AMERICAN UNIVERSITY OF BEIRUT

TRIAXIAL RESPONSE OF NATURAL CLAY REINFORCED WITH HEMP FIBERS UNDER FULLY DRAINED CONDITIONS

by
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AN ABSTRACT OF THE THESIS OF

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Different soil improvement techniques have been utilized to enhance the engineering properties of soft soils. Soil reinforcement by fibers is viewed as a viable and efficient ground improvement technique which is gaining more interest in the geotechnical community. Reinforcement of soils with discrete natural or synthetic fibers has been shown to increase soil strength and load-bearing capacity for applications involving earth retaining systems, pavement systems, earth slopes, and compacted clay liners and cover systems. An investigation of the published literature reflects a significant effort to study the response of fiber-reinforced clays using triaxial tests. Such tests are generally limited to investigating the undrained response of Kaolin clay and synthetic fibers with few tests targeting natural clay and natural fibers under fully drained conditions. There is a need for studying the response of fiber-reinforced clay systems under fully drained conditions to quantify the expected increase in shear strength for long-term stability assessment. In particular, there is a need for expanding the knowledge on the response of natural fibers within a drained testing setting whereby durability issues might affect the response of the composite.

The objectives of this dissertation are to (1) investigate the drained shear strength of clays that are reinforced with natural hemp fibers and which are compacted to different initial water contents, (2) quantify the level of improvement in the drained shear strength due to the addition of fibers, and (3) investigate the durability of hemp fibers for long term stability applications involving compacted clay systems. To achieve these objectives, a comprehensive and wide-reaching experimental drained triaxial testing program was conducted to study the load response of natural clay specimens that are reinforced with "natural" Hemp fibers. The parameters that were varied are the fiber content, confining pressure, and compaction water content. Results indicated that natural Hemp fibers are effective in increasing the drained shear strength of natural clay. The percent improvement in strength was found to be highly dependent on the adopted reinforcement schemes and the testing parameters. However, results of durability tests indicated that the improvement in strength due to the addition of hemp fibers is lost for specimens that are allowed to cure for 3 weeks prior to drained testing.

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CHAPTER 1

INTRODUCTION AND BACKGROUND

1.1 Introduction

Patel and Narayan (2005) defined Sustainable development as the ‘‘development that meets the needs of present without compromising the needs of future generations to meet their own needs.’’ Sustainable development helps in supporting the economic and social development and primarily protecting the environment. Sustainability concepts have emerged globally in an attempt to inhibit further climate change and depletion of natural resources. Numerous engineering domains, including geotechnical engineering and construction materials, are incorporating sustainable materials and systems.

In the geotechnical engineering industry, the use of inclusions (such as fibers) and additives that are commonly integrated into the soil for improvement purposes is on the rise. Replacing synthetic fibers with natural ones may present an appealing sustainable substitute as the latter would produce a minimal carbon footprint. Dittenber and GangaRao (2012) stated that natural fibers require only around 20–40% of the production energy as compared to synthetic fibers. In addition, the production of natural fibers is cost-effective and could improve the social/economic conditions of rural communities by encouraging local agricultural production of the needed materials. Natural fibers inclusion is viewed as a successful ground improvement technique because of its cost viability, simple flexibility, environment friendliness and reproducibility. In line with this trend, several studies have targeted the potential use of natural fibers (coconut fibers, palm fibers, straw fibers, bamboo fibers, cane fibers, etc.) instead of the commonly used synthetic fibers (polypropylene fibers, polyester fibers, glass fibers, scrap tire rubber fibers, polyvinyl fibers, etc.) for soil improvement purposes.

This study aims at exploring the use of natural “Hemp” fibers in improving the long-term drained load response of compacted natural clays. The proposed technique is expected to satisfy the abovementioned sustainability criteria. Hemp fibers can be produced while saving on expenses, natural resources and energy.

1.2 Fibers in Soil background

Earth reinforcement is a technique that is implemented in nature by animals, birds and the action of tree roots. Construction projects that adopted fiber inclusions are acknowledged to have existed in the fifth and fourth millenniums BC. Vidal of France introduced the concept of reinforced soils with continuous reinforcement (metallic strips, geomembranes, or geotextiles) in 1966. Fiber-reinforced soils have since been recommended for numerous geotechnical applications. Soil reinforcement with natural and synthetic fibers is a practical system for increasing the soil's strength and load-bearing capacity. Fiber inclusion in soils is viewed as a successful ground improvement technique because of its cost viability, flexibility, environment friendliness, and reproducibility. It is currently widely adopted in various applications involving the reinforcement of soils in earth retaining systems, pavement systems, earth slopes, and compacted clay liners and cover systems (Najjar et al. 2013, Sadek et al. 2010, Gregory 2006, Park and Tan 2005, Santoni et al. 2001, Rifai 2000, and Gregory and Chill 1998). The main advantage of soil reinforcement with discrete fibers compared to conventional systems of soil reinforcement that involve the usage of continuous planar inclusions is that discrete fibers contribute positively in the isotropic increase of the soil composite strength without introducing continuous planes of weakness (Maher and Gray 1990). The use of fibers is advantageous since they help in surpassing anchorage concerns given that they are mixed within the soil mass and compacted using a traditional tool as well as they are not easily damaged during execution (Gregory 2006).

The response of soil/fiber composite systems has been the matter of widespread experimental and numerical studies that aim at enhancing the design of these systems (ex. Gray and Al-Refeai 1986; Maher and Gray 1990; Michalowski and Zhao 1996; Michalowski and Cermak 2002 and 2003; Consoli et al. 2005; Chen 2008; Consoli et al. 2007; Russell et al. 2007; Michalowski 2008; Sadek et al. 2010; Najjar et al. 2013). Comprehensive experimental studies targeted the characteristics of fiber reinforced soil systems and they were mainly shifted towards the reliance on direct shear tests and triaxial tests.

Traditionally, fibers that have been investigated include synthetic fibers (ex. polypropylene, polyester, glass, rubber, and polyvinyl fibers). A broad assessment of the published literature on triaxial tests on fiber-reinforced clays indicates that almost all tests are conducted using Kaolin clay and synthetic fibers. These studies have failed to give sufficient importance to studying the behavior of natural clay that is reinforced with natural fibers. In modern days, environmental concerns and sustainability concepts have affected the field of civil engineering in general, and construction engineering and construction materials in particular. Thus, the reliance on using natural fibers (coconut fibers, palm fibers, straw fibers, bamboo fibers, Hemp fibers) is becoming a necessity because of the increasing demand for integrating sustainable materials in construction. The main concern in using natural fibers is whether they are durable or not with an evident need to investigate the durability of fiber-reinforced clay systems that are reinforced with natural fibers.

Type of reinforced soil (1)	Type of reinforcement (2)	Stress-deformation behavior of reinforcement (3)	Role and function of reinforcement (4)
Reinforced Earth® (Vidal, 1969)	Ideally inextensible inclusions (metal strips, bars, etc.) $E_R/E_S > 3{,}000^a$	Inclusions may have rupture strains which are less than the maximum tensile strains in the soil without inclusions, under the same opn. stress conditions, i.e.: $(\epsilon_R)_{RUP} < \epsilon_{MAX}$ Depending on the ultimate strength of the inclusions in relation to the imposed loads; these inclusions may or may not rupture.	Strengthens soil (increases apparent shear resistance) and inhibits both internal and boundary deformations. Catastrophic failure and collapse of soil can occur if reinforcement breaks.
"PLY-SOIL" (McGown, et al., 1978)	Ideally extensible inclusions (natural and synthetic fibers, roots, fabrics, "geotextiles") $E_R/E_S < 3{,}000$	Inclusions may have rupture strains larger than the maximum tensile strains in the soil without inclusions, i.e.: $(\epsilon_R)_{RUP} > \epsilon_{MAX}$ These inclusions cannot rupture no matter their ultimate strength or the imposed load.	Some strengthening . . . but more importantly provides greater extensibility (ductility) and smaller loss of post peak strength compared to soil alone or to reinforced earth.

[a] E_R/E_S = the ratio of reinforcement modulus (longitudinal stiffness) to average sand modulus. The limits shown are tentative; reinforcement/sand modulus ratios for all materials tested (including copper wires) ranged from 71 to 2,940.

Figure 1.1 Comparative Behavior of earth reinforcement (McGown et al. (1978))

The review of the literature also points to an important shortcoming in the published experimental work on fiber-reinforced clays. The majority of the experimental work on fiber-reinforced clays focuses on the undrained response of the composite. There is a clear need for studying the fully drained response of fiber-reinforced clay systems that have been compacted at different moisture content. The improvement brought by the addition of fiber to the response of the composite could be significantly affected by the drainage conditions and the rate of loading. An investigation of the drained response of fiber-reinforced clay systems is mandatory for the assessment of stability for long-term conditions.

Fiber-reinforced soil offers numerous advantages when compared to more traditional planar and oriented reinforcement options. These include:

a) Maintaining strength isotropy by preventing the occurrence of potential planes of weakness that can develop parallel to the planar oriented reinforcement as a result of the interface shear strength being usually less than that of the soil (Maher and Gray 1990).

b) Surpassing anchorage considerations: fibers are simply mixed within the soil mass while planar reinforcement necessitate enough embedment length and/or a properly designed anchorage to attain the required pullout resistance (Gregory 2006).

c) Using conventional equipment and standard compaction methods (Figure 1.2 and Figure 1.3) to complete the field spreading/mixing of fibers and the compaction of the composite, with fibers not prone to easy damage during the process (Gregory 2006 and Li 2005).

d) Cost effectiveness given the wide availability of fibers (Hejazi et al. 2012).

Figure 1.2 Field spreading and mixing of fibers (from GEOFIBERS job report – Riverview drive slope reconstruction)

Figure 1.3 Conventional tractor and roller compactor used for field placement of fibers (from YouTube, UAF College of Engineering & Mines, 2010)

However, planar reinforcement techniques and design procedures are more established and developed in the literature as compared to the fiber-reinforced soil. It could be argued that additional experimental, analytical, and theoretical work is needed to first understand and then predict the behavior of fiber-reinforced soils in order to advance suitable design/implementation methods.

1.3 Applications of Fiber-reinforced Soils

The use of discrete fibers for the purpose of reinforcing soils is not new. This ground improvement option, which was limited to few applications, has been recently generating more interest in the geotechnical engineering community, with a wider range of fiber types and potential engineering applications that include:

a) General slope stabilization and localized repair of shallow slope failures where fiber-reinforced soils present the advantage of allowing construction within irregular shapes and constricted areas (Figure 1.4 and Figure 1.5)

Figure 1.4 Repaired slope using Geofibers (from GEOFIBERS job report – Riverview drive slope reconstruction)

Figure 1.5 Lake Ridge parkway slope repair project (from Gregory, 2006)

b) Improving the performance of soil veneers such as landfill covers.

c) Construction of embankments (Figure 1.6). Fiber-reinforced soils have been used successfully on more than 50 embankment slopes in the United States between the year 1990 and 2006 (Gregory 2006).

Figure 1.6 Fiber-reinforced soil embankment, president George Bush Turnpike project (from Gregory, 2006)

d) Use within backfill material behind retaining walls to improve their characteristics and reduce the lateral earth pressures and associated displacements.

e) Strengthening of problematic soils to withstand overlying foundations.

f) Increasing the resistance of the pavement subgrades and structural layers.

g) Improving the seismic performance of geo-structures that benefit from gained ductility and strength.

1.4 Research Objectives and Scope of Work

A review of the literature indicates that most previous research studies were based on triaxial tests and studied the undrained behavior of the clay samples. Although undrained conditions generally govern the bearing capacity in the short term, there is a need to check whether the design of the fiber-reinforced system is stable in the long term after pore pressures dissipate as a result of drainage. Current design procedures for problems involving foundations or slopes that are supported or built with fibers lack a systematic approach for quantifying the effect of drainage and accounting for it in design. In addition, studies pertaining to the investigation of drained behavior on natural clay reinforced with natural fibers are scarce and limited.

The main objective of this research is to address the limitations found in the literature by investigating the practicality of using natural Hemp fibers in improving the response of natural clays under fully drained conditions. In this study, conventional drained triaxial tests will be conducted to study the performance of natural clay specimens that are reinforced with Hemp fibers under different confining pressures and different moisture and fiber contents. The parameters that will be varied in the study are the confining pressure (20, 100, and 200 kPa), compaction water content (14% 18% 20%) and Fiber content (1% 1.25% 1.5%).

1.5 Research Significance and Local Context

This study explores the use of natural “Hemp” fibers for improving the drained load response of compacted clays. This solution provides a possible alternative in some specific conditions and applications to more costly and environmentally unfavorable traditional alternatives such as complete replacement of the soft soil layer, vibro-replacement, grouting, soil mixing, geosynthetics, etc. These latter options consume energy during their manufacturing and/or implementation processes, exploit raw materials, and emit harming gases. Moreover, validating the effectiveness of sustainable Hemp-reinforced clay as a soil reinforcement scheme will lead to an increase in the demand for the industrial Hemp encouraging the Lebanese farmers to grow this plant as a substitute of the illegal drug species. Further, studies show that Hemp farms are beneficial to the environment since Hemp does not require any pesticides or chemical fertilizers, it replenishes the soil with nutrients and nitrogen, produces a lot of oxygen, and controls erosion of the top soil. Hemp would also be a cheap alternative compared to other types of fibers. A preliminary feasibility study set by the United Nations Development Program and the Lebanese Ministry of Agriculture shows that the cost of cultivating industrial Hemp is \$79 per 1000 m^2 of non-irrigated land while the products (seeds, stalks, fibers) are worth about \$192 (Awwad 2011). Industrial Hemp may also serve several other industries such as paper, textiles, clothing, biodegradable plastics, body products, health foods, bio-fuel, in addition to applications in concrete (Awwad 2011). Hemp-reinforced clay is considered to be a sustainable building material since it is expected to: (i) satisfy strength requirements and replace other environmentally unfavorable solutions; (ii) reduce energy and resources depletion; (iii) improve the economic and social conditions of rural/farming communities.

1.6 Research Methodology

The work in this dissertation is comprised of a comprehensive experimental testing program that consists of consolidated drained (CD) triaxial tests that were performed on control clay and Hemp-reinforced clay specimens. The main target behind the experimental program was to compare the drained response of hemp-reinforced clay that is compacted using the conventional impact Standard Proctor compaction methods with the undrained response of the same material done by Abou Diab et al. (2016). The target will be achieved by conducting an extensive study on the parameters (fiber content, water content, and confining pressure) that could affect the response of compacted clays that are reinforced with hemp fibers.

1.7 Thesis Outline

The thesis is divided into eight chapters. Chapter 1 is an introduction on the topic of fiber-reinforced clay, and a summary of the research objectives and methodology. Chapter 2 presents background and literature review on fiber-reinforced clay. Chapter 3 details the material, sample preparation, and presents an overview of the experimental program. Chapter 4 explains the triaxial testing setup and procedures. Chapters 5, 6 and 7 respectively discuss the results of the experimental investigation. Chapter 8 concludes the research and gives recommendations.

CHAPTER 2

LITERATURE REVIEW

2.1 Relevant Literature - General Context

The potential use of discrete fibers in applications involving the reinforcement of soils is garnering more interest in the geotechnical community. Fiber-reinforced soils have been used successfully on more than 50 embankment slopes in the United States between the year 1990 and 2006 (Gregory 2006). This interest in fiber reinforced soils has generated a number of research initiatives and experimental testing programs aimed at investigating the effect of adding discrete fibers on the shear strength and compressibility of soils.

Numerous studies have explored the behavior of fiber-reinforced sandy soils, (Maher and Gray 1990; Michalowski and Cermak 2002 and 2003; Consoli et al. 2005; Chen 2007; Consoli et al. 2007; Michalowski 2008; Sadek et al. 2010; Najjar et al. 2013; etc.). However, the potential for the use of soil-fiber systems in clays is clear and pressing, given that many engineered slopes or sloping geo-engineered system are constructed using clayey soils (Gregory 2006).

The recent relevant literature includes reports and findings from a number of experimental studies that investigated the response of clays reinforced with discrete fibers (ex. Maher and Ho 1994; Nataraj and McManis 1997; Prabakar and Sridhar 2002; Li and Zornberg 2008; Gregory 2006; Kumar et al. 2006; Punthutaecha et al. 2006; Tang et al 2007; Akbulut et al. 2007; Ozkul and Baykal 2007; Abdi el al. 2008; Chandra et al 2008; Attom et al. 2009; Viswanadham et al. 2009; Al-Mhaidib 2010; Jiang et al. 2010; Babu and Chouksey 2010; Amir- Faryar and Aggour 2012; Plé and Lê 2012; Pradhan et al. 2012; Jamei et al.

2013; Maliakal and Thiyyakkandi 2013; Mirzababaei et al. 2013; Qu et al. 2013; Anagnostopoulos et al. 2014; Najjar et al. 2014; Wu et al. 2014 , and Abou Diab et al. 2016.). Previous investigations have shown that fiber inclusion substantially improves the clay's response under both static and dynamic loading conditions.

Gray and Ohashi (1983) targeted in their comprehensive study the effect of fiber reinforcement in cohesionless soils. Moreover, they revealed that under static loading the inclusion of fibers expressively increased the peak shear strength of soils and limited their post peak drop in shear resistance. Furthermore, their study indicated that fiber content, orientation of fibers with respect to the shear surface, and fiber modulus were found to affect the contribution of fibers to increased strength. Gray and A1-Refeai (1986), Gray and Maher (1989), Maher and Gray (1990), and Noor Fatani et al. (1991) have extended the previous study and implemented its knowledge of the fiber/soil mechanism and the parameters affecting their response under static loading conditions.

The research efforts into the behavior of clays reinforced with discrete fibers included several experimental studies. Triaxial tests (TX) and direct shear (DS) tests were conducted in studies addressing the strength of the composite, whereas compaction tests, swelling/shrinkage tests, tensile tests, consolidation tests, hydraulic conductivity tests, etc. targeted other interesting aspects of behavior. The majority of the studies to date involved synthetic fibers, with only a few addressing or utilizing natural fibers.

2.2 Studies involving Clays reinforced with synthetic fibers

Anagnostopoulos et al. (2014) carried out a series of DS tests on sandy silt and silty clay samples reinforced with two different types of polypropylene fibers, added at 0.3% to 1.1% by weight of dry soil. They aimed to evaluate the influence of the fiber mechanical properties, the relative size of the fibers and grains, and the rate of shear on the response of polypropylene fiber-reinforced cohesive soils. The mixture of soil, fibers, and high quantity of water (w=35-40%) was poured inside the shear box and the desired normal stress was applied, then the consolidated specimen was sheared at a rate of 0.001 mm/min for drained conditions, and 1.2mm/min for undrained conditions. Results indicated the presence of threshold fiber content where the improvement in shear strength is maximal. The friction angle was more affected by fiber inclusion than the cohesive intercept. The reinforcement was more effective in undrained conditions, and was independent of the properties of the fibers. The improvement was found to be more pronounced for silty clay because of their smaller grains which resulted in a higher number of fiber-grain contact points.

Mirzababaei et al. (2013) investigated the response under unconfined compression (UC) of two clay soils (C and CB) reinforced with two types of carpet fibers (ABF and GBF) and compacted by impact using standard proctor hammer. The parameters which were varied were the dry unit weight of the composite, the compaction water content and the fiber content (1, 3, and 5% by weight of dry soil).

Results showed that increasing the fiber content caused a drop in the maximum dry density (MDD) and an increase in the optimum moisture content (OMC) of specimens (Figure 2.1). The UCS of specimens prepared at their respective MDD and OMC generally decreased with an increase in fiber content (Figure 2.2). The UCS increased with an increase in the dry unit weight, keeping the moisture content and the fiber content constant (Figure 2.3), whereas an increase in water content caused it to decrease. The UCS increased with fiber content (Figure 2.4) and the failure mode changed from an apparent shear plane to bulging (Figure 2.5) for specimens prepared at the same dry density or at the same water content.

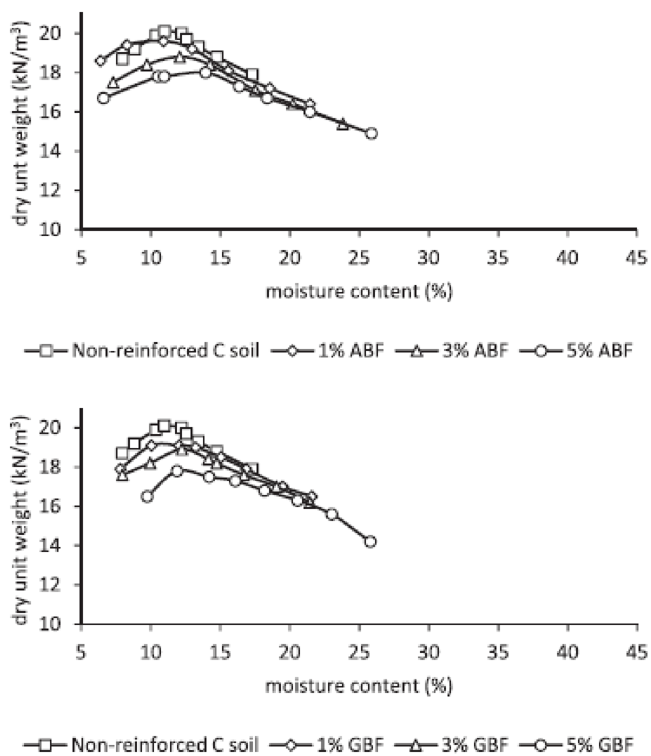


Figure 2.1 Proctor compaction curves for C soil reinforced with ABF/GBF fibers (Mirzababaei et al., 2013)

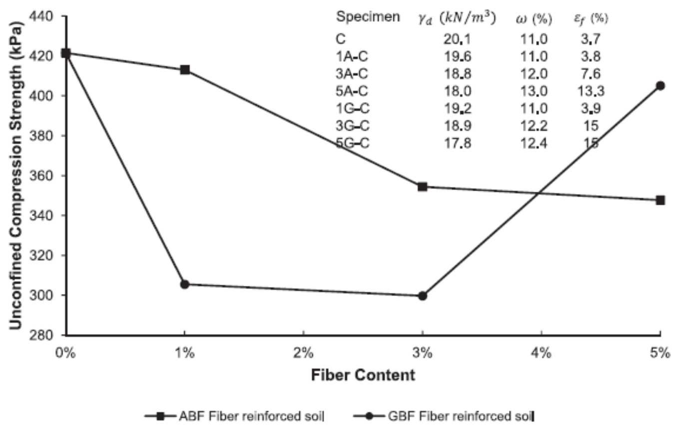


Figure 2.2 UCS of fiber-reinforced C soil specimens prepared at their respective MDD and OMC (Mirzababaei et al., 2013)

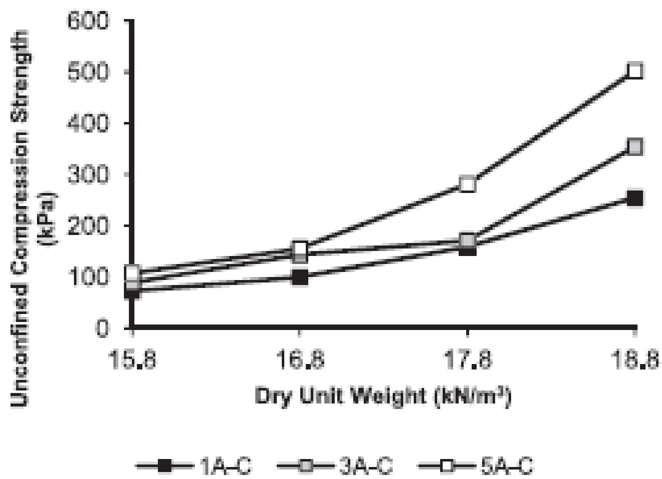


Figure 2.3 Influence of dry unit weight on UCS of fiber-reinforced C soil (Mirzababaei et al., 2013)

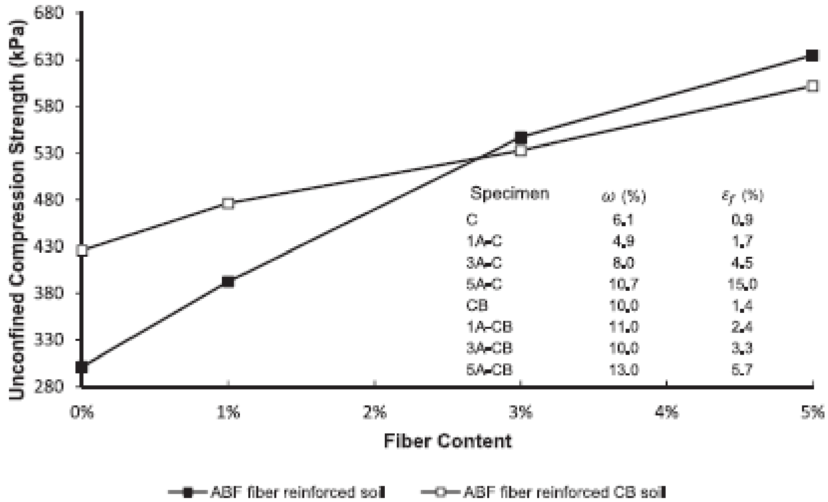


Figure 2.4 UCS of ABF fiber-reinforced C and CB soil prepared at a constant dry unit weight of 17.8KN/m3 (Mirzababaei et al., 2013)

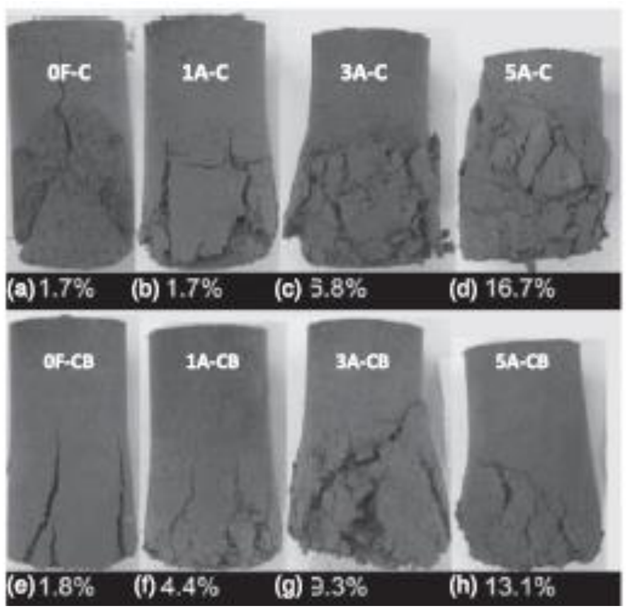


Figure 2.5 Failure patterns of ABF fiber-reinforced C and CB soil compacted at constant dry unit weight. Strain values are indicated. (Mirzababaei et al., 2013)

Plé and Lê (2012) conducted an experimental testing program to assess the feasibility of including fibers to improve the mechanical properties of the compacted clay serving as landfills' cover barriers. They performed a series of UC, TX (UU), and direct tensile tests on unsaturated clay mixed with polypropylene fibers and compacted wet of optimum under standard proctor energy.

Results showed that an increase in fiber content caused a slight increase in the MDD and a decrease in the OMC (Figure 2.6). The strength increased with increasing fiber content and with increasing confining pressure, but the relative improvement seemed be unaffected (Figure 2.7). The initial young modulus increased with fiber content, resulting in greater rigidity (Figure 2.8). The hydraulic conductivity increased with increasing fiber content but stayed within acceptable levels, for fiber contents up to 0.6%.

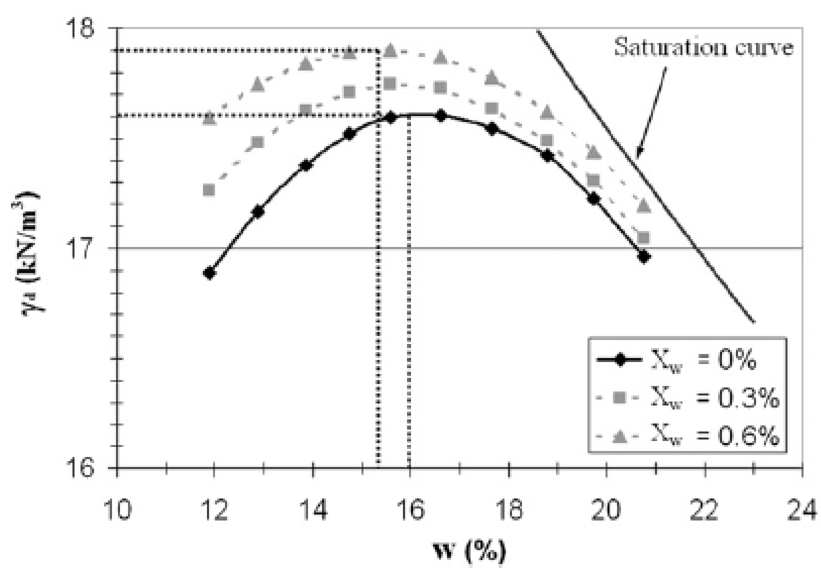


Figure 2.6 Standard Proctor compaction curves for control and reinforced soil (from Plé and Lê, 2012)

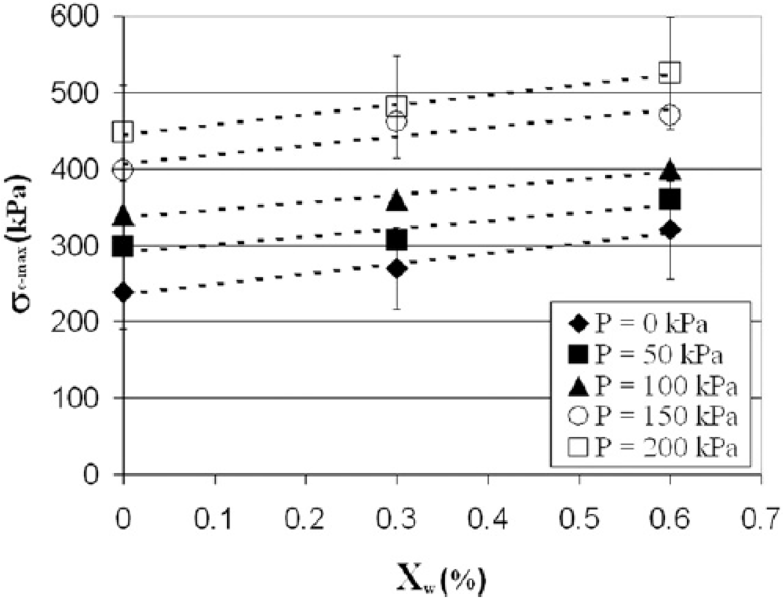


Figure 2.7 Variation of the maximum stress capacity with gravimetric fiber content under various confining pressures, L_f=12mm (from Plé and Lê, 2012)

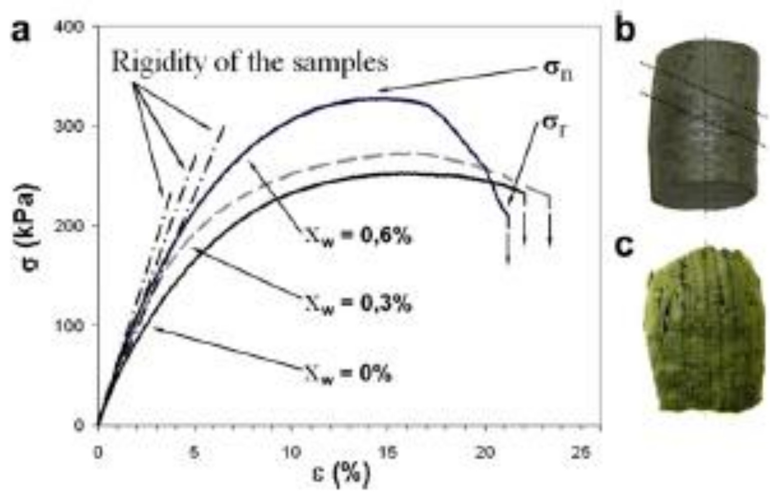


Figure 2.8 (a) Stress-strain curves for UC, (b) unreinforced specimen after test, and (c) reinforced specimen after test (from Plé and Lê, 2012)

Ekinci and Ferreira (2012) conducted a series of CU triaxial tests with pore pressure measurement on compacted, over-consolidated, highly compressible clay reinforced with polypropylene fibers added at a gravimetric content of 0.2%. Figure 2.9 shows the compaction procedure. Samples of unreinforced and fiber-reinforced clay were back-pressure saturated to reach a minimum B-value of 0.98. They were isotopically consolidated to an effective confining stress between 50 and 500 kPa, typical for geotechnical applications, then sheared undrained.

The reinforcement did not prove beneficial in terms of improvement in maximum deviatoric stress (Figure 2.10). Unreinforced samples developed shear planes at high confinement while all reinforced samples bulged. A study of the fibers alignment showed that they were mostly horizontal as a result of the compaction process.

Figure 2.9 Compaction procedure (Ekinci and Ferreira (2012))

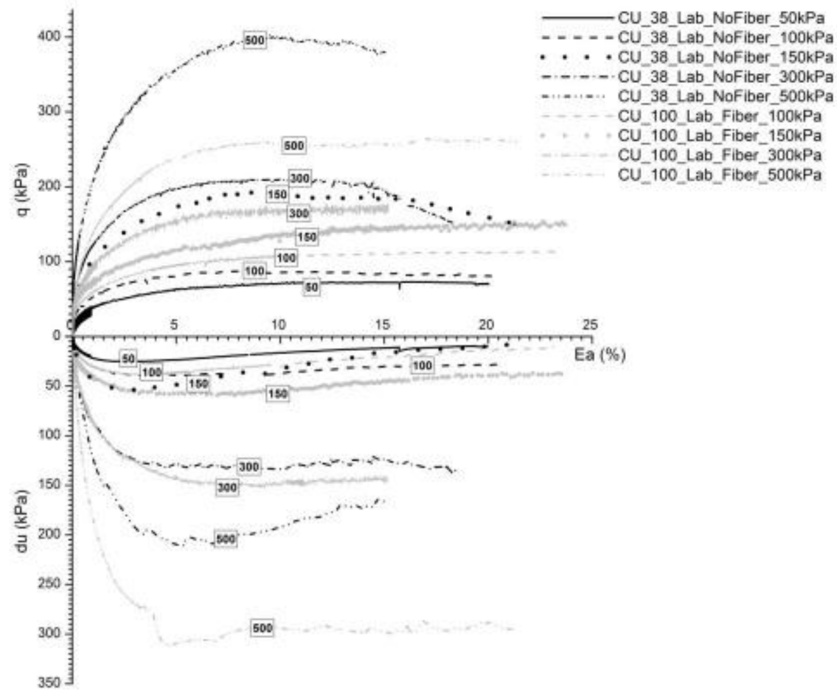


Figure 2.10 Stress-strain and pore-pressure response of clay/FRC samples (Ekinci and Ferreira (2012))

Pradhan et al. (2012) investigated the effect of the inclusion of polypropylene fibers on the strength characteristics of locally available clay by conducting a series of DS, UC and CBR tests. Fibers were added at 0.1-1% by weight of dry soil, with three aspect ratios (η): 75, 100 and 125 corresponding to fiber lengths of 15, 20, and 25 mm. Samples were compacted prior to testing at the optimum moisture content of 11% to the maximum standard proctor dry density of 17.7 KN/m^3.

Results from the direct shear tests showed that the shear strength increased with the fiber content and length up to 0.4% and 20 mm, respectively, after which the improvement decreased (Figure 2.11). Both the angle of internal friction and cohesion increased with increases in fiber content up to the optimum. The improvement in cohesion was more pronounced than that in internal friction angle.The UCS increased with fiber concentration and length up to a gravimetric content of 0.5% and a length of 20mm (η=100), after which the improvement decreased (Figure 2.12). The failure was defined at the peak stress or at that at 10% strain, whichever came first.For the soaked CBR, the optimum fiber properties were 0.8% gravimetric content and 20mm length (η=100).

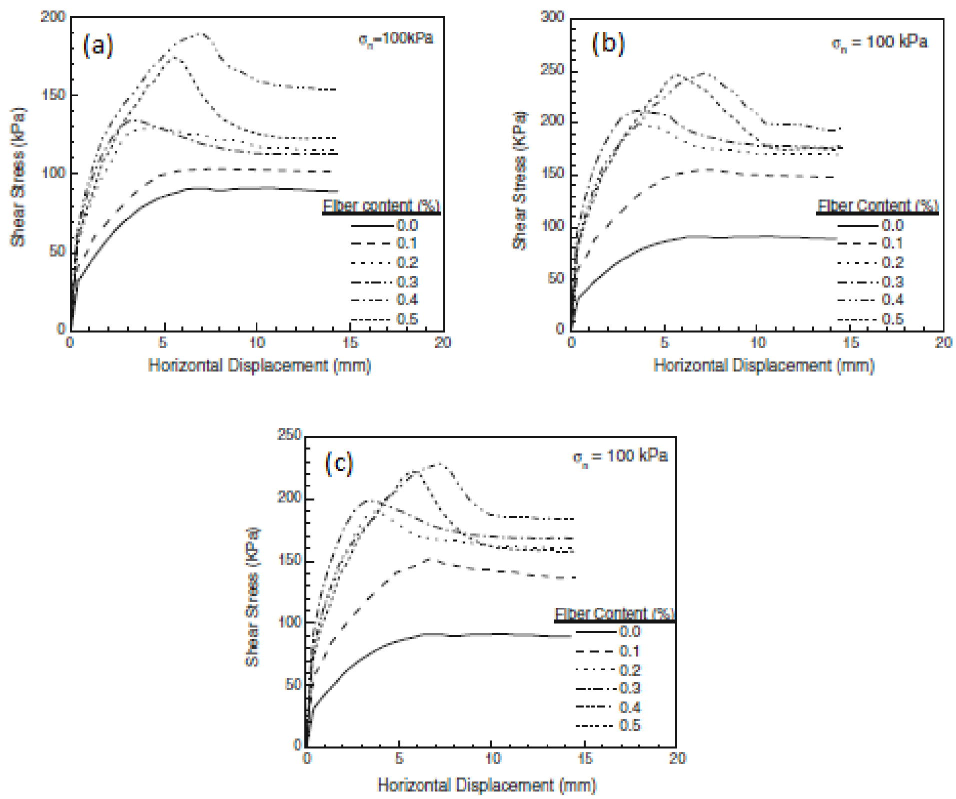


Figure 2.11 Stress-displacement curves for FRC from DS, (a) η=75, (b) η=100, and (c) η=125 (Pradhan et al., 2012)

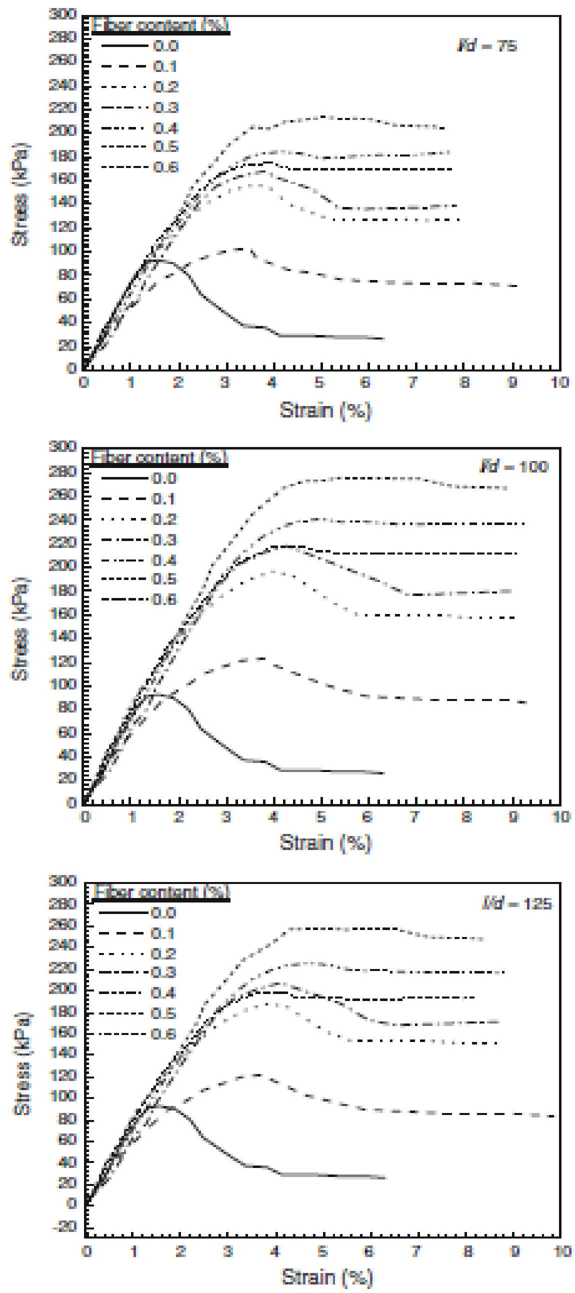


Figure 2.12 Stress-strain curves for FRC from UC tests (Pradhan et al., 2012)

Yang et al. (2011) added polypropylene fibers to "CL" sandy silt. Samples were prepared using a "pressure-based method" at optimum moisture content to 90% MDD and tested in an unconsolidated undrained (UU) triaxial setup. Fiber length was varied between 6, 12 and 19 mm while the gravimetric fiber content was varied between 0.1, 0.2, 0.3 and 0.4 %. The inclusion of fibers affected the cohesion (Figure 2.13) rather than the angle of internal friction (Figure 2.14) of the composite. The cohesion increased with gravimetric fiber content up to a threshold value and decreased afterwards (Figure 2.13). The intermediate aspect ratio was most effective at improving the soil at low fiber contents, while the highest aspect ratio was the most effective at higher fiber contents (Figure 2.13).

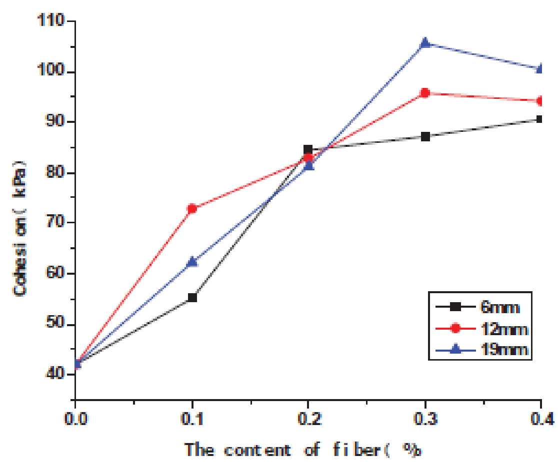


Figure 2.13 Effect of fiber content on the cohesion of clay/FRC specimens (Yang et al., 2011)

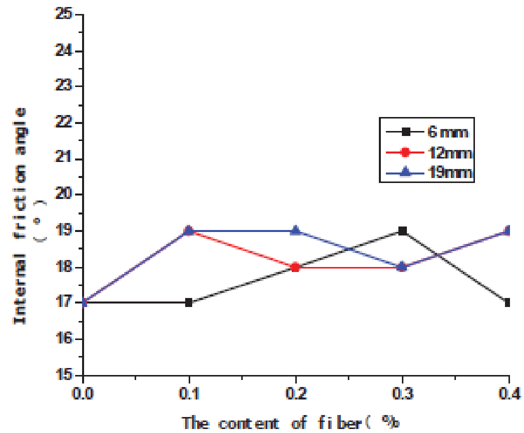


Figure 2.14 Effect of fiber content on the internal friction angle of clay/FRc specimens (Yang et al., 2011)

Maheshwari et al. (2011) conducted a series of UU and consolidation tests on polyester-reinforced highly compressible clay samples, and carried out model tests on square footings supported on such composites. All samples and in-situ soils were compacted at the OMC to the MDD by means of standard proctor compaction energy.

For UU testing, 12 mm long polyester fibers, with an average aspect ratio of 350 were added at various gravimetric contents, i.e. 0.2, 0.5, 1 and 1.5%. The deviatoric stress at failure (Figure 2.15), the percent improvement in deviatoric stress at failure, Cu and φu values increased with increasing fiber content up to a threshold value of roughly 0.5% and decreased afterwards. The percent improvement was more pronounced for the angle of internal friction as compared to the apparent cohesion. The maximal deviatoric stress increased with the confining pressure. The percent improvement was greatest at the low confining pressure of 50 kPa, decreased at 100 kPa confinement and then increased again for 200 kPa confinement.

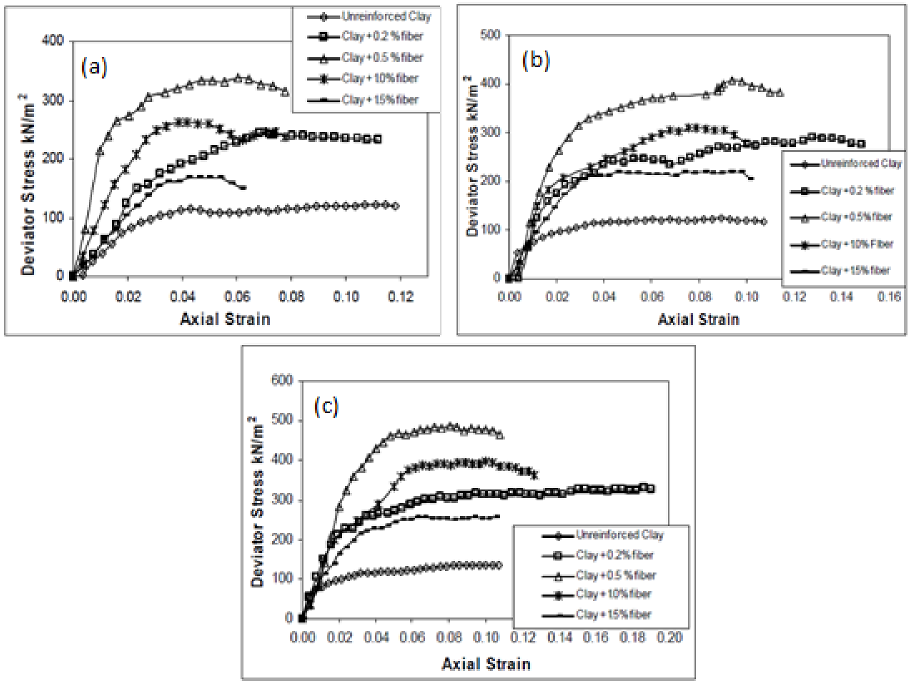


Figure 2.15 Stress strain curves for various fiber contents, (a) σ_3=50kPa (b) σ_3=50kPa (c) σ_3=50kPa (Maheshwari et al., 2011)

Jiang et al. (2010) carried a series of UC and DS tests on a clayey soil reinforced with polypropylene fibers (Figure 2.16). Fibers with different aspect ratios (i.e. length: 10, 15, 20 and 25 mm) and gravimetric contents (i.e. 0.1, 0.2, 0.3 and 0.4%) were added to the raw soil in order to study their influence on the strength of the composite material.

Jiang and his co-workers reported that the UCS and the shear strength parameters exhibited an initial increase followed by a decrease with increasing fiber content and fiber length (Figure 2.17 and Figure 2.18, respectively). The optimal gravimetric fiber content was found to be approximately 0.3% and the optimal fiber length was found to be 15mm.

Figure 2.16 Photo of polypropylene fibers (Jiang et al., 2010)

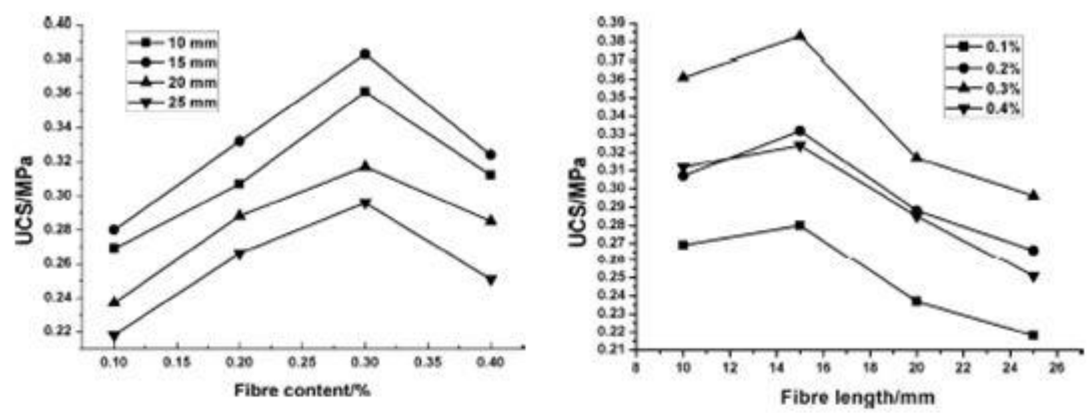


Figure 2.17 Effect of fiber content and fiber length on UCS of fiber-reinforced soil (Jiang et al., 2010)

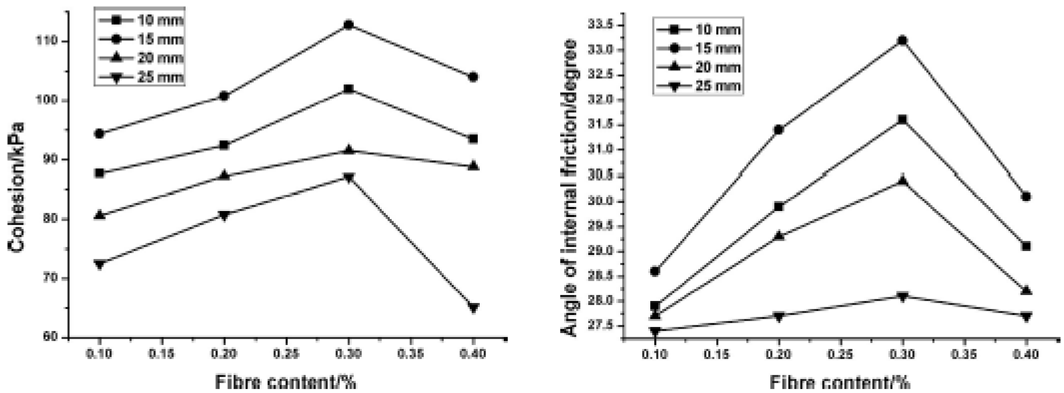


Figure 2.18 Effect of fiber content and fiber length on shear strength parameters of fiber-reinforced soil (Jiang et al., 2010)

Chandra et al. (2008) conducted a series of UC and CBR tests on various cohesive soils reinforced with polypropylene fibers. Polypropylene fibers were added at gravimetric contents of 0.75, 1.5, 2.25 and 3% and their aspect ratio was varied between 50, 84 and 100 corresponding to fiber lengths of 15, 25 and 30 mm, respectively. Specimens were compacted at the OMC to the MDD. The UCS and the CBR values of the reinforced soil increased with increasing fiber content and fiber aspect ratio.

Özkul and Baykal (2007) investigated the effect of fiber-shaped tire buffing inclusions on the shear behavior of silty, low plasticity, Kaolinite clay under both drained and undrained loading conditions. Fiber-shaped tire buffing was added at 10% by weight of dry clay. Control and reinforced samples were compacted at both standard and modified Proctor compaction energy, then CD and CU triaxial tests were conducted at confining pressures varying between 50 and 300 kPa. The MDD of mixtures, compacted at both high and low energy levels, were lower than those of the control clay. This was attributed to the lower specific gravity of rubber. All samples were prepared at water contents 1-2% wetter of their respective optimum values.

Results showed that for the case of the modified effort, the undrained peak shear strength of the composite was higher than the control clay and was reached at lower strains. However, the post-peak strength loss was greater. The failure mode was characterized by strain localization and development of shear planes (Figure 2.19).

The composite samples however, developed their peak shear strength characteristics at strain greater than the control samples, for the following cases: modified compaction effort under drained condition, and standard compaction effort under undrained conditions (Figure 2.20). The failure mode in those cases was ductile and characterized by bulging/barreling (Figure 2.19).

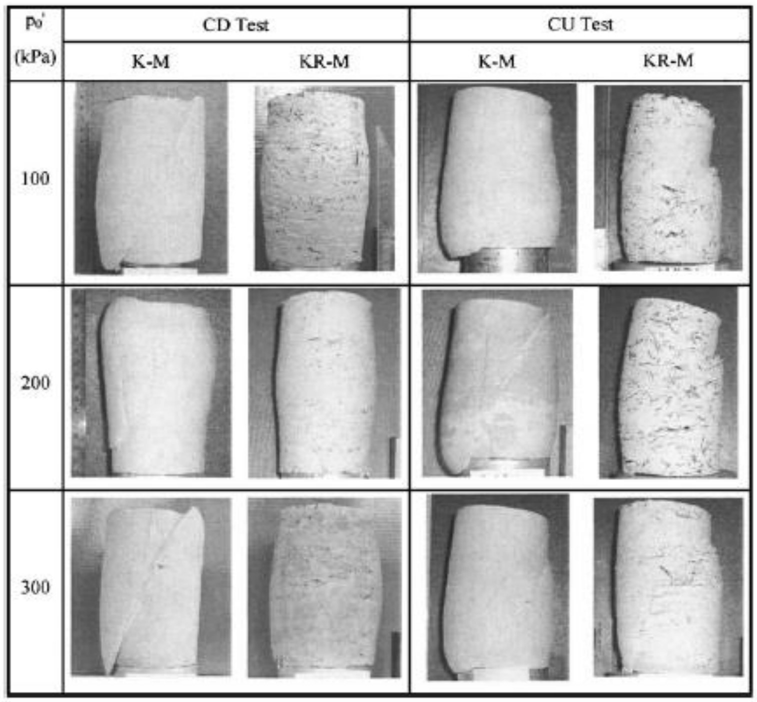


Figure 2.19 Modified energy samples after failure (Özkul and Baykal, 2007).K: kaolinite, KR: reinforced Kaolinite, M: modified

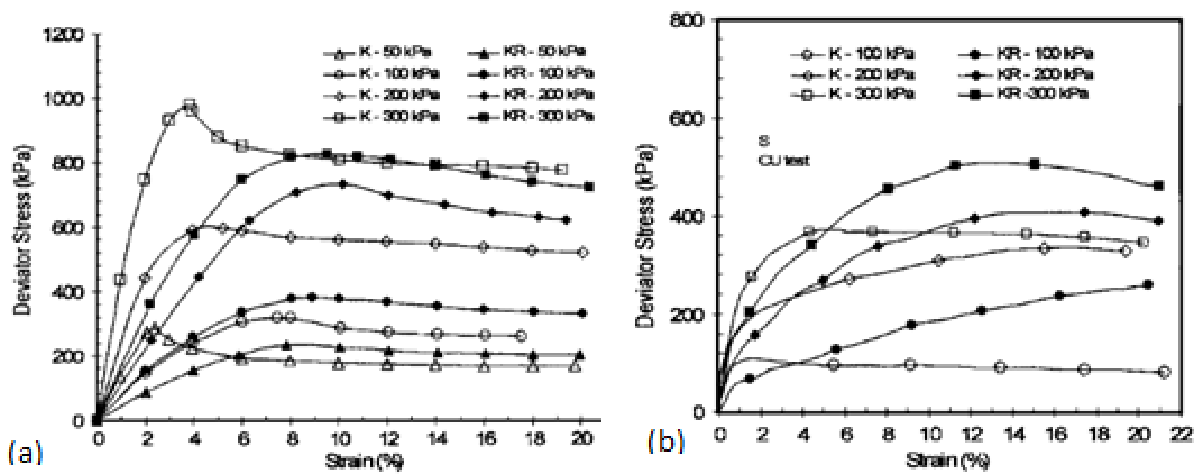


Figure 2.20 Stress-strain curves of (a) CD tests on samples compacted at modified Proctor, and (b) CU tests on samples compacted at standard Proctor (Özkul and Baykal, 2007).

Gregory (2006) proposed a conceptual model to predict the shear strength of the fiber-reinforced soil. He then conducted a series of CU tests on CH clay reinforced with polypropylene fibers for validation. Samples of unreinforced and fiber-reinforced soil were compacted at the OMC to 95% of the MDD using a rod to replicate the action of the sheep's foot roller on site. Results showed a difference between replicates and no significant improvement in the deviatoric stress of the soil following fiber inclusion.

Kumar et al. (2006) examined the effect of polyester fiber content and length on fiber-reinforced highly compressible clay. Fiber content was varied from 0.5 to 2% by weight of dry soil, and fiber length ranged from 3 to 12 mm. Kumar and his colleagues observed that the UCS strength of the clay increased as fibers were added and that the improvement in UCS increased with fiber content and fiber length (Figure 2.21). The effect of fiber inclusions on the value of OMC and MDD was negligible.

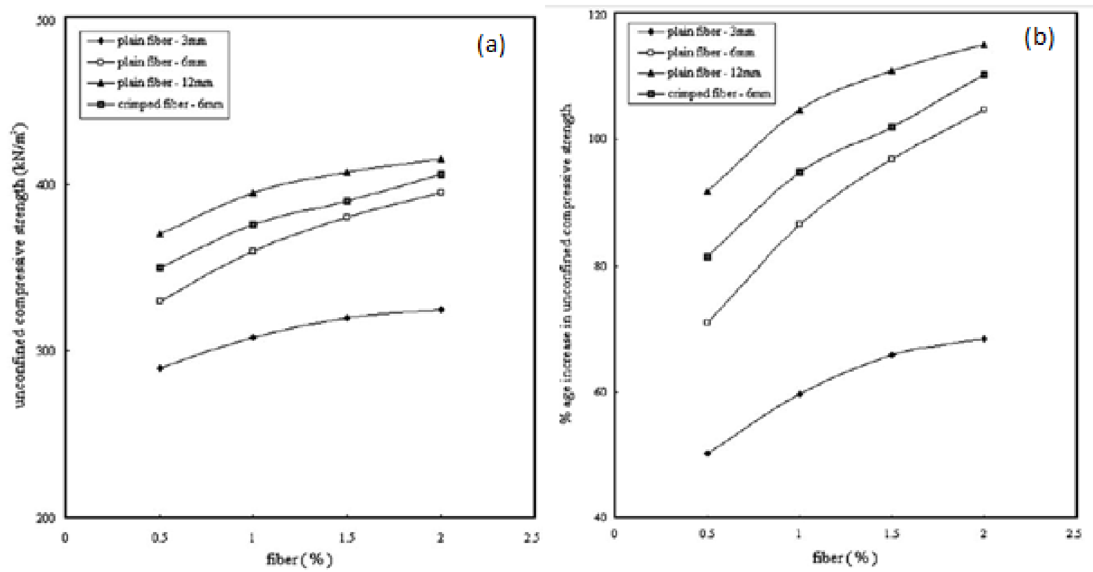


Figure 2.21 Effect of fiber physical properties on (a) UCS and (b) percentage of increase in UCS of clay (Kumar et al., 2006)

Li and Zornberg (2003) and Li (2005) carried out CU tests on different cohesive soils to validate a model proposed by Zornberg (2002) for shear strength prediction of fiber reinforced soils. Samples were compacted to 90% of the MDD of the raw soil prior to testing.

The shear strength of the fiber reinforced soil increased linearly with fiber aspect ratio and fiber content when failure was characterized by the pull-out of fibers. The ductility increased with fiber content and aspect ratio. The inclusion of fibers tended to restrain volume dilation in drained conditions and to increase the positive pore water pressure generated in undrained conditions.

In another separate project, Zornberg et al. (caee.utexas.edu) demonstrated the ability of fibers to minimize post peak shear strength loss of compacted fat clay to be used in blast protection berms throughout a series of CU tests with measurements of pore water pressures (test setup shown in Figure 2.22). Reinforced specimens exhibited bulging instead of localized shear failure bands, indicating an improved ductility (Figure 2.23).

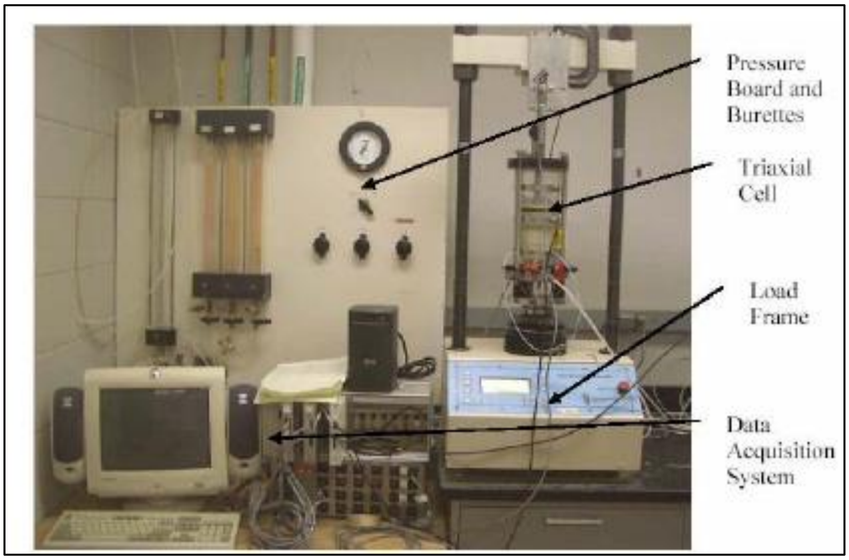


Figure 2.22 Triaxial test setup (Zornberg et al., caee.utexas.edu)

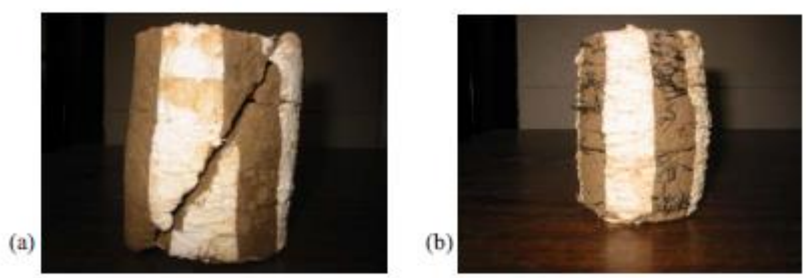


Figure 2.23 (a) Unreinforced and (b) fiber-reinforced specimen after testing (Zornberg et al., caee.utexas.edu)

Ang and Loehr (2003) performed a series of UC tests on specimens of compacted polypropylene fiber-reinforced silty clay at water contents varying from 4% dry to 4% wet of standard proctor OMC. Compaction characteristics did not vary with fiber inclusion. The UCS was found to increase with the fiber content and decrease with the water content (Figure 2.24). The variability of measured strengths was also assessed: it increased with increasing fiber content and decreasing specimen diameter.

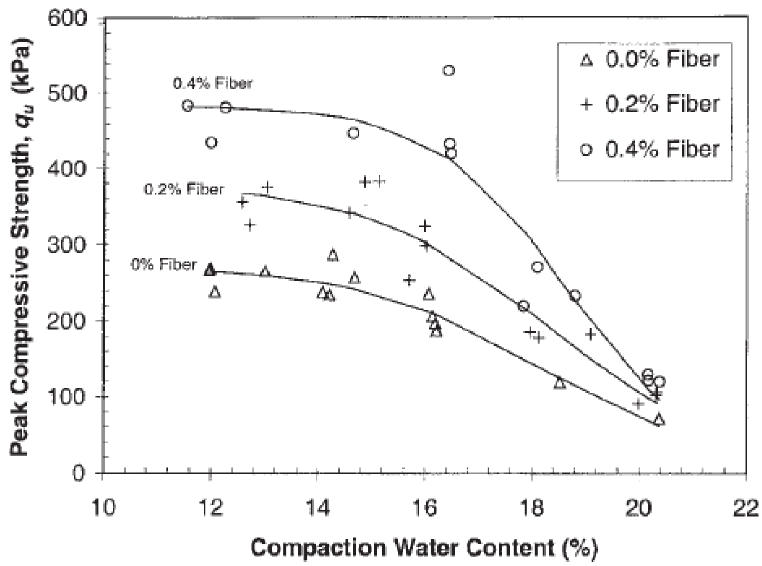


Figure 2.24 Effect of water and fiber content on the UCS of FRC (Ang and Loehr, 2003)

Results from DS, UC, and CBR tests conducted by Nataraj and McManis (1997) on compacted clay samples reinforced with polypropylene fibers indicated that fibers increased the peak shear strength, ductility, peak friction angle, cohesion, compressive strength, and CBR values of the clay soils. Standard Proctor compaction characteristics of the clay were not significantly affected by the inclusion of fibers (Figure 2.25). The UCS of clay-fibers mixtures compacted using the Harvard Miniature apparatus was found to increase with an increase in moisture content up to the optimum water content after which the strengths decreased (Figure 2.26).

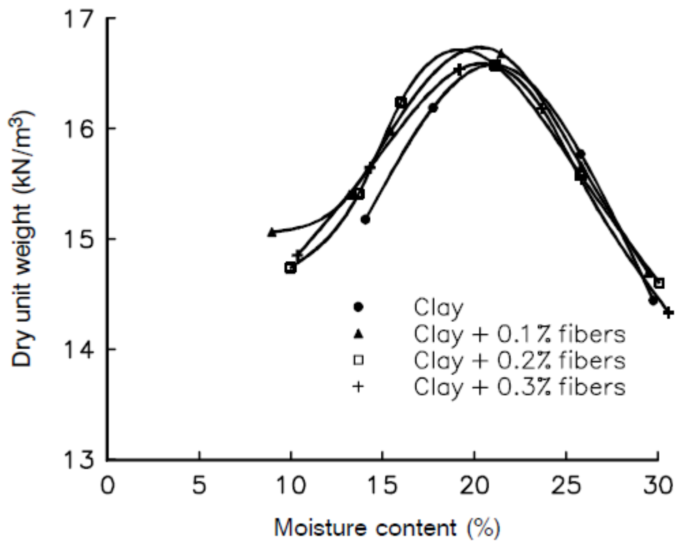


Figure 2.25 Compaction test results for fiber reinforced and unreinforced clay (Nataraj and McManis, 1997)

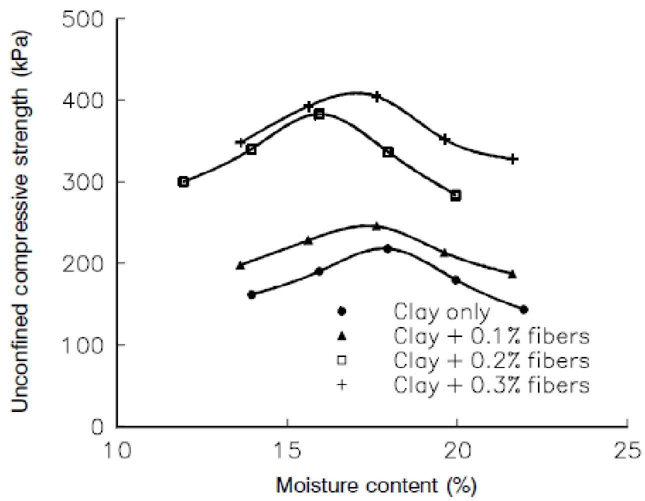


Figure 2.26 Variation of UCS with moisture content for fiber reinforced and unreinforced clay (Nataraj and McManis, 1997)

Alwahab and Al-Qurna (1995) tested silty clay mixed with polypropylene fibers in UC. Specimens were compacted at standard proctor energy for different moisture contents and various fiber lengths and contents. Tested fiber lengths were 5.8, 12.7, 19.2, 24.9 and 49.9 mm, and gravimetric fiber contents were 0.5, 1, 2, 3 and 5%.

Compaction curves showed a slight decrease in the MDD and an increase in the OMC with increasing fiber content. For specimens compacted at the OMC to the MDD of the control clay, the UCS increased with fiber content and fiber length up to threshold values (fiber content of 2% and length of 25mm) after which the improvement slowed or decreased (Figure 2.27). The UCS increased with moisture content up to the OMC after which it started to decrease (Figure 2.28). The inclusion of fibers changed the failure mode from distinct shear plane to bulging.

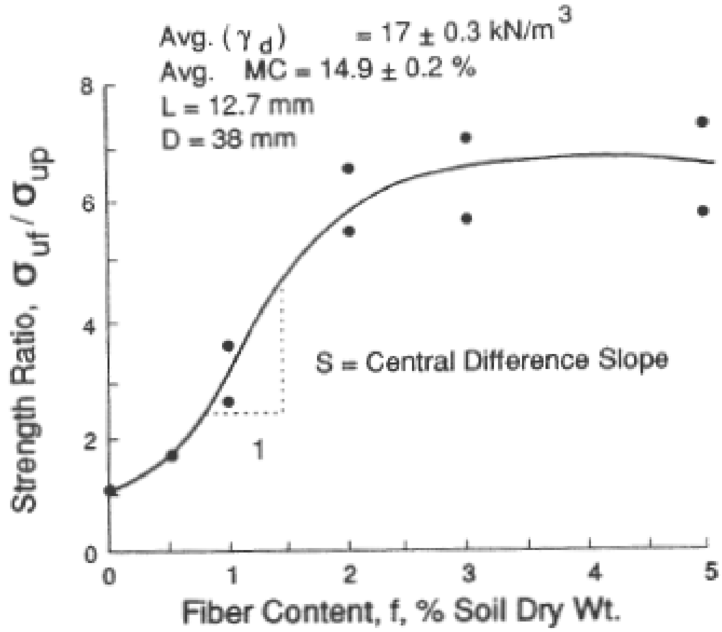


Figure 2.27 Effect of fiber content on the UCS (Alwahab and Al-Qurna, 1995), the strength ratio is the ratio of reinforced to plain soil strength

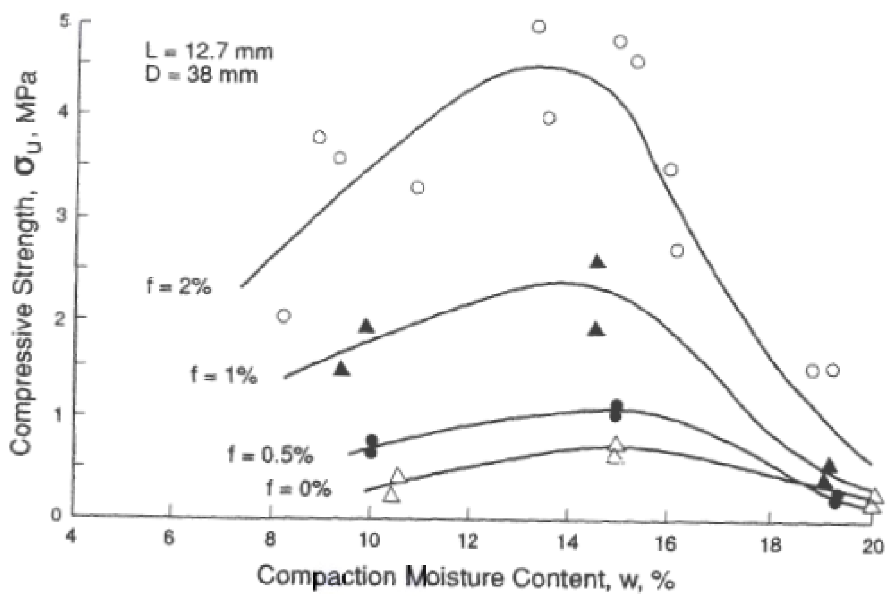


Figure 2.28 Effect of moisture content on UCS (Alwahab and Al-Qurna, 1995)

Maher and Ho (1994) conducted UC, splitting tension, three-point bending and hydraulic conductivity tests on Kaolinite and on different Kaolinite-fiber mixtures prepared by consolidation. Their observations can be summarized as follows: The inclusion of fibers significantly improved the engineering properties of the clay by increasing its peak compressive strength, stiffness, ductility, splitting tensile strength, and flexural toughness; an increase in fiber length reduced the improvement in peak compressive strength and tensile strength but increased the contribution to the ductility of the clay-fiber composite; an increase in fiber content enhanced the improvement in peak compressive strength, tensile strength and toughness of the clay-fiber composite; improvement in UCS and ductility by fiber inclusion was more pronounced at lower water contents (Figure 2.29); increasing fiber content had no significant effect on the compaction characteristics of the soil mixture; and hydraulic conductivity of clayey soil increased with inclusion of fibers.

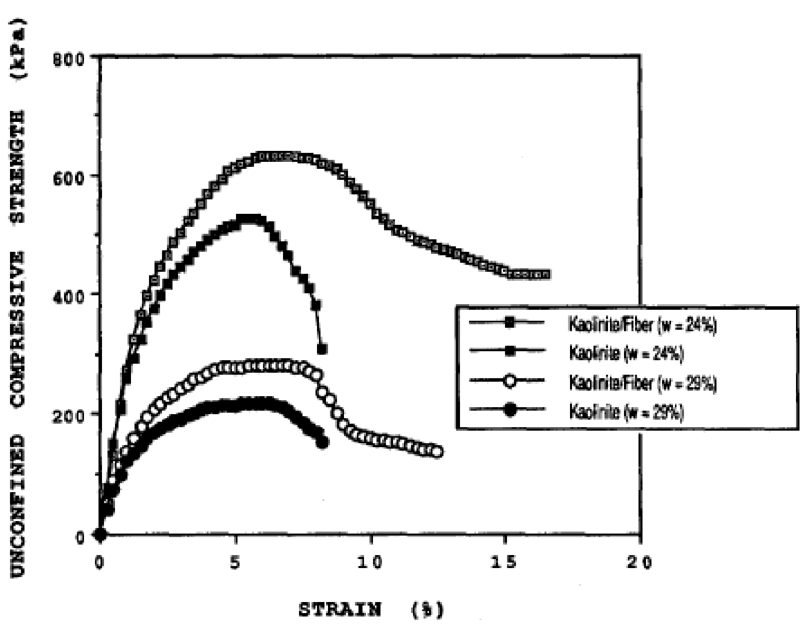


Figure 2.29 Effect of water content on the UCS of Kaolinite/fiber mixtures (Maher and Ho, 1994)

2.3 Studies involving Clays reinforced with Natural Fibers

Najjar et al (2014) assessed the potential use of Hemp fibers to improve the undrained load response of compacted clay by means of UU testing. The water content of the clay/fiber composite was close to the OMC of the raw clay and an equivalent energy corresponding to that of the standard proctor compaction test was applied.

The inclusion of Hemp fibers was beneficial to the ductility of the composite and its undrained shear strength. Improvement in the undrained shear strength increased with fiber content up to a threshold value (between 0.5 and 1% by weight of dry soil) after which no remarkable improvement was observed (Figure 2.30).

The addition of fibers helped in arresting the shear planes and "delaying" the failure (Figure 2.31).

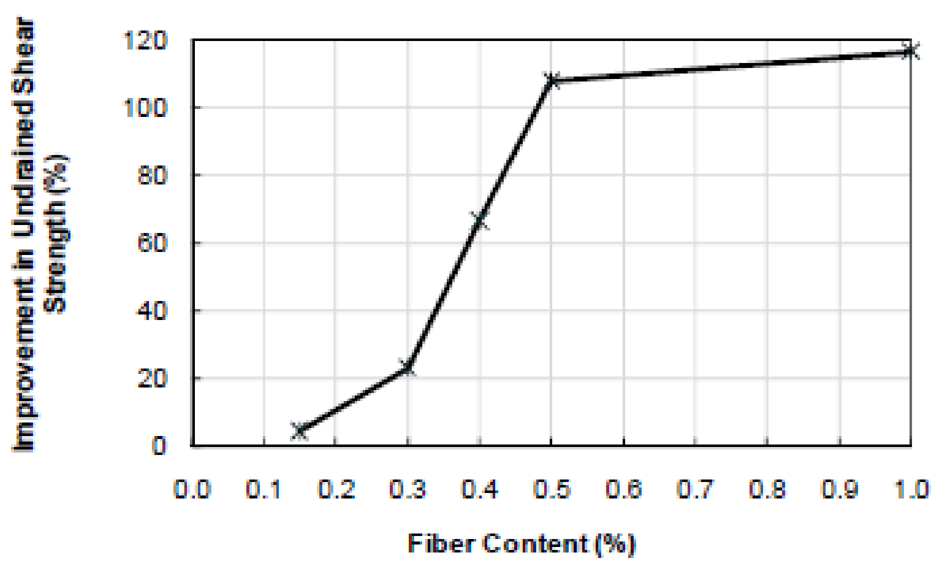


Figure 2.30 Variation of improvement in undrained shear strength with fiber content (Najjar et al., 2014)

Figure 2.31 Illustration of how fibers arrest the shear plane (Najjar et al., 2014)

Wu et al. (2014) used a UU triaxial setup to test CL clay reinforced with sisal fibers. Sisal fibers of various lengths (5, 10, and 15mm) were added at different gravimetric contents (i.e. 0.5, 1 and 1.5%). Samples were compacted at the OMC to the MDD of the control clay as determined from standard compaction test.

The results from those tests showed that the deviatoric stress at failure increased with fiber content and length up to thresholds of gravimetric content and length (fiber content of 1% and length of 10mm, respectively). The improvement was no longer pronounced after the optimum fiber content and it dropped after the threshold aspect ratio (Figure 2.32). The deviatoric stress at failure increased with confining pressure, but no trend was observed for the improvement in the deviatoric stress at failure versus the confining pressure. The value of the cohesion increased significantly with fiber content. The internal friction was not influenced by fiber inclusion.

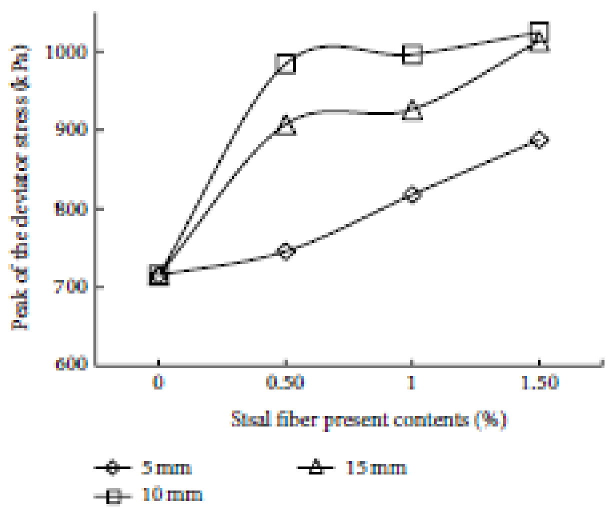


Figure 2.32 Effect of fiber content and length on the peak deviatoric stress of specimens (Wu et al., 2014)

Jamei et al. (2013) carried out a series of UU triaxial tests on clay specimens mixed with sisal fiber. The prepared samples were then subjected to double-side static compaction at OMC to reach the target MDD. No significant changes in dry density- moisture content relationships were noted after fiber inclusion. The inclusion of sisal fibers increased the apparent cohesion of the composite and its internal friction angle. The peak deviatoric stress increased with fiber content and confining pressure .The percentage of improvement in the strength increased with fiber content, but was greater at low confining pressures. Failure was assumed to correspond to 20% axial strain and was governed by the pull-out of fibers. Pull-out tests were also conducted to determine the interface parameters which were found to be dependent on the initial degree of saturation.

Maliakal and Thiyyakkandi (2013) conducted a series of CU triaxial tests on coir (coconut) fiber-reinforced clays. The parameters that were varied during the study were the fiber content, the fiber aspect ratio and the confining pressure. The gravimetric fiber content was varied between 0.5 and 2%. Three aspect ratios were investigated 50, 100 and 150, corresponding to coir fiber lengths of 12, 24 and 36 mm, respectively. Tests were conducted under confining pressures of 25, 50, 100 and 200 kPa. Three different types of clays were used. Samples were compacted to the MDD at OMC using standard proctor energy. The failure was assumed at peak deviatoric stress or at 20% axial strain, whichever was reached first.

Results indicated that the strength envelopes for the reinforced clay were curvilinear, with the transition occurring at the "critical confining pressure". Maximum improvement in strength occurred when the confining pressure was close to the critical value. The peak deviatoric stress increased with increasing aspect ratio and fiber content, with the gain in strength being smaller beyond a certain optimum fiber content and/or aspect ratio (Figure 2.33)

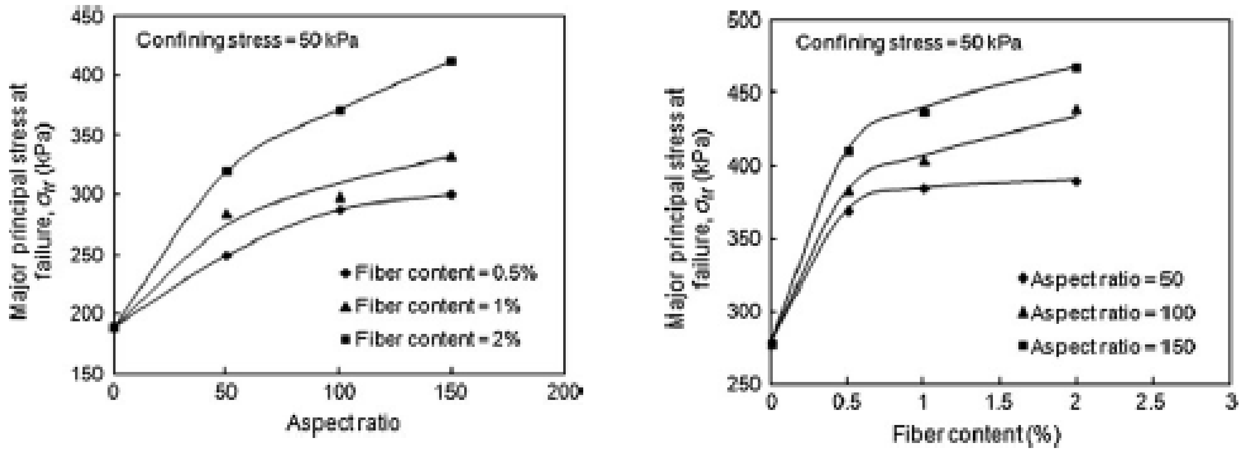


Figure 2.33 Effect of aspect ratio and fiber content on the major principal stress at failure (Maliakal and Thiyyakandi, 2013)

Babu and Chouksey (2010) prepared compacted samples of clay reinforced with coir fiber and tested them in a CU triaxial setup. Samples were compacted by impact at a constant water content to reach a target density. Results from their experimental study showed that the stress-strain response of clayey soils improved with fiber content (Figure 2.34).

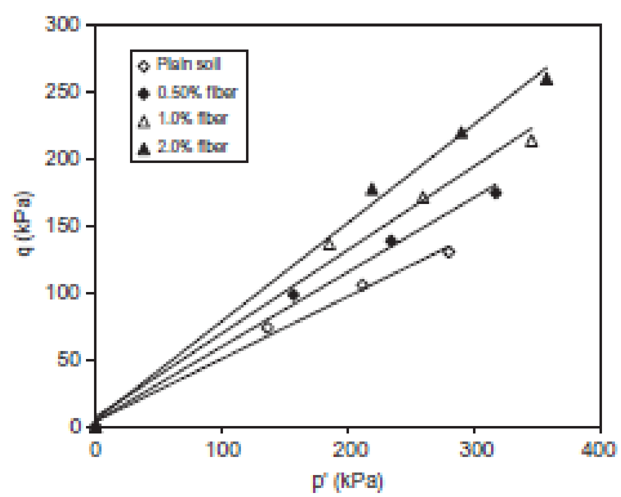


Figure 2.34 p'-q plot for different fiber contents (Babu and Chouksey, 2010)

Prabakar and Sridhar (2002) investigated the effect of sisal fibers on the behavior of CL clay. Fibers were of different aspect ratios, i.e. 40, 60, 80 and 100 (corresponding to 10, 15, 20 and 25 mm long fibers, respectively), and were added at different percentages, i.e. 0.25, 0.5, 0.75 and 1% by weight of raw soil.

Results from compaction tests using I.S. light compaction method (similar to the standard proctor compaction method) showed a decrease in the MDD of the raw soil and an initial increase in the OMC with addition of fibers. Further increase in fiber content or length led to additional decrease in the MDD and to a reduction in the OMC.

For the UU tests, all samples were compacted at the OMC and MDD of the raw soil. The peak deviatoric stress of the reinforced soil increased non-linearly with fiber content and aspect ratio up to threshold values after which they started to decrease (Table 2.1). The improvement in the deviatoric stress at failure was more affected by the change in the fiber content than by the change in the aspect ratio, and was more pronounced at low confining pressures (Table 2.1).

Table 2.1 Failure deviatoric stress and strength ratio (ratio of the shear strength of reinforced soil to that of unreinforced soil), (Prabakar and Sridhar, 2002)

SI. no.	Fibre length (mm)	Fibre content (%)	Failure stress (kPa)			Strength ratio		
1.	0	0	68.37	97.93	117.02	1.000	1.000	1.000
2.	10	0.25	103.99	122.12	142.30	1.521	1.247	1.216
3.	10	0.50	134.48	144.25	164.06	1.967	1.473	1.402
4.	10	0.75	166.41	195.08	221.87	2.434	1.992	1.896
5.	10	1.00	81.52	90.72	107.82	1.192	0.926	0.921
6.	15	0.25	106.79	133.57	146.33	1.562	1.364	1.250
7.	15	0.50	141.59	146.21	164.65	2.071	1.493	1.407
8.	15	0.75	182.55	200.56	224.91	2.670	2.048	1.922
9.	15	1.00	90.45	113.21	123.75	1.323	1.156	1.058
10.	20	0.25	114.39	118.35	151.20	1.673	1.209	1.292
11.	20	0.50	142.00	174.87	155.15	2.076	1.786	1.326
12.	20	0.75	222.21	220.89	261.65	3.250	2.256	2.236
13.	20	1.00	99.93	144.63	135.43	1.462	1.477	1.157
14.	25	0.25	97.50	110.47	124.28	1.426	1.128	1.062
15.	25	0.50	137.42	142.39	150.49	2.010	1.454	1.286
16.	25	0.75	150.88	157.86	165.27	2.207	1.612	1.412
17.	25	1.00	77.71	90.80	95.48	1.137	0.927	0.816
σ_3 (kPa)			69	138	207	69	138	207

Attom et al. (2009) studied three clayey soils (CH, CH, and CL) mixed with palmyra natural fibers and nylon synthetic fibers having an aspect ratio of 75 (30 mm and 15mm length for both fiber types respectively) added at different volumetric contents: 1, 2, 3, 4 and 5%.

Samples were compacted at OMC to the MDD using modified proctor procedure and some samples were prepared at various water contents. The UCS, stiffness and ductility of the composite increased with fiber content and the UCS decreased when the water content increased due to the lubricating effect which reduces load transfer between fibers and clay particles. The improvement in the UCS increased with increasing fiber content (Figure 2.35).

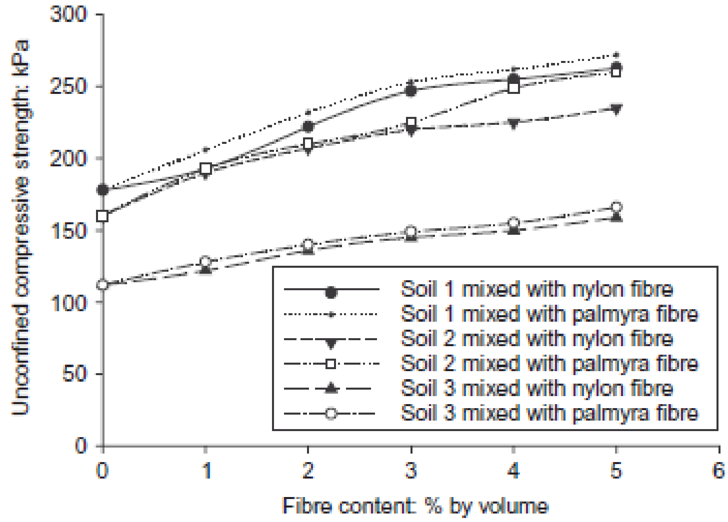


Figure 2.35 Effect of fibers on UCS of the three soils (Attom el al., 2009)

Yankai et al. (2014) performed unconsolidated undrained triaxial tests on control CL clay samples and samples reinforced with sisal fibers after being compacted. Results show that the deviatoric stress increased as increasing fiber content up to a threshold gravimetric content and length. After the optimum fiber content the improvement was no longer pronounced and it dropped after the threshold aspect ratio. As increasing the confining pressure, the deviatoric stress increased, but no trend was observed. The value of the cohesion increased significantly with fiber content for a given fiber length. The internal friction was not influenced by fiber inclusion.

Andersland and Khattack (1979) explored the response of Kaolinite clay reinforced with pure cellulose pulp fibers. The outcomes of the conducted triaxial tests revealed that fiber inclusions increased the peak strength of kaolinite under several testing conditions [(UU), (CU), or (CD)]. Furthermore, with increasing fiber content an improvement in the kaolinite ductility was noticed.

Ahmad et al. (2009) conducted a drained and undrained triaxial test program to evaluate the effect of reinforcing silty sand with oil palm empty fruit bunch fibers (OPEFB).Moreover, OPEFB were coated with acrylic butadiene styrene thermoplastic and the effect of coating was investigated. Results show that coating fibers increases the shear strength of silty sand much more than the uncoated ones. Moreover, coating fibers increases the interface friction between fiber and soil particles thus increasing the surface area.

Ramadan et al. (2017) conducted a short-term durability tests on hemp-fiber confined concrete cylinders after wetting and drying cycles in both water and seawater. Furthermore, they conducted a uniaxial tension test on hemp-fiber bundles to assess the effect of wetting and drying cycles. Concerning tensile testing results, the hemp-fiber bundles were totally ruined, after a prolonged exposure to water. Even more, epoxy coating was a suitable configuration to protect hemp-fiber bundles. Resistance to seawater was highly apparent in the conservation of tensile stresses.

In the most recent study, Abou Diab et al. (2016) conducted a comprehensive laboratory program consisting of 18 UU Triaxial tests. In this study, Samples of natural clay mixed with hemp fibers at different fiber content (from 0.5% to 1.5%) were prepared at different moisture content (14% 18% and 20%) and compacted using the standard proctor procedure. Outcomes revealed that adding fibers results in a significant improvement in the shear strength of up to 100% improvement with respect to the control samples. Moreover, this improvement increases with increasing the fiber content up to an asymptotic value of fiber content (1.25%) and this improvement also depends on the water content used in compaction.

2.4 Summary of Findings from the Experimental Literature Review

The aforementioned studies and others conducted on fiber-reinforced clays point to some commonalties in the findings in some aspects of behavior and to inconsistencies in other aspects. Some of the findings which are relevant to the objective of our study are summarized below:

- The inclusion of fibers in clays generally leads to an increase in strength.

- For a given aspect ratio (fiber length/diameter), the strength increases with fiber content up to certain fiber contents beyond which the improvement in strength reaches an asymptotic upper limit (Maliakal and Thiyyakkandi 2013; Najjar et al. 2014) or starts to decrease (Pradhan et al. 2012; Jiang et al. 2010; Prabakar and Sridhar 2002; Akbulut et al. 2007). Decay in shear strength gain is hypothesized to occur at high fiber contents because of a larger percentage of fiber-to-fiber rather than fiber-to-soil contacts (Gregory 2006).

- For a given fiber content, an increase in the fiber aspect ratio leads to higher strengths up to certain values of aspect ratio beyond which loss of strength is observed (Prabakar and Sridhar 2002; Akbulut et al. 2007; Jiang et al. 2010; Pradhan et al. 2012) or stability in strength (Maliakal and Thiyyakandi 2013). The ineffectiveness of the fibers at relatively high aspect ratios is attributed to several factors including the flexibility of the fibers, the inability to achieve proper mixing during sample preparation, and the tangling and kinking that is expected to occur for thin/long filament fibers.

- The effect of fiber inclusion on the stiffness of the FRC is not conclusive. Some researchers observed that incorporating fibers into clay increases its stiffness (Abdi el al. 2008; Attom et al. 2009; Plé and Lê 2012; and Najjar et al. 2014) while others observed reductions in the initial stiffness of the reinforced soil with the addition of fibers (Tang et al. 2007; Chandra et al. 2008; Jamei et al. 2013). The possible loss of stiffness could be attributed to reductions in the dry density and the existence of a non-uniform distribution of voids in some FRC compared to the control unreinforced clay (Chandra et al. 2008).

- Discrete randomly distributed fibers have been found to be effective at restraining the swelling tendency of expansive soils. Both swell percentage and swelling pressure decrease as the fiber content increases (Punthutaecha et al. 2006; Abdi el al. 2008; Viswanadham et al. 2009; Al-Mhaidib 2010). This reduction in volumetric strain can be attributed to the replacement of swelling clay particles by non-swelling fibers and to the resistance offered by fibers through clay–fiber contact (Viswanadham et al. 2009).

- The impact of fibers on the compaction characteristics is not conclusive. Some studies show that the addition of fibers leads to a decrease in the MDD and an increase in the OMC (Ozkul and Baykal 2007; Al-Mhaidib 2010; and Mirzababaei et al. 2013). Others indicate an opposite behavior with an observed increase in dry density in the presence of fibers (Nataraj and McManis 1997 and Plé and Lê 2012). The drop in the dry density is generally associated with the replacement of a percentage of soil solids with fibers of relatively low specific gravity (Prabakar and Sridhar 2002). Faryar and Aggour (2012) report that during the compaction of FRC, fibers will initially fill voids between the soil solids increasing the dry density. But with increasing the fiber content, fibers start to separate individual soil solids rather than fill the voids resulting in a reduction of the dry density. Some studies reported a null effect of fiber inclusion on the compaction characteristics of the soil (Kumar et al., 2006).

- Limited experimental programs addressed the influence of the saturation of the clay and the compaction characteristics on the effectiveness of the fiber in increasing the shear strength and stiffness. In the majority of the published studies, FRC samples were compacted at, or near, the optimum moisture content. In the few studies that tested clays that were compacted away from optimum, the improvement in the mechanical properties of the clay-fiber mixtures was found to be more pronounced in relatively unsaturated FRC that were prepared at lower compaction water contents (ex. Maher and Ho 1994; Attom et al. 2009; and Mirzababaei et al. 2013). It is hypothesized that the increase in strength due to the addition of fibers is less prominent in clays compacted at higher water contents due to the lubricating effect of water which reduces fiber/clay load transfer.
- Inclusion of fibers increases the ductility of the composite with the improvement being greater at higher fiber contents and aspect ratios. Acquired ductility enhanced the energy absorption capacity of the composite. It is manifested by the reduction in the post-peak strength loss and the increase of strain at failure or strain hardening behavior.
- Fiber inclusion enhances the tensile strength of the clay and thus impedes the development of desiccation cracks resulting from the clay shrinkage once it is subjected to drying (Ziegler et al. 1998 ; Tang et al. 2012)
- The hydraulic conductivity of the clay increases with fiber content but does not exceed the allowable limit for clay liners in the range of applicable fiber contents (Miller and Rifai 2004; Abdi et al. 2008)

2.5 Limitations of Experimental Work on FRC

Our investigation of the published experimental studies on FRC lead to the identification of the following possible limitations:

The first limitation is that the majority of the studies on FRC experimented with synthetic/manufactured fibers. Synthetic fibers typically used include polypropylene, polyester, glass, rubber, and polyvinyl fibers.

Recent studies have started investigating the potential use of natural fibers such as coconut fibers, bamboo fibers, cane fibers, palm fibers, sisal fibers, Hemp fibers, etc. to reinforce clayey soils (Hejazi et al. 2012). However, these endeavors are still in their early stages .Prabakar and Sridhar (2002), Jamei et al. (2013) and Wu et al. (2014) conducted unconsolidated undrained triaxial tests on clays that were reinforced with sisal fibers, while Babu and Chouksey (2010) and Maliakal and Thiyyakkandi (2013) conducted consolidated undrained triaxial tests on clays that were reinforced with coir fibers. The remaining three studies by Attom et al. (2009), Qu et al. (2013), Najjar et al. (2014) and Abou Diab et al. (2016) used unconfined compression, direct shear, and unconsolidated undrained triaxial tests conducted on clays reinforced with Palmyra, wheat straw, and Hemp fibers, respectively.

Most previous research studies that were based on triaxial tests studied the undrained behavior of the clay samples, because this behavior generally governs the bearing capacity in the short term. However, the assumption of fully undrained conditions may not apply in the long term.

Current design procedures for problems involving foundations on soft clay deposits that are reinforced with natural fibers lack a systematic approach for quantifying the effect of drainage and accounting for it in design. In addition, studies pertaining to the investigation of partially drained behavior on natural clay reinforced with natural fibers are scarce and limited.

The findings from the experimental studies point to some inconsistencies in some aspects of behavior of the FRC (the stiffness of the composite, existence of threshold fiber content/length, etc.). These studies also lack a systematic investigation of the parameters that are believed to influence the behavior of the FRC (compaction water content, confining pressure, etc.).

Thus, it could be safely argued that the experimental work that is published in the geotechnical literature on the behavior of clays that are reinforced with natural fibers is limited and needs to be supplemented with additional tests.

2.6 Natural Fibers Overview

Natural fibers are mainly composed of cellulose, hemicellulose, and lignin. They are harvested from the stem (flax, Hemp, jute), leaves (sisal), or seeds (coir). Their production is "low cost" and involves basic stages: plant growth, harvesting, decortication (separation of fibers from non-useful biomass), and supply. They are generally considered as waste material and their use saves on energy and natural resources.

A point that is consistently raised about the use of natural fibers is in reference to their potential degradation in the long-term, and the resulting effect on their mechanical properties. However, most of the reported durability studies in the literature to date were carried out on natural fibers that were fully exposed to severe conditions of weathering and water absorption, which may have accelerated their biodegradation by fungus and bacteria. There are no durability-related studies conducted on fibers embedded in a soil matrix (partial exposure). The soil sealing the fibers may reduce the extent and/or rate of their deterioration as testified by the clay bricks reinforced with natural straw and/or other fibers that were long used as earth materials for construction purposes.

In the long term applications targeted in this dissertation, any possible "long-term" degradation in the properties of natural fibers is expected to be of high relevance to the results. Nevertheless, the use of proper coatings and certain types of fiber modification (bleaching, alkalization, or silanes) are proven to be efficient solutions that may ensure the long term effectiveness of natural fibers (Dittenber and GangaRao 2012).

CHAPTER 3

MATERIALS AND SAMPLE PREPARATION

3.1 Introduction

Based on the literature review presented in Chapter II of this thesis, it is obvious that the mechanism that governs the behavior of fiber-reinforced clayey soil is not fully understood. Such an understanding is required in order to further develop standard practices and design methodologies for geotechnical systems that involve fiber-reinforced clay (FRC).

The experimental program in this research study was designed to supplement the limited data available in the literature on the strength and deformation properties of clay reinforced with natural fibers. In compliance with sustainability criteria, industrial Hemp was the fiber of choice. The main scope is to study the long term stability conditions of the FRC system. As such, the "consolidated drained" triaxial test was adopted as it provides a more realistic testing technique to predict the response.

In this chapter, the materials used in the testing program are described in detail, including the natural clay and natural Hemp fibers. The characterization tests conducted on the clay included specific gravity, grain-size analysis, Atterberg limits and standard proctor compaction tests. Both sieve analysis and hydrometer tests were conducted to establish the grain size distribution. Abou Diab et al. 2016 conducted Tensile strength tests on the Hemp fibers to determine their strength and stiffness properties.

An experimental testing program that is based on consolidated drained (CD) triaxial testing was designed to determine the strength of the Hemp-reinforced soil for various test conditions and reinforcement schemes. The parameters that were varied in the experimental program were (1) the compaction water content of the clay, (2) the fiber content, and (3) the applied confining pressure.

In what follows, a detailed description of the sample preparation and the adopted testing procedures is presented. The tests conducted in the experimental testing program are also presented along with their corresponding testing parameters/conditions and reinforcement schemes.

3.2 Test Materials

The materials used in this experimental study are a clayey soil termed henceforth "natural clay", and Hemp fibers .Their properties are presented in the following sections.

3.1.1 Natural Clay

The soil used in this experimental study was excavated from a construction site in kfarselwan, Lebanon. Specific gravity, sieve analysis, hydrometer, Atterberg limits and standard Proctor compaction tests were conducted to characterize the soil. Standard procedures proposed by the American Society for Testing Materials (ASTM) were implemented. The index properties of the soil are presented in Table 3.1 and the grain size distribution curves are presented in Figure 3.1. The soil is classified as inorganic clay of low plasticity (CL) as per the unified soil classification system (USCS). The clay has a plasticity index of 14% and a fines content of around 55%. The clay fraction within the fines is around 35%. The results of the standard Proctor test (Figure 3.2) indicate a maximum dry unit weight of around 16.8 KN/m^3 and optimum moisture content ~19%.

It should be noted that although the soil classifies as “clay”, it falls on the boundary between clay and silt on the USCS classification chart and includes a high percentage of sand. The effect of the presence of sand in the soil is expected to be clearly exhibited in several aspects of behavior including shear strength and permeability. The choice of this soil was intentional as it represents natural non-“ideal” and/or “synthetic” materials.

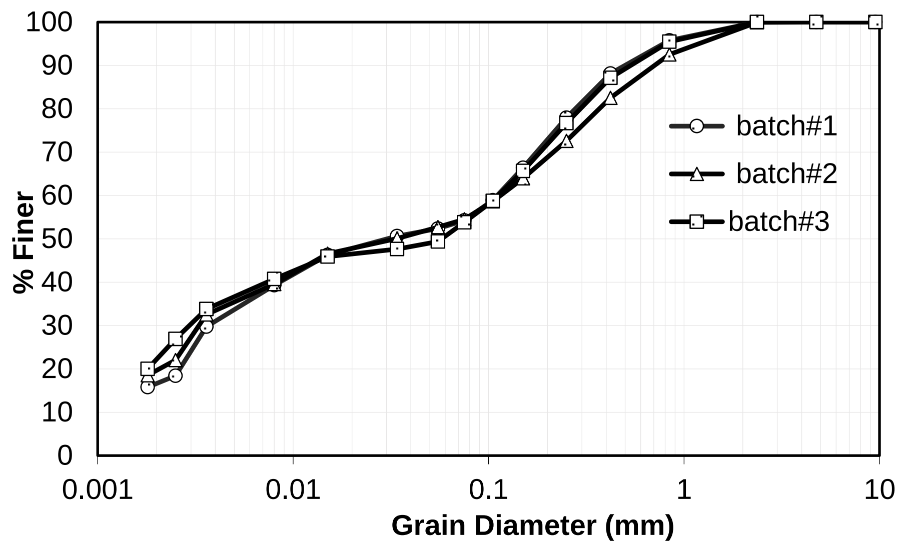


Figure 3.1 Grain size distribution curves of the natural clay

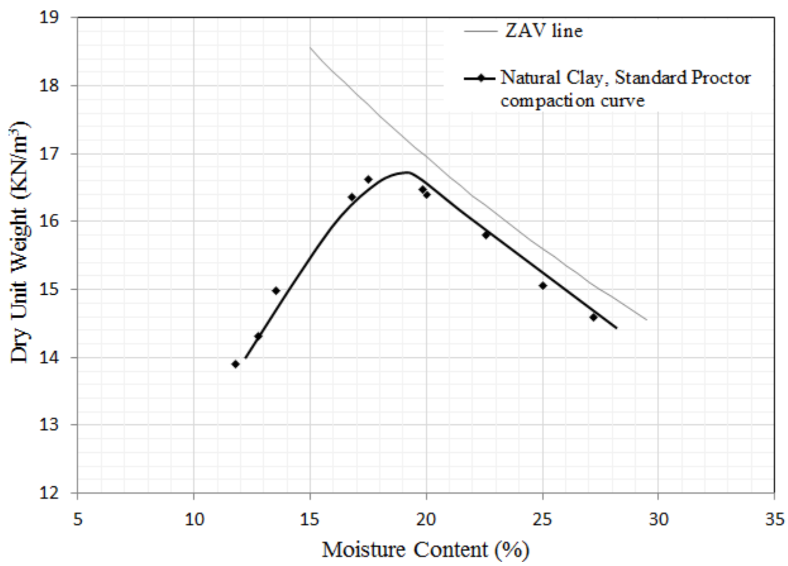


Figure 3.2 Standard Proctor compaction curve of the natural clay

Table 3.1 Material properties of the natural clay (Abou Diab et al. 2016)

	Batch #		
Clay	**1**	**2**	**3**
Liquid Limit,	35	33	34
Plastic Limit,	22	19	20
Plasticity	13	14	14
Specific	2.64	2.64	2.64
Sand, %	46	46	46
Silt, %	37	35	32
Clay, %	17	19	22
% Fines	54	54	54
Classification,	CL	CL	CL

3.1.2 Natural Hemp Fiber

Industrial Hemp (Figure 3.3) is used throughout this study. It is a variety of the Cannabis sativa plant species. As it is obvious from its taxonomy, industrial Hemp is specifically cultivated for industrial/commercial uses such as paper, textiles, rope, clothing, “biodegradable” plastics, paint, insulation, biofuel, food, construction materials, etc. Industrial Hemp approved for production secretes only minute amounts of the psychoactive drug THC (tetrahydrocannabinol). THC is the chemical responsible for most of marijuana's psychological effects. Typically, Hemp contains below 0.3% THC, while Cannabis grown for marijuana can contain anywhere from 6 or 7% to 20% or even more.

Industrial Hemp is favorably cultivated in temperate zones where it matures rapidly in about three to four months. Industrial Hemp is produced in many countries around the world, with Canada, France, and China being the main producers. In the United States, Hemp is mainly imported: industrial Hemp seems to face difficulty expanding as it faces challenges in regulations, traditional marketing and sales approaches (hempethics.weebly.com).

Industrial Hemp fibers were supplied from the Hemp Traders- USA (www.hemptraders.com) in the form of raw long fiber processed from the stalk (Figure 3.3b). The fibers were soaked in a sodium hydroxide solution (NaOH) at 6% by weight for 48 hours in order to remove all organic impurities. After treatment, the fibers were washed with water and left to dry. The fibers were then cut manually to lengths dictated by the predefined experimental program (Figure 3.3a).

Visual inspection of the Hemp fibers indicates that they have a rectangular cross section with relatively uniform thickness of about 0.13mm, and a relatively variable width. A statistical analysis of a random set of 250 fibers indicated an average width of 0.65mm and a standard deviation of 0.42mm (Figure 3.4).

The mechanical properties of the fibers were determined in the laboratory by Abou Diab et al. (2016). The tensile strength of the fibers was determined by carrying out tests on twenty randomly chosen Hemp fibers. The fibers were subjected to a pullout force by fixing one end and applying a tensile force to the other end through a clamp connected to a digital force gauge. The tensile strength of the fiber was calculated by dividing the peak rupture force recorded on the screen of the digital force gauge by the cross sectional area of the fiber. As a result, the ultimate tensile strength was found to have an average of 276 MPa with a standard deviation of 66MPa and a range varying from 181 to 415 MPa.

Moreover, the secant modulus of elasticity for another set of twenty fibers was determined using the same setup. These fibers were initially marked near their fixed end and a ruler was laid parallel to their axis in order to measure their elongation while applying the pull-out force. For each increment of the pull-out force recorded by the digital force gauge, the cumulative elongation was measured until rupture of the fiber. Based on these readings, a stress-strain curve was generated for each fiber and its modulus of elasticity was determined as the slope of the linear fitting to this curve. Consequently, the modulus of elasticity of the fibers was found to have an average value of 21.7 GPa with a standard deviation of 3.87 GPa, and a range varying from 16.1 to 29.8 GPa (Abou Diab et al. 2016). The properties of the Hemp fibers are summarized in Table 3.2.

Figure 3.3 Hemp fibers (a) raw and (b) cut

Table 3.2 Index properties of the Hemp fibers.

Fiber property	Value
Specific gravity	1.4
Ultimate tensile strength, MPa	276 (mean)
Elastic Modulus, GPa	21.7 (mean)
Thickness, mm	0.13
Width, mm	0.65 (mean)
Equivalent diameter, mm	0.41

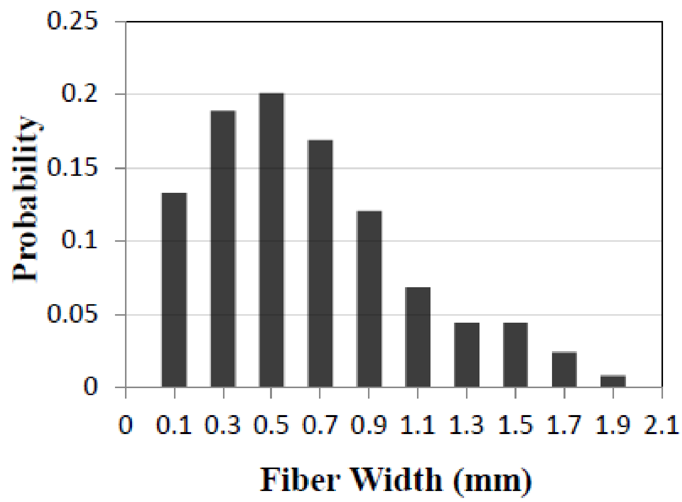


Figure 3.4 Probability distribution of the Hemp fiber widths (Abou Diab et al. 2016).

3.3 Overview of the Program and Sequence of Tests

Given the perspective and scope of the work presented in this dissertation, mainly addressing the long term response of FRC, the samples were tested in a consolidated drained (CD) triaxial setting (ASTM D7181-11). The CD test setup allows for an effective quantification of the degree of improvement in the drained shear strength of the compacted clay and clay/fiber composite for applications involving long-term stability of geotechnical systems (slopes, landfill covers, foundations, etc.). In this study, conventional drained triaxial tests were conducted to study the performance of natural clay specimens that are reinforced with Hemp fibers under different confining pressures and different moisture and fiber contents. The parameters that were varied in the study are the confining pressure (20, 100, and 200 kPa), compaction water content (14% 18% 20%) and Fiber content (1% 1.25% 1.5%). More details regarding the procedures and schedules of the three series of tests are presented in the following section.

The testing program that was adopted included three series of tests. The experimental program of testing is presented in Table 3.3, Table 3.4 and Table 3.5. In the first series of tests which included both control tests (not reinforced) and tests conducted on clay specimens that are reinforced with hemp fibers, the reinforced specimens are prepared at a fiber content of 1.25% and reinforced using 4cm long fibers. The choice of these fiber-reinforcement parameters is based on the results of Abou Diab et al. (2016) who showed that the optimal fiber-reinforcement scheme for undrained loading conditions consisted of 4cm long fibers and 1.25% fiber content by weight. The natural soil and natural fibers adopted in Abou Diab et al. (2016) will be identical to those adopted in our study.

Table 3.3 Triaxial soil testing program (Series 1) – Drained

Test No.	Confining pressure σ_3, (kPa)	Hemp fibers %	Drained	Target water content	Fiber length (mm)	Compaction Method
1	20	0	D	20%	40	Static
2	20	1.25%	D	20%	40	static
3	20	0	D	18%	40	static
4	20	1.25%	D	18%	40	static
5	20	0	D	14%	40	static
6	20	1.25%	D	14%	40	static
7	100	0	D	20%	40	Static
8	100	1.25%	D	20%	40	static
9	100	0	D	18%	40	static
10	100	1.25%	D	18%	40	static
11	100	0	D	14%	40	static
12	100	1.25%	D	14%	40	static
13	200	0	D	20%	40	Static
14	200	1.25%	D	20%	40	static
15	200	0	D	18%	40	static
16	200	1.25%	D	18%	40	static
17	200	0	D	14%	40	static
18	200	1.25%	D	14%	40	static

The objective of the first series of tests is to characterize the drained stress-strain behavior, volume change behavior, shear strength, and stiffness of the “natural clay” when reinforced with hemp fibers under different test conditions. All samples have a diameter of 7.2 cm and a height of 14.5cm and are tested at confining pressures of 20, 100 and 200kPa.

In a second series of drained tests, specimens reinforced with 4cm long fibers but at different fiber contents (smaller and greater than 1.25%) are tested to investigate whether the optimum combination of fiber length and content is affected by the drainage conditions. To limit the number of tests, this series of tests is restricted to specimens compacted at the water content of 18% (Table 3.4).

Table 3.4 Triaxial soil testing program (Series 2) – Drained

Test No.	Confining pressure σ_3, (kPa)	Hemp fibers %	Drained	Target water content	Fiber length (mm)	Compaction Method
19	20	1.00%	D	18%	40	Static
20	20	1.50%	D	18%	40	static
21	100	1.00%	D	18%	40	Static
22	100	1.50%	D	18%	40	static
23	200	1.00%	D	18%	40	static
24	200	1.50%	D	18%	40	static

The third and final series of tests is conducted to investigate the durability of the hemp when used in soil reinforcement. In these tests, specimens of fiber reinforced clay are prepared then sealed with Nylon sheets and covered with PVC tubes and left for a specific period of time in a control room prior to conducting Triaxial tests/ Tensile strength tests for fibers. The results of such tests are compared to identical tests, which were tested in the conventional approach (series 1) to portray any detrimental effect of time on the response of the specimens (Table 3.5).

Table 3.5 Triaxial soil testing program (Series 3) – Drained

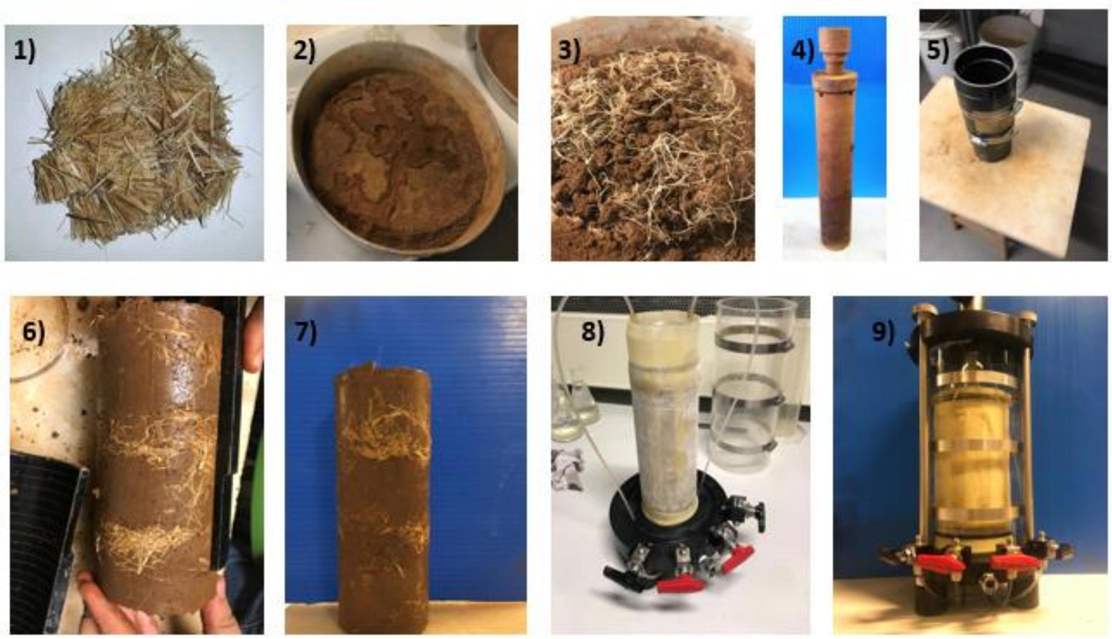
Test No.	Hemp fibers %	Drained	Target water content	Fiber length (mm)	Compaction Method	Age
D0 (Prev)	1.25%	D	18%	40	Static	0 days
D1	1.25%	D	18%	40	Static	15 days
D2	1.25%	D	18%	40	static	30 days
D3	1.25%	D	18%	40	Static	60 days
D4	1.25%	D	18%	40	static	90 days
D5	1.25%	D	18%	40	static	120 days
D6	1.25%	D	18%	40	static	150 days

3.4 Specimen Preparation

The specimen preparation steps are presented and described in this section. The clay was brought to the laboratory in a naturally wet state from a local construction site. It was oven dried, crushed, and sieved through a number 10 sieve. The dry clay was initially mixed with water and the fibers were then added gradually and mixed manually to ensure a homogenous mix with a random distribution of fibers. The mixture was then sealed and left aside for three to four hours to allow for water content homogenization.

The mix was then transferred to a split mold with a height of 210 mm and an internal diameter of 72.5mm and compacted in equal batches/layers with the tamper (impact). The exact diameter of the specimen was dictated by the size of the base cap in the triaxial cell. The two sections of the mold were lubricated from the inside and held in place using aluminum steel rings to ensure that lateral strains are negligible during compaction.

After compaction, the specimen was extracted from the split mold, trimmed to the required length (usually L/D ratio from 2 to 2.5), and assembled in the triaxial cell. The advantage of using a lubricated split mold was to ensure that the compacted clay/FRC specimen could be removed from the mold with minimal disturbance. For the range of fiber lengths considered in this study, the length to diameter ratio of 2-2.5 is likely to eliminate any possible fiber size effects in FRC specimens having a diameter of 70mm or greater (Zornberg, 2002; Ang and Loehr 2003; Jamei, Villard, and Guiras 2013).



CHAPTER 4

TRIAXIAL TESTING

4.1 Introduction

This chapter describes the method and steps to be followed in performing consolidated drained tests using the automated triaxial "TruePath" equipment. The step-by-step approach which describes the process from the initial stage of seating the test specimen to the final stage of shearing the specimen under drained conditions is designed to be a guide for future users of the "TruePath" equipment.

4.2 General Steps in Performing Consolidated drained (CD) Tests

After preparing the Natural Clay specimen as described in section 3.4, the triaxial cell (with the sample inside it) is placed in the automated triaxial "TruePath" system. The main components of the system are presented in Fig. 4.1. The "TruePath" system consists of four main parts which are the load frame with pressure transducer and the deformation sensor, the cell pump which provides the confining cell pressure to the cell chamber, the back/pore pump which provides the back pressure for the specimen and measures the pore water pressure through connecting a pressure transducer to valve#3 (as will be explained in a later stage), and the operating system which allows the user to perform the test and monitor its progress through the screen that displays all the stages of the test.

The triaxial test consisted of four stages which include seating, back pressure saturation, consolidation, and shearing. Each stage is characterized by a series Commands that appear on top of the screen and guide the user throughout the test. The four tabs, which represent each stage, become active after specimen and test data files are created. A specific tab representing a specific stage will become active only after the previous stage is completed. The following steps describe the detailed procedure to be followed in performing consolidated drained tests (CD) on consolidated clay samples.

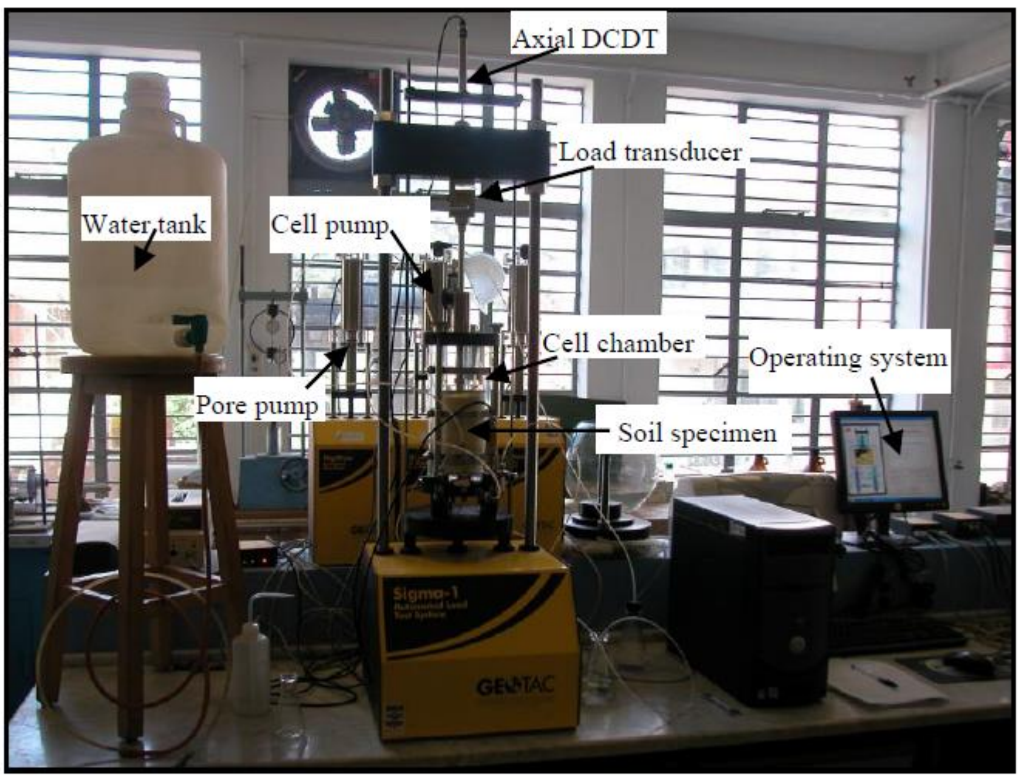


Figure 4.1 Automated triaxial equipment “TruePath”

4.3 Creating Specimen and Test Data Files

In order to view the test results while performing the test, the file called “graph initiative” should be deleted prior to the start of the test from the “TruePath” folder, which is located under the “program files” folder. The first step in performing the CD test involved setting all the sensors and load transducer readings to zero. This can be achieved by entering the “Set Up” menu and selecting “Sensor”. After highlighting the required sensor or transducer and pressing “Test”, a window will appear for the selected sensor. On this window, the “Take Zero” button should be pressed so that the sensor reading will indicate the average of ten consecutive readings that are almost zero. This process should be repeated for all the sensors, i.e. pore pressure, back pressure, cell pressure sensors, external load cell and axial DCDT. Figs. 4.2 to 4.4 show a step by step procedure for setting the sensors to zero readings.

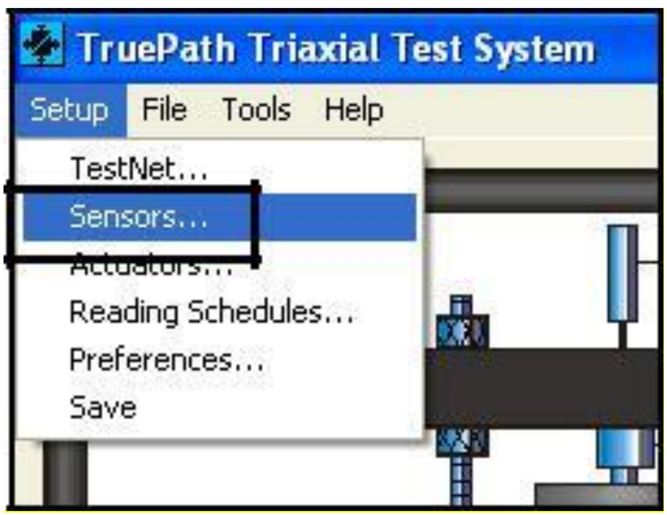


Figure 4.2 Selection of sensor button

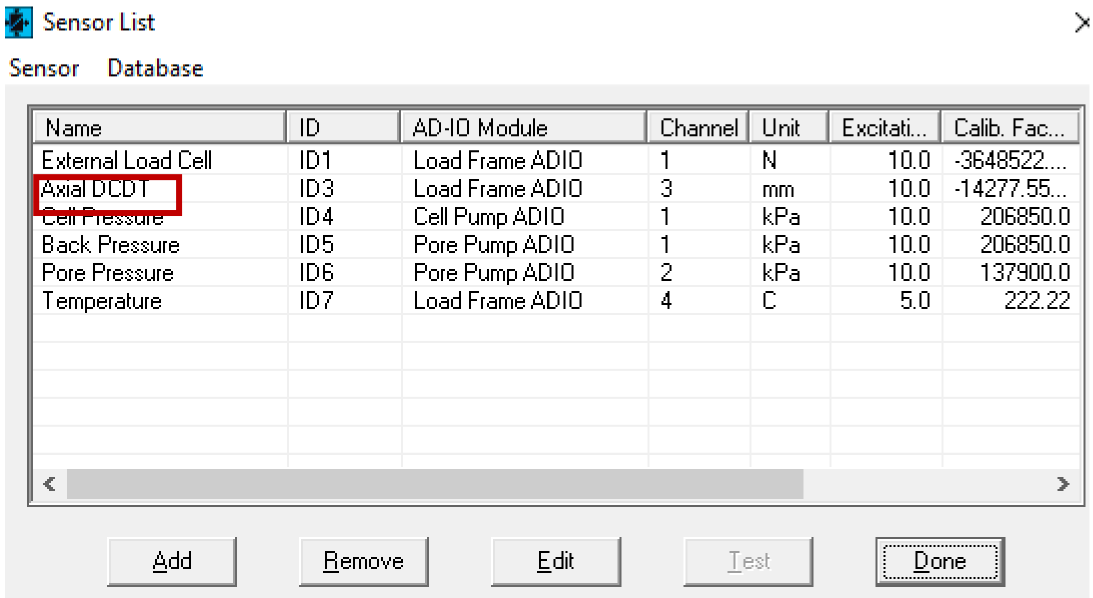


Figure 4.3 Selection of the Axial DCDT sensor

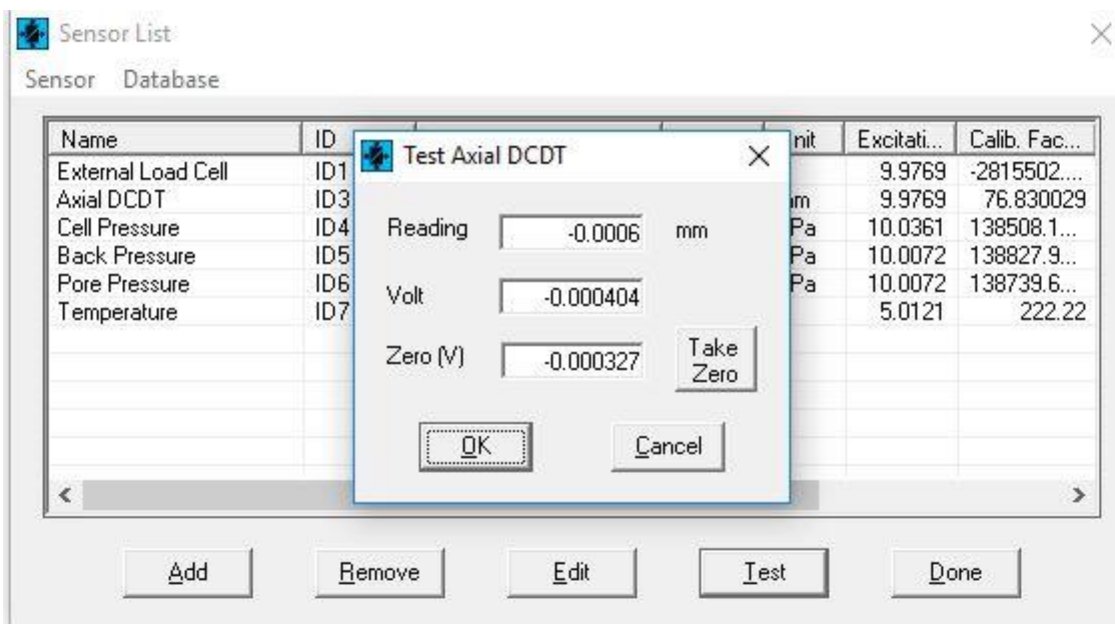


Figure 4.4 Initializing readings for the selected sensors

The second step involved accessing the "File" menu and choosing "Specimen Data" As shown in Fig. 4.5. Then the "specimen data" window will appear as shown in Fig.4.6 where the user has to click each box to fill the appropriate information, which includes the sample height (14.5 cm), sample diameter (7.25 cm), and the sample number and project number. The third step is also initiated from the "File" menu by selecting "Test Data" as shown in Fig. 4.5. A window will appear as shown in Fig. 4.7 where the user has to enter the control test parameters in the empty spaces.

The input data for the test consists of four categories that are included in one window. The user has to enter the following:

- The value of the target seating pressure which is defined as the seating confining pressure needed to keep the membrane pressed against the specimen during the flushing of the drain lines. A Seating pressure of 50 kPa is used for the samples consolidated at 100kPa and 200 kPa while 20 kPa was used for the samples consolidated at 20 kPa in the testing program.
- The value of the saturation/back pressure that is needed to saturate the sample. A pressure of (500 kPa) is chosen for the natural clay sample to ensure proper saturation.
- The type of consolidation (isotropic in this test program) and the value of the target effective stress that is needed to consolidate the sample. The test program involved three different confining cell pressures, 100 kPa , 200 kPa and 20 kPa. The stress rate for the target effective stress was chosen to be (5000 kPa) per hr. to guarantee instantaneous application of the consolidation pressure.
- The drainage conditions which were defined in this testing program to be "consolidated drained" (CD) condition, the loading direction which was chosen to be "compression", the maximum vertical effective stress which was taken as (1036 kPa),

the maximum strain which was taken as 20%, and the strain rate was taken as 0.25%/hr ((for all tests??) please check). It was also chosen that shearing will be terminated when either the maximum stress or the maximum strain is reached.

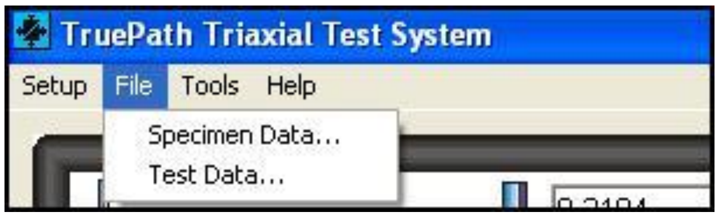


Figure 4.5 Entering file menu to select Specimen Data

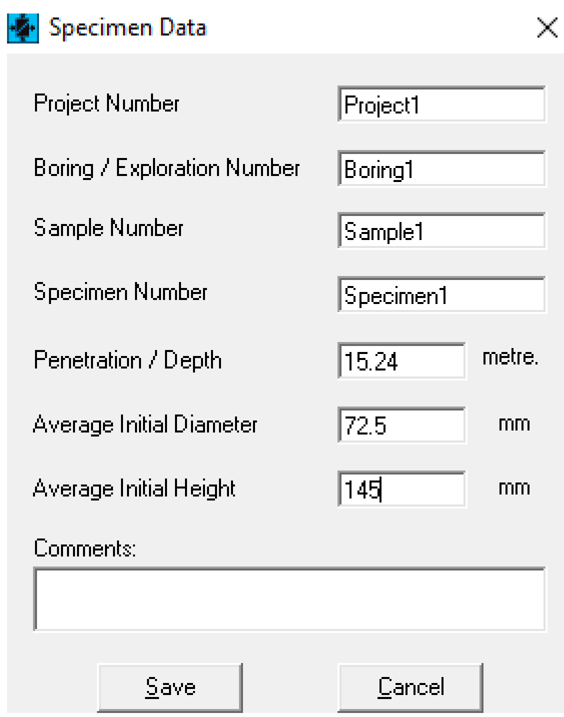


Figure 4.6 Writing the specimen data information

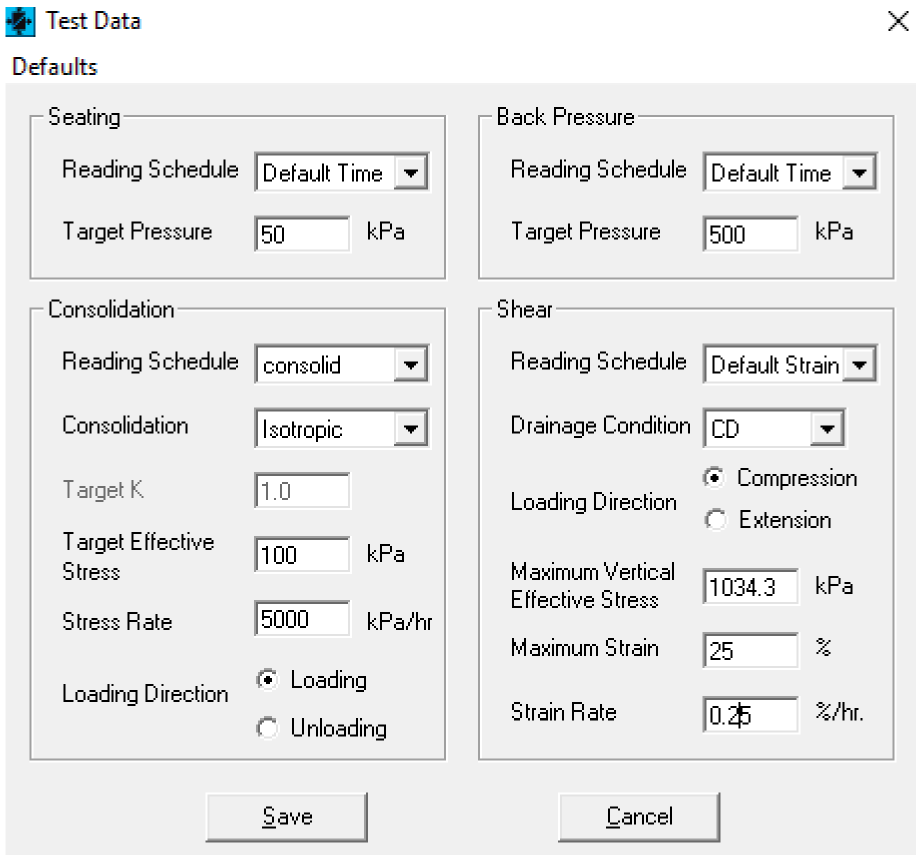


Figure 4.7 Entering the control test parameter

4.4 Seating Stage

After entering the specimen data, the "Seating" tab becomes active. The seating process involves seating the piston, adjusting the external load transducer, filling the cell with water, selecting the cell pressure, flushing the drains, and maintaining the volume of the sample.

4.4.1 Seating the Piston

The process of seating the piston involves locking the piston and minimizing the gap between the piston and the load button using manual control. This is achieved by entering the

"Tools" menu, selecting "Manual Mode", pressing on the "Load Frame" and then pressing on the first upward button. When the "Start" button is pressed, the platen will move upward till it reaches the load button. Figs. 4.8 through 4.9 show the sequence followed for reducing the gap between the piston and the load button.

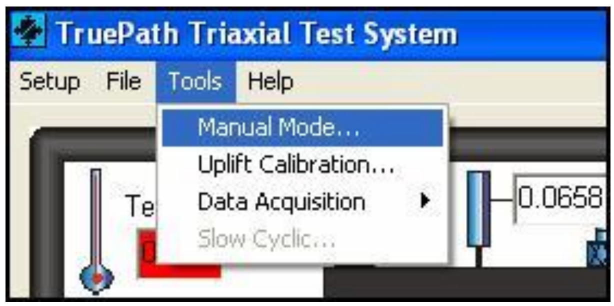


Figure 4.8 Selection for the manual mode

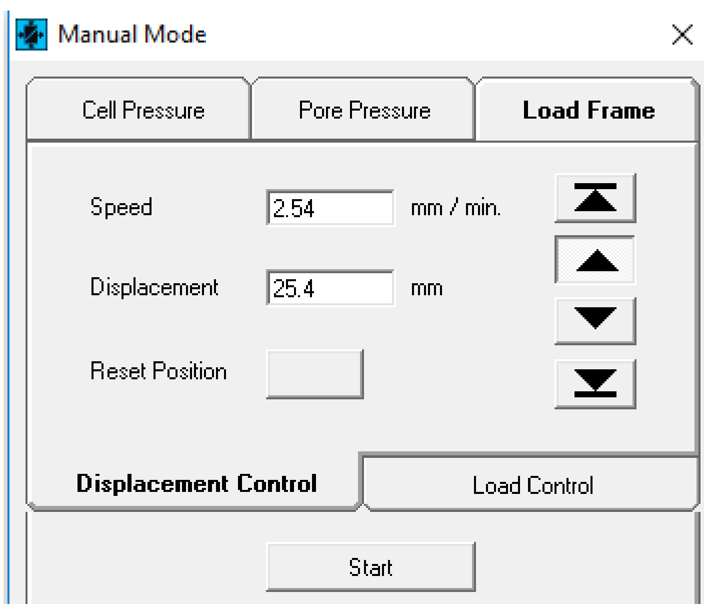


Figure 4.9 Reduction of gap between the piston and the load button

After reducing the gap, the “start” button is pressed as shown in Fig. 4.10 and another window will appear. In this window, the “Start” button has to be pressed again and the platen will move upward till it reaches the load button and the platen stops automatically when the load button is seated on the piston.

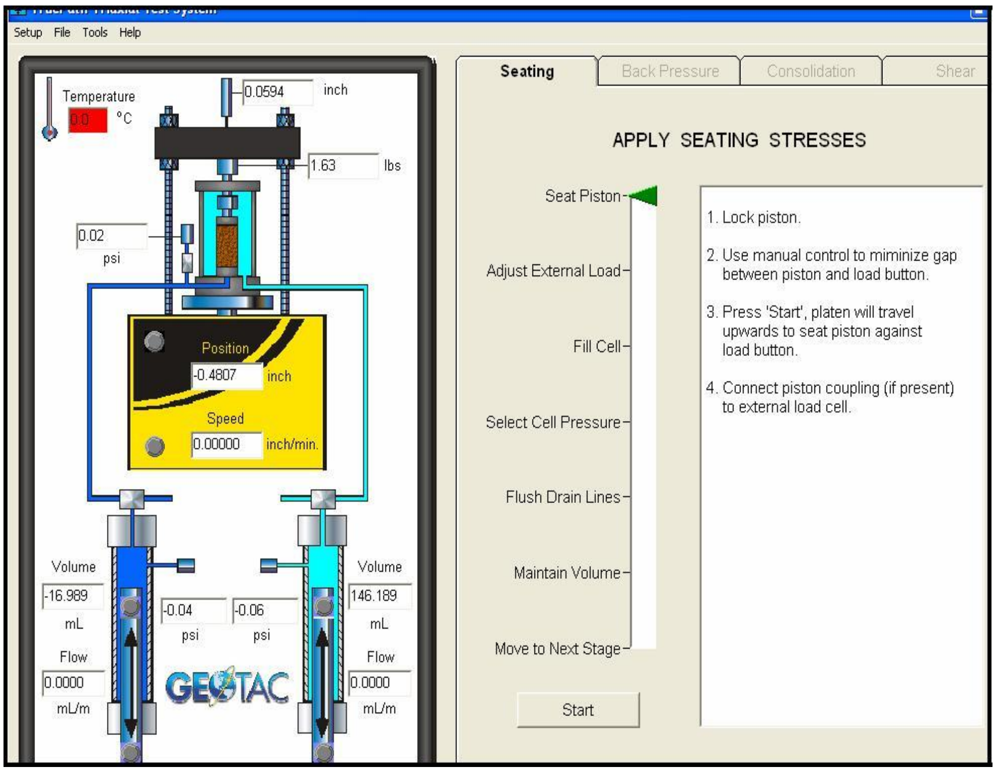


Figure 4.10 Window for seat piston

4.4.2 Adjust the External Load Sensor

When the “Adjust external load” button is pressed followed by pressing the “Start” button, the reading of the load cell becomes almost zero. The piston should be unlocked when the load cell reading approaches zero. Fig. 4.11 shows the procedure for adjusting the load.

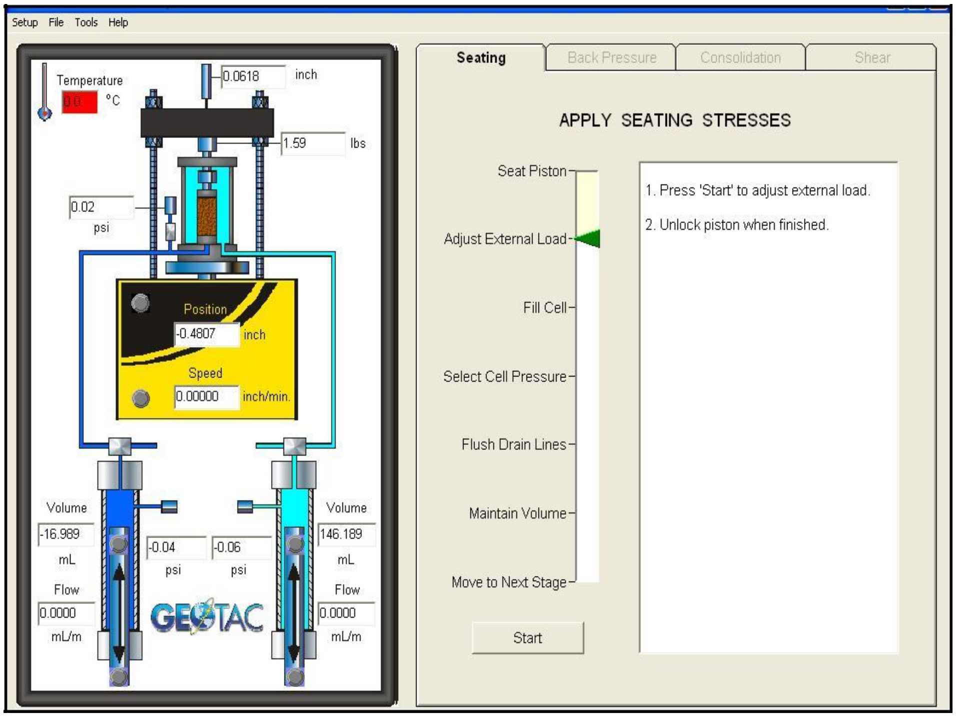


Figure 4.11 Adjustment for the external load transducer

4.4.3 Fill the Cell Chamber with Water

To fill the cell chamber with water, the “Fill Cell” button needs to be clicked and the ventilation air valve should be inserted into the top of the cell as shown in Fig.4.12. Then water should be supplied from an elevated water tank to the bottom quick connect of the cell through a plastic hose with a fitting on its top to allow entrance of the hose into the cell. The air in the cell is displaced by the water and is allowed to escape through the vent port. After filling the cell, water is allowed to flow out from the air vent port to ensure that all the air was driven out of the cell. The elevated water source should then be closed and the water hose is removed together with the air vent valve. The user can follow the step by step instructions that are displayed on the screen for the purpose of filling the cell with water as shown in Fig. 4.13.

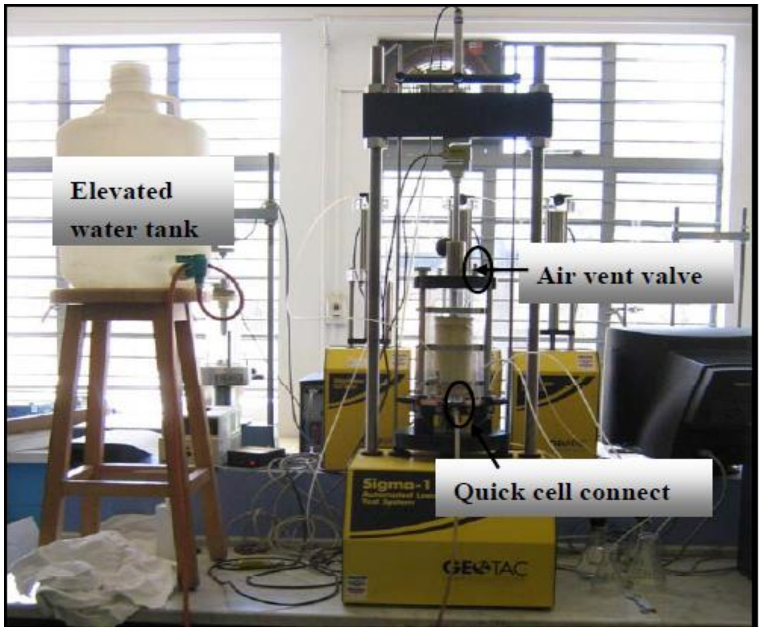


Figure 4.12 Filling the cell chamber with water

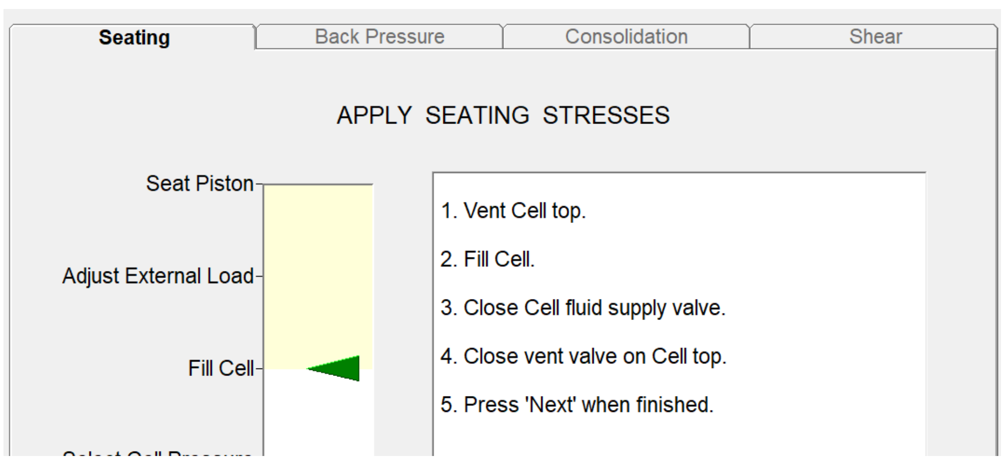


Figure 4.13 Steps for filling the cell chamber with water

4.4.4 Cell Pressure Selection

For the purpose of keeping the membrane pressed against the natural clay sample during the drain line flushing, a small confining pressure of 50 kPa (this is not correct. How do you run tests at 20 kPa If the seating is at 50 kPa?) is applied to the specimen. This can be achieved by opening the port valve of the cell pressure and connecting the cell pump pressure line to the cell bottom quick connect as shown in Fig. 4.14. The “Start” button should then be pressed to produce a window in which a pressure of 50 kPa should be entered. After about 3-5 minutes, the cell pressure will reach the required value and become stable. When this is achieved the user should press the “Done” button to complete the operation.

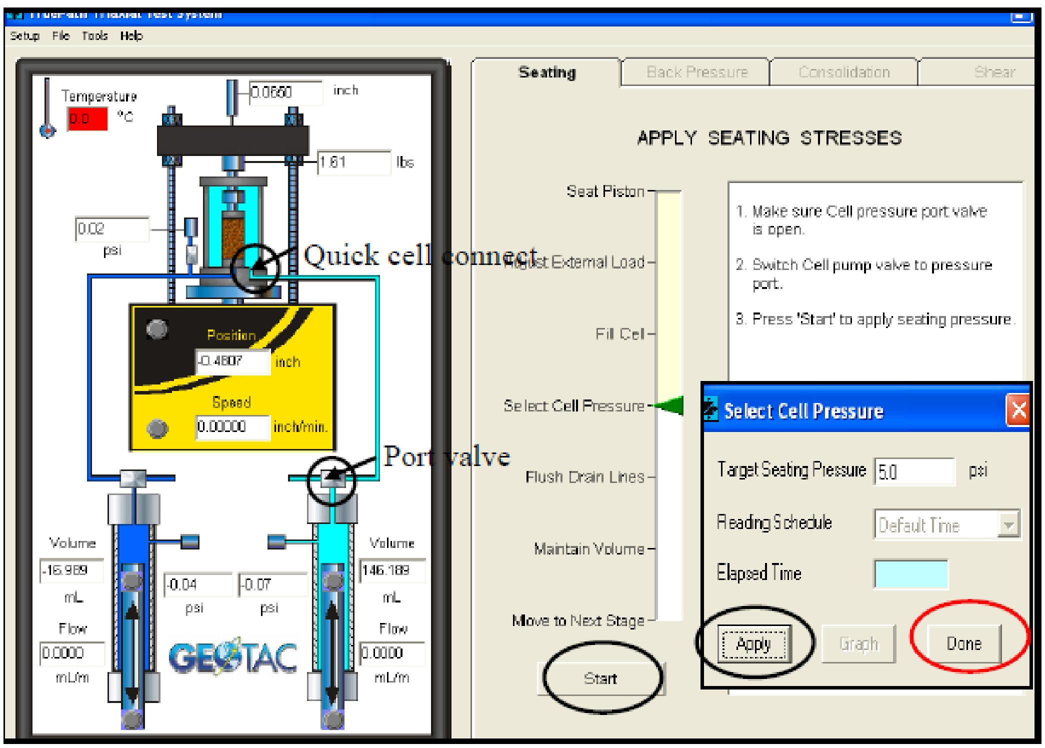


Figure 4.14 Application of initial confining pressure

4.4.5 Flushing the Drains

This technique is intended to force water to flow through the top and bottom drain lines using the bottom pump in order to expel air from these drain lines. First the bottom pump pressure line should be connected to the T fitting as shown in Fig. 4.15. Then the bottom pump valve is switched to the pressure line and the top drain inlet valve#1 and the top drain vent valve#4 are opened. An overflow tube is then attached to valve#4 and the “Start” button is pressed. Water should flow from the bottom pump into the T fitting through valve#1 and into the container through valve#4. In order to dislodge completely the air bubbles from the drain lines, the flow can be stopped and restarted simultaneously; Moreover, closing the vent valve #4 for one or two seconds and reopening it again while water is flowing from the top drain line valve can help in creating a pump pressure that speeds up the process of dislodging the air bubbles. After pressing the “Stop” button, valves#1 and 4 are closed and the bottom drain inlet valve #2 and bottom drain vent valve#3 are opened and the same procedure is repeat.

This technique is repeated until no more air bubbles are expelled through the drain lines. It is better to refill the bottom pump before completing the flushing step by switching the bottom pump to the refill container, pressing on “Tools” from the main menu, pressing “Manual Mode”, and selecting “Pore pump” (bottom pump). The “down” arrow is then clicked so that the bottom pump piston will move downward while water from the container will be drawn into the pump. The pore pump valve should then be returned to the pressure line, and flushing is continued if needed. Finally, the flushing stage should be terminated by closing valves#1 though 4 and pressing the “Done” button.

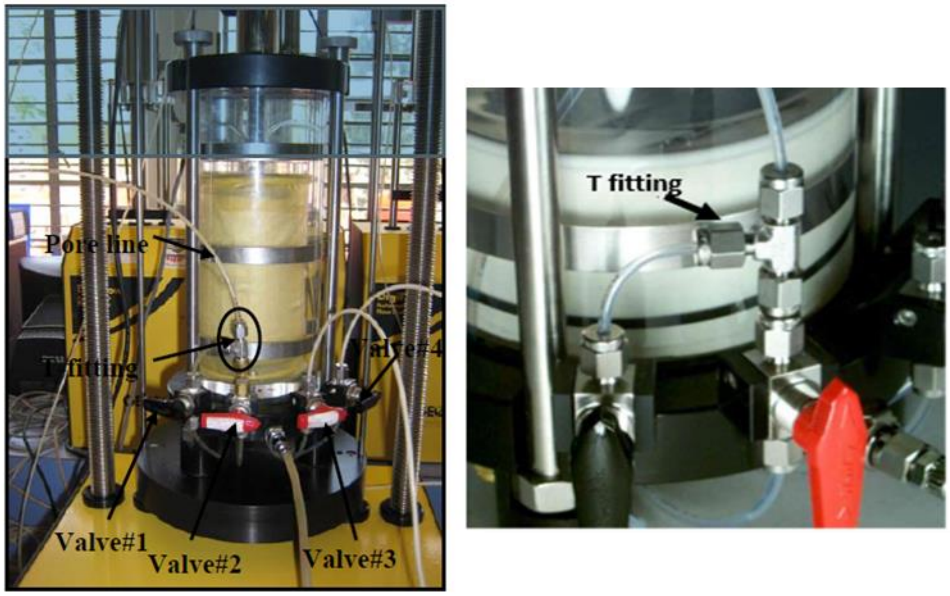
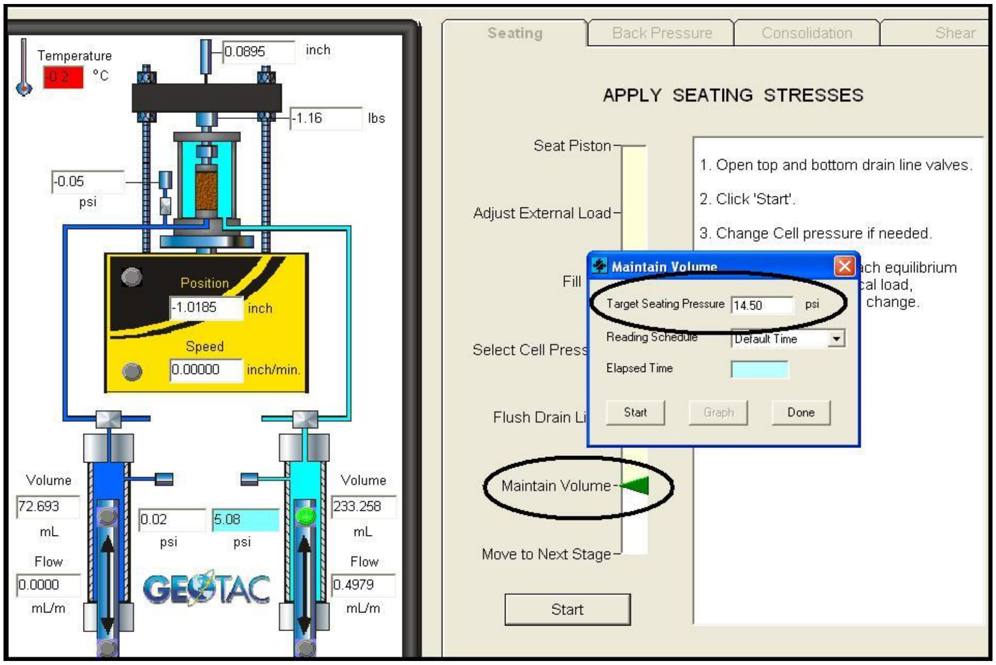


Figure 4.15 Flushing of the drains

4.4.6 Maintain the Volume

The ‘Maintain Volume’ step involves applying confining pressure to the specimen such that it does not swell. First, attach the pore pressure sensor to either the top or bottom vent drain valve #3 or #4. The “Maintain Volume” tab should be pressed as shown in Fig. 4.16 Next, the “Start” button is pressed and inlet drain valves #1, 2 and 3 (Pre pressure) are opened. The required confining cell pressure is then typed in the appropriate space and the “Start” button is pressed. This stage of seating may be as short as 15 minutes for soft specimens that are normally consolidated or may be as long as 24 hours for specimens that have a low hydraulic conductivity and are highly over consolidated. When the pore pressure and backpressure readings reach zero the maintain volume stage can be terminated. This is done by clicking on the “Stop” button and then on the “Done” button to end the maintain volume stage.



4.5 Back Pressure Saturation Stage

To ensure full saturation of the natural specimen, a back pressure/saturation pressure of 500 kPa is applied to the specimen using the back pump. The back pressure saturation stage consists of the following steps:

1. Check that inlet drain valves #1 and 2 are opened and make sure that the port valve of the bottom pump is opened and the pore pressure transducer pump valve#3 is opened, while drain valve #4 is closed.

2. Input the value of the required saturation pressure (500 kPa), and initiate saturation by click on the "Start" button as shown in Fig. 4.17

3. View the curve that shows the increase of back saturation pressure with time as shown in Fig. 4.18. The value of the back pressure can be checked either by looking at the curve or by looking directly at the bottom pressure transducer that is displayed on the left side of the screen. Usually a period of 24 hours is needed to reach the back pressure value or you can speed up the process by ramping the pressure in a certain rate such as 50 kPa/min .

4. Once the saturation pressure has reached its value, press on “Stop saturation”, and check the B value. To do that, click on “Check B” and enter a small increment of cell pressure (34 kPa) as shown in Fig. 4.17. Then, close drain inlet valves#1 and 2, and press on “Start”. The cell pump will instantaneously increase the cell pressure by 34 kPa, and the pore water pressure should indicate a similar increase of pore water pressure if the sample is completely saturated. The software calculates the B-value and reports its value every 15 Seconds on the screen. During this check, a B-value of 0.96 to 1 was generally obtained for tests conducted in this study.

5. After an acceptable B-value is ensured, click on “Done” and close the window for the B value check then re-open drain inlet valves#1 and 2, and press on “Done” to end the backpressure saturation stage. If saturation was not achieved using the initial specified backpressure of value 500 kPa, increase the saturation pressure by a certain increment and repeat the saturation process.

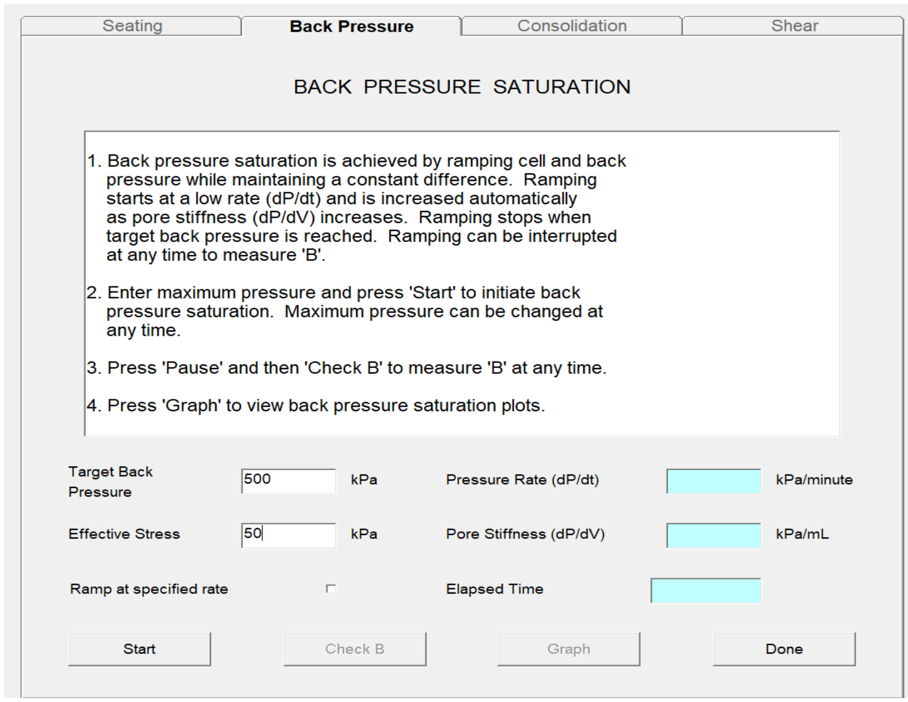


Figure 4.17 Window for backpressure saturation stage

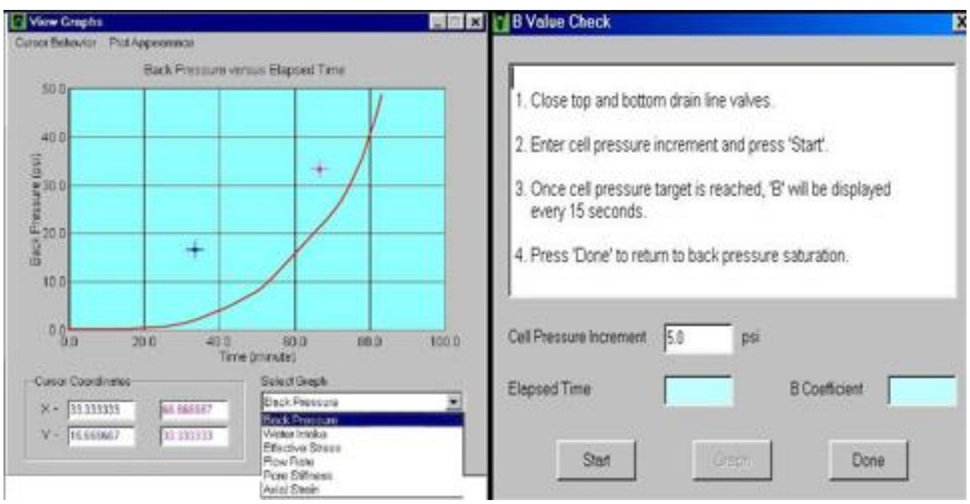


Figure 4.18 View the curve during the saturation process and Window for “B” value check

4.6 Isotropic Consolidation Stage

The consolidation stage is initiated by clicking on the “Consolidation” tab. First, the relevant data which includes the effective confining pressures and the stress rate that have been previously entered during the creation of the data test file should be checked. The activated window for isotropic consolidation is shown in Fig. 4.19. In this stage, the user can still change the target effective stress and the vertical stress rate, but cannot change the type of consolidation. Once all the input data is verified and consolidation is initiated, consolidation continues until the reading of the pore water volume intake for the pore pump becomes a constant. At this time, the isotropic consolidation stage can be assumed to be completed. A period of 3 days is usually needed to consolidate the natural clay specimens.

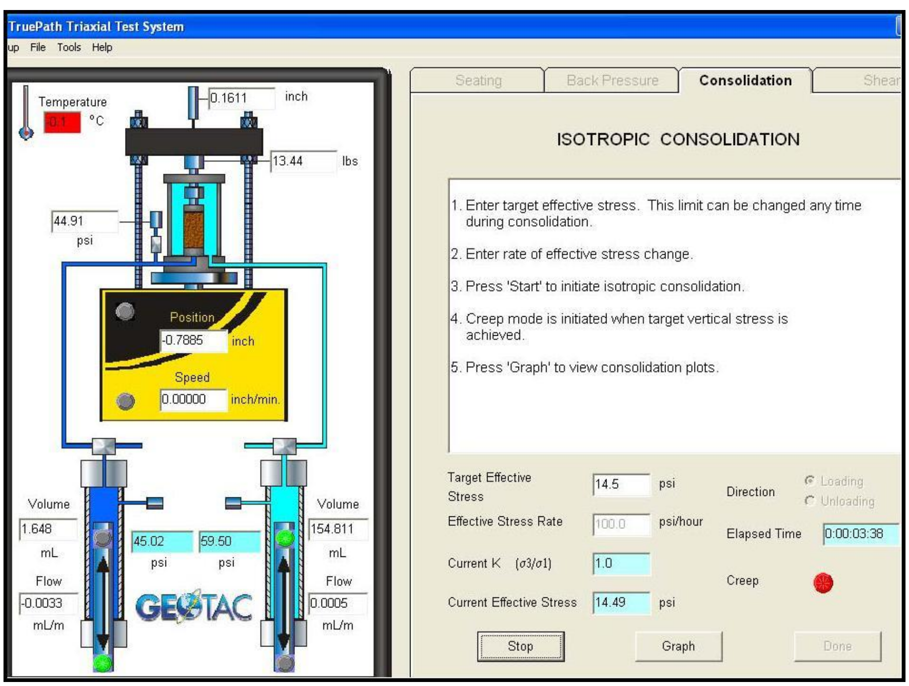


Figure 4.19 Window for isotropic consolidation

4.7 Drained Shearing Stage

At the end of the isotropic consolidation stage, a gap will form between the top cap of the specimen and the bottom of the loading piston. The user has to use the manual controls to close the gap and reestablish contact before starting the shearing stage. Once the window for the “drained shear” is activated, the user is required to enter the strain rate. In this research, a value of 0.25%/hour (confirm? Say why..) is used for the strain rate. Once the strain rate is chosen, cell valves#1 and 2 that are connected to the pore pump as well as valve#3 between the pore pressure sensor and the pore pump should be checked to be open. The “Start” button is then clicked as shown in Fig. 4.20 to initiate drained shearing. Different curves can be viewed while the test is in progress. These include curves that show the variation of the deviatoric stress and excess pore water with axial strain. When the strain reaches a percentage 20%, click on the “End test” tab to terminate the test and to close the software.

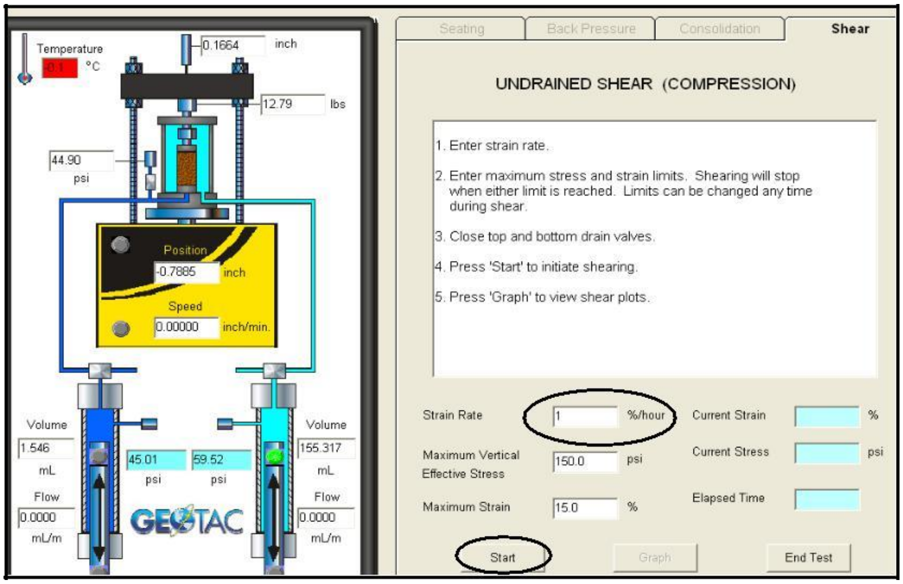


Figure 4.20 Window for drained shear test

4.8 Test Tear Down

Test tear down process involves removing pressures from the specimen and the triaxial chamber and the load frame. This can be accomplished as follows:

- Enter the True Path software and lock the cell piston.
- Select the manual controls, and choose cell pump. After that, choose "Pressure control" as shown in Fig. 4.21 and record a value of 0 kPa for the cell pressure and press start. Water will drain out from the cell chamber into the cell pump to reduce the cell pressure to zero.
- Use the manual control and reduce the pore pump pressure to zero.
- Use the manual control to lower the loading frame platen.
- Connect the top air vent valve and remove the hose from the bottom cell connect and replace it with a tube that discharges water into a container.
- After the water is drained out from the cell, remove the triaxial chamber from the loading frame, and dismantle the cell parts, wash them, and prepare them for another test.

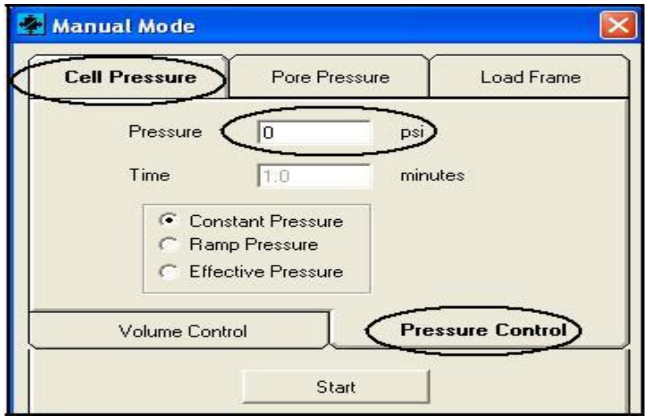


Figure 4.21 Window for unloading stage

4.9 Summary

A comprehensive description for operating the automated triaxial equipment "TruePath" was presented in this chapter in a simple way which includes a step-by-step procedure with figures and charts that facilitate the understanding of the testing process. The information presented in this chapter will make it easier for any future user to work and operate the "TruePath" equipment. However, reading the manual of the "TruePath" system is crucial and vital in order to complete all the required information that the user should know prior to operating the system.

CHAPTER 5

TRIAXIAL TEST SERIES 1

5.1 Introduction

The first series of tests was designed to investigate the effect of various combinations of input parameters on the load response of FRC composites which are compacted by impact. A total of 18 consolidated globally drained triaxial tests were carried out on both control (unreinforced) and fiber-reinforced natural clayey soil specimens while varying (1) the degree of saturation (compaction water contents, w: 14%, 18% and 20%), and (2) the confining pressure (σ_3: 20, 100, and 200 kPa). The parameters corresponding to each test together with the test results are presented in details in Appendix 1 (Table 10.1 for control samples and Table 10.2 for samples reinforced with 40mm long fibers, respectively). The choice of these fiber-reinforcement parameters is based on the results of Abou Diab et al. (2016) who showed that the optimal fiber-reinforcement scheme for undrained loading conditions consisted of 4cm long fibers and 1.25% fiber content by weight. The natural soil and natural fibers adopted in Abou Diab et al. (2016) will be identical to those adopted in our study for comparison purposes.

The energy used in compaction was calibrated by Abou Diab et al. (2016) for both unreinforced and fiber-reinforced clay to bring their dry unit weights up to the standard Proctor dry unit weights of the control clay at various water contents. As such, the number of hammer drops per layer was varied as specified in Tables 10.1, 10.2 in Appendix 1 to cater for the reduced split mold dimensions (as compared to the standard proctor mold) and for inclusion of fibers having a relatively low specific gravity. The number of layers was constantly kept as 3.

The initial dry unit weights and water contents of the specimens that were compacted by impact are presented in Figure 5.1 for control and reinforced specimens with 40mm long fibers (averaged for initially identical specimens). Results indicate that the calibrated compaction efforts provided a satisfactory representation of the target compaction characteristics with some inevitable variability in dry density around the standard proctor compaction curve.

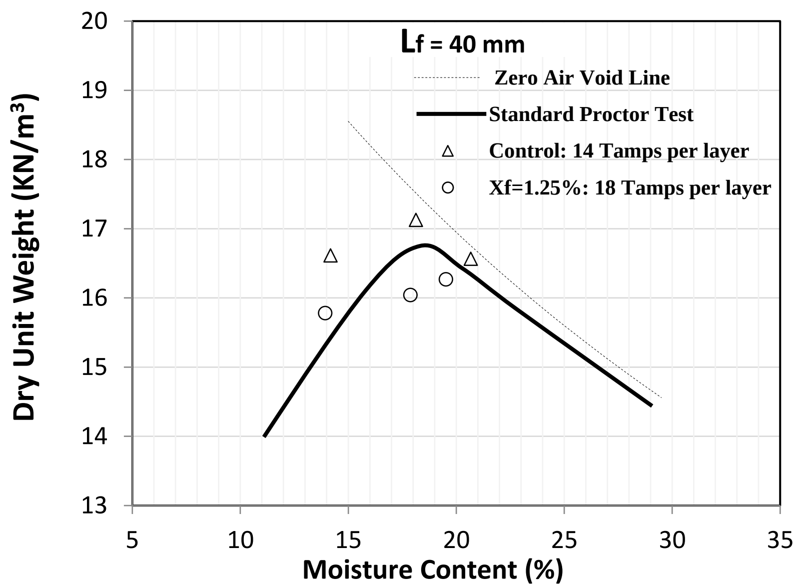


Figure 5.1 Dry unit weight- moisture content relationships of specimens compacted by impact, L_f=40mm (Series 1)

5.2 Test Results and Analysis

Results include a description of the stress-strain relationships, failure modes, an interpretation of the variation of the volumetric strain with axial strain, an analysis of the effect of various parameters on the variation of the deviatoric stress at failure and its degree of improvement.

5.2.1 Stress-Strain Relationships and Volumetric Change

Nine control samples were prepared at various compaction water contents (14%, 18%, and 20%) and subjected to CD triaxial testing under different confining pressures (20kPa, 100kPa, and 200kPa) to lay the base of the study and allow for assessment of the degree of improvement following fiber inclusion. Results show that the deviatoric stress at failure of compacted control clay varies between 82kPa and 99kPa under 20kPa confining pressure, and it increases to reach values between 208kPa and 248kPa under 100kPa confining pressure, and between 400kPa and 407kPa under 200kPa confining pressure; depending on the magnitude of the water content (see Table 10.1, Appendix 1).

The variation of the deviatoric stress and volumetric strain with axial strain is presented in Figures 5.2, 5.3, and 5.4 for control and fiber-reinforced clay specimens that are compacted at water contents of 20% (98% saturation), 18% (88% saturation) and 14% (68% saturation), respectively. These water contents correlate to specimens that are compacted wet of optimum, optimum, and dry of optimum, respectively.

Results indicate that the control specimens exhibited a strain softening response. This was observed for all water contents used, but was more evident for the specimens compacted dry of optimum. The peak deviatoric stresses were reached at axial strains of 13.78%, 14.19%, and 10.27% in control specimens tested at a confining pressure of 20kPa, for moisture contents of 14%, 18%, and 20% respectively. The associated axial strain values for a confining pressure of 100kPa are 18.63%, 20%, and 12.2%. However, at

200kPa the peak deviatoric stresses were reached at 20% axial strain for all moisture contents.

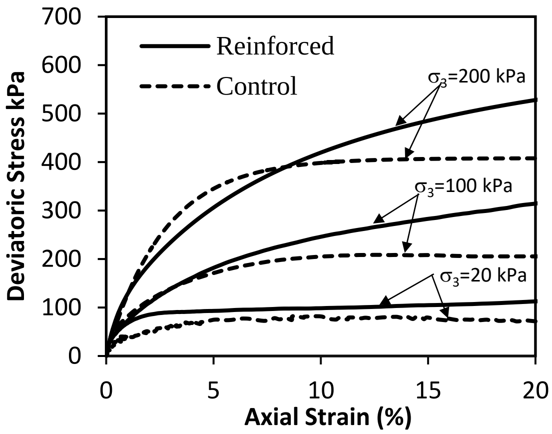
Reinforced specimens exhibited ductility in the stress-strain response with a strain hardening behavior and consistently showed a superior stress-strain response compared to the unreinforced specimens. The stress-strain curves of fiber-reinforced clay specimens exhibited improvements in the response, particularly at axial strains exceeding 5% where the stress-strain behavior shifts to a strain hardening response which continues all the way to the maximum strains applied (20%). The strain hardening response is more pronounced in tests conducted at higher confining pressures (100 kPa and 200 kPa) and results in improvements in the deviatoric stress and in the ductility of the mode of failure. These improvements in strength and ductility could be directly attributed to the positive reinforcing role of the fibers which were effective at reducing and re-distributing concentrated lateral strain (bulging) along the height of the specimen and allowing the reinforced clay to carry additional deviatoric load compared to the control clay specimens. (See Table 10.2, Appendix 1).

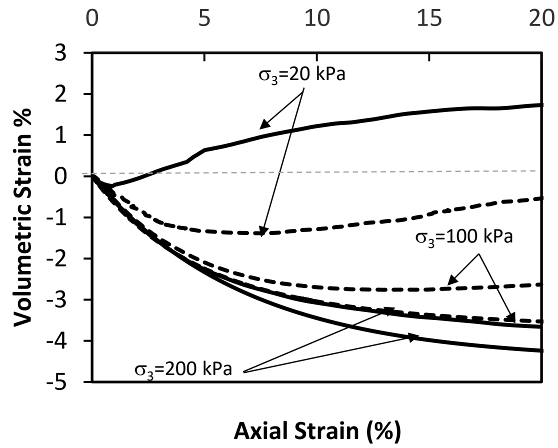
For σ_3=20kPa the deviatoric stress at failure increased for the case at dry of optimum (w=14%) from 82kPa (control sample) to 112 kPa (reinforced sample). Similarly, The deviatoric stress at failure for w=18% increased from 90kPa (control sample) to 143kPa (reinforced sample). For w=20%, the deviatoric stress at failure was 99kPa for control clay and improved to 113kPa for the reinforced sample.

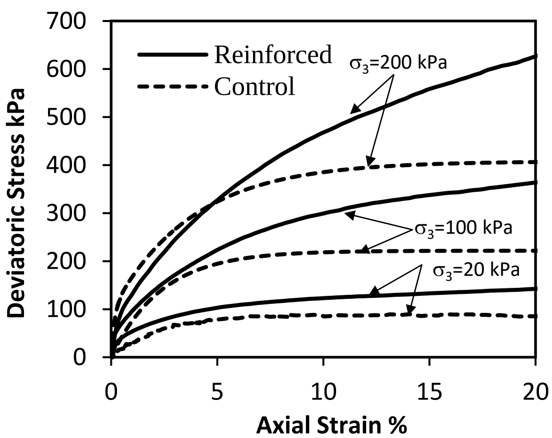
For σ_3=100kPa the deviatoric stress at failure increased for the case at dry of optimum (w=14%) from 248 kPa (control sample) to 256 kPa (reinforced sample). Similarly, The deviatoric stress at failure for w=18% increased from 221kPa (control sample) to 363kPa (reinforced sample). For w=20%, the deviatoric stress at failure was 208kPa for control clay and improved to 320kPa for reinforced sample.

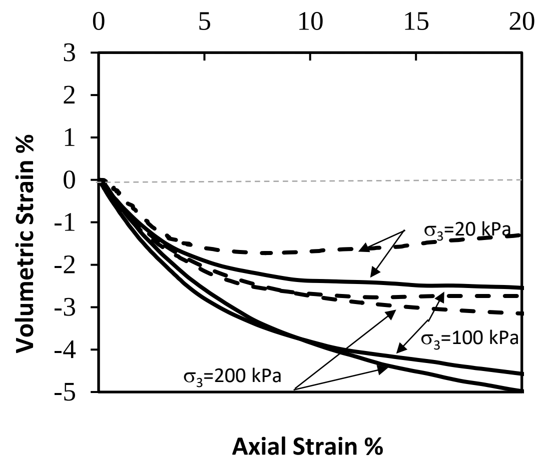
For σ_3=200kPa the deviatoric stress at failure increased for the case at dry of optimum (w=14%) from 400kPa (control sample) to 493kPa (reinforced sample). Similarly, The deviatoric stress at failure for w=18% increased from 406kPa (control sample) to 627kPa (reinforced sample). For w=20%, the deviatoric stress at failure was 407kPa for control clay and improved to 529kPa for reinforced sample.

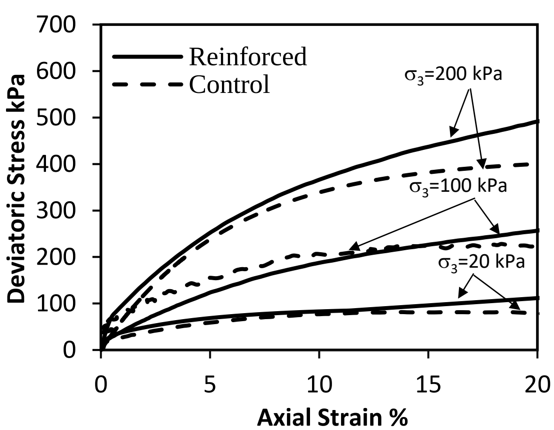
An analysis of the measured volumetric strains for control specimens indicates a general tendency for the clay to compress during drained shear, with the magnitude of the volumetric strains at failure being larger for tests at high confining pressures except for the dry of optimum case (w=14%). For fiber-reinforced specimens, results indicate that the inclusion of fibers in the clay instigated a tendency for additional contractive volume changes compared to control specimens. The only exception is the specimen compacted at water content of 20% and tested at a low confining pressure of 20 kPa. The volumetric strain response of tests conducted at a confining pressure of 20 kPa showed that fibers are efficient at arresting and reducing the dilative tendency of the clay, particularly for specimens that were compacted dry of optimum. It is worth noting that specimens compacted dry of optimum (low degree of saturation) exhibited higher volume changes and increased compressibility.

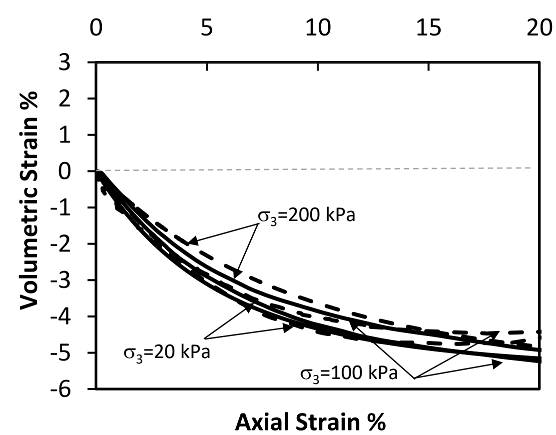

Reinforced
Control
σ3=200 kPa
σ3=100 kPa
σ3=20 kPa
Deviatoric Stress kPa
Axial Strain (%)


σ3=20 kPa
σ3=100 kPa
σ3=200 kPa
Volumetric Strain %
Axial Strain (%)


Reinforced
Control
σ3=200 kPa
σ3=100 kPa
σ3=20 kPa
Deviatoric Stress kPa
Axial Strain %

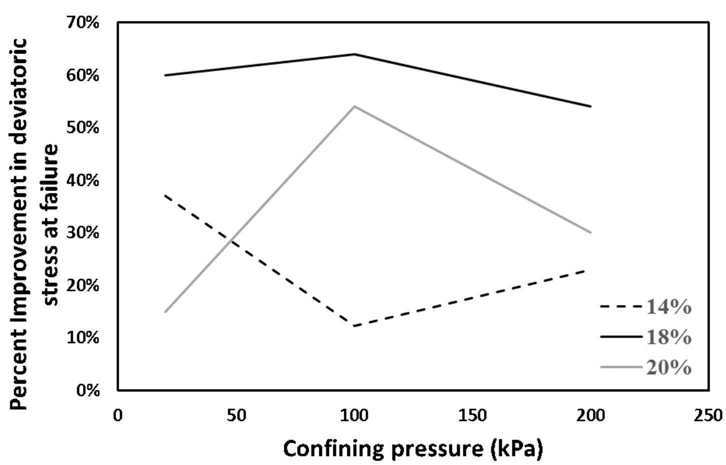

σ3=20 kPa
σ3=100 kPa
σ3=200 kPa
Volumetric Strain %
Axial Strain %


Reinforced
Control
σ3=200 kPa
σ3=100 kPa
σ3=20 kPa
Deviatoric Stress kPa
Axial Strain %


σ3=200 kPa
σ3=20 kPa
σ3=100 kPa
Volumetric Strain %
Axial Strain %

5.2.2 Improvement in deviatoric stress

The percent of improvement in the deviatoric stress at failure due to the inclusion of fibers was calculated for all the tests. Failure was defined in terms of the peak deviatoric stress or at 20% axial strain, whichever was reached earlier. The percent improvement was calculated as the difference between the deviatoric stress of the fiber-reinforced specimen and that of the control clay specimen normalized by the deviatoric stress of the control clay specimen. The resulting percent improvements are presented in Fig. 5.5 for the cases involving water contents of 14%, 18%, and 20%. Results on Fig. 5.5 indicate that improvements in the order of 15% to 54% were observed in the deviatoric stresses at failure for specimens compacted wet of optimum (w=20%). For specimens that were compacted at optimum (w=18%), the fibers were more efficient at improving the deviatoric stress with improvement levels ranging from 54% to 64%. For specimens prepared dry of optimum (14%), the improvement levels ranged from 3% to 37%.



For the purpose of exploring the effect of moisture content on the improvement levels, the variation of percent improvement with moisture content is shown in Figure 5.6. Results on Fig. 5.6 indicate that for samples compacted at 14% water content (dry of optimum) the highest improvement level reached was at 20 kPa confining pressure (37%). For samples compacted at optimum and wet of optimum the maximum improvement was reached at 100 kPa confining pressure of about 64% and 54%, for 18% and 20% respectively.

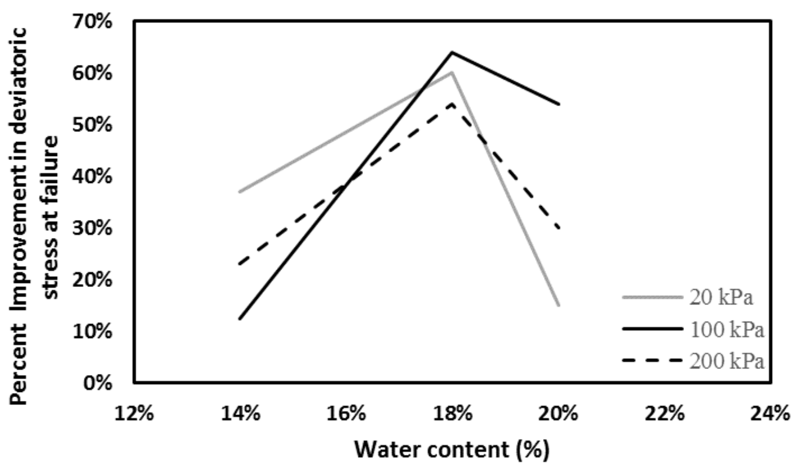


Figure 5.6 Improvement in deviatoric stress at failure for three different confining pressures at water contents of 14%, 18% and 20%.

5.2.3 Failure Modes

The mode of failure of the tested specimens is studied in Figures 5.7 (control clay) and 5.8 (fiber-reinforced clay), respectively. Failed specimens are shown from tests conducted at different confining pressures for clays compacted at a water content of 20%. Similar failure mechanisms were observed in the specimens compacted at a water content of 14% and 18%. The pictures of the failed control clay specimens show clear signs of bulging at the middle portion of the sample (Fig. 5.7), irrespective of the confining pressure. On the other hand, the bulging in specimens that were reinforced with fibers was found to be uniformly distributed along the height of the specimen indicating that the fibers played a significant role in transferring stresses and strains within the specimen, leading to an overall reduction in the intensity of localized bulging (Fig. 5.8). This change in the mode of failure in fiber-reinforced specimens resulted in improvements in the ductility of the stress-strain response as will be observed in the following section. Improvement in the ductility of the mode of failure of fiber-reinforced specimens was also reported by Abou diab et al. (2018) for triaxial tests conducted in an unconsolidated undrained setting using the same clay and fiber. It could thus be concluded that the inclusion of hemp fibers as reinforcing elements within compacted clays improves the ductility of the clay under both undrained and drained loading conditions.

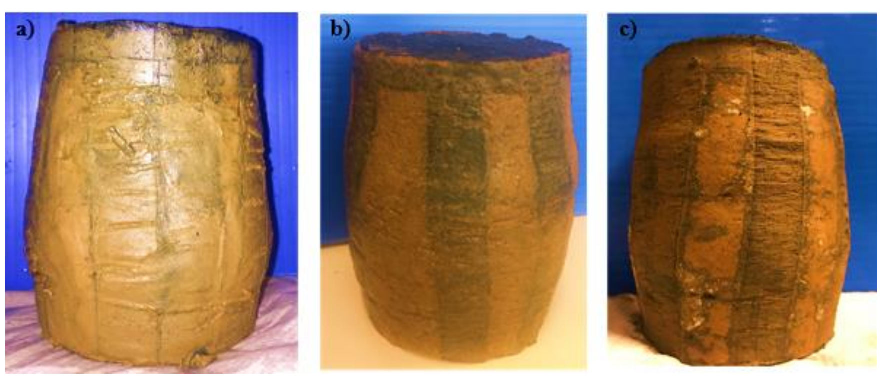


Figure 5.7 Mode of failure of control clay (w=20%) at (a) 20, (b) 100 & (c) 200 kPa

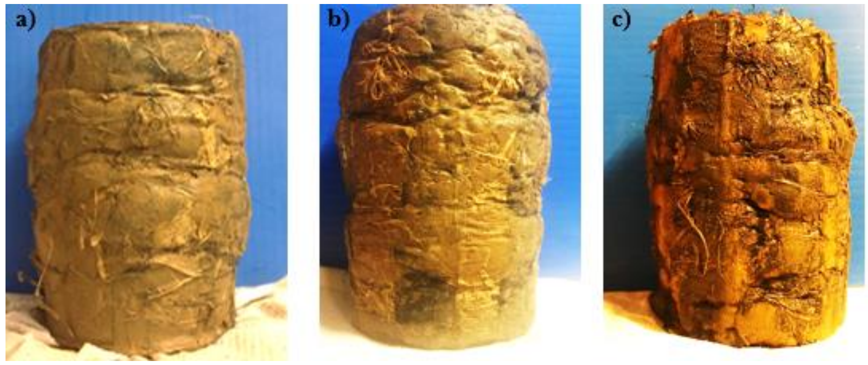


Figure 5.8 Mode of failure of fiber-reinforced clay (w=20%) at (a) 20, (b) 100 & (c) 200 kPa

5.2.4 Effect of Fibers on Stiffness

To quantify the effect of fibers on the drained stiffness of the compacted clay, a secant Young's modulus (defined as the stiffness at 1% strain) was calculated and compared to that of the control clay specimen. The comparison indicated that the effect of the fibers on the secant stiffness differs depending on the applied confining pressure. At the lowest confining pressure of 20 kPa, the fibers increased the stiffness of the composite by about 66% (w=20%), 86% (w=18%) and 45% (w=14%) compared to the control clay. For the higher confining pressures of 100 and 200 kPa, no improvements were observed in the secant stiffness. In fact, reductions in the order of 22% were observed in the secant stiffness for the case involving fiber reinforced clays tested at the highest confining pressure of 200 kPa at a compaction water content of 18% and a reduction in order of 51% were observed in the secant stiffness for the fiber reinforced clays tested at 100 kPa confining pressure at a compaction water content of 14% (refer to Figures 5.2 5.3 and 5.4). The possible loss of stiffness could be attributed to reductions in the dry density and the existence of a non-uniform distribution of voids in some FRC compared to the control unreinforced clay.

5.2.5 Failure Envelops

The drained Mohr circles at failure and the drained Mohr-Coulomb strength envelopes with their associated shear strength parameters (effective cohesion and friction angle) were determined and plotted on Figure 5.9 for both control and fiber-reinforced clay specimens. For the control clay, results on Figure 5.9 indicate that the effective cohesion (c') of the compacted clay ranges from 10 kPa (for w=14%) to 13 kPa (for w = 20%) to 18 kPa (for w = 18%), while the effective friction angle varies in the narrow range of 28 degrees (w = 18%) to 29 degrees (w = 20% & 14%). These shear strength parameters are typical for the drained response of compacted clays of low plasticity. The presence of a relatively large percent of sand in the clay could explain the drained friction angles that are slightly higher than normal for a typical pure clay.

The variation of the drained cohesive intercept and the friction angle with compaction water content and confining pressure is presented in Fig. 5.10. Upon reinforcement with hemp fibers, results on Fig. 5.10 show systematic increases in the drained cohesion and friction angle for the three water contents. The effective cohesion increases by 9 kPa (w=14%), 7 kPa (w=18%) and 10 kPa (w=20%), while the friction angle increases by about 7 degrees (w=18%) and 3 degrees (w=20% &14%) upon the inclusion of fibers. The higher increase in the friction angle for the compaction water content of 18% (optimum) may be related to (1) differences in the internal structure and dry density of the clay and (2) the "lubricating" effect which may reduce the interface friction between the hemp fibers and the clay at higher water contents (20%).

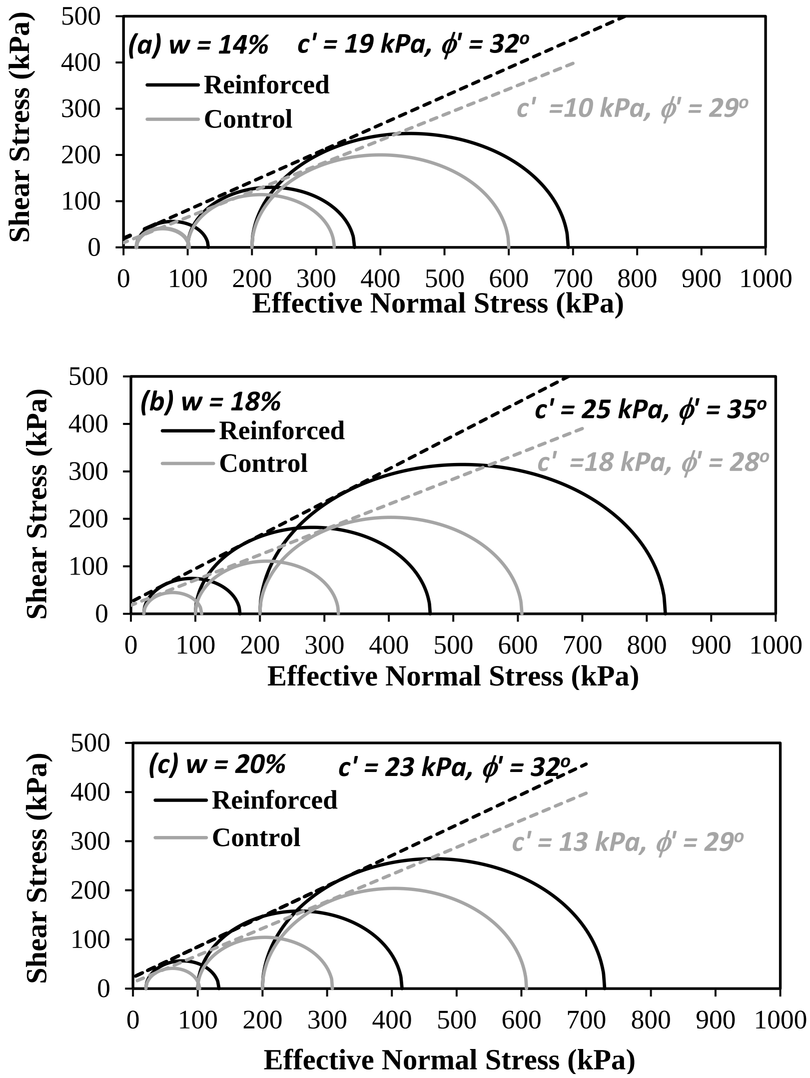


Figure 5.9 Mohr Coulomb's envelopes of samples prepared at (a) 14%, (b) 18%, and (c) 20% water content

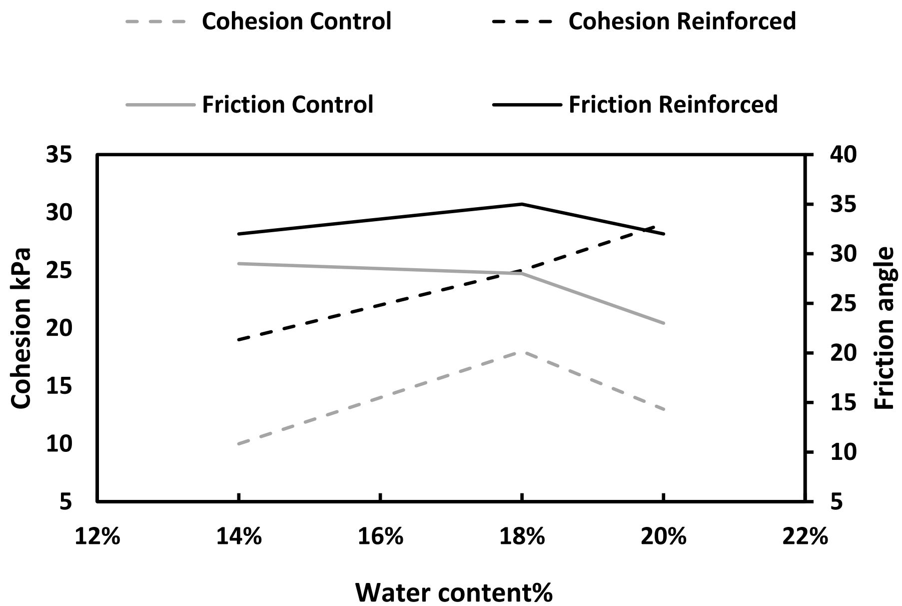


Figure 5.10 Variation of C’ and $\emptyset'^{\circ}$ with water content

5.3 Summary

Based on the results of consolidated drained triaxial tests (Series 1) that were conducted on control clay specimens and hemp-reinforced specimens, the following conclusion can be drawn:

1. The inclusion of hemp fibers in compacted clay improved the ductility. The addition of fibers reduced concentrated bulging and resulted in a stress-strain response that is characterized by strain hardening at axial strains exceeding 10%.
2. The deviatoric stress at failure of fiber-reinforced specimens was larger than that of control specimens with recorded improvements of up to 60%. These improvements

were found to be more pronounced in clay specimens that were compacted at 18% water content (optimum).

3. The effect of fiber inclusion on the stiffness was dependent on the confining pressure. No significant improvements in stiffness were observed for the tests conducted at high confining pressure of 100 and 200 kPa. For tests conducted at 20 kPa, improvements of up to 80% were observed for fiber-reinforced specimens.

4. Fibers are efficient at arresting and reducing the dilative tendency of the clay.

   With regards to volume change, the incorporation of fibers reduced the dilative tendency of the clay at low confining pressure and induced additional contractive volume changes in reinforced specimen.

5. The inclusion of hemp fibers within compacted clay specimens increased the drained cohesion and friction angle of the clay. The increase in the cohesive intercept (about 9 kPa) was not affected by the compaction water content. However, the increase in drained friction angle was more pronounced (around 8 degrees) in the specimen compacted at optimum (w=18%) in comparison to the specimen compacted wet of optimum w=20% and dry of optimum w=14% (about 3 degrees).

Chapter 6

TRIAXIAL TEST SERIES 2

6.1 Introduction

The second series of tests was designed to investigate the effect of varying fiber content on the load response of FRC composites which are compacted by impact. A total of 6 consolidated globally drained triaxial tests were carried out on fiber-reinforced natural clayey soil specimens while varying (1) the gravimetric fiber content X_f (%) (1% and 1.5%) and (2) the confining pressure (σ_3: 20, 100, and 200 kPa). The parameters corresponding to each test together with the test results are presented in details in Appendix 1 (Table 10.3). The choice of these fiber-reinforcement parameters is based on the results of Abou Diab et al. (2016) who showed that the optimal fiber-reinforcement scheme for undrained loading conditions consisted of 4cm long fibers and 1.25% fiber content by weight, so specimens reinforced with 4cm long fibers but at different fiber contents (smaller and greater than 1.25%) are tested to investigate whether the optimum combination of fiber length and content is affected by the drainage conditions. To limit the number of tests, this series is restricted to specimens compacted at the water content of 18%. Consequently, the results will be compared with the first Series for control and reinforced samples compacted at the same water content (w=18%) (Appendix 1 Table 10.4)

6.2 Test Results and Analysis

Results include a description of the stress-strain relationships, failure modes, an interpretation of the variation of the volumetric strain with axial strain, an analysis of the effect of various parameters on the variation of the deviatoric stress at failure and its degree of improvement.

6.2.1 Stress-Strain Relationships and Volumetric Change

Six reinforced samples compacted at various fiber contents (1%, 1.5%) and at 18% water content were subjected to CD triaxial testing under different confining pressures (20kPa, 100kPa, and 200kPa) for the purpose of investigating whether the optimum combination of fiber length and content is affected by the drainage conditions.

The variation of the deviatoric stress and volumetric strain with axial strain is presented in Figure 6.1 for fiber-reinforced clay specimens that are subjected to 20, 100, and 200 kPa confining pressures. First Series control (X_f =0%) and reinforced samples (X_f=1.25%) compacted at the same water content (w=18%) were plotted in Figure 6.1 for the purpose of comparison.

Results show that the deviatoric stress at failure of compacted reinforced clay varies between 170kPa and 217kPa under 20kPa confining pressure, and it increases to reach values between 431kPa and 437kPa under 100kPa confining pressure, and between 687kPa and 690kPa under 200kPa confining pressure; depending on the magnitude of the fiber content (1% and 1.5%) (see Table 10.3, Appendix 1).

Results on Fig. 6.1 indicate that varying the fiber content between 1% and 1.5% did not have a significant impact on the stress-strain response as was the case for the undrained tests conducted by Abou Diab (2016). In fact, results show that specimens reinforced with the lowest fiber content of 1% exhibited a slightly improved stress-strain response compared to the specimens reinforced with 1.25% and 1.5% for confining pressures of 20 and 200 kPa and a similar response of confining pressure of 100 kPa. These results indicate that the drained stress-strain response of fiber-reinforced clays may be less sensitive to the fiber content compared to the undrained response, and that the optimum fiber content in drained tests may slightly differ than that witnessed in identical undrained tests.

An analysis of the measured volumetric strains indicates that the contractive volumetric strains for control specimens were increased by the addition of fibers. The increase in volumetric strain does not seem to correlate well with the fiber content. For example, the increase in volumetric strain was the largest for specimens reinforced with 1.25% fibers by weight in the tests conducted at 20 kPa. For the tests conducted at 200 kPa, the specimens reinforced with 1.5% fibers exhibited the largest increase in volumetric strain. It should be noted however that the difference the volumetric strain response for specimens reinforced with different fiber contents can be considered to be minimal and may be due to uncertainties in the specimen preparation, dry density, fiber orientation and distribution, etc.

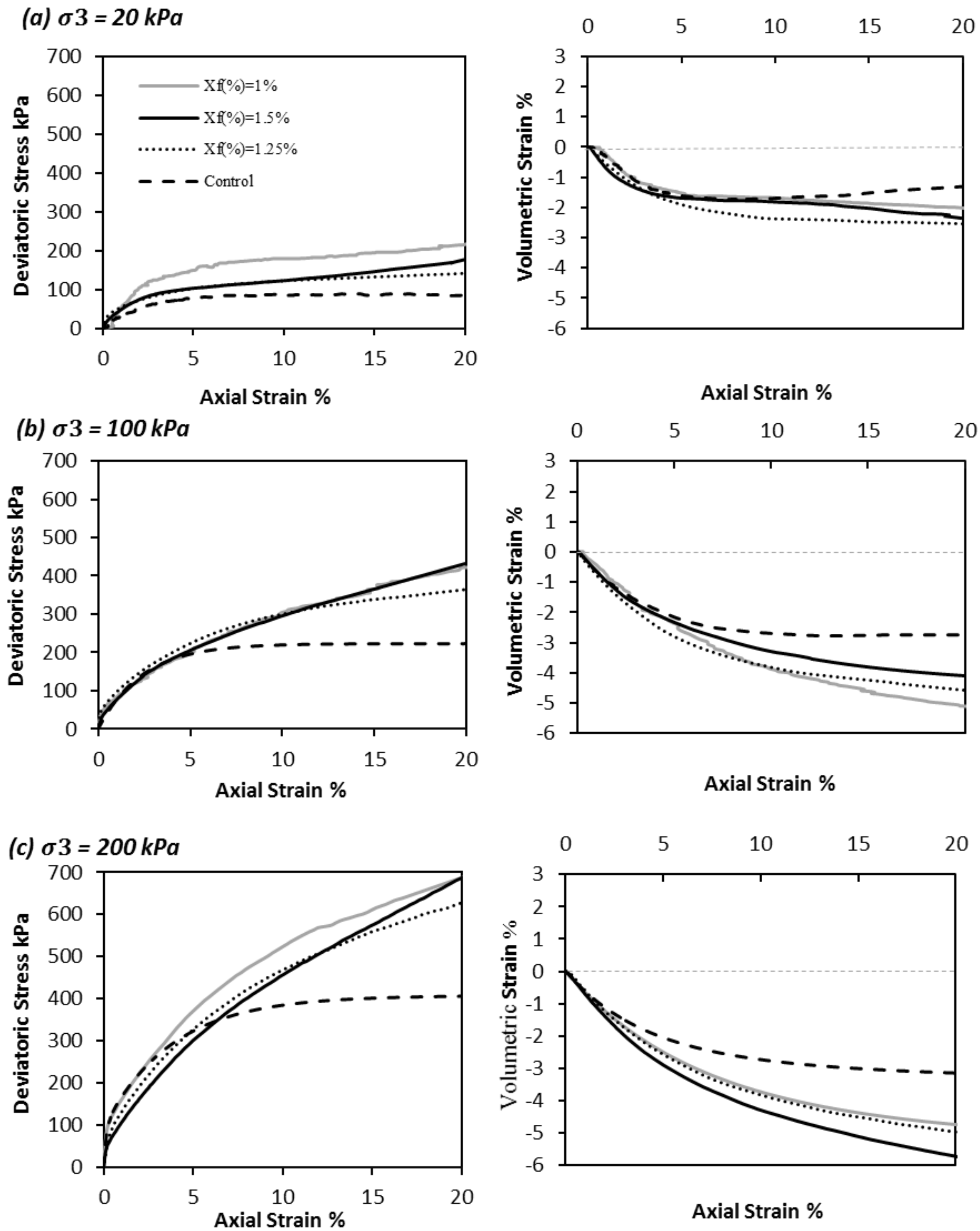


Figure 6.1 Variation of deviatoric stress and volumetric strain with axial strain for various fiber contents under different confining pressures

6.2.2 Improvement in deviatoric stress

The percent of improvement in the deviatoric stress at failure due to the different fiber contents was calculated for all the tests. Failure was defined in terms of the peak deviatoric stress or at 20% axial strain, whichever was reached earlier. The resulting percent improvements are presented in Fig. 6.2 for the cases involving three different confining pressures of 20, 100, and 200 kPa.

Results on Fig. 6.2 indicate that improvements in the order of 142% to 69% were observed in the deviatoric stresses at failure for specimens compacted at 1% fiber content at different confining pressures. For specimens that were compacted at 1.25% fiber content a drop in the improvement levels was observed ranging from 54% to 64%. For specimens prepared at 1.5% fiber content the improvement levels ranged from 70% to 97%.

To investigate the possibility of correlations between the observed improvements in deviatoric stress at failure and the dry densities of the specimens, the observed improvements were plotted versus the average dry density on Fig. 6.2. Results on Fig. 6.2 point to a possible correlation between the average dry density and the improvement in deviatoric stress. The improvement in response seems to decrease as the dry density decreases (particularly for the cases involving a fiber content of 1.25%).

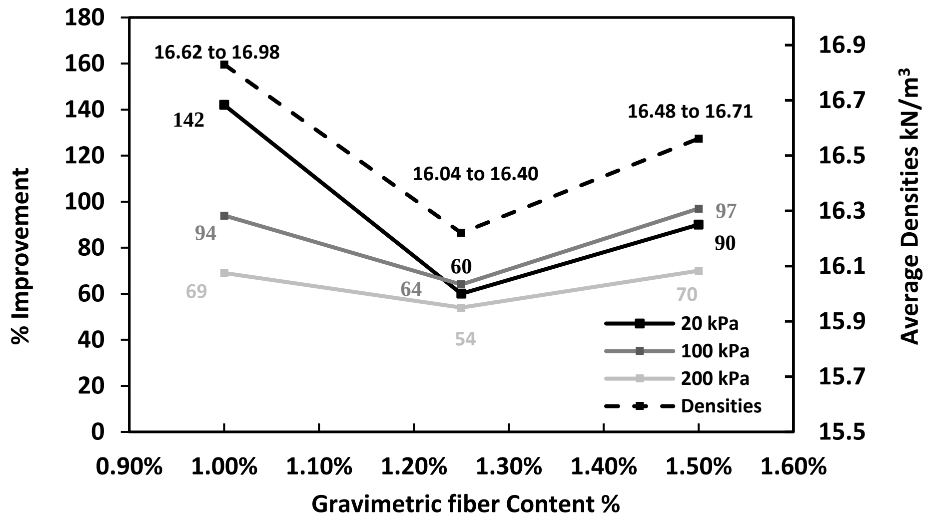


Figure 6.2 Improvement in deviatoric stress at failure and initial densities for fiber contents of 1%, 1.25% and 1.5% at different confining pressures.

6.2.3 Failure Modes

The mode of failure of the tested specimens is studied in Figures 6.3-6.5 (fiber-reinforced clay series 2). Similar failure mechanisms were observed in the specimens compacted at all confinement levels. Irrespective of the confining pressure the bulging in specimens that were reinforced with fibers was found to be uniformly distributed along the height of the specimen indicating that the fibers played a significant role in transferring stresses and strains within the specimen, leading to an overall reduction in the intensity of localized bulging.

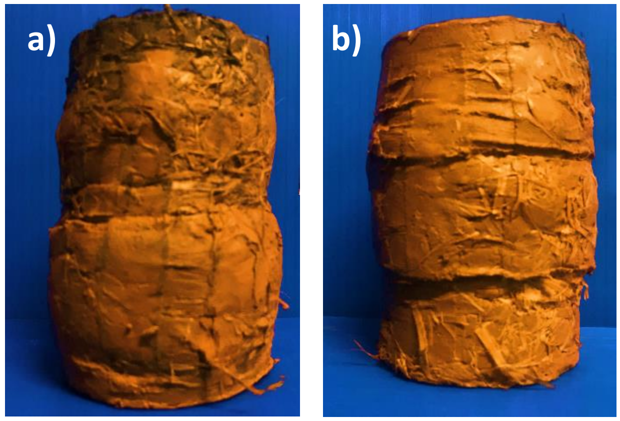


Figure 6.3 Mode of failure of Reinforced clay (w=18%, $\sigma 3$ =20 kPa) at (a) xf=1%, (b) xf= 1.5%

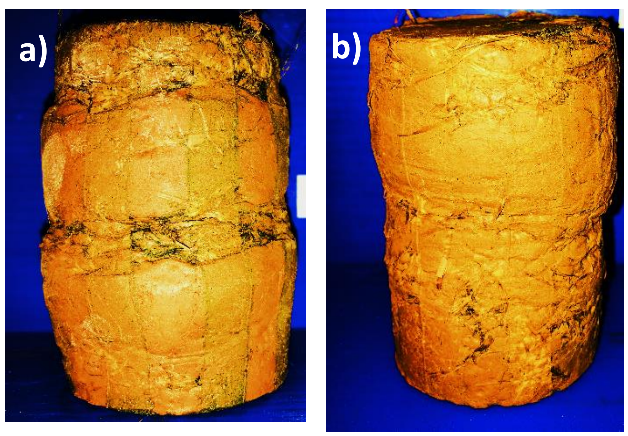


Figure 6.4 Mode of failure of Reinforced clay (w=18%, $\sigma 3$=100 kPa) at (a) xf=1%, (b) xf= 1.5%

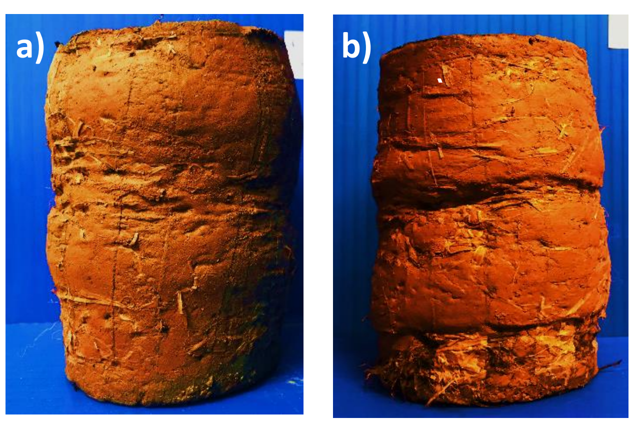


Figure 6.5 Mode of failure of Reinforced clay (w=18%, σ_3=200 kPa) at (a) xf=1%, (b) xf=1.5%

6.2.4 Failure Envelops

The drained Mohr circles at failure and the drained Mohr-Coulomb strength envelopes with their associated shear strength parameters (cohesion and friction angle) were determined and plotted on Figure 6.6. For the control clay, results from series 1 indicate that the effective cohesion (c’) of the compacted clay is in the order of 18 kPa (for w = 18%), while the effective friction angle is 28 degrees (w = 18%).

Upon reinforcement with different gravimetric content of hemp fibers, results on Figs. 6.6 and 6.7 show systematic increases in the drained cohesion and friction angle for the three fiber contents. The effective cohesion increases by 32 kPa (X_f=1%), 7 kPa (X_f=1.25%, series 1) and 14 kPa (X_f=1.5%), while the friction angle increases by about 6 degrees (X_f=1%) and 7 degrees (X_f=1.25%) & 9 degrees (X_f=1.5%) upon the inclusion of fibers. These results indicate that the cohesive intercept seems to be more affected by the fiber content than the friction angle. The higher increase in the cohesive intercept for the lower fiber content of 1% may be associated with the slightly higher dry densities and a possible more efficient fiber distribution within the soil specimen.

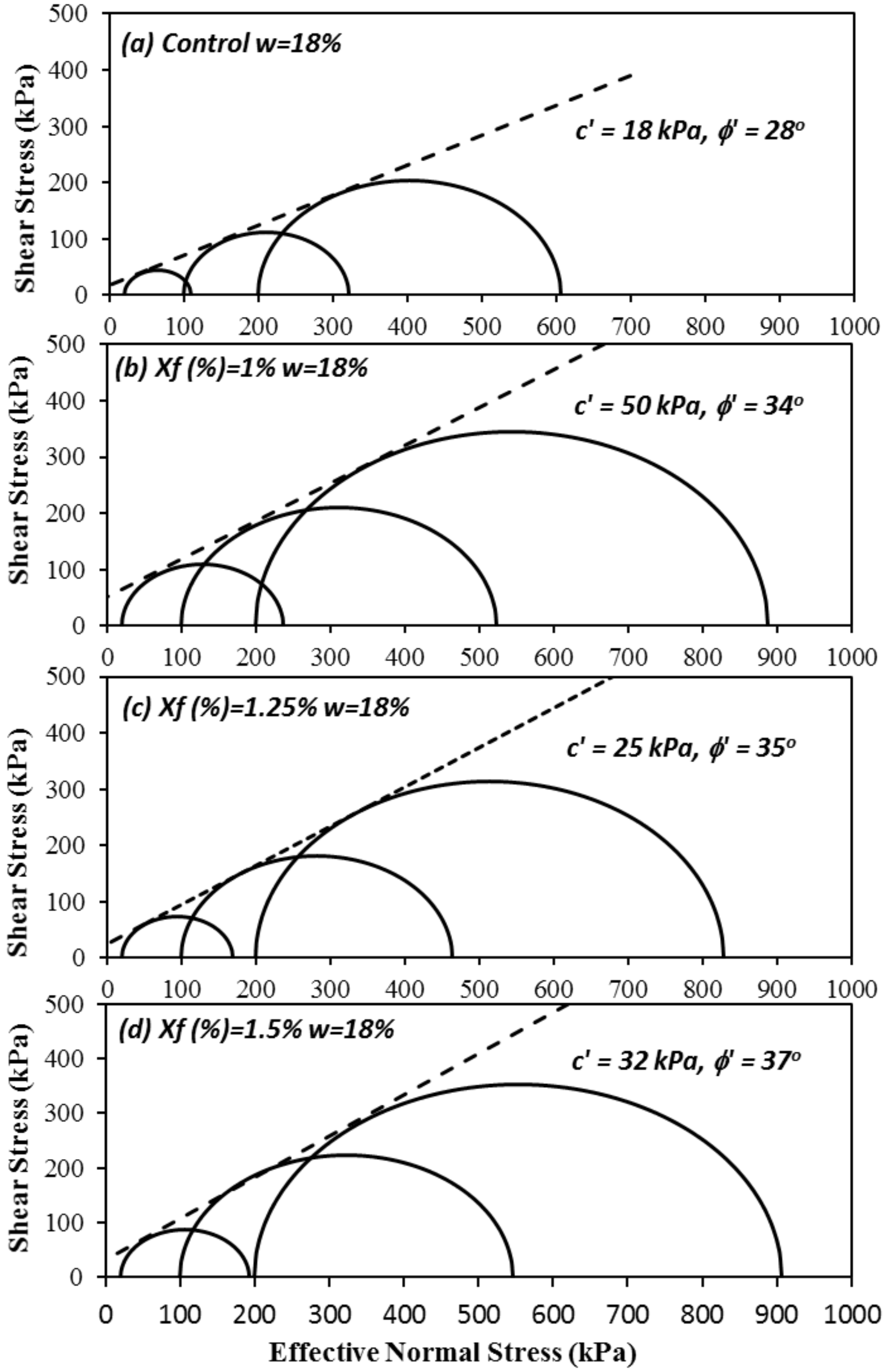


Figure 6.6 Mohr Coulomb's envelopes of series 2 samples

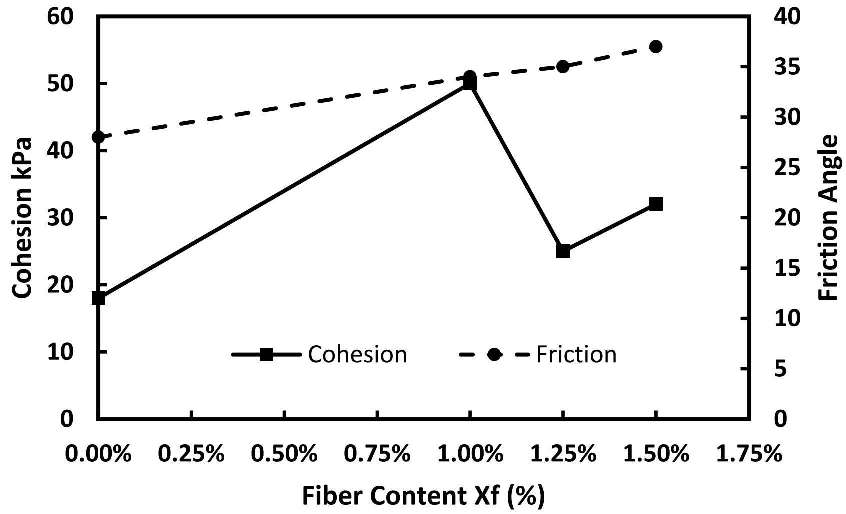


Figure 6.7 Variation of C’ and Ø'° with fiber content

6.3 Summary

Based on the results of consolidated drained triaxial tests (Series 2) that were conducted on reinforced specimens with different fiber content, the following conclusion can be drawn:

1. At low confining pressure (20 kPa) the best improvement was witnessed when the fiber content was 1% (up to 142%) while at higher confining pressures (100 & 200 kPa) almost the same improvement was noticed for 1% and 1.5% (in the range of 90%). For all confining pressures, the lowest improvement was noticed

at 1.25% fiber content. This drop in the improvement at 1.25% fiber content could be correlated to the reinforced samples initial densities.

2. The increase in the cohesive intercept was significantly higher in the specimens reinforced at the lowest fiber content of 1%. On the other hand, the friction angle increases consistently as the fiber content increased.

Chapter 7

TRIAXIAL TEST SERIES 3 “Durability”

7.1 Introduction

The third and final series of tests is conducted to investigate the durability of the hemp when used as a soil reinforcement. A total of 5 consolidated globally drained triaxial tests were carried out on fiber-reinforced natural clayey soil specimens compacted at 18% water content with 4cm long fibers and 1.25% fiber content by weight.The parameters corresponding to each test together with the test results are presented in details in Appendix 1, Table 10.5.

Most of the reported durability studies in the literature to date were carried out on natural fibers that were fully exposed to severe conditions of weathering and water absorption, which may have accelerated their biodegradation by fungus and bacteria. Moreover, no durability-related studies are conducted on fibers embedded in a soil matrix (partial exposure).

Durability studies must be conducted, preferably on fibers embedded in a soil matrix with partial exposure to simulate the setup of the real problem. The soil sealing the fibers may reduce the extent and/or rate of their deterioration as testified by the clay bricks reinforced with natural straw and/or other fibers that were long used as earth materials for construction purposes

7.2 Sample Preparation

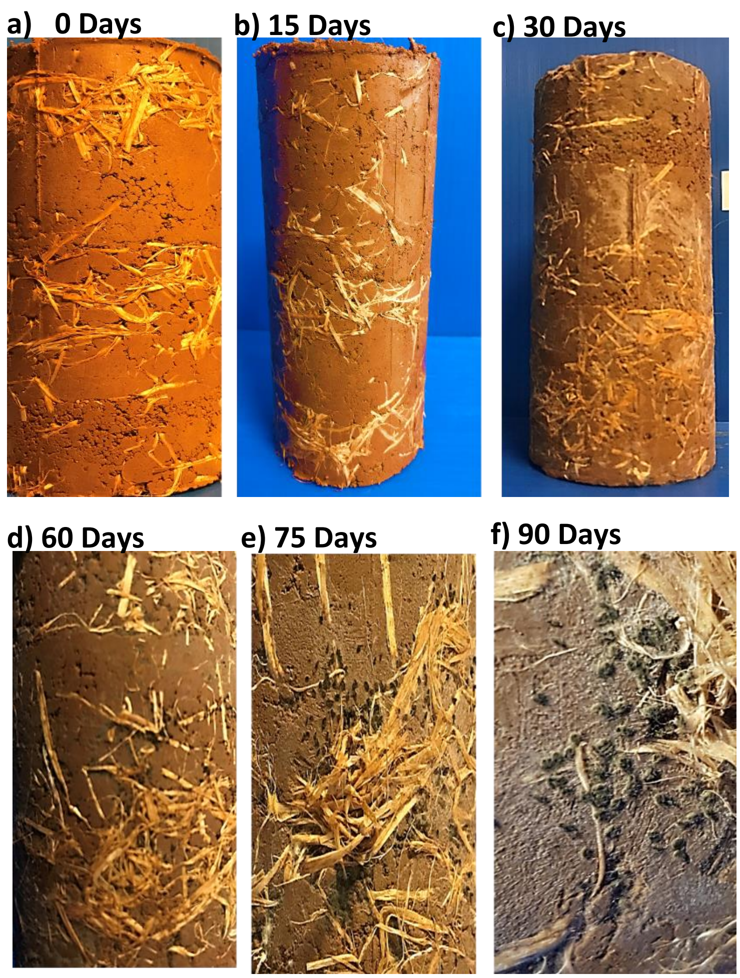
In the third series of tests, identical specimens of fiber reinforced clay are prepared, sealed with Nylon sheets and covered with PVC tubes (Figure 7.1). The specimens were then left for a specific period of time (Table 7.1) in a control room prior to conducting Triaxial tests/Tensile strength tests for fibers. The results of such tests are compared to identical tests, which were tested in the conventional approach (series 1, Table 10.6 in Appendix 1) to portray any detrimental effect of time on the response of the specimens.

Figure 7.1 Durability Samples

Table 7.1 Durability samples Age

Test	**Age**
D1	15 days
D2	30 days
D3	60 days
D4	75 days
D5	90 days

It is important to mention that natural Hemp fibers are mainly composed of organic materials such as cellulose, hemicellulose, and lignin. So with increasing time, a mold appears and feeds itself by producing chemicals that break down organic material and start to rot (figure 7.2). As the material rots, the mold grows. There are thousands of different kinds of molds. If the progression of the mold is observed for a few days, it will turn black. The tiny black dots are its spores, which can grow to produce more mold. In the next sections, we will understand whether molds affect the strength or not.



7.3 Test Results and Analysis

Results include a description of the stress-strain relationships, failure modes, an interpretation of the variation of the volumetric strain with axial strain, an analysis of the detrimental effect of time on the variation of the deviatoric stress at failure

7.3.1 Stress-Strain Relationships and Volumetric Change

Five reinforced samples compacted at 1.25% fiber content and at 18% water content were subjected to CD triaxial testing under one confining pressure of 100kPa and left for a specific period of time as mentioned before for the purpose of investigating the time effect on deviatoric stress.

The variation of the deviatoric stress and volumetric strain with axial strain is presented in Figure 7.3. Results from the first series (Age=0 Days) control (xf =0%) and reinforced samples (xf=1.25%) compacted at the same water content (w=18%) were plotted also for the purpose of comparison (Table 10.6 Appendix 1).

Results show that the stress-strain response of the fiber-reinforced specimen which was tested at 15 days was almost similar to that tested at zero days indicating no loss in load-carrying capacity. Interestingly, the results of the test specimens that were stored for more than 30 days prior to triaxial testing indicated a sudden reduction in load carrying capacity. In fact, the stress-strain curves for these tests approached that of the control clay specimen indicating zero fiber efficiency. These results are important since they indicate that the hemp fibers seem to have completely deteriorated and disintegrated in the soil specimens within a period of 30 days starting from soil sample preparation. It should be noted that the soil specimens in this time period were not subjected to any external moisture (on top of the as-compacted water content) and were wrapped with a rigid PVC tube with the same diameter to limit any swelling/softening of the specimens during the waiting period.

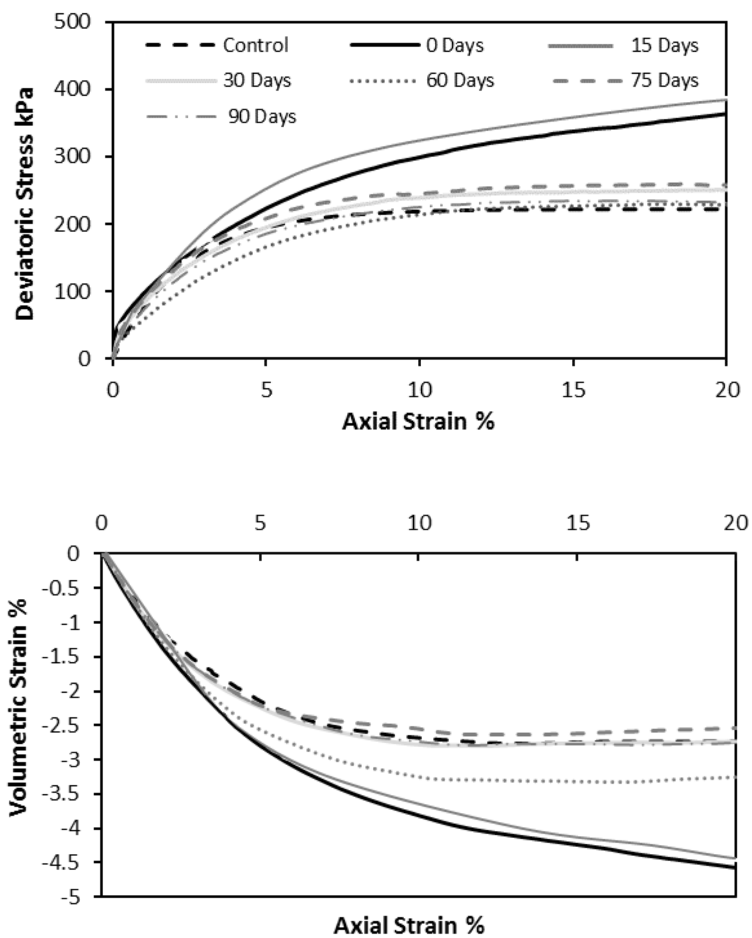


Figure 7.3 Variation of deviatoric stress and volumetric strain with axial strain at different ages under 100 kPa confining pressure

7.3.2 Fibers tensile strength

To shed light on any possible degradation that is occurring in the hemp fibers with time, a typical control clay specimen was prepared as was the case in Series 1 tests. The control clay specimen was then cut into two parts and fiber threads that were around 7 cm long (Fig. 7.4) were embedded between the two parts and the whole system was then sealed with Nylon sheets and included in a PVC tube as was the case with the durability specimens. The objective behind this exercise is to leave the sandwiched fibers for 90 days in the soil specimen and then measure their tensile strength after this prolonged period.

After 90 days, it was noticed that the fibers decreased in mass from 1.4 grams to 1 grams and decomposed and degraded to smaller filaments. The fibers were so fragile and easily breakable by hand with almost zero force applied. Pictures showing the state of the fibers after 90 days are shown in Fig. 7.5. It could be concluded the fibers that were sandwiched within a compacted clay specimen for 90 days degraded to a state that reduced their tensile strength almost to zero. These results explain the lack of improvement in the stress-strain response for the hemp-reinforced specimens that were tested in Series 3 after a waiting period that exceeded 30 days.

Figure 7.4 Embedding fibers in a soil matrix

Figure 7.5 Hemp fibers after 90 days age

7.3.3 Improvement in deviatoric stress

The percent of improvement in the deviatoric stress at failure due to the different ages was calculated for all the tests. Failure was defined in terms of the peak deviatoric stress or at 20% axial strain, whichever was reached earlier. The variation of deviatoric stress and the resulting percent improvements with time are presented in figure 7.6. Results on Fig. 7.6 indicate that the deviatoric stresses decreased with increasing time (after 15 days) from 363 to 233 kPa. Consequently, improvement levels decreased from 64% to 5.7%. This drop in the improvement could be correlated to the hemp fibers degradation with time.

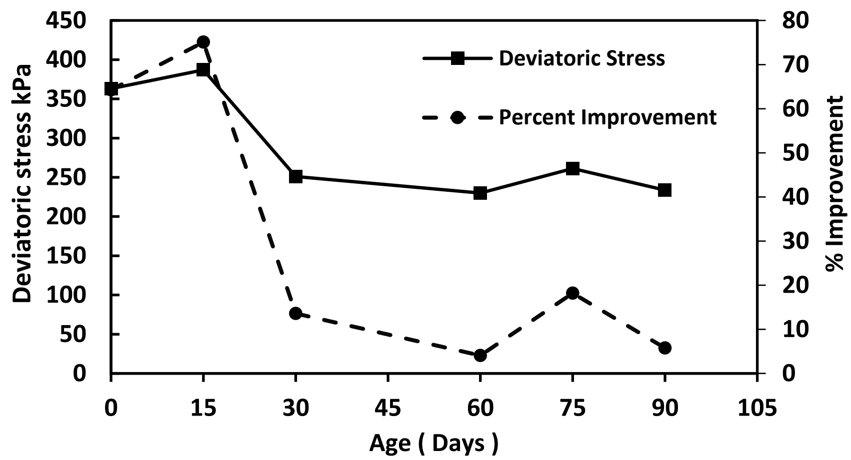


Figure 7.6 Variation of deviatoric stress and percent Improvement at failure with age in days

7.3.4 Failure Modes

The mode of failure of the tested specimens is studied and presented in figures 7.7. Failed specimens are shown from tests conducted at 1.25% fiber contents for clays compacted at a water content of 18% at 100 kPa applied confining pressure. The pictures of the failed control clay specimen from series 1 (0 age) show clear signs of bulging at the middle portion of the sample (Fig. 7.7 a). Initially the bulging in specimens that were reinforced with fibers was found to be uniformly distributed along the height of the specimen indicating that the fibers played a significant role in transferring stresses and strains within the specimen, leading to an overall reduction in the intensity of localized bulging (Fig. 7.7b). This change in the mode of failure in fiber-reinforced specimens resulted in improvements in the ductility of the stress-strain response. However, as the age of the samples prior to testing increases from 0 to 90 days the intensity of localized bulging started to increase again reaching a similar mode of failure as the control specimen. This means that the fibers are not functioning as they functioned initially.

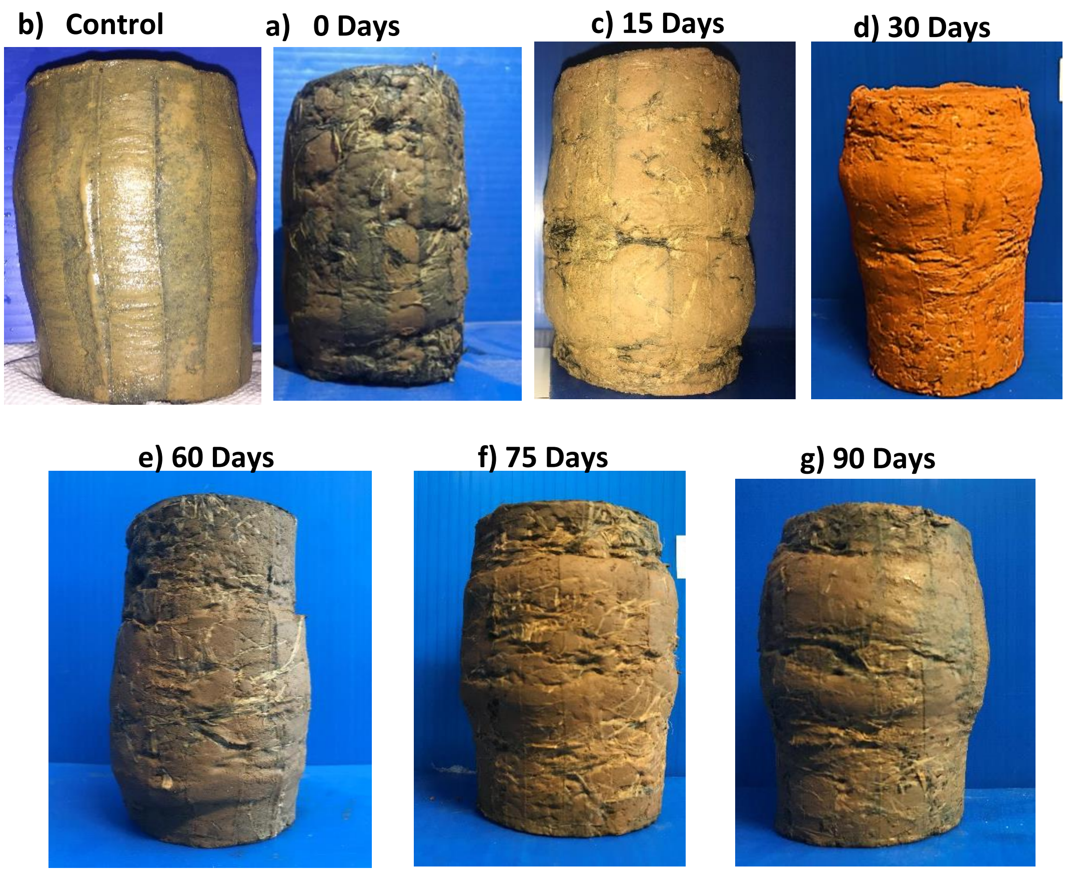


Figure 7.7 Durability samples failure modes

7.4 Summary

Based on the results of consolidated drained triaxial durability tests (Series 3) that were conducted on reinforced specimens compacted at 18% water content with 4cm long fibers and 1.25% fiber content, the following conclusion can be drawn:

1. Samples age has a detrimental effect on strain hardening response. When increasing the samples age the strain hardening response will decrease.
2. After 30 days, fibers were found to be inefficient increasing the deviatoric stress of the fiber-reinforced specimens with the measured stress at failure approaching the strength of the control clay specimen.
3. It is very likely that the drop in the improvement is associated with the hemp fibers degradation with time.

Chapter 8

COMPARISON WITH PREVIOUS STUDIES

8.1 Introduction

One of the main objectives of the experimental program was to compare the drained response of hemp-reinforced clay that is compacted using the conventional impact Standard Proctor compaction methods with the undrained response of the same material and prepared in the same way which was done by Abou Diab (2016). In the most recent study, Abou Diab (2016) conducted a UU Triaxial laboratory program where samples of natural clay mixed with hemp fibers at different fiber contents (only 1% ,1.25% and 1.5% will be presented in this study for comparison purposes) were prepared at different moisture content (14% 18% and 20%) and compacted using the standard proctor procedure. Samples were then sheared undrained at a strain rate of 1% per minute with a confining pressure of 20 and 100 kPa. The upper range of confining pressures was limited to 100kPa unlike our study, which was 200 kPa. All tests were terminated at a maximum axial strain of about 20%.

8.2 Comparing the Results of this study with Abou Diab (2016)

The comparison will be divided into two parts (comparison with Series 1 & Series 2). It will include a description of the stress-strain relationships, an interpretation of the variation of the volumetric strain with axial strain, an analysis of the effect of various parameters on the variation of the deviatoric stress at failure and its degree of improvement.

8.2.1 Comparing results of Series 1 with Abou Diab (2016)

Figure 8.1 shows the variation of the deviatoric stress and volumetric strain with axial strain for control clay specimens and for specimens reinforced with 1.25% fiber contents and compacted at three water contents: 14%, 18%, and 20%. In this figure, Abou Diab (2016) test results (UU tests) are plotted with the results of Series 1 (CD results) for a confining pressure of 20 kPa. The comparison for the tests conducted at 100 kPa are presented in Figure 8.2.

Results on Figs. 8.1 and 8.2 indicate that for identical specimens, the unconsolidated undrained shear strength of both control and reinforced clay specimens is greater than the consolidated drained strength. This result is expected for unsaturated compacted clay specimens whereby the initial matric suction and the possible generation of negative water pressure during undrained shearing may increase the effective stress of the clay resulting in an improved strength in unconsolidated as-compacted clay specimens compared to specimens that have been back-pressure saturated, consolidated, and sheared under drained conditions.

It is interesting to note that the contrast in the stress-strain response between undrained and drained tests increases as the sample becomes more and more unsaturated (specimens that are compacted dry of optimum). In addition, the difference in strength due to drainage and shearing conditions seems to be the largest for fiber-reinforced specimens. This indicates that saturating and consolidating the fiber-reinforced specimens and shearing them slowly (for up to two weeks) have a detrimental effect on the hemp-clay interface resistance and possibly on the tensile strength of the fibers themselves. These observations are applicable for both confining pressures analyzed (20 kPa and 100 kPa) as indicated in Figs. 8.1 and 8.2.

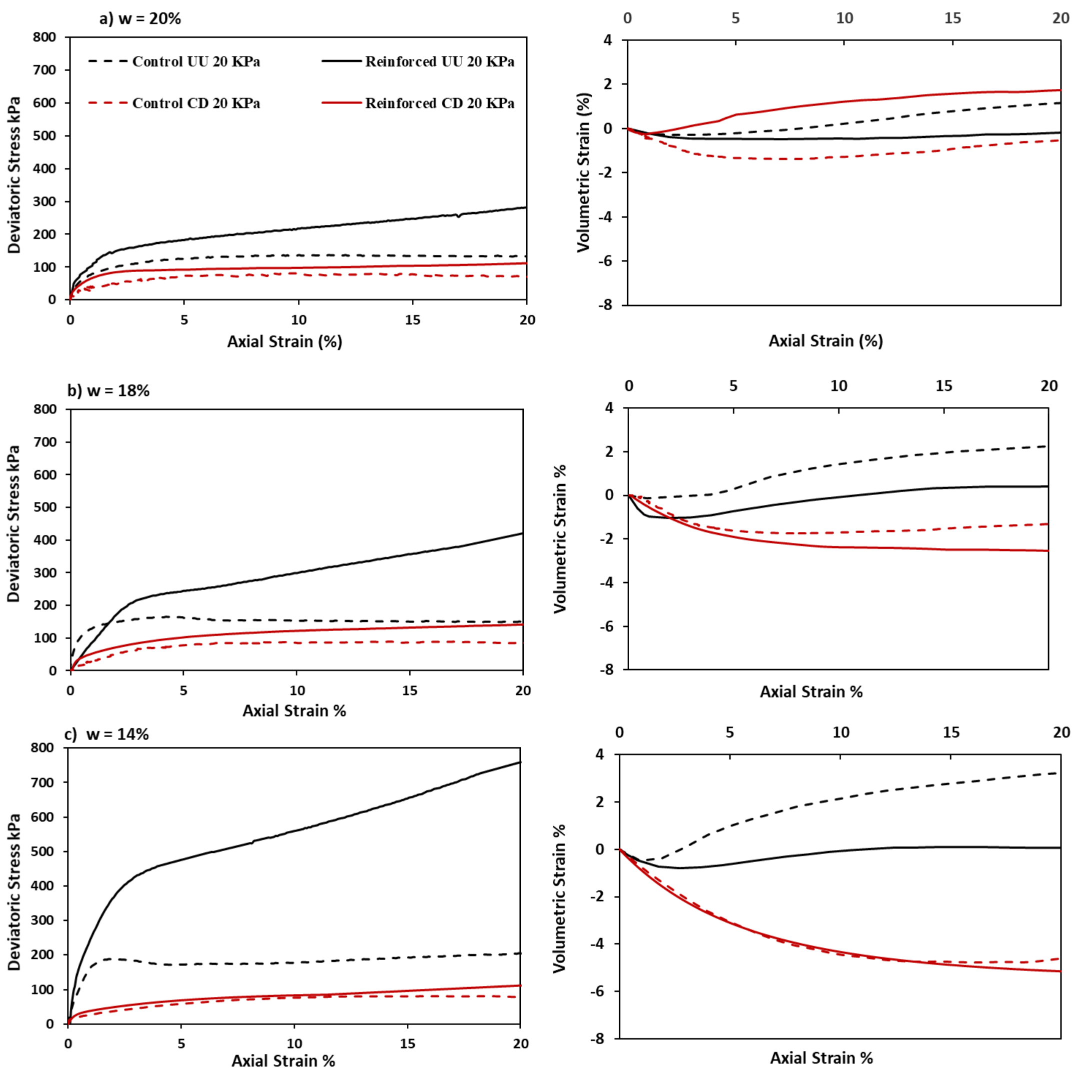


Figure 8.1 Variation of deviatoric stress and volumetric strain with axial strain at different water contents at confining pressure = 20 kPa for both UU (Abou Diab 2016) and CD tests

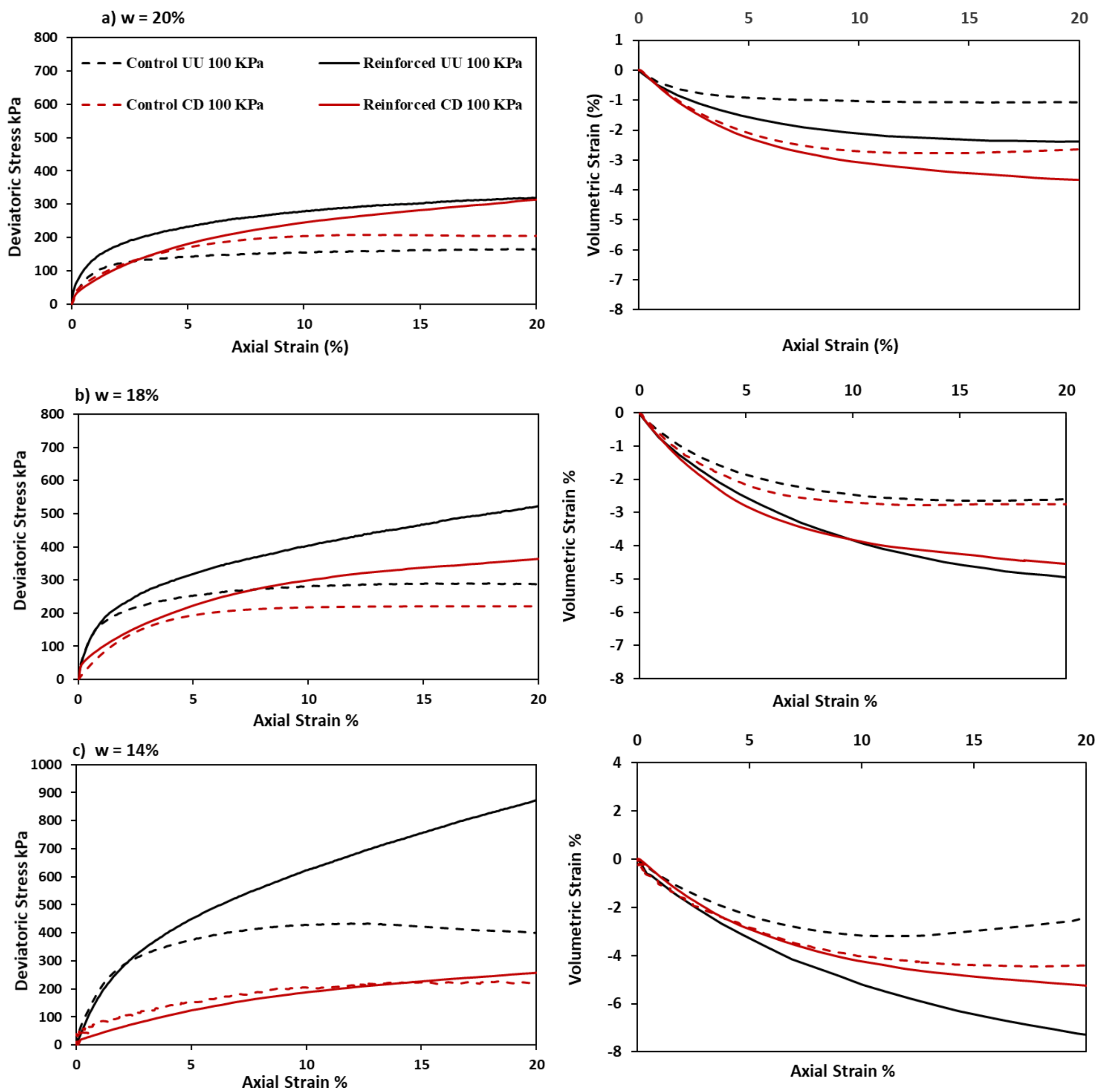


Figure 8.2 Variation of deviatoric stress and volumetric strain with axial strain at different water contents at confining pressure = 100 kPa for both UU (Abou Diab 2016) and CD tests

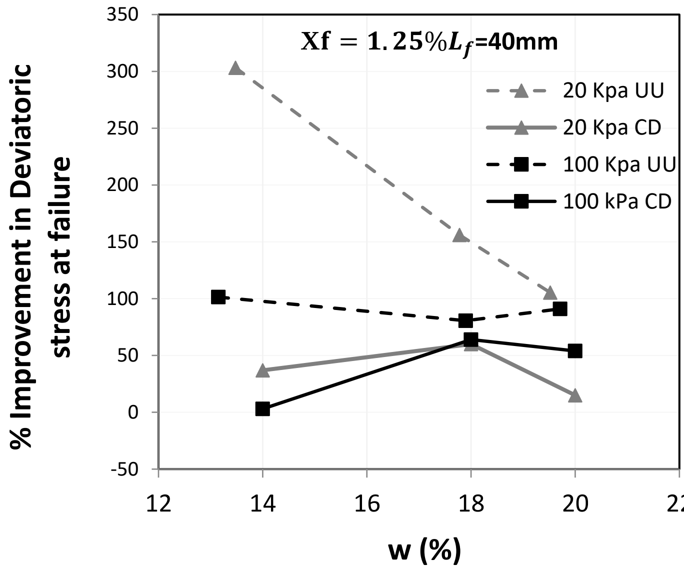
The variation of the percent improvement in the deviatoric stress at failure of the fiber-reinforced clay specimens versus fiber content at various water contents is shown in Fig. 8.3 for both Series 1 (CD) and Abou Diab 2016 (UU) test results. The improvement in deviatoric stress at failure of fiber-reinforced specimens at different moisture contents

(14%, 18%, and 20%) is measured in reference to the deviatoric stress at failure of the control samples tested at the same moisture contents of 14%, 18%, and 20%, respectively

For CD testing, the results on Fig. 8.3 indicate that improvements in the order of 15% to 54% were observed in the deviatoric stresses at failure for specimens compacted wet of optimum (w=20%). For specimens that were compacted at optimum (w=18%), the fibers were more efficient at improving the deviatoric stress with improvement levels ranging from 54% to 64%. For specimens prepared at dry of optimum (14%) the improvement levels ranged from 3% to 37%.

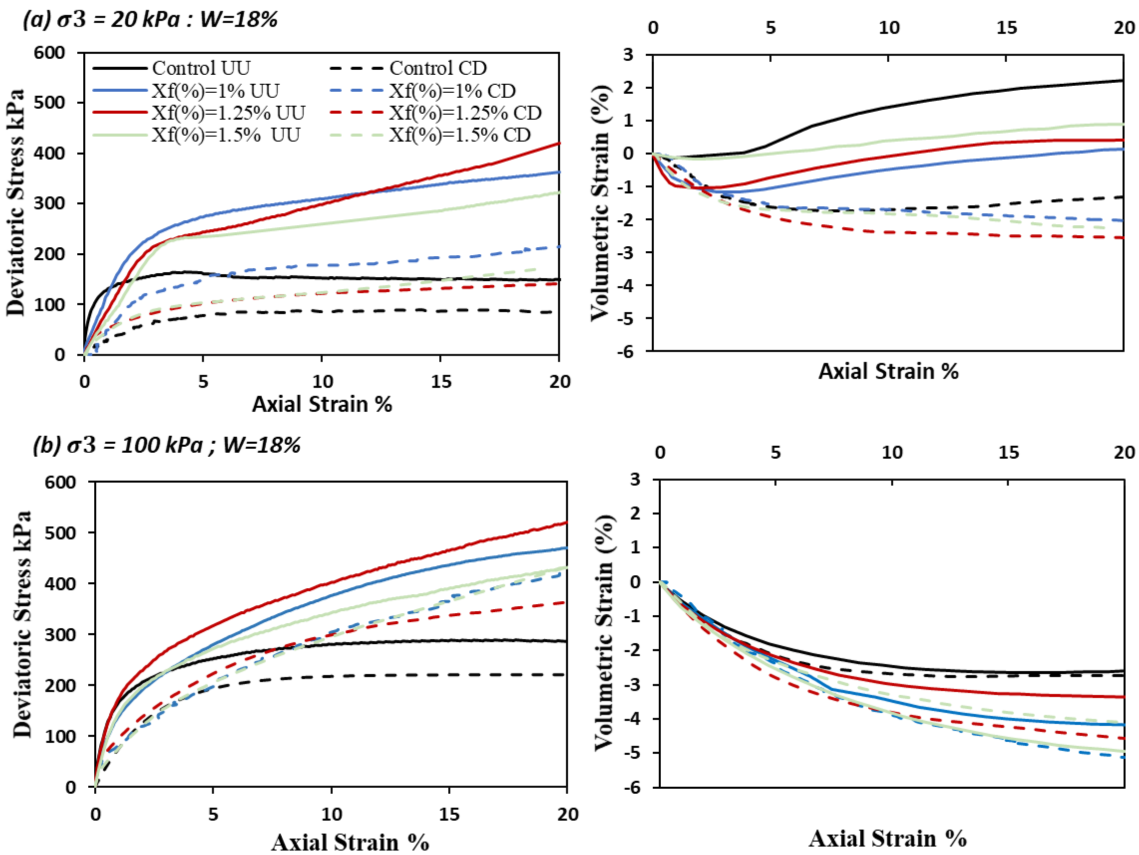
On the other hand, improvements in the order of 100% were noticed for fiber-reinforced clay sheared in an undrained triaxial setup (Abou Diab 2016). These improvements are so high compared to the drained triaxial setup. In the undrained tests the improvements are more evident for samples compacted dry of optimum (14%) and at 20 kPa confining pressure; however in the drained tests, the improvements are more evident at optimum(18%) and at 100 KPa confining pressure. These results confirm the hypothesis that in undrained tests, the initial matric suction and the possible generation of negative pore pressure (both are maximum at w = 14% and 20 kPa confining pressure) may have governed the efficiency of the fibers and dominated the shear strength response, whereas in drained tests, confining pressure and dry density (both maximum at optimum water content = 18% and largest confining pressure of 100 kPa) may have governed the response.

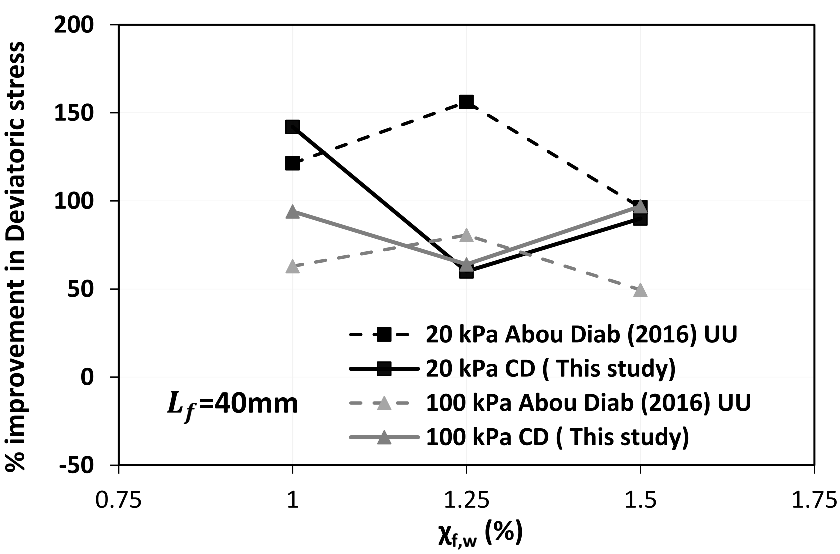
It is interesting to note that in both drained and undrained test setups, a slight decrease in the efficiency of the fibers at larger water contents (20%) was observed and could be related to the reduction of fiber/clay load transfer because of the lubricating effect of water (Attom et al. 2009) and to the alignment of clay particles when they are compacted at wet of optimum water content as compared to the flocculated structure of soil at dry of optimum.



8.2.2 Comparing results of Series 2 with Abou Diab (2016)

The fiber-reinforcement parameters of series 2 (1% & 1.5%) were based on the results of Abou Diab (2016) who showed that the optimal fiber-reinforcement scheme for undrained loading conditions consisted of 4cm long fibers and 1.25% fiber content by weight. As a result, specimens reinforced with 4cm long fibers but at different fiber contents (smaller and greater than 1.25%) were tested to investigate whether the optimum combination of fiber length and content is affected by the drainage conditions. The stress-strain results and the observed improvements in strength of Abou Diab (2016) for undrained tests were compared with the CD results on Figs. 8.4 and 8.5 for the three fiber contents tested. The results of all tests pertain to the optimum water content of 18%.


(a) σ3 = 20 kPa : W=18%
(b) σ3 = 100 kPa ; W=18%
Deviatoric Stress kPa
Volumetric Strain (%)
Axial Strain %
Control UU
Control CD
Xf(%)=1% UU
Xf(%)=1% CD
Xf(%)=1.25% UU
Xf(%)=1.25% CD
Xf(%)=1.5% UU
Xf(%)=1.5% CD


% improvement in Deviatoric stress
χf,w (%)
Lf=40mm
20 kPa Abou Diab (2016) UU
20 kPa CD (This study)
100 kPa Abou Diab (2016) UU
100 kPa CD (This study)

Results for UU tests indicate that fibers are efficient at increasing the deviatoric stress at failure in the fiber content range of 1% to 1.25%, with improvements increasing from 62% to 81% and from 121% to 156% at 100 KPa and 20 kPa confining pressure , respectively. Tests conducted at 1.5% fiber content resulted in a drop of the deviatoric stress at failure and thus in the degree of improvement as compared to tests conducted at fiber content of 1.25%, indicating a threshold fiber content at which a peak improvement in strength is observed. Results for CD tests indicate that improvements in the order of 142% to 69% were observed in the deviatoric stresses at failure for specimens compacted at 1% fiber content at different confining pressures. For specimens that were compacted at 1.25% fiber content a drop in the improvement levels was observed ranging from 54% to 64%. For specimens prepared at 1.5% fiber content the improvement levels increased slightly and ranged from 70% to 97%, indicating that a fiber content of 1% may be the optimal fiber content in consolidated drained tests.

8.3 Summary

The results presented in this chapter are somehow coherent with Abou Diab et al. (2016) regarding the stresses reached at respective strains which appear to decrease as the water content at compaction of the various samples increased. This may be attributed to the compaction effect that leads to an aligned structure of the clay particles at wet of optimum and to the lubricating effect of water that reduces the fiber/clay load transfer (Attom et al., 2009). Moreover, the magnitude of the volumetric strains decreased as the degree of saturation increased for the higher compaction water contents used. Further, the results of Abou Diab et al. (2016) are also in line with the CD results with regards to reinforcement effect of hemp fibers, which added ductility to the mode of failure by arresting the development of shear planes and reducing concentrated bulging in control clay specimens in both drained and undrained tests.

With regards to the improvement in strength due to the addition of fibers, the comparison between the undrained and drained results indicated that in the undrained tests the improvements are more evident for samples compacted dry of optimum (14%) and at 20 kPa confining pressure; however in the drained tests, the improvements are more evident at optimum (18%) and at 100 KPa confining pressure. These results confirm the hypothesis that in undrained tests, the initial matric suction and the possible generation of negative pore pressure (both are maximum at w = 14% and 20 kPa confining pressure) may have governed the efficiency of the fibers and dominated the shear strength response, whereas in drained tests, confining pressure and dry density (both maximum at optimum water content = 18% and largest confining pressure of 100 kPa) may have governed the response.

Finally, a comparison between the threshold/optimum fiber content observed in undrained tests and drained tests indicated that the threshold is 1.25% in undrained tests and 1.0% in drained tests. It should be noted however, that the differences between the stress-strain responses for specimens reinforced with 1% and 1.25% in both drained and undrained tests were relatively small and may have been affected by slight differences or uncertainties in dry density, fiber distribution, and fiber orientation. This leads to the conclusion that the optimum fiber content in both drained and undrained tests is in the range of 1.0 to 1.25%.

Chapter 9

CONCLUSIONS AND RECOMMENDATIONS

9.1 Introduction

In this Chapter, the conclusions and recommendations that resulted from this research are presented. The work presented in this thesis aimed at investigating the drained shear strength of natural clays reinforced with Hemp fibers, with primary focus on long term stability applications. The objective was to enhance our understanding of the mechanical characteristics of the composite material and thus promote its use as a sustainable, environmentally “friendly” soil improvement technique. The viability of natural Hemp fibers as construction material for geotechnical applications was adopted as one of key target elements of our research and has added local resonance given the Lebanese context and the widespread cultivation of illegal crops, for which industrial Hemp could provide a viable alternative. The migration of farming practices towards such a crop with many proven industrial applications, along with the newly established construction-related uses, will have a positive impact on the local community of farmers and small agri-businesses. It is critical to note here however, that the works presented in this thesis and the associated findings have potential impact and applications, which extend well beyond the local context into regional and international realms.

The use of natural fibers in soil reinforcement applications was considered as particularly interesting and relevant for the case of natural clayey soils .As such, the work presented in this thesis focused on fiber-reinforced clays (FRC).

9.2 Summary of the findings and recommendations:

Based on the results of consolidated drained triaxial tests (Series 1, 2 & 3) that were conducted on control clay specimens and hemp-reinforced specimens, the following findings, conclusions, and recommendations can be drawn:

- The inclusion of hemp fibers in compacted clay improved the ductility. The addition of fibers reduced concentrated bulging and resulted in a stress-strain response that is characterized by strain hardening at axial strains exceeding 10%.
- The deviatoric stress at failure of fiber-reinforced specimens was larger than that of control specimens with recorded improvements of up to 60%. These improvements were found to be more pronounced in clay specimens that were compacted around optimum.
- The effect of fiber inclusion on the stiffness was dependent on the confining pressure. No significant improvements in stiffness were observed for the tests conducted at high confining pressure of 100 and 200 kPa. For tests conducted at 20 kPa, improvements of up to 80% were observed for fiber-reinforced specimens.
- Fibers are efficient at arresting and reducing the dilative tendency of the clay.
  The incorporation of fibers reduced the dilative tendency of the clay at low confining pressure and induced additional contractive volume changes in reinforced specimen.
- The inclusion of hemp fibers within compacted clay specimens increased the drained cohesion and friction angle of the clay. The increase in the cohesive intercept (about 9 kPa) was not affected by the compaction water content. However,

the increase in drained friction angle was more pronounced (around 8 degrees) in the specimen compacted at optimum (w=18%) in comparison to the specimen compacted wet of optimum w=20% and dry of optimum w=14% (about 3 degrees).

- Samples age has a detrimental effect on durability of the fibers and thus on their efficiency in increasing the load carrying response of hemp-reinforced clay specimens. In samples that were allowed to rest for more than 30 days prior to triaxial testing, fibers were found to be inefficient at increasing the deviatoric stress of the fiber-reinforced specimens with the measured stress at failure approaching the strength of the control clay specimen. It is very likely that the drop in the improvement is associated with the hemp fibers degradation with time.
- It is recommended to further develop treatment techniques which might prolong the durability of natural fibers even under severe conditions.

In conclusion, additional investigation in future research studies should target the treatment techniques that would further prolong the durability of hemp fibers in order to keep this sustainable soil improvement measure as a feasible and promising soil improvement option for long term applications involving fiber-reinforced clay systems.

APPENDICES

APPENDIX 1

CD TRIAXIAL TEST RESULTS

Table 10.1 Control Samples Series 1 test data

Test No.	Compaction Method	No. of layers	Compaction effort / layer	Fiber type	Target w (%)	σ_3 (kPa)	$X_{f,w}$ (%)	L_f (mm)	Actual w (%)	γ_d initial (KN/m^3)	γ_d After Consolidation (KN/m^3)	γ_d after shearing (KN/m^3)	σ_d at failure (kPa)	Strain at failure (%)	Impr. (%)	$S_{r,i}$ (%)	$S_{r,c}$ (%)	$S_{r,F}$ (%)
1	Impact	3	14	-	14	20	-	-	14.18	16.61	16.61	17.41	81.53	13.78	-	65.0	86.0	84
2	Impact	3	14	-	18	20	-	-	18.13	17.13	17.14	17.41	89.58	14.19	-	89.5	98.3	98.2
3	Impact	3	14	-	20	20	-	-	20.68	16.57	16.56	16.78	98.98	10.27	-	93.2	100	100
4	Impact	3	14	-	14	100	-	-	14.39	16.56	16.85	17.62	248	18.63	-	64.8	71.0	67.1
5	Impact	3	14	-	18	100	-	-	18.3	17.44	17.7	18.2	221	20	-	95.3	96.7	96.4
6	Impact	3	14	-	20	100	-	-	19.99	17.15	17.36	17.85	208	12.2	-	99.0	100	100
7	Impact	3	14	-	14	200	-	-	15.46	16.87	18.06	18.89	400	20	-	73.3	86.2	84
8	Impact	3	14	-	18	200	-	-	17.73	17.29	17.93	18.51	406	20	-	90.1	98.4	98.2
9	Impact	3	14	-	20	200	-	-	19.67	17.03	17.87	18.53	407	20	-	95.7	99.57	99.5

Table 10.2 Reinforced Samples Series 1 test data

Test No.	Compaction Method	No. of layers	Compaction effort / layer	Fiber type	Target w (%)	σ_3 (kPa)	$X_{f,w}$ (%)	L_f (mm)	Actual w (%)	γ_d initial (KN/m^3)	γ_d After Consolidation (KN/m^3)	γ_d after shearing (KN/m^3)	σ_d at failure (kPa)	Strain at failure (%)	Impr. (%)	$S_{r,i}$ (%)	$S_{r,c}$ (%)	$S_{r,F}$ (%)
10	Impact	3	18	Hemp	14	20	1.25	40	13.94	15.78	15.77	16.63	112	20	37	55	79	76
11	Impact	3	18	Hemp	18	20	1.25	40	17.88	16.04	16.08	16.49	143	20	60	74	80	78
12	Impact	3	18	Hemp	20	20	1.25	40	19.53	16.27	16.23	16.74	112.84	20	14	84	87	99
13	Impact	3	18	Hemp	14	100	1.25	40	13.74	15.93	16.27	17.17	255.89	20	3	56	73	69
14	Impact	3	18	Hemp	18	100	1.25	40	18.5	16.4	17.53	18.38	363	20	64	85	90	89
15	Impact	3	18	Hemp	20	100	1.25	40	19.86	16.25	16.42	17.04	320	20	54	85	88	87
16	Impact	3	18	Hemp	14	200	1.25	40	15.43	16.49	17.58	18.48	492.6	20	23	69	89	87
17	Impact	3	18	Hemp	18	200	1.25	40	17.94	16.2	17.63	18.54	627	20	54	76	83	80
18	Impact	3	18	Hemp	20	200	1.25	40	19.66	17.42	18.02	18.81	528.63	20	30	100	100	100

Table 10.3 Reinforced Samples Series 2 test data

Test No.	Compaction Method	No. of layers	Compaction effort / layer	Fiber type	Target w (%)	σ_3 (kPa)	$X_{f,w}$ (%)	L_f (mm)	Actual w (%)	γ_d initial (KN/m^3)	γ_d After Consolidation (KN/m^3)	γ_d after shearing (KN/m^3)	σ_d at failure (kPa)	Strain at failure (%)	Impr. (%)	$S_{r,i}$ (%)	$S_{r,c}$ (%)	$S_{r,F}$ (%)
19	Impact	3	17	Hemp	18	20	1%	40	18.57	16.98	17.18	17.63	216.91	20	142.1	93.0	100.0	100.0
20	Impact	3	19	Hemp	18	20	1.50%	40	19.08	16.71	16.72	17.1	170	20	89.8	88.0	99.0	99.0
21	Impact	3	17	Hemp	18	100	1%	40	17.82	16.90	17.09	17.99	431	20	95.0	85.0	94	92
22	Impact	3	19	Hemp	18	100	1.50%	40	17.81	16.49	16.98	17.74	437	20	97.7	79.0	91.0	89
23	Impact	3	17	Hemp	18	200	1%	40	18.49	16.62	17.47	18.34	687.89	20	69.4	84.0	96.0	95
24	Impact	3	19	Hemp	18	200	1.50%	40	17.63	16.48	17.16	18.17	690	20	70.0	78.0	92	90

Table 10.4 Samples compacted at 18% water content from Series 1 Test data

Test No.	Compaction Method	No. of layers	Compaction effort / layer	Fiber type	Target w (%)	σ_3 (kPa)	$X_{f,w}$ (%)	L_f (mm)	Actual w (%)	γ_d initial (KN/m^3)	γ_d After Consolidation (KN/m^3)	γ_d after shearing (KN/m^3)	σ_d at failure (kPa)	Strain at failure (%)	Impr. (%)	$S_{r,i}$ (%)	$S_{r,c}$ (%)	$S_{r,F}$ (%)
2	Impact	3	14	-	18	20	-	-	18.13	17.13	17.14	17.41	89.58	14.19	-	89.5	98.3	98.2
5	Impact	3	14	-	18	100	-	-	18.3	17.44	17.7	18.2	221	20	-	95.3	96.7	96.4
8	Impact	3	14	-	18	200	-	-	17.73	17.29	17.93	18.51	406	20	-	90.1	98.4	98.2
11	Impact	3	18	Hemp	18	20	1.25	40	17.88	16.04	16.08	16.49	143	20	60	74	80	78
14	Impact	3	18	Hemp	18	100	1.25	40	18.5	17.27	17.53	18.38	363	20	64	94	100	100
17	Impact	3	18	Hemp	18	200	1.25	40	17.94	17	17.63	18.54	627	20	54	87	98	97

Table 10.5 Reinforced Durability Samples Series 3 test data

Test No.	Age (Days)	Compaction Method	No. of layers	Compaction effort / layer	Fiber type	Target w (%)	σ_3 (kPa)	$X_{f,w}$ (%)	L_f (mm)	Actual w (%)	γ_d initial (KN/m^3)	γ_d After Consolidation (KN/m^3)	γ_d after shearing (KN/m^3)	σ_d at failure (kPa)	Strain at failure (%)	Impr. (%)	$S_{r,i}$ (%)	$S_{r,c}$ (%)	$S_{r,F}$ (%)
25	15	Impact	3	18	Hemp	18	100	1.25%	40	17.81	16.64	16.91	17.71	386.93	20	75.1	81.2	95.7	95.0
26	30	Impact	3	18	Hemp	18	100	1.25%	40	17.61	17.05	17.24	17.73	251.00	20	13.6	86.0	94.3	93.8
27	60	Impact	3	18	Hemp	18	100	1.25%	40	17.50	16.38	16.64	17.21	229.97	18.09	4.1	76.6	91.2	90.4
28	75	Impact	3	18	Hemp	18	100	1.25%	40	17.31	16.72	16.86	17.29	261.17	18.71	18.2	80.1	86.5	85.5
29	90	Impact	3	18	Hemp	18	100	1.25%	40	17.71	16.63	16.84	17.32	233.69	16	5.7	80.6	86.0	84.9

Table 10.6 zero days age Samples from Series 1 Test data

Test No.	Compaction Method	No. of layers	Compaction effort / layer	Fiber type	Target w (%)	σ_3 (kPa)	$X_{f,w}$ (%)	L_f (mm)	Actual w (%)	γ_d initial (KN/m^3)	γ_d After Consolidation (KN/m^3)	γ_d after shearing (KN/m^3)	σ_d at failure (kPa)	Strain at failure (%)	Impr. (%)	$S_{r,i}$ (%)	$S_{r,c}$ (%)	$S_{r,F}$ (%)
5	Impact	3	14	-	18	100	-	-	18.3	17.44	17.7	18.2	221	20	-	95.3	96.7	96.4
14	Impact	3	18	Hemp	18	100	1.25	40	18.5	17.27	17.53	18.38	363	20	64	94	100	100

where

w: water content; σ_3: confining pressure; $\chi_{f,w}$: gravimetric fiber content; L_f: fiber length; γ_d: dry unit weight; σ_d: deviatoric stress at failure; $S_{r,I}$: initial degree of saturation; $S_{r,c}$: saturation after confinement; $S_{r,F}$: final degree of saturation; Impr.: Improvement in deviatoric stress, in percentage; T: tamps

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