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GEOMECHANICAL CHARACTERIZATION OF THE NATURAL HEMP-COMPACTED CLAY INTERFACE

by
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AN ABSTRACT OF THE THESIS OF

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The interface parameters that govern the shear resistance mobilized between compacted clays and various solid interfaces are of interest in many geotechnical engineering projects. For example, the interface shear strength is essential for the design and safety assessment of deep foundations, offshore and onshore pipelines, mechanically stabilized earth walls, and fiber-reinforced soils, among others.

Despite the wealth of data on the interface shear resistance between soils and solid interfaces, there is currently limited information on the interface resistance between clays and natural fibers which are currently being considered as a sustainable measure of soil improvement.

In this study, a comprehensive experimental program is implemented to investigate the interface shear strength between a sandy clay and natural hemp fibers. A series of direct shear tests are conducted on both clay-clay and clay-hemp interfaces. The parameters that are varied in the experimental program are (1) the water content of the compacted clay, (2) the applied normal stress, and (3) the rate of loading and type of test (unconsolidated undrained tests versus consolidated drained tests). Another series of tests is implemented to study the interface shear resistance using single fiber pull-out tests at different rates of loading. Results indicate that the interface shear strength parameters that characterize the fiber/soil interaction are significantly affected by the test mechanism and the rate of loading. Results from interface direct shear tests that were conducted at fast rates to simulate undrained loading indicate that the interface strength parameters were found to be similar to those obtained using similar drained tests. This observation is important since it indicates that interface direct shear tests may not be optimal for characterizing the undrained response of the clay/fiber interface since drainage at the interface may be inevitable given the setup of the direct shear test mechanism.

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CHAPTER I

INTRODUCTION

A. Introduction

Soil improvement measures have become important in the field of geotechnical engineering, since all engineering constructions involve the ground. Ground improvement is generally defined as a state modification of a natural or a mass behavior of ground materials in order to attain an accurate response to existing or projected engineering performance requirements. Hejazi et al. (2012) summarized different procedures for soil improvement. These are divided into (1) physical methods (vibration, thermo electrical, freeze and thaw), (2) mechanical methods that either utilize fibrous materials in the form of continuous reinforcement and randomly distributed fibers (synthetic and natural) or aim at increasing the density of the soil by applying short term external mechanical forces, and (3) chemical methods which could involve conventional materials, enzymes and polymeric resins. Soil improvement methods are summarized in figure 1.1.

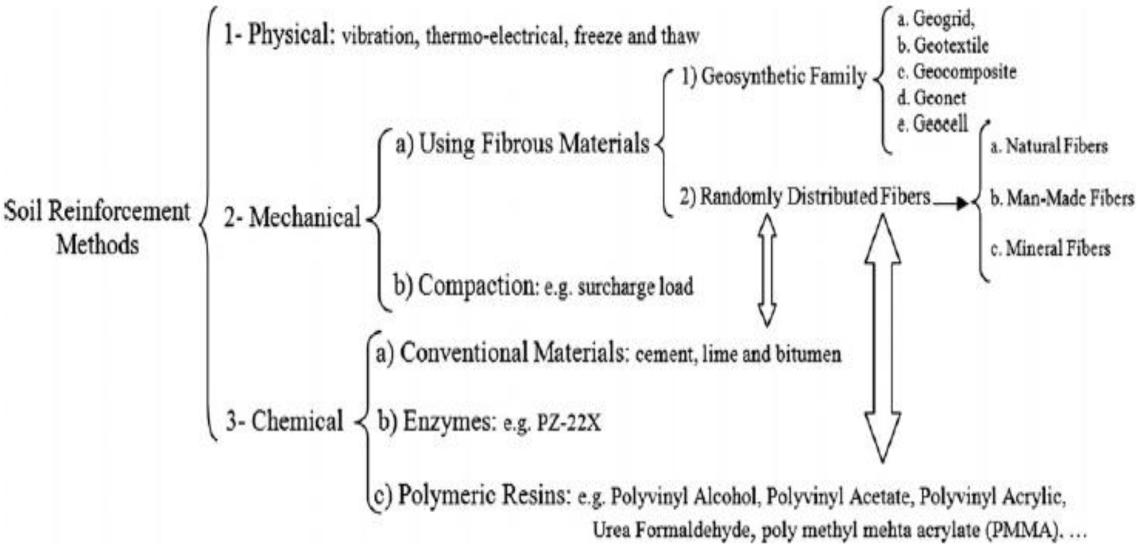


Figure 1.1Soil Improvement Methods.

B. Soil Reinforcement Using Inclusions

Soil reinforcement using inclusions has gained wide interest in a number of geotechnical applications. It consists of including certain materials with desired properties within soils that lack those properties. Therefore, soil reinforcement is defined as a technique to improve the engineering behavior of the soil by improving soil parameters that are generally related to shear strength, compressibility, density, and hydraulic conductivity. The aim of soil reinforcement is to improve stability of geotechnical systems, increase bearing capacity of foundations and reduce settlement and lateral deformation. Traditionally, geosynthetics, geotextiles, geofibers and geogrids have been widely used as inclusions in geotechnical engineering applications and other fields (Hossain et al. 2012). Different types of reinforcement materials are shown in figure 1.2.

Figure 1.2.Different types of soil reinforcement synthetic materials.

1. Soil reinforcement with geosynthetics and geotextiles.

The interaction between the soil and the reinforcement is very important in establishing the performance and design of the soil-reinforcement structure, and this interaction depends strongly on the nature and properties of the soil-reinforcement structure. Various types of geosynthetics and geotextile materials have been used with both cohesive and non-cohesive soils. The most commonly used geotextiles are High density polyethylene (HDPE), polypropylene (PP) and Polyester (PET), etc. However, new materials are always being fabricated and tested in order to improve the soil-reinforcement interaction properties.

The main applications of the soil-geosynthetics reinforcement system are mechanically stabilized earth walls (MSE) and reinforced soil slopes (RSS). However, geotextiles have shown a significant role in the modern municipal solid waste containment system, where bentonite clay sandwiched between two geotextiles known as Geosynthetics clay liners (GCLs), prevent underground water contamination , tolerate large deformations and improve the landfill stability (Lin et al. 2014).

2. Soil reinforcement with randomly distributed fibers

Recently, the use of randomly distributed fibers to reinforce both sands and clays has gained interest in the soil improvement community. Compared with the traditional reinforced materials, it is considered a simple procedure where soil is mixed with fibers. An advantage of using randomly distributed fibers is the absence of potential planes of weakness that can develop parallel to oriented reinforcement (Maher and Gray 1990).Fiber inclusion can provide some added strength but more important they present greater ductility (Ola 1989). Soils that are reinforced with fibers of relatively high tensile strength could exhibit an improved composite performance compared to the unreinforced soil due to the interaction between the soil and the fiber in the matrix.

The idea of using discrete fiber reinforcement stems mainly from the role that plant roots play in stabilizing the soil mass near the surface where the effective stresses are relatively low. Both synthetic (man-made fibers) and natural fibers have been widely used in geotechnical engineering applications that include pavement (road construction), retaining walls, and railway embankments, protection of slopes, earthquake, and foundations engineering (Hejazi et al 2012).

Discrete fibers can be used with a wide range of soil types. Synthetic fibers such as polypropylene (PP), polyethylene (PE), glass, nylon, steel and polyvinyl alcohol (PVA) fibers have been mostly used, on the other hand natural fibers such as coconut (coir) fiber, sisal (extracted from plant leaves), palm fibers, jute fibers, flax, barely straw, bamboo, and cane, some of which have been used from long times due to their availability and cost (Hejazi et al. 2012).

Whatever the used fibers and soil materials, the properties of the soil-fiber interface play an important role in dictating the response of the fiber-reinforced soil. The interaction mechanism between the soil and the reinforcement can be classified into two types: sliding of the soil over the reinforcement and pullout of reinforcement from the soil (Jewell et al. 1984). Any efforts that are aimed at establishing models that can predict the behavior of fiber-reinforced soils require information about the interface strength properties that describe the interaction between the fibers and the surrounding soils.

Many researchers have conducted experimental studies to explore the parameters affecting the response of the soil-fiber interface. The direct shear tests is considered as a suitable test to study the interaction between the soil and the reinforcement; since it can produce the shear mechanism along a potential failure plane in reinforced earth structure (Hossain et al. 2012). On the other hand, fiber pullout tests can mimic the slippage of the fiber along the surface of the soil and it is considered a simple and appropriate test to study the interaction between the fiber and the soil matrix (Tang et al. 2009).

There is a current drive in the construction industry that advocates the usage of environmentally friendly material as a substitution for synthetic materials. This drive has instigated emphasis on the important role that natural fibers could play in the modern construction industry (Hanafi et al. 1998). Hejazi et al. (2012) report that natural fibers have long been used in many developing countries in cement composites and earth blocks given their availability and low cost. Some studies (ex. Ghavami et al. 1999; Prabakar et al. 2002; Bouhicha et al. 2005; Wang et al. 2006; Lin et al. 2010; Dittenber et al. 2012; Hejazi et al. 2012; Najjar et al. 2014 etc.) have started investigating the potential use of natural fibers such as coconut fibers, palm fibers, straw fibers, bamboo fibers, and cane fibers to reinforce both sandy and clayey soils. Results of these studies indicate that natural fibers could provide soils with added shear strength and ductility.

Among the most commonly used natural fibers a study carried out by the United Nation Development Program (UNDP) and the Lebanese ministry of agriculture showed that investing in a 1000 square meters of non-irrigated land as a means to grow hemp costs about 79$ while the products are sold for approximately 192$ (Awwad 2011).

Hemp fibers are natural fibers that originate from the plant Cannabis Sativa. Hemp is legally planted in different countries including Spain, China, Japan, and France and is used in several industries such as paper, textiles, clothing, biodegradable plastics, construction, body products, health foods and bio-fuel. Hemp grows from a seed to maturity in approximately 12-16 weeks (approximately 100 days); an acre of hemp fibers can provide industry equivalent to what 4 acres of trees could. It is considered one of the most important types of natural fibers for industrial applications. It has been used in many civilizations for over 6000 years (Roulac 1997; Beckermann 2007).

According to Wang (2002), hemp has high tensile strength and strong tolerance for an alkali environment which makes it a good reinforcement material. Environmentally, hemp can perform better than glass by weight with respect to the life cycle analysis (LCA) (Anderson et al. 2004).Comparative studies between natural and glass fibers showed that natural fibers could replace glass fibers (Wambua et al. 2003).

Several research efforts were conducted to investigate the use of Hemp fibers in strengthening concrete as HFRC (hemp fiber reinforced concrete) (Li et al. 2005; Awwad et al. 2011). Recently, hemp is being considered as a potential soil reinforcement material. Najjar et al. (2014) and Bou Diab et al. (2016) investigated the improvement that is brought by the addition of hemp to the undrained shear strength of compacted clays that are reinforced with discrete hemp fibers. They concluded that hemp is very effective in increasing the undrained shear strength and ductility of the reinforced clay.

C. Scope of work

Although considerable research has been devoted to investigating the shear strength of fiber-reinforced soil as a whole matrix, less attention has been paid to the interface parameters between the soil and the fiber. In particular, previous research is lacking investigations of the interface properties between randomly distributed natural fibers and clayey soils. There is currently a need for understanding the mechanism that governs the interface behavior between natural fibers and clayey soils. This understanding is required for providing input to models that aim at predicting the shear strength of clays that are reinforced with randomly distributed natural fibers. Such understanding would allow for better design methodologies and predictions for geotechnical systems that involve fiber-reinforced clays in general and clays that are reinforced with natural fibers in particular. The interface shear strength between clays and natural fibers is expected to be a function of the clay properties (ex. plasticity, water content, and saturation), fiber properties (tensile strength, roughness, and aspect ratio), and the confining/normal stress acting on the clay/fiber interface, the drainage conditions and rate of loading.

The aim of this study is to experimentally investigate the interface resistance between Hemp fibers and clay. The interface shear strength will be measured using a series of direct shear tests and single fiber pull-out tests that will be conducted using a modified laboratory apparatus that is custom fabricated for this purpose. The parameters that will be varied in the experiments are (1) water content (degree of saturation), (2) rate of shearing/pullout (undrained versus drained), and (3) the applied normal stress

(20 kPa, 100 KPa, and 200 KPa). Both consolidated drained and unconsolidated undrained conditions will be conducted on clays and Hemp-clay interfaces, whereby the clay is compacted at different water contents (14%, 18%, and 20%) to simulate conditions that are at optimum and conditions that are wet and dry of optimum. Identical tests (same parameters) will be conducted using both direct shear and pullout to investigate the effect of the type of loading on the interface properties of the two materials.

D. Thesis Organization.

The thesis consists of five chapters. A literature review which includes the major experimental and analytical studies related to the interface shear strength between clay and different solid/fiber surfaces, investigated through experimental programs using direct shear and/or pullout test is presented in CHAPTER II. CHAPTER III includes the properties of the testing materials and a detailed description of the conducted testing program in addition to sample preparation and laboratory testing. CHAPTER IV presents the obtained results and their analysis and finally CHAPTERV includes the conclusions, recommendations and further research.

CHAPTER II

BACKGROUND
AND
LITEARTURE REVIEW

A. Introduction

The majority of the research studies that are published in the literature on soil-interface behavior have been conducted on sands, where both direct shear tests and pullout tests have been investigated (ex. Bosscher et al. 1987; Kishida et al. 1987; Jewell et al. 1987; O'Rourke et al. 1990; Pincus et al. 1995; etc.). These published studies indicate that the shear resistance between the sand and solid interfaces depends on the roughness of the interface material, sand type, grain size distribution, sand density, and the rate of loading. Fewer studies are available when it comes to the interface resistance mobilized between clays and solid or fiber interfaces.

Since many geotechnical applications involve the use of cohesive soils (ex. embankments, landfill cover systems, slopes repair, etc.), the assessment of the interface parameters between compacted clays and different solid interfaces has become an important component for solving soil-structure interaction problems. This chapter summarizes major experimental and theoretical studies conducted to investigate the interface behavior between clay and different surfaces using different tests methods.

Two major test methods are typically used in such investigations: The direct shear test and pullout test. The main studies associated with each of these tests methods are summarized and presented in chronological order. A final summary of major findings is also presented.

B. Interface Behavior Using Ring and Direct Shear Test

The direct shear test is considered to be a suitable test to investigate the shear strength of soils and to interpret interface parameters between a soil and different surfaces. The test allows testing soils under different confinement levels and controlled rates of shearing, across a predefined plane of failure.

The ring shear test allows for studying the interface parameters at large displacements because of its unlimited interface, which is an advantage over the direct shear test. In the direct shear test, the failure plane is located along the plane of separation of the two boxes leading to a change in the area and the shear stress distribution during shearing. Although the direction of shearing can be reversed to reach large displacements, secondary peaks in the shear stress versus displacement response are generally observed during reversals.

1. Interface behavior between clays and solid materials.

In order to investigate the behavior between the soil and a solid in contact with it, it is very important to study the ultimate shearing resistance at the interface between the soil and the adjacent solid material. This shearing resistance affects, if not governs, the stability of friction piles, retaining walls, anchor rods, submarine pipelines, and offshore gravity structures. Many researchers conducted direct shear tests to investigate the interface parameters between clay and different solid materials.

Lemos et al. (2000) used the ring shear apparatus to study the interface strength characteristics of clays with different plasticity, tested against materials such as glass, mild steel and stainless steel of varying roughness. All samples were consolidated prior to shearing. Shearing rates used ranged between 133mm/min up to 6000mm/min. The results of the tests are summarized in Tables 2.1 and 2.2 and they indicate the following:

- The peak interface shear resistance depends on the roughness of the interface material, the properties of the soil, e.g. grain size distribution and shape of the clay particles, the magnitude of the applied normal stress and the rate of shear displacement.
- Steel had lower interface strength characteristics compared to glass.
- Shearing the sample at fast shearing rates leads to an increase in the interface strength due to the probable generation of negative pore water pressures in the soil at or near the interface.

Table2.1 Lemos et al. (2000) test results on low plasticity clays.

Test no.	Soil type	Interface type	Roughness: μm		Soil-on-soil		Soil–interface			
			Initial CLA	Final CLA	Peak τ/σ'_N	Residual τ/σ'_N	Peak τ/σ'_N	Residual[b] τ/σ'_N	Residual[c] τ/σ'_N	Final displacement: m
1	CT	G	0·005	0·019	0·750	0·460	0·235	0·180	0·245	1·5
2	CT	G	0·005	0·029	0·750	0·460	0·242	0·186	0·390	4·5
3	HT	G	0·005	0·027	0·560	0·550	0·320	0·190	0·450	3·4
4	HT	G	0·027[a]		0·560	0·550	0·320	0·320	0·520	13·5
5	HT	G			0·560	0·550	0·308	0·307	0·454	5·6
6	HT	SSt	0·197		0·560	0·550	0·257	0·245	0·345	5·3
(cont)				0·109					0·560	12·5
7	HT	SSt	0·109[a]		0·560	0·550	0·296	0·255	0·500	7·8
8	HT	G	2·50		0·560	0·550	0·477	0·464	0·305	0·62
(cont)			–	0·80	–	–	–	–	0·435	3·7
9	MT	St	0·46	0·81	0·570	0·510			0·340	13·4
10	K1	G	7·0	7·0	0·550	0·270		0·238	0·242	1·5
11	K2	G	7·0	7·0	0·512	0·325	0·376	0·290	0·280	1·2

CT, Weathered Cowden till; HT, Happisburgh till; K1, Kalabagh Dam (sample no. 1); MT, Magnus till; G, glass; SSt, stainless steel; St, mild ste

[a] Same roughness as the one reached at the end of the previous test.

[b] True soil–interface residual shear strength.|

Table 2.2 Lemos et al. (2000) test results on London clay and Happisburgh till mixture.

Test no.	% LC	% HT	$(\tau/\sigma'_N)_P$	$(\tau/\sigma'_N)_R{}^R$	$(\tau/\sigma'_N)_R{}^C$	Displacement: m
12	100	0	0·320	0·191	0·186	0·97
13	75	25	0·352	0·220	0·250	0·60
14	50	50	0·350	0·245	0·279	0·83
15	25	75	0·418	0·292	0·341	0·65
16	0	100	0·477	0·464	0·435	3·70

The initial interface roughness for all tests had a mean value of CLA = 2·3 μm.
LC, London clay; HT, Happisburgh till; $(\tau/\sigma'_N)_P$, peak shear strength; $(\tau/\sigma'_N)_R{}^R$, true residual shear strength; $(\tau/\sigma'_N)_R{}^C$, residual shear strength with large displacements.

Hammoud et al. (2006) conducted consolidated drained shear tests using a rig shear apparatus with an applied normal stress of 70KN/m^2 and a rate of shearing 0.0356mm/min using four different types of clay sheared against four different stainless steel rings of different roughness. When soil-soil shear tests were conducted, samples exhibited a brittle behavior with measured internal friction angles varying between 17.8° to 26.9°. The ratio of the interface friction angle to the internal friction angle of the soil varied between 0.5 for smooth surfaces up to 1.08 for the rough ones. Results of the study are summarized in Table 2.3

Table 2.3 Hammoud et al. (2006) test results on different clays.

TEST	τ_p/σ_n'	φ_p' or δ_p' degrees	Δ_p mm	δ_p'/φ_p'	τ_r/σ_n'	φ_r' or δ_r' degrees	Δ_r mm	δ_r'/φ_r'
BOI	0.322	17.8	0.6	-	0.075	4.3	47	-
BSI4	0.349	19.2	0.5	1.08	0.106	6.1	60	1.42
BSI3	0.295	16.4	0.8	0.92	0.106	6.1	50	1.42
BSI2	0.283	15.8	0.4	0.89	0.104	5.9	92	1.37
BSI1	0.157	8.9	0.3	0.50	0.051	2.9	75	0.67
BCI2	0.345	19.0	0.5	1.07	0.110	6.6	55	1.47
BCI1	0.311	20.6	0.4	0.97	0.095	5.4	90	1.27
KOI	0.351	19.3	2.0	-	0.250	14.0	100	-
KSI4	0.368	20.2	2.0	1.05	0.228	12.8	100	0.91
KSI3	0.341	18.8	2.8	0.97	0.227	12.8	100	0.91
KSI2	0.330	18.3	2.0	0.94	0.216	12.2	95	0.87
KSI1	0.230	12.9	0.7	0.65	0.158	9.0	65	0.64
KCI2	0.336	18.6	2.4	0.96	0.235	13.2	60	0.94
XKIO	0.505	26.8	3.9	-	0.427	23.1	78	-
XKSI4	0.510	27.0	3.6	1.01	0.394	21.5	56	0.93
XKSI3	0.422	22.9	1.3	0.85	0.378	20.7	150	0.90
XKSI2	0.374	20.5	1.1	0.76	0.279	15.6	120	0.67
XKSI1	0.266	15.0	0.34	0.56	0.178	10.1	80	0.44
XKCI2	0.474	25.4	4.0	0.94	0.334	18.5	87	0.78
XKCI1	0.407	22.1	1.9	0.81	0.264	14.8	50	0.62
KCI1	0.334	18.5	2.3	0.95	0.221	12.4	50	0.88

2. Interface behavior between clays and geomembranes.

The possibility of using geomembranes in landfills and in reinforced earth walls has generated wide interest in studying the interface shear strength parameters that characterize the interface behavior between soils and geomembranes (Koerner et al. 1986; Ellithy et al.2000; Abu-Farsakh et al. 2007; Sharma et al. 2007; and Khoury et al. 2010).

<u>Koerner et al. (1986),</u> conducted consolidated drained direct shear tests with a shear rate of 0.06mm/min to study the interface strength between different low permeability soils and common liner materials (5 types of geo-membranes PVC, CPE, EPDM, HDPE, and embossed or textured HDPE). Their results (table 2.4) indicated that the adhesion between the different geomembrane materials and the soil is less than the cohesion of the soil unless the geo-membrane is very soft (PVC) or heavily textured (embossed HDPE), whereby the shear plane was observed to be within the actual soil. It was also found that the friction angles at the interface are relatively high for the applied normal pressures used.

Table 2.4 Koerner et al. (1986) test results on the interface shear strength between different cohesive soils and geomembranes.

	Soil No. 1 ML–CL				Soil No. 2 CL–ML				Soil No. 3 CL				Soil No. 4 SP–CH				Soil No. 5 CH–SP			
Description	c	$E_c(\%)$	ϕ	$E_\phi(\%)$	c	$E_c(\%)$	ϕ	$E_\phi(\%)$	c	$E_c(\%)$	ϕ	$E_\phi(\%)$	c	$E_c(\%)$	ϕ	$E_\phi(\%)$	c	$E_c(\%)$	ϕ	$E_\phi(\%)$
Soil-to-soil	9·0	100	38	100	12·0	100	34	100	20	100	30	100	25	100	24	100	28	100	22	100
	c_a	$E_c(\%)$	δ	$E_\phi(\%)$	c_a	$E_c(\%)$	δ	$E_\phi(\%)$	c_a	$E_c(\%)$	δ	$E_\phi(\%)$	c_a	$E_c(\%)$	δ	$E_\phi(\%)$	c_a	$E_c(\%)$	δ	$E_\phi(\%)$
Geomembrane-to-soil																				
PVC	8·5	94	39	100	3·7	31	23	69	14·0	70	16	53	7·0	28	24	100	12·0	43	17	77
CPE	8·0	89	40	100	3·2	27	24	71	13·0	65	17	57	8·0	32	23	96	10·0	36	19	86
EPDM	5·0	55	33	87	5·0	42	23	67	8·0	40	23	77	7·5	30	20	83	9·0	32	18	82
HDPE	5·0	88	26	68	2·0	17	23	67	14·0	70	15	50	3·0	12	21	88	14·0	50	15	68
Embossed HDPE	9·0	100	35	92	11·0	92	29	85	18·0	90	27	90	15·0	60	26	100	16·0	57	25	100

Note: c and c_a are in units of kN/m^2, ϕ and δ are in degrees.

Ellithy et al. (2000) studied the effect of the compaction moisture content on the geomembrane–clay interface shear strength using a direct shear test. In order to simulate different construction effects, they conducted both consolidated undrained (CU) and unconsolidated undrained tests (UU). The soil was compacted dry of optimum, wet of optimum and saturated. Soil was compacted into two layers in the shear box, and in the case where soil was tested against geomembrane, the geosynthetics were placed in the lower half of the shear box and clay was compacted in the upper half of the shear box. The soil was sheared against two types of geomembranes which were comprised of smooth and textured High Density Polyethylene (HDPE) by applying four normal stresses (25, 100, 250, and 500KPa).

Undrained shearing conditions were enforced by shearing the specimen with a fast rate so as not to allow the dissipation of pore water pressure during shear. The rate of shearing used to maintain “undrained” conditions was 2mm/min. The results of the shear strength properties of the Kaolinite clay under the UU and CU condition as shown in table 2.5, indicate that the shear strength decreases as the water content increases from the dry of optimum (28%), to the wet of optimum (32%), to saturated (34%) in both UU and CU tests.

Table 2.6 presents the interface shear strength parameters between the kaolinite clay and the smooth and textured HDPE under UU and CU conditions. The following parameters are presented:

- Interface Shear strength (τ) defined as: $\tau = a + \sigma \tan(\delta)$ where ;

  - a =shear strength intercept or adhesion (kPa)

  - δ = friction angle of the interface (˚)

  - σ =applied normal stress (kPa)

- Overall efficiency of the interface (E) defined as: τ of interface/τ of the clay.
- Adhesion efficiency (E_a) defined as: a of the interface $/c$ of the clay .
- Friction efficiency (E_δ) defined as: $\tan\delta$ of the interface $/\tan\phi$ of clay .

Based on the results the following conclusions can be established:

- For both textured and smooth HDPE, the interface shear strength decreases as the water content increases from the dry of optimum (28%), to the wet of optimum (32%), to saturated (34%).
- Using UU tests, the overall efficiency of the textured HDPE was around 1 for saturated and wet of optimum conditions under a normal stress of 250KPa, indicating that the failure plane was passing through the soil.

- In CU tests and under the same conditions as UU tests, an increase in the interface shear strength and the overall efficiency were observed compared to the associated UU test results.
- For the case where the soil was compacted dry of optimum, the efficiency of the interface was less than one.
- The shear strength of the clay and smooth HDPE interface in CU tests was higher than that of the UU tests for the wet of optimum and saturated cases, although the overall efficiency was lower. This was attributed to the effect of suction that can develop at the bottom of wet clay sliding over a smooth surface.
- Under all tested conditions, the friction efficiency (E_δ) of the textured HDPE was higher than that of the smooth HDPE where the values ranged from 0.55 to 0.67 for smooth HDPE versus 0.73 to 1.21 for textured HDPE.

Table 2.5 Ellithy et al. (2000) test results on the strength properties of the Kaolinite clay.

Test No	Conditions	c, (kPa)	Φ°	τ (kPa)
1	as-compacted (dry of optimum)/ unconsolidated/ undrained	28.7	32.1	185.5
2	as-compacted (wet of optimum)/ unconsolidated/ undrained	14.4	6.3	42.0
3	saturated/ unconsolidated/ undrained	18.5	1.9	26.8
4	as-compacted (dry of optimum)/ consolidated/ undrained	33.7	17.4	112.0
5	as-compacted (wet of optimum)/ consolidated/ undrained	8.4	12.9	65.6
6	saturated/ consolidated/ undrained	13.1	12.1	66.7

Table 2.6 Ellithy et al. (2000) test results on the strength properties of the Kaolinite clay / Geosynthetics Interfaces under UU and CU conditions.

Test No	Materials	Conditions	a_s (kPa) (E_a)	δ° (E_δ)	τ_s (kPa) (E)
10	Kaolin / Smooth HDPE	as-compacted (dry of optimum)/ unconsolidated/ undrained	3.4 (0.12)	19 (0.55)	89.5 (0.48)
11		as-compacted (wet of optimum)/ unconsolidated/ undrained	14.8 (1.03)	4.8 (0.76)	35.8 (0.85)
12		saturated/ unconsolidated/ undrained	10.7 (0.58)	2.2 (0.16)	20.3 (0.76)
13		as-compacted (dry of optimum)/ consolidated/ undrained	15 (0.45)	13.9 (0.79)	76.9 (0.69)
14		as-compacted (wet of optimum)/ consolidated/ undrained	11.9 (1.42)	8.3 (0.64)	48.4 (0.74)
15		saturated/ consolidated/ undrained	6 (0.46)	8.2 (0.67)	42.0 (0.63)
19	Kaolin / Textured HDPE	as-compacted (dry of optimum)/ unconsolidated/ undrained	17.4 (0.61)	24.5 (0.73)	131.3 (0.71)
20		as-compacted (wet of optimum)/ unconsolidated/ undrained	21.6 (1.5)	5.7 (0.91)	46.6 (1.10)
21		saturated/ unconsolidated/ undrained	9.9 (0.54)	3.2 (1.70)	23.9 (0.90)
22		as-compacted (dry of optimum)/ consolidated/ undrained	27.3 (0.81)	14.7 (0.84)	92.9 (0.83)
23		as-compacted (wet of optimum)/ consolidated/ undrained	20.9 (2.49)	13.5 (1.05)	80.9 (1.23)
24		saturated/ consolidated/ undrained	5.8 (0.44)	14.5 (1.21)	70.5 (1.06)

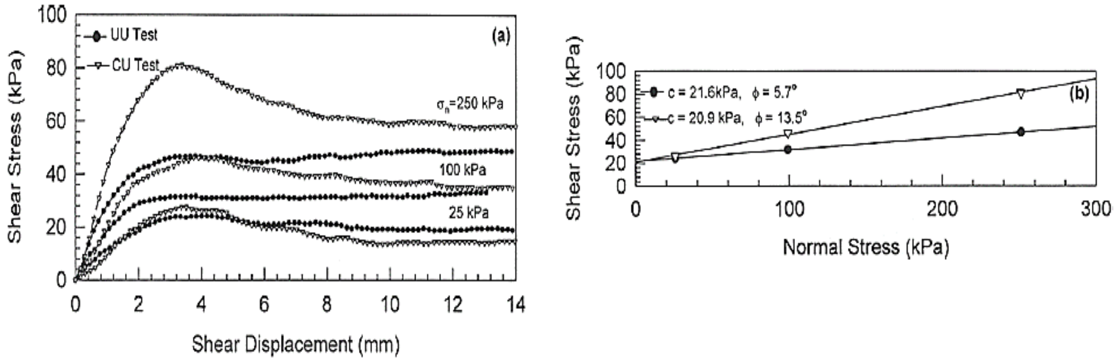


Figure 2.1: Ellithy et al. (2000) test results on the Kaolinite /Textured HDPE, compacted wet of optimum a) Shear Stress-Shear Displacement and b) Mohr-Coulomb Envelope

Ling et al. (2001) conducted a series of soil-soil and soil geomembrane (both smooth and textured) Polyvinyl Chloride (PVC) geomembranes interface tests, by applying three normal stresses (10, 25 and 50 kPa) using a modified direct shear test with soils of different plasticity indices. The soil samples were sheared with a rate of 0.117mm/min to simulate consolidated undrained conditions the following results were observed:

- The largest cohesion was observed in the soil-soil tests followed by the soil-textured PVC interface, followed by the soil-smooth geomembrane interface.
- The friction angle between the soil and the geomembrane was larger than the soil friction angle. This could be due to the spacing between the two boxes or "penetration"/indentation of soil particles into the geomembrane.
- The shear strength of the soil was the largest followed by the interface strength between the textured geomembrane and the soil followed by the interface resistance of the smooth geomembrane and the soil.

Abu-Farsakh et al. (2007) conducted interface direct shear tests to investigate the interface shear resistance between GCLs (woven geotextiles and geogrids) and four soil types (one sand and three clays). They varied the soil dry density and its moisture content. Interface characteristics such as the interface adhesion were measured. They conducted unconsolidated undrained (UU) direct shear tests, where no consolidation was allowed prior to shearing with a relatively fast shear rate of 0.85mm/min. The applied normal stresses on the clay sample were (25, 50 and 75kPa). The following results were reported:

- Interface efficiencies $c_i = \tan(\delta_a) / \tan(\phi)$ were greater than 0.7 in all cases, indicating good bonding between clay and geogrids.
- The slope of the interface shear failure envelope decreased with increasing moisture content. This was attributed to a possible decrease in soil suction accompanied with the increase in moisture content and development of excess pore water pressure in saturated clays (reducing the effective stresses and the shear resistance).
- Compacting soil dry of optimum enhances reinforcement efficiency.
- The reduction in the shear strength of the soil-geosynthetics interface was found to be larger than that of the soil-soil interface, implying that interface efficiency would decrease with the increase of the moisture content (or decrease of dry densities).

The Mohr-Columb failure envelopes for soil-soil and for soil-Geosynthetics interfaces are shown in figure 2.2 and the corresponding values of the soil cohesion and its internal friction angle, interface adhesion and the soil interface friction angle with different interfaces are shown in table 2.7.

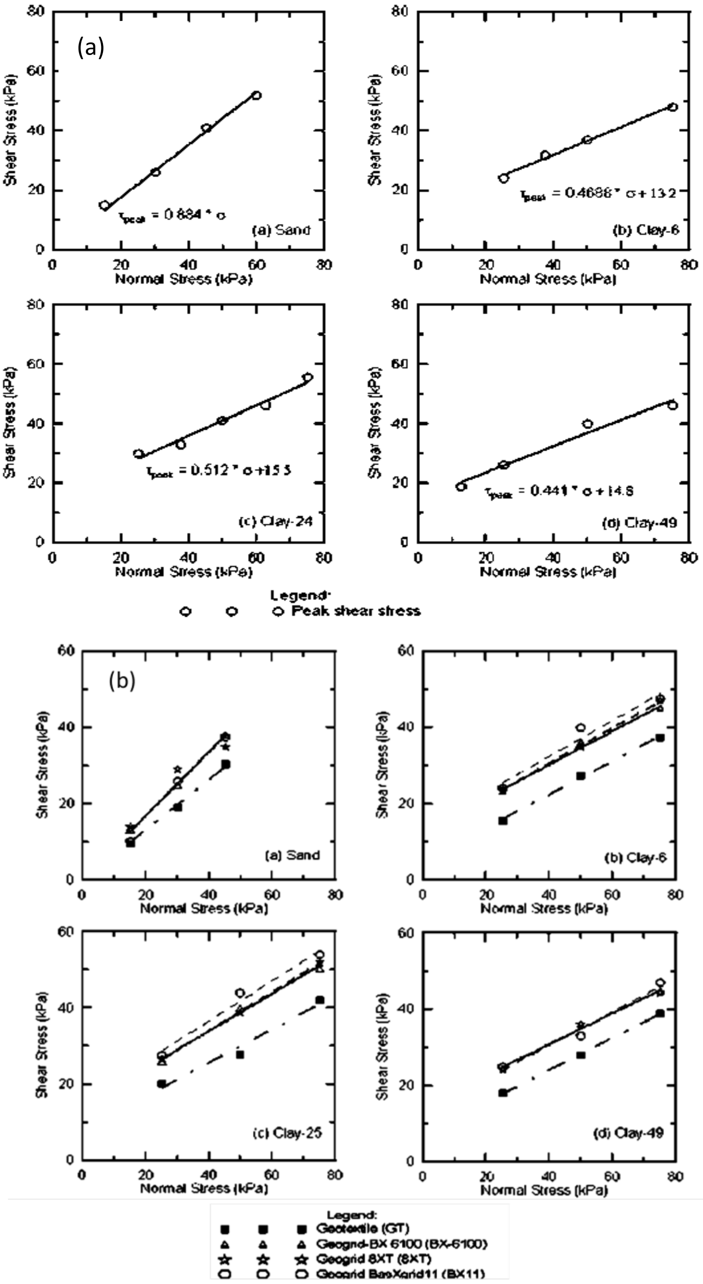


Figure2.2 Abu Farsakh et al. (2007) Shear stress versus normal stress curves for soil-soil (a)/ Soil- Geosynthetics (b). (Soils were compacted at their maximum dry unit weight and optimum moisture content).

Table 2.7Abu Farsakh et al. (2007) Interface test results of different geotextiles

	Soil data		GT		BX-6100		Miragrid 8XT		BasXgrid 11	
Soils	c (kPa)	ϕ (degrees)	c_a (kPa)	δ (degrees)	c_a (kPa)	δ (degrees)	c_a (kPa)	δ (degrees)	c_a (kPa)	δ (degrees)
Sand	0.0	41.5	0.0	33.4	0.0	40.1	0.0	40.1	0.0	39.9
Clay 6	13.2	25.6	10.7	21.7	9.1	18.9	10.1	24.1	9.2	18.0
Clay 25	16.7	27.1	8.1	23.7	8.9	21.4	10.3	25.8	12.7	30.7
Clay 49	14.8	23.8	7.3	22.8	14.0	13.3	13.9	18.5	8.4	23.0

Note: GT=geotextile; c_a=adhesion between geosynthetics and soils (kPa); and δ=interface frictional angle between geosynthetics and soils (degrees).

The authors recommend that the interface shear parameters of soils at 95% maximum dry density and at a moisture content that is 2% above optimum should be used in soil geosynthetics reinforcement design. This study is limited to the short term behavior of geosynthetics –reinforcement since it covers the unconsolidated undrained conditions.

Recently, Miller et al. (2006) and Khoury et al. (2010) modified a commercial direct shear apparatus to allow for suction-controlled testing to measure the interface resistance between unsaturated soil and stainless steel and geotextile interfaces. After consolidation, drained shearing was initiated under constant suction and constant normal at displacement rates equals to 0.005mm/min, while keeping both pore air and pore water pressure constant and recording volume change.

The shear strength expressed in terms of these two stress states is given by:

- $\tau=c'+(\sigma-u_a)\tan\phi'+(u_a-u_w)\tan\phi^b$ for soil shear strength
- $\tau=c_a'+(\sigma-u_a)\tan\delta'+(u_a-u_w)\tan\delta^b$ for interface shear strength

Where:

-c' /c_a' is the effective cohesion / adhesion intercept

- σ is the total normal stress

-u_a is the pore air pressure on the soil/ interface failure plane

-u_{aw} is the pore water pressure on the soil/ interface failure plane

- ϕ' / δ' is the effective angle of internal friction/interface friction angle with respect to the net normal stress (σ-u_a)

- ϕ^b / δ^b is the angle of internal friction/interface friction angle with respect to the matric suction (σ-u_a)

Their results indicate that:

- The peak shear strength of the soil-geotextile interface increases non-linearly with the soil suction, resulting in an increase in the soil adhesion (figure 2.3).
- The interface friction angle (δ'=32°) was smaller than the soil internal friction angle (ϕ'=36°) of comparable soil however this was not true for the adhesion and cohesion at zero suction (figure 2.4).
- A small decrease of water content was detected during shearing of both the soil and soil-geotextile interface due to the disruption of the menisci between soil particles causing the increase in pore pressure (decrease in suction). This resulted in water draining out of the sample.

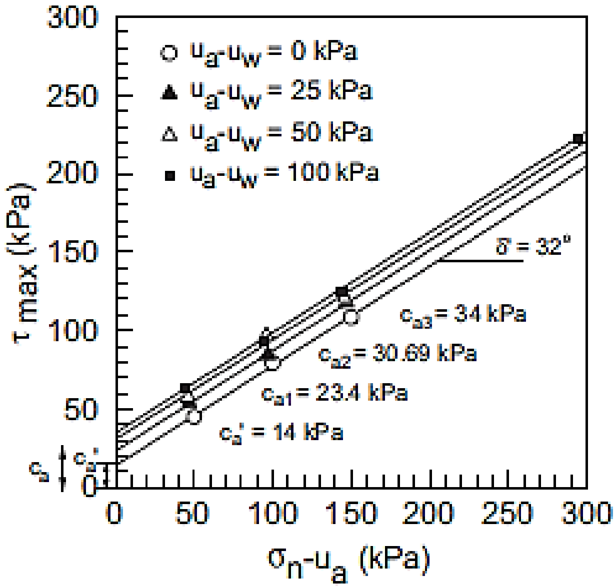


Figure 2.3Khoury et al. (2010) failure envelope for the soil geotextile interface at different suction values.

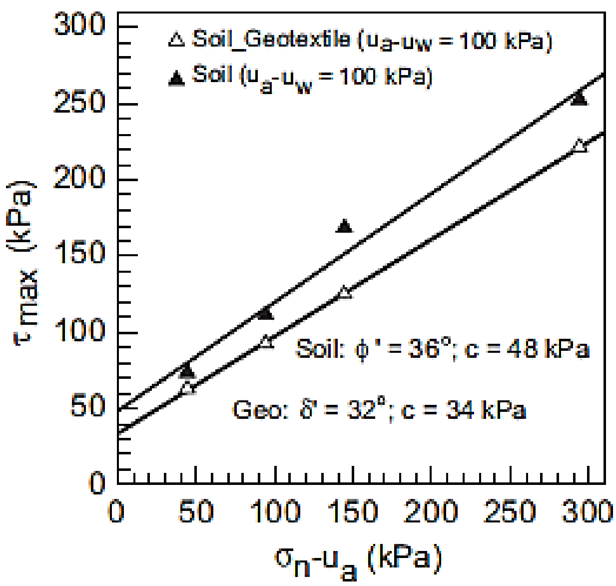


Figure 2.4 Khoury et al. (2010) failure envelope for the soil and geotextile interface at 100 kPa suction (u_a-u_w = 100kPa)

C. Interface Behavior Using Pullout Tests

Pullout tests have been adopted in research efforts exploring interface strength characteristics in order to develop a better understanding of the interface behavior between clays and solid/fiber interfaces. A pullout test is considered simple and appropriate for measuring the interaction between the "reinforcement" and the soil matrix (Tang et al. 2009). Most of pullout tests that are reported in the literature were conducted on backfill soils with different types of geotextiles, geogrids, and geosynthetics. The results obtained are generally compared to associated results from direct shear tests (Alfaro et al. 1995; Teixeira et al. 2006; Lopes et al. 2010; Hsieh et al 2011; Artidteang et al. 2013). The conducted tests indicated that the pullout resistance depends on the fiber diameter, elastic modulus of the fiber, and the applied normal stress. The pullout resistance increases with increasing normal stress, while the interaction coefficient generally decreases.

1. Comparison between Direct shear and pullout tests.

For certain types of reinforcement, the pullout resistance is composed of two components: the frictional resistance due to longitudinal members and a bearing resistance on the transverse members. The direct shear resistance is also composed of two components: a soil to reinforcement shear resistance along the plane of the reinforcement and a soil to soil resistance at grid openings (Alfaro et al. 1995).

Alfaro et al. (1995) conducted both pullout and direct shear tests in order to simulate both interaction mechanisms. A backfill soil composed of well graded gravel was compacted in a box, while the reinforcement was bolted on clamping plates for the direct shear test. For the pullout test, the clamping plates were placed inside the compacted soil and were extended behind the sleeve plate. The frictional interface properties obtained from pullout tests were higher than those from equivalent direct shear tests. This was attributed to the additional bearing resistance of the longitudinal members.

Lopes et al. (2010) conducted both pullout and direct shear tests to compare the friction parameters between soil and geosynthetics. The soil, a residual of granite, was tested against geotextile composed of non-woven polypropylene. The values of interface coefficients obtained from pullout and direct shear tests were obtained and compared. The interface coefficient and the ratio of the shear stress/ vertical stress at the maximum pullout force were lower than that obtained from the direct shear test. This was believed to be related to the deformation of the geosynthetics in the pullout tests, which is not considered in the direct shear tests. The interface properties in pullout, where the geosynthetics have a full plane of contact, cannot be obtained based on values obtained from the direct shear test.

Hsieh et al. (2011) studied the difference in the interface shear resistance behavior between uniform-graded granular soils and different types of geosynthetics. A series of direct shear tests and pullout tests were conducted by applying three normal stresses (49.05, 98.10, 147.15 kPa). The results indicate that the failure mechanism of direct shear test and pullout test are different. The pullout resistance is less than that of the direct shear resistance; this is due to the deformations along the geosynthetics causing the reorientation of the soil particles into a reduced shear strength mode at the soil geosynthetics interfaces.

Bergado and Chai (1994) defined the interaction coefficient from the pullout test as:

$$C_i = \frac{P_{(pullout)ult.}}{2WL(\sigma_n tan\varnothing + c)}, \text{ where}$$

- $P_{(pullout)\ ult.}$ is the ultimate measured pullout resistance from a pullout test;
- W is the width of the reinforcement;
- L is the embedment length of the reinforcement
- σ_n is the applied normal stress.

Artidang et al. (2012) investigated the tensile and the geotextile interface strength of a new type of geotextile called limited life geotextile (LLGs) made from natural Kenaf fiber. A series of direct shear tests were carried out to investigate the interface parameters between sand backfill material and Kenaf woven LLGs under three confining pressures (20, 40, and 60kPa). Another series of pullout tests were performed using confining pressures of 20, 40 and 60kPa representing the possible range of applied pressures in the field. It was found that the peak friction angle for sand was 35.63° with a corresponding cohesion of 11.33kPa while those of the interface with Kenaf LLGs were 27.66° and 9.26 kPa, repsectively. The interaction coefficient obtained from the direct shear test was 0.812. The pullout interaction coefficient for the Kenaf LLGs as defined by Bergado and Chai (1994), is 1.11, 1.07 and 0.88 for the applied normal loads of 20, 40, and 60 kPa respectively, indicating that the interface coefficient decreases with increasing normal stress.

In conclusion it was found that the results obtained from the direct shear tests and the pullout tests are different and both tests should be conducted to derive interaction parameters. This may be attributed to the different loading paths and interaction mechanism. The mobilized shear strain in the direct shear test has greater uniformity along the soil-geogrids interface compared to that of the pullout tests, whereas the mobilized strain in the pullout test is a combination of the shear strain along the interface and the reinforcement elongation.

2. Single fiber pullout tests in cohesive soils.

Proper understanding of the interfacial interactions between the reinforcement and the soil matrix is needed to predict the internal stability mechanism that is associated with soil/fiber interaction. For applications involving the use of randomly distributed fibers as “new” earth reinforcement materials, the single fiber pullout test seems to be an appropriate and simple test for modeling the interface mechanism.

Tang et al. (2009) tested the factors affecting the interface strength of a clayey soil reinforced with a polypropylene fiber using a single fiber pullout test. The specimen preparation technique and test setup are presented in figures 2.5 and 2.6, respectively, while the test results are presented in figure 2.7 and 2.8.The interfacial peak strength (IPS) and interfacial residual strength (IRS) were calculated according to the following equations:

$$\text{IPS}=\frac{N_{max}}{\pi dl} \quad \text{and} \quad \text{IRS}=\frac{N_r}{\pi dl}$$

Where N_{max} is the maximum load prior to the interface shear failure, d is the fiber diameter, l is the fiber embedded length and N_r is the residual load applied on the fiber. The fiber diameter was 0.048mm and the fiber embedded length was 5mm. Pullout displacement was measured by displacement transducers with a range of 50mm, where the free length of the fiber was constant in all tests l_0=50mm. The velocity of the loading disk was controlled at 1mm/min. All the tests were performed under undrained conditions, where they investigated the effect of soil water content and dry density on mechanical properties of fiber-soil interface and pullout response. Based on the test results, the following observations can be made

- The fiber surface roughness, fiber shape, and pull-out velocity were found to influence the interface strength of the fiber/composite.
- Both, the peak shear strength and residual shear strength of the interface decrease with an increase in the water content (figure 2.7). This was attributed to the effect of matrix suction that could be developed due to the capillary water between soil particles and the fiber surface, giving rise to an increase of effective stress on fiber/soil interface. The soil matrix suction is expected to increase with decreasing water content.
- The peak and the residual shear strength of the fiber/soil interface increase with increasing dry density (figure 2.8). This was due to the fact that higher dry densities are accompanied by lower void ratios and smaller pore diameter,

leading to an increase in the effective interfacial area between the soil and the fiber.

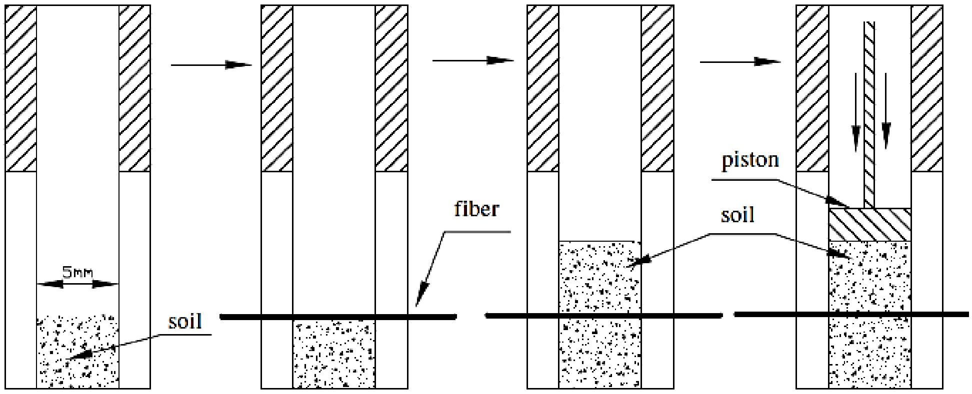


Figure 2.5 Tang et al. (2009) specimen preparing process.

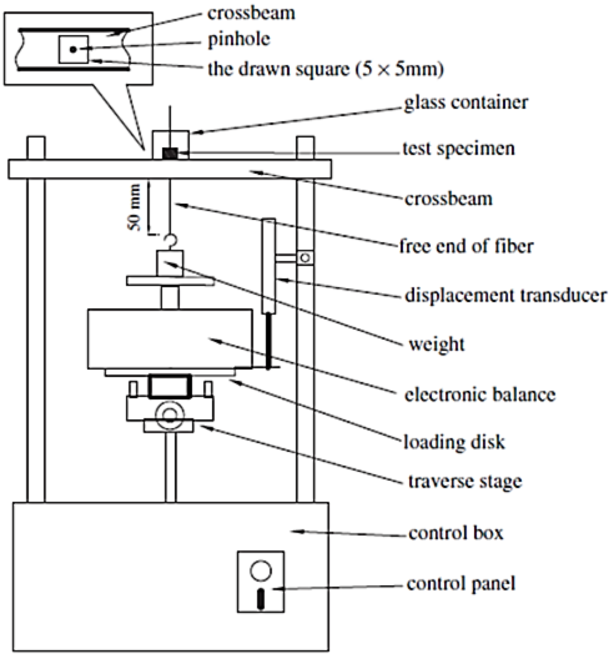


Figure 2.6 Tang et al. (2009) single fiber pullout test apparatus.

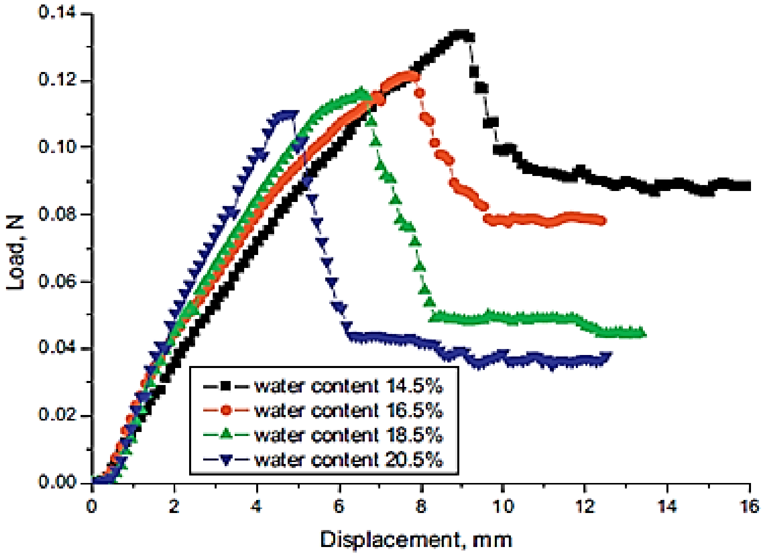


Figure 2.7 Tang et al. (2009) results showing the decrease in the peak shear strength of the fiber/soil interface with the increase in the water content.

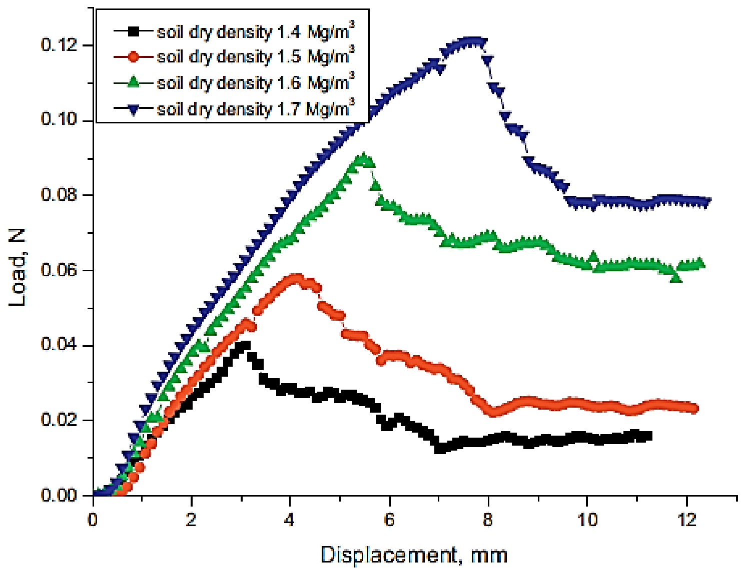


Figure 2.8 Tang et al. (2009) results showing the increase in the peak shear strength of the fiber/soil interface with the increase in the dry density.

Jamie et al. (2013) conducted unconsolidated undrained pullout tests on a single sisal fiber embedded in a natural clay with varied initial conditions (water content and dry density). A special soil box (figure 2.9) and pullout apparatus (figure 2.10) were designed to conduct the pullout tests. The soil was compacted in a special soil box designed for this single fiber pullout test, where a fiber is inserted between two layers of compacted clay. A normal load is applied and the fiber pullout is conducted using a tensile pullout system where the rate of shearing is controlled using a graduated oil can. Results (figure 2.11) indicate that the failure is based on slippage of the fiber when the applied normal pressure is below 125KPa, and that the interface strength decreases with the increase in the water content. The results also indicate that the shear strength of the fiber /clay interface can be reasonably well described used a Mohr-Coulomb criterion.

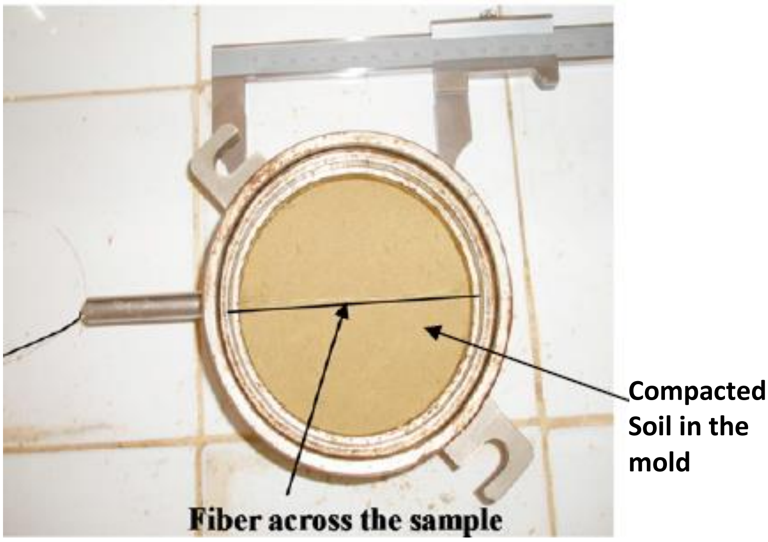


Figure2.9 Jamie et al. (2013), compacted soil in the mold including a single fiber

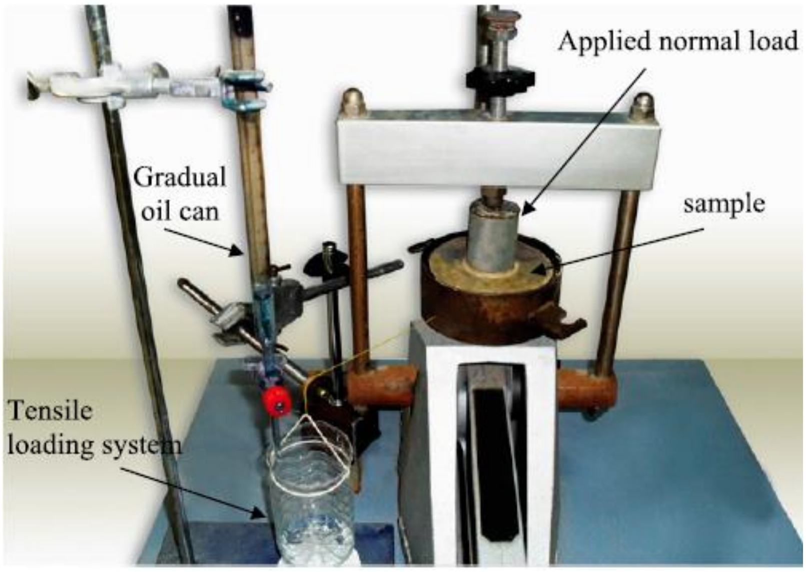


Figure 2.10Jamie et al. (2013) pullout test apparatus.

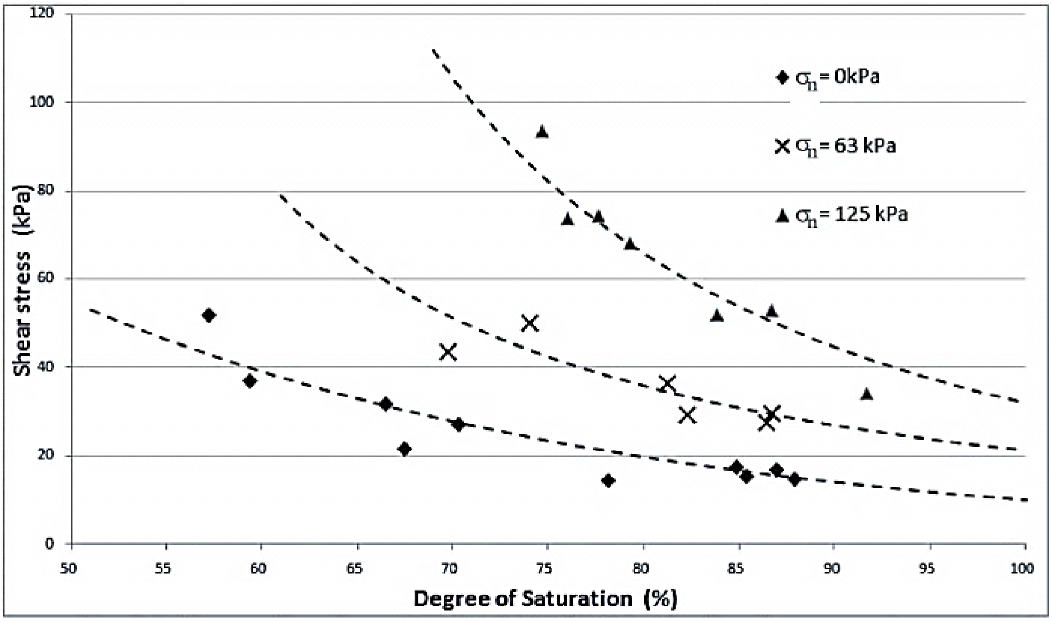


Figure 2.11 Jamie et al. (2013) results showing the decrease in the interface shear strength with increase in the degree of saturation (water content).

3. Pullout tests between cohesive soils and geotextiles

Pullout tests and direct shear tests are typically used in order to obtain the interface parameters between soils and reinforcement. The obtained parameters present actual and more reliable interface properties for designing reinforced soil-structures. However, many factors affect the interface behavior including the applied stress, moisture content and pullout rate.

Clancy and Naughton (2011) conducted pullout tests between marginal soils and both drainage reinforcement geosynthetics and geogrids. They investigated the effect of applied stress, pullout rate and moisture content of soil using an innovative pullout apparatus which facilitates pore pressure dissipation. The pullout mold is formed of a PVC pipe with top and bottom seals formed of seal caps and O rings. The bottom disc of the pipe was displaced using a pneumatic ram allowing the generation of excess pore pressure in the sealed soil contained in the pipe. The geosynthetics placed longitudinally and elongated out through the top cap. Pore water pressure was measured using pore water transducers placed in the soil around the geosynthetics which is attached through a clamp to a load cell that will record the pullout force. The pipe is placed in the triaxial apparatus where pullout is achieved by lowering the platen of the triaxial. The used apparatus is shown in figure 2.12.

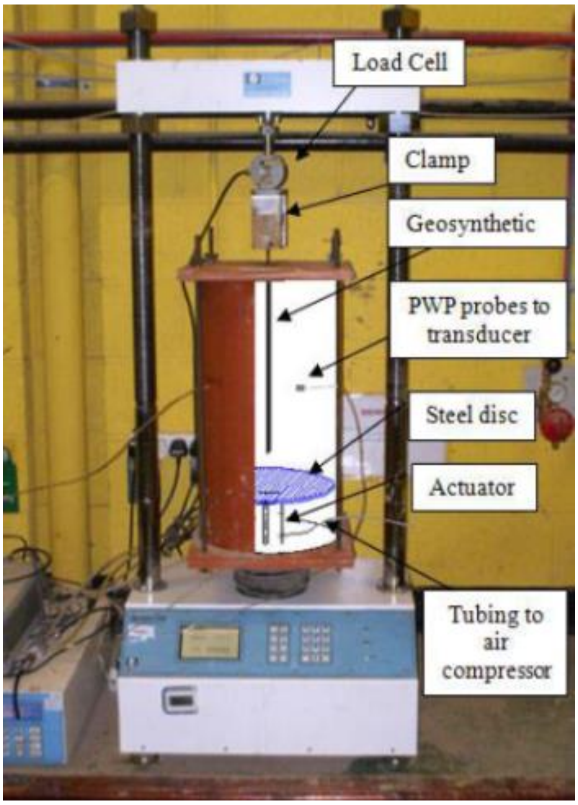


Figure 2.12. Clancy and Naughton (2011) Pullout Test Apparatus.

Clancy and Naughton (2011) ran two testing programs; the first one tested the peak pullout resistance of two geosynthetics, one with drainage capabilities and the other without. Different moisture contents, applied normal stresses and pullout rates (0.2mm/min, 2mm/min and 10mm/min) were investigated. In this first testing program soils were not allowed to consolidate prior to pullout and no dissipation of pore water pressure was permitted. In the second testing program they investigated the behavior of geocomposites with drainage capability where soils were allowed to consolidate and dissipation of pore water pressure was permitted. Water draining out of the sample was collected through drainage channels.

In testing program 1 the soil density was not varied and a wide range of water contents were investigated to allow the development of excess pore water pressures and monitor their dissipation. It was found that a significant decrease in pullout resistance was indicated at higher water contents. The pullout resistance decreases as the water content increases, irrespective of the confining stress. This was attributed to the water lubrication at the level of interface. The effect of the confining stress showed a decrease in pullout resistance with increasing confining stress, where higher confining stresses could generate higher pore water pressures, thus reducing the interaction between soil and geosynthetics.

The influence of the rate of pullout on the pullout resistance indicates that for the same confining stresses tests conducted at fast rate resulted in a significant increase in pullout resistance. This increase was attributed to the development of suction forces, particularly at lower moisture contents. Results are shown in figure 2.13, where the label shows the applied normal stress (kPa), pullout rate (S: slow, R: rapid and F: fast), and finally the tested water content (%).

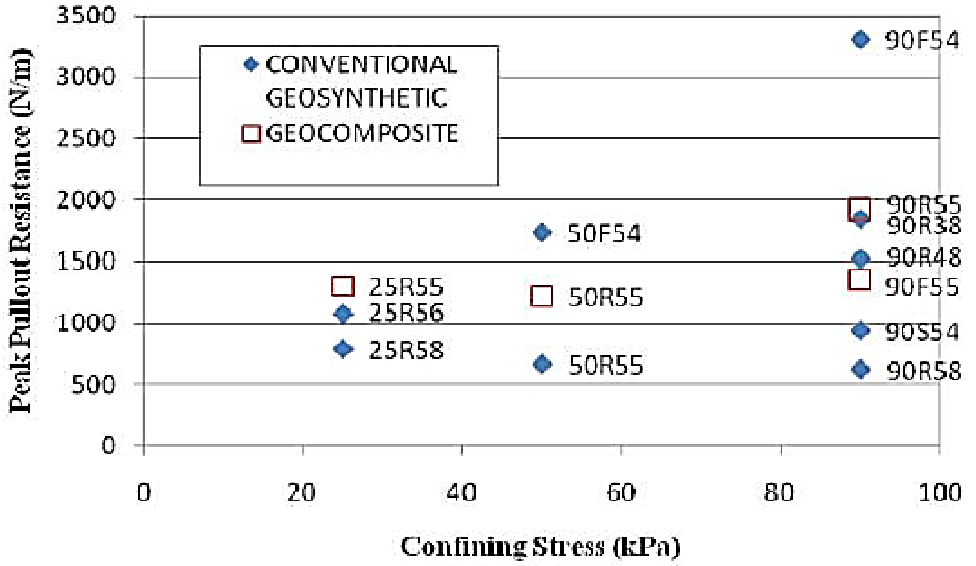


Figure 2.13 Clancy and Naughton (2011) Relationship between confining stress, pullout rate and moisture content with the peak pullout resistance.

The second testing program investigates the improvement in pullout resistance provided by using geocomposite with drainage capability, where the peak pullout resistance increases with an increase in the confining stresses. The comparison between tests conducted at the same water content, confining stresses, and pullout rates between test program 1 where no consolidation was allowed and test program 2 where consolidation was permitted on the same type of geocomposite showed an increase in the peak pullout resistance and no dissipation of pore water pressure where no consolidation or drainage was permitted, due to the development of suction forces.

Geocomposites with drainage capability resulted in higher peak pullout resistance than the conventional geocomposites for all cases of drainage (zero, partial and full) in marginal soils, where it is efficient at higher applied stresses due to rapid dissipation of excess pore water pressures through its drainage elements.

Recently Hatami and Ismaili (2015) conducted a series of small scale direct shear and pullout tests to investigate the effect of matric suction and the compaction water content of different un-saturated marginal soils on the interface behavior of reinforced soil slopes and embankments. A series of suction controlled tests on soil-geotextile interfaces were investigated. The same direct shear test equipment was used to conduct both interface and pullout tests where few modifications were added to allow pulling the geotextile as shown in figure 2.14 where a geotextile inserted between two layers of clay compacted in the mold and attached to a clamp, the two halves were bolted and the entire cell was moved relative to the clamped geotextile.

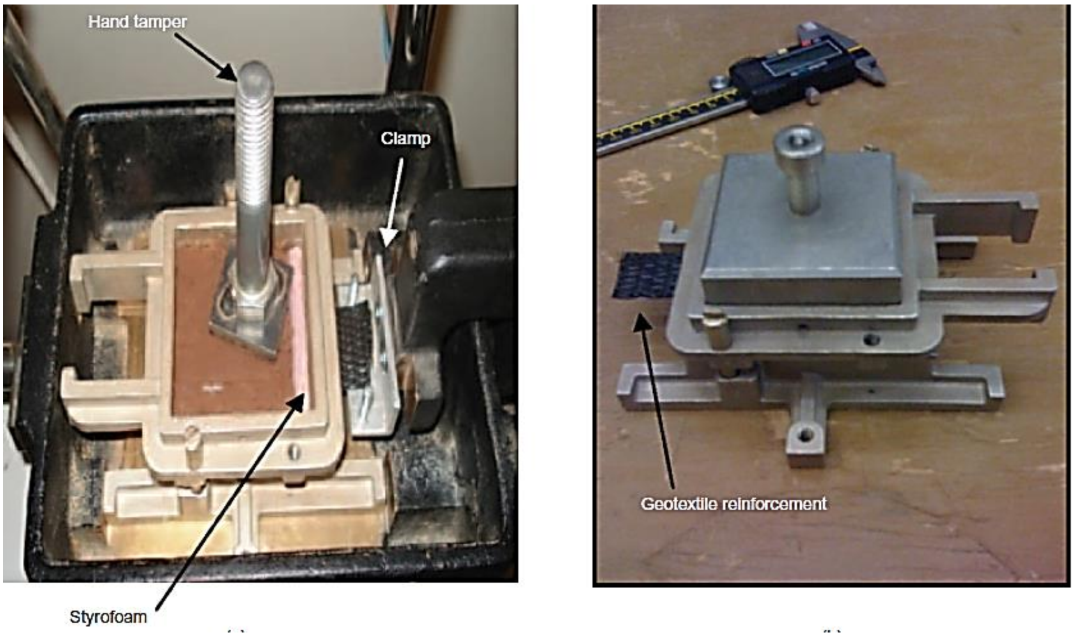


Figure 2.14 Hatami and Ismaili (2015) Pullout Set-Up.

Three different marginal soils classified as SC, CL-ML and CL according to the USCS, were tested and a woven polypropylene geotextile was used. Soils were mixed with water to reach a target water content. Three different water contents were investigated: 2% wet of optimum, optimum and 2% dry of optimum. Soils were compacted in the direct shear mold to reach the target dry unit weight. In pullout tests different boundary conditions and rates were studied (i.e. 0.065 mm/min and 1 mm/min shearing rates).Three normal stresses varied between 10 and 50kPa representing the top cover of reinforced soils where pullout failure is more likely to occur are applied. Load application and data collection is shown in figure 2.15.

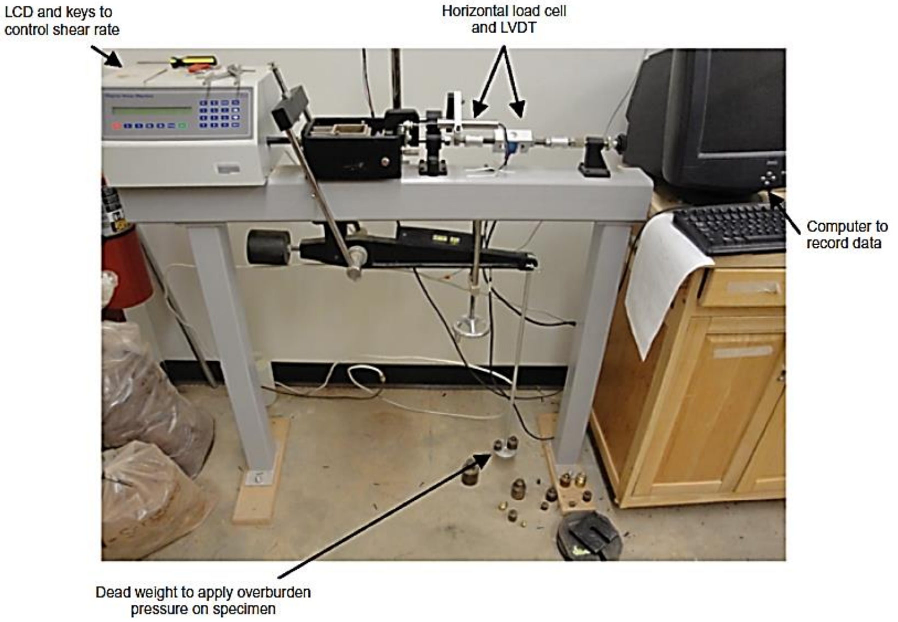


Figure 2.15 Hatami and Ismaili (2015) Load application and data collection.

For interface direct shear tests soils placed in the upper half sheared against the geotextile fixed to an aluminum block and placed in the lower half, after application of the normal stress the lower half is pushed until a maximum allowed displacement of 10mm is reached. Pullout tests results were presented as Mohr-Coulomb failure envelopes shown in figure 2.16 where matric suction values are shown for each test.

Results indicates that the interface shear strength increase with the increase of suction and increase of applied normal stress for all types of tested soils and different boundary conditions.

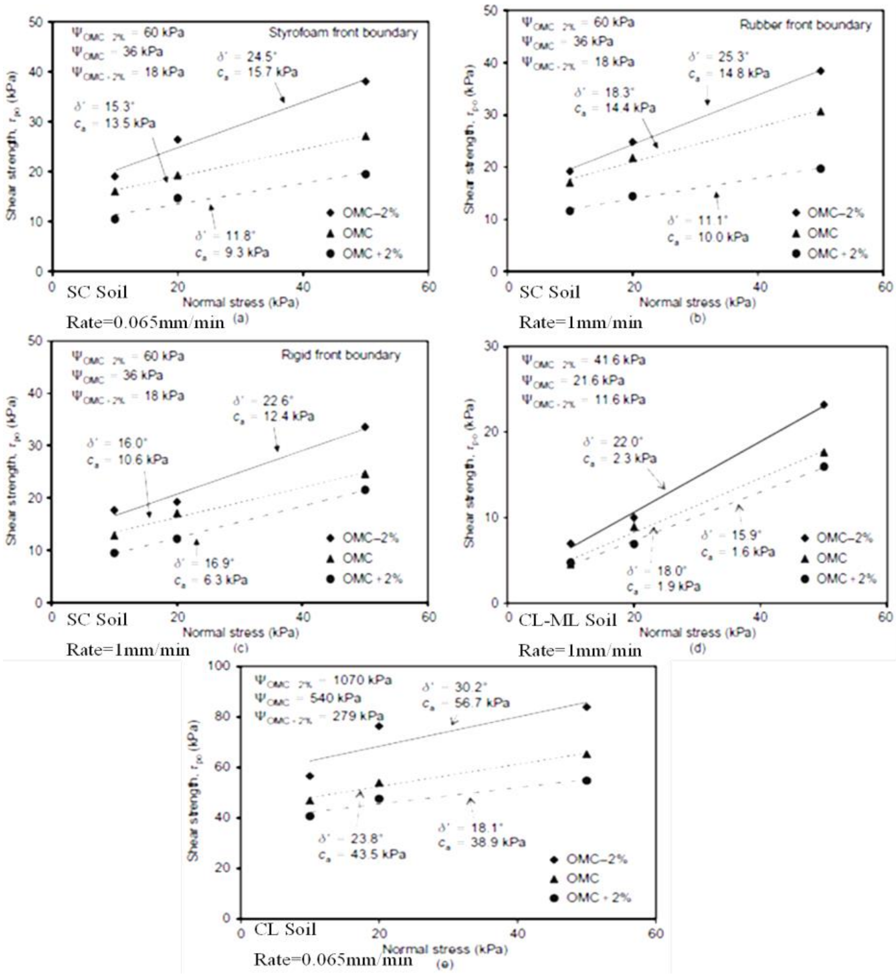


Figure 2.16 Hatami and Ismaili (2015) Mohr Coulomb Failure Envelopes from small scale pullout tests.

Results shown in figure 2.16 indicate that the friction angle and adhesion values for SC soil and geotextile at wet of optimum water contents are 56% and 49% smaller than those of dry of optimum, and for (CL-ML) soils the difference is 40% and 31% between wet and dry of optimum and those for CL soil 40% and 31%, respectively. Results from direct shear tests indicated that the interface shear strength was consistently higher with large values of matric suction (lower water contents). It is concluded that for both direct shear and pullout tests soil adhesion is influenced by matric suction.

The comparison between the obtained interface parameters from pullout and direct shear test are shown in table 2. 8. It is shown that the interface friction angle obtained from pullout tests is more significantly affected by suction and compaction water content than those obtained from direct shear tests, this was attributed to the fact that in pullout tests the geotextile is stretched resulting in additional penetration of soil particles in the geotextile layer.

Test information			Pullout tests						Interface shear tests	
Soil type	GWC	Suction (kPa)	Front boundary condition	τ_{po} (kPa)			δ' (°)	C_a (kPa)	δ' (°)	C_a (kPa)
				Overburden pressure (kPa)						
				10	20	50				
OUM-NCMA (SC)	OMC−2%: 10.6%	60	Styrofoam	19.0	26.4	38.0	24.5	15.7	28.9	5.1
			Rubber	19.2	24.8	38.4	25.3	14.8		
			Rigid	17.7	19.2	33.6	22.6	12.4		
	OMC: 12.6%	36	Styrofoam	16.0	19.3	27.1	15.3	13.5	27.1	3.5
			Rubber	17.0	21.8	30.7	18.3	14.4		
			Rigid	12.8	17.1	24.6	16.0	10.6		
	OMC+2%: 14.6%	18	Styrofoam	10.5	14.7	19.4	11.8	9.3	25.5	2.7
			Rubber	11.6	14.5	19.7	11.1	10.0		
			Rigid	9.5	12.2	21.5	16.9	6.3		
Minco silt (CL-ML)	OMC−2%: 10.7%	41.6	Rigid	7.0	10.0	23.2	22.0	2.3	30.2	8.0
	OMC: 12.7%	21.6	Rigid	4.6	8.9	17.7	18.0	1.9	30.3	6.2
	OMC+2%: 14.7%	11.6	Rigid	4.7	6.9	16.0	15.9	1.6	30.5	2.6
Chickasha clay (CL)	OMC−2%: 16%	1070	Styrofoam	56.5	76.3	83.8	30.2	56.7	21.0	34.8
	OMC: 18%	540	Styrofoam	46.8	53.9	65.2	23.8	43.5	20.0	27.0
	OMC+2%: 20%	279	Styrofoam	40.6	47.5	54.7	18.1	38.9	18.0	21.2

Table 2.8 Hatami and Ismaili (2015) Interface parameters obtained from direct shear and pullout tests.

Furthermore, a set of moisture reduction factors (MRF) was calculated which account for the reduction in the interface shear strength at water contents wet of optimum. It is concluded that this reduction is significant and needs to be accounted for in design. For the optimum+2% the MRF varies between 54 and 75% for SC soils and varies between 66% and 76% for (CL-ML) and 64% for CL soils. This reduction was directly attributed to the loss of matric suction (reduction in soil adhesion) at higher water contents.

D. Summary

Based on the above literature review it is clearly shown that for both direct shear and pullout tests the interface shear strength or the peak pullout resistance increase with the increase of the applied normal stress. Moreover, shearing the sample at a fast rate in direct shear test or using a fast pullout rate in pullout tests leads to an increase in the interface strength due to the probable generation of negative pore water pressures in the soil near/at the interface. Furthermore the increase of moisture content leads to a decrease in the shear strength for both direct shear and pullout test due to possible decrease in soil suction accompanied with the increase in moisture content and development of excess pore water pressures.

In general it is recommended that both direct shear and pullout test should be conducted to obtain interface parameters between soils and reinforcement materials in order to develop a better understanding of the interface behavior under different applied conditions. Moreover, the obtained interface parameters will indicate pragmatic and actual values to be used in models predicting the shear strength of reinforced soil composites in different field conditions.

CHAPTER III

EXPERIMENTAL PROGRAM
AND
SAMPLE PREPERATION

A. Introduction

Based on the background review and discussion presented in Chapter II of this thesis, it is clear that the mechanism that governs the interface behavior between natural fibers and clayey soils is not fully understood and/or definitively and clearly formulated. Such an understanding is required for providing input to models that aim at predicting the shear strength of clays reinforced with randomly distributed natural fibers. It is a prerequisite to developing better design methodologies and predictions for geotechnical systems that involve fiber-reinforced clays in general and clays that are reinforced with natural fibers in particular.

In this chapter, the materials used in the testing program are described in detail, including the natural clay and natural hemp fibers used in all tests. The characterization tests conducted on the clay soil include Atterberg limits, grain size distribution, specific gravity and hydrometer analyses. Tensile strength tests were conducted on the hemp fibers to determine their strength and stiffness properties with the fabric oriented in both the longitudinal and transverse directions. In addition, surface roughness measurements were conducted on hemp fiber surfaces.

An experimental testing program that is based on interface direct shear tests and single fiber pullout tests was designed and executed to define and determine the strength characteristics of the hemp-clay interface under various conditions of stress, preparation and testing. The single fiber pullout tests were conducted using a modified laboratory apparatus that was custom fabricated for this purpose. The parameters that were varied in the experimental program in addition to the type of test were (1) the compaction water content of the clay, (2) the applied normal stress, and (3) the rate of shearing / pullout.

In what follows, a detailed description of the sample preparation for each type of test is presented. The process includes mixing of the clay specimens, compaction of the clay into the test-specific mold (direct shear mold or the adapted 1-D consolidation mold), and incorporation of the hemp interface (direct shear) or hemp fiber (pullout). A detailed description of the testing protocols that were adopted in both tests is also presented in this chapter.

B. Test Materials

The experimental tests were conducted on two materials: natural clay and natural hemp fibers. In what follows is a presentation of the properties of each.

1. Natural clay

The clay was brought to the laboratory in a wet state from a local construction site in Kfarselwan, Lebanon. In order to determine its classification, both sieve analyses and hydrometer tests were conducted to establish its grain size distribution. Standard procedures proposed by the American Society for Testing Materials (ASTM) were implemented. Tests were replicated to ascertain results. The grain size distribution curves are shown in Figure 3.1. Index properties of the clay were determined in the laboratory using Atterberg limit tests and specific gravity tests (table 3.1). The soil thus tested classifies as "inorganic clay of low plasticity" (CL) in the unified soil classification system (USCS).

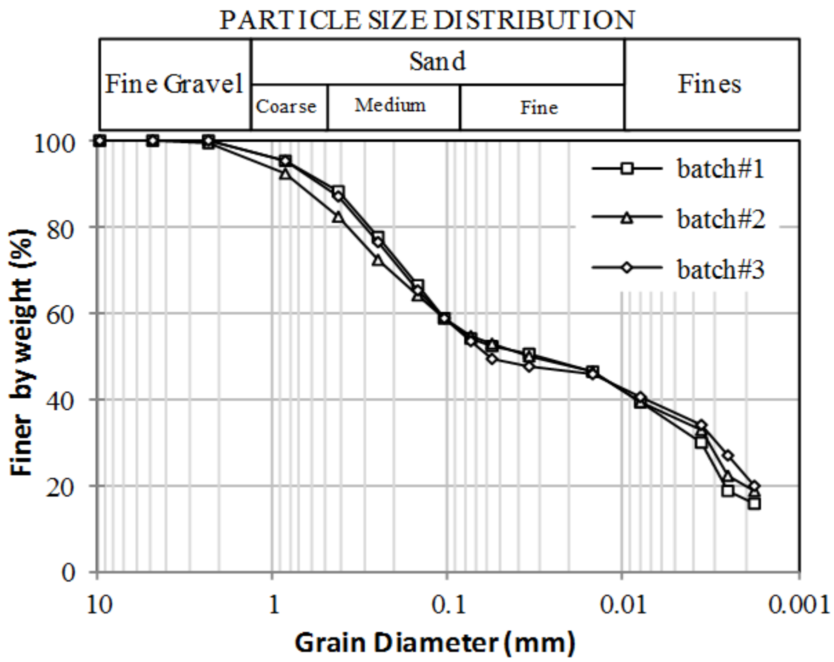


Figure 3.1 Grain size distribution curves of the used Clay.

Standard Proctor compaction tests (figure 3.2) were conducted to determine the full compaction characteristic curve and the maximum dry density and optimum moisture content. These which were found to be 16.8KN/m^3 and 19% respectively as shown in figure 3.2.

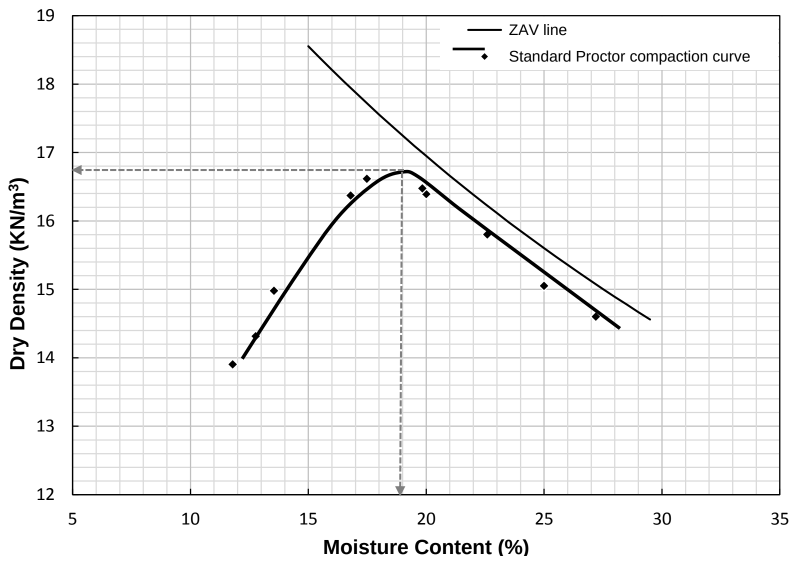


Figure 3.2 Standard Proctor compaction curve.

Table 3.1 Material properties of the used Clay.

	Batch #		
Clay Property	1	2	3
Liquid Limit,LL (%)	35	33	34
Plastic Limit,PL (%)	22	19	20
Plasticity Index, PI (%)	13	14	14
Specific gravity, G_s	2.64	2.64	2.64
Sand (%)	46	46	46
Silt (%)	37	35	32
Clay (%)	17	19	22
% Fines	54	54	54
Classification (USCS)	CL	CL	CL

2. Natural Hemp Fiber

Hemp fibers that are used in this study were imported from the Hemp-Traders-LA- USA as long fibers in the form of bast fiber bundle. Figure 3.3 shows a picture of hemp fibers as imported. Fibers were then treated in order to eliminate their organic impurities by soaking them in a sodium hydroxide solution (NaOH) at 6% by weight for 48 hours. After treatment, they were washed with clear water and left to air dry.

Hemp fibers, in general, have a rectangular cross section with a wide variability in dimensions. After a statistical sampling composed of measuring the thickness and the width of 250 single fibers, it was found that the average thickness is equal to 0.15mm and the average width is 0.65mm, with a standard deviation of 0.42mm, and a range of 2.4mm.

The mechanical properties of the hemp fibers were determined from tensile strength tests that were conducted on 20 individual fibers. The averaged ultimate tensile strength (UTS) was found to be equal to 276MPa with a standard deviation of 66MPa and a range varying from 181 to 415 MPa and the average modulus of elasticity was equal to 21.7 GPA with a standard deviation of 3.87 GPa, and a range varying from 16.1 to 29.8 GPa. The properties of hemp are summarized in table 3.2.

Figure.3.3.Hemp Fibers Used in this study

Table3.2. Mechanical Properties of Hemp

Hemp Fiber Property	
Specific gravity , G_f	1.4
Ultimate tensile strength (MPa)	276
Elastic Modulus (GPa)	21.7
Thickness(mm)	0.13
Width (mm)	0.65

In addition, fiber roughness was measured using a highly sensitive profilometer. Roughness was measured along the fiber surface in the longitudinal section, for both dry and wet conditions. Multiple measurements were considered along the fiber surface, average roughness for the fiber was found to be equal to 2.101μm for the case of dry fiber where the values ranges between 1.381μm and 2.891μm, while for the wet fiber the roughness was found to be equal to 1.806μm and the values ranges between 1.307μm and 2.485μm. Average roughness values are summarized in table 3.3.

Table3.3. Hemp Fiber Surface Roughness (μm)

Dry Longitudinal Section	2.101
Wet Longitudinal Section	1.806

In direct shear tests the interface between clay and Hemp was along the length of the fiber. For pullout tests where a fiber was inserted between two layers of compacted clay the interface surface is along the longitudinal section.

C. Experimental Testing Program

The aim of this study is to experimentally investigate the interface shearing resistance between Hemp fibers and clay. The interface shear strength was measured using a series of direct shear tests and single fiber pull-out tests where different parameters were varied.

1. Varied Parameters

The testing program investigated different parameters that were varied in the experiments under different testing conditions. The parameters included:

- The Compaction Water Content (degree of saturation)
- The Applied Normal Stress (20kPa,100kPa, and 200kPa)
- Rate of Shearing/pullout (Rapid/Undrained versus Slow/Drained)

<u>a. Testing Conditions</u>

i. Consolidated drained (CD) and unconsolidated undrained (UU) tests were conducted on the clay and Hemp-clay interfaces in order to simulate long and short term loading conditions, respectively.

ii. Clay samples were compacted at different water contents (14%, 18%, and 20%) to simulate conditions at optimum (w=18%), and dry (w14%) and wet of optimum (w=20%).

<u>b. Type of Tests</u>

Identical tests (same parameters) were conducted using both direct shear and pullout testing to investigate the effect of the type of loading on the interface properties at the contact of the two materials.

i. <u>Direct Shear Testing</u>

A series of 36 direct shear tests were performed on a compacted natural clay that was prepared by kneading using the miniature Harvard compactor. The tests were conducted under drained (Table 3.4(a)) and undrained (Table 3.4(b)) conditions, respectively. The tests included clay/clay and clay/hemp interface tests conducted at different water contents and normal stresses as indicated in the associated tables.

Table 3.4 Direct Shear Soil Testing Program (a) Drained Tests, (b) Undrained Tests.

(a)	Direct Shear Test/ Consolidated Drained		
Test #	Water Content Tested (%)	Confining Pressure σ (kPa)	Soil-Soil/Soil-Fiber
1	**20**	20	S-S
2		100	S-S
3		200	S-S
4		20	S-F
5		100	S-F
6		200	S-F
7	**18**	20	S-S
8		100	S-S
9		200	S-S
10		20	S-F
11		100	S-F
12		200	S-F
13	**14**	20	S-S
14		100	S-S
15		200	S-S
16		20	S-F
17		100	S-F
18		200	S-F

(b)	Direct Shear Test/ Unconsolidated UnDrained		
Test #	Water Content Tested (%)	Confining Pressure σ (kPa)	Soil-Soil/ Soil-Fiber
19	**20**	20	S-S
20		100	S-S
21		200	S-S
22		20	S-F
23		100	S-F
24		200	S-F
25	**18**	20	S-S
26		100	S-S
27		200	S-S
28		20	S-F
29		100	S-F
30		200	S-F
31	**14**	20	S-S
32		100	S-S
33		200	S-S
34		20	S-F
35		100	S-F
36		200	S-F

The objective of the direct shear tests was to characterize the parameters of the clay/Hemp interface in both drained and undrained conditions. The three applied normal stresses allow for the determination of the Mohr Coulomb envelopes and the associated shear strength parameters (soil cohesion and friction angle of clay tests, and the soil adhesion and interface friction angle of the soil-hemp tests).

ii. Pullout Testing

In the second series of tests (table 3.5), a total of 18 consolidated drained and unconsolidated undrained single fiber pullout tests were conducted using the same soil and test parameters of the direct shear tests. A modified pullout apparatus was designed and implemented to pull out a fiber horizontally from the clay using a pulley that is loaded with a container that is filled with water using a sensitive burette. A single fiber was inserted between two layers of compacted clay and then pulled out at different confining conditions and loading rates. Sample preparation and testing procedures are discussed in the following section.

The objective of this series of tests was to characterize the interface parameters of clay with the hemp fiber in both drained and undrained conditions where the fiber was pulled out horizontally from the compacted clay. The three applied confining pressures allowed for the evaluation of the Mohr Coulomb envelopes and for comparison with the direct shear test results. The measured peak pullout stress representing the ratio of the measured pullout force to the contact area represents the major output parameter from this series of tests.

Table 3.5 Pullout Soil Testing Program (Drained and Undrained Tests).

Pullout Test			
Test #	Water Content Tested (%)	Confining Pressure σ (kPa)	Drained/ Undrained
1	20	20	CD
2		100	CD
3		200	CD
4		20	UU
5		100	UU
6		200	UU
7	18	20	CD
8		100	CD
9		200	CD
10		20	UU
11		100	UU
12		200	UU
13	14	20	CD
14		100	CD
15		200	CD
16		20	UU
17		100	UU
18		200	UU

D. Sample preparation and Testing

The sample preparation procedure of the compacted clay and the associated hemp fiber/interface for direct shear and pullout tests are presented and described in this section.

1. Preparation of the Compacted Clay

The natural clay was oven dried and then crushed and sieved through a number 10 sieve. The dry clay r was stored in trays in the oven until sample preparation. Prior to sample preparation, the clay was removed from the oven and subjected to ambient temperature for 30 minutes. A key step in preparing the clay samples involves mixing the dry clay “powder” with water to achieve the desired moisture content as indicated by the experimental program for each test (tables 3.4 and 3.5). Water was sprayed and mixed manually to ensure a homogenous wet mixture. The mixture was then sealed and left for 30 min to ensure water content homogenization.

The mixed soil was then placed in a split mold in batches of equal thickness. For each batch, the clay was compacted using the Harvard apparatus which consists of a cylindrical rod which can apply a controlled tamping pressure by means of an air-pressurized system (figure3.4). The target dry unit weight of the raw clay specimens compacted by the Harvard apparatus was assessed under different combinations of number of layers, number of tamps per layer, and tamping pressure. The objective was to determine the correct combination of the number of layers, the number of tamps for each layer and the tamping pressure required to reach the target unit weight which was set in this study to be equal to 90-95% of the standard proctor dry unit weight as recommended by common construction specifications.

For the direct shear tests, the specimens were compacted in a split mold with a height of 21cm and an internal diameter of 7.15cm. The required kneading effort consisted of 5 layers with each layer being compacted by 25 tamps at a tamping rod pressure of 10 psi pressure each. For the pullout test, a custom-made split mold consisting of a galvanized steel pipe was used to prepare the compacted samples. Cylindrical specimens with a height of 20cm and an internal diameter of 8.25 cm were targeted. The required kneading effort for the pullout test specimens consisted of 6 layers with each layer being compacted with 25 tamps at 10 psi pressure each. The kneading effort was kept constant for all in the direct shear and the pullout tests.

Figure 3.4 Laboratory compaction tools the Harvard miniature apparatus.

Following the compaction process, the specimen is extracted from the split mold, trimmed to the required height, and assembled in the testing apparatus. The soil cuttings are then used to measure the moisture content of the specimen. The sample dimensions and weight are measured. The advantage of using a split mold is to ensure that the compacted clay specimen could be removed from the mold with minimal disturbance. Additional measures were taken to further eliminate the disturbance of the sample. For example, the internal surfaces of the mold were covered with plastic tape and then coated with a thin layer of lubricant in order to reduce the friction between the compacted clay and the inner surfaces of the mold and facilitate sample separation from the mold. The major steps that were followed during the process of sample preparation are presented in figure 3.5.

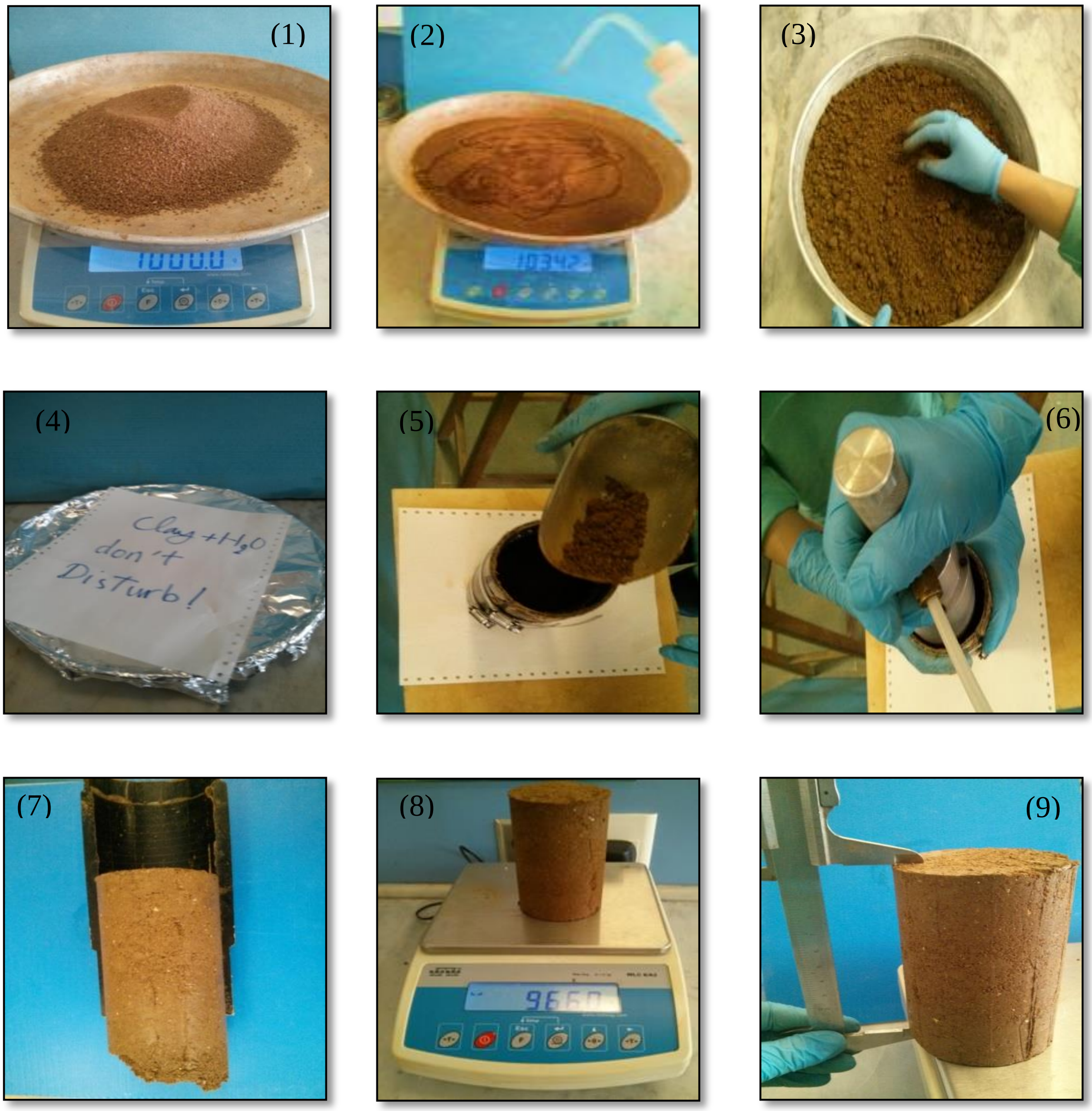


Figure 3.5 Procedures of compacted clay sample preperation.

2. Direct Shear Testing

In the direct shear tests, the parameters that were varied are the normal stress (20, 100, and 200 KPa), the clay water content (14%, 18%, 20%), the state of consolidation (consolidated, unconsolidated), and the shear displacement rate (slow, fast).

<u>a. Sample Preparation Prior to Placement in the Direct Shear Box</u>

Direct shear tests were conducted according to ASTM (D3080) using a circular shear box composed of two halves of equal thicknesses. The direct shear mold is of an internal diameter of 2.5” and a total height of 2”. Two porous stones that allow drainage and transfer shear stress from the insert were placed at the top and bottom boundaries of the sample. Two couples of screws were used to (1) fix the shear box during the application of normal stress and (2) create a space (gap) between the top and bottom halves of the shear box prior to shearing (figure 3.6).

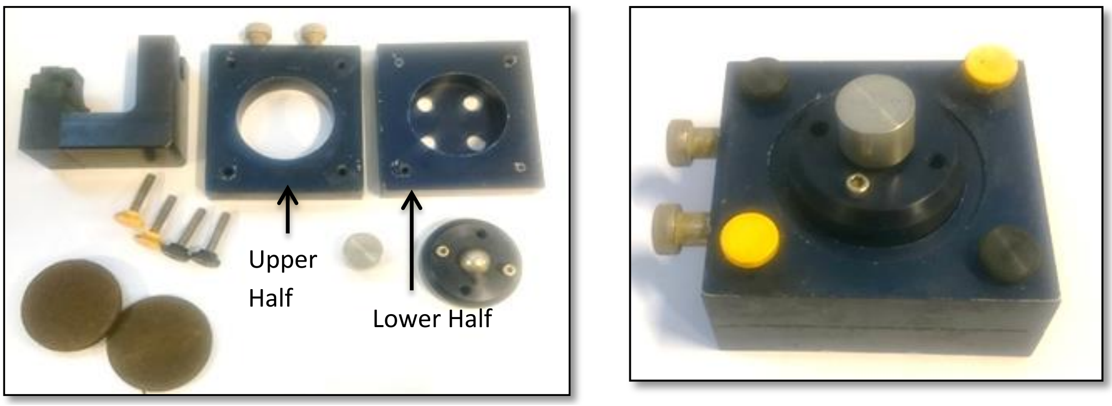


Figure 3.6 Direct shear box

It should be noted that the diameter of the sample after compaction in the split mode is generally 7.15 cm, while the diameter of the shear box is 6.35cm. As a result, the diameter of the specimen had to be reduced through a process of trimming. The step-by-step procedure that covers the process of extruding the compacted clay from the split mold up to the trimming of the specimen to the final state is illustrated in Figure 3.7. The inner sides of the mold were coated with a thin layer of lubricant to facilitate the trimming process and reduce sample disturbance.

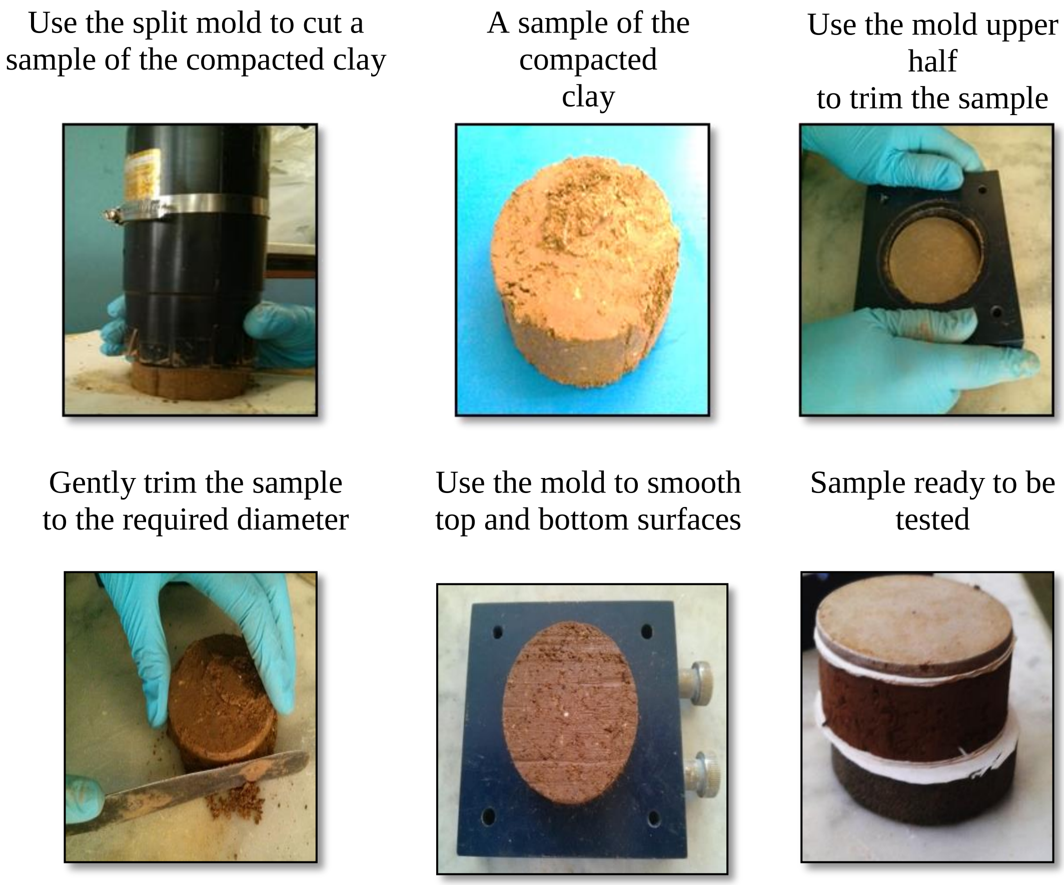


Figure 3.7 Step by step procedure to trim a sample from the compacted clay .

b. Consolidated Drained (CD) Direct Shear Testing

i. Control Tests

In the control tests that are aimed at measuring the response of the clay, the clay sample was placed in the specimen ring assembly and was sandwiched with two porous stones that were covered with filter paper. The normal stress was then applied by loading the top plate through a ball bearing adapter. The applied normal stress is controlled by Software that regulates the force applied to the top of the plate with the help of a vertical load cell. The system is equipped with a horizontal load cell that measures the horizontal force applied to the sample during the shearing stage as indicated in figure 3.8. The horizontal and the vertical load axes are adjusted from the software from the Tools menu.

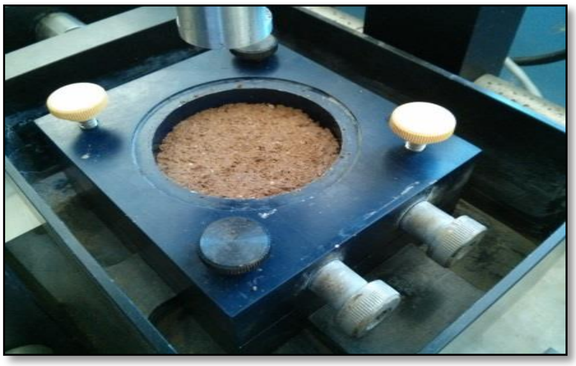
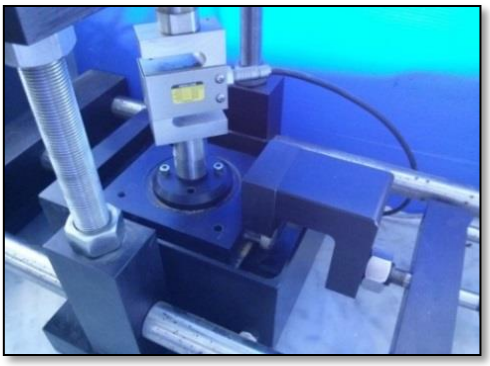



Figure 3.8.Setting the clay sample in the Direct Shear machine.

In the consolidated drained tests, the direct shear box is filled with water after the placement of the specimen ring assembly. The test consists of three stages: the seating stage, the consolidation stage, and the shearing stage. Each stage is characterized by a series of commands that appear on top of the screen and guide the user throughout the test. Three tabs, which represent each stage, become active after specimen and test data files are created. A specific tab representing a specific stage become active only after the previous stage is completed. The following steps describe the detailed procedure followed in performing consolidated drained tests (CD).

The first step was to create a specimen and test data file. After the adjustment of the specimen ring assembly to the required centered position, all the sensors and load transducer readings were set to zero. This step was accomplished by entering the "**Set Up**" menu and choosing "**Sensors**" leading to a dialogue box (see figure 3.9). After highlighting the required sensor and pressing "**Test**", a window of the selected sensor appears. In this shown window, the "**Take Zero**" button was pressed so that the sensor reading indicates the average readings that are almost zeros. This process should be repeated for the four sensors, i.e. horizontal load cell, vertical load cell, the vertical and horizontal load cells DCDT and DCDT2 respectively. Figure 3.9 shows a step by step procedure for setting the sensors reading to zero.

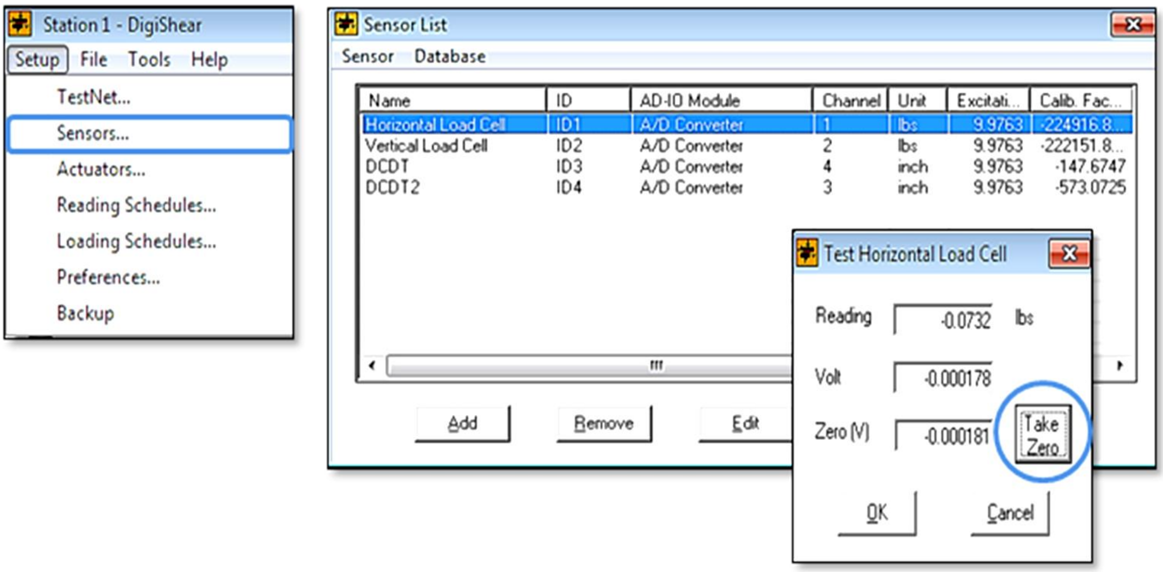
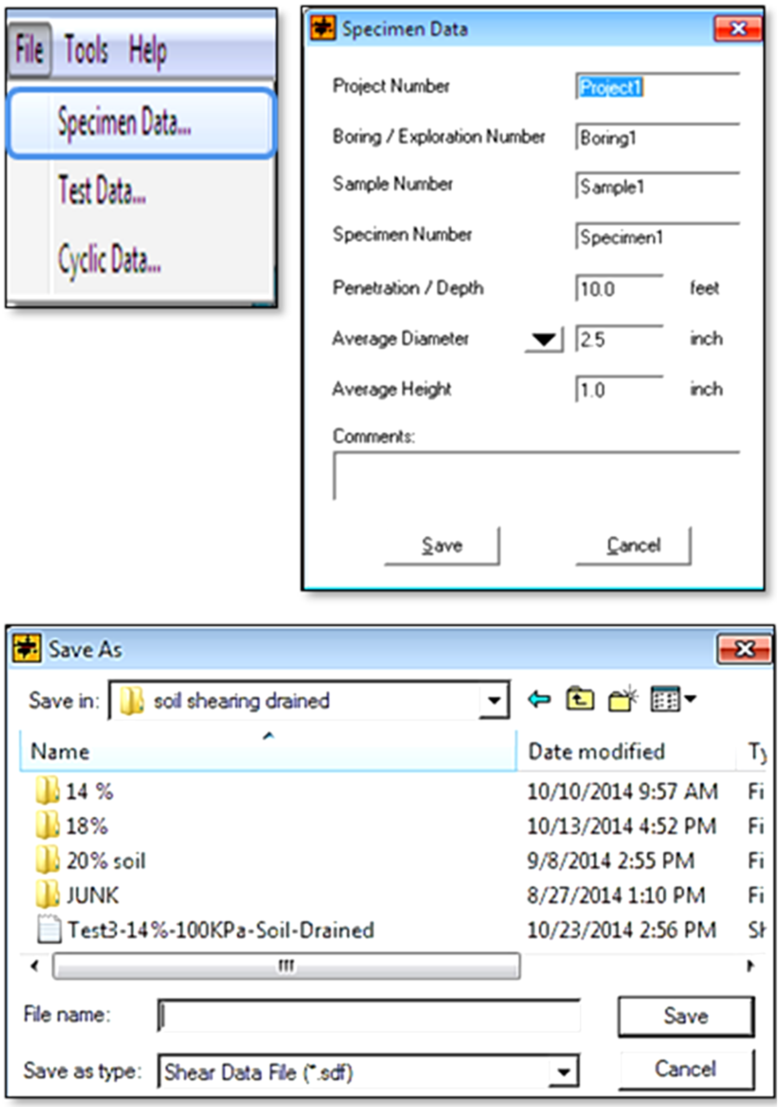


Figure 3.9.Setting sensors readings to zero.

The second step involves accessing the "**File**" menu and then choosing "**Specimen Data**". A specimen data dialogue box appears which the user fills to insert the properties of the tested sample including the samples average height, its average diameter, the sample number and project number. The file may then be saved in a selected folder as shown in figure 3.10.



Figures 3.10 Saving a specimen file data.

The third step was to select the "**File**" menu followed by "**Test Data**" in order to select the applied load as shown in figure 3.11. In this step, the parameters of the consolidation stage and shearing stage are determined including the loading schedule during the stage of consolidation and the displacement rate, displacement limit and load limit during the shearing stage. However, those parameters could be changed at a later stage.

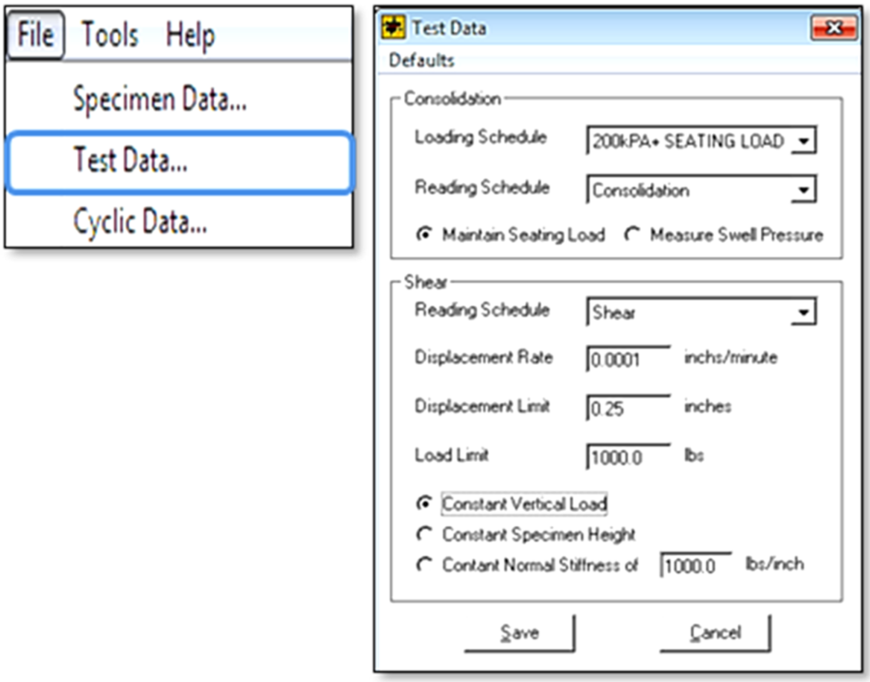
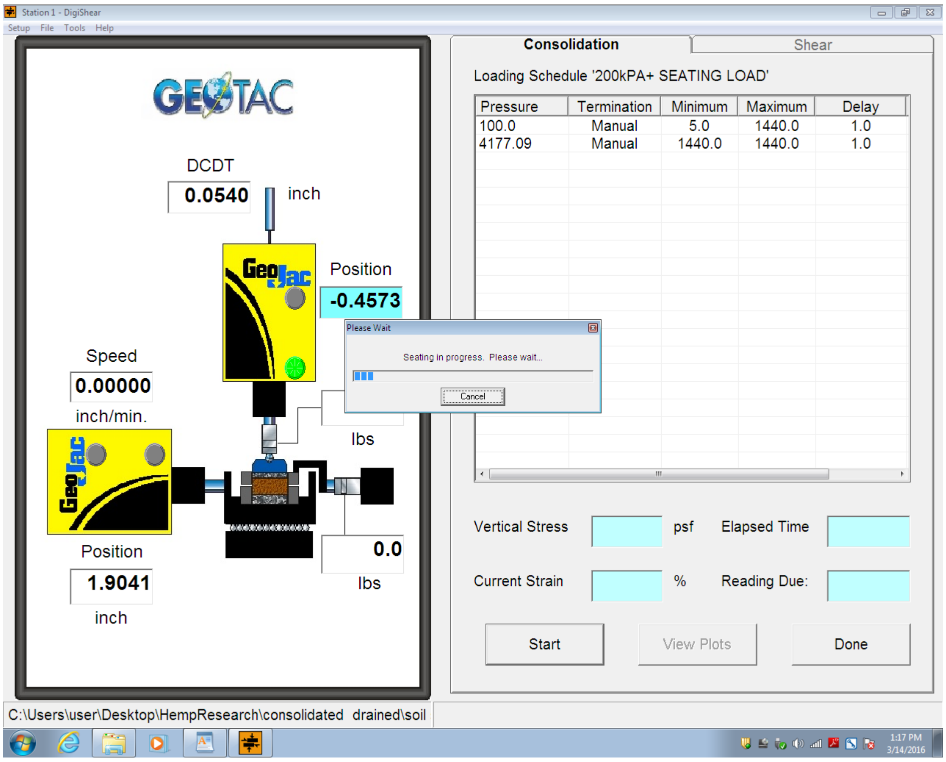


Figure 3.11.Saving a test data file.

In the test data dialogue box, in the consolidation section the tested load is chosen where it has been saved before in the loading schedule section. The applied load and the duration of its application are selected from this dialogue box. A seating load of a minimum value of 5kPa for a minimum period of 5 min was then applied to ensure that the vertical load is applied properly to the sample. To apply a seating load, the Tab “maintain a seating load” should be selected in the consolidation section in the test data dialogue box. The seating load stage is initiated after pressing the start button. A seating load in progress dialogue is shown in figure 3.12. In this example, a load of 5kPa was applied for a period of 5 min.



Figures.3.12. Initiating the seating load stage.

The other part of the Test Data dialogue box includes tabs related to the shearing section. In the shearing section, the displacement rate has to be input. Any value can be input at this stage since it can be changed later. In addition, a displacement limit of 2.5 inches and a load limit of 1000lb were assigned and the “constant vertical load” option selected (figure 3.11). On the screen, the consolidation tab is then highlighted and the values of the seating load and the applied load shown in psi units. However those assigned values should represent an equivalent value of stress in kPa.

After the completion of the seating load stage, the consolidation stage was initiated under the predetermined loads (mainly 20kPa, 100kPa and 200kPa). During consolidation the specimen was left to consolidate for a period of 24 hours under the applied load. The Truepath Software allows for the display of the deformation versus log time curve in the "view graph" button. When the curve reaches a steady state it means that there is no more deformation taking place in the sample and the consolidation is over as shown in figure 3.13.

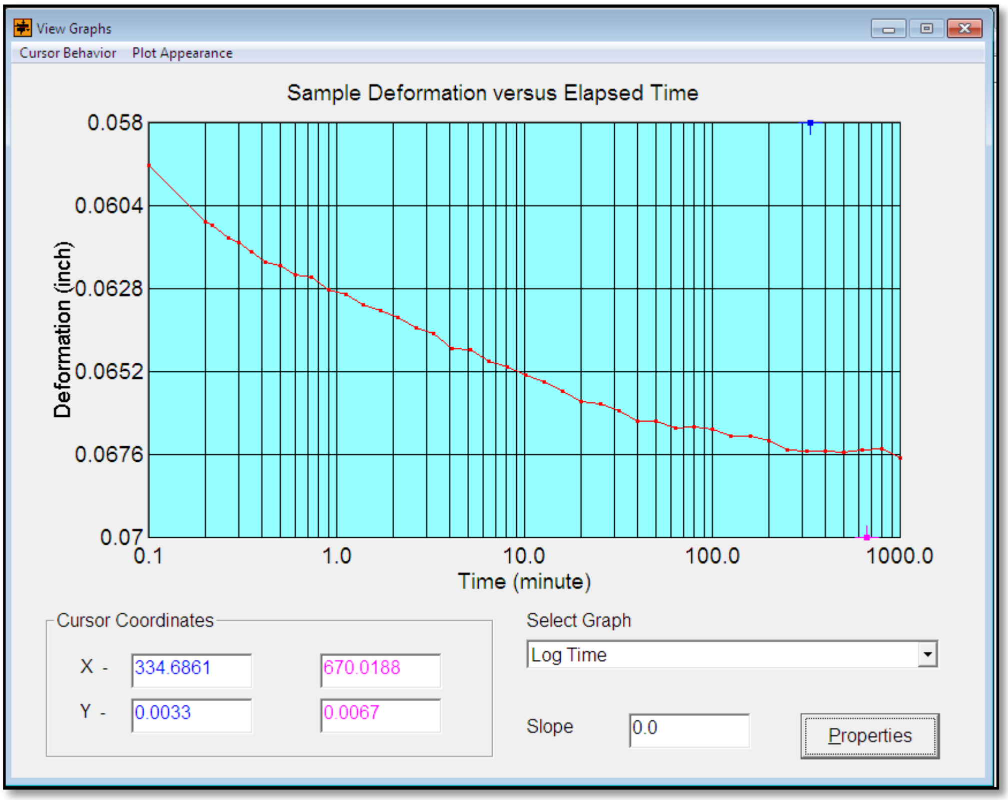


Figure.3.13 Deformation versus log time curve

The shearing stage can be activated after the completion of the consolidation stage which is dictated by (1) completion of the predetermined duration of load application or (2) manual cessation of the consolidation phase by pressing the “stop” button.

In the consolidated drained tests, shearing proceeded at a shearing rate that was selected in this project as that corresponding to the value of $50t_{50}$ as recommended by ASTM to ensure full dissipation of pore water pressures. The log time method was used to determine t_{50} which represents 50% consolidation. This value can be found graphically from the deformation-log time curve by observing the time that corresponds to 50% of the primary consolidation which is equal to the average deformation between 0% and 100%.

The corresponding rate of shearing R_d or the displacement rate in inch/min is equal to the ratio of the estimated relative lateral displacement at failure d_f (inch) over the total estimated elapsed time to failure t_f (min), where d_f was assumed to be equal to 0.2 inches and $t_f = 50.t_{50}$. After the calculation of the corresponding rate of shearing, the relative value is inserted at the space provided next to the displacement rate as shown on the screen of figure 3.14, in addition to the displacement limit and the load limit.

Before pressing the start box, the alignment screws (black couple) are removed and the gap screws (yellow couple) are used to separate the shear box halves approximately the diameter of the maximum sized particle in the test specimen or 0.025 in. [0.64 mm] as a minimum default value for fine grained materials. The shearing stage was then initiated. A warning message is generally displayed on the screen to remind the user to remove the alignment screws. The shearing stage is initiated with the predetermined displacement rate and was considered completed after reaching either the displacement limit or the load limit. The Software displays the shear force (lbs.) versus shear displacement (inches) curve in the "view graph" button. The obtained graph is shown in figure 3.15.

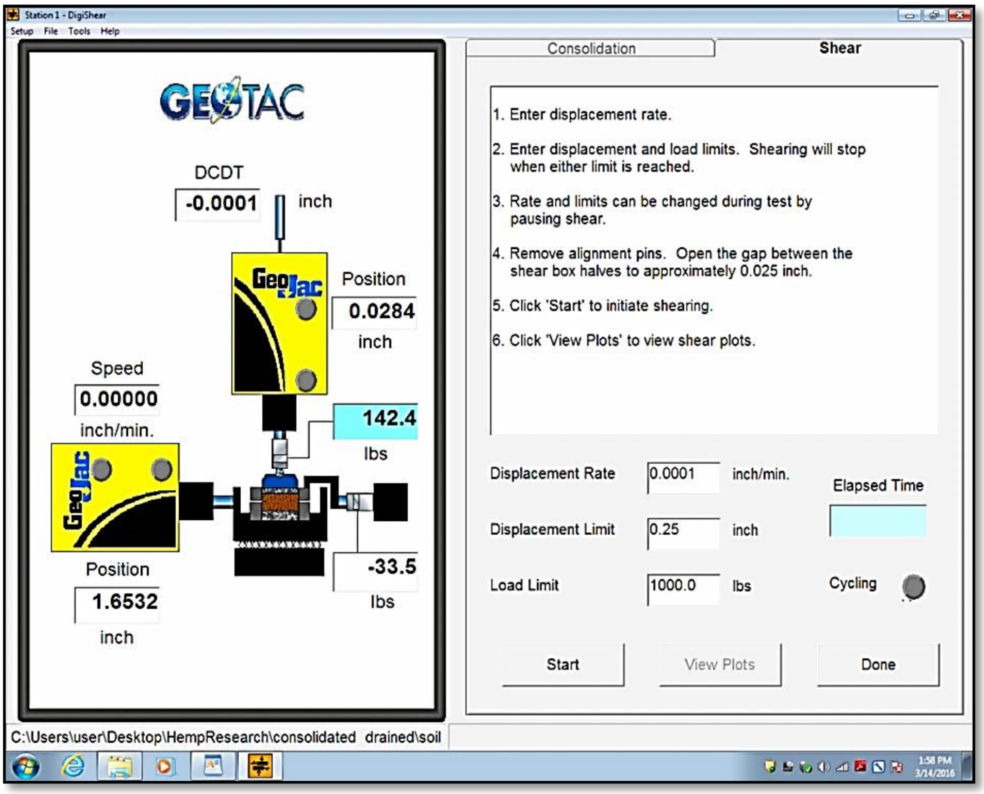


Figure 3.14. Inserting the corresponding values of shearing.

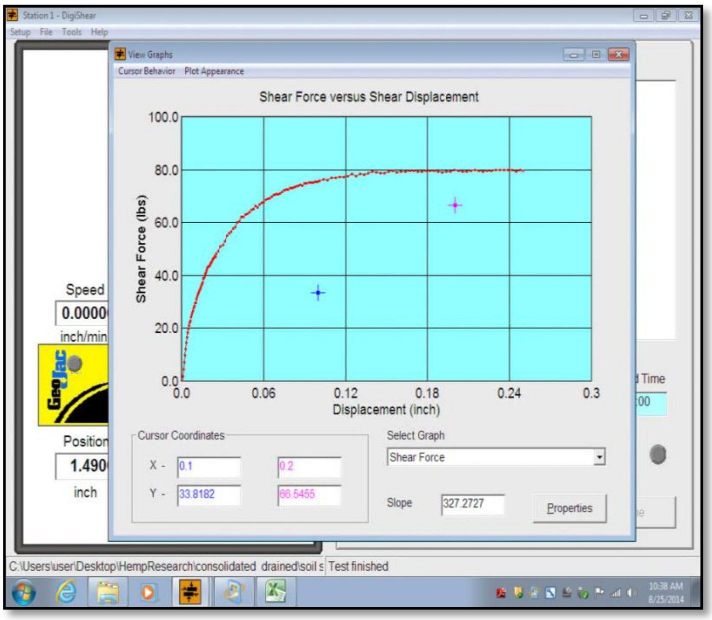


Figure 3.15.Shear force versus shear displacement curve.

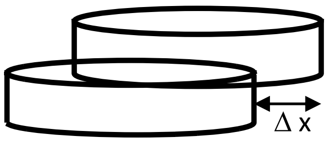
When shearing is completed, the normal force is removed from the specimen and the loading apparatus is disassembled. The shear box halves are separated in a sliding motion parallel to the failure plane to prevent damaging the sample. The sheared sample was removed in order to determine its water content and dry density. Figure 3.16 shows a sample after being sheared showing the “horizontal” shearing plane across the clay sample.

Figure 3.16 Horizontal failure plane of a sheared sample.

The saved file was imported to an associated “direct shear .xlt” file using the import tab and then the calculate tab. An excel file was then saved and data acquired by the horizontal LVDT, vertical LVDT, horizontal load cell, and vertical load cell recorded during both consolidation and shearing stages saved in the sheet “Data 1”. The shear stress data are calculated and saved in the “Result 1” sheet. However, the processed data needs to be corrected since the area decreases during shearing due to horizontal displacement.

For a circular direct shear box, the corrected area is calculated according to the following equation:

$A_c = r^2\ (2\alpha - \sin 2\alpha)$, where (1)



- $\alpha = \cos^{-1}(\Delta x/2r)$ in radians
- r is the radius in cm

The applied horizontal stress and vertical stress should be corrected by dividing the applied horizontal and vertical loads recorded by the load cells, by the corrected area. Corrected values were considered in all the results and data analyses presented in this thesis.

ii. Interface Tests

In the interface tests or the Clay/Hemp tests in which the interface resistance between the clay and the Hemp is targeted, the clay was prepared in the same way as for the control tests and trimmed to the required height which is almost half that of the sample tested in the clay/clay tests. The specimen was placed in the upper half of the shear box. The hemp fiber interface was then prepared by gluing a number of fibers to a prefabricated steel plate to be fixed within the lower part of the direct shear box as shown in figure 3.17. The two halves are fixed together using the gap screws and the reaction arm is used to fix the upper half of the shear box as shown in figure 3.17. The same procedures are followed to set the sample in the direct shear machine as before. A filter paper and a top porous stone were added to the upper surface of the sample in addition to the top cover and ball bearing adapter. The shear box was filled with water after the fixation of the specimen ring assembly.

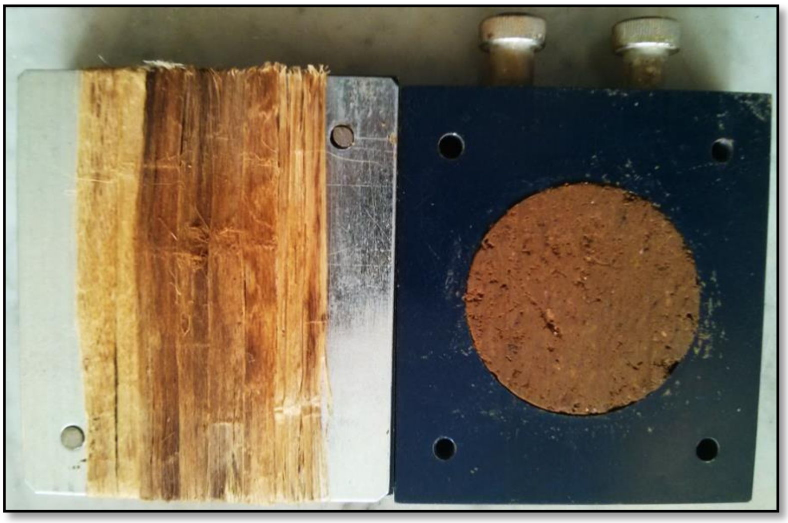
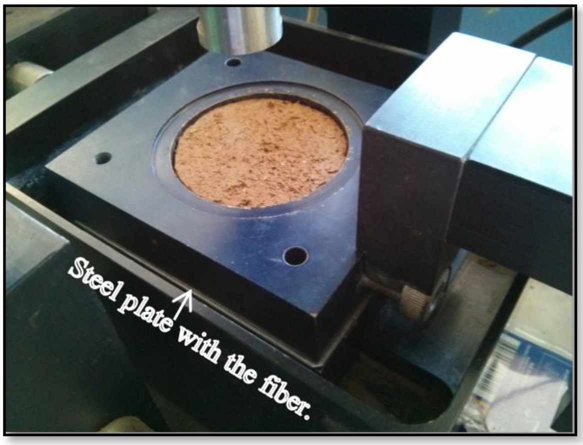


Figure.3.17. Clay in the upper half sheared to Hemp fixed in the lower half of the shear box.



Figs.3.18 Clay/Hemp interface tests in the direct shear machine.

For the consolidated drained test, the sample was left to consolidate prior to shearing and then sheared according to the calculated rate. The same steps in consolidation and shearing were followed as in the control tests. Clay fixed in the upper half box was sheared against Hemp in the lower half of the shear box leading to a shear force versus displacement curve. When shearing is over the sheared sample was removed in order to determine its water content and dry density. It should be noted that in direct shear interface tests, the area doesn't need to be corrected since there is no change in the contact area between the clay and the fiber during shearing as illustrated in figure 3.19.

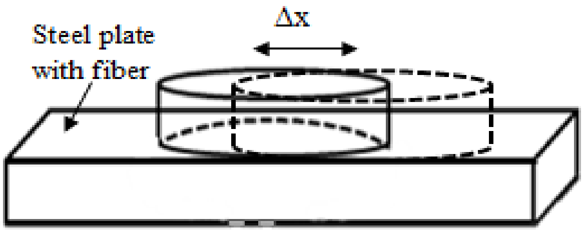


Figure 3.19.Contact area between clay and fiber in interface tests.

c. Unconsolidated Undrained (UU) Direct Shear Testing

i. Control Tests

The main difference between the consolidated drained and the unconsolidated undrained tests are the quick application of the normal stress and the fast shearing rate adopted. In addition, the direct shear box is not filled with water throughout the phases of the unconsolidated undrained tests. In the unconsolidated undrained direct shear tests the samples are not allowed to consolidate prior to shearing. After setting all sensors to zero and after creating a specimen data file and test data file, the seating stage starts. In order to ensure that the normal loading system is aligned and centered, a seating load of 5kPa was applied under which the specimen should not undergo significant compression. When the seating stage was over, the direct shear machine automatically moved to the consolidation stage. Since no consolidation is allowed, the "stop" button should be pressed to immediately move to the shearing stage.

After removing the alignment screws and creating a gap between the two boxes, the sample is sheared at a fast enough rate to minimize dissipation of pore pressures during shear. A displacement rate of 0.05inch/min and a shearing limit of 0.25 inch were adopted to ensure an undrained shearing (the total period of shearing was 5 min). A shear force versus shear displacement curve was thus obtained, and the sheared sample is removed in order to determine its water content and dry density.

ii. Interface Tests

In the interface tests, the clay and hemp were fixed as in the case of the consolidated drained tests. The hemp fibers used were assured to be dry prior to shearing to ensure undrained conditions. The samples were not allowed to consolidate prior to shearing and they were sheared under the same fast rate used in the clay specimens. A shear force versus shear displacement curve was obtained and the sheared sample was removed in order to determine its water content and dry density. As with the case of the drained interface tests, no area corrections are required in the data analysis.

3. Pullout Testing

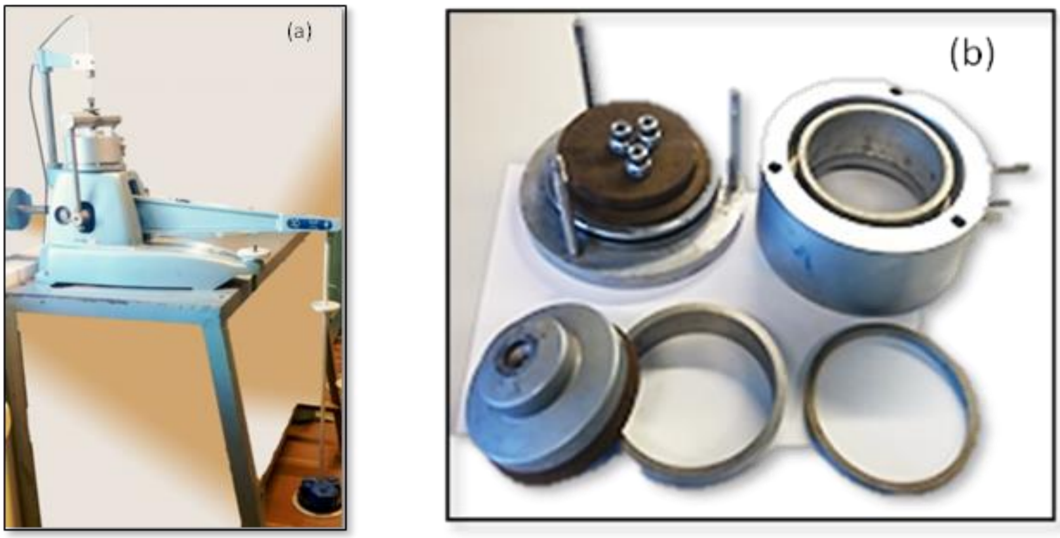
In the pullout tests, a single fiber that is placed between two layers of compacted clay was pulled out at different confining conditions and loading rates. Three normal stresses (20, 100, and 200 KPa) were applied on clay of different water contents (14%, 18%, and 20%). Consolidated drained and unconsolidated undrained conditions were investigated where shearing took place at different displacement rates.

a. Sample Preparation and Test Apparatus

A modified pullout mechanism was designed and implemented to pull out a fiber horizontally from the clay. The apparatus is based on the equipment and mold/ring used in conducting a 1-dimensional consolidation test (odometer). The custom-fabricated setup and ring are shown in figure 3.20. The mold/ring was modified to allow for the insertion of a single fiber between two layers of compacted clay. This was accomplished by drilling a 3-mm circular hole in the wall of the ring to facilitate the insertion of the fiber as indicated in figure 3.21. The ring has an inner diameter of 7.5cm and a height of 1.5cm. The hole is located exactly 1-cm above the bottom of the ring.

The specimen preparation is initiated by placing a 1-cm thick compacted clay specimen within the test ring. The fiber is then inserted horizontally within the fabricated hole and allowed to rest on the top of the 1-cm thick clay specimen, with a total horizontal embedment of 3.5cm within the clay specimen. A second compacted clay specimen with a thickness of 0.5cm is then placed on top of the bottom clay layer and fiber to produce a clay specimen with a total thickness of 1.5 cm with a single fiber that is sandwiched between the two specimens. This method of sample preparation was adopted to ensure that: (1) the fiber would be pulled out horizontally without any loss of horizontality, and (2) the clay specimen would remain below the water level needed to inundate the specimen during drained tests.

It should be noted that an additional prefabricated ring with a 7.5cm internal diameter and a 0.5cm thickness was fabricated to allow for preparing a compacted clay specimen with an associated thickness of 0.5cm.



Figures.3.20. Setup used in pullout test, (a) 1-Dimensional consolidation apparatus, (b) The used mold.

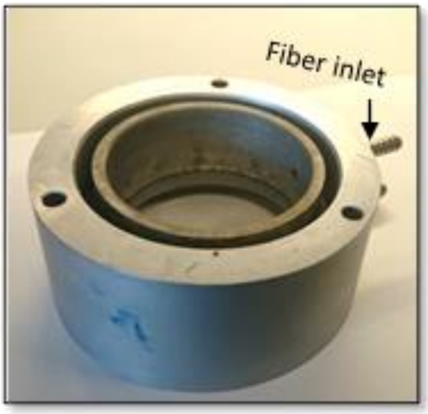


Figure 3.21 Test ring fabricated for fiber pullout tests.

The step-by-step procedure that was adopted in the process of preparing the pullout tests clay specimens is presented in figure 3.22. The pictures clearly depict how the fiber is inserted into the mold and laid out on top of the lower clay specimen in preparation for placing the upper part of the clay specimen.

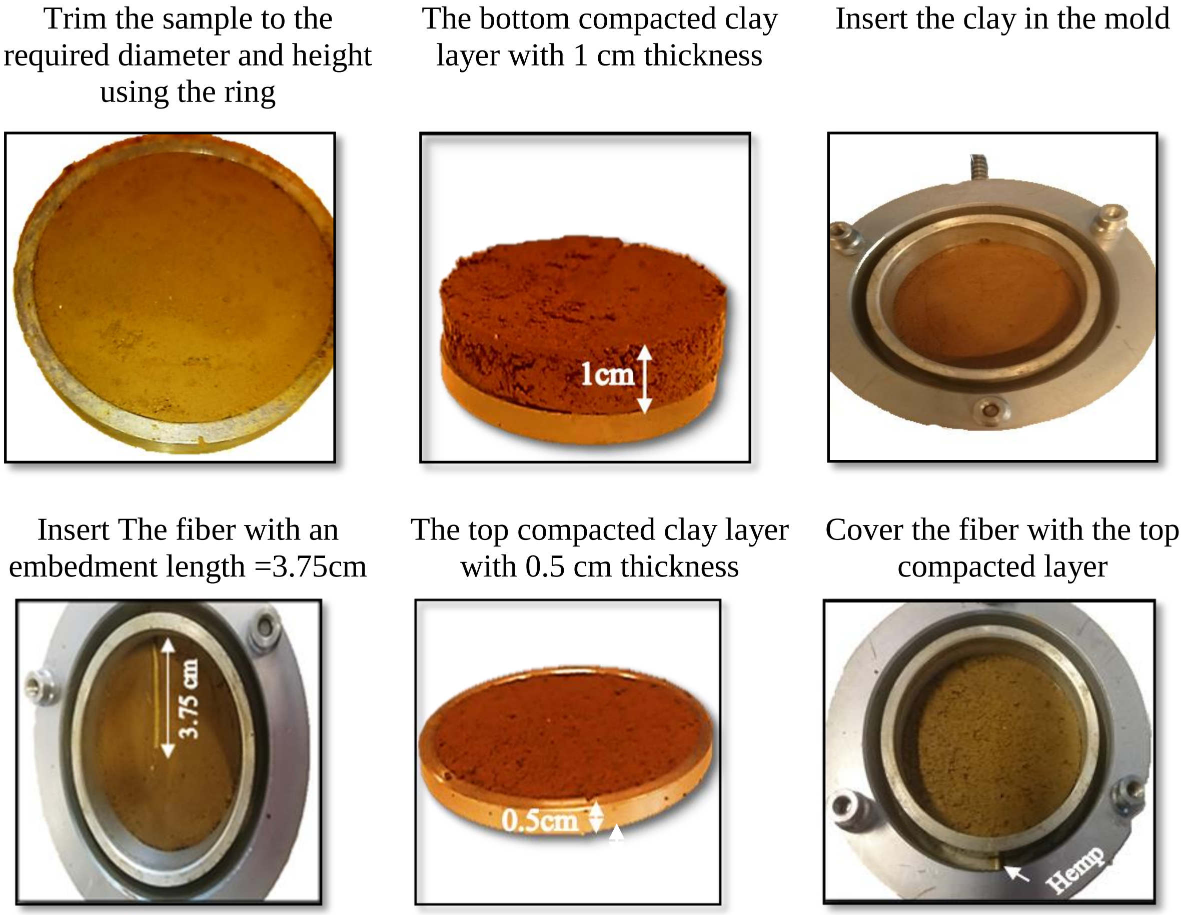


Figure 3.22 Step by step procedure for preparing the sample in the pullout mold.

Several mechanisms that would allow for pulling out the fiber from the soil were investigated. The first set of trials indicated that the main challenge in designing the pullout mechanism involved improvising a system that would allow for pulling out the fiber without risking the chance that rupturing the fiber prior to reaching the load necessary to fail the fiber in pullout as would be desired. The solution that was implemented involved the utilization of a junction tool that would act as a connection between the hemp fiber that is protruding out from the sample and a rod that can support high tensile loads without rupturing. In addition, to ensure that the pullout mechanism is horizontal and stable, the junction was cased with a steel jacket and the pullout rod was centered as shown in figures 3.23 and 3.24. One end of the fiber was covered with tape and fixed to the junction tool with screws. Tape was used to maximize the thickness, protect the fiber end from splitting, and ensure bonding between the fiber and the junction. Furthermore, the fiber was chosen with a highest width to ensure that the fiber won't rupture before failure occurs. The width of the tested fibers ranges between (1.8mm and 2.4mm).

Figure 3.23 Used junction tool.

Figure 3.24 Hemp attached from one side and rod from other side.

The pullout mechanism was achieved using a pulley system which allows for the transfer of vertical loads (applied through a container that is filled with water) to horizontal forces that would eventually allow for pulling out the fiber from the clay. The application of the load was controlled using a sensitive burette by controlling the rate at which water flow is allowed to the container. As mentioned previously, the pullout tests involved both consolidated drained and unconsolidated undrained tests. Each water content was tested under three applied loads (20kPa, 100kPa and 200kPa). The application of the normal stress was accomplished using the lever arm of the 1-D consolidation device. The weights associated with the application of the different normal stresses could be calculated according to the following equation and are shown in table 3.6

$$\sigma_n = \frac{m*\alpha*10*(10^{-3})}{A*(10^{-4})} \quad (2)$$

Where,

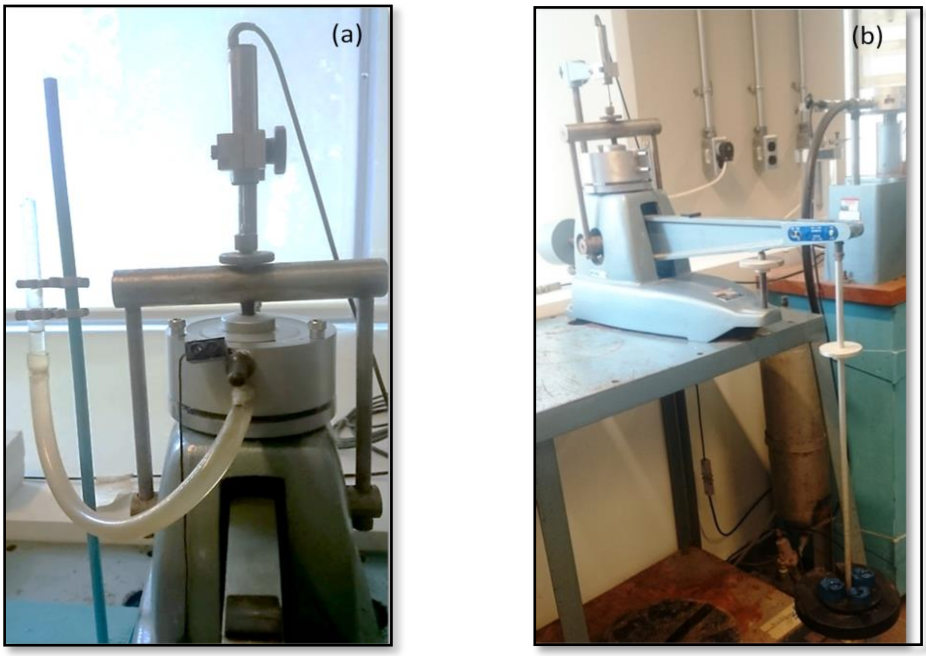
- σ_n is the required normal stress in kPa
- m is the needed weight in Kg
- α is the lever coefficient (equals to 10)
- A is the area of the sample in the mold = $\pi*d^2/4$ in cm^2

Table 3.6 Weights, equivalent to the applied stresses.

Applied σ_n (kPa)	20	100	200
Equivalent Weight (Kg)	0.885	4.42	8.835

b. Consolidated Drained Pullout Tests

After setting up the mold/ring in the 1-Dimensional consolidation device, water was added to the mold. The opening in the ring through which the fiber is placed was closed with a rubber stopper to prevent water losses during the test. Another inlet was also constructed in the side of the ring below the fiber at the level of the lower porous stone to act as a controlled drainage boundary. A drainage tube was attached to this inlet to act as a drainage boundary to and from the bottom of the specimen (figure 3.25).



Figures.3.25 Consolidated Drained test. (a) Mold filled with water, (b) Applied load.

The samples were allowed to consolidate under the applied normal stress. The vertical settlement was recorded versus time using an LVDT that was attached to the top of the sample (figure 3.25 (a)). A sample consolidation curve is shown in figure 3.26.

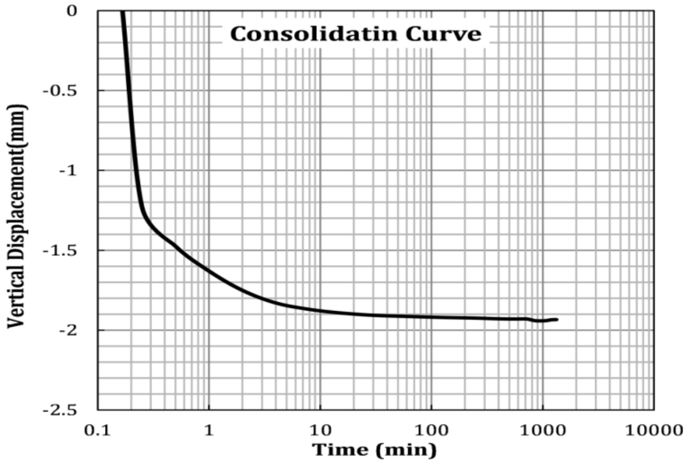


Figure 3.26 Vertical displacements (mm) versus log time (min) curve.

Samples were generally left for 24 hours to consolidate. After full consolidation was achieved, the fiber was pulled out horizontally using the pulley system shown in figure 3.27 Water was added gradually to the loading container using a sensitive burette with a capacity of 100 ml at a slow rate. The loading sequence that was adopted involved the application of variable load increments. At the beginning of the test where the fiber/clay interface is expected to exhibit a linear response, relatively large load increments were adopted. Each increment was applied for a period of 30 minutes to ensure that the pullout was being conducted at a rate that would ensure full drainage (dissipation of excess pore water pressure) at the interface between the fiber and the soil. As the test progressed, relatively lower load increments were adopted as the applied loads approached the ultimate pullout resistance. The final decision regarding the applied load increments and the associated time period required to conduct the test was made following a trial and error procedure.

During the addition of water, special care was exercised to ensure that the distance between the water container and the base was enough to keep the container hanging freely and vertically throughout the test. For this purpose an “S” shaped hook was used to act as a connection between the rod that was wrapped around the pulley and the hanging water container.

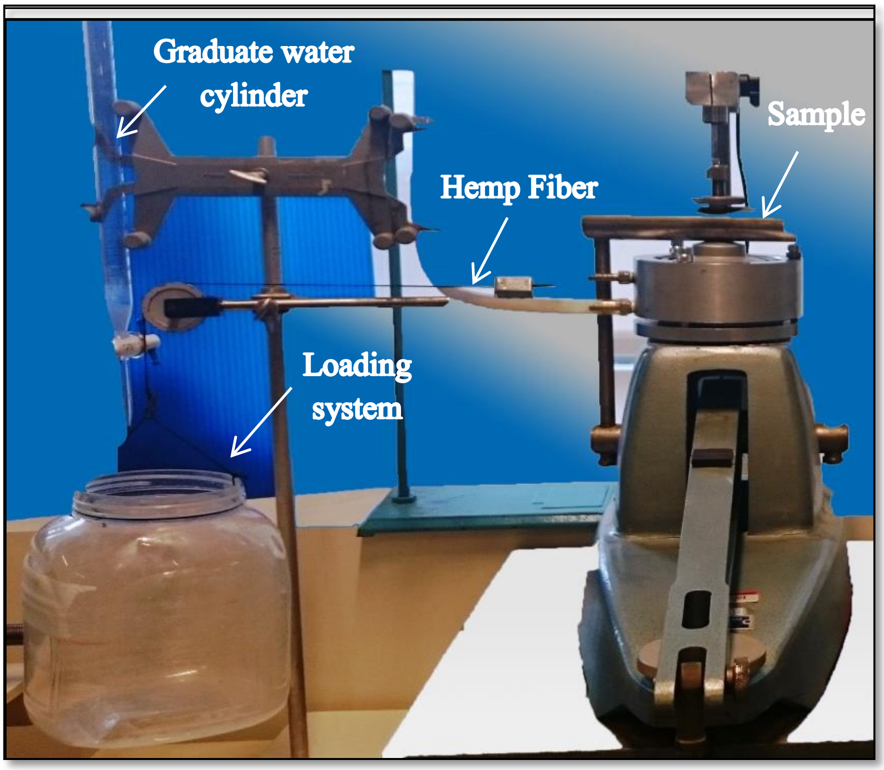


Figure 3.27 Pullout setup in Drained tests.

For all tests, the failure of the fiber was observed to occur by sudden slippage where the fiber pulls out under the weight of water. After pulling out the fiber the total weight of the water container was recorded. The fiber dimension after testing was measured. A comparison between the original and final width of the fiber indicated that the fiber width generally increases (where values ranges between 10 to 20% of its initial width) due to the normal stress applied on the fiber.

The ultimate pullout stress was calculated as the ratio between the force needed to pull out the fiber and the contact area with the compacted clay. The fiber has a rectangular cross section and is laid between two layers of clay as presented in figure 3.28. As a result four contact surfaces (two longitudinal and two transversal) exist between the clay and the fiber as shown in figure 3.29.

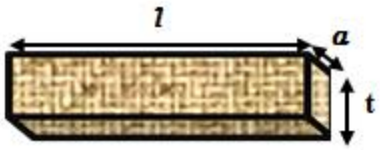


Figure 3.28 A sketch diagram showing fiber cross section.

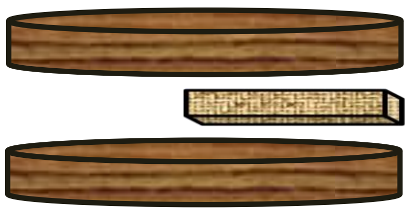


Figure 3.29 A sketch diagram showing fiber inserted between two layers of clay

The longitudinal contact surface area is equal to the product of the embedment length (l) and the average width (a), whereas the transversal contact surface area is equal to the product of the thickness (t) and the average width (a). The contact area A_c is the sum of transversal and longitudinal areas, and the total contact area is equal to two times Ac. The pullout force (F_{Po}) is equal to the product of the mass of water and gravitational acceleration.

The stress τ (kPa) needed to pullout the fiber is calculated according to the following equation

$$\tau = \frac{F_{po}}{2*(Ac)} \qquad (3)$$

Where,

- F_{po}=w*g in (N) ,where w is the weight of water (kg) and g is the gravity (N/kg.m)
- Ac=[(l*a)+(t*a)] in (m), where

  -l is the embedment length equals to3.75cm

  -a is the average width of the fiber measured after testing (mm)

  -t is the average fiber thickness, was found after a statistical distribution to be equal to 0.15mm.

After testing, the sample is removed in order to measure its water content and dry density as shown in figure 3.30.

Figure.3.30 Drained sample after fiber pullout

c. Unconsolidated-Undrained Pullout Tests

In the unconsolidated undrained tests, the samples were not allowed to consolidate under the applied normal stress prior to pulling the fiber. The weights corresponding to the different normal stresses were applied in the same manner as for the consolidated tests; however, water was not added to the mold and no consolidation was allowed. The applied normal stresses were calculated according to equation (2).

Following the application of the normal stress, the fiber was pulled out at a fast rate to ensure more-or-less undrained conditions at the interface between the single fiber and surrounding soil. The application of the pullout load was conducted using a continuous flow of water from a large water bottle with a tap that would allow for initiating and stopping water flow to the container. A drainage tube was attached to the tap and water flow was regulated such that the water container would be fully filled within a maximum time limit of 10minutes. The setup for undrained conditions is shown in figure 3.31

The initial water level in the supply water bottle was maintained constant for all tests to guarantee a constant pressure head. For all undrained tests, the failure of the fiber was observed to occur by sudden slippage where the fiber pulls out under the weight of water. After pulling out the fiber, the total weight of the water container was recorded and the fiber dimension after testing was measured.

Figure.3.31 Pullout setup in undrained conditions.

The pullout stresses were calculated according to equation (3). After testing the sample was removed in order to calculate its water content and dry density. Figure 3.32 shows an undrained sample after pulling out the fiber.

Figure.3.32Un-Drained sample after fiber pullout.

E. Summary

Clay properties were presented in this chapter including its grain size distribution, its classification and index properties. An overview of hemp characteristics and properties was also presented. This was followed by a detailed representation of the experimental testing program and a description of the sample preparation procedure and associated testing protocols. Finally, a thorough description of each type of test and its case-specific experimental challenges and data processing methodologies was included.

CHAPTER IV

TEST RESULTS
AND
DATA ANALYSIS

A. Introduction

In this chapter, the results of the experimental program are presented and analyzed with the objective of investigating the interface response between clay and hemp fibers using direct shear and single fiber pullout tests. A total of 36 direct shear tests which included clay/clay and clay/hemp tests were conducted in addition to 18 singe fiber pullout tests. The parameters that were varied in the two testing protocols were presented in Tables 3.4 a), 3.4 b) and 3.5 in the previous chapter.

This chapter is subdivided into four main sections. In the first and second sections, the results of the direct shear and pullout tests are presented, respectively. In the third section, a one-to-one comparison between the ultimate interface shear resistance resulting from identical direct shear and pullout interface tests was conducted to shed light on the effect of the test method on the ultimate interface strength characteristics and overall response. In the final section, interface coefficients and indicative values of interface efficiencies are derived from the results of the test program and compared with other values reported in the literature for different fibers and clay combinations.

In all the analyses that are conducted in this chapter, the parameters that are expected to influence the interface response are highlighted with special emphasis on the effects of the compaction water content and associated degree of saturation, initial dry density, applied normal stress, and drainage conditions and shearing rates.

B. Direct Shear Test Results

In the analysis of the direct shear test results, a set of symbols were consistently used throughout the chapter to designate the set of important parameters. In the adopted terminology, σ_n is the total applied normal stress (kPa), τ is the shear stress (kPa), w is the compaction water content (%), c and a are the total soil cohesion and the total interface adhesion, respectively and ϕ and δ are the total soil internal friction angle and total interface friction angle. A prime [’] is added to the mentioned parameters to differentiate between total and effective values.

The results of the tests were analyzed within three main contexts. First, the shear stress versus horizontal displacement response for both clay/clay and clay/hemp cases was analyzed with particular emphasis on the effects of drained and undrained loading and the compaction water contents adopted. Second, the maximum shear stresses that were measured in the different tests were used to determine the Mohr Coulomb failure envelopes for the clay and the clay/hemp interface. The shear strength parameters for the clay and the interface were then compared and contrasted for the different test conditions adopted.

1. Shear Stress versus Horizontal Displacement Response

The variation of the shear stress (kPa) with horizontal displacement (mm) is exhibited in Fig. 4.1 for the 36 direct shear tests that were conducted in this study. Results pertaining to clay/clay and clay/hemp tests for any given compaction water content are presented on the same figure for comparison. Results pertaining to the slow shearing conditions (drained tests) are presented in Fig. 4.1(a) while results pertaining to the fast shearing rates (undrained tests) are presented in Fig. 4.1(b).

a. General Response

Results pertaining to the drained tests on Fig. 4.1(a) indicate that for both clay/clay and clay/hemp tests, the shear stress versus horizontal displacement response are sensitive to the applied effective normal stress. This is expected for drained loading conditions where higher shear stresses are expected to be mobilized in the clay and at the interface for higher levels of confinement. No peaks were exhibited in the stress-displacement response despite the fact that the curves were observed to level out at the maximum applied horizontal displacement of 6mm to 7mm. This observation is valid irrespective of the effective normal stress and the compaction water content used.

On the other hand, results from clay/hemp interface tests indicate clear peaks in the shear stress at relatively small magnitudes of horizontal displacement. The post peak response of the interface was governed by softening, particularly for tests conducted at relatively large effective normal stresses (σ'_n =100kPa and 200kPa) irrespective of the water content. The peak interface shear stresses were mobilized at horizontal displacements in the range of 0.5mm to 1.5mm.

It should be noted that for all the drained direct shear tests, the peak interface stress is found to be smaller than the maximum associated clay/clay shear stress, indicating an interface efficiency that is smaller than 1.0, irrespective of the applied normal stress.

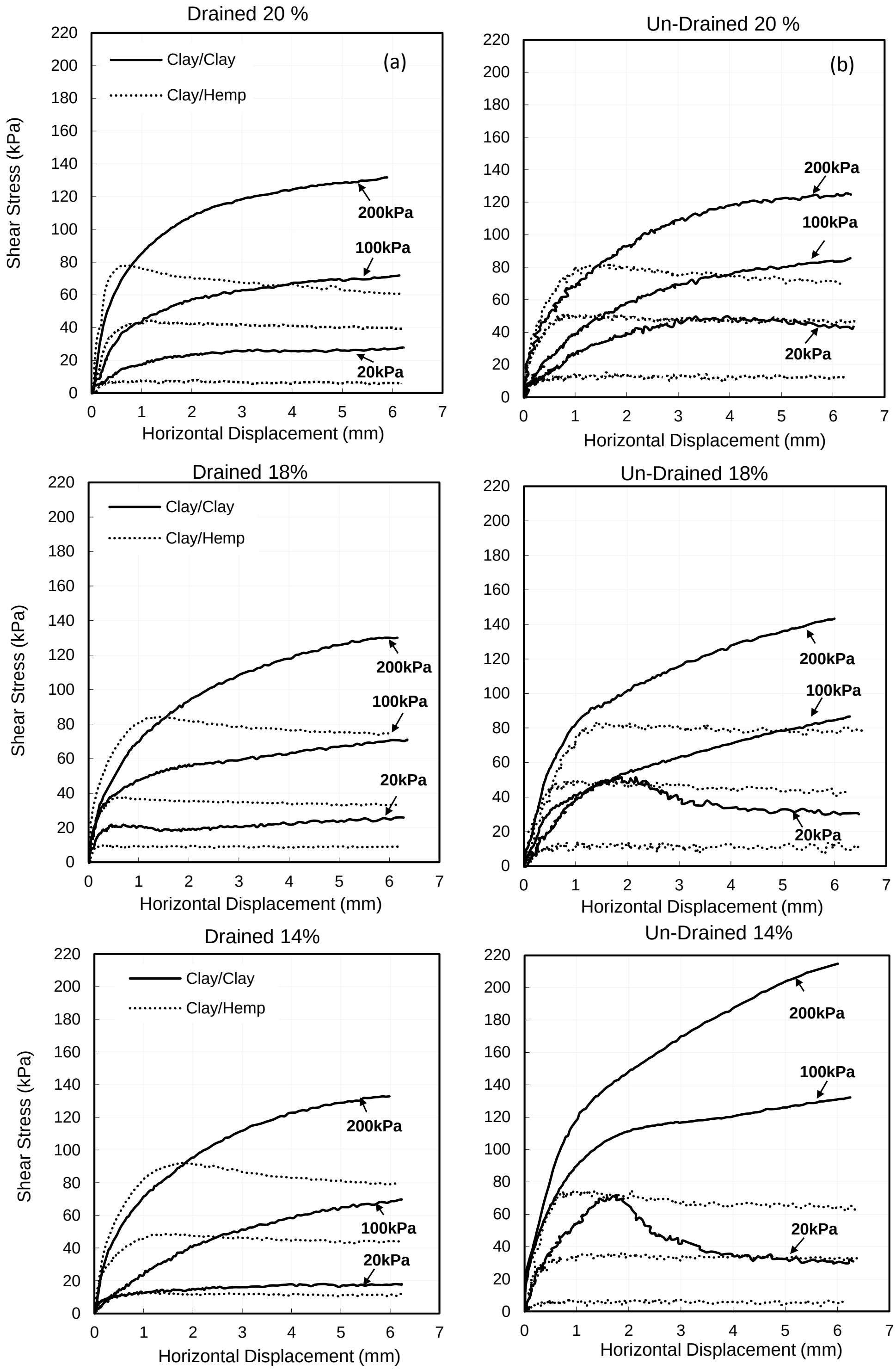


Figure 4.1 Variation of the shear stress with the horizontal displacement for (a) Drained and (b) Un-Drained Clay/Clay and Clay /Hemp tests under different water contents.

For the tests conducted using a fast shearing rate, the test specimens were not allowed to consolidate prior to shearing to model the “unconsolidated” response of the clay and the clay/hemp interface. The tests were aimed at mimicking more or less undrained conditions despite the fact that the specimens are free to drain in any direction during the test. Interestingly, results of the tests on Figure 4.1(b) indicate a relatively high and unexpected sensitivity of the load response to the applied total normal stress, despite the fact that the tests were designed to be unconsolidated and undrained. The sensitivity of the shear stress versus horizontal displacement response to the total normal stress diminishes as the compaction water content increases since the degree of saturation increases with increasing compaction water content, rendering the specimens less sensitive to the applied normal stress in a UU setting. This sensitivity of the response to the total normal stress is amplified by the fact that the clay includes more than 45% sand in its composition. For relatively unsaturated compacted clays, this percentage of sand will contribute to increasing the total stress friction angle of the clay.

With regards to the shape of the stress-displacement response, results of the fast clay/clay tests indicate that there are no peaks in the shear stress except for tests conducted at σ_n =20kPa at all water contents. At this low level of confinement, clear peaks in the measured shear stress are observed at horizontal displacements ranging from1.5mm for specimens compacted dry of optimum at w=14% and 3.5mm for specimens compacted wet of optimum at w=20%. The brittle, softening response that is exhibited in the unconsolidated undrained tests that were conducted at 20 kPa could be attributed to dilation of the unsaturated clay against the relatively low applied normal stress during shearing.

In the quick clay/hemp interface tests, the overall observed load response was found to be very similar to the response observed in the associated drained interface tests for all σ_n and all water contents. This observation is interesting since it indicates that the causes that led to differences in the response of the clay/clay specimens for different test conditions (drained versus undrained) did not have a similar impact on the response of the interface. This observation points to possible partial drainage that may have occurred at the interface in the quick tests. This issue will be investigated further in the following section.

b. Comparison Between Drained and Undrained Behavior

To allow for direct comparison between the results of the undrained and drained tests, the variation of the shear stress with the horizontal displacement for both testing conditions was compared in Fig. 4.2(a) (clay/clay) and 4.2(b) (clay/hemp), respectively. For clay/clay tests, the maximum observed shear stress for any given water content and applied normal stress is found to be larger for undrained tests compared to drained tests. Moreover, the difference between the drained and undrained responses seems to increase as the compaction water content decreases (from 20% to 18% to 14%) and as the applied normal stress decreases (from 200 kPa to 100 kPa to 20 kPa). To quantify the difference between drained and undrained loading conditions, a factor λ is defined as the ratio of the undrained maximum shear stress to the drained maximum shear stress at a given water content and normal stress. A quick analysis of the calculated λ factors indicates values that are approximately equal to 1 for w= 20% and σ_n =200kPa increasing to values as high as $\lambda \approx 4$ for w= 14% and σ_n=20kPa.

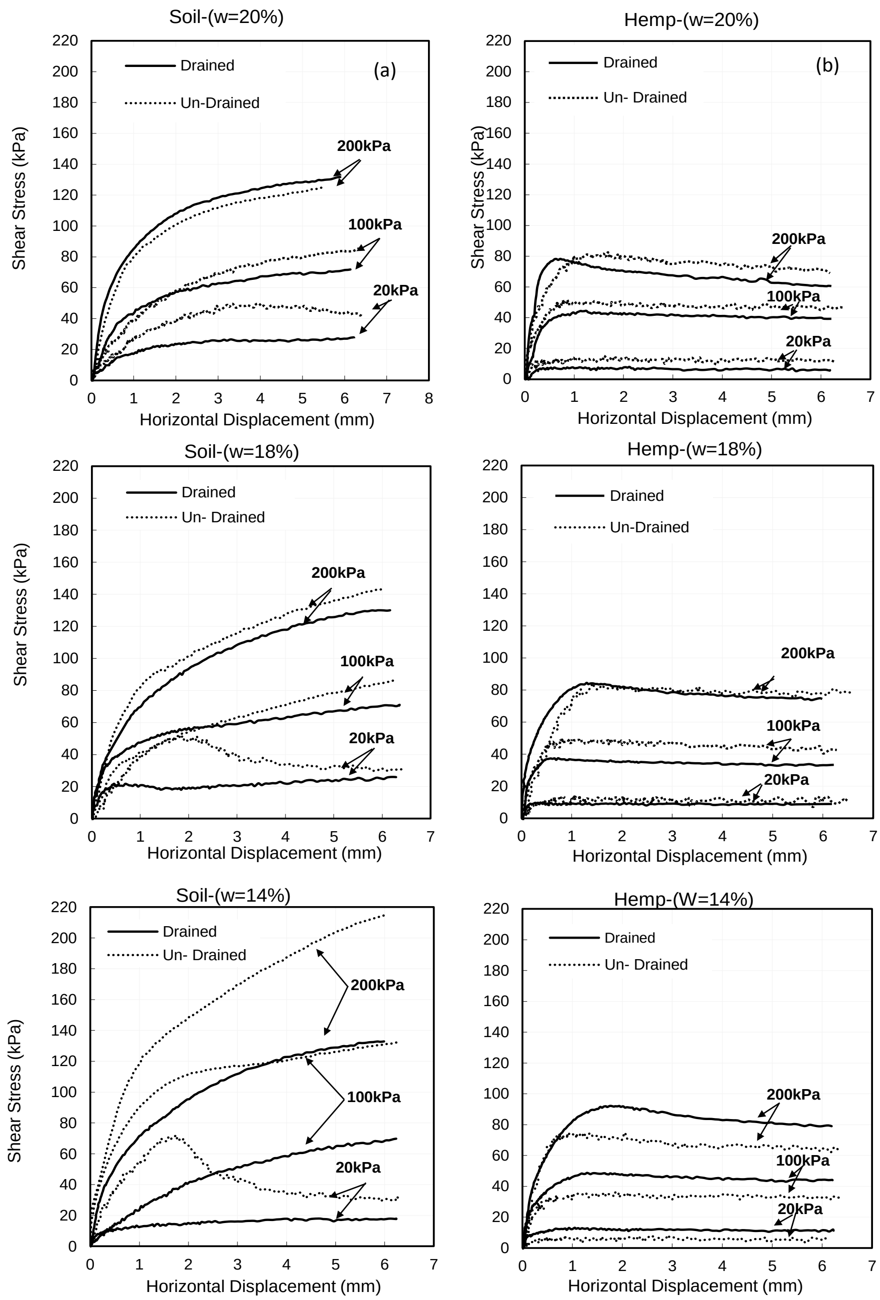


Figure 4.2 Variation of the shear stress with the horizontal displacement for (a) Clay/Clay (b) Clay /Hemp, with different water contents tested under Drained and Un-Drained conditions.

The differences in the observed response could be attributed to several factors. In the drained tests, the fact that the clay specimens are allowed to consolidate (increase in dry unit weight) is expected to have a positive impact on the response during drained shear. On the other hand, the uninhibited drainage and the slow shearing in drained tests do not allow for any buildup of negative pore pressures that could increase the effective stress and enhance the load response. This is expected to have a negative impact on the maximum stress carried by the clay during drained shear. For undrained tests on clay/clay specimens, clays are not given enough time to consolidate under the applied normal stress. As a result, the dry unit weights after the application of the normal stress and prior to shearing are expected be smaller than their consolidated counterparts. However, during shearing under quick rates, the relatively high matric suction values coupled with the likelihood of generating negative pore water pressures during shear are expected to result in a more effective response compared to drained tests. The positive effects of "undrained" shearing are expected to be more prominent for cases involving relatively unsaturated clays (w=14%) and relatively low normal stress levels (20 kPa). Similar observations were reported in Miller et al. (2006) and Khoury et al. (2010).

For the clay/hemp interface tests, the difference in the response between drained and undrained tests is minimal irrespective of the applied normal stress and the compaction water content (see Fig. 4.2(b)). A thorough analysis of the shear stress versus horizontal displacement relationships in Fig. 4.2(b) indicates that for the cases involving water contents of 18% and 20%, the curves corresponding to the "undrained response" exhibit a slightly improved interface resistance. However, for cases where the soil was compacted dry of optimum at w=14%, the drained response is consistently larger than the undrained response, despite the fact that the undrained clay/clay strength was much larger than the drained strength for this compaction water content.

The small difference between the “drained” and “undrained” load response of the interface tests could be attributed to possible drainage that might have occurred at the contact between the clay and the hemp surface at the micro level. Such drainage may be inevitable within an interface direct shear setup whereby the clay is sheared on a hemp surface that was prepared by gluing individual hemp fibers to a steel plate. Moreover, the hemp fibers themselves could act like a drainage conduit that could facilitate drainage of water from the thin clay surface that is in contact with the fibers. Drainage at interfaces between soils and solid interfaces was discussed by Miller et al. (2006), who detected a decrease of water content in the clay during interface shearing due to the disruption of the menisci between soil particles causing a decrease in matric suction which resulted in water draining out of the sample.

It could be argued that even if the drainage at the interface is minimal, it could be enough to reduce the positive effects of matric suction and negative pore pressures on the interface strength in fast tests. Any partial drainage at the interface could prohibit the interface material (hemp in this case) from mobilizing the full undrained shear strength of the clay that is in contact with it. The fact that the interface resistance for the cases of w=18% and 20% was still found to be slightly higher in the undrained tests reflects that partial drainage (rather than full drainage) may have dominated the interface response in the fast tests.

For the interface tests that were conducted with a water content of 14%, the interface strength in the drained tests was found to be larger than the interface strength measured in the tests sheared at a fast rate without allowing consolidation. These results are surprising given the fact that the clay/clay tests for w=14% indicated a much higher strength for the clay tested at a fast shear rate. These results can be explained by the following hypothesis. It could be argued that the clay that was compacted dry of optimum at w = 14% is expected to have a relatively high permeability and a relatively low dry unit weight. In fact, the coefficient of consolidation as obtained from the consolidation phase of the direct shear tests was determined to be 1.5 times larger for the clays compacted at 14% (compared to the clays compacted at 18% and 20%). This relatively higher permeability would amplify the partial drainage that exists at the interface in the fast tests, allowing the interface strength in the fast tests to approach that of the drained tests, and prohibiting the hemp interface from mobilizing the "undrained" strength of the clay that is in contact with it.

The fact that the interface strength in the undrained tests was lower than the drained tests could further be explained by the relatively small initial dry density (14.8 KN/m^3) for the w=14% samples. For the consolidated drained tests, increases in the initial dry density will result from allowing consolidation under the applied normal stresses, whereas in the unconsolidated undrained tests, the dry density is expected to remain relatively small even with the application of the normal stress. This could explain the superior drained response for w = 14%. For the tests conducted at the compaction water contents that are close to optimum (18% and 20%), the initial dry densities of the clay specimens are relatively large (values ranges between 16.5 KN/m^3 and 17 KN/m3) and the permeability is relatively low. As a result, allowing consolidation during the application of normal stress is not expected to have a significant impact on the interface response. In addition, the degree of partial drainage at the interface is expected to be smaller than the case involving w = 14%.

c. Effect of Water Content

To further investigate the effect of the water content on the clay/clay and clay/hemp responses, the stress-displacement curves for the clay/clay and clay/hemp tests were plotted on the same figure for comparison (Fig. 4.3). Each plot in Fig. 4.3 represents a constant value of applied normal stress.

For consolidated drained clay/clay tests that are conducted at σ_n =200kPa, there is no difference in response for w=14%, 18% and 20%. For the smaller normal stresses (20 kPa and 100 kPa), the response for tests pertaining to w=14% show divergence from the other water contents. This divergence is clearly visible for the lowest normal stress of σ_n =20kPa, where the maximum shear stress for w=14% was found to be the smallest followed by w=18% followed by w=20%. The more-or-less identical drained response of the w=14%, 18% and 20% specimens at the highest normal stress of σ_n=200kPa indicate that allowing the specimens to consolidate during the application of the normal stress, coupled with the slow drained shearing mechanism, may have allowed the response of the different specimens to converge despite the differences in the initial structure and dry density. As σ_n decreases to 100kPa and 20kPa, the ability of σ_n to normalize the behavior becomes less especially for w=14% which has initially a low dry density (high void ratio) and a flocculated structure. The case of σ_n =20kPa exhibits the scenario with the largest differences in clay/clay response at different water contents.

For the clay/hemp interface tests, it was observed that specimens that were compacted at w=14% consistently yielded the highest drained interface response. Relatively higher drained interface resistances were observed in the w=14% specimens despite the relatively lower initial clay dry densities. The reduction in the interface resistance with increases in compaction water content could be attributed to the lubricating effect of water which could negatively affect the efficiency of the interface at higher water contents. These results are supported by the results presented in Attom et al. (2007) who observed that the interface resistance decreases when the water content of the soil increases due to the effect of water lubrication, resulting in a reduction of the load transfer between clay particles and fibers at the level of interface.

For the clay/clay tests that were conducted at a fast shearing rate without allowing consolidation under the applied normal stresses, results pertaining to the case of w=14%, resulted in consistently higher stresses compared to clays compacted at w=18% and 20%, irrespective of the applied normal stress. For w=18% and 20%, similar stress-displacement responses were observed. The superior stress-displacement response witnessed for the w=14% specimens could be attributed to the generation of negative pore pressure and the effect of matric suction in the unsaturated specimens.

For clay/hemp tests, results from “undrained” interface tests that involved clays compacted at w=14% consistently gave smaller interface strengths than clays compacted at other water contents, irrespective of the applied normal stress. These results could be explained by the fact that clays compacted at w=14% exhibit relatively high permeability values (compared to w=18% and w=20%). As a result, the interface resistance may not be benefitting from the negative pore pressure generation or matric suction effects. High permeability will enhance the phenomenon of partial drainage that may be occurring at the interface leading to lower interface efficiencies for undrained tests.

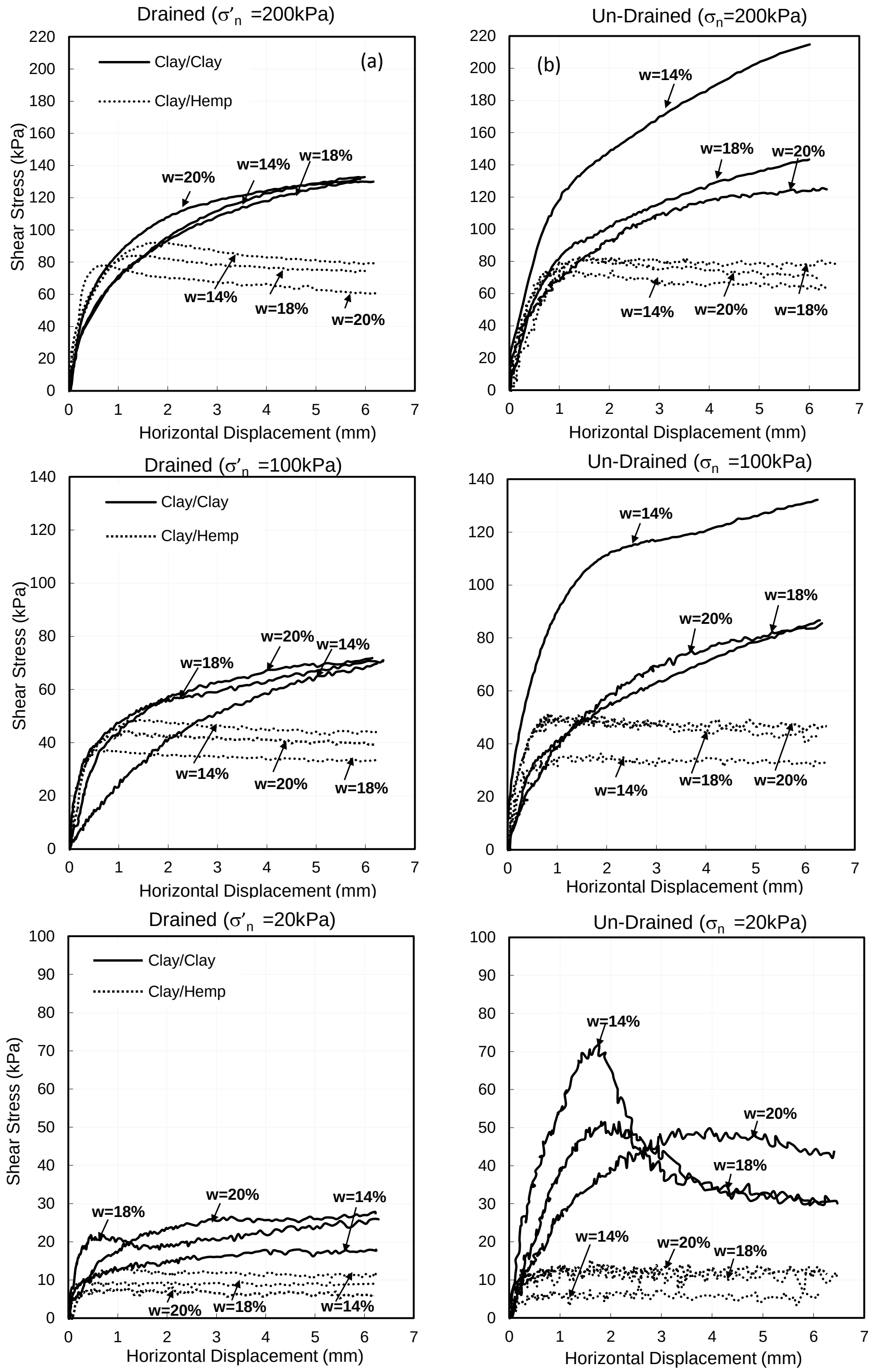


Figure.4. 3Variation of the shear stress with the horizontal displacement for (a) Drained and (b) Un-Drained Clay/Clay and Clay /Hemp tests under different normal stresses.

2. Maximum Shear Stresses and Associated Mohr Coulomb Failure Envelopes

The ultimate shear stresses observed for all direct shear tests (clay/clay and clay/hemp) that were conducted in this study are presented in Figure 4.4 as a function of the compaction water content for both drained and undrained loading conditions. The following major findings can be deduced from Figure 4.4:

- The ultimate clay/clay shear stresses observed in the quick tests are higher than their drained counterparts. This is attributed to the effect of matric suction and/or negative pore water pressure.
- The ultimate clay/clay shear stresses for the consolidated drained tests are not sensitive to the compaction water content. On the other hand, the ultimate shear stresses observed in the quick tests decreased significantly as the compaction water content increased from 14% to 18% and 20%.
- The maximum shear stresses that were measured in the clay/hemp interface tests do not seem to be significantly affected by the type of test (drained versus undrained). This indicates that possible partial drainage could have occurred at the interface between the clay and the fiber-coated interface even under quick loading conditions.
- Finally, the maximum interface shear stresses show minimal sensitivity to the compaction water content and the rate of shearing. The maximum interface stresses generally decrease with water content for the drained tests and increase with water content for the undrained tests. For the drained tests, the slightly reduced interface resistance at higher water content could be attributed to

increased lubrication at the interface. For the “undrained tests”, the slightly reduced interface resistance at lower water contents could be related to the lower initial dry density of the clay coupled with a relatively higher permeability which would enhance partial drainage at the interface and reduce the positive impact of matric suction and negative pore pressure on the interface resistance.

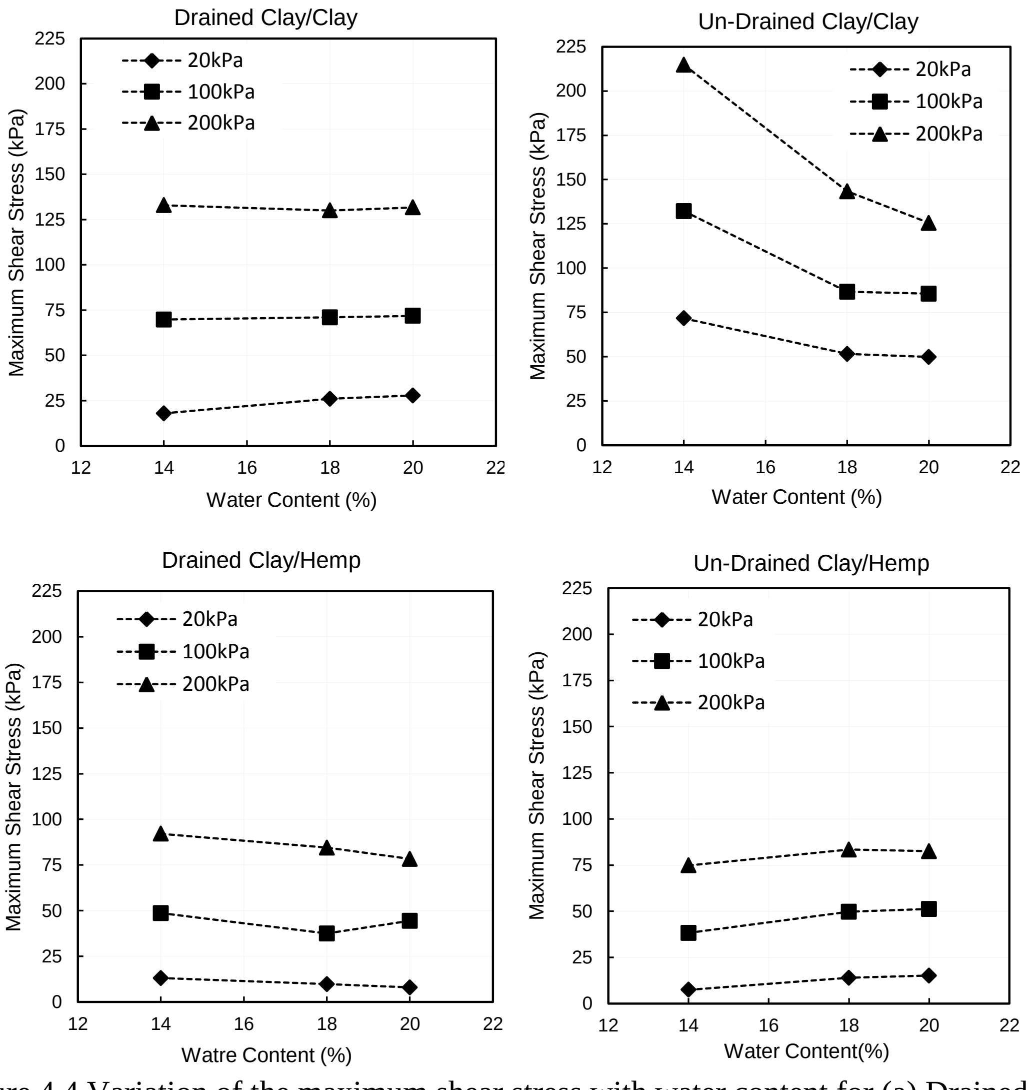


Figure 4.4 Variation of the maximum shear stress with water content for (a) Drained and (b) Un-Drained Clay/Clay and Clay /Hemp tests.

The maximum shear stresses presented in Fig. 4.4 were used to construct Mohr-Coulomb failure envelopes for the clay/clay and clay/hemp specimens using regression. The resulting shear strength parameters which include the cohesion intercept (c for clay/clay and a for hemp/clay) and the friction angle (ϕ for clay/clay and δ for hemp/clay) were determined form the resulting envelops. To allow for a one-to-one comparison between the clay/clay and clay/hemp envelops, the resulting Mohr-Coulomb failure envelops for the clay and the clay/hemp interface were plotted on Fig. 4.5 for cases involving different compaction water contents and test conditions. The same Mohr-Coulomb envelops were re-drawn on Fig. 4.6 to isolate the effect of the test conditions on the test results. The associated shear strength parameters are presented in table 4.1 and Table 4.2, for the clay/clay and clay/hemp tests, respectively.

A general graphical investigation of the shear strength envelops indicate that the envelopes of the interface tests were always lower than the envelopes of the clay/clay tests, indicating that the efficiency of the hemp fibers is less than unity, irrespective of the test conditions. The difference between the clay/clay and clay/hemp envelopes was more pronounced for the undrained cases, where the shear strength envelopes of the clay exhibited cohesive intercepts that were relatively large (38.6 kPa to 54.5 kPa). For the clay/clay case, a comparison between the drained and undrained envelops (Fig. 4.6) indicates that the undrained envelopes are predominately higher than their drained counterparts with the difference increasing as the compaction water content decreases. On the other hand, interface envelopes for drained and undrained tests were found to be very close to each other.

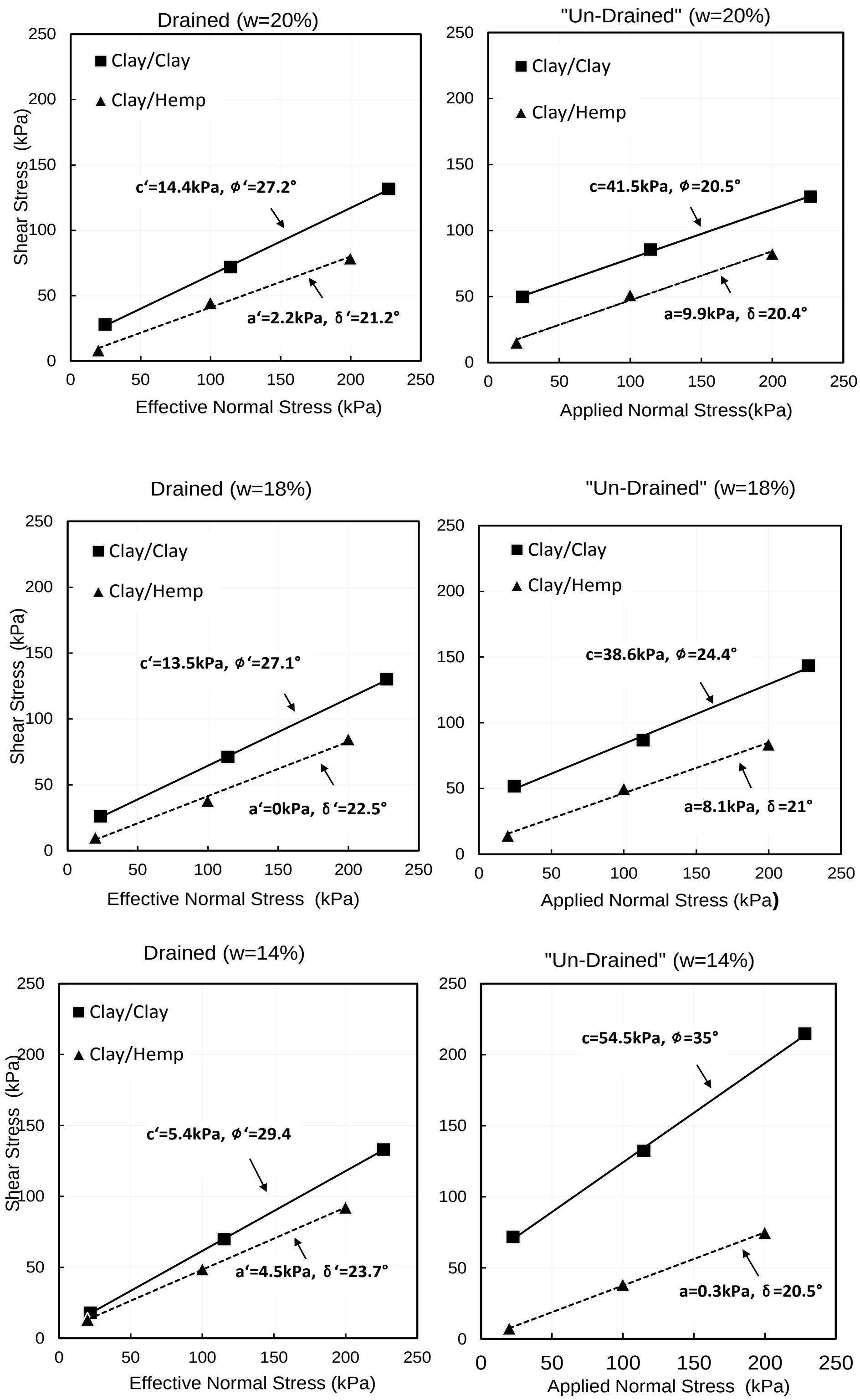


Figure 4.5Mohr- Coulomb Failure Envelopes. (a) Drained and (b) Un-Drained Clay/Clay and Clay /Hemp.

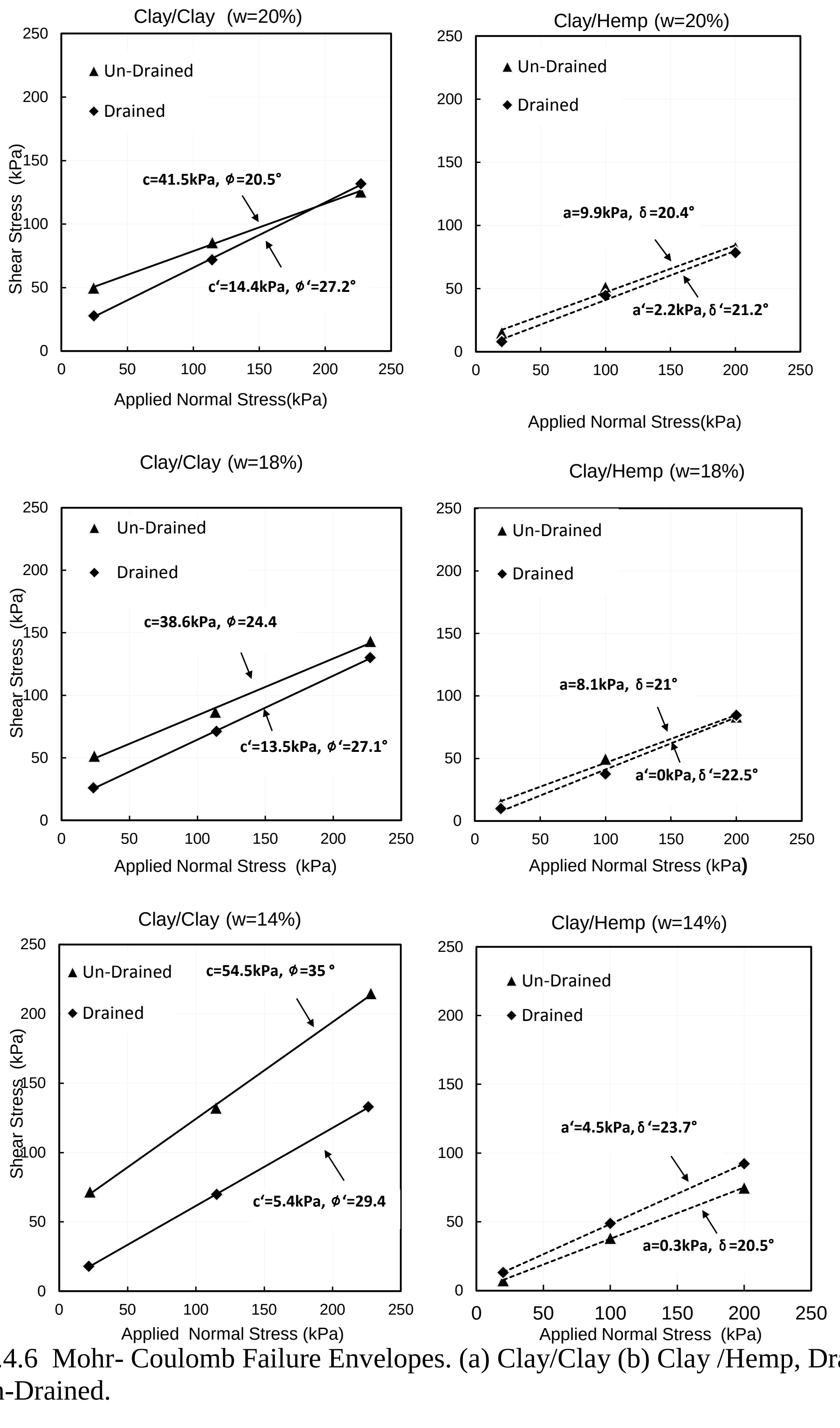


Figure.4.6 Mohr- Coulomb Failure Envelopes. (a) Clay/Clay (b) Clay /Hemp, Drained and Un-Drained.

The shear strength parameters for the clay tests are summarized in Table 4.1. The results indicate similar drained shear strength parameters c' and ϕ' for the tests conducted with a water content of 18% and 20%, with a c' of about 14 kPa and ϕ' of 27 degrees. These results may be considered to be realistic for low plasticity clay specimens that are compacted close to optimum. The relatively high drained friction angle could be attributed to the presence of more than 45% sand in the clay matrix. For clays that were compacted dry of optimum at a lower water content of 14%, a slightly lower c' value of 5.4 kPa and a slightly higher ϕ' of 29.4 degrees were obtained.

In the undrained tests and as expected, the total stress cohesive intercept c was relatively large (38.6 kPa to 54.5 kPa) compared to the drained tests. In addition, the total stress friction angles ϕ for the w=18% (ϕ = 24.4 degrees) and w=20% (ϕ = 20.5 degrees) cases were found to be slightly smaller than their drained counterparts. The difference between the total stress friction angles (ϕ = 24.4 versus 20.5 degrees) is related to the degree of saturation of the clay which is expected to be larger in the w = 20% tests. For the case involving a water content of 14%, the total stress Mohr-Coulomb envelope was very steep with a recorded friction angle of about 35 degrees. This excessively high friction angle could be attributed to the relatively low degree of saturation and the associated effect of matric suction coupled with the presence of a significant proportion of sand in the clay matrix. It is worth noting that for the cases involving w=18% and w=20%, ϕ' was found to be greater than ϕ and c' was found to be less than c. The opposite is true for the case involving a for w of 14% where c and ϕ were found to be greater than c' and ϕ'.

Table 4.1 Soil Cohesion and Internal Friction Angles for Clay/Clay tests (a) Drained, (b) Un-Drained

Clay/Clay	Drained		Un-Drained	
Water Content (%)	C′ (kPa)	ϕ' (°)	C (kPa)	ϕ (°)
20%	14.4	27.2	41.5	20.5
18%	13.5	27.1	38.6	24.4
14%	5.4	29.4	54.5	35

The interface shear strength parameters for the clay/hemp tests are summarized in Table 4.2. Results of the drained tests indicate that the drained adhesion intercept *a'* is relatively small and ranges from 0 to 4.5 kPa. The drained interface friction angle δ' varies in a relatively narrow range of 21.2 to 23.7 degrees, with the smaller values of δ' being associated with the higher water contents possibly due to the lubricating effect of water. For the unconsolidated quick interface tests, slightly higher adhesion values were obtained for the cases where the clay was compacted near optimum (w=18% and w=20%) with a total stress adhesion *a* of about 9 kPa. This adhesion was associated with total stress interface friction angles δ of 20.4˚ and 21˚, which are only slightly smaller than the associated range of δ' (21.2 and 22.5 degrees). For the case involving w=14%, the adhesion intercept *a* was zero and the associated interface friction angle was equal to 20.5˚. As mentioned previously, the possible partial drainage at the interface between the clay and the hemp interface in the undrained tests could explain the relatively similar interface shear strength envelopes of the drained and undrained tests.

Table 4.2 Soil Adhesion and Interface Friction Angles for Clay/Hemp tests (a) Drained, (b) Un-Drained

Clay/Hemp	Drained		Un-Drained	
Water Content (%)	a' (kPa)	δ' (°)	a(kPa)	δ(°)
20%	2.2	21.2	9.9	20.4
18%	0	22.5	8.1	21
14%	4.5	23.7	0.3	20.5

Interface parameters in literature varied according to the type of interface where soil adhesion in consolidated drained tests with different geomembranes varied between (2kPa to 18 kPa) and interface friction angle varied between (15˚ to 40˚). Un-drained interface parameters varied between a=3.4kPa to a=21.6kPa and δ varied between 2.2˚ to 40.1˚.

Our values belong to the interval of interface parameters found in literature taking into consideration that we are introducing a new natural material to the geotextiles and geomembrane family.

C. Pullout Test Results

As described in the previous chapter a custom fabricated apparatus was used to conduct horizontal pullout tests on single hemp fiber that was sandwiched between two layers of clay. The pullout force was calculated according to equation (2) and used to estimate the maximum pullout shear stress as indicated in equation (3). Slow and quick shearing conditions were adopted for clays compacted at water contents of 14%, 18% and 20% and subjected to three normal stresses (20kPa, 100kPa and 200kPa). The effect of water content, applied normal stress and the rate of shearing on the maximum pullout stress of single fibers was investigated and analyzed in the following paragraphs.

1. Effect of water content and test type on ultimate pullout stress

The ultimate pullout stresses that were measured in the slow and quick fiber pullout tests are presented in Figure 4.7(a) and 4.7(b), respectively. In tests where the fiber was pulled out slowly (drained conditions), the ultimate pullout stress was found to increase with the applied effective normal stress but was insensitive to the initial compaction water content of the clay. For tests where the fiber was pulled out at a fast rate, the magnitude of the measured ultimate pullout stress was larger than the corresponding drained condition. These results indicate that, unlike the case of the interface direct shear tests where partial drainage could have occurred at the interface, fast pullout tests may have exhibited more-or-less “undrained” behavior at the interface between the hemp fiber and the compacted clay. This hypothesis is reinforced by the observed relationship between the measured pullout stresses and the compaction water as exhibited in Figure 4.7(b). Results on Figure 4.7(b) indicate that the maximum pullout stress is highly dependent on the compaction water content particularly for the case of w=14%. A significant reduction in the ultimate pullout stress is clearly indicated in Figure 4.7b as the water content is increased from 14% to 18% and 20%. In fact, a comparison between the results of the fast fiber pullout tests (Fig. 4.7(b)) and the fast direct shear clay tests (Fig. 4.4(b)) show remarkable consistency with regards to the variation of the maximum shear stresses and ultimate pullout stresses with the compaction water content. This observation further validates the hypothesis of a true “undrained” behavior in the single fiber pullout tests.

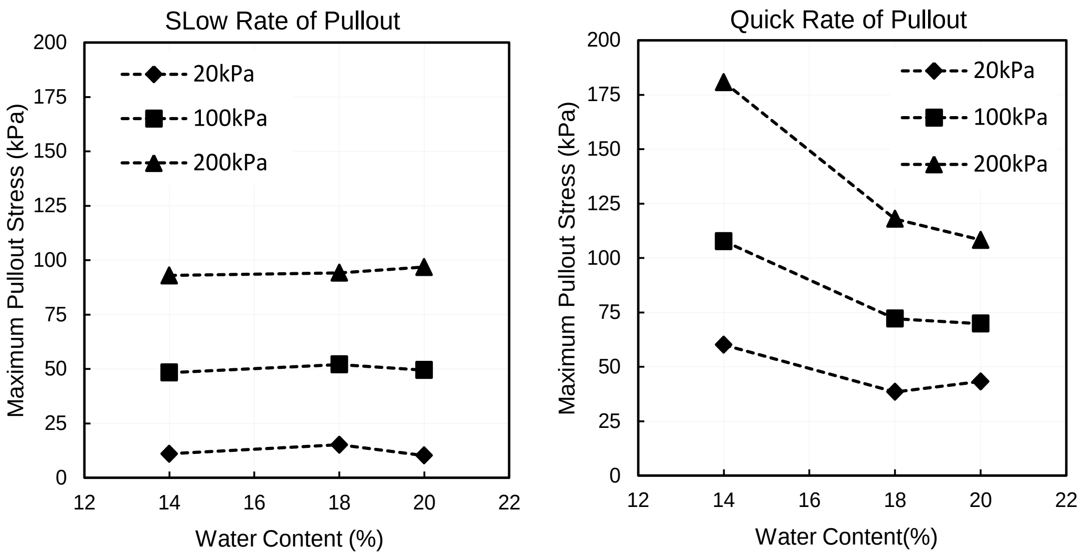


Figure.4.7 Maximum Pullout Stress versus Water Content at different applied normal stress for (a) consolidated drained tests, and (b) unconsolidated quick tests.

2. Mohr Coulomb Failure Envelopes for Pullout

Mohr Coulomb failure envelopes were established from the measured maximum pullout stresses and plotted on Figure 4.8(a) and 4.8(b) for slow and quick loading conditions, respectively. The interface shear strength parameters (*a* and δ) resulting from the pullout tests are summarized in table 4.3.

Results for the slow pullout tests where the clay at the interface is expected to be drained, the interface envelops for the three water contents are almost identical, with effective adhesion values *a′* ranging from 1kPa and 7kPa, and effective interface friction angles δ′ that range between 23.7˚ and 25.6˚. For fast rate pullout tests, the total stress adhesion *a* was relatively large (35 kPa to 44 kPa) compared to the drained pullout tests. In addition, the total stress interface friction angles δ for the w=18% (δ = 24 degrees) and w=20% (δ = 20 degrees) cases were found to be slightly smaller or equal to their drained counterparts. For the case involving water content of 14%, the total stress pullout envelop was very steep with a recorded interface friction angle of about 34 degrees. Moreover, the associated *a* = 44.4 kPa could also be considered to be relatively high. The excessively high interface friction angle and adhesion could only be put into context when compared to the equally high friction angle and cohesion witnessed in the undrained clay/clay direct shear tests. It could be argued that the hemp fibers in the fiber pullout tests were efficient at mobilizing a significant portion of the undrained shear strength of the surrounding clay. Similar results were reported in Tang et al. (2009) and Jamie et al. (2013) who conducted undrained pullout tests on a single fiber. It should be noted that the interface pullout strength parameters are in line with values reported in Hatami and Esmaili (2015).

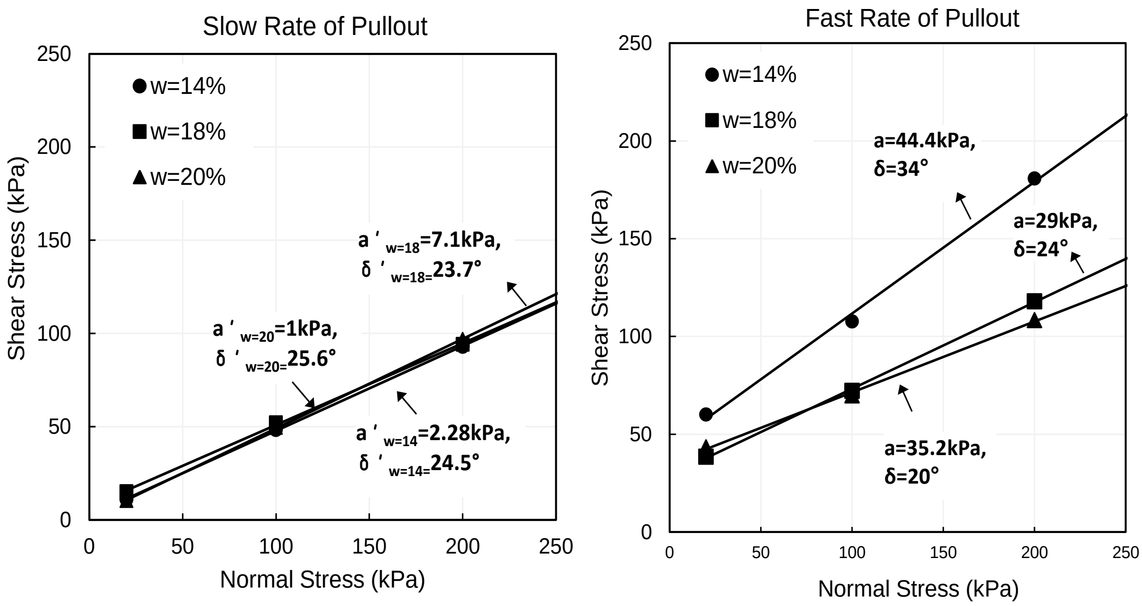


Figure.4.8 Mohr Coulomb failure envelops for (a) Slow rate pullout tests, (b) Fast rate pullout test.

Table 4.3 Soil Adhesion and Pullout Friction Angles for slow rate pullout tests and quick rate pullout test.

Pullout	Slow Rate		Quick Rate	
Water Content (%)	a′ (kPa)	δ′ (°)	a(kPa)	δ(°)
20%	1	25.6	35.2	20
18%	7.1	23.7	29	24
14%	2.28	24.5	44.4	34

D. Comparison between Direct Shear Results and Pullout Results

In this section, a one-to-one comparison is made between the results of the direct shear tests and those of the single fiber pullout tests to shed light on any major differences in the measured interface resistance from the two tests.

For that purpose, the maximum interface shear stresses that were measured in the direct shear tests were plotted with the ultimate single fiber pullout stresses on Figure 4.9 for both drained and undrained loading conditions. Plotted on the same figures are the maximum shear stresses that were measured in the associated clay/clay direct shear tests. The clay/clay stresses were added to the plots to act as a reference, since the clay/clay strength is estimated to represent the upper bound of any interface shear or pullout resistance.

Results on Figure 4.9 indicate that for slow drained tests, the pullout and direct shear stresses at failure are relatively close to each other, with identical values observed for the case with a water content of 14%. For w = 18% and 20%, the maximum interface pullout stresses were slightly and consistently larger than their direct shear counterparts. On the other hand, results from unconsolidated fast tests indicate a superior interface response in the single fiber pullout tests, with maximum pullout stresses that are significantly larger than the direct shear stresses. In fact, the maximum pullout stresses in fast tests approached the undrained maximum clay/clay stress for all water contents used.

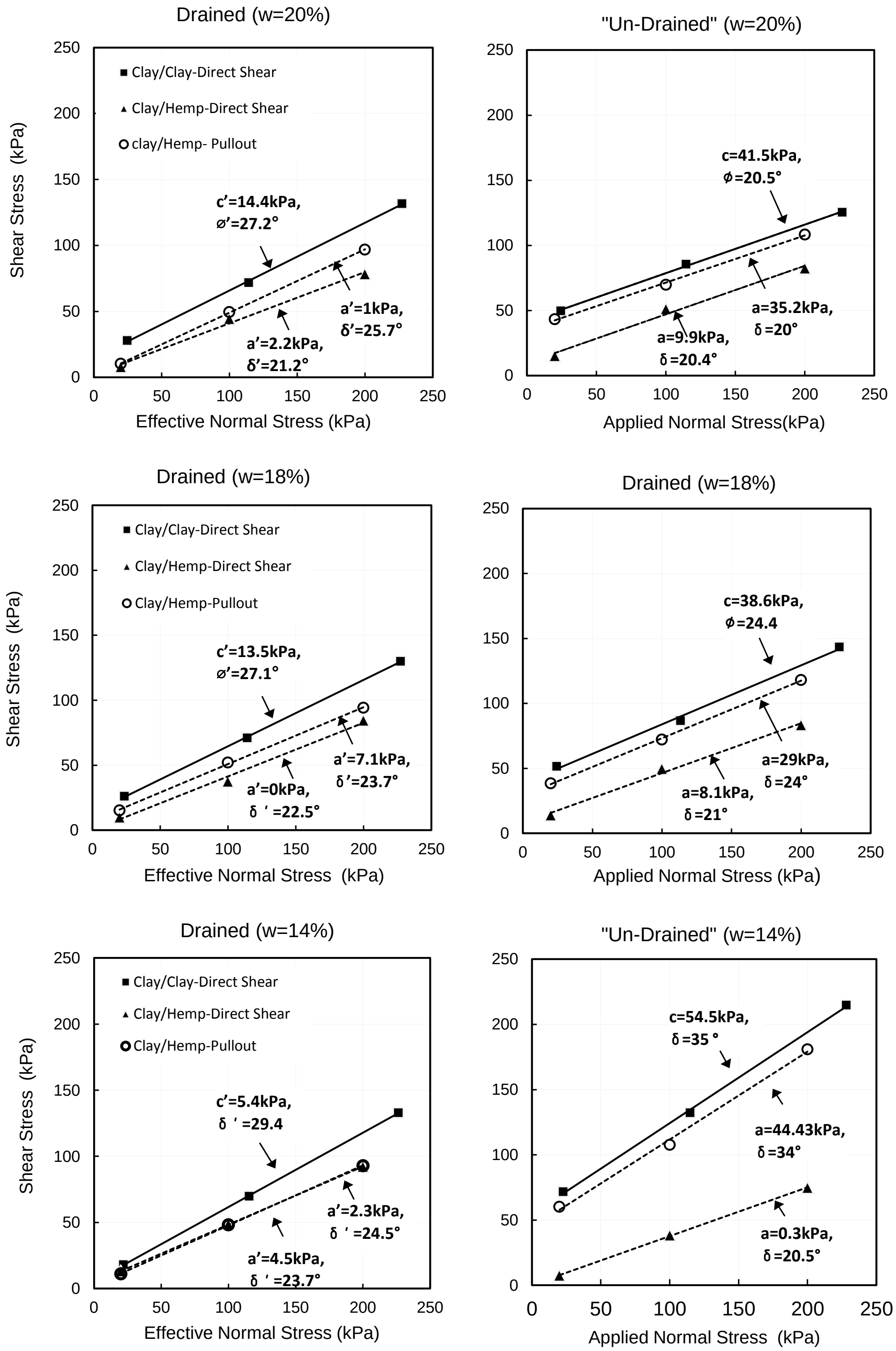


Figure.4.9 .Comparison between Results of Direct Shear and Single Fiber Pullout Tests for (a) Consolidated Drained Slow Tests and (b) Unconsolidated Quick Tests

To further understand the mechanisms governing the interface strength for pullout and direct shear tests conducted under different consolidation states and shearing rates, a parameter "λ" was defined as the ratio of the maximum undrained interface shear stress to the maximum drained interface shear strength. This ratio was calculated for direct shear tests and single fiber pullout tests and compared on Figures 4.10(a) (direct shear) and 4.10(b) (pullout).

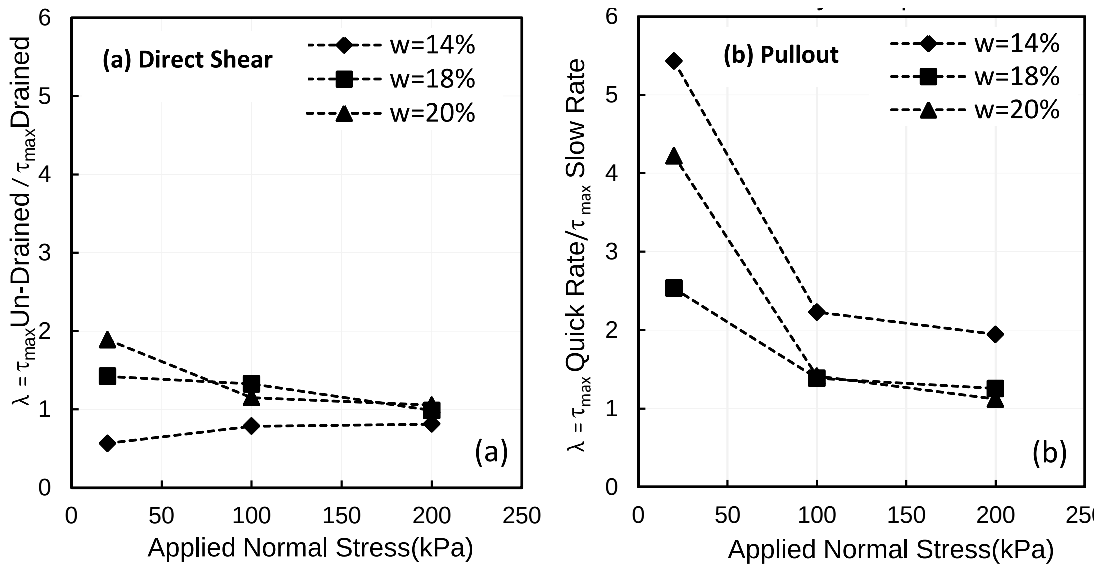


Figure.4.10. Ratio of the Maximum Undrained Shear Stress to Drained Shear Stress for (a) Interface Direct Shear Tests and (b) Single Fiber Pullout Tests

For the interface direct shear tests, results indicate that λ is slightly affected by the compaction water content of the clay. As *w* increased from 14% to 20%, the range of λ increased from 0.6 to 0.8 (w=14%) to 1.1 to 1.9 (w = 20%). If all the interface direct shear tests are considered, the average value of λ could be determined to be close to 1.0, indicating that the clay in the fast direct shear interface tests may not be really undrained due to partial drainage at the interface. For the pullout tests, λ was consistently found to be greater than 1.0, with a range of 2.5 to 5.4 for tests conducted at σ_n = 20 kPa decreasing to 1.1 to 2.0 for tests conducted at σ_n = 200 kPa. The relatively large values of λ for pullout tests are a direct effect of the significantly large ultimate pullout stresses that were measured in the fast single fiber pullout tests, where the hemp fibers were efficient at mobilizing the undrained shear strength of the clay.

The interface shear strength parameters that were determined from direct shear and pullout tests are compared in table 4.4 and table 4.5 for drained and undrained loading conditions, respectively. Results indicate that δ' values from pullout tests are slightly but consistently larger (by 1 to 3 degrees) than δ' for direct shear tests. The ranges of the effective adhesion (0 to 4.5 kPa for direct shear tests and 1.0 to 7.1 kPa for pullout) were found to be small and similar for the two test types. Similar observations were reported in Alfaro et al. (1995) where frictional interface properties from pullout tests were found to be higher than those determined from direct shear tests for geogrids that were pulled out from a dense granular backfill material.

For the unconsolidated fast tests, the total stress interface parameters shown in table 4.5 indicate that both a and δ are much larger in pullout (average a = 36 kPa and δ = 26 degrees) compared to direct shear (average a = 6 kPa and δ = 20.6 degrees). As mentioned in previous sections of this chapter, results indicate that for fast rates of loading, the response of the interface during pullout could be expected to be “truly” undrained allowing the interface strength to be directly correlated to the high undrained strengths that were measured in the fast clay/clay tests.

Table 4.4 Soil Adhesion and Interface Friction Angles for Drained Clay/Hemp Tests and Slow Rate Pullout Tests

Drained/Slow Rate	Direct Shear		Pullout	
Water Content (%)	a' (kPa)	δ' (°)	a' (kPa)	δ' (°)
14%	4.5	23.7	2.28	24.5
18%	0	22.5	7.1	23.7
20%	2.2	21.2	1	25.6

Table 4.5 Soil Adhesion and Interface Friction Angles for Un-Drained Clay/Hemp Tests and Quick Rate Pullout Tests

Un-Drained/Quick Rate	Direct Shear		Pullout	
Water Content (%)	a(kPa)	δ(°)	a(kPa)	δ(°)
14%	0.3	20.5	44.4	34
18%	8.1	21	29	24
20%	9.9	20.4	35.2	20

The results presented in the above section agree with findings reported in Hatami and Esmaili (2015) who conducted small scale interface direct shear and pullout tests on marginal soils with geomembranes. The parameters obtained from the tests on Chicasha Clay (CL) with different water contents and by controlled suction measurement indicate that the pullout shear strength parameters for dry of optimum a =56.7kPa and δ=30.2˚, for optimum a =43.5kPa and δ=23.8˚, and for wet of optimum a =38.9kPa and δ=18.1. While the results obtained from direct shear for the dry of optimum a=34.8kPa and δ=21˚, for optimum a=27kPa and δ=20˚, and for wet of optimum a=21.2kPa and δ=18.0. The decrease in strength as water contents increase was directly attributed to the reduction in the interface adhesion at lower suction values in pullout and direct shear tests.

E. Interface Coefficients and Overall Efficiency of Direct Shear and Pullout

Interface coefficients including $C_{i,c}$ and $C_{i,\phi}$ where $C_{i,c}$ represents the ratio of the soil adhesion to the soil cohesion and $C_{i,\phi}$ equal to the ratio of the tan (interface friction angle) over the tan (soil internal friction angle) are calculated for drained/ slow rate and un-drained/ quick rate for direct shear and pullout tests respectively at different water contents.

Those parameters are the key indices for modeling the shear strength of fiber reinforced soils and they vary according to the soil type and nature of reinforcement. In addition the overall efficiency "α", equal to the ratio of the interface shear strength over the clay shear strength is calculated for drained/ slow rate and un-drained/ quick rate for both direct shear tests and pullout tests.

1. Interface Coefficients $C_{i,c}$ and $C_{i,\phi}$

The interface coefficients were determined for the different cases analyzed in this study based on the direct shear and pullout tests. Since the interface resistance that was measured in the interface direct shear tests for unconsolidated fast tests was found to be non-representative of true undrained behavior, the interface coefficients from the direct shear tests were restricted to consolidated drained conditions. For the drained conditions, $C_{i,\phi}$ was calculated and plotted on Figure 4.11 for direct shear and single fiber pullout tests.

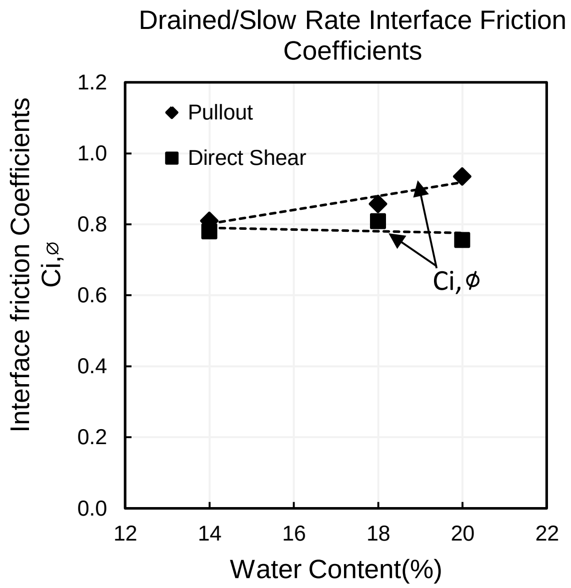


Figure.4.11 Interface Coefficient $C_{i,\phi}$ for Drained/Slow Rate Direct Shear and Pullout Tests.

Results on Figure 4.11 indicate that for the direct shear tests, an average interface coefficient $C_{i,\phi} \approx 0.78$ was obtained with minimum sensitivity to the compaction water content. For the single fiber pullout tests, the average $C_{i,\phi}$ was 0.85 with a range of 0.8 to 0.93. The interface coefficients in the pullout tests increased with increasing water content. Given the very small range of adhesion and cohesive intercepts that were obtained in the drained direct shear and pullout tests, the cohesive interface coefficient $C_{i,c}$ cannot be determined with any degree of confidence in the drained tests. In fact, it will not play any significant role in characterizing the drained interface behavior.

For the tests that were conducted under fast shearing/pullout conditions without allowing consolidation, the resulting interface coefficients were calculated and plotted on figure 4.12. As mentioned previously, $C_{i,c}$ and $C_{i,\phi}$ were only determined for the undrained pullout results since fast direct shear tests did not portray a true undrained behavior.

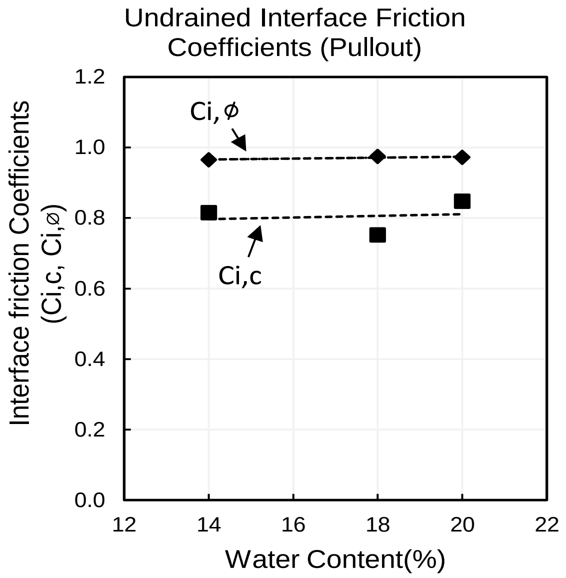


Figure.4.12 Interface Coefficients $C_{i,c}$ and $C_{i,\phi}$ for Fast Rate Pullout Tests .

Results of the quick fiber pullout tests indicate average interface coefficients of $C_{i,\phi} \approx 0.97$ and $C_{i,c} \approx 0.8$ with minimal variation with the compaction water contents. These values reflect a relatively high efficiency of the hemp fibers in mobilizing the undrained strength of the surrounding clay indicating high interface efficiency during fast shearing.

2. Overall Efficiency α

As a final indication of the For the direct shear tests, α is calculated as the ratio of τ_{max} (Clay/Hemp) / τ_{max} (Clay/Clay), while α for the pullout test was calculated as τ_{max} (Pullout)/ τ_{max} (Clay/Clay) where τ_{max} (Clay/Clay) is obtained from direct shear tests are shown in figure 4.13 .

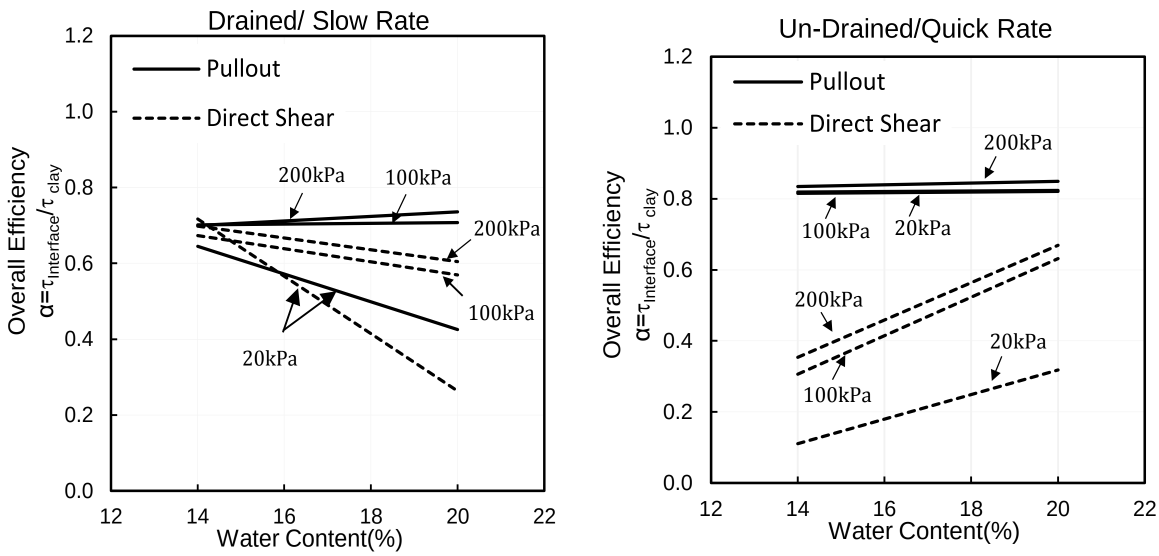


Figure 4.13 Overall Efficiency α of (a) Drained/Slow and (b) Undrained/Quick Direct Shear and Pullout Tests.

Results on Figure 4.13(a) indicate that the overall interface efficiency factor α for drained loading conditions was sensitive to the value of the compaction water content and the applied normal stress. For the small water content of 14%, the average alpha value was equal to about 0.7 with minimum variability at different normal stresses. For the case of the higher normal stresses of 100 kPa and 200 kPa, this average alpha value increases slightly with water content (alpha = 0.72 for w = 20%) for the drained pullout tests and decreases slightly with water content (alpha = 0.60 for w = 20%) for the drained direct shear tests. For the lower normal stress of 20%, alpha reduced significantly with water content particularly for the pullout test, reaching values close to 0.4 and 0.3 for the direct shear and pullout tests, respectively.

The alpha values that were calculated from the pullout undrained tests in Figure 4.13(b), an interface efficiency of about $\alpha \approx 0.8$ was determined irrespective of the compaction water content and applied normal stresses. The corresponding alpha values for the undrained direct shear tests were much lower and ranged from 0.1 to 0.35 at a water content of 14% and from 0.3 to 0.65 at the largest water content of 20%. It should be noted that the alpha values that were determined for the undrained direct shear tests may not be realistic nor representative of true undrained efficiencies, given the partial drainage that is hypothesized to exist at the interface. The trend of increasing alpha values with water content supports the hypothesis of interfacial partial drainage, since partial drainage is expected to become less significant for clays that are compacted wet of optimum (w=20%) where the permeability is expected to be relatively low. On the other hand, partial drainage is expected to be significant the case of w=14% and σ_n =20kPa, resulting in the lowest overall coefficient ($\alpha \approx 0.1$).

F. Summary

Results of 36 direct shear tests conducted on Clay/Clay and Clay/Hemp, prepared by compaction with different water contents, and tested under both consolidated drained and unconsolidated undrained conditions by applying different normal stresses were investigated. In addition 18 pullout tests where a fiber was inserted between two layers of compacted clay with different initial water contents and tested by pulling the fiber with both slow and quick rates with three applied normal stresses were inspected. Results obtained from both tests are analyzed and compared.

Results indicate that for drained conditions, the interface shear strength parameters could be determined from either direct shear tests or from single fiber pullout tests, with the drained interface friction angles being slightly larger (by 1 to 3 degrees) in the pullout tests. On the other hand, the true undrained interface response could only be reliably determined from results of pullout tests due to inevitable partial drainage that is expected to occur at the interface between the clay and the hemp-surface in the direct shear tests. The drained interface resistance between the hemp and the clay used in this study can be characterized by an interface friction angle of about 22 to 24 degrees and a small/negligible associated effective cohesion intercept. The undrained interface friction angle and cohesion are sensitive to the compaction water content. As a result, they can best be determined using interface coefficients of $C_{i,\phi} \approx 0.97$ and $C_{i,c} \approx 0.8$ with minimal variation with the compaction water contents.

- Peak shear stress of Clay/Clay and Clay /Hemp of direct shear test increase with the increase of applied normal stress for all tests irrespective of water content.
- Clay/Clay shear stress is greater than Clay/Hemp shear stress for all tests irrespective of water content.

- The un-drained Clay/Clay shear strength is greater than drained Clay/Clay shear strength for all tests irrespective of water content.
- For w=14% the undrained shear stress of Clay/Clay is the highest among all tests.
- For w=18% and w=20% the results of interface are consistent with Clay/Clay shear stress however, for w=14% drained shear stress of Clay/Hemp is greater than un-drained interface shear stress.
- For drained Clay/Hemp tests, clays compacted at w=14% gave higher strength compared to clays compacted at w=18% and w=20%.
- Interface friction angle δ′ decrease with the increase of water content.
- For w=18% and w=20% $a\approx$9Kpa and δ=20.5˚ where δ′ is greater than δ by 1˚to2˚ and ′ is less than a.
- For slow rate pullout tests, ultimate pullout shear strength is not sensitive to water content and increases with the increase of applied confinement.
- For fast rate pullout tests, ultimate pullout shear stress decrease with the increase of water content.
- For all tests fast rate pullout shear stress is greater than slow rate pullout shear stress.
- For slow rate pullout tests ′ ranges between (1kPa and 7kPa) and δ′ ranges between (23.7˚ and 25.6˚).
- For fast rate pullout tests a is much larger than ′ for all tests.

- λ of pullout tests is higher than λ of direct shear tests irrespective of water content.
- Mohr-Coulomb failure envelopes of drained direct shear interface tests are close to Mohr-Coulomb failure envelopes of slow rate pullout tests.
- Mohr-Coulomb failure envelopes of un-drained direct shear interface tests are below Mohr-Coulomb failure envelopes of fast rate pullout tests, where the difference increase with the decrease of water content.
- δ' of pullout tests are higher than δ' of direct shear tests for all water contents and (a, δ) of pullout are greater than (a, δ) of direct shear interface tests.
- For drained interface direct shear tests $C_{i,\phi}$ =0.78 and for slow rate pullout tests $C_{i,\phi}$ ranges between (0.8 and 0.93).
- For fast rate pullout tests interface coefficients $C_{i,\phi}$=0.97 and $C_{i,c}$~0.8.
- Overall efficiency of fast rate pullout tests $\alpha \approx 0.8$, and for un-drained direct shear test α increase with the increase of water content.
- The least obtained overall efficiency was observed for un-drained direct shear tests at w=14% and σ_n=20kPa.

CHAPTER V

CONCLUSION, RECOMMENDATION AND FURTHER RESEARCH

A. Introduction

This thesis presented the results of a conducted laboratory experimental program of 36 direct shear tests and 18 pullout tests conducted on compacted natural clay and natural hemp fibers. Direct shear tests were performed on clay/clay and clay/hemp, where in interface tests clays were sheared against hemp fixed in the lower part of the direct shear box. Pullout tests were performed using a custom fabricated apparatus by modifying the one-dimensional consolidation set-up, where a fiber was inserted between two layers of compacted clay and pulled using a pulley under the weight of water.

As expected the interface response was affected by the compaction water content or degree of saturation where three water contents were investigated (14%,18% and 20%), the rate of shearing/pullout (undrained versus drained), and the applied normal stress (20 kPa, 100 kPa, and 200 kPa). Results obtained from direct shear tests and pullout tests were discussed and compared.

This chapter includes the main concluding remarks and observations resulting from the conducted direct shear and pullout tests on compacted clay and natural hemp fibers. Recommendations and further research are also discussed.

B. Conclusions

Based on the results of 36 direct shear tests conducted on clay/clay and clay/hemp, in addition to 18 pullout tests the following conclusions can be drawn with regards to the reliability of the testing used procedure, the effect of water content, degree of saturation, dry density, permeability, generation of excess pore pressure, applied normal stress and rate of shearing:

1. As expected in all direct shear tests the maximum clay shear stress is higher than the peak interface stress with an interface efficiency smaller than 1.0 irrespective of the tested water content and the applied normal stress.

2. In general for drained clay/clay and drained clay/hemp tests the peak shear stress increase with the increase of effective normal stress for all tests irrespective of water content. A strain hardening behavior was observed for clay/clay tests where curves were observed to level out at the maximum applied horizontal displacement of 6mm to 7mm irrespective of the compaction water content and applied normal stress. Interface tests exhibited strain softening behavior during post peak shearing with clear peaks at σ'_n=200kPa and σ'_n=100kPa where the peak shear stress was mobilized at a horizontal displacement ranging from 0.5mm~1.5mm.

2. An unexpected sensitivity of the undrained shear stress of clay/clay was observed to the total applied normal stress despite the fact that the tests were designed to be unconsolidated and undrained. This sensitivity of the shear stress versus horizontal displacement response to the total normal stress is reduced at higher water contents (higher degree of saturation) where the performance is less sensitive to the applied normal stress exhibiting more unconsolidated undrained response. This sensitivity was attributed to the fact that the soil is not fully saturated especially at lower water contents and due to the presence of 45% sand in the soil matrix. For relatively unsaturated compacted clays, this percentage of sand will contribute to increasing the total stress clay friction angle. A strain hardening behavior was observed for the fast clay/clay tests for tests conducted at σ_n =100 and 200kPa however clear peaks were observed at σ_n=20kPa at displacements ranges from1.5mm for tests compacted at w=14% and 3.5mm for tests compacted at w=20%. This brittle response at σ_n=20kPa was attributed to dilation of the unsaturated clay against the relatively low applied normal stress during shearing. Undrained interface shear stress exhibited a strain hardening behavior that was more pronounced for σ_n =100 and 200kPa. In general the undrained interface response was similar to the drained interface response for all tested water contents and applied normal stress, which points to a possible drainage at the level of interface in fast shearing conducted tests.

3. The undrained shear stress of clay/clay is greater than the drained clay/clay shear stress for all tests irrespective of water content, where the difference between the drained and the undrained response increase as the water content decrease (from 20% to 18% to 14%) and as the applied normal stress decreases (from 200 kPa to 100 kPa to 20 kPa). This was attributed to the high matric suction values coupled with the generation of negative pore water pressures resulting in the increase of stresses during quick shearing, this was explicit at highly un-saturated clays (w=14%) and low applied normal stress at σ_n =20kPa where generation of negative pore pressure is expected. Moreover, the difference between the undrained clay/clay shear stress and the counterpart drained clay/clay shear stress increase as σ_n decreases where λ , defined as the ratio of the undrained maximum shear stress to the drained maximum shear stress at a given water content and normal stress was equal to =1 for w= 20% and σ_n =200kPa, while λ increases up to 4 for w= 14% and σ_n=20kPa.

4. For the undrained interface shear stress, a minimal difference between drained and undrained behavior was observed irrespective of the applied normal stress and the compaction water content. More specifically for tests conducted at w=18% and w=20% the undrained response was slightly higher than the drained response for cases involving σ_n=100kPa and σ_n=20kPa, while the opposite was true for tests conducted at w=14% for all applied normal stresses where the drained response was consistently higher than the undrained response. This slight difference highlights the potential of possible drainage at the level of interface where such drainage may be inevitable within an interface direct shear setup whereby the clay is sheared on a hemp surface in addition hemp fibers themselves could act like a drainage conduit that could facilitate drainage of water from the thin clay surface that is in contact with the fibers. This partial drainage prohibited the hemp from mobilizing the undrained interface behavior of the contact clay.

5. The drained interface shear stress of tests conducted at w=14% were consistently higher than the undrained interface shear stress for all applied normal stresses. It could be argued that the clay that was compacted dry of optimum at w = 14% is expected to have a relatively high permeability and a relatively low dry unit weight. This relative high permeability contributes in amplifying the partial drainage that exists at the interface in the fast tests, approaching the drained interface shear stress and preventing the hemp interface from mobilizing the "undrained" strength of the clay that is in contact with it. Furthermore, the drained response is benefitting from consolidation, allowing the increase of dry densities while in the undrained tests no consolidation is allowed prior to shearing maintaining relatively low dry densities. For the other water contents (18% and 20%) the initial dry densities are relatively large and the permeability is relatively low, where consolidation is not expected to have an impact on the drained interface response.

6. No difference in response was observed for drained clay /clay shear stress at w=14%, 18% and 20% at high confinement σ_n=200kPa. This was attributed to the effect of consolidation during the application of high normal stress, coupled with the slow drained shearing mechanism, permitting the response of the different specimens to converge despite the differences in the initial structure and dry density. As the applied normal stress decrease to σ_n=100kPa, and 20kPa the ability of σ_n to normalize the behavior becomes less especially for w=14% which has initially a low dry density (high void ratio) and a flocculated structure.

The case of σ_n =20kPa exhibits the scenario with the largest differences in clay/clay response at different water contents where the maximum shear stress for w=14% was found to be the smallest followed by w=18% followed by w=20%.

7. Highest response was observed for drained interface tests compacted at w=14%, where a smaller drained interface response was regarded for w=18% and w=20%, this was attributed to the detrimental effect of higher water content on drained interface strength due to lubricating effect of water resulting in the reduction of the load transfer between clay particles and fibers at the level of interface.

8. The “undrained” clay shear stress pertained to w=14% consistently yielded a higher strength compared to clays compacted at w=18% and 20%, irrespective of the applied normal stress. This was attributed to the generation of negative pore pressure and the effect of matric suction in the unsaturated specimens. For w=18% and 20%, similar stress-displacement responses were observed.

9. “Undrained” Clay/Hemp shear stress, for clays compacted at w=14% have the lowest undrained interface strength while clays compacted at w=18% and w=20% have similar behavior. Clays compacted at w=14%, are not benefitting from the negative pore pressure or matric suction effect due to possible drainage at the interface due to high permeability.

10. Envelopes of the interface clay/hemp tests were always lower than the envelopes of the clay/clay tests, indicating that the efficiency of the hemp fibers is less than unity, irrespective of the test conditions. The difference between the clay/clay and clay/hemp envelopes was more pronounced for the undrained cases, where the shear strength envelopes of the clay exhibited cohesive intercepts that were relatively large (38.6 kPa to 54.5 kPa).

11. Decrease in water content resulted in the increase of the difference between clay drained envelopes and undrained clay envelopes where it was observed that the undrained envelopes are predominately higher than their drained counterparts. Moreover, interface envelopes for drained and undrained tests were found to be very close to each other.

12. Similar drained shear strength parameters c' and ϕ' for the tests conducted with a water content of 18% and 20%, with a c' of about 14 kPa and ϕ' of 27 degrees. These results may be considered to be realistic for low plasticity clay specimens that are compacted close to optimum. The relatively high drained friction angle could be attributed to the presence of more than 45% sand in the clay matrix. For clays that were compacted dry of optimum at a lower water content of 14%, a slightly lower c' value of 5.4 kPa and a slightly higher ϕ' of 29.4 degrees were obtained.

13. As expected the total cohesive intercept c was relatively large (38.6 kPa to 54.5 kPa) compared to the effective values. In addition, for w=18% the total stress friction angles (ϕ = 24.4 degrees) and w=20% (ϕ = 20.5 degrees) were found to be slightly smaller than their drained counterparts. The difference between the total stress friction angles (ϕ = 24.4 versus 20.5 degrees) is related to the degree of saturation of the clay which is expected to be larger in the w = 20% tests. For w=14% a steeper total friction angle is observed of about 35 degrees, this was attributed to the relatively low degree of saturation and the associated effect of matric suction coupled with the presence of a significant proportion of sand in the clay matrix. For cases involving w=18% and w=20%, ϕ' was found to be greater than ϕ and c' was found to be less than c, while for the case involving w of 14% c and ϕ were found to be greater than c' and ϕ'.

14. Increase in water content resulted in the decrease of the drained interface friction angle δ' where it varies in a narrow range (of 21.2 to 23.7 degrees), due to the lubrication effect of water. The drained adhesion intercept a' is relatively small and ranges from 0 to 4.5 kPa.

15. Undrained interface parameters for w=18% and w=20% are close to each other where $a \approx$9kPa slightly higher than the effective cohesive intercept c' and δ=20.4˚ and 20.5˚ respectively which are slightly smaller than the effective δ' (21.2 and 22.5 degrees). For the cases involving w=14% a decreases reaching ~ zero value and δ=20.5˚ due to the possible partial drainage at the boundary of interface due to high permeability, this explains the relatively similar interface shear strength envelopes of the drained and undrained tests.

16. Increase in the applied effective normal stress resulted in the increase of the ultimate pullout stress in tests where the fiber was pulled out slowly however it was not sensitive to the compaction water content of the clay. Tests conducted with fast rate fiber pullout, resulted in higher value of the ultimate pullout compared to the corresponding ultimate pullout stress of tests conducted at slow rate. This indicated that unlike direct shear tests where partial drainage could have occurred at the interface fast pullout tests may have exhibited a true "undrained" behavior at the interface between the hemp fiber and the compacted clay.

17. Fast rate ultimate pullout stress is highly dependent on compaction water content particularly for the case involving w=14% compared to other water contents. Ultimate pullout resistance decrease as water content increase from 14% to 18%, to 20% with the drop being more evident for σ_n=200kPa. This decrease in ultimate pullout strength for w=14%, 18% and 20% is related to the high matric suction values at highly un-saturated clays.

18. Identical interface envelopes for the three water contents were observed in cases involving slow pull out of the fiber, with effective adhesion values a' ranging from 1kPa and 7kPa, and effective interface friction angles δ' ranges between 23.7˚ and 25.6˚. On the other hand, for tests involving fast rates of pullout the total stress adhesion a was relatively large (35 kPa to 44 kPa) compared to the drained pullout tests. In addition, the total stress interface friction angles δ for the w=18% (δ = 24 degrees) and w=20% (δ = 20 degrees) cases were found to be slightly smaller or equal to their drained counterparts. For cases involving w=14% the total stress pullout envelop was very steep with relatively high interface friction angle of about 34 degrees and relatively high associated adhesion intercept *a* = 44.4 kPa. These relatively high total interface parameters are directly correlated with the relatively equally high friction angle and cohesion witnessed in the undrained clay/clay direct shear tests for tests involving w=14%.

19. Failure envelopes for slow drained tests, of the pullout and direct shear tests were relatively close to each other, with identical values observed for the case involving w= 14%. For other water contents w = 18% and 20%, the maximum interface pullout stresses were slightly and consistently larger than their direct shear counterparts. On the other hand, a superior interface response in the single fiber pullout tests was observed for unconsolidated fast tests with maximum pullout stresses that are significantly larger than the counterpart direct shear stresses. Moreover, it was recognized that the maximum pullout stresses in fast tests approached the undrained maximum clay/clay stress for all water contents used.

20. The ratio of the maximum undrained interface shear stress to the maximum drained interface shear strength "λ" is slightly affected by the compaction water content of the clay for the interface direct shear tests where *w* increased from 14% to 20%, the range of λ increased from 0.6 to 0.8 (w=14%) to 1.1 to 1.9 (w = 20%). The average value of λ could be determined to be close to 1.0, when considering all direct shear tests indicating that the clay in the fast direct shear interface tests may not be really undrained due to partial drainage at the interface. On the other hand , "λ" for the pullout tests was consistently found to be greater than 1.0, with a range of 2.5 to 5.4 for tests conducted at σ_n = 20 kPa decreasing to 1.1 to 2.0 for tests conducted at σ_n = 200 kPa. These relatively large values of λ for pullout tests are a direct effect of the significantly large ultimate pullout stresses that were measured in the fast single fiber pullout tests, indicating a high efficiency of the hemp fibers in mobilizing the undrained shear strength of the clay.

21. The effective interface friction angle δ' values resulted from pullout tests are slightly but consistently larger (by 1 to 3 degrees) than δ' for direct shear tests. The ranges of the effective adhesion (0 to 4.5 kPa for direct shear tests and 1.0 to 7.1 kPa for pullout) were found to be small and similar for the two test types. Furthermore, the total interface parameters indicates that both a and δ are much larger in pullout (average a = 36 kPa and δ = 26 degrees) compared to direct shear (average a = 6 kPa and δ = 20.6 degrees). These results further indicates that for fast rates of loading, the response of the interface during pullout could be expected to be "truly" undrained allowing the interface strength to be directly correlated to the high undrained strengths that were measured in the fast clay/clay tests.

22. Interface coefficient $C_{i,\phi}$ equal to the ratio of the tan (interface friction angle) over the tan (soil internal friction angle) calculated for drained conditions resulted in an average value $C_{i,\phi} \approx 0.78$ for direct shear tests with minimum sensitivity to the compaction water content. For the single fiber pullout tests, the average $C_{i,\phi}$ was 0.85 with a range of 0.8 to 0.93. The interface coefficients in the pullout tests increased with increasing water content.

23. The undrained interface coefficients $C_{i,c}$, representing the ratio of the soil adhesion to the soil cohesion, and $C_{i,\phi}$ equal to the ratio of the tan (interface friction angle) over the tan (soil internal friction angle) were only calculated for quick pullout tests resembling true "undrained" behavior resulted in values of $C_{i,c} \approx 0.8$ and $C_{i,\phi \approx} 0.97$ with minimal variation with the compaction water contents. These values reflect a relatively high efficiency of the hemp fibers in mobilizing the undrained strength of the surrounding clay indicating high interface efficiency during fast shearing.

24. Overall interface efficiency factor α calculated as the ratio of τ_{max} (Interface) / τ_{max} (Clay) for drained loading conditions was sensitive to the value of the compaction water content and the applied normal stress. For the small water content of 14%, the average alpha value was equal to about 0.7 with minimum variability at different normal stresses. For the case of the higher normal stresses of 100 kPa and 200 kPa, this average alpha value increases slightly with water content (alpha = 0.72 for w = 20%) for the drained direct shear tests and decreases slightly with water content (alpha = 0.60 for w = 20%)

for the drained pullout tests. For the lower normal stress of 20%, alpha reduced significantly with water content particularly for the pullout test, reaching values close to 0.4 and 0.3 for the direct shear and pullout tests, respectively.

25. Undrained interface efficiency for quick pullout tests resulted in a value of about $\alpha \approx 0.8$ irrespective of the compaction water content and applied normal stresses. The corresponding alpha values for the undrained direct shear tests were much lower and ranged from 0.1 to 0.35 at a water content of 14% and from 0.3 to 0.65 at the largest water content of 20%. However, undrained interface efficiencies calculated from the corresponding direct shear tests may not be representative of true undrained efficiencies, given the partial drainage that is hypothesized to exist at the interface. The observed increasing trend of alphas with the water content in the case of direct shear tests supports the hypothesis of interfacial partial drainage, since partial drainage is expected to become less significant for clays that are compacted wet of optimum (w=20%) where the permeability is expected to be relatively low. On the other hand, partial drainage is expected to be significant in the case involving w=14% and σ_n =20kPa, resulting in the lowest overall coefficient ($\alpha \approx 0.1$).

C. Recommendations

Based on the reported results in this study a general conclusion can be declared stating the following hypothesis, tests conducted with a fast rate fiber pull out pronounces true “undrained” behavior where the ultimate pullout shear stress is benefitting from the high undrained shear strength of clay/clay, and this was attributed to the effect of matric suction where the maximum ultimate pullout shear stress was observed for tests involving w=14% where matric suction is expected to be highest for highly un-saturated soils. In contrary, and for direct shear tests despite the relatively fast shear rate “undrained” behavior was not declared where drained interface shear strength were close to undrained interface shear strength due to possible partial drainage at the interface boundary and this was asserted especially for tests where clays were compacted at w=14% and σ_n=20kPa, resulting in the least overall efficiency among all tests $\alpha \approx 0.1$ due to the fact that clays are of highest permeability and relatively low confinement compared to other tests.

Results indicate that for drained conditions, the interface shear strength parameters could be determined from either direct shear tests or from single fiber pullout tests, with the drained interface friction angles being slightly larger (by 1 to 3 degrees) in the pullout tests. On the other hand, the true undrained interface response could only be reliably determined from results of pullout tests due to inevitable partial drainage that is expected to occur at the interface between the clay and the hemp-surface in the direct shear tests. The drained interface resistance between the hemp and the clay used in this study can be characterized by an interface friction angle of about 22 to 24 degrees and a small/negligible associated effective cohesion intercept. The undrained interface friction angle and cohesion are sensitive to the compaction water content. As a result, they can best be determined using interface coefficients of $C_{i,\phi} \approx 0.97$ and $C_{i,c} \approx 0.8$ with minimal variation with the compaction water contents.

Drained interface shear strength calculated from direct shear tests decrease with the increase of water content where the drained interface shear strength for clays compacted at the dry of optimum (w=14%) was higher than that of clays compacted at optimum and wet of optimum (w=18% and w=20%) due to the detrimental effect of water lubrication at higher water contents resulting in decreasing the strength by reducing the load transfer between the fibers and clay particles.

Undrained interface shear strength increase with the decrease of water content from 20%, to 18% to 14% where the undrained interface shear strength for clays compacted at the dry of optimum (w=14%) was higher than that of clays compacted at optimum and wet of optimum (w=18% and w=20%) due to the effect of matric suction at highly unsaturated clays.

The interfacial behavior between natural compacted clay and natural hemp fibers resulted in an excellent response under “undrained” or fast rate conditions where the overall efficiency α ~0.8 obtained from pullout tests that resemble true “undrained” behavior. The interface coefficients between clay and hemp for short term behavior $C_{i,c}$ ~0.8 and $C_{i,\phi}$=0.97, are obtained from high quality pullout tests and they can be used as design parameters in models estimating the shear strength of fiber reinforced soils for cases involving the use of the tested natural soil reinforced with natural hemp fibers.

The custom fabricated pullout test used in this study resulted in good interface parameters especially in that resembling “undrained” or fast rate single fiber pull out. This indicates that this used apparatus is efficient in producing high quality results; in addition it is simple to be elaborated, more economical set up and certainly exists in all geotechnical laboratories. In general pullout results could be presented as Mohr coulomb failure envelopes at different confinements to obtain the interface parameters.

Interface parameters obtained in this study covers the effect of water content , rate of shearing (drained/slow rate versus un-drained/fast rate), applied normal stress and type of testing or interface behavior (direct shear versus pullout). The obtained parameters could be used to formulate a model for estimating the interface parameters between the tested clay and randomly distributed hemp fibers under different conditions. Both direct shear tests and pullout tests should be conducted to derive interaction parameters.

It is highly recommended that those specific lab tests both direct shear and pullout should be conducted to represent actual field conditions, in order to obtain interface parameters and interface coefficients when introducing new soil reinforcement materials. Those parameters present pragmatic peculiar indices to be inserted in models predicting shear strength of the new reinforced soil composites. This is an advantage by allowing engineers to obtain affordable, more accurate, and reliable interface parameters compared to estimated values used in the design of soil reinforced structures.

Dealing with unsaturated soils necessitate the conduction of laboratory tests to represent actual field conditions and produce reliable interface parameters since they are generally low permeable and are coupled with the generation of negative pore pressures due to matric suction. Therefore, using properties of fully saturated counterpart soils are not applicable in design and leads to inaccurate predictions of the soil structure behavior, uneconomical and unreliable factors of safety.

D. Further Research

- Since Hemp showed a great interfacial response with clays in short term behavior it is valuable for future research to support cultivation of cannabis legislation and highly recommend hemp as an effective soil reinforcement material.

- Pullout test apparatus used could be improved to allow the measurement of horizontal displacement associated with the fiber pullout.

- The same laboratory testing program is highly recommended to be repeated on different types of natural and synthetic fibers in order to obtain different interface parameters and interface coefficients. Moreover, this will allow the comparison of the behavior of natural versus synthetic fibers on the interfacial shear strength under different conditions.

- Finally, in addition to laboratory work it would be more significant if the findings and observations resulted from this laboratory research could be verified and confirmed with full scale field tests.

APPENDIX .1

Maximum Shear Stresses of Clay/Clay and Cay/Hemp from Direct Shear Tests

Table A.1.1 Maximum shear stress of drained clay/clay and clay/hemp for different water contents.

Drained		Clay/Clay		Clay/Hemp	
		σ'$_n$	τ'	σ'$_n$	τ'
w=20%	**20**	24.67	27.82	20	8.01
	100	114.34	71.83	100	44.48
	200	227.25	131.70	200	78.31

Drained		Clay/Clay		Clay/Hemp	
		σ'$_n$	τ'	σ'$_n$	τ'
w=18%	**20**	23.61	26.00	20	9.81
	100	114.28	71.00	100	37.53
	200	227.34	130.00	200	84.52

Drained		Clay/Clay		Clay/Hemp	
		σ'$_n$	τ'	σ'$_n$	τ'
w=14%	**20**	21.87	17.96	20	13.09
	100	115.39	69.77	100	48.70
	200	226.42	132.89	200	92.12

Table A.1.2 Maximum shear stress of undrained clay/clay and clay/hemp for different water contents

Un-Drained		**Clay/Clay**		**Clay/Hemp**	
		σ_n	**τ**	**σ_n**	**τ**
w=20%	**20**	24.4	49.8	20	15.1
	100	114.5	85.6	100	51.2
	200	227.0	125.5	200	82.5

Un-Drained		**Clay/Clay**		**Clay/Hemp**	
		σ_n	**τ**	**σ_n**	**τ**
w=18%	**20**	24.40	51.54	20	13.9
	100	113.36	86.67	100	49.7
	200	227.55	143.34	200	83.4

Un-Drained		**Clay/Clay**		**Clay/Hemp**	
		σ_n	**τ**	**σ_n**	**τ**
w=14%	**20**	22.66	71.69	20	7.42
	100	114.85	132.19	100	38.24
	200	228.21	214.82	200	74.80

APPENDIX .2

Maximum Shear Stresses of a Single Fiber Pullout Tests

Table A.2.1 Maximum shear stress of slow interface pullout tests at different water contents and applied normal stresses.

Slow Interface Pullout Test			
w(%)	sigma 3 (kPa)		
	20	100	200
14	11.07	48.32	93.00
18	15.20	52.10	94.18
20	10.25	49.49	96.87

Table A.2.2 Maximum shear stress of quick interface pullout tests at different water contents and applied normal stresses.

Quick Interface Pullout Test			
w(%)	sigma 3 (kPa)		
	20	100	200
14	60.12	107.71	180.78
18	38.48	72.16	118.00
20	43.28	69.85	108.31

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