And the top half was filed with sand with 56% relative density for GCL/S interface system (see Fig. 3(b)). Similarly, Powai soil was compacted in upper half of shear box to 90% of the maximum dry density for GCL/PS interface (see Fig.3(b), 4(b) and 4(c)). For both GCL/S and GCL/PS interfaces, three tests were conducted by applying normal stresses of 50 kPa, 100 kPa, and 150 kPa. These (both GCL/S and GCL/PS) samples were tested under dry and submerged condition, similar to first batch (see Fig. 4(d)).

A gap of 1 mm was maintained between lower half and upper half of shear boxes for all tests in order to avoid friction between the lower half and upper half. The upper half containing sand or Powai soil remained constant, whereas the lower half of shear box was moved with a strain rate of 1 mm/min. The shear force and horizontal displacement were measured by a proving ring and dial gauge respectively.



(a) Placement of GCL in lower half

(b) Placement of Powai soil in upper half

(c) GCL/Powai soil after shearing

(d) water jacket of GCL/Sand

Fig. 4. Interface and internal shear test specimen arrangement in the shear box.

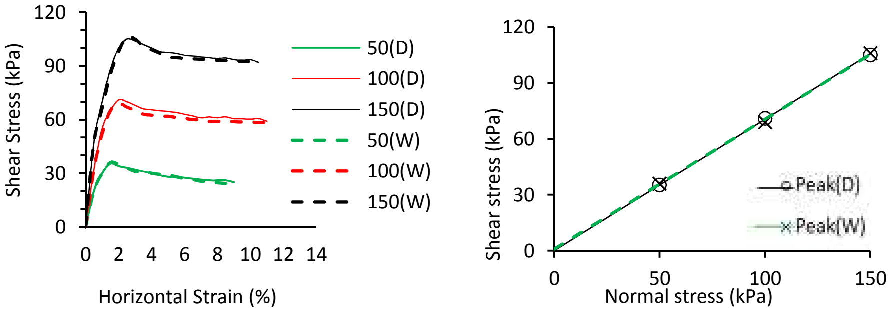
4 Results and discussion

4.1 GCL/Sand Interface Shear Strength

The variation of shear stress of sand to sand with different horizontal strain, obtained from the direct shear test is depicted in Fig. 5(a). From Fig.5(a), it is clear that sand in dry condition attained peak shear stress (35.32 kPa) at 1.5% horizontal strain, when sheared under 50 kPa normal stress. Similarly, the magnitude of peak stress is 71 kPa and reached at 2% horizontal strain, when sheared under 100 kPa normal stress. It is also clear that the magnitude of peak stress is 105 kPa and reached at 2.5% horizontal strain, when sheared under 150 kPa normal stress. The stress strain curves of submerged sand are similar to stress strain curve of dry sand under identical normal stress. The variation of shear stress of sand with normal stress, obtained from the direct shear test are presented in Fig. 5(b). From Fig. 5.b, it is clear that sand is a cohesionless material, hence its shear strength is mainly by governed by angle of internal friction. The cohesion and angle of internal friction of sand at 56% relative density are found to be 0 kPa and 35.22°, respectively, under dry condition. But, when the sand sample were kept submerged for 24 hours in water jacket before shearing, it develops a small apparent cohesion (1kPa). The effect of submergence on sand is negligible.

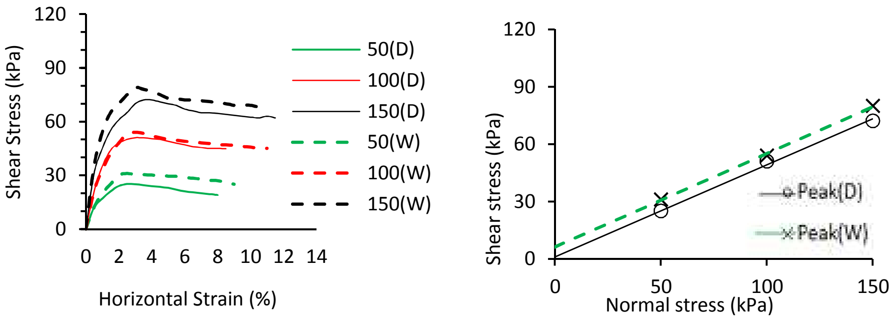
Fig. 6(a) and 6(b), represent the stress–strain curve and Failure envelopes of GCL/Sand interface. From Fig. 6.a, it is observed that the peak value of shear stress is reached at 2.5% horizontal strain for the normal stress of 50 kPa, at 3% horizontal strain for the normal stress of 100 kPa, and at 3.5% horizontal strain for the normal stress of 150 kPa for both dry and submerged condition. The magnitude of interface peak shear stress of GCL/Sand in dry condition is 25.01kPa, when sheared under the normal stress of 50 kPa. And, the magnitude of interface peak shear stress turns out to be51 kPa and 72.1 kPa, when sheared under 100 kPa and 150 kPa respectively. But, the magnitude of peak shear stress of GCL/Sand interface under 50 kPa, 100 kPa and 150 kPa normal stress turn out to be 31 kPa, 54 kPa and 80 kPa, respectively. This indicates that the shear stress of GCL/Sand interface increases when it subjected to submerged condition.

It is observed that the interface friction angle remains nearly same for both dry and submerged condition (i.e. 25.70° for dry condition and 26.10° for submerged condition), but and the value of adhesion increased slightly for submerged condition. The increased value of adhesion may be due to hydration of the bentonite [26-27]. The peak shear stress values in submerged condition are slightly higher than dry condition, because hydration of the bentonite in submerged condition gives extra adhesion to GCL/S interface.



(a). Stress–strain curve **(b).** Failure envelopes

Fig. 5. Direct shear test of sand to sand



(a). Stress–strain curve **(b).** Failure envelopes

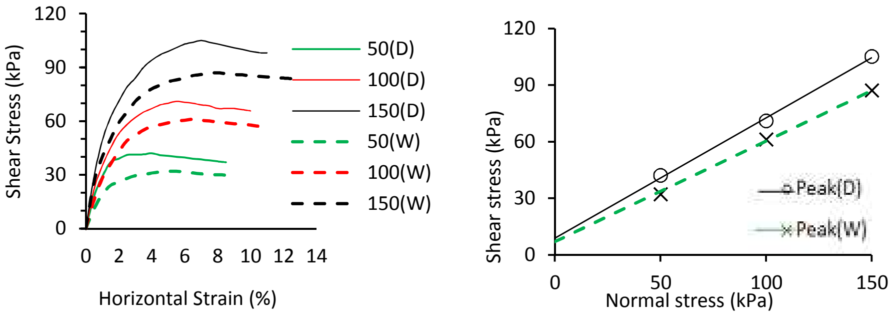
Fig. 6. Direct shear test of GCL/Sand interface

4.2 GCL/Powai soil Interface Shear Strength

The Stress–strain curve of Powai soil to Powai soil, obtained from the direct shear test is shown in Fig. 7(a). From Fig. 7(a), it is clear that Powai soil in dry state attained peak shear stress (42 kPa) at 4% horizontal strain, when sheared under 50 kPa normal stress. Similarly, the magnitude of peak stress is 72 kPa and reached at 5.5% strain, when sheared under 100 kPa normal stress. It is also clear that the magnitude of peak stress is 104 kPa and reached at 7.5% horizontal strain, when sheared under 150 kPa normal stress. The magnitude of peak shear stress under submerged condition turn out to be lower than the dry condition. From Fig. 7(a), it can be observed that the magnitude of peak shear stress is 32 kPa and appeared at 5% horizontal strain, when sheared under 50 kPa normal stress. Similarly, when the Powai soil sample sheared under 100 kPa normal stress, peak value of shear stress turns out to be 61 kPa at 6.5% horizontal strain. It is also clear that the magnitude of peak stress is 87 kPa and reached at 8% horizontal strain, when sheared under 150 kPa normal stress. Failure envelopes of Powai soil to Powai soil in dry and submerged conditions are presented in Fig. 7(b). The cohesion and angle of internal friction of Powai soil are 8.81 kPa and 32.33° when tested under dry condition. But, the cohesion and angle of internal friction of Powai soil decrease to 6.9 kPa and 28° when subjected to submerged condition.

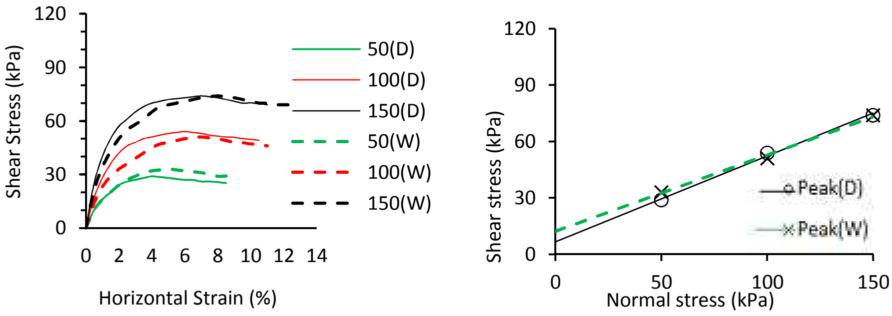
The stress–strain curves and Failure envelopes of GCL/Powai soil interface are shown in Fig. 8(a) and 8(b) respectively. When the sample sheared under a normal stress of 50 kPa, its peak value of shear stress become 29 kPa, at 4% strain. Similarly, for normal stress of 100 kPa and 150 kPa the shear stress become 54 kPa and 74 kPa, respectively. But, in case of submerged condition the peak value of shear stress occurs at little bit higher strain than dry condition (i.e. 8%, 7% and 5% horizontal strain for the normal stress of 150 kPa, 100 kPa and 50 kPa respectively).

Trends depicted in the Fig. 8(b), indicate that for Powai soil samples with GCL, the interface apparent adhesion increase, but the interfacial friction angle decrease, when it subjected to submerged condition. The results are similar to the results of the interface of GCL/Clayey Soil from the direct shear tests conducted by Chai and Saito (2016) [13].



(a). Stress–strain curve **(b).** Failure envelopes

Fig. 7. Direct shear test of Powai soil to Powai soil



(a). Stress–strain curve **(b).** Failure envelopes

Fig. 8. Direct shear test of GCL/Powai soil interface

4.3 Friction efficiency factors (E_{Φ})

The friction efficiency factors (E_{Φ}) were evaluated as per Mohr–Coulomb principle by using peak friction angle (Φ) of soil and interfacial friction angle (δ_{GCL}) of GCL/soil interface.

$$\text{Friction efficiency factors } (E_{\Phi}) = (\tan(\delta_{GCL}))/(\tan(\Phi)) \quad (1)$$

The peak friction angle (Φ) and interfacial friction angle (δ_{GCL}) evaluated for sand and Powai soil with GCL are presented in Table 1. It can be noted from the table that the friction efficiency factors (E_{Φ}) for submerged condition is higher than dry condition. These range of friction efficiency factors are consistent with the results obtained by Choudhary and Krishna [28], Lee and Manjunath [29], Evans and Fennick [30], Vaid and Rinnie [31], and Cazzuffi et.al. [32].

Table 1. Internal friction angle (Φ), interfacial friction angle (δ_{GCL}), and friction efficiency factors (E_{Φ}) values of GCL/Sand interface and GCL/Powai soil interface

Soil type	Φ(°)	δ_{GCL} (°)	E_{Φ}
Sand (Dry)	35.22	25.70	0.68
Sand (Submerged)	34.90	26.10	0.70
Powai soil (Dry)	32.33	24.57	0.72
Powai soil (Submerged)	28.00	22.10	0.76

5 Conclusion

This study presented the effect of submergence on the behaviour of GCL/S and GCL/PS interface through a series of laboratory tests. Considering the most important strength characteristics of the sand and Powai soil, and the GCL/S and GCL/PS interaction, the following conclusions can be made:

1.The effect of submergence on sand to sand is minimal and the angle of internal friction remain almost same for both dry and submerged condition. But, in case of Powai soil to Powai soil the angle of internal friction reduced by 4°.

2. It has been found that the Interface adhesion between GCL and soil (sand or Powai soil) increase significantly, due to hydration of bentonite. The interfacial friction angle (δ_{GCL}) found out to be lower than internal friction angle of soil (sand to sand or Powai soil to Powai soil).

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