Volume Change and Strength Behavior of Expansive Clay Stabilized with Scrap Tire Rubber

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ABSTRACT

Expansive soils in semi-arid climates are well known to cause detrimental damages in the form of cracks and differential movements of the civil engineering structures. In quest for economical and efficient methods for mitigation of swelling soils, there is a growing interest in using waste materials with a secondary aim of recycling the material. This study concentrated on using scrap tire rubber (STR) powder as a stabilizer for improving the locally available expansive soil. Using the scrap tire rubber in soil stabilization can be a feasible solution for recycling this waste material, hence reducing the environmental problems, as discarded waste tires impose a major environmental pollution. An experimental program was conducted to evaluate the mechanical characteristics of the expansive soil, including swell-shrinkage potential, hydraulic properties, and strength characteristics. Different percentages of STR (10%, 20%, and 30%) were mixed with natural expansive soil to evaluate its influence on physical and mechanical properties. The experimental findings displayed that 30% STR reduced the plasticity and swell potential considerably, while no change in compressibility parameters was recorded. The STR inclusion has been most effective in rendering the shrinkage potential, which causes settlement of light structures along their outer perimeter causing differential movements. Finally, the results show that modification of expansive soils using STR might be an effective technique in enhancing the soil characteristics against swelling-shrinking potential of subsoils on which roads and light weight structures are constructed. Therefore, the scrap tire rubber waste has a potential for stabilizing expansive soils, thus giving a two-fold advantage in enhancing a problematic soil and also resolving the problem of disposal of waste by recycling it. On the other hand, the results of unconfined compression, flexural strength and California Bearing Ratio tests demonstrated that the amounts and particle size of STR could not improve the strength properties, and that further research is recommended using different sizes and percentages of the scrap tire rubber waste.

Keywords: Expansive soil, scrap tire rubber and stabilization.

ÖZ

Yarı-kurak iklimlerde, şişen zeminlerin yapılarda, çatlaklara ve farklı hareketlere neden olan önemli hasarlara neden olduğu iyi bilinmektedir. Ekonomik ve etkin iyileştirme metodu arayışı içerisinde, ikincil bir hedef olarak geri dönüşümün de sağlanması amaçlanan, atık kullanımına önemli bir ilgi artışı vardır. Bu araştırmada atık kauçuk tozu kullanılarak yerel bir şişen zeminin iyileştirilmesi çalışılmıştır. Zemin stabilizasyonunda atık kauçuk kullanımı, bu atık malzemenin çevreye verebileceği önemli kirlilik ve hasarın islahı için kolay bir geri dönüşüm yöntemi olabilecektir. Bu deneysel çalışmada şişen zeminin mekanik davranışı şişme-büzülme, kompresibilite, dayanım ve hidrolik özelliklerin irdelenmesi ile çalışılmıştır. Farklı kauçuk tozu yüzdelikleri (%10, %20, %30) karıştırılarak şişen zeminin değişen fiziksel ve mekanik davranışı çalışılmıştır. Deney bulguları %30 kauçuk tozunun plastisiteyi ve şişme potansiyelini önemli miktarda düşürdüğünü, ancak kompresibilitenin bu katkıdan etkilenmediğini göstermiştir. Ayrıca, kauçuk tozu katkısı büzülme potansiyelini düşürmede çok etkili olmuştur. Büzülme potansiyeli özellikle hafif yapıların çevresinde iklimsel etkenlerle zeminin büzülmesiyle temellerde oturmalara neden olur. Sonuç olarak, kauçuk tozu kullanımının hafif yapıların altındaki şişen-büzülen zeminlerin iyileştirilmesinde oldukça etkili olabileceği, aynı zamanda bu malzemenin geri dönüşümünün de sağlanmış olacağı görüşüne varılmıştır. Ancak, dayanım deneyleri olan serbest basınç, eğilme ve CBR sonuçları kullanılan malzemenin mukavemet açısından yeterli olmadığını göstermiştir. Dolayısıyla farklı boyutlarda kesilmiş kauçuk parçaları ve farklı miktarlarla daha detaylı bir araştırma yapılması tavsiye edilmektedir.

Anahtar kelimeler: Şişen zemin, stabilizasyon, atık kauçuk.

DEDICATION

This thesis is dedicated to

My parents

My darling wife

Both my kids, (Aryan & Waryan)

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LIST OF SYMBOLS AND ABBREVIATIONS

STR	Scrap Tire Rubber
LL	Liquid limit
PL	Plastic limit
PI	Plasticity Index
LS	Linear shrinkage
G_s	Specific gravity
ASTM	American Society for Testing and Materials
OMC	Optimum Moisture Content
MDD	Maximum dry density
AEV	Air Entry Value
Fs	Flexural Strength
CBR	California Bearing Ratio
SWCC	Soil Water Characteristics Curve
$\Delta H/H_0$	Swell potential
UCS	Unconfined compressive strength
S	Degree of saturation
CEC	Cation Exchange Capacity
w	Gravimetric water content
ψ	Suction
ψ_o	Total suction
ρ_d	Maximum dry unit weight
θ	Volumetric water content

Chapter 1

INTRODUCTION

1.1 Background

The concept of expansive soil can be utilized for describing soils which include a significant amount of variation in volume because of exchanging the amount of the soil water content (Nelson and Miller, 1992). Expansive soils are mainly observed in the arid and semi-arid zones of the earth planet where, the rate of evaporation is more than the rate of precipitation annually. Furthermore, these types of soils cause problems to buildings and other engineering constructions especially lightweight structures because of their ability to uplifting the structures during wet season and shrink during summer season (Lucian, 2006). The arid zones are more subjected to expansive soil because of their conditions is appropriate for the materialization of clayey minerals of the smectite group like montmorillonite or other sorts of illites. These clays were known with their specific properties such as a very small particle size, a great specific surface area and a high level of cation exchange capacity (CEC) (Avsar, et al., 2009).

Annually, many damages due to swelling behavior of expansive soil have been noted obviously in the semi-arid zones which they are seen in the shape of cracking and crumbling of pavements, foundation of structures, highways, and channels. In addition, it causes damage in the irrigation systems such as water lines, reservoir linings and sewer lines (Cokca, 2001).

Nowadays, there is an international concentration in expansive clays and shales, engineers from different countries in the world have joined and started to share information to deal with this soil problematic and they try to share knowledge to produce the best design for structure on expansive soil especially in the developed countries such as Canada, Australia, South Africa, Israel and the United States. The first substantial international conference about expansive clay was one held at the Colorada Institute in Golden, Colorado in 1959. And the second conference on expansive clay were held at Texas A & M University in 1965 and 1969, the third one was held in Haifa, Israel, in 1973 (Chen, 1975). Later on fourth, fifth, sixth and seventh conferences were held in Denver (USA), Adelaide (Australia), New Delhi (India) and Dallas (USA) respectively.

One of the difficulties which civil engineers face is constructing of light structures on expansive soils. Therefore they are obliged to think about the most suitable foundation type for buildings and other structures. Another alternative for mitigating this problem is stabilizing soil by using most economical and effective ways by adding some none swell materials admixtures that modify volume change and other soil characteristics.

Soil stabilization is a geotechnical process aimed to improve one or more mechanical characteristics so as to improve engineering properties. Stabilization process has benefits of decreasing plasticity, lowering or sometimes increasing permeability, hence resulting in , higher soil strength, lower volume change due to temperature or moisture variations, increased workability of soil, thus lower thickness of pavement when used in road stabilization (Sachin et al., 2014). On the other hand, stabilization has a crucial role to reduce the harmful wastes accumulating in the environment.

Recently, many types of waste are investigated to be used as additive materials for stabilizing expansive soils, such as industrial wastes, marble dust, some plastic materials, fly ash and scrap tire rubber. There are many methods used to stabilize expansive soils, but generally they can be separated to two main groups: mechanical (physical) and chemical stabilization. The mechanical method includes replacement with non-expansive fill, compaction, soil reinforcement, addition of aggregates and mechanical remediation. Moreover, the most widespread physical stabilization methods are rewetting, removal and replacement (Nelson and Miller, 1992). In addition, the chemical stabilization enhances geotechnical properties of clay soils, by addition of some different materials such as fly ash, quick lime, Portland cement, bitumen, calcium chloride, chemical or bio-remediation, marble dust and scrap tire rubber (Barazesh et al., 2012).

This study concentrated on using scrap tire rubber (STR) powder as a stabilizer for improving the locally available expansive clay. The experimental program investigated on the physical and mechanical characteristics of the expansive soil, including swell-shrinkage potential, hydraulic properties, strength characteristics, and California Bearing Ratio (CBR). As Using the scrap tire rubber in soil stabilization can be a feasible solution for recycling this waste material, hence reducing the environmental problems, as discarded waste tires impose a major environmental pollution to the environment. Scrap tires stockpiled are found in many countries which endanger public health, as well as resulting in environmental problems including landscape deterioration. Moreover, available of huge amount of thrown scrap tire in landfill increases the risk of fires which poses danger to human health. In addition, the disposed waste tires, when mixes with water and organic materials, creates a favorable environment for mosquitoes to breed which may cause a very hazardous disease such as encephalitis (Guleria1 and Dutta, 2011).

More than 250 million scrap tires are manufactured yearly in the USA at the moment, Japan's unwanted scrap tire production in 2008 was nearly 106 million tires and in Korea the amount of used scrap tire production reached to almost 30 million tires in 2008 (Cosentino, et al, 2014; Kim, 2011). Huge number of produced scrap tires in the world create problems for all countries at different levels. In some developed countries, this issue has been studied and several alternatives were proposed to reuse the scrap tires as beneficial materials in various fields. For example, in the USA around 4.6 million tons of scrap tires was manufactured in 2007, in the same year, almost 89% of the manufactured scrap tire was passed to end-use markets. Nearly 12% of the scrap tire rubber manufactured in the USA is constructively used in civil engineering projects (Rubber Manufacturers Association, 2009). Furthermore, in regions like Colorado during the year 2008, approximately 5.7 million scrap tire facilities have been reported and during the same period, about 79% of the total amount of scrap tire was recycled and reused. However, the 21% of waste tire was stored in the storages. The 6.5% of the waste was used as crumb rubber for engineering applications (Snapp, 2009). The main advantages of waste tire rubber in civil engineering applications is not only due to environmental issues, but also due to the light weight of the material, which makes it favorable backfill material for retaining walls and embankment fills, as well as a stabilizing material of expansive soils, as documented in the present research.

1.2 Objective of the Thesis

The core objective of this investigation was to evaluate the effects of using scrap tire rubber material for stabilizing expansive soil and to observe the variation of the physical and mechanical soil characteristics after mixing with different percentages of tire rubber powder in order to conclude on an optimum percentage. The main emphasis in the experimental program however is given to the mitigation of swelling-shrinking potential of expansive soil to be lightly loaded.

1.3 Outline of the Thesis

This thesis consists of five chapters. Chapter 1 contains the background and objectives of the study. Chapter 2 represents a literature review of the expansive soils and stabilization of expansive soils with different additives specifically scraps tire rubber stabilization. Chapter 3 provides information on materials used and laboratory methodologies implemented on soil and the soil and scrap tire powder mixtures, to determine the material properties. Chapter 4 presents the results and discussions of the experimental study. Finally, Chapter 5 includes conclusions of the experimental findings and recommendations for future work.

Chapter 2

LITERATURE REVIEW

2.1 Review of Expansive Soil

Expansive soils are well-known for their composition containing the clay mineral montmorillonite, which are included in residual and sedimentary soils, such as shales and clay-stones. Expansive soils are mainly observed in arid and semiarid zones subjected to large climate change in different seasons. They are naturally located near the ground surface, hence affected by the temperature and environmental changes (Fredlund and Rahardjo, 1993).

There are many studies conducted to illustrate the behavior of expansive soil and many correlations have been found that are beneficial in categorizing potential swell of expansive soils. It can be possible to classify expansive soils by naked eye especially, by the experts in geotechnical engineering. They can observe the expansive soils visually by the following: (1) during the dry seasons, the shrinkage cracks can be observed easily, (2) the soil seems rock-hard when it dries, but it is very adhesive and soft when it gets wet, (3) always there are damages on the nearby buildings because of expansive soil (Wayne, 1984).

2.2 General Review of Clay Minerals

Clay minerals commonly have some particular characterstics such as (1) small particle size, (2) unit cells with a negative electrical charge balanced by taking cation from solution, (3) posses plasticity when mixed with water and (4) relatively high weathering resistance (Mitchell and Soga, 2005). The clay minerals which have high volume change are in the phyllosilicate group. The basic unit structure of phyllosilicates group is plate like structure and is consisted of two types of horizontal sheets. One of them is overcomed by silica tetrahedron block and the second one is controlled by aluminum and magnesium octahedron block. In silica tetrahedron block one ion of silicon atom is tetrahedrally surrounded by four oxgyen atoms. In the aliminum or magnesium octahedron block ions octahedrally are surrounded by six oxygen atoms or hydroxyl groups. Large number of octahedra joined together horizontally include the octahedral sheet. When clay minerals consist of trivalent cation or only aluminum, it is termed dioctahedral or gibbsite [Al 2(OH) 6]. When it contains magnesium or cation is divalent, then the structure is called trioctahedral or burcite [Mg3 (OH) 6] (Snethen et al., 1975).

Figure 2.1 and Figure 2.2 illustrate a silica sheet and silica tetrahedron, likewise an octahedron sheet and octahedron respectively. Furthermore, these figures represent the diagram of silica and octahedron sheets (Oweis and Khera, 1998).

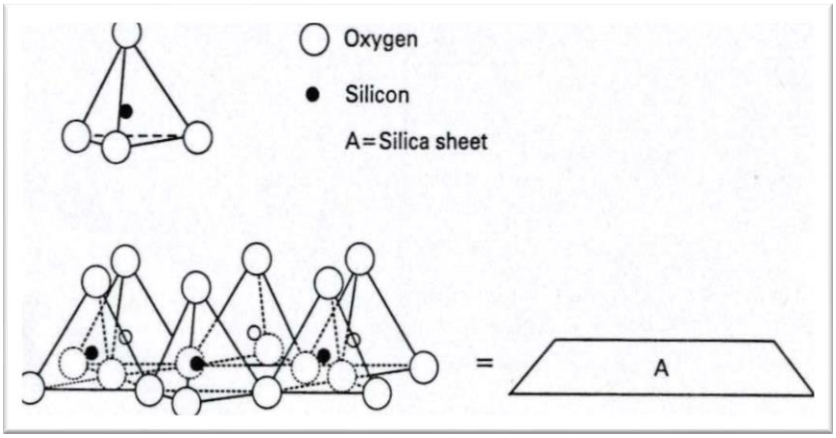


Figure 2.1: A silica tetrahedron and a silica sheet (Oweis and Khera, 1998).

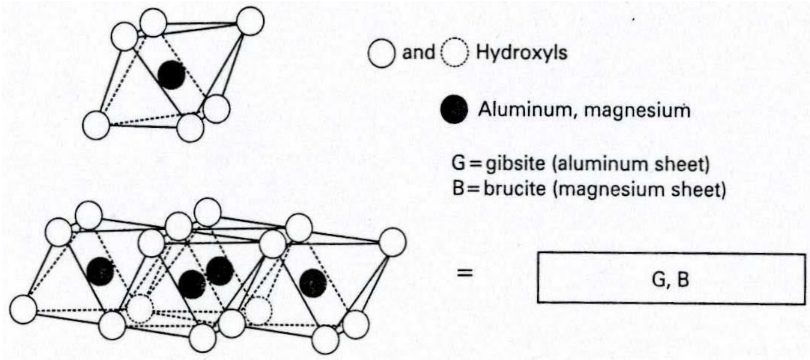


Figure 2.2: An octaheron and an octaheron sheet (Oweis and Khera, 1998).

Snethen et al., (1975) categorized the clay minerals to three main groups. (1) Two-layer clays which contain one silica tetrahedral layer jointed with one layer of octahedral aluminum. The best mineral as an example for this group is kaolinite, which aluminum is the main octahedral layer in its structure. (2) Three-layer clays, the mineral structure of this group consists of one octahedral layer between two other tetrahedral layers. The common minerals of this group are vermiculite, illite and montmorillonite. (3) Mixed-layer clays, produced as a result of combination and interstratification of two- and three-layer clay minerals as mentioned formerly. This combination may occur arbitrarily or systematically. The chlorite and montmorillonite-chlorite mineral are the examples for regular combination.

In categorization of clay minerals, three main principles can be considered which they are the dimensions of the layers, configuration of the crystal layers whether dioctahedral or trioctahedral and how many ions are available in the layers and the stacking arrangement of the layers and composition structure of their crystals.

Generally, the clay minerals are organized for three significant structural groups according to engineering applications as follows:

- The 1:1 Minerals or Kaolinite group – are the non-expansive mineral.

- Mica-like group - comprises illites and vermiculites, which are partially respect to swelling.
- Smectite group - comprises montmorillonites, which are highly expansive clay minerals (Nelson and Miller, 1992).

2.3 The 1:1 Minerals (Kaolinite Group)

The clay mineral kaolinite has a negligible interlayer swelling due to lack of exchangeable cations. Moreover, in the near octahedral and tetrahedral layers, the opposing electrical charge jointed the individual structure of double layers very strongly together. Therefore, volume change observed in this mineral is mostly because of water absorption on the edge of individual grains and there is no space for absorption of water or extraction of cations between the layers (Snethen et al, 1975). Therefore the kaolinite minerals can be considered as non-expansive clay minerals. Kaolinite crystals include the tetrahedron and octahedron sheets. Van der Waals forces and hydrogen bonds are stuck the layers together and the bonds are appropriately strong that it does not allowed each inter layers to swell in case it has water (Mitchell and Soga, 2005).

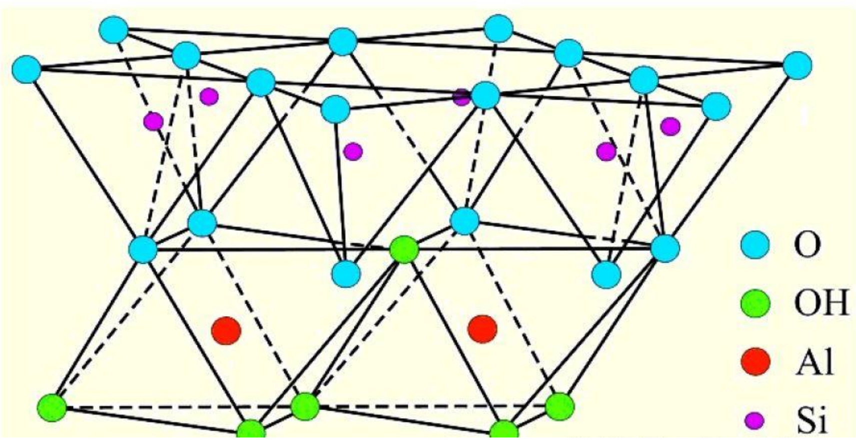


Figure 2.3: Diagrammatic skectch of the kaolinite structure (after USGS, 2001) (Mitchell and Soga, 2005).

Generally, the mineral particles in kaolinite group comprise of the basic units combined in the C direction. The kaolinite group includes different minerals such as kaolinite, serpentine, and halloysite. The main structural formula is (OH8 Si4 Al4 O10), and the structure diagram of kaolinite is displayed in Figure 2.4.

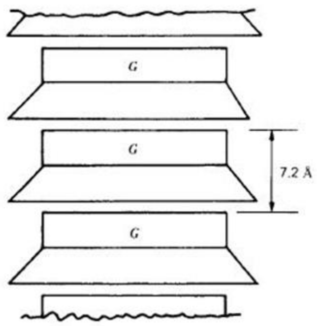


Figure 2.4: Schematic diagrams of the structures of kaolinite
(Mitchell and Soga, 2005).

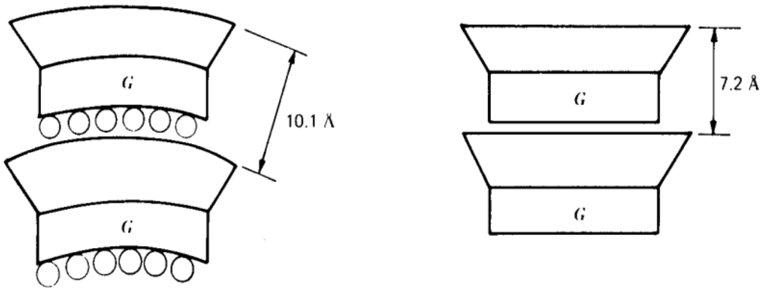
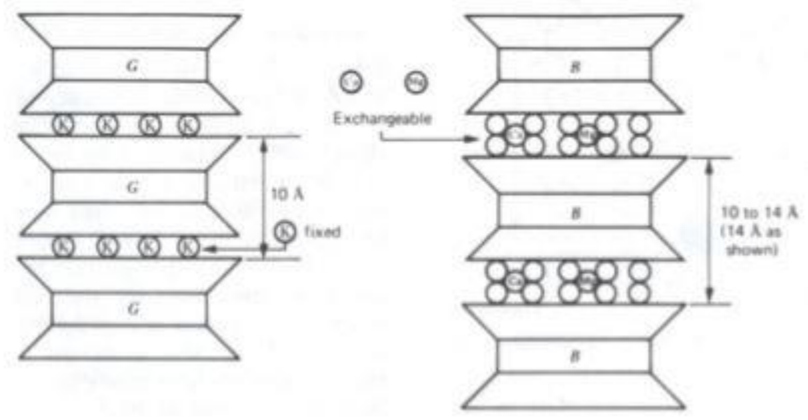


Figure 2.5: Schematic diagrams of halloysite structure (Mitchell and Soga, 2005).

Illite is the most abundant clay mineral found in clay in geotechnical engineering applications. There is a big similarity between the structure of illite and muscovite mica. The essential structure of illite consists of a sheet of gibbsite (aluminum) between double tetrahedron silica sheets. In the sheet of silica the aluminum is partially exchanged with silicon. There is a rather weak connection between jointed sheets because of non-exchangeable ions of potassium among them (Craig, 2004). The structures of different minerals of mica-like group are displayed in Figure 2.6.



(a) (b)

Figure 2.6: The structure diagram of mica-like minerals: (a) muscovite and illite and (*b*) vermiculite (Mitchell and Soga, 2005).

Montmorillonite and saponite are the most common minerals of smectite group. The structural composition of the smectite group minerals has a prototype structure which has the same as pyrophyllite structure, comprising of an octahedral sheet inserted in between two silica sheets, as displayed in the Figure 2.7 and in Figure 2.8 three dimensional composition structure is shown. The bonding system of this mineral group is van der Waals forces and by cations which may exist to equilibrate charge deficiencies in the structures. These bonds are very weak and simply separated because of cleavage or adsorption of water (Mitchell and Soga, 2005).

Montmorillonite is recognized as a dioctahedral clay mineral of smectite group which causes a partial replacement of aluminum by magnesium in the octahedral layer. According to some categorization approaches, the term “montmorillonite” is used as the name of three-layer clay minerals which consist of dioctahedral and trioctahedral (Patrick and Snethen, 1976).Montmorillonite’s structure is similar to that of illite. Aluminum is substituted partially by iron and magnesium in its gibbsite sheet, and also silicon is partially substituted by aluminum in the silica sheet.

Substantial amount of swelling is observed in the montmorillonite because of extra water being adsorbed in between combined sheets (Craig, 2004).

Montmorillonite and other minerals can be detected by various methods, the best being the X-ray diffraction (XRD). The existence of montmorillonite in the expansive (sediments or transport soils) is a function of (1) the processing of weathering and original material in the source region; (2) from the process of transportation to and inside of bowl of deposition; (3) volcanism; (4) digenesis; and (5) the impacts of tectonic and metamorphic processes on the sedimentation area (Patrick and Snethen, 1976).

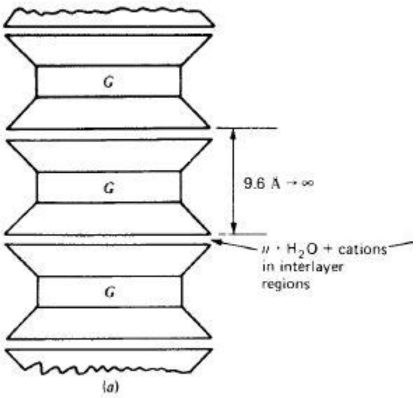


Figure 2.7: Schematic diagrams of the structures of the smectite minerals: montmorillonite (Mitchell and Soga, 2005).

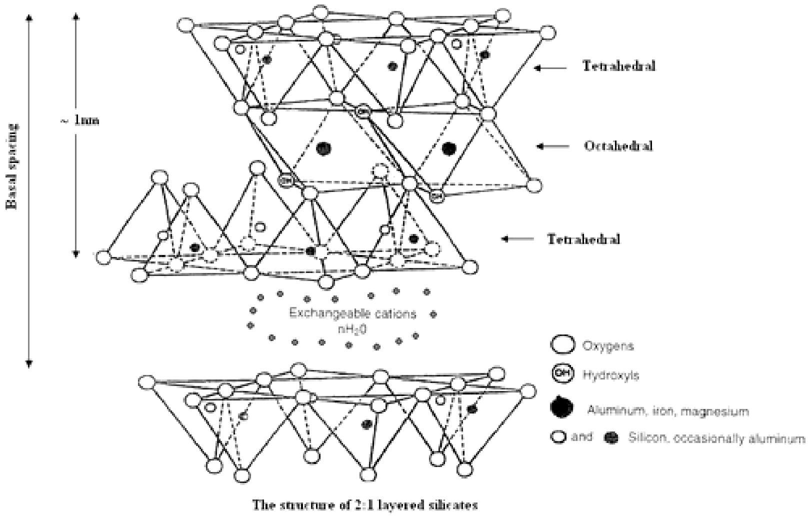


Figure 2.8: Diagrammatic sketch of the montmorillonite structure (Mitchell and Soga, 2005).

2.4 Mechanism of Swelling

Swelling is an exchange in the physical properties of clay minerals; it refers to increasing in the particles volume after absorbing water. Swelling in the clay minerals is mainly associated with cation exchange capacity of the minerals and diffused double layer. The mechanism of swelling in expansive clay minerals is intricate and effected by several features. Swelling is a variation in the particle volume and that is because of exchanging water system in the soil which disturbs the interior stress balance. There are other reasons impacting the swelling mechanism such as physical characteristics of soils including plasticity and density.

2.4.1 Cation Exchange Capacity (CEC)

The amount of exchangeable cations required for equilibrium of the negative charge deficiency on the clay particle surface is termed the cation exchange capacity (CEC) and it is usually indicated as milli-equivalents per 100 grams of dry clay. CEC can be determined either experimentally in the laboratory or theoretically if the composition of mineral is known (Nelson and Miller, 1992). The measure of CEC of some minerals is displayed in Table 2.1.

Cations which defuse the negative charge on the particle surface of soil in water are freely replaceable with cations from other molecules. The interchange reaction can be influenced by the concentration percentage of cation in the water and likewise it depends on the cations electrovalence (Terzaghi et al, 1996). The process of exchanging silicon to aluminum in the structure of clay minerals is known as isomorphous substitution, and the result is negatively charged surface of clay particles. The negative charge on the clay particles attracts positive ions which are known cations. This is more beneficial since it permits us to change soil structure properties chemically by alerting the cations which are held on the clay surface. The most popular soil cations are: hydrogen (H^+), sodium (Na^+), (calcium (Ca^{++}), magnesium (Mg^{++}), potassium (K^+), and ammonium (NH_4^+) (Mitchell and Soga, 2005).

Table 2.1: CEC of some clay minerals (meq/100g).

Clay Minerals	Cation Exchange Capacities (meq/100g)
Smectities	80-150
Vermiculites	120-200
Illites	10-40
Kaolinite	1-10
Chlorite	<10

Table 2.2: CEC of principle clay minerals (modified from Terzaghi et al., 1996).

Mineral	CEC (meq/100g)
Kaolinite	3 – 10
Illite	20 – 30
Montmorillonite	80 – 120

2.4.2 Diffuse Double Layer of Clay Minerals

The clay particles surface has been charged negatively because of isomorphous substitution process, which attracts positive ions in the solution. The zone between negatively charged clay surface particles and the attracted positive ions in any solution is described as "diffuse double layer" (DDL). The occurrence and durability of the DDL is dependent on: (1) the existence of the negatively charged clay particles and, (2) the availability of cations or counter-ions in the solution of soil which keep the equilibrium of the negative charge (Mitchell and Soga, 2005).

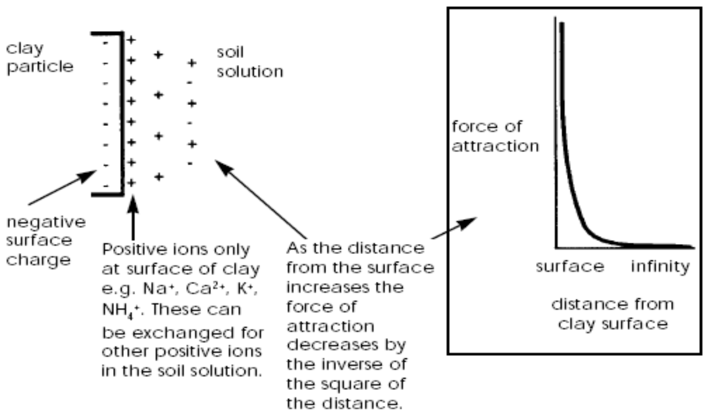


Figure 2.9: The diffuse double layer on the surface of clay particles (Mitchell and Soga, 2005).

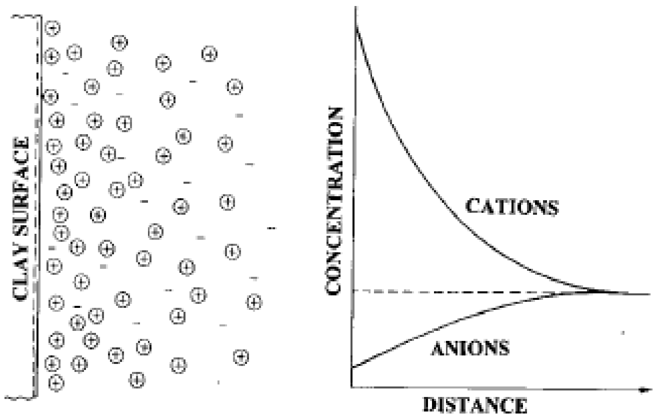


Figure 2.10: The Scattering of cation and anion adjacent to a clay particle (Mitchell and Soga, 2005).

Clay particle surface has negative charges, therefore when the clay is mixed with a solution; cations are attracted from the solution by clay surface to sustain electrical neutrality. As a result of attraction of cations to clay surface, the cation concentration will be bigger on the particle surface than bulk solution. As a consequence of reduction in cation concentration, cations will try to diffuse away from the surface of clay particles to the direction of solution.

2.5 Review of Expansive Soil Classification and Identification

Expansive soil classification systems are developed according to the problems which they impose on the foundations of structures (potential swell). There are various classification schemes, but the categorization system developed by the U.S Army Waterways Experiment Station is one of the one most extensively used in the United States. It has been preceded by O'Neill and Poormoayed (1980) and is presented in Table 2.3 (Das, 2011).

Table 2.3: Expansive soil classification system based on index tests (Das, 2011).

Liquid Limit	Plasticity Index	Potential Swell (%)	Potential Swell Classification
< 50	<25	<0.5	Low
50 – 60	25 - 35	0.5 – 1.5	Marginal
> 60	> 35	> 1.5	High

Potential swell = vertical swell under a pressure equal to overburden pressure

According to the classification system proposed by Chen (1975), the potential swell of expansive soil can be categorized by three varies approaches. The first approach is the mineralogical evaluation, which is a beneficial method for identification of the materials. However, it is not adequate when studying the natural soil. The second approach is known as indirect method. It is related to the evaluation of some physical properties of soil such as consistency indices and activity. The third approach is direct measurement, which provides useful data for engineering considerations (Chen, 1975).

Since the expansive soil has become a serious problem for engineering projects, the studying of expansive soil has dramatically increased and a huge number of researches have been published about the expansive soil worldwide annually. Among these publications, there are a lot of methods proposed for identifying and classifying the expansive soil and the swelling potential by researchers. The common criteria used to identify the expansive soils are documented in this study are as below.

(1) Atterberg limits; Holtz and Gibbs (1956) illustrated that plasticity index is the best parameter for finding the swelling behavior of all types of clays. Furthermore, it has been confirmed that the only test which can be used as initial indication for swelling behavior of most clays is the plasticity index. The effects of plasticity index of clays on the swelling potential were displayed in the Tables 2.4 and 2.5.

Table 2.4: Swelling potential and plasticity index relationships (Holtz and Gibbs, 1956).

Swelling potential	Plasticity index
Low	0 – 15
Medium	10 – 35
High	20 – 55
Very high	35 and above

Table 2.5: Expansive soil classification based on plasticity index (Holtz and Gibbs, 1956).

Degree of Expansion	Holtz and Gibbs	Chen	IS 1498
Low	<20	0 – 15	<12
Medium	12 – 34	10 –35	12 -23
High	23 – 45	20 – 55	23 - 32
Very high	>32	>35	>32

(2) Linear shrinkage is another criterion which can be used for evaluating the swelling potential and degree of expansion of soils. According to the theory it seems that the shrinkage properties of clay can be used as a reliable index to measure the swelling potential. It was Altmeyer (1955) first who suggested that it can be possible to evaluate the swelling potential according to different values of shrinkage limits and linear shrinkage as illustrated in Table 2.6 (Chen, 1975).

Table 2.6: Relationship between shrinkage limit, linear shrinkage and degree of expansion (Chen, 1975).

Shrinkage Limit (%)	Linear Shrinkage (%)	Degree of Expansion
<10	> 8	Critical
10 -12	5 – 8	Marginal
>12	0 – 5	Non-critical

(3) Swell potential is another well-known criterion for identification of expansive soil, this method commonly known as Seed, Woodward, and Lundgren method (Woodward, and Lundgren, 1962), the potential swell of a soil sample is recognized from correlations between swell percentages from oedometer test in laboratory and compacted samples with optimum moisture content and maximum dry density. The potential swell is classified as in Table 2.7.

Table 2.7: Relationship between swell potential and degree of expansion (Woodward, and Lundgren, 1962)

Swell Potential (%)	Degree of Expansion
0 – 1.5	Low
1.5 – 5	Medium
5 – 25	High
>25	Very high

(4) Sidharan, and Prakash (2000) have documented some parameters together that illustrate the classification and identification of expansive soil. They got benefit from early works which have been done about categorization of expansive soil according to different characteristics of soil. The common criteria are displayed in Tables 2.8-2.9.

Table 2.8: Soil expansion prediction by other measures (Sidharan, and Prakash, 2000).

Degree of Expansion	Colloid content[10] %minus 0.001mm	Shrinkage Limit[10] (%)	Shrinkage Index[4] (%)	Free Swell	Percent Expansion In odometer As per Holtz and Gibbs	Percent Expansion In odometer As per Seed et al
Low	<17	>13	<15	<50	<10	0 -15
Medium	12 – 27	8 -18	15-30	50-100	10- 20	1.5 -5.0
High	18 – 37	6 -12	30-60	100-200	20 -30	5 -25
Very high	>27	<10	>60	>200	>30	>25

Table 2.9: Proposed expansive soil classification (Das, 2011).

Odometer % Expansion	Free Swell Ratio	Clay type	Soil Expansivity
<1	$\leq$ 1.0	Non-Swelling	Negligible
1 – 5	1.0 – 1.50	Mixture of Swelling And Non-Swelling	Low
5 - 15	1.50 – 2.0	Swelling	Moderate
15 -25	2.0 – 4.0	Swelling	High
> 25	> 4.0	Swelling	Very High

(5) USBR Classification System; United State Bureau Reclamation suggested a categorization system for expansive soils. This classification founded on the colloidal contends less than 2ɱm, plasticity index, and SL shrinkage limit and the rate of expansion or swell percent amount. The USBR system is indicated in Table 2.10.

Table 2.10: USBR classification method (Snethen, 1977).

Colloid Content %- 1 µm	PI (%)	SL (%)	Probable Expansion %	Expansion
<15	<18	<15	<10	Low
13 – 23	15 – 28	10 – 16	10-20	Medium
20 – 31	25 – 41	7 – 12	20-30	High
> 28	> 35	<11	>0	Very high

2.6 Damage of Structures on Expansive Soils

Expansive soils are defined as a soil type which has potential to significant volume change during the dry and wet seasons. During wet season the clay particles of expansive soil absorb water and they start to swell by some mechanical and chemical reactions between the minerals in the clay particles. Also during dry season inversely the water content evaporates and the soil shrinks causing cracks in the soil structure close to the ground surface. Shrinking of expansive soil is presented in Figure 2.11.

Figure 2.10: Mechanism of shrinking expansive soils.

According to the geotechnical engineering evaluations, mostly the lightweight structures undergo the damages due to expansive soils. The mechanism of the structural damaging by the expansive soil starts after a structure is constructed on the expansive soil surface which is subjected to climatic influences. As a result, the soil starts to move under the structure and this soil movement is recognized according to (a) short-term effects and (b) long-term effects. If the structure is built in the dry season, and the wet season follows, the expansive soil surrounding the building take the rainwater and swells after swelling it starts to up lifting the foundation and this effect is known as the edge heave. In contrast, if it is constructed in wet season followed by dry period, it causes the expansive soil to shrink at the edge and the foundations settles. This phenomenon is known as the edge-shrink effect (Figure 2.12). Additionally, in the long term effect, the moisture content continuously gathers under the center of foundation by capillary action within a long period until it will be in equilibrium. It means the soil under the center of foundation swells with time whereas; at the foundation's edges the moisture variation continues to happen. This is called center-heave condition as shown in Figure 2.13 (Vankataramana, 2003).

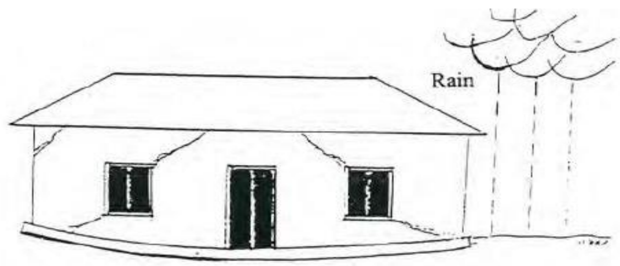


Figure 2.11: Edge heave condition (Vankataramana, 2003).

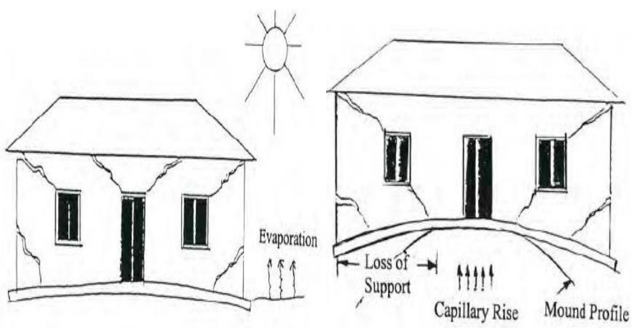


Figure 2.12: Edge shrink and center heave conditions (Vankataramana, 2003).

The damages of expansive soil appear initially as cracks on the internal and external building walls, particularly at the weakest parts of the structures. All types of cracks are shown in the Figure 2.14. The size of cracks due to expansive soil growth with time up to reach the maximum damage of failure (Elarabi, 2000).

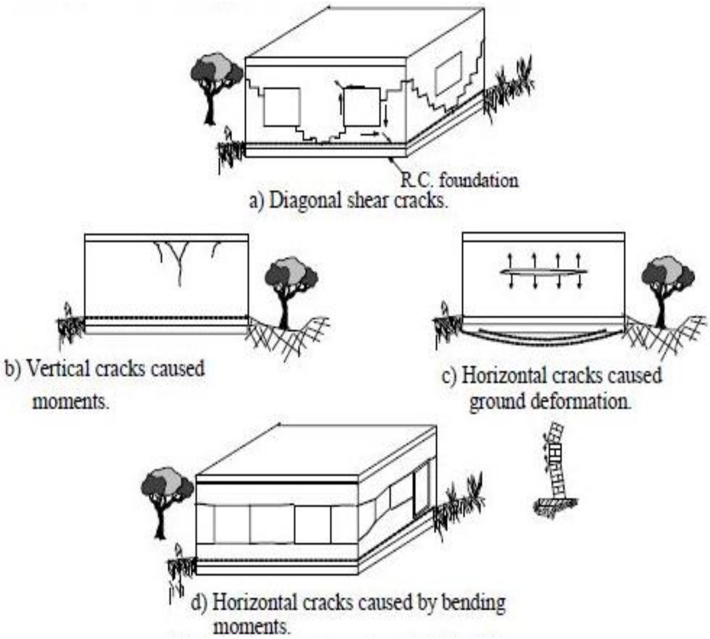


Figure 2.13: Common types of cracks in buildings with shallow foundations (Hussein Elarabi, 2000).

2.7 Stabilization of Expansive Soils

Soil stabilization is the geotechnical process which is implemented in order to improve the one or more mechanical and chemical soil characteristics; so as it becomes applicable to engineering applications. Additionally, soil stabilization is an engineering technique in which the soil can be mixed with an appropriate amount of different stabilizers to eliminate volume changes, and increases some other physical and chemical characteristics of soils. Using of stabilizers is not a modern technique; actually there are many additive materials which have been used in the past for stabilizing the expansive soil such as plant saps, animal dung, natural oils, and crushed anthills (Khandaker and Hossain, 2011).

2.7.1 Mechanical (Physical) Stabilization of Expansive Soils

Mechanical methods are used when the present soil zone is a moderately expansive and near the surface. The mechanical method can be practiced by two different techniques; the first one is removing and replacing the expansive soil and then compacted according to engineering standards. The second technique of mechanical stabilization is applied by changing the nature of the soil, which is implemented by compaction or rewetting.

2.7.2 Chemical Stabilization of Expansive Soils

The chemical stabilization is the oldest and most common technique used to improve the expansive clays. This method participates in enhancement of geotechnical soil properties, such as strength of soil which depends on interaction between clay particles. The improvement can be performed at macroscopic and microscopic levels. At microscopic level the conflict of interaction between particles is improved by clay stabilization methods. Moreover, at the macroscopic level, the interaction among particles is reinforced by using some convinced equipment and this process usually known as reinforcement of soil. Chemical stabilization has a significant role in development of expansive clay and it strengthens soils by modifying the clay structure, producing a new material with different characteristics (Arash, et al, 2012).

Nowadays, the most popular method of stabilization is chemical improvement, and it consists of mixing the soil with one or a group of additives in the form of powder, slurry or liquid for enhancing or overcoming the clay swelling, improving the strength, permeability, and other physical or chemical characteristics of expansive clays. The most commonly used stabilizers are lime, fly ash, cement, marble dust, silica fume and waste products such as tires rubber powder. Currently many researchers have being reviewing the effects of natural and fabricated materials as a stabilizer to modify expansive clays. The common additive materials which have been used for improving expansive clays are documented in the following sections.

2.8 Lime Stabilization of Expansive Soils

Lime stabilization is one of the oldest techniques which is used for stabilizing the expansive clays. Lime when added to fine-grained soil with existing water, starts some reactions with soil particles, the two initial reactions being the cation exchange capacity and flocculation-agglomeration, which occur immediately and result in rapid developments in some soil characteristics (Mallela et al., 2004). Al-Rawasa et al. (2005) studied the stabilization of expansive soil using lime, cement, combinations of both. They mixed soil with different percentages of lime and cement. At the end of all experiments they observed that by mixing 6% lime, both the swell percent and swell pressure decreased dramatically to zero. Abass (2013) used hydrated lime for improving heavy clays (natural soil + bentonite). The results of the research showed that the addition of lime is effective in reducing the swell potential of expansive soil. Al-Mukhtar et al. (2010) conducted several laboratory tests on highly expansive soil improved with lime so as to deal with the lime-soil reaction during a short time and the process of pozzolanic reaction during a long time. The test results illustrated that 5% lime is adequate for short time reaction involved with cation exchange in the soil. By increasing the pozzolanic reaction causes production of new minerals, as well as calcium aluminate hydrates (CAH). Finally the results of this study obviously show that soil characteristics continue to be enhanced with lime addition.

2.9 Stabilization of Expansive Soils using Fly Ash

Fly ash is an industrial waste which is produced because of some chemical reactions in thermal power plant projects. Using fly-ash is more useful and very essential in terms of recycling this waste which can be hazardous for the environment. Hakari and Puranik (2012) carried out a study about the influence of fly ash on stabilization of black cotton soils. The physical and chemical characteristics of the soils were studied such as Atterbergs limits, compaction characteristics, unconfined compression strength, and CBR test which they effected by adding flay ash. Three different soil samples were chosen which they have different properties, the fly ash mixed with soils in different percentages which includes 0, 10, 20, 30, 40, 50, and 60% by weight and they observed that adding fly ash percentage eliminates the Liquid limit with adding fly ash up to 30% and then decreasing of liquid limit was noted as marginal for more adding fly ash. The plastic limit of the soils starts to decline steady with rising the fly ash percentage. Similarly, the plasticity index decreased after mixed the soil with different fly ash contents up to 30% and then the reducing trend negligible with further adding fly ash. Shrinkage limits other properties which it was affected with mixing fly ash with soils, from the results it can be observed that increasing the fly ash content due to enhanced steady of the shrinkage limit, increasing shrinkage limit may be occur because of clay particles flocculation due to free lime which available in the fly ash. The effects of fly ash content on the compaction characteristics can be concluded as below from the results, for all soil samples the maximum dry density has been improved with increasing Fly ash content up to 30-40%, then it starts to reduce with further adding fly ash. However, the optimum moisture content for all samples was decreased with increasing the fly as percentages. Impacts of fly ash on Strength characteristics can be explained according to unconfined compression strength data, for the results can be seen that the value of unconfined test growths with increasing the Fly ash content for all soil samples, at 28 days curing time the unconfined value is greater than the value in 7 and 14 days curing, and the optimum fly ash contents are 10 and 20%, because in larger percentages of fly ash, the increasing of strength is not considerable. Similarly to compaction results, the California bearing ratio value increased with adding Fly ash content up to 30-40%, and after these percentages, it starts to eliminate with further increasing of fly ash percentages.

Yashwantsinh (2013) conducted the same study about stabilization of expansive clay using fly ash material. They conducted some essential tests to deal with the influence of fly ash to mitigate the expansive clay. In the research, highly expansive soil mixed with different percentages of fly ash (15%, 20%, and 30%). According to the test results, the unconfined compression stress of expansive clay rose from 114 kN/m^2 to 123 kN/m^2 after added 20% fly ash. And the liquid limit and plastic limit were reduces from 74.4% to 72.5%, 38.4% to 32.93% respectively with increases of fly ash up to 30%. Moreover, the results depict that maximum dry density at 14% optimum moisture content, was growth from 1.68 g/cm^3 to 1.71 g/cm^3, after increasing percentage of fly ash to 30%. Das and Parhi (2013) chose an activated fly ash at different alkali concentration for improving the local expansive soil properties. During their research, they have done some tests to observe the effects of fly ash in treatment of physical and chemical characteristics of expansive clay, like Atterberg limits, free swell index, and proctor compaction test. At the result of their experimental study the following conclusions were drawn. An increase in percentage of Alkali Activated Fly ash (AAFA) decreases the plasticity and liquid limit of expansive clay. And the plastic limit and optimum moisture content goes up, with adding the AAFA content. In contrast, the maximum dry density was decrease, with increasing AAFA percentage. Furthermore, the addition of AAFA causes of dramatically decreases in the free swell index (FSI) of expansive clay. In the presence experimental study the FSI of used expansive soil was dropped considerable from 66.67% to 23.07 by adding AAFA up to 15%.

2.10 Burnt Brick and Marble Dust Stabilization of Expansive Soils

Bhavsar et al. (2013) have done an experimental research in India to determine the potential of burnt brick dust as additive material for improving behavior of expansive clays. The experiments were done to evaluate the influence of using burnt brick dust to enhance the expansive soil characteristics include linear shrinkage, swelling potential, Atterberg's limits, and compaction test. The expansive soil was mixed with different percentages of burnt brick dust which consists 30, 40, and 50%. The results display the substantial decrease in swelling with adding burnt brick dust (BBD) up to 50%. The Atterberg's limits and linear shrinkage were decrease by increasing the BBD content. An increasing in BBD content causes reduction in the optimum moisture content however, maximum dry density improved with increasing the BBD percentage. Singh and Yadav (2014) have investigated to improve expansive clay, but they used different additive material and the main objective of their research was evaluating the feasibility of marble dust as a clay stabilization material. In this experimental work, natural expansive soil were mixed with marble dust in different percentages like (0, 10, 20, 30, and 40%); the test results revealed that marble dust has a significant potential influence to enhance the expansive soil properties. The analysis of results indicated that liquid limit and plasticity index were observed to reduce from 57.67% to 33.90% and 28.35% to 16.67% respectively, when decreasing the marble dust content. The shrinkage limit of the black cotton has increased from

8.06% to 18.39% by adding 40% of marble dust. Adding the marble dust content causes the appreciable reduction in the value of differential free swell from 66.6% to 20%.

2.11 Stabilization of Expansive Soils using different Additive Materials

Fattah et al. (2010) in the Kurdistan region of Iraq have conducted an experimental work for improving the expansive soil at Hamamuk earth dam by using four different additives; gasoline fuel, steel fibers, cement, and injection by cement grout. The natural expansive soil was mixed with 5% of cement, steel fiber, and cement grout, and with 4% gasoline fuel. The friction angle, adhesion, and cohesion between the clay particles were studied after adding the additive materials. The test results provided these conclusions. The compression index (C_c) and rebound index (C_r) were reduced with adding the additive materials to the expansive soil. The friction angle between clay particles was not influenced with the improvement due to the surface area of cement being larger than soil. Since the adhesion between soil particles and additive materials were changed, the cohesion between particles was slightly influenced by adding the stabilizer materials to the expansive soil. Abbawi, (2013) used silica-fume as additive material to stabilize the expansive soil. The natural expansive soil which was chosen for this experimental research was categorized as (CH) with liquid limit 51% and plastic limit 27%. Natural expansive soil samples were mixed with different silica-fume contents (10%, 20%, 25%, 30%, and 50%) to observe the influences on geotechnical characteristics such as compaction, Atterberg's limits, unconfined compression, and swelling behavior. The effects of silica fume on stabilizing expansive soil obviously can be perceived from the experimental results. The vertical swelling and compressibility properties were decrease with increasing the silica fume content. Maximum dry unit weight and liquid limit of the soil specimens have decreased by adding silica fume. The optimum moisture content of the soil rose with increasing silica fume content. The unconfined compressive strength changed from 83.9 kPa to 144.6 kPa in the treated soil sample. An increase in the silica fume content up to 50% caused reduction in the compression index and swell index in stabilized samples. Gandhi (2013), mitigated the geotechnical properties of expansive soil by using two industrial waste products, rice husk ash and marble dust. Expansive soil samples were mixed with two different additives rice husk and marble dust with percentages of 0%, 10%, 20%, and 30% for both of them. And several geotechnical experiments carried out to determine the effects of both additive materials in enhancement of expansive soil. The results of this experimental research drew the follow conclusion. Liquid limit of the soil reduced about 30% with adding 20% of marble dust while in the same portion of adding rice husk it decreased almost 26%. By using both of them the plastic limit reduced in the near amount from each other. A decrease in the shrinkage limit was observed about 23% after adding of 30% off marble dust whereas; it decreased to about 17.5% with adding rice husk ash. It has been noted that the free swell index decreased considerable to about 80% by addition of 20% of marble. While in the rice husk ash using the reduction of free swell index is lesser than in the marble dust and it is near 38%. At the end of the all results it can be conclude that Marble dust has more influence than Rice husk ash for improving of expansive clays.

2.12 Stabilization of Expansive Soils using Scrap Tires Rubber

Nowadays, due to the substantially increasing of the world population, and considerably rising of people’s requirements, a huge amount of industrial and agricultural wastes are gathered. The accumulated waste materials impose significant environmental problem. There are a lot of studies on possible methods of recycling or reusing these wastes safely. There are many methods and fields suggested to reuse the waste materials, but their use in the construction field is most applicable. One of the common waste materials which uses in the construction application is the scrap tires rubber. The utilizing of scrap tire rubber in the geotechnical engineering is more substantial methods for recycling the worrying waste materials. Hopefully many researches have been carried out which they emphasize that reuses of waste materials more feasible and successful in the construction application, especially it has a crucial role in the stabilizing of asphalt and Bitumen, also there are many researches were conducted about effects of scrap tires rubber in road construction, however, the number of investigations which depict the potential of scrap tires rubber in modifying the expansive soil still limited. The most important studies about the effects of scrap tire rubber in improvement of soil characteristics were documented as follow. Trouzine et al (2012) has evaluated influences of scrap tire rubber (STR) on the enhancement of swelling behavior of clay particles in the expansive soil. They chose two soil samples from “Ayaida and bentonite clays in the north –west of Algeria”, the two samples of clay were mixed with different STR contents (10%, 20%, 25% and 50%). These experiments were executed for both cases of natural soil and treated soil. Grain size distribution, Atterberg limit, specific gravity, loading–unloading test, and swell-consolidation tests. The results reveal that swelling potential and swelling pressure decrease with increasing STR content. An increase in the STR percentage causes the substantial rising in the compression and recompression indices of the expansive soils. Producing a new subgrade soil by using the expansive soil- rubber (ESR) mixture, is the main application of scrap tire rubber in the study conducted by Carraro et al., (2013), since the ESR can reduce the swell percentage and swell pressure without any harm to the shear strength of the ESR mixture. They performed the experimental and empirical field works for their researches, and the main conclusions drawn were that in both compaction tests (standard and modified), there is a linear decrease in the maximum dry unit weight of the ESR mixture with the rubber percentage. The optimum moisture content slightly increased when the percentage of ESR is increased from 0 to 20% in the modified compaction effort, while in the standard compaction test effort the optimum moisture content decreased faintly. An increasing in the rubber content causes reduction in the stiffness of compacted expansive soil. The Poisson's ratio of materials reduces slightly with addition of rubber percentage. Furthermore, the swell pressure and swell percentage of the expansive clay reduced with increasing the rubber content. Characteristics of stiffness and shear strength of combination soil-rubber (ESR) have been assessed systematically in a research by Friel et al. (2011). The three different percentages of rubber used were 0, 10 and 20%. The influences of percentage of rubber and effective stress have been systematically calculated by doing isotropically undrained triaxial consolidation tests at three different levels of effective stresses such as 50, 100, and 200 kPa. The outcomes of this systematic research illustrate that an increase in the rubber content increased the slope of the critical-state lines and friction angle between particles. For all three levels of effective stress when the rubber percentage increases the large strain stiffness of ESR reduced by almost 12 to 58%. The ESR mixtures can be used in the engineering applications because it is quite stronger than the expansive soils and it can be utilized as back filling materials for supporting the retaining and foundation walls. Kumar et al. (2013) have mixed expansive soils with different additive materials such as lime + fly ash + waste tire rubber to modify the strength of expansive clay. The results of this research show that the maximum dry density increased when the additive materials are used. An increment in the additive materials improves the shear parameters of the expansive soils. Finally, they observed that the additives of fly ash, lime, and waste tire rubber has potential impact on the characteristics of expansive soils, therefore, utilizing these materials provides a two-fold benefit in modifying the expansive soil and also reducing the waste disposal issue. The strength and dynamic behavior of reinforced soils with contained waste fiber materials were concentrated in an experimental study which was conducted by Akbulut et al. (2007). Both natural soil and reinforced soil samples were exposed to shear box, unconfined compression, and resonant tests, to illustrate the effects of waste fiber materials on strength and dynamic manner of clays. In this investigation the scrap tires rubber, polyethylene, and polypropylene fibers were implemented as reinforcement. The test results show these variations which produced because of using scrap tires rubber. The unconfined compression strength value of the soils was increase after adding scrap tires rubber percent up to 2% and then reduced. Generally, the cohesion and internal friction angle value incremented with adding scrip tire rubber. The damping ratio and shear modulus of reinforced soil improved with increasing the scrap tire rubber content. Friel et al., (2014) took two samples of expansive clay from Loveland and Colorado were mixed with granular rubber to produce expansive soil-rubber (ESR) mixture and the effects of compaction effort on the shear strength and stiffness of the (ESR) were assesses. The test outcomes reveal that; an increase in the compaction effort which applied to the ESR mixture, improved strength of mixture, especially at the large strain. The stiffness of the ESR mixture increases faintly with addition of compaction effort. The stiffness of the granulated rubber has a vital influence on the stiffness of the ESR mixture, while, the particle size of granulated rubber is not observed that have crucial effect on the physical properties of the ESR mixture. Kalkan (2013) used another modification technique is the mixture of silica fume and scrap tire rubber to enhance the geotechnical characteristics of clayey soil. In this experimental research natural clay and treated clayey soil samples were tested and the shear box, unconfined compression, oedometer, and the falling- head permeability tests were performed. The results depict that the mixture of silica fume and scrap tire rubber improved both the unconfined compressive strength and shear strength of the clay. It was detected that the silica fume- scrap tire rubber mixture reduced both the swelling pressure and hydraulic conductivity. Finally, it is concluded that the silica fume-scrap tire rubber mixture plays a vital role in stabilization of clayey soil in the geotechnical implementations. Purushotham et al. (2012) investigated the influence of crumb rubber powder (CRP) on improvement of clay problems. The results of this experimental work have yielded these conclusions: the liquid limit and plasticity index reduced with increasing CRP content. An increase in CRP from 5% to 25% caused reduction in the maximum dry density of the clay. The optimum moisture content reduced with addition of CRP up to 10%, and adding CRP more than 10% increased the optimum moisture content. With addition of 5% CRP the unconfined compressive strength was improved.

Chapter 3

MATERIALS AND METHODS

3.1 Introduction

The main goal of this experimental study is to investigate the influence of the scrap tire rubber in modifying expansive soils. The natural expansive soil samples were mixed with different scrap tire rubber percentages which include 0%, 10%, 20%, and 30% of dry mass the soil and 0% percentage was considered as the control sample which is obtained from a pre-determined expansive soil area in the South Campus of Eastern Mediterranean University. Both treated and untreated soil samples were tested to study the variations in the physical and mechanical and hydraulic properties of the soil after mixing with different percentages of stabilizing material. The experimental part of this study consists of determination of physical, mechanical and hydraulic properties of expansive soil with 0%, 10%, 20% and 30% scrap tire rubber inclusions. This chapter gives information on the materials and the testing methodologies used.

3.2 Sample Preparation

The expansive soil which was investigated in this research work was brought from the Eastern Mediterranean University South Campus, Famagusta, North Cyprus. It was dug to a depth of 1.5 m to 2.5 m by using an excavator to obtain a soil sample clear of debris, vegetation and roots. A number of tests were conducted to evaluate the physical properties of the soil to identify and classify it. All the tests were performed according to the Standards of American Society for Testing and Materials (ASTM). Moreover, the soil is classified as borderline of MH-CH according to index properties and the Unified Soil Classification System (USCS). The soil samples which were inspected in this research were dried at 50° C for more than five days, then pulverized by using a grinder machine and the pulverized soil was kept at 50º C in order to protect from humidity. Some of the properties of natural expansive soil used in the present work are depicted in Table 3.1. The excavation and sample obtaining operation is shown in Figure 3.1.

Table 3.1: Physical properties of the expansive soil.

Properties	Expansive Soil
Liquid limit (%)	64
Plastic limit (%)	32
Plasticity iIndex (%)	32
Linear shrinkage (%)	15
Specific Gravity	2.69
Natural moisture content (%)	32
Optimum moisture content (%)	22
Maximum Dry Density (kN/m3)	15.36
Grain Size distribution:	
Gravel (%)	0
Sand (%)	0
Silt (%)	55
Clay (%)	45

Figure 3.1: The location of the expansive soil gathered from EMU South Campus.

3.3 Preparation of Scrap Tire Rubber (STR)

The scrap tire rubber utilized in this experimental work was manufactured by a recycling plant in Haspolat area near Nicosia in a local factory which it has named Rubber Land, the outside area of the factory and inside machines of the factory was displayed in Figure 3.2. It is an economical material that can be used in construction industry for different purposes. The scrap tire rubber used in this work has particle sizes ranging from 0.20 mm to 1.10 mm according to the sieve analysis test which was done for the scrap tire rubber. Generally, there are two essential methods for recycling the scrap tires to produce the tire rubber crumble, tire chips, and shredded rubber tire. The first method is “Ambient Scrap Tire Processing”, in which the scrap tires are subjected to reduction in size at or near the ambient temperatures, which is increased to 120º C maximum. In this method cooling process is not implemented which would make the scrap tire brittle. The whole ambient process step by step is displayed in Figure 3.3. The second method is “Cryogenic Tire Recycling”; in this method all the tires are cooled down to below -80º C (-112º F), the decrease of temperature changes the physical properties of tires and the tire rubber becomes approximately a brittle material and the process of size decreasing can be easily performed by pulverizing and breaking. The complete process which explains the steps is showed in Figure 3.4 (Reschner, 2008).

Figure 3.2: Rubber Land Factory, Haspolat, Nicosia.

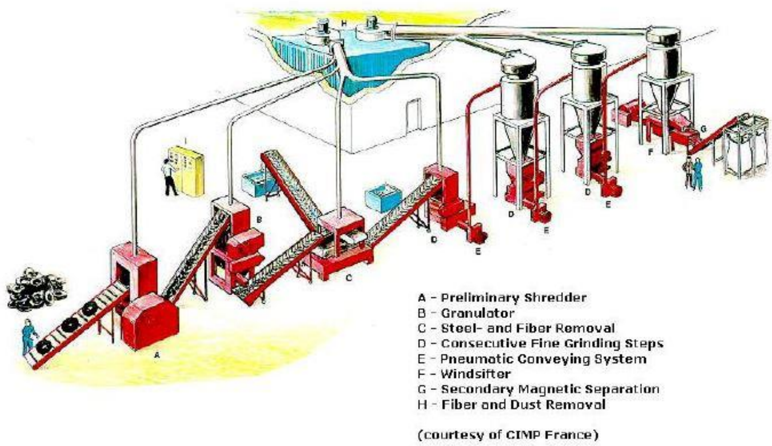


Figure 3.3: Diagram of an Ambient Scrap Tire Rubber Processing Plant (Reschner, 2008).

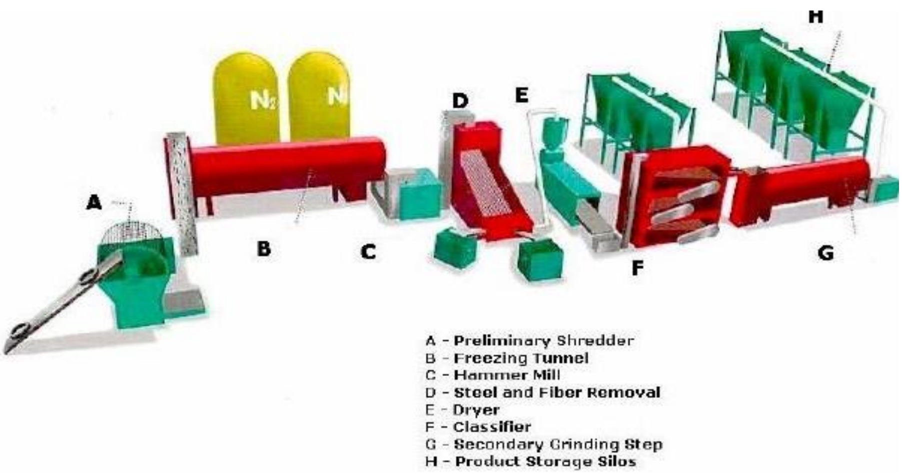


Figure 3.4: Diagram of a cryogenic scrap tire processing plant (Reschner, 2008).

3.4 Experimental Program

The methodology used in this research work includes experiments to determine physical, mechanical and hydraulic soil properties and the effect of increasing percentages of scrap tire rubber) addition. All the tests were carried out according to the American Society for Testing and Materials (ASTM). Liquid limit, plastic limit, specific gravity, linear shrinkage, hydrometer, standard Proctor compaction, oedometer (one-dimensional swelling, one-dimensional consolidation, and volumetric shrinkage), unconfined compression, and flexural strength tests and total suction measurements by chilled mirror hygrometer were conducted on the both treated and untreated soil samples.

For determining the specific gravity of scrap tire rubber and the Soil-STR mixture, the standard procedure could not be used, as the STR floated on the surface of the water as shown in Figure 3.5. Therefore for finding the specific gravity of mixtures, an analytical approach was used by considering each component's dry mass and the values of specific gravity according to Equation 3.1 (Sellaf et al., 2014) .

$$G_{s\ mixture} = \frac{Ms1+Ms2}{\left(\frac{Ms1}{Gs1}\right)+\left(\frac{Ms2}{Gs2}\right)} \tag{3.1}$$

where:

G_s: Specific gravity

M_s: Dry mass of specimen (kg).

Figure 3.5: Determination of Gs of Soil- STR mixtures with rubber floating on the surface of water.

3.4.1 One - Dimensional Swell Test

For assessing the swelling potential of the expansive soil, one-dimensional swell test is conducted in accordance to ASTM D4546–14. In this experimental work the samples compacted at optimum water content to maximum dry density with standard Proctor energy, were extruded to inside the s metal rings of 50 mm diameter and 20 mm height and the specimen high was 15 mm so as to the enough space is provided to the soil for swelling, as shown in Figure 3.6. The prepared ring samples were placed in the consolidation cells and saturated under 7 kPa surcharge pressure. Each sample was kept on porous stone and filter paper in both top and bottom of the specimen to prevent the particles from dispersion and allow the water to pass through the sample. The consolidometer devices were connected to the computer to monitore swell at different time intervals. The swelling of the samples started soon after the addition of distilled water to the consolidation cells.

Figure 3.6: One-dimensional swell test procedure.

3.4.2 One- Dimensional Consolidation Test

One-dimensional consolidation test is the significant test to predict and evaluation the magnitude and ratio of the differential and total settlement of a structure and earth fill. Assessments of these parameters are importance in the design of engineered structures. In this experiment procedure the rate and value of consolidation of the soil specimens were calculated when the soil samples were restrained horizontally and axially were drained. After the soil samples have reached the maximum swelling value directly the consolidation stage starts according to the (ASTM D2435/D2435M – 11), the first stage of consolidation is loading process, the arrangement of loading process was found on ΔP/P =1, and each load increasing was applied for 24 hours, 1, 2,4,8,16,32, and 64 Kg in the loading process were increased. And in the unloading process the loads were decreased from 64, 32, 16, and 0. The data were recorded by the computer program called MPX 2000, and they analyzed by the Data System 7 (DS7) software of ELE international as shown in Figure 3.7 From the recorded data, consolidation curve (pressure-void ratio relationship) could be plotted.

(a) (b)

Figure 3.7: (a) DS7 software usage for data analysis, (b) Consolidation test setup.

3.4.3 Volumetric Shrinkage Test

Volumetric shrinkage can be defined as the reducing in the volume of the soil when it dried, because of decreasing the water content of the soil samples. This process starts when the soil sample completely swelled; the sample was taken out from odometer device and the average of diameter and high, and mass of the specimen were measured and it was left in the oven at 40°C to desiccate. All the measurements were taken in the interval time and daily until the soil sample reached to fully dry and the evaporation process has finished. Furthermore, based on these measurement data some relationships were established, the most essential one is the curve between the void ratio and water content during desiccating process. This procedure was repeated to treated soil samples. The volumetric shrinkage procedure which consists of taking specimen mass and dimension measurement were revealed in Figure 3.8.

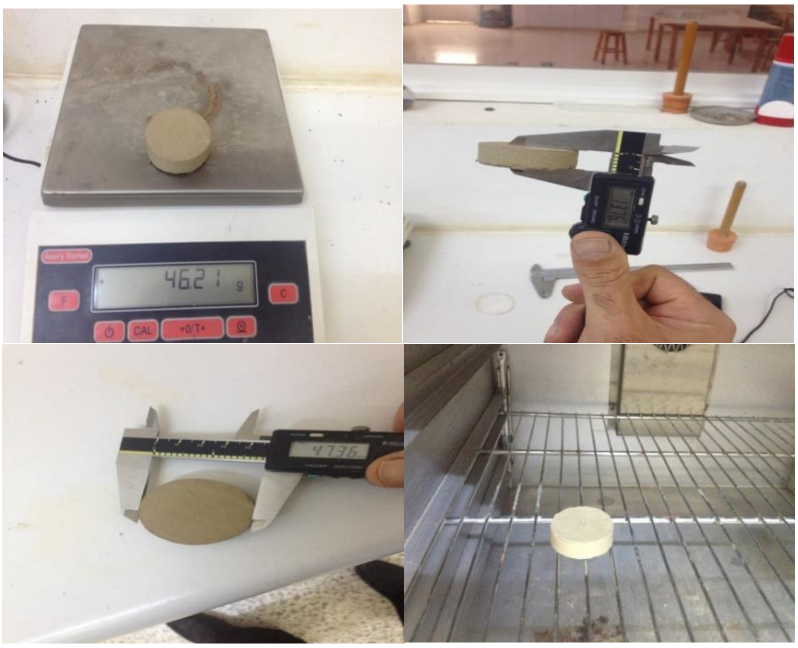


Figure 3.8 Volumetric shrinkage test procedure.

3.4.4 Suction Measurement

For the first time the significance of soil suction concept was identified by engineers in the 1950's. In general, soil suction comprises of matric and osmotic suction constituents. Also total suction is the sum of both osmotic and matric suction. Additionally, soil suction can be recognized as the free energy state of soil water or the relative vapor pressure of the moisture of soil. In this research work for measuring the soil suction (chilled-mirror psychrometer) was implemented to guess total and matric suction of samples so as to draw the soil water characteristic curves of compacted natural expansive soil and mixtures of it with 30% STR.

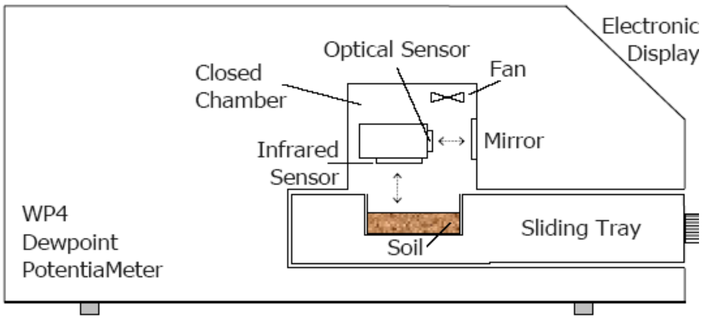
3.4.5 Chilled-Mirror Psychrometer

Dewpoint potentiameter WP4-T (Decagon Devices, Inc.) was utilized as a quick and very accurate device for calculating the total soil suction of the natural and mixed prepared specimens. In order to get reliable results and avoid from more error the device should be calibrated before using, the calibration and verification of the device conducts by using potassium chloride (KCl) solution at temperature of 25˚ C.

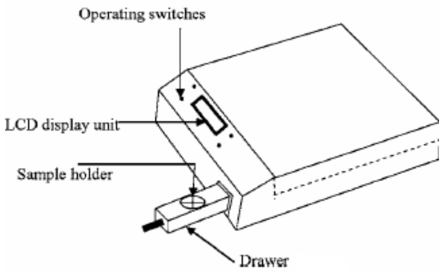
The samples were prepared with a simply way by dividing the sample in to two pieces and filling half of the disposable specific plastic cups of the equipment. And after the samples were put in the devices drawer, it is closed and it starting to work for 5-15 minutes for keeping temperature equilibrium between soil sample and measuring chamber. After that the predictable suction (MPa) and temperature (°C) would be shown on the LCD device screen followed by alarm ring with a green flash, demonstrating the results. Figure 3.9 displays the chilled mirror device and temperature plate equilibration.

(a)

Figure 3.9: (a) chilled mirror device test procedure, (b) detail of the inner part of chilled mirror psychrometer, (c) Schematic view of chilled mirror psychrometer.



(b)



(c)
Figure 3.9: (Cont.)

3.4.6 Flexural Strength

This test technique utilizes for finding the flexural strength of the soil and the test procedure was modified from ASTM C348-14, the standard test method for “flexural strength of hydraulic-cement mortars”. The samples of dried soil were mixed with water at optimum moisture content which was found from Proctor compaction test.to mellow. After mellowing period of 24 hours, the samples were compacted in a mold used for preparation of flexural test beams by static compaction. The specimens were prepared by compacting the soil mixtures directly in the mold the special mold and rectangular hammer used for specimen preparation are depicted in Figure 3.10. The amount of soil compacted statically in the mold was determined according to the maximum dry density and optimum water content which were determined in Proctor compaction test. The compacted soil beams prepared for flexural strength test had dimensions of 160×40×40 mm with 100 mm distance between the supports on which the beam was seated. The computer system was used to calculate the applied load and the deflection of the beam during the test procedure after that the tensile strength of the samples were determined using Equation 3.2.

$$fs = 1.5 * \frac{FI}{bd^2} \qquad (3.2)$$

where,

fs: Flexural strength

f: Applied load,

l: Distance between two supports,

B: Beam width, and

d:Beam height

The test setup and the soil beam failure under flexure are shown in Figure 3.11. This test procedure was repeated for finding the flexural strength of treated soil samples with different amounts of STR.

Figure 3.10: Special mold for preparing compacted soil beam and the equipment used for static compaction.

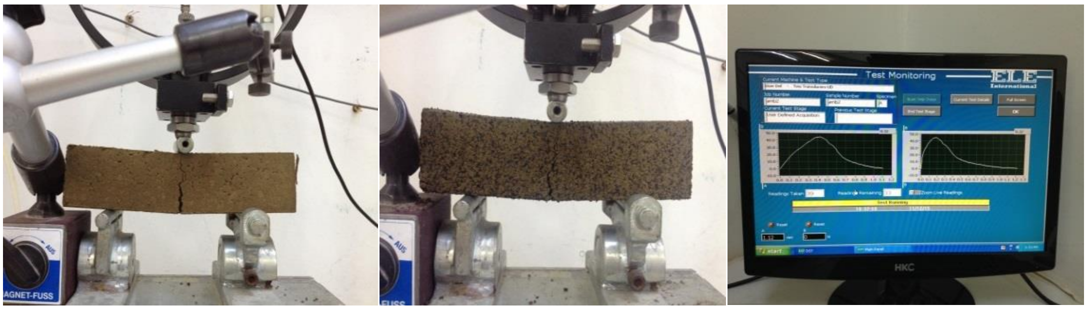


Figure 3.11: Flexural test set up and application of load on soil beam to failure with load versus deflection curve obtained during testing.

3.4.7 California Bearing Ratio Test (CBR)

The CBR test is one of the tests, which has a significant role in the pavement design, since the CBR test is utilized for assessing the strength of sub-base, subgrade, and other essential materials which are used in pavement construction. Additionally, the CBR test can be defined as the percentage of applied load per unit area which is desired to penetrate into a compacted soil sample by using a round plunger with diameter 50 mm and the machine used for performing the penetration at the rate of 1.27 mm/min. In this experimental work, the CBR test was performed according to the ASTM D1883-14 standard specification. Both natural samples and soil-STR mixtures were subjected to the CBR test in order to conclude the effect of STR on the CBR value. For preparation of the specimens for CBR test, method C in the Test Method D698 (Proctor compaction test) was used as was mentioned in the ASTM D1883-14 standard for CBR. According to the Method C, the mold which was used was 152.4 mm diameter and soil was compacted in three layers with 56 blows each. After completion of compaction process the specimens were placed in the CBR test machine and the test was performed in un-soaked condition under the surcharge of 4.5 kg. The sample was penetrated by a piston of diameter 50 mm. at a rate of 1.27 mm/min. The applied loads were recorded which corresponded to the penetrations of

0.50, 1.00, 1.50, 2.00, 2.5, 3.00, 4.00, 5.00, 7.5, 10.00, and 12.50 mm during the test procedure. Equation 3.3 was used for converting the dial gage readings of the applied load to Newton (N). The CBR test procedure is depicted in Figure 3.12.

$$y = 2.6476x + 0.2625 \qquad (3.3)$$

where:

y: applied load with (N).

x: applied load with dial gage readings deviation.

Figure 3.12: CBR test procedure.

Chapter 4

RESULTS AND DISCUSSIONS

4.1 Introduction

As it was explained in the previous chapters, this experimental investigation evaluates the suitability of waste scrap tire rubber (STR) to be implemented as soil modifying agent to mitigate expansive soils. In order to emphasize the effect of STR on the behavior of an expansive soil gathered from EMU South Campus, North Cyprus, and a series of tests were carried out. The experimental findings are presented and interpreted in this chapter, discussing both positive and negative effects of different percentages of STR on the characteristics of the expansive soil. The experimental program of this study consists of determination of physical properties and compaction parameters, volume change (swell-shrinkage and compressibility), strength characteristics (unconfined compressive strength and flexural strength tests) and hydraulic properties (SWCC and hydraulic conductivity).

4.2 Grain Size Distribution

The sedimentation test was carried out since all the material passed through sieve No.200. A hydrometer was used to classify the particles which are smaller than 0.002 mm (sieve No.200 according to ASTMD422–63). The results demonstrated in Figure 4.6 indicate that the expansive soil consists of 45% clay and 55% silt. Sieve analysis was conducted on the STR alone and the results are depicted in Figure 4.1. Based on the sieve analysis the STR is determined to consist of medium-coarse poorly graded sand size particles. Therefore, the particle sizes of each soil-tire mixture are as given in Table 4.1.

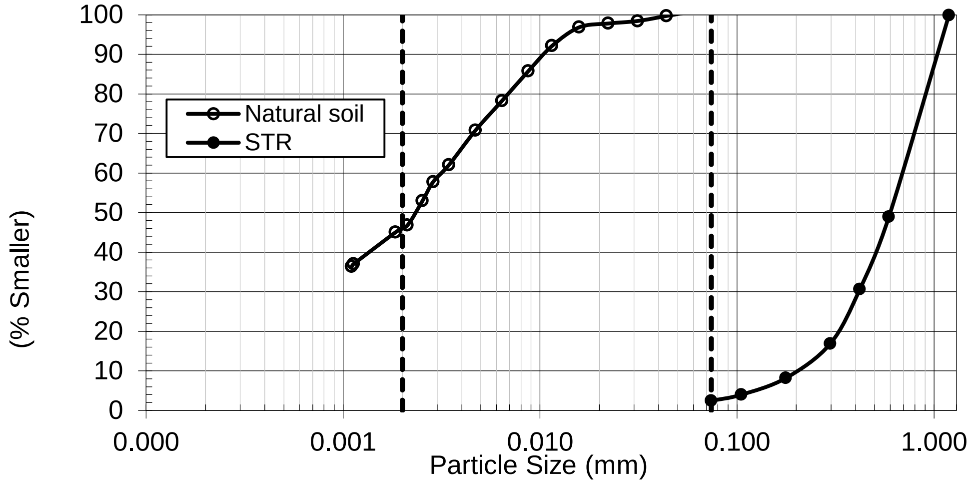


Figure 4.1: Particle size distribution of natural soil and STR

Table 4.1: Particle sizes of soils uses based on sieve and sedimentation analyses.

Soil	Sand size (%)	Silt size (%)	Clay size (%)
N	0	55	45
N+10%STR	10	50	40
N+20%STR	20	44	36
N+30%STR	30	39	31

4.3 Atterberg Limits

The results of liquid limit and plastic limit tests for natural soil and soil mixed with different percentages of STR (10, 20, 30, and 35%) are displayed in Figure 4.2. The results depict that liquid limit decreased from 65% to 46% when the percentage of STR increased from 10% to 35%. Hence a reduction of 28% can be observed. Similarly increasing in STR content decreased the plastic limit from 33% to 24%, by adding 30 to 35% STR to the expansive, causing a reduction of approximately 23%. Hence plasticity reduced from 32% to 21% as displayed in Figure 4.2. Table 4.2 depicts al the Atterberg limits and the qualitative classification of swell potential based on the classification schemes of Chen (date?) and Holtz and Gibbs (date?). The swell potential altered from high to low the type of expansive soil used in this research can be classified according to the Unified Soil Classification System (USCS) and the plasticity chart. The results of Atterberg limits show that the expansive soil is on the border of high plasticity clay (CH) and high plasticity silt (MH), as displayed in Figure 4.2. After the soil was mixed with different percentages of STR, the classification of the clay has changed to low plasticity clay (CL) with the 30% to 35% STR inclusion, using the classification scheme in Table 4.3 which is based on past experience as cited in Mansour (2011).

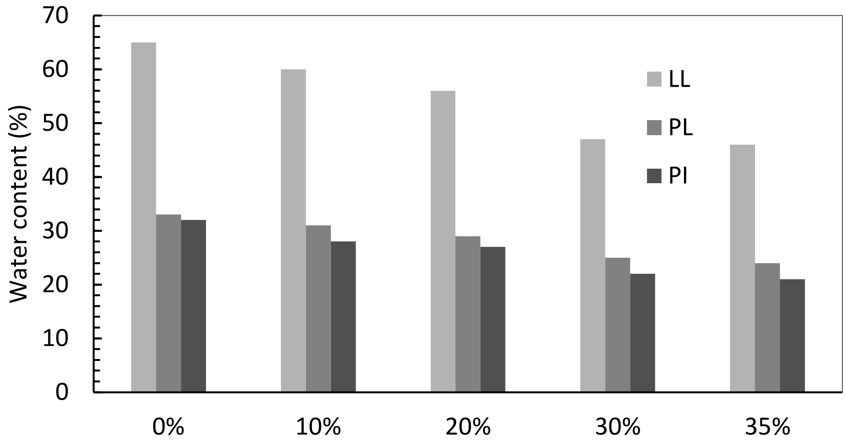


Figure 4.2: Atterberg limits versus percent STR added.

Table 4.2: The results of Atterberg limits of treated and untreated samples.

Soil	Liquid Limit	Plastic Limit	Plasticity index	Swell potential
Natural soil	65	33	32	High
N.Soil+10%STR	60	31	28	Marginal
N.Soil+20%STR	56	29	27	Marginal
N.Soil+30%STR	47	25	22	Low
N.Soil+35%STR	46	24	21	Low

Table 4.3: Classification of expansive soils (Mansour, 2011).

Classification	Plasticity index (%)	Liquid limit (%)
Non-expansive	0-6	0-25
Low	<25	25-50
Marginal	25-35	50-60
High	>35	>60

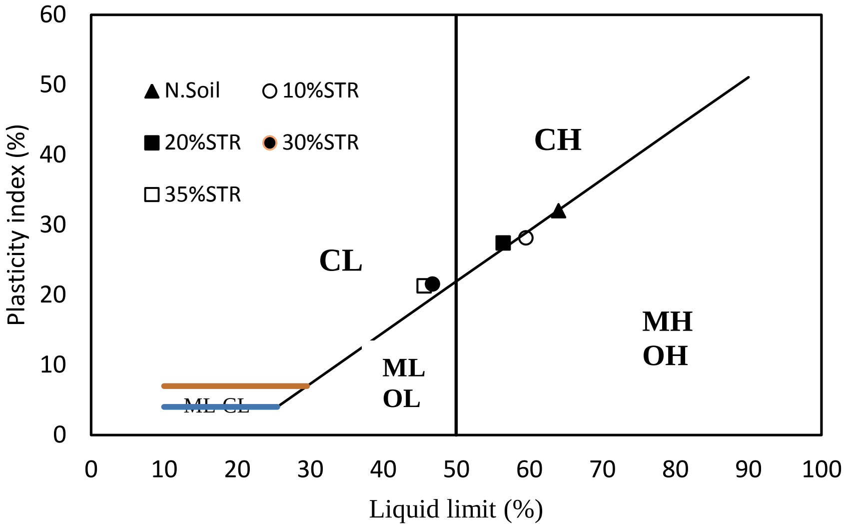


Figure 4.3: Unified Soil Classification System (USCS) with plasticity chart.

4.4 Linear Shrinkage

This test was conducted to determine the linear shrinkage of the drying soil. Linear shrinkage is the reduction in the length of the sample when completely dries. For implementing this experiment, the British Standard (BS-1377; 75) was utilized. The linear shrinkage test was conducted on the natural and mixed soil. According to the results the maximum reduction in shrinkage limit has been recorded in the 20% STR which it decreased from 15.72% to 11.29%. Generally, the percentage of shrinkage decreases with increasing the STR% about almost 28%. The results obtained are depicted in Figure 4.4.

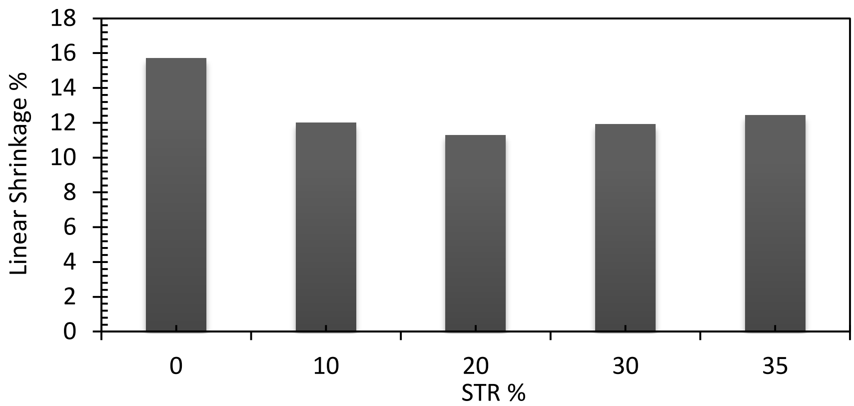


Figure 4.4: Variation of linear shrinkage with increasing STR content.

4.5 Specific Gravity

The Specific gravity of the natural expansive soil determined according to the ASTM D854–14 standards is 2.69. However, this method was not applicable to find the specific gravity of the soil-STR mixture since the STR is light weight material and floats on the water surface inside the pycnometer. Therefore for finding the specific gravity of the mixtures, a theoretical method was applied which was mentioned in the literature (Sellaf et al., 2014). It can be calculated by using each component's dry mass and the values of specific gravity according to Equation 4.1, and the scrap tire rubber's specific gravity was assumed as 1.16, taken from Guidance Manual for STR (2008). (Carroro et al., 2011) was used 1.16 as the STR's specific gravity. The calculated specific gravity of the natural soil and mixed soil are given in Table 4.4. Figure 4.5 depicts the appreciable reduction in the specific gravity of the expansive soil mixture, with increasing STR content, which can be attributed to the reduction of soil solid particles, hence the density of the soil. Since the soil particles are replaced by the STR particles which have lower mass than the soil particles. .

$$G_s\ (\text{Mixture}) = \frac{(Md1+Md2)}{\left(\frac{Md1}{Gs1}\right)+\left(\frac{Md2}{Gs2}\right)} \tag{4.1}$$

Table 4.4: Specific gravity of natural and stabilized samples.

Natural Soil	STR	10%STR	20%STR	30%STR
2.69	1.16	2.38	2.13	1.93

4.6 Compaction Characteristics

The compaction test was implemented on both natural soil and on its mixture with STR in different percentages. The results of the Proctor compaction test have been used to evaluate the maximum dry density (MDD) and optimum moisture contents (OMC) which are presented in the Figure 4.5. Consequently the Table 4.3 demonstrates a summary of the compaction results. From the results it can be observed that increase in the STR content added to the mixture, causes reduction in the maximum dry density of the soil. By adding 30% STR to the soil, MDD has reduced 15.36 kN/m^3 to 13.33 kN/m^3, approximately 15% reduction occurs. However, there seems to be no appreciable change in OMC.

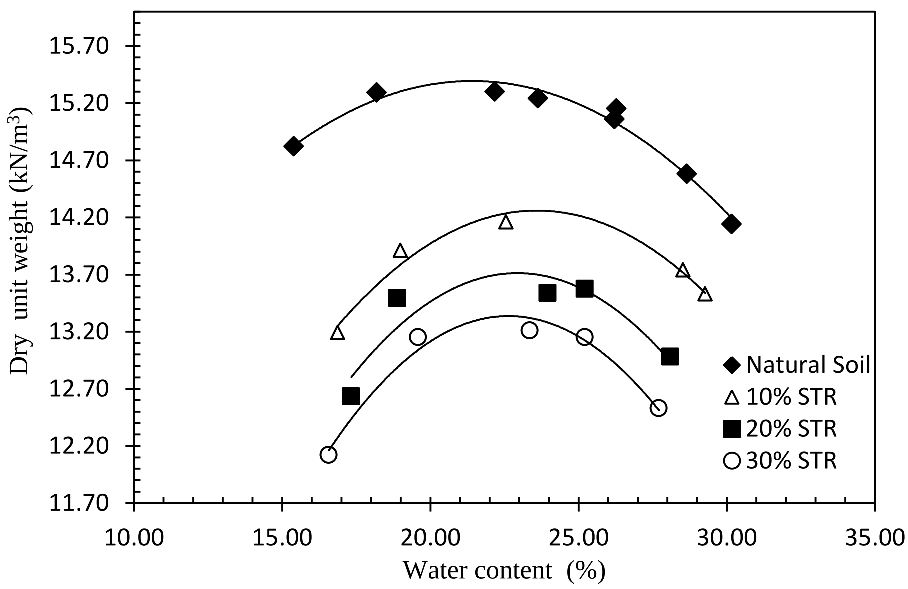


Figure 4.5: Standard Proctor compaction curve

Table 4.5: Compaction test results.

Samples	Optimum Moisture Content (%)	Maximum Dry Density (kN/m^3)
Natural soil	22.00	15.36
10% STR	23.60	14.26
20% STR	23.00	13.71
30% STR	22.70	13.33

4.7 Unconfined Compression Test

The results of unconfined compression test are interpreted as axial stress versus axial strain curve. This experiment was conducted on the natural soil and mixtures of soil with different percentages of STR. Figure 4.6 represents the variations which were occurred in the values of unconfined compressive strength of the presence soil after it was mixed with different STR% contents. It is obviously noted that the natural soil sample was failed at the highest point which it is 460.56 kPa and by adding a 10% of STR, it was reduced to 369.50 kPa, and when the percentage of STR was increased to 20% again the axial stress has reduced to almost 347 kPa. Conversely, the maximum reduction in the axial stress of the mixture was recorded, when the STR% was increased to 30%. However, based on the classification based on unconfined compressive strength as cited in Das and Sobhan (2014) given in Table 4.6, the unconfined compressive strength of 200 kPa obtained with 30% STR addition is still within the consistency range of stiff to very stiff. The stabilized soil failed at a higher failure strain indicating that the STR% enhances the ductility behavior of the expansive soil. The axial strain at failure versus STR percentages is presented in Figure 4.6. According to these observations a higher amount of STR would yield a lower unconfined strength, therefore to maintain a stiff consistency and strength, 30% STR is deduced to be the maximum percentage to be used.

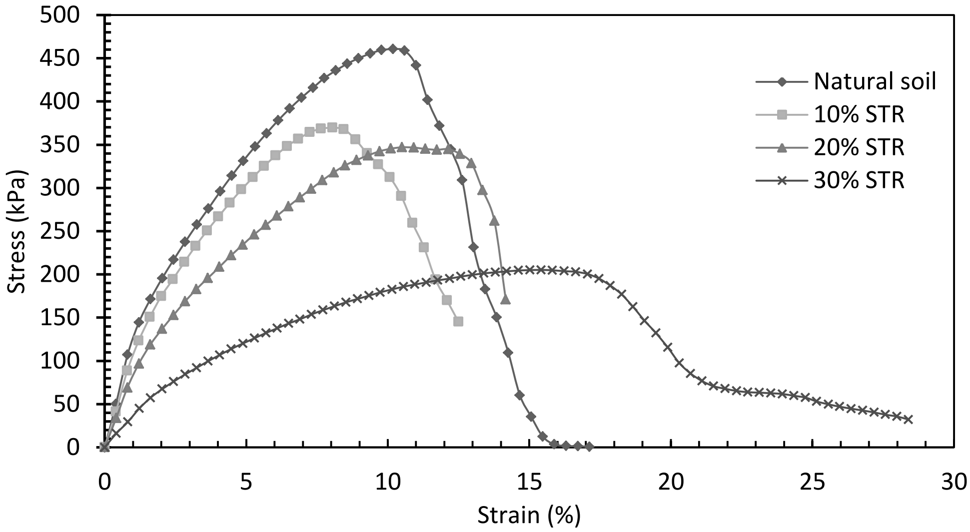


Figure 4.6: Unconfined compression test results.

Table 4.6: General relationship of consistency and unconfined compressive strength. (Das and Sobhan (2014).

Consistency	q_u (kN/m^2)
Very soft	0-25
Soft	25-50
Stiff	100-200
Very stiff	200-400
Hard	>400

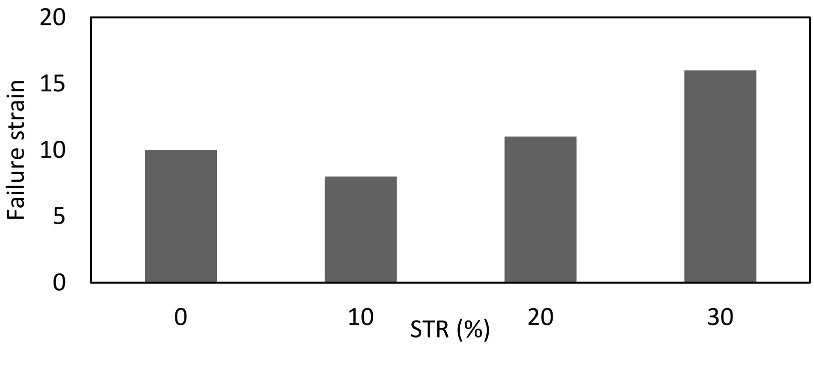


Figure 4.7: Deformation versus STR content in the unconfined compressive strength.

4.8 Flexural Strength Test

Tensile strength is an important characteristic of compacted soils to be exposed to heavy loads under structures such as airfield pavements, highways, landfills and embankments. Therefore, the stabilized soils under tension should be studied under tension. In this study, tensile testing is determined indirectly by applying flexural strength test which is an indirect method. It was conducted on the natural soil and soil-STR mixtures. The flexural strength versus deflection curves are depicted in Figure 4.8. The highest strength obtained is 134 kN/cm^2 for natural soil. It is observed that in the natural specimen sudden failure has occurred at approximately 0.55 mm deflection. Flexural strength reduces from 134 kN/m2 to 46, 20, and 18 kN/m^2 for 10, 20, and 30% STR additions respectively. Conversely, the deflection amount at failure as well as crack depth occurring at failure increased by adding STR. The crack depth was measured manually using digital vernier and the crack measurements were made at the failure time. According to the documented results, the crack depth of natural sample is 21.20 mm which increased with adding STR. At 20% STR the maximum crack depth was recorded which increased to 34.62 mm. Table 4.7 presents the results of flexural strength test for all samples.

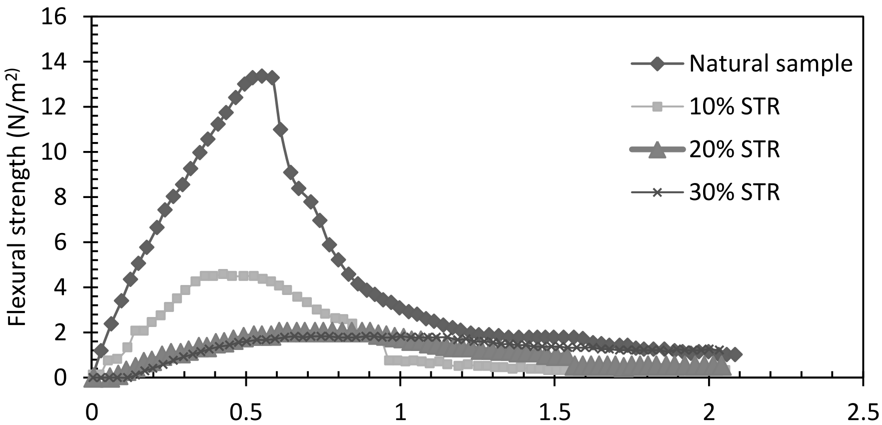


Figure 4.8: Flexural strength test results.

Table 4.7: Maximum flexural strength and crack depth of natural and mixed samples.

Samples	Max. Applied Load (N)	Maximum Flexural Strength (N/cm^2)	Crack Depth (mm)
Natural soil	60.18	13.35	21.20
10% STR	19.56	4.56	31.38
20% STR	9.62	2.00	34.62
30% STR	8.55	1.82	32.50

4.9 California Bearing Ratio Test (CBR) results

CBR test was performed on both natural soil samples and soil-STR mixtures. In both cases the unsoaked samples have been tested and the results which corresponded to the 2.54 mm penetration is considered as the CBR value. It can clearly be observed that increase in the STR percentages reduces the CBR value from 1.64 to 0.637 when the STR was added from 0% up to 30%. According to the test results presented in Table 4.8, 10-20% STR gives the best CBR value if this is the target, which is in good agreement with Prakash, et al. (2013). According to these data 30% STR addition reduces the CBR value significantly.

Table 4.8: CBR test results at 5.08 and 2.54 mm penetration

Samples	CBR% value at (2.54mm) penetration	CBR% Value at (5.08mm) penetration
Natural soil	1.660	1.520
10% STR	0.927	0.883
20% STR	0.913	0.880
30% STR	0.637	0.610

4.10 One- Dimensional Swell Test

For evaluating swelling behavior of expansive soil, the one- dimensional swell test was performed according to ASTM D4546-14 on both treated and untreated samples. The swelling curves have been plotted as percent swell strain ($\Delta H/H_0$ *100) versus time. The total swell is comprised of the initial, primary, and secondary swell stages. The primary swell is the basic constituent of the total swell the secondary swell stage occurs progressively and it takes a very long time to complete. According to the results presented in Figure 4.9 primary swell potential of expansive soil has been substantially reduced by adding STR up to 30%, the swell percentage being reduced from 6.95% to almost 0.91%. Therefore using 30% STR decreases swell potential by 86.90% at the primary stage of swelling. Moreover, there is an appreciable reduction in time of completion of primary swell. For STR additions less than 30% the swell potential observed is higher than 1.5% which is the recommended maximum primary swell percentage. Overall, the demonstrated outcomes indicate that STR additive is significantly effective for reducing the swelling behavior of expansive clays, most effective has been observed with the 30%STR.

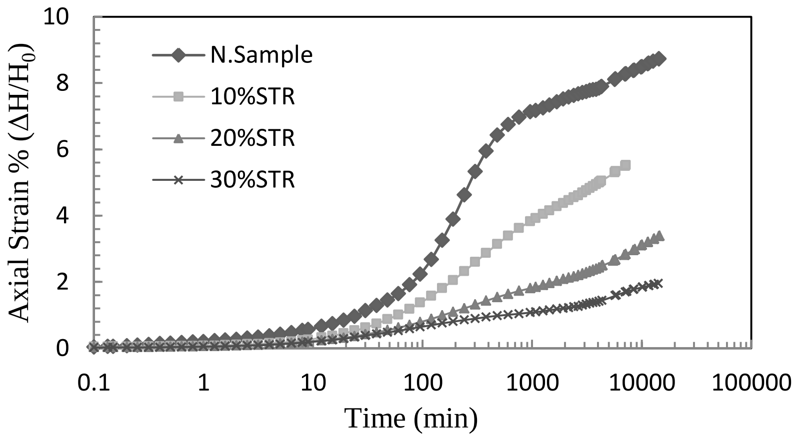


Figure 4. 9: One- dimensional swell strain versus logarithm of time.

These results are in good agreement with previous research findings of Trouzine, et al. (2012), who have observed the swell potential of the soil being reduced about 36.8% with 25% scrap tire fiber addition.

The shape of swell strain versus time graph resembles the shape of a rectangular hyperbola. Then the time/swell versus time relationship would be a straight line, based on Kondner (1963), who stated that the non-linear stress-strain curves of soils could be linearized by plotting the results in this way. This plot could be used to predict the ultimate swell percentage, which includes the expected secondary swell value from the reciprocal of the slope of the straight line obtained (Nagaraj et al., 2010; Dafalla and Al-Shamrani, 2011; Muntohar, 2003).

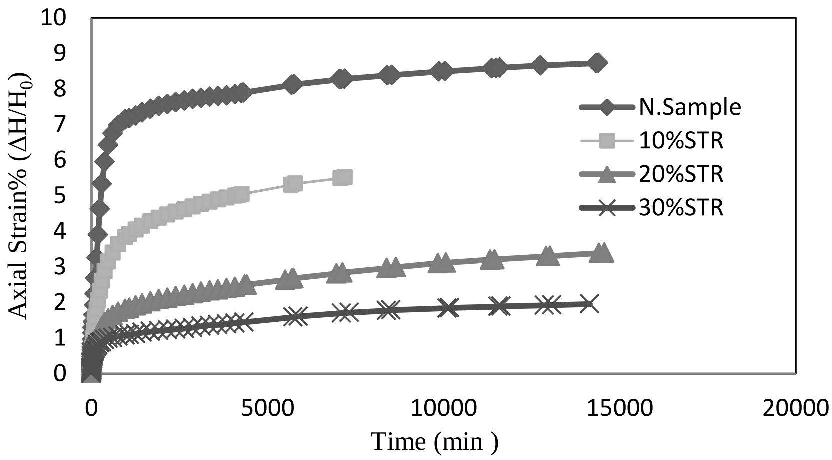


Figure 4.10: One- dimensional swell strain versus time.

From the reciprocal of the straight line the ultimate swell can be predicted by applying the hyperbolic model calculated in Equation 4.2 (Muntohar,2003; Murugan, 2009).

$$S\ (t) = \frac{dh(t)}{h_0} = \frac{t}{(a+bt)} \qquad (4.2)$$

Komine and Oggata (1994) proposed to obtain the maximum swell by finding the limiting value at infinite time as given in Equation 4.3, where t is the time from the start of water inundation, S(t) is the vertical swell at time t, and “a” and “b” are constants obtained from straight line fits giving the highest R^2 value.

$$S_{max}=\lim_{t\to\infty}\left(\frac{1}{\frac{a}{t}+b}\right) \;=\frac{1}{b} \qquad (4.3)$$

Theoretically, it takes infinite time to reach the ultimate swell value, which cannot be practically measured in the laboratory. Figure 4.12 depicts the time/swell (%) versus time plots, which are fitted with straight lines with fitting parameters “a” representing the ordinate and “b” the slope of the lines. The experimentally determined swell percentages, as well as times of completion of each swell type, as well as straight line fitting parameters and the predicted maximum swell percentages are presented in Table 4.9.

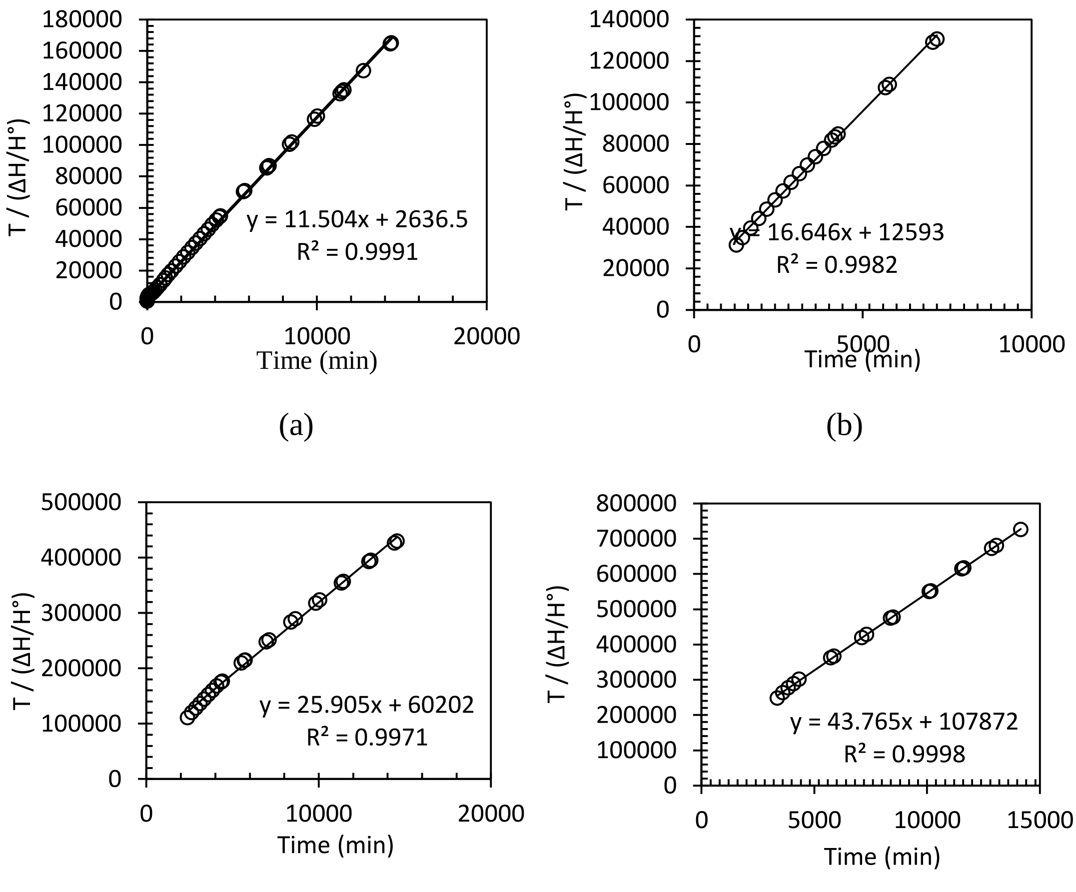


Figure 4. 11: Time/swell versus time relationships of (a) natural soil, (b) 10% STR, (c) 20% STR, (d) 30% STR added soils.

Table 4.9: Swell properties and predicted ultimate swell values.

Swell properties	N	N+10%STR	N+20%STR	N+30%STR
Initial swell (%)	0.73	0.61	0.31	0.12
Initial swell time (min)	53	45	28	10
Primary swell %)	6.95	3.62	1.67	0.91
Primary swell time (min)	560	650	700	400
Max. swell measured (%)	8.65	5.48	3.38	1.95
Hyperbolic constant, b	11.50	16.65	25.91	43.77
Hyperbolic constant, a	2637	12,593	60,202	107,872
Ultimate swell predicted (%)	8.69	6.00	3.86	2.28
R^2	0.9991	0.9982	0.9971	0.9998

4.11 One- dimensional Consolidation Test Results

Consolidation tests have been carried out on swelled samples. The average test results are displayed as void ratio versus applied pressure in log scale as plotted in Figure 4.13. The parameters obtained from consolidation results are compression index (C_c), rebound index (C_r), and swell pressure (p_s') as depicted in the Table 4.10.

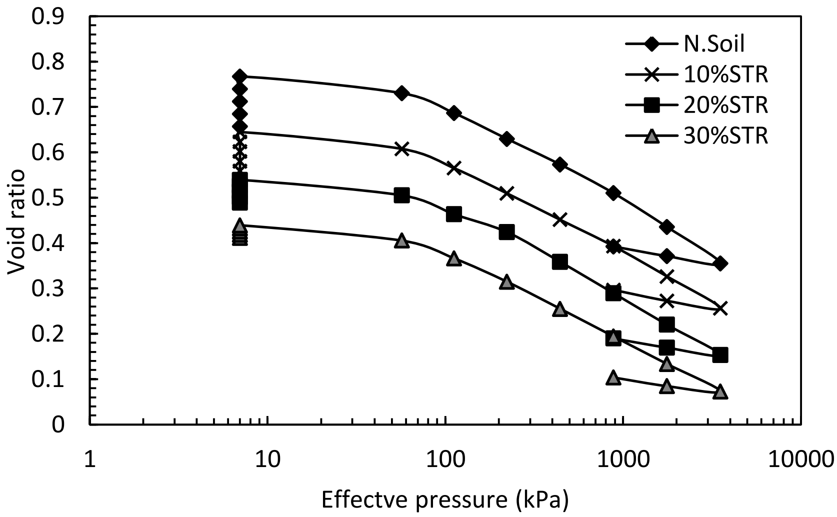


Figure 4. 12 Consolidation test results.

Table 4.10: Swell-consolidation test parameters.

Parameters	Natural Soil	10% STR	20% STR	30% STR
C_c	0.22	0.20	0.22	0.20
C_r	0.08	0.07	0.08	0.06
$p_{s'}$ (kN/m^2)	200	120	85	55

Generally it can be perceived that the compression and rebound indices remained almost the same after STR addition, whereas a significant reduction in swell pressure from 200 kPa to 55 kPa was observed when the STR% increased up to 30%.

4.12 Volumetric Shrinkage Test Results

The compacted soil samples which were completely swelled in the oedometers were dried at 40ºC, taking readings of mass, height, and diameter at different time intervals along the drying path, until the volume change ceased. The results have been plotted as volumetric, axial, and diametric strains versus time. The main target of conducting the shrinkage test is to study the behavior of mixtures of expansive soil and STR during desiccation and to assess the improvement in volume change when soil is treated. The parameters which have been obtained from shrinkage test represent the effect of STR on the expansive soil. Figure 4.14 demonstrates the relative variation in volumetric strain ($\Delta V/V_0$), axial strain ($\Delta H/H_0$), and diametric strain ($\Delta D/D_0$) versus. It is also noted that there were no cracks formed when the samples were reached to fully dried condition.

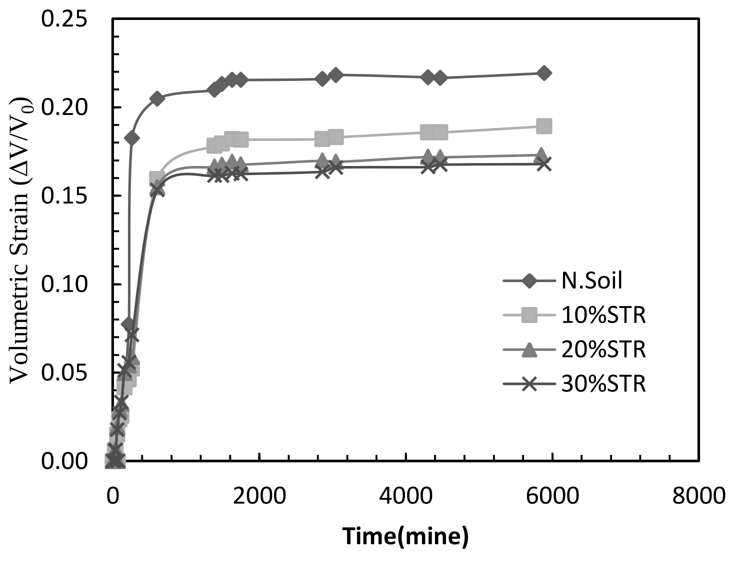


Figure 4. 13: The relationship between shrinkage strains and time.

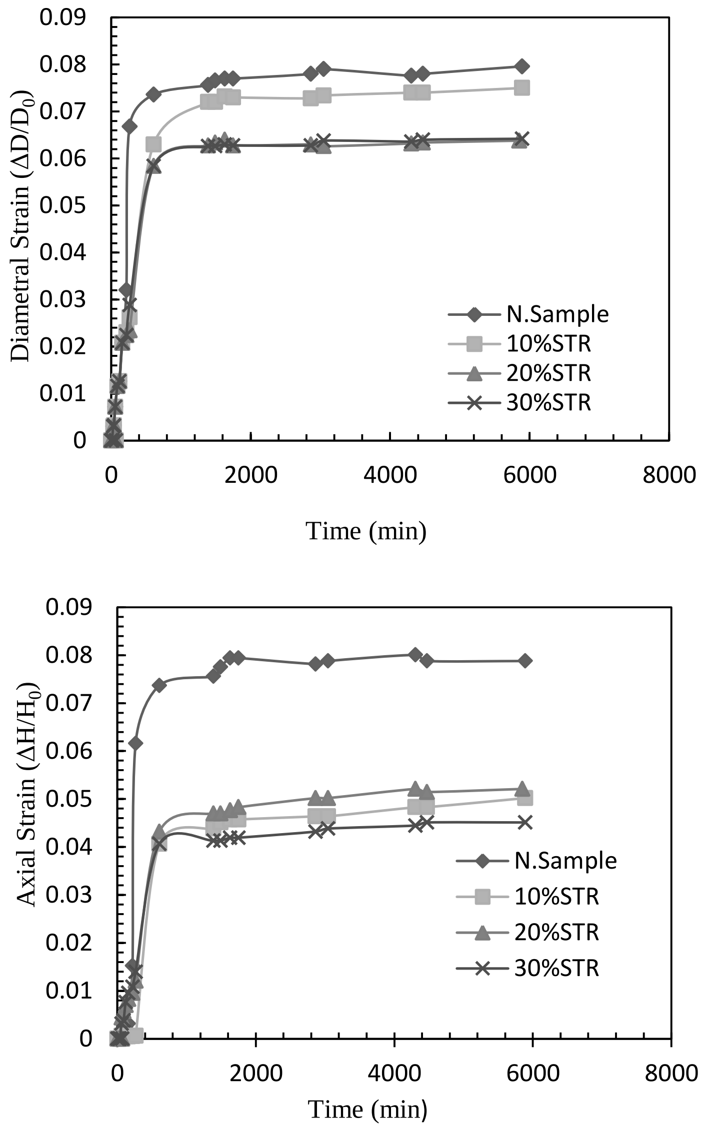


Figure 4.13: (Cont.)

4.13 Shrinkage Curve

Shrinkage curve is the relationship between void ratio and the water content. Figure 4.14 displays the shrinkage curves for natural soil and mixtures with different STR%. The results show that the void ratio of the soil decreases with reducing water content during drying phase. SoilVision software version 4.21 was used to fit hyperbolic model to the test data. The shrinkage starts at the saturation water content and proceeds until volume change ceases at the shrinkage limit and the final void ratio is observed at the end of the drying process.

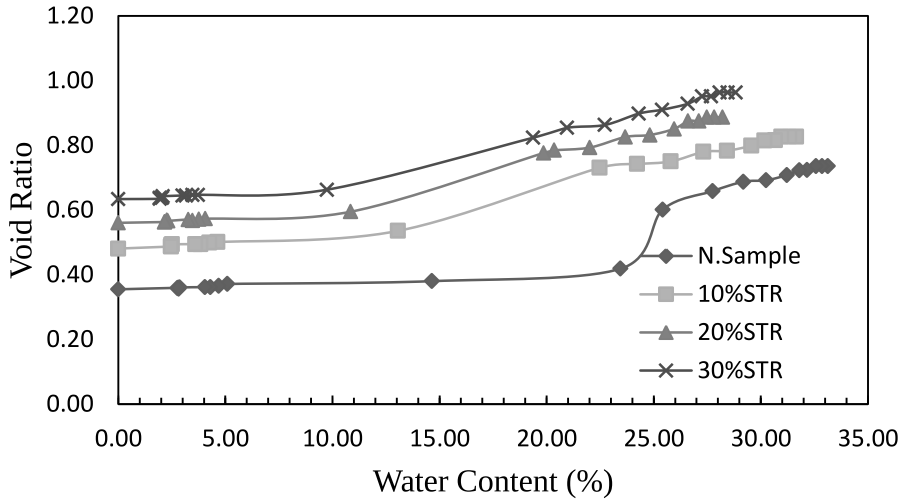


Figure 4.14: Shrinkage curve for natural and mixed soil.

The hyperbolic model fitted to the shrinkage data is represented by Equation 4.3.

$$e(w) = a_{sh}\left[\frac{w^{c_{sh}}}{b_{sh}{}^{c_{sh}}}+1\right]^{\left(\frac{1}{c_{sh}}\right)} \tag{4.3}$$

where,

The parameter a_{sh} is the minimum value of void ratio of the dried sample, b_{sh} parameter is the minimum water content at which the changing volume of specimen is stopped or it is the slope of the tangent line from saturation states, and the csh is the inflection of the shrinkage curve, (Fredlund et al., 2002). Figure 4.15 displays the shrinkage curve of natural soil; it can be observed that the void ratio of the sample decreased considerably from 0.74 in saturation state to almost 0.35 at dried condition, representing 53% reduction.

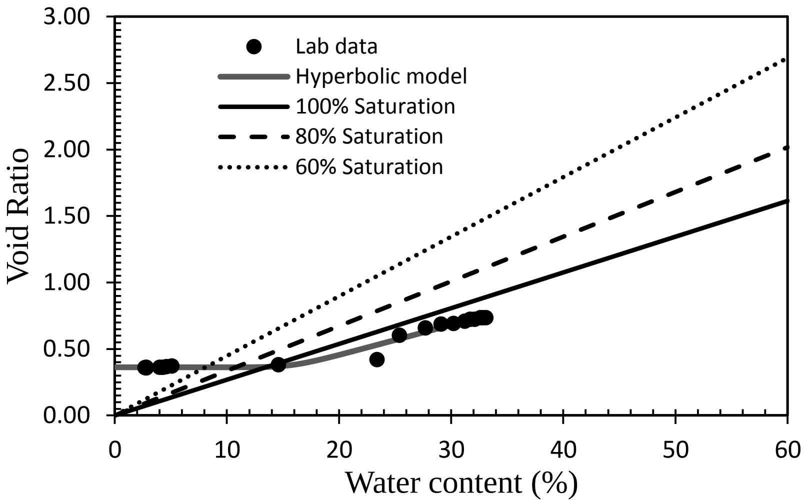
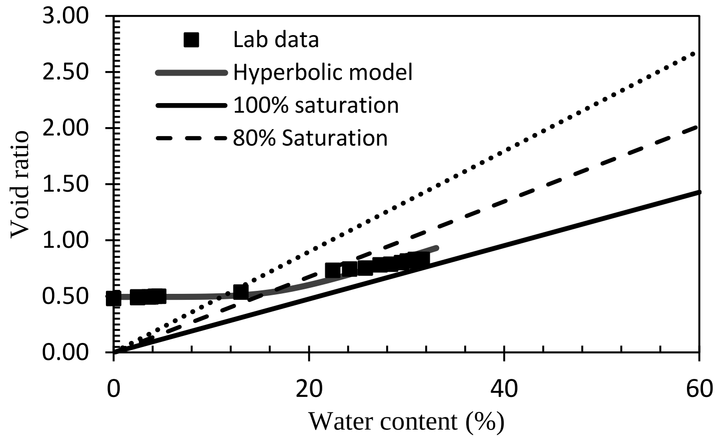
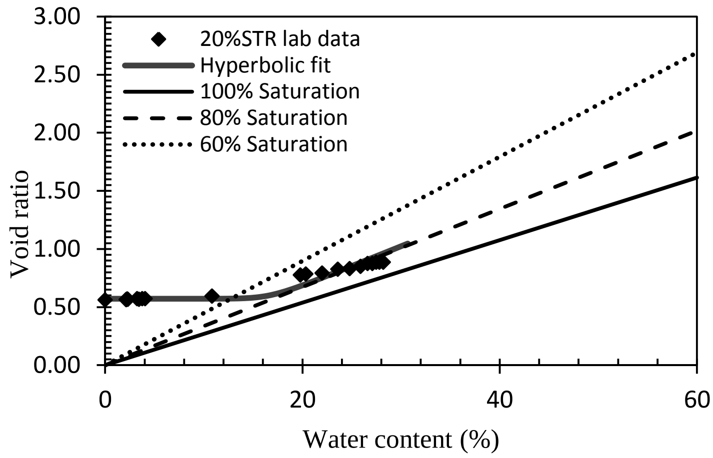


Figure 4.15: Shrinkage curve of natural soil with hyperbolic fit.

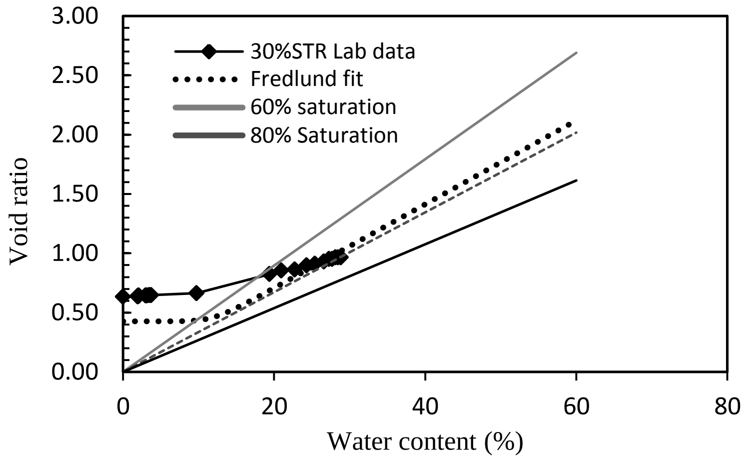
Figure 4.16 (a) shows the shrinkage curve for 10% STR sample with hyperbolic fit curve. It can be noted that the void ratio of the mixed soil sample with 10% STR has been reduced significantly from 0.83 in the full saturation condition to 0.48 when dried, a reduction of 42 % has occurred. Figure 4.16 (b) demonstrates the shrinkage curve of the treated soil sample with 20%STR, the void ratio of the soil specimen reducing from 0.88 to 0.56 which is a reduction of 36%. Figure 4.16 (c) displays the shrinkage curve of soil specimen which mixed with 30% STR with a reduction of 34% in the void ratio. The results indicate that the change in void ratio with respect to the initial value reduces when STR increases, therefore it can be deduced that STR is mitigating the soil shrinkage. The hyperbolic model parameters are presented in Table 4.11. It can be observed that the a_{sh} parameter, which it is the minimum void ratio at drying condition increases with adding STR% from 0.361 to 0.648. The b_{sh} parameter also increases with STR% content, which represents the shrinkage limit; the decrease in volume change also reduced the formation of cracks in the treated specimens.



(a)



(b)



(c)

Figure 4.16: Shrinkage curves of (a) 10% STR, (b) 20% STR and (c) 30% STR with hyperbolic fit curves.

Table 4.11: Hyperbolic fit parameters of natural soil and mixtures.

Specimens	a_{sh}	b_{sh}	c_{sh}	Shrinkage Limit	Shrinkage Error ($R^{2)}$
N.Soil	0.361	0.159	11.593	0.159	0.97
N+10%STR	0.495	0.177	5.697509	0.177	0.95
N+20%STR	0.572	0.168	13.17344	0.168	0.91
N+30%STR	0.648	0.183	6.08953	0.183	0.94

4.14 Soil Water Characteristics Curve (SWCC)

Expansive soils are unsaturated soils in semi-arid zones. Therefore, it is important to establish the water content-suction variation of expansive soil when their stabilization is studied, since suction has a crucial factor in influencing the mechanical properties including hydraulic conductivity, strength and volume change.

SWCC is a quantity of water storing capability of soil for certain soil suction. It illustrates the association between the gravimetric water content, w, or volumetric water content, θ and the matric suction, ψ_m $(u_a\text{-}u_w)$ or the total suction(which is sum of matric and osmotic suctions), ψ_t. The function of SWCC is used for prediction of hydraulic conductivity, volume change, compressibility and shear strength of unsaturated soils. Therefore, SWCC could be considering as one of the most significant parameters of unsaturated soils.

There is a hysteresis between the drying and wetting paths of soils. When the soil water content decreases, suction increases following drying path (desorption). In contrast, when the water content increases, it causes reduction in suction of soil during wetting path (adsorption). The SWCC curve has a breaking point relating to the matric suction at the start of desaturation, termed the air-entry value (AEV), and is recognized as the suction at which air enters in to the largest pores of the soil (Fredlund and Rahardjo 1993).

In this experimental study natural soil and mixture of soil with 30% STR were subjected to the chilled-mirror potentiometer test to evaluate the SWCC. The results are plotted and fitted by the two most commonly used models using SoilVision software. The models used are Fredlund and Xing (1994) and van Genuchten (1980). Fredlund and Xing (1994) presented an equation with three parameters which fit a wide range of soils and also is improved to be more precise in high ranges of suction (Equation 4.4).

$$w_w = w_s \left[1 - \frac{\ln\left(1+\frac{\psi}{h_r}\right)}{\ln\left(1+\frac{10^6}{h_r}\right)} \right] \left[\frac{1}{\left[\ln\left[\exp(1) + \left(\frac{\psi}{a_f}\right)^{n_f} \right] \right]^{m_f}} \right] \tag{4.4}$$

where,

w_w is the gravimetric water content at any soil suction,

ws is saturated gravimetric water content,

af is a soil parameter which is primarily a function of the air entry value in (kPa)

nf is a soil parameter which is primarily a function of the rate of water extraction from the soil once the air entry value has been exceeded

mf is a soil parameter which is primarily a function of the residual water content

hr is suction at which residual water content occurs (kPa).

van Genuchten equation (1980) presented an equation with flexibility of fitting a wide range of soils by using three parameters (Equation 4.5).

$$w_w = w_{rvg} + (w_s - w_{rvg})\left[\frac{1}{\left[1+\left(a_{vg}\psi\right)^{n_{vg}}\right]^{m_{vg}}}\right] \tag{4.5}$$

where,

w_w is the gravimetric water content at any soil suction,

w_{rvg} is residual gravimetric water content,

w_s is saturated gravimetric water content,

a_{vg} is a soil parameter which is primarily a function of the air entry value in (kPa)

n_{vg} is a soil parameter which is primarily a function of the rate of water extraction from the soil once the air entry value has been exceeded

m_{vg} is fitting parameter

Figure 4.17 and 4.18 are displaying the SWCC's for natural soil and soil mixed with 30% STR. Comparing the Fredlund and Xing (1994) and van Genuchten (1980) models, it can be observed that the SWCC of natural soil is very steep in both fitting models when desaturating. It explains that this soil has a high sensitivity to decrease of water content, whereas, the soil sample with 30%STR shows a lower difference between saturation condition and desaturation condition than natural soil sample. The slope of desorption line has decreased with 30%STR addition, from 1.065% to 0.7032% in the Fredlund fit and reduced from 0.7333 to 0.4035 in van Genuchten fit which indicate slower drying with STR addition. AEV is also reduced considerably when STR is used, hindering the air entry. The parameters of SWCC are depicted in the Tables 4.12-4.13.

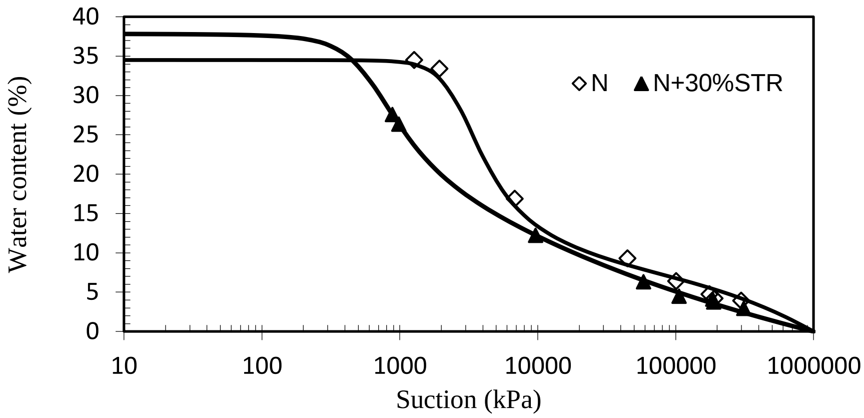


Figure 4.117: SWCC laboratory data fitted by Fredlund and Xing model.

Table 4.12: Fredlund and Xing model parameters.

Soil	Slope	AEV (kPa)	w_r	R^2
N	1.0653	1854	28.9	0.9953
N+30%STR	0.7032	363	14.30%	0.9988

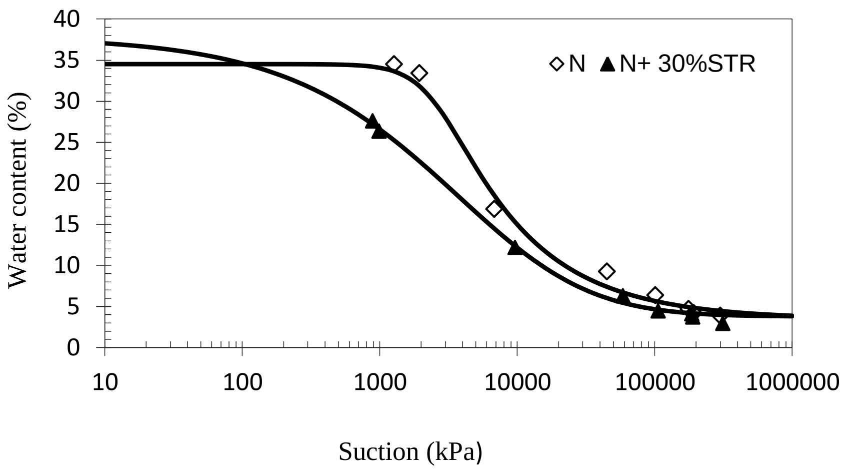


Figure 4.18: SWCC laboratory data fitted by van Genuchten model

Table 4.13: van Genuchten model parameters

Soil	Slope	AEV (kPa)	w_r (%)	R^2
N	0.7333	1318	10	0.9956
N+30%	0.4035	200	10	0.9959

Chapter 5

CONCLUSION

This investigation presents the findings of an experimental study on the possible use of waste material scrap tire rubber (STR) for modifying the properties of expansive soils. Using waste material in engineering applications plays an important role in protection of environment from pollution, since huge amounts of municipal and industrial wastes are discarded to landfills without considering any precautions for environmental protection. Waste tires are also main sources of environmental pollution, which occupy vast areas in scrap yards or landfill areas. Therefore, they are ground in different sizes and recycled either as a soil stabilizing agent or in producing rubber tiles for playground base or paving garden pathways. The STR used in this study was ground to poorly graded medium sand size in a newly established tire processing plant in Haspolat area. It is added to an expansive soil gathered from EMU South Campus in proportions of 10%, 20% and 30%, and the following conclusions are derived from the testing program:

1. STR decreased the plasticity index of the expansive soil, classified as high plasticity silt (MH) to low plasticity clay (CL). It was observed that STR amount less than 30% is still high plasticity, whereas inclusion of 35% STR yields almost the same result as 30% addition. Therefore, it is deduced that 30% could be accepted as the optimum amount to reduce the swell potential from high to low.

2. Regarding compaction characteristics, maximum dry density has reduced substantially, due to reduction of average unit weight of solids in the mixture of soil and STR. Therefore, higher the STR content, lower is the density of the mixture.

3. Addition of scrap tire rubber to expansive clay specimens caused a considerable reduction in the swell potential. The time of completion of the primary swell potential of natural soil, which is the major component of total swell, has reduced with 30% STR addition. However, the ultimate swell predictions and maximum measured swell percentages yielded increasing percent error with increasing STR content. This is attributed to the insufficient time allowed for observing maximum swell. STR particles hindered easy access of water to saturate and swell the specimens easily. Based on these observations, a higher percentage of STR could have yielded better results as swell potential less than 1.5% is the maximum value recommended.

4. According to consolidation test results, addition of STR to expansive soil does not have significant effect on the compression and rebound indices. However, swelling pressure of the soil has reduced substantially when mixed with different STR contents.

5. Shrinkage behavior of the soil was studied thoroughly by measuring water content, volume change and time readings. Regarding the shrinkage curves fitted by SoilVision software, the shrinkage limit has increased with increasing STR content. It can be concluded that there is a significant decrease in volume change with the increasing amount of STR which reduced the formation of cracks in the treated specimens.

6. In studying the soil-water characteristics curve, it can be concluded that increasing the STR causes a significant reduction in the air entry value from 1854 kPa to 363 kPa, indicating that air can penetrate easier in stabilized soil. Conversely, the slope of SWCC decreases from 1.065 to 0.703 which indicates a slower drying and reduction in the unsaturated hydraulic conductivity functions with respect to suction.

7. The strength behavior of STR added expansive soil was studied in unconfined compression, flexural strength and CBR tests. The results displayed no improvement in strength characteristics.

Finally, it can be concluded that the scrap tire rubber waste has a potential for stabilizing the properties of expansive soil. Thus providing a two-fold advantage, which are enhancing a problematic soil and resolving a problem of waste disposal by recycling it. Furthermore, it can potentially reduce stabilization costs. Since the results show that STR improves the swelling behavior of expansive soil and it does not enhance the strength of the soil, therefore this stabilization technique might be recommended for light weight structures, light traffic road base stabilization and backfilling for retaining structures.

As a future work, it is recommended to conduct a more detailed research considering effect of different sizes/shapes and percentages of STR. Durability of this technique should also be investigated considering climatic effects.

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