ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

FACULTY OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

Investigation on Improving the Geotechnical Properties of Black Cotton Soil by Blending with Pumice

A thesis submitted to the school of graduate studies of Addis Ababa University
in partial fulfillment of the requirements for the Degree of
Master of Science in Civil Engineering

By
Desta Berhane

Advisor:
Dr. – Ing. Henok Fikre

June 2018

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Declaration

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor Dr. -Ing. Henok Fikre and has not been presented as a thesis for a degree in any other university. All sources of materials used for this thesis have also been duly acknowledged.

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ABSTRACT

Soil stabilization is the process of improving the shear strength parameters of soil and thus increasing the bearing capacity of soil. It is required when the soil available for construction is not suitable to carry structural load. Soils exhibit generally undesirable engineering properties. Soil Stabilization is the alteration of soils to enhance their physical properties.

Stabilization can increase the shear strength of a soil and/or control the shrink-swell properties of a soil, thus improving the load bearing capacity of a sub-grade to support pavements and foundations. Soil stabilization is used to reduce permeability and compressibility of the soil mass in earth structures.

In this thesis study, pumice has been used to stabilize black cotton soil due to its mineralogical content which will act to have a cementitious property and will undergoes pozzolanic reaction and able to alter the black cotton soil property to a certain degree. The natural pumice material excavated from quarry source and passing 4.75mm sieve was used as stabilizing material. On the process of testing of the soil-pumice mix, the pumice material was subjected to compaction effort which causes crushing of the pumice to a certain extent. Hence, all test results and conclusion were drawn based on partially crushed pumice material in the soil-pumice mix. The effect of pumice as a form of powder and mixed with black cotton soil was not conducted and determined. Hence, further investigation shall be carried out to determine the outcomes of powdered pumice material to its stabilization property after mixing with expansive clay soil.

In this regard an attempt has been made to evaluate the effect of pumice as a stabilizer on the geotechnical properties of black clay soil in stabilized form specifically strength, compaction and swelling characteristics. Therefore, series of laboratory tests such as Atterberg limits, free swell index, compaction, unconfined compressive strength, soaked CBR and swelling pressure tests for different percentages of pumice were performed .The soil was stabilized with pumice in stepped concentration of 10%, 20%, 30%, 40%, 50%, and 60% by dry weight of the soil. Analysis of the results shows slight improvement on the geotechnical properties of pumice stabilized soil. Pumice slightly reduces the index properties of the black clay soil as well as the heaving tendency of the soil. Pumice is known to its light weight property due to that the MDD of the soil decreases as the ratio of the pumice increased in the clay soil. Moreover, the CBR-Swell, free swell and one dimensional swelling pressure shows a decrease as the pumice ratio increases in the clay soil.

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Aberrations and Symbols

AASHTO	American Association of State Highway & Transport Officials
Ac	Activity
ASTM	American Society for Testing and Materials
Al_2O_3	Aluminum oxide
BS	British Standards
Ca	Calcium
CaO	Calcium Oxide
Ca(OH)	Calcium Hydroxide
CaC2	Calcium Carbide
CBR	California Bearing Ratio
Cc	Cubic Centimeter
CaC2	Calcium Carbide
C_3S	Tricalcium silicates
C_2S	Dicalcium silicates
(C_3A)	Tricalcium aluminates
(C_4AF)	Tetra calcium aluminoferrites
FS	Free Swell
Fe_2O_3	Iron oxide
H_2O	Water
KHC	Kansas Highway Commission
kPa	Kilo Pascal
LL	Liquid Limits
MDD	Maximum Dry Density
Mg	Magnesium
NMC	Natural Moisture Content
mm	Millimeter
Na	Sodium
OMC	Optimum Moisture Content
PI	Plastic Index
PL	Plastic Limits
PFA	pulverized fly ash
psi	Pound per Square Inch
SI	shrinkage index
SiO_2	Silicon Oxide
SL	Shrinkage Limit

UCS Unconfined Compressive Strength

Chapter one

1 Introduction

1.1 General

Expansive soil is a clay soil that changes its volume when the water content of the soil changes. Usually, the soil will shrink when water (or moisture) content is reduced and will swell when the water content increases. The degree of expansiveness depends on the content of the active clay mineral called Montmorillonite

The stress caused by alternate heaving and shrinkage of the foundation soil creates stress on the structures. Usually, structures are not designed to withstand this stress or it is difficult to account quantitatively. As a result, the structures are damaged due to the additional stress.

Damage due to expansive soils could occur on any type of buildings, road and other civil engineering constructions that are not properly designed and/or constructed. However, light structures like one or two story buildings, warehouses retaining walls and buried facilities are more vulnerable to damages because these structures couldn't exert a heavy pressure to counteract the uplift pressure from the expansive soils. Therefore, in this study more emphasis is given to reduce and minimize the effect of volume change and to increase the bearing capacity and shearing resistance of these soils by mechanical stabilization using pumice material. The pumice material source which was used in this stabilization study has been collected from Nazareth area. At the quarry site it was observed that the pumice material stands vertically to a height of 4 to 5m without any collapse and this can be one of the indications of pumice material has a cementitious property in addition to its mineralogical composition to react pozzolaniclly with expansive clay minerals. The pumice material used in the stabilization of black cotton soil was partially crushed due to the application of compaction effort during the process of testing. Hence, all test results as well as the conclusion of this thesis study was reached based on a partially crushed pumice material. Therefore, further investigation shall be carried out in order to determine the effect of fully crushed pumice (pumice as a form of powder) on its behavior to stabilize expansive clay soil. The expansive clay soil was stabilized with pumice material in stepped concentration of 10%, 20%, 30%, 40%, 50% and 60% by dry weight of the soil.

1.2 Objective of the study

The general objective of this research is to assess and determine the effect of pumice stabilization on black cotton soil with the addition of pumice material with an increments of 10% to expansive black cotton soils which are collected from Koyefech and Mekanisa areas. In addition the thesis work aimed at evaluating on the extent of the engineering property change of this expansive clay soil after the soil being stabilized with pumice material. The result of the study will come up with remedial measures for those construction which will be conducted on expansive soil areas of the country.

1.3 Methodology

In order to assess the effect of expansive soil stabilization by pumice material various testes have been conducted on the soil sample collected from the aforementioned areas. The following tests were conducted on the soil samples collected from Koyefech and Mekanissa area before and after the samples were blended with pumice material. particle size determination, Atterberg limits, Free swell, Linear shrinkage, Moisture-density relationship, One-point CBR test, CBR swelling, remolded Swelling potential and remolded UCS tests.

1.4 Scope of the research

The research addresses the general objectives and tries to investigate the degree of stabilization of expansive clay soil with the addition of pumice material based on the existing theories and principles of stabilization. The study includes different tests conducted before and after the addition of pumice material to the expansive clay soil and comparison shall be done to assess the improvements found after stabilization.

1.5 Organization of the thesis

The thesis consists of five chapters. The first chapter is the introduction part and it discusses the objective, scope and the methodology of the study in general. The second chapter is literature review of method of soil stabilization in detail. The third chapter deals with the general review on expansive soils. The fourth chapter comprises laboratory data collection and analysis of the laboratory test results of the different blended soils with pumice material at various ratios starting from 10% to 60%. The fifth chapter consists of the conclusion and recommendation of the thesis.

Chapter Two

Literature Review

2.1 Stabilizers

According to Chen, F.H. (1981) and Perloff W.H. (1976) soil stabilization may be defined as the alteration or preservation of one or more soil properties to improve the engineering characteristics and performance of a soil. Stabilization, in a broad sense, incorporates the various methods employed for modifying the properties of a soil to improve its engineering performance. Soil stabilization refers to the procedure in which a special soil, cementing material, or other chemical materials are added to a natural soil to improve one or more of its properties. One may achieve stabilization by mechanically mixing the natural soil and stabilizing material together so as to achieve a homogeneous mixture or by adding stabilizing material to an undisturbed soil deposit and obtaining interaction by letting it permeate through soil voids. Soil stabilizing additives are used to improve the properties of less-desirable soils. When used these stabilizing agents can improve and maintain soil moisture content, increase soil particle cohesion and serve as cementing and water proofing agents. A difficult problem in civil engineering works exists when the sub-grade is found to be clay soil. Soils having high clay content have the tendency to swell when their moisture content is allowed to increase. Many research have been done on the subject of soil stabilization using various additives, the most common methods of soil stabilization of clay soils in pavement work are cement and lime stabilization. The high strengths obtained from cement and lime stabilization may not always be required, however, and there is justification for seeking cheaper additives which may be used to alter the soil properties. Lime or calcium carbonate is oldest traditional chemical stabilizer used for soil stabilization. The study provides details of different types of soil stabilizing methods.

2.2 Soil

Soil is a mixture of minerals, organic matter, gases, liquids, and countless organisms that together support life on Earth. Soil continually undergoes development by way of numerous physical, chemical and biological processes, which include weathering with associated erosion. Most of stabilization has to be undertaken in soft soils (silty, clayey peat or organic soils) in order to achieve desirable engineering properties.

According to Sherwood fine-grained granular materials are the easiest to stabilize due to their large surface area in relation to their particle diameter. A clay soil compared to others has a large surface area due to flat and elongated particle shapes. On the other hand, silty materials can be sensitive to small change in moisture and, therefore, may prove difficult during stabilization. Peat soils and organic soils are rich in water content, high porosity and high organic content. The consistency of peat soil can vary from muddy to fibrous, and in most cases, the deposit is shallow, but in worst cases, it can extend to several meters below the surface. Organic soils have high exchange capacity; it can hinder the hydration process by retaining the calcium ions liberated during the hydration of calcium silicate and calcium aluminate in the cement to satisfy the exchange capacity. In such soils, successful stabilization has to depend on the proper selection of binder and amount of binder added.

2.3 Soil Stabilization

According to Lambe, T.W (1958) and FM5-410, (2012) soil stabilization is a method of improving soil properties by blending and mixing other materials. Soil stabilization is the process of improving the shear strength parameters of soil and thus increasing the bearing capacity of soil. It is required when the soil available for construction is not suitable to carry structural load. Soil stabilization is used to reduce permeability and compressibility of the soil mass in earth structures and to increase its shear strength. Thus to reduce the settlement of structures. Soil stabilization involves the use of stabilizing agents (binder materials) in weak soils to improve its geotechnical properties such as compressibility, strength, permeability and durability.

2.4 Soil Stabilization Methods

According to Sherwood (1993) and Evans, W.C. (2005), in road construction projects, soil or gravelly material is used as the road main body in pavement layers. To have required strength against tensile stresses and strains spectrum, the soil used for constructing pavement should have special specification. Through soil stabilization, unbound materials can be stabilized with cementitious materials (cement, lime, fly ash, bitumen or combination of these). The stabilized soil materials have a higher strength, lower permeability and lower compressibility than the native soil. The method can be achieved in two ways, namely;

1) In situ stabilization and

2) Ex - situ stabilization.

Note that, every soil properties cannot be improved for better by the process of stabilization. The decision to technological usage depends on which soil properties have to be modified. The chief properties of soil which are of interest to engineers are volume stability, strength, compressibility, permeability and durability. Some stabilization techniques are listed below;

a. Mechanical Stabilization
b. Stabilization by using different types of admixers

(I) Lime Stabilization

(II) Cement Stabilization

(III) Chemical Stabilization

(IV) Fly ash Stabilization

(V) Rice Husk ash Stabilization

(VI) Bituminous Stabilization

(VII) Thermal Stabilization

(VII) Electrical Stabilization

(IX) Stabilization by Geo-textile and Fabrics

(X) Recycled and Waste Products etc.

2.4.1 Mechanical Stabilization

Mechanical Stabilization is the process of improving the properties of the soil by changing its gradation. This process includes soil compaction and densification by application of mechanical energy using various sorts of rollers, rammers, vibration techniques and sometime blasting. The stability of the soil in this method relies on the inherent properties of the soil material. Two or more types of natural soils are mixed to obtain a composite material which is superior to any of its components. Mechanical stabilization is accomplished by mixing or blending soils of two or more gradations to obtain a material meeting the required specification.

2.4.2 Stabilization by using different types of admixers

2.4.2.1 Lime Stabilization

Lime provides an economical way of soil stabilization. The method of soil improvement in which lime is added to the soil to improve its properties is known as lime stabilization. The types of lime used to the soil are hydrated high calcium lime, monohydrated dolomite lime, calcite quick lime, dolomite lime. The quantity of lime is used in most soil stabilizer is in the range of 5% to 10%. Lime modification describes an increase in strength brought by cation exchange capacity rather than cementing effect brought by pozzolanic reaction. In soil modification, as clay particles flocculates, transforms natural plate like clays particles into needle like interlocking metalline structures. Clay soils turn drier and less susceptible to water content changes. Lime stabilization may refer to pozzolanic reaction in which pozzolana materials reacts with lime in presence of water to produce cementitious compounds. The effect can be brought by either quicklime, CaO or hydrated lime, Ca (OH).

Slurry lime also can be used in dry Soils conditions where water may be required to achieve effective compaction. Quicklime is the most commonly used lime; the followings are the advantages of quicklime over hydrated lime higher available free lime content per unit mass - denser than hydrated lime (less storage space is required) and less dust - generates heat which accelerate strength gain and large reduction in moisture content according to the reaction equation below $CaO + H_2O \rightarrow Ca(OH)_2 +$ Heat (65kJ / mol) Quicklime when mixed with wet soils, immediately takes up to 32% of its own weight of water from the surrounding soil to form hydrated lime; the generated heat accompanied by this reaction will further cause loss of water due to evaporation which in turn results into increased plastic limit of soil i.e. drying out and absorption.

Sherwood investigated the decrease in plasticity as brought about in first instance by cation exchange in which cations of sodium and hydrogen are replaced by calcium ions for which the clay mineral has a greater water affinity. Even in soils (e.g. calcareous soils) where, clay may be saturated with calcium ions, addition of lime will increase pH and hence increase the exchange capacity. Like cement, lime when reacts with wet clay minerals result into increased pH which favors solubility of siliceous and aluminous compounds. These compounds react with calcium to form calcium silica and calcium alumina hydrates, a cementitious product similar to those of cement paste. Natural pozzolanas materials containing silica and alumina (e.g. clay minerals, pulverized fly ash, PFA, blast furnace slag) have great potential to react with lime. Lime stabilizations technology is mostly widely used in geotechnical and environmental applications. Some of applications include encapsulation of contaminants, rendering of backfill (e.g. wet cohesive soil), highway capping, slope stabilization and foundation improvement such as in use of lime pile or lime-stabilized soil columns.

However, presence of Sulphur and organic materials may inhibit the lime stabilization process. Sulphate (e.g. gypsum) will react with lime and swell, which may have effect on soil strength.

2.4.2.2 Cement Stabilization

Soil cement stabilization is soil particles bonding caused by hydration of the cement particles which grow into crystals that can interlock with one another giving a high compressive strength. In order to achieve a successful bond the cement particles need to coat most of the material particles. To provide good contact between soil particles and cement, and thus efficient soil cement stabilization, mixing the cement and soil with certain particle size distribution is necessary.

The main compounds of Portland cement are tricalcium silicates (C_3S), dicalcium silicates (C_2S), tricalcium aluminates (C_3A), and tetracalcium aluminoferrites (C_4AF) (where C = Ca, S = SiO_2, A = Al_2O_3, and F = Fe_2O_3). Prusinski and Bhattacharja (1999) state that when cement is mixed with water, and then hydrates, the most important products from the chemical reaction include calcium silicate hydrate and calcium hydroxide from the two calcium silicates. The calcium silicate hydrate stabilizes the soil by forming a hard structure around the soil particles, and the calcium hydroxide stabilizes the soil through ion exchange, flocculation of the clay particles, and over the long term, by secondary cementing material formed by release of silicates from the clay and their combination with calcium from the calcium hydroxide. Herzog and Mitchell (1963) believe that the calcium hydroxide generated from the hydration of the calcium silicates is more reactive than hydrated lime, since the calcium hydroxide created from the calcium silicates is very fine and well dispersed throughout the soil.

Soil-cement is a highly compacted mixture of soil/aggregate, cement, and water. Soil-cement is sometimes called cement-stabilized base, or cement-treated aggregate base. Soil-cement becomes a hard and durable material as the cement hydrates and develops strength. Cement stabilization is done when the compaction process is continuing. As the cement fills the void between the soil particles, the void ratio of soil is reduced. After this when water is added to the soil, cement reacts with water and goes hard. So, unit weight of soil is increased. Because of hardening of cement shear strength and bearing capacity is also increased. Cement helps decrease the liquid limit and increase the plasticity index and workability of clayey soils. Cement reaction is not dependent on soil minerals, and the key role is its reaction with water that may be available in any soil. This can be the reason why cement is used to stabilize a wide range of soils.

Numerous types of cement are available in the market; these are ordinary Portland cement, blast furnace cement, sulfate resistant cement and high alumina cement. Usually the choice of cement depends on type of soil to be treated and desired final strength. Hydration process is a process under which cement reaction takes place. The process starts when cement is mixed with water and other components for a desired application resulting into hardening phenomena. The hardening (setting) of cement will enclose soil as glue, but it will not change the structure of soil. The hydration reaction is slow proceeding from the surface of the cement grains and the center of the grains may remain unhydrated.

Cement hydration is a complex process with a complex series of unknown chemical reactions. However, this process can be affected by

(a) Presence of foreign matters or impurities
(b) water-cement ratio
(c) Curing temperature
(d) Presence of additives
(e) Specific surface of the mixture.

Depending on factor(s) involved, the ultimate effect on setting and gain in strength of cement stabilized soil may vary. Therefore, this should be taken into account during mix design in order to achieve the desired strength. Calcium silicates, C_3S and C_2S are the two main cementitious properties of ordinary Portland cement responsible for strength development. Calcium hydroxide is another hydration product of Portland cement that further reacts with pozzolanic five materials available in stabilized soil to produce further cementitious material. Normally the amount of cement used is small but sufficient to improve the engineering properties of the soil and further improved cation exchange of clay. Cement stabilized soils have the following improved properties:

(a) Decreased cohesiveness (Plasticity)

(b) decreased volume expansion or compressibility

(c) Increased strength.

2.4.2.3 Chemical Stabilization

Chemical stabilization of soil comprises of changing the physico-synthetic around and within clay particles where by the earth obliges less water to fulfill the static imbalance. Calcium chloride being hygroscopic and deliquescent is used as a water retentive additive in mechanically stabilized soil bases and surfacing. The vapor pressure gets lowered, surface tension increases and rate of evaporation decreases. The freezing point of pure water gets lowered and it results in prevention or reduction of frost heave. The depressing the electric double layer, the salt reduces the water pick up and thus the loss of strength of fine grained soils. Calcium chloride acts as a soil flocculent and facilitates compaction. Frequent application of calcium chloride may be necessary to make up for the loss of chemical by leaching action. For the salt to be effective, the relative humidity of the atmosphere should be above 30%. Sodium chloride is the other chemical that can be used for this purpose with a stabilizing action similar to that of calcium chloride. Sodium silicate is yet another chemical used for this purpose in combination with other chemicals such as calcium chloride, polymers, chrome Lignin, alkyl chlorosilanes, siliconites, amines and quaternary Ammonium salts, sodium hexametaphosphate, phosphoric acid combined with a wetting agent.

2.4.2.4 Calcium Carbide

Calcium carbide is used for the Speedy Moisture Test (ASTM D 4944), but it apparently has not been used for soil stabilization. Nevertheless, calcium carbide should stabilize soil in a manner similar to lime, and it could actually be more effective than lime.

When calcium carbide reacts with water in the soil, the end products are acetylene gas and hydrated lime, with production of quicklime during an intermediate step, as demonstrated in the equation below.

$CaC_2(s) + 2H_2O \longrightarrow C_2H_2\ (g) + Ca\ (OH)_2\ (aq) = CaC_2 + H_2O \longrightarrow C2H_2 + Cao + Cao + H_2o \longrightarrow$
$Ca\ (OH)_2$

More water is consumed in these chemical reactions than is consumed by quicklime hydration alone. In addition, more heat is generated by the calcium carbide reactions, which would evaporate more water than would be evaporated by quicklime hydration alone. Furthermore, if the acetylene gas were captured and combusted, even more water could be driven off. As an alternative to combusting the acetylene gas, it may be possible to polymerize the acetylene gas in the clay, since acetylene gas consists of unsaturated monomers that can be polymerized under the right conditions. Such polymers could stiffen and strengthen the mixture.

2.4.2.5 Sodium Silicates

Ding et al. (1996) found that the addition of only sodium silicates to hydrated clay may actually negatively affect soil stabilization. Clay particles typically have a net negative charge on their face and a positive charge along their edges due to broken bonds. When sodium silicates are added to hydrated clay, the negative silicate ions from the sodium silicates are attracted and attach to clay particle edges causing entire clay particles to become negatively charged. If the entire clay particles have a negative charge, they will repel one another and the clay structure will become dispersed and weak.

Although sodium silicates may weaken clay when added alone, Ruff and Davidson (1961) affirm that sodium silicates may strengthen clay if lime is added along with the sodium silicates. The lime can be used as a source of calcium ions, and with the presence of both calcium ions and silicate ions, calcium silicate gel can form, hydrate, and harden, thereby cementing the clay particles together.

2.4.2.6 Super Absorbent Polymers

According to Dr. Joseph Rafalko (personal communication, 2006), soil could possibly be stabilized with calcium and super absorbent polymers, such as sodium or potassium polyacrylic acids. This combination of calcium and super absorbent polymers could stabilize the soil by absorbing excess water, exchanging ions with the clay particles, and hardening a polymer network throughout the soil. When the polymers absorb water, a weak gel is formed, but calcium from other sources, such as quicklime or calcium carbide, could possibly crosslink the polymers of sodium or potassium polyacrylic acid together to form a harder material. Very little research exists on soil stabilization with calcium and sodium or potassium polyacrylic acids, but Lambe (1951) studied a similar material, calcium acrylate, as a potential soil stabilizer. The calcium acrylate should theoretically exchange ions with the soil as well as crosslink other polymers of calcium acrylate together. Lambe (1951) found that the addition of calcium acrylate did significantly increase the compressive strength. However, calcium acrylate treated soil may lose strength over time, which may be a problem for a soil treated with calcium and sodium or potassium polyacrylic acid as well (Karol 2003).

2.4.2.7 Dispersants/Superplasticizer/Water Reducers

Naudts et al. (2002) maintain that dispersants, superplasticizers, and water reducers are all common cement admixtures that increase the workability and increase the strength of a cement mixture at low water-to-cement ratios. More specifically, Gallagher (2000) indicates that dispersants increase the workability by coating the cement particles with a negative charge so they will repel one another, which also prevents flocculation of micro fine cement particles. If cement particles are prevented from flocculating into larger particles, the overall surface area may not be reduced and chemical reactions may not be slowed.

2.4.2.8 Accelerators

Accelerators are cement admixtures that decrease set time and increase rate of strength gain for a concrete mixture. Mamlouk and Zaniewski (1999) report that the most common type of accelerator is calcium chloride, which can reduce the typical final set time from six hours to three hours with a dosage rate of 1% of the cement weight, and to two hours with a dosage rate of 2% of the cement weight. The Materials Group (1999) at the Federal Highway Administration state that calcium chloride can slightly increase the workability of the cement and reduce the amount of water needed, similar to the effect of dispersants, superplasticizers, and water reducers, but to a lesser degree. However, exposure of metals to this accelerator should be limited, since calcium chloride can corrode and weaken certain metals. Other non-chloride accelerators can be used when contact with metal is unavoidable, such as in reinforced concrete, but unfortunately, these accelerators are often not as effective as calcium chloride.

2.4.2.9 Fly ash Stabilization

Fly ash stabilization is gaining more importance recent times since it has wide spread availability. This method is inexpensive and takes less time than any other methods. It has a long history of use as an engineering material and has been successfully employed in geotechnical applications. Fly ash is a byproduct of coal fired electric power generation facilities; it has little cementations properties compared to lime and cement. Most of the fly ashes belong to secondary binders; these binders cannot produce the desired effect on their own. However, in the presence of a small amount of activator, it can react chemically to form cementations compound that contributes to improved strength of soft soil. However, soil fly ash stabilization has the following limitations:

(a) Soil to be stabilized shall have less moisture content; therefore, dewatering may be required.

(b) Soil-fly ash mixture cured below zero and then soaked in water are highly susceptible to slaking and strength loss

(c) Sulfur contents can form expansive minerals in soil-fly ash mixture, which reduces the long term strength and durability.

2.4.2.10 Rice Husk ash Stabilization

Disposal of solid waste on the land fill can be minimized if the waste is having desirable properties such that they can be utilized for various geotechnical application viz. Land reclamation, construction of embankment etc. There are several methods used for improving geotechnical properties of problematic soils that includes densification (such as shallow compaction, dynamic deep compaction, pre-loading), drainage, inclusions (such as geosynthetics and stone columns), and stabilizations. Chemical stabilization of the problematic soils is especially significant in concerning with the treatment of soft fine-grained, expansive soils, and collapsible loess deposits. Soil stabilization is the process which is used to improve the engineering properties of the soil and thus making it more stable. Soil stabilization is required when the soil available for construction is not suitable for the intended purpose. It includes compaction, preconsolidation, drainage and many other such processes.

Rice husk ash (RHA) is a pozzolanic material that could be potentially used in soil stabilization, though it is moderately produced and readily available. When rice husk is burnt under controlled temperature, ash is produced and about 17%-25% of rice husk's weight remains ash. Rice husk ash and rice straw and bagasse are rich in silica and make an excellent pozzolana. Pozzolanas are siliceous and aluminous materials, which in itself possess little or no cementations value, but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperature to form compounds possessing cementations properties. The Rice Husk Ash would appear to be an inert material with the silica in the crystalline form suggested by the structure of the particles, it is very unlikely that it would react with lime to form calcium silicates. It is also unlikely that it would be as reactive as fly ash, which is more finely divided. So Rice Husk Ash would give great results when it used as a stabilizing material. The ash would appear to be a very suitable light weight fill and should not present great.

2.4.2.11 Bituminous Stabilization

Bituminous soil stabilization refers to a process by which a controlled amount of bituminous material is thoroughly mixed with an existing soil or aggregate material to form a stable base or wearing surface. Bitumen increases the cohesion and load-bearing capacity of the soil and renders it resistant to the action of water. Bitumen stabilization accomplished by using asphalt cement, asphalt cutback or asphalt emulsions. The type of bitumen to be used depends on the type of soil to be stabilized, method of construction and weather conditions. In frost areas, the use of tar as binder must be avoided because of its high temperature maximum susceptibility.

Asphalts and tars are bituminous materials which are used for stabilization of soil, generally for pavement construction. Bituminous materials when added to a soil, it imparts both cohesion and reduced water absorption.

2.4.2.12 Thermal Stabilization

Thermal change causes a marked improvement in the properties of the soil. Thermal stabilization is done either by heating the soil or by cooling it.

Heating: As the soil is heated, its water content decreases. Electric repulsion between clay particles is decreased and the strength of the soil is increased.

Freezing: cooling causes a small loss of strength of clayey soils due to an increase in inter particles repulsion. However, if the temperature is reduced to the freezing point, the pore water freezes and the soil is stabilized.

2.4.2.13 Electrical Stabilization

Electrical stabilization of clayey soils is done by a process known as electro-osmosis. As a direct current (DC) is passed through a clayey soil, pore water migrates to the negative electrode (cathode). It occurs because of attraction of positive ions (cations) that are present in water towards cathode. The strength of the soil is considerably increased due to removal of water. Electro-osmosis is an expensive method, and is mainly used for drainage of cohesive soils. Incidentally, the properties of the soil are also improved.

2.4.2.14 Stabilization by Geo-textile and Fabrics

Geotextiles are porous fabrics made of synthetic materials such as polyethylene, polyester, nylons and polyvinyl chloride. Woven, non-woven and grid form varieties of geotextiles are available. Geotextiles have a high strength.

When properly embedded in soil, it contributes to its stability. It is used in the construction of unpaved roads over soft soils. Reinforcing the soil for stabilization by metallic strips into it and providing an anchor or tie back to restrain a facing skin element. Past research has shown that the strength and load-bearing capacity of subgrades and base course materials can be improved through the inclusion of non-biodegradable reinforcing materials, such as fibers, geotextiles, geogrids, and geo-composites. Use of these materials can improve the performance and durability of future highways and may reduce the cost of construction. At present, most of the research on these materials is based on tests conducted in the laboratory that are only partially complete. Further laboratory tests and evaluations will be necessary to develop design specifications based on material properties, and these specifications will need to be verified using large-scale field tests.

2.4.2.15 Recycled and Waste Products

Improved chemical and mechanical stabilization techniques are needed for such waste materials as crushed old asphalt pavement, copper and zinc slag, paper mill sludge, and rubber tire chips. The need to recycle many potentially hazardous materials, it will be necessary to develop a realistic, economical and effective means of assessing the risk of pollution posed by these materials through leachates and emissions. In some cases, risk evaluation is hampered by restrictive environmental constraints, and this issue needs to be addressed as well.

Chapter Three

3. General review on expansive soils

Introduction

Most soil classification systems arbitrary define clay particles as having an effective diameter of two microns (0.002mm) or less. Particle size alone doesn’t determine clay mineral. Probably the most important grain property of fine-grained soils is the mineralogical composition (chen, 1988). For small size particles, the electrical forces acting on the surface of the particles are much greater than the gravitational force. These particles are in a colloidal state. The colloidal particles consist primarily of clay minerals that were derived from parent rock by weathering.

The three most important groups of clay minerals are Montmorillonite, Illite, and Kaolinite. Montmorillonite is the clay mineral that is mostly present in expansive soil. When these minerals are exposed to moisture, water is absorbed between the inter-layering lattice structures and exerts an upward pressure, which is the cause for most damages associated with expansive soil.

3.1 Origins of expansive soils

3.1.1 Parent material

The parent materials that give rise to expansive soil are classified into two. The first group comprises the basic igneous rocks, which are low in silica, generally about 45% to 25% and rich in metallic base such as pyroxenes, amphiboles, biotitic and olivine. Such rocks include the gabbro, basalts and volcanic glass. The second group includes sedimentary rock that contains montmorillonite as a constituent. These include shales and claystone, and limestone and marls rich in magnesium.

3.1.2 Weathering and climate

The weathering process by which clay is formed includes physical, biological and chemical process. The most important weathering process responsible for the formation of montmorillonite is the chemical weathering, which include hydrolysis, hydration, oxidation, carbonation and solution, of parent rock mineral which generally consists of ferromagnesium mineral, calcic feldspars, volcanic glass, volcanic rocks and volcanic ash. The formation is aided in alkaline environment, presence of magnesium ion and lack of leaching. Such condition is favorable in semi-arid regions with relatively low rain fall or seasonal moderate rainfall particularly where evaporation exceed precipitation. Under these conditions enough water is available for the alteration process but the accumulated cations will not be removed by rainwater.

3.1.3 Clay mineralogy

The clay minerals are classified as follows:

A: Two-layer clays which consists of one tetrahedral layer bounded to one aluminum octahedral layer.

Kaolinite is the most common mineral under this category.

B: Three-layer clays which consists of one octahedral layer sandwiched between two tetrahedral layers.

Illite, montmorillonite and vermiculite are the common mineral under this category.

C: Mixed-layer clays which consists of interstratifications of the two and three-layer clay minerals previously described. The mixing may be regular or random. Common mineral under these classes are chlorite, montmorillonite-chlorite.

The clay mineral Kaolinite exhibits very minor interlayer swelling. This is explained by the virtual absence of ionic substitution in either the tetra- or octahedral layers which results in more or less complete electrical neutrality and the absence of compensating cations. Also, the individual two layer structures are more tightly bonded together by the opposing electrical charges on the adjacent octa- and tetrahedral layers.

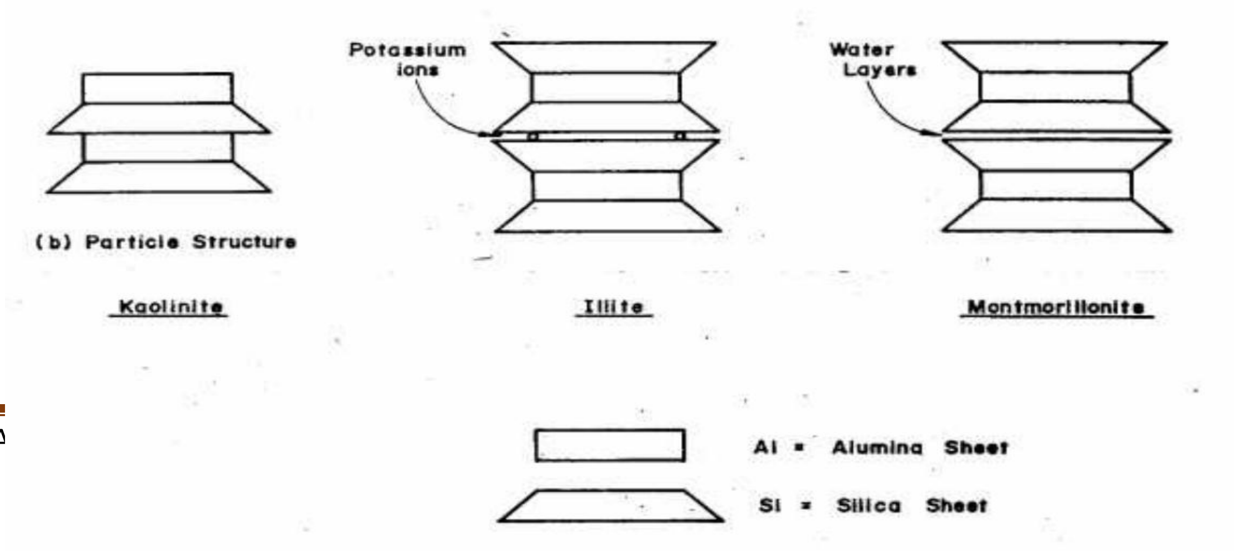


Fig. 3.1 Symbolic Structure of Clay minerals (Teferra, and Leikun, 1999)

Illite also a three-layer clay mineral, it exhibits very minor interlayer swelling. This result from the presence of non-hydrated K^+ ions in the interlayer positions within the hexagonal openings of the tetrahedral layer. The K^+ satisfies charge deficiencies residing mainly on the tetrahedral layer and is thus tightly bonded. The clay mineral responsible for the most damage of expansive soil is montmorillonite.

Montmorillonite is a diocthedral and usually contains some magnesium substituted for aluminum in the octahedral layer. This substitution results in a lattice charge deficiency that is neutralized by the presence of cations such as Na+, Ca++, or Mg++ on the interlayer positions. Although these ions possess ionic radii that would permit occupancy of the space within the hexagonal opening at the surface of the tetrahedral layers. The ions are hydrated and as a result of increased ionic radii must occupy space on and above the tetrahedral layers. Such a position props adjacent layers apart and permit access of more water to interlayer positions. (Grim, 1962)

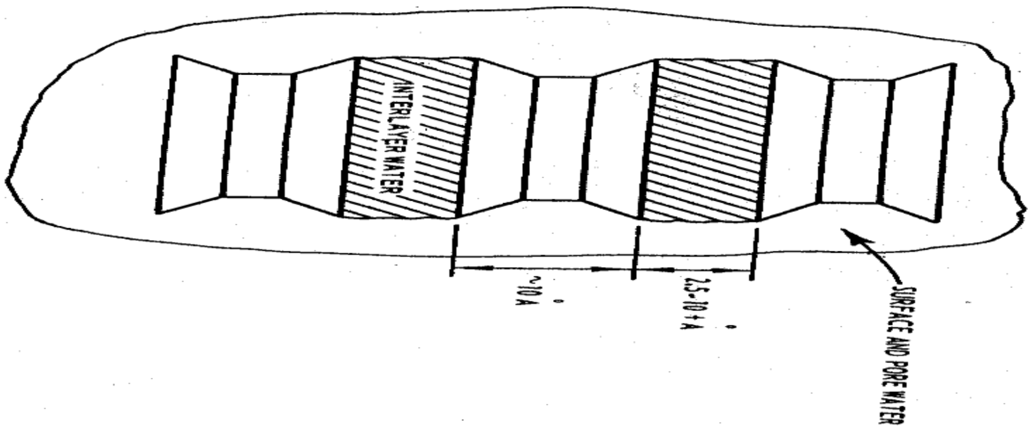


Fig. 3.2 Deflocculated clay mineral showing surface and interlayer water

3.2 Identifications of expansive soil

3.2.1 Field identification

It is evident that expansive soil deposits can be recognized in the field through visual inspections. The method is simple and easy to use.

Some of the important field identification method that indicate the potential for expansiveness of a soil are the following:

- A shiny surface is easily obtained when a partially dry piece of the soil is polished with a smooth object such as the top of a finger nail
- The wet samples of the soil is sticky and it will be relatively difficult to clean the soil from the hands

- The appearance of cracking in nearby structures
- They usually have a color of black and gray
- In the regions where there is seasonal moisture variation
- Open or closed fissures, (a joint or similar discontinuity)
- Slickenside, (highly polished or glossy fissure surface)
- Shattering or micro-shattering, (presence of fissures forming granular fragments of clayey soils)

3.2.2 Laboratory identification

Generally, there are three different method of identifying expansive soil in the laboratory.

3.2.2.1 Mineralogical identification

This method is used for identifying the mineralogy of clay particles such as characteristic crystal dimensions, characteristic reaction to heat treatment, size and shape of clay particles and change deficiency and surface activity of clay particle. These properties are a fundamental factor controlling expansive soil behavior.

The various techniques under these methods are

- X-ray diffraction
- Differential thermal analysis
- Dye absorption
- Electron microscope
- Base exchange capacity, etc

But these methods are not suitable for routine tests because of the following reason; they are time consuming, require expensive test equipment and, the results are interpreted by specially trained technicians.

3.2.2.2 Indirect methods

These methods include simple soil property test that a practicing engineer resort to use for identifying expansive soil. Such tests are easy and can be performed in most soil mechanics laboratory, and yield an excellent indices of expansive properties. The various tests under these methods are:

3.2.2.2.1 Atterberg limits

In this method, measurement of the plasticity and the shrinkage characteristics of the soil are conducted for identification of all engineering soils and provide a wide acceptable means of rating. Especially when they are combined with other tests they can be used to classify expansive soils. The different types of limits under this method are Liquid limit, Plastic limit and Shrinkage limit.

3.2.2.2.2 Linear shrinkage tests

Linear shrinkage, is the change in length of a soil sample as it dries to the shrinkage limit (SL), expressed as a percentage of the original length. This test also in combination with other tests is used for classification of expansive soil.

3.2.2.2.3 Free swell tests

The free swell test may be considered as a measurement of volume change in clay upon saturation and is one of the most commonly used simple tests to estimate the swelling potential of expansive clay. The free swell is then given by:

$$FS = (V - V_o)/V_o * 100\%$$

Where FS= free swell, %

V= soil volume after swelling, cm^3

V_O=volume of dry soil, $10cm^3$

3.2.2.2.4 Colloid content test

This test is used to determine the quantity of material in a soil sample that is smaller than a selected size, expressed as a percentage by weight of the total sample. Sizes used are 2µm (0.002mm) and 1µm (0.001mm); the upper limit of the clay range is generally considered to be 2 to 5 µm. The test usually requires hydrometer analysis.

3.2.2.3 Direct measurement

The most accurate and dependable method of determining the swelling potential and the swelling pressure of expansive clay is by direct measurement. The method quantitatively evaluates the volume change characteristics of expansive soil. The test can be done using consolidometer but care should be taken on the test procedure. A standardized procedure that consider the factors that affect the shrink swell potential as well as simulate the expected loading condition should be adopted.

3.2.2.3.1 Classification of expansive soil

The parameters determined from expansive soil identification tests have been combined in a number of different classification schemes. But before using any soil classification system, engineers should understand the data base from which it was derived and establish its limitations;

otherwise, poor reliability and lack of confidence in the system may result. The different classification system are categorized into two:

1. General classification systems which have evolved over many years and are based largely on correlation with actual performance.
2. Those devised specifically for classification of expansive soils. Theses system are based on indirect and direct prediction of swell potential, as well as combinations, to arrive at a rating.

3.2.2.3.2 General classification system

The most widely used general classification systems are

i. Unified soil classification systems

In these classification system a correlation is made between swell potential and unified soil classification as follows;

Table 3.1 Unified soil classification

Category	soil classification in unified system
Little or no expansion	GW, GP, GM, SW, SP, SM
Moderate expansion	GW, SC, ML, MH
High volume change	CL, OL, CH, OH

ii. AASHTO classification

This recommended practice describes a procedure for classifying soils into seven groups based on laboratory determination of particle-size distribution, liquid limit, and plasticity index. Evaluation of soils within each group is made by means of a “group index,” which is a value calculated from an empirical formula. The group classification, including group index should be useful in determining the relative quality of the soil material. However, for the detailed design of important structures additional data concerning strength or performance characteristics of the soil under field conditions will usually be required.

The classification is made by using the test limits and group index values. The liquid limit and plasticity index ranges for the A-4, A-5, A-6, and A-7 soil groups are shown graphically in Figure 2.3. Soils that lies on A6, A7 and borderline soils A-4, A-6, and A-7 may be considered as potentially expansive.

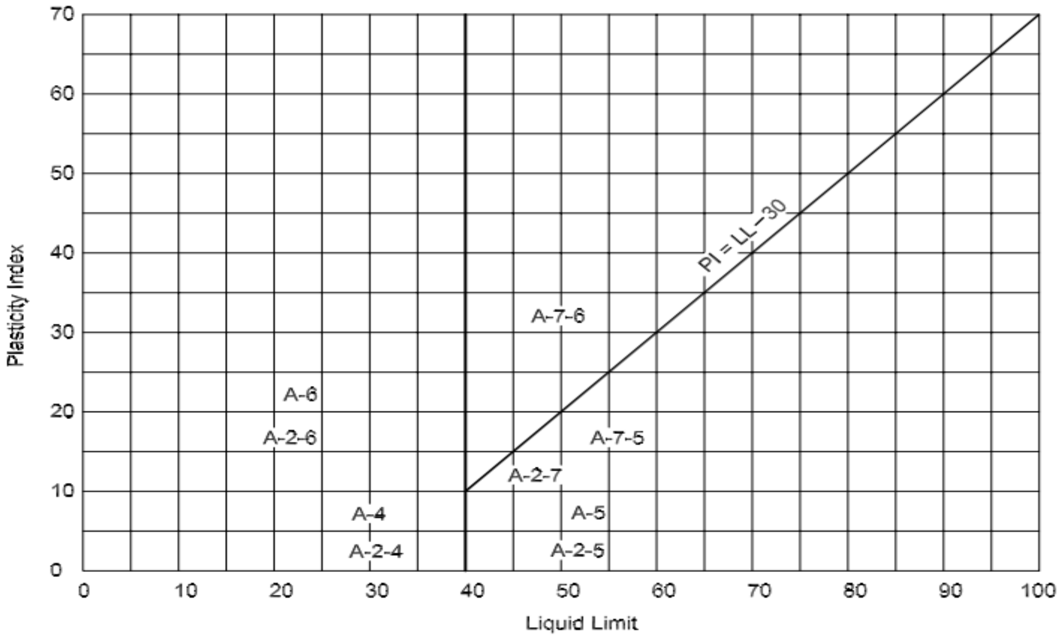


Fig. 3.3 Liquid limit against plasticity index to the AASHTO classification method

3.2.2.4 Classification specific to expansive soil

The above classification system may give an initial alert that the soil may have expansive character and doesn't provide useful information. A parameter determined from the expansive soil identification tests have been combined in a number of different classification schemes to give qualitative rating on the expansiveness of the soil. But the direct use of such classification systems as a basis for design may lead to an overly conservative construction in some places and inadequate construction in some areas (Nelson, 1992). Hence, it is very important to emphasize that design decision has to be based on predicting testing and analysis, which provide reliable information.

3.2.2.4.1 Classification based on indirect predictions of swell potential

An indirect prediction of swell potential includes correlations based on index properties, swell, physical indicator and a combination of them. Some of such classification systems are:

i. Skempton (Mckeen, 1976)

This method is developed, by combining Atterberg limits and clay content into a single parameter called Activity. Activity is defined as;

Activity, Ac = PI/ percentage by weight finer than 2µm

Skempton suggested three classes of clays according to their activity.

Activity	Potential of expansion
Ac<0.75	Low (inactive)
0.75< Ac<1.25	Medium (normal)
Ac>1.25	High (*active)

Table 3.2 Skempton Method of classification of expansive soil

ii. Altmeyer (Mckeen, 1976)

He suggest rating for degree of expansion based on shrinkage limit (SL) and linear shrinkage:

Table 3.3 Altmeyer Method of classification of expansive soil

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

The classification system developed based on single property alone such as: based on activity (Skempton, 1953), based on shrinkage limit and linear shrinkage (Altmeyer, 1956), based on index property (Kanntey and Brink, 1952), etc. (Nelson, 1992) are difficult to use alone as a classification system because they may lead to wrong conclusion.

A better indirect classification system can be developed by combining index property, swell and physical indicator. Good examples for such classification are:

iii. Bureau of reclamation Method

This method involves direct correlation of observed volume change with colloidal content, PI, and SL. The degree of expansion and limits of correlated properties are shown in the following tabulation;

Table 3.4 classification based on bureau of reclamation method

Colloid content, %-1μm	PI, %	SL, %	Probable expansion %	Degree of expansion
<15	<18	>15	<10	Low
13-23	15-28	10-16	10-20	Medium

20-31	25-41	7-12	20-30	High
>28	>35	<11	>30	Very High

Experience has shown that this method correlates reasonably well with expected behavior and provides a good indicator of potential volume change. The major criticisms of the method are that the colloidal content indicates amount but not the type of clay constituents and that the hydrometer test is not a routine test in many agency laboratories.

iv. Chen Method

In an effort to simplify the USBR method (i.e. eliminate the need for hydrometer analysis) and to provide some relative measure of soil density, a correlation was made between odometer swell data and percent passing the No. 200 sieve, LL, and standard penetration resistance. The resulting classification of the degree of expansion is as follows:

Table 3.5 Chen Method of classification of expansive soil

< No 200, sieve, %	LL %	Standard penetration blows	Probable Expansion %	Degree of expansion
<30	<30	<10	<1	Low
30-60	30-40	10-20	1-5	Medium
60-95	40-60	20-30	3-10	High
>95	>60	>30	>10	Very High

v. Louisiana Department of Transportation

The Louisiana Department of Transportation uses the Atterberg Limits (liquid limit (LL) and plasticity index (PI)) balanced with field experience to identify potentially expansive soils.
The criteria used for identifying and classifying potential swell are:

Table 3.6 Louisiana Department of Transport Method of classification of expansive soil.

LL %	PI %	Potential Swell classification
20-49	15-24	Low to medium
50-70	25-46	High
>70	>46	Very high to severe

vi. Kansas highway commission

The Kansas Highway Commission (KHC) also uses the Atterberg limits (PI) to indicate potentially expansive soils. In addition, KHC generally follows up with an odometer-type volume change test (1-psi surcharge) to better estimate the quantity of anticipated swell. The KHC criteria for potential swell are:

Table 3.7 Kansas highway commission Method of classification of expansive soil.

PI %	Potential Swell classification
<15	Low or non
15-35	Moderate
>35	High

vii. Raman

This method uses the Atterberg limits (LL, PI, and shrinkage limit (SL)), but in a different configuration. Raman defines the shrinkage index (SI) as the difference between the LL and the SL (LL- SL). The criteria recommended by Raman for identification/classification are:

Table 3.8 Raman Method of classification of expansive soil.

PI, %	SI, %	Degree of expansion
<12	<15	Low
12-23	15-30	Medium
23-32	30-40	High
>32	>40	Very high

viii. Sowers

Sowers' early work used only the PI as an indicator of potential swell. In his later work he combines the PI with the SL to provide additional accuracy. The criteria are:

Table 3.9 Sowers Method of classification of expansive soil.

PI, %	SI, %	Potential Volume change
>12	<15	Probably Low
10-12	15-30	Probably moderate
<10	>30	Probably High

One interesting note in Sowers' work is his reference to the water plasticity ratio or liquidity index (LI). His data indicate that little swell will occur when the soil moisture reaches a value which results in a LI of 0.25.

ix. Dakshanamurthy and Raman

This method is based on a modification of Casagrande's plasticity chart, which includes PI and LL, with the addition of the SI (LL- SL). Figure 3.4 is a graphical representation of the recommended criteria. A simplification of the procedure using the LL is:

Table 3.10 Dakshanamurthy and Raman Method of classification of expansive soil.

LL, %	Potential Swell Classification
0-20	Non swelling
20-35	Low swelling

35-50	Medium swelling
50-70	High swelling
70-90	Very high swelling
>90	Extra high swelling

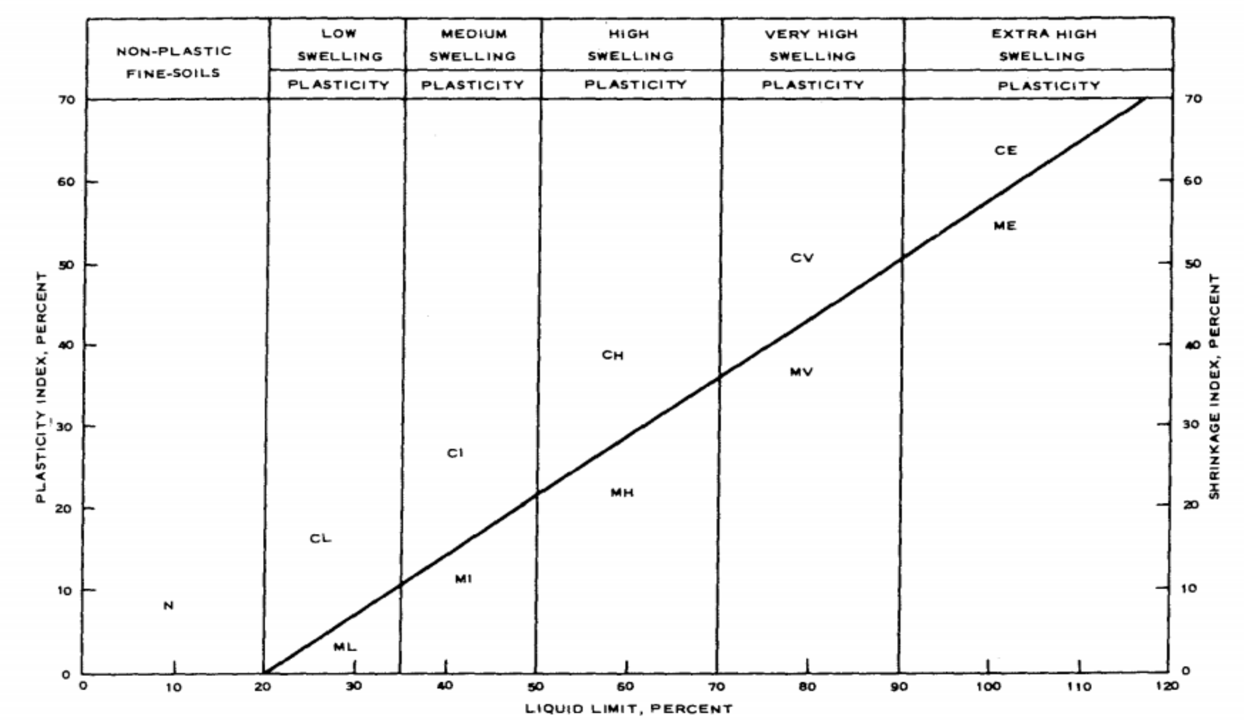


Fig. 3.4 Chart for potential expansiveness of soil

x. Anderson and Thomson

The authors of this method use the PI alone, but rather than qualitatively describe the degree of expansion alone, they add a categorization of measured potential swell. The recommended criteria are:

Table 3.11 Anderson and Thomson Method of classification of expansive soil.

PI, %	Potential swell, %	Degree of expansion
<20	<1.5	Low
20-31	1.5-4.0	Medium
31-39	4.0-6.0	High
>39	>6.0	Very high

xi. Ranganatham and Satyanarayana

This method uses the Atterberg limits (LL and SL) in combination to define the SI (LL - SL). These authors were the first to use the SI for identification of expansive soils. The criteria are:

Table 3.12 Ranganatham and Satyanarayana Method of classification of expansive soil.

SI, %	Potential Swell Classification
<20	Low
20-30	Medium
30-60	High
>60	Very high

xii. Saito and Miki

This method defines the plasticity ratio (PR) as the PI divided by the PL and uses this ratio to correlate with measured swell. The corresponding criteria for potential swell are:

Table 3.13 Saito and Miki Method of classification of expansive soil.

PR	Potential volume change	Potential swell classification
<0.6	<3	Low
0.6-1.0	3-10	Medium
1.0-2.0	10-50	High
>3.0	>50	Very high

xiii. Vijayvergiya and Ghazzaly

The method defines a swell index for an expansive soil as the ratio of the natural water content w_i to the LL and correlates it with odometer swell and swell pressure data. Rather than a specific degree of expansion, limits of probable swell and swelling pressure are defined as shown in-the following tabulation:

Table 3.14 Vijayvergiya and Ghazzaly Method of classification of expansive soil.

W_i/LL	Probable swell pressure	Probable Swell, %
>0.37	<0.3	<1
0.37-0.5	0.3-1.25	1-4
0.25-0.37	1.25-3.0	4-10
<0.25	>3.0	>10

3.3 Prediction of Heave

As many prediction methods in use today involve direct measurements of swelling pressure with a consolidometer and most of the structural damages occur when the swelling pressure is greater than the foundation pressure, assessing the swelling pressure is an important task in dealing with expansive soil.

3.3.1 Consolidometer Testing.

Evaluation of soil volume changes by consolidometer testing is the most widely used method for predicting heave. The different types of techniques under these methods are:

i. Zero swell test

This test is conducted by applying a small incremental load of 6.9kpa (1psi) on a compacted specimen. Water is then added to the sample. As expansion starts, pressure is added in small increment to prevent swelling. This is continued until the specimen cease to swell. The total load required to prevent swelling divided by the area of the sample defines the swelling pressure. (Mckeen, 1976).

ii. Swell-Consolidation test

In this test the sample under a 6.9kpa applied load is wetted and allowed to fully swell. At this point a standard consolidation test is conducted by applying incremental loads starting with 25kpa and ending with 1600kpa. The pressure required to revert the specimen to its initial void ratio (height) is used to define the swelling pressure. (Mckeen, 1976).

iii. Restrained swell test

This test consists of successively increasing the load on the specimen allowing it to attain equilibrium deformation at each pressure level. At a prescribed applied pressure, the sample is inundated and permitted to fully swell. The process is repeated with various inundation pressures on identical samples. Here the swell potential is calculated as the ratio of maximum expansion to the sample initial height. The pressure resulting in no expansion defines the swelling pressure. (Mckeen, 1976)

iv. Double-odometer test

The test involves a pair of nominally identical specimens. The first is loaded in the as- compacted state by incremental vertical pressure with equilibrium deformation recorded at each pressure level. The second specimen is fully inundated with no seating load and the maximum swell is recorded. The difference between the percent changes in specimen height at each stress level is used to define specimen height at each stress level is used to define the swell potential. Furthermore, the stress at which the percent settlement of the firs specimen equals the percent swell of the second specimen i.e. Difference equals zero, is used as a measure of swell pressure. (Mckeen, 1976)

3.4 Mechanics of swell

Soil volume change result from an imbalance in internal energy of the system (soil/ water/ plants/ air). Energy imbalances important in engineering result from moisture movement caused by loads, desiccation, and temperature changes. Response to a specific set of conditions is determined by the composition, structures, and geologic history of the soil. The largest component of volume change is that of the clay micelle which surrounds the individual clay particles in the soil. Water is forced out of the micelle by loads, desiccation, or temperature along energy gradient and reduction in volume results. When these influences are removed or reduced, the energy gradients are reversed, the available water is forced into the clay micelle and swell is produced. (Mckeen, 1976).

The natural micro scale mechanisms, which contribute the major portion of volume changes in expansive soils, are (Snethen, 1975) Osmotic repulsion: it is a pressure gradient developed in the double-layer water due to variations in the ionic concentration in the double layer.

Clay particle attraction: as clay particles possess a net negative charge on their surfaces and edges which result in attractive forces for various cations and in particular for dipolar molecules such as water.

Cation hydration: it is physical hydration of cations substituted into or attached to the clay particles.

Capillary imbibition: it is a movement of water into a mass of clay particles resulting from surface tension effects of water and air mixtures in the pores of the clay mass.

3.5 Factors influencing swelling and shrinking of a soil

The factors influencing the shrink swell potential of a soil can be considered in three different groups.

1. Soil characteristic that influence the basic nature of the internal force field. These includes
   - Clay mineralogy
   - Soil water chemistry
   - Soil suction
   - Plasticity
   - Soil structures and fabrics
   - Dry density
2. The environment factor that influence the changes that may occur in the internal force system. These include.
   - Initial moisture condition
   - Moisture variation
   - Climate
   - Ground water
   - Drainage and manmade water source
   - Vegetation
   - Permeability
   - Temperature

3. State of stress, which includes
    - Stress history
    - Surcharge load
    - Soil profile

As swelling pressure is the built in property of expansive soil and will not affected by Placement condition or environmental condition, only initial dry density and the amount and the type of clay mineral affect the swelling pressure [Chen, 1975, Yehyese.H, 2001].

3.6 Effect of initial dry density

The dry density is an important factor in determining the magnitude of volume change. The swell or the swelling pressure of an expansive soil increases with increasing dry density for constant moisture content. The reason is that higher densities result in closer particles spacing, therefore causing greater particle interaction.

Chapter Four

4.0 LABORATORY TEST RESULTS AND DISCUSSIONS

4.1 General

In this chapter laboratory test results are presented and their analysis is briefly discussed. The relevant engineering property of the soil is evaluated both for natural and stabilized soil samples separately. The tests include Atterberg limits, free swell, compaction, UCS, One dimensional swell potential and CBR. All the tests were conducted on black clay soil mixed with different percentage of pumice material.

4.2 Properties of Materials Used in the Study

4.2.1 Natural Soil (Black cotton Soil)

Soil samples were collected from Nifasilk Lafto Sub City around, Koyefech (E-0479583, N-0984817) and Mekanisa (E-0470218, N-0990702) areas where Black cotton soil is found. One test pit was excavated at each locations and disturbed samples were taken. Disturbed samples were air dried to constant moisture and sieved with different sieve sizes after pulverizing depending on the requirement of specific test procedures.

Laboratory tests were conducted on disturbed and remolded soil samples. Tests such as Atterberg limits, particle size analysis, linear shrinkage, free swell, moisture-density, CBR, remolded UCS and remolded swelling potential were conducted on disturbed samples.

From the summary of test results, the natural soil can be characterized as highly expansive and high plastic. On the particle size distribution curve almost 94 % of the soil is passing through No. 200 sieve for Koyefeche soil sample and 88.2% for Mekanissa Soil Sample; it exhibits a liquid limit of 86% for Koyefech and 84% for Mekanissa soil samples and a plasticity index of 43% for Koyefech soil sample and 40% for Mekanissa soil sample. Hence, these values indicate that the soil is highly plastic clay. Accordingly both soil samples falls under the A-7-5 soil class based on AASHTO soil classification system.

4.2.2 Pumice used for blending.

Pumice material has been used in this thesis work which was found from Nazareth which is 100km from Addis Ababa. As it was mentioned in the previous sections of this thesis study the pumice material is blended with black cotton soil with a ratio of 10% to 60%. The black cotton soil were collected from Koyefeche and Mekanissa areas.

From the summery of test result for the pumice soil Atterberg limits test of non-plastic (NP), Optimum moisture content of (OMC) 40.2%, maximum dry density (MDD) 0.795% and CBR value and CBR-Swell test of 50% and 0.8% respectively.

4.3 **Laboratory Test Results of Native soil and Blended Soil.**

4.3.1 Effect of Pumice on Particle Size Analysis

This method covers the determination of the particle size distribution of fine and coarse aggregates by sieving. Air dried sample passing 4.75mm sieve was used for particle size analysis. Accurate determination of material finer than the 75 micrometer (No. 200) sieve cannot be achieved by use of this method alone. ASTM/AASHTO T-27 was employed. The test was conducted According to ASTM C 136-84a/AASHTO T-27 and the results obtained are plotted below. The summary of the test results is shown in the Fig 4.1 below and the laboratory test analysis are given in Appendix 4 and 8.

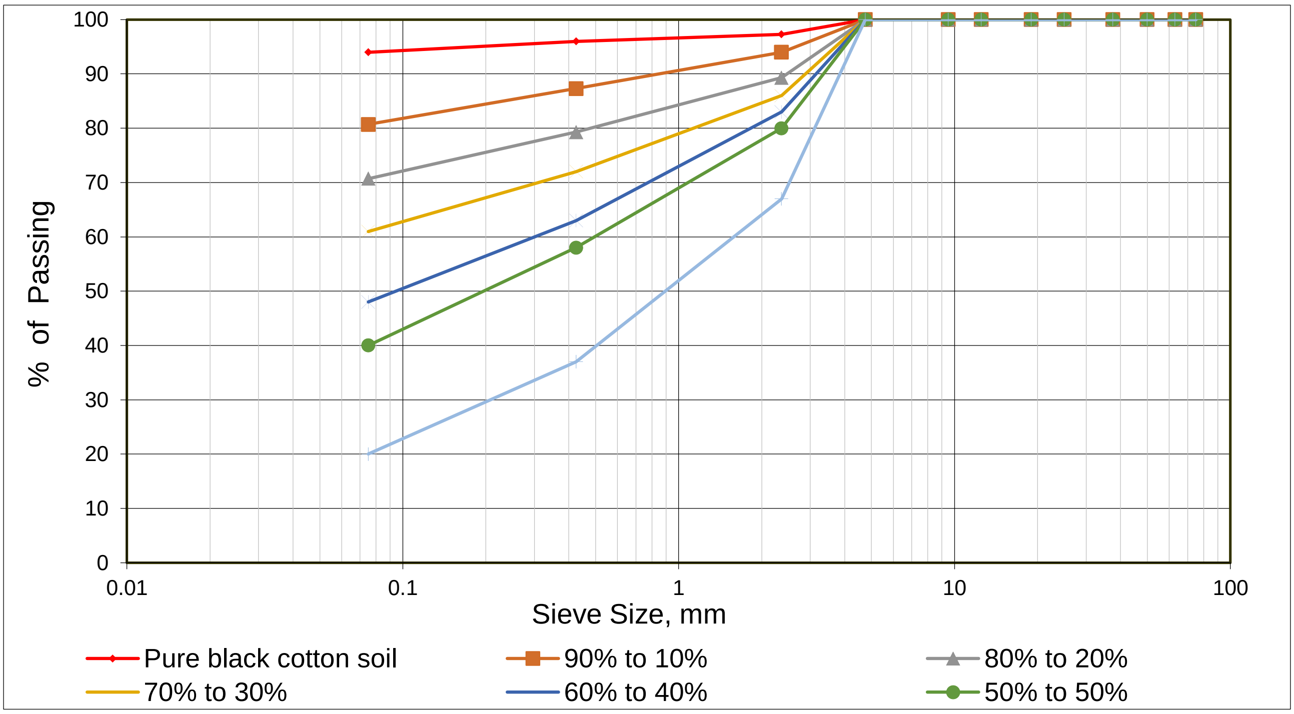


Fig. 4.1 Particle Size distribution for Koyefechi Test pit

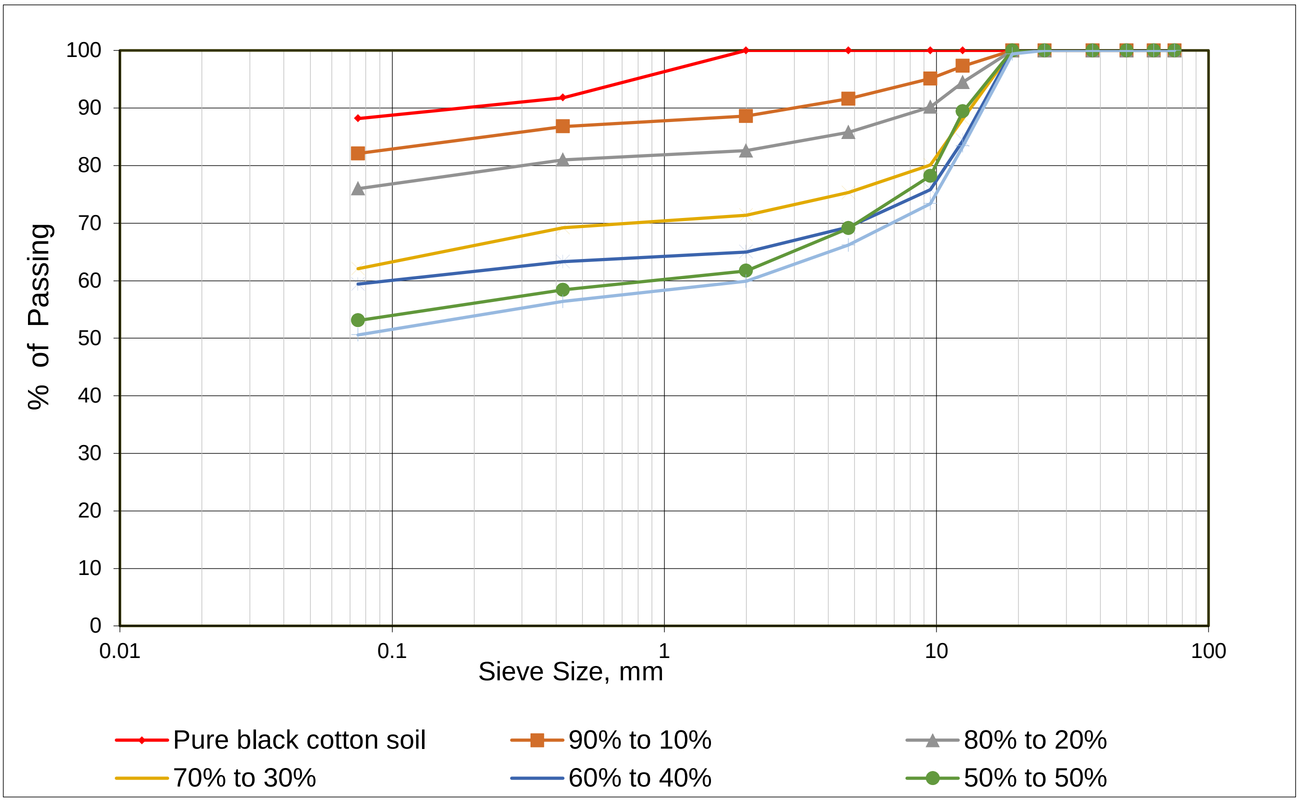


Fig. 4.2 Particle Size distribution for Mekanisa Test pit.

4.3.2 Effect of Pumice on Atterberg limits and linear shrinkage

The Atterberg limits are basic measures of the critical water contents of a fine-grained soil its shrinkage limit, plastic limit, and liquid limit. As a dry, clayey soil takes on increasing amounts of water, it undergoes distinct changes in behavior and consistency. Depending on the water content of the soil, it may appear in four states: solid, semi-solid, plastic and liquid. In each state, the consistency and behavior of a soil is different and consequently so are its engineering properties. The test was performed in accordance to AASHTO T-89 and T-90.

Shrinkage due to drying is significant in clays, but less so in silts and sands. If the drying process is prolonged after the plastic limit has been reached, the soil will continue to decrease in volume. The linear shrinkage is a way of quantifying the amount of shrinkage likely to be experienced by clay materials. Such a value is also relevant to the converse condition of expansion due to wetting. The test was carried according to test method of BS 1377: part 2: 1990. The summary of the test result is tabulated below and the laboratory test analysis are given in Appendix 5 and 9.

Table 4.1 Summary of Atterberg Limits and linear shrinkage for Koyefech Test pit.

No	Stabilization Description	Liquid limit, %	Plastic limit, %	Plastic index, PI, %	Linear Shrinkage LS, %
1	Pure black cotton soil	86	43	43	20
2	90% to 10% (black cotton to pumice	84	43	41	19
3	80% to 20% (black cotton to pumice)	83	43	40	16
4	70% to 30% (black cotton to pumice)	78	40	38	14
5	60% to 40% (black cotton to pumice)	73	39	34	12
6	50% to 50% (black cotton to pumice)	67	35	32	11
7	40% to 60% (black cotton to pumice)	63	36	27	7

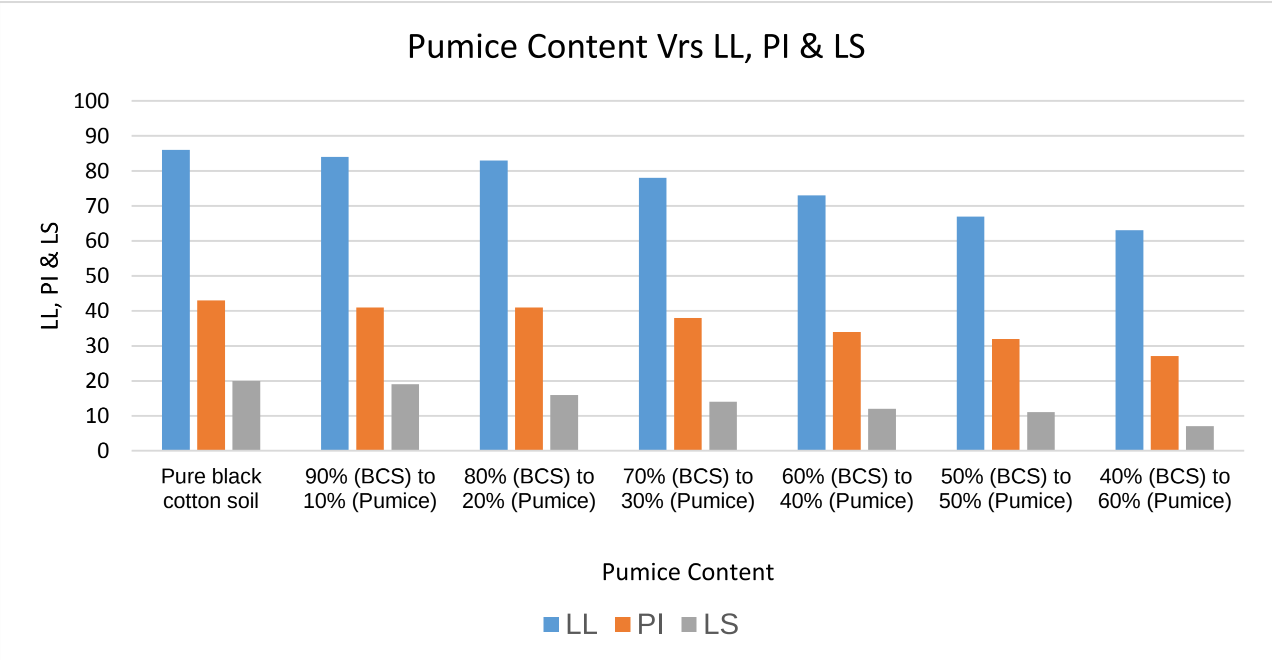


Fig. 4.3 Pumice content vs LL, PI & LS for Koyefeche Test Pit.

Table 4.2 Summary of Atterberg Limits and linear shrinkage for Mekanisa Test pit.

No	Stabilization Description	Liquid limit, %	Plastic limit, %	Plasticity index, PI, %	Linear Shrinkage LS, %
1	Pure black cotton soil	84	44	40	19
2	90% to 10% (black cotton to pumice	81	41	40	18
3	80% to 20% (black cotton to pumice)	76	40	36	16
4	70% to 30% (black cotton to pumice)	71	37	34	15
5	60% to 40% (black cotton to pumice)	68	35	33	13
6	50% to 50% (black cotton to pumice)	62	31	31	10
7	40% to 60% (black cotton to pumice)	63	32	31	10

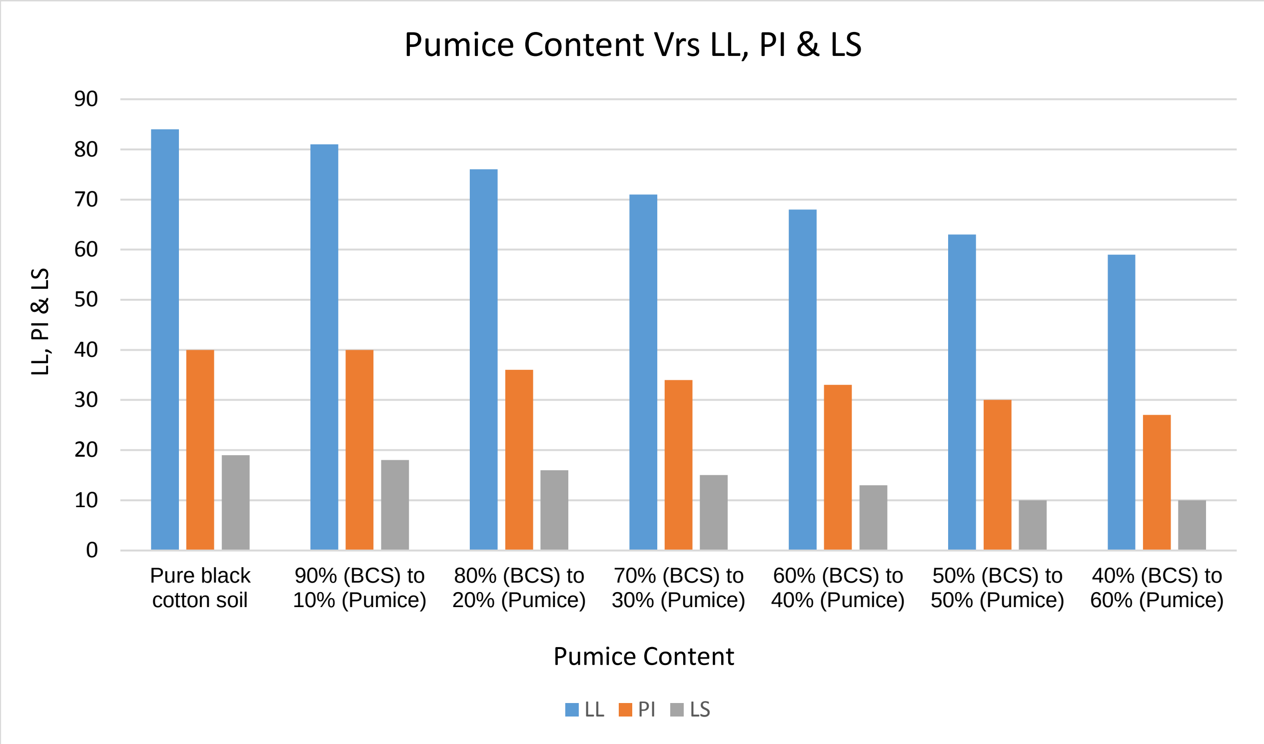


Fig. 4.4 Pumice content vs LL, PI & LS for Mekanisa Test Pit.

The variation of plasticity index and liquid limit for soil-pumice mix with different percentages are shown in the Figure above for both soil samples collected from the corresponding study areas. It is observed that the liquid limit and plastic index decrease with increase in pumice content for soil-pumice stabilized samples. The liquid limit decrease for soil sample collected from Koyefeche area from 100% to 84% and the plastic index decrease from 100% to 63% whereas the liquid limit decreases from 100% to 75% for the soil sample collected from Mekanissa area and the plastic index decrease from 100% to 78%. However, from the test result the rate of decrease is not significant.

4.3.3 Effect of Pumice on Free Swell

Swelling tendency were also determined from the samples passing 425 micrometer sieve and oven dried. The 10ml of soil sample was put in water for 24 hours and swelling was examined as percentage of volume change to the original volume. The results obtained are as follows.

Table 4.3 Summary of Free Swell for Koyefech and Mekanisa Test pits.

No	Stabilization Description	Free Swell (Koye Fech Test pit)	Free Swell (Mekanisa Test pit)
1	Pure black cotton soil	130	120

2	90% to 10% (black cotton to pumice)	100	110
3	80% to 20% (black cotton to pumice)	90	90
4	70% to 30% (black cotton to pumice)	80	80
5	60% to 40% (black cotton to pumice)	70	70
6	50% to 50% (black cotton to pumice)	60	50
7	40% to 60% (black cotton to pumice)	50	30

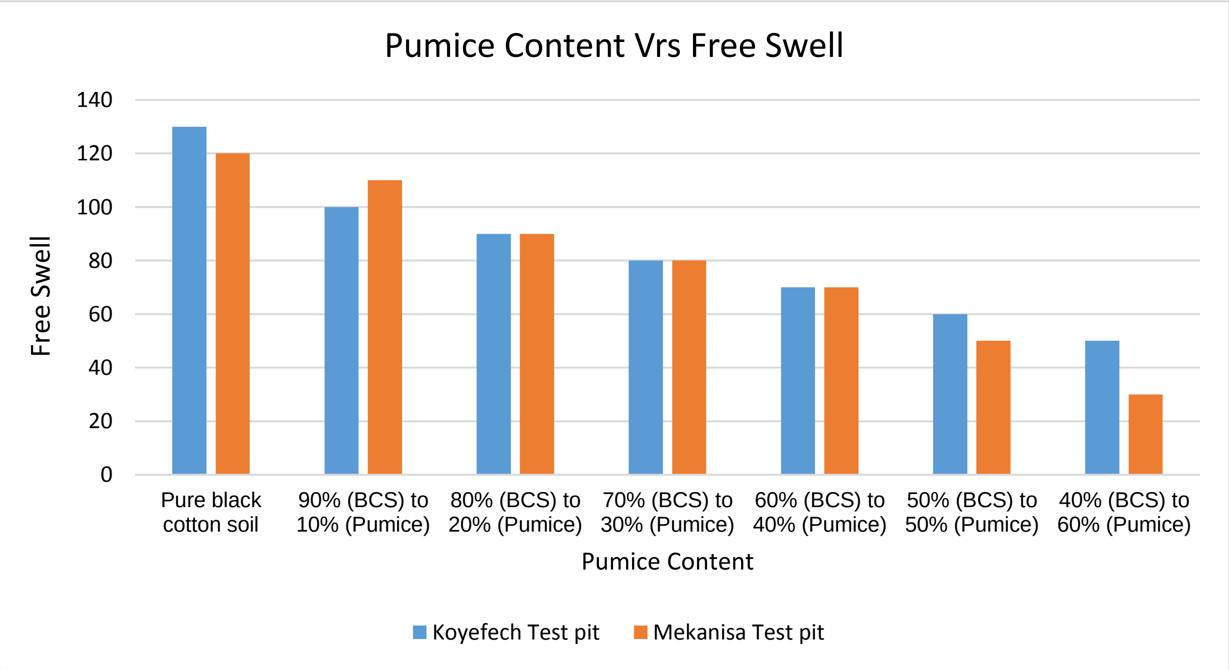


Fig. 4.5 Pumice content vs free swell for both test pits.

The effect of pumice on the free swell of the soil is shown in the figure 4.5, the application of pumice results in modest reduction in the free swell of the soil. The reduction in free swell is directly proportional to the quantity of pumice added to the soil. The highest reduction in free swell is attained when the expansive soil is treated with 60% of pumice for both soil samples i.e. Pumice reduces the heave by 38% for Koyefeche soil sample and by 25% for Mekanissa soil sample from its natural state (130%) and (120%) for Koyefeche and Mekanissa soil sample respectively. The free swell index gradually decreased from 130% to 50% and from 120% to 30% for up to 60% pumice content for the Koyefe and Mekanissa soil soil sample respectively. This reduction in free swell is caused due to the reduction of water absorbing clay particles in the soil-pumice mix.

4.3.4 Effect of Pumice on Modified compaction

Modified proctor compaction test which simulates heavy compacting effort were used to obtain the moisture-dry density relationship of the specific soil samples accordance to AASHTO T-180 method in a 4 inches diameter mold. The soil were compacted with different moisture content in five layers each suffering 25 blows and with different pumice ratio mix with an increment of 10%. After obtaining the density and moisture of the each compacted soil sample, the following relationships for dry density and moisture content were obtained.

Figure 4.7 and 4.9 shows the variation of MDD of expansive soil with addition of different percentage of pumice material. MDD goes on decreasing irrespective of the increase in percentage addition of pumice. The maximum reduction of MDD for pumice treated soil is 17% and 32% for Koyefeche and Mekanissa soil sample respectivly which was obtained for 60% of pumice stabilized clay soil. Generally the maximum dry density of soil decreases gradually with an increase of pumice content for both soil samples. This is due to comparatively low specific gravity and light weight behavior of pumice material. Pumice (with lower specific gravity) fills the soil voids and it contributes to a decrease in density.

Figure 4.6 and 4.8 shows the variation of OMC of expansive soil with addition of different percentage of pumice material. The OMC of treated soil with pumice decreases from 30% to 22.4% and from 29% to 19% for Koyefeche and Mekanissa sample respectively with increased in pumice content from 0% to 60%. Generally the OMC of the soil decreases with an increase of pumice content for all soil samples collected from the corresponding study area. The decrease of optimum moisture content is caused by the decrease in water absorbing clay particles as the pumice content increases in the soil-pumice mix. The summary of the test result is tabulated below and the laboratory test analysis are given in Appendix 6 and 10.

Table 4.4 Summary of OMC and MDD for Mekanissa Test pit.

No	Stabilization Description	OMC, %	MDD, g/cc	Wet density, g/cc
1	Pure black cotton soil	29	1.42	1.720
2	90% to 10% (black cotton to pumice)	27	1.27	1.625
3	80% to 20% (black cotton to pumice)	25	1.23	1.588
4	70% to 30% (black cotton to pumice)	23	1.21	1.493

5	60% to 40% (black cotton to pumice)	21	1.16	1.391
6	50% to 50% (black cotton to pumice)	19	1.11	1.385
7	40% to 60% (black cotton to pumice)	19	0.97	1.345

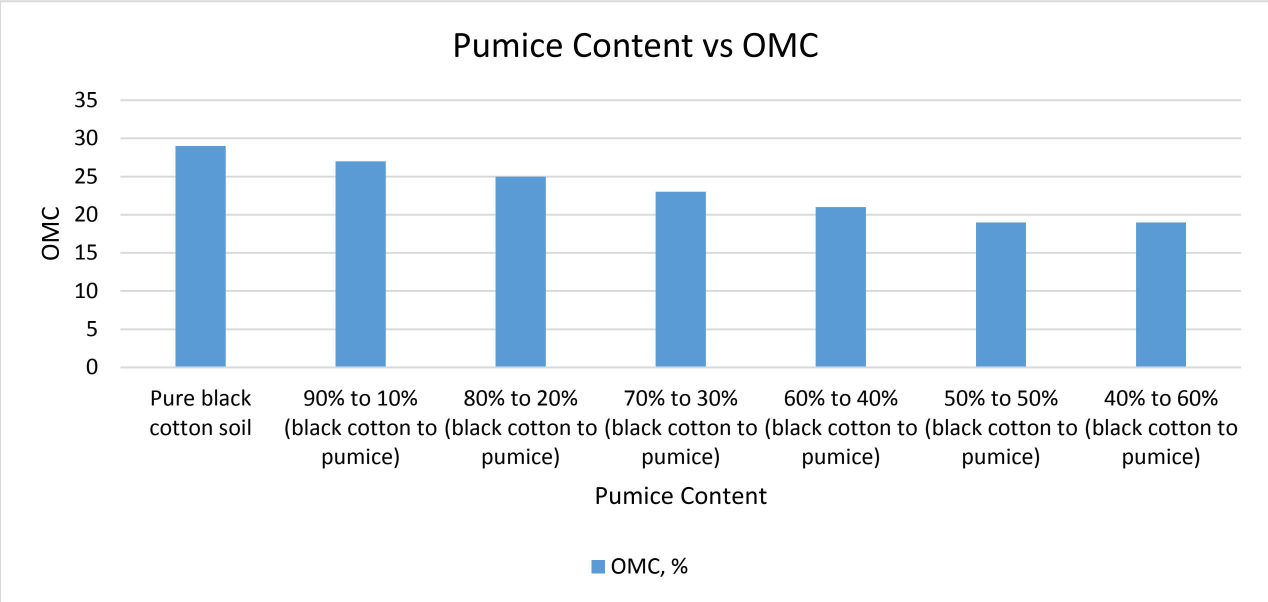


Fig. 4.6 Pumice content vs OMC for Mekanissa Test pit.

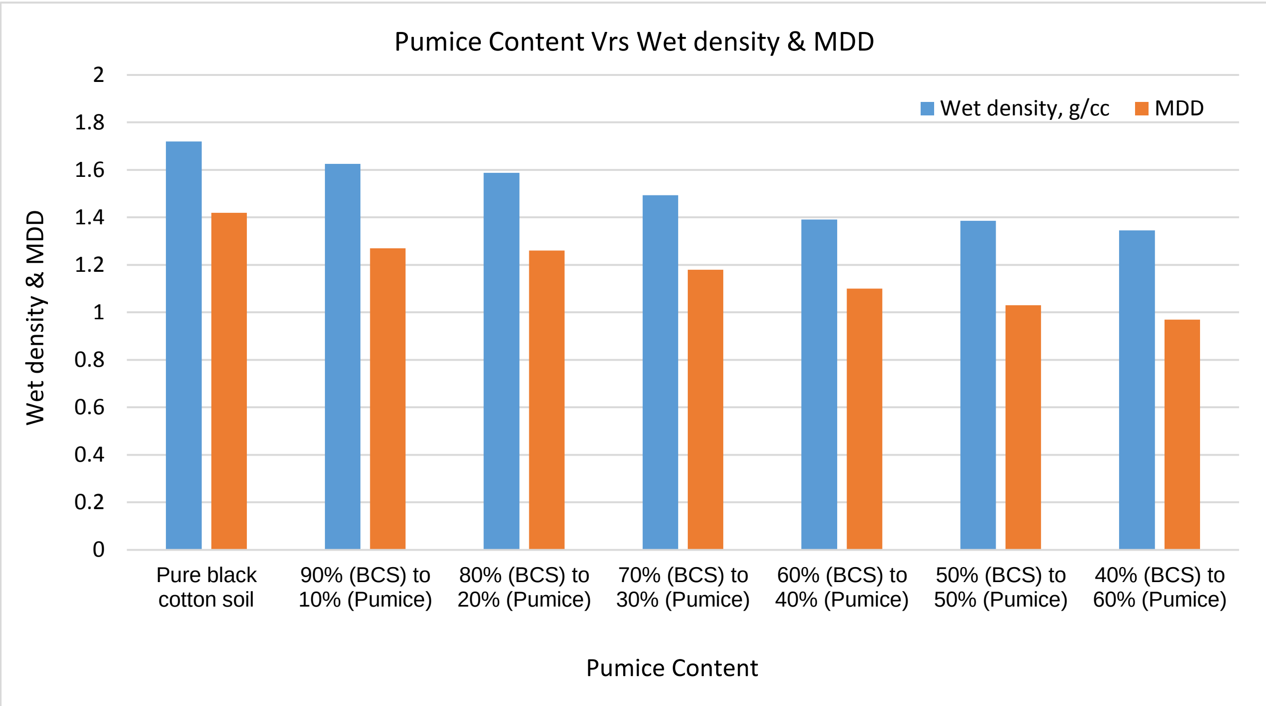


Fig. 4.7 Pumice content vs Wet density & MDD for Mekanissa Test pit

Table 4.5 Summary of OMC and MDD for Koyefech Test Pit.

No	Stabilization Description	OMC, %	MDD, g/cc	Wet density, g/cc
1	Pure black cotton soil	30.0	1.32	1.72
2	90% to 10% (black cotton to pumice)	28.6	1.26	1.63
3	80% to 20% (black cotton to pumice)	27.5	1.23	1.59
4	70% to 30% (black cotton to pumice)	24.8	1.20	1.49
5	60% to 40% (black cotton to pumice)	24.4	1.13	1.39
6	50% to 50% (black cotton to pumice)	23.6	1.13	1.39
7	40% to 60% (black cotton to pumice)	22.4	1.10	1.34

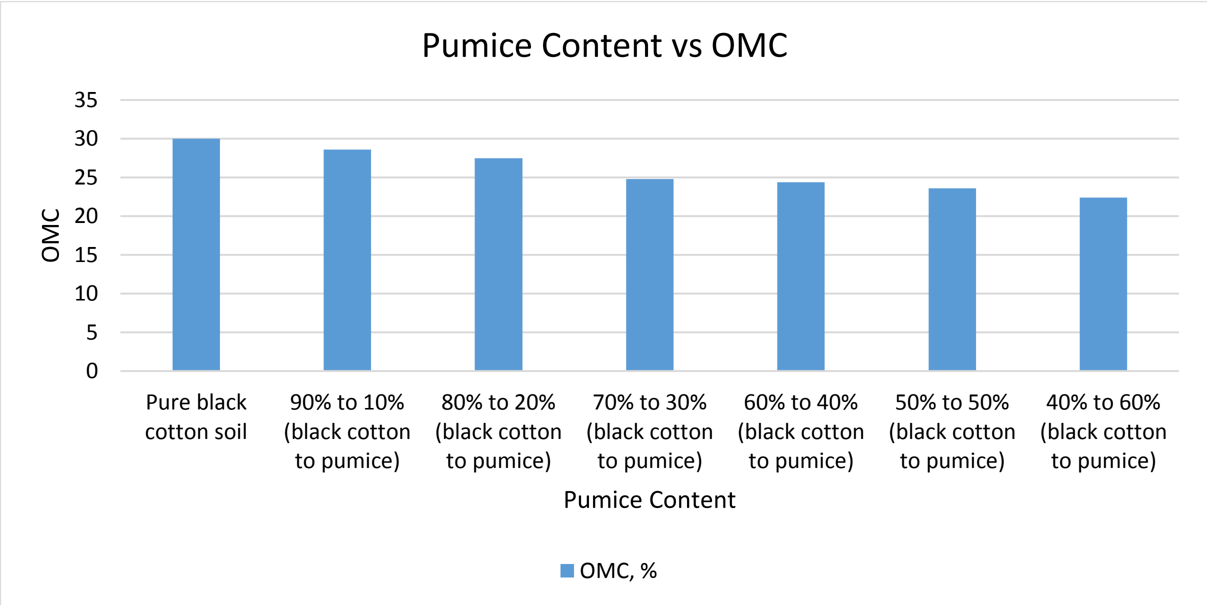


Fig. 4.8 Pumice content vs OMC for Koyefech Test pit

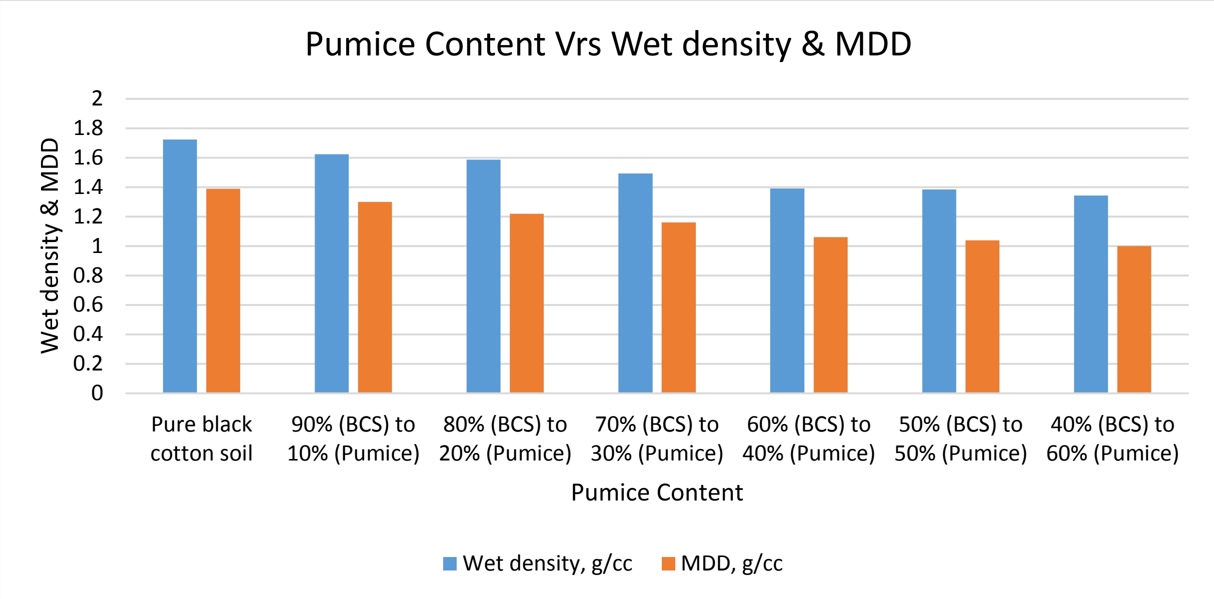


Fig. 4.9 Pumice content vs Wet density & MDD for Koyefech Test pit

4.3.5 Effect of Pumice on California Bearing Ratio (CBR)

The CBR is a measure of shearing resistance of the material under controlled density and moisture conditions. The CBR test in this thesis work were done according to AASHTO T-193. The CBR value for 2.54mm and 5.08mm are recorded. This load is expressed as a percentage of standard load value at a respective deformation level to obtain CBR value. One point CBR test have been done for all samples to determine the strength character of the black cotton soil alone and in the stabilized case. The density versus CBR were plotted and the CBR for 56 blows is determined from the graph of maximum dry density. The summary of the test result is tabulated below and the laboratory test analysis are given in Appendix 7 and 11.

The variations of California Bearing Ratio (CBR) with different percentage of soil and pumice combinations are shown in Figures 4.10 and 4.11 for both soil samples collected from Koyefech and Mekanissa respectively. The maximum California Bearing Ratio (CBR) value of 28% and 11% for Koyefeche and Mekanissa respectively is found to occur with the combination of 60% pumice contents under soaked condition. Even though CBR value increases, the rate of increment is not as high as the percent of pumice increment in the soil-pumice mix. The load bearing capacity of the sample shows insignificant increment with pumice treatment and also curing has almost no effect on the soaked CBR values. Addition of cement or lime to the soil –pumice mix may improve better the soaked CBR value due to the generation of cementitious gel between the pumice and Calcium contained in the cement/lime.

The variation of CBR-Swell with different pumice content is shown in Figure 4.12. However, the CBR-Swell shows a decrease as the content of pumice material is increases in the soil-pumice mix for both soil samples collected from the study areas. But the decrease in the CBR-swell is achieved by adding a higher ratio of pumice content in the soil and this is not promising result.

Table 4.6 Summary of CBR and CBR Swell for Koyefech Test Pit

No	Stabilization Description	CBR at 93% of MDD	CBR Swell, %
1	Pure black cotton soil	0	13.44
2	90% to 10% (black cotton to pumice)	0	11.04
3	80% to 20% (black cotton to pumice)	0	9.22
4	70% to 30% (black cotton to pumice)	3	7.96
5	60% to 40% (black cotton to pumice)	5	2.80
6	50% to 50% (black cotton to pumice)	15	1.88
7	40% to 60% (black cotton to pumice)	28	1.42

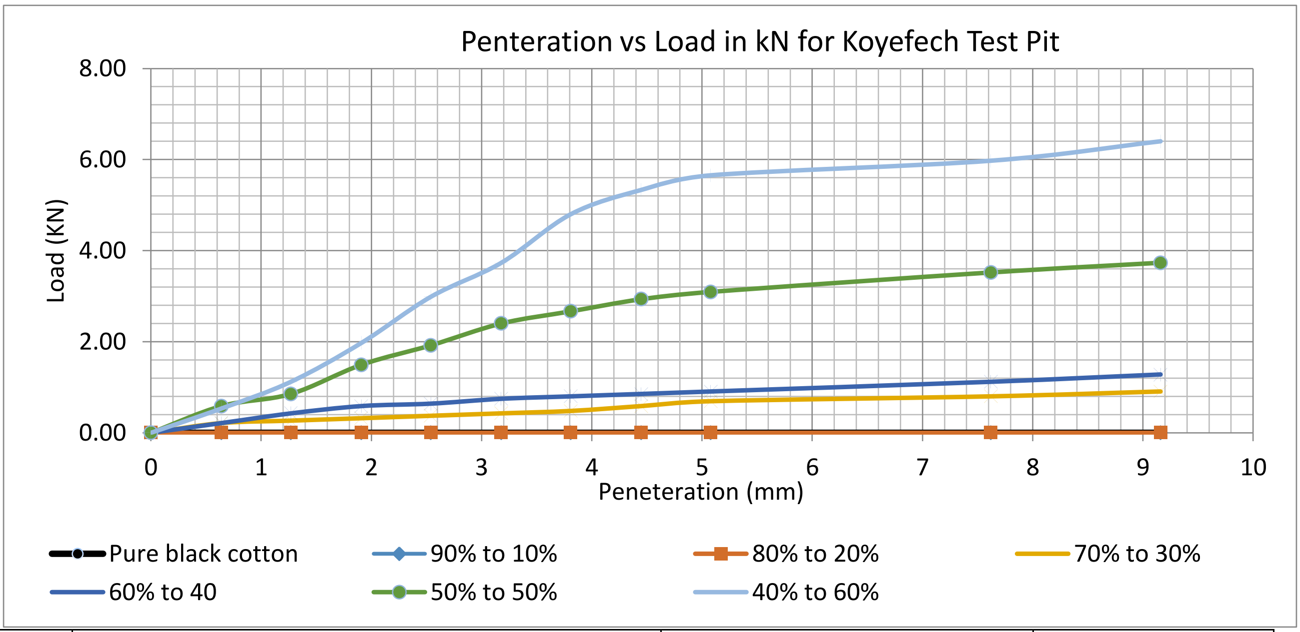


Fig. 4.10 CBR Test for Koyefech Test Pit.

Table 4.7 Summary of CBR and CBR Swell for Mekenisa Test Pit.

No	Stabilization Description	93% of MDD CBR, %	CBR Swell, %

1	Pure black cotton soil	2	11.85
2	90% to 10% (black cotton to pumice)	3	10.95
3	80% to 20% (black cotton to pumice)	4	10.34
4	70% to 30% (black cotton to pumice)	5	8.88
5	60% to 40% (black cotton to pumice)	6	5.48
6	50% to 50% (black cotton to pumice)	8	3.10
7	40% to 60% (black cotton to pumice)	11	2.40

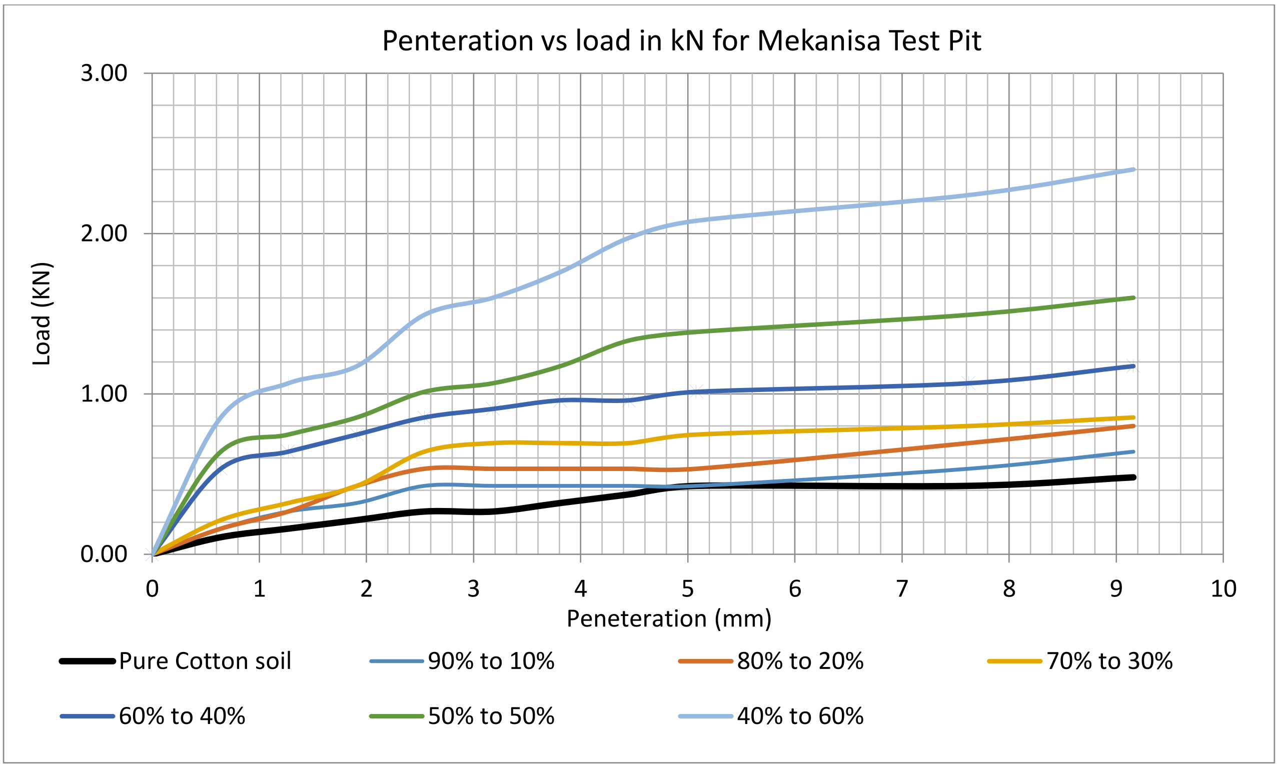


Fig. 4.11 CBR Test for Mekanisa Test Pit.

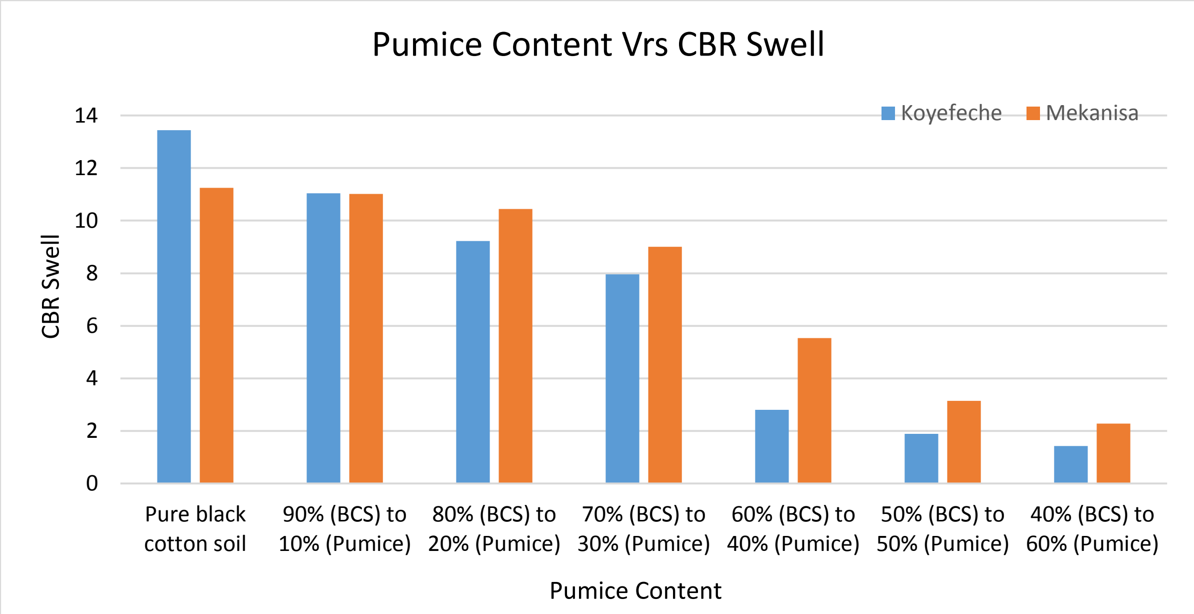
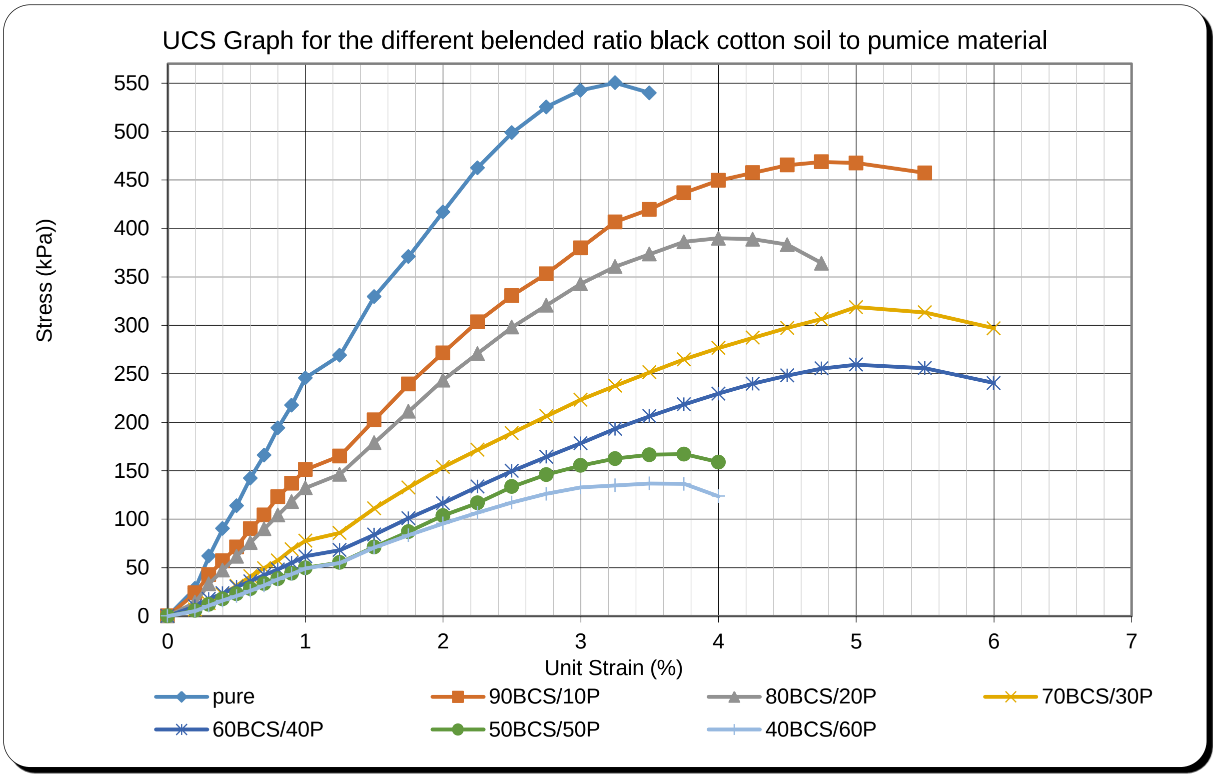


Fig. 4.12 CBR Swell vs. Pumice content for both test pits.

4.3.6 Effect of Pumice on Unconfined Compressive Strength (UCS)

The Unconfined Compressive Strength (UCS) test is an unconsolidated and undrained load test performed for determining the unconfined compressive strength of cohesive soil samples where the lateral confining pressure is equal to zero during the test. UCS is the compressive stress at which an unconfined cylindrical soil sample fails in a load test. UCS is taken as the maximum load attained per unit area during loading. For UCS test specimens height-to-diameter ratio shall be between 2 and 2.5, the shear strength were calculated to be half of the compressive stress at failure. The UCS tests were performed on both un-stabilized and pumice stabilized soil materials which are blended at different ratios. The tests were carried out according to ASTM Standard D2166.

Unconfined compressive strength test result is shown in Figures 4.13 to 4.14. These figures illustrate the stress-strain behavior of original and pumice treated soil. Initially the stress is gradually increases with the increase of strain. After attaining the peak stress, it decreases with the increase of strain for all the combination of pumice and soil. All the specimen shows shear failure. The variation of unconfined compressive strength for soil at different percentages of pumice is also shown in Figure 4.15. There is a rapid decrease of unconfined compressive strength from 267kPa to 68.5kPa for Koyefeche soil sample and from 268.5kPa to 63.34kPa for Mekanissa soil sample with the addition 60% pumice stabilizer un-socked soil-pumice mix. May this result be changed if the socked condition were done because of the curing of cementitious nature of pumice when the minerals combine with water particles. The reason for the decrease for the unconfined compressive strength of the soil-pumice mix is due to the fact that the addition of more cohesion less material to the clay soil reduce its natural cohesive force between the particles of clay soil. The summary of the test result is shown in the Fig. below and the laboratory test analysis are given in Appendix 12 and



13.

Fig. 4.13 Unit Strain Vs Stress for Koyefeche soil Sample

Table 4.8 Summery of UCS Values for Koyefech Sample

No	Stabilization Description	UCS (kPa)
1	Pure black cotton soil	552
2	90% to 10% (black cotton to pumice	469

3	80% to 20% (black cotton to pumice)	389
4	70% to 30% (black cotton to pumice)	315
5	60% to 40% (black cotton to pumice)	260
6	50% to 50% (black cotton to pumice)	170
7	40% to 60% (black cotton to pumice)	137

Table 4.9 Summery of UCS Values for Mekanissa Sample

No	Stabilization Description	UCS (kPa)
1	Pure black cotton soil	536.5
2	90% to 10% (black cotton to pumice	450.56
3	80% to 20% (black cotton to pumice)	362.36
4	70% to 30% (black cotton to pumice)	311.3
5	60% to 40% (black cotton to pumice)	250.06
6	50% to 50% (black cotton to pumice)	163.7
7	40% to 60% (black cotton to pumice)	126.68

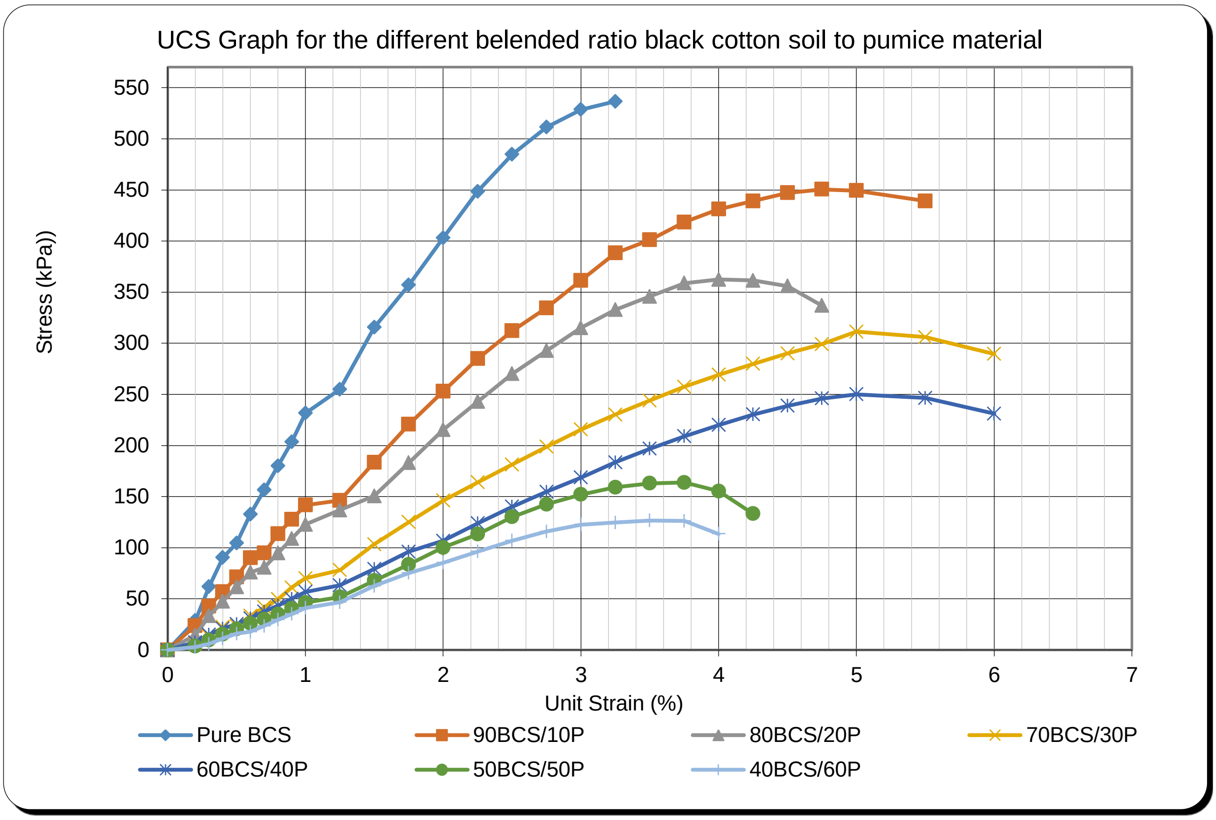


Fig. 4.14 Unit Strain Vs Stress for Mekanissa soil Sample

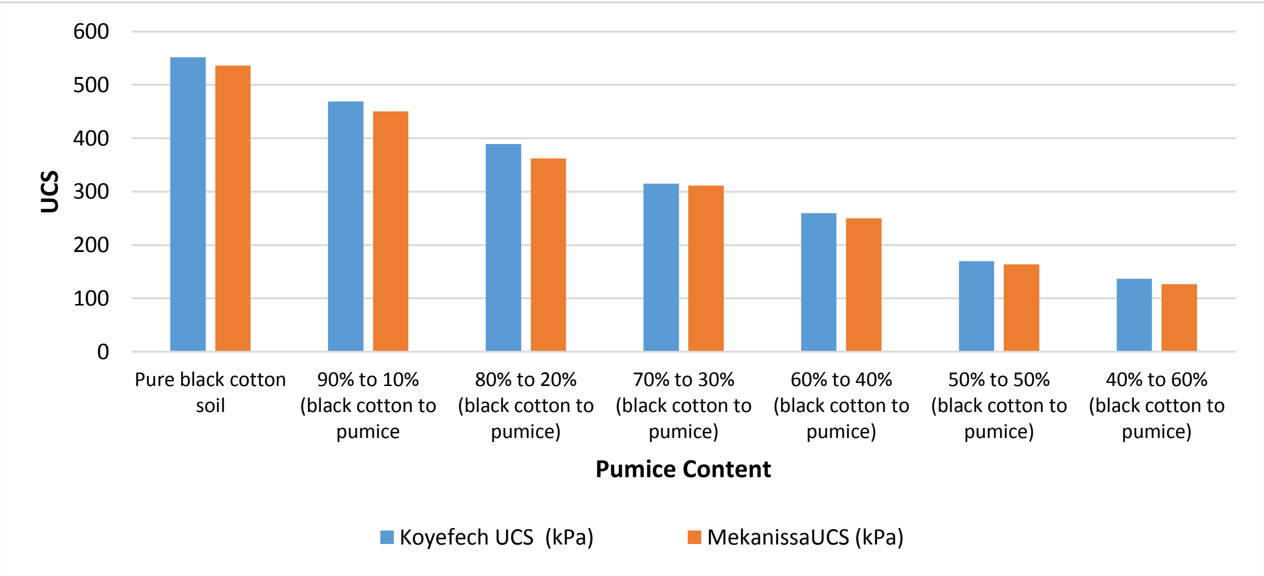


Fig. 4.15 Variation of UCS with pumice content for both study areas

4.3.7 Effect of Pumice on Swelling Pressure (One dimensional Swell)

The soil swell/collapse strains measured from these test methods can be used to develop estimates of heave or settlement for a confined soil profile subject to one-dimensional heave or settlement, or stress-induced settlement following wetting-induced heave/settlement. They can also be used to estimate the pressure that would be necessary to prevent swelling. Selection of test method, loading, and inundation sequences should, as closely as possible, simulate field conditions because relatively small variations in unit weight and water content, or sequence of loading and wetting can significantly alter the test results. One dimensional swell tests were performed on both unstabilized and pumice stabilized soil materials which are blended at different ratios for both study areas as mentioned above. The tests were carried according to ASTM Standard D4546. The summary of the test result is tabulated below and the laboratory test analysis are given in Appendix 14 and 15.

The one dimensional swelling pressure as shown in Figure 4.17 decreases with increase in pumice treatment. The swelling pressure decreased from 400kPa at 0% pumice content for both soil sample to 100kPa and 125kPa at 60% pumice content for Koyefeche and Mekanissa soil samples respectively. This implies that as more pumice is added, the lesser the swelling potential of the treated soil.

Table 4.10 One dimensional swell Test Result for Koyefeche Soil Sample

No	Stabilization Description	Swelling Potential (kPa)	Dry density (g/cc)	Final Void ratio (%)
1	Pure black cotton soil	400	1.230	1.177
2	90% to 10% (black cotton to pumice)	350	1.264	1.052
3	80% to 20% (black cotton to pumice)	300	1.226	1.078
4	70% to 30% (black cotton to pumice)	250	1.137	1.233
5	60% to 40% (black cotton to pumice)	200	1.056	1.358
6	50% to 50% (black cotton to pumice)	150	1.012	1.430
7	40% to 60% (black cotton to pumice)	100	0.981	1.455

Table 4.11 One dimensional swell Test Result for Mekanissa Soil Sample

No	Stabilization Description	Swelling Potential (kPa)	Dry density (g/cc)	Final Void ratio (%)
1	Pure black cotton soil	400	1.370	0.935
2	90% to 10% (black cotton to pumice)	400	1.400	0.864
3	80% to 20% (black cotton to pumice)	325	1.234	1.091
4	70% to 30% (black cotton to pumice)	275	1.145	1.209
5	60% to 40% (black cotton to pumice)	200	1.109	1.237
6	50% to 50% (black cotton to pumice)	150	1.062	1.289
7	40% to 60% (black cotton to pumice)	125	1.030	1.330

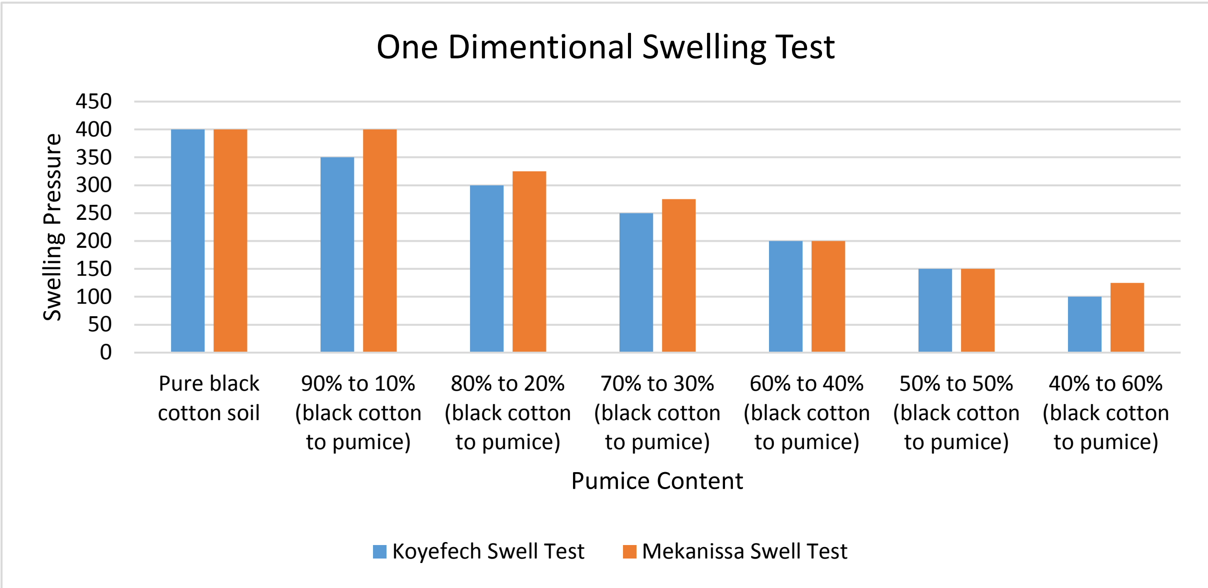


Fig. 4.16 Variation of one dimensional swelling pressure with the increment of pumice ratio for Koyefeche and Mekanissa soil sample

Chapter 5

Conclusions and Recommendations

5.1 Conclusions

Based on the test results obtained from the investigation of the selected soil treated with Pumice material the following conclusions can be drawn. In the conclusion part each stabilized clay material is compared with that of the pure expansive clay soil (non-stabilized state) and hence, all figures are expressed in percentage to show clearly the extent of pumice Stabilization.

1. Atterberg limit shows reduction as the ratio of pumice increases in the mechanical stabilization process. The liquid limit is reduced by 27% (from 100% to 73%) for Koyefeche soil sample and by 30% (from 100% to 70%) for soil sample collected from Mekanisa area. The plastic index also reduced by 37% (from 100% to 63%) for soil sample collected from Koyefeche area and by 32% (from 100% to 68%) for soil sample collected from Mekanisa area as the ratio of the pumice material increased by 10%. As a result of pumice material has a non-plastic nature a decrease in liquid limit and plastic index is observed as the pumice content increase in the soil-pumice mix.

2. The linear shrinkage of the soil samples collected from Koyefech area is reduced by 65% (from 100% to 35%) and by 47% (from 100% to 53%) for Mekanissa soil sample. However, as the linear shrinkage test result of the stabilized expansive clay soil is compared with table 3.3 (Altmeyer Method of classification of expansive soil) in chapter three, for koyefech soil sample the degree of expansion fall in marginal and for that of the Mekanissa soil sample it falls in the critical degree of expansion.
3. The free swell test for both study area shows a decrease in the range of 130% to 30% as the pumice content increases from 0% to 60%. Hence, this indicates that the addition of pumice material in expansive clay soil reduces the heaving potential of the soil.
4. However, the compaction test result shows a reverse observation as the above. As the ratio of pumice increases in the soil mix for both soil samples collected from Koyefeche and Mekanisa, the samples show a decrease in the maximum dry density. This is due to the fact that pumice has low density or due to its light weight behavior. Moreover, the void ratio also shows an increase as the ratio of pumice material added to the expansive clay soil increased, this phenomena is occurred due to the fact that pumice is produced when lava with a very high content of water and gases is discharged from a volcano. As the gas bubbles escape, the lava becomes frothy. When this lava cools and hardens, the result is a very light rock material filled with tiny bubbles of gas. Hence, more voids are available in the

pumice, then as the ratio of pumice material added to the expansive clay soil increases the void space in the mix (soil to pumice) is increased and it is difficult to reduce the void space during remolding process (compaction). Therefore, due to this reason it is observed that on the one dimensional swell test the void ratio is decreased only for 90% to 10% (black cotton to pumice) mix ratio for both soil samples collected from the corresponding study area. But, for the rest of the trials as the pumice percentage increases in the mix the void ratio also shows an increment.

5. Laboratory test was carried out to estimate the CBR value for soil samples collected from Koyefeche and Mekanisa Test pits. Accordingly, the test result shows small increment of CBR value for both soil samples collected from the corresponding areas which were treated with different incremental percentage of pumice material. However, the result shows that the increase in CBR value is small after the expansive clay soil is treated with pumice material.

6. On the other hand CBR-swell value for both samples shows a significant reduction as the ratio of pumice material increase. The CBR-Swell for Koyefeche soil sample reduced by 89% (from 100% to 11%) and that of the Mekanisa soil sample reduced by 80% (from 100% to 20%). The free swell test result also reduced by 62% (from 100% to 38%) for Koyefeche soil sample and by 75% (from 100% to 25%) for Mekanissa soil sample. This reduction in CBR-Swell and free swell is caused by the minimization of clay particles which are capable of absorbing large quantities of water (such as Monmorillonite) and they partly replaced by non-water absorbing minerals. This result indicates that blending of pumice material to expansive clay soil minimize the heaving tendency which occurs due to seasonal moisture variations.
7. From UCS test, it is observed that the value of the test result decreases for each 10% increment of pumice material added to the expansive clay soil samples collected from both study areas. For Koyefeche soil sample the UCS value decreases by 75% and by 76% for Mekanissa soil sample. The decrease in UCS value of the black cotton soil is due to the fact that the expansive clay soil decrease in its ability to bind/cohesion as the percent of pumice material increases in the soil mix (more cohesion less material is added).
8. Furthermore, one dimensional swell test have been also carried on the soil samples collected from the corresponding study areas. One dimensional swelling pressure test result is reduced by 75% for Koyefech soil sample and by 69% for Mekanissa soil sample at 40% black cotton soil to 60% pumice material. However, the decrease in one dimensional swelling pressure is 50% at 60% black cotton soil to 40% pumice material ratio by weight. It can be concluded that in line with the test result the change in volume/heave in black cotton soil decreases as the ratio of pumice material increases in the process of stabilization.

9. From the above index properties, CBR test, CBR-Swell test, one dimensional test results and other pertinent tests done in this study, it is concluded that the change in the engineering property of the expansive clay soil to its reduction in heaving potential is caused at higher rate of pumice content mixed to the soil samples collected from the study areas.
10. In general, all the above conclusions was reached using naturally occurring pumice material passing 4.75mm sieve size and crushed partially during the process of compaction. However, if the pumice material in this thesis study has been used as a form of powder pumice the reaction between the pumice material and black cotton soil may speeded up and the cementitious property of the pumice was maximized. Hence, the stabilization process will better and all the testes discussed above may be changed and a better result with a minimum pumice content will be expected.

5.1 Recommendations

The present work has attempted to obtain the optimum blending proportion of Pumice material with expansive clay soil (Black cotton soil) which results a reduction in the swelling behavior of these soil samples collected from certain portion of Addis Ababa sub city around Nifasilk Lafto. However, due to financial constraints and time limitations the present research work did not cover the whole aspects of expansive soil treatment with pumice. In view of this, it would be desirable to consider the following recommendations for better use of pumice material as a stabilizing agent for expansive clay soils.

1. Pumice stabilization of expansive subgrades should be conducted throughout the country and the mix designs should be formulated. With the necessary investigations and results guidance can be developed for the country to mitigate serious hazards due to highly expansive soils.
2. During this thesis study, pumice was used as a stabilizing agent to expansive clay soil. Natural pumice material passing sieve size 4.75mm was used in this thesis study. On the process of testing of the soil-pumice mix, the pumice material was subjected to compaction effort which causes crushing of the pumice to a certain extent. Hence, all test results and conclusion were drawn based on partially crushed pumice material in the soil-pumice mix. The effect of crushed pumice mixed with black cotton soil was not conducted and determined. Hence, further investigation shall be carried out to determine the effect of pulverized pumice material to its stabilization property after mixing with expansive clay soil.
3. All teste results conducted in this thesis study was without curing the sample to a certain period of time except for that of CBR test. Hence, anyone who has the interest to study on pumice material as a stabilizing agent to expansive clay soil shall conduct by including the cured case.

4. As investigated in this research work Pumice is not an effective standalone mechanical stabilizer for highly plastic expansive soils. Therefore, it is recommended to study use of pumice by mixing with other admixture stabilizer.