ADAMA SCIENCE AND TECHNOLOGY UNIVERSITY

SCHOOL OF CIVIL ENGINEERING AND ARCHITECTURE

DEPARTMENT OF CIVIL ENGINEERING

MAJOR GEOTECHNICAL ENGINEERING

Stabilization of Local Expansive Subgrade Soil using Marble waste powder with Lime

by
Amdegebreal Girmaw

September, 2016

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TITLE: Stabilization of Local Expansive sub-grade Soil Using Marble waste powder with Lime

A Thesis Submitted to Adama Science and Technology University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering (Geotechnical Engineering)

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DECLARATION

I Amdegebreal Girmaw declare that, this is my own original work towards the MSc and that, to the best of my knowledge. It contains no material presented and will not be presented by me to any other University for similar or any other degree award, except where due acknowledgment has been made in the text.

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Date: _______________

ACKNOWLEDGEMENT

First of all I thank almighty God with his mother St. Virgin Mary and St. Gebreal for take care of my health and offering me travelling mercies for all these years and the rest of my life.

I wish to express my heartfelt gratitude to Dr. Argaw Asha for his perfect assistance, guidance, recommendations, critical comments and support throughout this research and preparation of this thesis without hesitation from his very busy time, I have no special word to thank him.

I would like to thank all my lecturers for their positive and helpful approaches for my education, Especially Dr. Addiszemen Teklay and Dr. Addisalem Zeleke.

I am grateful to my organization Wolkite University giving me this chance and Wolkite university civil engineering and architecture department. Also, I wish to extend my gratitude towards ASTU, EIABC, material research and training center (MRTC) laboratory technicians.

My thankfulness goes to Mafcon engineering for laboratory testing material support to perform the test, Gast Solar Mechanics and Ethiopia marble processing enterprise for material support.

I also would like to state my appreciation to Mr. Tesfay K., aby F., Dinkie A., Eestifanos B., Molalgn N., Abebaw A. and Abay A. for their encouragement.

And finally I would like to extend my heartfelt thanks to my father Ato Girmaw Endalew for his unlimited support and initiation for this work, and also I thank my whole family for their help.

LIST OF SYMBOLS AND ABBREVIATION

SML – Soil stabilized marble waste powder with lime

AASHTO - American Association of State Highway and Transport officials.

Al_2O_3 - Alumina

ASTM - American Standards of Testing Materials

MWP – Marble Waste Powder

CaO - Calcium Oxide

CBR - California Bearing Ratio

CH - Inorganic clays of high plasticity

UCS - Unconfined Compressive strength

ERA - Ethiopian Road Authority

FS - Free Swell Index

KPa - Kilo Pascal

MDD - Maximum Dry Density

MgO - Magnesium Oxide

OMC - Optimum Moisture Content

PI - Plasticity Index

SiO2 - Silicon Dioxide

LL – Liquid Limit

USCS - unified soil classification system

MOWUD – Ministry of Work and Urban Development

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ABSTRACT

Expansive soils are the most problematic soils due to their property of swelling and expansion with the influence of variable moisture, a number of civil engineering structures were destroyed. A billions of US doralls spent worldwide each year to mitigate the problem.

The presence of expansive subgrade soil results pavement distress and damage. Removing the expansive soil and replacing with the competent material is applied to mitigate the problem which is very expensive and time consuming for long hauling distance and thick layer expansive soil.

This study presented stabilization of local expansive subgrade soil using marble waste powder with lime. The marble waste powder was collected in Addis Ababa from Ethio-marble processing enterprise Gulele branch and the lime was collected at Gast Solar Mechanics in Addis Ababa. Free swell index test, Atterberg limit test, Proctor test, unconfined compressive test, California Bearing Ratio Tests, swelling potential and swelling pressure test were used to evaluate properties of treated and untreated soils. The expansive subgrade soil was treated using 5%, 10%, 15%, 20%, and 25% marble waste powder with fixed 3% lime respective combinations by weight of the soil. The optimum percent combination for this study was 10% marble waste powder with 3%lime based on soaked CBR swell, soaked CBR, swelling pressure and swelling potential test result values. Optimum proportion of stabilizers improve CBR Value from 0.65% to 4.19%, reduce swelling pressure from 1000kpa to 440kpa, increases MDD from 1.21 to 1.29, and reduce PI from 78% to 48.4%.

Keywords*: marble waste powder, lime, expansive soil, CBR, UCS, swelling pressure, MDD, OMC*

1. INTRODUCTION

1.1. Background

Almost every civil engineering structures, bridges, highways, buildings, canals, dams, walls, or tunnels, must be in or on the surface of the earth. For satisfactory performance, each structure must have suitable foundation soil.

Expansive soils are soils that swell when subjected to moisture and shrink when they dry. The presence of expansive soil doesn't cause a problem at constant moisture content. However, at the situation of repeated moisture variations swelling and shrinkage are not fully reversible processes. The process of shrinkage causes cracks, which on re-wetting, do not close-up perfectly and hence cause the soil to bulk-out slightly, and also allow enhanced access to water for the swelling process. In geological time scales shrinkage cracks may become in-filled with sediment, thus imparting heterogeneity to the soil. When material falls into cracks the soil is unable to move back, thus resulting in enhanced swelling pressures. This cyclic movement create a problem for civil engineers to construct on expansive soils and cause considerable damages to civil engineering structures including pavements. Construction of pavements over expansive soils leads poor performance due to development of cracks induced by moisture variation. The presence of clay mineral montmorillonite is known to be responsible for expansive nature of these soils.

Expansive soils are widely distributed all over the world both in tropical and temperate zones. In Ethiopia, expansive soils form major soil groups. even though the extent and range of distribution of this problematic soil has not been studied thoroughly (Tilahun, 2004) the high lands mostly in the western, central and southwestern part of the country covered by expansive soils with major clay mineral component of "montmorillonite".

A great deal of distress or damage is reported all over the world on structures founded on or at shallow depth in expansive and shrinkage soils. This is due to seasonal volumetric changes resulting in vertical and horizontal movement of the expansive soils by adsorbing and losing moisture. Maintenance and repair requirements of untreated expansive soil can be extensive, and expenses can grossly exceed the original cost of any mitigation measures. It has been estimated that the annual damage to structural on expansive soil are $1000 million in USA, £150million in UK and 1000 million pounds in worldwide (Gourley *et al.*, 1993). Reports show that, the situation causes greater financial loss to property owners than earthquake, flood, hurricanes and tornados combined in typical year. The road sector in Ethiopia too is suffering from the high shrink-swell behavior of this expansive soil. Many damages occur each year and road construction over such expansive soil creates serious problems including increasing cost of construction and maintenance (MoWUD, 2009).

According to Chen (1988) the problems that this soils bring include cracking, heaving and breaking of pavements, buildings, building foundations, reservoir linings, and channels. This shows how civil engineers confronted with the expansive soil problems.

It is very difficult to have a single mitigating measure in dealing with expansive soils due to limited understanding of their behavior. Several mitigating measures can be taken to minimize effect of expansive soils before and after construction of civil engineering structures. This include chemical stabilization, compaction control, soil replacement, pre-wetting, surcharge loading, blending with non-swelling soil and use of geosynthetics (Zonberg and Gupta, 2009). In Ethiopia no systematic records of application of these methods and monitoring of their performance are available to evaluate the effectiveness of the practice.

Soil stabilization is a technique aimed at increasing or maintaining the stability of soil mass and chemical physical alteration of soil to enhance their engineering properties. Marble waste powder with lime is a chemical method of stabilization which is effective for minimizing detrimental effect of expansive soil.

1.2. Problem statement

Pavements lied on expansive subgrades are subjected to distresses .they develop lateral and longitudinal cracks. During road construction works the practice in Ethiopia where expansive subgrade soils are encountered, excavate the top unsuitable material and replace with selected fill. However in this method, unsuitable materials are indiscriminately disposed of a long road way thereby destroy farmlands with attendant negative environmental consequences, and materials for replacement may be hauled from a long distance which is more costly, time taking, machinery intensive. In recent years the Authority has started stabilizing expansive soil with chemical additives on road sections. Foreign chemical additive supplying companies had contracted with authority and was doing stabilization of expansive sub-grade soil on trial road sections in the Addis Ababa city road, Ambo Gedo road section. This shows the problem due to expansive soils is serious in the road sector development activity in country. Based on the above conditions the road development sector have initiated to search and evaluate for the various alternatives which enables to improve the engineering characteristics of the expansive sub-grade soil.

The removal problem from working site and environmental consequence of Waste Marble Powder in Ethiopia which was used for stabilization in this study.

1.3. Objective

The main objective was stabilizing weak local expansive subgrade soils using waste marble powder with lime to minimize damage of pavements. Specifically, the investigation seeks to;

- Characterize both the stabilized and non-stabilized expansive subgrade soil by conducting laboratory test.
- Determine how various proportions of marble waste powder with fixed lime content affect index and engineering properties of expansive subgrade soil.
- Provide safe disposal mechanism for marble processing factories and reduce environmental hazardous effect of marble waste powder by using it as soil stabilizer material.
- Determine the optimum proportion of Marble Waste Powder with lime mix to stabilize expansive subgrade soil.

1.4. Justification

Expansive soils are a problematic soils having a great deal of damage report all over the world to civil engineering structures due their swelling and shrinkage behavior. Their existence on a road construction corridor must therefore be given utmost attention. The currently used procedure for the design and construction of pavements on expansive soils don't systematically consider the variety of factors and conditions which influence volume changes as evidenced by the continued occurrence of failures in areas where expansive soils exist. Therefore most accurate method of testing, identifying and treating expansive clay is needed to improve highway design, construction, and maintenance.

Soil replacement method of mitigating expansive soil is highly practiced in Ethiopia can be time consuming and expensive, considering the time and cost of excavation and disposal of unsuitable material, and hauling of suitable selected fill to replace the unsuitable material. In addition the practice may be of doubtful effectiveness. Marble waste powder with lime stabilization is effective way of improving expansive soil. This investigation will initiate research in to the effectiveness of marble waste powder with lime stabilization of local expansive subgrade soils.

1.5. Scope of the work

This research is limited to exploring the behavior of one deposit of highly expansive black cotton soil, located in Addis Ababa, bole sub-city, around Bole Arabsa condominium site. It was also limited to secondary data and laboratory test result interpretation of soil stabilized by marble waste powder with lime and unstabilized soil.

1.6. Methodology of the report

This research was done using literatures, research bulletins, previous studies, different books and free website sources. Expansive subgrade soil sample was collected from Bole-Arabsa project – 13 new condominium site, Bole sub-city Addis Ababa, Ethiopia by conducting a serious of laboratory tests. The sample was prepared by sun and air drying, pulverization and sieving the soil to the required particle size based on the outlined in ASTM. Classification of soil was determined by running index property and grain size analysis tests. Expansive sub-grade soil was treated by 5%, 10%, 15%, 20%, and 25%

Marble Waste Powder with fixed 3% lime by weight. Atterberg limit test, specific gravity test, free swell test, moisture-density test, unconfined compressive test, California bearing ratio test and swelling pressure test was done for treated and un treated soil samples. Laboratory test results were analyzed and conclusions are drawn based on the analyzed test results.

1.7. Organization of the report

This thesis work is organized in six chapters. The first chapter gives a brief description of the thesis background, problem statement, objectives, scope and methodology employed. The second chapter explains about expansive soils, distributions, damaging effect, and mitigation measures. The third chapter discusses material used for this study, standards, test methodologies, experimental designs and parameters for both untreated and treated soil. The fourth chapter reports the test results obtained, analysis of results and discussion of results. Finally, the fifth and six chapter discuses about conclusions and recommendations respectively.

2. LITERATURE REVIEW

2.1. Distribution of expansive soil

2.1.1. Worldwide distribution of expansive soil

Potentially expansive soils can be found almost anywhere in the world and they are a worldwide problem in both residual and transported soils. They can be found both in tropical and temperate climates. The countries in which expansive soils have been reported are as follows (Chen, 1988):

- Argentina
- Iran
- Australia
- Mexico
- Burma
- Morocco
- Canada
- Rhodesia
- Cuba
- South Africa
- Ethiopia
- Spain
- Ghana
- India
- U.S.A.
- Israel
- Venezuela

In the underdeveloped nations, much of the expansive soil problems may not have been recognized. It is to be expected that more expansive soil regions will be discovered each year as the amount of construction increases.

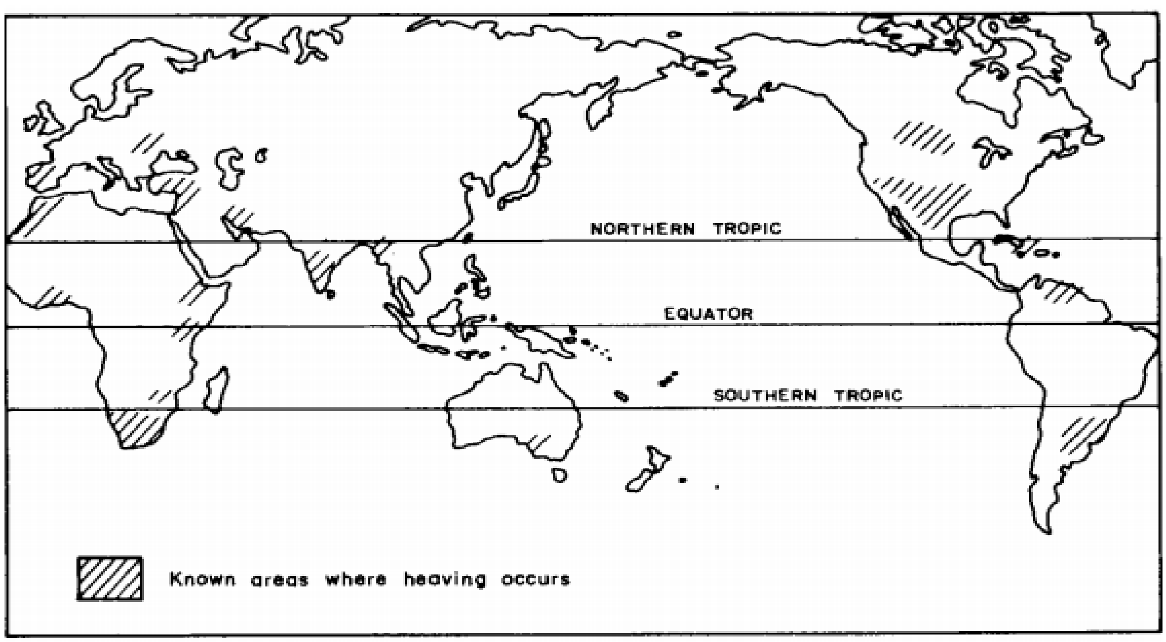

Figure 2. 1 worldwide distribution of expansive soils ((Donaldson, 1969)

2.1.2. Distribution of expansive soil in Ethiopia

In Ethiopia, Expansive soil is known to be widely spread. Although the extent and range of distribution of this problematic soil has not been studied thoroughly. Most of the recent construction are being carried out and central part of Ethiopia following the major trunk roads like Addis-Ambo, Addis-Woliso, Addis-Debre Birhan, Addis-Goha tsion-Gojjam, Addis-Modjo are covered by expansive soils. Also areas like Mekele and Gambela are covered by expansive soils.

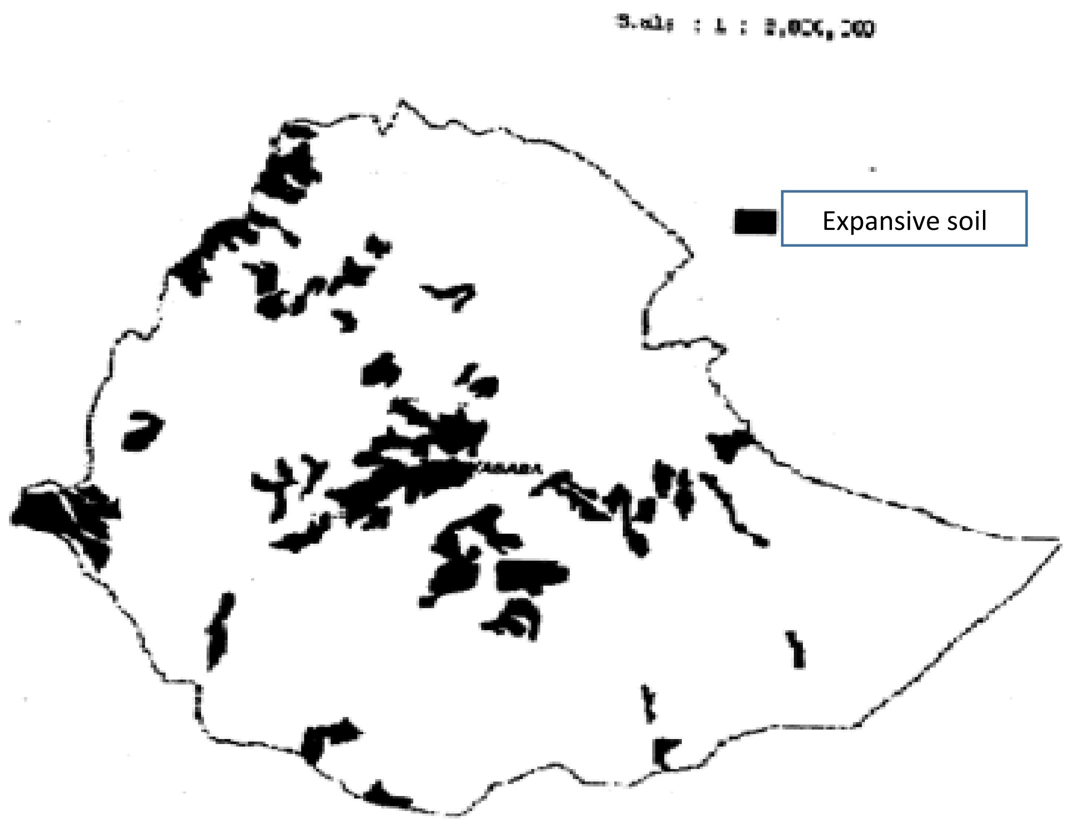

Figure 2. 2 Distribution of expansive soil in Ethiopia (Daniel, Jully 11, 2003)

The southern, south-east and south-west part of the city of Addis Ababa areas. The distribution is shown in Figure 2.3.

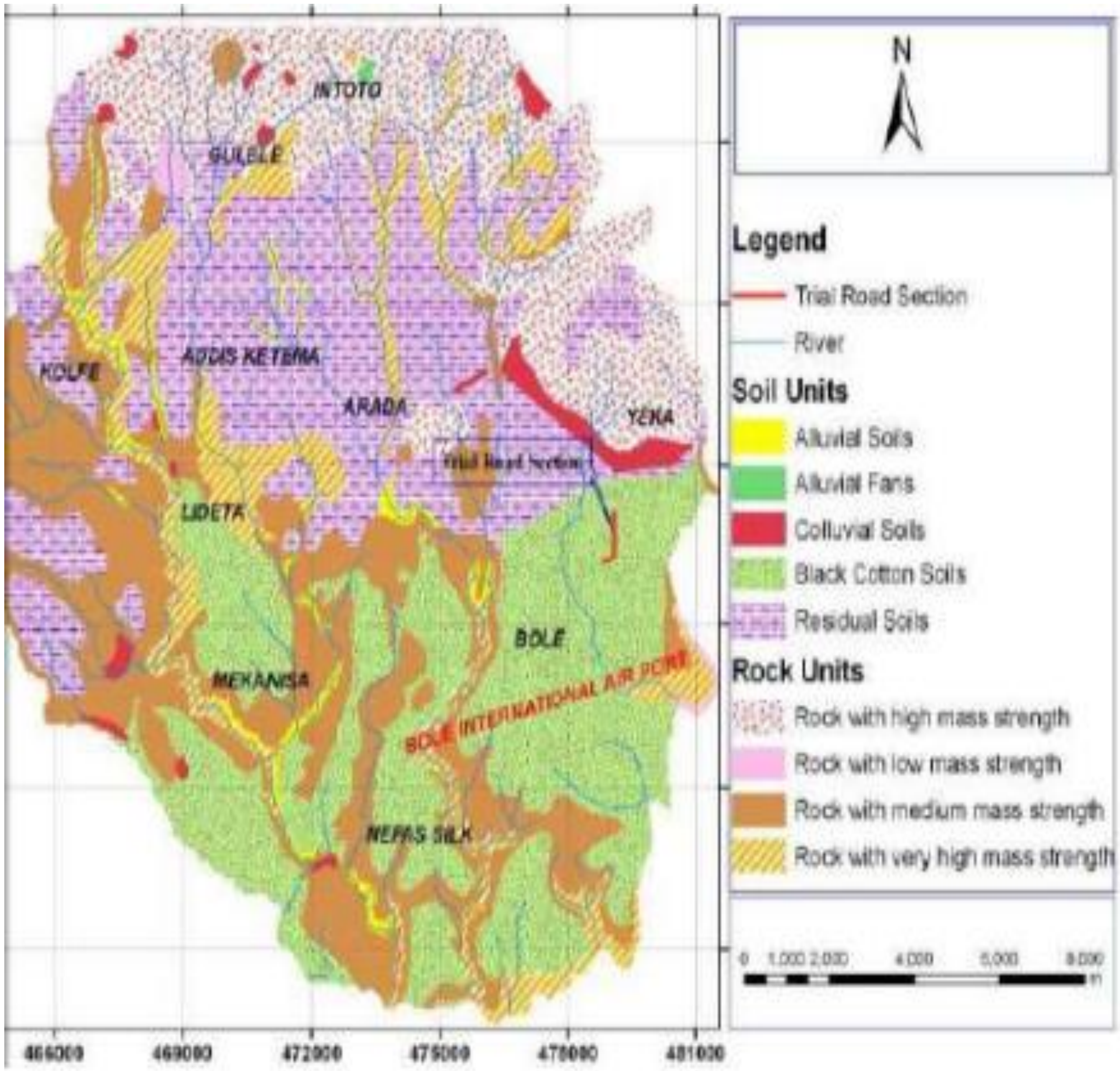

Figure 2. 3 Engineering Geological Map of Addis Ababa (Kebede Tsehayu and Tadesse Hailemariam, 1990)

2.2. Mineralogical composition of expansive soils

Soils are composed of a variety of minerals which will not expand or expand in the presence of moisture. The term clay can refer both to a size and to a class of minerals. As a size term, it refers to all constituents of a soil smaller than a particular size, usually 0.002 mm in engineering classifications. As a mineral term, it refers to specific clay minerals that are distinguished by small particle size, a net electrical charge, plasticity when mixed with water and high weathering resistance (Mitchell and Soga, 2005). A number of clay minerals are expansive. These includes bentonite, montmorillonite, beidellite, vermiculite, nontronite, illite, and crolite. There are also some sulphate salts that will expand with changes in temperature. Significant expansion of soils therefore depends upon the minerals in the soil. The clay mineral montmorillonite is the major component of expansive soil. The basic idealized crystalline structural unit of a clay mineral is composed of a silica tetrahedron block and an aluminum octahedron block. Aluminum octahedron block may have Aluminum (Al3+) or magnesium (Mg2+). If only aluminum is present, it is called gibbsite [Al2 $(OH)_6$]; if only magnesium is present, it is called brucite [Mg_3 $(OH)_6$]. Various clay minerals are formed as these sheets stack on top of each other with different ions bonding them together (Oweis and Khera, 1998). A silica tetrahedron and a silica sheet, also an octahedron.

Three important structural groups of clay minerals are described for engineering purposes as follows:

I. **Kaolinite group** - generally non-expansive
II. **Mica-like group** - includes illites and vermiculites, which can be expansive but generally do not pose significant problems.
III. **Smectite group** - includes montmorillonites, which are highly expansive and are the most troublesome clay minerals (Nelson and Miller, 1992).

I. Kaolinite group

Kaolinite

Kaolinite crystals consist of tetrahedron and octahedron sheets. The bonding between successive layers is by van der Waals forces and hydrogen bonds. The bonding is sufficiently strong that there is no interlayer swelling in the presence of water (Mitchell and Soga, 2005).

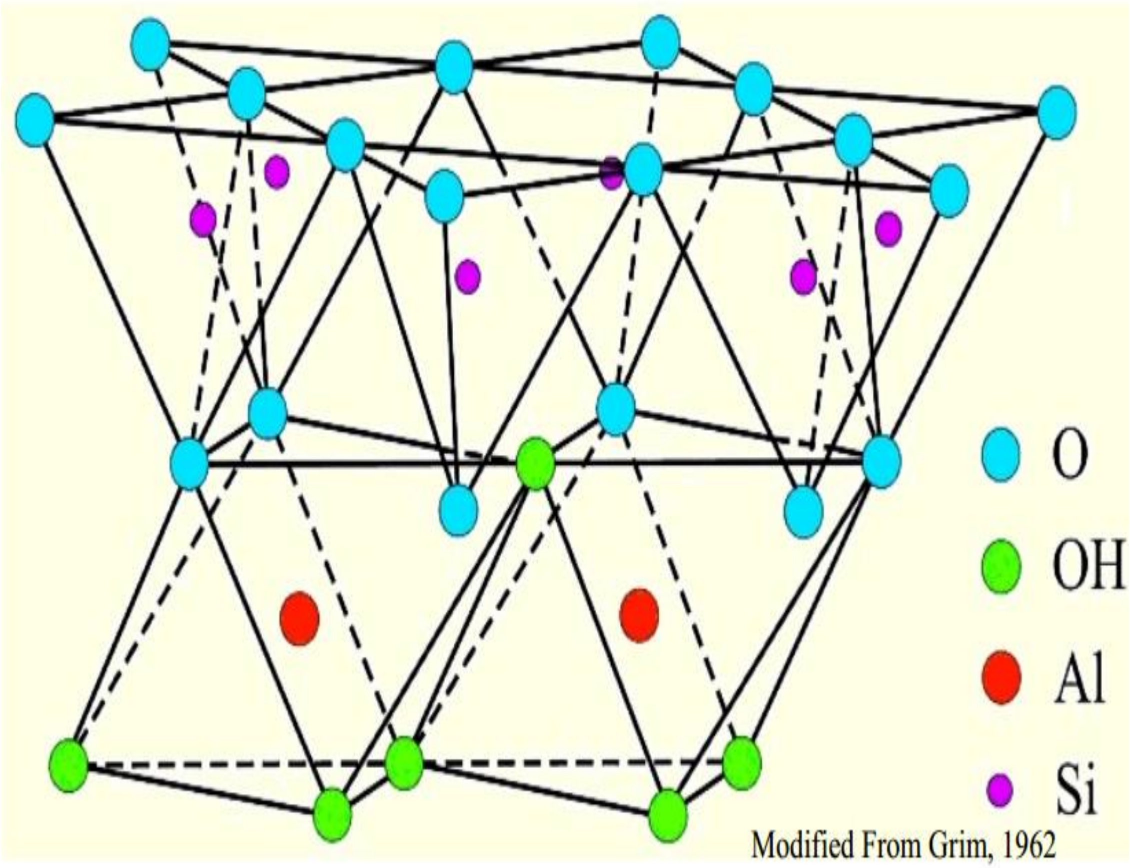

Figure 2. 4 Diagrammatic Sketch of the Kaolinite (after USGS, 2001)

II. Mica-like group

Illite

Illite has a basic structure consisting of a sheet of alumina octahedrons between and combined with two sheets of silica tetrahedrons. In the octahedral sheet there is partial substitution of aluminum by magnesium and iron, and in the tetrahedral sheet there is partial substitution of silicon by aluminum. The combined sheets are linked together by fairly weak bonding due to (non - exchangeable) potassium ions held between them (Craig, 1997).

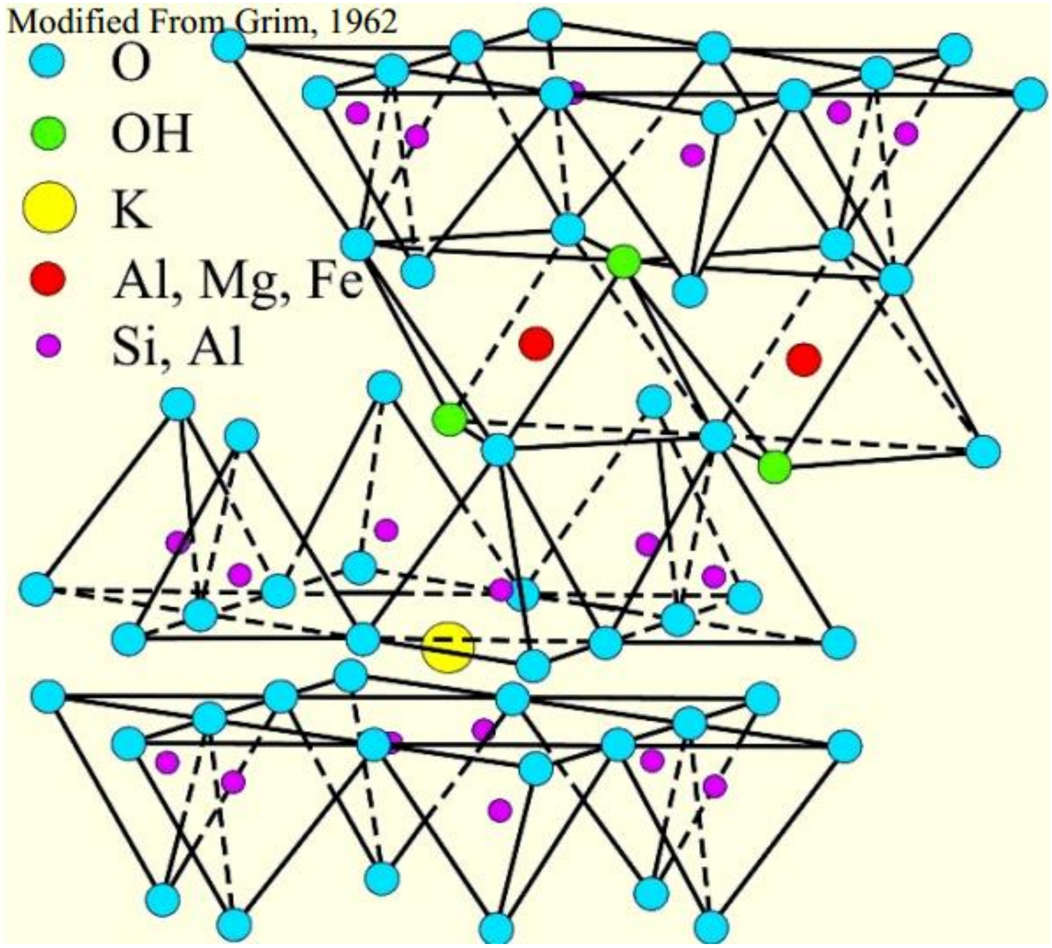

Figure 2. 5 Diagrammatic Sketch of the Illite (after USGS, 2001)

III. Smectite group

Montmorillonite

Montmorillonite is formed from weathering of volcanic ash under poor drainage conditions or in marine waters. The basic building sheets for smectite are the same as for illite except there is no potassium ion present. The space between the combined sheets is occupied by water molecules and exchangeable cations. There is a very weak bond between the combined sheets due to these ions. Considerable swelling of montmorillonite can occur due to additional water being absorbed between the combined sheets (Craig, 1997; Oweis and Khera, 1998)

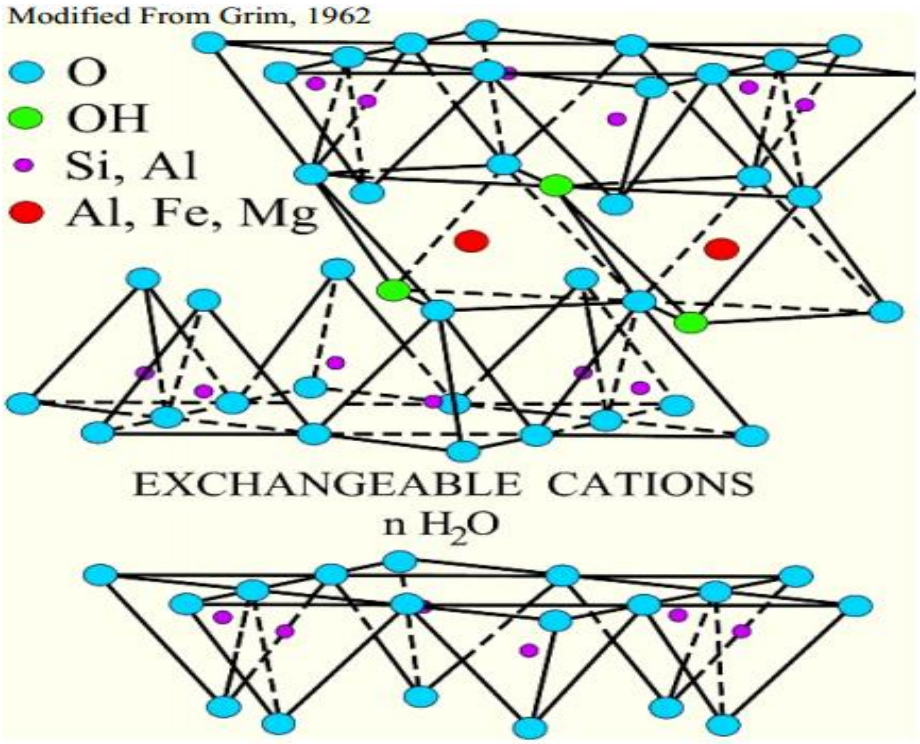

Figure 2. 6 Diagrammatic Sketch of the Montmorillonite (after USGS, 2001)

2.3. Identification

Expansive soils can be recognized both visually and series of laboratory test identification methods.

2.3.1. Field identification

Expansive soils can be visually identified (Wayne et al. 1984):

1) Wide and deep shrinkage cracks occurring during dry periods

2) Soil is rock-hard when dry, but very sticky and soft when wet

3) Damages on the surrounding structures due to expansion of soils

4) Grey or black in color

2.3.2. Laboratory identification

These are three methods of identifying expansive soils in laboratory test method. These are mineralogical identification, indirect methods and direct measurement methods.

2.3.2.1. **Mineralogical identification:**

Mineralogical identification can be useful in the evaluation of clay mineralogy quantitatively such as crystal dimensions, characteristic reaction to heat treatment, size and shape of clay particles and change deficiency and surface activity of clay particle, but is not sufficient in itself when dealing with natural soils. The various methods of mineralogical identification are important in a research laboratory in exploring the basic properties of clays, but are impractical and uneconomical for practicing engineers. Techniques which may be used as mineral identification methods are as follows. (Chen, F.H., 1988, John, D., Nelson 1992).

- X-ray diffraction.
- Differential thermal analysis.
- Dye adsorption.
- Chemical analysis and
- Electron microscope resolution.

Due to their property of time consumption, expensiveness, need of skill technician for result interpretation, this test cannot be considered as a routine tests.

2.3.2.2. Indirect techniques

These are Simple soil property tests can be used for the evaluation of the swelling potential of expansive soils. Such tests are easy to perform and should be included as routine tests in the investigation of construction sites in those areas having expansive soil. Such tests may include:

- Atterberg limits tests
- Linear shrinkage tests
- Free swell tests
- Colloid content tests and

- activity index

i. **Atterberg limit test**: Holtz and Gibbs demonstrated in 1956 that plasticity index and liquid limit are useful indices for determining the swelling characteristics of most clays. Seed, Woodward, and Lundgren have demonstrated that the plasticity index alone can be used as a preliminary indication of swelling characteristics of most clays (Chen, 1988). Relation between swelling potential of clays and plasticity index can be established as follows:

Table 2. 1 Relation between swelling potential of clays and plasticity index.

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

ii. **Linear Shrinkage**: The swell potential is presumed to be related to the opposite property of linear shrinkage measured in a very simple test. In theory it appears that the shrinkage characteristics of the clay should be a consistent and reliable index to the swelling potential.

It was suggested by Altmeyer in 1955 (cited by Chen 1988) as a guide to the determination of potential expansiveness for various values of shrinkage limits and linear shrinkage as follows:

Table 2. 2 Relationships between degree of expanision vs linear shirnkakege

Shrinkage limit as a percentage	Linear shrinkage as a percentage	Degree of expansion
Less than 10	Greater than 8	Critical
10 -12	0 -8	Marginal
Greater than 12	0 - 5	Non-critical

iii. **Free swell:** This test is a simple test conducted in laboratory for determination of degree of expansiveness. Free swell tests consist of placing a known volume of dry soil in water and noting the swelled volume after the material settles, without any surcharge, to the bottom of a graduated cylinder. The difference between the final and initial volume, expressed as a percentage of initial volume, is the free swell value. The swell test is very crude and was used in the early days when refined testing methods were not available. Experiments indicated that a good grade of high swelling commercial bentonites will have a free swell value from 1200 to 2000 percent. Soils having free swell value as low as 100 percent can cause considerable damage to lightly loaded structures, and soils having free swell value below 50 percent seldom exhibit appreciable volume change even under very light loadings.(Chen. F.H, 1998, Nelson 1992).

Free swell (%) = $\frac{vf-vi}{vi}$ where

V_f = final volume reading after 24 hours

V_i = initial volume seated on cylindrical plate (10cc)

Table 2. 3 Relation B/N % Free Swell and Condition of Expansiveness

Free swell (%)	Expansiveness
<50	Not expansive
50-100	Marginal
>100	Expansive

iv. **Colloid content:** The grain size characteristics of a clay appear to have a bearing on its swelling potential, particularly the colloid content. Seed, Woodward, and Lundgren (1962) believed that there is no correlation between swelling potential and percentage of clay sizes. However, for a given clay type, the amount of swell will increase with the amount of clay (Chen, 1988)

Activity index: Atterberg limits and clay contents have been combined in a single parameter called activity ratio sometimes called activity index (A) developed by skempton (1953).

Activity (Ac) = $\frac{plasticity\ index}{\%\ clay}$. The % clay is defined as % of particles less than 0.2μm.

According to skempton (1953):

Table 2. 4 Relation between Activity Ratio Vs Swelling Potential

Activity ratio (Ac)	Class
<0.75	Inactive
0.75<A<1.25	Normal
>1.25	Active

Active clays provide a potential for expansion. Ca-saturated Montmorillonite has an estimated Ac value of 1.5 and Na-saturated montmorillonite has an activity ratio greater than 7. Kaolinite has Ac value of 0.3 and illite has Ac value of about 1. But this classification doesn't accurately estimate for most mixed mineralogy soils (Thomas, 1998).

2.3.2.3. Direct methods

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 a clay under various conditions.

Table 2. 5 Swelling Potential Prediction in Soils

Parameter	reference	Degree of expansion			
		low	medium	high	Very high
LL (%)	Chen,1975	<30	30-40	40-60	>60
PI (%)	Chen, 1975	0-15	10-35	20-55	>55
	Holtz and Gibbs, 1956	<20	12-43	23-45	>45
Swell percent (%)	Thomas et al, 2000	<3	3-9	6-9	>9
Swell pressure(Kpa)	Thomas et al,2000	<81	81-153	153-225	>225

Note Potential expansion is given for a confined sample with vertical pressure equal to overburden pressure expressed as a percentage of simple weight. Studied soil properties and their proposed relation to degree of expansion are summarized in above table (Dinkie A., June 2015).

2.4. Damaging effect of expansive sub-grade soils

Expansive soils are problematic soils because of their inherent potential to undergo volume changes corresponding to changes in the moisture regime. When they imbibe water during rainy season, they expand and exert enough force on pavement structures to cause damage. On evaporation thereof in dry season, they shrink and remove support from pavements and other structures and result in damage subsidence. The fissures in the soil also develop and facilitate deep penetration of water when moist conditions or runoff occur. Because of this alternate swelling and shrinkage, both light and heavy structures; like railroads, highways, airport pavements and lined and unlined irrigation canals and other civil engineering structures severely damaged.

Bulman (1980) presented problems associated with black clay soils of Africa as sub-grade soil under paved roads. According to him the problems are not only because of its swelling characteristics due to moisture variations and a very low bearing strength when saturated. But also difficulties during the construction stage is that it is difficult to handle if too wet or too dry. He also indicated the difficulty to raise the moisture of such soils to optimum for compaction in arid climates and thus results in low densities in compacted fills under road pavements built on black clay soil. The other problem of expansive soil is its susceptibility to erosion. Highly expansive clays, such as black cotton soils, when dry exhibit a sand like texture and are susceptible to erosion to a much greater extent than that normally anticipated from their plasticity and clay content (ERA, 2002).

Expansive soils have a low CBR value in the range of 2 to 4% that they have low strength to support the loads transmitted from the pavement structure and results in excessive deformation beyond permissible limits (Rao, 2007; Seehra, 2008). The low load bearing capacity of expansive soils is affected by many factors such as moisture content, degree of compaction, soil type etc. Volumetric changes of expansive soil which leads to pavement distortion, cracking and general unevenness due to seasonal wetting and drying are explained by Rao (2007). This soil absorbs water heavily, swell, become soft, easily compressible and lose strength greatly.

Seehra (2008) and Rao (2007) described the effect of water to pavements. In their study they proved that water is the worst enemy of road pavement, particularly in expansive soil areas and Poor drainage facility and results in damages of pavements. Furthermore, Seehra (2008) described the paths that water enters in to pavements. Water penetrates into the road pavement through the top surface, side berms and from sub-grade due to capillary action. Therefore, the road surfacing must be impervious, side berms paved and sub-grade well treated to check capillary rise of water.

2.5. Mitigating the effect of expansive sub-grade soils

The following are some of the methods used to mitigate the detrimental effect of expansive sub-grade soils.

- Soil replacement
- Surcharge loading
- Pre-wetting
- Moisture control
- Compaction control

- Grouting
- Mechanical stabilization
- Chemical stabilization
- Application of geosynthetics (e.g. geogrids and geotextiles)

2.5.1. Soil replacement

This method eliminates the swelling problems of expansive subgrade soil by replacing natural expansive subgrade with non-expansive material. The method is economical if expansive stratum or layer is thin and the replacement materials are available. The required depth of excavation depends upon the expansiveness of the soil and the anticipated weight of back fill and the structure which will counteract the uplift forces of swelling soil. The selection of the particular non-expansive backfill material is critical.

2.5.2. Surcharge loading

Swelling of expansive soils can be prevented by loading pressure greater than the swelling pressure. However, pavement loads are insufficient to prevent the expansion and usually applied in case of large structures.

Sallberg, (1965) mention that pavement designs developed by the California Division of Highways are based partly on the requirement that the pavement weigh enough to prevent expansion of subgrade. But the use of this method is for soils with low expansive pressure and care should take when balancing the pavement weight and swell pressure.

2.5.3. Pre-wetting

This method allows desiccated swelling soils to reach equilibrium prior to placement of roadway or structure. Ponding is the most common method applied for accelerating the swelling. But the how long the material should be ponded and what depth the moisture should penetrate to be effective are still unknown (snethen et al, 1975).

2.5.4. Moisture control

This method use moisture barriers such as bituminous or full depth bituminous pavements for controlling volume change as an effective way. However where the capillary moisture or high water tables preclude effective sealing of the expansive subgrade from moisture accumulations, a membrane obviously will be ineffective. Bituminous products appear to be the most widely used material for membranes. To be effective, complete sealing across ditches and up the back slopes is required (snethen et al, 1975).

2.5.5. Compaction control

This method is usually the most economical means to achieve particle packing for both cohesion less and cohesive soils and usually uses some kind of rolling equipment. Dynamic compaction is a special type of compaction consisting of dropping heavy weights on the soil. The swell pressure or volume change of compacted clays at optimum water contents can be reduced by compacting the soil to low or medium densities above optimum.

2.5.6. Grouting

Initially this was the name for injection of a viscous fluid to reduce the void ratio to cement rock cracks. Currently this term is loosely used to describe a number of processes to improve certain soil properties by injection of a viscous fluid, sometimes mixed with a volume of soil. Most commonly, the viscous fluid is a mix of water and cement or water and lime, and/or with additives such as fine sand, bentonite clay, or fly ash. Bitumen and certain chemicals are also sometimes used. Additives are used either to reduce costs or to enhance certain desired effects.

2.5.7. Mechanical stabilization

Mechanical stabilization is the most widely used method of stabilization. It is the process of improving the properties of the soil by changing its gradation. Two or more types of natural soils are mixed to obtain a composite material which is superior to any of its components. To achieve the desired grading, sometimes the soils with coarse particles are added or the soils with fine particles are removed. Mechanical stabilization is also known as granular stabilization. For the purpose of mechanical stabilization, the soils are subdivided into two categories:

1) Aggregates: These are the soils which have a granular bearing skeleton and have particles of the size larger than 75μ.

2) Binders: These are the soils which have particles smaller than 75 µ size. They do not possess a bearing skeleton. The aggregates consist of strong, well-graded, angular particles of sand and gravel which provide internal friction and incompressibility to a soil. The binders provide cohesion and imperviousness to a soil. These are composed of silt and clay. The quantity of binder should be sufficient to provide plasticity to the soil, but it should not cause swelling. Proper blending of aggregates and binders is done in order to achieve required gradation of the mixed soil. The blended soil should possess both internal friction and cohesion. The material should be workable during placement. When properly placed and compacted, the blended material becomes mechanically stable. The load-carrying capacity is increased. The resistance against the temperature and moisture changes is also improved. Mechanical stabilization is the simplest method of soil stabilization. It is generally used to improve the sub-grades of low bearing capacity. It is extensively used in the construction of bases, sub-bases and surfacing of roads.

2.5.8. Chemical stabilization

In chemical stabilization, soils are stabilized by adding different chemicals. The main advantage of chemical stabilization is that setting time and curing time can be controlled. However, Chemical stabilization is generally more expensive than other types of stabilization. The most common used chemicals are calcium chloride, sodium chloride, sodium silicates, polymers and chrome lignin.

Soil improvement by means of chemical stabilization can be grouped into three chemical reactions; cation exchange, flocculation - agglomeration, pozzolanic reactions.

2.5.8.1. 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 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 and divalent cations are held more tightly than monovalent cations (Mitchell and Soga, 2005).

A typical replace ability series is:

$Na+ < Li+ < K+ < Rb+ < Cs+ < Mg2+ < Ca2+ < Ba2+ < Cu2+ < Al3+ < Fe3+ < Th^{4+}$

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

Ca2+ + Na+-Clay $\rightarrow$ Ca2+ 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.

2.5.8.2. Flocculation and Agglomeration

Cation exchange reactions result in the flocculation and agglomeration of the soil particles with 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 change in texture, a significant reduction in the swelling of the soil occurs.

2.5.8.3. Pozzolanic Reactions

Time depending 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 produces calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH).

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

Ca^{2+} + 2$(OH)^-$ + Al_2O_3 (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, strength of the soils increases (Hadi et al, 2006; Sivapullaiah, 2006).

2.5.9. Lime stabilization

Lime stabilization is done by adding lime to a soil. It is useful for stabilization of clayey soils. When lime reacts with soil, there is exchange of cations in the adsorbed water layer and a decrease in plasticity of the soil occurs. The resulting material is more friable than the original clay, and is, therefore, more suitable as subgrade. Lime is produced by burning of lime stone in kilns. The quality of lime obtained depends upon the parent material and the production process.

There are basically 5 types of limes.

(I) High calcium, quick lime (CaO)

(II) Hydrated , high calcium lime [$Ca(OH)2$]

(III) Dolomitic lime ($CaO + MgO$)

(IV) Normal, hydrated dolomitic lime [$Ca(OH)_2 + MgO$]

(V) Pressure, hydrated dolomitic lime [$Ca(OH)_2 + MgO2$].

The quick lime is more effective as stabilizer than the hydrated lime; but the latter is more safe and convenient to handle. Generally, the hydrated lime is used. It is also known as *slaked lime.* The higher the magnesium content of the lime, the less is the affinity for water and the less is the heat generated during mixing. The amount of lime required for stabilization varies between 2 to 10% of the soil. However, if the lime is used only to modify some of the physio-chemical characteristics of the soil, the amount of lime is about 1 to 3%.

The following amount may be used as a rough guide.

i. 2 to 5% for clay gravel material having less than 50% of silt- clay fraction.
ii. 5 to 10% for soils with more than 50% of silt-clay fraction.
iii. For soils having particle size intermediate between (i) and, (ii) above, the quantity of lime required between 3 to 7%.
iv. About 10% for heavy clays used as base and sub -base.

Lime stabilization is not effective for sandy soils. However, these soils can be stabilized in combination with clay, fly ash or other pozzolanic materials, which serve as hydraulically reactive ingredients.

The chemical theory involved in the lime reaction is complex (Thompson, 1966, 1968). The main reactions include cation exchange, flocculation and pozzolanic reactions (cited in Nelson and Miller, 1992). The cation exchange and flocculation concepts, primarily effects of stabilizer, were stated in Section 2.5.7.1 and 2.5.7.2, respectively. Also, in Section 2.5.7.3, the pozzolanic reactions for lime stabilized soils were presented. These three stabilization steps are valid for stabilization of expansive soils using waste limestone dust and waste dolomitic marble dust (cited in Onur Baser, 2009).

2.5.10. Marble waste powder stabilization

Marble is a highly crystalized, metamorphosed limestone transformed through the heat and pressure into a dense, variously colored, crystallized rock. It is predominantly composed of calcite with minor impurities. Pure calcite is white in color. Iron and magnesium and some silicate minerals give a significant green color; graphite gives dark, pyrite greenish grey and hematite color marble pink. Some rare colors like sky-blue are due to impurities or “failure” within the calcite crystals (Heldal, and Wale, 2000 and Ministry of Mines 2002).

Extensive literature is available on soil improvement by the application of additives, notably cement and lime. Lately, many researchers have reported on additives that could substitute lime as a soil modifier. Such materials include fly ash (Çokça, 1999; Indraratna et al. 1991, 1995), rice husk (Muntohar, 1999); (Muntohar and Hantoro 2000), marble dust (Okagbue and Onyeobi, 1999), and limestone ash (Okagbue and Yakubu, 2000) (Cited in Okagbue, 2007).

A lot of research work has been done worldwide in the direction of utilizing of marble dust waste into the soil stabilization technique (*Parte Shyam Singh*, *2014*). The effect of marble dust with rice husk ash on expansive soil has been studied by Sabat and Nanda (2011). CBR and UCS values increases substantially due to addition of marble dust and rice husk with expansive soil. The waste material marble dust used as a stabilizer in expansive soil. Vishwakarma and Rajput (2013) studied the utilization of waste marble slurry to enhance the soil properties. The main objective of their study was to utilize the waste material like marble slurry to improve the characteristics of black-cotton soil (*Parte Shyam Singh, 2014)*. Gupta and Sharma (2014)studied the influence of marble dust, fly-ash and Beas sand on sub grade characteristics of expansive soil.), and Palaniappan (2009) and Agrawal *et al.* (2011), had investigated that marble dust powder is effective waste material in the stabilization of expansive soil. Which improve the index and engineering properties like as LL, PL, SL, compaction, swelling characteristics (*Parte Shyam Singh, 2014)*. Many researchers (Çelik and Sabah, 2007; Zorluer and Usta, 2003; Oates, 1998; Almedia et al., 2007; Tegethoff, 2001) have reported that marble has very high lime (CaO) content up to 55 % by weight. Thus, stabilization characteristics of dolomitic marble dust is mainly due to their high lime (CaO) content (cited in Basar, 2009).

2.5.10.1. Production of Waste Marble Dust

Waste marble dust can be produced from any rock that can be polished in marble plants (Oates, 1998). Limestones, schists, travertines or even granites can be considered as marble in the business world (Onargan et al., 2005). The production of fine particles (<2 mm) while cutting marble is one of the major problems for the marble industry. When 1 m^3 marble block is cut into 2 cm thick slabs, the proportion of fine particle production is approximately 25 % (Kun, 2000). While cutting of marble blocks water is used as cooler. But, the fine particles can be easily dispersed after losing humidity, under atmospheric conditions, such as wind and rain. Thus, fine particles can cause more environmental pollution than other forms of marble waste (cited in Çelik and Sabah, 2007).

Figure 2. 7 Partial view of how the factory remove the marble waste powder from working site.

3. METHODOLOGY

3.1. Materials used for testing

3.1.1. Expansive subgrade soil sample

The expansive soil used in this study was collected at depth of 1.5m below natural ground from Bole-Arabsa project-13 new condominium site where current massive government saving house construction and urban development on progress, Bole sub-city, Addis Ababa, Ethiopia. Due to previous studies implications of Bole area most similar range of engineering and index properties and economic constraints, One borehole sample was dug manually and a representative bulk disturbed sample of black cotton expansive clay soil

placed in sacks and transported to laboratory for test and analysis.

Figure 3. 1 Partial view of bole arabsa project-13 new condominium site expansive soil

3.1.2. Lime

The hydrated lime was collected in Addis Ababa at Gast solar mechanics.

3.1.3. Marble Waste Powder

Marble waste powder were collected from ***National Mineralogical Corporation,*** Ethiopian Marble processing enterprise, Gulele branch, Addis Ababa, Ethiopia.

3.1.4. Availability of marble waste powder 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).

Known deposit of Marble in Ethiopia

- Daleti Marble
- Bapuri Marble
- Enkonte Marble
- Tulu Mon Marble
- Naeder Marble
- Bulen Marble
- Mora Marble
- Zigi Marble
- Baruda Marble
- Mankush Marble
- Newi Marble
- Adiwonyane Marble
- Taget Marble
- Akmara Marble
- Endetukri/ Tekeze Marble

Most of the outcrops are hilly to cliff-forming, white to gray, coarse grained composed of dominantly calcite. Pink, greenish and sky-blue varieties and dolomitic marbles are locally present (Heldal, Wale and Zewdie 1987; Heldal, Wale, 2000). The difference between Marble deposit of northern and western Ethiopia is the age, the grain size and color. These variations are accounted for the low degree of metamorphism. White, yellow and violet verities of colors are common. (Heldal, and Wale, 2000). Marble is quarried for a variety of architectural and artistic purposes in Ethiopia. They are essentially small-scale operations, applying hand-held drill hammers and wedges as the primary tools, without using more sophisticated methods such as blasting and/or sawing. Wedging along vertical and horizontal drill-hole lines, helped by natural joints, makes primary cuts. Final shaping of blocks is made with secondary drilling and wedging.

There are 14 marble processing companies in Ethiopia. The country export marble and present home demand for the mineral product is estimated at 211,252sq.m per annum. The demand is expected to reach 729.299sq.m by the year 2020.

The production of fine particles (<2 mm) while cutting marble is one of the major problems for the marble industry. When 1 m^3 marble block is cut into 2 cm thick slabs, the proportion of fine particle production is approximately 25 % (Kun, 2000). Therefore much amount of waste marble powder is produced to prepare the above mentioned quantity products.

3.2. Preparation and designation of test samples

The sample was air and sun dried and all lumps broken. The marble waste powder and the hydrated lime was directly fetched from the sack and measured. In this test the percentage of the lime was tried to fix by its PH value. As outlined in ASTM D 6276-99a (PH =12.4) for subgrade was achieved at 6% lime content. However, when this amount of lime mixed with marble waste powder and soil, the optimum moisture content and maximum dry density can’t be achieved due high crystalized lime content of marble waste powder. Therefore the maximum percent of lime content used in this study was fixed by trial of doing series of compaction tests and it was 3% lime. 5%, 10%, 15%, 20%, and 25% marble waste powder with fixed 3% lime was mixed with expansive soil. Lime and waste marble powder was added based on total weight of the mixture. Stabilizers used for all tests done in this study, fixed 3% lime content was mixed with variable (5%, 10%, 15%, 20%, and 25%) marble waste powder. The sample row was designed SML-0 as unsabilized samples and stabilized samples as SML-5, SML-10, SML-15, SML-20, SML-25, and SML-30 for 3% fixed lime and 5%, 10%, 15%, 20%, and 25% marble waste powder dry mix as shown in the table 3.1.

Table 3. 1 Summery of sample designation for all tests

Sample ID	Quantity of soil Sample (%)	Quantity of lime (%)	Quantity of Marble waste Powder in (%)	Total mix (%)
SML-0	100	0	0	100
SML-5	92	3	5	100
SML-10	87	3	10	100
SML-15	82	3	15	100
SML-20	77	3	20	100
SML-25	72	3	25	100
SML-30	67	3	30	100

3.3. Test procedure

For tests conducted, ASTM and AASHTO standards were used and summarized in the table below.

Table 3. 2 Laboratory Tests Conducted

Category	Test	Standard test used
Index properties	Atterberg limit	ASTM D 4318
	Grain size analysis	ASTM D 422
	Specific gravity	ASTM D 854-00
Mechanical properties	Compaction	ASTM D 698
	CBR	AASHTO T 193
	24-hour Free swell	
	UCS	ASTM D 2166-00
	One dimensional swell consolidation	ASTM D 2435-02
	PH	ASTM D 6276-99a

3.3.1. Atterberg limits test

Atterberg limits test were conducted for both unstabilized and stabilized expansive soil samples according to the procedure in ASTM D 4318 for testing liquid limit and plastic limit where plasticity indices were calculated for air dried soil sample from liquid limit and plastic limit values. 100g of sample for liquid limit and 20g of sample for plastic limit was used.

3.3.2. Grain size analysis test

The standard test was performed for untreated soil sample as outlined in ASTM D 422. Grading test was conducted by wet sieve analysis including hydrometer meter test for fine particles that passed in #200 sieve.

3.3.3. Specific gravity test

The laboratory test was performed to determine the specific gravity of soil by using a water pycnometer. Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. This test was conducted for treated and untreated soil according to outlined on ASTM D 854-00. In this test method the determination of the specific gravity of soil solids that pass the 4.75-mm (No. 4) sieve is by means of a water pycnometer.

3.3.4. Compaction test

Standard compaction test for moisture density relationship was conducted using procedure outlined in ASTM D 698. The treated and untreated samples were set curing room for 24 hours in order to get uniform moisture distribution. A soil at a selected water content is placed in three layers into a mold, with each layer compacted by 25 blows of 24.4-N rammer dropped from a distance of 305-mm.

3.3.5. California bearing ratio test

A laboratory determination of one-point method California bearing ratio (CBR) of dynamically compacted soils on CBR molds by modified metal rammers was used to indicate the CBR value according to AASHTO T193. 6kg of sample was prepared using optimum moisture content determined from standard compaction test for both treated and untreated samples. CBR was immediately measured and soaked CBR was measured after soaked for 96 hours under surcharge load of 4.54 kg.

3.3.6. Twenty four hour free swell test

Free swell tests consist of placing 10cc of dry soil passing a sieve size 0.425mm (No. 40) into a 100cc graduated jar filled with water and noting the swelled volume after 24 hour material settles, without any surcharge, to the bottom of a graduated cylinder. The difference between the final and initial volume, expressed as a percentage of initial volume was measured.

3.3.7. Unconfined compressive strength

The unconfined compressive strength test was made according to ASTM D 2166-00. The sample was remolded with 38mm diameter and height of 76mm which was prepared with optimum moisture content determined from standard compaction test for both untreated and treated soil samples. A strain-controlled axial load was applied until peak stress was obtained to get approximate value of the strength of sample soils in terms of total stresses.

3.3.8. One-dimensional swell consolidation

Both treated and untreated soil sample test was performed according to ASTM D 2435. The samples were prepared with moisture content determined from standard compaction test. The sample was compacted and remodeled by consolidometer ring. Samples taken for moisture content determination. The consolidometer ring was weighted and taken to consolidation Machine where porous stones were placed above and bottom of it. Then the sample was allowed to swell under 7kpa setting pressure. Initial reading was taken immediately and final reading was taken after specimen stopped to swell. Then loading continued with loads of 0.5kg, 1kg, 2kg, 4kg, 8kg, 16kg, and reading taken for 24hours. Unloading the loads applied and reading taken for 24 hours.

4. RESULTS AND DISCUSSION

4.1. Property of Bole-arabsa condominium project-13 untreated soil

In this section some test of AA bole-arabsa untreated expansive soil were presented.

4.1.1. Particle size analysis

This test was performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of the finer particles using ASTM D 422 - Standard Test Method for Particle Size Analysis of Soils.

Table 4. 1 Mechanical Sieve Analysis Test

Sample mass taken=500

Sieve No	Sieve Opening (mm)	Mass of Retained soil (g)	Percentage Retained (%)	Cum. Percentage Retained (%)	Percentage Passing (%)
3"	75	0	-	-	100.00
2"	50	0	-	-	100.00
1.5"	37.5	0	-	-	100.00
1"	25	0	-	-	100.00
3/4"	19	0	-	-	100.00
1/2"	12.5	0	-	-	100.00
3.8"	9.5	0	-	-	100.00
No 4	4.75	0	-	-	100.00
No 8	2.36	0	-	-	100.00
No 10	2	9.8	2.21	2.21	97.79
No 20	0.85	4.4	0.99	3.20	96.80
No 40	0.425	3.4	0.77	3.97	96.03
No 60	0.25	2.1	0.47	4.44	95.56
No 100	0.15	2.4	0.54	4.98	95.02
No 200	0.075	2.5	0.56	5.54	94.46
Pan	-------	7.1	-	-	-------
	total mass=	443.65			

Table 4. 2 Hydrometer analysis for soils pass in #200 sieve

Hydrometer Analysis								
Specific Gravity of soil = 2.66					Test Temperature (deg. c) 20.8			
Elapsed Time (min)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
0.75	1.0322	-0.0025	1.0297	7.8031	0.0135	0.0435	95.31	94.12
1	1.0319	-0.0025	1.0294	7.8825	0.0135	0.0379	94.35	93.17
2	1.031	-0.0025	1.0285	8.1205	0.0135	0.0272	91.47	90.32
4	1.03	-0.0025	1.0275	8.3850	0.0135	0.0195	88.26	87.16
8	1.0285	-0.0025	1.0260	8.7817	0.0135	0.0141	83.45	82.41
15	1.0275	-0.0025	1.0250	9.0462	0.0135	0.0105	80.25	79.25
30	1.0255	-0.0025	1.0230	9.5752	0.0135	0.01	73.84	72.92
60	1.0242	-0.0025	1.0217	9.9191	0.0135	0.0055	69.67	68.80
120	1.0234	-0.0025	1.0209	10.1307	0.0135	0.0039	67.11	66.27
240	1.022	-0.0025	1.0195	10.5010	0.0135	0.0028	62.62	61.84
480	1.021	-0.0025	1.0185	10.7655	0.0135	0.0020	59.42	58.67
1440	1.0202	-0.0025	1.0177	10.9771	0.0135	0.0012	56.85	56.14

The grain size test result showed, 58.67% clay size, 35.79% silt size, 5.54% and 0% gravel which is poorly graded.

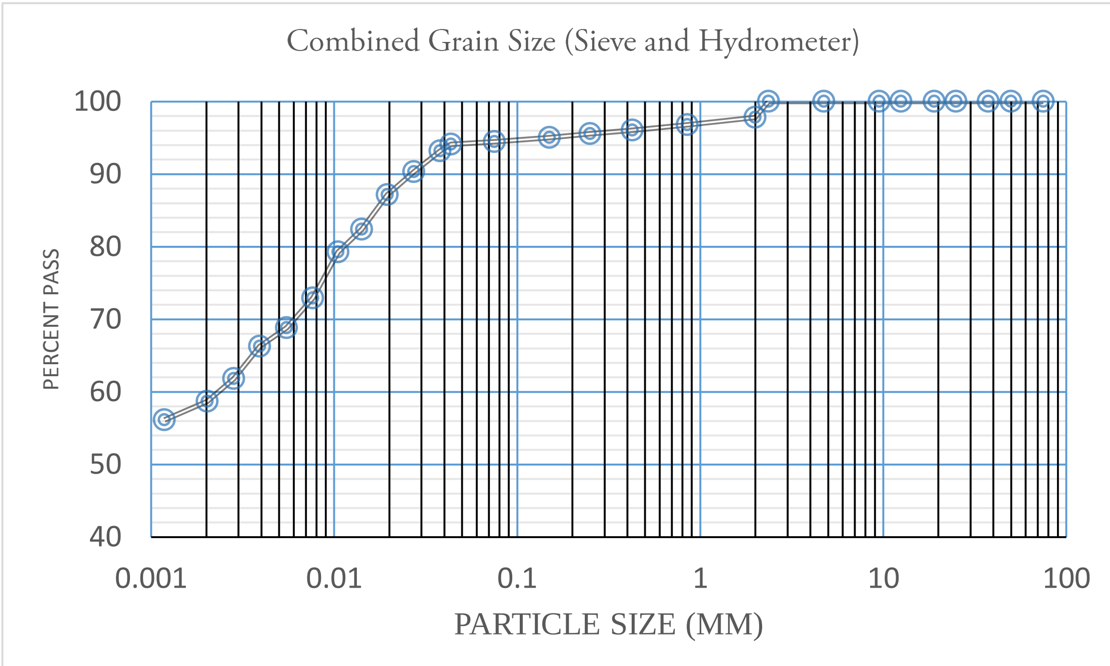

Figure 4. 1 Combined Grain Size (Sieve and Hydrometer) of AA *Bole Arabsa* Expansive Soil

4.1.2. Laboratory compaction test

Standard compaction laboratory test was performed to determine the relationship between the moisture content and the dry density of a soil for a specified compactive effort using ASTM D 698-00.

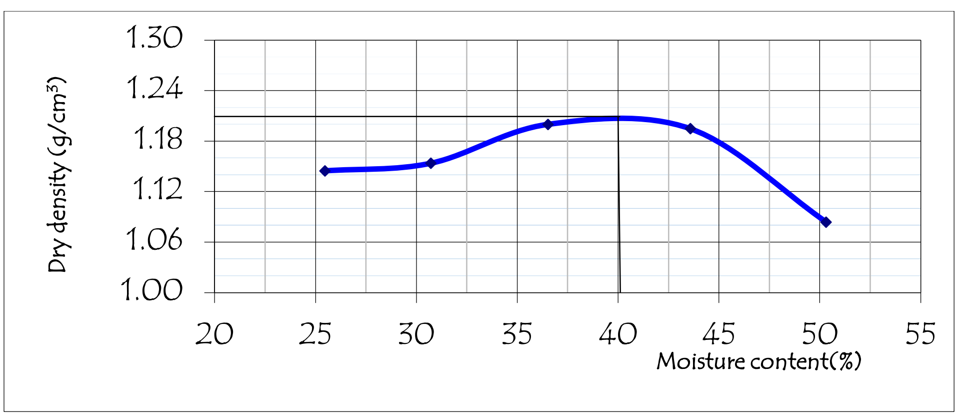

Figure 4. 2 Moisture Density Relationship of AA *Bole Arabsa* Expansive Soil

Optimum moisture content (%) = 40 Maxim um dry density, (g/cm^3) = 1.21

4.1.3. Specific gravity of soil solids by water pycnometer

The specific gravity of bole-arabsa soil using a water pycnometer as outlined in ASTM D 854-00 is shown in the table below.

Table 4. 3 Determination of Specific Gravity Test

Determination No.	1	2
Pycnometer No.	B7	A6
Temperature, T_x (^oc)	25.2	25.2
Weight of pycnometer + water at T_x , W_{pw} (atT_x) (g)	144.00	146.90
Weight of pycnometer + soil + water, W_{pws} (g)	159.6	162.5
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.99879	0.9988
Specific gravity of soil at 25.2°c.	2.66	2.66
Average specific gravity of soil G20°C=K · Gt		2.66

4.1.4. Atterberg limits

The test was performed to determine the plastic and liquid limits of a fine grained soil. The liquid limit (LL) is arbitrarily defined as the water content, in percent, at which a pat of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (1/2 in.) when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. The plastic limit (PL) is the water content, in percent, at which a soil can no longer be deformed by rolling into 3.2 mm (1/8 in.) diameter threads without crumbling. The test was performed using ASTM D 4318 - Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. Atterberg limit test results of a natural soil are high and which are not recommended for subgrades use based on the standards.

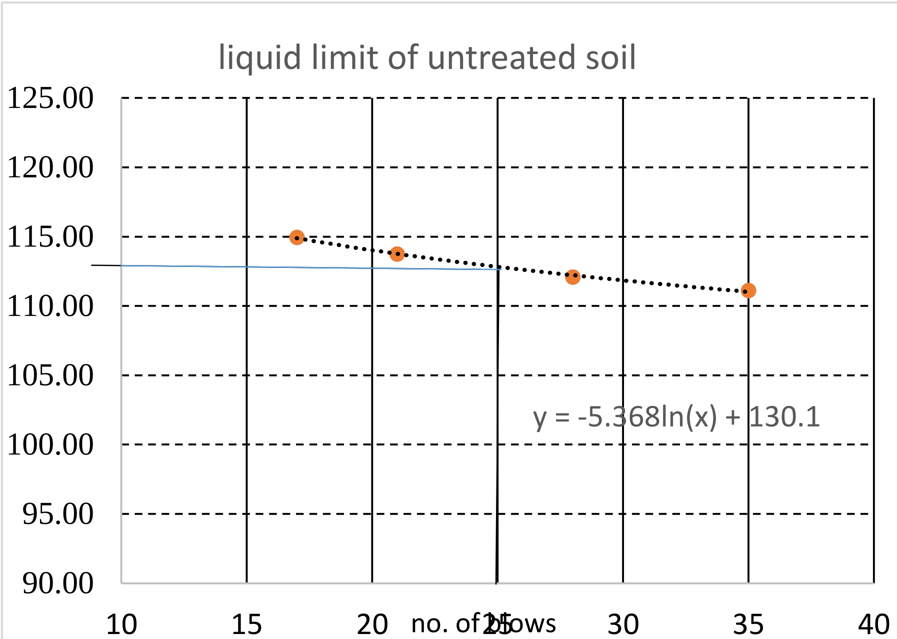

Figure 4. 3 Liquid Limit of Untreated Addis Ababa *Bole Arabsa* Untreated Soil

Liquid Limit, % = 113 Plastic Limit, % =35 PI, % =78

4.1.5. Classification

The soil was classified based on AASHTO and USCS classification system.

Table 4. 4 Summary of *Bole-Arabsa* Untreated Soil Classification

USCS soil classification system	AASHTO soil classification system
CH	A-7-6[20]

The soil classified CH based on USCS and A-7-6[20] based on AASHTO are poor subgrade material which require replacement or treatment for subgrade use.

4.1.6. Free swell index

This test was conducted in laboratory for determination of degree of expansiveness. Free swell tests consist of placing a known volume of dry soil in water and noting the swelled volume after 24 hour material settles, without any surcharge, to the bottom of a graduated cylinder. The difference between the final and initial volume, expressed as a percentage of initial volume, is the free swell value. The soil expands 95% which is expansive and results considerable damage to civil engineering structures.

4.1.7. Unconfined comprehensive strength

The test was performed using ASTM D 2166-00. The average value qu is 77.65 kpa.

Table 4. 5 Unconfined Compressive Strength Value of Bole-Arabsa Untreated Soil

Sampling		disturbed	Cross- Sectional Area , m^2		0.001134
Diameter of sample , mm		38	Ring Calibration Factor, kN/div		0.00133
Length of sample , mm		76	Rate of Strain, mm/min		0.80
Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m2]	Axial Stress [kPa]
0.00	0.00	0	0.0000	0.001134	0
0.20	0.26	4	0.0053	0.001137	4.68
0.40	0.53	11	0.0146	0.001140	12.83
0.60	0.79	18	0.0239	0.001143	20.94
0.80	1.05	27	0.0359	0.001146	31.33
1.00	1.32	35	0.0466	0.001149	40.51
1.20	1.58	42	0.0559	0.001152	48.48
1.40	1.84	48	0.0638	0.001155	55.25
1.60	2.11	54	0.0718	0.001159	61.99
1.80	2.37	61	0.0811	0.001162	69.84
2.00	2.63	68	0.0904	0.001165	77.65
2.20	2.89	68	0.0904	0.001168	77.44
2.40	3.16	66	0.0878	0.001171	74.96
2.60	3.42	60	0.0798	0.001174	67.96

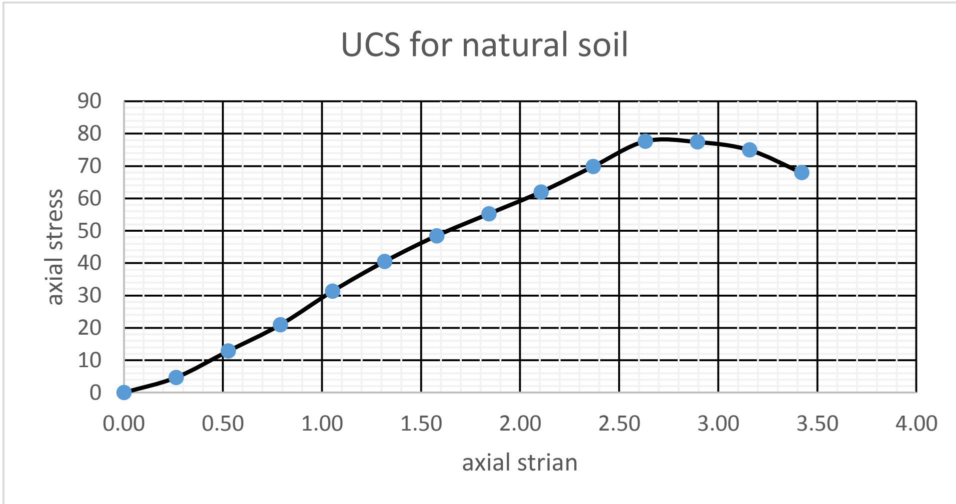

Figure 4. 4 Unconfined Compressive Strength of *Bole-Arabsa* Untreated Soil

4.1.8. California Bearing Ratio

California bearing ratio is the ratio of force per unit area required to penetrate in to a soil mass with a circular plunger at the rate of 1.27mm /min. one point unsoaked CBR value of bole-arabsa untreated CBR value (%) = 8.15

Table 4. 6 Unsoaked CBR Test Values

Blows/ Layer	56/5	Ring calibration (N/Div.)	= 25.8		
CBR Value, %	**8.15**	Plunger area (mm^2)	1963.45		
Penetration (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.0	0	0.00		
0.64	31.2	805	0.41		
1.27	37.5	968	0.49		
1.91	40.5	1045	0.53		
2.54	42.8	1104	0.56	6.9	8.15
3.18	44.5	1148	0.58		
3.81	45.6	1176	0.60		
4.40	47.2	1218	0.62		
5.08	48.6	1254	0.64	10.3	6.20
7.62	53.2	1373	0.70		
10.16	67.0	1729	0.88		
12.70	71.0	1832	0.93		

4.2. Effect of lime on soil PH

The PH value increases rapidly with increasing lime content. The minimum quantity of lime required for subgrade stabilization is determined by Initial Consumption of Lime (ICL) and recommended in ASTM D 6276 to be a minimum PH of 12.4 for lime soil mixture, in our case 6% lime content gave that result. But, due to high crystalized lime content of marble waste powder the lime content was fixed to be 3% using compaction trail tests for all treated soil sample tests. The effect of lime on soil are shown in the figure 4.5.

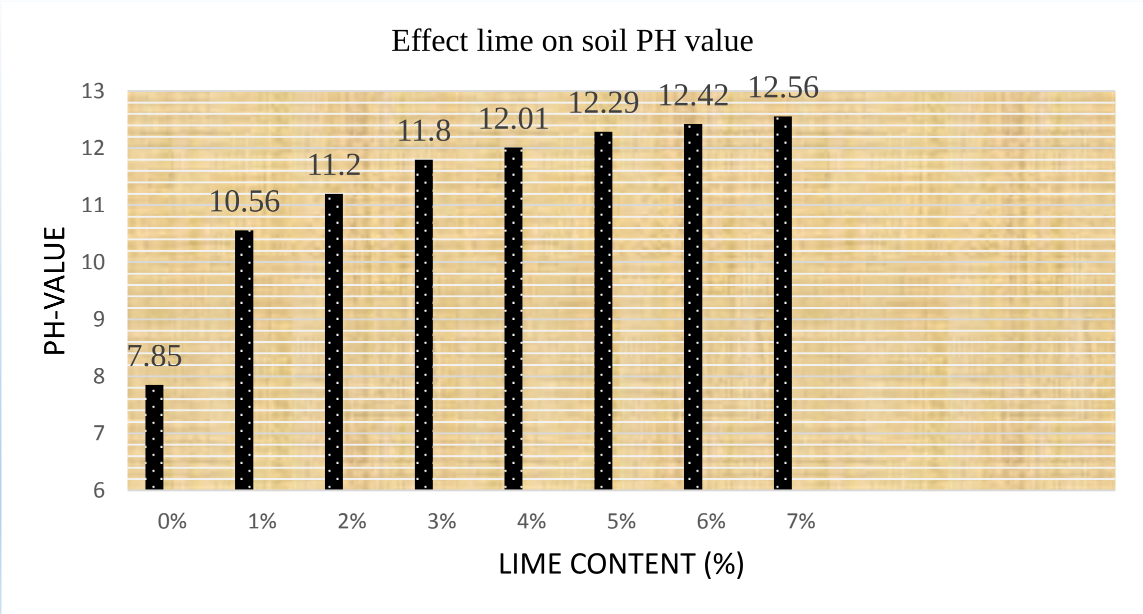

Figure 4. 5 The Effect of Lime on PH Value of a Soil

4.3. Effect of stabilizer on Atterberg limit test

A significant decrease in the value of liquid limit was observed .there was a nominal increase in value of plastic limit with the increase of marble waste powder. The increase in plastic limit and reduction in liquid limit caused a net decrease in plasticity index. The liquid limit decreased from 112.8% to 71.6% while the plastic limit shows an increased from 34.7% to 44.4%, resulting in reduction of plasticity index from 78.1% to 27.1% for 5-25% of marble waste powder with fixed 3% lime powder. Chen (1988) explained that the flocculation of the clay particles result in reduction of plasticity index. Stabilizer effect on liquid limit, plastic limit and plasticity index are shown in table4.7.

Table 4. 7 Effect of Stabilizer on Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI) For Local Expansive Subgrade Soil.

stabilizer	LL	PL	PI	(%) Change LL	(%) Change PL	(%) Change PI
SML-0	112.8	34.7	78.1			
SML-5	97.8	37.4	60.4	-12.67	7.78	-22.66
SML-10	87.4	39.0	48.4	-22.87	12.39	-38.02
SML-15	85.8	39.6	46.2	-23.94	14.12	-40.84
SML-20	78.7	41.1	37.6	-30.23	18.44	-51.85
SML-25	71.6	44.4	27.1	-36.79	27.95	-65.3

4.4. Effect of stabilizers on moisture density relationship characteristics

The characteristics Maximum Dry Densities (MDD) and Optimum Moisture Contents (OMC) of the different percentage of stabilizers on soil is shown in table 4.8.

Table 4. 8 Summary of Compaction Test Results

Sample Id	Maximum dry density MDD	Optimum moisture content (OMC)
SML-0	1.21	40.00
SML-5	1.27	38.87
SML-10	1.29	37.50
SML-15	1.33	33.80
SML-20	1.37	32.00
SML-25	1.43	27.57
SML-30	1.46	16.09 = Existing moisture Cont.

The maximum dry density (MDD) increased as the amount of marble waste powder increased with fixed 3% lime powder mix. The optimum moisture content decreased for soil sample mixed from 5% to 25% marble waste powder with fixed 3% lime of mixture.

But, further increment of marble waste powder addition to the sample made the compaction curve uncommon which had no definite peak to determine MDD and OMC as shown in figure 4.9.

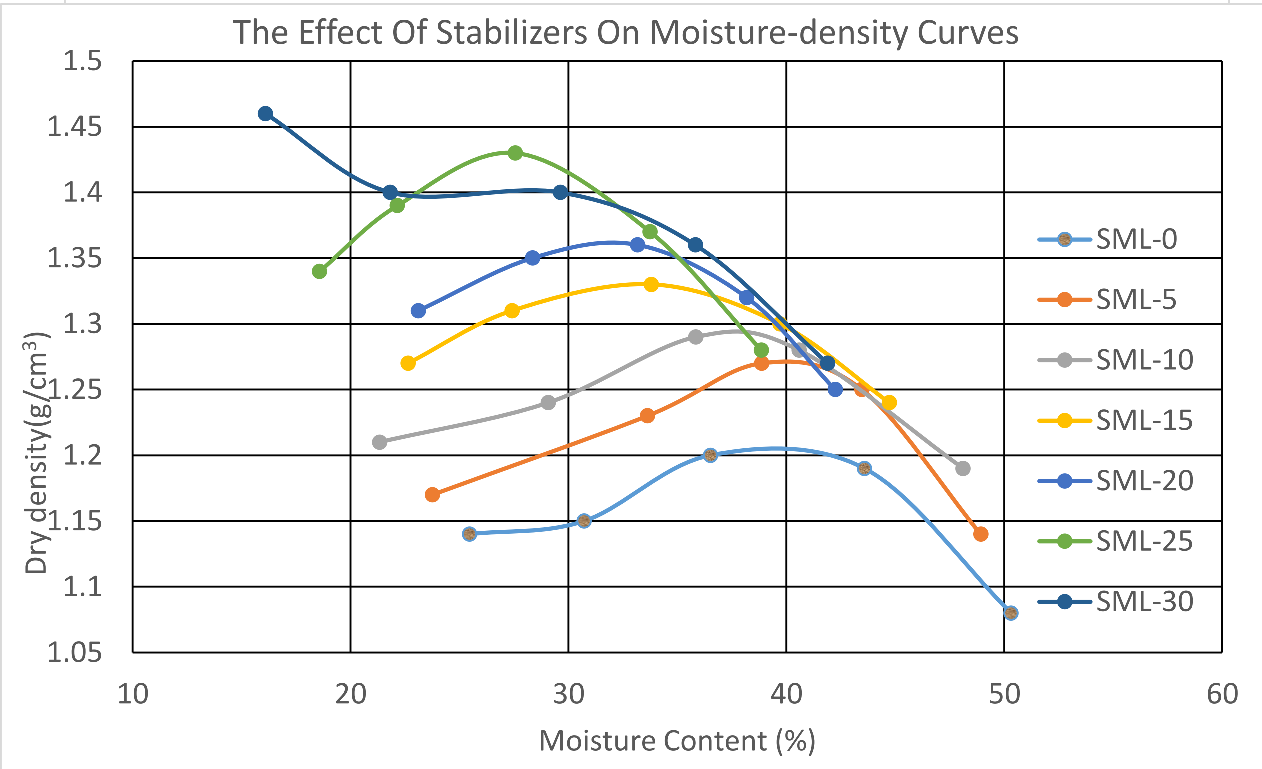

Figure 4. 6 The Effect of Stabilizers on Moisture Density Curve

4.5. Effect of stabilizers on unconfined compressive strength (UCS) of soil

The determined unconfined compressive strength values are increased with the increment of stabilizers for soil sample mixed with 5% - 20% marble waste powder with fixed 3% lime powder. Further addition of marble waste powder with fixed 3% lime showed decrement in value of UCS as shown in figure 4.7

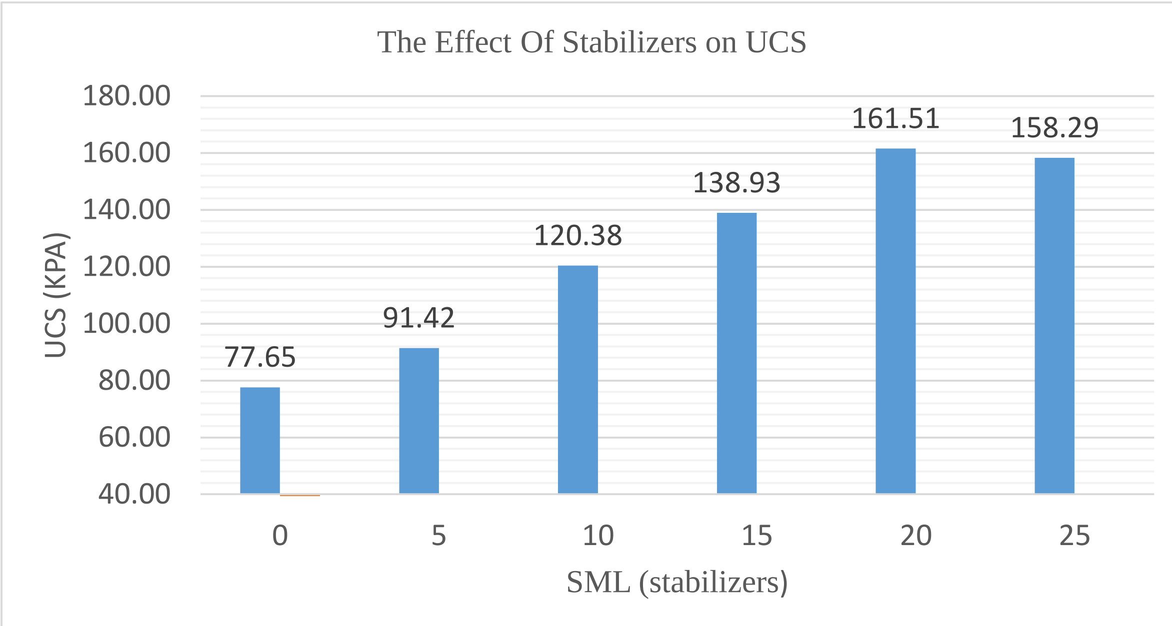

Figure 4. 7 Effect of Stabilizer on UCS

4.6. The effect of stabilizers on 24hour free swell index (FSI)

The 24 hour free swell index decreased with increased value of marble waste powder for soil sample mixed 5 to 20% MWP with fixed 3% lime powder. At SML-25 (25% MWP with 3% lime) the FSI showed increment.

Table 4. 9 Summary of 24 Hour FSI Test

Sample ID	SML-0	SML-5	SML-10	SML-15	SML-20	SML-25
Initial volume(ml)	10	10	10	10	10	10
Final volume(ml)	19.5	19	17	16	15	16
% Swell	95	90	70	60	50	60

4.7. Effect of stabilizers on California Bearing Ratio (CBR)

One-point CBR tests were done according to outlined AASHTO T193, ASTM D1883 and referenced from British standard (BS 1377: Part 5:1990). The CBR tests samples were prepared with optimum moisture and maximum dry density using standard mold with an internal diameter of 152.4 mm and a height of 177.8 mm then specimens were compacted in 5 layers with 56 blows per layer. Two samples are usually prepared for CBR tests; one is tested directly after sample preparation and the other after soaking in water for 96 hours. The unsoaked CBR values increased with increment of marble waste powder mixed with 3% lime powder from SML-5 to SLM-20. Further increment of marble waste powder decreased the CBR value. Poor subgrade strength (0.65% soaked CBR vale) natural soil 96 hour soaked samples increment of stabilizer from SML-5 to SML-10 showed effective increment on CBR values and when stabilizers further increased the CBR values showed decrements. The effect of stabilizers on CBR values shown in the table 4.10 below.

The CBR swell rapidly decreased for SML-5 and SML-10 samples. However, further increment of stabilizer increase the value of the CBR swell.

Table 4. 10 Summery Soaked and Unsoaked CBR Laboratory Test Values

Stabilizers (%)	CBR values (%)		CBR swell (%)	% reduction in swell
	unSoaked sample	Soaked sample		
SML-0	8.15	0.65	7.32	-
SML-5	9.81	2.38	1.98	72.95
SML-10	14.47	4.19	1.49	79.65
SML-15	15.52	3.31	2.98	59.28
SML-20	20.8	3.14	3.04	58.47
SML-25	18.5	3.12	14.99	-104.78

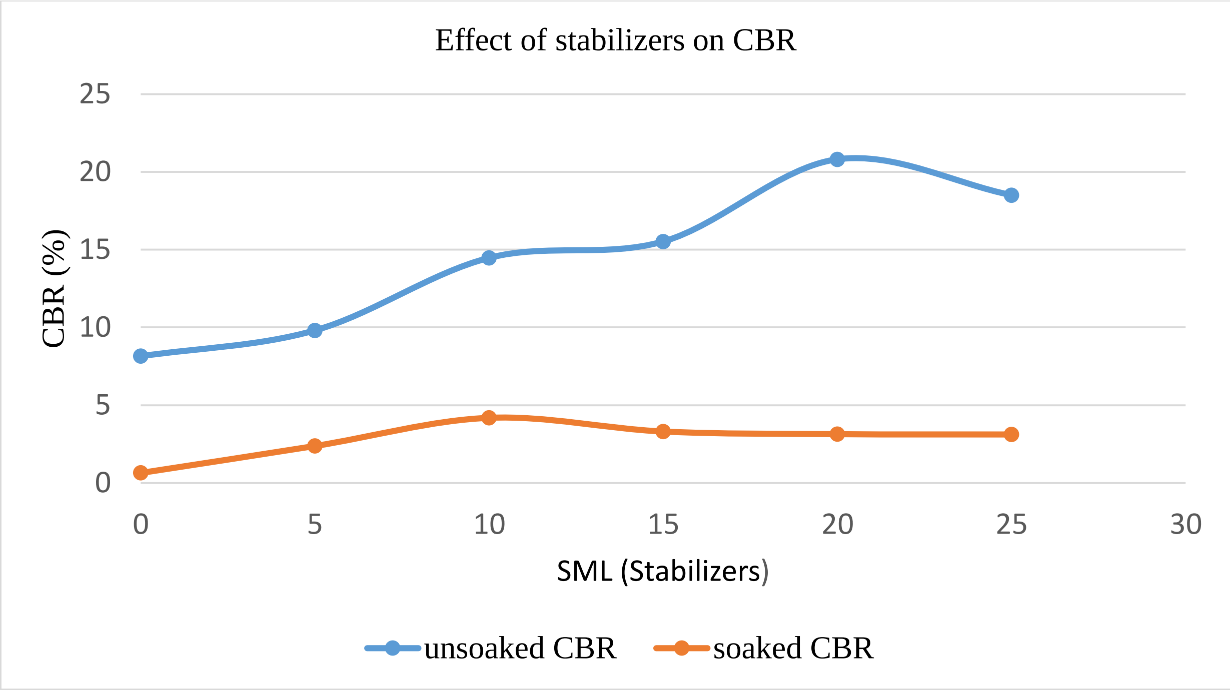

Figure 4. 8 Summery of Effect of Stabilizers on CBR

4.8. Effect of stabilizers on swelling pressure test

One dimensional consolidation test was performed according to outlined in ASTM D-2435. Swell pressure test samples was prepared using optimum moisture content determined from standard Proctor's test. 50 mm diameter and 18 mm diameter samples were used. In this test the samples were remolded using consolidation ring height of 20mm. the remolded sample with consolidometer ring placed on the consolidometer and subjected to under applied load of 7 kPa is wetted and allowed to fully swell. After sample was fully swelled a standard consolidation test was conducted by applying incremental loads starting with 50 kPa and ending with 1600kPa. The pressure required to revert the specimen to its initial void ratio (height) is used to define the swelling pressure. The swelling pressure tests were conducted on the treated and untreated expansive soil samples. The summaries of swelling pressure and swelling potential test results of untreated soil sample have been given in the table 4.11.

Table 4. 11 Swelling Test Result for Untreated Soil

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height, H_v (mm)	Void Ratio, E
Loading					
7	12.801	0.00	18.00	8.67	0.93
7	14.849	2.05	20.05	10.72	1.15
50	14.725	1.92	19.92	10.60	1.14
100	14.569	1.77	19.77	10.44	1.12
200	14.319	1.52	19.52	10.19	1.09
400	13.810	1.01	19.01	9.68	1.04
800	13.180	0.38	18.38	9.05	0.97
1600	12.010	-0.79	17.21	7.88	0.85
Unloading					
1600	12.010	-0.79	17.21	7.88	0.85
400	12.281	-0.52	17.48	8.15	0.87
100	12.520	-0.28	17.72	8.39	0.90
7	13.409	0.61	18.61	9.28	1.00

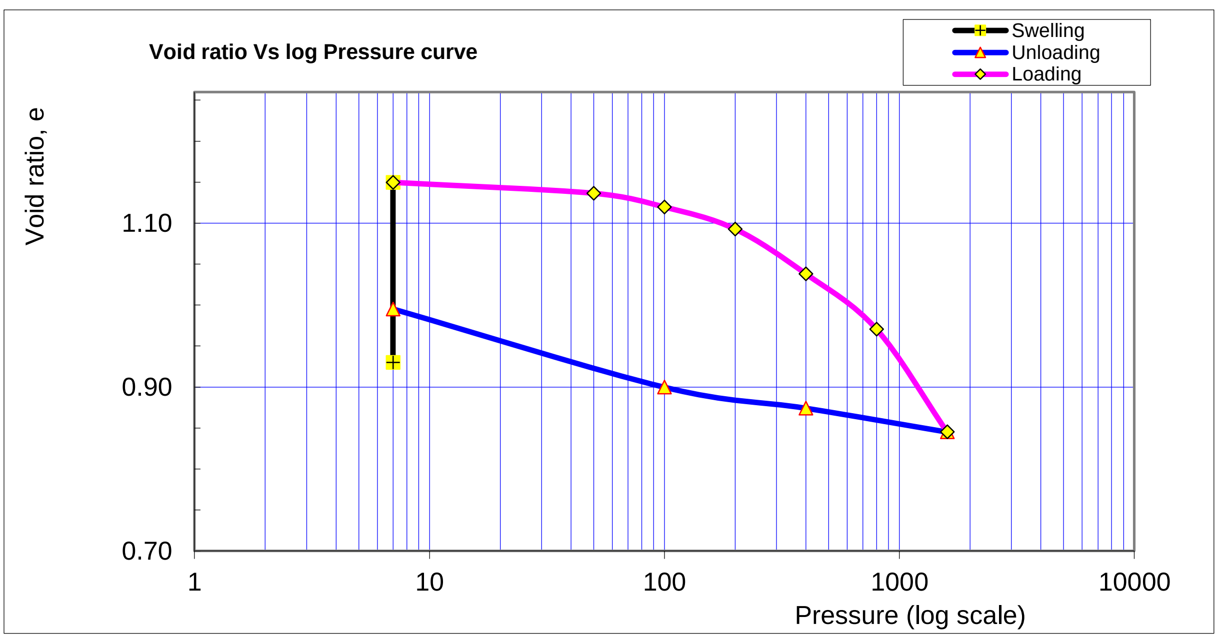

Figure 4. 9 Void Ratio Vs Log-Pressure Graph of Untreated *Bole-Arabsa* Soil

Swell pressure with the addition stabilizers of expansive soil samples was presented in table 4.12. As the percent of stabilizers increased the swelling potential decreased. Soil sample had swelling potential of 11.39% and gradually reduced to 4.67% and 2.95 with the addition of stabilizers and the untreated soil sample had 1000kpa laboratory swelling pressure and decreased to 750kpa at stabilizer SML-5 and 440kpa at stabilizer SML-10. However, it has been observed that further increment of stabilizers results increased in swelling pressure and swelling potential indicating that, 10% marble waste powder with 3% lime powder is the optimum mix for stabilizing A-7-6[20] soil stabilization for swelling pressure and swelling potential properties of the soil.

Table 4. 12 Summary of the Effect of Stabilizers on Swelling Potential

Sample id	Swelling potential (%)	Swelling pressure (kpa)	% decrease swelling potential	% decrease in swelling pressure
SML-0	11.39	1000	--------	---------
SML-5	4.67	750	58.99	25
SML-10	2.94	440	74.18	56
SML-15	3.44	600	69.79	40
SML-20	-	-	-	-
SML-25	-	-	-	-

5. CONCLUSIONS

The expansive soil taken from ***Bole-Arabsa*** was inorganic clay with high plasticity. It is very low strength, 0.65% CBR (96-hour soaked) value which is unsuitable for pavement foundation usage. The soil was stabilized with marble waste powder with lime and the following conclusions can be drawn based on different test results and investigations of this study.

i. Index properties: the stabilizers decreased liquid limit from 113% to 71.6%, plasticity index from78.1% to 27.15 and increased liquid limit from 34.7% to 44.4%.

ii. Twenty four hour free swell index: stabilizers decreased FSI up to SML-20 from 95% to 50% and further increased marble waste powder with fixed 3% lime results increment in FSI value.

iii. Compaction**:** the addition of stabilizers increased in MDD with increment of stabilizers.

iv. CBR and Swelling pressure: showed positive effect on subgrade soil from SML-0 to SML-10, CBR increased six times than natural soil from 0.65% to 4.19% and swelling pressure decreased from 1000kpa to 440kpa. But further increment of stabilizer showed negative effect on the property of subgrade soil.

v. Based on soaked CBR value, CBR swell, swelling pressure and swelling potential test values the optimum mix is 87% soil + 10% marble waste powder + 3% lime.

vi. Finally, marble waste powder is used as a stabilizer for local expansive subgrade soil of A -7-6[20] which is improved.

6. RECOMMENDATION

10% marble waste powder with 3% lime is the optimal stabilization of the A-7-6(20) expansive subgrade soil. With this, the material is suitable for use as sub-grade material for road construction. Further studies should be conducted to determine the effect of adding marble waste powder with lime on some other expansive soils.

- The mineralogy of waste marble powder and lime was not investigated in terms of suitability in this study. It may be helpful to study mineralogical suitability of marble waste powder and lime.
- The environmental impacts of the additive application were not taken into account in the study, Further studies of this aspect may be important.
- The results of this study were obtained based on immediate compaction after treating the soil. The effect of delay between mixing and compaction of the soils should be investigated.
- All the test result obtained in this study were for non-cured samples. It may be useful to study the effect of curing of marble waste powder with lime stabilized soil sample.
- Finally, the finding of this study should be considered as only indicative, as small soil sample, limited tests, and limited parameters were implemented.

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APPENDIX

Laboratory Test Results

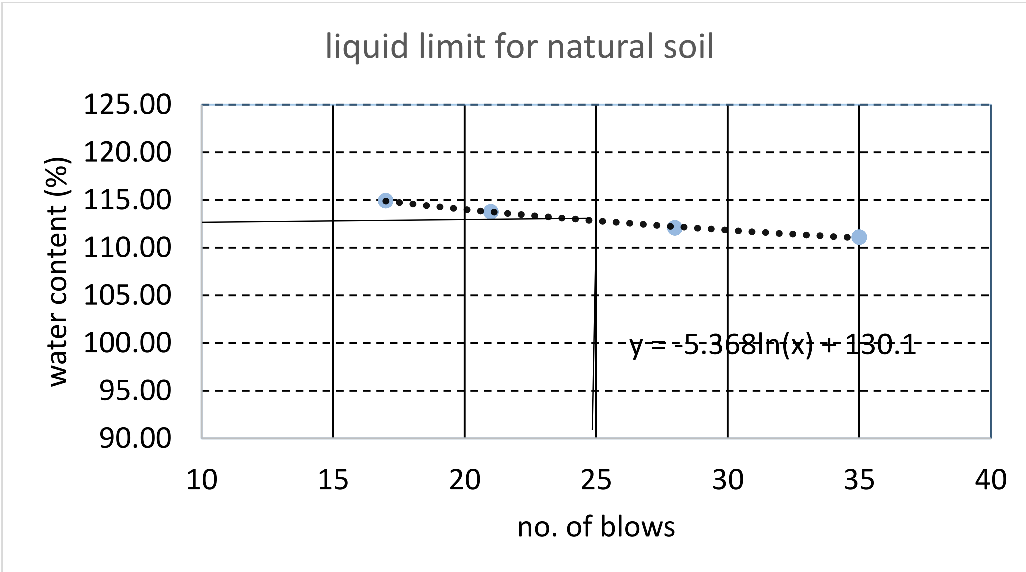

Figure 1 Liquid limit for natural soil.

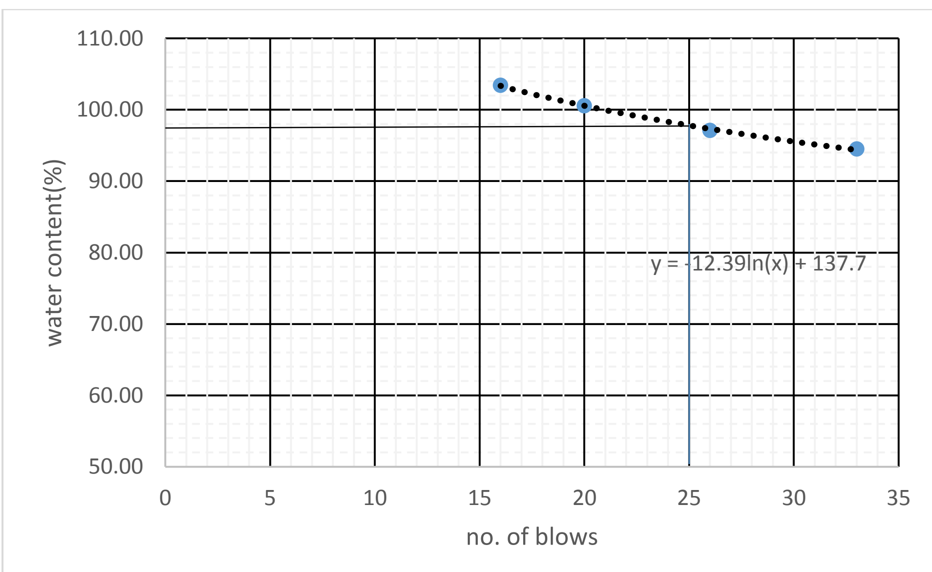

Figure 1 Liquid limit for 5% marble waste powder +3% lime + 92% soil

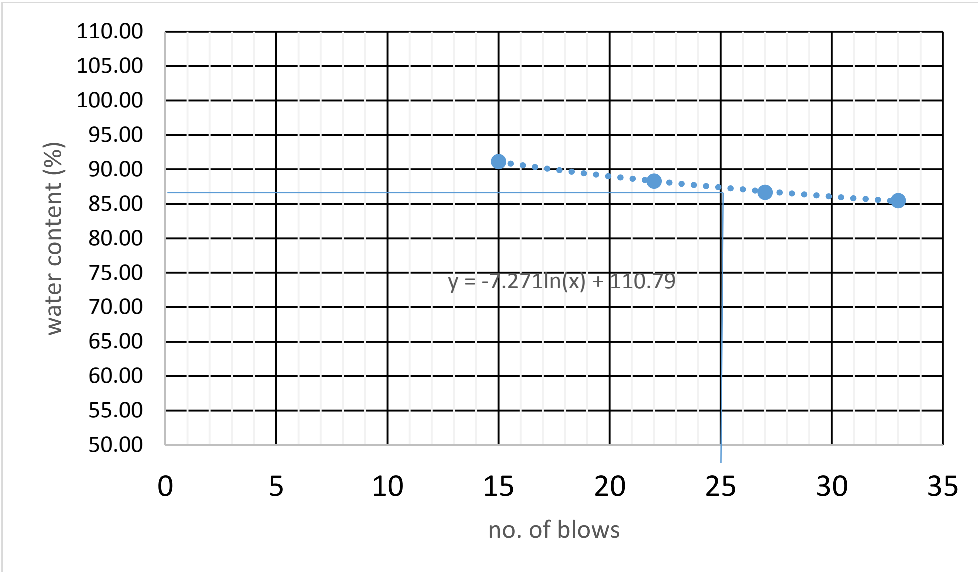

Figure 2 Liquid limit for 10% marble waste powder +3% lime + 87% soil

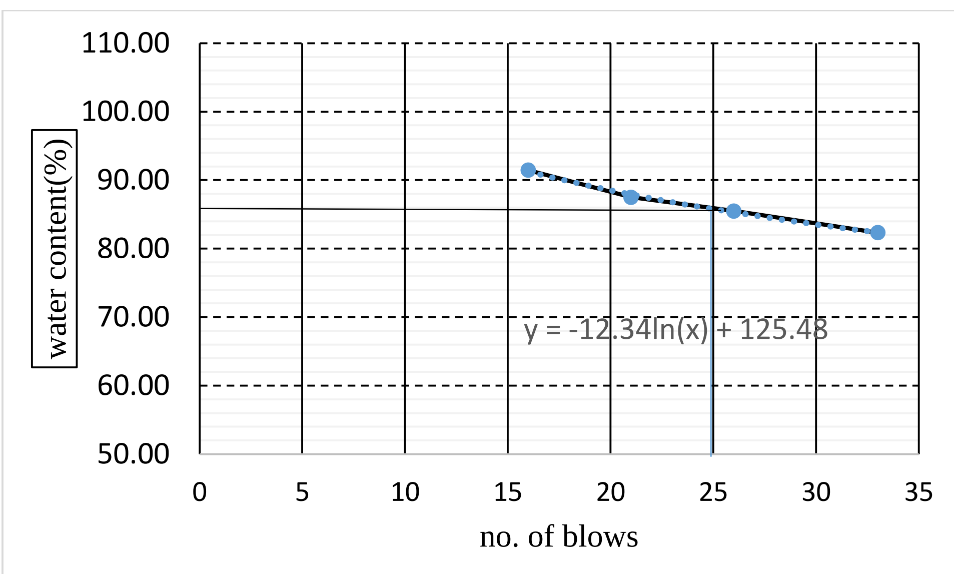

Figure 3 Liquid limit for 15% marble waste powder +3% lime + 82% soil

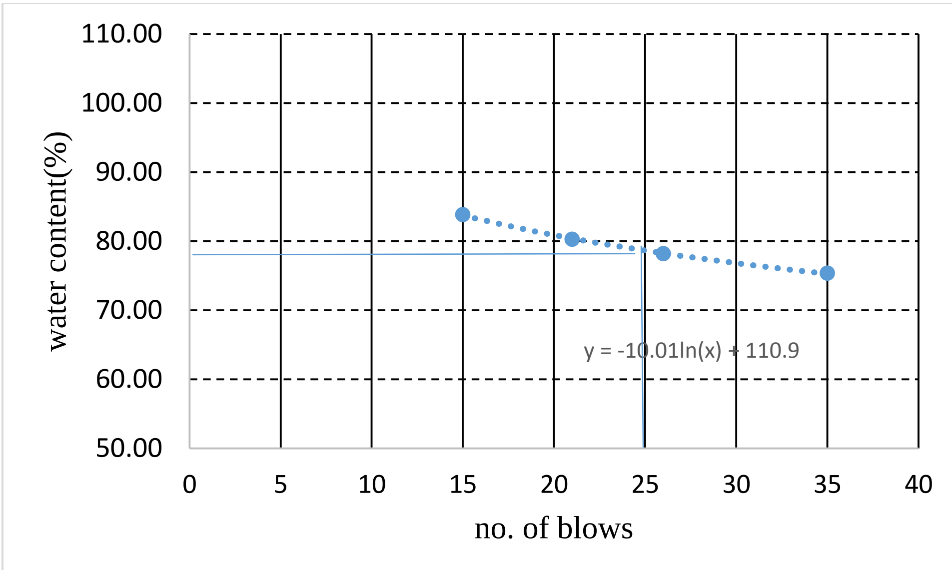

Figure 4 Liquid limit for 20% marble waste powder +3% lime + 77% soil

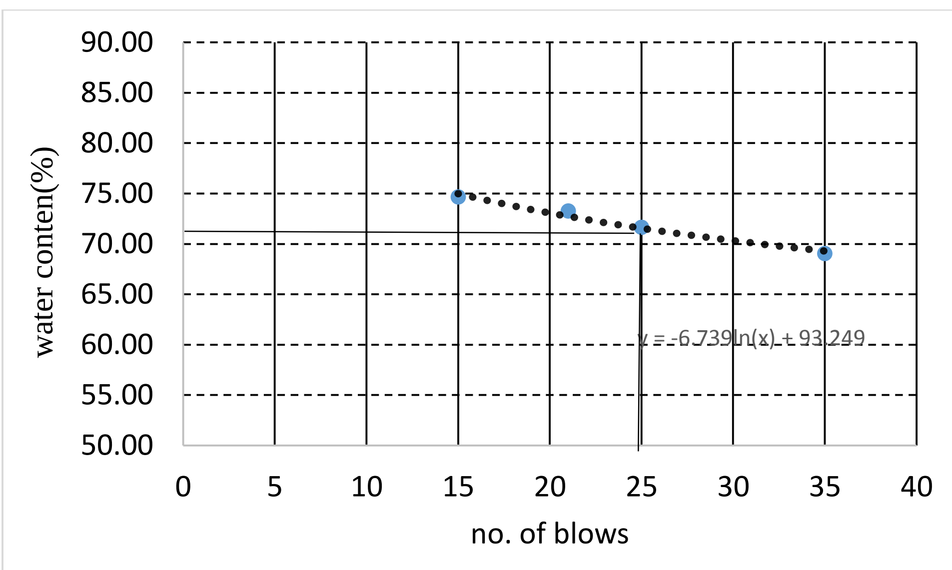

Figure 5   Liquid limit for 25% marble waste powder +3% lime + 72% soil

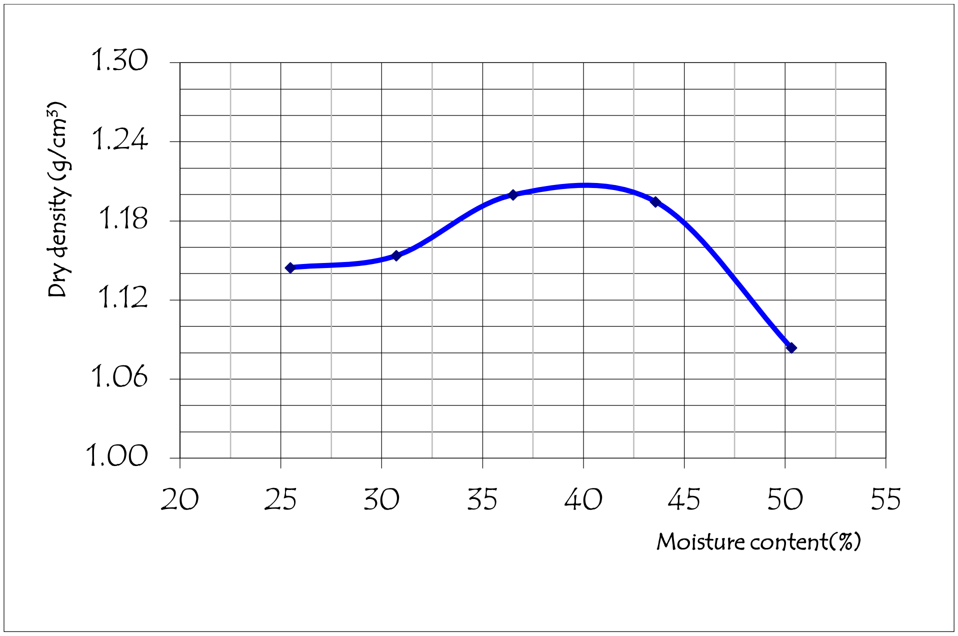

Figure 6 compaction curve for natural soil.

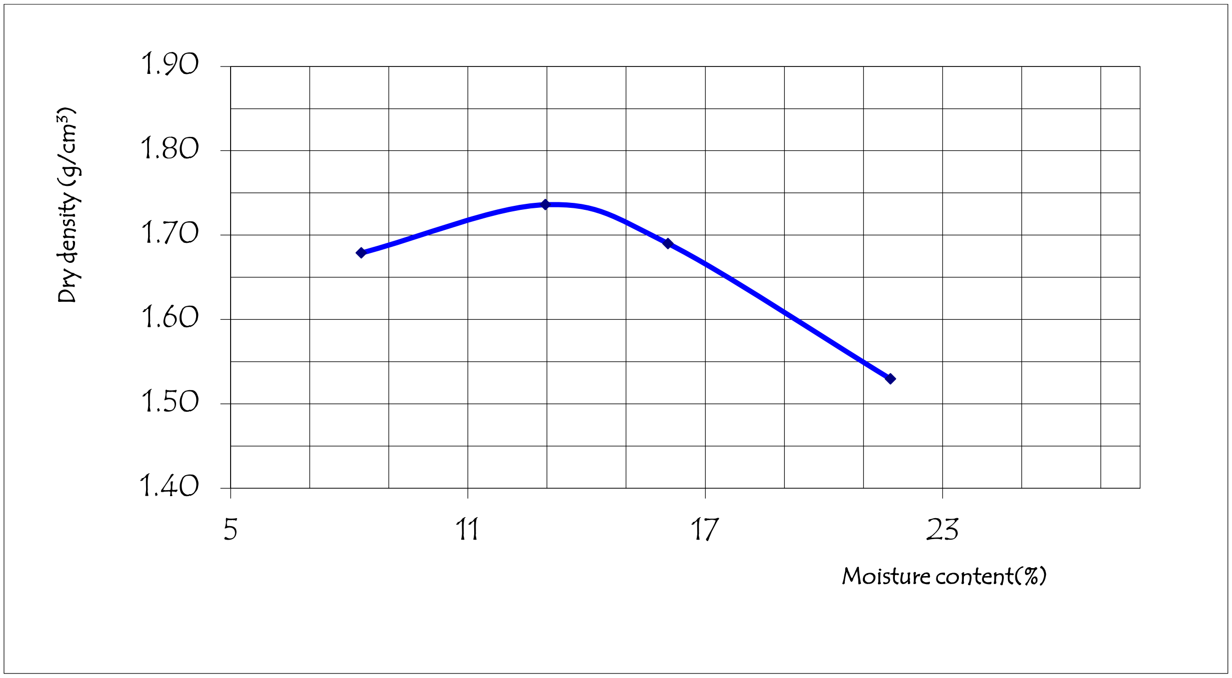

Figure 7 Compaction curve of marble waste powder

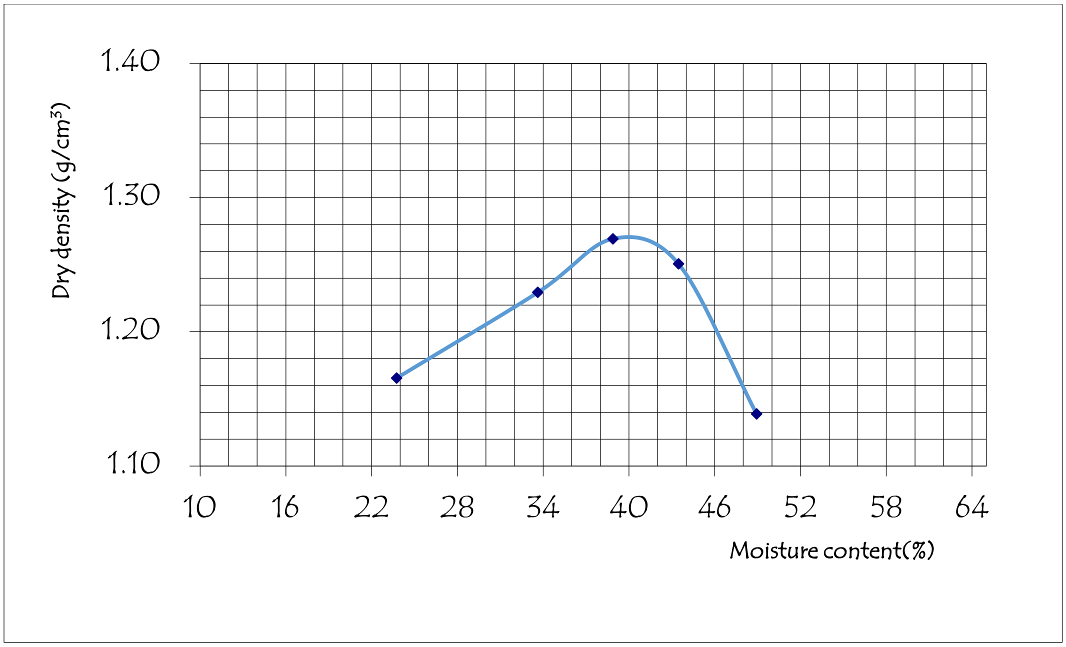

Figure 8 compaction curve for 5% marble waste powder + 3% lime + 92% soil

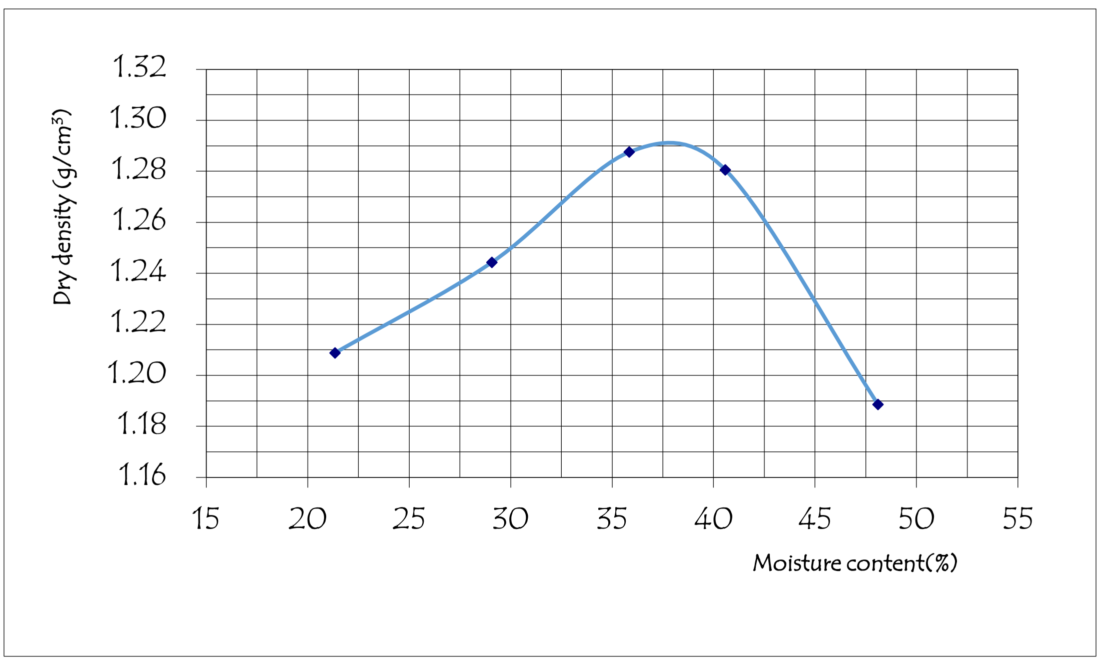

Figure 9 compaction curve for 10% marble waste powder + 3% lime +87% soil

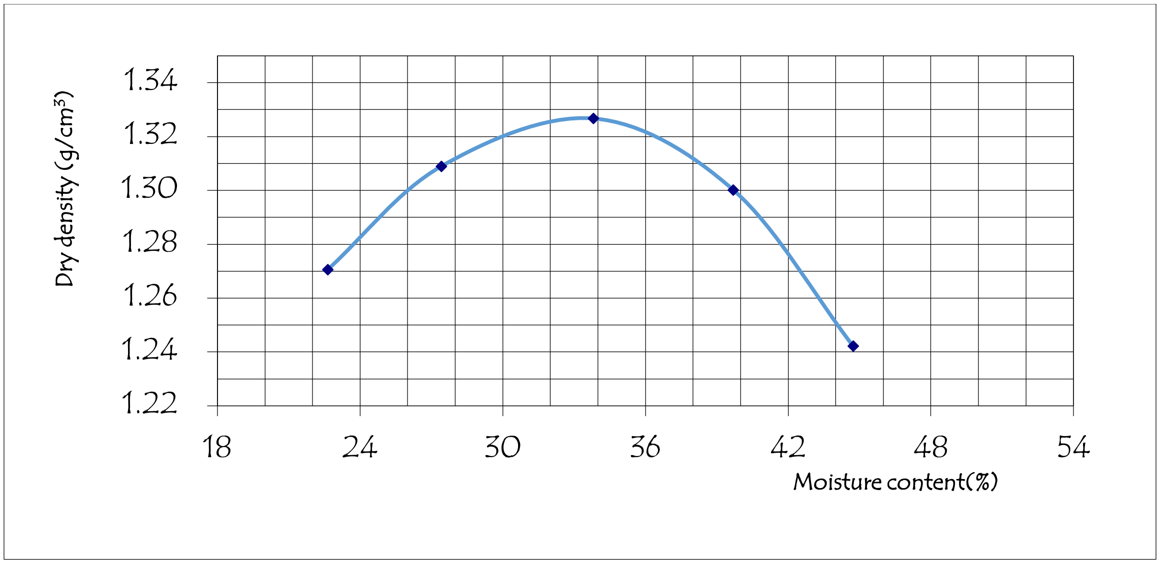

Figure 10 compaction curve for 15% marble waste powder + 3% lime + 82% soil

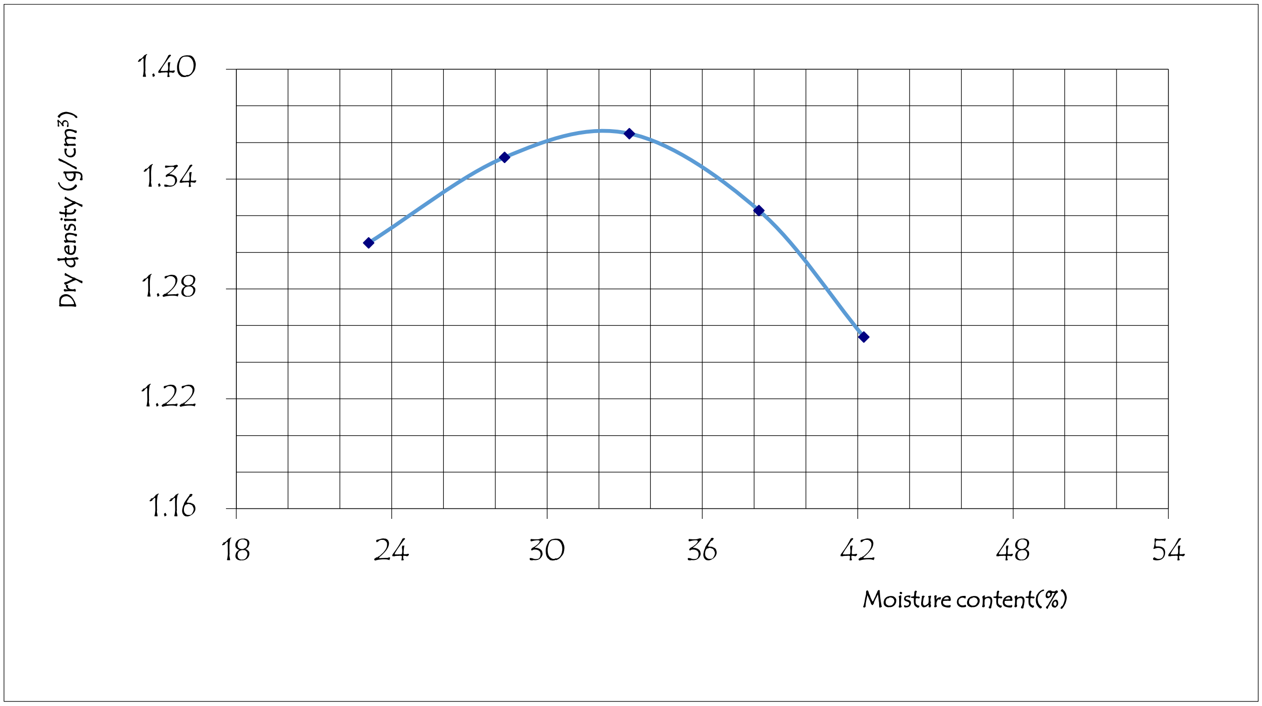

Figure 11 compaction curve for 20% marble waste powder + 3% lime +77% soil

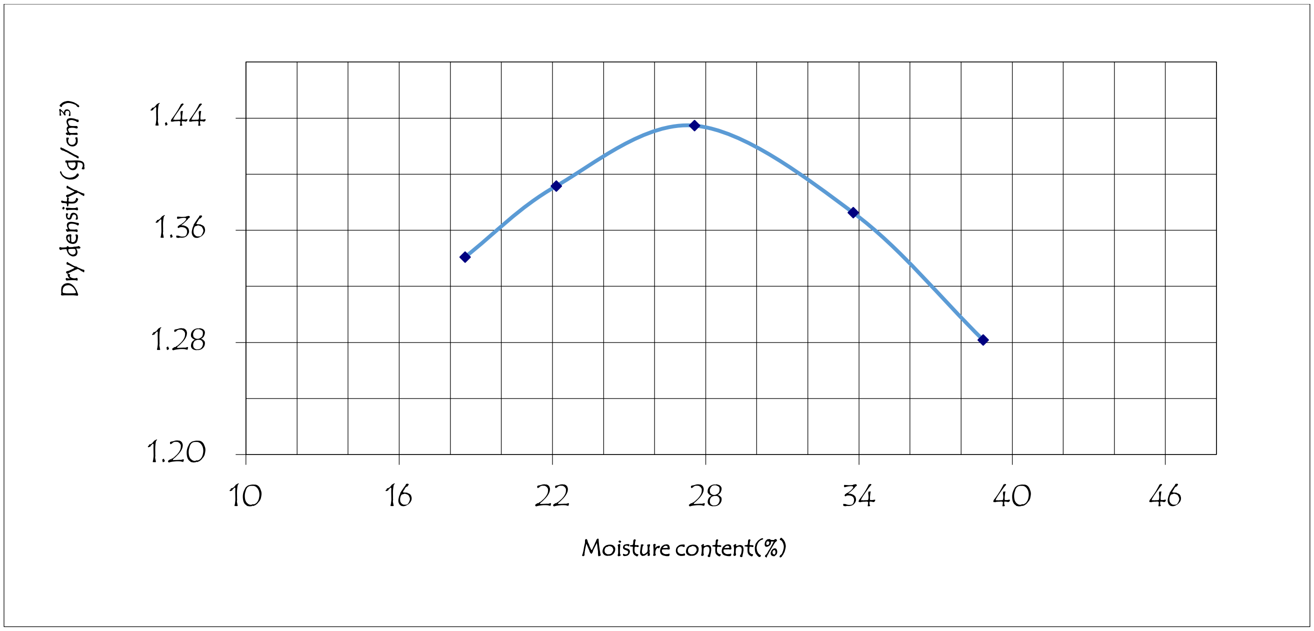

Figure 12 compaction curve for 25% marble waste powder + 3% lime + 72% soil

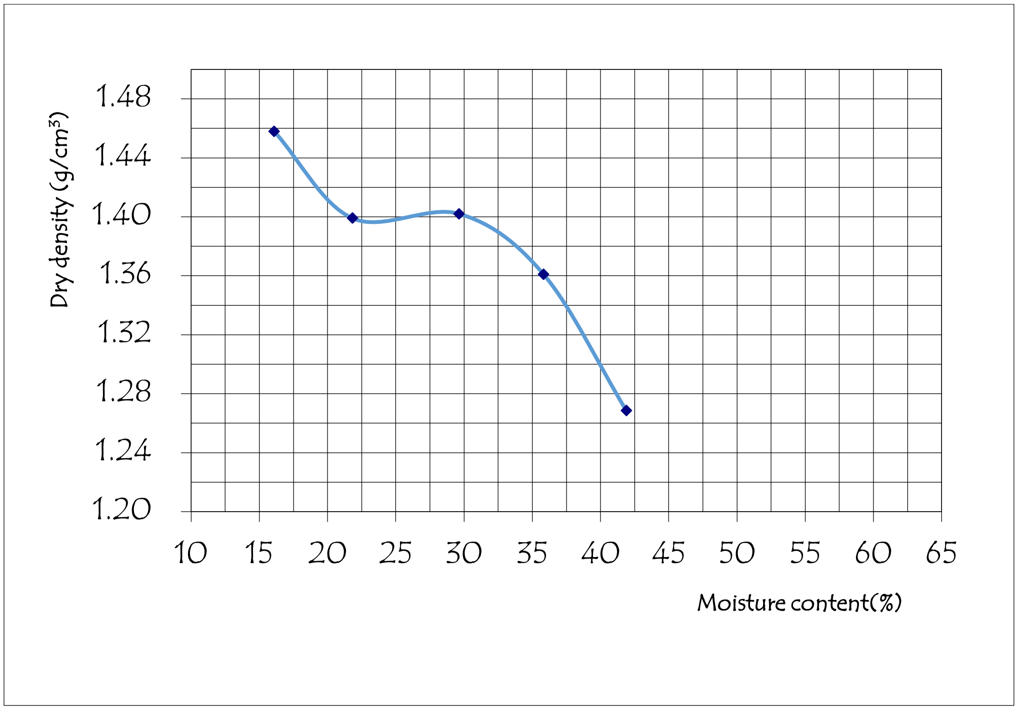

Figure 13 compaction curve for 30% marble waste powder + 3% lime + 67% soil

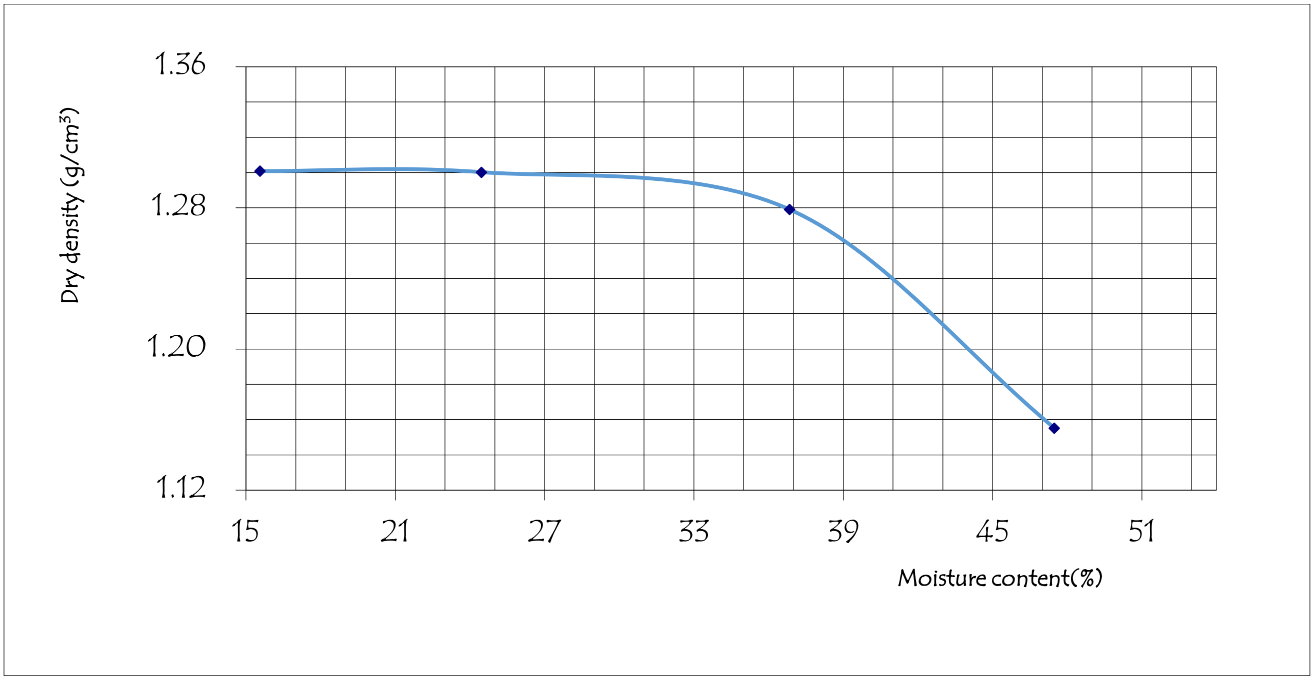

Figure 14 compaction curve of 4% lime + 5% marble waste powder + 91% soil

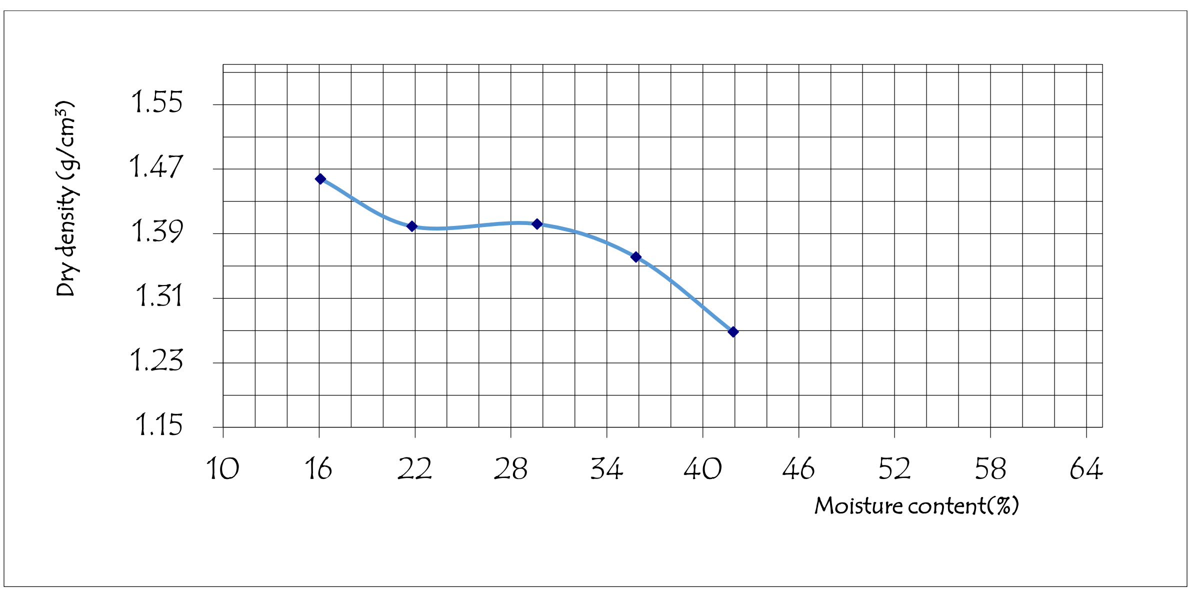

Figure 15 compaction curve of 5% lime + 5% marble waste powder + 90% soil

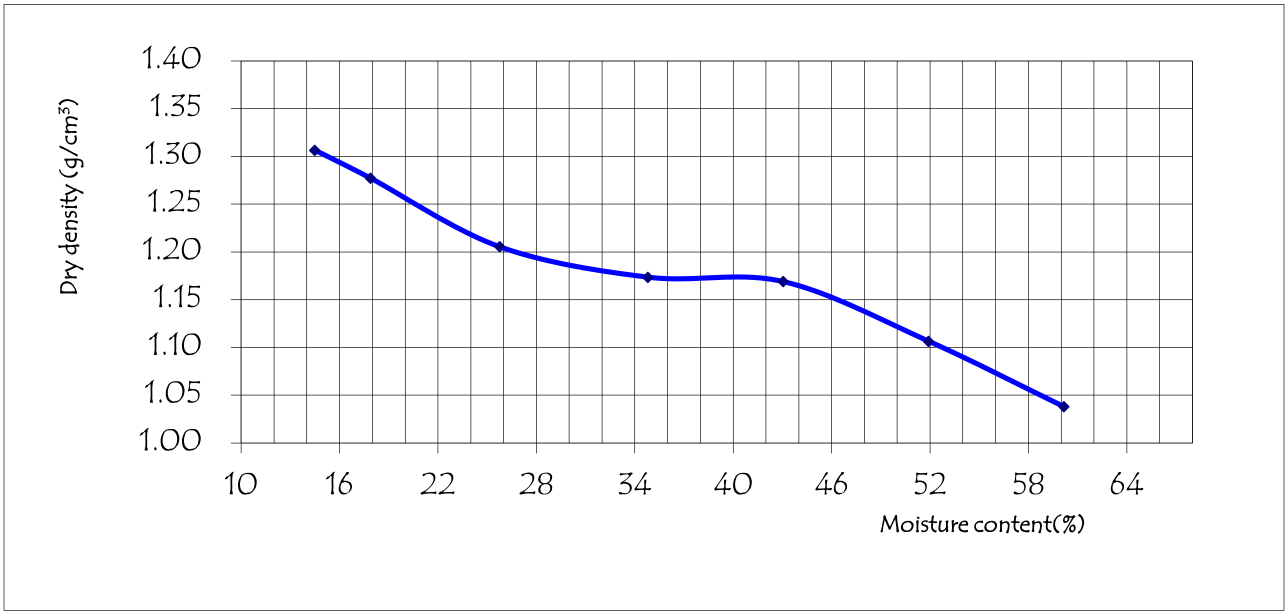

Figure 16 compaction curve of 6% lime + 5% marble waste + 89% soil

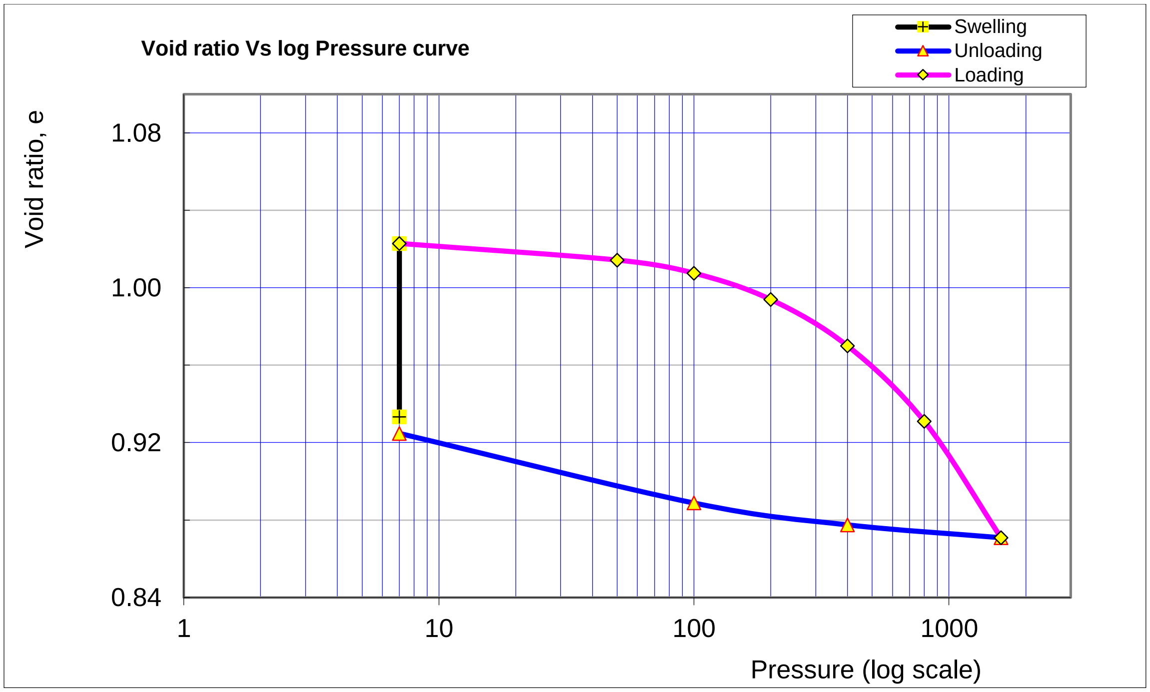

Figure 17 Void ratio versus swelling pressure for SML-5

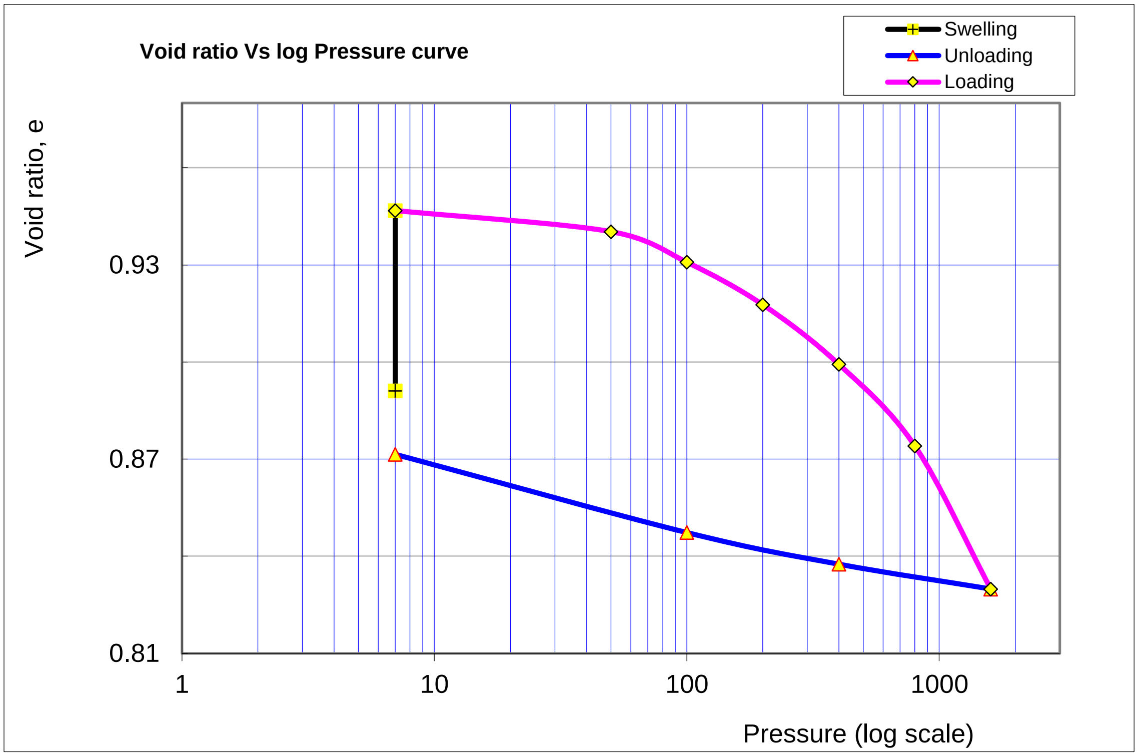

Figure 18 Void ratio versus swelling pressure for SML-10

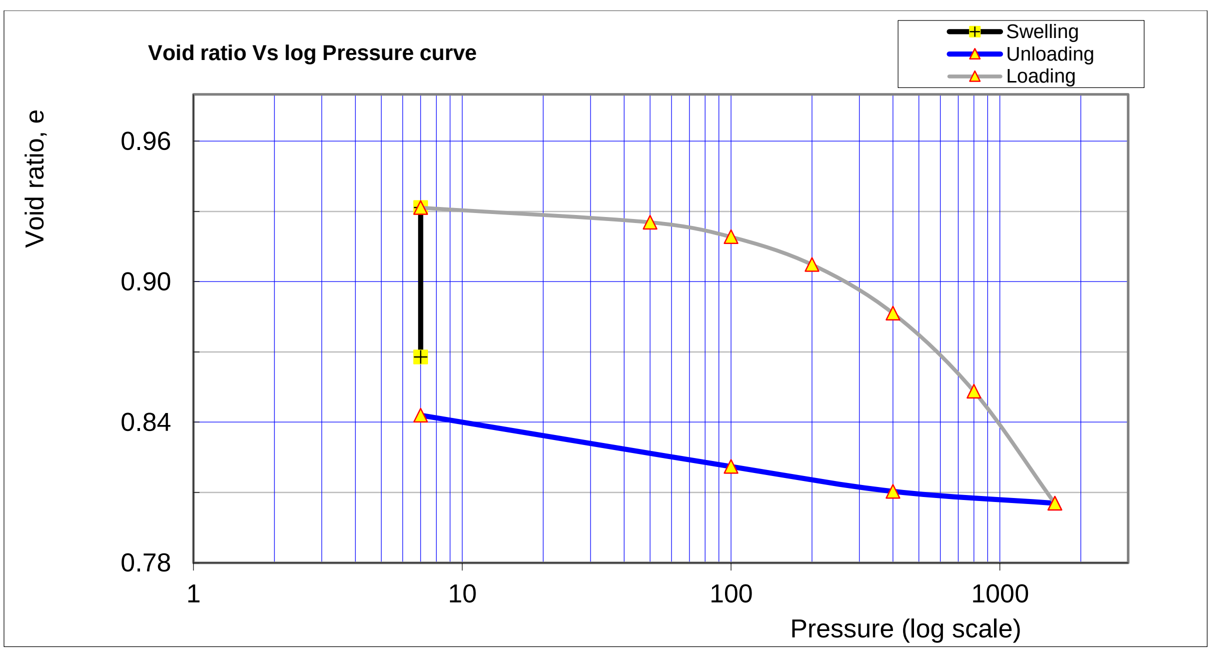

Figure 19 Void ratio versus swelling pressure for SML-15

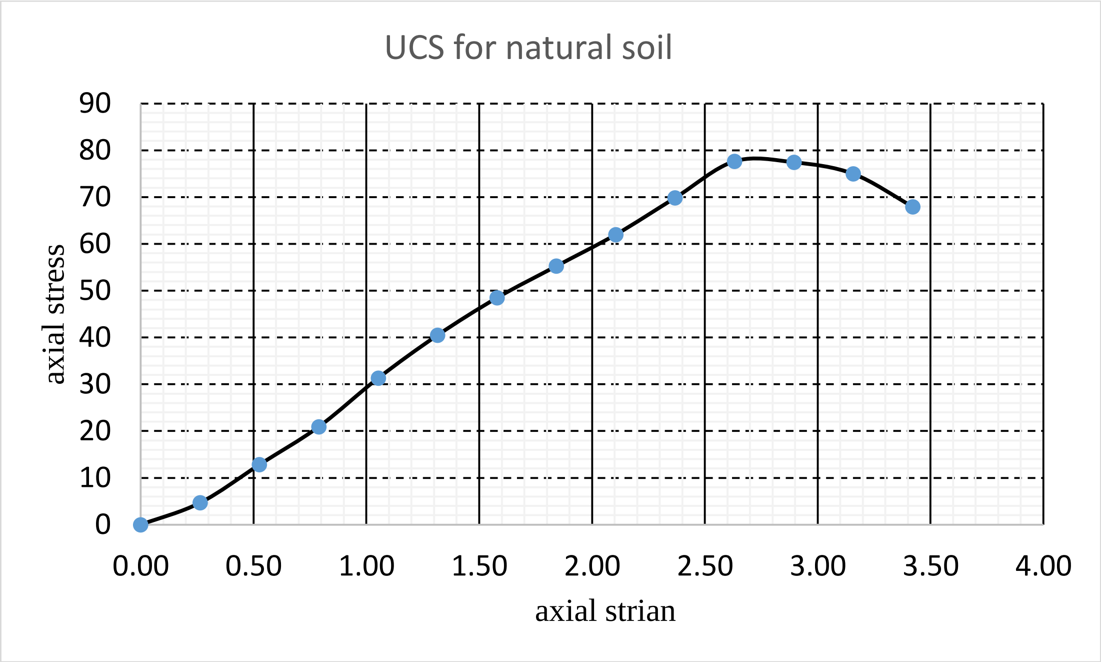

Figure 20 stress strain curve for natural soil

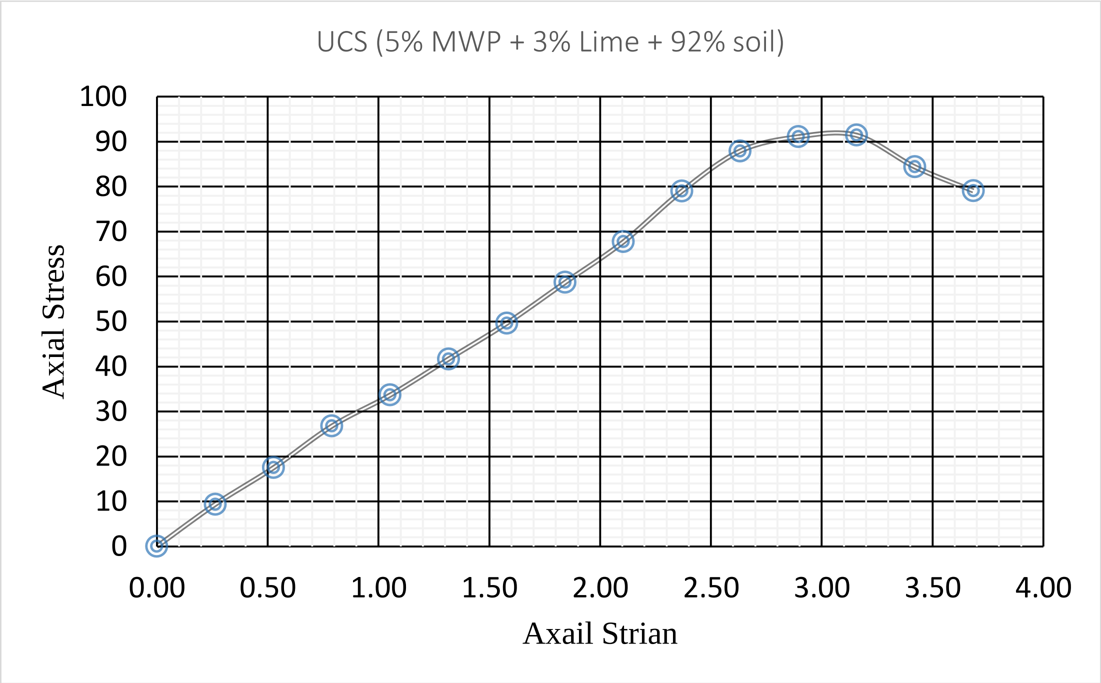

Figure 21 stress strain curve for 5% marble waste powder +3%lime + 92% soil

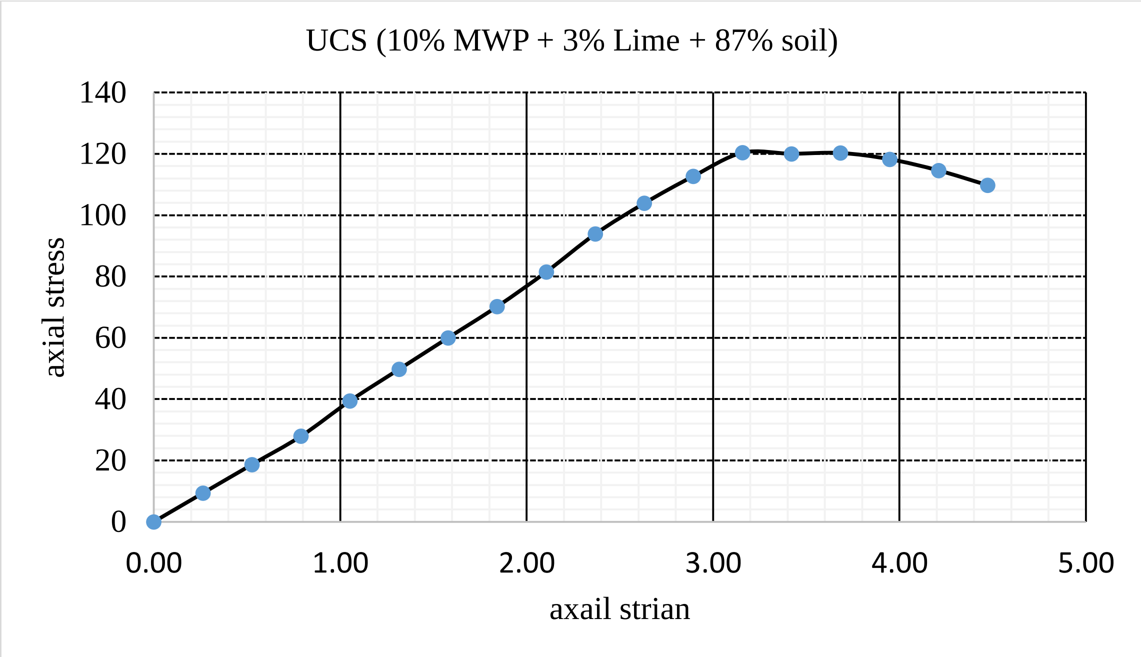

Figure 22 stress strain curve for 10% marble waste powder +3%lime + 87% soil

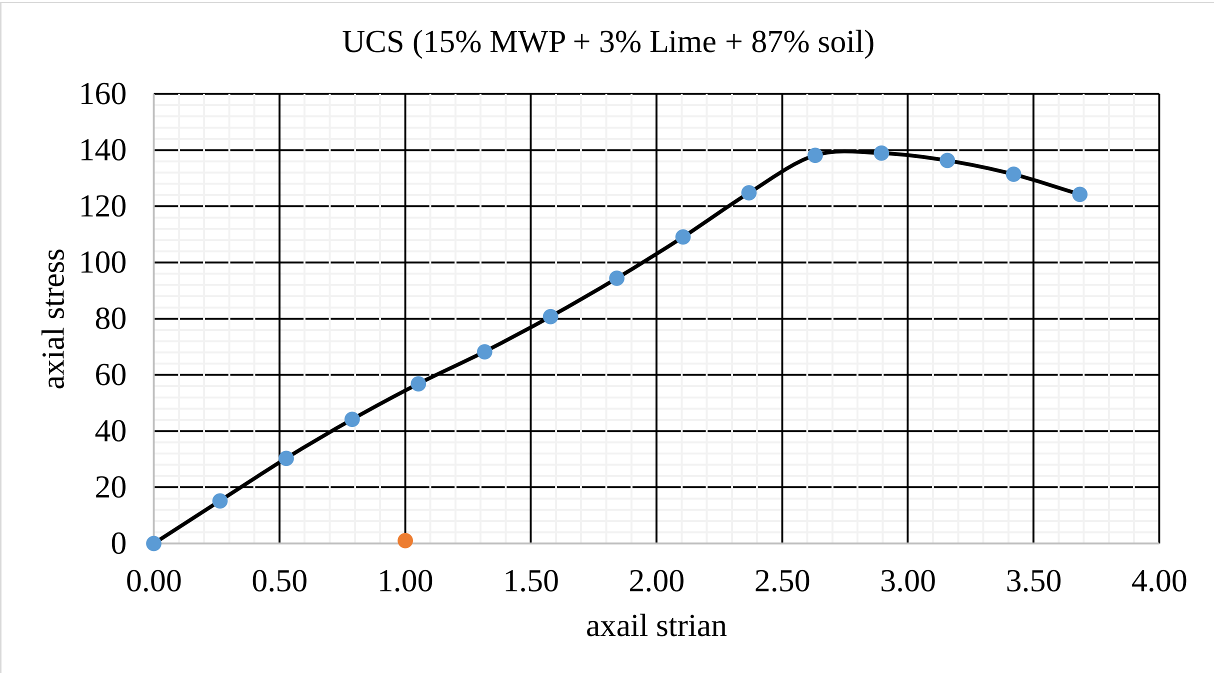

Figure 23 stress strain curve for 15% marble waste powder +3%lime + 82% soil

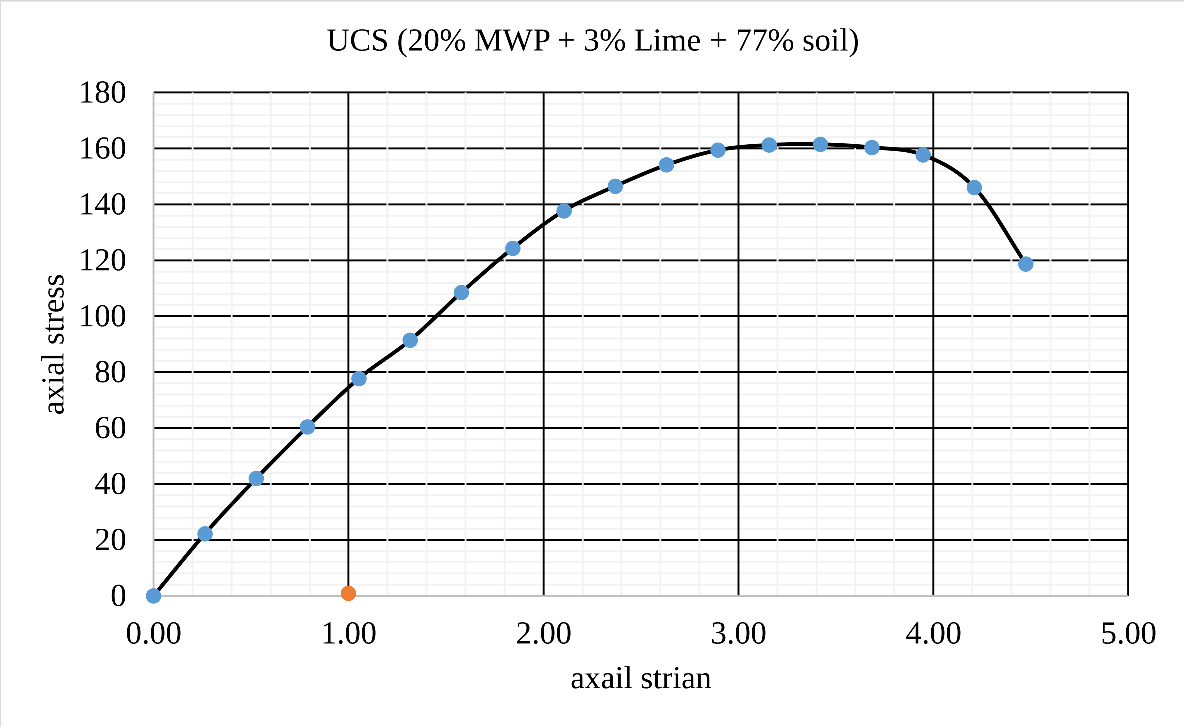

Figure 24 stress strain curve for 20% marble waste powder +3%lime + 77% soil

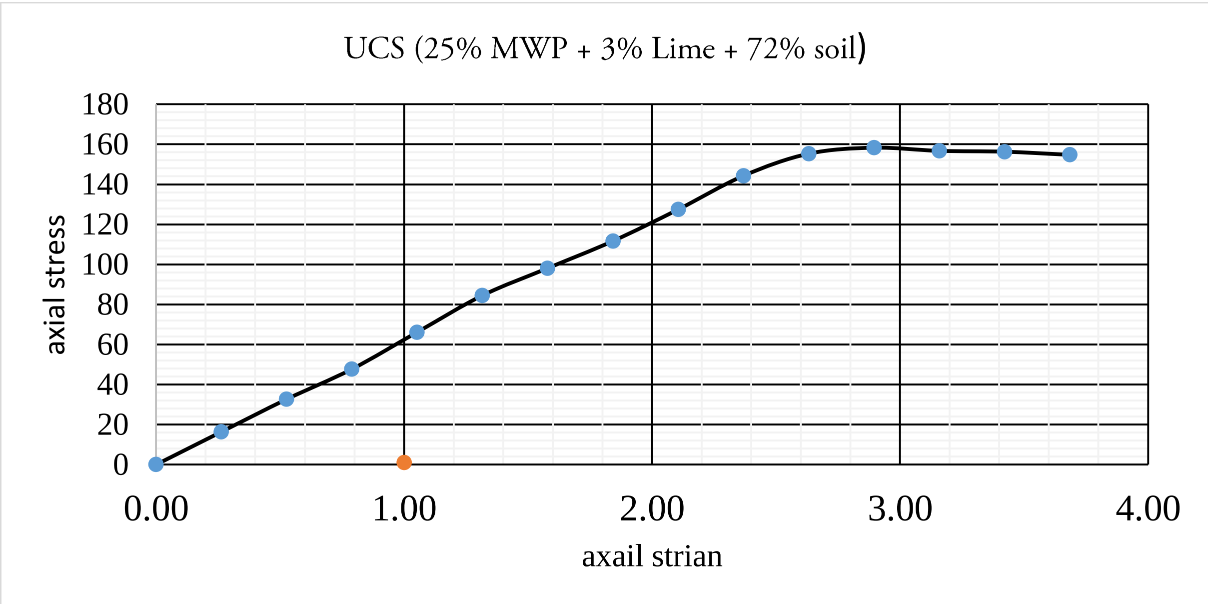

Figure 25 stress strain curve for 25% marble waste powder +3%lime + 72% soil

ADDIS ABABA UNIVERSITY **EIABC** **M.R.T.D** **Date tested: 27/08/2008** **Tested by: Amdegebreal Girmaw** Specific Gravity Determination of marble powder		
Determination No.	1	2
Pycnometer No.	9A	A5
Temperature, T_x(^{o}c)	23	23
Weight of pycnometer + water at T_x , W_{pw} (atT_x) (g)	143.00	144.64
Weight of pycnometer + soil + water, W_{pws} (g)	158.8	160.5
Weight of dry soil , w_s (gm)	25	25
Conversion factor , K	0.9975702	0.9976
Specific gravity of soil at 25.2°c.	2.72	2.74
Average specific gravity of soil G20°C=K · Gt		2.72

Table 1 specific gravity determination for marble waste powder