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Interface shear strength properties of composite compacted silty soil liners using Cordoba local soils and smooth HDPE Geomembrane

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ABSTRACT
The environmental performance of waste disposal facilities lies in the correct design and construction of their liners. There are several liners design recommendations prepared by regulatory agencies. Typical liner design includes single liners and composite liners. Single liners are constructed by a layer of low hydraulic conductivity compacted soils with final thickness according to regulation's requirement or by a geomembrane. Composite liners include in their design combination of compacted soil layers and geosynthetic layers with different function. Most common composite liners are built by a compacted soil layer and a geomembrane. Hydraulic performance of liners lies on their hydraulic conductivity and the amount of leakage due to geomembrane imperfections, while mechanical performance depends on soil internal friction angle and soil – geosynthetic interface strength. Liner stability against sliding is of uttermost importance in side slope liners. It is a common practice to mix local soils with different amount of expansive clays in order to reach the target value of hydraulic conductivity. Clay addition affects soil shear strength and soil – geosynthetic interface strength. This research analyzes the effect of clay addition in shear strength of geomembrane – compacted soil liner. Effects of amount of clay addition, shear velocity and moisture condition are explored by means of direct shear tests between soil samples and a high density polyethylene smooth geomembrane.

1. INTRODUCTION

Sanitary landfills are primary Municipal Solid Waste (MSW) treatment facilities in developing countries. The objective of these facilities is to safely isolate MSW for preventing soil, groundwater and air contamination. The efficiency of contaminant isolation in landfills relies on the correct design of side and bottom liner systems and on the cover system. The liner system is the single most important element of a landfill. Both liners and covers system are composed of a series of layers constructed with different materials were each one of those has a specific function.

Typically, geomembranes (Gm), low hydraulic conductivity compacted clay liners (CCL), and geosynthetic clay liners (GCL) are employed as low hydraulic conductivity barriers having the primary function of contaminant retention. Non-woven geotextiles are used as protection layer and also in drainage layer. Sand layers are mainly employed as drainage layers.

There are available different liner and cover systems designs recommended by local and international regulations (USEPA 1993, Benson et al. 1999). Liner and cover systems can be designed as single or composite liners. A single liner is composed of either a CCL or by a single Gm. Composite liners are composed by two or more layers, combining CCLs and geosynthetics (Giroud and Bonaparte 1989). The common practice is to construct compacted clay liners on the prepared slopes of the natural ground and then place the geomembrane liners on the CCL surface. Subsequent construction of the landfill includes the placement of soil cover and waste layers up to various heights above the geomembrane liners (Dixon et al. 2006).

Because of the design criteria and construction methods, there are present many interfaces among geosynthetic products and geosynthetic/soil layers which can be potential failure surfaces, due to their low interface resistance. As a result, the soil/geosynthetic and geosynthetic/geosynthetic interface friction is an important variable in the proper design of cap and liner systems (Koutsourais and Sprague 1991, Bergado et al. 2006, Koerner 2012).

Liner stability gained importance after the Kettleman Hills landfill failure. The stability of landfill liner system is influenced by many variables and the most important factors that influence the multiple layer landfills lining system stability are:

- Interfaces shear strength between various geosynthetic materials.
- Interfaces shear strength between geosynthetics and soil materials
- Internal shear strength of solid waste

- Slope and height of waste fill during each lift.

After a deep investigation, Mitchell et al. (1990) determined that most critical interface combinations controlling the stability of the fill and liner system were the high density polyethylene geomembrane (HPDE Gm) / geotextile (Gtx), HDPE Gm / geonet and HDPE Gm / CCL under saturated conditions.

The most critical interfaces with the lowest frictional resistance in the nonhazardous landfill are the Gtx – Gm and the Gm – CCL. Failures result in additional costs, and at worst they can cause significant environmental damage (Dixon et al. 2006). After several experiences, it is clear that potentially critical interfaces are located between Gtx and Gm and Gm and CCLs (Masada 1994; Bergado et al. 2006).

All the properties of geosynthetics, except the interface shear strength (which depend upon the properties of the material in contact with the geosynthetic) can be controlled during the manufacturing process. Therefore it is of uttermost importance using the appropriate values of interface shear – strength parameters in the design of slopes incorporating one or more geosynthetic layers. It is widely accepted that those parameter must be experimentally determined for each particular project (Sia and Dixon 2007).

Compacted clay liners in Cordoba city and mostly in the central area of Argentina are constructed directly by local fine soil compaction. Local soil is mostly loessical silt with silt size particles and a few amounts of sand and clay particles (Francisca 2007). Because of its grain size distribution, particles form an open fabric with macro pores between silt and sand particles jointed by clay bridges and salt crystals. When loess is wetted, salts dissolve and clay tends to expand. Hence, soil macro-structure collapses (Kane, 1973). In the dry state, loess presents a rigid structure that can be excavated with slope angles as steep as 90º (Rinaldi et al. 2007). In undisturbed state, loess presents hydraulic conductivities around 1×10^{-5} m/s, while compacted hydraulic conductivity is close to 1×10^{-6} m/s. Nieva and Francisca (2007) found that in less than the 20% of cases it was possible to reach hydraulic conductivities less of 1×10^{-7} m/s for compacted loessical silts, and therefore in most cases it is necessary to amend the soil by adding bentonite and a Gm in order to accomplish common regulations for landfill liners (Montoro et al. 2015).

In spite, the Gm – CCL interfaces are among the most critical interfaces for liner stability, there are limited results for the interaction between cohesive soils and geomembranes, although they are used routinely in design (Ling et al. 2001).

The objectives of this research are to analyze changes of interface strength parameters for HDPE smooth Gm and CCL composed of compacted blends of loessical silty soils and bentonite, considering the effect of different bentonite content, the effect of shear velocity and considering also, two different moisture conditions, CCL samples at optimum compaction moisture content and under inundated conditions.

2. BACKGROUND

Interface shear strength can be determined by analyzing the failure shear stress at different normal stresses in a Mohr space. The interface shear strength can therefore be computed considering the Mohr – Coulomb failure criteria (Equation 1) (Koutsourais and Sprague 1991, Koerner 2012).

$$\tau = \sigma_n \tan \delta + \alpha \qquad [1]$$

Were τ is the shear stress on the failure plane, σ_n is the normal effective stress to the failure plane, δ is the interface friction angle and α is the adhesion intercept. In spite that, for nearly all soils, Mohr's envelope is curved and requires interpretation, it is broadly accepted representing the interface strength by a straight line (Dixon et al. 2006). Under drained conditions, the cohesive intercept for sands, silts and normally consolidated clays approaches zero (Koutsourais and Sprague 1991).

The friction efficiency (E_φ) and the cohesion efficiency (E_α) can be calculated using the following equations:

$$E_\varphi = (\tan \delta / \tan \varphi) \qquad [2]$$

$$E_c = \alpha / c \qquad [3]$$

There are many factors affecting the direct shear strength between soils – Gm interfaces, such as the types of soil and Gm, moisture content and soil compaction among others (Seed 1991, Ling et al. 2001). For soil/Gm interfaces, the higher the soil shear strength, the higher the interface shear strength (Bacas et al. 2015). The friction angle is a function of normal stress thus revealing that the Mohr failure envelope is indeed curved for the soils. Negative adhesion can also be produced by best – fit lines through limited test data. If negative adhesion are ignored this will result in an overestimate of shear strength and hence potentially unsafe designs (Dixon et al. 2006).

The interface failure mechanism for smooth geomembranes was primarily particle sliding with limited particle rolling. The mechanism of particle movement at the interface is directly determined by the magnitude of normal stress relative to the critical stress; below the critical stress sliding without surface damage is the primary mode of translation while above the critical stress failure mechanism changed from soil particles sliding at the surface of the geomembrane to particles embedded into the geomembrane and plowing trenches along the direction of shear (Fleming et al. 2006, Punetha et al. 2017).

For smooth geomembranes, particle sliding along the interface was limited to a zone of two particle diameters adjacent to the geomembrane. As the surface roughness increased, the affected zone increased to about six particle diameters from the interface (DeJong and Westgate 2005). At high normal stress, after peak, the particle slippage dominates interface movement for smooth surfaces (DeJong and Wastegate 2005).

The dilation at the interface is a primary component of interface friction and that the highest interface friction angles are developed between layers where a significant amount of dilation occurs. Dense soils under low confining pressures tend to dilate resulting in a high frictional resistance. Soil particles are not able to dilate when the confinement is increased. Therefore, at higher normal stresses, the interface friction angle tends to approach a constant residual value. Strain softening behavior, very small dilatancy and nonlinear failure envelopes may have place at normal stress range (25 – 500kPa) (Bacas et al. 2015).

There are many articles published dealing with the analysis of Gm – soil interface behavior. Most of those articles analyze interface strength between smooth Gm and sand drainage layers. For Gm – sand layers reported interface angles can be as high as 29º showing a very little adhesion intercept of 0.5 kPa (Cen et al. 2018). These interface friction angles can diminish to around 20º under saturated conditions (Fleming et al. 2006). Friction efficiency use to range between 0.3 to 0.67 (Mitchell 1990; Hsieh and Hsieh 2003).

In spite the importance of the Gm – CCL interface there are very limited published research dealing with the study of Gm – fine soil interface resistance. Bacas (2015) determined interface friction angles ranging between 6 – 11º for fine soils with IP ranging between 17 to 33.5. Bergado (2006) reported a decrease of the interface friction angle from 10.54º under dry conditions to 8.2º under saturated conditions. Interface friction angle of Gm/CCL are very influenced by moisture condition, it importantly decreases under saturated conditions while under unsaturated conditions suction effect results in higher interface shear strength mostly in the low normal stress range (Sharma et al. 2007).

Pore pressure development and dissipation are expected to present an important effect, but since most analysis are performed under total stress approach, rate of shearing show limited effect on interface strength (Fleming et al. 2006).
Under landfill operating conditions liner are in close contact with leachate. It was shown by Masada et al. (1994) that prolonged exposure of the geomembranes to the landfill leachate reduced the interface friction angle.

3. MATERIALS AND METHODS

3.1 Tested Soils

This article analyzes the interface shear strength of composite liners considering a liner composed by two low hydraulic conductivity layers: a geomembrane and a compacted clay layer.

Compacted clay liners in Córdoba area are usually constructed by direct compaction local loessical silt. However, the required hydraulic conductivity can be reached in less than the 20% of the cases by only soil compaction, therefore some minor content of bentonite use to be included. Typically, it is included between 2 to 10% of bentonite by dry weight to the silt – bentonite blend (Francisca and Nieva 2007).

Cordoba loessical silt is mainly composed by sand sized particles (1.5 – 4 %), silt sized particles (40 - 80%) and clay sized particles (2 – 25 %) (Teruggi 1957). Employed silt samples were obtained from a trench excavated at the campus of the Universidad Nacional de Córdoba.

Bentonite added to the mixtures was a natural sodium bentonite provided by "Bentonitas del Lago". According to the manufacturers this bentonite has at least 95% content of natural sodium montmorillonite. These samples were formed in the cretacic age and were obtained from Pellegrini Lake, which is located in Río Negro Province (Lombardi et al. 2003).

In this research only three compacted silt – bentonite blends were considered: silt, silt + 5% bentonite and silt + 10% bentonite.

Hydraulic behavior of this silt – bentonite blends have been extensively studied by (Francisca and Nieva 2007, Francisca and Glatstein 2010, Musso et al. 2016, Glatstein et al. 2017). Mechanical behavior and strength of these silt – bentonite blends were also studied by Juarez et al. (2018). Table 1 depicts the main characteristics of these basic materials and the three mixtures considered in this research.

Also, it was studied in this research the interface strength between a smooth geomembrane and a drainage material such as fine sand.

Geomembrane tested in this research was a 1.5 mm thick high density polyethylene smooth geomembrane (Figure 1).

Table 1. Characteristics of tested soils.

Characteristics	Loessical Silt	Bentonite	Silt – 5% Bentonite	Silt – 10% Bentonite	Fine sand	Standard
Liquid limit (%)	27.8	343	35.2	43.7	NA	ASTM D4318
Plastic limit (%)[1]	26.2	89	32.0	35.6	NA	ASTM D4318
Plasticity index (%)	1.6	254	3.2	8.1	NA	ASTM D4318
Passing sieve 200 (%)	83.4	100	84.2	84.9	3.12	ASTM D6913
% of particle diameter < 2 µm	7.2	100	11.6	15.6	0	ASTM D7928
Specific Gravity	2.65	2.74	2.65	2.66	2.65	ASTM D854
Maximum dry Unit weight [kN/m³]	16.9	NA	16.9	16.6	NA	ASTM D698
Optimum compaction moisture [%]	17.6	NA	19.4	19.4	NA	ASTM D698
Activity	0.22	2.54	0.52	0.52	NA	NA
Internal friction angle φ [º]	24	NA	31.3	27	NA	ASTM D3080
Cohesion [kPa]	21.4		1.42	16.93	NA	ASTM D3080
SUCS	ML	CH	ML	CL	SP	ASTM D2487

Figure 1: Picture of the tested 1.5 mm thick HDPE smooth geomembrane.

3.2 Soil sample preparation

Soils used to prepare tested specimens were initially oven dried overnight. After that, blends with different content of bentonites were prepared by mixing the required weight of silt and bentonite. Weights were referred to dry condition. After having a uniform mixture of silt and bentonite, samples were gradually moistened and mixed until reaching the required optimum moisture content for compaction, then mixtures were stored in plastic bags in a humid chamber during 24 hs. in order to allow bentonite particles hydration.

Soil samples were compacted directly inside the shear box applying a static vertical load with a press to get the target density.

3.3 Interface shear tests

Interface shear tests were performed in accordance with the ASTM D5321.Tests were performed in an ELE shear machine which was modified and fully automated.

A common shear box typically employed for direct shear test of soils was employed. The shear box has a circular shape with an inner diameter of 63.4 mm. Standard test method suggest that boxes should have a minimum dimension greater than 300 mm, 15 times the d_{85} of the coarsest soil used in the test, or a minimum of five times de maximum opening size of the geosynthetic tested. However, smaller shear boxes can be used for tests were no scale or edge effects are expected, since there are no effects of aperture and rib size in the case of a continuum geomembrane and fine grained soils (Hsieh and Hsieh 2003). Shear boxes of similar sizes were employed for Gm – CCL interface tests by several authors (Koerner et al. 1986; Izgin and Wasti 1998; Ling et al. 2001; DeJong and Westgate 2005, Sharma et al. 2007).

This shear box has movable lower box with horizontally supported upper box. Soil was compacted in the upper box which has a height of 30 mm. The lower box was replaced by an acrylic block and geomembrane samples were glued and fixed by means of screws directly to the lower box.

Figure 2 shows pictures of the employed shear box, the shear test device and a sketch of the test set up for the interface shear strength parameter determination.

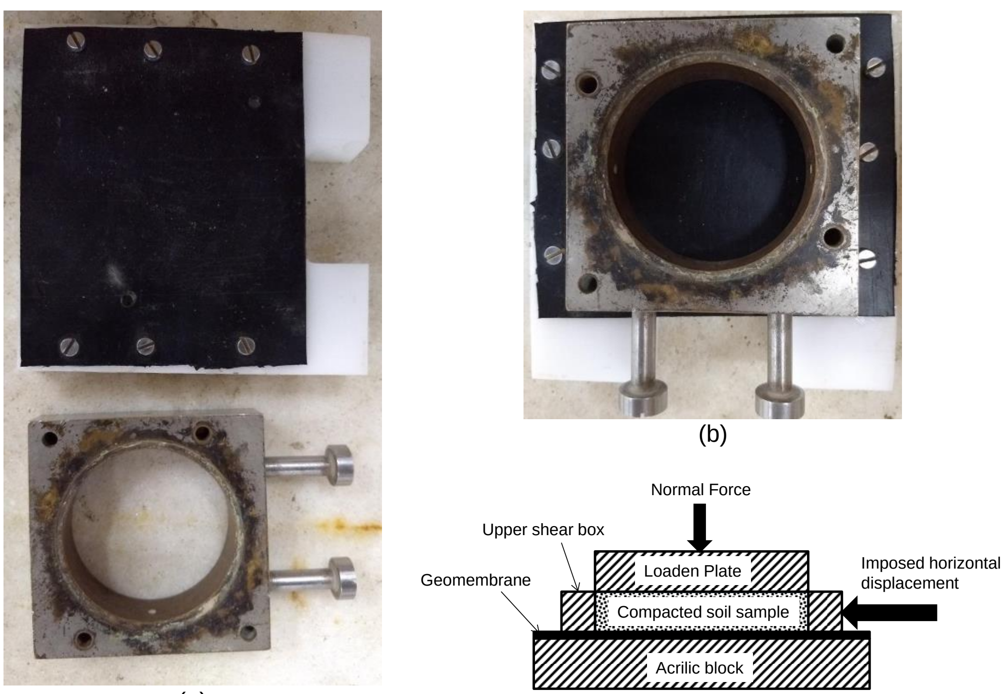

Figure 2: lower and upper halves of the employed shear box (a), shear box mounted (b), interface shear test sketch (c).

Normal stresses were applied using a rigid loading platen. Between three to five different normal stress were applied: 60 kPa, 124 kPa, 217 kPa, 310 kPa and 528 kPa. These magnitudes of normal stresses are representative of the vertical stresses at which landfill liners are subjected during operational conditions (Bacas et al. 2015).

Tests were performed under two different moisture conditions: samples tested at optimum compaction moisture content and samples tested after inundating the box. Consolidation of soil sample was measured and once it finished shear displacement was applied at a constant velocity. Tests were performed under four different strain velocities: 0.002 mm/min, 0.02 mm/min; 0.2 mm/min and 1 mmm/min.

Shear force was measured using a load cell while vertical and horizontal displacements were measured using two linear variable differential transformers (LVDTs).

A total of 28 tests were performed.

It is important to highlight the limitations of the testing procedure. One of the most important features is that the principal stresses are not defined, and that it is not possible measuring pore pressure or either to control its dissipation, therefore results are reported for total stress conditions. The direct shear device also induces no uniform stresses along the length of the specimen as a result of the rotation of the loading plate during shearing. Nevertheless, the strength parameters obtained from direct shear tests are widely used for design because the setup resembles that of the field conditions. (Ling et al. 2001).

4. RESULTS AND ANALYSIS

Interface shear resistance parameters were determined by fitting a straight line for each pair of normal stress and its associated failure shear stress in a Mohr space. For determining failure shear stress, specific horizontal shear stress was plotted against specific shear deformation. Every one of the resulting shear strain curves showed a peak shear stress meaning that almost all interfaces tested present a dilative behavior. Only peak shear stresses will be considered for the analysis.

The peak shear strengths was usually reached at 0.2 – 0.28 mm for silt samples under inundated condition while it was reached at 0.26 – 0.62 for the silt - 5% bentonite specimen under the same condition. For the case of samples tested at optimum compaction moisture, shear strength was reached at 0.41 – 0.58 mm for the silt specimen and 0.34 – 0.77 for the silt – 5% bentonite specimen.

For the case of Gm – loose fine sand and dense fine sand, shear strength was reached at 2.35 – 5.83 mm.

Figure 3 presents the shear strength curves determined for different specimens. Figure 3a presents the results corresponding to Gm – loose and dense fine sand under inundated conditions, Figure 3b present the corresponding shear strength failure envelope. Figure 3c and 3d present the same information for the case of silt specimen and silt +5% bentonite specimens under inundated conditions and Figure 3e and 3f the corresponding results for the case of the silt specimen and silt + 5% bentonite specimen under optimum compaction moisture.

From the analysis of the figures it is clear that Gm – sand interfaces present higher shear strength, also, maximum shear strength develops at higher horizontal displacement. In all cases, silt and silt – bentonites specimens present less shear strength than for the case of fine sand. Saturated silt specimen show more stiffness at the initial portion of the curves when compared with the silt – bentonite specimen under inundated condition. In almost all cases silt specimens showed higher shear strength than silt – bentonite specimen regarding if they were under compaction moisture content or inundated conditions. In all presented cases, shear strength was mainly supplied by soil sliding against Gm surface. Neither damage nor plowing was evident at the end of tests.

Shear failure envelopes showed non – zero adhesion intercept, and for the case of the inundated specimens the adhesion parameter was negative.

Shear velocity is expected to have an important effect on shear strength since the device has no possibility of controlling pore pressure. As long as shear velocity increases it is expected that pore pressure also increases and undrained condition prevail in the interface. Figure 4 shows the effect of shear velocity on shear strength for specimens of silt and silt – 5% bentonite tested under a normal stress of 217 kPa at compaction moisture and inundated conditions. It can be observed that due to undrained conditions shear strength decreases as shear velocity increases under inundated conditions. Same effect occurs when considering specimen tested at optimum compaction moisture content, however the decrease in shear strength is much more less in important.

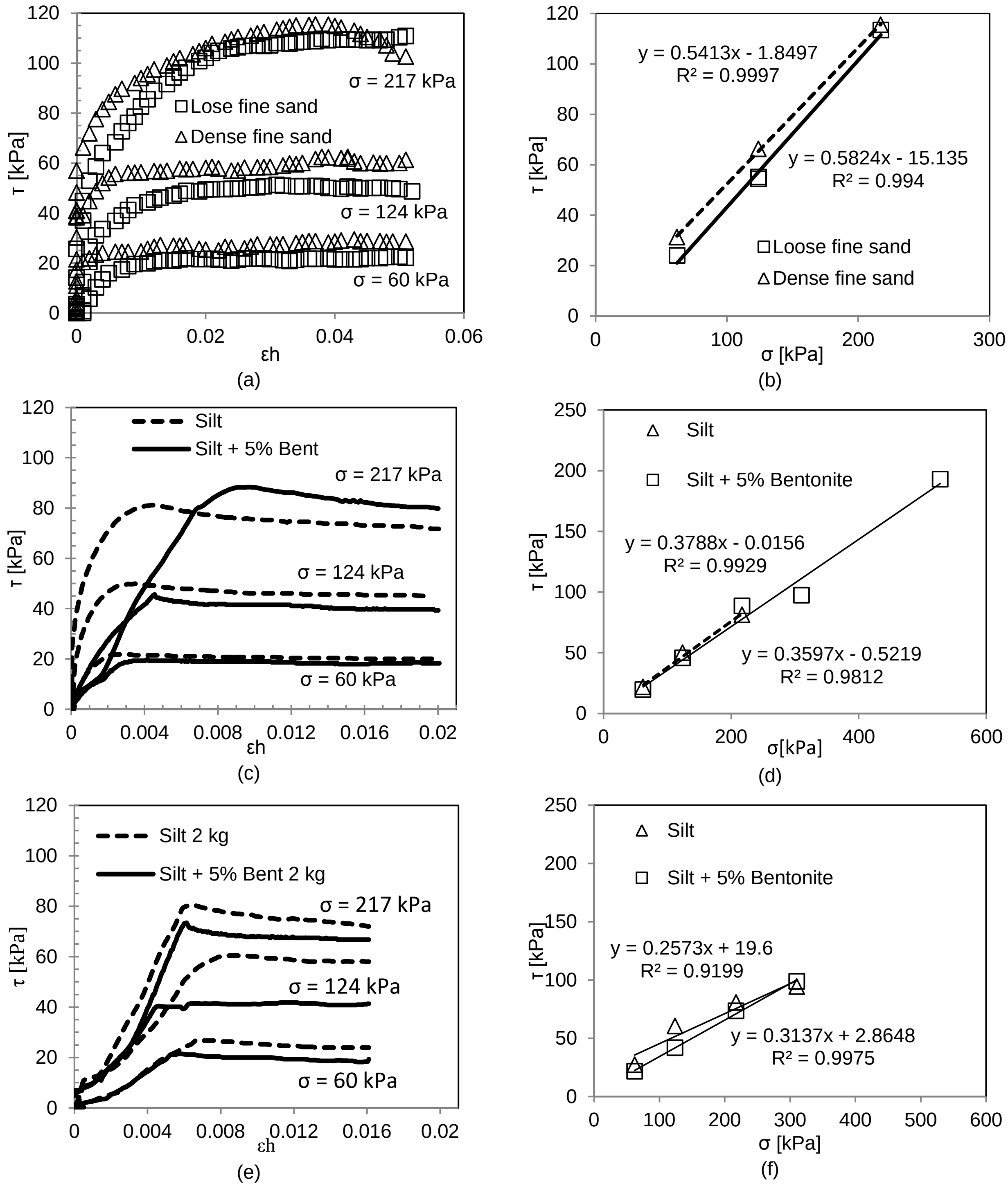

Figure 3: Shear stress – strain curves and failure envelopes. (a) and (b) loose and dense fine sand under inundated condition; (c) and (d) silt and silt – 5% bentonite under inundated condition; (e) and (f) silt and silt – 5% bentonite tested at optimum compaction moisture content.

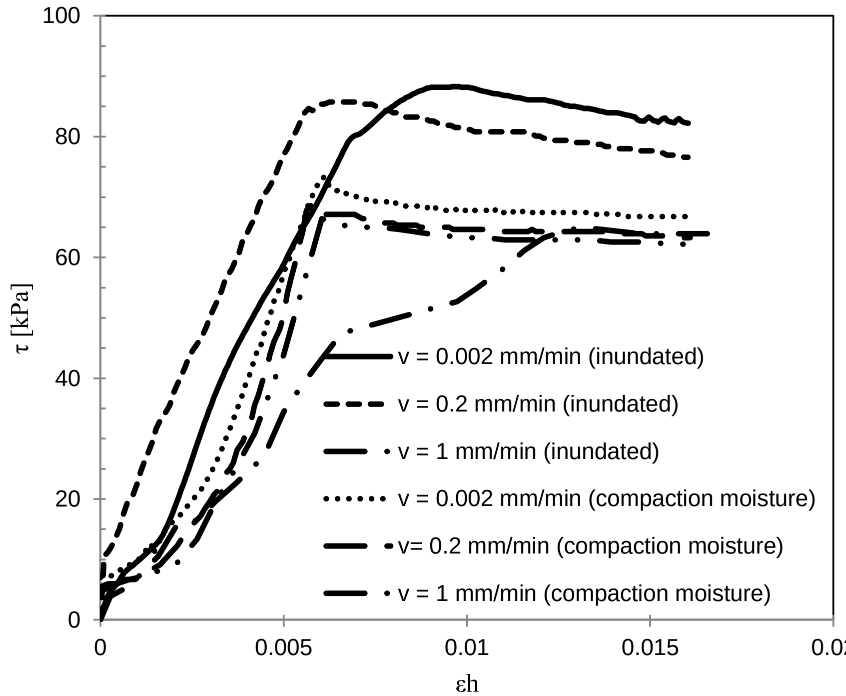

Figure 4: Change in shear strength for different shear velocities for the silt – 5% bentonite specimen at inundated and optimum compaction moisture content test condition under normal stress of 217 kPa

Table 2. Interface shear strength parameters

Specimen of CCL	Testing condition	Shear velocity [mm/min]	δ [º]	α [kPa]	E_{φ}	E_{α}
Loose fine sand	MC[1]	1	28.36	-8.33	0.44	NA
Loose fine sand	I[2]	1	30.1	-14.19	0.91	NA
Dense fine sand	MC	1	38.1	-24.3	0.36	NA
Dense fine sand	I	1	28.42	-1.85	0.6	NA
Silt	MC	0.002	14.4	19.6	0.71	0.11
Silt	MC	0.02	12.6	22.23	0.62	0.13
Silt	MC	0.2	11.2	24.22	0.55	0.14
Silt	MC	1	11.7	21.79	0.58	0.13
Silt	I	0.002	20.7	-0.016	0.85	NA
Silt	I	0.02	19.8	4.46	0.81	0.21
Silt	I	0.2	21	0.36	0.86	0.02
Silt	I	1	14	19.16	0.56	0.9
Silt + 5% Bentonite	MC	0.002	17.4	2.86	NA	0.02
Silt + 5% Bentonite	MC	0.02	16.6	2.97	NA	0.02
Silt + 5% Bentonite	MC	0.2	16.5	2.6	NA	0.02
Silt + 5% Bentonite	MC	1	16.6	2.97	NA	0.02
Silt + 5% Bentonite	I	0.002	19.78	-0.52	0.59	-0.37
Silt + 5% Bentonite	I	0.02	18.12	6.32	0.54	4.45
Silt + 5% Bentonite	I	0.2	15.6	30.5	0.46	21.48
Silt + 5% Bentonite	I	1	16.8	19	0.50	13.38
Silt + 10% Bentonite	MC	0.002	23.6	-8.21	NA	NA
Silt + 10% Bentonite	MC	0.02	20.6	5.4	NA	NA
Silt + 10% Bentonite	MC	0.2	23.6	-4.23	NA	NA
Silt + 10% Bentonite	MC	1	21.3	6.41	NA	NA
Silt + 10% Bentonite	I	0.002	20.2	-3.5	0.72	NA
Silt + 10% Bentonite	I	0.02	13.7	21.6	0.48	1.28
Silt + 10% Bentonite	I	0.2	17.6	0.31	0.62	0.02
Silt + 10% Bentonite	I	1	16.6	17.8	0.59	1.05

[1]Inundated, [2]Compaction moisture.

Table 2 summarizes all obtained results for interface shear strength parameters. In every case failure envelope presented adhesion intercept. For the case of Gm – sand interfaces, in every case, the adhesion intercept was negative. Interface friction angle was higher for the case of the Gm – sand interfaces. Interface friction angle increases with the increase in bentonite content under optimum compaction moisture testing condition. That is because analysis is performed under total stress approach and soil suction increase with the increase of bentonite content affecting the interface friction angle under unsaturated condition (Juárez et al. 2018). The opposite occurs when inundated condition is considered, interface friction angle tend to decrease with the increase in bentonite content. Also, for the case of silt specimens and silt – 5% bentonite, higher interface friction angles were determined under inundated conditions. That trend was not observed for the case of the silt – 10% bentonite since the lower hydraulic conductivity of the material may be affecting the pore pressure dissipation under shear.

The general trend also shows that friction angle decrease with the increase in shear velocity being more important the change under inundated conditions. It is important to highlight this fact because under normal landfill operation leachate level use to be formed in the bottom of landfills, affecting the lower region of side liners. Particularly, this region is the zone of the liner subjected to the more important normal stresses; therefore if leachate level is not controlled and geomembranes have defects in the zone, there will be some risk of instabilities and sliding failures.

Interface friction efficiencies were all in the range of previously published results. Determined efficiencies ranged between 0.44 up to 0.91 showing in every case a slight tendency to decrease with the increase in shear velocity.

There is not a clear trend respect to the variation of the adhesion intercept parameter.

5. CONCLUSIONS

This research analyzed the change in interface shear strength between a smooth Gm and CCL composed of blends of silty soil with different content of bentonite and with sand layers. The effect of bentonite content, shear velocity and moisture condition was explored. The main conclusions can be summarized as follows:

- Horizontal deformation for reaching peak shear stress increases with the increase in bentonite content which indicates that bentonite inclusion reduces the specimen stiffness, however fine soil samples showed less horizontal deformation than the sand specimens.
- Regarding the moisture condition of the tests in every situation the failure shear stress decreases with the decrease of the particle size and the increase in bentonite content.
- The only observed failure mechanism was the sliding of soil particles against Gm surface. Neither damage nor plowing was observed in the Gm after dismantling the samples.
- Shear stress decreases as shear velocity increase for samples under inundated conditions. This behavior was attributed mainly to the pore pressure development and undrained conditions for the samples. Changes in maximum shear stresses for the increase in velocity are more important as the bentonite content increases since undrained conditions are more evident for higher clay content.
- The analysis of interface shear strength parameters showed that all failure envelopes presented an adhesion parameter. Most of them presented negative values also. Interface friction angle increases with the increase in bentonite content under optimum compaction moisture content conditions due to the stiffening of particle fabrics because of suction increase; the opposite trend was evident for the case of samples tested under inundated conditions.
- Interface friction angle decrease with the increase in shear velocity, and those changes were more important for specimens tested under inundated conditions.

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