Effect of Polypropylene Fiber and Posidonia Oceanica Ash on the Behavior of Expansive Soils

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ABSTRACT

This study presents an experimental study evaluating the effect of synthetic and natural additives on the behavior of an expansive soil from a local deposit. The synthetic additive used is polypropylene fiber which constitutes the major part of this research work. The initial phase of the experimental program includes the study of the effect of polypropylene fiber on maximum dry density and optimum moisture content with different fiber inclusions. Dynamic compaction tests have been conducted on an expansive soil sample with percentages of 0%, 0.5%, 0.75%, and 1% polypropylene fiber additions (by dry weight of the soil). The specimens used for volume change and strength tests were conducted on specimens prepared by static compaction at optimum water content and maximum dry density obtained by dynamic compaction of Standard Proctor procedure. The second phase of the experimental program focuses on the strength and volume change behavior of unreinforced and reinforced specimens. In the third phase, the effect of the polypropylene fiber on soil- water characteristic curve is studied. Finally it is concluded that mitigation of expansive soils using polypropylene fiber might be an effective method in enhancing the compression, tension and Air entry values of the subsoils on which roads and light buildings are constructed. The natural additive used in this study is abundantly found Posidonia oceanica (sea weed) which is carried and deposited at the shores all along the coastline of Cyprus. The weed is burnt and its ash is used to investigate the potential effect on physical properties and swell behavior of the expansive soil used in this study. Despite the difficulties encountered in representative specimen preparation due to random distribution of fiber filaments, it is observed that there is a future prospect in the use of this environmental friendly additive for soil mitigation.

Keywords: polypropylene fiber, compressive strength, static compaction, dynamic compactions, Soil water characteristic curve,

ÖZ

Bu çalışma sentetik ve doğal katkıların yerel şişen zeminlerin davranışına olan etkilerini incelemektedir. Çalışmanın önemli bir kısmı sentetik katkı olan polipropilen fiberin etkisini içermektedir. Araştırmanın ilk aşaması farklı yüzdeliklerde polipropilen fiber katkının maksimum kuru birim hacim ağırlığı ve optimum su muhtevasına etkisini içerir. Şişen zeminlerle karıştılan 0%, 0.5%, 0.75%, and 1% oranlarında polipropilen fiberle elde edilen karışım kompaksiyon deneylerine tabi tutulmuş ve her karışımın maksimum kuru birim hacim ağırlığı ve optimum su muhtevası elde edilmiştir. Araştırmanın ikinci aşamasında birinci aşamada bulunan maksimum kuru birim hacim ağırlığı ve optimum su muhtevasında statik kompaksiyon yöntemi ile sıkıştırılmış numuneler hazırlanmıştır. Bu numuneler hacim değişikliği (şişme-büzülme ve kopressibilite) ve serbest basınç deneylerinde kullanılarak katkısız ve katkılı zemin numunelerinin davranışları irdelenmiştir. Üçüncü aşamada ise polipropilen fiberin zemin-su karakteristik eğrisine olan etkisi incelenmiş ve sonuç olarak zeminlerin iyileştirilmesinde polipropilenin etkili olduğu gözlemlenmiştir.

Doğal katkı malzemesi olarak bir çeşit deniz bitkisi olan ve dalgaların sahile taşıyıp çevre kirliliği yarattığı Posidonia oceanica (PO) kullanılmıştır. PO 550 °C derecede yakılarak elde edilen külün şişen zeminle %5 ve %10 oranlarında karışımı incelenmiş ve potansiyel bir katkı malzemesi olabileceği, ayrıca çevre kirliliği yaratan bu malzemenin geri dönüşümünün sağlanmış olabileceği sonucuna varılmıştır.

Anahtar kelimeler: polipropilen fiber, serbest basınç, statik kompaksiyon, dinamik kompaksiyon, zemin-su karakteristik eğrisi.

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AEV	Air Entry Value
ASTM	American Society for Testing and Materials
G_s	Specific gravity
k	Coefficient of hydraulic conductivity
k_s	Saturated coefficient of hydraulic conductivity
LL	Liquid Limit
OMC	Optimum Moisture Content
PL	Plastic Limit
PO	Posidonia oceanica
$\Delta H/H_0$	Swell potential
q_u	Unconfined compressive strength
S	Degree of saturation
SWCC	Soil water characteristic curve
w_o	Optimum water content
w	Gravimetric water content
ψ	Suction
ψ_o	Osmotic suction
ψ_t	Total suction
ψ_m	Matric suction
ρ_d	Maximum dry density
θ	Volumetric water content

Chapter 1

INTRODUCTION

1.1 Aim and scope of the study

This research explores the importance of using fibers to reinforce expansive soils. Mitigation and stabilization of expansive soils are the focus of this study. This study emphasizes the effect of polypropylene fibers on strength behavior, one dimensional consolidation, one-dimensional swell, shrinkage, soil-water characteristic curve and of expansive soils. Another scope of this study is to monitor the influence of Posidonia oceanica ash on soil properties based on the changes of physical properties, swell and unconfined compressive strength.

1.2 Background

1.2.1 Expansive Soil and its Stabilization

Expansive soils are the main cause of damages to variety of civil engineering structures such as spread footings, roads, highways, and airport runways. Expansive soils are usually found in arid and semiarid regions of tropical and temperate climate zones (Abduljauwad, 1993). Swelling and shrinking behavior of the expansive soils is caused by the montmorillonite mineral (Chen, 1988).

Stabilization by chemical additives, pre-wetting, compaction control, preloading, water content prevention are general ground improvement methods that are the solution of swelling problems (Yucel Guneya et al., 2005).There has been a growing interest in recent years in the influence of chemical modification of soils which upgrades and enhances the engineering properties. The changes of soil properties by adding chemicals such as cement, fly ash, lime, or their combination, often shift the physical and chemical properties of the soil such as the cementation of the soil particles. Especially use of lime admixture has proved to be one of the most economical method for improving the geotechnical properties of expansive soils. Leroueil and Vaughan (1990), Basma and Tuncer (1991), Nalbantoglu and Tuncer (2001), Bilsel and Oncu (2005), Rao and Shivananda (2005) have examined the compressibility behavior of lime-stabilized soils.

According to Gordon and McKeen (1976) cement and lime show different behavior in soil stabilization. Cement contains the necessary ingredients for the pozzolanic reactions, whereas lime can be effective only if there are reactants in the soil.

Recently there is a growing attention to soil reinforcement with different types of fiber. According to Heineck et al. (2005) experimental results gathered in recent years show the potential of different types of fiber in reinforcing problematic soils. In order to wholly understand the strength behavior of fibered and non- fibered soils; Prabakar and Sridhar (2002) has carried out a sequence of experimental works on a non-expansive soil and assessed the suitability of sisal fiber, which is a natural fiber of Agavaceae family traditionally used in making twine and ropes, as a reinforcement material and resulted in a considerable enhancement of the failure deviator stress, as well as shear strength parameters c and φ.

Freilich and Zornberg (2010) observed an increase of shearing strength of the soils with the presence of randomly distributed polypropylene fibers. Polypropylene fiber, which is a kind of thermoplastic polymer, appears to be a great potential for reducing the detrimental effects on buildings, earth retaining structures and roadways induced by expansive soils (Loehr, 2000). However, there is limited research done on fiber reinforcement of fine grained soils, particularly on its effects on compaction characteristics, strength and hydromechanical properties. In this experimental investigation, the aim was to study the effect of soil reinforcement with the use of polypropylene fiber on the improvement of physical and mechanical properties of a clay sample obtained from an expansive clay deposit in Famagusta, North Cyprus. The experimental program was carried out on compacted soil specimens with 0%, 0.5%, 0.75%, and 1% polypropylene fiber additives, and the results of unconfined compression, compaction, and suction measurement tests on 0%, 0.5%, 1% fiber are discussed.

Posidonia oceanica is common seaweed of the Mediterranean Sea, which grows all along the coastal area and forming widespread meadows starting near the water surface to depths of 40m (Duarte, 1991). Among all the aquatic plants, Posidonia oceanica is the most plentiful seagrass type in the basin of Mediterranean sea, approximately covering 40,000 km^2 area of the seabed (Cebrian and Duarte, 2001). The leaf rejuvenation cycle of Posidonia oceanica process typically occurs in fall, when an increase in wave action causes the seaweeds to transport. Indeed, noticeable deposits of Posidonia oceanica leaf usually piles up along the coastal areas (Ott, 1980).

In this study the use of these litters in soil stabilization and in geotechnical engineering will be analyzed. The ash content achieved by oven dried crushed pieces in 550 degree Celsius for the duration of 24 hours has been used to monitor its affect on Atterberg limits, grain size distribution, swell potential, and compressive strength of the soil-PO ash mixture of 5 and 10%.

1.3 Outline of the Thesis

This research includes the sets of experimental works on fiber reinforced expansive soils as well as the analysis of effect of Posidonia oceanica ash on the physical properties of these types of soils.

This study is comprised of five chapters. Chapter 2 includes a literature survey on expansive soils and method of their improvements including chemical stabilization, soil reinforcement, and the combination of both methods. In the last section of this chapter, information on certain topics of unsaturated soils is also given, which includes methods of suction measurement, soil-water characteristic curve, and different empirical models of soil-water characteristic curves.

Description of the materials and methodology used in this study are included in Chapter 3. Chapter 4 contains various data analysis and discussion of the results obtained from measurements of physical properties of reinforced and unreinforced samples. Furthermore, Chapter 5 consists of the overall conclusions of this study together with some recommendations for future work.

Chapter 2

LITERATURE REVIEW

2.1 Expansive Soils

Expansive soils are the main cause of damages to variety of civil engineering structures including spread footings, roads, highways, airport runways, and earth dams constructed with expansive soils. High plastic clays and clay shales, marls, clayey siltstones and saprolites (classified as expansive soils) mainly consist of montmorillonite mineral.

Expansive soils are usually found in arid and semi-arid regions of tropical and temperate climate zones. Shrinkage and swelling behavior of expansive soils due to climate changes cause movements in a foundation, which increase the possibility of damage to the civil engineering structures. Movements in foundations are generally the cause of the major structural damages related to expansive soils. Changes in moments and shear forces occur due to the differential movements which are caused by concentration of loads that were not previously accounted in standard design. There are different types of damages due to constructing on expansive soils. These damages can be classified as appearance of cracks in pavements and floor slabs, beams, walls, and drilled shafts; wedged or misaligned doors and windows; and steel or concrete failure.

Lateral forces may possibly initiate collapsing on the basement and retaining walls, principally in over consolidated and nonfissured soils. The extents of damages to structures are widespread, prejudicing the usefulness of the structure, and influence by environmental conditions. Maintenance and repairing are required expenses that may grossly exceed the original cost of the foundation (USA army technical report, 1983). Problems of expansive soils result from a wide range of factors such as shrinkage and swelling of clay soils resulting from moisture changes, type of the clay size particles, poor surface drainage of the soil strata, resulting from applied load. Other factors include pressure of the backfill soil, soil softening, weather, vegetation the amount of aging (Chen, 1988; Lucian, 1996; and Day, 1999).

The depth of active zone is significant in controlling the swell potential of the soil profile. The region that is near enough to the ground surface is defined as the active zone or seasonal zone in which the soils experience a moisture change due to precipitation or evapotranspiration in cycle with the climate changes (Hamilton, 1977; Day, 1999 and Chen, 1988).

It has been proved that every year in the USA, billions of dollars is spent for repairing the damages caused by expansive clays, more than any other natural hazards (Jones and Holtz, 1973; Chen, 1988 and Day, 1999). Damages related to expansive soils have not been recognized until 1930s.The first observation about soil heaving was observed in 1938 (Chen, 1988). From then the researches on expansive soils have been started. Swelling and shrinking behavior of the expansive soils is caused by montmorillonite mineral.

According to Chen (1988) montmorillonite is made up of a central octahedral sheet, which has a 2 to 1 lattice structure that is occupied by aluminum or magnesium, sandwiched between two sheets of tetrahedral silicon. It consists of three-layer clay mineral which has a structural configuration and chemical makeup, which permits a large amount of water to be adsorbed in the interlayer and peripheral positions on the clay crystalline, resulting in the remarkable swelling of soil (Patrick and Snethen, 1976).

One of the most expansive type of clay mineral is montmorillonite and its structural formula is Al4Si8O20 $(OH)_{4n}$(H2O). Montmorillonite mineral has the exchange capacity of is 80 ~ 150meq per 100 g (Li et al., 1992). Expansive soils are distributed all around the world. Northern Cyprus is one of the countries that existence of expansive soils has been reported in recent papers and conferences. As Jones and Holtz (1973); Chen (1988) stated this type of soil is named as “hidden hazard” that cause loss of millions of dollars every year in U.S.A. Many factors can influence the behavior of the soils; these factors are as follows: the existence of type and quantity of minerals, the specific surface area, soil structure and exchangeable cations’ valency has an effect on the mechanism of the swelling (Mitchell, 1993).

Expansive soils are made up of clay particles that result from the alteration of parent materials. Alteration takes place by several processes: weathering, diagenesis, hydrothermal action, neoformation, and post depositional alteration (Grim, 1968). Most clay minerals are transported by air or water to areas of accumulation. Once deposited, the materials are subjected to the local conditions of accumulation (overburden), followed by erosion which makes up the geologic stress history of the materials (Tourtelot and Harry, 1973).

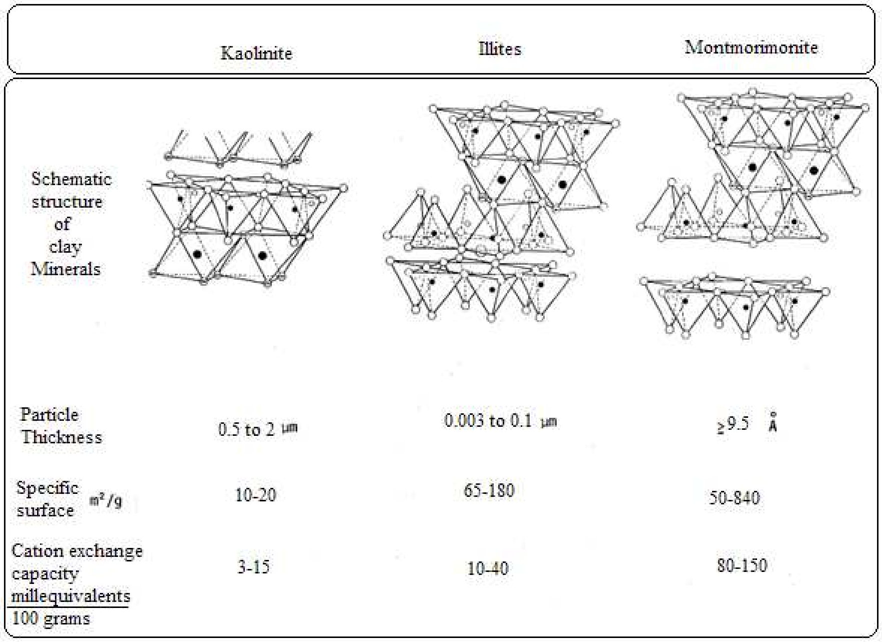

Figure 2.1: Schematic diagram and properties of clay minerals

2.2 Stabilization of Expansive Soils

Soil stabilization with the use of chemical additives, pre-wetting, Compaction control, preloading, water isolation is common ground improvement methods that are the solution of swelling problems (Guney et al., 2006). Among stabilization techniques chemical stabilization is the most frequently used since it provides fast, efficient, repeatable, and reliable result in improving soil properties (Hausmann, 1990). Most of the researches have been on chemical stabilization of expansive soils by cement, lime, fly ash, slag, and bituminous materials.

2.2.1 Chemical Stabilization

2.2.1.1 Lime

Lime stabilization is an effective method to stabilize expansive soils. The aim of lime treatment is to strengthen and minimize the volume change of soil in railroad beds, pavement subgrades, and slopes. This treatment is not always successful because the usefulness depends on the reactiveness of the soil with lime and the distribution of lime mixed with the soil (USA Army Technical Manual, 1983).

Depending on the composition of the soil, the reactions that occur between lime and soil can be as follows: ion exchange, flocculation, carbonation, and pozzolanic reaction. Cation exchange and agglomeration/flocculation reactions occur when lime is added to the soil. After mixing this reduction in plasticity and improvement in the workability of practically all fine-grained soils occur immediately (Thompson, 1964).

Stabilization of clayey soils with lime or cement can improve subgrade properties even at lower cost than removing or replacing material or increasing thickness of the base to reduce subgrade stress (Prusinski and Bhattacharja, 1999). Due to this reason many researchers have focused on stabilization by use of lime or cement in 19^{th} century. Series of laboratory tests have been performed by Locat et al. (1996) in order to predict the mechanical behavior of dredged sediments used in reclamation projects. He has observed a linear relationship between preconsolidation pressure and lime concentration and curing time. He noticed an increase in hydraulic conductivity by introducing lime because of flocculation reaction and formation of secondary minerals.

Kelley (1976) identified soil layers that has been stabilized with lime can perform very well and can survive with high strength properties for even more than 40 years. Extensive experimental study by Thompson and Dempsey (1969) and Little (1995) has verified that once the soil is stabilized with lime the rate of strength reduces due to the wet-dry and freeze-thaw cycles.

Locat et al. (1990) has investigated on the addition of quicklime to sensitive clays and verified that even if the water content is above the liquid limit, a significant increase in strength can be achieved if enough lime are supplied and sufficient time is given for curing. In soil liming process, the recognition of strength enhancement is based on the detection of physical and chemical properties of soil particles. It has been found that there is a correlation between water content and strength, at a specific time.

Kassim and Chern (2004) have highlighted the essential assessment of lime stabilization sustainability with respect to mineralogical influences. For this purpose, different amounts of lime contents have been added to the soil and an increase of 2.5 to 11 times of untreated soils in unconfined compressive strength has been observed. After 14 days, the formation of calcium aluminates silicate hydrate (CASH) observed from x-ray diffraction test, which indicated that, a new product can form with the addition of lime. One of the advantageous of stabilizing soils with lime is that it can transmit a yield stress to the clay soils (Okumara and Terashi, 1975; Balasubramaniam et al., 1989; Rao et al., 1993).

According to Vaughan (1988), if the soil of loaded less than its yield stress very small deformations can be observed. When the soil is loaded to its yield stress, the bonds are destroyed progressively and large strains develop. Rao et al. (2005) have examined the compressibility of soils improved with lime and proposed a framework for saturated lime enhanced clays. Lime stabilization reactions in lime stabilized specimens are observed to cause an improvement in the yield strength in about the range of 3900–5200 and the compressive behavior of these specimens in are conformed to framework for saturated lime enhanced clays (Rao and Shivananda, 2005).

The effect of cyclic wetting-drying on swelling behavior of lime-enhanced clay soils has been examined by Guney et al. (2006) by measuring the swell potential and swelling pressure. In each cycle, the samples have been led to dry in the room temperature to their initial water content and shrink to their initial height and volume, which is known as ‘partial shrinkage’. As a result the effect of lime has been lost after completion of the first cycle and improvement of swelling potential has been observed with an increase in number of cycles. Conversely, the swell potential and pressure of the natural soil samples have reduced after the first cycle and equilibrated after the completion of fourth cycle. Applicability of the in-situ method of lime stabilization has been performed in Ankara province, Yukar Yurtcu village road by (Kavak and Akyarli, 2007). Amount of 5 percent lime has been chosen to apply on section of the road with a thickness of 30 cm and length of 200 m. Results of California Bearing ratio (CBR) tests illustrated an increase that reach 16 and 21 times of the initial CBR values measured after 28 days. Similar enhancements have been observed in unconfined compression and plate loading tests. The results verified the behavior of the surface treatment with lime and its applicability.

Stabilization with lime is not only used for improvement of clayey soils but also used for improvement of sandy soils. Arabani and Karami (2007) have studied some important geotechnical properties of clayey sand such as unconfined compressive strength, tensile strength, CBR, and elastic-plastic behavior. Samples of soils with desired gradation have been taken from field and reconstituted in the laboratory. The mixes were improved with hydrated lime and treated. Different tests were performed on natural and cured samples. A relationship between the results of uniaxial load test, tensile strength, and CBR of the tested specimens has been established. In addition, results of the unconfined compression test and the indirect tensile strength test proved that raise in clay content up to a certain percent, in the clay-sand fills, tends to increase compressive and tensile strength of the materials.

Amu et al. (2008) investigated the lime treatment of lateritic soil mixed with portions of palm kernel shells (PKS). Lime with percentage of 2,4,6,8, and 10 % by weight were added. Although liquid limit, optimum moisture content, and shear strength increased with addition of lime and PKS, a reduction in maximum dry density (MDD) and unsoaked CBR values as well as compressive strength have been observed. Concluding that, PKS is not a good supplement for lime.

2.2.1.2 Fly Ash Treatment

Fly ash which is a chemical additive has been formed by compounds such as silicon and aluminum, which is a consequence of the coal combustion. Its role in stabilization process is to act as a pozzolan and/or as a filler to reduce air voids (U.S Army technical report, 1984). Fly ash is a kind of alkaline material which is mainly composed of spherical non-crystalline silicate, aluminum as well as iron oxides. It can provide multivalent cation (Ca^{+2}, Al^{+3}, etc.) under ionized conditions, which would support flocculation of clay particles by cation exchange (Cokca, 2002).

Kumar and Sharma (2004) have studied the improvement of properties of expansive soils with the addition of fly ash as an effective additive. They have estimated the effect of the fly ash on the properties such as swelling parameters, plasticity index, compaction characteristics, strength behavior, and hydraulic conductivity of fly ash enhanced expansive soils. They observed that hydraulic conductivity, the plasticity and swelling parameters of the mix reduced and the dry density and compressive strength enhanced with an increase in fly ash content. The more the ash contents the more the penetration resistance of the blends for given water content. An excellent relationship has been obtained between the measured and predicted undrained shear strengths.

There are different types of fly ash. Tuncer et al. (2006) have compared the effect of fly ash type on CBR values of the mixtures prepared with the different types of off-specification Dewey and King fly ashes and fly ash Class C Columbia fly ash. Observation illustrated that mixtures prepared with 10 and 18% Dewey or King fly ashes have higher CBR values than the values obtained from Columbia fly ash. Thus, Dewey or King fly ashes are sufficient for stabilizing soft soils (Tuncer, Acosta and Benson, 2006). Coal fly ash has been widely used for stabilization of different kinds of soils, since it has some pozzolanic properties. It is an artificial pozzolan when it is mixed with lime and water, a cementitious compound will form. Coal fly ash is one of the most commonly used, pozzolan in the world (Okunade, 2010).

Nalbantoglu and Gucbilmez (2002) reported on the swell potential and compressibility of Degirmenlik soil (LL=67.8, PI=45.6) stabilized with fly ash. Reduction of swell potential parallel to the enhancement of cure time has been concluded. After curing 7 days, swell values of 4.8% and 3.7% were observed for 15% and 20% fly ash addition, respectively. The compression (C_c) and rebound (C_r) indices decreased as curing time and fly ash content increased.

Zia and Fox (2000) evaluated the swell potential of low plasticity (PI=0) Indiana loess-fly ash mixtures. Swell was measured during soaking of CBR samples. Ten-percent fly ash addition caused a swell reduction of 55% compared to loess alone. With higher compaction swell magnitude for the 10% samples increased. Samples containing 15% fly ash actually exhibited a 255% increase in swell potential over the loess soil. Zia and Fox (2000) attribute this behavior to the formation of ettringite.

2.2.1.3 Cement Treatment

The behavior of the soil can alter with the addition of Portland cement to soil. These changes are caused by the hydration of the cement, and therefore the amount of cement is crucial on the behavior of the soil (McKeen, 1976). To reduce volume changes and to increase the shear strength of expansive soils, cement may be added in a cast that degree of soil stabilization by lime alone might not be sufficient. Usually the combination of lime-cement or lime-cement-fly ash may be used as the overall additive; however the greatest combination can merely be determined by a laboratory study (Army USA, Technical Manual, Foundations in Expansive Soils, 1983). The main distinction between cement and lime-stabilization is that in cement-stabilization, the cement requires some ingredients for the pozzolanic reactions, but in lime-stabilization, the soil should provide part of the reactants. Therefore, cement/soil mixtures can solidify faster than lime/soil mixtures, although both mixtures may gain strength with time.

2.2.1.4 Bituminous Material

Granular materials of base and sub-base bituminous materials are extensively used to stabilize soils for different applications. Firstly clays must be modified with lime into a granular material as the bituminous materials cannot be used directly with fine-grained soils (Army USA, Technical Manual, Foundations in Expansive Soils, 1983). Determination of the plasticity index and the grain size distribution of the soil might be helpful in selecting the best additives as summarized in Figure 2.2 by Dunlap et al. (1975).

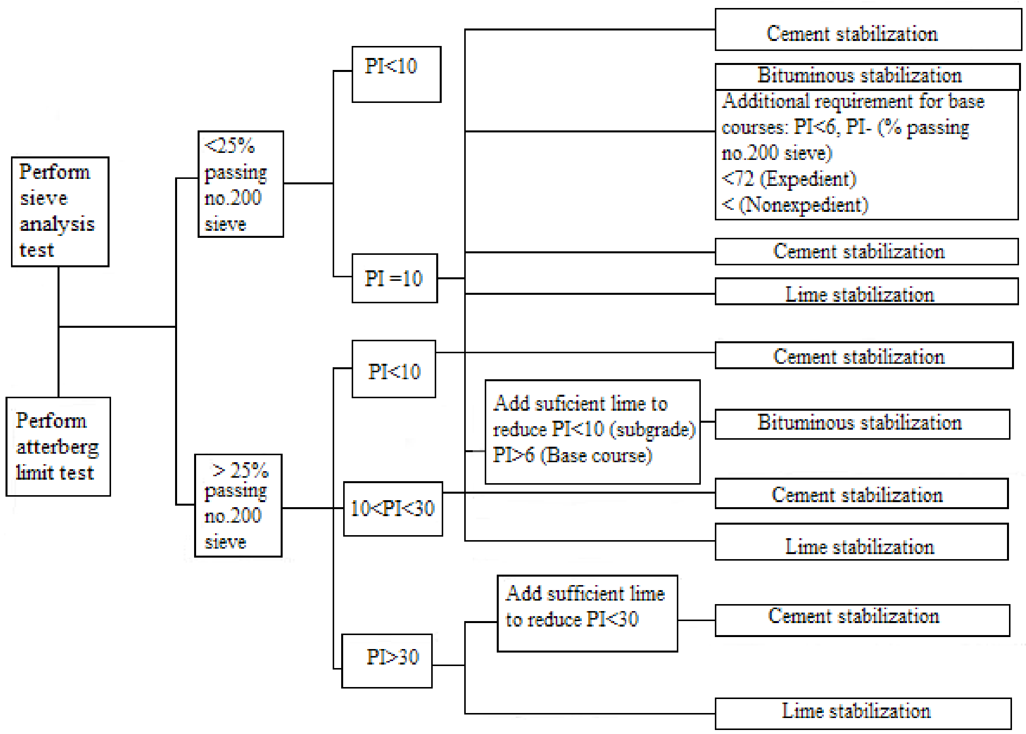

Figure 2.2: Selection of stabilizer (after Dunlap et al. 1975)

2.2.1.5 Posidonia Oceanica Ash

Posidonia oceanica meadows are the most suitable aquatic plant for bio-monitoring. The bio-monitoring is occurring through the control and inspection of this kind of seaweed because of its distribution, acceptable size, easy collection, availability and sensitivity to adjustments of littoral zone.

The presence of this organism in a particular environment indicates that its ecological requirements are fulfilled, while its vanishing confirms an alteration in the environment; which is the basis of "sentinel species" (Blandin, 1986).

In winter and spring seasons this type of seagrass pile up near the Mediterranean Sea. There exists a great deal of research on Posidonia oceanica. Posidonia oceanica meadows play an ecological, sedimentary role (Bell and Harmelin-Vivien, 1983; Grissac and Boudouresque, 1985; Gambi et al., 1989; Romero et al., 1992; Duarte, 2002). The chemical composition of the Posidonia oceanica ash shows it mostly contains: 71% of nitrogen and carbon, 29% of phosphorus and 14% of hydrogen. Other analysis indicates 29% of phenolic compounds and stress enzymes (Pergent-Martini et al., 2005).

Another aspect is the study of Posidonia oceanica contamination which indicates that in particular different amounts of mercury, copper, cadmium, lead, zinc, iron, chromium and/ or titanium (Martinia et al., 2005).

Many literatures about ecological and biological aspects are concerned of some issues such as deposition of Carbon and nutrient in a Mediterranean Posidonia oceanica (Gacia et al., 2002). Investigational proof of particle resuspension reduction within a Posidonia oceanica has been documented by Terrados and Duarte (2000). Impact of infield experimental works on the Posidonia oceanica, and chemical, physical and spectroscopic properties of Posidonia oceanica and their possible recycle has been analyzed by Cocozza et al. (2010). There are many more researches about ecological, nutritional, and biological aspects of Posidonia oceanica; however it cannot be found a research about the application of Posidonia oceanica ash in geotechnical science.

The ash content, which can be stated as the initial dry weight percentage, can be achieved by burning in a muffle furnace at 550$^{\circ C}$ for 12 h (Cocozza et al., 2010). Posidonia oceanica is common seaweed of the Mediterranean Sea, which grows all along the coastal area and forming widespread meadows starting near the water surface to depths of 40 m (Duarte, 1991). Among all the aquatic plants, Posidonia oceanica is the most plentiful seagrass type in the basin of Mediterranean Sea, approximately covering 40,000 km^2 area of the seabed (Cebrian and Duarte, 2001). The leaf rejuvenation cycle of Posidonia oceanica process typically occurs in fall, when an increase in wave action causes the seaweeds to transport. Indeed, noticeable deposits of Posidonia oceanica leaf usually piles up along the coastal areas (Ott, 1980).

In this study the use of these litters in soil stabilization and in geotechnical engineering will be analyzed. The ash content achieved by oven dried crushed pieces in 550$^{\circ C}$ for the duration of 24 hours has been used to monitor its affect on Atterberg limits, grain size distribution, swell potential, and compressive strength of the soil-PO ash mixture of 5 and 10%.

2.3 Expansive Soils Reinforced by Geotextiles and Geomembranes

2.3.1 Introduction and Historical Development

According to the American Society of Testing Materials (ASTM D4439), geomembranes are impermeable synthetic liners or barriers that have been coated with a geotechnically engineered material to control fluid mitigation in human made structure or system.

According to Koerner (1980), designing with geosynthetics is generally prepared from very flexible continuous polymeric sheets. By saturating geotextiles with elastomer sprays or bitumen composites geomembranes can be produced.

Koerner (1980) classified seven fundamental sorts of geomembranes: Chlorinated polyethylene, chlorosulfonated polyethylene, ethylene interpolymer alloy, high-density polyethylene, polypropylene, polyvinyl chloride, and low-density polyethylene

Most of the geosynthetics are made of synthetic polymers for instance polypropylene, polyester, polyethylene, polyamide, PVC, etc. These materials are extremely opposed to biological and chemical degradation. By the method used to combine the filaments or tapes into the planar textile structure, geotextiles can be produced. Classification of soil synthetics and other soil inclusions can be observed in Figure 2.3.

Geomembranes were produced and used in Europe in different situations. The Dutch widely used them in protecting the dyke surge construction in North Sea areas in the early 1950s. In the 1960s; the Du Pont Company in the United States has been recognized as the manufacturer of geomembranes products. Polypropylene, which was bonded and coated with ethyl vinyl acetate (EVA) to give it an impermeable surface, has been produced. DuPont called this product “geomembranes typer”. Dr. Harry Tan, the DuPont Company’s geotechnical consultant, viewed the typer as a means to control the moisture alterations in expansive soils (Steinberg, 1998) Geotextiles are convincingly impermeable and therefore they propose many solutions to the challenge of expansive soils.

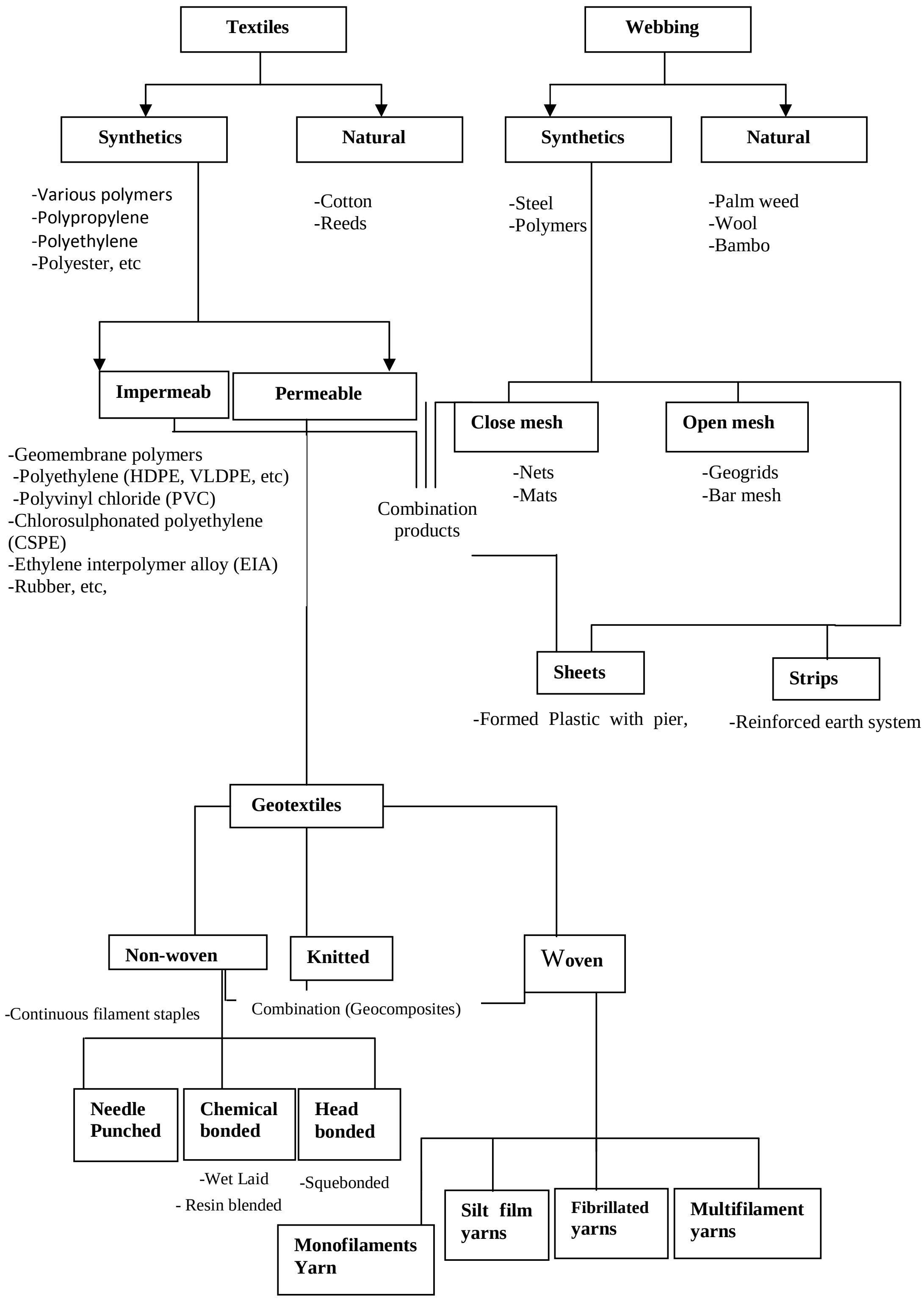

Figure 2.3: Classification of soil synthetics and other soil inclusions (Holtz and Robert, 2001)

2.3.2 Overview of the Literature

According to Heineck et al. (2005) experimental results collected over the last 20 years illustrate the potential of polypropylene fiber for soil reinforcement. Researchers such as Gray and Ohashi (1983), Gray and Al Refeai (1986); Maher and Gray (1990); Al Refeai (1991); Maher and Ho (1994); Ranjan et al. (1994); Michalowaski and Zhao (1996); Morel and Gourc (1997); Consoli et al. (1998, 2002, 2003b); Zornberg (2002); and Michalowaski and Cermak (2003) who have investigated the use of polypropylene fiber for soil stabilizations.

Heineck et al. (2005) reported that no influence on initial stiffness of the materials has been observed with the addition of the polypropylene fibers at very small strains. In contrast, a noticeable effect on the ultimate strength of reinforced soil has been recorded at very large horizontal displacements, despite the fact that no loss in shear strength was indicated. Another concept, which has been studied by Zaimoglu (2010), was freeze and thaw behavior of polypropylene fiber reinforced fine-grained soils. It was found that during freeze and thaw cycles with an increase in fiber content improvement unconfined compressive strength of specimens can be observed. Alternatively, the results showed a constant value of initial stiffness of the stress–strain curves with addition of fiber.

One of the main concerns in cold climates is freeze and thaw phenomena is that an alteration of the soil properties including permeability, water content, stress–strain behavior, failure strength, elastic modulus, cohesion, and friction angle can happen once the soil freezes. Effect of freeze and thaw cycles on fiber reinforced expansive soils has been investigated by Ghazavi and Roustaie (2010). They reported that the reduction of unconfined compressive strength of clay samples can be observed with an increase of number of freeze–thaw cycles. Moreover, an increase in unconfined compressive strength of soil results in the reduction of the frost heave. Furthermore, the results inferred that addition of fiber can affect the strength of soil in opposition to freeze–thaw cycles.

The study of the deformation behavior of moist-compacted soil liners with and without inclusion of discrete and randomly distributed fibers for waste containment systems has been done by Viswanadham et al. (2009) at the onset of non-uniform settlements in a geotechnical centrifuge. Based on the findings it was concluded that there is a considerable potential for fiber reinforcement to lessen and to retard soil crack potential in a randomly reinforced soil liner, while retaining its hydraulic performance at the same time. Swelling behavior of geofiber-reinforced soils has also been studied by these researchers for fibers of different aspect ratios. It was observed that a reduction in heave occurred at low aspect ratios, where swelling pressure was at its maximum value. Finally, with the use of the soil-fiber mechanism by which fibers has restrain swelling of expansive soil is explained.

In order to comprehend the strength behavior of soils and to evaluate the appropriateness of using sisal fiber as reinforcement material Prabakar and Sridhar (2002) conducted a series of experiments on soil samples of different percent sisal fiber. The results have shown a significant development in the failure deviator stress and shear strength parameters c and φ of the studied soil. Consequently the sisal fiber can be classified as a superior material to reinforce soils.

An increase of shear strength of the soil reinforced with polypropylene fiber was presented by Freilich et al. (2010). According to this study, the presence of the fibers altered the behavior of the clay during shearing, which consequently caused changes to the generation of the pore pressure.

Loehr et al. (2000) investigated that with addition of fiber reinforcement the reduction of swell potential of soils can be observed. Significant reduction of volume changes were reported with inclusion of discrete fibers in expansive clays once subjected to one-dimensional free swell. Hence, there is a great potential for reducing the harmful effects on buildings, earth retaining structures and roadways with a high potential for controlling volume change behavior. Fiber dosage rates are important as well as the issue of adequate sample size for testing of fiber-reinforced soils.

Harianto et al. (2008) used polypropylene fiber (C_3H_6) to reinforce soils to overcome problems related to desiccation cracking of the compacted Akaboku soil. He established that the highest crack depth occurs to a depth of almost 50% of the thickness of the unreinforced soil. The authors concluded that the potential application of reinforcing soils with fibers can be counted as a presented method to restrain desiccation cracks which can be faced in landfill cover barriers.

Fiber reinforcement of soil is not just applicable to clayey soils but also to reinforcing sands against settlements. Consoli et al. (2003) discussed the settlement of thick homogeneous layers of compacted polypropylene fiber reinforced based on their research on sandy soils. They have observed a visible stiffer response with an increase in settlement. In triaxial tests, an increase in the lateral stresses underneath the plate has been out looked.

In most cases, fiber reinforcement of sand is against liquefaction potential of the soils. Yetimoglu and Salbas (2002) have studied strength behavior of the fiber-reinforced sand and observed an increase in residual shear strength angle. Static liquefaction of fiber-reinforced sand under monotonic loading has been analyzed by Ibraim et al. (2010). He explores the opportunity of improving the monotonic undrained response of loose clean sand by absorption of the sand with discrete flexible fibers. The potential for the occurrence of liquefaction in both compression and extension triaxial loadings has shown a significant reduction.

Diambra et al. (2010) have tested the effect of short polypropylene fibers in triaxial tension and compression. The role of fibers in strengthening the sand was significant in compression while restrained in extension where it depends mainly on tensile strains.

Kim et al. (2008) used waste fishing net as a fiber reinforcement to improve mechanical behavior of lightweight soils (dredged clayey soil, cement, and air-foam). He investigated the strength behavior of reinforced and unreinforced lightweight soils. The results indicated that with about 0.25% the maximum compressive strength has been obtained. The compression properties of lightweight soil, including the yield stress and compression index, did not depend on the type of curing.

Sulphate rich expansive soils has been stabilized with the use of Class F fly ash, bottom ash, polypropylene fibers, and nylon fibers and it has been recognized as potential stabilizers in improving the volume changes which has been done by Punthutaecha et al. (2006). Two different type of subgrade soils from two locations in Texas have been chosen for a comprehensive experimental study. Swelling-shrinking and plasticity have been reduced by 20–80% with an introduction of ash stabilizers; while introduction of fibers bring about various improvements. The mixed class F fly ash and nylon fibers were the most efficient materials to be used on both Dallas and Arlington soils, where the soil properties have been considerably improved from an average to a moderate level.

Shenbaga and Gayathri (2003) have carried out an investigation on the effect of randomly distributed fiber inclusions on the geotechnical behavior of two different Indian fly ashes. With the introduction of fibers to raw fly ash specimens the strength increased and brittle behavior has been altered into ductile behavior.

Investigations on the influence of fly ash, lime, and polyester fibers on compaction and strength properties of expansive soil show that expansive soil can be effectively stabilized by the combinations of fibers, lime, and fly ash (Kumar, Walia and Bajaj, 2007).

Strength and mechanical behavior of cemented and clay reinforced by discrete short polypropylene fiber (PP-fiber) have been investigated by Tang et al. (2007). They have found that the bond strength and friction at the interface appear to be the principal mechanism managing the benefits of fiber reinforcement. The friction at the interface in fiber-reinforced non-cemented soil shows different behavior than fiber-reinforced cemented soil. Several factors which play an important role in altering micromechanical properties of fiber/matrix interface can be explained as binding of materials in soils which cause an increase in the normal stresses around the fiber body, the effective contact area of the interface and fiber and the surface roughness. Repeated loading of the sub-grade soils in road pavement is a serious issue causing the pavements to lose strength and reach the fatigue level.

Dall'Aqua, Ghataora and Ling (2010) performed series of cyclic loading tests on fiber-reinforced soils and came to the conclusion that reinforced and stabilized soils reach to a sufficient strength after soaking which can be applied in the upper parts of a pavement Consoli, Bassani and Festugato (2010) confirmed the differentiations in the strength of cemented sandy soils with and without addition of fiber. The amount of cement, porosity, water content, and voids/cement proportion were distinguished as controlling parameters. Then it was inferred that the unconfined compressive strength increased linearly where there is a reduction cement amount and the enhancement in porosity for both the fiber reinforced and unreinforced specimens. The outcome of the tests on the study of the mechanical behavior of lime treated and reinforced soil has clearly shown that inclusion of polypropylene fiber and lime in soil can recover compression and shear strength, and reduce the swelling and shrinkage potential and it also can change the failure feature of soil from brittle to ductile behavior. Thus, polypropylene fiber and lime mixture can be considered as an efficient method of stabilization (Cai et al., 2006).

Ayyappan et al. (2010) have done investigation on engineering behavior of soils reinforced with mixture of polypropylene fibers and fly ash for the purpose of road construction. Primary conclusions obtained from this investigation indicate that as length of the fiber increases the peak compressive strength decreases, while the strain energy absorption capacity improve in all combinations of soil and fly ash.

2.4 Unsaturated Soil Mechanics

Soils situated above the ground water table and compacted soils are basically unsaturated, and as a result of evaporation and transpiration of vegetations, they have negative pore-water pressures. The top soil located near the surface can highly be affected by climate changes which subsequently alter the shear strength and volume change properties (Rahardjo et al., 2002).

The general field of soil mechanics can be divided into two parts: saturated soils and unsaturated soils. There are basic differences in the nature and engineering behavior of the saturated and unsaturated soils due to existence of two phases in saturated soils and three phases in unsaturated soils. Soils close to the ground surface in arid and semi-arid climates are subjected to negative pore-water pressures (suction) and possible desaturation. The soil–water characteristic curve (SWCC) is the correlation between suction and water content for an unsaturated soil (Ng and Menzies, 2007) Terzaghi (1943) stated that “the theories of soil mechanics provide us only with the working hypothesis, because our knowledge of the average physical soil properties of the subsoil and the orientation of the boundaries between the individual strata is always incomplete and often utterly inadequate.”

Constitutive relations for the classical soil mechanics were proposed in 1970s (Fredlund and Rahardjo, 1993). Primarily, the study of seepage, shear strength, and volume change problems are the main focus of the constitutive surfaces. Progressively the behavior of unsaturated soils could be classified as an addition to saturated soils (Fredlund and Morgenstern, 1976).

Numerous studies extend volume change and shear strength in the form of elastoplastic models from the saturated soil collection to unsaturated soil states (Alonso et al., 1990; Wheeler and Sivakumar, 1995; Blatz and Graham, 2003).

Seepage modeling for soils is the first of the unsaturated soils problems. The 1990s was a period that there has been an emphasis on the performance of unsaturated soil mechanics into regular geotechnical engineering. The primary stages in the development of a science suggested by Nishimura and Fredlund (2000) consist of establishing the stress state variables, constitutive relations, formulation, solution, design, verification and monitoring and implementation. Research is required for all of the mentioned stages in order to establish a practical, efficient, cost-effective, approach (Fredlund, 2006).

2.4.1 Soil Suction

Soil suction is a measure of the free energy of the pore-water in a soil. Soil suction is the tendency of soil to retain water and provide information on soil parameters that are influenced by water; for example, volume change, deformation, and strength characteristics of the soil. Soil suction is dependent on the initial matric suction (Ψ) defined as the negative pore-water pressure in the soil due to capillary and adsorption forces. It is the difference between pore-air (u_a) and pore-water pressure (u_w).

Osmotic suction (Ψ_o) corresponds to the negative pressure of the soil which is related to the amount of dissolved salt in pore-water. Once the soil looses its moisture and gets dry, the concentration of dissolved ions as well as the osmotic component of suction increases (Peroni and Tarantino, 2003). Total suction includes osmotic suction and matric suction. Low and high ranges of suction can be determined from different methods. In low suction ranges, measurement methods are generally based on passage of free water, such as axis-translation technique, and only matric suction can be measured. In high ranges of suction total suction is measured and measurement techniques are based on vapor migration, such as in psychrometer. Therefore it is important to know for which ranges what is the suitable method. Methods of suction measurements are summarized in Table 2.1.

2.4.2 Suction Measurements

There are different direct and indirect suction measurement techniques. Table 2.1 shows summary of different methods of suction measurement. Suction can be measured as total suction or matric suction. Measuring matric suction includes tensiometer; axis translation techniques, electrical/thermal conductivity sensors, and contact filter paper techniques. Tensiometers measure negative pore water pressure directly. Axis translation techniques depend on controlling the dissimilarities between the pore-air pressure and pore-water pressure, and measuring the corresponding water content of soil in balance with the applied matric suction. Electrical or thermal conductivity sensors, often called as ‘‘gypsum block’’ sensors, are used to indirectly correlate matric suction to the electrical or thermal conductivity of porous medium surrounded in a mass of unsaturated soil. Finally, the contact filter paper technique depends on the water content measurement of small filter papers in direct contact with soil specimens. In each of these cases, water content corresponding to the measured suction is considered to produce data points along the soil-water characteristic curve. The characteristic curve can either be a wetting or drying cycle depending on the wetting path during the measurement (Lu and William, 2004).

Table 2.1: Common laboratory and field suction measurement techniques (Lu and William, 2004)

Suction Component Measured	**Technique/Sensor**	**Practical Suction Range (kPa)**	**Laboratory/ Field**	**References**
Matric suction	Tensiometers	0–100	Laboratory and field	Cassel and Klute (1986); Stannard (1992)
	Axis translation techniques	0–1,500	Laboratory	Hilf (1956); Bocking and Fredlund (1980)
	Electrical /thermal conductivity sensors	0–400	Laboratory and field	Phene et al. (1971a, 1971b); Fredlund and Wong (1989)
	Contact filter paper method	Entire range	Laboratory and field	Houston et al. (1994)
Total suction	Thermocouple psychrometers	100-8000	Laboratory and field	Spanner (1951)
	Chilled-mirror hygrometers	1,000–450,000	Laboratory	Gee et al. (1992); Wiederhold (1997)
	Resistance/ capacitance sensors	Entire range	Laboratory	Wiederhold (1997); Albrecht et al. (2003)
	Isopiestic humidity control	4,000–400,000	Laboratory	Young (1967)
	Two-pressure humidity control	10,000–600,000	Laboratory	Likos and Lu (2001, 2003b)
	Noncontact filter paper Method	1,000–500,000	Laboratory and field	Fawcett and Collis-George (1967); McQueen and Miller (1968); Houston et al. (1994); Likos and Lu (2002)

2.4.3 Soil-water Characteristic Curve

The soil-water characteristics curve defines the correlation between soil suction and the degree of saturation, *S*, or gravimetric water content, *w*, or the volumetric water content, θ. SWCC is a relationship that shows the behavior of the soil during wetting and drying. Soils with low water content have higher suction values and vice versa. It consists of the two paths of drying (adsorption) SWCC and wetting (desorption) SWCC. The wetting path is started from oven-dried condition. Therefore, from oven-dried condition the wetting process continues until full saturation. Oven-dried soils normally have suction of 1000000 kPa, which is the last value on the x-axis of the soil water retention curve. Croney and Coleman (1961) indicated that when water content is zero, total suction for a most range of soils is vaguely below 1000000 kPa.

Fredlund and Rahardjo (1993) also indicated suction values of 98000 kPa for various sand and clay soils for zero water content. Thermodynamic considerations also support these values (Richards, 1965). Type, texture and mineralogy of soils affect the soil water retention and the suction values. Compaction with different water content values cause differences in fabric of the soil (Lambe, 1960; Gens et al., 1995; Delage and Graham, 1996), due to different void ratios and compaction energy needed. Soil-water characteristic curve consists of different stages that can be seen in Figure 2.4.

With the use of well-known models, most of the properties of the unsaturated soils such as hydraulic conductivity and the shear strength functions can be estimated by the use of SWCC (Vanapalli and Fredlund, 2000). The stress states in soil-water and pore size distribution are essential information relating to the amount of water existing in pores at any suction value (Sillers et al., 2001).

Soil-water characteristic curve consists of three stages of capillary or saturation zone, desaturation zone, and zone of residual saturation. Capillary zone is the zone at which soil remains saturated or does not lose its moisture due to capillary forces and the waters inside pores that are in tension. After the air entry value, which is the desaturation zone, soil specimens start to dry or desaturate. This is because at air entry value the air starts to replace its position with water.

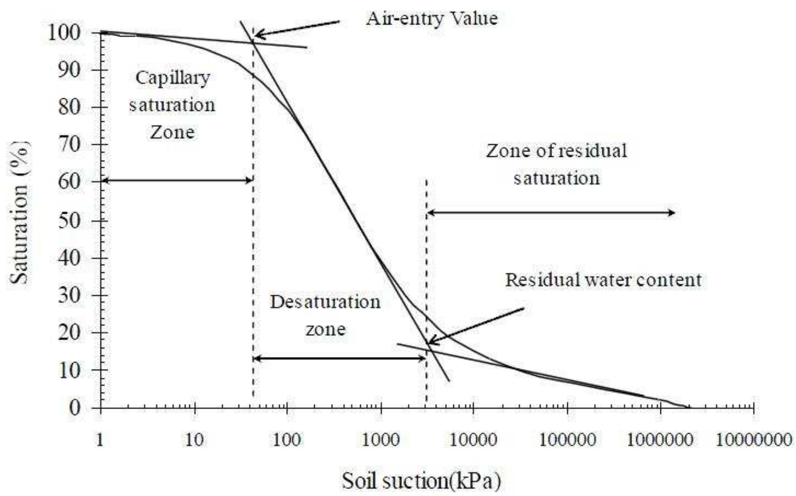

Figure 2.4: Typical soil- water characteristic drying curve, illustrating the regions of saturation and desaturation (Sillers et al., 2001)

2.4.4 Soil-Water Characteristic Curve Models

Nishimura and Fredlund (2000) review different existing mathematical equations and models that have been proposed to describe SWCC. Table 2.2 shows the different models of SWCC, from which unsaturated hydraulic properties of the soils can be estimated.

Table 2.2: Some of the models proposed for SWCC (after Fredlund, 2000)

Author (S)	**Equation**	**Soil Parameter**
Fredlund and Xing (1994)	$w_w = w_s\left[1-\frac{\ln\left(1+\frac{\psi}{h_f}\right)}{\ln\left(1+\frac{10^6}{h_f}\right)}\right]\left[\frac{1}{\left[\ln\left[\exp(1)+\left(\frac{\psi}{a_f}\right)^{n_f}\right]\right]^{m_f}}\right]$	a_f, n_f, m_f
van Genuchten (1980)	$w_w = w_{rvg} + (w_s - w_{rvg})\left[\frac{1}{\left[1+(a_{vg}\psi)^{n_{vg}}\right]^{m_{vg}}}\right]$	av_g, nv_g, mv_g
Mualem (1976)	$w_w = w_{rm} + (w_s - w_{rm})\left[\frac{1}{\left[1+(a_m\psi)^{n_m}\right]^{\left(1-\frac{1}{n_m}\right)}}\right]$	a_m, n_m, m_m=1/(1-n_m)
Gardner (1958)	$w_w = w_{rg} + (w_s - w_{rg})\left[\frac{1}{\left[1+a_g\psi^{n_g}\right]}\right]$	a_g, n_g
Burdine (1952)	$w = \frac{w_s}{\left[1+\left(\frac{\psi}{a_b}\right)^{n_b}\right]^{m_b}}$	a_b, n_b, m_b =2/(1-n_b)

Chapter 3

MATERIALS AND METHODS

3.1 Materials

3.1.1 Soil Sample

The soil used in this research has been obtained from the campus of Eastern Mediterranean University in North Cyprus. The physical properties of the soils are shown in Table 3.1. As indicated, the soil has high plasticity index. Identification, description, and classification of these types of soils are based on Atterberg limits. According to Nelson and Miller (1992) classification method given in Table 3.2, the soil used in this study is clay with high swell potential. Linear shrinkage can be used as a reference to estimate the probable swell percentage and the degree of expansion by Altmeyer (1955) criteria given in Table 3.3. The linear shrinkage was determined to be 21% which is greater than 8% indicating that a probable swell would be greater than 1.5. Hence, it may experience a critical degree of expansion.

Table 3.1: Engineering properties of the soil used in this study

Property	
Specific Gravity	2.56
Gravel (>200 μm), (%)	0
Sand (75-200 μm), (%)	8
Silt (2-75 μm), (%)	40
Clay (<2 μm), (%)	52
Liquid limit, (%)	57
Plastic limit, (%)	28
Plasticity index, (%)	29
Linear shrinkage, (%)	20
Optimum moisture content, (%)	24
Maximum dry density, (gr/cm^3)	1.497
Soil classification (USCS)	CH

Table 3.2: Expansive soil classification after Holtz and Gibbs (1956) (Nelson and Miller, 1992)

Data from Index Tests based on vertical loading of 6.9 kPa			**Probable Expansion (% Total Volume Change)**	**Degree of Expansion**
Colloid Content (% minus 0.0001 mm)	**Plasticity Index**	**Shrinkage Limit**		
> 28	> 35	< 11	>30	Very high
20-31	25-41	7-12	20-30	High
13-23	15-28	10-16	10-20	Medium
< 15	< 18	> 15	<10	Low

Table 3.3: Expansive soil classification after Altmeyer (1955) (Nelson and Miller, 1992)

Linear shrinkage	**SL (%)**	**Probable Swell (%)**	**Degree of Expansion**
< 5	> 12	< 0.5	Non critical
5-8	10-12	0.5-1.5	Marginal
> 8	< 10	> 1.5	Critical

Based on Chen (1965) with liquid limit of 57, which is between 40 and 60, again the soil is identified to possess high degree of expansion potential as shown in Table 3.4. Furthermore, Chen (1988) suggested a simplified classification scheme for expansive soils, which was only based on plasticity index as given in Table 3.5. With 29% plasticity index the soil is in the range of high swell potential.

Table 3.4: Expansive soil classification after Chen (1965) (Nelson and Miller, 1992)

Laboratory and Field Data				
Percentage Passing No. 200 Sieve	**Liquid Limit (%)**	**Standard Penetration Resistance (Blows/ft)**	**Probable Expansion (%Total Volume Change)**	**Degree Of Expansion**
> 95	> 60	> 30	> 10	Very high
60-95	40-60	20-30	3-10	High
30-60	30-40	10-20	1-5	Medium
< 30	< 30	< 10	< 1	Low

Table 3.5: Expansive soil classification after Chen (1988) (Nelson and Miller, 1992)

Swelling Potential	Plasticity Index
Low	0-15
Medium	10-35
High	20-55
Very high	35 and above

3.1.2 Polypropylene Fiber

The polypropylene fiber used in this study is the most commonly used synthetic material due to its low cost and hydrophobic and chemically inert nature which does not allow any reaction with soil moisture or leachate. The other properties are the high melting point of $160^{\circ C}$, low thermal and electrical conductivity, and high ignition point of $590^{\circ C}$. The physical properties also include the specific gravity of 0.91, and an average diameter and length of 0.06 mm and 20 mm respectively.

3.1.3 Posidonia Oceanica Ash

Posidonia oceanica is common seaweed of the Mediterranean Sea, which grows all along the coastal area and forming widespread meadows starting near the water surface to depths of 40 m (Duarte, 1991). Among all the aquatic plants, Posidonia oceanica is the most plentiful seagrass type in the basin of Mediterranean sea, approximately covering 40,000 km^2 area of the seabed (Cebrian and Duarte, 2001). The leaf rejuvenation cycle of Posidonia oceanica process typically occurs in fall, when an increase in wave action causes the seaweeds to be transported. In addition, Deposits of Posidonia oceanica piles up along the coastal areas (Ott, 1980). Posidonia oceanica used in this study has been collected from the East coast of North Cyprus and transported to the laboratory in plastic bags. The PO has been washed several times to remove the soluble salts, air dried and crushed to small pieces in a food processor to obtain maximum amount of ash which could be produced in a small muffle furnace. Ash content, which is expressed as a percentage of the initial dry mass was obtained by combustion in a muffle furnace at 550$^{\circ C}$ for 24 hours. Final step was to grind the ash in to powder form using a wooden pestle. The final product can be depicted in Figure 3.1.

Figure 3.1: Posidonia oceanica ash

3.2 Methods

3.2.1 Experimental Study on Polypropylene Fiber Reinforced Clay

3.2.1.1 Sample Preparation

One of the drawbacks of using fibers in soil stabilization research is the difficulty in achieving uniform distribution in compacted soil during sample preparation. Trimming samples after performing dynamic compaction is extremely difficult which yields poor quality specimens. Therefore, the optimum moisture content and maximum dry density of soil with and without addition of fiber were obtained from Standard Proctor test according to the ASTM D698-91. To achieve the amount of pressure needed to reach to the maximum dry density at optimum water content, samples of reinforced and unreinforced soil have been compacted statically. The static compaction done by modifying the CBR instrument (Figure 3.3) was preferred due to easiness in achieving uniform fiber distribution in the molds. The pressure applied to each sample has been monitored and recorded by a dial gauge. From the graph, the pressure required to obtain maximum dry density has been obtained and implemented to compact samples directly in the mold of required size. Therefore with this method trimming the samples is not required. For well mixing water is introduced to the pulverized soil and has been cured for minimum of 24 hours. Polypropylene fibers are added to the cured soil and have been mixed in a mixer to have the best distribution of the fibers. Figure 3.2 presents the view of fiber mixed soil and as compacted unreinforced and reinforced specimens. The blade of food processor have been wrapped by stretch film and electrical tape to avoid the breakage and cutting of fibers.

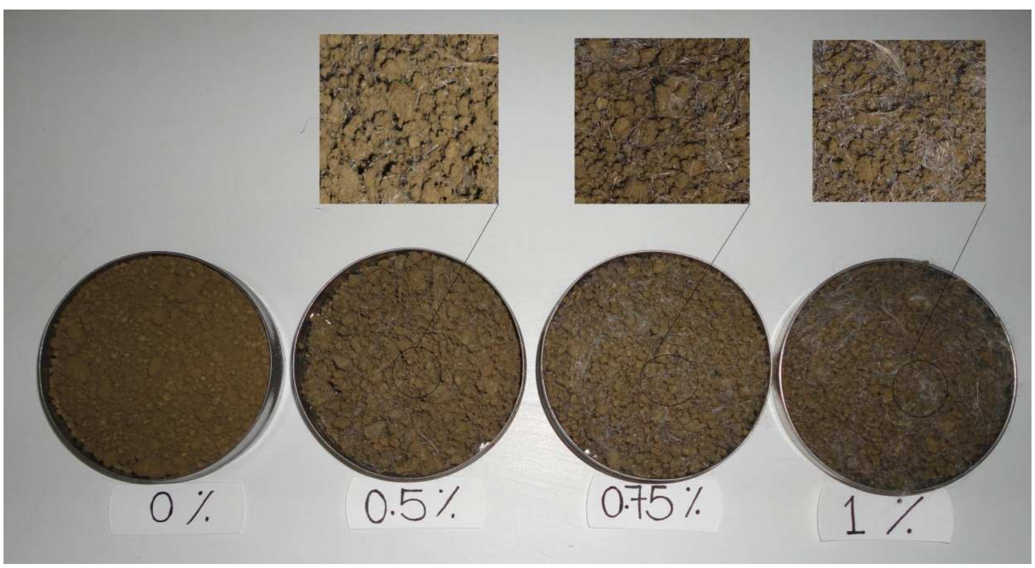

Figure 3.2: Distribution of polypropylene fibers in soil-fiber mixtures

3.2.1.1.1 Dynamic Compaction

Standard Proctor test (ASTM D 698-91) was used to obtain the relationship between water content and dry unit weight of samples. Mixtures of soil with different water contents were compacted in standard molds in three layers (25 blows on each layer with thickness of 5-8 cm) with the application of an automatic dynamic compacter hammer of 2.5 kg dropping from 305 mm height that produce a compactive effort of 600 kN-m/m. The compacted samples were then trimmed, weighed and divided into three and from each part top, middle and bottom samples have been taken to measure the water contents according to ASTM D2216. The maximum dry densities and the corresponding optimum water contents were determined from the relationship of molding water content (w_c) and dry density (ρ_d).

3.2.1.1.2 Static Compaction

After obtaining the compaction characteristics by dynamic compaction, these values have been used to obtain samples with static effort. Samples at the optimum water content are subjected to static compaction under increasing pressure in static compaction molds with a diameter of 50 cm and height of 10 cm. The amount of applied pressure has been obtained from the calibration curve. Then the specimens have been trimmed and the dry densitities have been calculated to obtain the curve of maximum dry density versus compactive effort in kPa.

to 2.5.

Figure 3.3: CBR equipment and the mold used for static compaction

3.2.1.2 Unconfined Compression Test

The unconfined compressive strength determination for the intact, remolded, or reconstituted samples with the application of the axial load is done according to the ASTM D2166-06. In this test method, unconfined compressive strength is taken as the maximum load obtained per unit area or the load per unit area at 15 % axial strain, whichever occurs first during the performance of a test. Samples have been prepared in optimum moisture content and compacted statically by CBR instrument to achieve maximum density required for the unconfined compression mold. Compaction of the specimens has been performed directly in the mold and has been extruded by an extruding machine. Meanwhile specimens are monitored to have a minimum diameter of 30 mm or the height over diameter ratio should be between 2 to 2.5.

3.2.1.3 One-dimensional Swell

To determine swell potential of the soil specimens free swell tests were held according to ASTM D4546-08. The samples, which have been compacted statically to their maximum dry density, were soaked under 7-kPa surcharge pressure in metal rings with a diameter of 50 mm and height of 19 mm. The height of each sample is trimmed to 15 mm so that there would be enough space for the soil to swell. Each sample is placed on the porous stone and a filter paper at top and bottom of the specimen to avoid dispersion of the particles (Figure 3.4). Swell starts soon after the specimen is imbibed in distilled water introduced to the cell. Displacement of the samples can be monitored by a set of dial gauges. Displacement is measured at different time intervals until the volume change is constant.

Figure 3.4: One- dimensional swell equipment

3.2.1.4 One-dimensional Consolidation

These experiments cover processes for obtaining the value and degree of consolidation of soil when restrained laterally and drained axially while subjected to incrementally applied loads. Preparation and the instrumental set up are the same as one-dimensional swell test. This test is applied according to ASTM D2435 – 04. Once the swell is completed the specimens are loaded and the consolidation process is started. After application of 64 kg of weight the unloading part of the procedure is started. The soil which has been compressed cannot go back to its original height in the beginning of consolidation test but swells following another path, the unloading part of the curve of void ratio versus logarithm of pressure. Compression index, C_c and rebound index, C_r, are the two indices of the soil, which are the slope of the loading and unloading curves respectively.

3.2.1.5 Saturated Hydraulic Conductivity

The hydraulic conductivity of fine grained soils can be determined directly by falling head method. It can also be found indirectly from the consolidation data. The hydraulic conductivity of unreinforced and reinforced soils is obtained indirectly from the standard one-dimensional consolidation test. The following expression is used in the calculations:

$$k = c_v \, m_v \, \gamma_w \qquad (3.1)$$

where:

k is the hydraulic conductivity,

c_v is the coefficient of consolidation,

γ_w is the unit weight of water,

3.2.1.6 Split Tensile Strength

Split tensile strength is a method to measure the tensile strength of concrete. This method has been adopted to measure the tensile strength of unreinforced and reinforced soil specimens (according to the ASTM D3967-08). The aim is to exhibit the effect of polypropylene inclusion on the tensile strength. Specimens of 50 mm in diameter and 100 mm in height have been prepared and been tested in the split tensile strength instrument shown in Figure 3.5. The most important problem encountered in this test, which is actually designed for testing concrete samples, is the deformation of soil specimens under compression. Therefore, a correction factor has been applied to allow for the deformations under compression (Frydman, 1964).

Figure 3.5: Split tensile strength equipment

3.2.1.7 Suction Measurements

In this study, contact filter paper method with Whatman #42 is used according to ASTM D5298 –10 to measure matric suction. Suction of the soil has been obtained under controlled temperature and zero applied stress. After completion of the swell test, the samples are removed and weighed. Then they have been placed in the oven at 40$^{\circ C}$ to simulate an average temperature, which can be reached during summer in North Cyprus. At certain time intervals, samples have been taken out for suction measurement. The sample is packed in a container with three filter papers placed in intimate contact with the specimen. The two filter papers other than the sacrificial filter paper, in closest contact with specimen to avoid soil particle dispersion and contamination of the other filter papers, are used for suction measurements. The schematic test apparatus is shown in Figure 3.6. The specimens are secured from moving by bubble wrap filling placed in the gap around them, as can be seen in Figure 3.7.

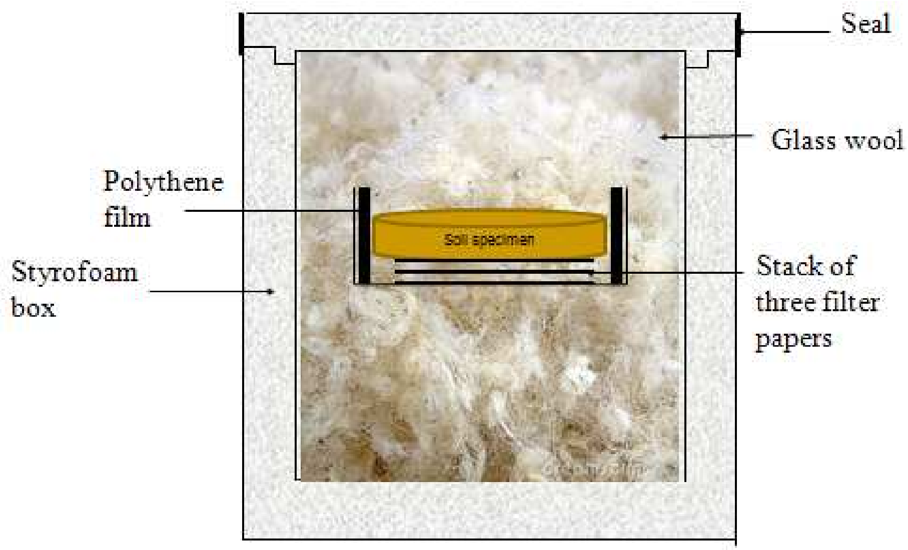

Figure 3.6: Schematic figure showing the location of filter papers (Bilsel, 2002)

Figure 3.7: Packing of soil samples for suction measurement with filter paper method

Water content of filter papers determined after an equilibration period of 7 days are correlated to soil suction using the calibration curve that has been obtained by using pressure plate equipment and vapor equilibration technique given in Figure 3.8.

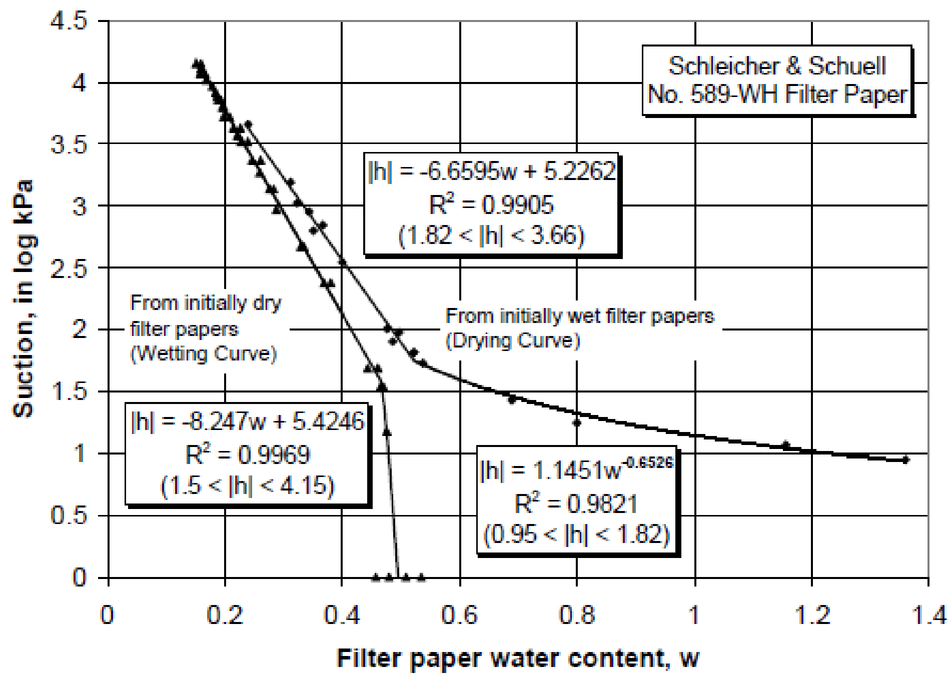

Figure 3.8: Drying and wetting suction calibration curves (Bulut et al., 2001)

3.2.1.8 Shrinkage

Volume change behavior of the unreinforced and reinforced compacted samples during drying phase has been analyzed in this study by recording diametrical, axial and volumetric deformations of the specimens. Mass, diameter and height of the specimens have been measured at specific time intervals, once they are in the drying process, in order to obtain strain-time and void ratio-water content relationships, which are essential in studying the behavior of desiccating soil. Void ratio versus water content is the shrinkage line and the experimental data can be fitted by a model from SoilVision.

3.2.2 Expansive Soil Stabilized with Posidonia Oceanica Ash

3.2.2.1 Specific Gravity

Specific gravity of the soil in natural state and with different percentages of 5% and 10% Posidonia oceanica ash has been determined according to the ASTM D 854 – 06.

3.2.2.2 Atterberg Limits

The most basic and preliminary tests are the Atterberg limit tests which are the liquid and plastic limits of soils. ASTM D4318 – 05 is the standard method used in this study. Plasticity index which is the difference between the liquid limit and plastic limit is used to identify the type of soil based on unified soil classification system. Recently, soil stabilization using additives, mainly for the purpose of recycling the waste, is one of the most popular environmental friendly methods. In this study, the Posidonia oceanica ash, the preparation of which has been explained earlier, has been used. Figure 3.9 depicts the ash added to the soil and the mixture used for Atterberg limit tests. Effect of PO ash on Atterberg limits has been studied by adding 5% and 10% of PO ash by dry mass of the soil and the changes of the Atterberg limits have been recorded.

Figure 3.9: Mixture of Posidonia oceanica ash and soil for Atterberg limit analysis

3.2.2.3 Grain Size Distribution

The hydrometer analysis, information about grain size distribution of the soil, has been implemented according to ASTM D422-54T. Particle sizes larger than 75 μm (retained on No. 200 sieve) is determined by sieve analysis, while the distribution of particle sizes smaller than 75 μm can be determined by a sedimentation process, using hydrometer analysis. Hydrometer analysis has been carried out with 0%, 5%, and 10% Posidonia oceanica ash addition to see the effect of PO ash on grain size distribution of the expansive soil.

3.2.2.4 Linear Shrinkage

Linear shrinkage of the soil mixtures with 0%, 5% and 10% Posidonia oceanica ash has been tested according to the BS-1377: 90. The soil that is used in this test should have number of drops of less than 25 according to liquid limit experiment. Usually the amount of remaining soil sample from liquid limit test is used for the linear shrinkage. The length of the mold is measured by compass and the soil mixture is placed inside the mold. Afterward the mold is left for 1 day in room temperature and after in oven with $50^{℃}$, then placed in the oven with $110^{℃}$.

3.2.2.5 Swell and Unconfined Compressive Strength

Samples of swell and consolidation tests have been compacted at the optimum moisture content and maximum dry density of original soil (0% ash) with the static compaction equipment. The same procedure is followed as explained in sections 3.2.1.3 and 3.2.1.4.

3.3 Computer Programs

3.3.1 SoilVision Program

SoilVision 3.34 Software, which is a knowledge-based system database for estimating unsaturated and saturated properties of the soil, has been used in this research. SoilVision is a valuable program that is able to model soil- water characteristic curves with different well-known models. In many cases, in the early design stages, the soil-water characteristic curves and hydraulic conductivity values are not available. With the help of SoilVision, these properties of the soil can be predicted based on the grain-size distribution and other simple index properties such as volume- mass relationships.

Chapter 4

RESULTS AND DISCUSSIONS

4.1 Fiber Reinforcement

This study has assessed the suitability of synthetic and natural additives, polypropylene fibers and Posidonia oceanica to be used as soil stabilizing agents to mitigate expansive soils. The emphasis however has been on the use of polypropylene fibers on the behavior of local expansive soils abundantly found in North Cyprus. Most of the research works on polypropylene added expansive soils are carried out on fiber reinforcement with cement, lime and fly ash as complementary materials. There is few study related to the compressibility and volume change behavior as well as unsaturated properties of expansive soils reinforced with polypropylene fiber. Therefore, this thesis work will include the hydro-mechanical properties of fiber reinforced soils in the initial part of this study in addition to physical properties. The most important mechanical behavioral properties studied include the compaction, volume change (compressibility, swelling-shrinking), and strength characteristics (tensile and unconfined compressive strength), and hydraulic properties include the soil-water characteristic study and saturated hydraulic conductivity.

4.1.1 Compaction Characteristics

Standard Proctor test (ASTM D 698-91) was used to obtain the relationship between water content and dry unit weight of samples. Mixtures of soil with different water contents were compacted in standard molds in three layers (25 blows on each layer of thickness of 5-8 cm) using an automatic dynamic compactor with a hammer of 2.5

kg dropping from 305 mm height producing a compactive effort of 600 kN-m/m. The compaction test has been performed on unreinforced and reinforced soil specimens with different fiber contents of 0.5%, 0.75%, and 1% of dry unit weight. The test results are presented in Figure 4.1, from which the maximum dry density and optimum moisture content values are obtained. Polypropylene fiber is an impermeable material, therefore changes in optimum moisture content is not significant. However maximum dry density has been reduced as fiber content increases, which can be attributed to the reduction of average unit weight of solids in the mixture of soil and fiber.

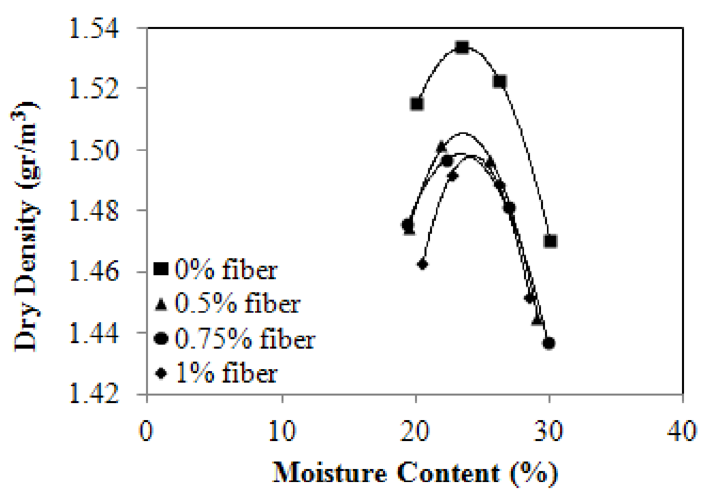

Figure 4.1: Standard Proctor compaction curve

Figure 4.2 shows the pressure versus dry density obtained from static compaction results. This test has been performed by the use of California Bearing Ratio test equipment to find the pressure required to obtain maximum dry density from Standard Proctor test. The specimens were statically compacted at the optimum moisture content and the required pressure to achieve the maximum Proctor density has been obtained. As it is observed from the figure, higher amount of pressure is needed to obtain maximum dry density of fiber reinforced samples in comparison to unreinforced specimens. This is due to the resistance of reinforced soil specimens to compression.

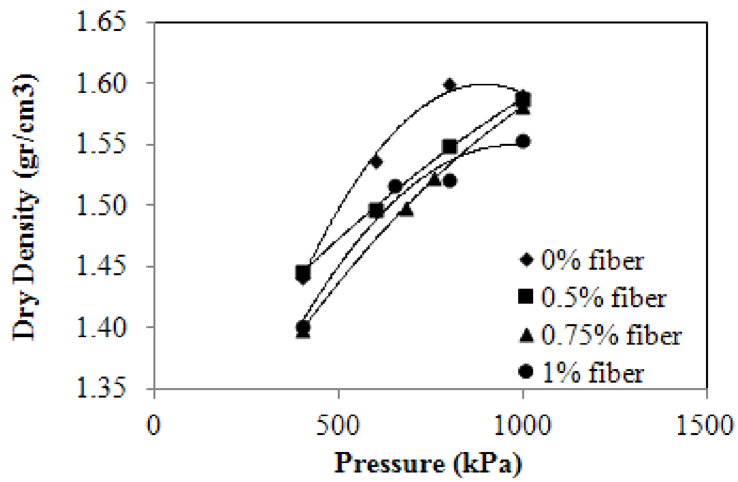

Figure 4.2: Dry density versus static compaction pressure

4.1.2 Volume Change Behavior

4.1.2.1 Swelling

To investigate the swelling characteristics of original and fiber reinforced specimens, one dimensional swell test was carried out using oedometer. Samples were statically compacted at optimum moisture content in consolidation rings of 50 mm inner diameter and 19 mm of height. The samples were left under a low surcharge of 7 kPa and full swell was measured. Specimens of different fiber inclusions have been swelled until the increase in free swell with time became marginal. Figure 4.3 presents the free swell response as one-dimensional swell ($\Delta H/H_0$) percent with respect to logarithm of elapsed time in minutes for different fiber contents. The results show an increase of swell with 0.5% and 0.75% fiber contents, whereas a sudden reduction with 1% fiber content is detected. According to Ghazavi and Roustaie (2009) a reduction in swell percentage has been obtained with 3% of polypropylene fiber and an enhancement of swell percent with 1% and 2% of polypropylene fiber. Sample size is a factor which can affect the swell percentage of the soil samples, as Loehr et al. (2000) pointed out that samples of 10.2 cm showed a reduction in swell percentage versus time with the increase of fiber content, whereas the same soils tested with a dimension of 6.4 cm indicated an increase in swell percent of the soil specimens.

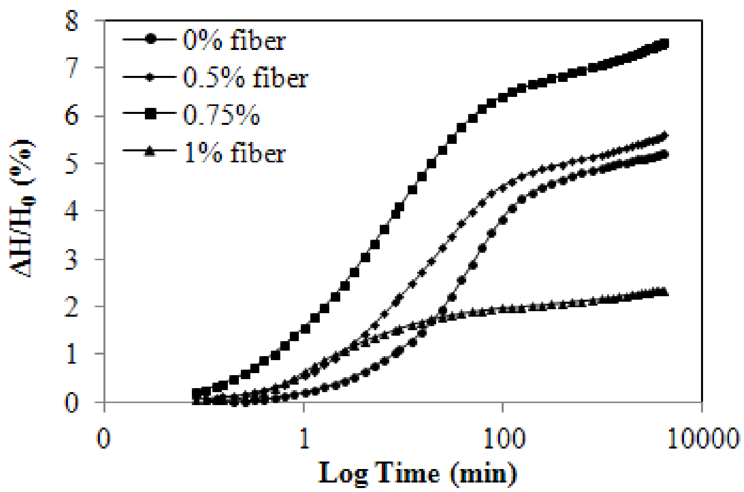

Figure 4.3: Percent swell of soil specimens versus logarithm of time

Further study of Figure 4.3 depicts that primary swell of the 0.5% and 0.75% fiber contents has increased while a reduction can be observed for 1 % fiber content. The secondary swell, which is the amount of swell beyond the completion of primary swell, has been reduced by the increase of fiber content. Therefore, it can be concluded that the efficiency of fiber inclusions should be evaluated in a longer time period, with the completion of secondary swell period, and not with the primary swell. Time for completion of primary swell (t_s) is noticeably reducing with increasing fiber content, which also indicates the efficiency of using fiber reinforcement in controlling the swell of expansive soils. Values for primary swell and time for primary swell and percent secondary swell for each sample are presented in Table 4.1. The primary swell percentage of 0.75% fiber added specimens is not in good agreement with the other results, which can be concluded as an experimental nonconformity.

Table 4.1: Primary swell, primary swell time and secondary swell of different fiber contents

Fiber content (%)	**Primary swell (%)**	**t_s (min)**	**Secondary swell (%)**
0	4.4	150	0.9
0.5	4.6	80	0.8
0.75	6.3	50	0.6
1	1.6	6	0.4

4.1.2.2 Shrinkage

The main purpose of the shrinkage test is to obtain the constitutive law that links the water content to the strain. Tracking of volume change behaviour was used to assess the effect of fiber content on shrinkage behaviour of expansive soil used in this study. Statically compacted soil specimens have been subjected to one- dimensional swell test. Upon completion of the swelling phase, samples have been dried in the oven at $40^{°C}$. Height, diameter and mass of the specimens were measured at different time intervals. Figure 4.4, shows the results of volumetric, diametric, and vertical strain versus time relationships.

Studying the shrinkage strains, it can be concluded that longer duration for shrinking process of the reinforced samples is required, and that the axial shrinkage is more influenced by reinforcement than the lateral (diametric) shrinkage.

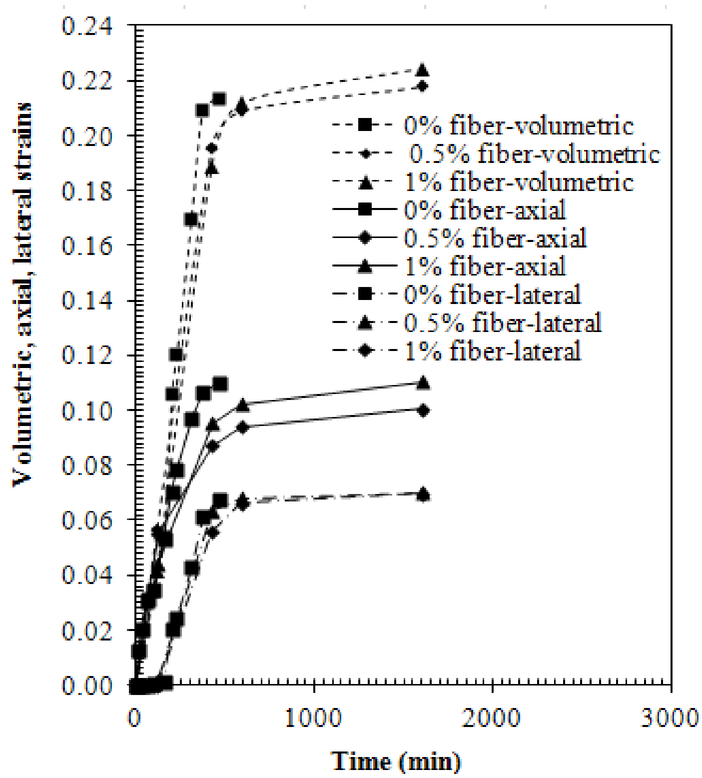

Figure 4.4: Volumetric, axial and diametric strain versus time relationships

Void ratio versus water content relationship from an initial high water content to completely dry condition, is the "shrinkage curve", which is in the form of a hyperbolic curve. Fredlund et al. (1997, 2002) proposed an equation to best-fit data for the shrinkage curve given in Equation 4.1.

$$e(w) = a_{sh}\left[\frac{w^{c_{sh}}}{b_{sh}^{\ c_{sh}}} + 1\right]^{\left(\frac{1}{c_{sh}}\right)} \tag{4.1}$$

where,

a_{sh} = the minimum void ratio, (e_{min}), b_{sh} = slope of the line of tangency, (e.g., drying from saturated conditions), c_{sh} = curvature of the shrinkage curve, and w = gravimetric water content.

The hyperbolic fits to the experimental shrinkage data are depicted in Figure 4.5. The model parameters a_{sh}, b_{sh} and c_{sh} determined from the SoilVision software, and are given in Table 4.2.

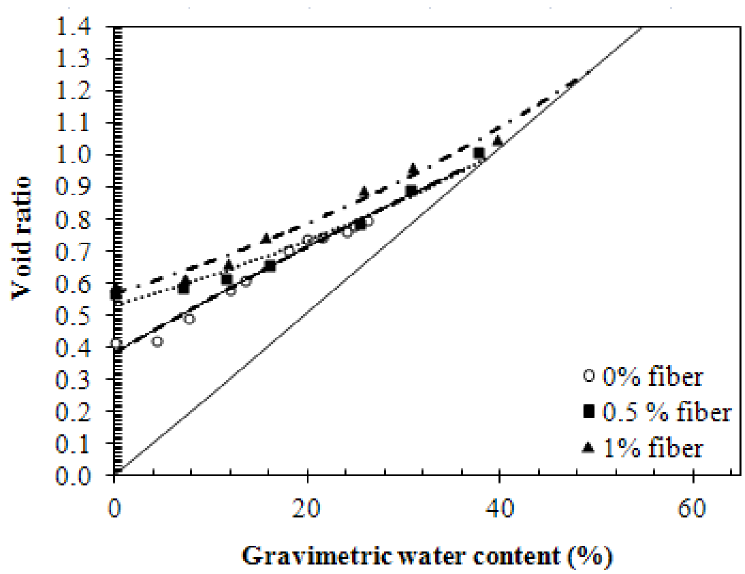

Figure 4.5: Shrinkage curves of unreinforced and reinforced specimens

The parameter a_{sh} represents the minimum void ratio the dried specimens attained, and the b_{sh} values are the minimum water content values at which volume change commences. Studying the parameters in Table 4.2 it can be observed that the a_{sh} parameter, which represents the minimum void ratio at the end of drying increases with fiber content. The b_{sh} parameter also increases with the fiber content, which represents the shrinkage limit, hence indicating the reduction in shrinkage as fiber content increases. Therefore, it can be concluded that there is a significant decrease in volume change with the increasing amount of fiber addition. The reduction in volume change reduced the formation of cracks in the reinforced specimens, which propagated visibly in the unreinforced specimens as observed in Figure 4.6

Table 4.2: Shrinkage model parameters

Fiber content(%)	a_{sh}	b_{sh}	c_{sh}
0	0.46	0.1522616	2.688935
0.5	0.59	0.2208801	3.622919
0.75	0.63	0.2382464	2.745497
1.0	0.67	0.2422591	3.285588

Figure 4.6: Crack pattern on unreinforced and reinforced specimen

4.1.5.1 Soil-water Characteristic Curve

Contact filter paper of Whatman #42 is used according to ASTM D5298 – 10 to measure matric suction. Suction of the reinforced and unreinforced specimens have been obtained under controlled temperature and zero applied stress. After completion of the swell test, the samples are removed and weighed. Then they have been placed in the oven at 40$^{\circ C}$ to simulate the maximum temperature, which can be reached during summer in North Cyprus. Drying soil-water characteristic curves have been obtained and modelled by the use of SoilVision software (Figure 4.7). The best fit could be given by van Genuchten model which indicates the highest R^2 value than other existing models. An increase in air entry value (AEV) with the addition of fiber has been observed which indicates that drying process starts later than the unreinforced soils. A distinct behavior observed in residual water content is that it does not change with the addition of fiber. Thus it can be concluded that residual water content is rarely altered with fiber addition. According to Puppala et al. (2006), the presence of fibers in the fly ash treated soils has no significant influence on volumetric water content of combined stabilized or treated soils, as well as no significant changes in gravimetric water content with the fiber reinforced and unreinforced specimens. Almost the same observation is made in this study, having only a slight increase in the gravimetric water content values of 1% fiber stabilized specimens.

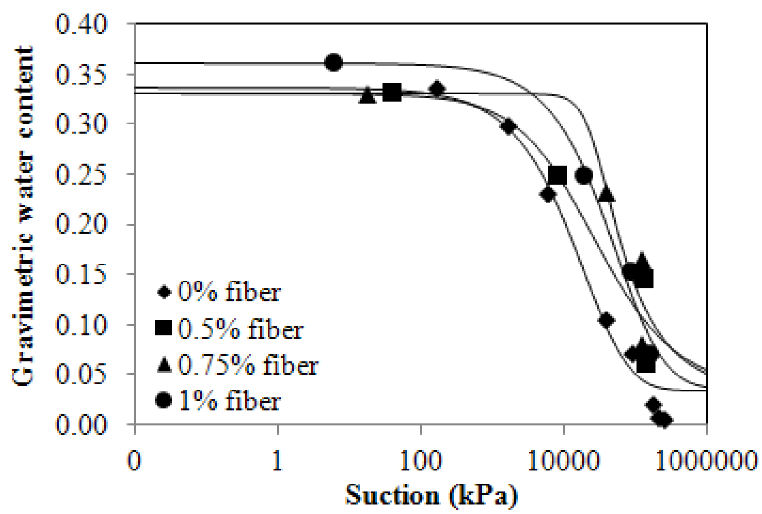

Figure 4.7: Soil-water characteristic curves of different polypropylene fiber contents

According to the van Genuchten (1980) model fitting parameters, given in Table 4.3, the air entry value (AEV) suction increases with the increase in fiber content, which indicates a good bonding between the soil and the fiber, hindering the air entry into the specimens. The n_{vg} parameter represents the slope of the SWCC, and an increase in slope indicates that desorption rate increases with the fiber content.

Table 4.3: Soil-water characteristic curve model parameters (van Genuchten, 1980)

Fiber Content	a_{vg}	n_{vg}	m_{vg}	Residual water content (%)	van Genuchten Error	van Genuchten AEV (kPa)
0%	$6.06x10^{-06}$	0.795	5.66	10	0.98	1964
0.50%	$53.1x10^{-06}$	0.837	0.7959	10	0.92	2090
0.75%	$39.1x10^{-06}$	3	0.2545	10	0.89	17523
1%	$4.65x10^{-06}$	0.847	3.2285	10	0.98	6134

4.1.2.3 One-dimensional Consolidation

The effect of polypropylene fiber on compressibility properties of expansive soils has been investigated by one-dimensional consolidation test. Consolidation pressures up to a maximum of 1568 kPa have been applied during the process. Figure 4.8 gives the results of void ratio versus logarithm of consolidation pressure. Table 4.4 depicts the results of this experiment in which a considerable reduction in the compression and rebound indices can be observed with the increase in fiber content. The expansive soil used in this study which has high swell potential is also highly compressible and this is an undesirable mechanical behavior especially in construction of road pavements, since it causes cracks due to fatigue behavior. Thus fiber inclusions have improved the compressibility behavior.

Table 4.4: Swell pressure and preconsolidation pressure

Fiber content	Compression index (C_c)	Rebound index (C_r)	Preconsolidation pressure (kPa)	Swell pressure (kPa)
0%	0.317	0.131	830	200
0.5%	0.265	0.088	199	80
0.75%	0.230	0.058	505	68
1%	0.186	0.046	120	40

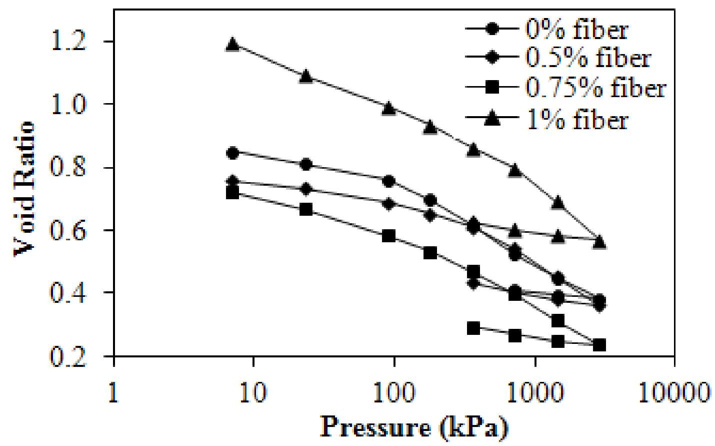

Figure 4.8: Void ratios versus effective consolidation pressure

Further study of these results can lead to the confirmation of considerable reduction of swelling pressures as well which is in agreement with the swell potential reductions. Reduction in the apparent pre-consolidation stress, which can be considered as a degree of bonding in compacted soils is also observed which is due to non-pozzolanic nature of the fiber. Overall, a marked improvement can be observed in the compressibility and swelling properties of the soil which has been mixed with 1% polypropylene fiber.

4.1.3 Hydraulic Conductivity

The hydraulic conductivity of the soil has been determined indirectly by use of consolidation test results. The following expression was used in the calculations:

$$k = c_v \; m_v \, \gamma_w \tag{4.2}$$

where k is the saturated hydraulic conductivity, c_v is the coefficient of consolidation, a_v is the coefficient of compressibility, m_v is the coefficient of volume change, and γ_w is the unit weight of water. The results are shown in Table 4.5 which shows the variation of the hydraulic conductivity for unreinforced and reinforced soils with the enhancement of fiber additions. In pavement design, lower stress ranges are important for analysis since it is the surface soil that is in contact with the main layer of the pavement. The results show variations of hydraulic conductivity with different fiber contents. This behavior might be due to the variation in distribution of the fibers in each sample. Comparing the k obtained for 0-200 kPa and 200-400 kPa ranges of stresses reveals that the addition of 0.5% and 1% fiber reduced the hydraulic conductivity, whereas in the case with 0.75% fiber content a tremendous increase in hydraulic conductivity is observed. This might be attributed to the possibility of experimental nonconformity, as fiber inclusion and uniform mixing is very difficult and may lead to unexpected results.

Table 4.5: Saturated hydraulic conductivity and efficient of the consolidation

Stress ranges (kPa)	a_v	m_v (m^2/kN)	c_v (m^2/s)	k (m/s)
0% Fiber				
0-200	$7.5x10^{-4}$	$1.58x10^{-3}$	$2.69x10^{-3}$	$11.87x10^{-4}$
200-400	$5.0x10^{-4}$	$1.68x10^{-3}$	$2.69x10^{-3}$	$8.41x10^{-4}$
400-800	$2.5x10^{-4}$	$1.79x10^{-3}$	$2.69x10^{-3}$	$4.48x10^{-4}$
800-1569	$1.2x10^{-4}$	$1.88x10^{-3}$	$2.69x10^{-3}$	$2.20x10^{-4}$
0.5% Fiber				
Stress ranges (kPa)	**a_v**	**m_v (m^2/kN)**	**c_v (m^2/s)**	**k (m/s)**
0-200	$4.10x10^{-4}$	$1.81x10^{-3}$	$2.98x10^{-3}$	$7.40x10^{-4}$
200-400	$2.60x10^{-4}$	$1.86x10^{-3}$	$2.98x10^{-3}$	$4.84x10^{-4}$
400-800	$2.00x10^{-4}$	$1.96x10^{-3}$	$2.98x10^{-3}$	$3.91x10^{-4}$
800-1569	$1.43x10^{-4}$	$2.09x10^{-3}$	$2.98x10^{-3}$	$2.99x10^{-4}$
0.75% Fiber				
Stress ranges (kPa)	**a_v**	**m_v (m^2/kN)**	**c_v (m^2/s)**	**k (m/s)**
0-200	$10.0x10^{-4}$	$1.72x10^{-3}$	$2.61x10^{-3}$	$17.14x10^{-4}$
200-400	$3.00x10^{-4}$	$1.79x10^{-3}$	$2.61x10^{-3}$	$53.47x10^{-4}$
400-800	$1.75x10^{-4}$	$1.88x10^{-3}$	$2.61x10^{-3}$	$3.28x10^{-4}$
800-1569	$1.30x10^{-4}$	$2.02x10^{-3}$	$2.61x10^{-3}$	$2.63x10^{-4}$
1% Fiber				
Stress ranges (kPa)	**a_v**	**m_v (m^2/kN)**	**c_v (m^2/s)**	**k (m/s)**
0-200	$9.00x10^{-4}$	$1.60x10^{-3}$	$3.09x10^{-3}$	$14.47x10^{-4}$
200-400	$4.00x10^{-4}$	$1.68x10^{-3}$	$3.09x10^{-3}$	$6.71x10^{-4}$
400-800	$1.25x10^{-4}$	$1.73x10^{-3}$	$3.09x10^{-3}$	$2.16x10^{-4}$
800-1569	$1.69x10^{-4}$	$1.86x10^{-3}$	$3.09x10^{-3}$	$3.14x10^{-4}$

4.1.4 Strength Behavior

4.1.4.1 Unconfined Compressive Strength

Unconfined compression, which is a method that covers the determination of the unconfined compressive strength of cohesive soil in the intact, remolded, or reconstituted condition, using strain-controlled application of the axial load, is done according to the ASTM D2166-06. The unconfined compressive stress is where an unconfined cylindrical specimen of soil will fail in a simple compression test. In this test method, unconfined compressive strength is taken as the maximum load attained per unit area or the load per unit area at 15% axial strain, whichever is secured first during the performance of a test. Samples have been prepared in optimum moisture content and compacted statically by CBR instrument to achieve maximum density required for the unconfined compression mold.

Figure 4.9 demonstrates the stress strain relationship of fiber reinforced and unreinforced soils. Unconfined compressive strength has been observed to increase with increase in fiber content. Therefore soil specimens with 1% fiber have the highest compressive strength and hence the cohesion. These values are displayed in Figure 4.10. Failure of the fiber reinforced specimens is observed to occur at higher deformations than the unreinforced soil, which indicates an increase in ductility of the soil after reinforcement. This behaviour is shown as a plot of strain at failure versus fiber content in Figure 4.11. This observation is further substantiated with the reduction in modulus of elasticity with the fiber addition, indicating reduction in brittle behaviour and therefore enhancement of ductility of the fiber reinforced specimens.

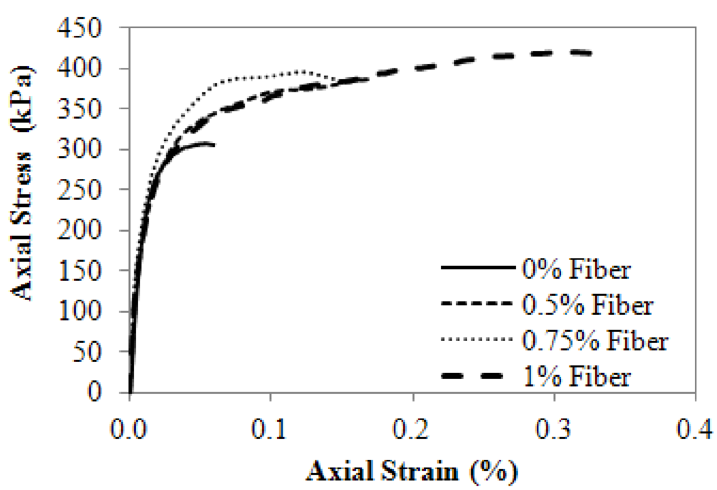

Figure 4.9: Stress-Strain relationship of original and fiber reinforced soils

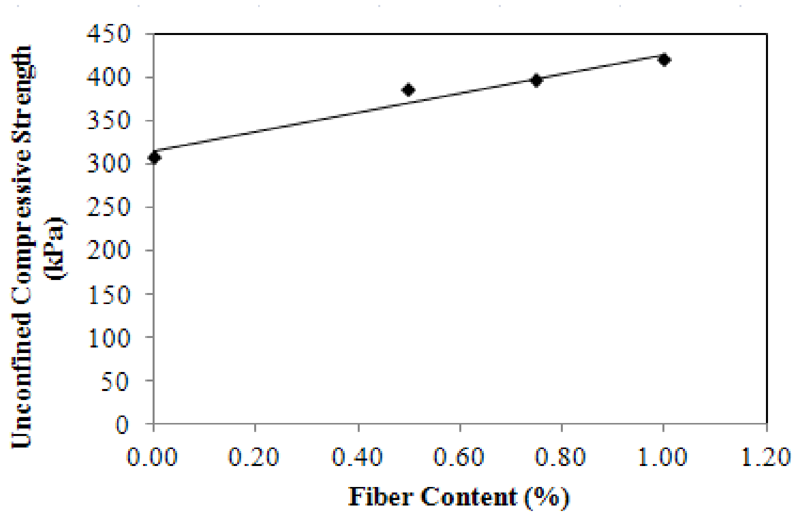

Figure 4.10: Unconfined compressive strength versus fiber content

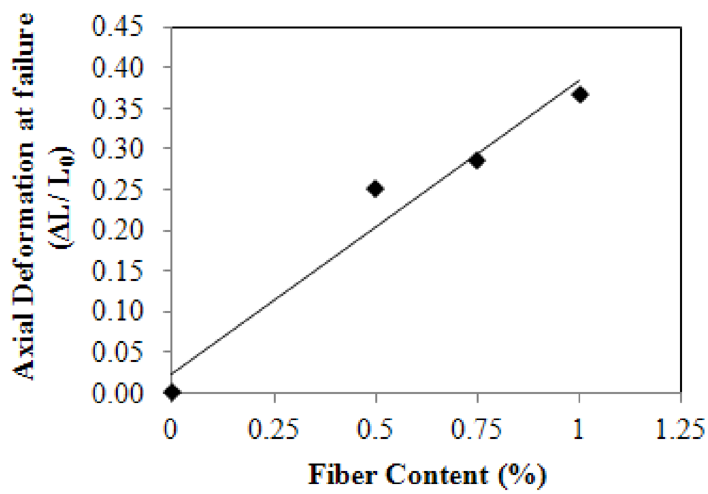

Figure 4.11: Deformation versus fiber content

4.1.4.2 Tensile Strength

Split tensile strength is a method to measure the tensile strength of concrete. This method has been used according to the ASTM D3967-08 to measure the tensile strength of reinforced and unreinforced soil specimens. Since polypropylene fiber is improving the shrinkage behavior of soil as well as concrete; therefore it is important to measure this parameter to present the resistance of this material to tension. Samples of 50 mm diameter and 100 mm height have been prepared and tested in the split tensile strength equipment. The split tensile strength is calculated by the following equation:

$$\text{Split tensile strength} = 2P/\pi td \qquad (4.3)$$

where P is the failure load; t is thickness or length of the specimen; and d is the diameter of the specimen. This test method is developed for assessment of concrete tensile strength therefore a correction factor is required when testing soils to account for the ductile behavior. Equation 4.2 is used to determine the correction factor g(x) (Frydman 1964).

$$g(x)=(-\ d/2a)\{2x- \sin 2x -2y/d\ \log_e \tan(\pi/4 + x/4)\} \qquad (4.4)$$

where,

x= (a/y) and “a” is defined as width of flattening and “y” is the distance between the flattened portions. Figure 4.12 presents the value of split tensile strength versus fiber content and it is indicated that with the addition of fiber the tensile strength of the soil has increased significantly.

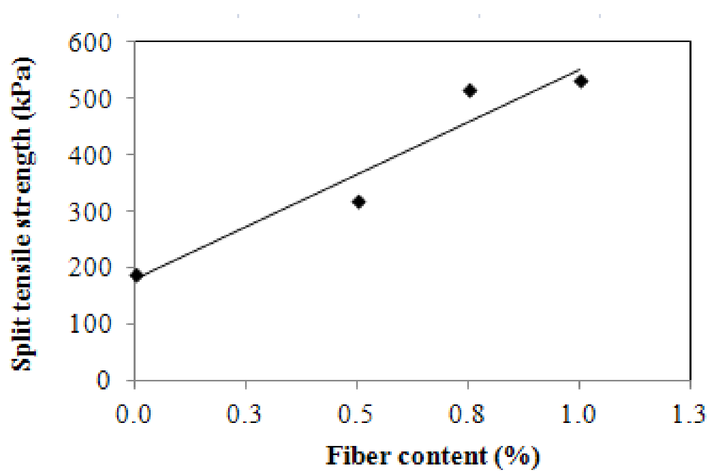

Figure 4.12: Split tensile strength versus fiber content

Figure 4.13 illustrates the ratio of unconfined compressive strength over split tensile strength of the reinforced and original soil with the variation of fiber content. The results show an increase of the ratio with an increase in fiber content of the specimens. This indicates that reinforcement of the soils with polypropylene fiber is more effective in improving tension than compression, and that increases the ductility of the soil, which is very important mainly during desiccation, preventing shrinkage settlements causing detrimental damage to structures, such as roads and pavements.

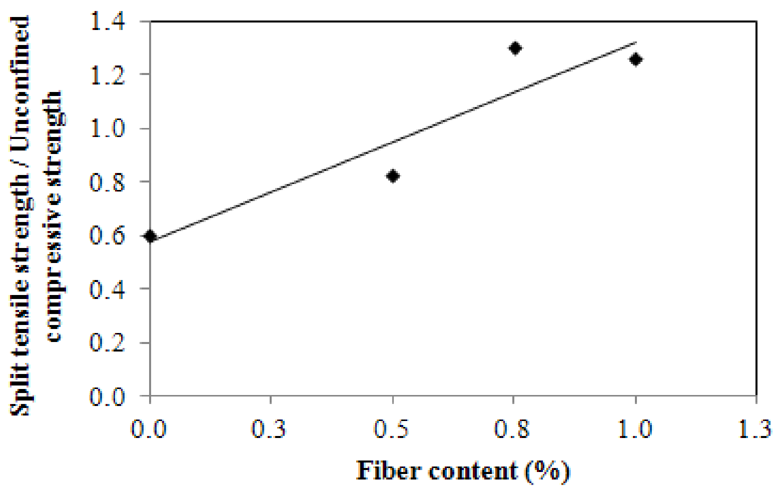

Figure 4.13: Split tensile strength/unconfined compressive strength ratios

The increase in ductility can also be observed in Figure 4.14, which shows the specimens at the end of split tensile testing with varying fiber inclusions. The increase in deformations in the transverse direction is very evident, hence indicating the enhancement in ductility.

Figure 4.14: Failed unreinforced and reinforced samples after split tensile strength

4.2 Use of Posidonia Oceanica Ash

The second phase of the study includes research on potential use of a natural material, Posidonia oceanica (seaweed), in the form of ash, as an innovative material for mitigation of expansive soils. This is aimed to recycle a natural waste material found abundantly along the shores of Cyprus.

4.2.1 Specific Gravity

Specific gravity of the soil in original state and with different percentages of 5% and 10% Posidonia oceanica ash has been determined according to the ASTM D 854 – 06. Figure 4.15 shows that with addition of Posidonia oceanica ash the specific gravity of the soil mixtures has been reduced tremendously, which indicates the reduction in the soil density and soil solid particles. This behavior is due to the replacement of the Posidonia oceanica ash with the clay particles of the soil where the ash has lower mass in proportion to the soil particles.

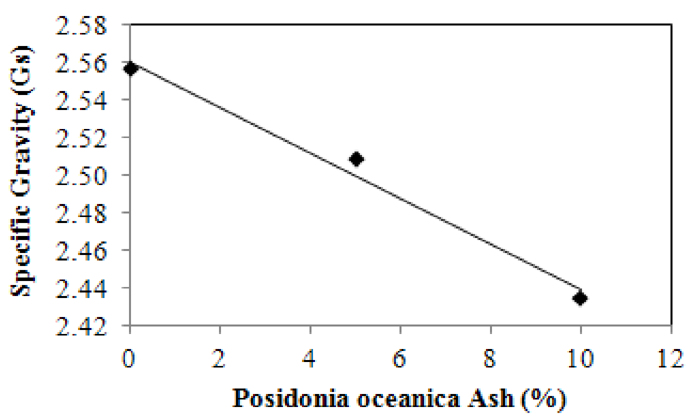

Figure 4.15: Specific gravity versus Posidonia oceanica ash

4.2.2 Grain Size Distribution

Particle sizes larger than 75 μm (retained on the No. 200 sieve) is determined by sieve analysis, while the distribution of particle sizes smaller than 75 μm can be determined by a sedimentation process, using a hydrometer analysis according to ASTM D422-54T. Hydrometer analysis has been done with 0%, 5%, and 10% Posidonia oceanica ash additions to determine the effect of PO ash on grain size distribution of the expansive soil and the results are presented in Figure 4.16.

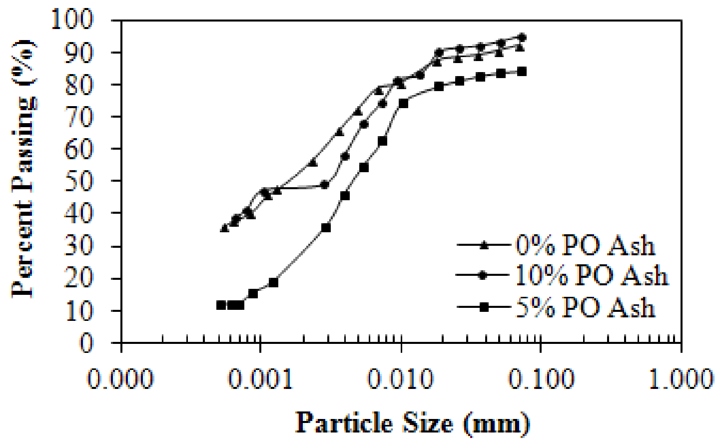

Figure 4.16: Grain size distribution of soil mixed with different percentage of Posidonia oceanica ash

4.2.3 Atterberg Limits

Once the soil is obtained from the field, the very first laboratory experiment is determination of Atterberg limits, which is referred to as liquid limit and plastic limit. With the use of this test, we can obtain the water content at which soil behaves like liquid, plastic or semi plastic.

Effect of PO ash has been analyzed on Atterberg limits by adding PO ash of 5% and 10% by dry unit weight of the soil and the changes of the Atterberg limits have been recorded. As it is stated in Figure 4.17 the soil classification has changed from CH to MH. There is a reduction trend in plasticity index of the mixture shown in Figure 4.18.

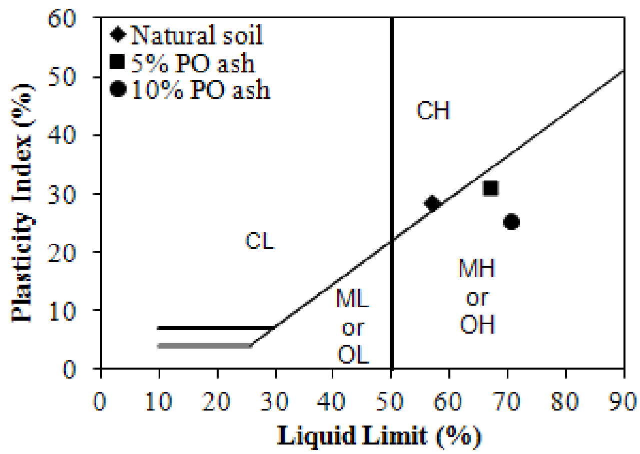

Figure 4.17: Cassagrande Plasticity Chart

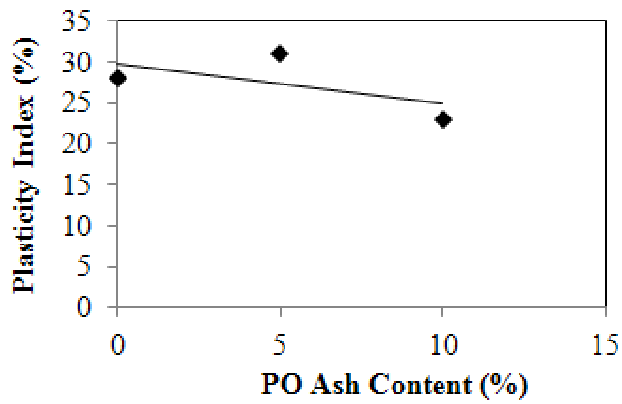

Figure 4.18: Plasticity index versus Posidonia oceanica ash

4.2.4 Linear Shrinkage

Linear shrinkage of the soil mixtures with 0%, 5% and 10% Posidonia oceanica ash has been determined. The length of the mold is measured by compass and the soil mixture is placed inside the mold. A reduction in shrinkage limit has been observed from 21% to the shrinkage limit of 18% and 17% for 5% and 10% Posidonia oceanica ash, respectively. Therefore a reduction of linear shrinkage is observed by addition of the PO ash.

4.2.5 Swell

Samples of swell and consolidation test have been compacted at the optimum moisture content and maximum dry density of the natural soil (0% ash) statically.

Figure 4.19 indicates the swell percentage versus logarithm of time for different Posidonia oceanica ash contents. It is observed that the primary swell has increased with 5% Posidonia Oceanica ash whereas there is a reduction in primary swell with addition of 10% ash. Additionally, changes in secondary swells of stabilized samples are not tangible.

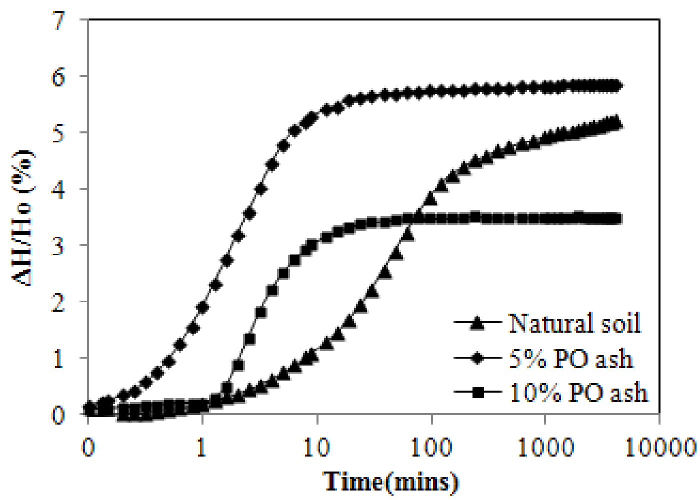

Figure 4.19: Swell percentage versus time

4.2.6 Unconfined Compressive Strength

Stress strain relationships of original soil and the soil ash mixtures can be observed in Figure 4.20 where it is indicated that with 5% PO addition the unconfined compressive strength increases slightly, yet the sample attains a brittle behaviour. Additionally with the inclusion of 10% PO ash an increase in compressive strength has been monitored. Furthermore, there is a significant improvement in the modulus of elasticity of the specimens. Figure 4.21 shows the unconfined compressive strength increments with ash addition.

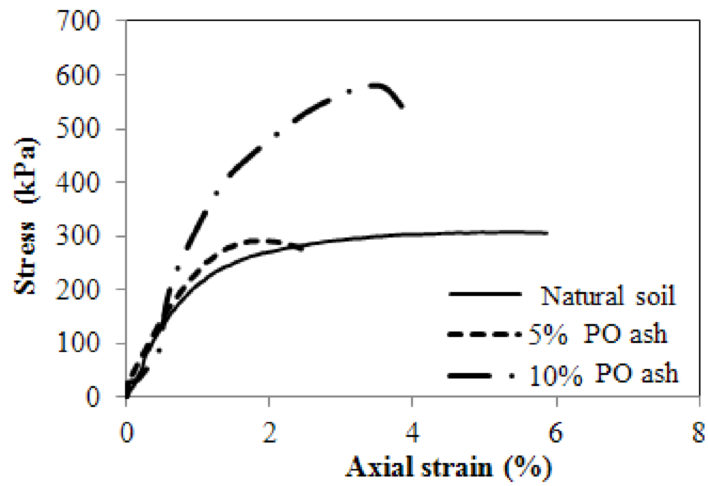

Figure 4.20: Stress-strain relationship under unconfined compressive strength test

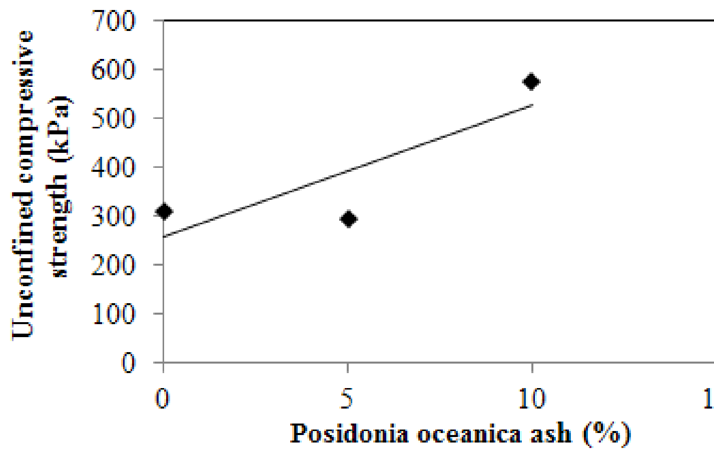

Figure 4.21: Unconfined compressive strength versus percent Posidonia oceanica ash

Chapter 5

CONCLUSIONS

This research was intended to consider the most suitable percentage of a synthetic (polypropylene fiber), and the suitability for use of a natural waste additive (Posidonia oceanica) for the use in the improvement of swelling-shrinking soils in North Cyprus. In the first phase of this study, a series of tests were carried out on unreinforced and reinforced soil mixed with different amounts of polypropylene fiber. The experimental program included all the tests required for physical identification, swell-shrinkage, suction measurement, compressibility, unconfined compression test and tensile strength test. In the second phase of this experimental work Posidonia oceanica ash is used, which is a type of sea weed found in abundance along the shores of North Cyprus. Its suitability for soil improvement is sought so that this naturally found waste causing visual pollution could be recycled in an efficient way.

The results of the experimental program carried out on unreinforced and reinforced soil can be concluded as follows:

1. Regarding compaction characteristics, optimum water content is not influenced by polypropylene fiber inclusion whereas maximum dry density has been reduced. This can be attributed to the reduction of average unit weight of solids in the mixture of soil and fiber.

2. Studying the influence of polypropylene on volume change in swell-shrinkage behavior, the overall conclusion is that primary swell, time of completion of primary swell and secondary swell decrease considerably with fiber addition.

3. Fiber inclusions have improved the shrinkage properties, and hence reducing potential settlements which might occur due to environmental effects.

4. Based on consolidation test results there is marked reduction in compression index in addition to the rebound index. The apparent preconsolidation stresses obtained, which give a measure of bonding or cementation also reduce. The reductions in swell pressures are in good agreement with the swell potentials obtained from one-dimensional swell tests.

5. Unconfined compressive strength increased with inclusion of polypropylene fiber. Maximum unconfined compressive strength value can be observed with 1% fiber content, which is approximately 1.5 times of the unreinforced soil.

6. From the analysis of split tensile strength, it is observed that the maximum value of the split tensile strength obtained for 1% fiber inclusion is 2.7 times of the natural soil. Increase in the ratio of tensile strength to compressive strength indicates that reinforcement of the soils with polypropylene fiber is more effective in improving tension than compression. This behaviour increases the ductility of the soil, which is very important mainly during desiccation, preventing shrinkage settlements causing detrimental damage to structures, such as roads and pavements.

7. The results obtained from the second phase of this research, where the influence of PO ash on the properties of the expansive soils has been analyzed, are as follows: The Atterberg limits results indicate a reduction in plasticity index as well as specific gravity, one dimensional swell analysis shows that with the addition of 10% PO ash primary swell has reduced tremendously, in a case that no secondary swell could be observed for the soil samples mixed with both 5% and 10% PO ash, unconfined compressive strength has enhanced to approximately 2 times of the natural soil in comparison with 10% PO ash addition.

8. The use of Posidonia oceanica, abundantly found along the coastline of North Cyprus, has proved to be a potential material for further research on soil improvement.

Finally, it can be concluded that there is a potential for use of polypropylene fiber to reinforce expansive soils in North Cyprus. However, further research is recommended with higher fiber contents, which would give lower swell potentials, and for substantiating the conclusions herein. Accumulation of Posidonia oceanica along the coastline of Cyprus, even though non-toxic, is a visual pollution. Recycling this natural waste as an additive in soil stabilization is a very innovative topic which needs further attention.

REFERENCES

Abduljauwad, S.N. (1993). Treatment of calcareous expansive clays. In Fly ash for soil improvement. Edited by K.D. Sharp, ASCE, American Society of Civil Engineers. Geotechnical Special Publication. 36, 100–115.

Alonso, E.E., Gens, A., and Josa, A. (1990). A constitutive model for partially saturated soils. Géotechnique. 40, 405–430.

Al-Refeai, T. (1991). Behavior of granular soils reinforced with discrete randomly oriented inclusions. Geotextiles and Geomembranes. 10, 319–333.

Amu, O.O., Adeyeri, J.B., Oduma E.W. and Fayokan O.A. (2008). Stabilization characteristics of lime on palm kernel blended lateritic soil. Trends applied science. 3, 182-188.

Arabani, M., and Veis Karami, M., (2007). Geomechanical properties of lime stabilized clayey sands. The Arabian Journal for Science and Engineering. 32 (1B).

Ayyappan, S., Hemalatha, K., and Sundaram, M. (2010). Investigation of Engineering Behavior of Soil, Polypropylene Fibers and Fly Ash -Mixtures for Road Construction. International Journal of Environmental Science and Development. 1 (2).

Balasubramaniam, A.S., Phien-wej, N., Indraratna, B., and Bergado, D.T. (1989). Predicted behavior of a test embankment on a Malaysian marine clay. Int. Symp. on Trial Embankments on Malaysian Marine Clays. Kuala Lumpur, Malaysia, 2, 1-1-1-8.

Bell, J.D., and Harmelin-Vivien, M.L. (1983). Fish fauna of french mediterranean Posidonia oceanica seagrass meadows feeding habits. Téthys, 11 (1), 1–14.

Bilsel, H. (2002). Hydraulic properties of soils derived from marine sediments of Cyprus. Journal of arid enviroment. 56 (1), 27-41.

Blanche, KR. (1992). Preliminary observations on the distribution and abundance of seaweed flies (Diptera: Coelopidae) on beaches in the Gosford district of New South Wales, Australia. 17 (27) 34.

Blandin, P. (1986). Bioindicateurs et diagnostic des systèmes écologiques. Bull. Ecol. 17 (4), 211–307.

Blatz, J. A., and Graham, J. (2003). Elastic-plastic modelling of unsaturated soil using results from a new triaxial test with controlled suction. Geotechnique. 53(1), 113-122.

Cai, Y., Shi, B., Ng, C.W.W., and Tang, C.S. (2006). Effect of polypropylene fiber and lime admixture on engineering properties of clayey soil. Engineering Geology. 87, 230-240.

Ceccherelli, G., and Cinelli, F. (1999). Effects of posidonia oceanica canopy on caulerpa taxifolia size in a north-western mediterranean bay. Journal of Experimental Marine Biology and Ecology. 240, 19–36.

Chen, F.H. (1988). Foundations; swelling soils . Amsterdam and New York and New York, U.S.A.: Elsevier DOI: ISBN 0444430369.

Cocozza, C., Parente, A., Zaccone, C., Mininni, C., Santamaria, P., and Miano, T. (2010). Chemical, physical and spectroscopic characterization of posidonia oceanica (l.) del. residues and their possible recycle. Biomass and bioenergy. 35, 799-807.

Consoli, N.C., Casagrande, M.D.T., Prietto, P.D.M., and Thomé A. (2003b). Plate load test on fiber-reinforced soil. Journal of Geotechnical and Geoenvironmental Engineering. 129 (10), 951–955.

Consoli, N.C., Bassani, M.A.A., Festugato, L. (2010). Effect of fiber-reinforcement on the strength of cemented soils. Geotextiles and Geomembranes. 28 (4), 344-351.

Consoli, N.C., Montardo, J.P., Prietto, P.D.M. and Pasa, G.S. (2002). Engineering behavior of a sand reinforced with plastic waste. Journal of Geotechnical and Geoenvironmental Engineering. 128 (6), 462–472.

Consoli, N.C., Prietto, P.D.M., and Ulbrich, L.A. (1998). Influence of fiber and cement addition on behavior of sandy soil. Journal of Geotechnical and Geoenvironmental Engineering. 124 (12), 1211–1214.

Consoli, N.C., Vendruscolo M.A., and Prietto, P.D.M. (2003a). Behavior of plate load tests on soil layers improved with cement and fiber. Journal of Geotechnical and Geoenvironmental Engineering. 129 (1), 96–101.

Croney, D., and Coleman, J.D. (1961). Pore pressure and suction in soil. In: Pore Pressure and Suction in Soil. Institution of Civil Engineers, London. 31–37.

Dall'Aqua, G.P. and Ghataora G.S., and Ling, U.K. (2010). Behavior of fiber-reinforced and stabilized clayey soils subjected to cyclic loading. Studia Geotechnica et Mechanica. xxxii(03).

Day, R.W. (1999). Geotechnical and foundation engineering. McGraw-Hill, New York

De Falco, G., Simeone, S., Baroli, M. (2008). Management of beach-cast Posidonia oceanica seagrass on the island of Sardinia (Italy, Western Mediterranean). J Coast Res. 24 (4C), 69-74.

De Lumley, H., Pillard, B., Pillard, F. (1969). L'habitant et les activite´s de l'homme du Lazaret. Une cabanne acheule´enne de la Grotte du Lazaret (Nice). Me´moires de la Socie´te´ Pre´historique Franc¸aise. (7),183-222 [in French].

Delage, P., and Graham, I. (1996). State of the art report – Understanding the behavior of unsaturated soils requires reliable conceptual models. In Proceedings of the 1st International Conference on Unsaturated Soils. (3), 1223–1256.

Diambra, A., Ibraim, E., Muir Wood D., and Russell, A.R. (2010). Fiber reinforced sands: Experiments and modelling. Geotextiles and Geomembranes. 28 (03), 238-250.

Diviacco, G., Tunesi, L., Boudouresque, CF. (2006). Feuilles mortes de Posidonia oceanica, plages et re´ensamblement. Pre´servation et conservation des herbies a` Posidonia oceanica. 61-69.

Duarte CM. (2004). How can beaches be managed with respect to seagrass litter? European seagrasses: an introduction to monitoring and management. Copenhagen: EU project Monitoring and Management of European Seagrass Beds. 83.

Duarte, C.M. (1991a). Seagrass depth limits. Aquat. Bot. 40, 363–377.

Duarte, C.M. (1999). Seagrass ecology at the turn of millenium: challenges for the new century. Aquat. Bot. 65, 7–20.

Duarte, C.M. (2002). The future of seagrass meadows. Environ. Conserv. 29 (2), 192–206.

Dunlap, W.A., Epps, J.A., Biswas, B.R., Gallaway, B.M. (1975). United States Air Force Soil Stabilization Index System: A Validation. Report No. AFWL-TR-73-150, Air Force Weapons Laboratory, Kirtland Air Force Base, New Mexico.

Edil, T.B., Acosta, H.A., and Benson, C.H. (2006). Stabilizing soft fine-grained soils with fly ash. Journal of materials in civil engineering. 18 (2), 283-294.

El-Deek, A.A., and Mervat, A.B. (2009). Nutritional and biological evaluation of marine seaweed as a feedstuff and as a pellet binder in poultry diet. International Journal of Poultry Science, 8 (9), 875-881.

Emmanuel, A.O. (2010). Geotechnical properties of some coal fly ash stabilized southwestern nigeria lateritic soils. Modern Applied Science. 4 (12), 66-73.

Erdal, C. (2002). Relationship between methylene blue value, initial soil suction and swell percent of expansive soils. Turkish J. Eng. Env. Sci. 26, 521-529.

Fredlund, D.G. (1997). An introduction to unsaturated soil mechanics. Geotechnical Special Publication. Geo-Logan.

Fredlund, D.G. (2002). Use of the soil-water characteristic curve in the implementation of unsaturated soil mechanics, Proceedings of the 3rd International Conference on Unsaturated Soils.

Fredlund, D.G., and Morgenstern, R. (1976). Constitutive relations for volume change in unsaturated soils. Canadian Geotechnical Journal. 13, 261 -276.

Fredlund, D.G., and Rahardjo, H. (1993). Soil mechanics for unsaturated soils. John Wiley and Sons Inc., New York, NY.

Freilich, B.J.L., and Zornberg, J.G. (2010). Effective shear strength of fiber-reinforced clays. 9_{th} International Conference on Geosynthetics.

Frydman, S. (1964). The applicability of the Brazilian (indirect tension) test to soils. Aust. J. appl. Sci. 15, 335.

Gacia, E., and Duarte, C.M. (2001). Sediment retention by a mediterranean posidonia oceanica meadow: The balance between deposition and resuspension. Estuarine, Coastal and Shelf Science. 52, 505–514.

Gacia, E., Granata, T.C., and Duarte C.M. (1999). An approach to measurement of particle flux and sediment retention within seagrass (posidonia oceanica) meadows. Aquatic Botany. 65, 255–268.

Gambi, M.C., Buia, M.C., Casola E., and Scardi, M. (1989). Estimates of water movement in Posidonia oceanica beds: a first approach, International Workshop Posidonia oceanica Beds. 101–112.

Gens, A., Alonso, E. E., Suriol, J. and Lloret, A. (1995). Effect of structure on the volumetric behavior of a compacted soil, in: Unsaturated Soils: Proc. 1st Int. Conf. on Unsaturated Soils. 83–88.

Ghazavi, M., Roustaie, M. (2010). The influence of freeze and thaw cycles on the unconfined compressive strength of fiber reinforced clay. Cold Region science and technology. 61, 125-131.

Gray, D.H., and Al-Refeai, T. (1986). Behavior of fabric vs. fiber-reinforced sand. Journal of Geotechnical Engineering. 112 (8), 804-820.

Gray, D.H., and Ohashi, H. (1983). Mechanics of "fiber reinforcement in sand. Journal of Geotech. Engng. 109 (3), 335-353.

Grim, R.E. (1968). Clay Mineralogy, McGraw-Hill Book Co., Second Edition.

Guidetti, P. (2001). Detecting environmental impacts on the mediterranean seagrass posidonia oceanica: the use of reconstructive methods in combination with ‘beyond baci’ designs. Journal of Experimental Marine Biology and Ecology. 260, 27–39.

Guney, Y., Sari, D., Cetin, M., and Tuncan, M. (2006). Impact of cyclic wetting–drying on swelling behavior of lime-stabilized soil. Building and Environment. 42 (2), 681-688.

Hamilton, J.J. (1977). Foundations on swelling or shrinking subsoils. CBD-184, Canadian Building Digest, National Research Council, Canada.

Harianto, T., and Hayashi, S., and Du., Y.J., and Suetsugu, D. (2008). Water Air Soil Pollut. 194, 141–149.

Hausmann, E. (1990). A contemporary perspective on techniques for the clinical assessment of alveolar bone. Journal of Periodontology. 149–156.

Heineck K.S., Coop, M.R., and Consoli, N. (2005). Effect of microreinforcement of soils from very small to large shear strains. Journal of Geotechnical and Geoenviromental Engineering. 131 (8), 1024-1033.

Holtz, W.G., Robert, D. (2001). Geosynthetics for Soil Reinforcement Seattle, Washington. 3-4.

Ibraim, E., Diambra, A., Muir Wood D., Russell, A.R. (2010). Geotextiles and Geomembranes. 28, 374–385.

Jeudy de Grissac, A., Boudouresque, C.F. (1985). Rôles des herbiers de phanérogames marines dans les mouvements des sédiments côtiers: les herbiers à Posidonia oceanica. Les aménagements côtiers et la gestion du littoral, Coll. pluridisciplinaire franco-japonais océanographie. 143–15.

Jones, D.E., and Holtz, W.G. (1973). Expansive Soils - the hidden disaster. Civil Engineering. 43 (8), 49-51.

Kaniraj, S.R., and Gayathri, V. (2003). Geotechnical behavior of fly ash mixed with randomly oriented fiber inclusions. Geotextiles and Geomembranes. 21, 123-149.

Kassim, K.A., and Kok, K.C. (2004). Lime Stabilized Malaysian Cohesive Soil, Jurnal Kejuruteraan Awam. 16 (1), 13-23.

Kavak, A., and Akyarli, A. (2007). A field application for lime stabilization. Environmental Geology. 51, 997.

Kelley, C.M. (1976). A Long Range Durability Study of Lime Stabilized Bases at Military Posts in the Southwest. National Lime Association Bulletin 328.

Kim, Y.T., Kim, H.J., and Lee, G.H. (2008). Mechanical behavior of lightweight soil reinforced with waste fishing net. Geotextiles and Geomembranes. 26 (06), 512-518.

Kirkman, H., and Hendrick, G.A. (1997). Ecological significance and commercial harvesting of drifting and beach-cast macroalgae and seagrasses in Australia: a review. J Appl Phycol. 9 (3) 11-26.

Koerner, R.M., and Welsh, J.P. (1980). Constrcution and geotechnical engineering using synthetic fabrics. Wiley, New York.

Kumar, A., Walia, B.S., and Bajaj, A. (2007). Influence of fly ash, lime, and polyester fibers on compaction and strength properties of expansive soil. Journal of Materials In Civil Engineering. 19 (03), 242–248.

Lambe, W.T., and Whitman, R.V. (1969). Soil mechanics. John Wiley and Sons, New York.

Last, J.C., and Duarte, C.M. (2001). Detrital stocks and dynamics of the seagrass posidonia oceanica (l.) delile in the spanish mediterranean. Aquatic Botany. 70, 295–309.

Little, D.N., Scullion, T., Kota, P.V., and Bhuiyan, J. (1995). Guidelines for Mixture Design and Thickness Design for Stabilized Bases and Subgrades. Report No. 1287-3F, Texas Transportation Institute.

Locat, J., Berube, M.A., and Choquette, M. (1990). Laboratory investigations on the lime stabilization of sensitive clays: shear strength development. Canadian Geotechnical Journal. 27, 294–304.

Locat, J., Tremblay, H., and Leroueil, S. (1996). Mechanical and hydraulic behavior of a soft inorganic clay treated with lime. Canadian Geotechnical Journal. 33, 654–669.

Lucian, C. (1996). Structural survey and geotechnical investigation works. A case of the Regional Office Block for Coast Region (Consultant Report), Kibaha, Tanzania.

Maher M.H. and Gray D.H. (1990). Static response of sands reinforced with randomly distributed fibers. Journal of Geotechnical Engineering. 116 (11), 1661–1677.

Maher, M.H., and Ho, Y.C. (1994). Mechanical Properties of Kaolinite /Fiber soil composite. Journal of Geotechnical Engineering. 120 (8), 1381-1393.

Marba, N., Hemminga, M.A., Mateo, M.A., Duarte, C.M., Maas, Y.E.M., Terrados, J., and Gacia, E. (2002b). Carbon and nitrogen translocation between seagrass ramets. Mar. Ecol. Prog. Ser. 226, 287– 300.

Mateo, M.A´., Sa´nchez-Lizaso J.L., Romero, J. (2003). Posidonia oceanica “banquettes”: a preliminary assessment of the relevance for meadow carbon and nutrients budget. 56, 85-90.

Michalowski, R.L. and Zhao, A. (1996). Failure of fiber-reinforced granular soils. J. Geotech. Geoenviron. 122 (3), 226–234.

Michalowski, R.L., and Cermak, J. (2003). Triaxial compression of sand reinforced with fibers. Journal of Geotechnical and Geoenvironmental Engineering, 129 (2), 125–136.

Mitchell J.K. (1993). Fundamentals of soil behavior. New York: John Wiley and Sons Inc.

Morel, J.C., and Gourc, J.P. (1997). Mechanical behavior of sand reinforced with mesh elements. Geosynthetics International. 45, 481–508.

Moreno, D., Aguilera, P.A., and Castro, H. (2001). Assessment of the conservation status of seagrass (posidonia oceanica) meadows: implications for monitoring strategy and the decision-making process. Biological Conservation. 102, 325–332.

Nalbantoglu, Z., and Gucbilmez, E. (2002). Utilization of an industrial waste in calcareous expansive clay stabilization. Geotechnical Testing Journal, 25 (1), 78–84.

Narasimha Rao, S., Rajasekaran, G., and Prasad, C.V. (1993). Lime column method of stabilization in a marine clay, Proceedings Eleventh Asian Geotechnical Conference, Singapore. 397–402.

Nishimura, T. and Fredlund, D.G. (2000). Relationship between shear strength and matric suction in an unsaturated silty soil. Unsaturated Soils for Asia.

Okumara, T., and Terashi, M. (1975). Deep lime mixing method of stabilization for marine clays, Proceedings Fifth Asian Regional Conference on Soil mechanics and Foundation Engineering. (2), 69–76.

Olive, W.W., Chleborad, A.F., Frahme, C.W, Schlocker, J., Schneider, R.R., Shuster, R.L., (1989). Swelling clays map of the conterminous United States. US Geological Survey Publication.

Ott, J.A. (1980). Growth and production in Posidonia oceanica (L.). Mar Ecol (1), 47-64.

Pergent-Martini, C., Leoni, V., and Pasqualini, V. (2005). Descriptors of posidonia oceanica meadows: Use and application. Ecological Indicators. 5, 213–230.

Phani Kumar, B. R., Radhey, S., and Sharma, M. (2004). Effect of fly ash on engineering properties of expansive soils. Journal of Geotechnical and Geoenviromental Engineering. 130 (7), 764–767.

Prabakar, J., and Sridhar, R.S. (2002). Effect of random inclusion of sisal fiber on strength behavior of soil. Construction and Building Materials. 16, 123–131.

Prabakar, J., Sridhar, R.S. (2002). Effect of random inclusion of sisal fiber on strength behavior of soil. Construction and Building Materials. 16, 123–131.

Procaccini, G., and Mazzella, L. (1998). Population genetic structure and gene flow in the seagrass posidonia oceanica assessed using microsatellite analysis. Marine ecology progress series. 169, 133-141.

Prusinski J.R., and Bhattacharja, S. (1999). Effectiveness of portland cement and lime in stabilizing clay soils. Transportation research report. No 1652.

Punthutaecha, K., Puppala, A.J., Vanapalli, S.K., and Inyang, H. (2006). Volume change behaviors of expansive soils stabilized with recycled ashes and fibers. J. Materials in Civil Engineering. 18 (2), 295-306.

Rahardjo, Y.S.P., Weber, F.J., le Comte, E.P., Tramper, J., Rinzema, A. (2002). Contribution of aerial hyphae of Aspergillus oryzae to respiration in a model solid-state fermentation system. Biotechnol Bioeng. (78) 539-544.

Ranjan, M., Wong, J., Shi Y.B. (1994). J. Biol. Chem. 269, 24699–24705.

Rao, S.M., and Shivananda, P. (2005). Compressibility behavior of lime-stabilized clay. Geotechnical and Geological Engineering. 23, 309–319.

Richards, F.A. (1958). Dissolved silicate and related properties of some western North Atlantic and Caribbean waters. J. Marinc Rcs. (17), 449-465.

Romero, J., Pergent, G., Pergent-Martini, C., Mateo, M.A., Regnier, C. (1992). The detritic compartment in a Posidonia oceanica meadow: litter features, decomposition rates and mineral stocks. Mar. Ecol. 13 (1), 73–83.

Ruiz, J.M., Perez, M., Romero, J. (2001). Effects of fish farm loadings on seagrass (Posidonia oceanica) distribution, growth and photosynthesis. Mar. Pollut. Bull. 42 (9), 749-760.

Sahin Zaimoglu, A. (2010). Freezing–thawing behavior of fine-grained soils reinforced with polypropylene fibers. Cold Regions Science and Technology. 60 (1), 63-65 .

Sillers, W.S., Fredlund, D.G., Zakerzadeh, N. (2001). Mathematical attributes of some soil-water characteristic curve models. Geotechnical and Geological Engineering, (19), 243-283.

Tang, C., Shi, B., Gao, W., Chen, F., Cai, Y. (2007). Strength and mechanical behavior of short polypropylene fiber reinforced and cement stabilized clayey soil. Geotextiles and Geomembranes. 25, 194–202.

Terrados, J., Borum, J. (2004). Why are seagrasses important? – Goods and services provided by seagrass meadows. European seagrasses: an introduction to monitoring and management. Copenhagen: EU project Monitoring and Management of European Seagrass Beds. 8-10.

Terzagi, D.K. (1943). Theoretical Soil Mechanics, New York, Wiley.

Thompson, M.R., and Dempsey, B. J. (1969). Autogenous Healing of Lime-Soil Mixtures. Highway Research Record. 263.

Tourtelot, H.A. (1973). Geologic Origin and Distribution of Swelling Clays. Proceedings of a workshop on Expansive Clays and Shales in Highway Design and Construction. 1, 44-69.

USA army report. (1983). Foundations in expansive soils Technical Manual TM 5-818-7.

Vanapalli, S.K., and Fredlund (2000). Comparison of different procedures to predict unsaturated soil shear strength. ASTM Proceedings, Unsaturated Soils.

Vaughan, P.R., Maccarini, M., and Mokhtar, S.M. (1998). Indexing the engineering properties of residual soil. Engineering Geology Special Publications.

Viswanadham, B.V.S., Phanikumar, B.R., Mukherjee, R.V. (2009). Swelling behavior of a geofiber-reinforced expansive soil. Geotextiles and Geomembranes. 27 (1), 73-76.

Wheeler, S.J., and Sivakumar, V. (1995). An elasto-plastic critical state framework for unsaturated soil. Géotechnique. 45 (1), 35–53.

Yetimoglu, T., and Salbas, O. (2002). A study on shear strength of sands reinforced with randomly distributed discrete fibers. Geotextiles and Geomembranes, 21 (2).

Zia, N., and Fox, P.J. (2000). Engineering Properties of Loess-Fly Ash Mixtures for Road base Construction, Transportation Research Record. 1714, 49-56.

Zornberg, J.G. (2002). Discrete framework for limit equilibrium analysis of fiber reinforced soil. Geotechnique. 52 (8), 593–604.