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Impact of Compaction Energy on SWCC of Stabilized Expansive Soil using Marble Waste

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On Thesis Title

IMPACT OF COMPACTION ENERGY ON SWCC OF STABILIZED EXPANSIVE USING MARBLE WASTE

By

Bayou Alehegn Wubet

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Approval of Thesis for defense result

I hereby confirm that the changes required by the examiners have been carried out and incorporated in the final thesis.

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As members of the board of examiners, we examined this thesis entitled "**Impact of Compaction Energy on SWCC of Stabilized Expansive Soil using Marble Waste by Bayou Alchegn Wubet**. We hereby certify that the thesis is accepted for fulfilling the requirements for the award of the degree of Master of Science in "**Geotechnical Engineering**".

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Declaration

This is to certify that the thesis entitled "**impact of compaction energy on SWCC of stabilized expansive soil using marble waste** on, submitted in partial fulfillment of the requirements for the degree of Master of Science in Geotechnical Engineering under the Faculty of Civil and Water Resource Engineering ,Bahir Dar Institute of Technology , is a record of original work carried out by me and has never been submitted to this or any other institution to get any other degree or certificates. The assistance and help I received during the course of this investigation have been duly acknowledged.

Bayou Alehegn Wubet		**02/13/2022**
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ABSTRACT

In tropical countries, including Ethiopia Expansive soils are difficult deposits for civil engineering construction because they swell and shrink during the wet and dry seasons, respectively. The moisture fluctuation of the soil can be controlled by using different techniques such as chemical and mechanical stabilization using optimum compaction effort to reduce swell-shrinkage behavior of expansive soil. But the selected stabilization techniques and comp active effort should be economical in addition to being effective. Currently, the cost of additives like cement and lime is increasing. It is best to try other locally available materials, such as industrial waste, as a stabilizing agent. In addition, some stabilization methods do not consider the fact that expansive soil exists in an unsaturated state. The use of the soil-water characteristic curve (SWCC) as a tool to evaluate the impact of compaction energy on the stabilization of expansive soil using waste marble is examined in this study in order to establish the relationship between unsaturated soil theory and engineering problems related to expansive soils that exist in an unsaturated condition. The soils collected from Bahir Dar City(at Ayer tena Kebele) are classified as A-7-5 according to AASHTO and based on USCS test pit-1, which classified as CH, and test pit-2 and test pit-3, which were classified as MH for specific areas. In the current study, the seven-day effect of curing condition, effective marble content, and compaction parameter on the SWCC of stabilized soil was investigated. For evaluation of the effectiveness of marble waste as a stabilizer, it was added to natural soil with 5%, 10%, 15%, 20%, and 25% of the dry weight of the soil for a test of Atterberge limits, linear shrinkage, free swell, specific gravity, and 5%, 10%, 15%, and 20% mw on compaction, unconfined compressive strength, California bearing ratio, and pressure tests. With the addition of 25% marble waste, the free swell index value decreased from 125% to 44% and the liquid limit decreased from 96% to 46%, indicating that the soil changed from high swelling potential to low swelling potential. The unconfined compressive strength value and the CBR value are directly proportional to the amount of marble waste. Because of the effect of pore size distribution, soils compacted with modified and standard energy at OMC in both treated and untreated soils have a slightly higher AEV value than standard compaction and a lower rate of desaturation.

Keywords: SWCC, Compaction energy, Residual water content, AEV, Marble waste

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LIST OF SYMBOLS AND ABBREVIATIONS

SWCC	=	Soil water characteristic curve
MDD	=	Maximum dry density
GS	=	Specific gravity of soil
USP	=	Unsaturated soils property
ASTM	=	American Society for Testing and Materials
Ψr	=	Residual suction
e	=	Void ratio
m	=	Soil parameter related to residual water content condition
n	=	Soil parameter related to the rate of desaturation
a	=	Soil parameter related to the air entry of the soil.
AASHTO	=	American Association of State Highway and Transportation Officials
PPA	=	Pressure Plate Apparatus
AEV	=	air entry value
Θs	=	Saturated water content
Θr	=	Residual water content
Ua-uw	=	matric suction
OMC	=	Optimum moisture content
Ψ	=	soil suction
MW	=	Marble waste

CHAPTER ONE
INTRODUCTION

1.1 Background of the Study

Early soil mechanics research and applications (e.g., the 1930s and 1940s) focused mainly on the behavior of saturated soils. There are numerous soil materials encountered in engineering practice whose behavior is not consistent with the principles and concepts of classical, saturated soil mechanics (linear approach). In theory, the saturated soil state is an ideal special case of unsaturated soils; this is often assumed in the traditional design approach with zero suction. However, this may not necessarily be the case in many situations. This is a simplification, not reflective of site conditions. It is the presence of more than three phases that results in a material that is difficult to deal with in geotechnical engineering practice. Engineering problems in embankments and highway construction, which have occurred in many countries in the tropical world, have been caused by engineering harm brought by failure to understand unsaturated soil properties. Soils that are unsaturated form the largest category of materials which do not adhere in behavior to classical, saturated soil mechanics (Fredlund et al.1994).

Partially in Ethiopia, the soils pose a serious threat to the lightly loaded structures constructed above them (especially lightweight residential buildings). The alternate swell and shrink behavior caused by moisture fluctuations in the ground leads to foundation failures of the structures built on these soils. In the area alone, the damage associated with expansive soil problems causes a great economic loss (Dagmaw 2007). Most of the geotechnical infrastructure in tropical regions like Ethiopia is also founded on unsaturated expansive soils. Soil Water Characteristic Curve (SWCC) is an important mechanical property of unsaturated soil that describes a highly nonlinear relationship between matric suction and water content (Hernandez et al. 2011). SWCC is not only able to describe the relationship between the strength of unsaturated soil and water content but also the distribution of water within the body of unsaturated soil. Unsaturated soil mechanics has primarily utilized the SWCC for the estimation of unsaturated soil property functions, which are subsequently used in numerical modeling solutions of geotechnical engineering problems (Hongliang et al. 2016).

Accurately determining the SWCC of unsaturated soils has important practical significance for effectively preventing engineering disasters caused by rainfall. Most of the infrastructure is founded on unsaturated soils. Even though constitutive relationships that utilize the concepts of unsaturated soils have been proposed for the classic areas of interest to geotechnical engineers, the application or implementation of these proposals into engineering practice has been rather slow. One of the reasons for the delay in the application of unsaturated soil mechanics in practice is, no doubt, the time required for the determination of the SWCC in the laboratory, and also the specialized equipment and training needed. SWCC is generally obtained by laboratory tests, due to the discrete test points being difficult to accurately fit into the highly nonlinear curves, and laboratory tests are also quite expensive. (Tao et al.2017).

SWCC framework is available to describe the engineering behavior of unsaturated soils in terms of two stress state variables, namely net normal stress, $(\sigma - u_a)$, and suction, $(u_a - u_w)$ (Lu and Likos, 2006). This approach is however costy and time consuming. Engineering properties of unsaturated soils such as the shear strength, coefficient of permeability and consolidation properties can be predicted reasonably well using the saturated soil properties and the soil-water characteristic curve (Fredlund 1997).

According to (Lu and Griffiths, 2004, Zhou and Jian-Lin, 2005) was suggestion various influences on the SWCC as a source of information, that the curves obtained from conventional tests often cannot be directly applied. The effects of void ratio, initial water content, stress state and high suction were studied in the work revealing that water content and stress state are more important than the other effects. Expansive soils occur both in temperate and tropical climates throughout the world. Problems associated with these soils have been reported in Africa, Australia, Europe, India, Israel, South America, United States as well as some regions in Canada. In tropical region expansive soils are known as black cotton soils and they are the major problematic soils. Due to changing moisture conditions, these soils show very strong shrinkage and swelling characteristics. In Ethiopia, expansive soils are found in the central, north-western, and eastern highlands of Ethiopia; in the western lowlands around Gambella and in some parts of the rift valley (ERAmanual2013). The shrink-swell behavior of unsaturated expansive soils caused most light-weight structures to crack. Those soils need to be treated.

The treatment mechanism of expansive soil problems, with the addition of industrial waste, agricultural waste, and controlling compaction effort, On the other hand, marble waste production is industrial waste, which increases time to time and affects the environment. So I want to stabilize such expansive soils with marble waste and evaluate the effect of compaction energy on SWCC. Unsaturated soils may be defined as the soil which has four phases: soil, water, air, and air-water interface, or the "contractile skin". The contractile skin is considered a fourth phase since it has definite bounding and different properties from the contiguous materials. The presence of small air pockets in soil renders the soil unsaturated (Fattah et al., 2015). The soil below the water table is fully saturated and the pore water pressure has a positive value. The ground water table is considered as the line at which the pore water pressure will be equal to zero (relative to atmospheric). Above the water table, the soil will be in an unsaturated state where the pore water pressure has a negative value. Depending on the nature of the material and suction range, laboratory measurements of the soil-water characteristic curve (SWCC) can be time-consuming and expensive, especially for residual soils, in which a wide range of particle sizes, compaction efforts, and soil structures typically result in SWCCs that cover a wide range of suction. The soil-water characteristic defines the relationship between the soil suction and gravimetric water content, w, or the volumetric water content. The soil-water characteristic curve was fitted with different models, and then the effect of degree compaction on the curve was evaluated. Most research shows that the compaction degree on the specimen will decrease with higher water content, and from the gravimetric water content-matric suction curve, it is found that compaction degree has an effect on air-entry value and water storage capacity (Choi etal 2015).

1.2 Statement of the Problem

During the last decade, geotechnical damage due to shrink-swell action has been observed clearly in the semiarid and tropical regions in the form of cracking and heaving of pavements, roadways, building foundations, reservoir linings, irrigation systems, water lines, and sewer lines. The problems also typically express the soil water characteristic curve (unsaturated soil property function). The soil-water function is highly nonlinear and relatively difficult to obtain accurately. Since the matric potential extends over several orders of magnitude for the range of water contents commonly encountered in practical applications, In addition, SWCC is influenced by different factors such as soil structure (and aggregation), initial water content, void ratio, soil mineralogy, particle size distribution, and compaction energy that have potentially affected the features of the SWCC. Compaction energy is one of the most significant parameters focused on the engineering properties of stabilized expansive soil. The compaction energy approach is for the stabilization of expansive soil up to a particular limit, since the above limit can increase swell potential. Compaction characteristics of high expansive soils are studied with emphasis on the relationships between moisture content and dry density of a soil at a range of compaction energy levels. In Ethiopia, available information on the impact of compaction energy on SWCC for stabilization of expansive soil test evaluation and factors affecting the soil water characteristic curve is insufficient to allow a detailed geotechnical explanation. On the other hand, the amount of marble waste production has increased from time to time. When demand for marble products rises, the generation of waste marble material also rises. The proportion of marble discharged as waste during block production also increased from 25% to 30% of the overall production volume. The results will increase environmental pollution and to take money, time for disposal system of annual production. So the research focused on the impact of compaction energy on SWCC stabilized expansive soil using marble waste in the cause of Bahidar (Ayer Tena Kebele). There is a significant research gap in Ethiopia regarding the nature of the soil-water characteristic curve on stabilized expansive soil and the effect of compaction.

1.3 Objective of the Study

1.3.1 General Objectives

The general objective of the research is to investigate the impact of compaction energy on SWCC of stabilize expansive soils using marble waste in the cause of Ayer Tena Kebele.

1.3.2 Specific Objectives

- To evaluate effectiveness of marble waste treatment on expansive soil treatment
- To evaluate the effect of compaction energy of stabilized expansive soil on SWCC
- To measured SWCC for different type of compaction energy on stabilized soil
- To evaluate compacted SWCC parameters for treated and untreated expansive soil and to introduce the applicability of SWCC in the effectiveness of treatment

1.4 Scope of the Study

This study focuses on investigating the effect of compaction energy on the SWCC of stabilized expansive soil by using marble waste. This was considered to reduce the impact of marble waste on the environment in addition to the stabilization of expansive soil. However, due to the limitations of the pressure plate apparatus and the time required to complete the desired number of tests, taking one sample that is highly expansive soil from the three samples and stabilized by MW to conduct index tests; standard compaction tests; modified compaction tests, particle size distribution; free swell tests, and suction measurement tests by pressure plate apparatus with a suction range of (33–1400) kPa.

1.5 Significant of the study

The most stabilized practice and compaction conditions are adapted above the ground water table. This area is also considered as a negative pore water pressure interaction (unsaturated soil). Unsaturated soil is also expressed by SWCC in which it is influenced by pore size distribution, void ration, compaction effort, soil plasticity, soil clay mineralogy, and grain size distribution. In Ethiopia, available information about SWCC for treatment soil test evaluation and factors affecting the soil water characteristic curve is insufficient to allow a detailed geotechnical explanation. Hence, this research is used to significant benefit to understand the effects of unsaturated expansive soil treatment practice on SWCC in the study area. Furthermore, the study will provide preliminary information about the impact of compaction energy on SWCC.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

In this chapter deals about the description of expansive soils origin, their distribution, identification and classification, damages and stabilization techniques. And also, the previous researches done on the classical approach of assessing on impact of compaction energy on stabilizer effect is reviewed. The describes about the nature and the behavior of SWCC, by different models proposed for predicting SWCCs over the entire suction range followed by pressure plate suction method (ASTMD6836) used to the laboratory to determine soil-water retention properties. Fredlund et al (1993) Stated that the general field of soil mechanics can be subdivided into saturated soils and unsaturated soils. The spaces and pores between soil particles can be filled with water or air or combination of both fluids if all spaces are filled with water, the soil is so called saturated. Otherwise it is unsaturated. In arid and semi- arid areas, the ground water table is usually located deeply under the ground surface. The soils above the ground water table have negative pore water pressure. Changes in climate conditions significantly affect the water contents of soils in the unsaturated zone. Upon wetting, commonly e.g. rain water infiltration, the negative pore water pressure increase towards positive values. Many soils exhibit significant swelling or expansion and great loss in shear strength when wetted for instance, expansive soils.

Unsaturated soil is defined as a soil consisting of three phases, namely solid particles, pore water, and pore air. It is a soil for which the degree of saturation S, the volume of pore water divided by the volume of pore air is less than unity (Blight et al, 2013). It has been generally recognized that the mechanical behavior of unsaturated soils is influenced by the pore-water suction, or negative suction. The structure is constructed in unsaturated soils. However, the principle of saturated soil mechanics is adopted in the geotechnical practitioners in designing the structures. To follow a sustainable approach and to reduce cost it is necessary to apply the principle of unsaturated soil mechanics in designing structures. In the past half century, unsaturated soil mechanics has emerged as a flourishing expansion of classical soil mechanics in dealing with mechanics of soil under partially saturated conditions. The Soil Water Characteristic Curve (SWCC) describes the functional relationships between soil water content and matric potential under equilibrium conditions. The SWCC is an important soil property related to pore space distribution (sizes, inter-connectedness), which is strongly affected by texture, void structure and other factors such as organic content. The soil-water function is highly nonlinear and relatively difficult to obtain accurately. (Chen et al., 2006). Many studies were conducted to assess the influence of different factors on the water retention properties of the soil in geotechnical engineering applications. Numerous investigations noted that the (Malaya and Sreedeep, 2012). Standard laboratory equipment based on the axis translation technique, such as the pressure plate, volume extractor, suction-controlled oedometer, modified triaxial apparatuses, and standard tension meters are the most common devices used to control or measure matric suction. Depending on the nature of the material and suction range, laboratory measurements of the soil-water characteristic curve (SWCC) can be time-consuming and expensive, especially for residual soils, in which a wide range of particle sizes and soil structures typically results in SWCCs that cover a wide range of suction.

2.2 Nature of Expansive Soil behavior

When the moisture content of expansive soils varies, the volume changes dramatically expansive soils expand when the moisture content rises and shrink when the moisture level lowers. The most well-known example of expansive soils is black cotton soil, which is grey to black in color and receives its name from India, where these soils are suited for cotton cultivation (Alemayehu and Mesfin1999). Changes in moisture cause these soils to swell and shrink, causing damage to civil engineering projects, especially light-weight structures and pavements (Jhon and Debora, 1992). Expansive soils behave differently from other normal soils due to their tendency to swell and shrink. Because of this swelling and shrinking behavior, expansive soils may cause structural damage to lightweight structures, basement construction, public utilities retaining walls due to pressure exerted on vertical walls and loss of residual shear strength, causing instability of slopes.

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2.3 Origin of Expansive Soil

Expansive soils are created by a combination of conditions and processes that lead to the formation of clay minerals with a certain chemical makeup that expand when exposed to water. Clay mineralogy is influenced by the parent material‟s composition, as well as the amount of physical and chemical weathering the components have experienced (Chen, 1988). The nature of the parent material is significantly more important during the early stages than it is after years of intense weathering. Expansive soils can be linked to two types of parent materials Basic igneous rocks such as basalt, dolerite sills and dykes, gabbros, and minerals such as feldspar and pyroxene; decomposition of these minerals produces an important mineral known as montmorillonite clay minerals, which are one of the smectite group and are highly expansive and effective for swelling behavior, as well as other secondary minerals (Chen, 1988). This type of soil can be found in many places around the world, especially in dry and semi-arid climates. Vertisol, high content loamy soil Smectite covers over 2.4 million square kilometers (240 million hectares) worldwide (Buol et al., 2003). Australia (80 million hectares), India (73 million hectares), Sudan (50 million hectares) ha), Southern and Western States of the United States (1,280,000 ha), Egypt, Ethiopia, Chad, Ghana, Cuba, Puerto Rico and Taiwan have great expanse (Hussein, 2010). Shale, swollen bedrock, containing a unique type of mineral called claystone, Rich in magnesium, contains montmorillonite, and physically the formation of expansive soil forms a second group of sedimentary rocks (Chen, 1988).

2.4 Composition of Clay Mineralogy

Clay minerals were first classified according to the size of their crystals. Minerals with particle sizes less that 0.02mm were identified as these minerals. The use of a petrographic microscope imposed this limit since the smallest particle that could be visually discerned was of this size. Clays were minerals that couldn‟t be processed in the way they were in the nineteenth century. Chemical examinations of fine grain size materials were performed, and the results were generally positive. The crystal structure and mineralogical family, on the other hand, remained largely unknown. This was primarily due to impurities in clay aggregates, which could be found as additional phases or in multiphase assemblages (Budhu, 2007).Clay soils have two primary molecular configurations in their lattice structure. These are the alumina octahedron and the silica tetrahedron. Silicate sheets are thin layers of silica tetrahedrons in which adjacent tetrahedrons share three oxygen ions. Alumina sheets are built up of alumina crystals, which have an aluminium ion in an octahedral arrangement surrounded by six oxygen or hydroxyl atoms (Craig, 2004 and Murray, 2007). Electrically negative tetrahedral sheets (also known as silica sheets) and electrically neutral octahedral sheets (also known as gibbsite sheets) Layer structures are created by joining a silica sheet with one or two gibbsite sheets. Although Kaolinite and illite are stacks of these layers with different sorts of bonding between them, Montmorillonite is the clay mineral particle that causes the most expanding damage (Craig, 2004).

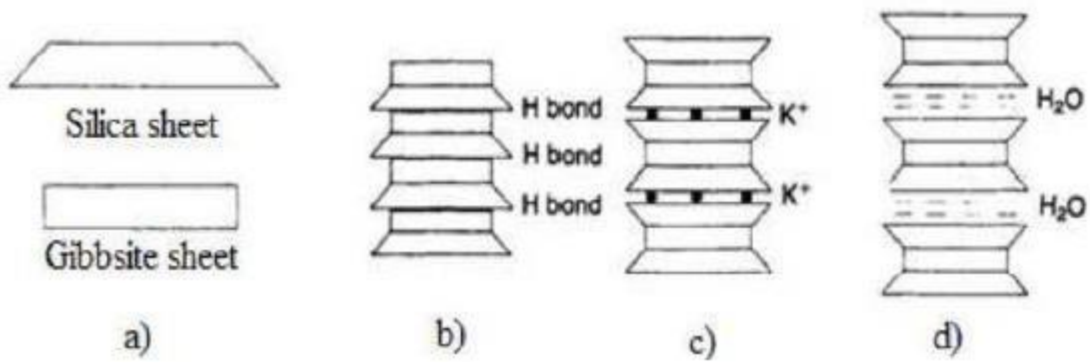

Figure 2. 1 Principal Clay minerals: a) Symbolic representation of the sheet structures; b) Kaolinite; c) Illite d) Montmorillonite (Craig, 2004)

2.5 Distribution of Expansive Soils in Ethiopia

Expansive soils are created by a combination of conditions and processes that lead to the formation of clay minerals with a certain chemical makeup that expand when exposed to water. Clay mineralogy is influenced by the parent material‟s composition as well as the amount of physical and chemical weathering the components have experienced. In Ethiopia, over 13.8 million hectares of land, which represents 12.5 % of the total land area, is covered by expansive soils (Negawo et al., 2017). The southern, south-east, and south-west parts of Addis Ababa, as well as the central portion of Ethiopia, are covered with expansive soils, which can be found along important trunk routes like Addis-Ambo, Addis-Woliso, Addis-Debre Berhan, Addis-Gohatsion, and Addis-Modjo. Parts of Mekele, Gondar, Bahirdar, Debreberihan, and Gambela have also been recorded to be partially covered by expanding soils (Zewdie, 2004).

2.6 Structural Damage by Expansive Soils

If there are no safeguards taken, expansive soils, which inflate when water is absorbed and shrink when removed, inflict serious damage to structures such as buildings, tunnels, pavements, and channels. As the position of the ground water table changes with the seasons, foundations resting on expansive soils may shift unevenly in the vertical direction and show signs of undesirable fractures. According to observations of structural unit deformations, both heave and settlement can occur in structures built on expanding clay. Upward heave is the most common structural deformation seen in structures. According to published data, heaving of the structure typically begins one year after construction. It should be emphasized that cracks are more prone to forming in lightly loaded structures than in heavily loaded buildings, as the former is simpler to move vertically due to underlying expansion soil pressure (Hussein, 2010).

2.7 Identification and Classification of Expansive Soils

Expansive soils can be identified using a variety of techniques. The methodologies for identifying and classifying expansive soils are classified into two categories: field identification and laboratory identification.

2.7.1 Field Identification of Expansive Soils

Expansive soils can be found in the field during both dry and rainy seasons, primarily during the reconnaissance and preliminary inquiry stages (Chen, 1988 and Murthy, 2003).The Following are some observations:

- They might be grey or black in color. Desiccated surfaces having open or closed fissures, as well as ground heave due to seasonal moisture variations.
- These soils become extremely sticky and difficult to navigate during rainy seasons.
- During dry seasons, shrinkage cracks appear on the ground surface, with a maximum width of 20 mm or more and extending deep into the ground.
- Cracks emerge in neighboring structures (mostly on building walls, foundation and grade beams, and longitudinal cracks appear at road shoulders & the centors.

2.7.2 Laboratory Identification

Mineralogical, direct, and indirect identification techniques are used in the laboratories to identify expansive soils.

a) Mineralogical Composition

Swelling potentials are influenced by the mineralogical composition of expansive soils. The mineralogy of expansive soils can be identified using five methods (Chen, 1988), which are listed in Table 2.1 Mineralogical analysis is the study of materials to determine mineral composition and mineral structure. This analysis can be used to identify mineral species, and understand their characteristics and properties. Mineralogical characteristics of expansive soils were examined using the X-ray diffraction (XRD) techniques can confirmed the presence of major non-clay minerals (quartz and calcite) and clay minerals (kaolinite, illite, and montmorillonite/smectite) in the expansive soils.

Table 2.1: Techniques for identifying the mineralogy of Expansive soils (Chen 1988)

Techniques	**Property and parameter determined**
X ray diffraction	The proportion of various minerals presented in colloidal clay
Differential thermal analysis	The control of materials which undergo characteristics change on heating. measure area and amplitude of reaction
Dye absorption	Identified by characteristics colors formed by des that are absorbed by the minerals of the soil sample.
Chemical analysis	Determine the nature of isomorphism and to show the origin and location of the charge on the lattice.
Electron microscope evaluation	Will show up distinct morphological characteristic‟s includes size and shape of clay particles

b) Indirect Methods

The indirect approaches, which include index properties, potential volume change (PVC) methods and activity methods, are the second method. These methods, as shown in Table 2.2, are important tools in evaluating the swelling potential of soils, according to Chen (1998).

Table 2.2 Indirect Methods of Expansive Soil Identification.

Tests	Properties investigated
Atterberge limit	Liquid limit, plastic limit plastic index and shrinkage limit
Potential of volume change(PVC)	Free swell(FS)= $(\frac{vf-vi)}{vi}$ * 100,vf = final volume in water, vi =initial volume in water, Free swell index(FSI)= $(\frac{vw-vk}{vk})$ * 100 , vw = $final\ volume\ in\ water$ Free swell ratio (FSR) = $(\frac{VW}{VK})$ * 100 vk, final volume in kerosene.
Activity method	Activity (AC)= $\frac{\text{plastic index}}{(\%\text{byweightfiner than } 0.02\text{mm})}$

C) Direct Method

The most satisfactory and convenient method of determining the swelling potential and swelling pressure of an expansive clay is by direct measurement. Direct measurement of expansive soils can be achieved by the use of the conventional one-dimensional consolidometer. Such a device enables an easy and accurate measurement of the swelling potential of clay under various conditions (Senthen, etal 1975).

2.8 Classification of Expansive Soils

A number of different classification schemes have been created using the parameters determined by expansive soil identification tests. General classification systems that have evolved over time largely based on performance correlation. According to index properties, soils are classified in two general schemes: The Unified Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials Method (AASHTO). Soils with a USCS classification of CL or CH and an AASHTO classification of A-6 or A-7 may be considered potentially expansive (Nelson and Miller, 1992). Other classification schemes have been developed specifically for expansive soil classification.

These systems, which include classification based on Chen (1988), Skempton, United States Bureau of Reclamation (USBR) Wiseman et al. (1985), IS 1498 (1970), Craig (1997) are based on indirect and direct predictions of swell potential, as well as combinations, to arrive at a rating.

Table 2.3 Classification of Expansive soil based on Skempton Method.

Degree of activity	Activity
Inactive clay	<0.75
Normal clay	0.75-1.25
Active clay	>1.25

Table 2.4 Classification of Expansive Soils based on USBR Method (1997)

Colloidal content<0.001mm	Plastic index	Shrinkage limit	Probable expansion percent total volume change	Degree of expansion
>28	>35	<11	>30	Very high
20-13	25-41	7-12	20-30	high
13-23	15-28	10-16	10-30	Medium
<15	<18	>15	<10	low

Table 2.5: Classification of Expansive soils according to Chen (1988).

The percentage passing No 200	Plastic index (%)	Percentage of swell	Swelling pressure (kPa)	Degree of expansion
.95	>55	>10	>1000	Very high
60-90	20-55	3-10	250-1000	high
30-60	10-35	1-5	150-250	medium
<30	0-15	<1	50	low

Table 2.6: Shrinkage limits and Degree of Expansion according to (IS1498-1970).

Shrinkage (%)	Linear shrinkage (%)	Degree of expansion
<10	>8	Critical
10-12	5-8	Marginal
>12	0.5	Non critical

Table 2.7: Atterberg limit results and Degree of Expansion according to Wiseman et al. (1985).

Index test	Usually non problematic	Almost always problematic
Plastic index	<20	>32
Shrinkage limit	>13	<10
Free swell	<50	>100

Table 2.8 Prediction of the degree of expansion in fine-grained soils based on IS (1970).

Free swell (%)	Degree of expansion
Less than 50%	low
50-100	medium
100-200	high
Greater than 200	Very high

Table 2.9: Classification of Soils based on free swell ratio based on Craig (1997).

Free swell ratio	**Expansion potential**	**Clay content**
<1	negligible	Non swelling
1-1.5	low	Mixture of non - swelling & swelling
1.5-2	moderate	swelling
2-4	high	swelling
>4	Very high	swelling

2.9. Stabilization of Expansive Soils

If good earth is not available at the construction site, it needs to be stabilized for use as an engineering material. Soil stabilization is the process of treating soil to maintain or improve the performance of the soil as a construction material. These treatments are generally classified into two processes: (1) soil modification and (2) soil stabilization. Soil stabilization is the process of blending and mixing materials with soil to improve certain properties of the soil.

The process may include the blending of soils with commercially available admixtures that may alter the gradation, texture, or plasticity, or act as a binder for the cementation of the soil. Soil modification is the stabilization process in which improvement in some properties of the soil occurs but does not result in a significant increase in soil strength and durability. Soil properties like strength and compressibility, Workability, swelling potential, and volume change tendencies may be altered by various soil stabilization and modification methods. Soil stabilization using chemical admixtures is the oldest and most widespread method of ground improvement. Chemical stabilization is the mixing of soil with one or a combination of admixtures of powder, slurry, or liquid for the general objectives of improving or controlling its volume stability, strength, stress-strain behavior, permeability, and durability (Winterkorn and Pamukçu, 1990). Soil improvement by means of chemical stabilization can be grouped into three chemical reactions; cation exchange, flocculation-agglomeration, and pozzolanic reactions.

2.9.1 Cation Exchange

The excess of ions of opposite charge (to that of the surface) over those of like charge present in the diffuse double layer are called exchangeable ions. These ions can be replaced by a group of different ions having the same total charge by altering the chemical composition of the equilibrium electrolyte solution (Winterkorn and Pamukçu, 1991). Negatively charged clay particles adsorb cations of a specific type and amount. The ease of replacement or exchange of cations depends on several factors, primarily the valence of the cation. Higher valence cations easily replace cations of lower valence. For ions of the same valence, the size of the hydrated ion becomes important; the larger the ion, the greater the replacement power. If other conditions are equal, trivalent cations are held more tightly than divalent cations, and divalent cations are held more tightly than monovalent cations (Mitchell and Soga, 2005). A typical replaceable series is.

$$Na^+ < Li^+ < K^+ < Rb^+ < Cs^+ < Mg^{2+} < Ca^{2+} < Ba^{2+} < Cu^{2+} < Al^{3+} < Fe^{3+} < Th^{4+}$$

The exchangeable cations may be present in the surrounding water or be gained from the stabilizers. An example of the cation exchange (Sivapullah, 2006).

$$Ca^{2+} + Na^+\text{-Clay} \rightarrow Ca^{2+}\,\text{Clay} + (Na^+)$$

The thickness of the diffused double layer decreases as replacing the divalent ions (Ca2+) from stabilizers with monovalent ions (Na+) of clay. Thus, swelling potential decreases.

2.9.2 Flocculation and Agglomeration

Cation exchange reactions result in the flocculation and agglomeration of the soil particles, with a consequent reduction in the amount of clay-size materials and hence the soil surface area, which inevitably accounts for the reduction in plasticity (Terzaghi and Peck, 1967). Due to the change in texture, a significant reduction in the swelling of the soil occurs.

2.9.3 Pozzolanic Reactions

Time-dependent pozzolanic reactions play a major role in the stabilization of the soil since they are responsible for the improvement in the various soil properties (Show et al., 2003). Pozzolanic constituents produce calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH).

$$Ca^{2+} + 2(OH)^- + SiO_2 \text{ (Clay Silica)} \rightarrow CSH$$

$$Ca^{2+} + 2(OH)^- + Al_2O_3 \text{ (Clay Alumina)} \rightarrow CAH$$

The calcium silicate gel formed initially coats and binds lumps of clay together. The gel then crystallizes to form an interlocking structure; thus, the strength of the soil increases (Hadi et al., 2006; Sivapullaiah, 2006). Chemical additives and mechanical stabilization techniques are commonly used to improve the engineering properties of expansive soils. Traditional stabilizers like hydrated lime, Portland cement, and fly ash; non-traditional stabilizers like sulfonated oils, ammonium chloride, enzymes, polymers, and potassium compounds; and byproduct stabilizers like cement kiln dust and lime kiln dust, among others. Chemical stabilizers" effectiveness is influenced by the soil"s physical and chemical characteristics. The mineralogy of fine-grained soil particles, as well as soil plasticity and grain size distribution, are among these features. Lime can be used to effectively stabilize soils that have high plasticity and a large proportion of fine-grained soil particles (Bell, 1996). Mechanical stabilization is a process that improves the soil"s stability and shear strength without changing its chemical properties. Compaction, mixing or blending of two or more gradations, applying geo-reinforcement, and mechanical remediation are the main methods of mechanical stabilization (Little and Nair, 2009; Guyer, 2011; and Makusa, 2012).

Expansive soils, according to Nelson and Miller (1992), Christopher (2005), Hussein (2010), and Guyer (2011), pose a serious threat to civil engineering infrastructure around the world due to moisture fluctuation.

2.9.4 Soil Stabilization using Lime

Lime, cement, and a combination of the two are commonly used as soil stabilizers in many countries. On the other hand, cement stabilization is more expensive than lime stabilization and more difficult to apply to fine-grained clays. In granular soils, lime is far more effective (Mc Keen, 1976). Lime is a widely used additive to improve the properties of expansive soils. Clay soils can be stabilized with lime in the form of quicklime (calcium oxide – CaO), hydrated lime (calcium hydroxide Ca [OH]2), or lime slurry. Quicklime is generated by chemically converting calcium carbonate (limestone-CaCO3) into calcium oxide. Hydrated lime is also generated when quicklime chemically interacts with water. Lime stabilization results in long-term changes in clay properties (LMA, 2004). The chemical theory involved in the lime reaction is complex (Thompson, 1966, 1968). The main reactions include cation exchange, flocculation, and pozzolanic reactions (Nelson and Miller, 1992). These three stabilization steps are valid for the stabilization of expansive soils using waste limestone dust and waste dolomitic marble dust.

2.9.5 Soil Stabilization Using Waste Marble Dust

Many researchers have reported that marble has a very high lime (CaO) content, up to 55% by weight (elik and Sabah, 2007; Zorluer and Usta, 2003; Oates, 1998; Almedia et al., 2007; Tegethoff, 2001; Okagbue and Onyeobi, 1999). Thus, the stabilization characteristics of waste limestone dust and waste dolomitic marble dust are mainly due to their high lime (CaO) content. It is an effective stabilizer. On the other hand, in the development of the construction industry, the marble and granite industry is one of the most environmentally unfriendly industries. Cutting the stones produces heat, slurry, rock fragments, and dust. In addition, it affects soil fertility and reduces permeability. Fine particles, in particular, can cause more pollution than other types of marble waste because they are easily dispersed after being lost in some atmospheric conditions, such as wind and rain, unless properly stored in sedimentation tanks and further utilized.

2.9.6 Availablity of Marble waste in Ethiopia

Extensive deposits of marble are found in the Precambrian metamorphic terrain of northern and western Ethiopia (T. Heldal, H. Wale, and S. Zewdie, 1987; T. Heldal, H. Wale, 2000). Based on their suggestion, the country of Ethiopia can export marble, and its present domestic demand for the mineral product is estimated at 211,252 sq. m. per annum. During this time, there will be 34 marble processing companies in Ethiopia. Waste marble dust can be produced by any rock that can be polished in marble plants (Oates, 1998). In business (Onargan et al.2005). Industrial wastes (by-products) can be used solely or as admixtures so that natural sources are used more efficiently and the environment is protected from waste deposits. The increasing demand for marble products in the construction industry raises the generation of waste marble dust. During the cutting process of marble blocks, the marble dust and water mix together and become waste marble. Based on an Ethiopian mining company, about 25–30% of waste marble is marble dust. The production of fine particles are less than 2 mm while cutting marble is one of the major problems for the marble industry. When a 1 m3 marble block is cut into 2 cm thick slabs, the proportion of fine particle production is approximately 25 % (Kun, 2000). While cutting marble blocks, water is used as a cooler. But, the fine particles can be easily dispersed after losing humidity under atmospheric conditions such as wind and rain. Fine particles can thus pollute the environment more than other types of marble waste (cited in Eli and Sabah, 2007).

2.9.7 Previous Works on Stabilization of Expansive Soil

Okagbue and Onyeobi (1999) evaluated three different red tropical soils with varying proportions of MWP (0%, 2%, 4%, 6%, 8%, 10%) and after 28 days of curing of the treated soils. Results showed that soil plasticity was reduced by 33%, shear strength increased by 46%, and the California Bearing Ratio (CBR) increased by 55%. Based on this Results we showed the highest improvement at 8% MWP content and that 28-day curing improved results by over 80%.

Patel and Bhavsar (2014) stabilized black cotton soil by replacing 30% of the soil with marble waste powder (MWP) and brick dust (BD). Results for MWP gave a reduction of 10% in swelling, 17.7% in shrinkage, 4.08% in plasticity, and 5.72% in optimal moisture content (OMC) with an increase of 11.69% in maximum dry density (MDD). Similarly, for BD, there was a reduction of 10% in swelling, 14.3% in shrinkage, 5.57% in plasticity, 5.72% in OMC, and an increase of 11.69% in maximum dry density.

Ali, R. (2014) investigated the effects of MWP and Bagasse Ash (BA) on the stabilization of expansive soils at 4%, 8%, and 12%. The addition of 12% MWP decreased soil uplift pressure from 9.02 to 5.56 psi, while the addition of 12% BA decreased soil uplift pressure from 9.02 to 4.72 psi. MDD increased with the addition of MWP and BA and reached its maximum approximately at 8% addition but declined with the addition of 12% marble waste product and bagasse ash.

Perlindh (2004) investigated the compaction of stabilized soil is important to achieve good quality and to obtain the desired service life of the stabilized material. Stabilization changes the compaction properties to give a material that needs more compaction energy compared to untreated soil to achieve the same dry density.

Minhas A (2016) studied the effect of MWP on the stabilization of alluvial soils. When there was no mix in the soil, OMC was 8%. But by the addition of MWP, the OMC got up to 12% steadily with all the percentages of MWP (5%, 10%, and 15%). CBR test results also improved and bearing capacity decreased if MWP was added.

Singh and Arora (2017) added rice husk (RH) ash, fly ash (FA), and MWP to plastic soil. They found that the optimum percentage of RH, FA, and MWP in the stabilization of soil is found to be 10%, 8%, and 20%. The soaked unconfined compression strength (UCS) of RH increased up to 20% with the addition of MWP. The soaked UCS decreased as more MWP was added, while it increased by 14% as more FA content was added.

Magdi, M. (2018) evaluates the use of marble waste powder (MWP) to stabilize expansive clay. Soil samples were mixed with varying proportions of MWP (0%, 10%, 15%, and 20% by dry weight). Laboratory experiments including sieve analysis, Atterberg‟s limits, Standard Proctor, Unconfined Compression Strength (UCS), and Free Swell were performed on treated and untreated soils. The results revealed that the addition of 20% MWP to the soil significantly reduced the soil plasticity and free swell index by almost 12%, while the UCS greatly increased by almost 3.5 times the initial value. It‟s concluded that the use of MWP will improve the expansive soil properties and is beneficial for economic and environmental considerations.

Mada (2016) investigated the performance of marble dust to improve the problematic nature of expansive soils, which are characterized as A-7-5 according to the AASHTO classification. The expansive subgrade soil is blended with an increasing percentage by weight (5% to 30%) of marble dust. The test results indicate that the swelling potential of the natural soil changed from high to medium, CBR increased from 0.9% to 2.25%; the CBR swell reduced from 8.6% to 5.3%, LL reduced from 88% to 63%, PI reduced from 52% to 34%. The natural subgrade soil is stabilized with 30% marble dust.

An un-soaked CBR test is conducted and it has been noted that the subgrade class improves. This results in a significant reduction in project cost due to a reduction in pavement thickness.

Shitaye (2020) investigated eucalyptus wood ash as a stabilizer for expansive soil in Woreta town. The soil was stabilized with 5%, 10%, 15%, and 20% wood ash by weight of dry soil. In order to evaluate the soil samples, atterberg limit, free swell, compaction, California Bearing Ratio, and unconfined compressive test tests were conducted. Based on the result, plastic limit, specific gravity, unconfined compressive strength (UCS), CBR value, and the optimum moisture content (OMC) are directly proportional to the addition of wood ash. But after 15% of wood ash, The added UCS and CBR values are decreased. On the other hand, the liquid limit, plastic index, linear shrinkage, free swell, MDD, and CBR swell values of treated soils are inversely proportional to the percentage of wood ash.

Negussie and Dinku (2014) investigated the performance of engineering expansive soil using 6% lime content and found that this was an optimum lime content based on Ph test reduction in PI. The reduction of PI decreased with increasing curing duration due to carbonation reaction aggravated by premature wetting of samples during soaking. Sodium silicate reduced PI values associated with increased curing duration. Maximum reduction of PI occurred at 4% lime and 1% sodium silicate cured for 28 days. MDD increased and OMC decreased with increased lime content Neither sodium silicate nor applying sodium silicat in combination with lime is a suitable means of expansive (montmorillonitic) clay stabilization.

Wubshet and Tadesse (2014**)** investigated the effects of lime and bagasse ash on the engineering properties of the soil with 3% lime and 15% bagasse ash for 7-day curing periods. The results were an increase in OMC and CBR values; and a decrease in MDD and plasticity of the soil for all additives. But there was also a tremendous improvement in the CBR value when the soil is stabilized with a combination of lime and bagasse ash.

2.10 Unsaturated soils

In recent decades, a theoretical framework for unsaturated soil mechanics has emerged. According to a popular definition, solids, water, and air are the three phases of an unsaturated soil. However, recognizing the existence of a fourth phase, the air-water interface or contractile skin, may be more accurate. (Fredlund and Morgenstem,1977). In geotechnical engineering, the constitutive equations for volume change, shear strength, and flow for unsaturated soil have been widely recognized (Fredlund and Rahardjo 1993a). According to Fredlund and Rahardjo (1993), many materials encountered in engineering practice behave in ways that contradict the assumptions and concepts of classical, saturated soil mechanics. In most cases, the existence of more than two phases results in a difficult-to-work-with material in engineering practice. Unsaturated soils are the most common type of material that does not behave according to traditional, saturated soil mechanics. The majority of the soil in the Bahir Dar area is composed of residual fine soils such as clay and silt-clays built on basaltic bedrock. Their negative pore-water pressures, which are considered unsaturated soils, are the key factors contributing to their peculiar behavior. When the influence of matric suction is taken into account, the engineering behavior of collapsible, residual, compacted, and expansive soils that are often unsaturated can be better understood (Fredlund, 2000). The SWCC for a given soil and the properties of the unsaturated soil have been linked in laboratory investigations (Fredlund and Rahardjo, 1993b). Bearing capacity, volume change, permeability, and shear-strength functions are examples of these types of properties. It is critical to gain a better understanding of how unsaturated soil behaves and works. Effective stress (-uw) governs soil behavior below the water table, whereas two separate stress factors, net normal stress (-ua) and matric suction (ua–uw), influence soil behavior above the water table (Jennings and Burland, 1962; Fredlund and Morgenstem, 1977). The soil is at or near saturation condition and behaves as if it is saturated at low matric suction, where the suction is less than the air-entry value of the soil. The soil begins to desaturate at matric suction greater than the soil‟s air-entry value (Thamer et al. 2006). The characterization of the engineering behavior of unsaturated soil is totally dependent on the soil-water characteristic curve (SWCC), which is a graphical relationship between water content (either gravimetric or volumetric) or degree of saturation and soil suction. Different SWCC models can be used to fit the experimental SWCC data (Malaya and Srideep,2010).

Using multiple regression analysis, equations were derived for the use of different parameters based on predictors derived from GSD and PI(grain size distribution and plastic index). The soil water characteristic curve (SWCC) is often predicted from the pore size distribution (PSD) of the soil. It is necessary to consider the pore-size distribution changes in predicting SWCCs. (Li and Zhang, 2007).

2.10.1 Role of Climate on Unsaturated soils

The climate has a big impact on whether a soil is saturated or not. Evaporation from the ground surface or evapotranspiration from a vegetation cover remove water from the soil. These processes cause water to flow upward from the earth. Rainfall and other forms of precipitation, on the other hand, provide a downward flow into the soil.

The pore-water pressure conditions in the soil are mostly determined by the difference between these two flux conditions on a local scale. A net upward flux causes the soil mass to gradually dry, crack, and desiccate, whereas a net downward flux eventually saturates the soil mass. The net surface flux, among other things, influences the depth of the water table. A hydrostatic line drawn parallel to the groundwater table denotes a state of equilibrium in which no flux exists at the ground surface. The pore-water pressures become more negative during dry periods. During wet periods, the situation is the polar opposite (Fredlund and Rahardjo1993).According to Fredlund and Rahardjo (1993), arid and semi-arid areas have a deep groundwater table. The pore-water pressures are negative in soils above the water table. Due to excessive evaporation and evapotranspiration, the soils have become de saturated. Climatic variations have a significant impact on the water content of the soil near the ground surface.

2.10.2 Phases of Unsaturated Soils.

According to Fredlund and Rahardjo (1993), an unsaturated soil consists of three phases:a) solids, b) water, and c) air. However, recognizing the existence of a fourth phase, namely the air-water interface or contractile skin, may be more effective. The contractile skin interacts with the soil particles and influences the soil‟s mechanical behavior while the air phase is continuous. The fluid becomes highly compressible when the air phase contains occluded air bubbles.

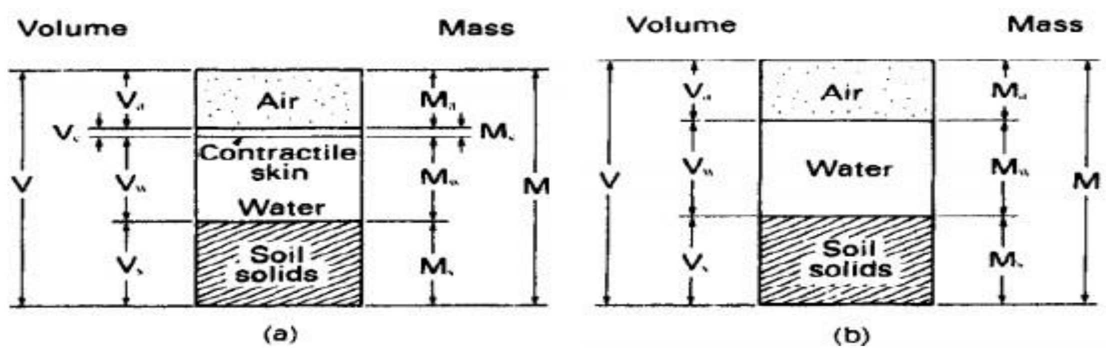

Figure 2.2 (a) four phase unsaturated soil system; (b) simplified three phase diagrams (Fredlund and Rahardjo, 1993*).*

2.10.3 Unsaturated Soil in Engineering Practice

For many years, unsaturated soils were either neglected in civil engineering design and construction analyses, or they were handled incorrectly from a saturated soil mechanics perspective. However, in the last 30 to 40 years, rapid advances in our understanding of unsaturated soil behavior have prompted today‟s civil engineers to recognize that there is now an opportunity to tackle problems involving unsaturated soil on a much more rational basis. The growing body of information about the underlying principles of unsaturated soil mechanics is progressively being applied to a wide range of real-world engineering issues (Ning and William, 2004). Stress-related and deformation-related issues are two common categories of engineering challenges affecting primarily unsaturated soils. Slope stability and land sliding under changing climatic conditions, lateral earth pressure and retaining structure stability, excavation and borehole stability, bearing capacity for shallow foundations under moisture loading, and stress wave propagation in unsaturated soil are among those stress-related problems. Deformation problems included swelling and shrinkage of expansive soil, desiccation cracking of clay, collapsing soil, soil compaction, and consolidation and settlement of unsaturated soil (Ning and William, 2004). Soil types such as saturated sands, silts, clays, and dry sands have been the focus of traditional soil mechanics. Theories relating to these types of soils in either a completely dry or completely saturated state were widely researched. It has recently been demonstrated that soils that do not fit into these categories require special consideration. Unsaturated soils account for the majority of these soils. Because of the complexities of their behavior, engineering with unsaturated soils has always been empirical. The natural laws controlling the behavior of an unsaturated soil are altered since it contains more than two phases. The connection between water and air as the soil desaturates is crucial to the behavior of an unsaturated soil. The SWCC describes this relationship. According to laboratory experiments (Fredlund and Rahardjo 1993), there is a link between the SWCC and unsaturated soil properties.

2.10.4 Soil Water Characteristic Curves (SWCC)

SWCC is the relationship between matrix suction and water content, and it reflects the capacity of holding water under the matrix suction (Tan et al., 2005). The SWCC describes the relationship between gravimetric water content, *w*, or volumetric water content,, or degree of saturation, *S*, and soil suction (Fredlund and Xing, 1994). Matric suction is the difference between pore air pressure and pore water pressure. For common geotechnical engineering practice, pore air pressure is equal to atmospheric pressure and is considered zero. For unsaturated soils, pore water pressure is always negative (less than atmospheric pressure). This negative pore water pressure is termed suction (Fredlund and Xing, 1994). Much important information about the soil body, such as permeability, strength, volume change, stress state, and granular distribution, is obtained from SWCC (Zhou, 2005).

The SWCC of unsaturated soils is the main content of its constitutive relations (Chen, 2001; Chen et al., 2003). The SWCC of unsaturated soil has great significance in explaining and predicting the engineering characteristics of unsaturated soil, including permeability, seepage flow, strength, and volume change (Zhou, 2005).

2.10.4.1 Nature of soil water characteristics curve

The SWCC is a hysterical relationship, not a single-valued, unique relationship. The non-uniformity of pore-size distribution in the soil causes hysteresis in the SWCC. As a result, a single stress state designation for a soil cannot be determined solely by measuring its water content. In other words, along what is known as a scanning curve (shown in Figure 2.3(a) below), it is impossible to tell whether the soil is now on the drying curve (desorption), the wetting curve (adsorption), or somewhere in between the two bounding curves. It is clear that the adsorption curve"s finishing point differs from the desorption curve"s beginning point this is due to air trapped in the soil (Fredlund and Xing, 1994). The effect of hysteresis is ignored due to its complexity, and only desorption SWCC is employed to estimate soil suction (Fredlund et al., 2011).

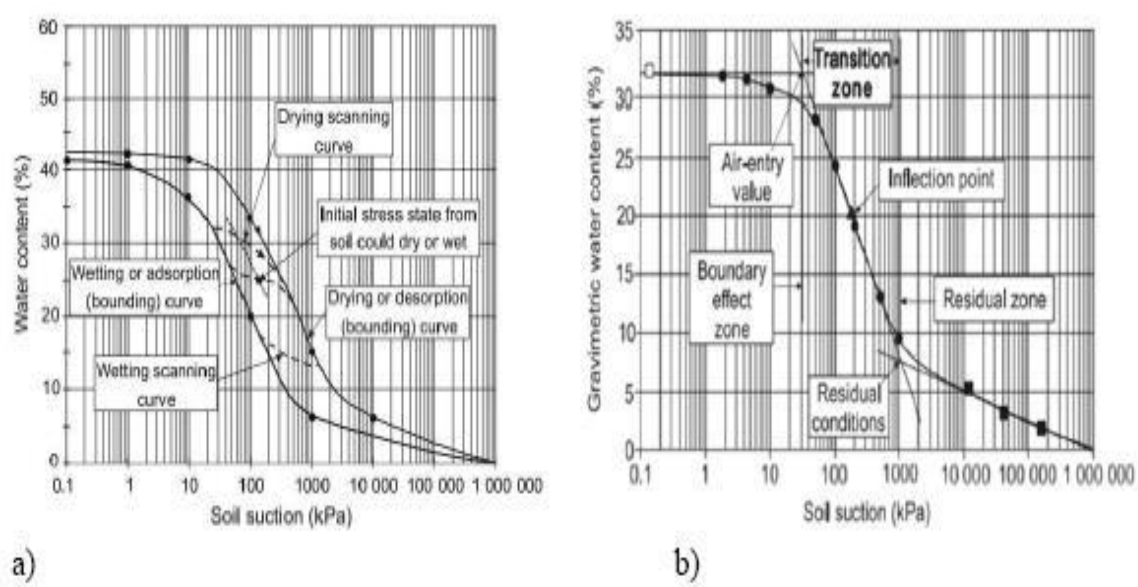

Figure 2.3 a) Effect of hysteresis on SWCC (Fredlund 2000) & b) Desorption of SWCC (Fredlund et al.2011).

There are two major variations in slope along the SWCC, as represented by the curve in Figure 2.3 (b). As suction is increased, the first point is referred to as the soil‟s “air-entry value,” where the largest voids begin to desaturate. The second stage is known as “residual conditions,” and it refers to the point at which water removal from the soil becomes much more difficult. (i.e., it requires significantly more energy for water removal). The SWCC is divided into three zones by variations in slope: the „„„boundary effect zone” in the lower suction range; the „„„transition zone” between the air-entry and residual values; and the „„„residual zone” at high soil suctions up to 106 kPa (Fredlund et al., 2011). The water content at zero matric suction is known as the saturated water content, and it represents the complete ability of the soil pores to hold water (Fredlund and Xing, 1994). The soil storage potential is represented by the ratio of change in soil matric suction to water content. To put it another way, the steepness of the slope throughout a range of soil suctions represents the soil storage potential (Leong and Rahardjo, 1997). The behavior of SWCC on a semilogarithmic scale, Figure 2.4 shows standard SWCCs for sandy, silty, and clayey soils. The plasticity and percentage of fines of the soil enhance the saturated water content and the air-entry value. The rate of desaturation reduces as the fines of the soil rise. As a result, a soil with high plasticity can hold water even at higher matric suctions. The shape of the SWCC is further influenced by the initial water content, density, stress history, soil state and structure, and pore size distribution (Fredlund and Xing, 1994).

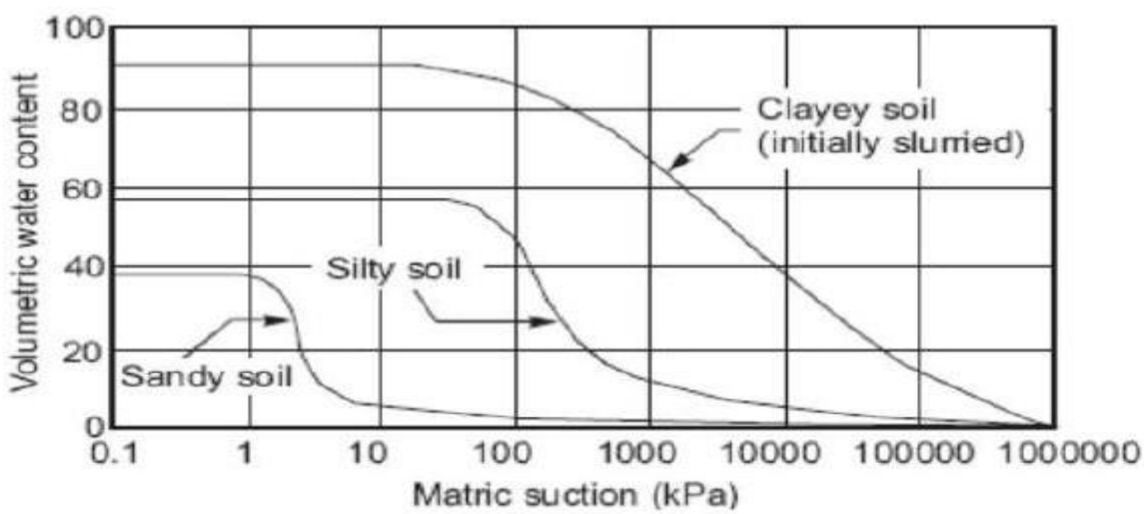

Figure 2.4 Soil Water Characteristic Curve for a Sandy Soil, Silty Soil and Clayey Soil (Fredlund and Xing, 1994).

2.10.4.2 Factor that Affecting the Soil-water Characteristic curve

The soil-water characteristic curve (SWCC) gives the relationship between the amount of water in the soil (i.e., gravimetric or volumetric water content) and soil suction (i.e., matric suction at low suction and total suction at high suction). Many properties of a partially saturated soil, such as the coefficient of permeability, shear strength and volume strain, pore size distribution, the amount of water contained in the pores at any suction, can be obtained from the SWCC. However, many factors that influence the solid water characteristics curve, such as soil structure (and aggregation), initial water content, void ratio, soil type, mineralogy, and compaction method, can have a significant impact on SWCC features. (Zhou and Jian 2005). The grain-size-distribution (GSD) of a soil is intimately related to its pore size distribution, and hence, the GSD holds a close relationship with the soil-water characteristic curve (SWCC). In addition, the plasticity index (PI) is a measure of the water holding capacity of the soil, and therefore, it plays an important role in shaping the SWCC.

a) Effect of grain size distribution on SWCC

A soil with a uniform grain-size distribution (the steeper slope in grain-size distribution) has less hysteresis and a greater slope of drying SWCC than that of a non-uniform soil. The soil mixtures were gap-graded in grain-size distribution and bimodal characteristics, but their SWCCs did not exhibit a bimodal characteristic. The gap-graded nature of the grain-size distribution of the mixtures did not affect the conventional sigmoidal shapes of the SWCCs. The pore-size distribution of the mixtures indicated that large pores within the compacted mixtures were filled with fine-grained soil. The air-entry value of the compacted mixtures was found to be high due to the high dry density of the specimens. The different particle sizes of the soil are used in the evaluation of the air-entry value of the mixtures. The dry density and coefficient of uniformity of the grain-size distributions were found to affect the air-entry values of the mixtures and the SWCC fitting parameters n and m, respectively (Chaminda Pathma 2020).

b) Effect of initial water content on soil water characteristics curve

Soil water characteristics curve and hydraulic conductivity depend on the soil and are strongly influenced by the initial water content and the dry density. The influence of the compaction conditions on the microstructure of compacted clays was carried out by Lambe (1954) and Seed & Chan (1959).

They showed that the pore size distribution depends strongly on the initial water content. The soil is compacted to a wet optimum; the Proctor test usually does not distinguish between intra- and inter-aggregate pores. Matric suction arises mainly from capillary forces. The SWRC of compacted fine-grained soils reflects the pore size distribution. Hence, the SWRC is strongly influenced by the initial water content (Vanapalli et al. 1999, Tinjum et al. 1997, Tarantino & Tombolato 2005, Romero et al. 2011) as well as by the initial dry density (Tinjum et al. 2014, Romero et al. 1999, Miller et al. 2002, Sun et al. 2006, Miao et al. 2006.

Samples were compacted and prepared with the required initial water content and dry density as shown below. The gravimetric water content and suction value are different. The drying paths of the samples were compacted at the same initial water content and differed only in the low suction range at matric suctions that are smaller than 1500 kPa.

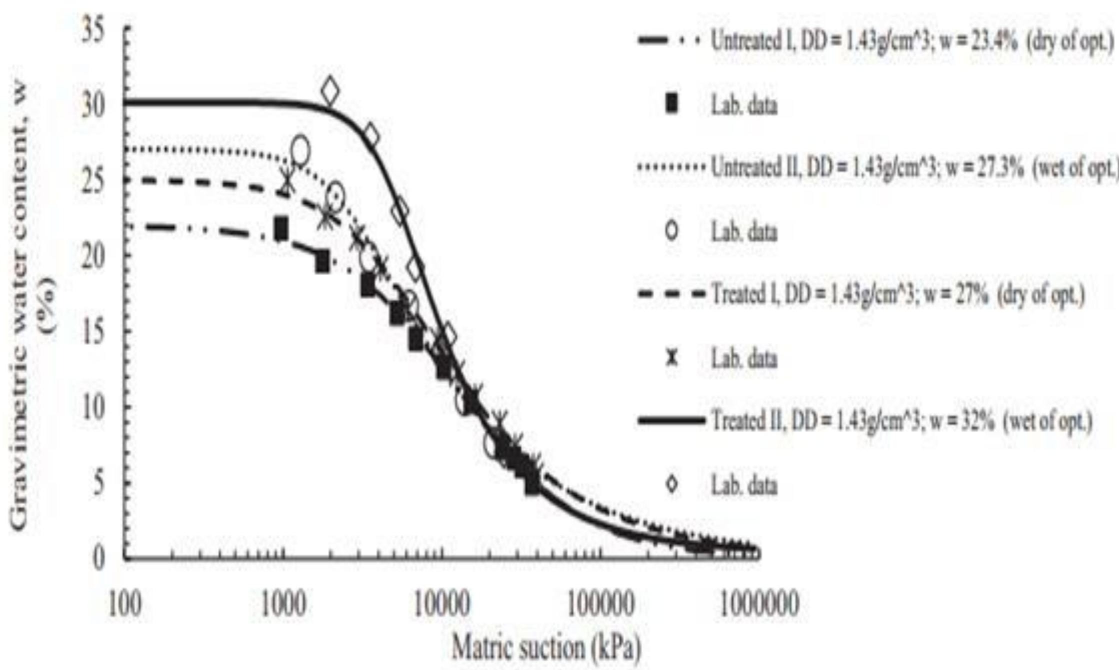

Figure 2.5 effect of initial moisture content on SWCC of stabilized soil (Zhang et al 2017)

The initial water content has a considerable influence on the shape of SWCC curves, which means that samples with higher initial water content on the SWCC have a steeper curve. The air-entry value also increases with initial water content. The resistance to de-saturation is relatively low in the dry of optimum specimens in comparison to the optimum and wet of optimum specimens. So, for soils of high initial water content, the effect of de-saturation is more obvious, especially at low suction values.

Previous studies on the measurement of soil–water characteristic curves (SWCC) of soil mixtures with two distinct pore-size distributions showed that the measured soil–water characteristics were bimodal (Burger and Shackelford, 2001; Zhang and Chen, 2005).

C) Soil Type and Compaction Conditions on Soil Water Characteristic

The determination of unsaturated soil behavior was investigated for conditions covering a range of compactive efforts and water content. Tests were conducted to determine the variation of water content and pore water suction for compacted soils.

The soils had varying amounts of clay fraction with plasticity ranging from low to high plasticity. The experimental data were fitted to common models for the water content-pore water suction relationship. However, the individual parameters obtained from the curve fits varied significantly between models. The soil water characteristic curves, SWCCs, were more sensitive to changes in compaction effort than changes in compaction water content. At similar water contents, the pore water suction increased with increasing compaction effort for each compaction condition and soil type. For all compaction conditions, the lowest plasticity soils retained the smallest water content and the highest plasticity soils retained the highest water content at a specified suction (Miller et al., 2002, Yaldo, 1999). The SWCC is a typical curve that describes the relationship between water content and pore water suction for unsaturated soils. It has several defining parameters, including air-entry suction head, residual water content (ɵr), and saturated water content (ɵs). It can also be seen that when the matric suction is relatively low (lower and stays at a certain value, water content is larger when the compaction degree is lower.

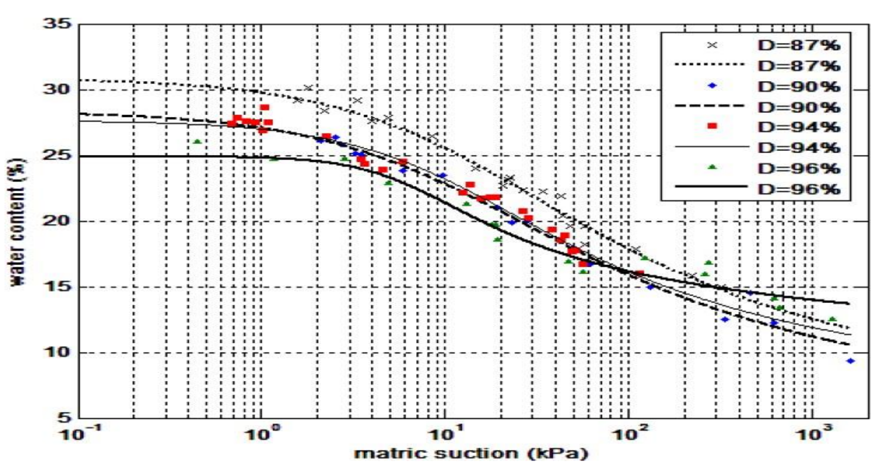

Figure 2.6 Fitted SWCC based on degree of compaction (Fredlund 1994)

d) Influence of clay Mineral content on SWCC

The influence of clay content on the SWCC of sand–natural clay mixtures was marked by an increase in water retention capacity with increase in clay content ranging between 0% and 60% (Elkade2015). This increase in water retention capacity was attributed to the formation of micro pores due to the addition of clay. The chemical behavior is mainly attributed due to the presence of clay minerals such as Montmorillonite and is hypothesized that there is a strong correlation between soil water–suction relationship and percent clay mineral such as Montmorillonite (Pedarla et al. 2016). According to Pedarla (2016) high plasticity expansive clays are selected and studied on the mineralogy composition of unsaturated soil properties specially montmorillonite clay has great effect on SWCC. The clayey soils with higher percentages of Montmorillonite content exhibited higher swelling behavior with affects the soil water retention capacity.

2.11 Model of soil water characteristics curve

SWCC has been studied by a number of researchers around the world. Various SWCC models have also been created. Brooks and Corey (1964), Van Genuchten (1980), and Fredlund and Xing (1994) are among others who can be mentioned the popular model. In this study the SWCC is predicted using the Fredlund and Xing (1994) model since his model serve as wide range of soil to wide range of suction.

a) Brooks and Corey SWCC Fitting Model

SWCC is supposed to be an exponentially decreasing function of water content, whereas at suctions smaller than the air-entry value, it remains constant. Furthermore, the model is in effective when the degree of saturation is smaller than the residual value (Brooks and Corey, 1964). According to the findings, the model is more suited to coarse-grained soils than fine-grained soils (Fredlund and Xing, 1994).

$$\Theta = \frac{\theta - \theta_r}{\theta_s - \theta_r} = \left(\frac{\psi}{a}\right)^{-n} \text{ when } \psi > a \qquad (2.1)$$

$$\Theta = 1 \text{ and } \theta = \theta_s \text{ when } \psi < a \qquad (2.2)$$

where

Θ : normalized water content; ψ : soil suction; θ : volumetric water content;

θ_r :residual volumetric water content; θ_s : saturated volumetric water content;

a and n :equation parameters

Here **'a"** is related to the air-entry value, which is the suction required to remove water from the largest pores or matric suction for which air starts to enter largest pores in the soil. The pore size distribution of the soil is related to **'n"**. The greater the value of „n," the more homogeneous the pore sizes in the soil are, and the SWCC within the desaturation zone is steeper.

b) Van Genuchten (1980) SWCC Fitting Model

This model is commonly used for modeling and understanding the behavior of unsaturated soils. The model is continuous, fitting the SWCC over the complete range of soil suction, using fitting parameters a, n, and m. The SWCC of expansive soil found in Bahir Dar, according to Amlak (2020), never reached zero water content at a maximum suction of 106 kPa by the Van Genuchten (1980) SWCC fitting model expressed normalized water content or effective saturation.

$$\Theta = \frac{\theta - \theta r}{\theta s - \theta r} \left\{ \left[\frac{1}{1 + \left(\frac{\varphi}{a}\right)^n} \right] \right\}^m \qquad \textbf{2.3}$$

Θ = normalized water content

θ r = residual water content

θs = saturated water content

θ = volumetric water content

a,n,m are equation parameter

Equation 2.3 and Equation 2.2 employ the same definitions for the above parameters, with the exception of m, which is connected to the model"s asymmetry.

c) Fredlund and Xing (1994) SWCC Fitting Model

In terms of three parameters identified as **a, n**, and **m**, the Fredlund and Xing (1994) model formulation provides a continuous SWCC as a function of gravimetric water content over the entire soil suction range of 0 – 10^{6} kPa.

The following is a representation of the model equation (Equation 2.4). The desorption curve is the focus on this paper. And Fredlund and Xing (1994) model is used to fit the curves.

$$\omega(\psi) = c(\psi)\frac{\omega_s}{\left\{\ln\left[e+(\psi/a)^n\right]\right\}^m} \tag{2.4}$$

Where $\omega\varphi$= gravimetric water content at any soil suction, ψ, e= irrational constant equal to 2.71 ωs=saturated gravimetric water content, a = fitting parameter indicating the inflection point that bears a relationship to air entry value and is greater than the air-entry value, n= fitting parameter related to the rate of desaturation, m = fitting parameter related to the curvature near residual conditions and φ= correction factor directing the SWCC to 106 kPa at zero water content, and given by:-

$$C(\psi) = 1 - \frac{\ln(1+\psi/\psi_r)}{\ln\left[1+(10^6/\psi_r)\right]} \tag{2.5}$$

$\varphi\gamma$ = is the suction corresponding to the residual water content, ωr.When utilizing the volumetric water content and degree of saturation to fit the curve, the term ($\omega\varphi$ ωs) in Equation 2.4 will be substituted by $\theta\omega,\ \theta s$ and S(ψ) and So, respectively Sillers et al. (2001) found that the Fredlund and Xing (1994) equation is well ω fitted for experimental data for various soils over a wide range of suction and requires fewer iterations to reach the best–fit parameters than Van Genutchen‟s equation (1980). Table 2.10 lists some of the most generally proposed soil-water characteristic curve formula by Sillers (1996) presents a more extensive set of equations that could be used.

Table 2.10 Summary of SWCC fitting equations from Sillers (1996).

Name	Equation Description	Fitting parameters
Gardner (1958)	$w(\psi)=\frac{w_s}{1+a\psi^n}$	a, n
Brooks and Corey (1964)	$w(\psi)=w_s$ for $\psi>\psi_{ae}$ $w(\psi)=w_s\left(\frac{\psi}{a}\right)^{-n}$ for $\psi\leq\psi_{ae}$	a, n
Brutsaert (1966)	$w(\psi)=\frac{w_s}{1+\left(\frac{\psi}{a}\right)^n}$	a, n
van Genuchten(1980)	$w(\psi)=\frac{w_s}{\left[1+(a\psi)^n\right]^m}$	a, m, n
Van Genuchten-Mualem (1980)	$w(\psi)=\frac{w_s}{\left[1+(a\psi)^n\right]^{(1-1/n)}}$	a, n
Genuchten-Burdine (1980)	$w(\psi)=\frac{w_s}{\left[1+(a\psi)^n\right]^{(1-2/n)}}$	a, n

2.12 Suction Measurement device

Several measurement techniques are available to measure the suction of a soil sample. The method selected should depend upon the suction desired. Different methods are used to determine the total suction and matric suction, respectively. In this study, the pressure plate extractor is used for suction measurement among different techniques. The direct method for measuring matric suction and the indirect method for measuring matric, osmotic, and total suction are the two basic types of suction measurements (Pan et al., 2010). The axis translation technique, the tension meter, and the suction probe are examples of direct approaches. Time domain reflect meters, electrical conductivity sensors, thermal conductivity sensors, and in-contact filter paper technology are all indirect methods of detecting matric suction. Matric suction is due to the surface tension forces present in unsaturated soils (surface tension effects are also referred to as capillarity) at the interfaces (menisci) between the water and the gas (usually air) phases.

Total suction is also defined as the negative pressure which must be applied to a pool of pure water at the same elevation and temperature in order for it to be in equilibrium with the soil water. This negative pressure is needed to balance the suction forces acting within the soil due to capillarity (matric suction) and the suction induced by different concentrations of salts in the pore water in the soil and the pure water outside (osmotic suction). The difference between total suction and matric suction is made up by osmotic suction (Huatet et al., 2013). The pressure difference across the water/air interface (ua – uw) controls matric suction, where ua is the pore air pressure and uw is the pore water pressure. In general, the pore air pressure (ua) will be at atmospheric pressure in the field (ua = 0), so the negative pore water pressure (-uw) will describe the suction. However, it can be difficult to explicitly calculate extremely negative pore water pressures, as cavitation can occur in conventional measurement systems. The mechanism by which bubbles form when water is subjected to tensile stresses is cavitation. Therefore, suctions are often measured (or controlled) in the laboratory by elevating the pore air pressure (ua) within the sample, a technique known as “axis translation”(Bujang et al. 2013).

a) Pressure Plates Extractors.

The axis translation technique involves using some variation of a pressurized chamber to apply air pressure to a material while keeping the water pressure at a constant value (usually zero). A soil sample is placed within the chamber onto a saturated high air entry disk that will allow for the flow of water through the saturated pore spaces but prevent air up to a rated value (air entry value) of matric suction. The air pressure inside of the chamber is elevated to a desired value while keeping the pore water pressure at a constant value (normally atmospheric pressure). The difference between the applied air pressure and the constant pore water pressure is the matric suction (ua-uw) at the existing water content or degree of saturation. As the pore air pressure is elevated, water is expelled from the soil sample through the saturated high air entry disk, and volume outflow measurements are able to be determined. The air entrance value of the ceramic disk is the only constraint to the measurement of matric suction using this technique. The axis-translation technique is used to operate the pressure plate device. This theory states that a matric suction can be applied to soil by regulating pore gas pressure (ug) and pore water pressure (uw), so that the difference between the two equals the desired matric suction, that is, = ug-uw. The approach is known as null-type axis-translation because the water pressure in the water compartment is kept as near to zero as possible (Fredlund and Rahardjo, 1993).

b) Tensiometer

A tensiometer is a device that measures the negative pore-water pressure of soil directly. The essential premise is that the water pressure contained in a high air entry material will equalize with the soil water pressure, allowing negative soil water pressures to be measured (Pan et al.2010). Only the value of the matric suction component in the soil with a suction range of 0–1500 kPa is provided by this measurement.

C) Suction Probe

Ridley and Burland (1993) invented the suction probe to measure suction. It is relatively simple and convenient to make direct measurements of matric suction using a suction probe. A suction probe consists of a pressure transducer with a high-air entry ceramic disk mounted at the tip of the transducer. The diaphragm of the pressure transducer responds to the pressure applied. The equilibrium between the pore-water pressure in the soil and the pore-water pressure in the water compartment provides the basis for suction measurements. Water flows from the water compartment into the soil, or vice versa, before equilibrium is reached. The pore water pressure is determined via the suction probe (uw). Since the applied air pressure (ua) is known and the matric suction is the difference between the pore-air and pore-water pressures (ua–uw), the matric suction can be calculated (Pan et al., 2010).

d) **Filter Paper Testing**

Soil suction can be measured with filter paper, which is a simple and inexpensive approach. This technique is used to measure total and matric suctions, and the test is carried out in line with ASTM D 5298–94. Higher suction, up to 106 kPa, can be measured with filter paper. Filter paper is allowed to absorb moisture from a soil specimen, When the soil and filter paper have reached equilibrium, the suction in the filter paper equals the suction in the soil (Ridley and Wray, 1995).

2.13 Importance of soil water characteristics curve

As the soil suction changes, unsaturated soil properties such as volume change, permeability, and shear-strength qualities are changed, and these changes can be linked to the quantity of water present in the soil pores. Each of the volume-mass variables needs to be taken into consideration when estimating unsaturated soil property functions in geotechnical engineering. The shrinkage curve can be used in conjunction with the (gravimetric) water content versus soil suction relationship (i.e. w-SWCC) to provide a more complete understanding of unsaturated soil behavior. Changes in the degree of saturation during soil drying have an effect on overall volume. The estimation of various unsaturated soil property functions, USPFs, requires the use of the saturated soil properties along with mathematical algorithms related to one or more of the volume mass SWCCs. However, the cost of directly determining unsaturated soil property functions (USPF) is beyond the financial budget of most clients (Fredlund, 2020). As a result, instead of laboratory tests measuring gravimetric water content vs soil suction, referred to as the soil-water characteristic curve (w-SWCC), and (ii) void ratio versus water content, referred to as the shrinkage curve (SC), as well as saturated soil parameters, were used to forecast USPF (Fredlund, 2020). SWCC is being used by researchers to forecast the engineering properties of unsaturated soil. Furthermore, the curve fitting parameters were employed to evaluate the improvement effect of the stabilizer on the expansive soils by determining SWCC and SC. In addition, after the establishment of SWCC and SC functions, the stabilizer effect on swelling pressure and volume change behavior of natural soil has been identified (Zhang et al., 2017).

2.14 Compaction Energy on soil water characteristics curve

In compacted soils, the compaction energy and the initial water content may control the SWCC shape at some level of suction. The soil water characteristic curves relate the amount of water that can be retained in the pores of a porous material to the soil water suction. This amount of water can be quantified in gravimetric or volumetric water content ways (Marinho & Stuermer 1993). Compacted specimens simulating optimum, dry-of-optimum and wet-of-optimum conditions showed two forms of SWCCs are unimodal and bimodal SWCCs. The unimodal SWCC was observed to be dominant for sand–clay mixtures with low clay content (less than 5%) and for specimens compacted at wet-of-optimum conditions. The bimodal SWCC was observed for optimum and dry-of-optimum water content specimens. The effect of the compaction procedure on the suction of two of the soils is shown in figure 2.7. The Goose Lane clay and the Champaign till were compacted using both, kneading compaction and static compaction. The results indicate the compaction curves obtained using the kneading method on the Goose Lane clay were similar to those obtained using static compaction. Although other results showed an influence of the compaction energy on the value of suction for the same water content, the shape of the SWCC is a biomodal curve.

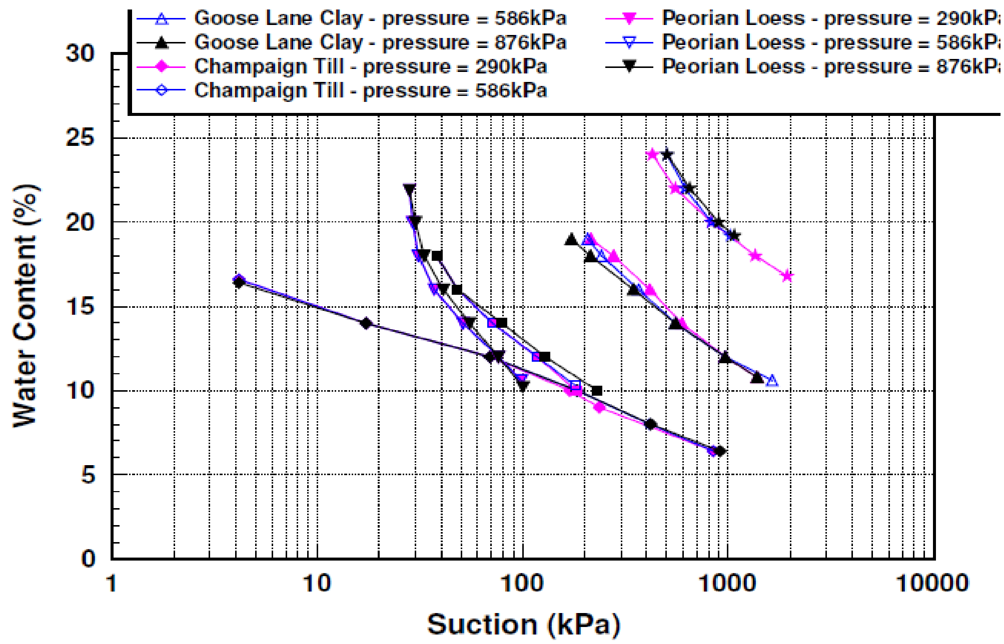

Figure 2.7 gravimetric water content and suction on compacted soil (Olson & Langfelder1965).

The shape of the SWCC is further influenced by the initial water content, density, compaction energy, clay mineral content, stress history, soil state structure, and pore size distribution (Fredlund and Xing, 1994). Compacted state samples are preferable for SWCC determination of improved expansive soil using different compaction energies (modified and standard). Because compacted soil samples were done to compare the results with treated compacted soil samples to describe the influence of additives and compaction energy (Zhoun and Yu 2005). In this stage, SWCC was evaluated using different compaction energies with optimum moisture content and the maximum dry density of stabilized and un stabilized soil. From Fredlund and Xing‟s (1994) model equation fitting parameters, a is related to the air-entry value of soil, which is the suction value at which air starts to enter the largest pores in the soil, n is related to the rate of desaturation, and m is a fitting parameter related to the curvature near residual conditions were evaluated.

Leong & Rahardio (2002) studied the influence of compaction energy on the SWCC of a mudstone residual soil using modified and standard compaction. On fine-grained soil, the soil pore size distribution typically decreased as compaction energy increased. Therefore, have different storage capacities of SWCC on the same soil when compacted with different energies. The results indicate that the SWCC behavior of fine-grained soil is more sensitive to compaction energy and compaction parameters. Therefore, the SWCC is a measure of the ability of soil to retain water under different suction levels by applying different compaction energies and stabilizing conditions using pressure plate apparatus.

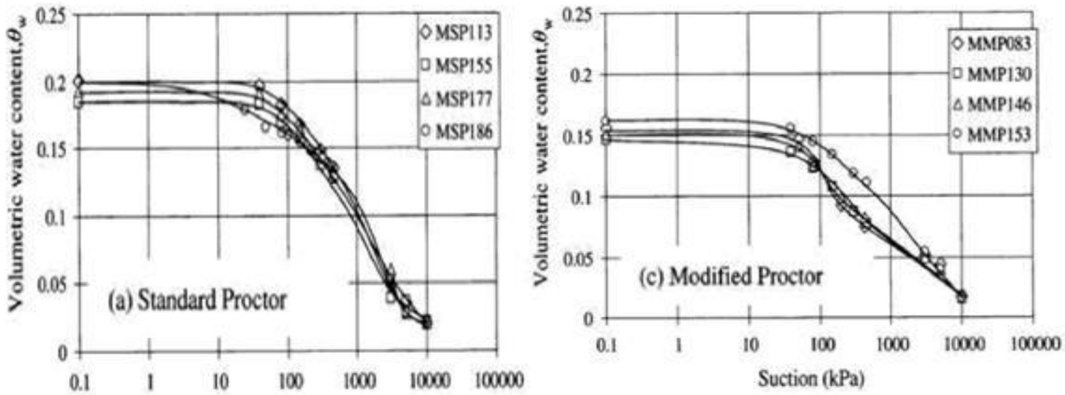

Figure 2. 8 variation of modified compaction effort on SWCC (Leong& Rahardio (2002)

2.15 Previous Studies of SWCC on stabilized expansive soil

Several studies have been conducted in recent years to incorporate unsaturated soil mechanics principles into conventional engineering practice. The use of SWCC to predict the unsaturated soil properties has been done by different researchers. Among the number of research studies done so far, the studies of Rahardjo (2002), Bilsel and Oncu (2005), Puppala et al. (2006), Thudi (2006), Khattab et al. (2006), Yang et al. (2011), and Khattab and Aljobouri (2012). Table 2.11. Mavroulidou et al. (2013), Elkady et al. (2015), Zhang et al. (2017) (Agus & Schanz, 2006; Montanez, 2002; Pei-yong & Qing, 2009) are reviewed and summarized in Table 2.11. Zhang et al. (2017) were studied The drying and wetting soil water retention curves (SWRCs) of statically compacted lime stabilized in London Clay specimens using the contact filter paper method, pressure plate apparatus, and a suction-controlled triaxial system containing the axis translation approach were all used in a series of tests. The flocculation and chemical bonding effects of the lime treatment boosted volumetric stability but decreased water retention ability.

Yang et al. (2011) To conduct the free expansion ratio test, lime and fly ash were introduced to the baise expansive soil in four different doses. The Pressure plate apparatus was used to conduct dewetting SWCC tests in the range of 5–1000 kPa. The improved expansive soils‟ expansibility and soil-water characteristic curve are developed.

The SWCC fitted by the Van Genuchten (VG) model for two modifiers with various mixing quantities is studied based on the idea of unsaturated soil.

Puppala et al. (2006) suggest that using pressure plate apparatus can evaluate the volumetric water content of fly ash-treated soils. The result shows that volumetric water content decreased with an increase in the percentage of fly ash stabilizers. The AEV for stabilized soils with a higher percentage of fines is typically larger than those with no fines. AEV was initially increasing and then started decreasing as the bottom ash stabilizer increased. As bottom ash increased, the n values increased, but for fly ash, an indefinite pattern was observed. The SWCC results are used to interpret how stabilizer treatment effects expansive soil behavior. Additionally, using multiple linear regression analysis, connections were established between basic soil and stabilizer parameters such as water content, dry density, liquid limit, plastic limit, and stabilizer doses and the model constants of Fredlund and Xing‟s SWCC formulation. Unfortunately, some of the previous studies only looked at the improving effect of stabilizers on SWCC. In addition, only a few researchers looked into the combined effect of the compaction energy of stabilized expansive soil on-SWCC.

Vanapalli et al. (1999) have reported that the soil water characteristic curve depends on several factors, such as soil structure, initial water content, void ratio, type of soil, texture, mineralogy, stress history, and method of compaction. According to them, the initial molding water content and the stress history have the most influence on the soil structure, which in turn dominates the nature of the soil-water characteristics for fine-grained soils.

Gurtug et al. (2004) reviewed the compaction behavior of fine-grained soils. They noted that both swelling pressure and swell potential are considerably influenced by variations in compaction energy. The greater the plasticity index of the soil, the greater the swell potential of the soil. They confirmed that an increase in compaction energy improves some engineering properties of soil significantly, but it also reveals an undesirable increase in the swell potential.

Abdulrahman et al. (2014) investigated stabilized clay soil by gypsum compacted by modified and standard energy with optimum moisture content. The results showed the AEV is significantly changed with the pore size distribution of soil that depends on the compaction energy.

Table2.11 Summary of previous studies on the evaluation effect of different stabilizers on the SWCC

Author /year/	Material to be used	Methods	Final goal
Taban (2016)	Different sands(based on particle size distribution data)	Genetic programming	Estimation of van Genuchten SWCC model
Jingsongqiam (2011)	Silt & clay	filter paper	Evaluate the effect of compaction degree on soil-water characteristic curve
Bilsen & Oncu (2005)	Lime	Filter paper	Develop SWCC for both stabilized & non stabilized
Yasora and Perera(2005)	Unsaturated soils	multiple regression analysis, equation	Estimation of the SWCC using GSD and PI-based conceptual model.
Tsa & Petry (1995)	Highly expansive clay mixed with bentonite	Pressure pate extractor with modify compaction	Evaluate the compacted soils, on matric and total suction & on SWCC
Guntur (2009)	Clay and sand mi	Pressure plate &index test, filter paper method	To evaluate the influence of stress state in the compression index on SWCC
Josip etal (2018)	Weathered residual soil	Filter paper & psychomotor	Evaluate suction range of disturbed & undisturbed sample and desaturation effect on SWC
Abdulrahman et al. (2014	Clay soil mixed with gypsum	Pressure plate	AEV is more influenced on modified compaction than standard compaction
Augustin etal 2020	Slity and clay soil	Pressure plate	AEV depends on the property of soil.

Lin & Cerato (2012)	Fly ash and expansive soil	Pressure plate apparatus	Air entry values for fly-ash treated soils decreased than natural soil samples for lower curing periods & Air entry value start increasing for higher curing period. The "**n** parameter related to rate of desaturation has increased with the fly-ash content increased.
Elkady etal. (2015)	Lime and expansive soil	Pressure plate Filter paper	The examination indicated that AEV evaluated from S-SWCC is consistently higher than that deduced from w-SWCC. The percent difference between AEV (w) and AEV(S) was observed to be maximum for untreated expansive clay where samples experience appreciable volume change.
Khattab etal (2012)	Lime, cement, clay soil	Vapour equilibrium and Osmotic solution.	The permeability of natural soil was found greater than lime – cement treatment.
Tinjum (1997)	Highly expansive soil Lime, Cement and cla soil	Pressure plate Vapour equilibrium and Osmotic solution	Evaluate the effect of compaction effort and soil plasticity on SWCC. The results show the reduced pore size distribution becomes high AEV The permeability of of natural soil was found to increase with lime, cement, and lime-cement treatment.
Thud (2006)	Lime, cement, clay	Pressure plate Filter paper	Volumetric water content of treated soil decreased with increased dosage of lime &cement
Khattab 2002	Lime pulse expansive soil	Pressure plate apparatus	Evaluate the Percent of lime and curing effect on SWCC. AEV is influenced by the increase percent of lime
Miao etal 2006	Remolded expansive soil	Pressure plate	The slope of SWCC depends on the dry density of the soil and the AEV increase with dry density.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Introduction

Unsaturated expansive soils annually cause major economic losses and structural damage due to exhibiting large amounts of swelling and shrinkage when subjected to variations in water content. These problems are typically expressed by the soil water characteristics curve for the estimation of unsaturated soil functions (i.e. shear strength, swelling pressure, and permeability functions). The unsaturated soil properties determined in this research are:- Gravimetric water content versus soil suction is referred to as the soil-water characteristic curve (w-SWCC) for treated and non-treated soils, and it evaluates the compaction parameter on the SWCC. Expansive soils, in general, experience volume changes as a result of the wetting and drying processes since the water holding capacity in their pores, the swell-shrink property of expansive soil causes instabilities in most light-weight buildings. The SWCC of both untreated soil and treated soils is determined using a pressure plate test device in the suction range of 33–1400 Kpa. For this section, the materials, methods, and procedures used to conduct all the specified laboratory tests are summarized.

3.2 Materials

Expansive soils were used in the laboratory tests. Expansive soil samples were collected from three different test pits in Ayer tena Kebele at a depth of 1.5 m, which is found in the south-eastern part of Bahir Dar city in the Amhara region, Ethiopia.

Figure 3.1 black cotton soil on site

Marble waste:

The utilization of waste marble is an industrial production and unutilized waste materials, especially in Kokeb marble industries, have resulted in increasing environmental pollution. Increasing the utilization rate of such wastes appears to be a solution to environmental problems and will decrease construction costs. Increasing demand for marble products in the construction industry rises the generation of waste marble dust during the cutting process. As a result, marble waste is collected from the Kokeb industry found in Bahirdar city.

3.3 Description of the study area

The geographical location of Bahir Dar is at 11° 35‟ 37.1”N, 37° 23‟ 26.77”E north latitude and east longitude. The town is located about 565 km north-west of Addis Ababa and it lies in an area characterized by generally flat topography in Ayer Tena kebele. The most abundant soils are expansive blak cotton soils and non-expansive red clay minerals (Desalegn 2004).

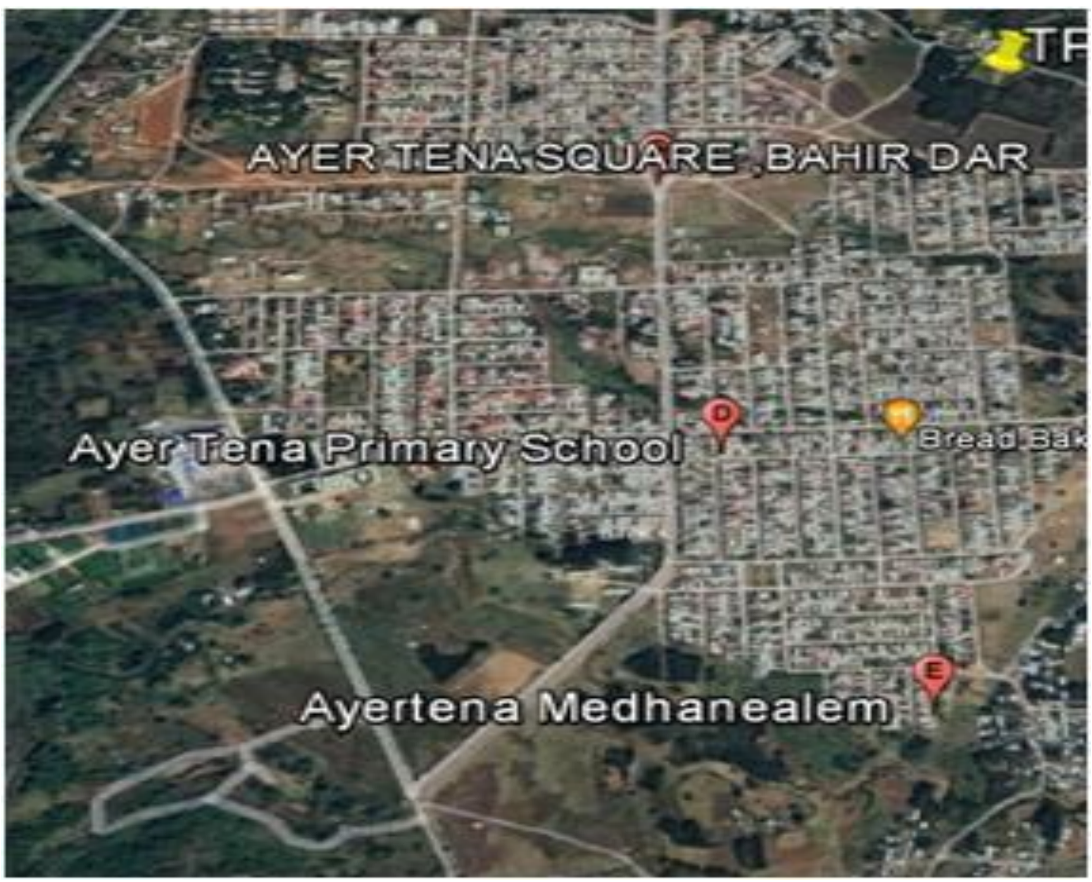

Figure 3.2 Location of test (GIS and Google earth)

3.3 Soil sampling

In this research representative disturbed expansive soil samples collected from five test pits in different location, soil sampling was carried out. The test pit locations were determined according to ASTM D 420 -98. Following the identification of the test pit (TP) locations, samples of disturbed were obtained at depth of 1.5m and the exact locations of the test pits were tracked and recorded using the Global Positioning System (GPS) tracker software. The samples were then ready to be transported from the site to the laboratory for investigation in accordance with ASTM D 4220.

3.4 Test methods

The marble dust was mixed with the expansive soil in different proportions (5%, 10%, 15%, and 20%, 25%) by the dry weight of the soil. To achieve the objective of the study, a series of laboratory tests were conducted on untreated and treated expansive soil, performed for free swell index sieve analysis, Atterberg limit, compaction characteristic, unconfined compressive strength, and California bearing ratio. Curing methods were also considered on marble treated soil samples by applying an impervious plastic bag to cover the marble-treated soil samples for 7 days. In addition, the Standard Proctor test method and modified test method that are specified in ASTM D 698 and ASTM D1557 were performed. Both compaction tests are performed following the compaction procedures for the appropriate grain size distribution of soil with an appropriate mold size. The number of layers of soil, the number of blows, the weight of hammers, and the hammer dropping height are considered. A representative compaction parameter was taken from each compacted specimen that was used to strength evaluate both treated and untreated samples.

3.5 Soil Sample Preparation

Soil samples were prepared according to ASTM D 421 – 85, before being treated and tested. This Method involves exposing soil samples to the air to dry them at room temperature, then breaking up the air-dried soil aggregates with a rubber-covered pestle in the mortar. The dried soils are then sieved to separate into several laboratory tests, such as Atterberg limits (LL, PL), Free swell (FS), Specific Gravity, Linear Shrinkage (LS), and proctor compaction test.

3.6 Standard Laboratory tests

The laboratory tests were divided into two phases: Natural Moisture Content (NMC), Atterberg Limit (LL, PL), Grain Size Analysis, Specific Gravity (Gs), Free Swell (FS), Linear Shrinkage (LS), proctor compaction tests, and Unconfined Compressive Strength tests (UCS) for both natural and marble-treated expansive soils were investigated in the first phase. The second phase includes determining SWCC for different compaction parameters for untreated and marble-treated soil samples using the Pressure Plate Apparatus and Ring method, respectively.

3.6.1 Natural Moisture Content (NMC)

The laboratory procedure to measure the water content comprises of measuring the moist soil sample as collected from the site and drying the moist soil in an oven at 105℃ for 24 hours according to ASTM D 2216. The loss of mass due to drying is referred to as mass of water. The mass of water divided by the mass of the dry specimen yields the water content.

3.6.2 Specific gravity

The specific gravity of a substance is defined as the ratio of the density of the substance to the density of water. The specific gravity of soil solids is utilized for performing weight-volume calculations in soils. In this research, the measurement of specific gravity of solid soil will be performed according to the ASTM D854 standard which is the standard test method for the specific gravity of soils.

3.6.3 Particle size distribution

Grain size analysis of disturbed sample was performed using ASTM D 422-63. Grain size analysis were performed in two stages: Sieve analysis is used for particles bigger than 0.075mm; it involves shaking the soil through a stack of wire screens with known-size openings, and the percentage finer can be determined from the mass of sample retained on the sieve, the mass of sieve, and the total mass of sample, while for particles smaller than 0.075mm, sedimentation (Hydrometer Analysis) is used. Combined analysis is necessary for soil samples that contain a quantifiable amount of their grains that are both coarser and finer than 0.075mm in size.

3.6.4. Free Swell

One of the most common easy tests for estimating the swelling capacity of expansive clay is Free Swell. This test was carried out in accordance with Indian standard (IS1977). The technique is taking two oven dried soil samples, passing them through a 425 μm sieve, and placing 10cc of each into two 100ml graduated cylinder.

3.6.5 Atterberg Limits (LL and PL)

This test is carried out in accordance with ASTM D 4318. For the liquid and plastic limits, a soil sample was air dried, and 200g of the material passing through a No. 40 sieve (425μm aperture) was obtained and thoroughly combined with water on a flat glass plate to make a homogeneous paste. The liquid limit (LL) is the water content in percent at which soil in a standard cup cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm. On a semi–logarithmic graph, the determined moisture content and the corresponding number of blows are plotted, then LL is determined as the moisture content corresponding to 25 number of blows from the graph.

3.6.6 Linear Shrinkage (LS)

Linear shrinkage is a measurement of how much a sample shrinks in length after it has dried completely, expressed as a percentage of its original length. The test was carried out in accordance with IS 2718, an Indian standard.

3.6.7 Soil Classification

There are various classification schemes, each of which employs a distinct set of fundamentals. The Unified Soil Classification System (USCS) and the American Association of State Highway and Transport Official (AASHTO) both employ index property to classify soils.

3.6.8 Standard Compaction Test

According to ASTM D 698, this laboratory test is used to establish the relationship between a soil‟s moisture content and dry density for a given compaction effort. The sample is compacted in the Standard Proctor Test by a 2.5 kg hammer falling 308mm into a soil-filled mold. The mold is filled with three equal layers of Soil, each of which receives 25 drops of hammer. The dry density vs. moisture content curve is then plotted to determine the Maximum Dry Density (MDD) and Optimal Moisture Content (OMC).

3.6.9 Modified Compaction test

According to ASTM D1557 Modified Compaction Test procedure, equipment & function is essentially the same as that used for the Standard compaction test except the use of (volume of mold Hammer mass, drop of hammer height) and layers of soil to be used.

3.6.10 Unconfined Compressive Strength Test (UCS)

According to the ASTM standard, unconfined compressive strength (UCS) is the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test. This method is applicable only to cohesive materials that will not expel water during loading. The tests are conducted for both the natural soil and soil treated with different percentages of marble waste (for uncured, 7-day curing periods) in accordance with ASTM D 2166 testing procedures. The unconfined compressive strength of cohesive soil is calculated using strain-controlled axial load application in the undisturbed, remolded, or compacted state.

3.6.11 California Bearing Ratio Test

The California Bearing Ratio (CBR) test is a strength test that compares a material's bearing capacity to that of well-graded crushed stone. The CBR test was conducted as per AASHTO T193-99. The three-point CBR test was used (10, 30, and 65 blows per layer). Its goal was to evaluate the relative stability of fine crushed rock as a base material. It is now widely used throughout the world for evaluating the stability and strength of subgrade soil and other flexible pavement materials for pavement design.

3.7 SWCC Determination

As per ASTM D6836, the determination of the SWCC has been standardized. In the standard, there are five methods, from A to E, to determine the SWCC. Methods B and C use pressure plate extractors and differ in terms of the measurement method. Method B measures the volume of water outflow from the specimen (volumetric), whereas Method C measures the weight of the specimen (gravimetric). Methods B and C are used for suction in the range of 0 to 1500 Kpa. In this study, I was used to method C type measurement.

3.7.1 Soil Preparation and mixing design

The known weight of pulverized dry soil was taken. The dry weight of marble waste was calculated for different percentages and added to the dry soil and compacted with the required optimum moisture. SWCCs are determined for untreated natural soil and treated soil samples by adding the content of 5%, 10%, 15%, and 20% with marble waste for 7-day curing periods, following the drying path as per ASTM D 6836 for a suction range of 33–1400 kPa. The pressure plate apparatus has a ring dimension of 1 cm in height and 5 cm in diameter. The pressure plate apparatus works on the principle of axis-translation technique. This principle stating that matric suction (ψ) can be applied to a soil by controlling the pore gas pressure (ug) & the pore water pressure (uw).

3.7.2 Pressure Plate Apparatus (Axis translation techniques)

The pressure plate apparatus is used to determine soil water retention capacity. It comprises a pressure chamber enclosing a water-saturated porous plate, which allows water but not air to flow through its pores. The porous plate (ceramic plate) is at atmospheric pressure at the bottom, while the top surface is at the applied pressure of the chamber. Soil samples are placed in retaining rings in contact with the porous plate and allowed to soak up water by immersion in water. The porous plate with saturated soil samples is then placed in the chamber and a known air gas pressure is applied to force water out of the soil through the plate. Water flows out of the soil until equilibrium between the force exerted by the air pressure and the force by which soil water is being held by the soil (ψ) is attained. Pressure manifolds are an essential component in pressure plate extraction systems. They are required to control the air pressure from a compressor or compressed air vessel into the extractor. The pressure required for the test is applied through an air compressor. It is connected to the test chamber through the connecting hose. The pressure manifold, shown in Figure 3.3, consists of a pressure gauge to measure the pressure in a compressed gas and displays the air pressure level that is in the compressor tank; a pressure regulator which regulates the air pressure; an air filter which keeps small dirt particles out of the regulators; and several control valves. SWCC can be experimentally determined by plotting in terms of the degree of saturation (S) or gravimetric water content or volumetric water content versus matric suction on a semi-log plot. In my research, SWCC plots gravimetric water content versus matric suction for each treated, untreated, soil samples by different compaction effort. The pressure plate apparatus works on the principle of axis-translation technique. This principle states that a matric suction (ψ) can be applied to a soil by controlling the pore gas pressure (ug) and the pore water pressure (uw) so that the difference between the pore gas pressure and pore water pressure equals the desired matric suction, that is, $\psi = ug - uw$.

The pore gas pressure was raised to apply the suction via the axis translation principle using Method C in ASTM D 6836. This test was carried out in the soil laboratory of Bahir Dar Institute of Technology.

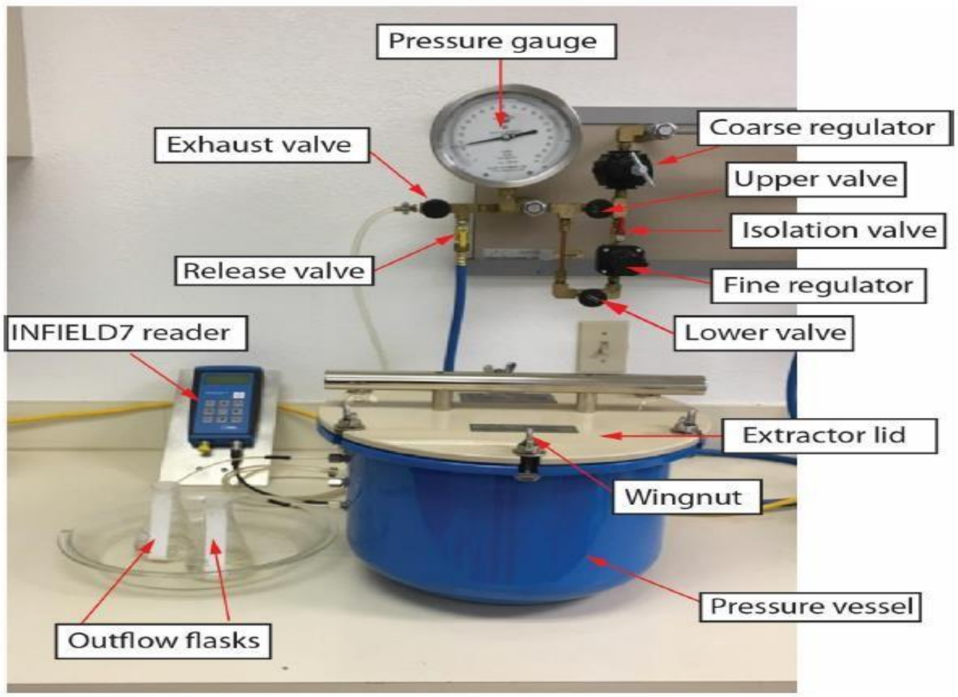

Figure 3. 3 Laboratory setup of the pressure Plate apparatus of regulated air system

3.7.3 Test procedure

Both treated and untreated soil samples compacted with the required compacted parameter and compacted energy then compacted samples were placed on the bench, and retaining rings were gently placed on the soil surface, according to ASTM D 6836. Soil that protruded beyond the retaining ring‟s edge was cut with a trimming spatula so that the ring could slide over the soil specimen with ease. The top of the specimen that flushed with the top of the retaining ring was trimmed. The specimen‟s mass in the retaining ring was measured and recorded to the nearest 0.01 g. Method D2216 and Equation 3.1 were used to calculate the gravimetric water content and dry density of the remaining material.

$$\rho d = \frac{Mm}{V(1+\mathrm{W}m)} \quad \text{--} \quad 3.1$$

Where: Mm and wm are the mass and gravimetric water content of the moist soil in the retaining ring after the specimen has been prepared (trimmed or compacted in ring). V is the volume of the retaining ring. The ceramic plates were first soaked for 4 hour after the sample was placed in the retaining ring, and then the soil specimens were placed in contact with a water saturated porous plate or membrane and saturate for at least 48 hours. After the sample was saturated, the weight of the specimen was measured, and the saturated water content was calculated using Equation

$$\mathrm{W}sat = Msat\,(\frac{1+\mathrm{W}m}{Ms}) - 1 \qquad 3.2$$

Where Ms is the mass of soil in the retaining ring after saturation and Wm= mass of water.
Before closing the chamber and starting the test, all surplus water on the plate (or membrane) was removed. The matric suction was applied to the soil by controlling the pore gas pressure. Water flows from the specimen when the matric suction is applied until the equilibrium water content corresponding to the applied suction is reached. Then water stopped flowing from the specimen, and equilibrium was established. Clamp the outflow tube to prevent backflow. When equilibrium was achieved, the pressure was drained, and the pressure chamber was opened. Using a wide-blade spatula, the specimens and their holding rings were swiftly removed from the porous plate (or membrane). The specimens were immediately weighed. The specimens were placed back on the porous plate because they had been used during the entire test. Using a wide-blade spatula, the specimens and their holding rings were swiftly removed from the porous plate (or membrane). The specimens were immediately weighed. This method was carried out again and again until all successes had been determined. After all equilibrium had been established, the soil specimens were immediately transferred to covered moisture cans in order to avoid changes in the water content. The specimens were weighed and placed in a drying oven for 24 hours. The dry specimens were removed from the oven and immediately weighed. The gravimetric water content of the soil specimens was calculated. Wet of specimen-(dry specimen)/(dry specimen)*100 ,Calculate bulk density as the mass of the compacted specimen divided by the volume of the ring. Calculate particle density $(\text{Ø}) = \frac{\mathrm{ps-pb}}{\mathrm{ps}} = 1 - \frac{\mathrm{pb}}{\mathrm{ps}}$,

Volumetric moisture content = gravimetric moisture content * bulk density of soil.

Degree of saturation= $(\frac{\mathrm{Gs*wi}}{\mathrm{e}})$ where Gs = specific gravity, wi= initial water content.

Void ration= $\frac{(GS*6\mathrm{W})}{6d} - 1$ where ∂W and ∂d is unit weight of water & dry density of soil.

CHAPTER FOUR

RESULT AND DISSCUSSION

4.1. Introduction

Different laboratory tests were carried out on the untreated natural soil sample and samples treated with different percentages of marble waste. The tests conducted include the basic index and classification tests; FS tests; LS tests; SWCC tests; and standard and modified compaction tests. This section presents the results from the different laboratory tests; the interpretation and analysis of the test results; and a discussion of the findings from the test results. A comparison of the findings from this study with previous research findings is also provided in this section.

4.2 Test Results for the Natural Soil Sample

4.2.1 Soil Index and Classification Test Results

The grain size analysis test was conducted to determine the particle size distribution of the three soil samples. Figure 4.1 shows the particle size distribution curves of the five soil samples.

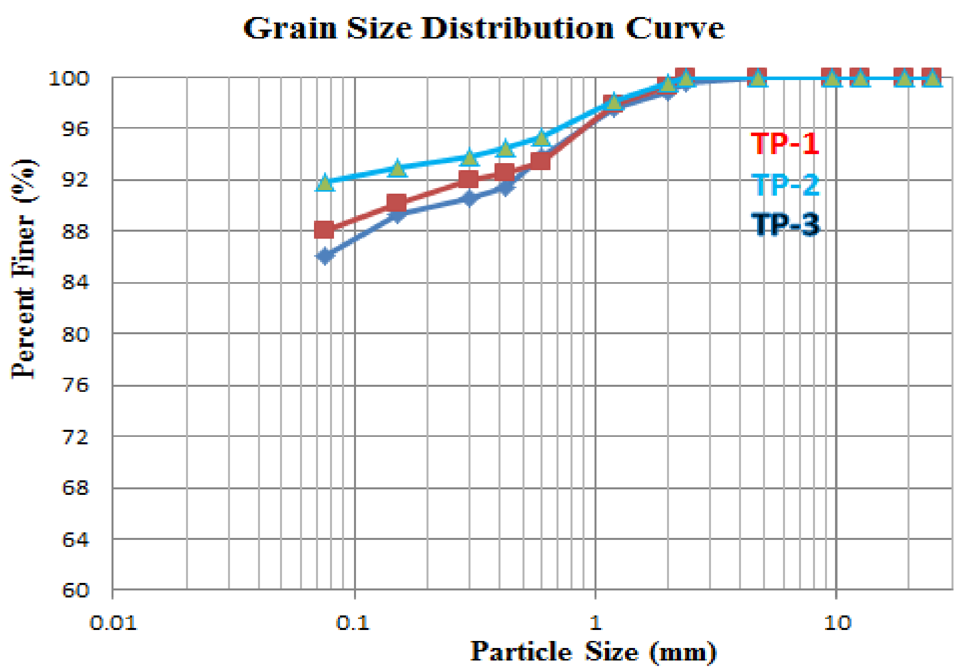

Figure 4.*1* Particle size distribution curve of expansive soils

By conducting wet sieve analysis, as shown in Figure 4.1, the grain size distribution curve shows that for the three soil samples, the percentage of passing in 2 mm, 0.425 mm, and 0.075 mm sieves. The percentage of finer is presented in the table below based on the ASTM test standard.

Table 4.1 Properties of natural expansive soil

Property of grain size distribution	**value**	
Sand (%) Silt(%) Clay(%)	14 32 56	Test pit-1
Sand (%) Silt (%) Clay (%)	20 42 49	Test pit -2
Sand(%) Silt(%) Clay(%)	14 33 53	Testpit-3

4.2.2 Atterberg limits linear shrinkage and Free swell Index

The liquid limit, plastic limit and plasticity index test results linear shrinkage and free swell index for all the natural soil samples are presented in Table 4.2 as follows.

Table 4.2:- Summary of different laboratory test results of untreated natural soil samples

TPNo	Nmc %	GS	LL%	PL%	PI%	LS%	FS%	AASHTO	UCS
TP -1	42	2.51	94.8	40	54.8	21.4	125	A-7-5	CH
Tp -2	44	2.61	90.3	51	39.3	18.9	115	A-7-5	HM
TP-3	40	2.4	93.68	51	42	16.7	104	A-7-5	HM

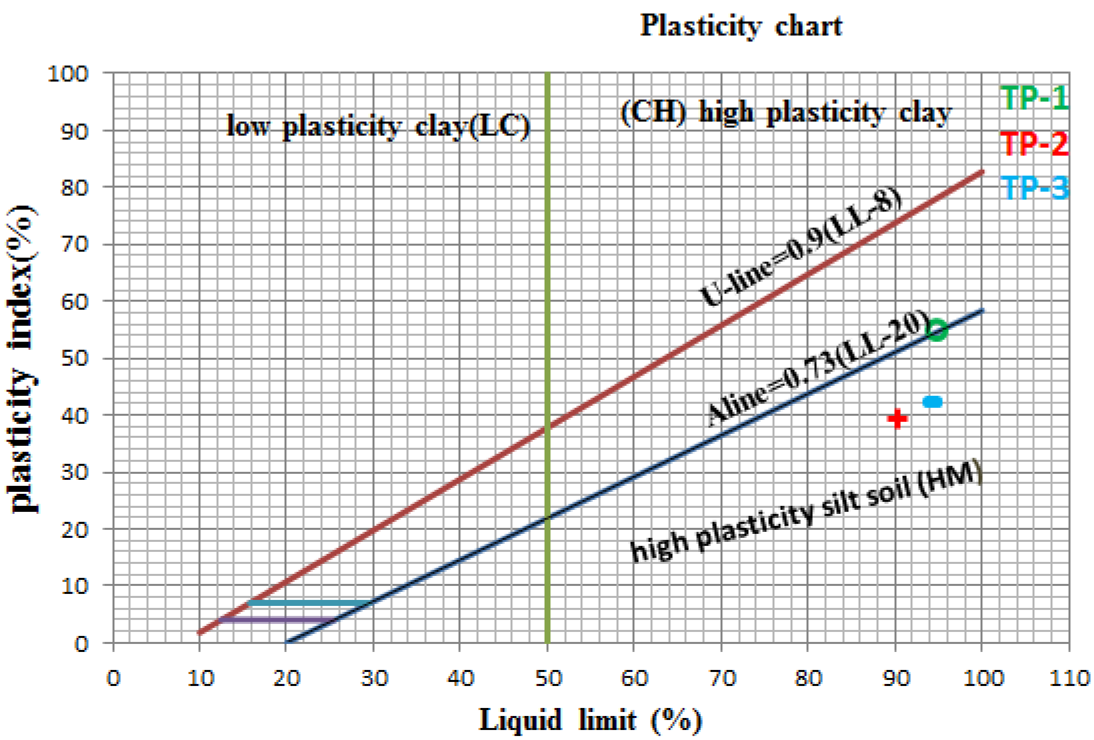

Table4. 3: Compaction parameter of untreated soil samples.

Test pit No	OMC	MDD(g/cm3)
TP-1	20	1.49
TP-2	26	1.54
TP-3	24	1.52

The compaction energy changes from standard test method to modified compaction test method the dry density of the spacmen increased by 7% .The test result shows natural moisture greater than OMC indicated that the sub grade layer is below the minimum requirement based on ERA.

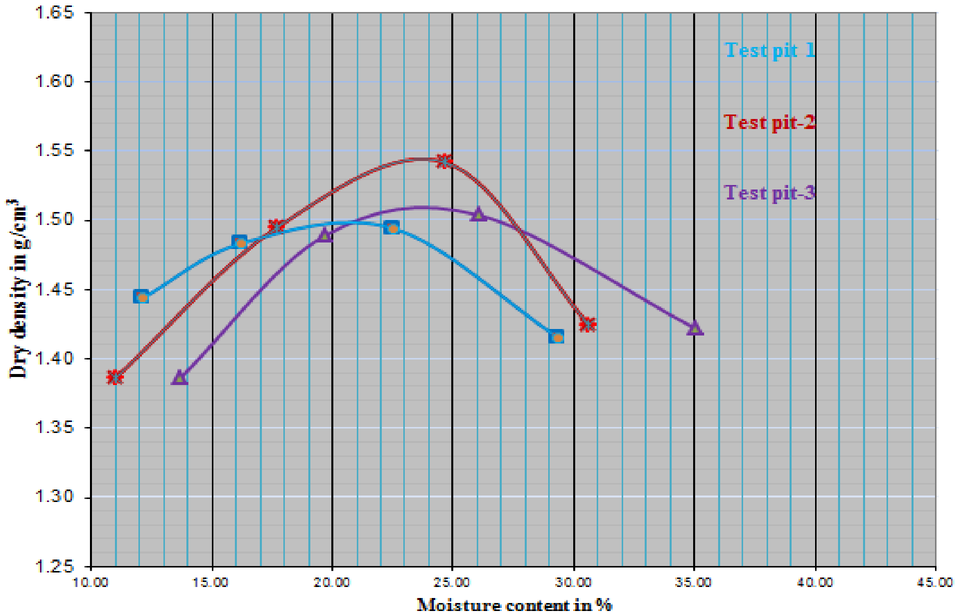

Figure 4.2 Compaction parameter of untreated testing soil

4.2.4 Unconfined compressive strength

The unconfined compressive strength (UCS) test results of the natural soil samples compacted at optimum water content correspond to 200.35kPa, 190.31kPa and 180.75 kPa for TP-3, TP-2, TP-1 respectively. According to the ERA 2013 manual, if the unconfined compressive strength soil test value is less than 250 kPa and it is soft clay, the soil must be treated.

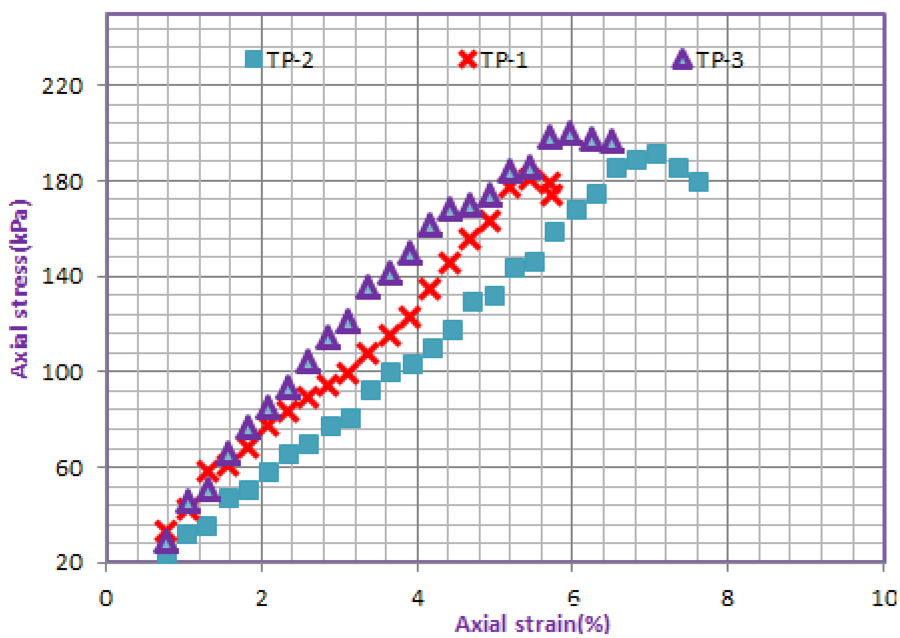

Figure 4.3 Unconfined compressive strength test

4.2.5 CBR test result for natural soil sample

CBR tests are conducted on the compacted specimens using the optimum moisture content obtained from the compaction test. The results of the CBR and CBR-swell test results of the natural soil are summarized in Table 4.4 at the standard penetration point with respect to the standard blows.

Table 4. 4 CBR test result of untreated soil sample

Test pit	CBRvalue@5.08			CBR @2.54mm penetration			CBR swell
	10 blow	30 blow	65 blow	10 blow	30 blow	65blow	
Test pit -1	1.3%	1.61%	2.1%	1.34%	1.7%	2.2%	8.6
Test pit -2	2.24%	2.21%	2.49%	2.34%	2.4%	2.7%	5.8
test pit-3	2.16%	2.3%,	2.6%	2.18%,	2.54%,	2.88%	6.7

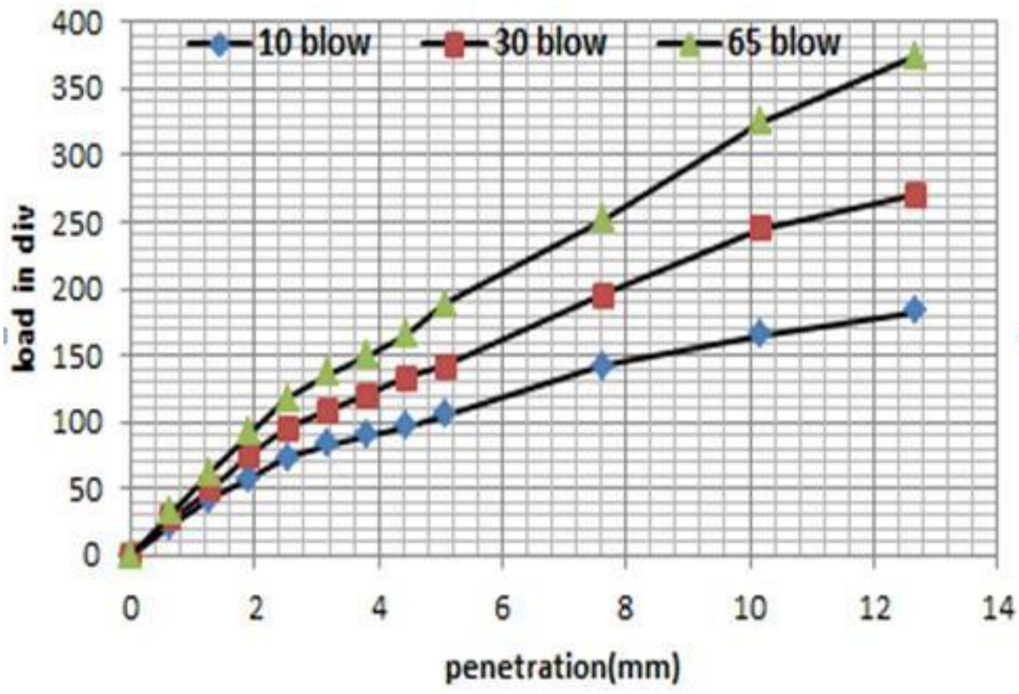

Figure 4.4 CBR test pit -1 result

4.2.6 Discussion on Test Results of Natural Soil

Based on the field observation and the conducted laboratory test results, the natural soil samples in the study area are found to be expansive soil. The color of the soil is dark brown and the cracks that are also observed are some of the indicators of the expansive soil according to field identification techniques. Similarly, the liquid limits of 94.8%, 90.3%, and 93.12% and the plasticity index of 54.8%, 39.3%, and 42% for test pit 1, test pit-2, and test pit-3, respectively, show the expansive nature of the soil. All the above soils are classified as A-7-5 according to AASHTO, which indicates poor subgrade strength and, based on USCS test pit-1 is highly plasticity clay (CH) and test pit-2, test pit-3 MH are (high plasticity silt soil). Such soils with high-plasticity silt contents are common to many areas in Ethiopia. These high-silt and clay soils frequently have low strengths and minimal bearing capacity. The strength test results of CBR and UCS according to ASHTO and ERA design standards belong to poor subgrade and soft clay respectively. Based on the above classification systems, the soil that falls under this category has poor engineering property to be used as a sub-grade material. The liquid limit values of the soils are above 55% and their plasticity index values are greater than 20% which indicate that the soils have high swell potential (ERA 2002). The free swell index test results also show that the soils have high swelling potential with an FSI value greater than 100% based on the Indian standard. Similarly, the CBR and CBR swell test results indicate that the soils have low bearing capacity and high swelling potential based on the ERA manual, less than three and less than two, respectively. Therefore, appropriate treatment methods are required to minimize the problems associated with these soils in the study area before constructing structures. So, in this section, choose only one test pits (TP-1) that have been stabilized by marble waste at various percentages of 5%, 10%, 15%, and 20% by dry weight of soil for strength test and from 5% -25% mw for index tests.

4.3 Marble waste as Stabilizer

For this study, the marble waste is used as a stabilizer for the expansive soil in the study area. Before treating the expansive soil by using marble waste, it is important to know the properties and chemical composition of the marble waste. The marble wastes collected from the Bahir-Dar Kokeb factory have the chemical compositions and physical properties listed below.

The addition of marble waste material as an additive to problematic soil is a developing method used to stabilize soil in geotechnical engineering. Many researchers support the addition of marble waste to problematic soil as a method used to stabilize soil.

Table 4.5:- Oxide composition and Physical property of marble waste (Begashaw Worku 2019)

Oxides	**Values in percent**
Silica (SiO_2)	7.84
Alumina (Al_2O_3)	0.01
Iron II Oxide (Fe_2O_3)	0.32
Lime (CaO)	49.4
Magnesia (MgO)	0.60
Sodium Oxide (Na_2O)	0.01
Potassium Oxide (K_2O)	0.40
Manganese Oxide (MnO)	0.04
Phosphorus Penta Oxide (P_2O_5)	0.01
Tin Oxide (TiO_2)	0.03
Water (H_2O)	0.28
Loss on ignition (LOI)	40.23
Physical property	
Specific gravity	2.74

4.3.1 Effect of marble waste on Free Swell Index of the Soil

The free swell index (FSI) test results show that as the amount of marble waste increases, the free swell index value decreases. The maximum decrement in the free swell index of the soil was the addition of 25% of marble waste. The free swell index value of the natural soil is reduced from 125% to 44%. The free swell index value shows that the soil changes from high swelling potential to low swelling potential with the addition of 25% of marble. The effect of marble waste on the free swell index of soil is shown in Figure 4.5. This result was confirmed by Singh and Yadav (2014). The differential free swell value of black cotton soil stabilized by marble dust decreased from 66.6% to 20.0%, showing an appreciable decrease in swelling behaviour.

The addition of 25% marble waste reduces the free swell ratio of a soil from 2.2 to 1.44. Based on the free swell ratio test result, the soil changes from high swelling potential to low swelling potential based on Sridharan & Prakash (2000). The test result shows when the percentage of marble waste increased the free swell ratio value till know reduced.

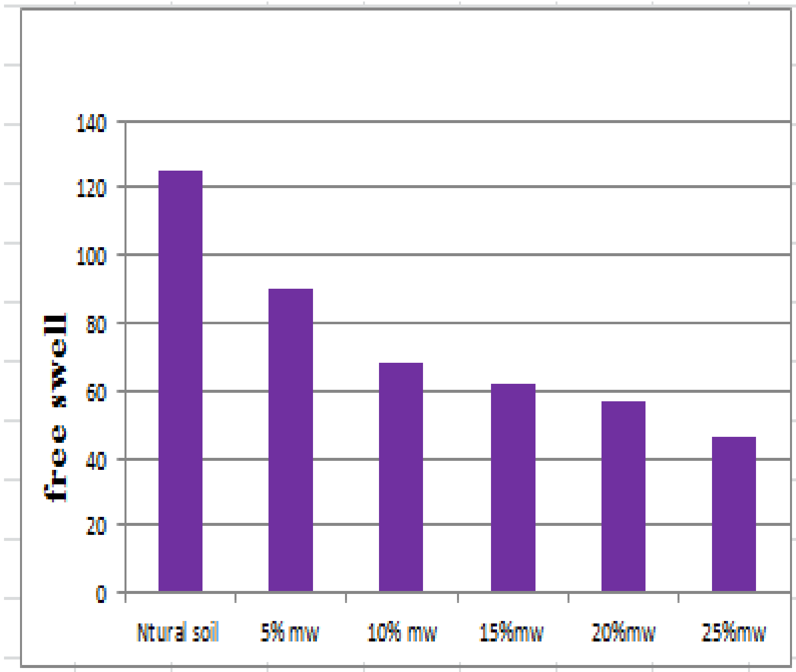

Figure 4.5 The effect of marble waste on the free swell index.

4.3.2 Effect of marble waste on the Atterberg Limits of the Soil

The effect of the addition of marble waste on the plasticity index of the soil is shown in Figure 4.6 for 7-day cured samples. Based on test results, as the percentage of marble waste increased, the plasticity index decreased. The plasticity index test result showed that the treated soil changed from very high swell potential to medium swell potential. The liquid limit decreases from 96% to 46%, the plastic limit decreases from 58% to 24%, and the plasticity index decreases from 38% to 22% as the percentage of marble waste increases by 25%.

4.3.3 Effect of marble waste on linear shrinkage of soil

Linear shrinkage tests were carried out to determine the one-dimensional shrinkage of stabilized expansive soils in terms of marble content. The test trough was filled with a stabilized soil specimen at the liquid limit. The wet material was placed in a drying oven and dried at a temperature of 110 ±5°C for about 24 hours until all shrinkage stopped. The test result shows linear shrinkage decreased from 33.33% to 7.6% with the addition of 25% marble waste.

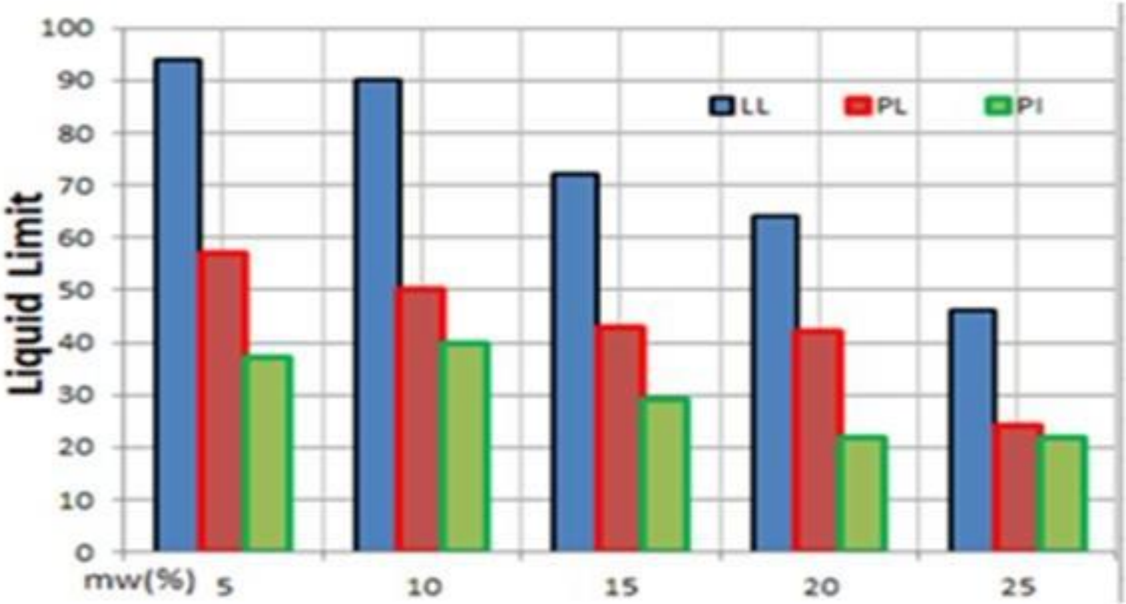

Figure 4.6 Effect of marble waste on the Atterberge limit test result.

The reduction in the plasticity index of the soil is mainly due to the replacement of plastic soil particles with non-plastic particles of marble waste and the availability of calcium in marble waste for the occurrence of cat ion exchange. These results were confirmed by Altu Saygili (2015) and Okagbue (1999), who concluded that the plasticity of soil was reduced by 34 % to 23 % and the liquid limit was reduced from 57.6% to 33.9%.

4.3.3 Effect of Marble waste on compaction parameter of the Soil

a) Maximum dry density (MDD)

The maximum dry density increases from a natural soil value of 1.43 g/cm3 to 1.92 g/cm3 with the addition of 20% marble waste by dry weight of the soil sample. The effect of additional marble waste on the maximum dry density and moisture content is shown in Figure 4.7, but at a particular point of the maximum dry density of the soil might be a unique curve due to the water absorption capacity of the soil increasing because of the addition of marble waste. This results in an increase in the moisture content and reduces the maximum dry density.

(Ramoo & Ravi 2018) suggested that the Optimum Moisture Content (OMC) of clay accelerated from 18% to 24% and the Maximum Dry Density (MDD) increased by up to 10% when using 20% marble waste.

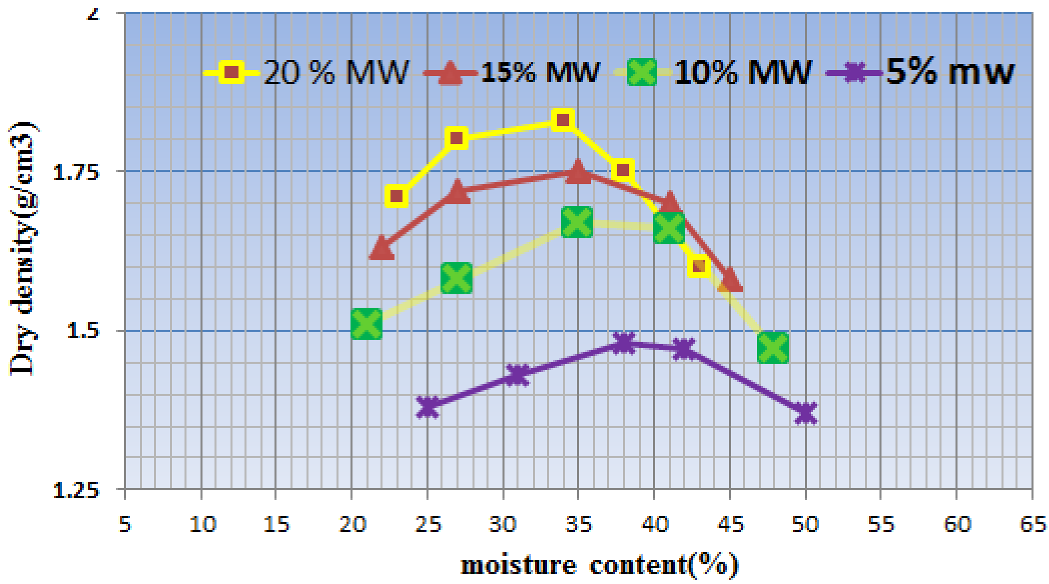

Figure 4. 7 Effect of marble waste stabilizer on compaction parameter.

c) Optimum moisture content (OMC)

The optimum moisture content initially increased from a natural soil average value of 20% to 38% with the addition of 5% marble waste by dry weight of soil sample. Since the absorption capacity of marble mixed soil has increased. The effect of the addition of marble waste on the optimum moisture content: after adding some percent of marble waste, as shown in the above figure, the optimum moisture content of the soil decreased. This might be through the addition of marble waste that makes the particle spacing closer or filling the voids of the soil.

4.3.4 Effect of marble waste on unconfined compressive strength

The determined unconfined compressive strength values are increased with the increment of stabilizers for soil samples mixed with 5%–20% marble waste. The strength of the material is simultaneously increased. But at 15% of marble waste, the unconfined compressive strength is increased by two times the untreated soil. This result also satisfied the minimum standard subgrade strength based on the ERA manual 2013. The increase in unconfined compressive strength of the soil might be due to strong bondage formed between the soil particles and marble waste, and it may also be because the fine parts of marble waste fill the voids of the soil that increases the unconfined compressive strength value and reduces swelling. This result concurs with Singh et al. (2017) who investigated the effect of marble dust on the strength of low plastic silt soil. The optimum percentage of marble dust was found to be 15% marble waste in an unconfined compressive test.

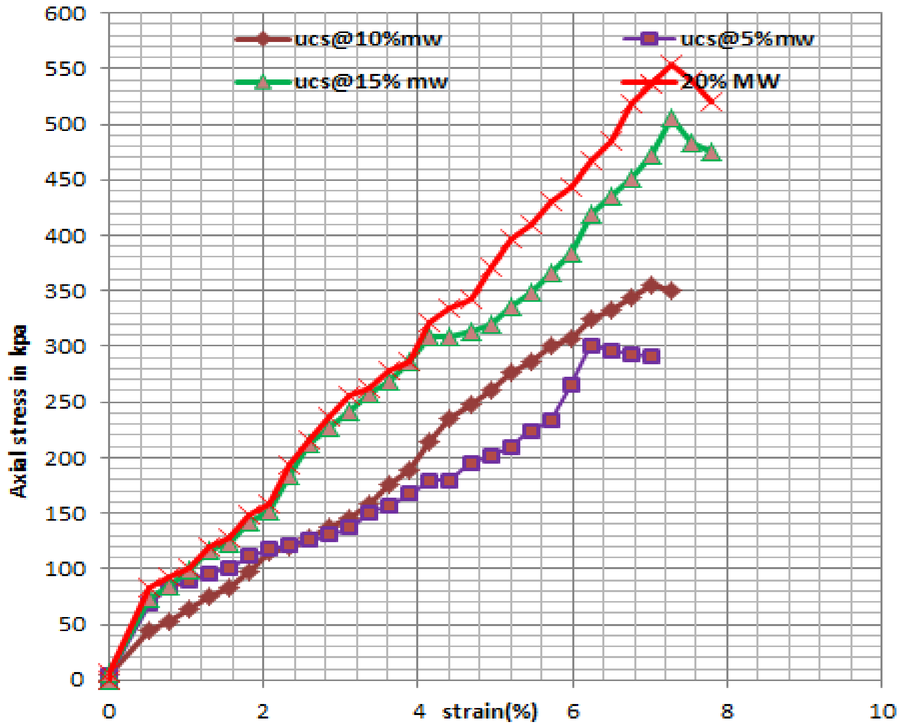

Figure 4. 8 Effect of marble waste on unconfined compressive strength test result.

4.3.5 Effect of marble waste stabilization on CBR and CBR Swell of the soil

a. CBR Value

The CBR value increases from 1.34 % to 15.75 % with the addition of 20% of marble waste. According to ERA pavement design manual volume I (2013), the natural soils are classified under S1 and, with the addition of 20% of marble waste, the class changes to S4. The effects of the addition of waste marble on the CBR values are increased. The CBR values of the soil are due to the soil voids being filled by fine particles of marble waste, so the soil‟s water-entry ability is reduced for the stabilized soil. Therefore, the CBR value of the stabilized soil sample increases. Similarly, the internal bonds between the soil and marble waste increased due to cation exchange between the soil and marble waste, which correspondingly increased the soil strength in terms of CBR. The result was agreed by Ahmed and Fares (2020). The California bearing ratio (C.B.R) test result of marble waste and expansive soil mixture improved remarkably with the maximum value of 12.5% at 25% addition of marble dust.

In addition, the stabilizing clayey soil by marble dust using the optimum values, maximum dry density, CBR and unconfined compression strength obtained were 1.76 g/cm3, 8.4% and 777.11KN/m2 respectively at 15% marble powder addition (Yashdeep & Soni, 2017).

Table 4. 5 CBR value of treated soil stabilized with 10% marble waste (TP-1)

Penetration	Load in division	Load in division	Load in division
in mm	for 10 blows	for 30 blows	for 65 blows
0	0	0	0
0.64	95	115	130
1.27	188	244	270
1.91	289	362	398
2.54	**380**	**425**	**480**
3.18	440	481	562
3.81	490	530	628
4.45	530	578	690
5.08	**565**	**615**	**712**
7.62	595	647	735
10.16	625	667	770
12.7	635	690	800

CBR@2.54	CBR@ 5.08
6.93%	6.87%
7.75%	7.48%
8.75%	8.57%

b. CBR Swell

The CBR-Swell value of the soil decreases from 8.6 % to 2.0% with the addition of 15% of marble waste. The test results show that with increasing of compaction effort (from 10 blows to 65 blows) in addition to marble waste the CBR-swell value reduces for the soil treated with the same amount of waste marble. The value of CBR-Swell with the addition of 15% of marble waste, that is 2.0%, is equal to the maximum value of mentioned in the ERA Geotechnical Design Manual. The CBR tests were performed by utilizing the OM of the compaction test. The effects of marble content on CBR value and swelling potential were shown to show that the test results improved the CBR values and decreased the swelling potential of the soil.

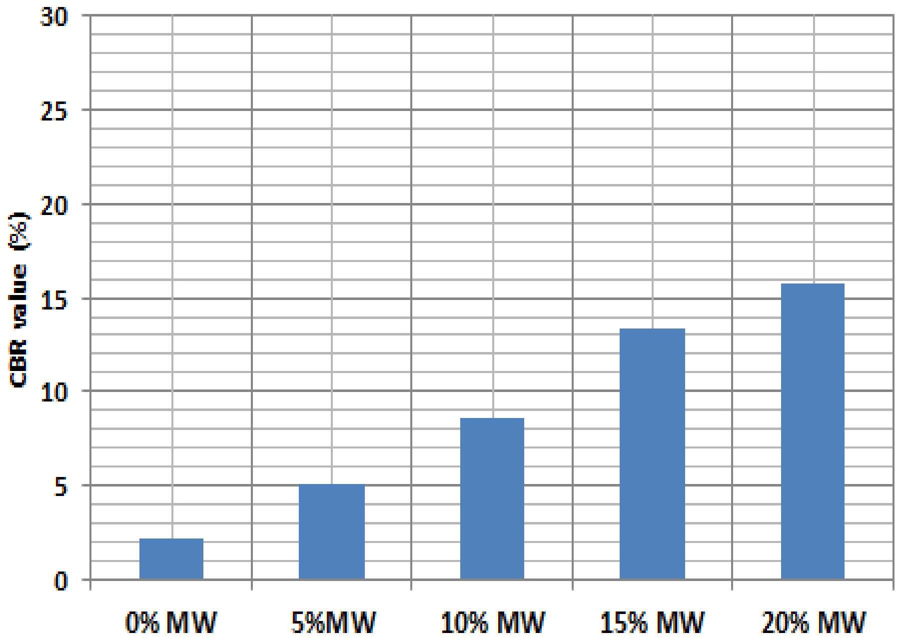

Figure 4. 9 Effect of marble waste on CBR value.

4.3.6 Summary of test result expansive soil stabilized by marble waste.

Soil description	LL(%)	PL(%)	PI(%)	UCS (kPa)	CBR(%)	Free swell(%)
Natural soil	94.8	40	54.8	180.75	2	125
5%mw+soil	92	58	34	280	5.3	97
10%mw+ soil	90	50	40	360	8.7	72
15%mw+soil	72	41	31	510	13.4	61
20%mw+ soil	63	43	20	560	15.75	58

Based on the above stabilized laboratory test result, marble waste at 20% mw is an effective stabilizer on the strength test result. As a result, the marble waste treated expansive soil meets the minimum criteria specified by the Ethiopian Roads Authority pavement design manual (2002) specification for materials suitable for use as subgrade material, and it belongs to the S4 subgrade strength class, where the CBR value is greater than 15%. The unconfined compressive strength result was also greater than the minimum standard mentioned in the ERA manual and ASTM.

4.5 Pressure Plate Test Results

The SWCCs are a measure of the ability of soil to retain water under different suction levels. The working principle was based on the pressure plate using axis translation techniques. The outflow collection system was flushed to remove any air bubbles during the saturated specimen condition. The initial pressure was applied and measured. The moist soil was recorded until outflow stopped (i.e., the soil specimen was in equilibrium with the applied pressure). Then they measured gravimetric water content, degree of saturation, volumetric water content, and void ratio at each applied suction pressure. This process was repeated from 33 kPa until 1400 kPa was achieved. Expansive soil samples were mixed with various percentages of marble prepared at optimum water content, using standard and modified Proctor compaction efforts.

Table 4. 6 Compaction effort on both treated and untreated soil sample on TP-1

% of marble waste	Modified compaction		Standard compaction	
	OMC (%)	MDD (g/cm3)	MDD (g/cm3)	OMC (%)
0%	22	1.34	1.36	20
5%	31	1.52	1.47	26
10%	33	1.55	1.54	30
15%	32	1.68	1.6	18
20%	28	1.82	1.69	21

Table 4.7 A summary of measured gravimetric water content for untreated soil samples.

Suction(kPa)	Undisturbed	Modified compaction	Standard compaction
33	51.33	42.3	45.8
200	44.21	40.44	39.8
300	38.46	38.88	34.36
500	33.92	35.73	31.23
800	31.21	32.87	27.34
1000	29.34	29.4	26.11
1200	28.78	28.53	25.41
1400	27.86	27.62	24.87

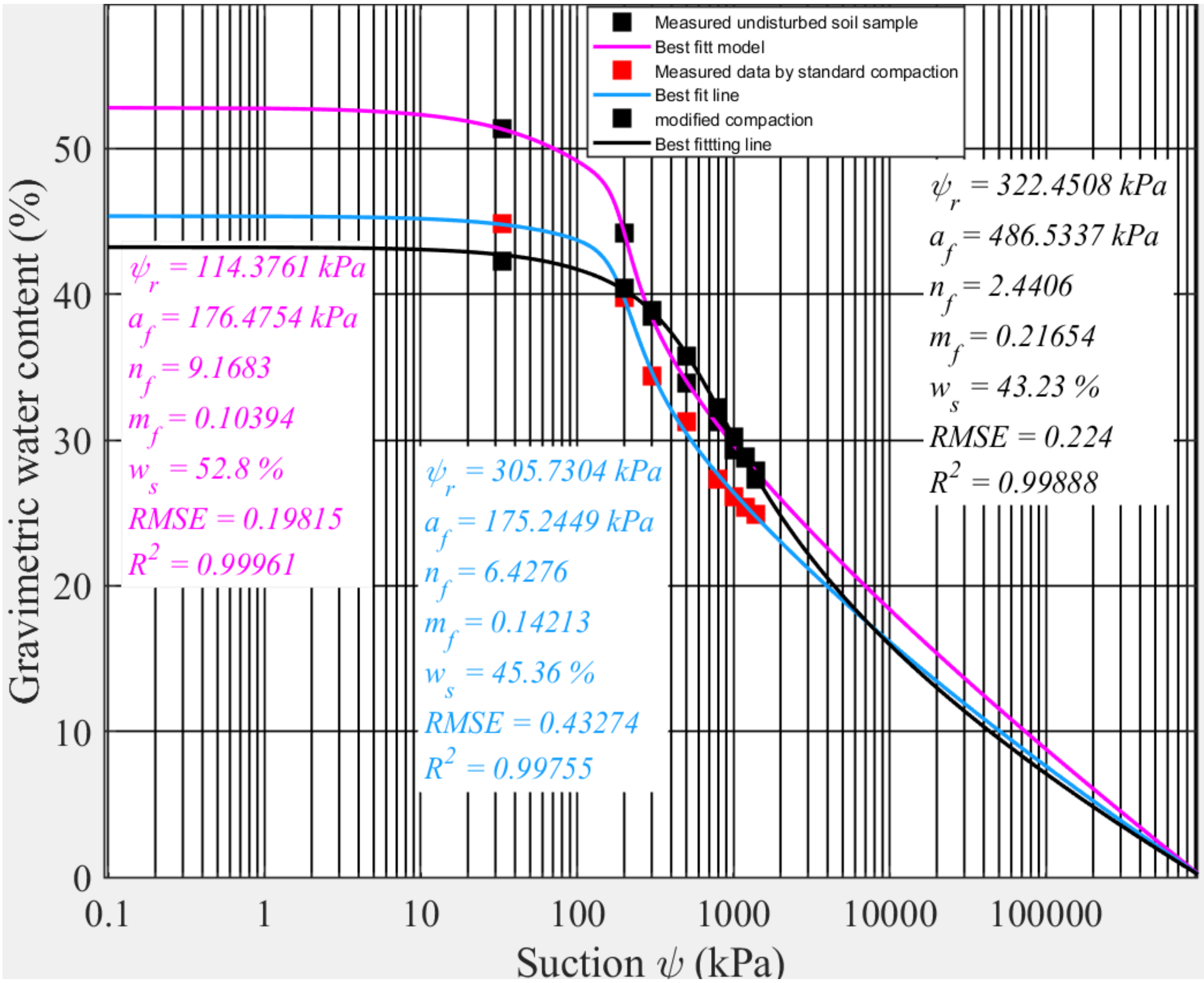

Figure 4.10 Combined fitting model for SWCC on undisturbed, Standard & Modified test

Fitting parameter	Undisturbed soil sample	Modified compaction	Standard compaction
af (kPa)	176.47	486.5	175.24
n	9.16	2.44	6.42
m	0.10	0.216	0.14
Wsat	52,8	43.23	45.36
ψr(kPa)	114.37	322.45	305.73
SSE	0.122	0.78	1.43
RMSE	0.198	0.22	0.432
AEV(kPa)	100	110	200

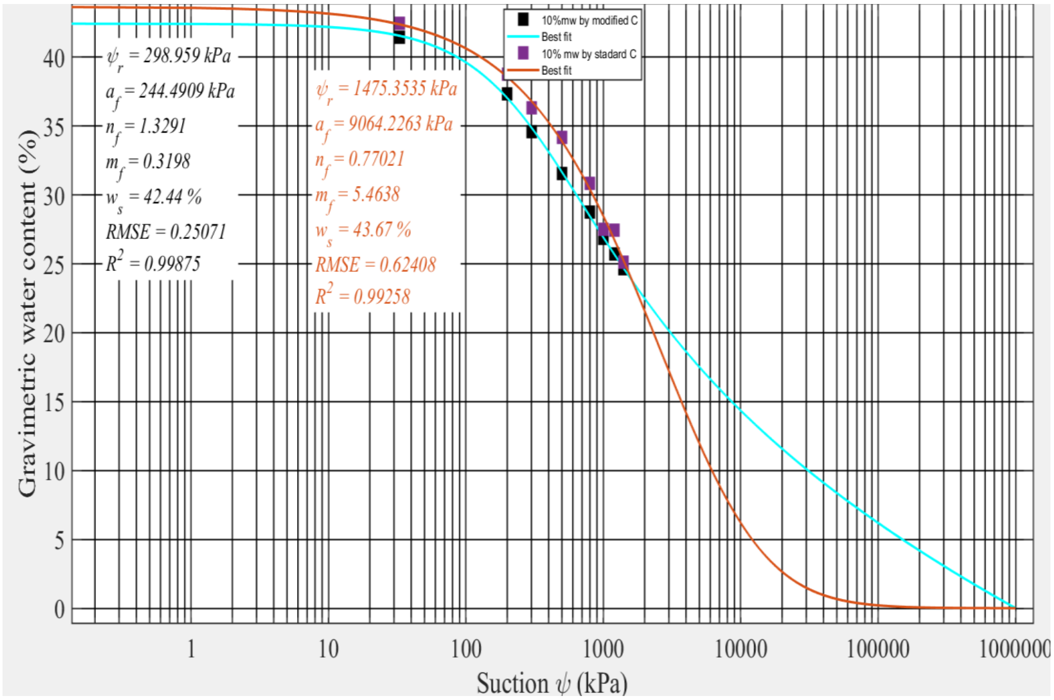

Figure 4.11 SWCC model fitting by 10 % mw standard & modified effort.

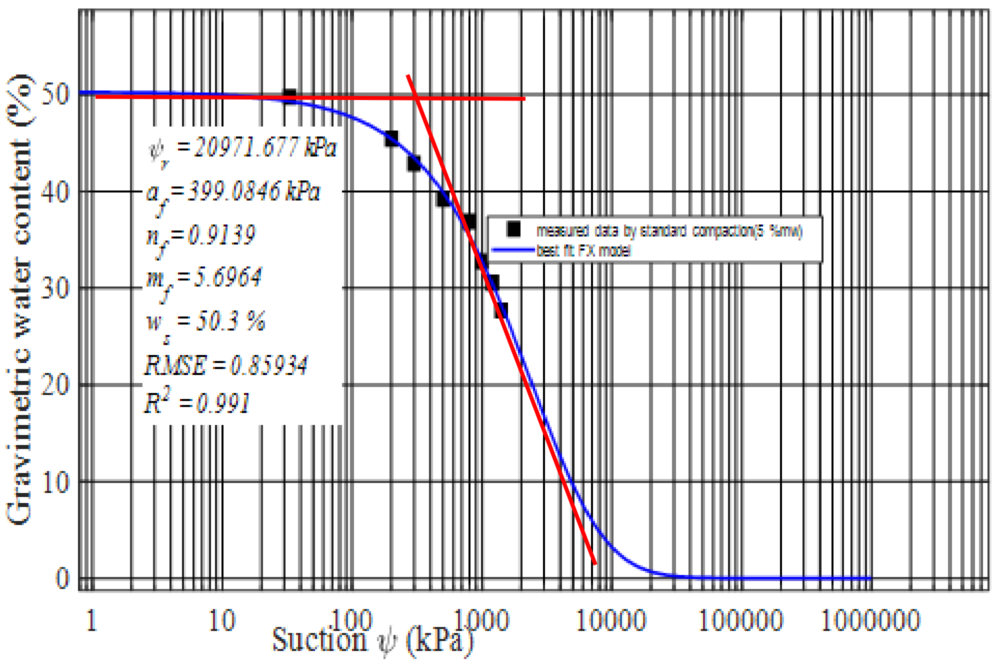

Figure 4.12 SWCC fitting model on modified compaction effort

Table 4.8 Measured Gravimetric Water Content from the pressure plate test result.

modified compaction compaction energy								
Suction (kpa)	At different marble waste with 7 day curing period							
	5%mw		10%mw		15%mw		20%mw	
	Measured	predicted	Measured	predicted	Measured	predicted	Measured	predicted
	(θg)	(θg)	(θg	(θg)	(θg)	(θg)	(θg)	(θg)
33	49.78	50.23	40.44	41.11	38.82	39.11	34.56	34.62
200	47.67	47.44	37.34	37.87	35.23	35.72	31.78	31.91
300	46.73	46.68	34.45	34.62	30.34	30.66	29.43	29.18
500	45.33	45.61	30.66	30.22	28.67	28.58	27.77	27.34
800	44.23	44.34	27.86	28.12	27.22	27.83	26.54	28.49
1000	40.23	42.81	26.48	26.64	26.44	26.39	25.23	25.45
1200	38.88	39.2	25.72	25.43	24.58	24.28	23.81	23.22
1400	27.68	27.3	24.89	24.11	23.19	22.73	21.67	21.14
	Ws=50.23		**ws=42.44**		**ws=40.67**			**ws=37.34**

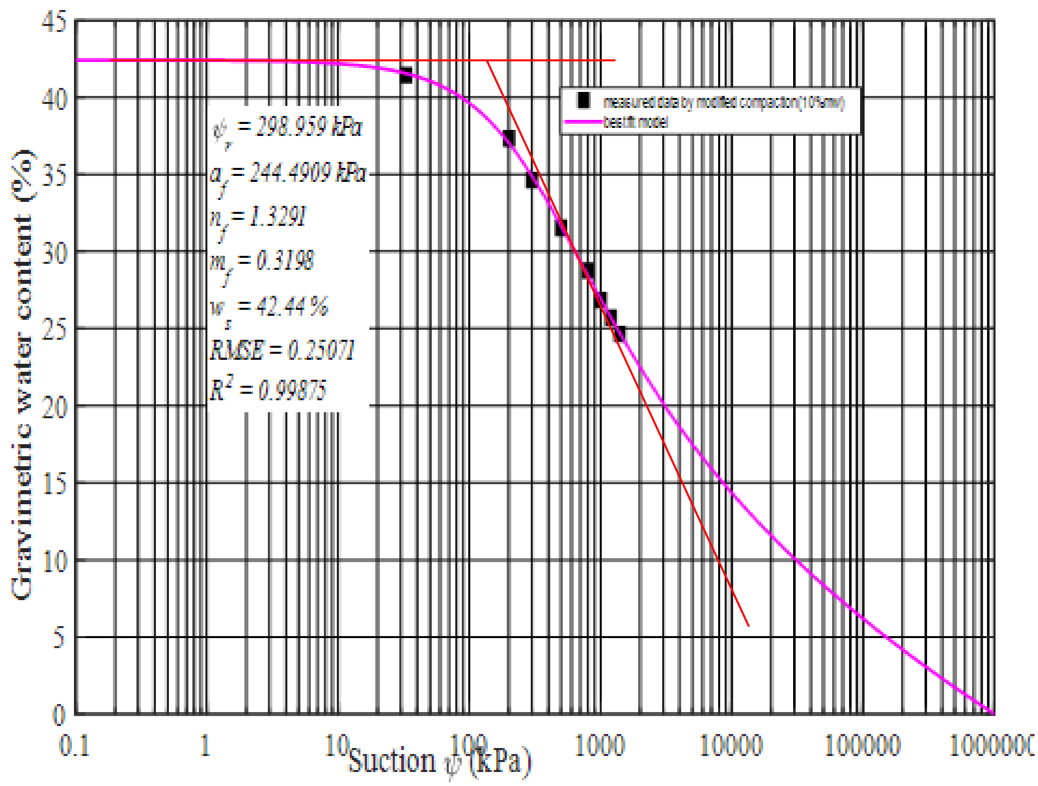

Figure 4. 13 SWCC fitting model with a 10% mw by modified compaction effort.

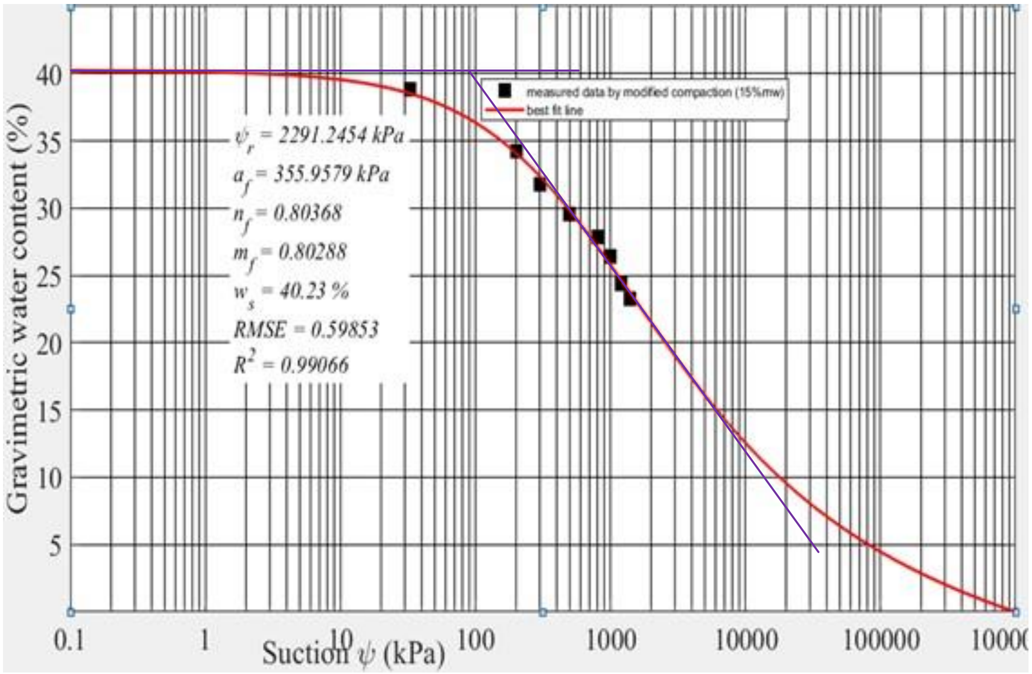

Figure 4.14 SWCC fitting model with 15% mw by modified compaction effort.

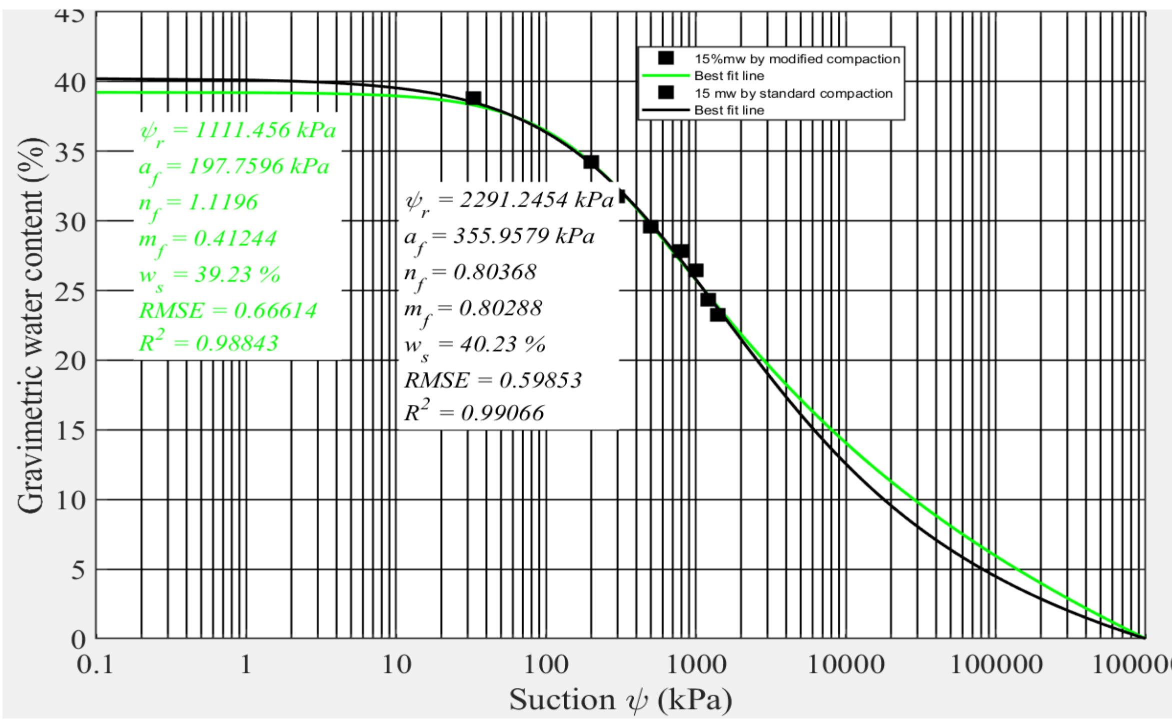

Figure 4.14 SWCC fitting model with 15% mw by modified& standard effort

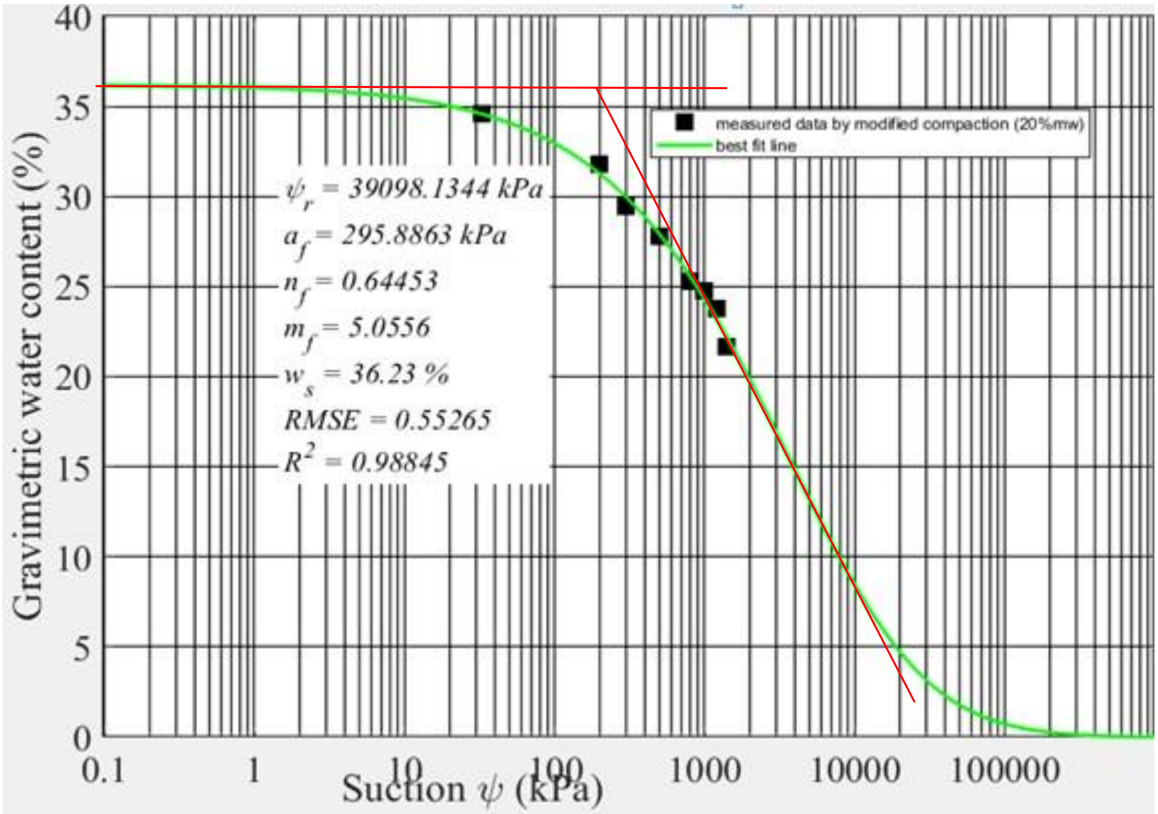

Figure 4. 14 SWCC fitting model with 20 % mw by modified compaction effort

Table 4.9 Measured Gravimetric Water Content obtained from the pressure plate test by standard compaction energy the treated soil sample.

Suction(bar)	**Measured Gravimetric water content**			
	5% mw	10% mw	15% mw	20%mw
33	52.12	42.23	39.39	36.67
200	47.67	38.72	36.78	34.83
300	44.43	36.13	34.31	30.75
500	35.67	32.76	29.21	27.64
800	32.34	30.56	27.83	25.92
1000	27.48	27.32	26.25	24.73
1200	26.71	26.24	25.87	24.13
1400	25.23	24.89	24.23	23.22

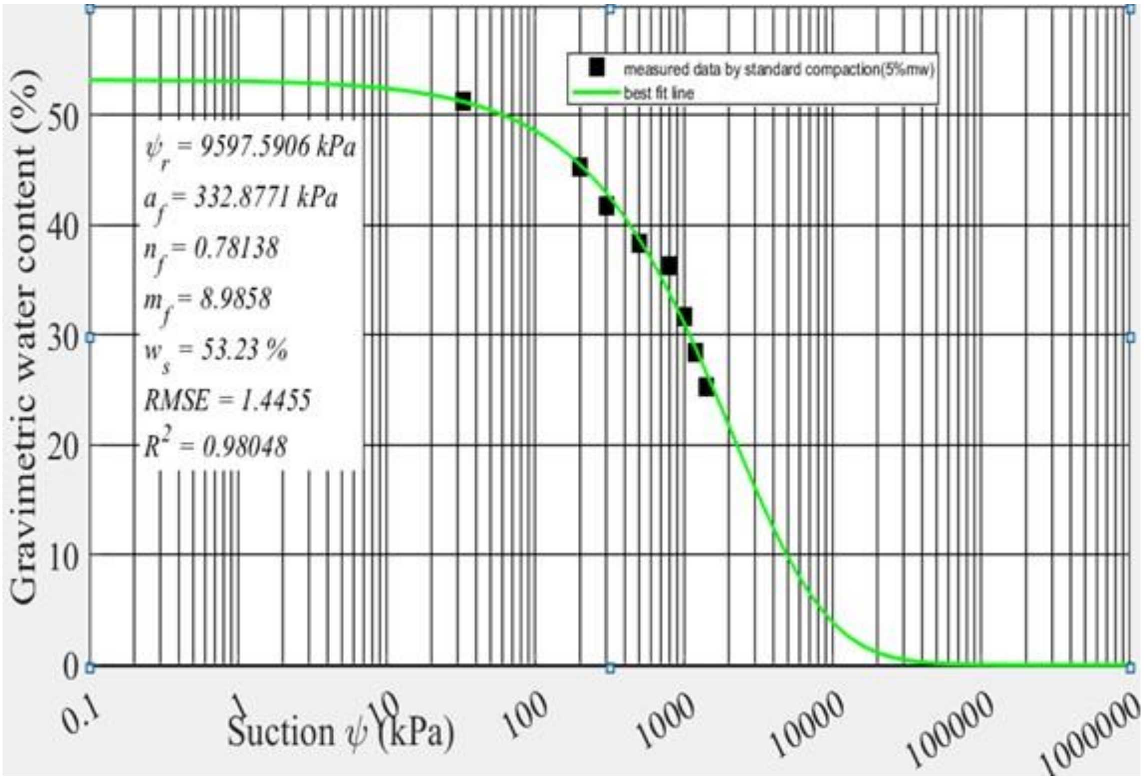

Figure 4.15 SWCC fitting model soil stabilized 5% mw by standard compaction effort

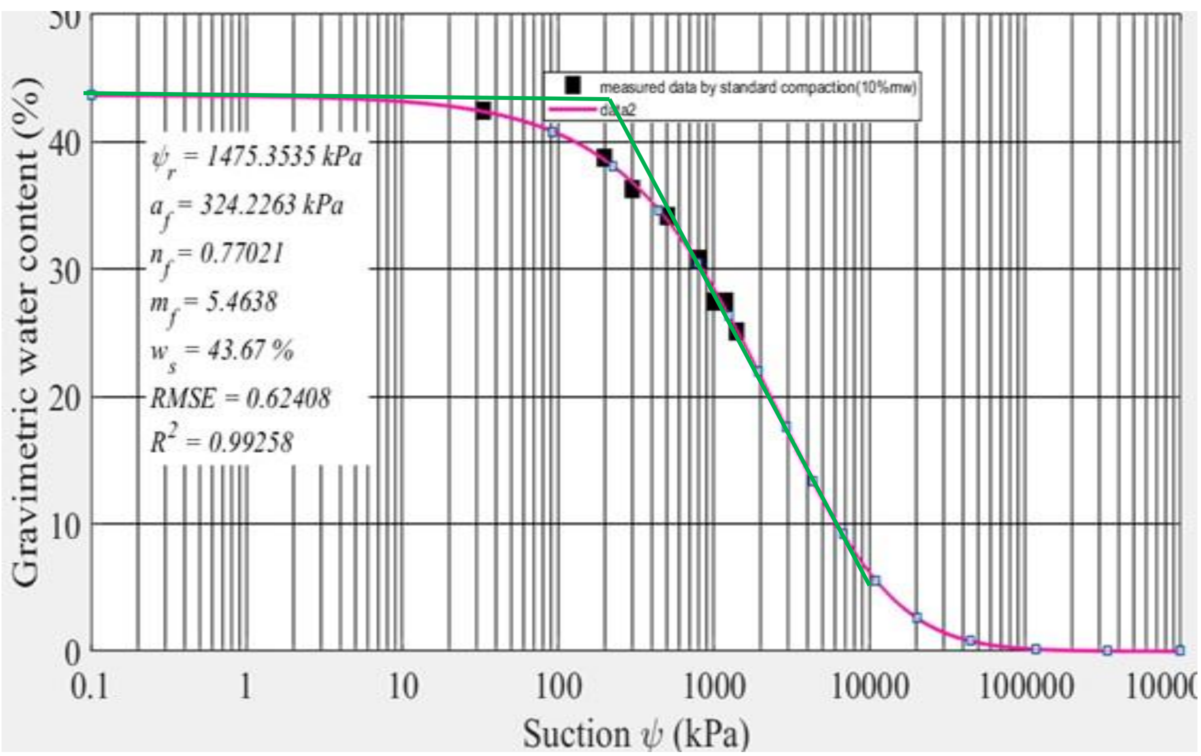

Figure 4. 16 SWCC fitting model soil stabilized 10% mw by standard compaction effort

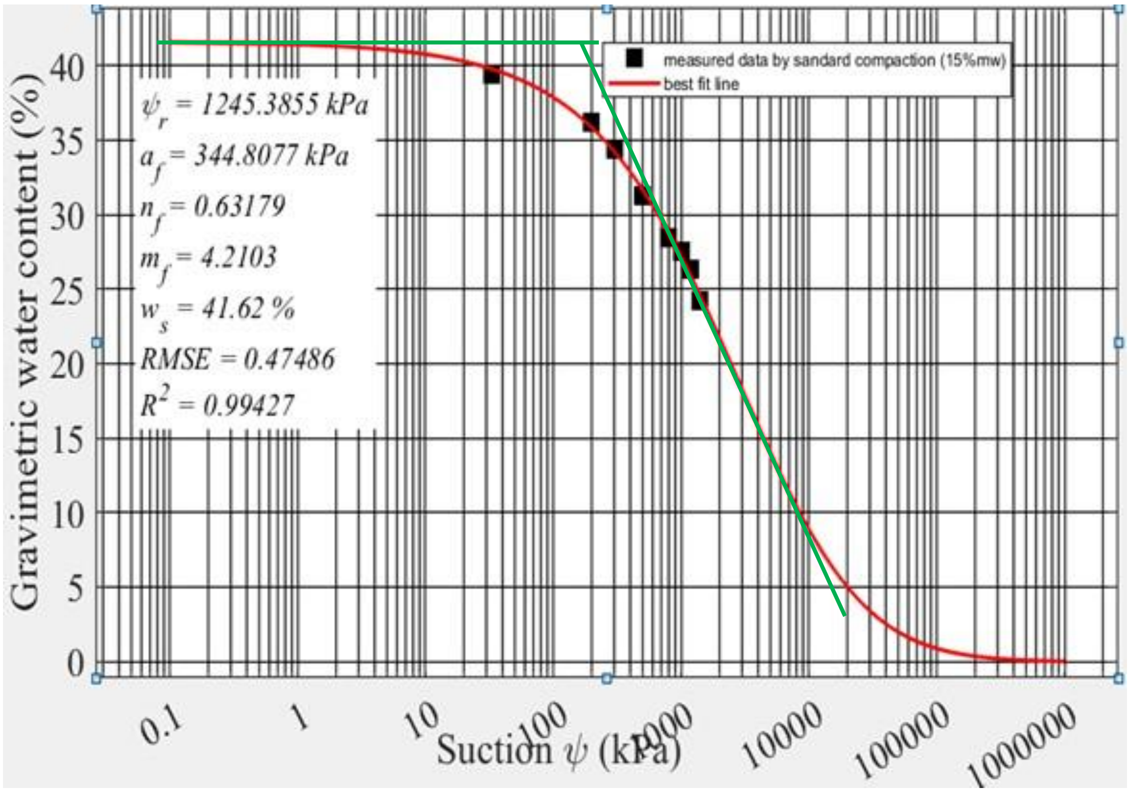

Figure 4. 17 SWCC fitting model soil stabilized with 15% mw by standard compaction effort

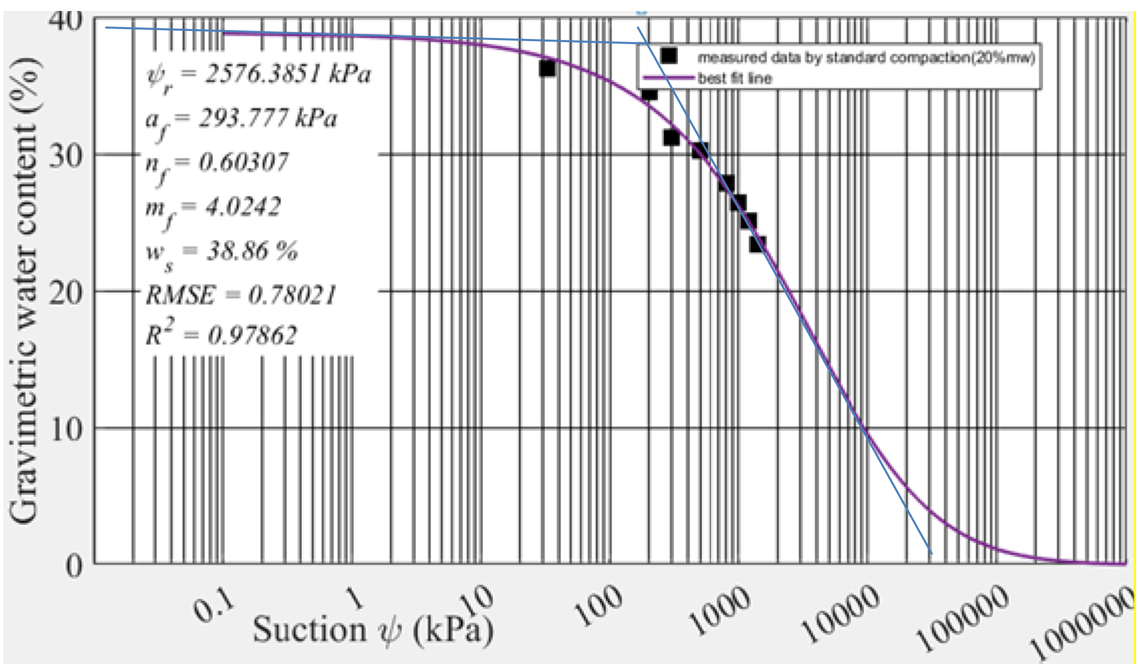

Figure 4.18 SWCC fitting model soil stabilized with 20% mw by standard compaction effort.

Table 4.10 Summary of SWCC fitting model parameters for measured data

Modified Compaction	Fitting parameter			
	Ws	a (kPa)	n	m
5% mw	50.3	399	0.91	5.69
10% mw	42.4	244.49	1.32	0.31
15% mw	40.23	355.95	0.8	0.8
20% mw	36.23	295.88	0.64	5.05
Standard Compaction				
5%mw	53.23	332.87	0.78	8.91
10%mw	43.67	324.22	0.77	5.46
15%mw	41.6	344.8	0.63	4.2
20%mw	38.86	293.77	0.61	4.02

4.10 Summary of SWCC parameter Test Results by Different Researchers

Table 4.11 Abdulrahman et al.'s (2014) gypsum stabilized soil SWCC parameter

		SWCC Parameters		
Gypsum + soil	Curing period	a	n	m
Natural soil	-	190	1.5	0.9
5% gypsum	7	200	1.7	0.78
15% gypsum	7	210	1.45	0.78
25% gypsum	7	240	1.28	0.81

Table4.12 Botao Lin (2012) soil water characteristics curve of parameters of untreated soil.

Test sample(CH)	AEV(Kpa)	**a** (Kpa)	n	m	Wr(kpa)
Carnisaw	43	77	1.48	0.10	10^6
Eagle ford	74	111	2.85	0.15	10^6
Hollywood	70	121	1.59	0.19	10^6
Heiden	55	112	1.06	0.23	10^6

Table 4.13 Botao Lin (2012)'s curve parameter for SWCC stabilized by fly ash.

Test sample(CH)	AEV(Kpa)	a (Kpa)	n	m	Wr(kpa)
Carnisaw	43	67	1.48	0.10	10^6
Eagle ford	23	44	1.51	0.12	10^6
Hollywood	7	11	1.77	0.1	10^6
Heiden	50	19	1.27	0.09	10^6

Table 4.14 Amir et al. (2018) SWCC parameters for natural and lime-treated soil

		SWCC Parameters		
Lime + soil	Curing period	a	n	m
Natural soil	-	283.7	0.68	0.26
3% lime	7	277	0.9	0.22
5% lime	7	317	0.92	0.18
7% lime	7	385	1.56	0.18

4.11 Result discussion on SWCC of Compacted and Stabilized Soil

The SWCC for marble treated soil samples is determined at 5%, 10 %, 15%, and 20 % by dry mass content with a 7-day curing period. As shown the above the measured data value and Fredlund and Xing (1994) model fit are used, and the corresponding data are presented in the Tables. The SWCCs are considered only the dried portions of untreated and marble-treated soil samples compacted by optimum parameters and are presented as a relationship between gravimetric water content and suction (w-SWCC). The data points represent measured experimental data by solid and dot lines represent the best-fit curves using Fredlund and Xing‟s (1994) equation respectively. The fitting parameters are obtained using the matlab program. In SWCCs, both AEV and residual water content are obtained using a free-hand sketch as shown in Figure 4.18. The w-SWCCs for marble-treated soils show a definite AEV residual suction and residual water content. W-SWCC for marble treated soils shows a sharp desaturation slope between the AEV and residual suction. The w-SWCC of the marble-treated soil sample as the content of marble increases with respect to the untreated natural soil sample is attributed to the lower initial water content of the marble-treated sample with a reduced rate of desaturation. During the saturation stage, higher water uptake took place in the standard compaction effort than in the modified compaction effort. Since the comparatively looser state of the soil sample. This is due to the fact that marble-treated soils have voids filled by fine particles and less absorption of water at saturation stage than standard compaction effort of soils. According to Zhang et al. (2017), a reduced AEV implies lower water retention at low suctions. This is due to the flocculation and chemical bonding creating an open structure. The effect of stabilization and compaction on the SWCC of fitting parameters ("a,n,m) is directly related to the air entering level, the rate of desaturation or the water retention property of the soil. The "n" value of the compacted soil sample is smaller compared to the undisturbed soil sample, implying that the compacted soil sample shows a low rate of desaturation (i.e. water is removed from the pores at a slower rate than the natural state). (The n value 9.16, 6.42, 2.44 undisturbed, standard, & modified respectively.) The fitting parameter "m" is related to the curvature near residual conditions. The "m" values are 0.1, 0.21 and 0.14 for undisturbed modified and standard compacted soil samples, respectively. According to Fredlund and Xing (1994), low values of "m" indicate moderate slopes of the SWCC.

This shows soil structure and compaction effects has a significant change in "m" value compared to the other fitting parameters, "a" and "n."

For soil compacted with standard and modified Proctor efforts at optimum water content, slightly higher air-entry suction exists for the soil compacted with modified Proctor effort. Higher compactive efforts also result in a curve that appears to be slightly flatter. The residual suction corresponds to the residual water content, which is defined as the water content beyond which a significant increase in suction is accompanied by a small change in water content (Fredlund et al., 2011). The residual suction values are 114.37 kPa, 305.73kPa, 322.45 kPa for undisturbed standard and modified compacted soil samples, respectively. This difference in residual water content is due to the initial saturated water content difference between disturbed and compacted soil samples. When the specimen is compacted at a higher compaction degree, the slope of the SWCC and the rate of desaturation is smaller.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The following conclusions can be drawn from the test results of this study:

- Expansive soil stabilization by using marble waste not only improved expansive soil problem but to reduce waste disposal problems.
- The Liquid limit, free swell ratio, linear shrinkage, compaction parameter, CBR & UCS value of expansive soil were improved by increasing the percentage of marble waste content.
- The strength test value of expansive soil (CBR & UCS) improved by at least 15% MW to meet the ERA standard's minimum requirement.
- The engineering properties of soil were significantly improved by compaction energy and treatment mechanism.
- The gravimetric water content of the marble-treated soil compacted with modified and standard energy gravimetric water content decreased with an increase in the percentage of marble as well as a low rate of desaturation (n).
- The effects of marble waste stabilization with different compaction energy on the SWCCs are analyzed. The results indicate that there is a change in the SWCC parameters as the treatment percentage is changed.
- Soils compacted with standard and modified effort at OMC slightly higher air entry suction exist for soils compacted by modified effort. Higher compaction effort also results in a curve that appears to be slightly flat curve.
- In genera impact of compaction energy on SWCC of stabilized and un stabilized soil was observed to have a significant influence on the SWCC response.

5.2 Recommendations

- In addition to the growing cost of stabilizers, industrial waste is the main problem with related to the disposal system. So, the research evaluation is a critical action for the light weight construction and waste disposal system. This research is one of the platforms to recommend that using locally available industrial waste as stabilizers in construction is a useful technology to save time and cost for waste disposal system. This research shows that the SWCC behavior can be used to understand the stabilization effects on expansive soils and that SWCC is influenced by the compaction energy, compaction parameter, and dosage of stabilizers. The studies can be further extended to better understand the engineering behavior of unsaturated expansive soils stabilized by different industrial and agricultural wastes. In addition, the following specific recommendations have been made for future research.
- Further detailed investigation is recommended effect of stress history evaluation on use of SWCC as a tool to check improvement effect on expansive soils.
- The effect of compaction energy and compaction parameters on both SWCC and shrinkage curve behavior on the combined effect on SWCC need to be investigated.
- More studies can be needed to enable better interpretation of engineering properties and behavior such as shear strength, permeability, and consolidation on different effects of temperature and sampling depth on SWCC of stabilized soils.
- The effect of swelling pressure and swelling properties on the AEV based on different models.

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APPENDIX

Appendix A: Natural Soil Laboratory Test Results

a). Natural moisture content test results

Table A1:- Moisture content determination for test pit-1

	Can no	A1	xz	34
A	Can weight(g)	22.6	22.43	12
B	Can plus wet soil(g)	55.34	64.22	39
C	Can plus dry soil(g)	45.86	52.13	31.7
D=B-C	Mass of water (g)	9.48	26.03	7.7
E=C-A	Mass of dry soil(g)	23.3	29.52	19.3
D/E	Moisture content (%)	40.8	40.75	39.7

Average moisture **40.34**

Table A2:- Moisture content determination for test pit-2

	Can no	zz	aa	zd
A	Can weight	22.64	22.67	12.5
B	Can plus wet soil	89.2	95.25	63.51
C	Can pluse dry soil	69	73.4	48.39
D=B-C	Mass of water	20.13	22	15.2
E=C-A	Mass of dry soil	46.42	50.83	36.6
D/E	Moisture content	43.36	42.8	43.35
	Average moisture =**42.53**			

Table A3:- Moisture content determination for test pit-3

	Can no	53	RR	RD
A	Can weight	22.61	22.61	22.34
B	Can+wet soil	95.31	96.83	122.24
C	Can+dry soil	72.9	74.4	96.45
D=B-C	Mass of water	22.41	22.38	30.8
E=C-A	Mass of dry soil	50.32	51.83	69

D/E	Moisture	44.5	43.16	44.53

a) Grain size distribution test results

Table A4:- Sieve Analysis for test pit-1

Mass of soil before washing = 500g

Mass of soil after washing = 45 g

Sieve size	Sieve wt	S +sample	Wt retained	Percent retained	Cumulative retained	% of passing
4.75	346	346	0	0	0	100
2.36	335.2	335.2	0	0	0	100
2	319.36	319.36	0	0	0	100
1.18	280.68	288.68	0	0	0	100
0.6	393.36	413.54	20.18	4.03	4.03	95.96
0.425	377.74	382.01	4.27	0.85	4.89	95.1
0.3	262.2	264.93	2.84	0.56	5.46	94.53
0.15	265.77	276.2	10.71	2.14	7.6	92.39
0.075	256.87	261.27	4.4	0.88	8.45	91.52
pan	263.35	266.75	457.28	91.51	100	0

Table A4:- Sieve Analysis for test pit-2 before wash=500 gram, after wash 53.45 gram

Sieve Opening (mm)	Mass of Sieve (g)	Mass of sieve with Retained soil (g)	Mass of Retained soil (g)	Percent Retained (%)	Cumulative Percent Retained (%)	Percent Finer (%)
4.75	412.39	412.39	0	0	0	100
2.36	335.27	335.27	0	0	0	100
2	319.48	323.14	3.66	0.73	0.73	99.27
1.18	280.68	287.93	7.25	1.45	2.18	97.82
0.6	393.36	414.54	21.18	4.24	6.42	93.58
0.425	377.75	382.01	4.26	0.85	7.27	92.73
0.3	262.09	264.93	2.84	0.57	7.84	92.16
0.15	265.79	274.48	8.69	1.74	9.58	90.42
0.075	256.86	262.43	5.57	1.11	10.69	89.31
Pan	263.35	266.75	446.55	89.31	100	0
Sum			500	100		

53.45

Table A5:- Sieve Analysis for test pit-3

Mass of soil before washing = 500g

Mass of soil after washing = 135.8gram

S size(mm)	S weight	WS+soil	Weight retained	Percent retained	Cumulative retained	% of passing
4.75	567	567	0	0	0	100
2.36	334.4	338	3.6	2.65095729	2.65095729	97.349
2	334.6	337.3	2.7	1.98821797	4.63917526	95.3608
1.18	353.2	358	4.8	3.53460972	8.17378498	91.8262
0.6	318.2	328.2	10	7.36377025	15.5375552	84.4624
0.425	265.2	269.4	4.2	3.09278351	18.6303387	81.3697
0.3	262	273	11	8.10014728	26.730486	73.2695
0.15	265.6	278	12.4	9.13107511	35.8615611	64.1384
0.075	264.4	273.2	8.8	6.48011782	42.3416789	57.6583
pan	300.3	378.6	78.3	57.6583211	100	0
			135.8	100		

b)Free swell test result

Table A6-free swell ration test result

station	intial value	final value	(vf-vi)/vi*100
Station 1	10	21	**110**
Station-2	10	18	80
Station -2	10	21	110
Station-3	10	22	120
Station-4	10	20	100

Table A7 Linear shrinkage test result

Test pit	No-1	No-2	No-3	No-4	No-5
Intial value	14	14	14	14	14
Final value	10.5	11.5	13	12	11.5
Linear shrinkag	33.3%	21.7%	7.6%	16.6%	21.7%

	trial	1	2	3	4
	blows	18	24	28	31
	Can no	k	m	a	34
A	Mass of can	12	22.9	23.4	22.9
B	Can +wet soil	18.83	33.17	32.1	27
C	Can+ dry soil	15.2	28.14	27.86	25.3
D=B-C	Mass of water	3.3	5.03	4.16	2.12
E=C-A	Mass of dry soil	3.4	5.22	4.44	2.34
D/E	Moisture content	97.3	96	93	90.23

Average moisture=94%

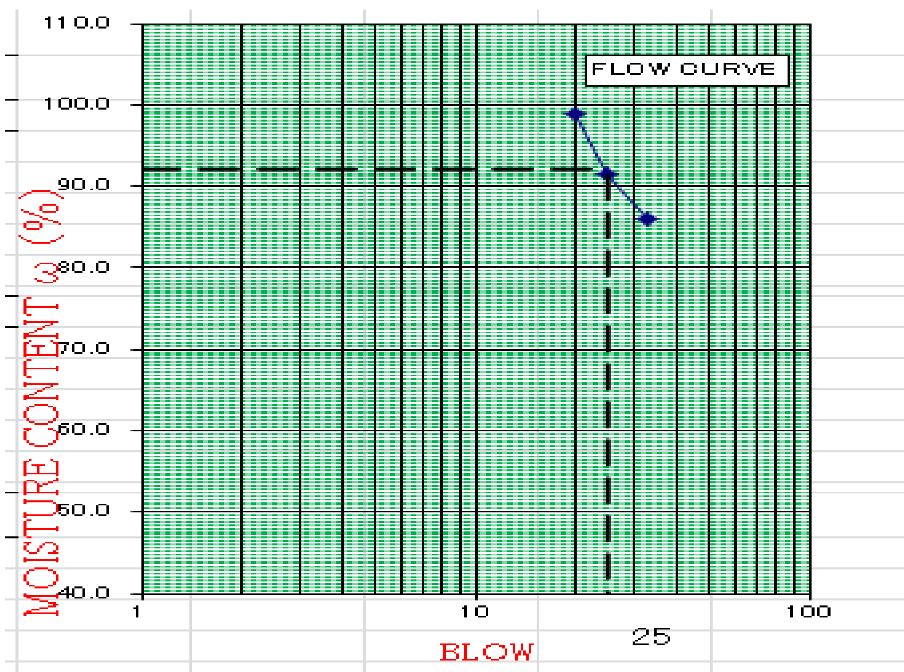


Figure d1 liquid limit test result

Table A9- Liquid limit test result for test pit-2

	Test Trail	1	2	3	4
	Number of blows	17	19	23	27
	Can no	F	R	12	M
A	Mass of can	4.37	13.43	13.5	13.67
B	Can+ wet soil	13.57	22.43	24.48	20.3
C	Can+ dry soil	9.12	18.4	19.2	17.1
D=B-C	Mass of water	4.45	4.29	5.22	3.2
E=C-A	Mass of dry soil	4.75	4.71	5.79	3.5
D /E	Moisture content	93.4	91	90.2	89

Average liquid limit =90%

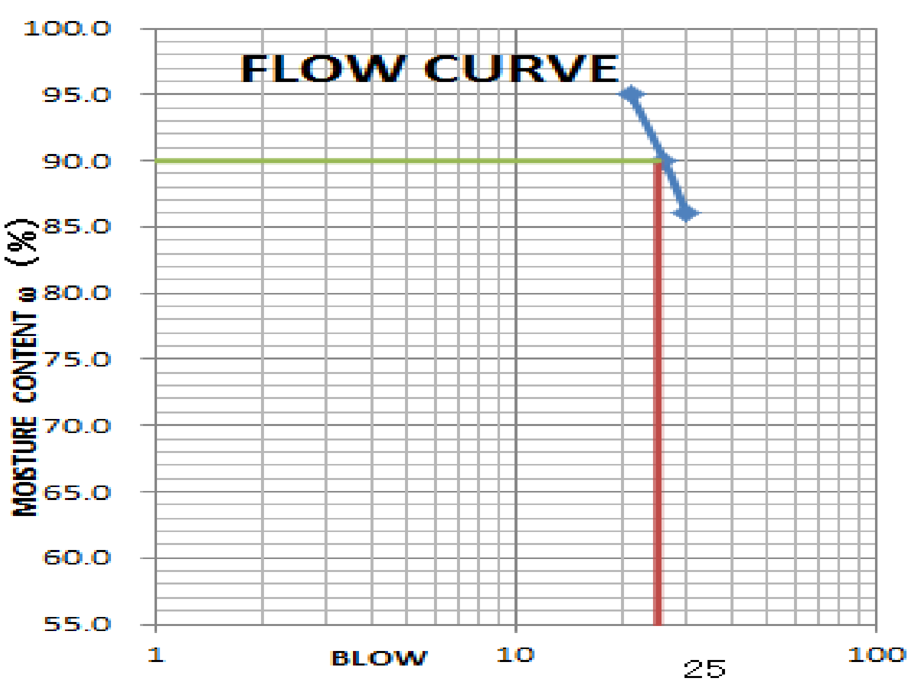


Figure d2- liguid limit test result

Table A10 Liquid limit test result for test pit-3

	Trial test	1	2	3	4
	No of blows	16	20	24	28
	Can no	9	z	1	l
A	Can wt	22.32	13.4	26.63	22.56
B	Can+wet soil	31.21	22.66	30.16	32.46
C	Can+dry soil	26.85	18.1	26.54	27.75
D=B-C	Mass of water	4.36	4.51	3.62	4.71
E=C-A	Mass of dry soil	4.53	4.82	3.92	5.17
D/E	Moisture content	97	93	92	91

Average liquid limit =93%

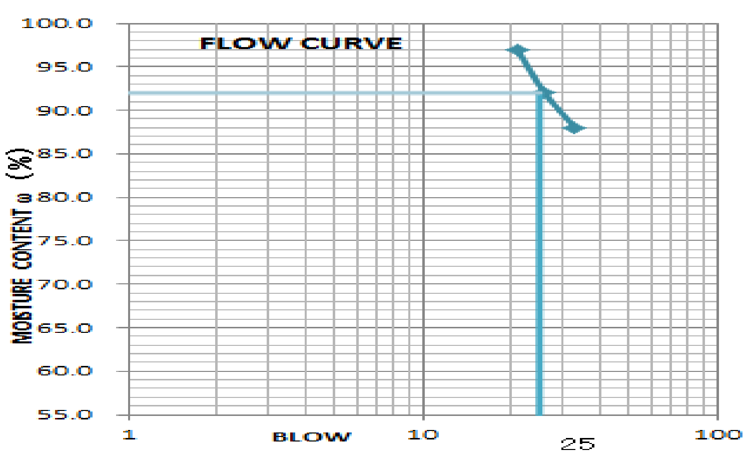


Figure -d3 liquid limit test result

Test pit 11 plastic limit result for pit -1

	Test trial	1	2
	Can no	sz	lm
A	Mass of can	13.2	12
B	Mass of can+ wet soil	22	15
C	Mass of can +dry soil	18.78	18.88
D=B-C	Mass of water	2.34	2.23
E=C-A	Mass of dry soil	5.61	5.67
D/E	Plastic limit	40.3	40.21

c) **Compaction test result (moisture determination)**

Table A.12:- Compaction test results for test pit-1

Can no	A3	**AS**	RT	ER
Mass of can	41.2	40.4	41	42
Can +w soil	180	180	180	180
Can+ dry soil	160	158	156	154
Moisture (%)	19.4	24	27	35

Density determination					
Mass of mold	Volume of soil	Mass of mold +soil	Mass of soil	Bulk density	Dry density
5036	2123	9011	3872	1.87	1.56
		9380	4341	2.04	1.62
		9490	4451	2.1	1.64
		9395	4356	2.05	1.52

Average specific gravity of expansive soil

Test trials	1	2	3
Mass of Density bottle (g)	60.94	55.04	51.77
Mass of Density bottle + Dry soil (g)	85.95	79.96	76.56
Mass of Density bottle + Dry soil + Water (g)	180.17	192.97	189.72
Mass of Density bottle + Water (g)	164.52	177.34	174.13
Temperature (Tx 0c)	24	24	24
Specific gravity @Tx, G	2.672	2.683	2.695
Average Specific gravity,	2.68		

Table A. 13:- standard Compaction test results for test pit-1

moisture content determination				
Can no	10	23	35	78
Can wt	13.7	14	22	20
Mass of can+wet soil	108	118	119	123
Mass of can+dry soil	91.58	97.49	96.23	96.26
Moisture content	21.24	27.36	30.94	36.3

Density determination

Mass of mold	Volume of soil	Mass of mold +soil	Mass of soil	Bulk density	Dry density
4796	944	6214	1418	1.52	1.24
		6497	1702	1.8	1.44
		6604	1808	1.92	1.45
		6579	1783	1.88	1.38

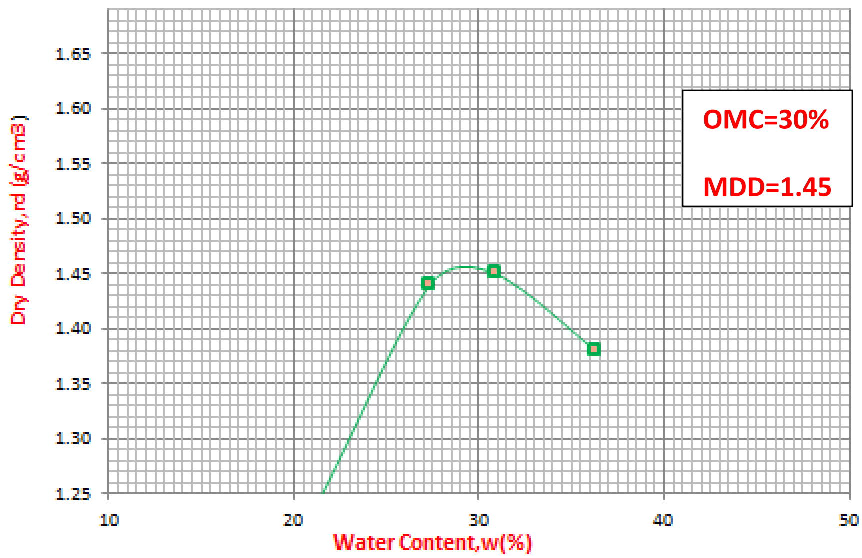


Figure e-2 Standard compaction test graph

Table A.14-standard Compaction test results for test pit-3

Moisture content determination

Can no	23	6	45	m
Can wt	22.4	11.83	22.34	22.51
Can+soil	102.93	117.73	122.46	102
Can+dry soil	89.56	100	99.35	80.9
Moisture content	19.9	20.43	26.9	36.55

Density determination

Mass of mold	Volume of soil	Mass of mold +soil	Mass of soil	Bulk density	Dry density
4796	944	6252	1456	1.54	1.28
		6579	1783	1.89	1.45
		6628	1824	1.92	1.49
		6571	1781	1.88	1.37

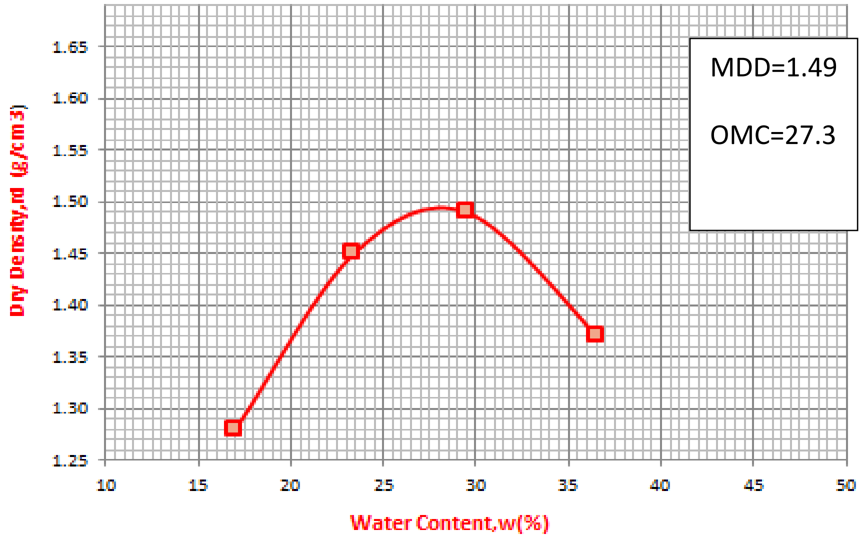


Figure e-3 Standard compaction test graph

E) Unconfined compressive strength test results (TP1)

Shape of specimen cylindrical

Diameter of specimen 38mm

Intial area of specimen AO=1136.5

Initial height of specimen 76mm

Deformation Dial ,chan. L (mm)	**strain**	**Strain(%)**	**Corrected area**	**Proving Ring Dial in devision**	**Applied Axial Load P(N)**	**Load per Unit Area N/mm^2**	**KN/m^2**
1	2 = chan.L/Lo	0	3 =Ao/1-E	4	5 =4*44.48	6 =5/3	
0.2	0.0027	0.2597	1139.1	0.25	11.20	0.00983	11.76
0.4	0.0054	0.5195	1142.2	0.5	22.40	0.01961	19.61
0.6	0.0081	0.7792	1145.3	0.85	38.08	0.03325	33.25
0.8	0.0108	1.0390	1148.4	1.35	60.48	0.05266	52.66
1	0.0135	1.2987	1151.6	1.5	67.20	0.05836	58.36
1.2	0.0162	1.5584	1154.7	1.95	87.36	0.07565	61.23
1.4	0.0184	1.8182	1157.3	2.3	103.04	0.08903	68.34

1.6	0.0211	2.0779	1160.4	2.57	115.14	0.09922	77.34
1.8	0.0237	2.3377	1163.6	2.83	126.78	0.10896	83.23
2	0.0263	2.5974	1166.7	3.17	142.02	0.12172	89.67
2.2	0.0289	2.8571	1169.9	3.5	156.80	0.13403	94.56
2.4	0.0316	3.1169	1173.0	3.73	167.10	0.14245	99.87
2.6	0.0342	3.3766	1176.2	4.19	187.71	0.15959	107.98
2.8	0.0368	3.6364	1179.5	4.4	197.12	0.16713	115.23
3	0.0395	3.8961	1182.7	4.67	209.22	0.17690	122.56
3.2	0.0041	4.1558	1140.7	4.85	217.28	0.19048	134.32
3.4	0.0447	4.4156	1189.2	5.1	228.48	0.19213	145.67
3.6	0.0474	4.6753	1192.5	5.4	241.92	0.2028703	155.67
3.8	0.0500	4.9351	1195.8	5.56	249.088	0.2083042	163.56
4	0.0526	5.1948	1199.1	5.9	264.32	0.2204299	177.33
4.2	0.0553	5.4545	1202.5	6	268.8	0.2235434	180
4.4	0.0579	5.7143	1205.8	6.45	288.96	0.2396397	179.4

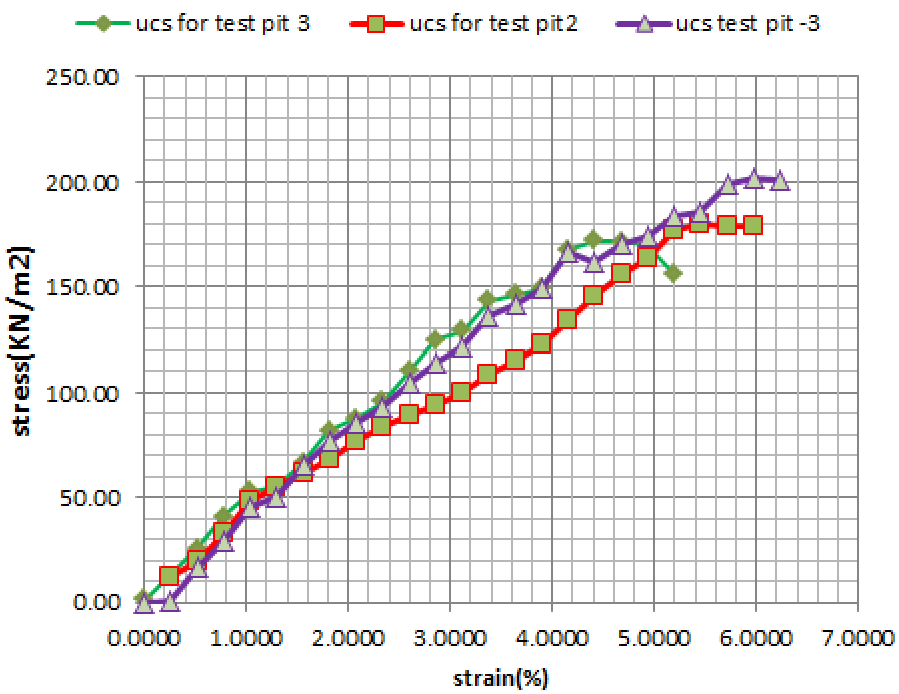


Figure –e3 Un confiend compressive test result

Table A16 un confiend compressive strength test result (TP-2)

Deformation Dial ,chan. L (mm)	Unit strain ,E	% strain	Cross sectional Area, A (mm^2)	Proving Ring Dial in division	Applied Axial Load P(N)	Load per N/mm^2	Unit Area KN/m^2
1	2 = chan.L/Lo	0	3 =Ao/1-E	4	5 =4*44.48	6 =5/3	0
0.2	0.0026	0.2597	1139.0	0.25	11.20	0.00983	0.86
0.4	0.0053	0.5195	1142.0	0.5	22.40	0.01961	17.18
0.6	0.0079	0.7792	1145.0	0.85	38.08	0.03326	29.04
0.8	0.0105	1.0390	1148.1	1.35	60.48	0.05268	45.88
1	0.0132	1.2987	1151.1	1.5	67.20	0.05838	50.71
1.2	0.0158	1.5584	1154.2	1.95	87.36	0.07569	65.57
1.4	0.0184	1.8182	1157.3	2.3	103.04	0.08903	76.93
1.6	0.0211	2.0779	1160.4	2.57	115.14	0.09922	85.50
1.8	0.0237	2.3377	1163.6	2.83	126.78	0.10896	93.65
2	0.0263	2.5974	1166.7	3.17	142.02	0.12172	104.33
2.2	0.0289	2.8571	1169.9	3.5	156.80	0.13403	114.57
2.4	0.0316	3.1169	1173.0	3.73	167.10	0.14245	121.44
2.6	0.0342	3.3766	1176.2	4.19	187.71	0.15959	135.68
2.8	0.0368	3.6364	1179.5	4.4	197.12	0.16713	141.70
3	0.0395	3.8961	1182.7	4.67	209.22	0.17690	149.57
3.2	0.0041	4.1558	1140.7	4.85	217.28	0.19048	166.98
3.4	0.0447	4.4156	1189.2	5.1	228.48	0.19213	161.56
3.6	0.0474	4.6753	1192.5	5.4	241.92	0.2028703	170.12
3.8	0.0500	4.9351	1195.8	5.56	249.088	0.2083042	174.20
4	0.0526	5.1948	1199.1	5.9	264.32	0.2204299	183.83
4.2	0.0553	5.4545	1202.5	6	268.8	0.2235434	185.91
4.4	0.0579	5.7143	1205.8	6.45	288.96	0.2396397	198.74
4.6	0.0605	5.9740	1209.2	6.6	295.68	0.2445278	202.22
4.8	0.0632	6.2338	1212.6	6.89	308.672	0.2545572	200.93
5	0.0658	6.4935	1216.0	7	313.6	0.2578947	200.32

Table A17 **California Bearing ration test result for pit 1**

MDD =1.54 OMC=28%

No of blows	10	30	60
Mold no	3	4	5
Mold volume	2123	2123	2123
Mass of mold	6151	6460	6134
soil +mold	10023	10603	10403
Mass of soil	3872	4145	4253
Bulk density	1.826	1.93	2.01
Moisture determination			
canno	12	a	23
canwt	36	36	36
Can+wet soil	153.5	162.42	168.8
Can+dry soil	127.18	134.3	140.52
Ma of dry soil	91.19	98.61	104.14
Mass of watre	25.97	27.8	28.34
Moisture (%)	28.47	28.23	27.14
Dry density	1.42	1.52	1.57

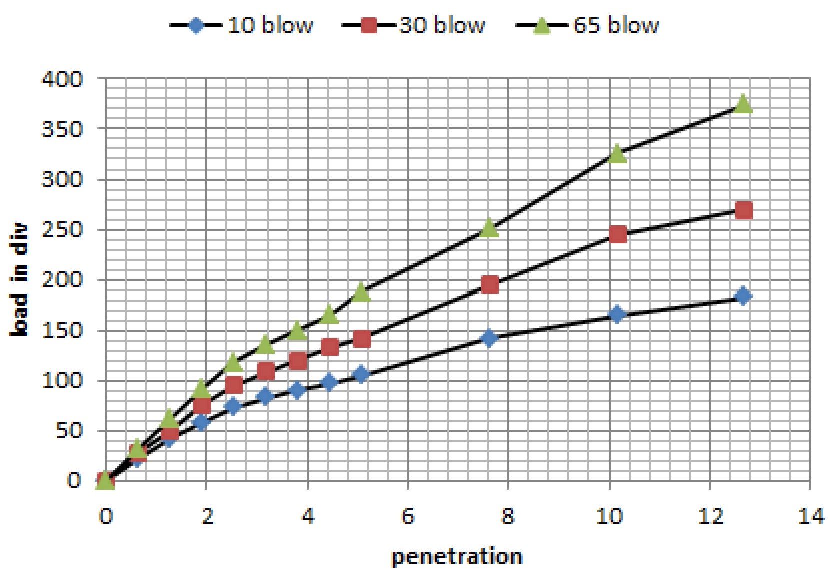


Figure e4:- CBR test result for test pit-1

Penetration	Load in division	Load in division	Load in division
in mm	for 10 blows	for 30 blows	for 65 blows
0	0	0	0
0.64	46	69	88
1.27	87	97	107
1.91	110	115	129
2.54	**128**	**131**	**147**
3.18	134	144	166
3.81	149	152	182
4.45	157	162	196
5.08	**165**	**178**	**220**
7.62	200	234	240
10.16	223	240	256
12.7	250	253	267

cbr@ 2.54	cbr@5.08
1.3	1.27
1.7	1.72
2.1	2.2

Table A18 Test pit -1 CBR test result

Penetration	Load in division	Load in division	Load in division
in mm	for 10 blows	for 30 blows	for 65 blows
0	0	0	0
0.64	40	63	77
1.27	68	94	98
1.91	88	117	129
2.54	**120**	**142**	**158**

3.18	138	158	172
3.81	150	164	185
4.45	162	173	200
5.08	**178**	**189**	**232**
7.62	200	213	251
10.16	230	245	263
12.7	249	270	374

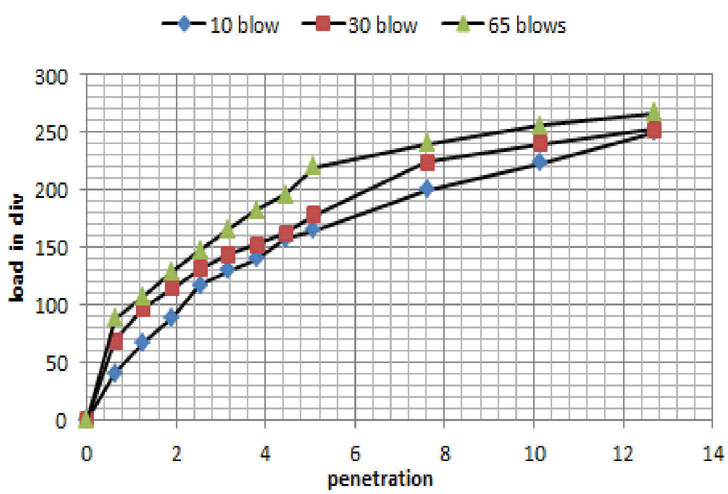


Figure e5 CBR test result at pit -2

cbr@2.54	cbr@5.08
2.1	**2.24**
2.39	2.21
2.68	2.49

Appendix B: Stabilized Soil Laboratory Test Results

Table B1-Liquid limit test result for 10% of marble waste for 7 days cured soil samples

Can no/blows	A (30)	f (26)	b (20)
Can wt	37.2	37.3	37.2
mass of wet soil	54.6	56	57
Mass of dry soil	47.40	48	48.68
moisture	70.6	74.8	72.5

Average liquid limit=72.63

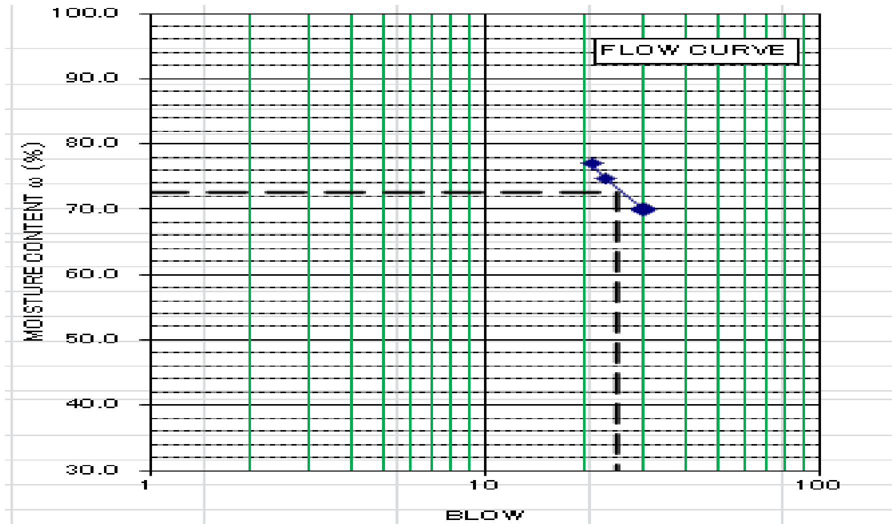


figure B1 liquid limit result

Table B2 Liquid limit test result for **15** % of marble waste for 7 days cured soil samples

Can no/blows	A (30)	f (24)	b (21)
Can wt	37.2	37.3	37.2
mass of wet soil	56.3	50	53.8
Mass of dry soil	49.20	45.5	46.9
moisture	60.2	54.2	71.9

Average liquid limit 62.1

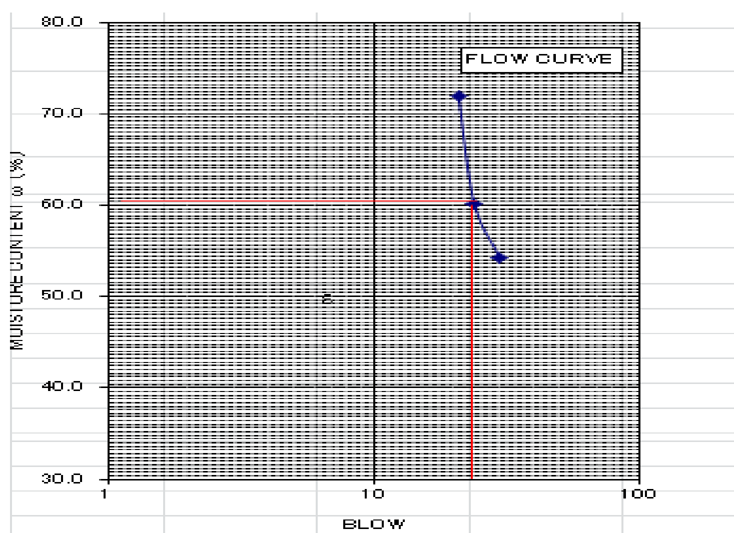


figure B-2 Liquid limte test result for 20% marble waste +80 expansive soil

Table B3-Liquid limit test result for **20 %** of marble waste for 7 days cured soil samples

Can no/blows	A (33) **Average liqui**	f (26) **limite 46 %**	b (23)
Can wt	37.4	37.4	37.3
mass of wet soil	51.2	51	53
Mass of dry soil	47	46.8	4.8
moisture	44.1	44.7	49.5

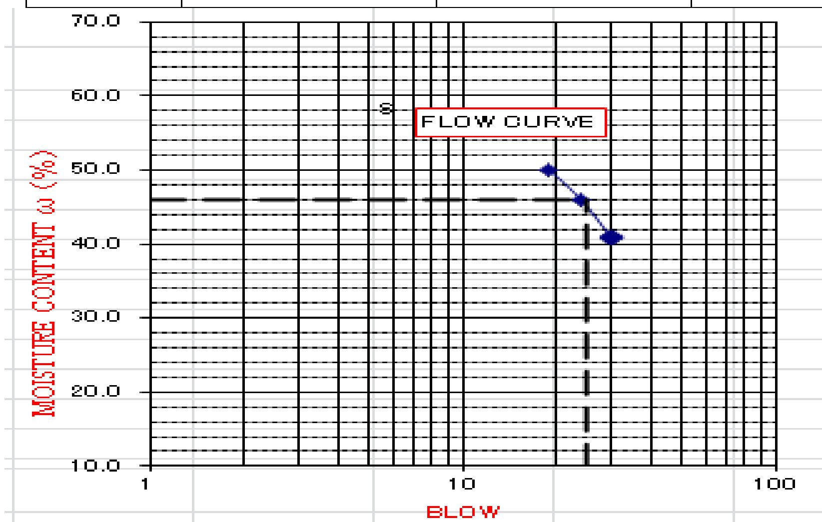


Figure B3-Liquid limite test result

Compaction test results (5%mw)

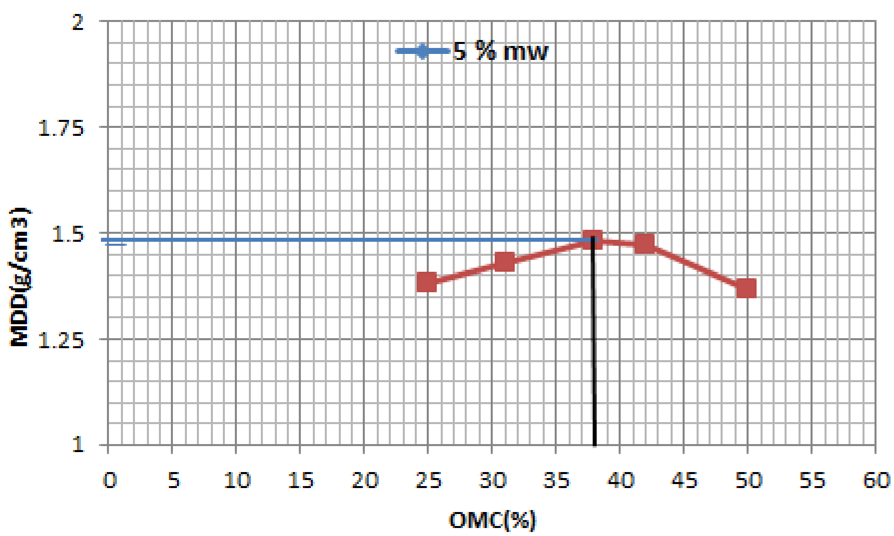


Figure B4-standard compaction test result

Table B4 Compaction parameter of stabilized soil by 10% mw

Can no	Can wt	Can+ wet soil	Can+ dry soil	Moisture content	Mass of soil+ mold	Mass of soil	Bulk density	Dry density
23	23	102.9	89.56	15.4	5862	1674	1.77	1.51
L1	11.8	117.73	100	20.43	6091	1903	2.01	1.67
45	22.3	122.46	99.35	27	6247	2059	2.18	**1.71**
mm	22.5	102	80.9	34	1961	1961	2.08	1.55

MDD @ 10 % mw=1.71 g/cm3

OMC@10 % MW=28

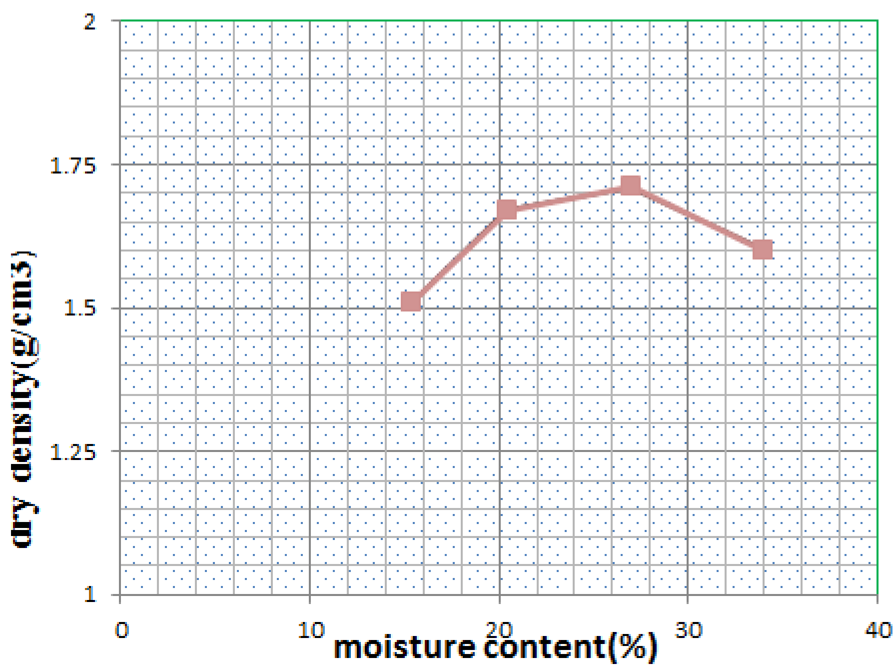


Table B5 Method C-standard compaction test result 15% marble waste

C. No	Can.Wt	C.wt+wet soil	C.wt+dry soil	Moisture content	Mould+bas+ soil	massof wet soil	Bulck density	Dry density
11	40.9	92.3	84	18.4	5888	1711	1.81	1.78
2c	40.8	108	95	23	6333	2156	2.28	1.88
2w	40.9	114	96	27.8	6800	2623	2.78	**1.92**
2w	41.2	131	103.4	34	6755	2523	2.67	1.83
22	38	133	121	38	6721		2.64	1.77

MDD @ 15% MW=1.92
OMC@15% MW=

Table 6B on unconfined compressive strength test result @5% mw

Deformation Dial ,chan. L (mm)	Unit strain ,E	% strain	Cross sectional Area, A (mm^2)	Proving Ring Dial in devision	Applied Axial Load P(N)	Load per Unit Area N/mm^2	KN/m^2
1	2 = chan.L/Lo	0	3 =Ao/1-E	4	5 =4*44.48	6 =5/3	0
0.2	0.0026	0.0034	1139.0	1.8	80.64	0.07080	6.22
0.4	0.0053	0.5195	1142.0	2.2	98.56	0.08630	75.57
0.6	0.0079	0.7792	1145.0	2.43	108.86	0.09507	83.03
0.8	0.0105	1.0390	1148.1	2.64	118.27	0.10302	89.73
1	0.0132	1.2987	1151.1	2.84	127.23	0.11053	96.01
1.2	0.0158	1.5584	1154.2	3	134.40	0.11644	100.88
1.4	0.0184	1.8182	1157.3	3.33	149.18	0.12890	111.38
1.6	0.0211	2.0779	1160.4	3.55	159.04	0.13705	118.11

1.8	0.0237	2.3377	1163.6	3.67	164.42	0.14130	121.44
2	0.0263	2.5974	1166.7	3.82	171.14	0.14668	125.72
2.2	0.0289	2.8571	1169.9	4	179.20	0.15318	130.94
2.4	0.0316	3.1169	1173.0	4.2	188.16	0.16040	136.74
2.6	0.0342	3.3766	1176.2	4.64	207.87	0.17673	150.25
2.8	0.0368	3.6364	1179.5	4.88	218.62	0.18536	157.16
3	0.0395	3.8961	1182.7	5.23	234.30	0.19811	167.51
3.2	0.0041	4.1558	1140.7	5.46	244.61	0.21444	187.99
3.4	0.0447	4.4156	1189.2	5.66	253.57	0.21323	179.30
3.6	0.0474	4.6753	1192.5	6.2	277.76	0.23293	195.33
3.8	0.0500	4.9351	1195.8	6.42	287.616	0.24052	201.14
4	0.0526	5.1948	1199.1	6.73	301.504	0.25144	209.69
4.2	0.0553	5.4545	1202.5	7.23	323.904	0.26937	224.02
4.4	0.0579	5.7143	1205.8	7.56	338.688	0.28088	232.94
4.6	0.0605	5.9740	1209.2	8.66	387.968	0.32085	265.34
4.8	**0.0632**	**6.2338**	**1212.6**	**9.88**	**442.624**	**0.36503**	**301.03**
5	0.0658	6.4935	1216.0	9.77	437.696	0.35995	296.01
5.2	0.0684	6.7532	1219.4	9.74	436.352	0.35783	293.44
5.4	0.0711	7.0130	1222.9	9.72	435.456	0.35609	291.19

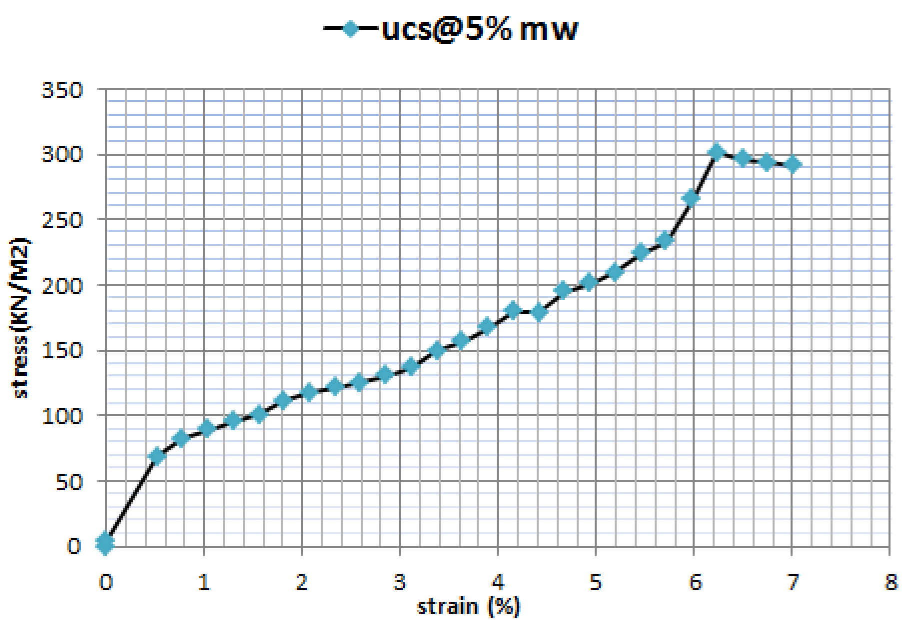


Figure 8B unconfiend compressive strength result

Table 7B unconfined compressive strength test result @10% MW

Deformation Dial ,chan. L (mm)	**Unit strain ,E**	**% strain**	Cross sectional Area, A (mm^2)	Proving Ring Dial in devision	Applied Axial Load P(N)	Load per N/mm^2	Unit Area KN/m^2
1	2 = chan.L/Lo	0	3 =Ao/1-E	4	5 =4*44.48	6 =5/3	0
0.2	0.0026	0.0034	1139.0	1.2	53.76	0.04720	4.14
0.4	0.0053	0.5195	1142.0	1.32	59.14	0.05178	45.34
0.6	0.0079	0.7792	1145.0	1.56	69.89	0.06104	53.30
0.8	0.0105	1.0390	1148.1	1.88	84.22	0.07336	63.90
1	0.0132	1.2987	1151.1	2.2	98.56	0.08562	74.38
1.2	0.0158	1.5584	1154.2	2.48	111.10	0.09626	83.40
1.4	0.0184	1.8182	1157.3	2.9	129.92	0.11226	97.00
1.6	0.0211	2.0779	1160.4	3.44	154.11	0.13281	114.45
1.8	0.0237	2.3377	1163.6	3.64	163.07	0.14015	120.45
2	0.0263	2.5974	1166.7	3.88	173.82	0.14899	127.70
2.2	0.0289	2.8571	1169.9	4.21	188.61	0.16122	137.81
2.4	0.0316	3.1169	1173.0	4.48	200.70	0.17110	145.86
2.6	0.0342	3.3766	1176.2	4.89	219.07	0.18625	158.34
2.8	0.0368	3.6364	1179.5	5.44	243.71	0.20663	175.19
3	0.0395	3.8961	1182.7	5.87	262.98	0.22236	188.01
3.2	0.0041	4.1558	1140.7	6.23	279.10	0.24468	214.50
3.4	0.0447	4.4156	1189.2	7.44	333.31	0.28028	235.69
3.6	0.0474	4.6753	1192.5	7.88	353.024	0.29604	248.25
3.8	0.0500	4.9351	1195.8	8.34	373.632	0.31246	261.30
4	0.0526	5.1948	1199.1	8.86	396.928	0.33102	276.05
4.2	0.0553	5.4545	1202.5	9.24	413.952	0.34426	286.30
4.4	0.0579	5.7143	1205.8	9.77	437.696	0.36299	301.03
4.6	0.0605	5.9740	1209.2	10	448	0.3705	306.40
4.8	0.0632	6.2338	1212.6	10.65	477.12	0.39347	324.49
5	0.0658	6.4935	1216.0	11	492.8	0.40526	333.28
5.2	0.0684	6.7532	1219.4	11.44	512.512	0.42029	344.66
5.4	0.0711	7.0130	1222.9	11.88	532.224	0.43522	**355.89**
5.6	0.0737	7.2727	1226.4	11.76	526.848	0.4296	350.31

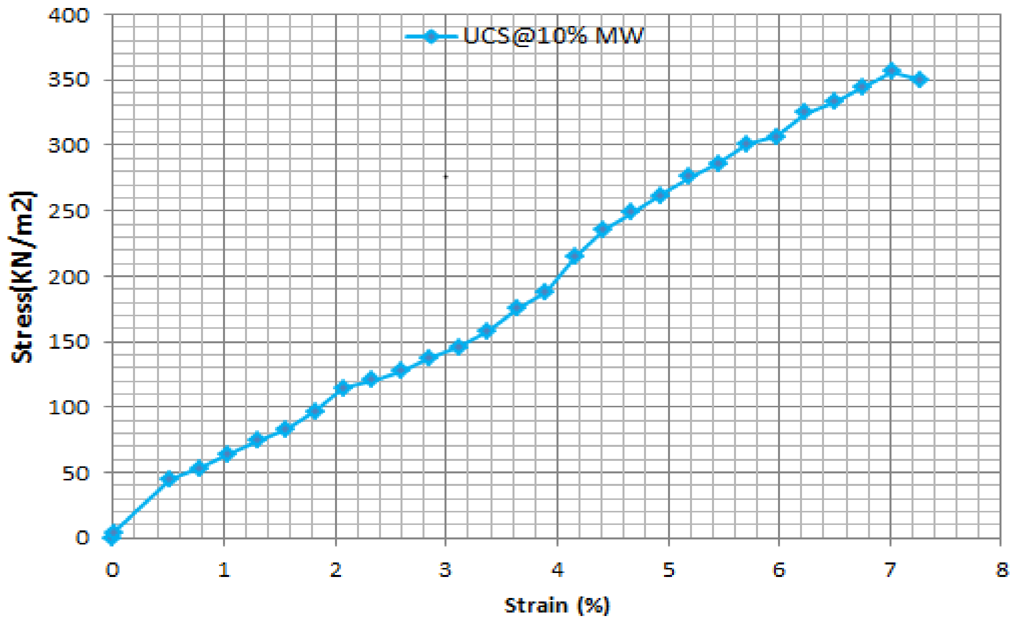


Figure 9B un confiend compressive strength test result @10%mw

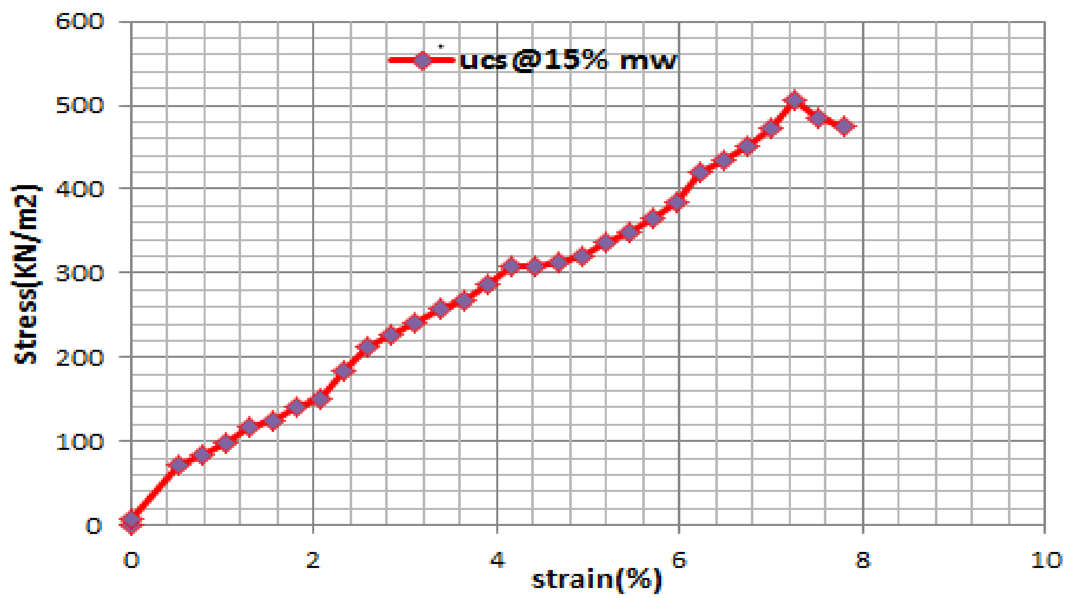


figure 10B UCS test result @15% MW

Table 8B Test pit -1 CBR result @ 5% MW stabilized

Penetration	Load in division	Load in division	Load in division
in mm	for 10 blows	for 30 blows	for 65 blows
0	0	0	0
0.64	40	60	81
1.27	65	138	162
1.91	89	188	240
2.54	121	290	338
3.18	145	330	376
3.81	167	373	414
4.45	211	404	465
5.08	235	439	506
7.62	300	504	592
10.16	356	559	638
12.7	402	608	675

cbr@2.54	cbr@ 5.08
2.2	2.85
5.2	5.3
6.1	6.1

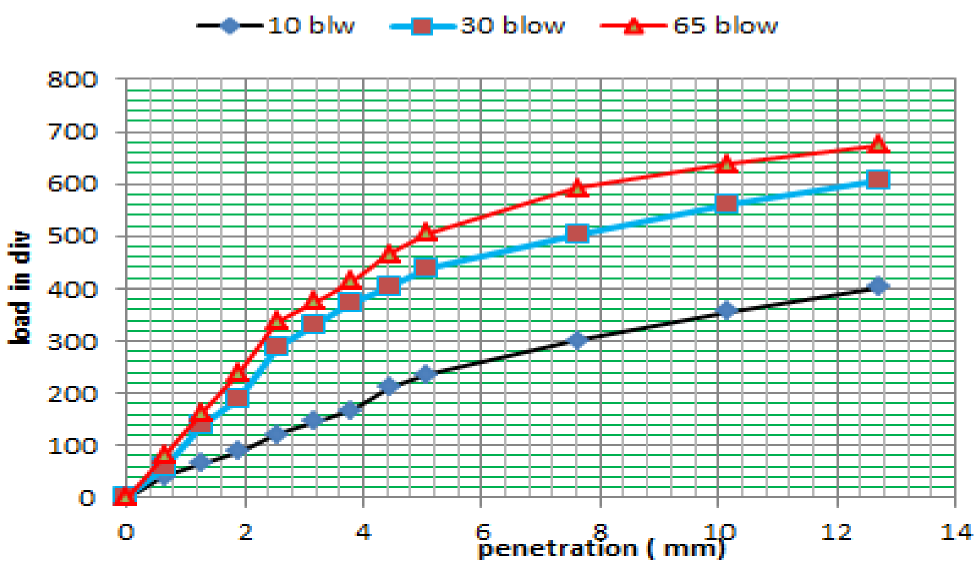


Table 18 Test pit -1 CBR result @15% MW stabilized

Penetration	Load in division	Load in division	Load in division
in mm	for 10 blows	for 30 blows	for 65 blows
0	0	0	0
0.64	150	178	230
1.27	320	389	450
1.91	500	590	620
2.54	**632**	**770**	**737**
3.18	700	854	860
3.81	788	888	910
4.45	812	900	976
5.08	**850**	**968**	**1015**
7.62	894	1000	1034
10.16	900	1118	1200
12.7	980	1150	1340

cbr@ 2.54	**cbr@ 5.08**
12%	**10.80%**
12.50%	**11.76%**
13.40%	**12.34%**

APPENDEX C

DETERMINATION OF SOIL WATER CHARACTERISTICS CURVE .

Determination of Saturated Water Content.

$$w_{sat} = \frac{M_{sat}\left(1+w_m\right)}{M_m} - 1$$

$$\rho = \frac{M_{sat}}{V} \quad , \quad \rho_d = \frac{\rho}{\left(1+w_{sat}\right)} \; , \; e = \frac{G_s * \rho_w}{\rho_d} - 1 \; , \; S = \frac{G_s * w_{sat}}{e}$$

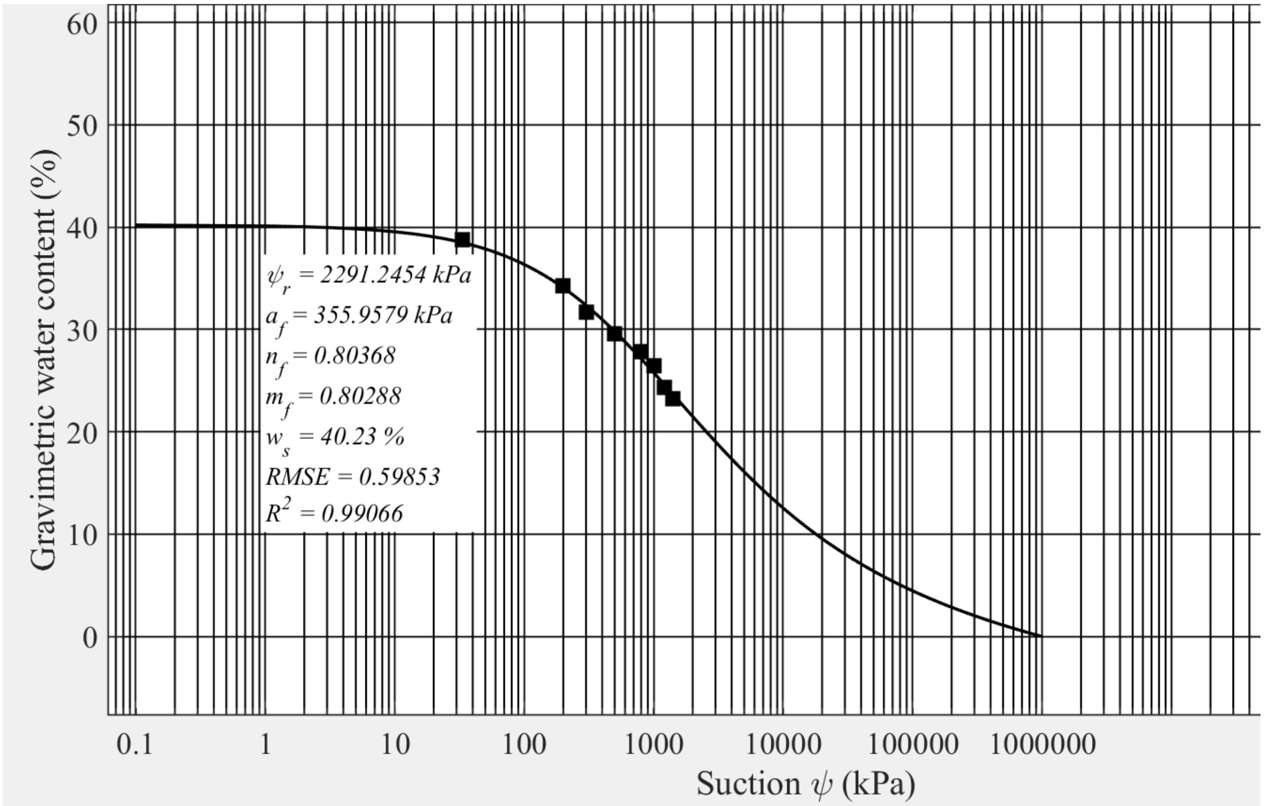


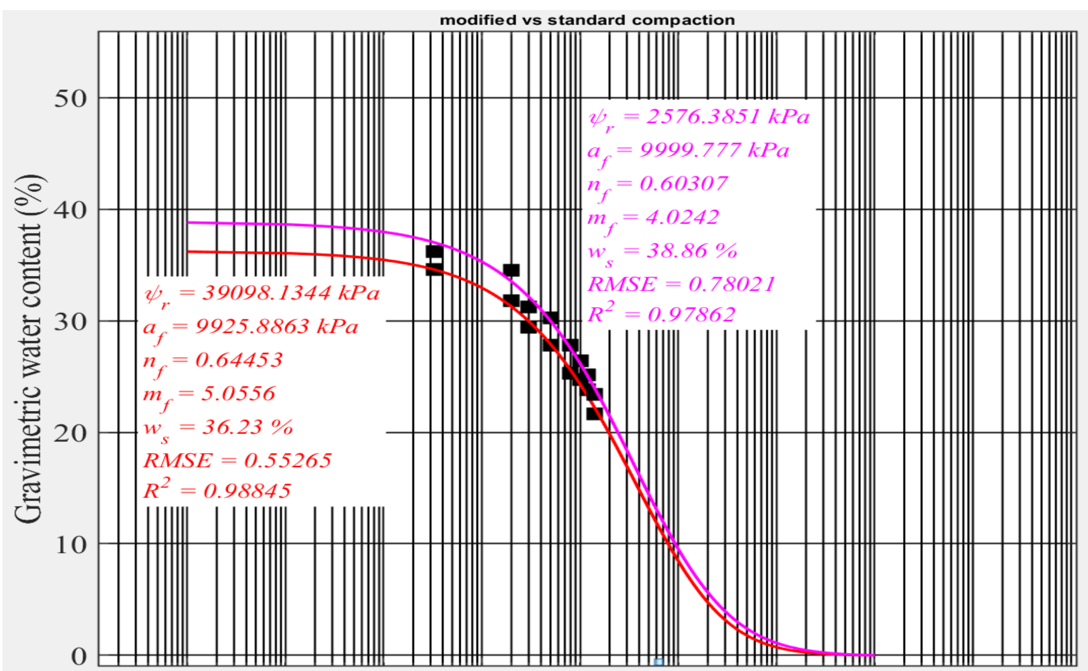
Figure c-1 Modified compaction test result

SWCC test data of modified & standard compaction from pressure plate

			After equilibrium at different suction								
Cn	Before saturation	After saturation	At 0.33bar	At 200 bar	At 300bar	At 500 bar	At 800bar	At 1000 bar	At 1200 bar	At 1400 bar	condition
O	165	183	180	176.2	174	170	168.	166.	164.8	163	Modified compact
L1	158	172	169	163	162.8	160	158	157.2	156.4	155.8	Standard compact
M3	166	182	179	172.8	170.3	168.4	165.8	163	162.2	161.8	Modified compact
L6	157	171	165.8	164.7	163	161	159	157	155	154.2	Standard compact
11	163	177	173	168	167.6	165	163	160	158.9	157	Standard compact

modified compaction

Suction (kpa)	At different marble waste with 7 day curing period							
	5%mw		10%mw		15%mw			20%mw
	Measured (θg)	predicted (θg)	Measured (θg	predicted (θg)	Measured (θg)	predicted (θg)	Measured (θg)	predicted (θg)
33	51.23	51.19	42.23	42.44	39.45	39.42	38.4	38.55
200	47.98	47.44	40.44	41	38.34	38.23	36.21	36.12
300	46.73	46.68	38.88	38.68	37.64	37.78	34.82	34.77
500	45.33	45.61	36.22	36.64	35.23	35.28	34.23	34.21
800	44.23	44.34	30.81	30.9	33.34	33.41	28.21	28.21
1000	42.8	42.81	28.93	28.91	32.8	32.78	26.44	26.34
1200	38.7	39.2	27.23	27.41	31.9	32.1	25.4	25.41
1400	28	27.3	26.87	26.23	28.7	28.64	23.4	23.51



SWCC Moisture determination of undisturbed natural soil

Can no	Mass of can	Can+ wet soil	Can +dry soil	Moisture content	Average moisture	Applied pressure (bar)
2	36	49.7	45	0.522222	0.51	0.33
4	36	49.3	45	0.477778		
4	36	48.7	44.8	0.443182	0.44	2
2	36.2	48.6	44.8	0.44186		
2	35.8	48.2	44.8	0.377778	0.42	3
4	36.45	48.6	44.8	0.45509		
3	35.6	46.5	44.8	0.184783	0.36	5
4	35.6	46.6	44.8	0.195652		
2 3	35.6	46.4	44.8	0.173913	0.32	8
3	36	46.3	44.78	0.173121		
4 2	36.41	46	44.8	0.143027	0.31	10
4	36.33	46	44.8	0.141677		
2 4	35.9	45.8	44.8	0.11236	0.29	12
4	36	45.8	44.8	0.113636		
3	36	45.7	44.8	0.102273	0.27	14
4	36	45.7	44.8	0.102273		

Modified compacted soil sample data from pressure plate

Suction (kPa)	Ring no	Mass of ring	Ring +soil before saturation	Ring +soil after saturation	Ring +soil after equilibrium	soil after equilibrium	Dry mass of soil	Moisture
0.1	10	10.11	43.24	44.28	44.28	33.45	23.28	40.57
	7	10.11	43.03	43.56	43.56	32.98	22.99	38.42
33	10	10.11	43.24	44.28	42.51	31.68	23.28	36.08
	7	10.11	43.03	43.56	43.05	32.47	22.99	35.24
200	10	10.11	43.24	44.28	42.13	31.30	23.28	34.45
	7	10.11	43.03	43.56	41.48	30.90	22.99	32.41
400	10	10.11	43.24	44.28	41.02	30.19	23.28	29.68
	7	10.11	43.03	43.56	40.17	29.59	22.99	28.71
800	10	10.11	43.24	44.28	39.33	28.50	23.28	22.42
	7	10.11	43.03	43.56	38.74	28.16	22.99	22.49
1000	10	10.11	43.24	44.28	39.03	28.20	23.28	21.13
	7	10.11	43.03	43.56	38.48	27.90	22.99	21.36
1200	10	10.11	43.24	44.28	38.68	27.85	23.28	19.63
	7	10.11	43.03	43.56	38.25	27.67	22.99	20.36
1400	10	10.11	43.24	44.28	38.42	27.59	23.28	18.51
	7	10.11	43.03	43.56	38.15	27.57	22.99	19.92