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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Stabilization of Expansive Subgrade soil using Waste Ceramic Powder

(A case study on the road segment in Ethio ICT Village)

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A Thesis Submitted to

School of Graduate Studies in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

(Geotechnical Engineering)

ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

This is to certify that the thesis prepared by Bililign Firdawek entitled ‘Stabilization of Expansive Subgrade Soil Using Waste Ceramic Powder (A case study on the road segment in Ethio ICT Village)’, submitted in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering complies with the regulations of the University and meets the accepted standards with respect to originality and Quality.

Approved By the Board of Examiners.

Dr. - Ing. Samuel Tadesse.

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DECLARATION

I here by declare that the thesis entitled Stabilization of Expansive Sub-grade Soil Using Waste Ceramic Powder (A case study on the road segment in Ethio ICT Village) has been carried out by me under the supervision of Dr.-Ing. Samuel Tadesse, during the year 2019 as part of Master of Science Program in Geotechnical engineering. I further declare that this work has not been submitted to any other University or institution for the award of any degree. All quotations and their sources are specifically acknowledged by means of references.

Place: Addis Ababa

Bililign Firdawek.

2019 G.C

Table of Contents *Page*

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS

AASHTO	American Association of Highway and Transportation Officials
ASTM	American Society for Testing and Materials
BS	British Standard
CAH	Calcium aluminate hydrate
CEC	Cation Exchange Capacity
CBR	California Bearing Ratio
CSH	Calcium Silicate Hydrate
ERA	Ethiopian Roads Authority
FSI	Free swell index
FSR	Free swell ratio
IS	Indian Standard
LL	Liquid Limit
PL	Plastic Limit
MDD	Maximum Dry Density
NGL	Natural Ground Level
OMC	Optimum Moisture Content
PI	Plastic Index
PL	Plastic Limit
SP	Swelling pressure
UCS	Unconfined compressive strength
USA	United States of America
WCP	Waste Ceramic Powder

Units

Gm	Gram
KN	Kilo Newton
mm	millimeter
g/cm^3	Gram per centimeter cube
µm	micrometer
KN/m^2	Kilo Newton per meter square
kPa	Kilo Pascal
mm	millimeter
meq	Milliequivalent
^{o}C	Degree Centigrade
cc	Centimeter cube
cm^3	Centimeter cube

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ABSTRACT

Subgrade materials are expected to have basic desirable characteristics related to strength, stiffness and swelling. If these properties are not fulfilled, engineers are expected to come up with ground improvement methods. The growing cost of traditional stabilizing agents and the need for the economical utilization of industrial and agricultural wastes for beneficial engineering purposes have prompted an investigation into the stabilizing potential of waste ceramic powder in highly expansive clay soil. This research work is aimed to evaluate the suitability of waste ceramic powder for stabilization of expansive clay soil. The preliminary investigation of the soil to be stabilized shows that it belongs to A-7-5 class of soil in the AASHTO soil classification system. Soils under this class are generally of poor engineering use. Atterberg limits, free swell, free swell index, free swell ratio, compaction and CBR tests were used to evaluate properties of stabilized soil. The soil was stabilized with waste ceramic powder in stepped concentration of 5%, 10%, 15%, 20%, 25% and 30% by dry weight of the soil. All stabilized soil samples were also cured for 7 days for Atterberg limits, compaction and CBR tests.

Analysis of the results shows that slight improvement on the geotechnical properties of waste ceramic powder stabilized soil. Waste ceramic powder reduces plasticity index, swelling and CBR swell and it increases MDD with decrease in OMC and CBR. Curing has an insignificant effect on the geotechnical properties of waste ceramic powder stabilized soil. From this study it was found out that waste ceramic powder stabilized soil do not show significant change for both index and strength property.

Additional study is also incorporated as a supplementary work to investigate the effect of applying 3% lime as an activator in combination with 15% waste ceramic powder on the geotechnical properties of the soil for uncured and cured soil samples. The results indicate that lime in combination with waste ceramic powder is suitable for improving the plasticity index, swelling and CBR. The strength values (CBR) also increased with curing ages, thus indicating that the blend has a potential for time-dependent increase in strength that will reduce the quantity of stabilizer needed for the construction of roads over the expansive soil. Therefore, this study shows that lime in combination with/plus waste ceramic powder can be effectively used to improve expansive soils with low soaked CBR value and high plasticity.

Key words: Expansive Soil, Soil Stabilization, Waste Ceramic Powder, Atterberg limit, Compaction and CBR value.

CHAPTER ONE

INTRODUCTION

1.1 General Background

The need to bring down the growing cost of soil stabilizers and the cost of waste disposal has lead to intense global research towards economic utilization of wastes for engineering purposes. The safe disposal of industrial and agricultural waste products demands urgent and cost effective solutions because of the debilitating effect of these materials on the environment and to the health hazards that these wastes constitute. In order to make deficient soils useful and meet geotechnical engineering design requirements researchers [46-47,50] focused more on the use of potentially cost effective materials that are locally available from industrial and agricultural waste in order to improve the properties of deficient soils.

Subgrade is the native material underneath a constructed road or pavement. It is the foundation of the pavement structure and called formation level [7,11]. Subgrade function is to prevent excessive rutting and shoving during construction, provide good support for placement and compaction of pavement layers, limit pavement rebound deflections to acceptable limits and restrict the development of excessive permanent deformation (rutting) in the road structure during its service life. The quality of the subgrade will greatly influence the pavement design, performance and its service life [2].

Roads constructed on expansive soils areas are known for bad condition and unpredictable be havior for which the nature of the soil contributes to some extent. The failures of pavement, in form of heave, depression, cracking and unevenness are most likely to happen by the expansive soil in the subgrade. To eliminate the danger from such these soils, a technique of soil stabilization needs to be taken out. Soil stabilization is a collective term for any physical, chemical, or biological method or any combination of such methods employed to improve or change certain properties of natural soil to make it serve adequately an intended engineering purpose. There are many techniques of soil stabilization like mechanical, chemical and physical stabilization. Stabilization waterproofs the soil improves its strength, workability and durability and helps to reduce its volume change due to temperature or moisture [43].

There are a lot of industry which produce ceramic like Tabor Ceramic (older one).Arerity Industrial Park, East African Zone (DIYUANN CERAMIC PLC) and other Industrial Park. DIYUANN CERAMIC PLC is over 400,000 m^2 in Eastern Industrial Zone, Oromia Region, Ethiopia. The production capacity is 60,000 m^2 per day. They use the resourse material which comes from around Mojo and Debrezeyit like Talk, Feldspar, Calcite, Quartz, Muscovite and other material which are important for the ingredient of ceramic production. It has been estimated that about 30% of the daily production in the ceramic industry goes to waste. These discarded materials, most of which cannot be recycled within the plant, constitute industrial waste which is often landfilled. Meanwhile, the waste has been categorized under pozzolana with about 3.13 % calcium oxide (CaO), 0.75% iron oxide (Fe_2O_3), 1.82% magnesium oxide (MgO), 67.51% silicon oxide (SiO_2) and 16.92% aluminumoxide (Al_2O_3) (from Ethiopian geological survey). The utilization of this pozzolana as a replacement for traditional stabilizers, such as cement and lime, will go a long way in actualizing the dreams of most developing countries of scouting for cheap and readily available construction materials. Waste ceramic powder has been used in concrete as a partial replacement material for cement [34].

The three most commonly used stabilizers for expansive soils are bitumen, lime and cement. [41,43] have reported that the stabilization of expansive soil with lime or cement is effective. Unfortunately, the costs of these stabilizers are on the high side making them economically unattractive as stabilizing agents. Recent trend in research works in the field of geotechnical engineering and construction [19,47] focuses more on the search for cheap and locally available materials such as waste ceramic powder, bagasse ash, sugarcane straw ash, fly ash, rice husk ash, coconut husk ash etc. as stabilizing agents for the purpose of full or partial replacement of traditional stabilizers like cement and lime. Engineering properties of soils are commonly altered when these wastes are introduced as admixtures with lime or cement. Moreover, since recently, several studies have confirmed applicability of these wastes as a cement replacing material in concrete technology. Therefore, this study will be geared towards evaluating some of the engineering properties of waste ceramic powder stabilized expansive soil. If the study leads to positive outputs, waste ceramic powder can be used as soil stabilizing agent replacing the rather costly chemicals employed such as cement, lime, etc.

1.2 Justifications for the Thesis

- Cost savings, because waste ceramic powder is typically by far cheaper than traditional stabilizers such as cement and lime;
- The production of traditional stabilizers, such as cement and lime, is environmental unfriendly processes;
- The extraction of substantial amounts of non-renewable natural resources for road construction creates significant damaging impacts on the local environment and its inhabitants;
- Waste management can be done economically;

Therefore, using waste ceramic powder for improving engineering properties of the soils is an economical solution for Ethiopia as it is available in large quantity.

1.3 Objectives of the Research

1.3.1 General Objective

The general objective of this study is to evaluate the suitability of waste ceramic powder as a stabilizing agent for expansive soil.

This is achieved through the following specific objective:

1.3.2 Specific Objectives

The specific objectives of this study are:

1. To evaluate the effect of waste ceramic powder on the properties of the expansive soil using Atterberg limits, free swell, free swell index, compaction and CBR as measuring parameters.

2. To compare the changes in properties of expansive soil with respect to waste ceramic powder stabilized soil.

3.To investigate constituents of the waste ceramic powder.

4.To come up with a recommendable optimum proportion by weight of the waste ceramic powder required to improve expansive soils to meet the specification requirement of subgrade material

1.4 Research Methodology

In order to achieve the above objectives of the study the following methodologies were adopted:

i) Literature review: pertinent literature pertaining to expansive soils has been identified and reviewed.

ii) Sampling and testing: material sampling and testing methods are undertaken as per ASTM, AASHTO and IS standards. Soil samples were collected from Addis Ababa Infront of Bole International Airport in Ethio ICT Village. The chemical additive, waste ceramic powder taken was collected from East Africa Industry.

iii) Sample preparation of the experimental work involved air drying, pulverization and sieving of the natural soil sample to the required particle sizes. Classification of soil was made by running grain size distribution and Atterberg limit tests. Then Atterberg limit, free swell, free swell ratio, free swell index, compaction and California bearing ratio tests are carried out on natural soil as well as on soil- waste ceramic powder mix to study the effect of the stabilizer (waste ceramic powder).

iv) Analysis and discussion of test results: based on the theories and laboratory tests performed, the results obtained have been analyzed and discussed thoroughly.

v) Formulation of conclusions and recommendations based on the results obtained.

vi) Finally compiling and writing of the thesis work.

1.5 Scope of the Study

This study has been supported by literature survey and a series of laboratory tests. However, the findings of the research are limited to one soil sample considered in this research which is expansive clay. The results are also specific to the type of additives used and test procedures that have been adopted in the experimental work. Therefore, findings should be considered indicative rather than definitive for field applications.

The scope of this thesis is limited to DIYUAN CERAMIC PLC which is found in Eastern Industry Zone which produce mainly **wall** and **floor ceramic tile**. The research have used only wall and floor ceramic wastage.

1.6 Organization of the Thesis

The presentation of this thesis work is organized in seven Chapters. The first Chapter gives a brief description of the thesis background, objectives, scope and methodology employed. Chapter two and Chapter three present conceptual background on expansive soils and soil stabilization respectively. Important details from previous studies are also included in Chapter three. The fourth Chapter briefly describes the characterization of materials used for the study and laboratory testing procedures followed. The fifth Chapter presents the test results obtained; analysis of results and discussions of results with respect to the theoretical background and findings of previous studies. Chapter six presents additional work. Finally, conclusions and recommendations drawn from the research are presented in Chapter seven.

CHAPTER TWO

IDENTIFICATION OF EXPANSIVE SOILS

2.1 Introduction

Expansive soil refers to a soil that has the potential for swelling and shrinking due to changing moisture condition. Expansive soils cause more damage to structures particularly pavements and light buildings than any other natural hazard, including earthquakes and floods. It has been reported that the damage caused by these soils contribute significantly to the burden that the natural hazard pose on the economy of countries where the occurrence of these soils is significant [44]. Ethiopia is amongst the list of countries where the occurrence and spatial distribution is recognized as significant. Expansive soils can be found anywhere in the world but they are basically confined to semi–arid and arid regions. These areas are naturally characterized by marked dry and wet seasons with low rainfall, poor drainage and exceedingly great heat. The climate condition is such that the annual evapo-transpiration exceeds the precipitations [18]. Two groups of parent materials have been associated with the formation of expansive soils. The first group comprises sedimentary rocks of volcanic origin which can be found in North America, South Africa and Israel, while the second groups of parent materials are basic igneous rocks found in India and Southwestern [18]. The most well known example of expansive soils is the black cotton soil which is dark grey to black in color and the name originated from India where locations of these soils are favourable for growing cotton.

2.2 Origin of Expansive Soils

The origin of expansive soils is related to a combination of conditions and processes that result in the formation of clay minerals having a particular chemical makeup which, when in contact with water, expands. Variations in the conditions and processes may also form other clay minerals, most of which are non expansive. The conditions or processes, which determine the clay mineralogy, include composition of the parent material and degree of physical and chemical weathering to which the materials are subjected.

i. Parent Material

The constituents of the parent material during the early and intermediate stages of the weathering process determine the type of clay formed. The nature of the parent material is much more important during these stages than after intense weathering for long periods of time [18]. The parent materials that can be associated with expansive soils are classified into two groups. The first group comprises the basic igneous rocks and the second group comprises the sedimentary rocks that contain montimorillonite as a constituent [18]. The basic igneous rocks are comparatively low in silica, generally about 45 to 52 percent. Rocks that are rich in metallic base such as the pyroxenes, amphiboles, biotite and olivine fall within this category. Such rocks include the gabbros, basalts and volcanic glasses [18]. The sedimentary rocks that contain montimorillonite as constituent include shale and clay stones. Limestone and marls rich in magnesium can also weather to clay. These parent materials contain varying amounts of volcanic ash and glass, which can subsequently be weathered to montimorillonite. The volcanic eruptions sent up clouds of ash, which fell on the continents and sea. Some of fine grained sediments which accumulated to form these rocks also contain montimorillonite derived from weathering of continental igneous rocks and from ash, which fell on the continental areas [18].

ii. 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 of parent rock mineral. The parent material 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 exceeds precipitation. Under these conditions enough water is available for the alteration process but the accumulated cations will not be removed by rain water [18].

2.3. Clay Mineralogy

The term clay can refer both to a size and to a class of minerals. As a Size term, it refers to all constituents of a soil smaller than a particular size, usually 0.002 mm in engineering classifications. As a mineral term, it refers to Specific clay minerals that are distinguished by small particle size, a net Electrical charge, plastic when mixed with water and high weathering resistance [41]. Minerals are crystalline and make up the solids constituent of a soil. The mineral particles of fine grained soils are platy. Minerals are classified according to chemical composition and structure. Most minerals of interest to geotechnical engineers are composed of oxygen and silicon. Silicates are a group of minerals with a structural unit called the Silica Tetrahedral. A central silica cation (positively charged ion) is surrounded by four oxygen anions (negatively charged ions), one at each corner of the tetrahedron. Silicate minerals are formed by addition of cations and interaction of tetrahedrons. Silica tetrahedrons combine to form sheets, called silicate sheets, which are thin layers of silica tetrahedrons in which three oxygen ions are shared between adjacent tetrahedrons. Silicate sheets may contain other structural units such as alumina sheets. Alumina sheets are formed by combination of alumina minerals, which consists of an aluminum ion surrounded by six oxygen or hydroxyl atoms in an octahedral [29]. Most soil classification system arbitrarily defines clay particles as having an effective diameter of two microns (0.002mm) or less. Particle size alone does not guarantee the classification of fine grained materials. The most important grain property for fine grained soils is the mineralogical composition [18].

Clay minerals are classified as follows:

- Two-layer clays which consists of one tetrahedral layer bonded to one aluminum octahedral layer. Kaolinite is the most common mineral under this catagory
- Three-layer clays which consist of one octahedral layer sandwiched between two tetrahedral layers. Illite, montmorillonite and vermiculite are the common minerals under this category.
- Mixed-layer clays which consist of interstratifications of the two- and three- layer clay minerals previously described. The mixing may be regular or random. Commonmineral under these classes are chlorite, montmorillonite-chlorite of the clay mineral.

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. Illite, also a three-layer clay mineral, exhibits very minor interlayer swelling. This results 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 minerals responsible for most of the damages incurred by expansive soils is montmorillonite.

Montmorillonite is a die octahedral 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. The mineral configurations of these clay types are shown in Figure 2.1 [31].

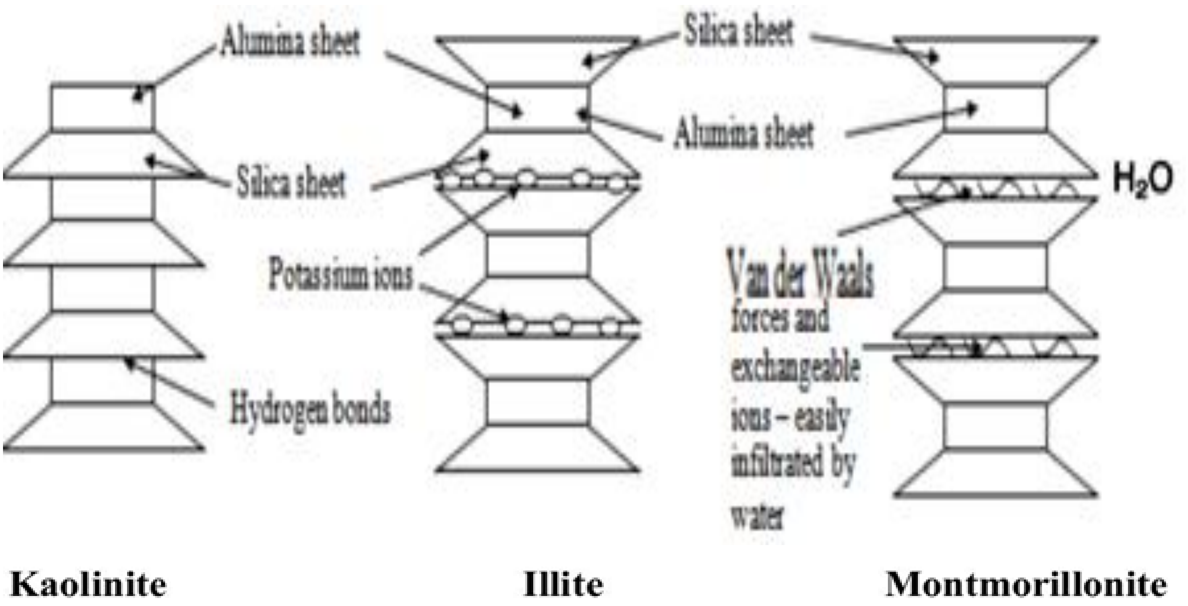


Figure 2.1: Schematic Representation of Clay Minerals [23].

Clay minerals are predominantly silicates of aluminum and/or iron and magnesium. Some of them also contain alkalies and/or alkaline earths as essential components. These minerals are predominantly crystalline in such a way that the atoms composing them are arranged in definite geometric patterns. Most of the clay minerals have sheet or layered structures. A few have elongated tubular or fibrous structures [45].The basic idealized crystalline structural unit of a clay mineral is composed of a silica tetrahedron block and an aluminum octahedron block. Aluminum octahedron block may have Aluminum (Al^{3+}) or magnesium (Mg^{2+}). If only aluminum is present, it is called gibbsite [$Al_2(OH)_6$]; if only magnesium is present, it is called brucite [$Mg_3(OH)_6$].

Various clay minerals are formed as these sheets stack on top of each other with different ions bonding them together. A silica tetrahedron and a silica sheet, also an octahedron and an octahedron sheet are presented in Figure 2.2 and Figure 2.3, respectively. Also, these figures consist of schematic representations of silica and octahedron sheets [41].

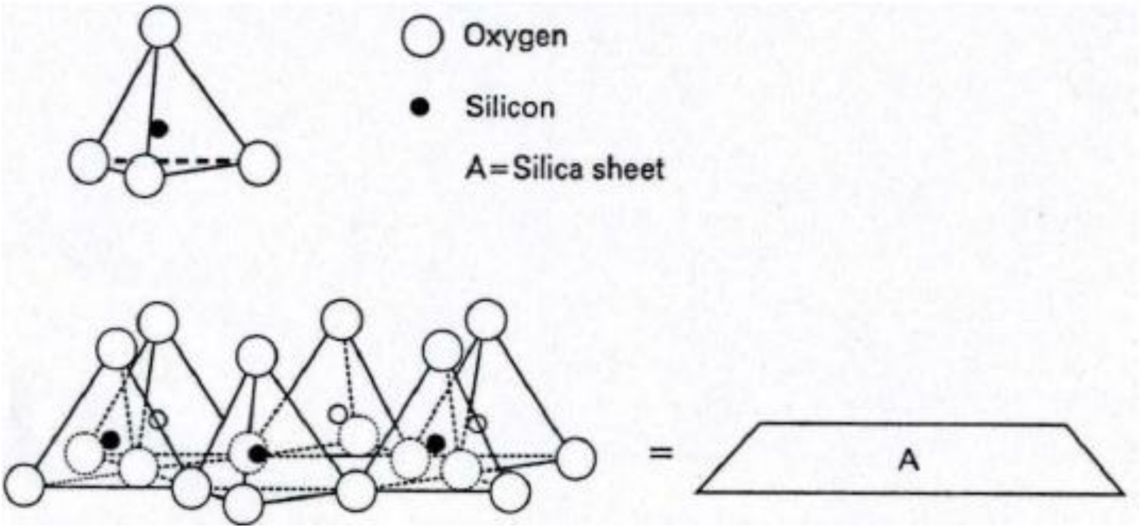


Figure 2.2: a Silica Tetrahedron and a Silica Sheet [49].

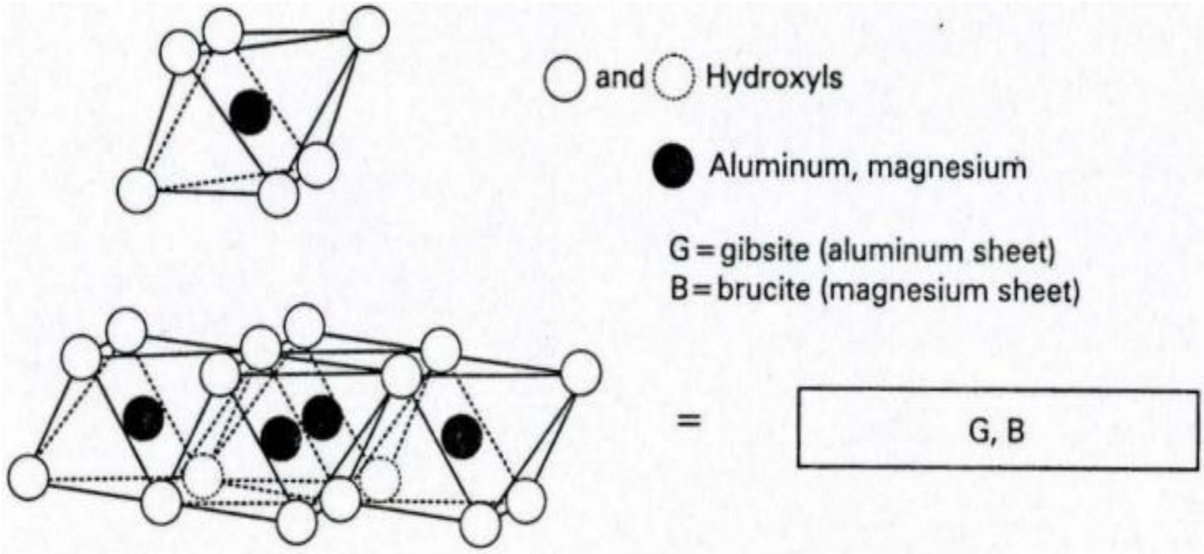


Figure 2.3: An octahedron and an octahedron Sheet [49].

2.4 Clay Structures

Dispersed and flocculated structures are the two elementary structures of clay particles as shown on Figure 2.4. If the magnitude of the attractive force is lesser than the magnitude of the repulsive force, the final result will exactly be repulsion. The individual clay particles will settle and form a dense layer at the bottom; however, they will separately remain from their surroundings. This is referred to as the dispersed state of the soil. On the other hand, flocs will be formed and these flocs will settle to the bottom, if the net force between the particles is attraction, which is called flocculated clay [25].

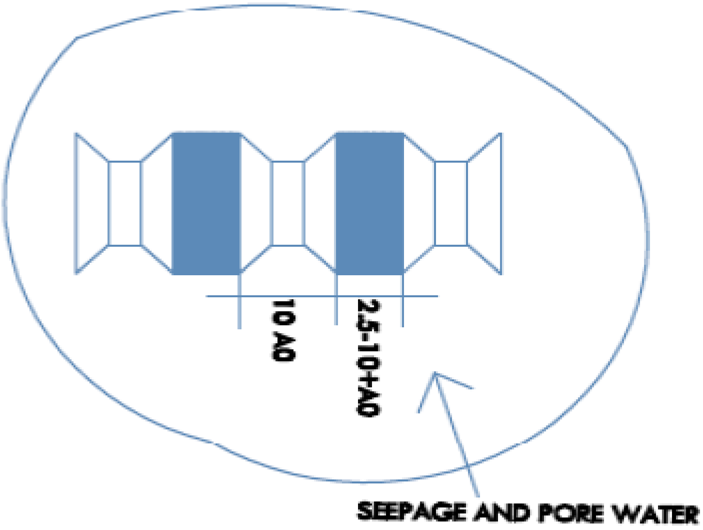


Figure 2.4: Deflocculated clay mineral showing surface and interlayer water

There are three structural groups of clay minerals that are important for engineering purposes. These three groups are

- **Kaolinite group -** usually non expansive.
- **Mika like group** - includes illites and vermiculites, which can be expansive but generally do not pose significant problems.
- **Smectite group** - includes montmorillonites, which are highly expansive and are the most troublesome clay minerals

2.4.1 Kaolinite Group

The kaolin minerals are a group of clay minerals consisting of hydrous aluminum silicates. A common kaolin mineral is kaolinite. Kaolite is a typical two layer mineral having a single tetrahedral sheet joined by a single octahedral sheet to form what is called a 2 to 1 lattice structure. The bonding combination of hydrogen and Vander Waals forces results in considerable strength and stability with little tendency for interlayers to take on water and swell. The bonding is sufficiently strong that there is no interlayer swelling in the presence of water. Kaolinite is the least active of the clay minerals. Kaolinite can be produced by weathering of certain of the more active clay minerals as well as being directly formed as a by product of rock weathering. Kaolinite tends to be found in regions of heavier rainfall [46].

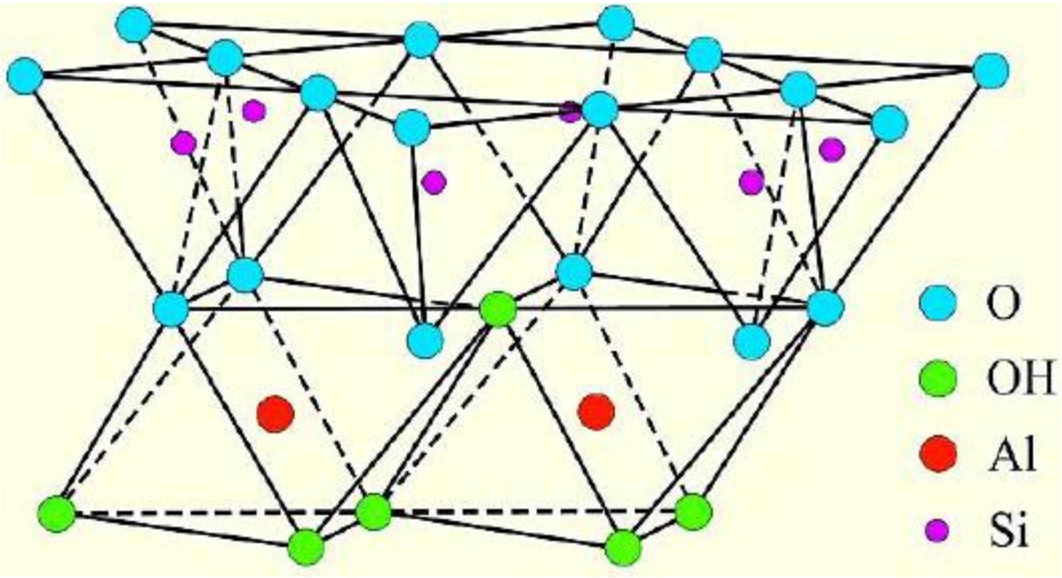


Figure 2.5: Diagrammatic Skectch of the Kaolinite [46].

2.4.2 Illite Group

Illite has a basic structure consisting of a sheet of alumina octahedrons between and combined with two sheets of silica tetrahedrons. This clay mineral has a structure similar to montmorillonite, but the layers are more strongly bonded together. In terms of cation exchange capacity, inability to absorb and retain water and in physical characterstics. Illite is intermediate in activity between clays of kaolin and montmorillonite groups. In the octahedral sheet there is partial substitution of aluminum by magnesium and iron. And in the tetrahedral sheet there is partial substitution of silicon by aluminum. The combined sheets are linked together by fairly weak bonding due to (non-exchangeable) potassium ions held between them [41].

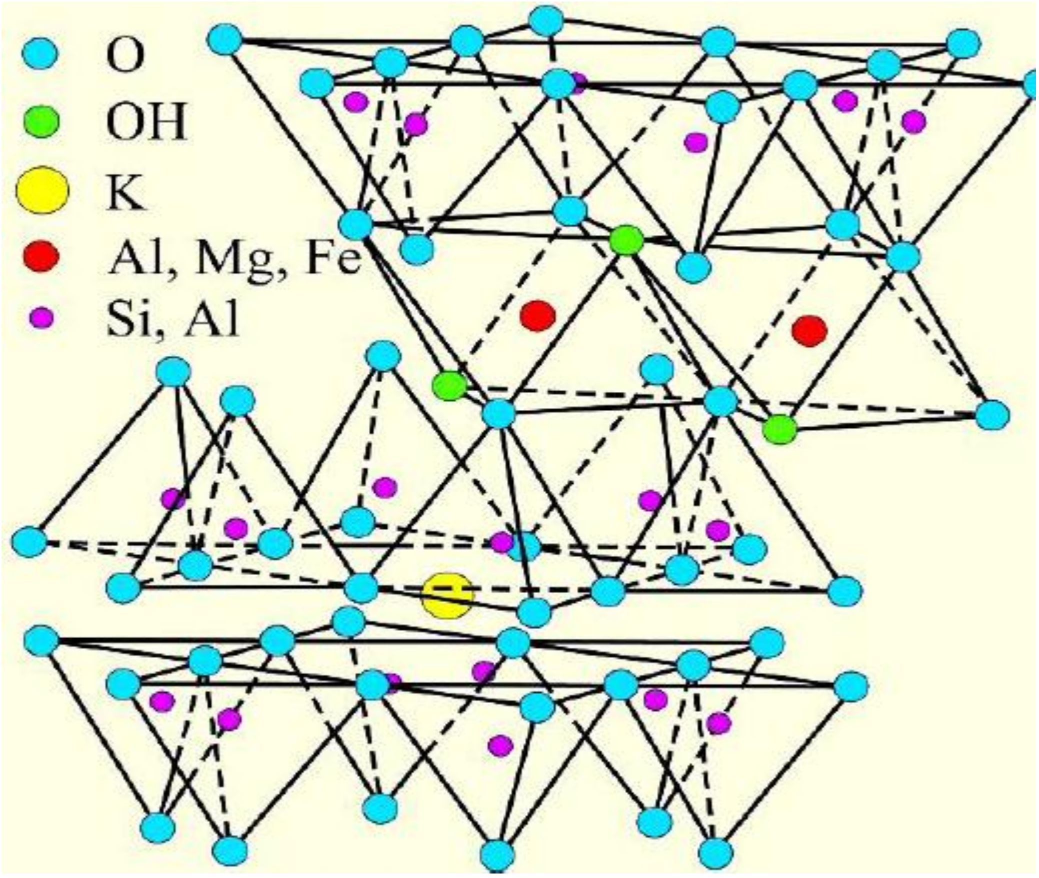


Figure 2.6: Diagrammatic Skectch of the Illite [46].

2.4.3 Smectite Group

Montmorillonite are a group of clay minerals that are characterized by weekly bonded layers. It is formed from weathering of volcanic ash under poor drainage conditions or in marine waters. Each layers consists of two silica sheets with an aluminium (gibbsite) sheet in the middle. Water and exchangeable cations can enter and separate the layers, creating a very small crystal that has a strong attraction to water. Montmorillnite has the highest activity and it can have the highest water content, greatest compressibility and lowest shear strength of all clay minerals [41]. The space between the combined sheets is occupied by water molecules and exchangeable cations. There is a very weak bond between the combined sheets due to these ions. Considerable swelling of montmorillonite being can occur due to additional water absorbed between the combined sheets [41].

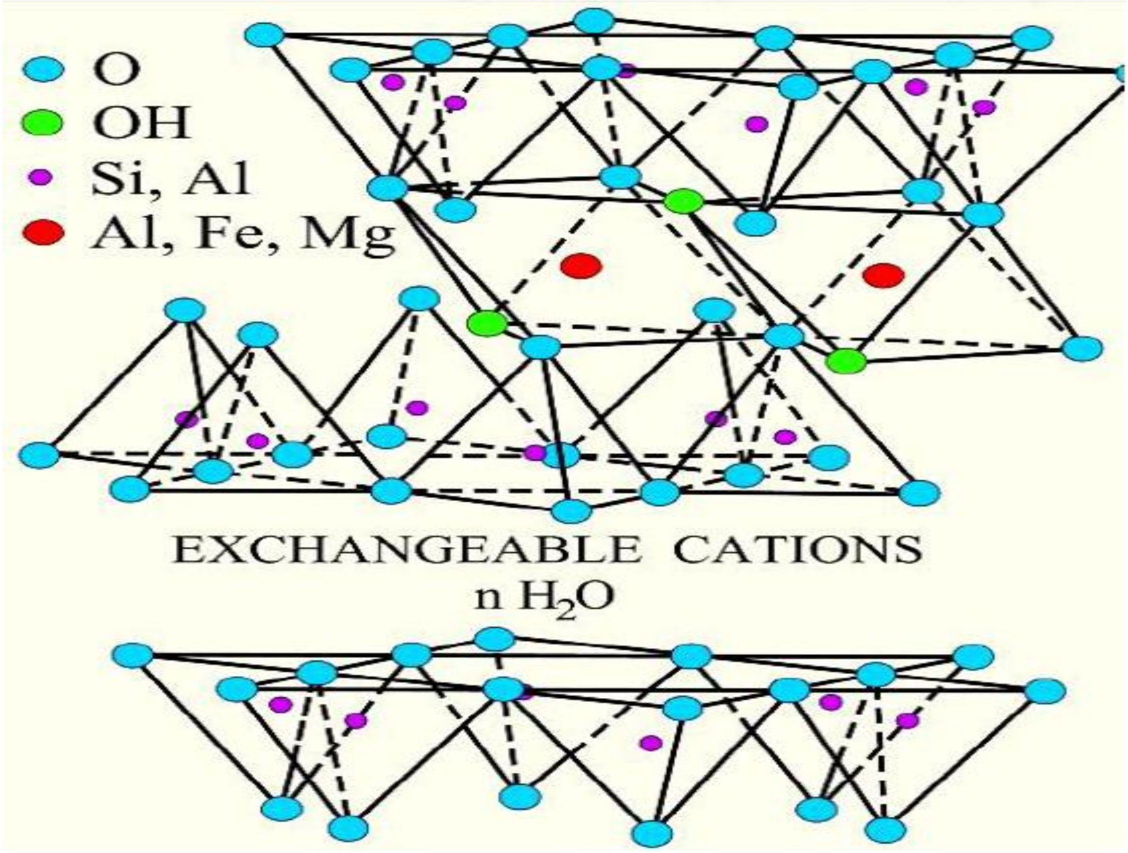


Figure 2.7: Diagrammatic Skectch of the Montmorillonite [46].

2.5. Cation Exchange Capacity (CEC)

The cations can be arranged in a series in terms of their affinity for attraction as below that shows some cations are strongly more attracted than the others:
$Al^{3+} > Ca^{2+} > Mg^{2+} > NH^{4+} > K^{+} > H^{+} > Na^{+} > Li^{+}$, [25].
This series illustrate that hydrogen clay can be transformed to sodium clay by a constant percolation of water including dissolved Na^{+} salts. Such changes can facilitate decreasing the permeability of a soil. However, all adsorbed cations are not exchangeable. The quantity of exchangeable cations in a soil determines exchange capacity [42].

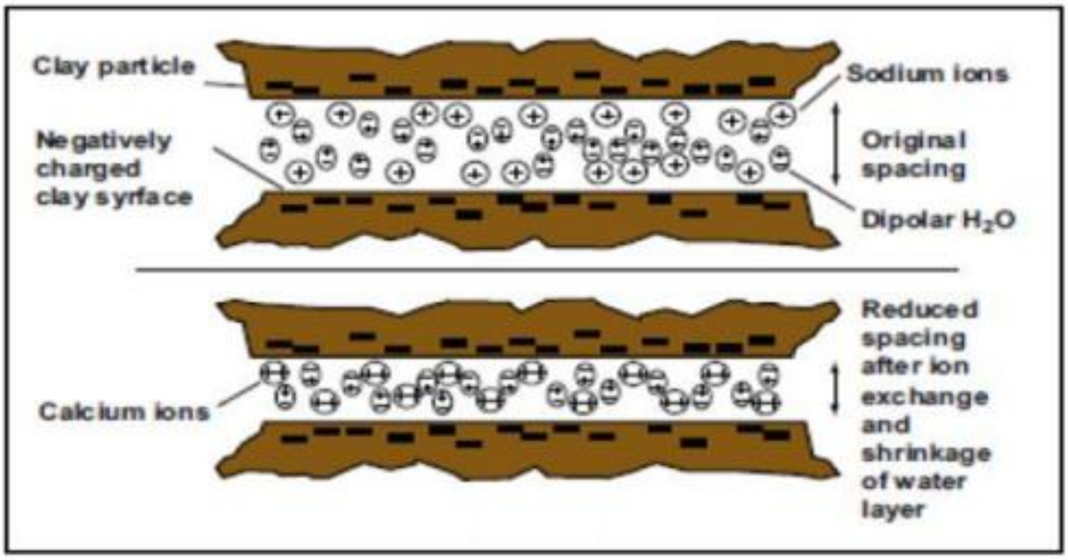


Figure 2.8: Cation Exchange [42].

CEC is term that referred to the quantity of exchangeable cations required to balance the charge deficiency on the surface of the clay particles. Higher CEC, higher surface activity and consequently higher water absorption potential usually belong to clays with larger specific surface area. Cation exchange capacity (CEC) values have been used to explain the effect of the new pozzolanic reaction products on the particle size and the swell potential of the treated soils. Typical values of CEC for three basic clay minerals are tabulated in Table 2.1 as follow [44].

Table 2.1: Typical CEC values of basic clay minerals [44].

Clay Mineral	CEC(meq/100gm)
Kaolinite	3-15
Illite	10-40
Montmorillonite	80-150

2.6. Distribution of Expansive Soil

Expansive soils are widespread in African continent, occurring in South Africa, Ethiopia, Kenya, Mozambique, Morocco, Ghana, Nigeria etc. In other parts of the world case of expansive soils have been widely reported in countries like USA, Australia, Canada, India, Spain, Israel, Turkey, Argentina, Venezuela etc [12,43]. The aerial coverage of expansive soils in Ethiopia is estimated to be 24.7 million acres as cited by [43]. They are widely spread in central part of Ethiopia following the major truck roads like Addis-Ambo, Addis-Wolliso, Addis–Debrebirhan, Addis-Gohatsion, Addis-Modjo are covered by expansive soils. Also areas like Mekele and Gambella are covered by expansive soil. The distributions are shown in Figure 2.9 [51,54].

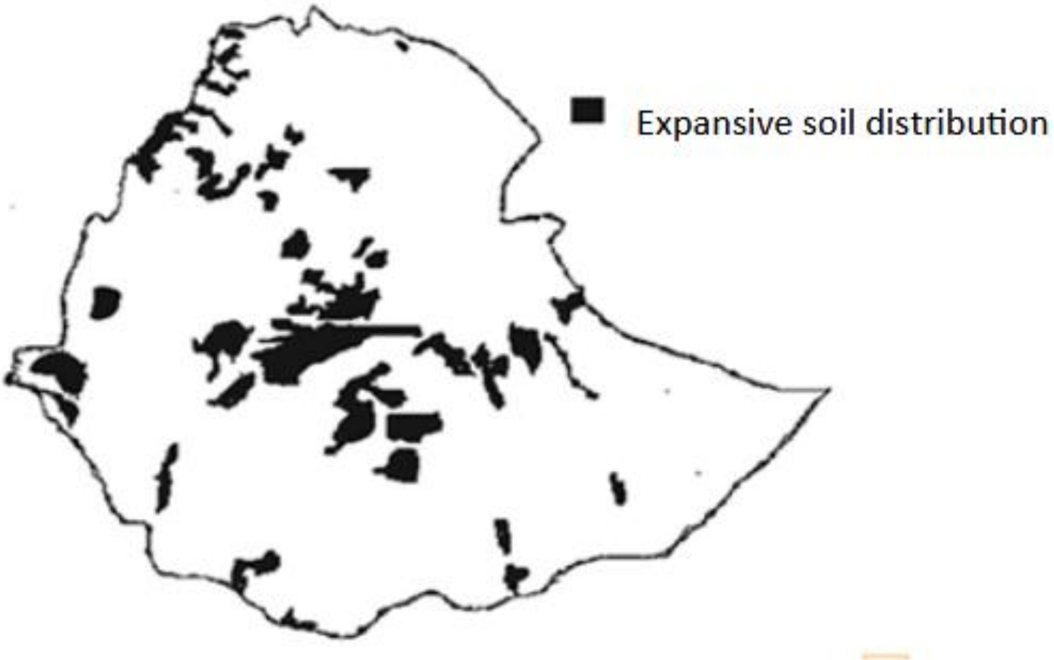


Figure 2.9: Distribution of expansive soil in Ethiopia [51,54].

2.7 Identification of Expansive Soils

Investigation of expansive soils generally consists of two important phases. The first is the visual identification and recognition of the soil as expansive and the second is sampling and measurement of material properties to be used as the basis for design. The theme of this topic is to discuss different ways that are commonly used to identify expansive soils.

2.7.1 Field Identification

Soils that can exhibit high swelling potential can be identified by field observations, mainly during reconnaissance and preliminary investigation stages. Important observations include [18,44]:

- Usually have a color of black or grey.

- Wide or deep shrinkage cracks.
- High dry strength and low wet strength.
- Stickiness and low trafficability when wet.
- Cut surfaces have a shiny appearance.
- Appearance of cracks in nearby structures.

Arid and semi-arid areas are particular trouble spots because of large variations in rainfall and temperature.

2.7.2 Laboratory Identification

Laboratory identification of expansive soils can be categorized into mineralogical, indirect and direct methods.

2.7.2.1 Mineralogical Identification

Clay mineralogy is a fundamental factor controlling expansive soil behavior. Clay minerals can be identified using a variety of techniques. The techniques that can be used are [18,44]:

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

But these methods are not suitable for routine tests because of the following reason;

- They are time consuming;
- They require expensive test equipment; and
- The results can only interpreted by specially trained technicians.

2.7.2.2 Indirect Methods

In this method simple soil property tests can be used for the evaluation of swelling potential of expansive soils using swelling test. Such tests are easy to perform and should be included as routine tests in the investigation of expansive soils. Such tests may include [18,44]:

i) Atterberg Limits

In this method, measurement of the atterberg limits of the soil are conducted for identification of all 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 relation between the swelling potential of clays and the plasticity index is shown in Table 2.1 below.

Table 2.2: Relation between the swelling potential of clays and the plasticity index [18].

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

ii) 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.

Experiments indicated that a good grade of high swelling commercial bentonite will have a free swell of from 1200 to 2000 percent. Soils having a free swell value as low as 100 percent can cause considerable damage to lightly loaded structures, and soils having a free swell value below 50 percent seldom exhibit appreciable volume change even under very light loadings. The free swell percentage can be computed using Equation (2.1) from the relationship between initial and swelled volume [18,44].

$$FS=\left(\frac{(V_F-V_I)}{V_I}\right)\times 100 \tag{2.1}$$

Where: FS = free swell, V_I =initial volume & V_F =final volume

iii) Free Swell Index

Free swell index is also one of the most commonly used simple tests to estimate the swelling potential of expansive clay. The procedure involves in taking two oven dried soil samples passing through 425µm sieve, 10cc each were placed separately in two 100ml graduated soil sample. Distilled water was filled in one cylinder and kerosene in the other cylinder up to 100ml mark. The final volume of soil is computed after 24hours to calculate free swell index. The free swell index is then calculated using Equation (2.2) [4].

$$\text{Free swell Index} = \frac{V_w - V_k}{V_k} \text{x } 100 \tag{2.2}$$

Where:

V_w = final volume in water

V_k = final volume in kerosene

The relation between the degree of expansion and differential free swell index is shown in Table 2.3. It is normal to quantify 10cc as the volume occupied by 10g of soil. This does not account for variations of density [4].

Table 2.3: Degree of expansion and differential free swell index [23].

Free Swell index(%)	Degree of expansion
Less than 20	Low
20 to 35	Moderate
35 to 50	High
Greater than 50	Very high

iv) Free Swell Ratio test

To determine the swell property, Sridharan and Prakash proposed the free swell ratio method of characterizing the soil swelling. Free swell ratio is defined as the ratio of sediments volume of 10cc oven dried soil passing through 425µm sieve in distilled water to that of Kerosene Equation (2.3).

$$\text{Free swell ratio} = \frac{V_w}{V_k} \text{x } 100 \tag{2.3}$$

Where

V_w = final volume in water

V_k = final volume in kerosene

The relation between the degree of expansion and differential free swell ratio is given in Table 2.4.

Table 2.4: Classification of Soils based on free swell ratio [23].

Free Swell Ratio	Soil Expansivity	Clay Type
<1	Negligible	Non-Swelling
1.0-1.5	Low	Mixture of non Swelling & Swelling
1.5-2.0	Moderate	Swelling
2.0-4.0	High	Swelling
>4	Very high	Swelling

2.7.2.3 Direct Methods

These methods offer the most useful data by direct measurement; and tests are simple to perform and do not require complicated equipment. Testing should be performed on a number of samples to avoid erroneous conclusions. Direct measurement of expansive soils can be achieved by the use of conventional one-dimensional consolidometer.

2.8 Classification of Expansive Soils

Parameters determined from expansive soil identification tests have been combined in a number of different classification schemes. The classification system used for expansive soils are based on indirect and direct prediction of swell potential as well as combinations to arrive at a rating. There are a number of classification systems. The following are some of the common methods.

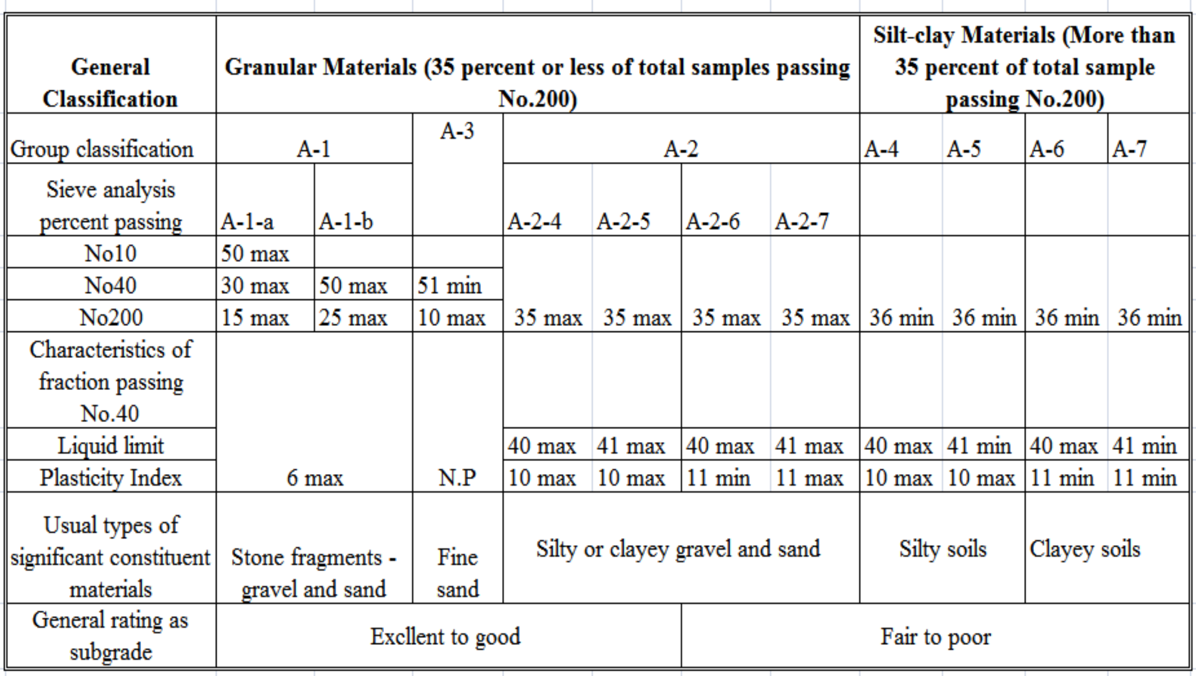
2.8.1 Classification Using General Methods

The most widely used general classification systems are:

i. AASHTO Classification

As shown on Table 2.5 soils rated A-6 or A-7 by AASHTO can be considered potentially expansive [44].

Table 2.5: AASHTO soil classification chart [44].



General Classification	Granular Materials (35 percent or less of total samples passing No.200)							Silt-clay Materials (More than 35 percent of total sample passing No.200)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
Sieve analysis percent passing	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				
No10	50 max										
No40	30 max	50 max	51 min								
No200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No.40											
Liquid limit				40 max	41 max	40 max	41 max	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P	10 max	10 max	11 min	11 max	10 max	10 max	11 min	11 min
Usual types of significant constituent materials	Stone fragments - gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Exclent to good					Fair to poor					

ii. Unified Soil Classification Systems

In this classification system a correlation is made between swell potential and unified soil classification as follows.

Category	**Soil classification system**
Little or no expansion	GW, GP, GM, SW, SP, SM
Moderate expansion	GW, SC, ML, MH
High volume change	CL OL, CH, OH
No rating	Pt

The above classification system can be summarized as follow:

a. All clay soil and organic soils exhibit high volume change.

b. All clayey gravels and sands and all silts exhibit moderate volume changes.

c. All sands and gravels exhibit little or no expansion.

2.8.2 Classification Specific to Expansive Soil

The above classification system may give an initial alert that the soil may have expansive character but it does not 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 [44]. Hence, it is very important to emphasize that design decision has to be based on predicting testing and analysis, which provide reliable information. An indirect prediction of swell potential includes correlations based on index properties, swell and a combination of them. Some of such classification systems are:

i) Method of Chen

[18] Presented a single index method for identifying expansive soils using only plasticity index. Chen suggested four classes of clays according to their plasticity indices shown in Table 2.6.

Table 2.6: Relation between the swelling potential of clays and the plasticity index[18].

Swelling Potential	Plasticity index
Low	0-15
Medium	10-35
High	20-35
Very high	35 nd above

ii) Method of Daksanamurthy and Raman (1973) Daksanamurthy and Raman (1973).

Presented a single index method for identifying expansive soils using only liquid limit. They suggested four classes of clays according to their liquid limits as shown in Table 2.7 [4].

Table 2.7: Relation between the swelling potential of clays and the liquid limit [4,18].

Swelling Potential	Liquid Limit
Low	$20<LL \leq 35$
Medium	$35<LL\leq50$
High	$50<LL\leq70$
Very high	$LL>70$

iii) USBR Method

This method is developed by Holtz and Gibbs; it is based on direct correlation of observed volume change with colloid content, plastic index and shrinkage limit. The classification is as given in Table 2.8.

Table 2.8: Classification based on bureau of reclamation method [18, 23].

Colloid Content (%)	Plasticity index (%)	Shrinkage Limit (%)	Probable Expansion (%)	Degree of Expansion
<18	>15	>15	<10	Low
15-28	15-28	10-20	10-20	Medium
25-41	25-41	20-30	20-30	High
>35	>35	>30	>30	Very high

iv) Method of Seed et al

After an extensive study on swelling characteristics of remolded, artificially prepared and compacted clays [18] have developed a chart based on activity and percent clay sizes as shown in Figure 2.10. The activity here is defined as:

$$Ac=\frac{PI}{C-10} \qquad (2.4)$$

Where

A_c= activity

C= percentage of clay size finer than 0.002mm

PI= plasticity index

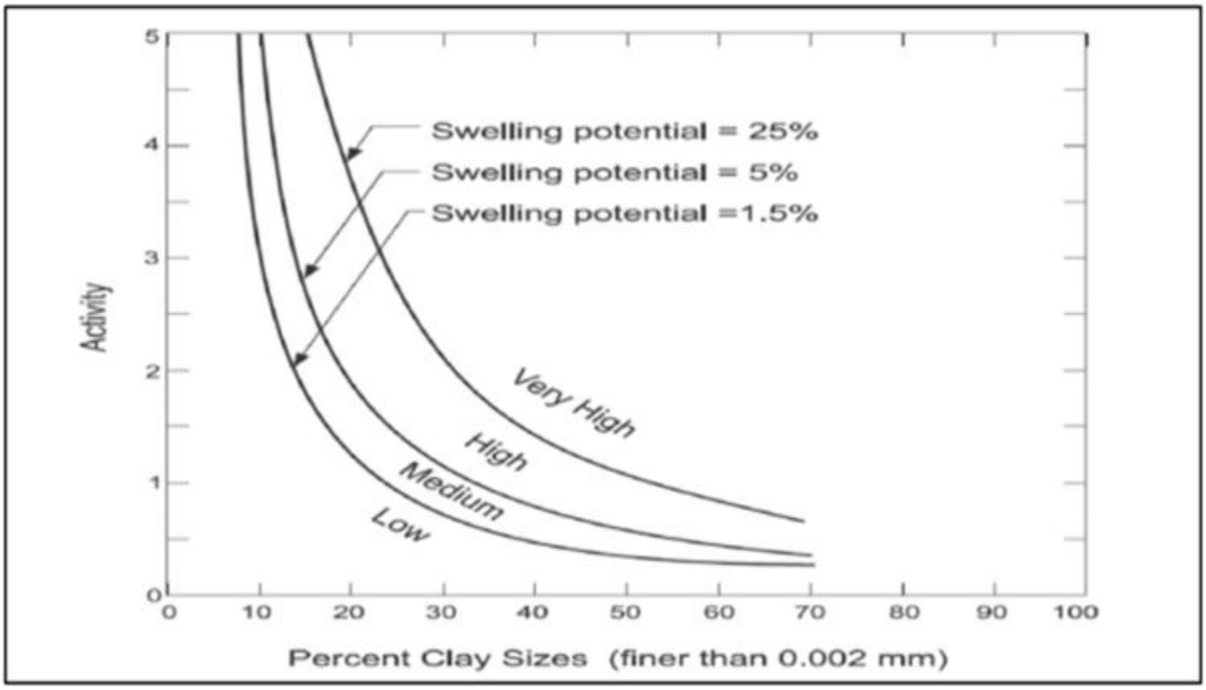


Figure 2.10: Classification chart for swelling potential [18].

v) Method of Skempton

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

$$\text{Activity (Ac)} = \frac{\text{Plasticity Index}}{\text{Percentage by Weight finer than } 2\mu\text{m}} \tag{2.5}$$

Skempton suggested three classes of clays according to their activity shown in Table 2.9.

Table 2.9: Relation between clay activity and potential of expansion [18].

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

CHAPTER THREE

REVIEW ON SOIL STABILIZATION

3.1 Introduction

Generally, the long-term performance of any geotechnical structures depends on the soundness of the underlying soils. Unstable/expansive soils can create significant problems for pavements or structures. The black cotton soil is an expansive soil with low bearing capacity when it is subjected to moisture, has the ability to absorb and dissipate water with subsequent change in volume. Construction of any structure on this type of soil requires either replacement of the soil by importing a better foreign one or by addition of chemical(s) that will improve the soil towards the desired property. The successful construction of highways requires the construction of a structure that is capable of carrying the imposed traffic loads. One of the most important layers of the road is the actual foundation, or subgrade. Subgrade soil form the integral part of the road pavement structure as it provides the support to the pavement from beneath. The main function of the subgrade is to give adequate support to the pavement and for this; the subgrade should possess sufficient stability under adverse climate and loading condition. If these structures are founded on soil with low bearing capacity, they are likely to fail either during or after construction, with or without application of wheel load on them. Where the pavement is founded in an inherently weak soil, this material will be typically then removed and replaced with a stronger granular material or improving the soil towards the desired property by addition of chemical(s) [20]. This removal and replacement technique can be both costly and time consuming. Where aggregates are scarce, the use of these non-renewable resources is viewed as non-sustainable, particularly if haulage distances are significant. An alternative to the removal and replacement option is to chemically stabilize the host material. This eliminates the requirement to replace the material, and ensures the engineering characteristics and performance of the host material is enhanced to allow for its use within the pavement structure [20].

3.2 Soil Stabilization

Soil stabilization is the alteration of one or more soil properties, by mechanical or chemical means, to create an improved soil material possessing the desired engineering properties. The process may include blending of soils to achieve a desired gradation or mixing of commercially available additives that may alter the gradation, texture or plasticity, or act as a binder for cementation of the soil [33].

3.2.1 Uses of Stabilization

Pavement design is based on the premise that minimum specified structural quality will be achieved for each layer of material in the pavement system. Each layer must resist shearing, avoid excessive deflections that cause fatigue cracking within the layer or in overlying layers, and prevent excessive permanent deformation. As the quality of a soil layer is increase, the ability of that layer to distribute the load over a greater area generally increase so that a reduction in the required thickness of the soil and surface layers may be permitted. Commonly, improvement attained from soil stabilization can be summarized as; [33]

- Quality improvement: the most common improvements achieved through stabilization include reduction of plasticity index or swelling potential, and increases in durability and strength with a better soil gradation. In wet weather, stabilization may also be used to provide a working platform for construction operations [44].
- Thickness reduction: the strength and stiffness of a soil layer can be improved through the use of additives to permit a reduction in design thickness of the stabilized material compared with an unstabilized or unbound material. The design thickness can be reduced if the strength, stability and durability requirement of a base or subbase course is indicated to suitable by further analysis [33].

3.4 Types of Soil Stabilization

The two frequently used methods of stabilizing soils are stabilization by compaction or stabilization by chemical additives.

3.4.1 Mechanical Stabilization

Mechanical stabilization can be defined as a process of improving the stability and shear strength characteristics of the soil without altering the chemical properties of the soil. The main methods of mechanical stabilization can be categorized in to compaction, mixing or blending of two or more gradations, applying geo-reinforcement and mechanical remediation [33,39].

3.4.2 Chemical Stabilization

Soil stabilization using chemical admixtures is the oldest and most widespread method of ground improvement. Chemical stabilization is mixing of soil with one or a combination of admixtures of powder, slurry or liquid to improve or control its stability, strength, swelling, permeability and durability. Soil improvement by means of chemical stabilization can be grouped into three chemical reactions;cation exchange, flocculation agglomeration pozzolani c reactions [34,40].

a) Cation Exchange

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

$Na^{+}<Li^{+}<K^{+}<Rb^{+}<Cs^{+}<Mg^{2+}<Ca^{2+}<Ba^{2+}<Cu^{2+}<Al^{3+}$, [25].

Cation exchange includes an immediate reaction of the clay with the stabilizer within few minutes of mixing, resulting in a soil with improved texture. The tetrahedral (T) and octahedral (O) combination of clay minerals in 1:1 (1T and 1O) or 2:1 (2T and 1O) have charge deficiency that results in the attraction of the cations or water molecule. Generally, sodium or potassium (Na^+ or K^+) are prevalent in clay minerals along with water. However, these cations can be replaced by the higher valance cations like Al^{+3}, Ca^{+2}, Mg^{+2} etc. so called cation exchange. During this process calcium rich chemical stabilizer provides enough cations to replace the monovalent cations resulting in a reduced thickness of diffused double layer [25].The calcium is released in suspension of stabilizer-soil-water and will be available for the stabilization of soil.

The exchangeable cations may be present in the surrounding water or be gained from the stabilizers.

An example of the cation exchange;

$Ca^{2+} + Na^{+}\text{-Clay} \text{------------------}\blacktriangleright Ca^{2+}\text{-Clay} + Na^{+}$

The thickness of the diffused double layer decreases as replacing the divalent ions (Ca^{2+}) from stabilizers with monovalent ions (Na^+) of clay. Thus, swelling potential decreases [34,40].

b) Flocculation and Agglomeration

Cation exchange reactions result in the flocculation and agglomeration of the soil particles with consequent reduction in the amount of clay-size materials and hence the soil surface area, which inevitably accounts for the reduction in plasticity. Due to change in texture, a significant reduction in the swelling of the soil occurs [34,40].

Flocculation and agglomeration is the rearrangement of the clay particles from face to face orientation to more compact edge-face orientation. The fine grained soil changes to the more coarse grained with much more improved strength/stiffness as well as workability [17]. As cation exchange, flocculation and agglomeration is also a short-term process, which takes place within few hours of mixing the stabilizer and water with subgrade soil. The flocculation and agglomeration of clay particles is shown in Figure 3.1.

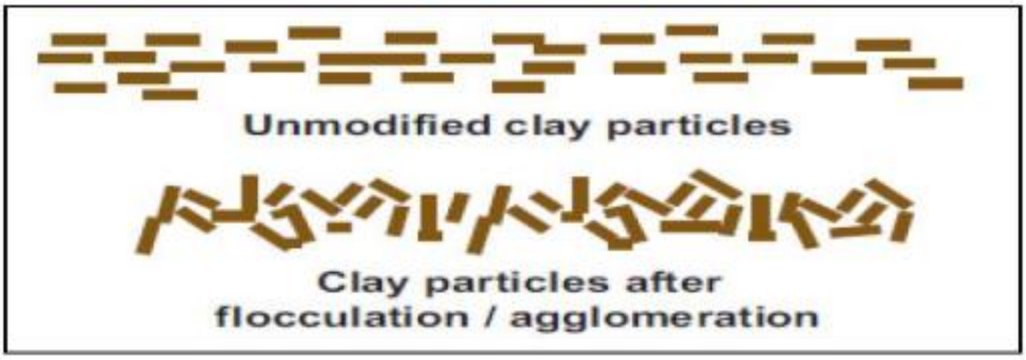


Figure 3.1: Flocculation and agglomeration of clay particles [17].

c) Pozzolanic Reactions

Time dependent pozzolanic reactions play a major role in the stabilization of the soil, since they are responsible for the improvement in the various soil properties. Pozzolanic reaction is a long-term process that produces more stable hydrates and aluminates of calcium after few months of mixing of soil, stabilizer and water. The PH environment in the system initiates further reaction of the silica and alumina with the clay particles, hence proving extra strength to the stabilized soils .The minimum PH of 12.4 is necessary in order to maintain the pozzolanic reaction [27]. Fly ash stabilization of the soil is similar to cement; however the strength provided is less than the cement. Depending upon the reactivity, the fly ashes are classified as self-cementing (**C-class**) and/or non-self-cementing (**F-class**). Generally, **C-class** fly ash is applied with either cement or lime, whereas F-class does not includes any activating stabilizers. Pozzolanic constituents produce calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH).

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

$Ca^{2+} + 2(OH)^{-} + Al_2O_3$ (Clay Alumina)----------------------► CAH

The calcium silicate gel formed initially coats and binds lumps of clay together. The gel then crystallizes to form an interlocking structure which increases the soil strength [26,36].

3.4.2.1 Lime Stabilization

Lime is one of the oldest and still popular additives used to improve fine-grained soils. Lime, either alone or in combination with other materials, can be used to treat a range of soil types. Lime treatment of soil facilitates the construction activity in three ways. First, a decrease in the liquid limit and an increase in the plastic limit results in a significant reduction in plasticity index. Reduction in plasticity index facilitates higher workability of the treated soil.

Second, as a result of chemical reaction between soil and lime a reduction in water content occurs. This facilitates compaction of very wet soils. Further, lime addition increases the optimum water content but decreases the maximum dry density and finally immediate increase in strength and results in a stable platform that facilitates the mobility of equipment [11].

3.4.2.1.1 Stabilization Mechanism

Laboratory testing indicates that lime reacts with medium, moderately fine and fine-grained soils to produce decreased plasticity, increased workability and increased strength [37]. Strength gain is primarily due to the chemical reactions that occur between the lime and soil particles. These chemical reactions occur in two phases, with both immediate and long-term benefits. The first phase of the chemical reaction involves immediate changes in soil texture and soil properties caused by cation exchange. The free calcium of the lime exchanges with the adsorbed cations of the clay mineral, resulting in reduction in size of the diffused water layer surrounding the clay particles. This reduction in the diffused water layer allows the clay particles to come into closer contact with one another, causing flocculation/agglomeration of the clay particles, which transforms the clay into a more silt-like or sand-like material. Overall, the flocculation and agglomeration phase of lime stabilization results in a soil that is more readily mixable, workable and ultimately compactable. According to [27] practically all fine-grained soils undergo this rapid cation exchange and flocculation/agglomeration reactions when treated with lime in the presence of water. The second phase of the chemical reaction involves pozzolanic reactions within the lime-soil mixture, resulting in strength gain over time. When lime is combined with a clay soil, the pH of the mixture increases. When the pH reaches 12.4, the silica and alumina from the clay become soluble and are released from the clay mineral. In turn, the released silica and alumina react with the calcium from the lime to form cemented material, which strengthens in a gradual process that continues for several years. As long as there is sufficient calcium from the lime to combine with the soluble silica and alumina, the pozzolanic reaction will continue as long as the pH remains high enough to maintain the solubility of the silica and alumina [36]. Strength gain also largely depends on the amount of silica and alumina available from the clay itself; thus, it has been found that lime stabilization is more effective for montmorillonitic soils than for kaolinitic soils [37].

3.4.2.1.2 Mixture Design and Strength Characteristics

When using lime as a stabilizer, the goal of the mixture design is to find the optimum lime content to adequately stabilize the soil to meet desired strength requirements. Strength requirements can vary from project to project depending on the intended use of the subgrade and the overall costs associated with construction. In some instances it may be desirable to achieve the strongest subgrade possible in order to minimize pavement thickness or increase service life of the pavement. In other instances, it may be desirable to reach a lower strength level that reduces the life cycle costs for the pavement. In still other instances, the goal may be to only improve the workability of the soils such that they are compactable. Therefore, a wide range of lime contents can be used to produce various desired results. The most common approach to mixture design is to determine the optimum lime content that provides the maximum strength. The optimum lime content is first estimated by measuring the pH of several soil-lime mixtures with varying lime contents. The lowest lime content that provides a pH of 12.4 is then used as the starting point for determining the optimum lime content. The Texas Procedure, as summarized by [21], first estimates the optimum lime content using the plasticity index of the soil and the percentage of soil passing the No. 40 sieve as indicated in Figure 3.2. After estimating of the optimum lime content, strength testing is then used to verify the actual optimum lime content.

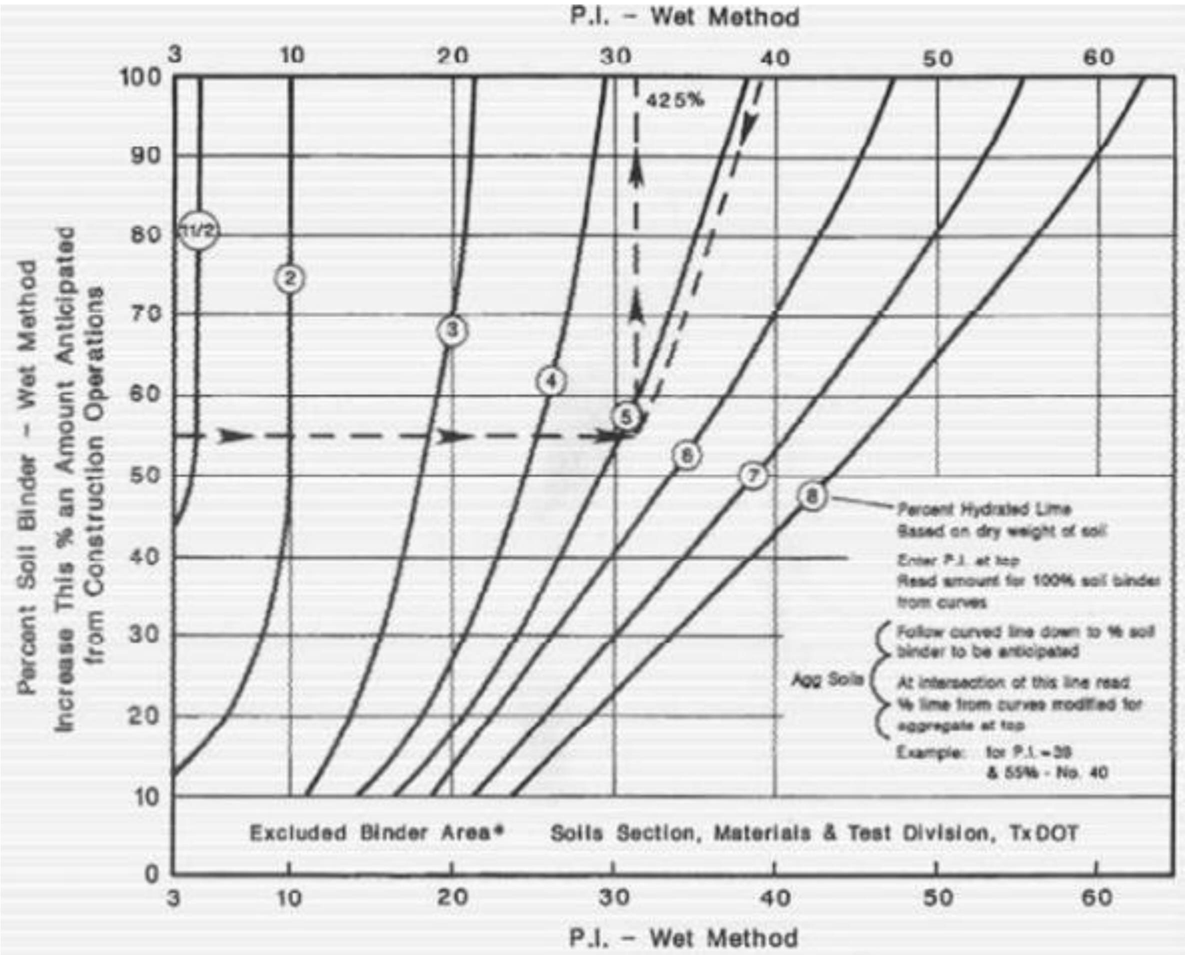


Figure 3.2: PI wet method to calculate amount of lime for stabilization [55].

3.4.2.1.3 Curing Time

Lime addition causes significant improvement in a short time in clayey soil properties by reducing plasticity and increasing strength of soil. [14] indicated that the optimum gain of strength in clay is achieved with the lime addition of 4-6%. The increase in strength is affected by the amount of mix water, the curing time length and the temperature at which curing takes place. When the mix water content is just in excess of the optimum moisture content, the gain in strength is the highest. According to the tests conducted by Bell, the most notable increase of strength occurs within the first 7 days [14,36].

3.4.2.1.4 Previous Studies

Tesfaye, A., (2001) studied improvement of expansive soil by addition of lime and cement on black cotton soil from different parts of Addis Ababa. Index properties, compaction characteristics and swelling pressure of soil-cement and soil-lime were determined using Atterberg limit test, moisture-density relations, free swell and swelling pressure tests. The conclusions and findings drawn from the study are;

- Expansive soil becomes moderately active to inactive based on the amount of lime and cement added.
- Swelling pressure of expansive soil decreases with increasing lime, cement and molding water content.
- 4-6% of lime and 9-12% of cement yielded significant improvement on plasticity and swelling properties of expansive soils.

Nebro, D., (2002) evaluated lime and liquid stabilizer called Con-Aid for stabilization of potentially expansive subgrade soil on samples collected form Addis-Jimma road which had indicated different pavement damages exacerbated by the presence of expansive soils. The experimental study involved Atterberg limit test, moisture-density relation, UCS, CBR and CBR swell. The findings and conclusions of the study can be summarized as follows:

Addition of lime reduced maximum dry density and increased the optimum moisture content.

- 4% of lime by dry weight of the soil was optimum lime content to stabilize the soil even though increased quantity of lime led to increased strength.
- Addition of lime reduces the swelling potential but no significant improvement in the engineering properties of the soil was attained by addition of Con-Aid.

Argu, Y., (2008) studied stabilization of light grey and red clay subgrade soil collected from Addis Ababa using SA-44/LS-40 chemical and lime. The experimental study involved Atterberg limit, moisture-density relation, swelling pressure and CBR tests. The conclusions and findings drawn from the study are;

- 8% lime yielded significant improvement on plasticity, swelling and strength properties of expansive soils.
- The applications of SA-44/LS-40 chemical alone are ineffective in improving the soaked CBR value of the red clay and light grey soils.

- The application of 0.30lit/m3 of SA-44/LS-40 chemical and 2% lime is an optimum proportion in increasing the soaked CBR value and reducing the swelling pressure of the light grey clay soil.
- The application of 0.08lit/m3 of SA-44/LS-40 chemical and 4% lime is an optimum proportion in increasing the soaked CBR value of the red clay soil.

Nigussie, E., (2011) evaluated the effect of sodium silicate and its combination with cement/lime for soil stabilization collected from Addis Ababa. The experimental study involved Atterberg limit, moisture-density relation and CBR tests. The conclusions and findings drawn from the study are;

- 6% lime yielded significant improvement on plasticity and strength properties of expansive soils.
- No significant improvement in the engineering properties of the soil was attained by addition of sodium silicate.

3.5 Industrial and Agricultural Waste as a Soil Stabilizing Material

Recent research works in the field of geotechnical engineering and construction materials focuses more on the search for cheaper and locally available materials, agricultural and industrial wastes, for use in construction industry. The use of different industrial and agricultural wastes has become a common practice in the construction industry. Fly ash, sugarcane waste ceramic powder, coconut husk ash and rice husk can be sited as examples, those by-products are increasingly playing a part in road construction and concrete technology, hence minimizing the problem of resource depletion, environmental degradation and energy consumption. This research focuses on the potential utilization of waste ceramic powder in soil stabilization, specifically expansive clay.

3.5.1 Waste Ceramic Powder

The ceramic powder is the principal waste of the ceramic industry which is generated as unwanted dust during the process of dressing, polishing and other processes. The ceramic waste powder have pozzolanic properties, which can be used to improve strength and durabili ty of expansive soil.

Presently, in ceramic industry the production goes as waste, which is not undergoing the recycle yet. The principle waste coming into the ceramic industry is the ceramic powder, specifically in the powder forms. Ceramic wastes are generated as a waste during the process of dressing and polishing. It is estimated that 15 to 30% waste are produced of total raw material used, and although a portion of this waste may be utilized on-site, such as for excavation pit refill, The disposals of these waste materials acquire large land areas and remain scattered all around, spoiling the aesthetic of the entire region. It is very difficult to find a use of ceramic waste produced.

3.5.1.1 Waste Ceramic Powder as a Soil Stabilizing Material

These days sustainability plays the major role in every aspect of human activities. Many technologies came to end because they were not in harmony with the idea of sustainable development. Sustainability is concerned about the world we will be leaving behind for future generations. It focuses on the social, environmental and economic issues of human activities. Therefore it requires every activity to be environmental friendly, economical and safe for the social.

Waste ceramic powder contains large amount of silica which is the most important component of cement replacing materials. Despite this abundance and silica content, relatively little has been done to examine the potential of this material for soil stabilization.

3.5.1.2 Previous Studies

Sandeep Sharma, VedParkash &Vishal Kumar (2017). Soil Stabilization of Clayey Soil using Ceramic Dust and Cement: In this thesis, the comparative study has been done of Soil-ceramic-cement mixture to find out the compaction characteristics, strength parameter and unconfined compressive strength of this soil mixed with different material at different percentage. Based on this the following conclusion can be made on the basis of test performed in laboratory:

1) From the proctor test the OMC & MDD of the parent soil are 22.727 % & 1.676 gm/cc respectively. It has been observed that the maximum dry density (MDD) increase by the addition of ceramic dust in parent soil at various percentage.

2) In Procter test, with increase in ceramic dust with cement and soil the Optimum Moisture Content value has been decreasing trend than the parent soil.

3) In UCS, with increase in ceramic dust with cement and soil the UCS value has been increasing trend than the parent soil.

Agil.R and A. Kumar (2017). Expermental study on cement replacement of cement with ceramic waste powder. This research work is concerned with the experimental investigation on strength of concrete and optimum percentage of the partial replacement by replacing cement via 10%, 15%, 20%, of ceramic waste. Keeping all this view, the aim of the analysis is to study the performance of concrete while replacing the ceramic waste with different proportions in concrete.

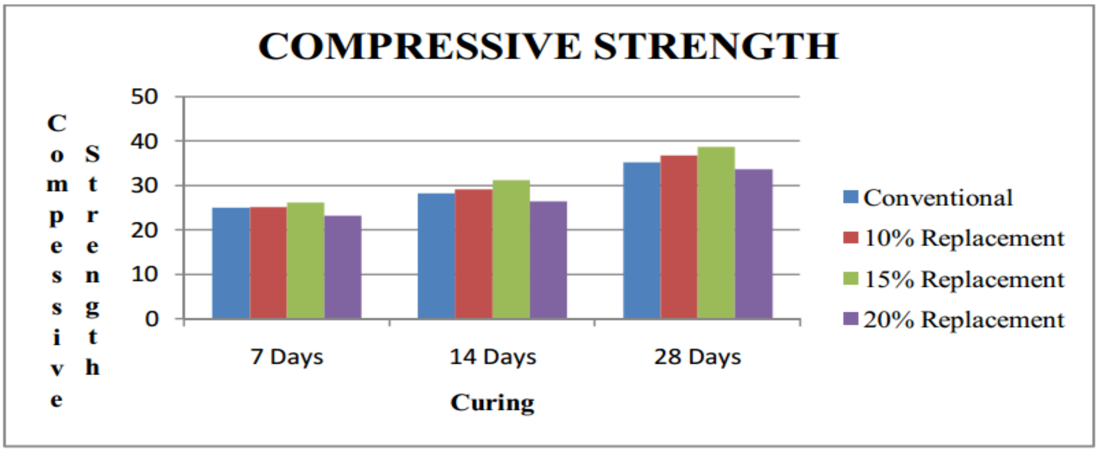


Figure3.3: compressive strength of concrete.

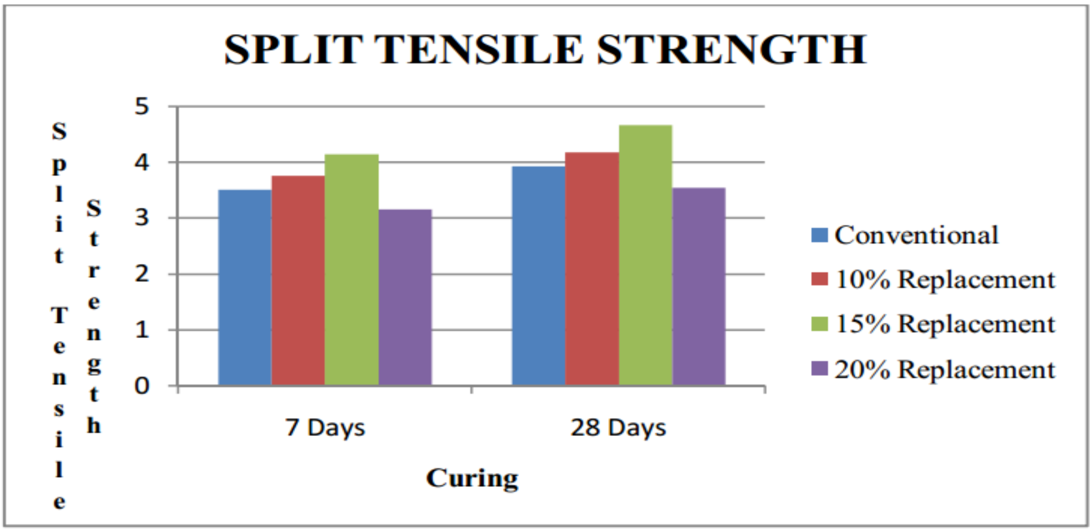


Figure 3.4 Tensile strength of concrete.

He conclude that the most optimal dosage for the partial alternative of cement by ceramic tile powder is 15 % for both compressive and tensile strength.

Amitkumar D. Raval, & eta al (2016). Use of Ceramic Powder as a Partial Replacement of Cement the OPC cement has been replaced by ceramic waste powder accordingly in the range of 0%, 10%, 20%, 30% 40%, & 50% by weight for M-25 grade concrete. The Compressive Strength of M25 grade concrete increased when the replacement of cement with ceramic waste up to 30% by weight of cement and further replacement of cement with ceramic powder the compressive strength decreased.

Ajay Upadhyay & Suneet Kaur (2016). Review on Soil Stabilization using ceramic waste with the addition of ceramic waste liquid limit, plastic limit and plasticity index of the clayey soil decreases.

- Optimum moisture content of the clayey soil decreases as the percentage of ceramic waste increases and maximum dry density obtained at certain optimum content of ceramic waste and decreases beyond this optimum content of ceramic waste.
- California bearing ratio of the clayey soil increases with the increase in the percentage of ceramic waste.
- The unconfined compressive strength of the clayey soil increases as percentage of ceramic waste dust increases.
- The differential free swell of clayey soil decreases as the percentage of ceramic waste increases.

Chen, James A. & Idusuyi, Felix O (2015). Studied the effects of waste ceramic powder on the index and engineering properties of expansive soils and obtained the following results. The MDD goes on increasing and OMC goes on decreasing with increase in percentage of addition of ceramic dust. The soaked CBR goes on increasing with increase in percentage of addition of ceramic dust. There is 150% increase in soaked CBR value as compared to untreated soil, when 30% ceramic dust was added. The Free swell and swelling pressure of the soil also decreased with increase in waste ceramic dust. With increase in percentage of WCD, the MDD of soil goes on increasing. The MDD increases from 15.6 kn/m^3 to18.1 kn/m^3 when WCD were increased from 0 to 30%. The reason of such behavior is due to replacement of WCD particles having high specific gravity (2.92) with soil particles having low specific gravity (1.9).

Table 3.1 Index and Engineering Properties of Stabilized Soil.

WCD (%)	**LL (%)**	**PL (%)**	**PI(%)**	**FS (%)**	**MDD (g/m^3)**	**OMC (%)**	**CBR (%)**
0	71	32	39	117	15.6	20.4	1.6
5	65	29	36	108	15.8	19.8	1.8
10	54	26	28	101	16.1	19.4	2.1
15	51	24	27	67	16.5	19	2.5
20	47	22	25	52	17.2	18.5	2.8
25	43	20	23	44	17.5	18.2	3.5
30	35	18	17	36	18.1	17.6	4.1

Source: Field Data (2015)

Key: WCD: Waste Ceramic Dust, LL: Liquid Limit, PL: Plastic Limit, PI: Plasticity Index, FS: Free Swell, SP; Swell Pressure, MDD; Maximum Dry Density, OMC: Optimum Moisture Content, CBR: California Bearing Ratio.

From the construction waste management and economic point of view, it is recommended that up to 30% of waste ceramic dust can be utilized for strengthening the sub grade of flexible pavement with a substantial save in cost of construction.

T.G. Rani, G.V.R. Prasada Raju &eta al (2014). Carried out studies on an expansive soil mixed with tile waste from 0 to 30% at an increment of 10%. From the analysis of test results it was found that index properties, OMC, and swelling pressure decreased, MDD and CBR increased with an increase in tile waste.

Prasad eta al. (2014). Evaluated the effect of tile waste on clayey soil. The clayey soil available locally was blended with tile waste from 0 to 30% at an increment of 10%. The liquid limit and plastic limit were decreased irrespective of the percentage of addition of tile waste. The MDD attained at 20% tile waste and OMC was decreasing with increase in percentage of tile waste. The soaked CBR was increased with increase in percentage of addition of tile waste. The CBR value has increased by 105% as compared to untreated soil, when 20% tile waste was mixed. There was 48 % decrease in swelling pressure of soil as compared to untreated soil, when 20% tile waste was added. From the above analysis it was found that tile waste up to 20% can be utilized for strengthening the clayey soil sub grade of flexible pavement with considerable save in cost of construction.

Sabat and Bose (2014). Studied the effects of ceramic dust on compaction properties, UCS, CBR, Ps and durability of fly ash-lime stabilized expansive soil. The optimum percentages of fly ash, lime and ceramic dust were found to be 10%, 5% and 35% respectively with improvement in strength, swelling and durability characteristics.

Sabat (2012). Stabilized expansive soil mixed with waste ceramic dust. The locally available clayey soil was mixed with ceramic dust from 0 to 30% with an increment of 5%. Effect of ceramic dust on consistency limits, compaction characteristics, unconfined compressive strength, California bearing ratio, shear strength parameters and swelling pressure of clayey soil was evaluated. From the results of tests it was found that liquid limit decreases from 62% to 35%, plastic limit decreases from 30% to 20%, PI decreases from 32% to 15%. The compaction characteristics were also improved. The MDD increases from 15.6 kn/m^3 to 18.1 kn/m^3, OMC decreased from 20.4% to 17.6%. The UCS increases from 55 kn/m^2 to 98kN/m^2. The soaked CBR value increased from 1.6% to 4%. There was 150% increase in soaked CBR value. The cohesion decreased from 18 kn/m^2 to 13.5 kn/m^2. When ceramic dust increases from 0 to 30% the angle of internal friction increased from 130 to 17.70. The swelling pressure decreases from 130 kn/m^2 to 24 kn/m^2 when ceramic dust increases from 0 to 30%. The economic analysis for stabilized was conducted and it was found that ceramic dust up to 30% can be utilized for strengthening the sub grade of flexible pavement with a substantial save in cost of construction.

3.5.1.3 Availability of Waste Ceramic Powder in Ethiopia

Production of waste ceramic powder can be obtained from the following ceramic industry

1. Ethio Ceramic PLC
2. East African Zone (DIYUANN CERAMICS PLC)
3. Arerity Industry Park
4. Tabor Ceramic PLC
5. Other Industry etc.

To respond to the Belt and Road strategy of Chinese government, DIYUANN CERAMICS PLC was established in March 2016 in Ethiopia with the planning amount of 218 million USD invested by China Rongguang Group, Henan Rongqing Industry Co.Ltd and Hong Kong Huazheng Enterprise Limited. It specializes in the production of wall and floor tiles, sanitary wares, pottery and other ceramic products. The factory is over

400,000 m^2 in Eastern Industrial Zone, Oromia Region, Ethiopia. The production capacity is 60,000 m^2 per day. They use the resourse material which come from around Mojo and Debrezeyit like Talk, Feldspar, Calcite, Quartz, Muscovite and other material which is important for the ingredient of ceramic production.

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CHAPTER FOUR

MATERIALS AND METHODS

4.1 Introduction

In this chapter, the materials used and the methods adopted for the research are described with respect to their sources.

4.2 Study design

The research study was conducted by using both experimental and analytical method.

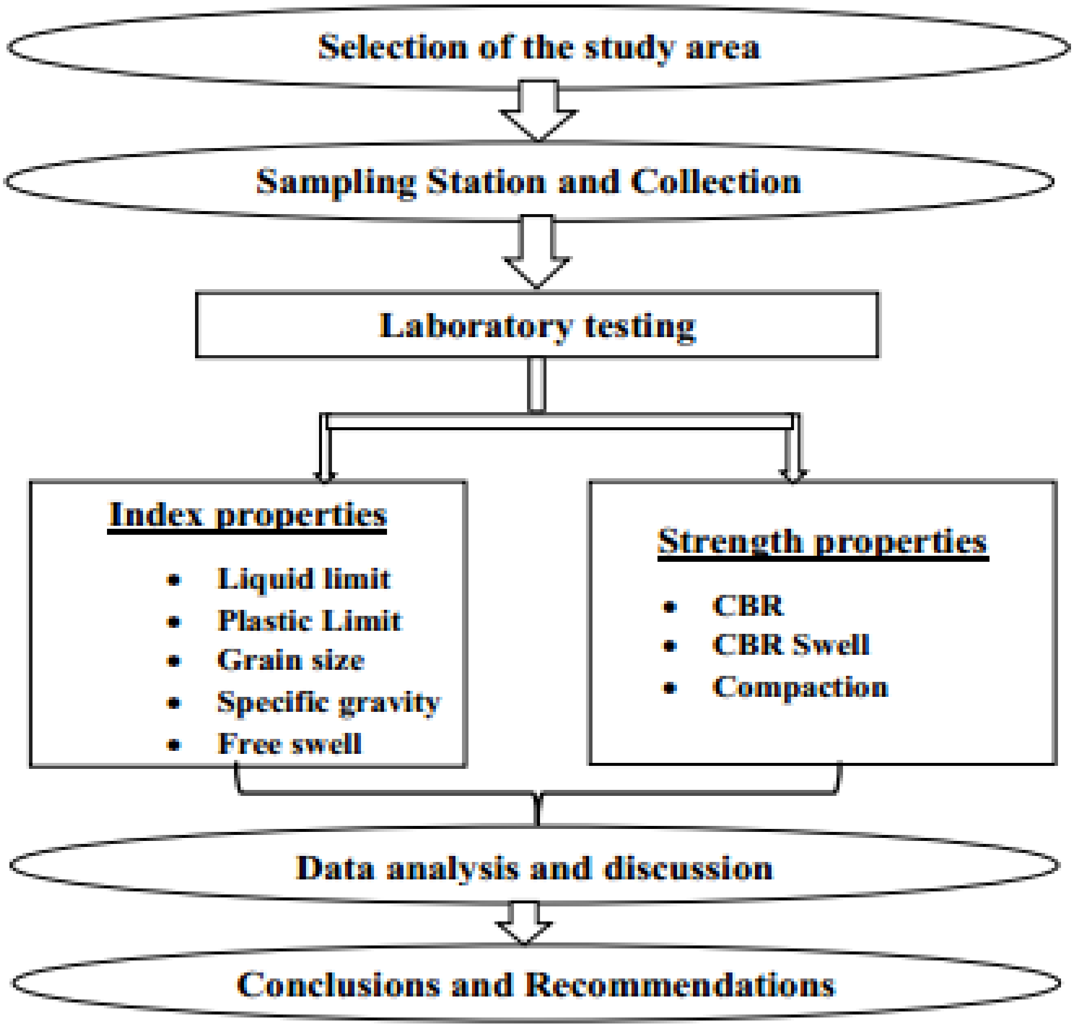


Figure 4.1: Flow chart of the study.

4.3 Materials

4.3.1 Expansive Soil and Its Location

The expansive soil sample used for this research work is collected from Addis Ababa, around Goro Ethio ICT Village at 8º 58’ 12.42’’N and 38º 50’ 38.39’’E from two stations. The soil is grayish black in colour highly plastic clay. Disturbed soil samples are collected from two stations the two stations are 100 meter apart from each other at a depth 1m below NGL in order to avoid the inclusion of organic matter. The sample stations are shown in Picture 4.2 a.

a) The two Sample stations which are 100m apart each other.

b) Ethio ICT Village

c) East African Industry (DIYUAN CERAMIC PLC) Around Dukem.

Picture 4.2: Location of Study area and source of Waste Ceramic Powder.(Source Ethiopia-Google map)

Picture 4.3: An overview of the sample station.

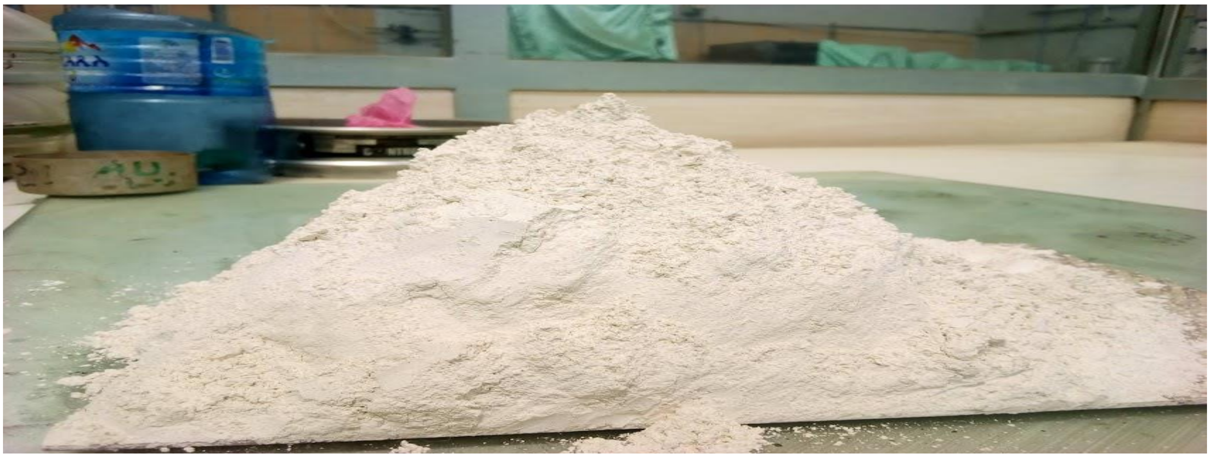
4.3.2 Waste Ceramic Powder

Waste ceramic powder as shown in Picture 4.4 was obtained from East African Industry (DIYUANN CERAMIC PLC) which is located in Oromiya Regional State around Dukem. For this research, 15kg waste ceramic was taken from the wastage accumulated and change it to powder using grinder. Then, it was properly packed in sacks and transported to the laboratory.

Picture 4.4:Views from waste ceramic powder from disposed site.

4.3.3 Lime powder

The lime used in this study was purchased from the open market from authorized dealers in Addis Ababa.



Picture 4.5:Lime Powder.

4.4 Methods

4.4.1 Sample Preparation

Prior to treatment and testing, the soil samples were prepared in accordance with the method described in AASHTO T87-86. This method involves:

- Air drying of samples and/or oven drying at 60ºC or less;
- Breaking up the soil aggregates by rubber covered mallet. Then, sieve analysis is performed to separate the dried soils into two groups. The first group involves preparing uniform samples for Atterberg limits, free swell, free swell index and free swell ratio tests. And the other for compaction and California bearing ratio tests.
- Then, soil and waste ceramic powder are mixed manually to get uniform mix ratio for each test.

4.4.2 Moisture Content

The test is conducted in accordance with AASHTO T265 Small representative sample of the natural soil and soil- waste ceramic powder mixture specimens are obtained and oven-dried at 105 ± 5°C for at least 12 hours. The samples were then reweighed, and the difference in weight was assumed to be the weight of the water driven off during drying. The difference in weight was divided by the weight of the dry soil, giving the water content of the soil a dry weight basis.

4.4.3 Atterberg Limits Testing

The test includes the determination of the liquid limits, plastic limits and the plasticity index for the natural soil and the soil- waste ceramic powder mixtures. The tests are conducted for uncured and 7 days cured stabilized soil samples in accordance with AASHTO T89-90 and T90-96 testing procedures.

4.4.3.1 Liquid Limit

The soil sample for liquid limit is air dried and 200g of the material passing through No. 40 sieve (425µm aperture) was obtained and thoroughly mixed with water to form a homogeneous paste on a flat glass plate. A portion of the soil water mixture is then placed in the cup of the Casagrande apparatus, leveled off parallel to the base and divided by drawing the grooving tool along the diameter through the centre of the hinge. The cup is then lifted up and dropped by turning the crank until the two parts of the soil come into contact at the bottom of the groove. The number of blows at which that occurred was recorded and a little quantity of the soil was taken and its moisture content determined. The test is performed for well–spaced out moisture content from the drier to the wetter states. The values of the moisture content (determined) and the corresponding number of blows is then plotted on a semi–logarithmic graph and the liquid limit is determined as the moisture content corresponding to 25 blows. The same procedure is also carried out for the treated soil with increment of waste ceramic powder content.

4.4.3.2 Plastic Limit.

A portion of the natural soil and the soil–waste ceramic powder mixture used for the liquid limit test is retained for the determination of plastic limit. The ball of the natural soil and the soil–waste ceramic powder mixture is moulded between the fingers and rolled between the palms of the hand until it dried sufficiently, even though the soil is already relatively drier than the ones used for liquid limit. The sample is then divided into approximately two equal parts. Each of the parts is rolled into a thread between the first finger and the thumb. The thread is then rolled between the tip of the fingers of one hand and the glass. This continued until the diameter of the thread is reduced to about 3mm. The movement continued until the thread shears both longitudinally and transversely. The crumbled natural soil and soil– waste ceramic powder mixture is then put in the moisture container and the moisture content determined. The same procedure is also carried out for the treated soil with increment of waste ceramic powder content.

4.4.3.3 Plasticity Index

The plasticity index of the natural soil and the soil–waste ceramic powder mixture is the difference between the liquid limits and their corresponding plastic limits. The plasticity indexes of the samples are calculated as:

$$\mathbf{PI = LL - PL} \qquad (4.1)$$

4.4.4 Particle Size Distribution

The test includes the determination of the particle size distribution for the natural soil. The tests are conducted in accordance with AASHTO T88-93 testing procedures. Approximately, 50gm of dry soil passing No. 200sieve is treated with a dispersing agent for 18 hours. A hydrometer analysis is then performed to measure the amount of silt and clay size particles. The sample is then washed through a series of sieves with progressively smaller screen sizes to determine the percentage of sand-sized particles in the specimens.

4.4.6 Free Swell Index

The test includes the determination of the free swell index of the natural soil and the soil waste ceramic powder mixture. The tests are conducted in accordance with IS: 2720 (Part 40) 1977 testing procedure. Two samples of oven dried soil 10cc each, passing through 425 micron sieve are taken. One is put in a 100cubic centimeters graduated glass cylinder containing kerosene. The other sample is put in a similar cylinder containing distilled water. Both the samples are left undisturbed for 24 hours and then their volumes are noted. Then free swell index is determined using Equation (2.2). The same procedure is also carried out for the treated soil with increment of waste ceramic powder content.

4.4.7 Free Swell Ratio Test

The test procedure followed for the determination of free swell ratio is in accordance with IS: 2720 (Part 40) 1977. In this study, recommended procedure of Sridharan and Prakash is adopted.10gm oven dried soil passing through 425 micron is added to 100ml of distilled water in a jar and another 10gm of same sample is added to 100ml of Kerosene. After 24hours, sediment volumes of samples are measured to determine free swell ratio. Free swell ratio is the ratio of change in volume in water to change in volume in kerosene after 24 hours. Then free swell ratio is determined using Equation (2.3). The same procedure is also carried out for the treated soil with increment of waste ceramic powder content.

4.4.8 Free Swell Tests

The test includes the determination of the free swell for the natural soil and soil- waste ceramic powder mixture. This test has not yet been standardized by AASHTO and ASTM. The method was suggested by Holtz and Gibbs, (1956) to measure the expansive potential of cohesive soils. The free swell test gives a fair approximation of the degree of expansiveness of the soil sample. The procedure consists of pouring very slowly of 10 cubic centimeters of that part of the dry soil passing No. 40 sieve in to a 100 cubic centimeters graduated measuring cylinder and letting the content stand for approximately 24 hours until all the soil completely settles on the bottom of the graduating cylinder. Then the final volume of the soil is noted. Finally, free swell value is calculated using Equation (2.1).

4.4.9 Specific Gravity

Specific gravity which is the measure of heaviness of the soil particles are determined by the method of pychnometer method using a soil sample passing No. 10 sieve and oven dried at 105 degree centigrade. The test includes the determination of the specific gravity for the natural soil and the soil waste ceramic powder mixture. The test is conducted in accordance with AASHTO T100-93 testing procedure.

4.4.10 Compaction

The test includes the determination of the maximum dry density and the optimum moisture content for the natural soil and the soil-waste ceramic powder mixture. The tests are conducted for uncured and 7days cured stabilized soil samples in accordance with AASHTO T99-94 testing procedures.

4.4.10.1 Maximum Dry Density

The maximum dry density is conducted for both the natural and soil- waste ceramic powder mixture of about 2.5kg, by varying the moisture content. The sample is then compacted into the 944 cubic centimeters (of mass m1); in three layers of approximately equal mass with each layer receiving 25 blows. The blows are uniformly distributed over the surface of each layer. The collar is then removed and the compacted sample leveled off at the top of the mould with a straight edge. The mould containing the leveled sample is then weighed to the nearest 1g; m^2One small representative sample is then taken from the compacted soil for the determination of moisture content. The same procedure is repeated until minimum of five sets of samples are taken for moisture content determination. The bulk density is then calculated for each compacted specimen using:

$$\gamma_b = \frac{(m1-m2)}{944} \qquad (4.2)$$

Where;

m_1= mass after compacted the soil with mould

m_2= mass of the mould.

$\gamma_b =$ Bulk unit weight

The dry density is also calculated using the following equation:

$$\gamma_d = \frac{(\gamma b)}{(1+\frac{\omega}{100})} \qquad (4.3)$$

Where;

ω is the moisture content of each compacted specimen.

$\gamma_d =$ Dry unit weight

$\gamma_b =$ Bulk unit weight

The values of the dry densities as obtained from equation above are plotted against their respective moisture contents and the dry densities, MDD is deduced as the maximum point on the resulting curves as shown in Appendix 4.

4.4.10.2 Optimum Moisture Content

The corresponding value of moisture contents at maximum dry densities, which is deduced from the graph of dry density against moisture content, gives the optimum moisture content.

4.4.11 CBR and CBR-swell

The CBR and CBR-swell tests are conducted in accordance with AASHTO T193-93 for the natural soils and soil-waste ceramic powder mixture. For stabilized soil samples tests were conducted for uncured and 7 days cured soil samples. The CBR is expressed by the force exerted by the plunger and the depth of its penetration into the specimen; it is aimed at determining the relationship between force and penetration. 5.0kg of the natural soil and the soil-waste ceramic powder mixture are mixed at their respective optimum moisture contents in 2124 cubic centimeters mould. The samples are compacted in three layers with 56 blows from the 2.5kg rammer. The CBR test indirectly measures the shearing resistance of a soil under controlled moisture and density conditions. The CBR is obtained as the ratio of load required to affect a certain depth of penetration of a standard penetration piston into a compacted specimen of the soil at some water content and density to the standard load required to obtain the same depth of penetration on a standard sample of crushed stone.

In equation form, this is:

CBR= (test load on the sample/ standard load on the crushed stone)*100 % (4.4)

CBR tests were conducted on the compacted specimens at the optimum moisture content using standard compaction test. The compacted soil samples of the CBR mold are soaked for 96 hours in a water bath to get the soaked CBR value and the CBR swell of the soil. The CBR swell of the soil is measured by placing the tripod with the dial indicator on the top of the soaked CBR mold. The initial dial reading of the dial indicator on the soaked CBR mold is taken just after soaking the sample. At the end of 96 hours the final dial reading of the dial indicator is taken hence the swell percentage of the initial sample length is given by:

$$\text{CBR Swell} = \left(\frac{\text{Change in height in mm during Soaking}}{116.3\text{mm}}\right) \times 100\,\% \qquad (4.5)$$

For which 116.4mm is the height of soil specimen.

CHAPTER FIVE

TEST RESULTS AND DISCUSSIONS

5.1 Introduction

This chapter presents the results of laboratory tests and a discussion pertinent to the results. The relevant engineering properties of the soil are evaluated both for natural and stabilized soil samples separately. The tests include Atterberg limits, free swell, free swell index, free swell ratio, moisture density relationship (compaction) and California bearing ratio (CBR). Atterberg limits, moisture density relationship (compaction) and CBR are conducted for uncured and 7 days cured soil samples.

5.2 Properties of Material Used in the Study

5.2.1 Natural Soil

The results of the tests conducted for identification and/or determination of properties of the natural soil before applying waste ceramic powder are presented in Table 5.1. The soil is grayish black in color. As shown in Figure 5.3 on the particle size distribution curve almost 90.5% of the soil is passing through No. 200 sieve; it exhibits a liquid limit of 106.5%, a plastic limit of 44% and plasticity index of 64.5%.Liquid limit less than 35% indicates low plasticity, between 35% and 50% intermediate plasticity, between 50% and 70% high plasticity and between 70% and 90% very high plasticity [44], Hence, these values indicate that the soil is highly plastic clay. Accordingly the soil falls under the A-7-5 soil class based on AASHTO soil classification system. Soils under this class are generally classified as a material of poor engineering property to be used as a sub-grade material. Results that are related to swelling characteristics of the soil are also indicate that the soil is highly expansive clay with a free swell of about 140% and free swell index of 120%. The soil has a maximum dry density of 1.29 g/cm^3, optimum moisture content of 32.5%, unsoaked CBR value of 19.5% and soaked CBR value of 1.02%. The general relationship between CBR values and the quality of the subgrade soils used in pavement applications is as follows [14].

CBR-values	**Quality of Subgrade**
0 – 3 %	very poor subgrade
3 – 7 %	poor to fair subgrade
7 – 20 %	fair subgrade
20 – 50 %	good subgrade
> 50	excellent subgrade

Hence, the soil was found to be highly plastic expansive clay with low bearing capacity when it is soaked and high swelling potential and fell below the standard recommendations for most geotechnical construction works especially highway construction. Therefore, the soil requires initial modification and/or stabilization to improve its workability and engineering property.

Picture 5.1: Soil Sample From Station One.

Table 5.1:Geotechnical properties of the natural soil for station 1.Below 1m from NGL

Property	Quantity for Station one
Percentage passing No.200 sieve, %	90.5
Liquid limit, %	106.5
Plastic limit, %	44
Plasticity index, %	64.5
AASHTO soil classification	A-7-5
Specific gravity	2.70
Free swell, %	140
Maximum dry density, g/cm3	1.29
Optimum moisture content, %	32.5
Soaked CBR value, %	1.02
Unsoaked CBR value, %	19.5
CBR-swell,%	9.2
Colour	Grayish black

Picture 5.2: Soil Sample From Station Two.

Table 5.2 :Geotechnical properties of the natural soil for station 2. Below 1m from NGL

Property	Quantity
Percentage passing No.200 sieve, %	88.74
Liquid limit, %	100
Plastic limit, %	43
Plasticity index, %	57
AASHTO soil classification	A-7-5
Specific gravity	2.7
Free swell, %	127.5
Maximum dry density, g/cm3	1.32
Optimum moisture content, %	36
Soaked CBR value, %	1.4
Unsoaked CBR value, %	18.4
CBR-swell,%	9.2
Colour	Grayish black

From the two test station I used station one for taking soil sample for stabilizing with waste ceramic powder because station one is slightly higher expansive than station two by controlling swelling test.

5.2.2 Geotechnical properties of Waste Ceramic Powder

Locally collected waste ceramic powder from the industry around Dukem East African Industry was used in the experiment. The geotechnical properties of the waste ceramic powder was used in the experimental programme are given below. As shown in Table 5.5.The main composition element of waste ceramic powder are SiO_2(67.51%) and Al_2O_3 (16.92%) which give a total of 84.43% which is greater than 70% which implies it satisfys the criteria of Pozzolan as shown at Appendix 6.

Table 5.3: Geotechnical Properties of Waste Ceramic Powder.

No.	Properties	Value
1	**Grain size Analysis**	
	% passing Sieve No.200	84.8
2	Free Swell (%)	5.02
3	Specific Gravity	3.06
4	**Atterberg Limit**	
	Liquid limit (%)	28.5
	Plastic limit (%)	18.4
	Plastic index (%)	10.1
5	**Compaction Characteristics**	
	Optimum Moisture Content	14.8
	Maximum Dry Density	2.58
6	CBR (%)	12.9
7	CBR Swelling (%)	0.132

From the above conducted test the waste ceramic powder fall under non plastic group. Details of the test results are shown in Appendix 1.

Table 5.4: Geotechnical Properties of Lime

No.	Properties	Value
1	Free Swell (%)	2.50
2	Specific Gravity	1.86
3	**Atterberg Limit**	
	Liquid limit (%)	4.5
	Plastic limit (%)	2.8
	Plastic index (%)	1.7
4	**Compaction Characteristics**	
	Optimum Moisture Content	24.5
	Maximum Dry Density	1.18
5	CBR (%)	39.6
6	CBR Swelling (%)	0.18

From the above conducted test the waste ceramic powder fall under non plastic group. Details of the test results are shown in Appendix 2.

Table 5.5: Oxide Composition of Waste Ceramic Powder.(Source from Ethiopian Geological Survey)

Constituents	% Composition
SiO_2	67.51
Al_2O_3	16.92
Fe_2O_3	0.75
CaO	3.13
MgO	1.82
K_2O	1.31
Na_2O	4.8
P_2O_5	0.023
SO_3	0.01
Cl_2	0.043
MnO	0.12
TiO_2	0.04
L.O.I	1.2

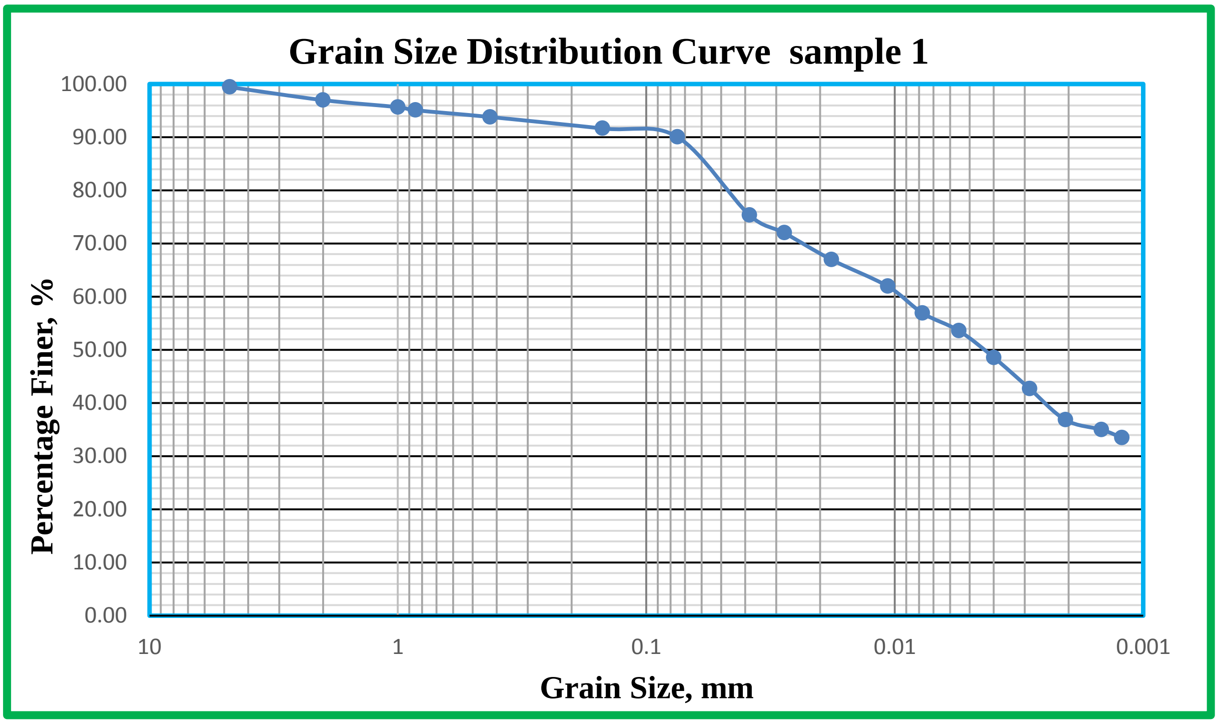


Figure 5.3: Particle size distribution curve of the expansive soil

5.3. Effect of Waste Ceramic Powder on Soil classification

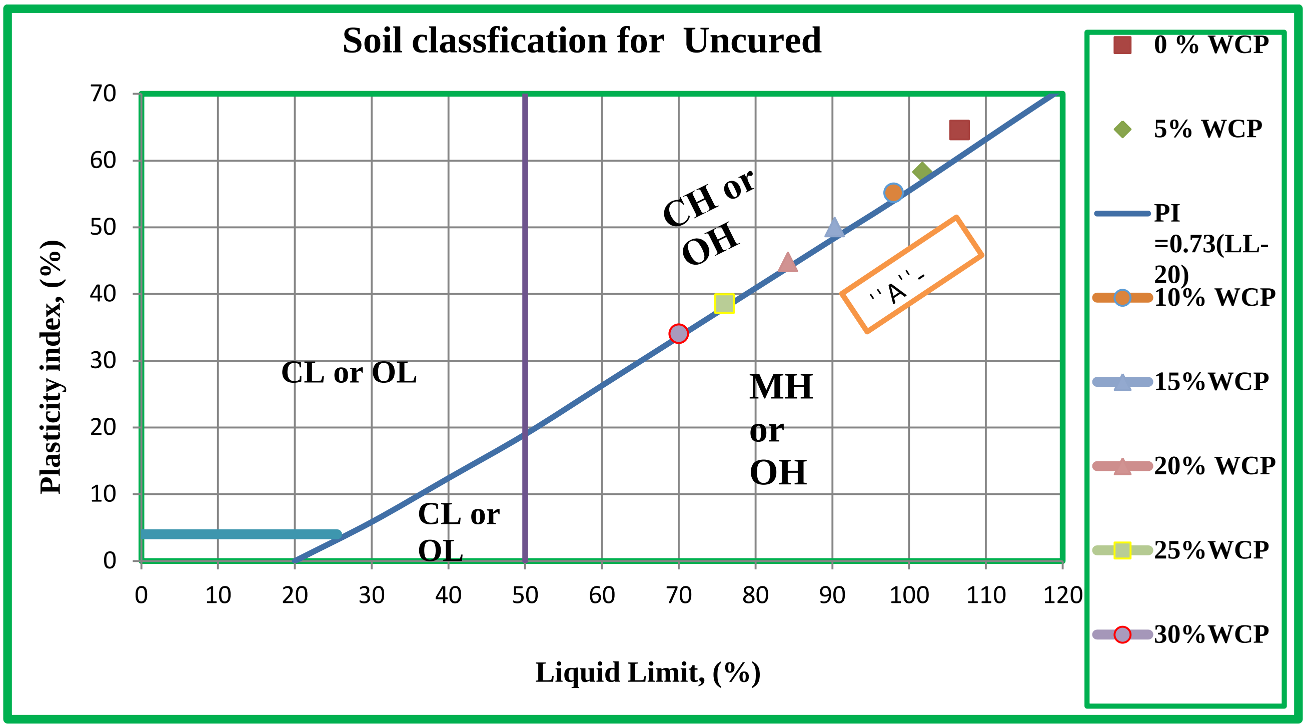


Figure 5.4: Plasticity chart indicating the positions of soil with waste ceramic powder for Uncured.

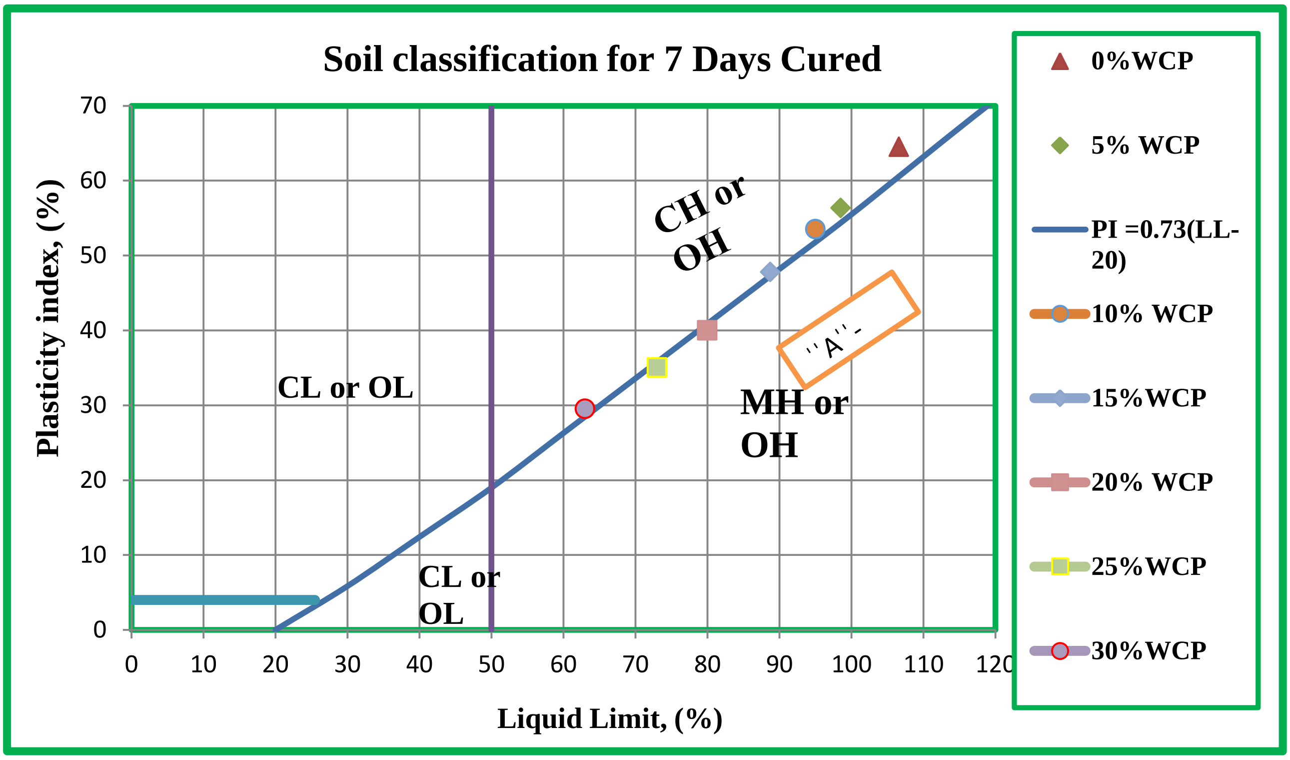


Figure 5.5: Plasticity chart indicating the positions of soil with waste ceramic powder.

As shown in Fig 5.5, the stars indicate where the relation between the soil's plasticity index and liquid limit lies on the graph and Slightly below and above the A line graph for each step of waste ceramic powder.

5.4. Effect of Waste Ceramic Powder on Atterberg Limits.

The effect of waste ceramic powder on the plasticity index of the soil is shown in Figure 5.6 for both uncured and 7 days cured soil samples. As shown in the figure plasticity index generally decreased with increment in waste ceramic powder content. As we seen from the graph, the addition of waste ceramic powder decreases the plasticity index of the expansive soil. The decrease is observed to be more with the increase in the quantity of waste ceramic powder up to 30%.

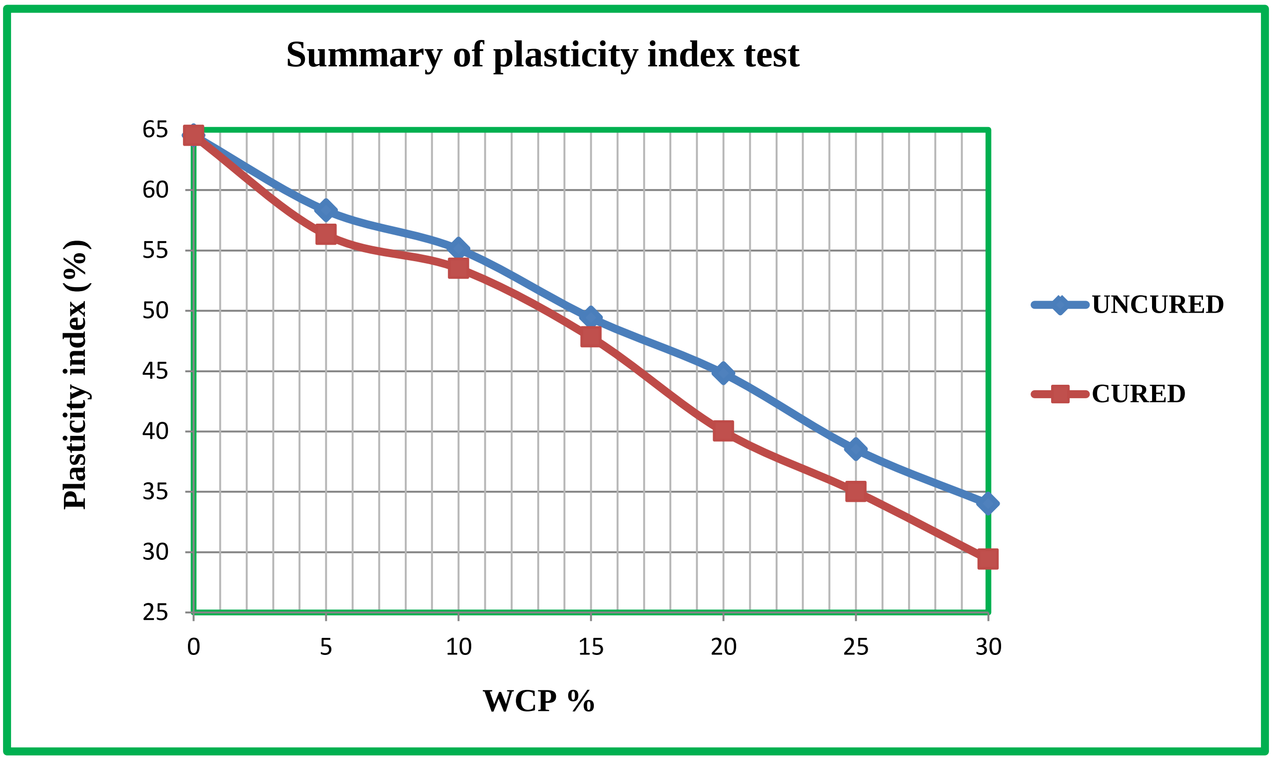


Figure 5.6: Variation of plasticity index with addition of different waste ceramic powder contents

In general, the plasticity of the soil is decreased by the addition of waste ceramic powder content. This is clearly shown by the fact that plasticity index of treated soil decreased with increasing additive quantity. These effects are due to the partial replacement of plastic soil particles with waste ceramic powder which is non plastic material and flocculation and agglomeration of clay particles caused by cation exchange may be the other cause. Details of the Atterberg limit test results are shown in Appendix 3.

5.5 Effect of waste ceramic powder on Swelling Characteristics

5.5.1 Free Swell

The effect of waste ceramic powder on the free swell the soil under investigation is shown in the Figure 5.7, the reduction in free swell is directly proportional to the quantity of waste ceramic powder. The highest reduction in free swell is attained when the expansive soil is treated with 30% waste ceramic powder which is 106 % reduction compared to untreated in the Figure 5.7, the reduction in free swell is directly proportional to the quantity of waste ceramic powder.

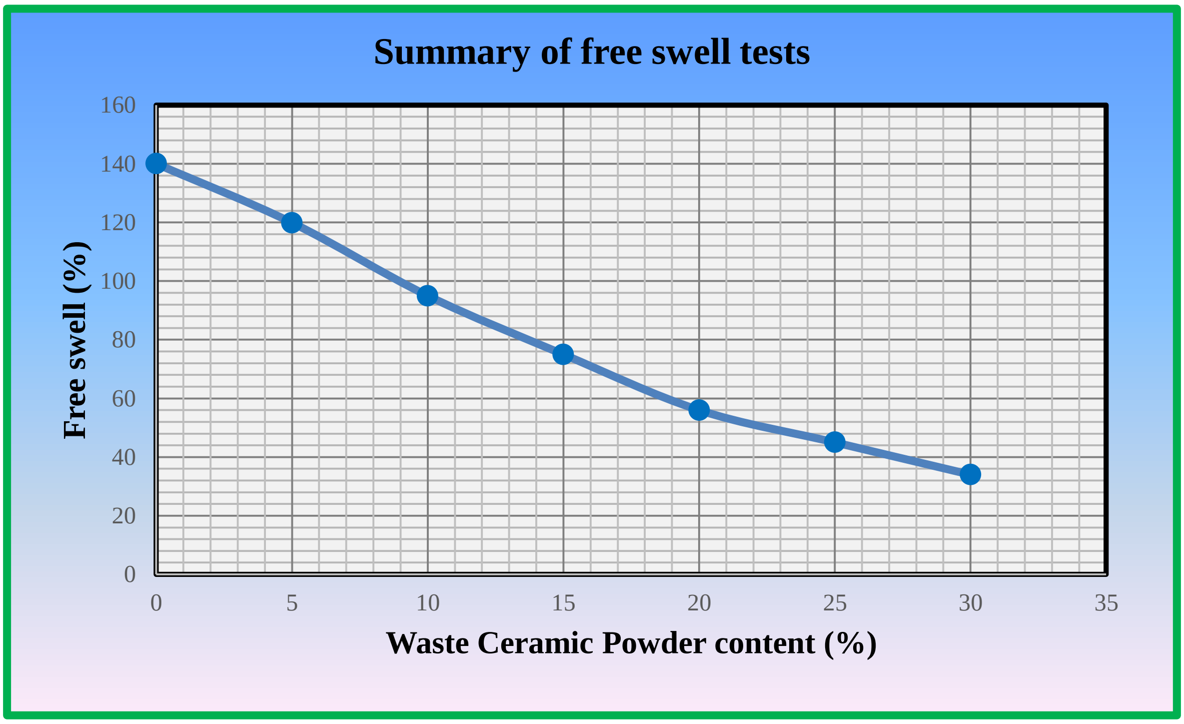


Figure 5.7: Changes in the free swell index with varying percentage of waste ceramic powder.

From the above swelling characteristics (free swell index) the decrease in swelling are mainly due to;

- Waste ceramic powder decreases swell potential of expansive soils by replacing some of the volume that is previously held by expansive clay minerals and by cementing the soil particles together.

- The reason may be due to cation exchange in the waste ceramic powder-soil mix.

5.5.2 Free Swell Index

The effects waste ceramic powder on the free swell index of the expansive soil is shown in figure 5.8 .The free swell index value decreased from 120% to 18% with increased waste ceramic powder content from 0% to 30%. From the analysis of test results the free swell index of expansive soil is 120%. According to swell classification based on free swell index in Chapter two section 2.7.2.2. Table.2.3 the soil has high degree of expansion. As the percentage of waste ceramic powder increases, the free swell index value decreases and decreased up to 34% at 30% waste ceramic powder and makes the soil to be moderate swelling.

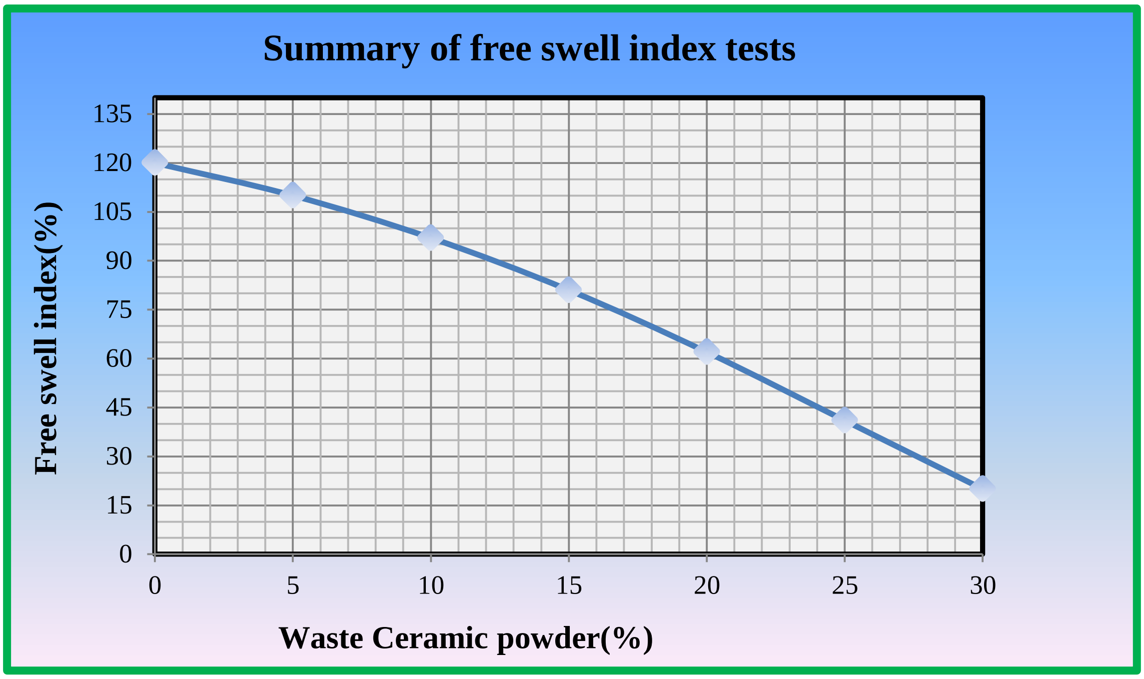


Figure 5.8: Changes in the free swell index with varying percentage.

From the above swelling characteristics (free swell index) the decrease in swelling are mainly due to;

- Waste ceramic powder decreases swelling potential of expansive soils by replacing some of the volume that is previously held by expansive clay minerals and by cementing the soil particles together.
- The reason may be due to cation exchange in the waste ceramic powder-soil mix.

5.5.3 Free Swell Ratio

As it is shown in Figure 5.9 when waste ceramic powder is added to the soil the free swell ratio decreases. The free swell ratio decreases from 3.1 to 1.52 with increased waste ceramic powder content from 0% to 30%. From the analysis of test results the free swell ratio of expansive swell classification based on free swell ratio in Chapter two in section 2.7.2.2 Table 2.2, the soil is considered as high swelling soil. As the percentage of waste ceramic powder increases, the free swell ratio value decreased up to 1.52 at 30% waste ceramic powder and makes the soil medium swell.

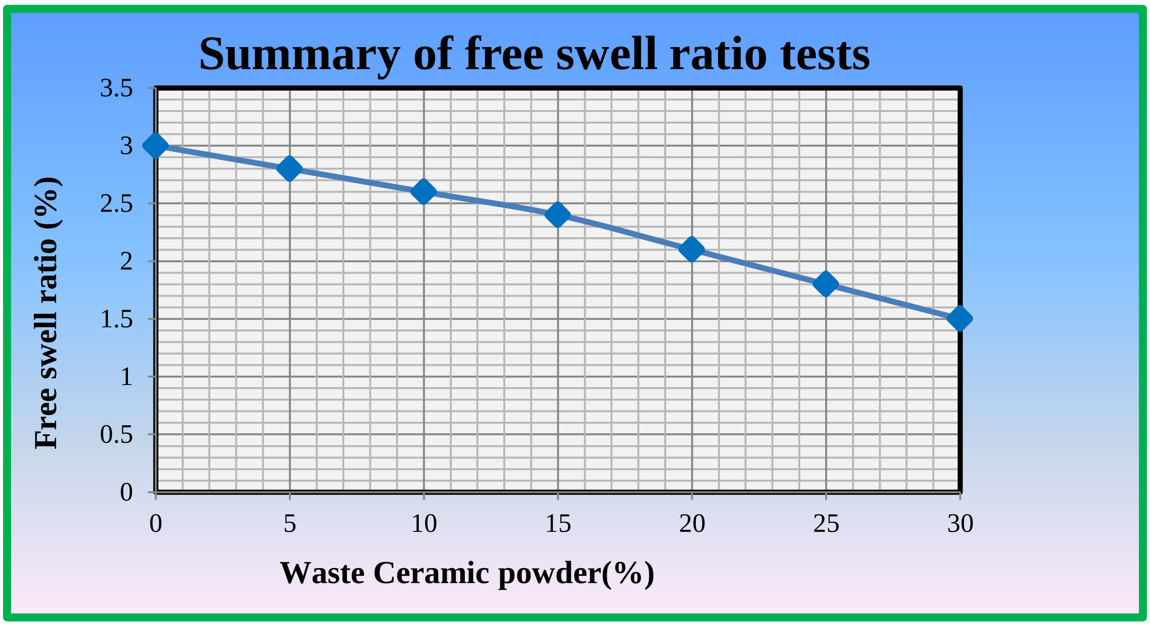


Figure 5.9: Effect of addition of waste ceramic powder on free swell ratio of expansive soil

From the above swelling characteristics (free swell ratio) the decrease in swelling are mainly due to;

- Waste ceramic powder decreases swell potential of expansive soils by replacing some of the volume that is previously held by expansive clay minerals and by cementing the soil particles together.

- The reason may be due to cation exchange in the waste ceramic powder-soil mix.

5.6 Effect of waste ceramic powder on Specific Gravity

The effect of waste ceramic powder on the specific gravity of the expansive soil is shown in Figure 5.10 Specific gravity increase from 2.70 to 2.801 with increased waste ceramic powder content from 0% to 30%. As it is shown in the Figure 5.10, the increase in specific gravity is directly proportional to the quantity of waste ceramic powder this is due to the heaveness of the waste ceramic powder which has specific gravity of 3.06.

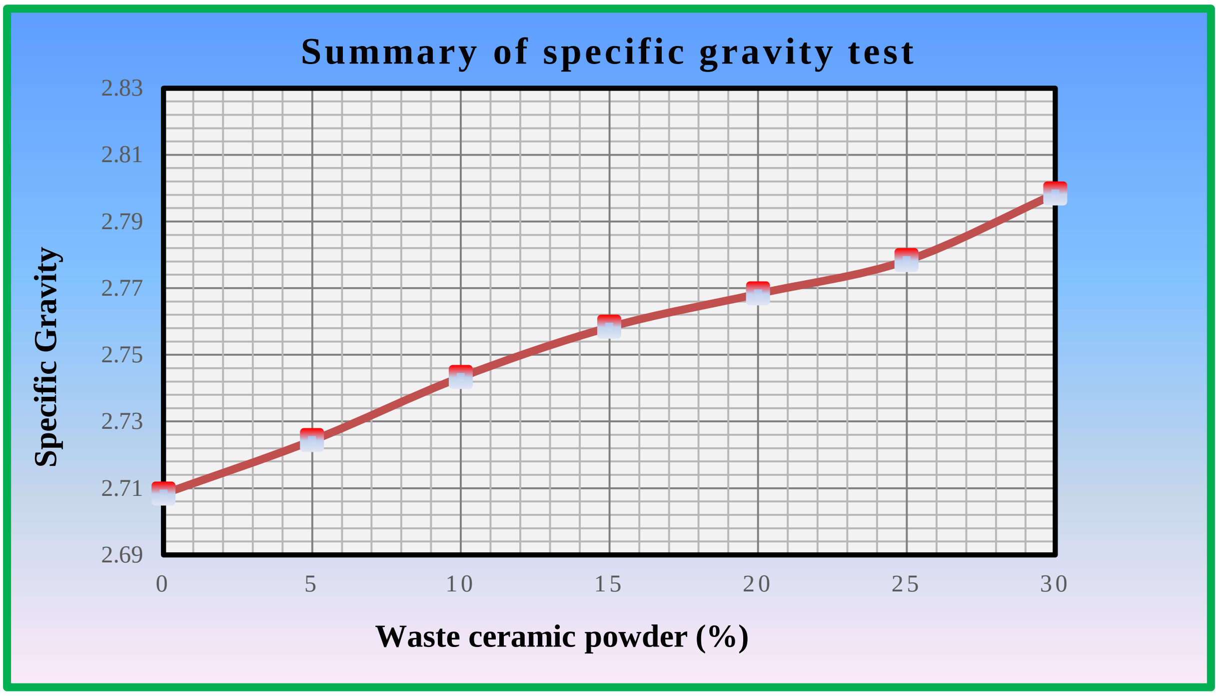


Figure 5.10: Variation of specific gravity of soil with waste ceramic powder contents.

This increase in specific gravity of the soil waste ceramic powder mix is due to the higher value of specific gravity of waste ceramic powder (3.06).

5.7 Effect of waste ceramic powder on Compaction Characteristics

5.7.1 Maximum Dry density

The effect of waste ceramic powder on the maximum dry density of the expansive soil is shown in Figure 5.11 for uncured and 7 days cured soil sample. As shown in the Figure 5.11, maximum dry density increase from 1.29g/cm^3 to 1.42 g/m^3 for uncured and to 1.44 g/cm^3 for 7 days cured soil samples with increased waste ceramic powder content from 0% to 30%.

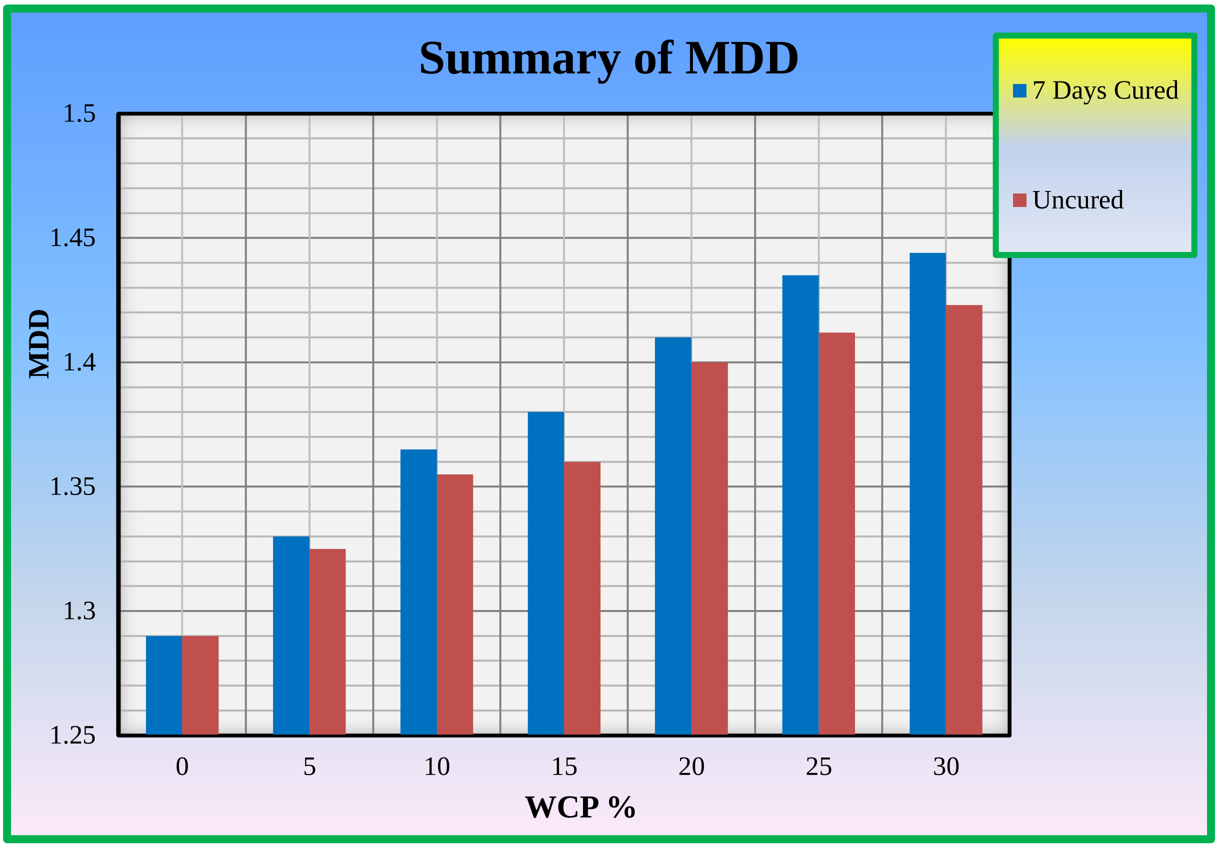


Figure 5.11: Variation of MDD with application of different waste ceramic powder contents

The increase in the maximum dry density is mainly due to;

- the partial replacement of comparatively heavey waste ceramic powder with the light weight soil ;
- comparatively low specific gravity value (2.7) of soil than that of replaced waste ceramic powder (3.06);
- it may also be attributed to coating of the soil by the waste ceramic powder which result to large particles with small voids and hence high density;

5.7.2 Optimum Moisture Content

The effect of waste ceramic powder on the optimum moisture content for the soil waste ceramic powder mixtures are shown in Figure 5.12. The optimum moisture content decrease from 32.5% to 26.3% for uncured and 32.5% to 25.6% for 7 days cured soil samples with increased waste ceramic powder content from 0% to 30%.

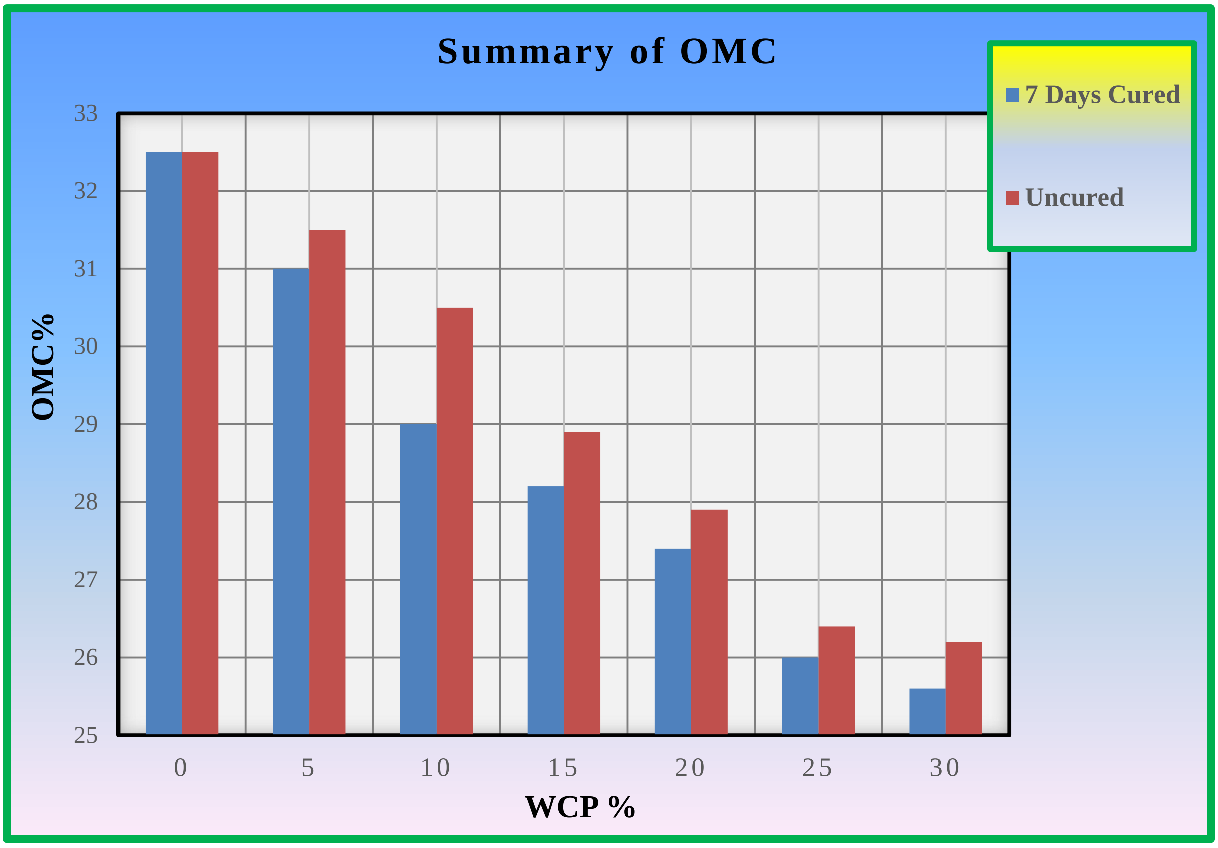


Figure 5.12: Variation of OMC with application of different waste ceramic powder contents

The decrease in the optimum moisture content was mainly due to:

- The optimum moisture content of soil decrease with an increase waste ceramic powder, because waste ceramic powder is denser than the soil. The more denser the more preventing permeability of water.
- The decrease of water content may also be attributed by the pozzolanic reaction of waste ceramic powder with the soil.
- The decrease in OMC due to addition of waste ceramic powder caused by high absorption of water by waste ceramic powder. This implies that less water is needed in order to compact the soil with waste ceramic powder mixture. So waste ceramic powder effectively dries wet soils and provides an initial rapid strength gain, which is useful during construction in wet, unstable ground conditions. In general it can be utilized in improving the workability of wet soils.

The summaries of compaction curves show in the Figure 5.13 and Figure 5.14 for uncured and 7 days cured samples respectively; treating soil with waste ceramic powder give the bell shaped curves for both cured and uncured samples respectively. These curves shifted to the left with respect to untreated soil sample, which also means addition of waste ceramic powder increase the maximum dry density and decrease the optimum moisture content. The increase in maximum dry density and decrease corresponding optimum moisture content of all treated soils is related to the additive quantities. Details of the maximum dry density and optimum moisture content results are shown in the Appendix 4.

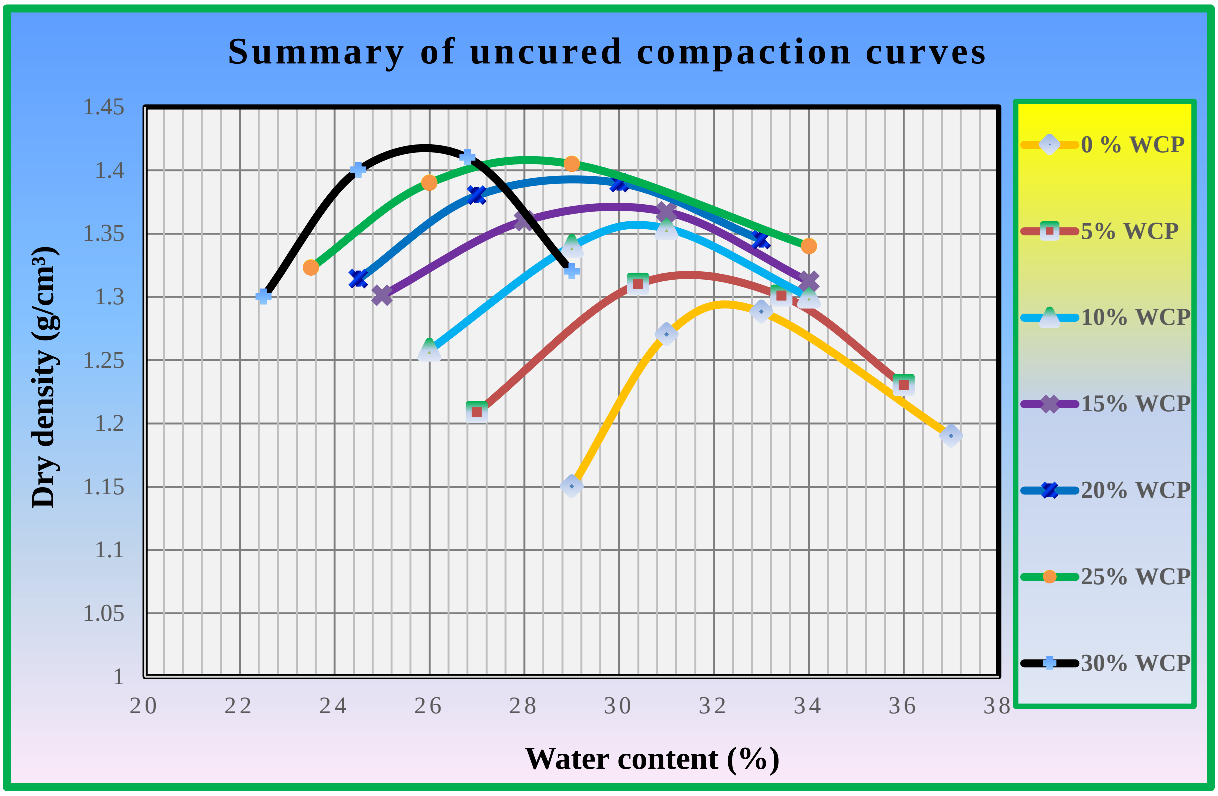


Figure 5.13: Summary of compaction curves with application of different waste ceramic powder contents for uncured samples.

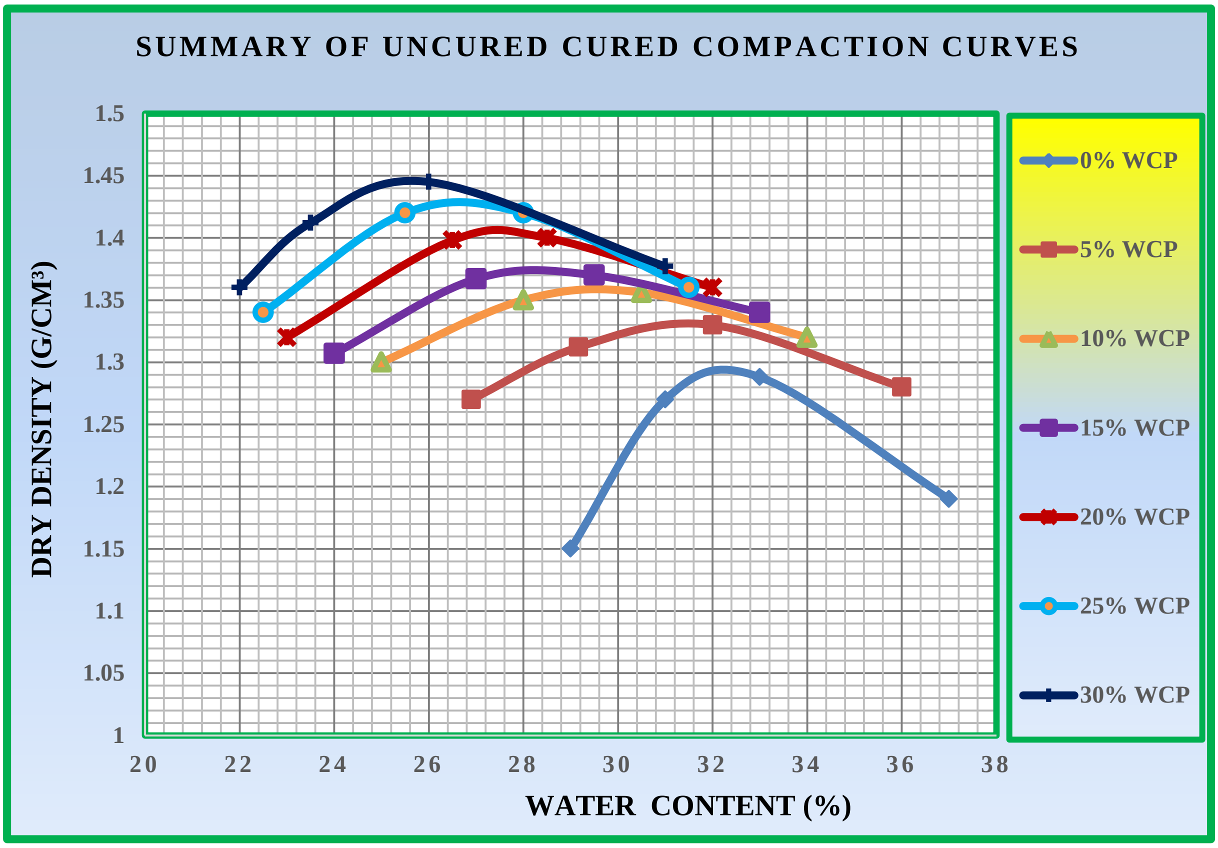


Figure 5.14: Summary of compaction curves with application of different waste ceramic powder contents for 7 days cured samples.

5.8 Effect of waste ceramic powder on CBR and CBR-Swell

5.8.1 CBR Values

The effect of waste ceramic powder on the CBR values of the soil waste ceramic powder mixtures are shown in Figure 5.15. When treating the expansive soil at 30% waste ceramic powder content the uncured soil waste ceramic powder sample gives a peak CBR value of 3.2% while that of 7 days cured CBR value is 4.44% from a value of 1.02% for the natural soil. The CBR of the expansive soil increased with all higher waste ceramic powder contents. This shows that the load bearing capacity of the sample increased significantly with waste ceramic powder treatment and also curing has little effect on the CBR values. Hence, the waste ceramic powder treated black cotton soil meet the minimum criteria as specified by Ethiopian Roads Authority pavement design manual (2002) specification for materials suitable for use as sub grade material of not less than 3% CBR determined at MDD and OMC, the waste ceramic powder could be used in admixture stabilization with a more potential stabilizers, reduce the cost of stabilization.

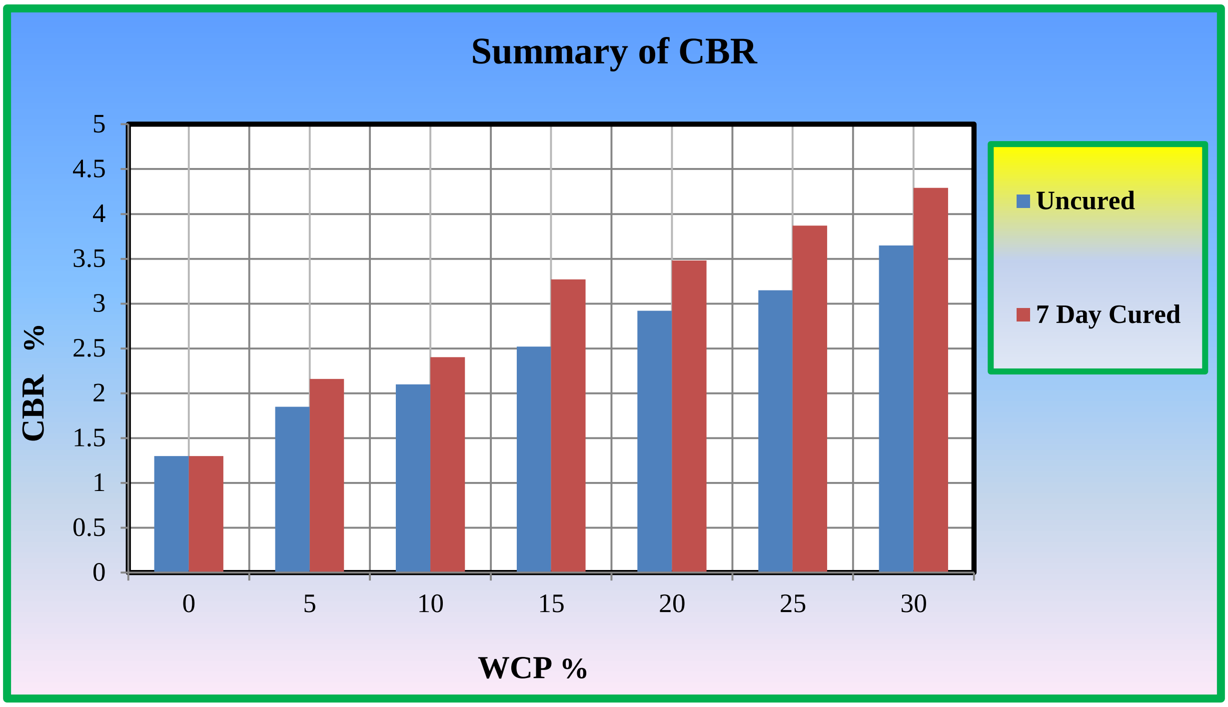


Figure 5.15: Variation of soaked CBR with application of different waste ceramic powder contents.

The reason for this improvement is due to the pozzolanic reactions of waste ceramic powder with soil. This results in agglomeration of clay particles and causes the increase in strength gain. The reason for the slight increases in strengths could be due to inadequate amounts of calcium required for the formation of CSH which is the major element for strength gain. Detailed test results are given in Appendix 5.

5.8.2 CBR-Swell

The effect of waste ceramic powder on the CBR-swell for the soil waste ceramic powder mixtures are shown in Figure 5.16 for both uncured and 7 days cured samples. When treating the expansive soil at 30% waste ceramic powder content the uncured soil- waste ceramic powder sample gives CBR-swell of 3.2% while that of 7 days cured CBR-swell is 2 % from a value of 9.3% for the natural soil. The CBR-swell of the expansive soil decreased with all higher waste ceramic powder contents. This shows that the swelling potential of the sample decrease with waste ceramic powder stabilization.

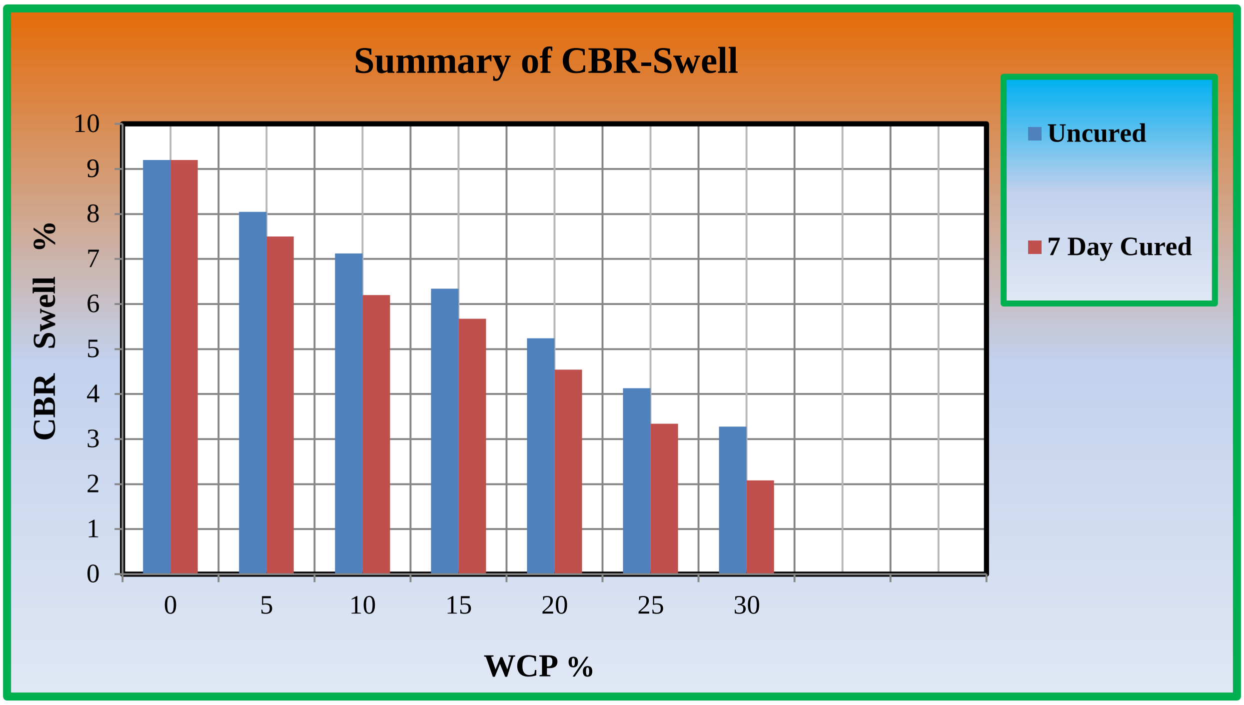


Figure 5.16: Variation of soaked CBR-swell with application of different waste ceramic powder contents.

Waste ceramic powder decreases CBR-Swell of expansive soil. This is due to cation exchange and flocculation and agglomeration of the soil particles. This is also due to replacement of some of the volume that is previously occupied by expansive clay minerals by waste ceramic powder. It decreases CBR-swell of expansive soil. This is also due to cation exchange and flocculation and agglomeration of the soil particles and the replacement of some of the volume that is previously occupied by expansive clay minerals by waste ceramic powder. Therefore, waste ceramic powder treated expansive soil satisfy the minimum criteria as specified by Ethiopian Road Authority pavement design manual (2002) specification for materials suitable for use as sub grade material of not less than 3% CBR determined at MDD and OMC. And From the literature review section 2.10, according to [16] the quality of CBR subgrade increased from very poor subgrade to poor subgrade.

CHAPTER SIX

EFFECT OF LIME ON WASTE CERAMIC POWDER TREATED SOIL, ANOTHER PERSPECTIVE

6.1 Introduction

The reason why the soil-waste ceramic powder mixture is not more successful soil stabilizer could be explained by chemistry. The pozzolanic reactions are dependent on both access to water, pozzolanic material and the content of calcium ions, and this is simply there is no enough calcium in waste ceramic powder. This shows that a certain amount of calcium from cement or lime is necessary. The amount of waste ceramic powder that is going to be mixed with lime was determined according to Texas Department of Transportation manual. The ratio of lime to fly ash that will yield the best engineering property is determined by using lime to fly ash ratios of 1:3, 1:4 and 1:5. According to Texas Department of Transportation manual typical proportions for the fly ash lime blends are 2% to 8% lime blended with 10% to 15% fly ash [55].

Researches done around Bole area indicate that the optimum lime content required to stabilize expansive soil is mostly around 6-8% [43] or from Chapter three section 3.4.2.1.2 in Figure 3.2 the optimum lime content required to stabilize soils having a PI of 64.5% and 99.4% of soil passing No. 40 sieve is almost above 8%. So half of the optimum lime content recommended to stabilize soil around Bole area is used to activate the pozzolanic soil- waste ceramic powder mix to form more cementitious compounds [40].

6.2 Test Results and Discussion

6.2.1 Atterberg Limits

Variations of plasticity index with the addition of 3% lime, 15% waste ceramic powder and 3% lime in combination with/plus 15% waste ceramic powder are presented in Figure 6.1 for both uncured and 7 days cured samples. As shown in the figure plasticity index generally decreased with the addition of additives. The decrease in plasticity index indicates an improvement in the workability of the soil. Atterberg limit graphs are shown in Appendix 3.

When black cotton soil treated with 3% lime the plasticity index decreased from a natural soil value of 64.5% to 35.4% for uncured sample and to 34.5% for 7 days cured sample. While with the addition of 15% waste ceramic powder the plasticity value of 64.5% to 49.4% for uncured samples and to 47.8%. For a similar class soil, the plasticity index 32% without curing and to 29.7% combination with/plus 15% waste ceramic powder.

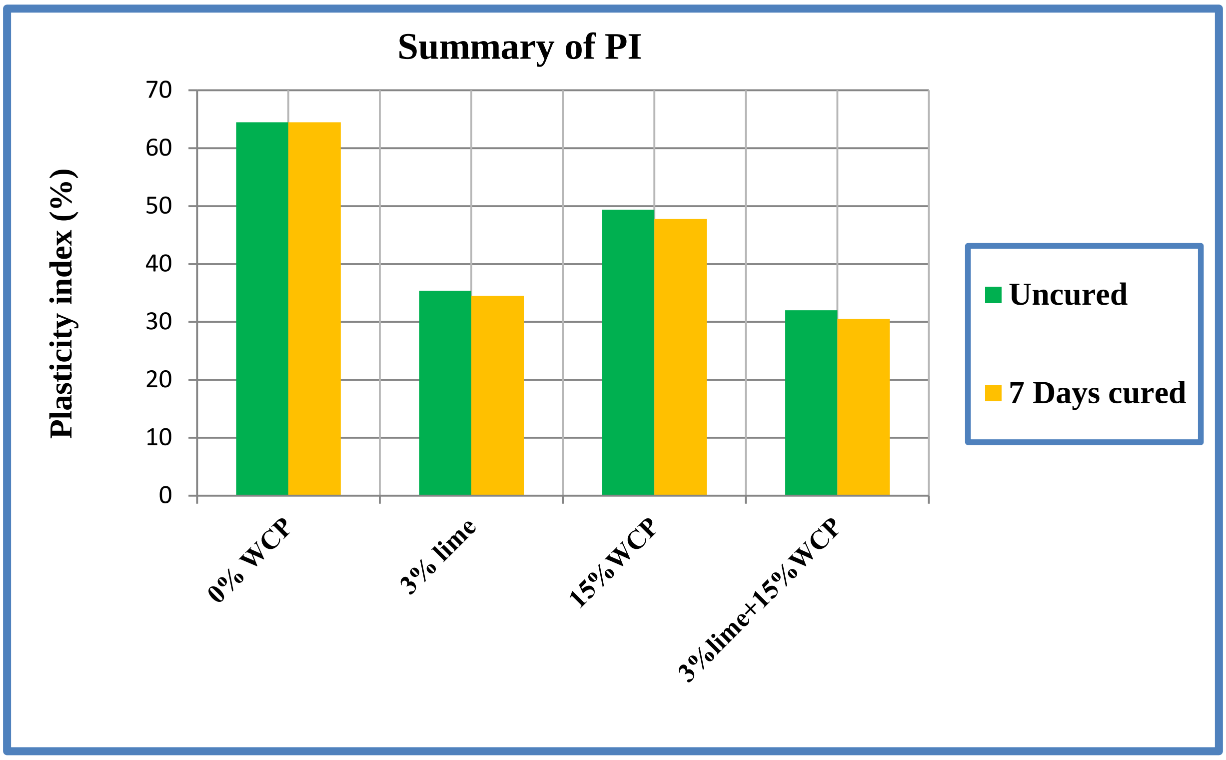


Figure 6.1:Effect of addition of waste ceramic powder and lime on plasticity index

Significant reduction in plasticity associated with a with lime is expected as more calcium is available for cation exchange to take place and also this effect could be attributed to the combined action of partial replacement of plastic soil particles with non-plastic particles of minerals of the soil. These led to flocculation and agglomeration of the clay particles which in turn reduces the plasticity of the treated soil.

6.2.2 Compaction Characteristics

6.2.2.1 Maximum Dry Density

The effects of waste ceramic powder on the maximum dry density of the soil-lime, soil-waste ceramic powder and soil-lime-waste ceramic powder mixtures are shown in Figure 6.2 for both 7 days cured and uncured soil samples. As shown in the figure, for uncured soil sample maximum dry density decreases from a natural soil value of 1.29g/cm^3 to 1.24 g/cm^3, increase to 1.37 g/cm^3 and increase to 1.325 g/cm^3 with the addition of 3% lime, 15% waste ceramic powder and 3% lime plus 15% waste ceramic powder respectively. For 7 days cured soil samples maximum dry density decreases from a natural soil value of 1.29 g/cm^3 to 1.19 g/cm^3, increase to 1.38 g/cm^3 and increase to 1.36 g/cm^3 with the addition of 3% lime, 15% waste ceramic and 3% lime plus 15% waste ceramic powder respectively. The addition of lime and waste ceramic powder together led to increase of the maximum dry density compared to the addition of lime.

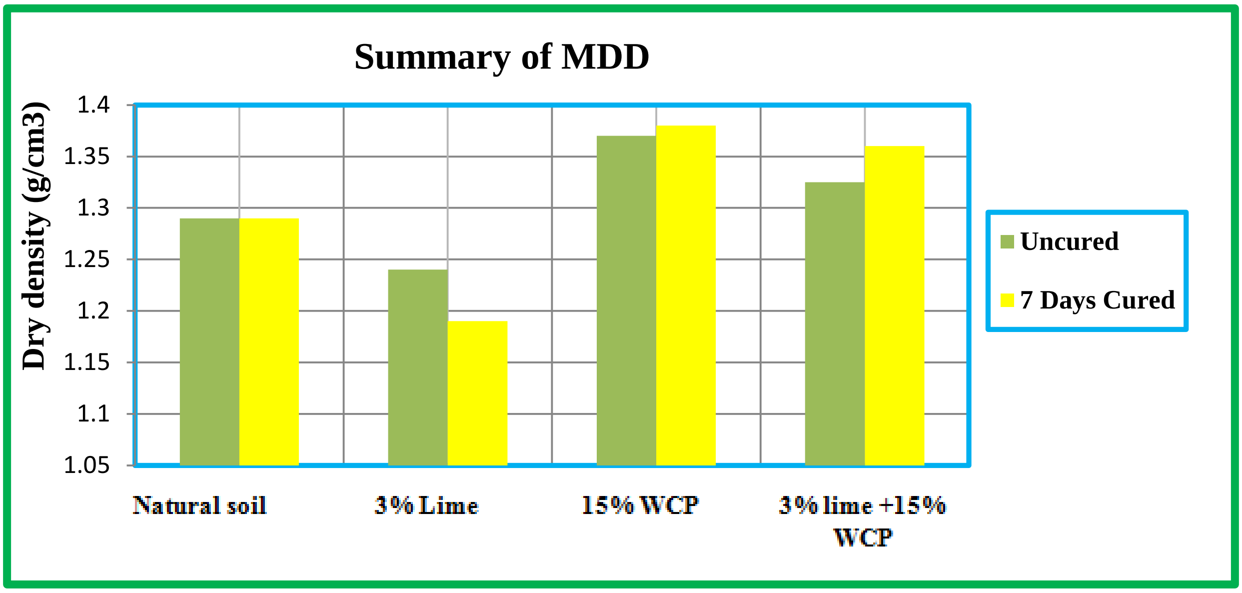


Figure 6.2: Effect of addition of waste ceramic powder and lime on MDD

6.2.2.2 Optimum Moisture Content (OMC)

The effects of waste ceramic powder on the optimum moisture content for the soil-lime, soil-waste ceramic powder and soil-lime-waste ceramic powder mixtures are shown in Figure 6.3 for 7 days cured and uncured soil samples. As shown in the figure, for uncured soil sample optimum moisture content increases from a natural soil value of 32.5 % to 33.5 %, decrease to 30% and decrease to 32.4% with the addition of 3% lime, 15% waste ceramic powder and 3% lime plus 15% waste ceramic powder respectively. For 7 days cured soil sample optimum moisture content increases from a natural soil value of 32.2% to 37.5%, decrease to 28% and decrease to 30.7% with the addition of 3% lime and 15% waste ceramic powder and 3% lime plus 15% waste ceramic powder respectively. The addition of lime and waste ceramic powder together led to a slight decrease of the optimum moisture content compared to the addition of lime.

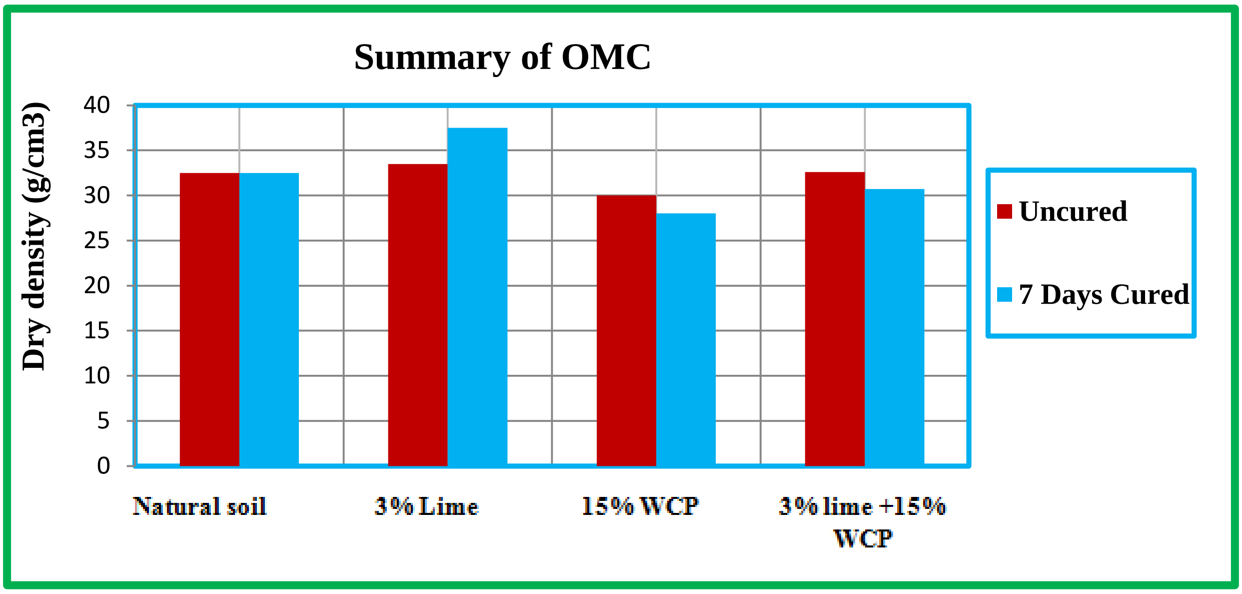


Figure 6.3: Effect of addition of waste ceramic powder and lime on OMC

The summaries of compaction curves shown in Figures 6.4 and 6.5 for uncured and 7 days cured samples respectively. Details of the maximum dry density and optimum moisture content results are shown in the Appendix 4.

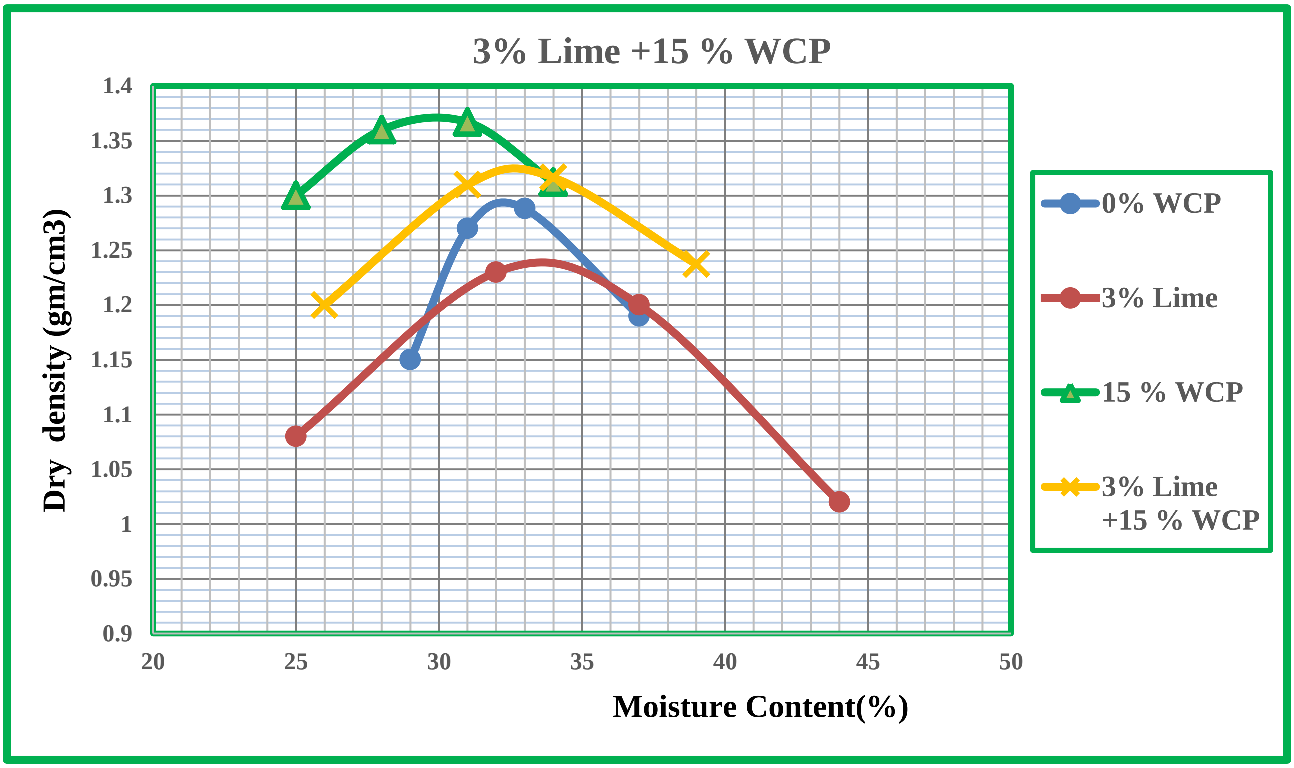


Figure 6.4: Summary of compaction curves with application of different waste ceramic powder and lime content for uncured samples.

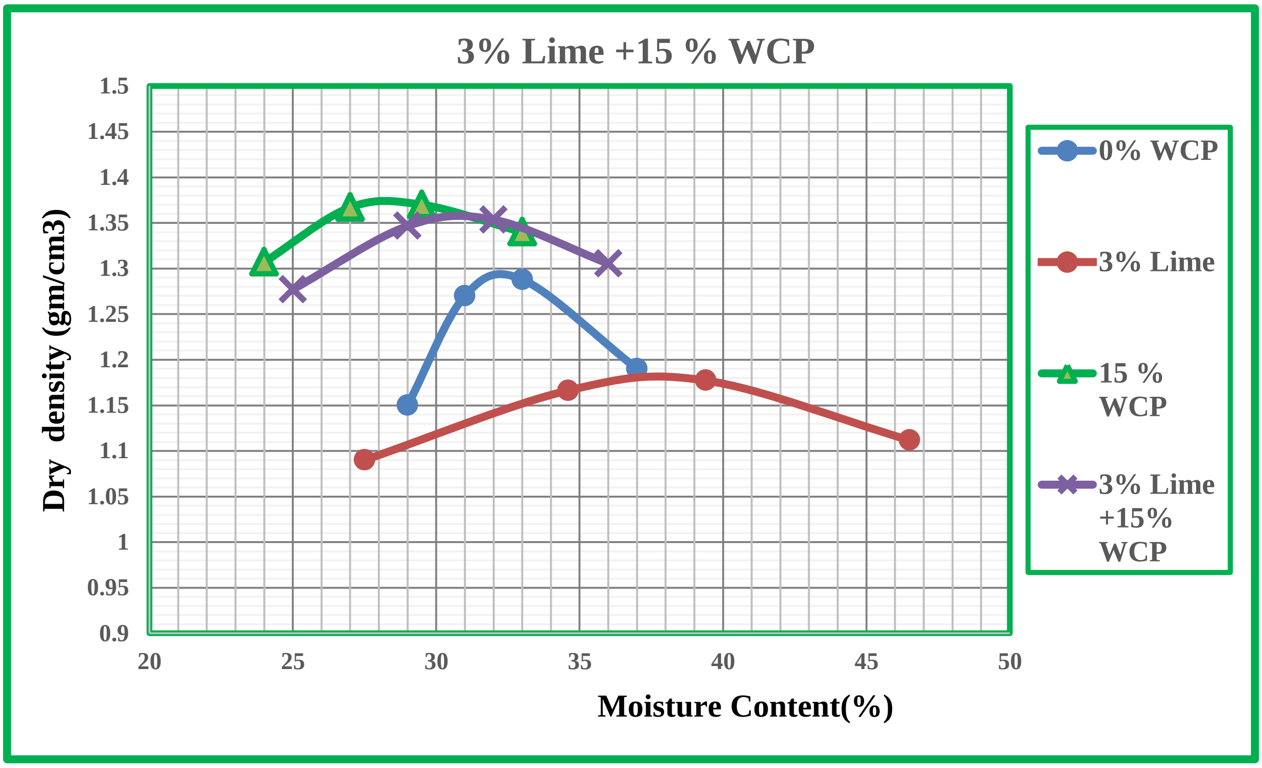


Figure 6.5: Summary of compaction curves with application of different waste ceramic powder and lime contents for 7 days cured samples.

The decrease in OMC is due to pozzolanic reaction of silica and alumina in waste ceramic powder and soil with calcium of the lime to form calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) which are the cementing agents. The increased in density of all treated soils is mainly due to the partial replacement of comparatively heavy waste ceramic powder with the light weight soil particles.

6.2.3 CBR and CBR-swell

6.2.3.1 CBR values

Results presented in Figure 6.6 show that generally the CBR of all treated soil increases with addition of lime, lime plus waste ceramic powder. Results also show that curing enhances the strength development of expansive soil treated with lime and lime plus waste ceramic powder but curing has an insignificant change when expansive soil is treated with waste ceramic powder only. As shown in the figure, the uncured CBR value increases for all mixes from a natural soil value of 1.02% to 14.73%, to 2.36% and to 20.4% with the addition of 3% lime, 15% waste ceramic powder and 3% lime plus 15% waste ceramic powder respectively. For 7 days curing period the CBR values increased from the natural soil value of 1.02% to 22.72%, to 3.27% and to 28.7% with the addition of 3% lime, 15% waste ceramic powder and 3% lime plus 15% waste ceramic powder respectively. The addition of lime and waste ceramic powder together led to a more increase of the CBR value compared to the addition of lime and waste ceramic powder separately. This shows that the load bearing capacity of the sample increased considerably with waste ceramic powder and lime treatment and curing has a significant effect on the CBR values. The combination of waste ceramic powder and lime can strongly improve the strength of expansive soils. Detailed test results are given in Appendix 5.

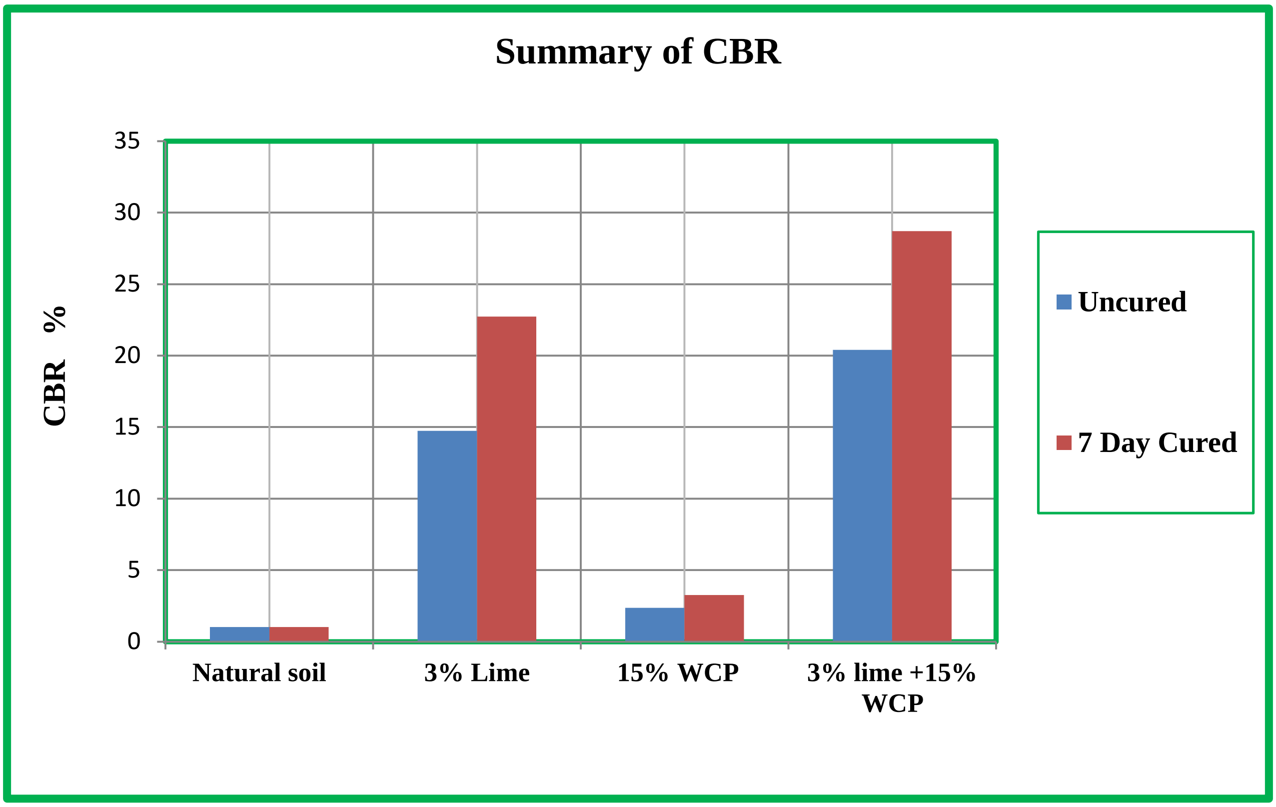


Figure 6.6: Effect of addition of waste ceramic powder and lime on CBR values

The improvements in the CBR values of samples satisfy the minimum requirements that qualify them as road construction materials and showed that the soil sample was effectively stabilized by lime in combination with waste ceramic powder. This increase was as a result of the formation of cementitious compounds, calcium-silicate-hydrates (CSH) and calcium-aluminate-hydrate (CAH), by calcium from lime and the readily available silica and/or alumina from both the soil and waste ceramic powder. CSH and CAH are cementitious products similar to those formed in portland cement. They form the matrix that contributes to the strength of stabilized soil layers. It was also observed that the CBR value increased with curing age for all mixes. This is attributed to the pozzolanic reaction between the lime, soil and waste ceramic powder resulting in the formation of more cementitious compounds. Big differences in CBR values between cured and uncured sample for lime and lime plus waste ceramic powder magnifies the importance of curing.

6.2.3.2 CBR- Swell

Results presented in Figure 6.7 show that generally the CBR-swell of all treated soil decreases with addition of lime, lime plus waste ceramic. Results also show that curing further reduces the CBR-swell of expansive soil treated with lime and lime plus waste ceramic powder but curing has an insignificant change when expansive soil is treated with waste ceramic powder alone. As shown in the figure, the uncured CBR-swell decrease for all mixes from a natural soil value of 9.2% to 5.85%, to 6.35% and to 2.09% with the addition of 3% lime, 15% waste ceramic powder and 3% lime plus 15% waste ceramic powder respectively. For 7 days curing period the CBR-swell decreased from the natural soil value of 9.2% to 2.3%, to 5.6% and to 0.81% with the addition of 3% lime, 15% waste ceramic powder respectively. This shows that the swelling capacity of the expansive soil decreased considerably with waste ceramic powder and lime treatment and curing has also a significant effect on the CBR-swell values.

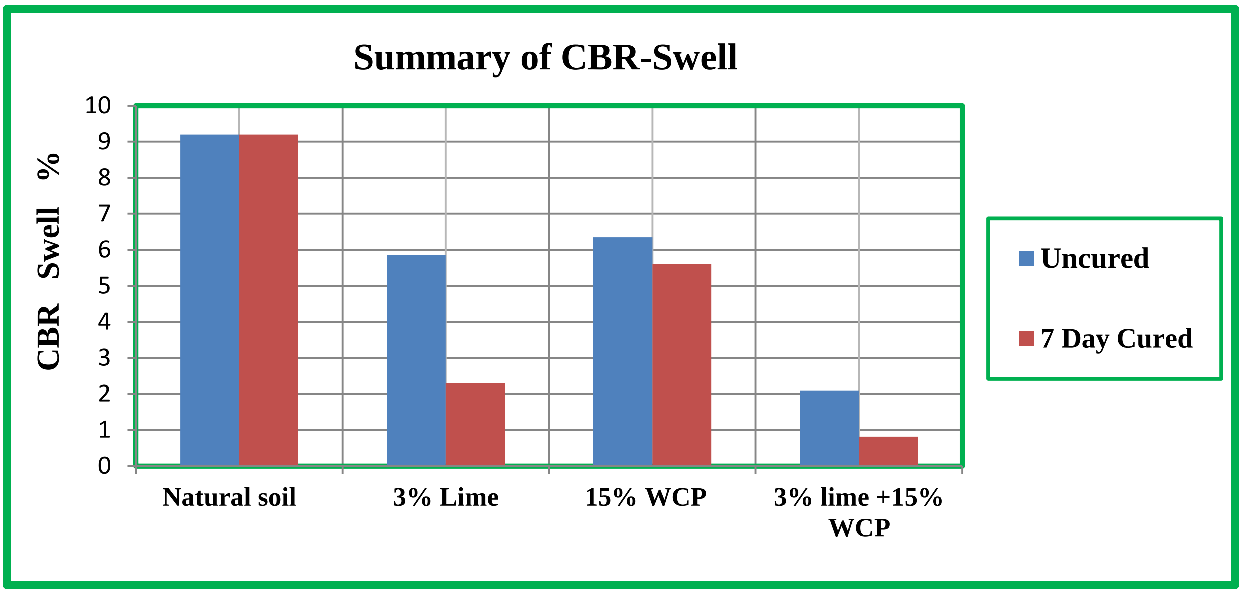


Figure 6.7: Effect of addition of waste ceramic powder and lime on CBR-swell

The decrease in CBR-swell of expansive soil is due cation exchange and flocculation and agglomeration of the soil particles. This is also due to replacement of some of the volume that is previously occupied by expansive clay minerals by waste ceramic powder

CHAPTER SEVEN

CONCLUSION AND RECOMMENDATION

7.1 Conclusion

The following conclusions can be drawn from the results of the study/investigation carried out within the scope of the study.

1. The plasticity index slightly reduced with increased in waste ceramic powder content and curing has also an insignificant effect on the plasticity of the expansive soil.
2. The optimum moisture content decreased while the maximum dry density values increased with increment of waste ceramic powder content.
3. Free swell, free swell index and free swell ratio of the stabilized samples decreased with increasing waste ceramic powder content.
4. CBR values slightly increased with the addition of waste ceramic powder. The change in CBR value is not significant for both cured and uncured samples. Addition of waste ceramic powder alone does not improve the strength of soils due to presence of only reactive silica with low amount of calcium content in waste ceramic powder.
5. The plasticity index significantly decreased with addition of waste ceramic powder combined with lime and increased curing period. However, the addition of waste ceramic powder alone has a minor effect on the plasticity index of expansive soil.
6. The addition of lime and waste ceramic powder together led to a more decrease of the maximum dry density and increase in optimum moisture content compared to the addition of lime and waste ceramic powder separately.
7. The addition of waste ceramic powder in combination with lime improved the CBR value. The improvement is more significant when the sample is cured. Hence, combination of waste ceramic powder and lime can strongly improve the strength of the expansive soil.
8. Unlike lime in combination with waste ceramic powder the improvement achieved by waste ceramic powder alone on the poor geotechnical properties of expansive soil was limited because lower amount of calcium in the waste ceramic powder. Hence, improvements achieved with up to 30% waste ceramic powder content were not satisfactory. However, the rate of swelling and heave decreased with increasing waste ceramic powder content of stabilized expansive soil.

9. In this investigation waste ceramic powder stabilized expansive soil does not bring significant change for use it as a sub-grade material. Therefore, waste ceramic powder is not an effective standalone stabilizer for highly plastic expansive soils. However, waste ceramic powder plus/in combination with lime can effectively stabilize this soils. The expansive soil stabilized with waste ceramic powder plus/in combination with lime can be used as a good subgrade material. So, combining two local materials (waste ceramic powder and lime) can effectively improve the poor geotechnical properties of this soils and help in increasing land resources availability for construction projects and reduce the amount of lime needed for the stabilization purpose.

9.2 Recommendations

Based on the findings of this research, the following recommendations are forwarded:The results obtained during this investigation as discused in the previous sections showed.

1. Waste ceramic powder as investigated in this research work can be used as a soil stabilizing material in combination with lime with economical and environmental benefits. Therefore concerned bodies different ceramic industries and government entities should be made aware about this potential soil stabilizing material and promote its standardized production and usage.

2. The Ceramic industry in collaboration with higher education organizations in the country should work together and establish a research team to further study the use of waste ceramic powder as a soil stabilizing material on different types of soils.

3. When pavement layers are treated with cementitious stabilizers in Ethiopia, good quality materials should be used and attention needs to be given to construction procedures such as compacting pavement layers and curing properly.

The following topics are recommended for future studies

- The stabilizing potential of waste ceramic powder from different sources like Tabor ceramic, Ethio-ceramic, Arerty industrial park and the coming new industries should be studied.

- Stabilizing soils with cement and waste ceramic powder on different types of soils.
- Stabilizing soils with cement kiln dust and waste ceramic powder on different types of soils.
- Stabilizing soils with waste ceramic powder and marble waste dust on different types of soils.
- Stabilizing soils with cement, lime and waste ceramic powder on different types of soils.

REFERENCES

[1] AASHTO, “Standard specification for transportation materials and methods of sampling and testing,” America, 1995.

[2] Abd EI-Aziz, M.A. and Abo-Hashema, M.A. (2013) “Measured Effects on Engineering Properties of Clayey Subgrade using Lime-Homra Stabiliser,” International Journal of Pavement Engineering, 14(4), 321-332.

[3] Akshaya Kumar Sabat “Stabilization of expansive soil using waste ceramic dust”. Electronic journal of Geotechnical Engineering (EJGE), vol.17, pp 3915-3925,

[4] Al-Rawas, A.A., and Mattheus, F.A., “Expansive Soils Recent Advanced in Characterization and treatment,” Proceedings and Monographs in Engineering, Water and Earth Sciences, Netherlands, 2006.

[5] Amadi, A.A., Lubem, S. (2014) “Assessing Stabilization Effectiveness of Combined Cement Kiln Dust and Quarry Fines on Pavement Subgrades Dominated by Black Cotton soil,” Geotechnical and Geological Engineering, DOI 10.1007/s10706-014-9793-0.

[6] American Society for Testing and materials (ASTM, 1992). Annual Book of Standards Vol.04,08. Philadelpha.

[7] Amrendra Kumar, Dr. Ravi Kumar Sharma and Babitasingh (2014), “Compaction and sub-grade characteristics Of Clayey Soil Mixed with Foundry Sand and Fly Ash And Tile waste”, International Conference on Advances in Engineering & Technology - 2014 (ICAET-2014),Page PP 01-05 eISSN:2278-1684.

[8] Amu, O.O., et al., “Geotechnical properties of lateritic soil stabilized with sugarcane straw ash,”American Journal of Scientific and Industrial Research, 2011.

[9] Ashango, A.A. and Patra, N.R. (2014) “Static and Cyclic Properties of Clay Subgrade Stabilised with Rice Husk Ash and Portland Slag Cement,” International Journal of Pavement Engineering, DOI: 10.1080/10298436.2014.893323.

[10] ASTM C618 – 03, “Specification for Fly Ash and Raw or Calcined Natural Pozzolanas for Use as a Mineral Admixture in Portland Cement Concrete,” American Society for Testing and Materials, Vol.04.02, Philadelphia, 2003.

[11] Babitasingh, Ravi kumar (2014), “Evaluation of Geotechnical properties of clayey soil blended with waste materials”, Jordan Journal of civil engineering, volume-8 Nov-2 2014.

[12] Balasingam,M.,and Farid, S.,2008;Teferra,A..,and Leikun,M.,1999.

[13] Baser, O.(2009) Stabilization of expansive soils using waste marble dust, Master of Science thesis submitted to Civil Engineering Department, Middle East, Technical University.

[14] Bell, F.G, “Lime Stabilization of Clay Minerals and Soils,” Journal of Engineering Geology, Vol. 42, pp. 223-227 1996. As cited by Thian, S.Y, and Lee C.Y, “Shear Strength Behaviour of Mining Sand Treated with Clay and Hydrated Lime,” International Journal of Civil and Structural Engineering Volume 3, No 1, 2012.

[15] Binici, H. (2007) “Effect of crushed ceramic and basaltic pumice as fine aggregates on concrete mortars Properties,” Construction and Building Materials, Vol. 21, pp 1191-1197.

[16] Bowles, J., “Engineering properties of soil and their measurements,” McGraw-Hill Boston, 4th edition, 1992.

[17] Brooks, R.M., “Soil Stabilization with Fly ash and Rice Husk Ash, “International Journal of Research and Reviews in Applied Sciences, Volume 1, Issue 3, 2009.

[18] Chen, F.H., “Foundation on expansive soils,” Elsevier, Amsterdam, 1988 & Chen, James A. & Idusuyi, Felix O.(2015),”Effect of waste ceramic dust on index properties of shrink swell soils.”ISSN 2504-8848 Vol. 1 No.8 2015.

[19] Chittaranjan, M., et al., “Agricultural wastes as soil stabilizers, ”International Journal of Earth Sciences and Engineering, Volume 04, No 06 SPL, pp 50-51, October 2011.

[20] Christopher, M.G., (2005), “Stabilization of Soft Clay Subgrades in Virginia Phase I Laboratory Study,” MSc. Thesis, Virginia Polytechnic Institute and State University, Blacksburg, Virginia, 2005.

[21] Chou, L., “Lime Stabilization: Reactions, Properties, Design, and Construction,” State of the Art Report 5, Transportation Research Board, Washington, DC, 1987.

[22] Chusilp, N., et.al, “Development of waste ceramic powder as a pozzolanic material in concrete,” Asian Journal on Energy and Environment, p. 149-159, 2009.

[23] Craig, R.F, soil mechanics 6th edition spon press London 1997.

[24] Dallas, N. Little, & Syam, N., “Recommended Practice for Stabilization of Subgrade Soils and Base Materials, “Texas Transportation Institute Texas A&M University College Station, Texas, 2009.

[25] Das.B.M. (2008). Advanced Soil Mechanics,3rd Edition. New York: CRC Press.

[26] Department of the Army, the Navy, and the Air Force, “Technical Manual Soil Stabilization for Pavements,” October 1994.

[27] Eades and Grim,1960 cited in Harty , 1970.

[28] Ethiopian Roads Authority, "Pavement Design Manual," Addis Ababa, 2002.

[29] Fransiscus S. Hardianto and Wayne A. Ericson (1994). Stabilization of phosphatic clay using Lime Column. Bromwell & Carrier, Inc.#02-088-102.

[30] Gourley, C., et.al, 1993 "Expansive Soils," first international symposium on engineering characterization of arid soils, transport research, UK.

[31] Grim, R.M., "Applied clay mineralogy," McGraw-Hill company, 1962.

[32] Govindarajan., D., and Jayalakshmi, G., "XRD, FTIR and SEM studies on calcined sugarcane bagasse ash blended cement," Scholars Research Library, Department of Physics, Annamalai University, AnnamalaiNagar, Tamilnadu, India, 2011.

[33] Guyer, J. P.,"Introduction to Soil Stabilization in Pavements," New York, 2011.

[34] Hailu, B., "Waste ceramic powder as cement replacing material," MSc. Thesis, Addis Ababa University, Addis Ababa, 2011.

[35] Laxmikant, Y., et al., "Comparison of Fly Ash and Rice Husk Ash Stabilized Black Cotton Soil, "International Journal of Earth Sciences and Engineering, Volume 04, No 06 SPL, pp. 42-45, 2011.

[36] Lees, G., et al., "Sodium Chloride as an Additive in Lime-Soil Stabilization," Highway Engineer, Vol. 29, No. 12, pp. 2-8, 1982.

[37] Little, D.N., "Handbook for Stabilization of Pavement Subgrades and Base Courses with Lime," Kendall/Hunt, Iowa, 1995.

[38] Lyon Assoiciates, Inc, "Laterite and Lateritic soils and other problems soils of Africa," Baltimore, 1971. As cited by Nebro, D.Muazu.T, "Stabilization of Potentially. Expansive Subgrade Soil Using Lime and Con-Aid," MSc. Thesis, Addis Ababa University, Addis Ababa, 2002.

[39] Makusa, G.P., "Soil Stabilization Methods and Materials in Engineering Practice," Department of Civil, Environmental and Natural resources engineering, Luleå University of Technology, Luleå, Sweden, 2012.

[40] Meron, W.,"Bagasse ash as a Sub-grade Soil Stabilizing Material," MSc. Thesis, Addis Ababa University, Addis Ababa, 2016.

[41] Muni Budhu (2007).Soil Mechanics and Foundations. John Wiley &Sons.Inc

[42] Murthy, V.N.S., "principles and practices of soil mechanics and foundation engineering," New York,

[43] Nebro, D., "Stabilization of Potentially Expansive Subgrade Soil Using Lime and Con-Aid," MSc. Thesis, Addis Ababa University, Addis Ababa, 2002.

[44] Nelson, D., and Miller, J., "Expansive Soils: Problems and Practices in Foundation and Pavement Engineering," New York, 1992.

[45] NG. Pulling, (2005). Determination of optimum concentration of lime solution for soil stabilization, Master s Thesis: University Technology: Malaysia.

[46] Onur Baser (2009).Stabilization of expansive soils with waste marble dust. A Thesis submitted to School of Graduate Studies of Middle east technical University In Partial Fulfillment of the Requirements for the degree of Master of science in civil.eng.

[47] Osinubi, K. J., "Lime modification of black cotton soils," Spectrum Journal, Vol. 2, pp. 112 – 122, 1995.

[48] Osinubi, K.J., et.al, "Bagasse Ash Stabilization of Lateritic Soil," Appropriate Technologies for Environmental Protection in the Developing World, p. 281-290, 2009.

[49] Oweis,I.S.,Khera R.P. Geology of waste management,2nd edition, PWS Publishing Company,Boston,1998.

[50] Sabat, A.K. (2012). "Stabilization of expansive soil using waste ceramic dust" Electronic Journal of Geotechnical Engineering, Vol. 17, Bund. Z, 3915-3926.

[51] Teklu b bn , D., "Examining the Swelling Pressure of Addis Ababa Expansive Soil," MSc. Thesis, Addis Ababa University, Addis Ababa, 2003.

[52] Tesfaye, A., "Chemical Treatment of "Black Cotton" Soil to make it Usable as a Foundation Material," MSc. Thesis, Addis Ababa University, Addis Ababa, October 2001.

[53] Texas Department of Transportation, ". f. (2005). ftp://ftp.dot.state.tx.us/pub/txdot-info/cmd/tech/stabilization.pdf. Retrieved Jan 29, 2015.

[54] Tilahun, D., "Influence of Drainage Condition on Shear Strength Parameters of Expansive Soils," MSc. Thesis, Addis Ababa University, Addis Ababa, 2004.

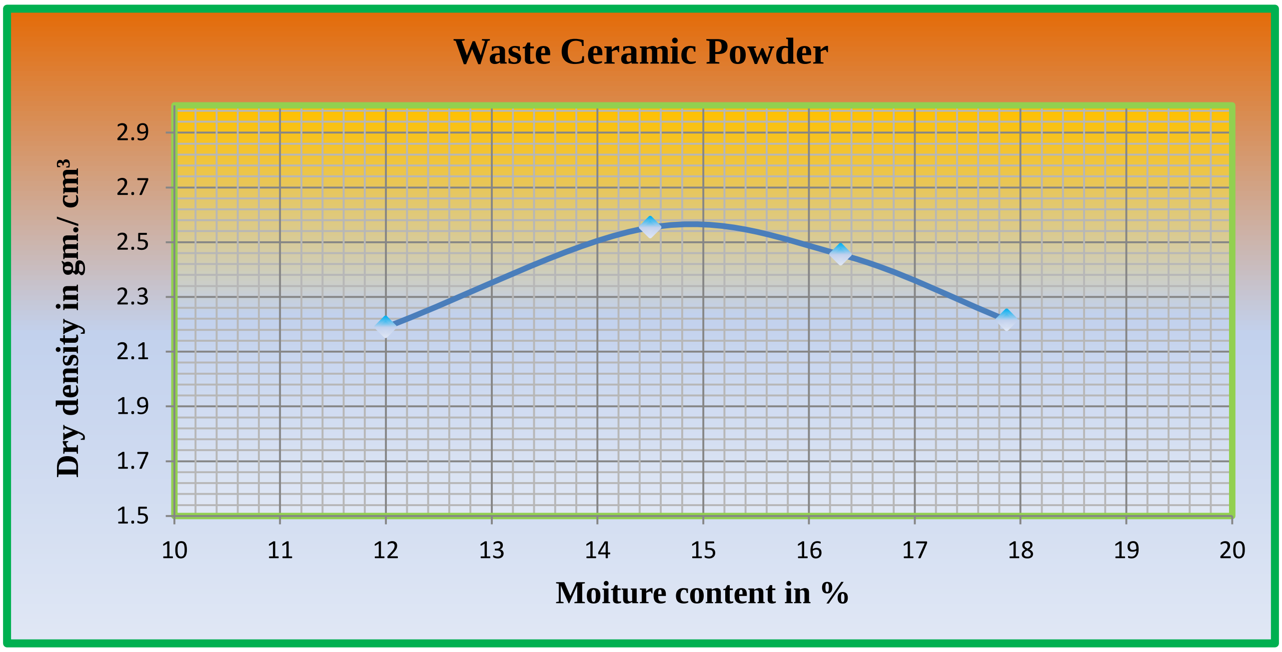
[55] Veisi, M., et al., "Accelerated Stabilization Design of Subgrade Soils," Center for Transportation Infrastructure Systems (CTIS) the University of Texas, Research Report 0-55691, 2010.

[56] Whitlow, R., "Basic Soil Mechanics,"3rd ed. Edinburgh Gate: Addison Wesley Longman Limited, 1995.

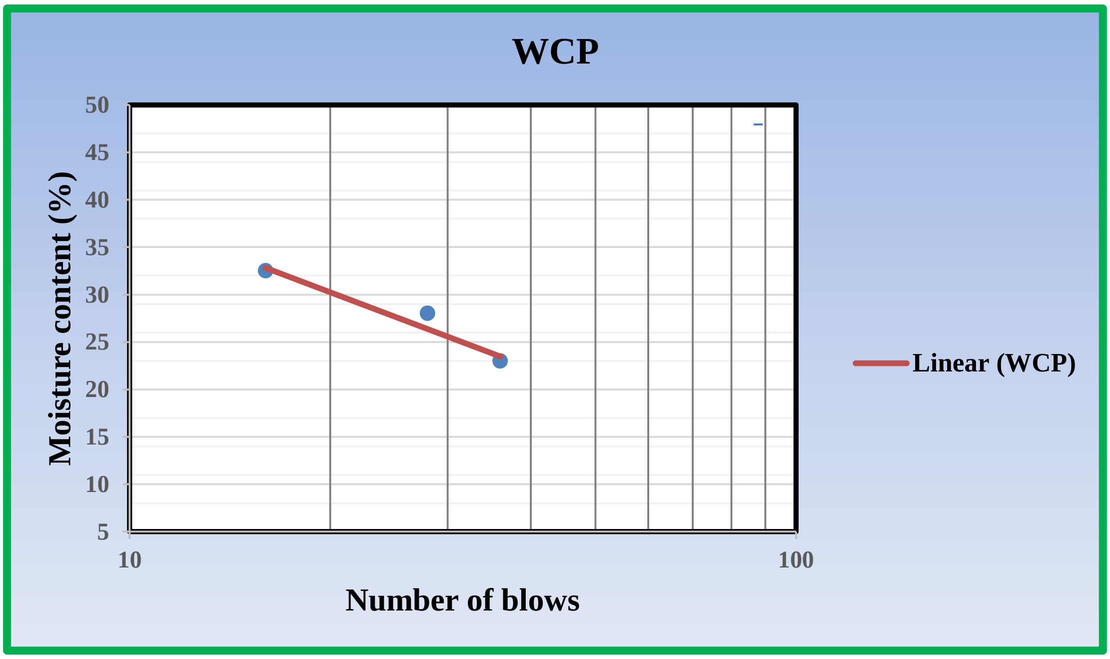
Appendix

Appendix 1: Geotechnical properties of Waste Ceramic Powder.

A) Compaction of Waste Ceramic Powder



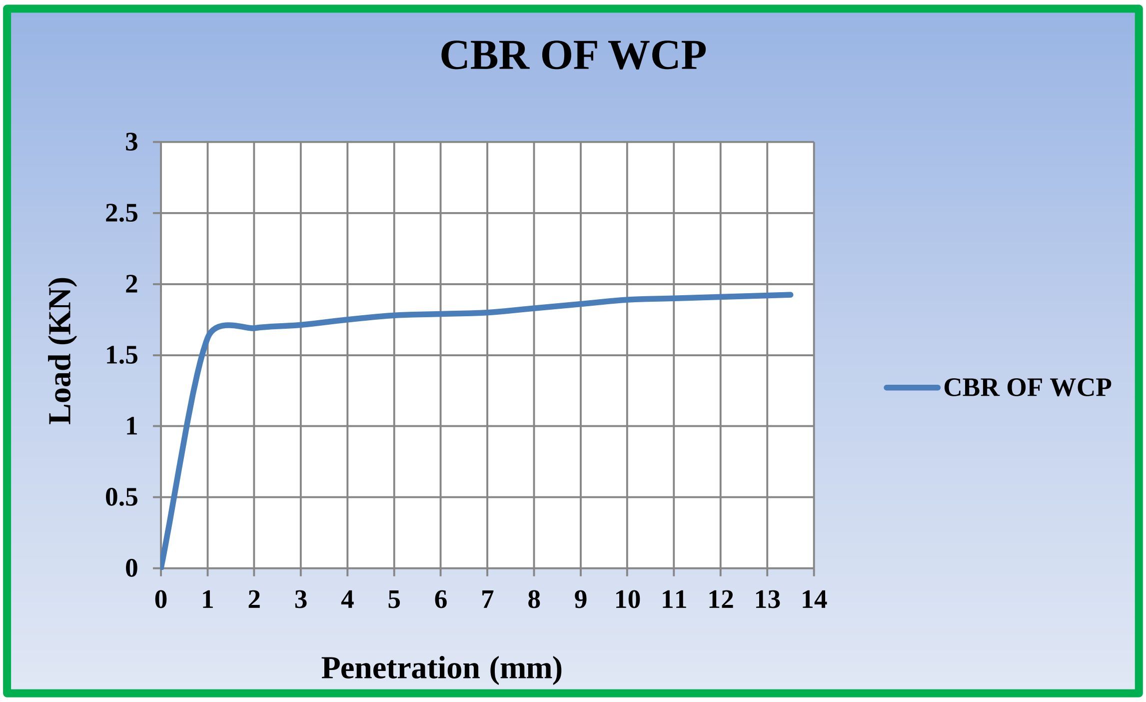
B)Atterberg Limit of Waste Ceramic Powder



LL	27.5
PL	17
PI	10.5

B) California Bearing Ratio of Waste Ceramic Powder

		CBR OF WCP				
Standard Load(KN)		**Load (KN)**		**CBR(%)**		**CBR-Swell (%)**
2.54	**5.08**	**2.54**	**5.08**	**2.54**	**5.08**	
13.24	20	1.71	1.78	12.9	9.06	0.132

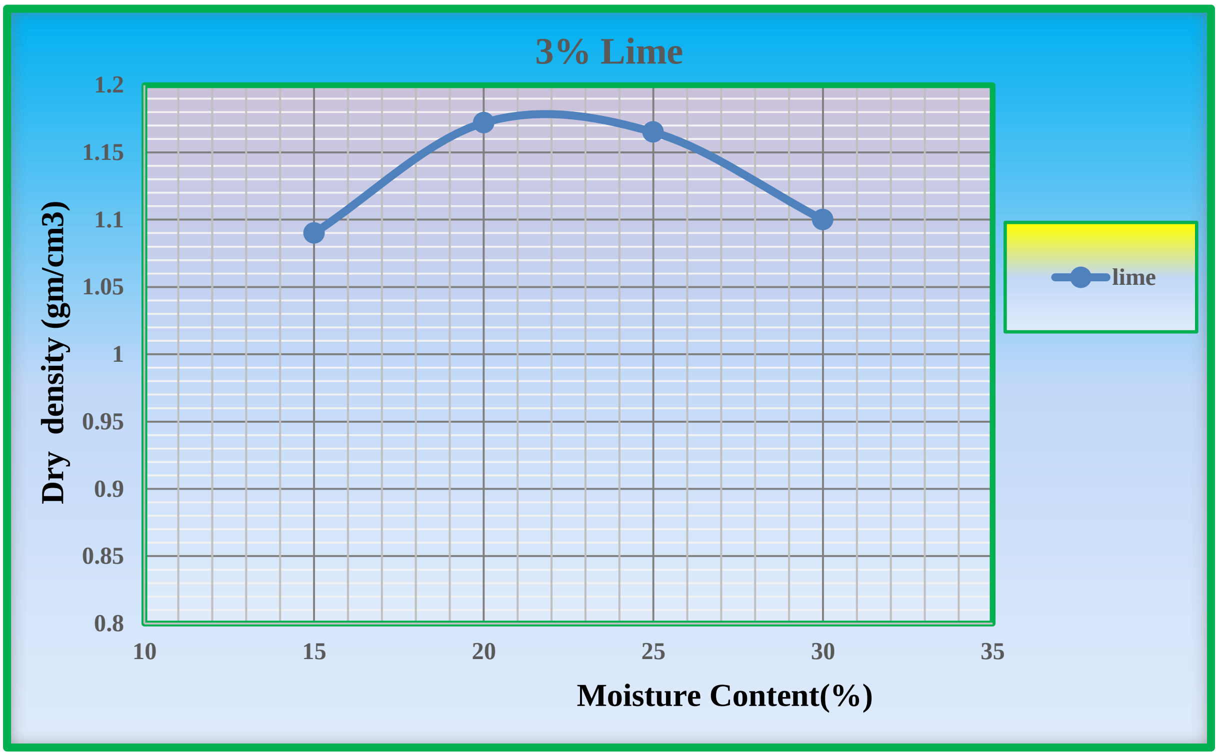


$$\mathbf{CBR=\frac{\text{Load reading from penetration point at 2.54 and 5.04}}{\text{Standard Load at 2.54 and 5.04}}*100}$$

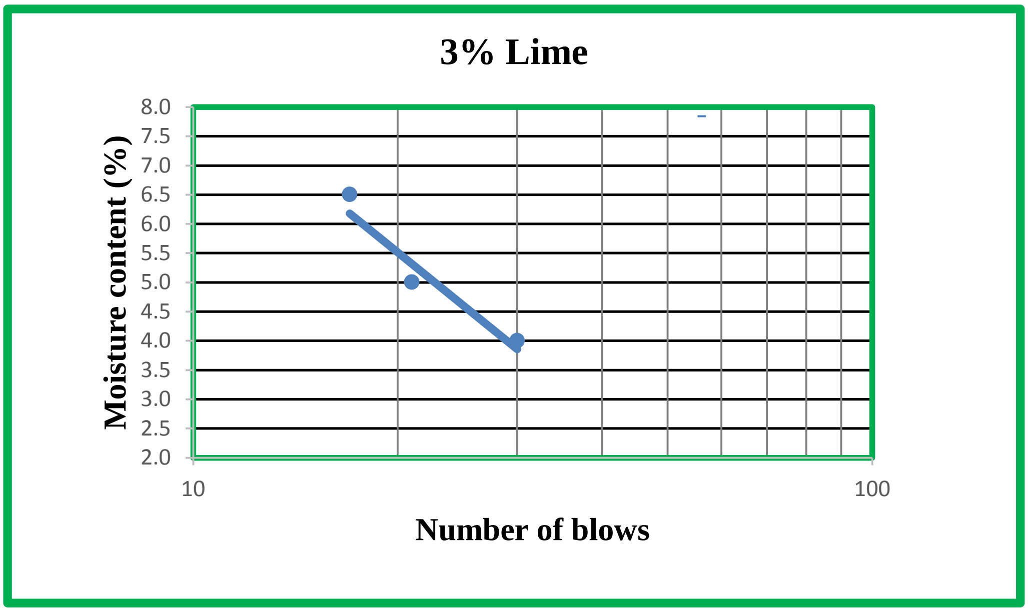
CBR=1.71/13.24=12.9

Appendix 2: Geotechnical properties of Lime.

A) Compaction of Lime



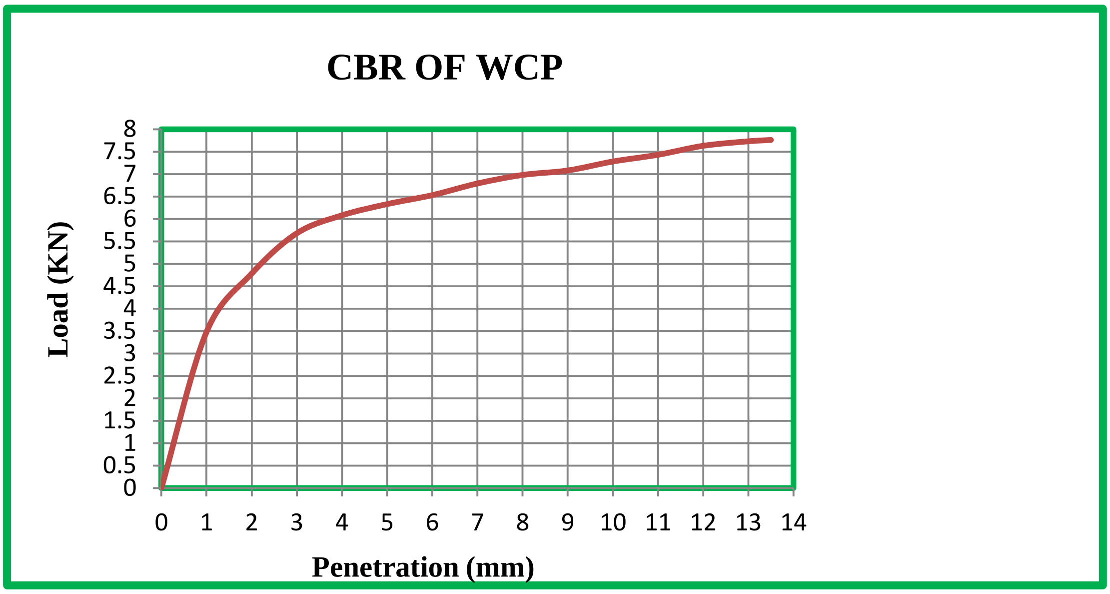
B) Atterberg Limit of Lime



LL	**4.5**
PL	**2.8**
PI	**1.7**

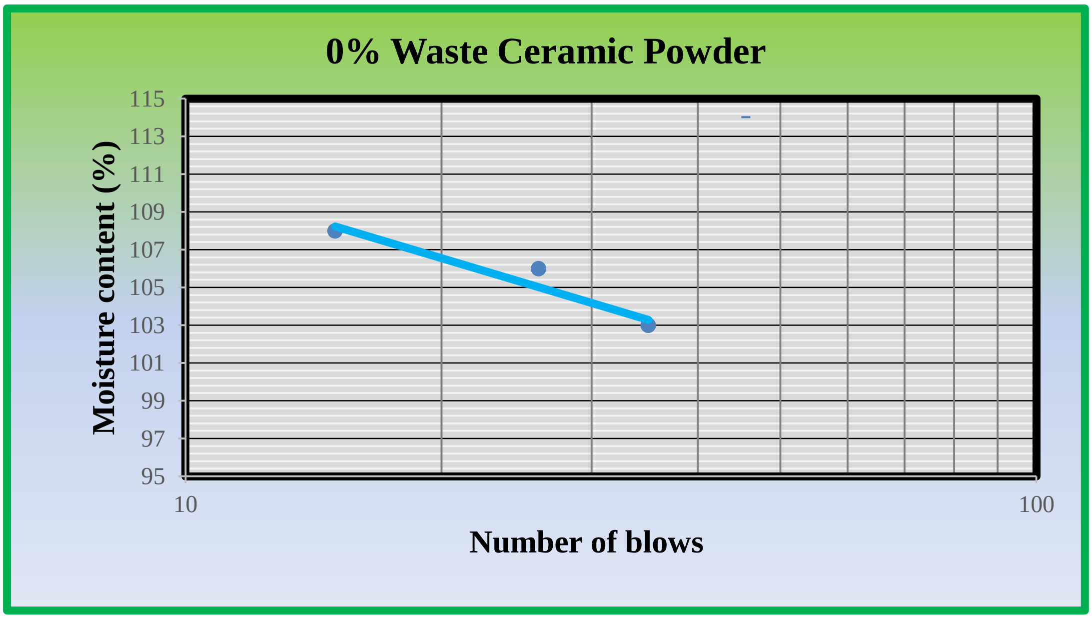
C) California Bearing Ratio of Lime

			CBR OF LIME				
Curing Period	**Standard Load(KN)**		**Load (KN)**		**CBR(%)**		**CBR-Swell (%)**
	2.54	**5.08**	**2.54**	**5.08**	**2.54**	**5.08**	
o day	13.24	20	5.25	6.4	39.6526	32	0.18



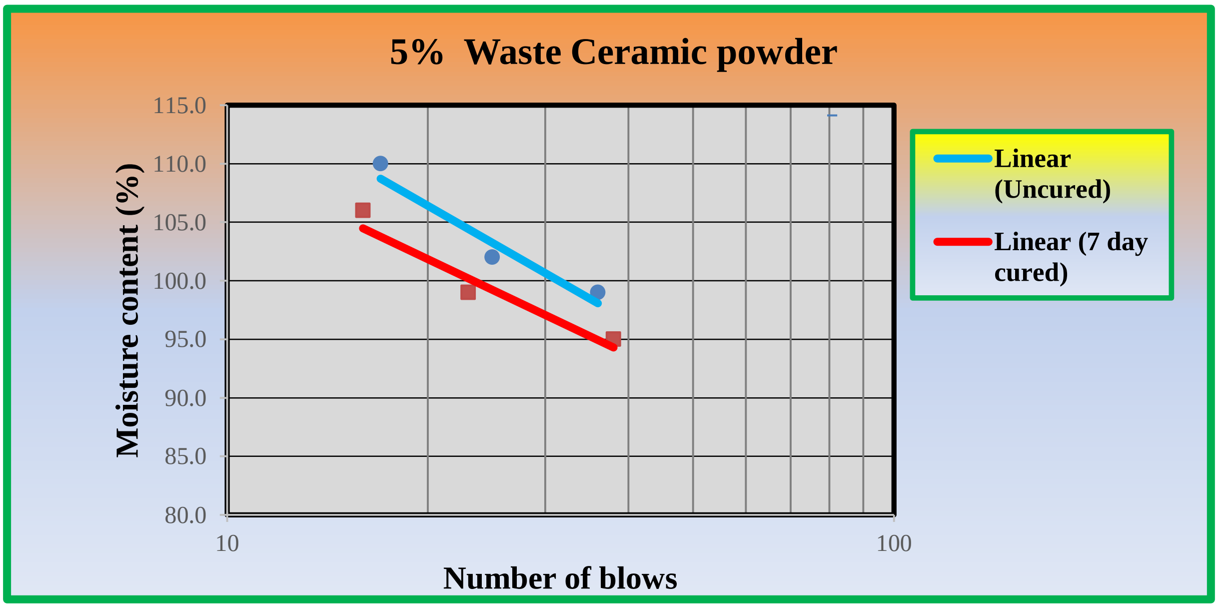
Appendix 3: Atterberg limits with varying percentage of Waste Ceramic Powder.

a) Waste Ceramic Powder Content = 0%



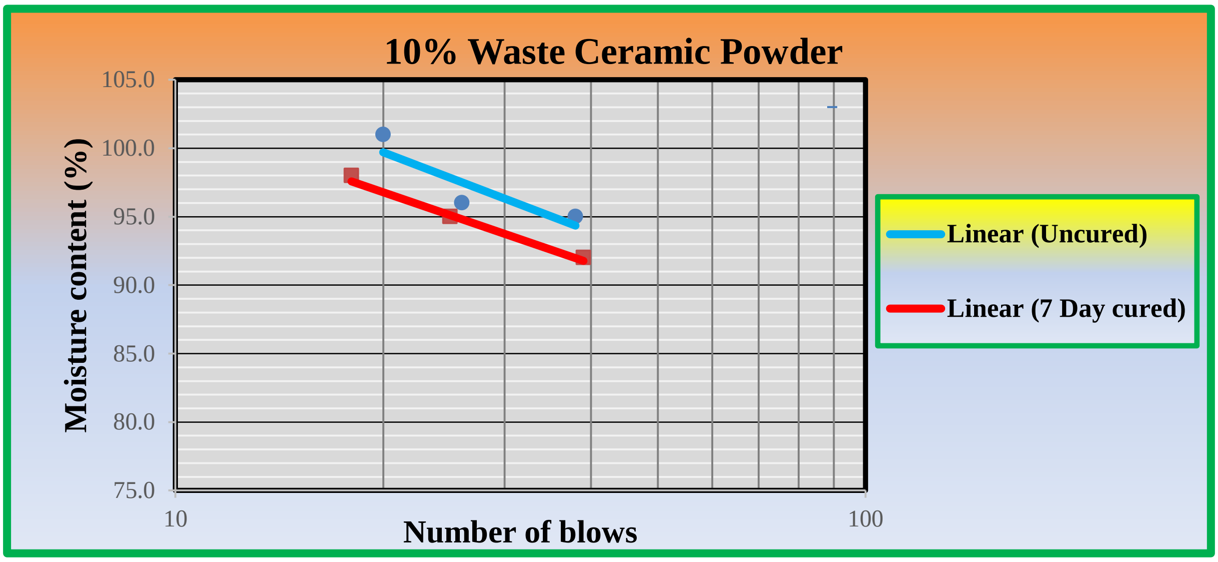
LL	106.5
PL	42.0
PI	64.5

b) Waste Ceramic Powder Content = 5%



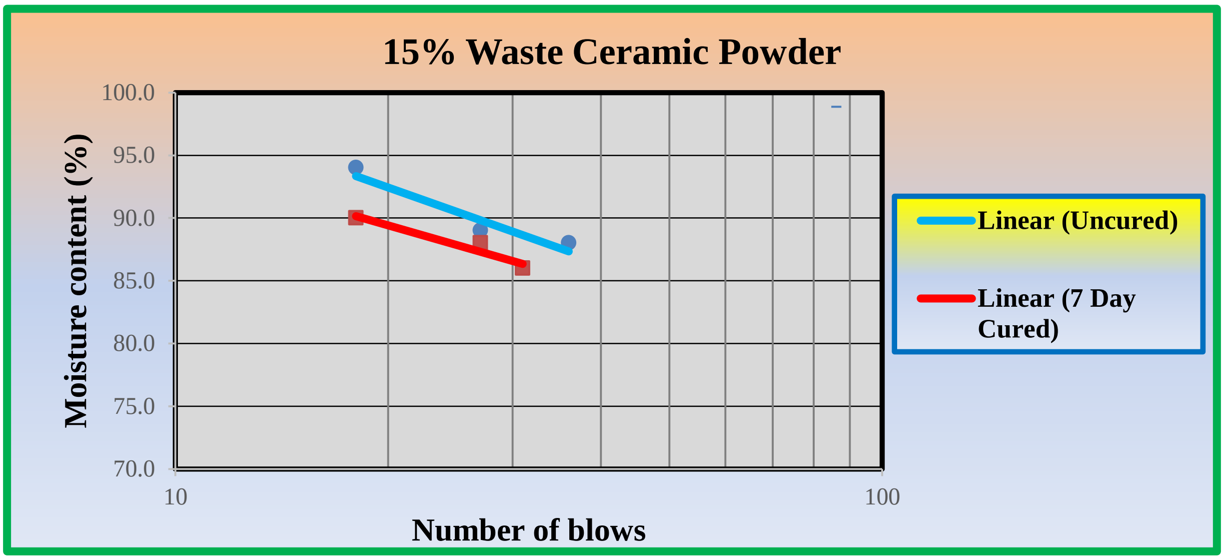
	Uncured	7 Day Cured
LL	101.75	98.53
PL	43.5	42.22
PI	58.3	56.3

c) Waste Ceramic Powder Content = 10%



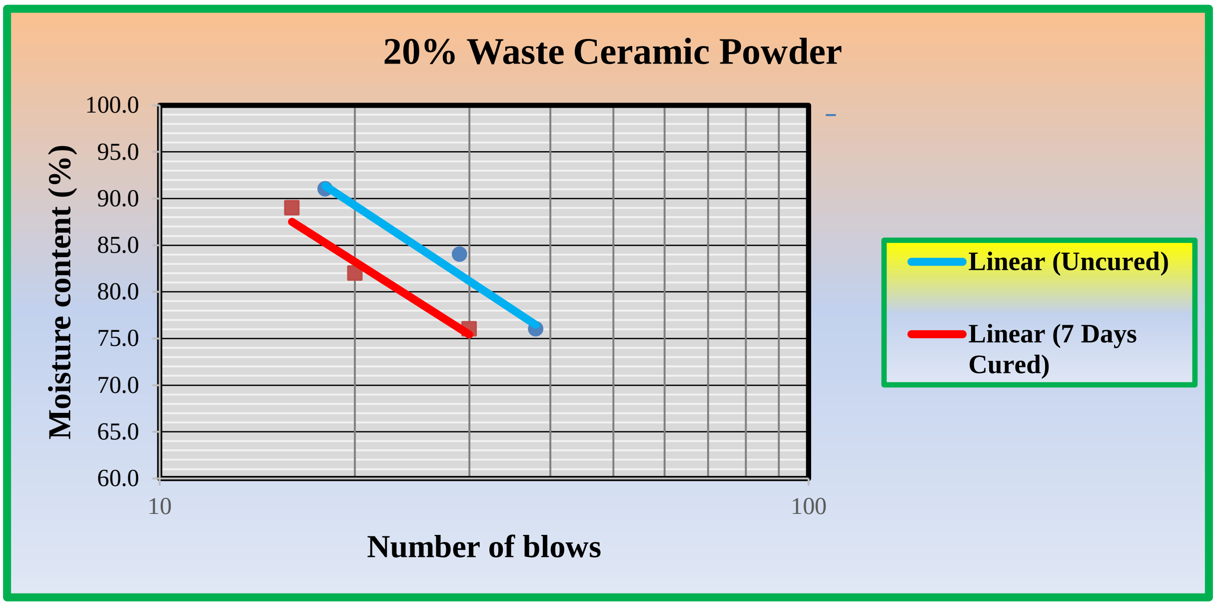
	Uncured	7 Day Cured
LL	98	95
PL	42.9	41.5
PI	55.1	53.5

d) Waste Ceramic Powder Content = 15%



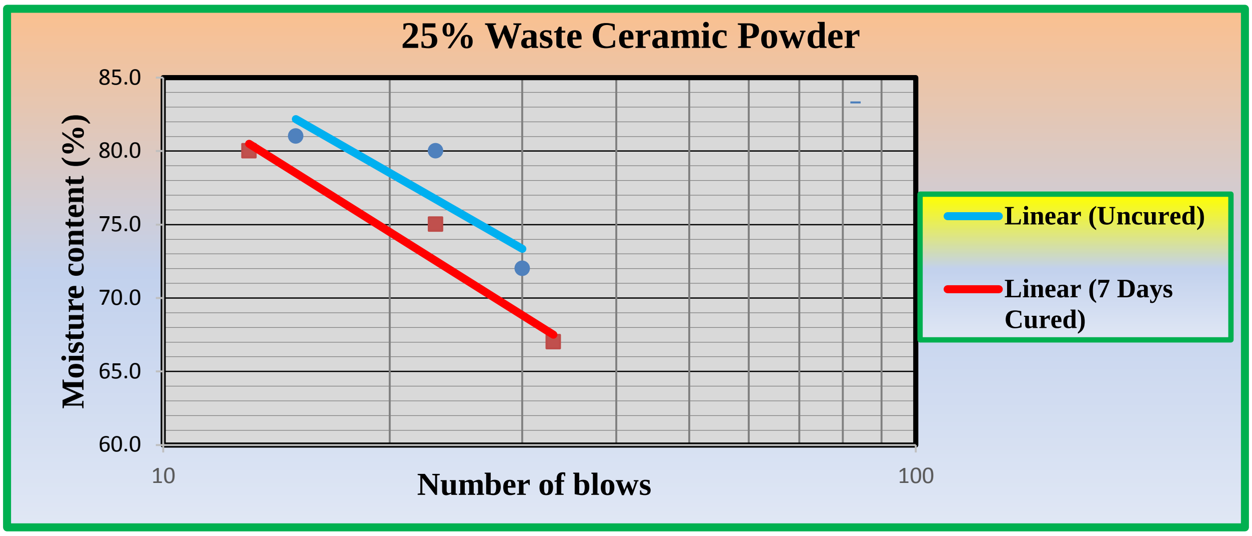
	Uncured	7 Day Cured
LL	91.4	88.75
PL	42	41
PI	49.4	47.8

e) Waste Ceramic Powder Content = 20%



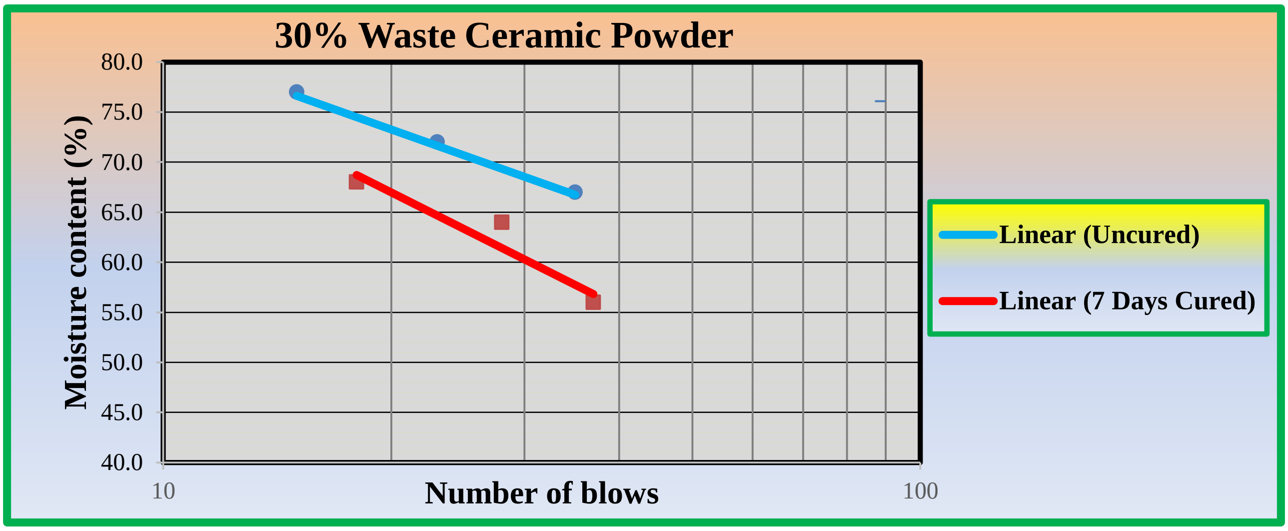
	Uncured	7 Day Cured
LL	84.3	80
PL	39.5	40
PI	44.8	40.0

f) Waste Ceramic Powder Content = 25%



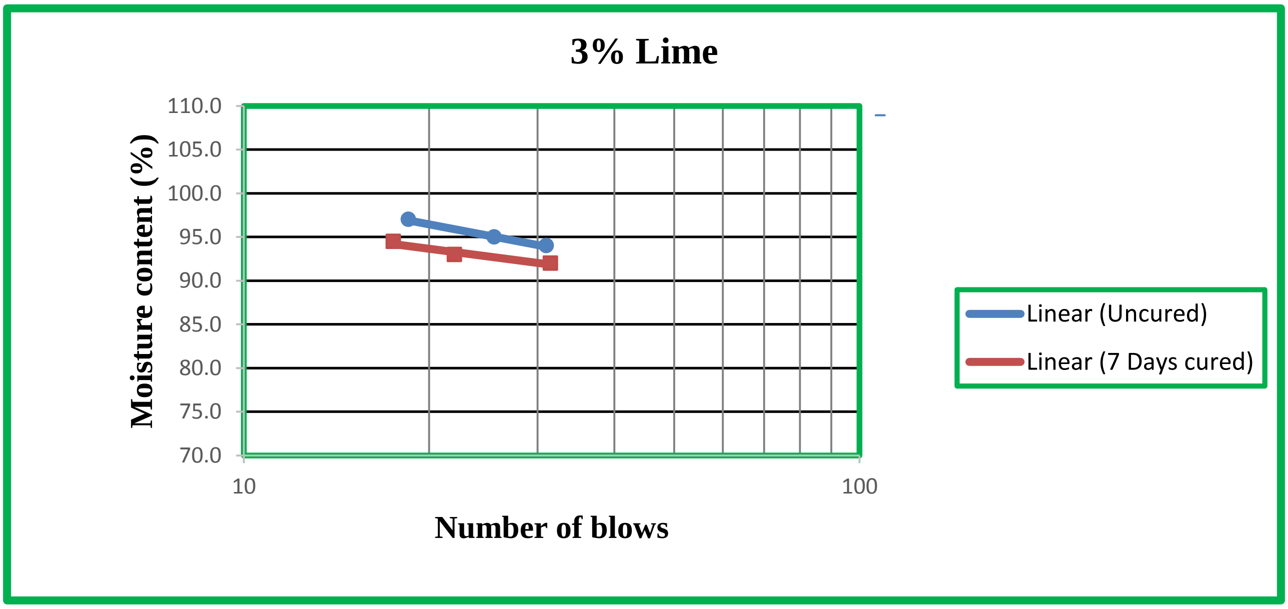
	Uncured	7 Day Cured
LL	76.00	73
PL	37.5	38
PI	38.5	35.0

g) Waste Ceramic Powder Content = 30%



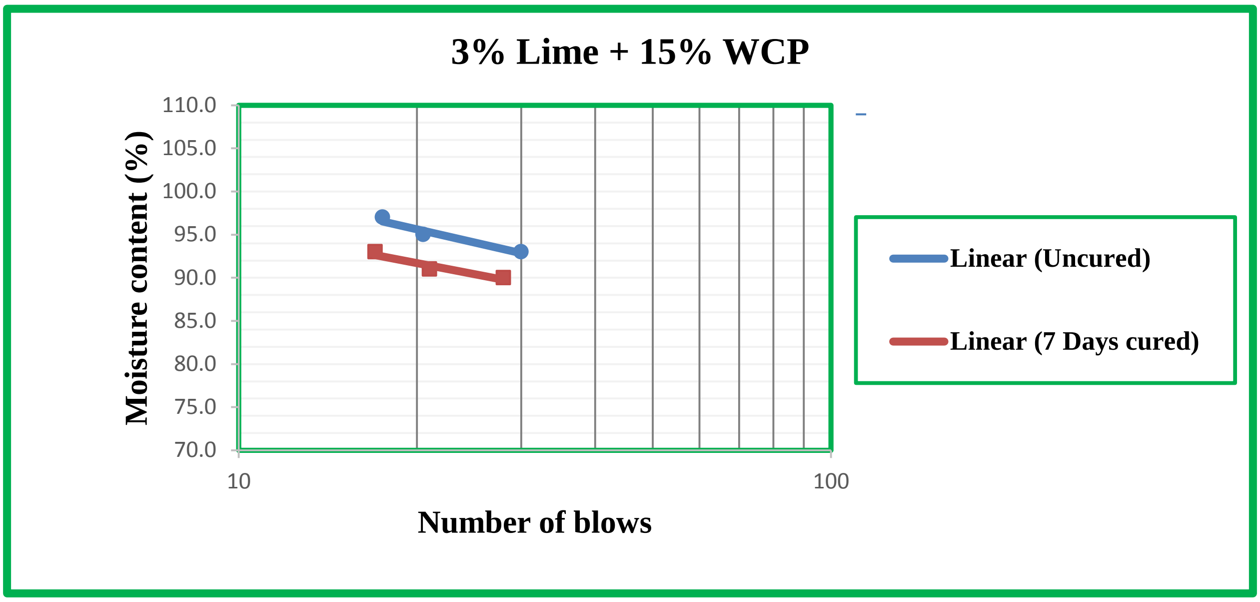
	Uncured	7 Day Cured
LL	70	63
PL	36	33.5
PI	34.0	29.4

h) 3% Lime



	Uncured	**7 Day Cured**
LL	94.9	92.5
PL	59.5	58
PI	35.4	34.5

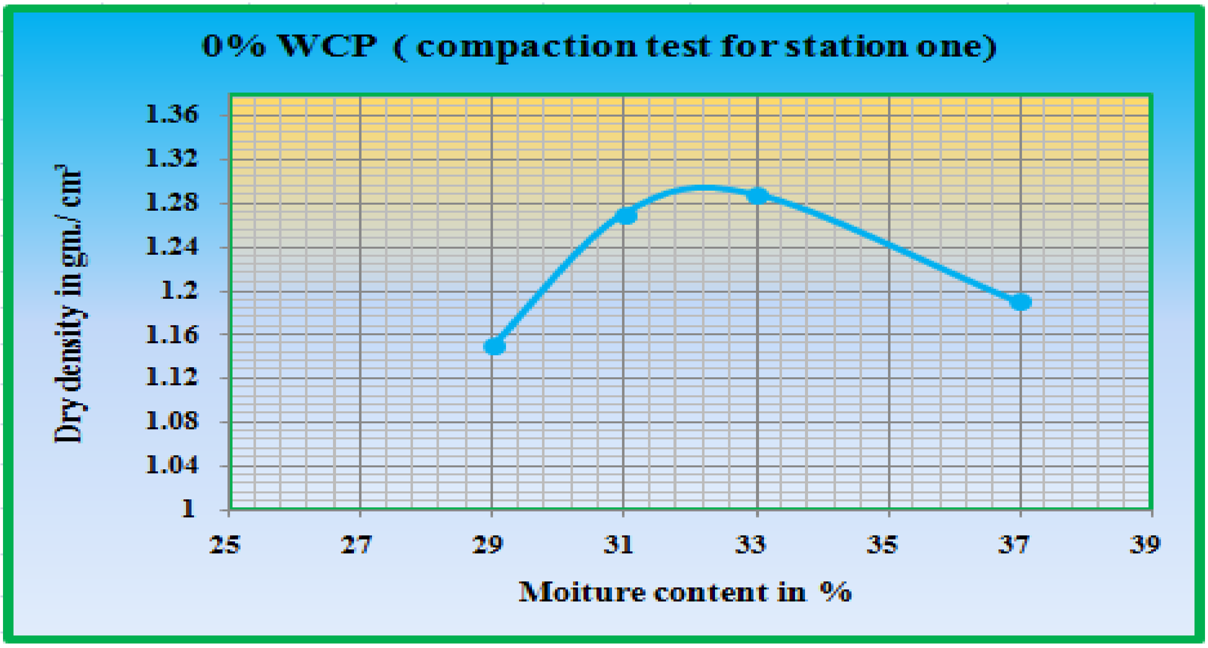
i) 3% Lime +15% WCP



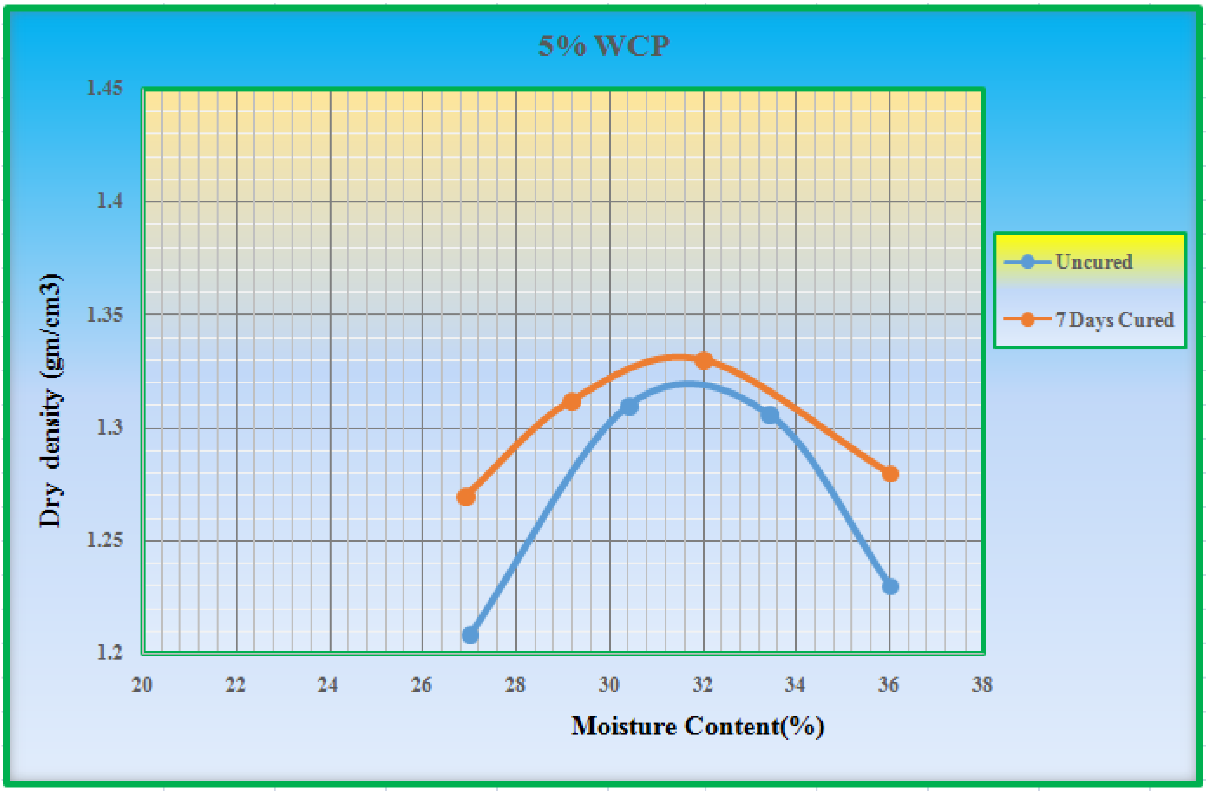
	Uncured	**7 Day Cured**
LL	93.5	90
PL	61.5	60.3
PI	32	29.7

Appendix 4: Compaction curves with varying percentage of Waste Ceramic Powder Content :

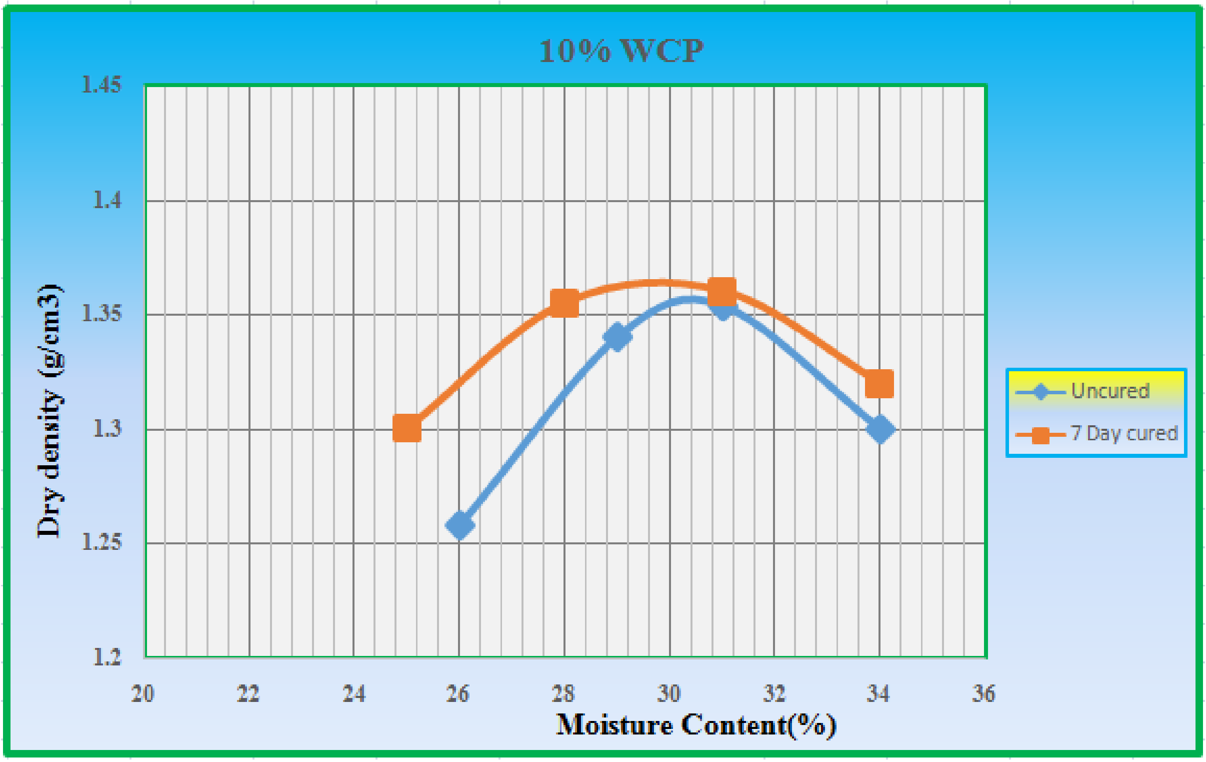
a) Waste Ceramic Powder Content =0%



b) Waste Ceramic Powder Content =5%



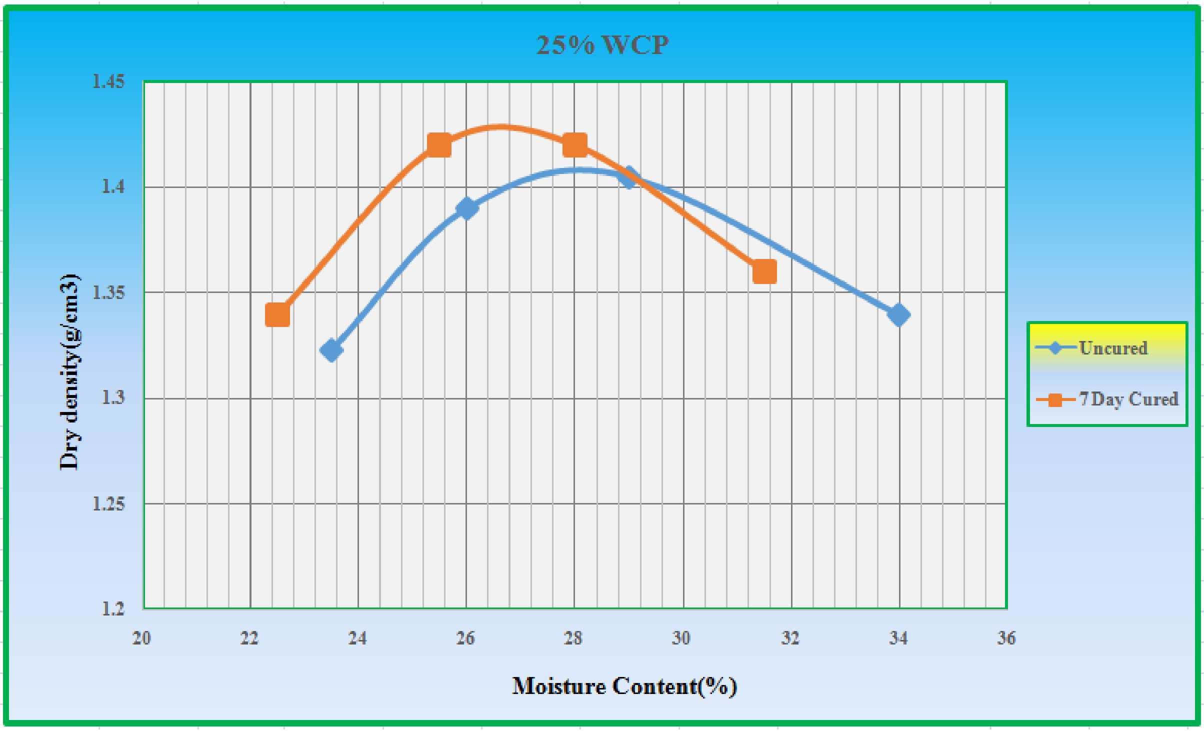
c) Waste Ceramic Powder Content =10%



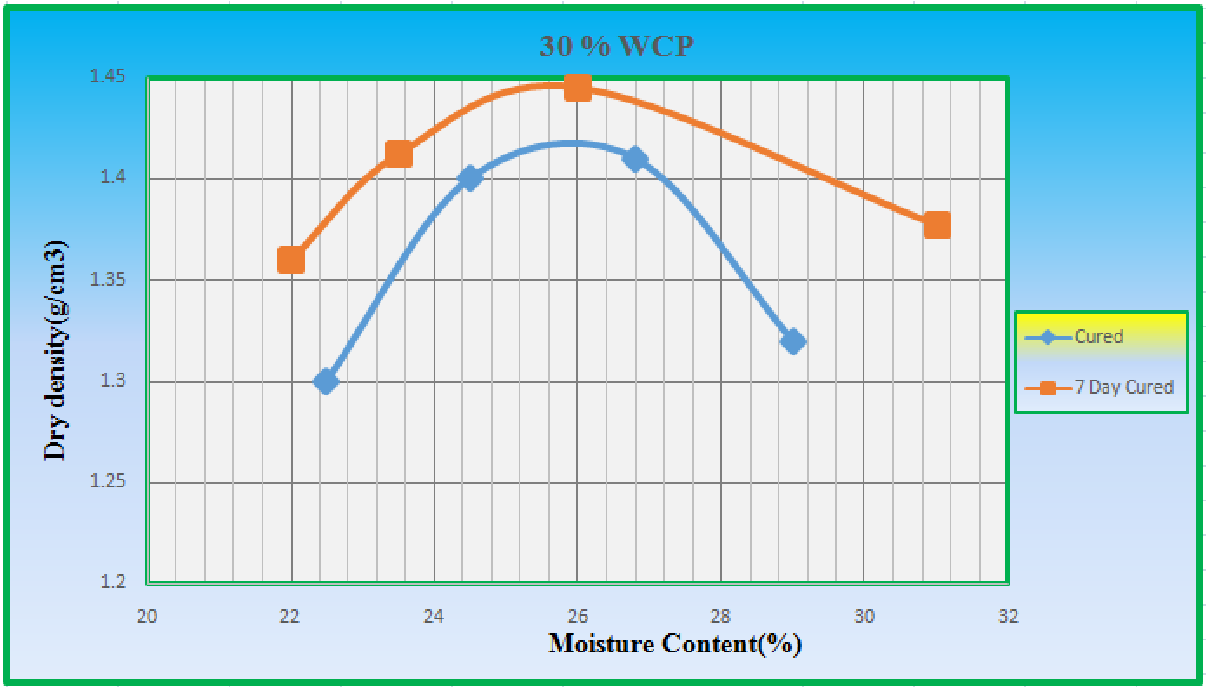
d) Waste Ceramic Powder Content =15%

e) Waste Ceramic Powder Content =20%

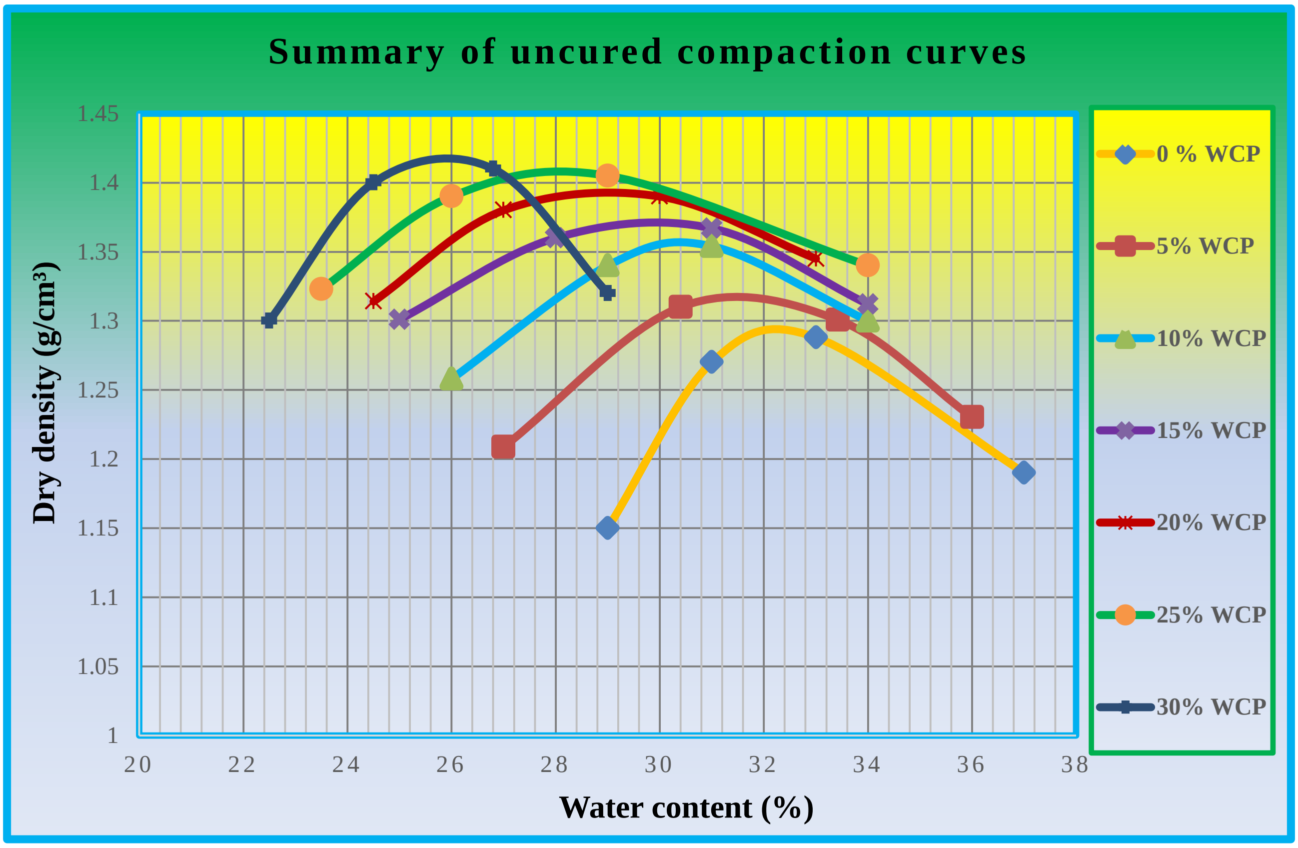
f) Waste Ceramic Powder Content =25%



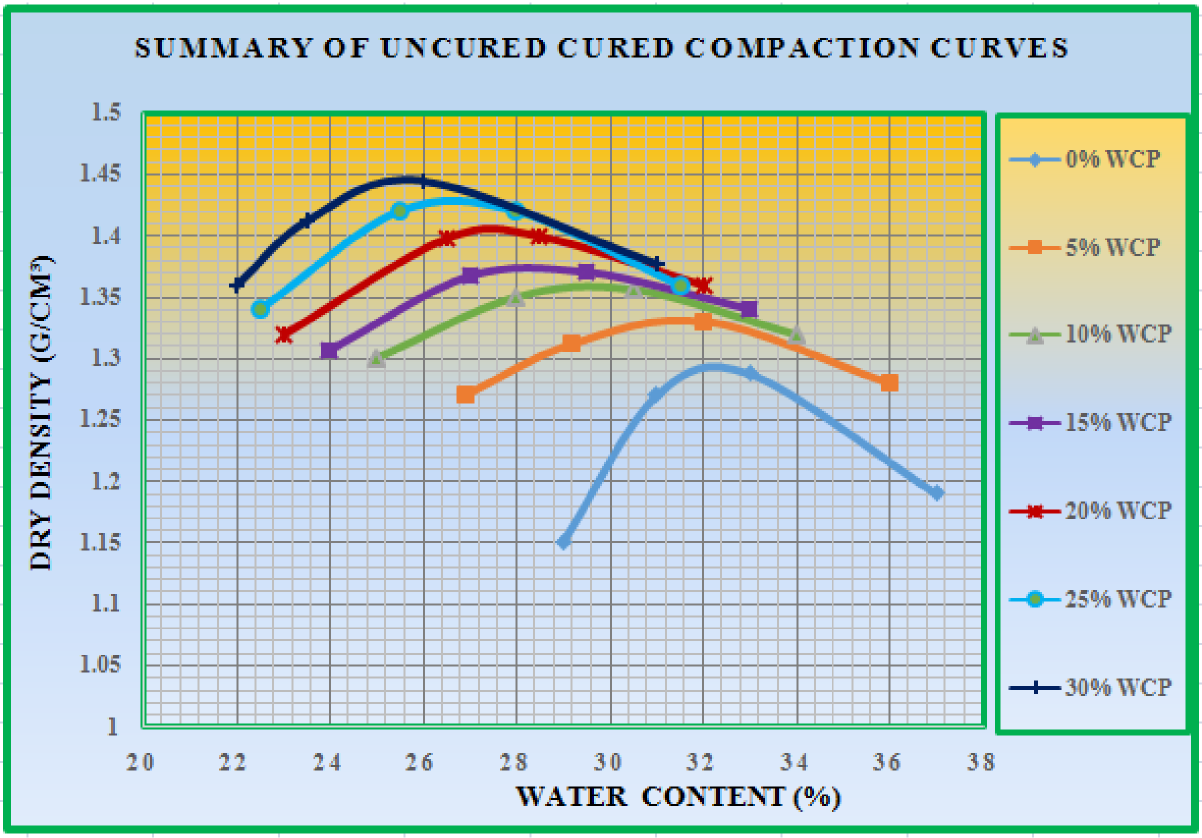
g) Waste Ceramic Powder Content =30%



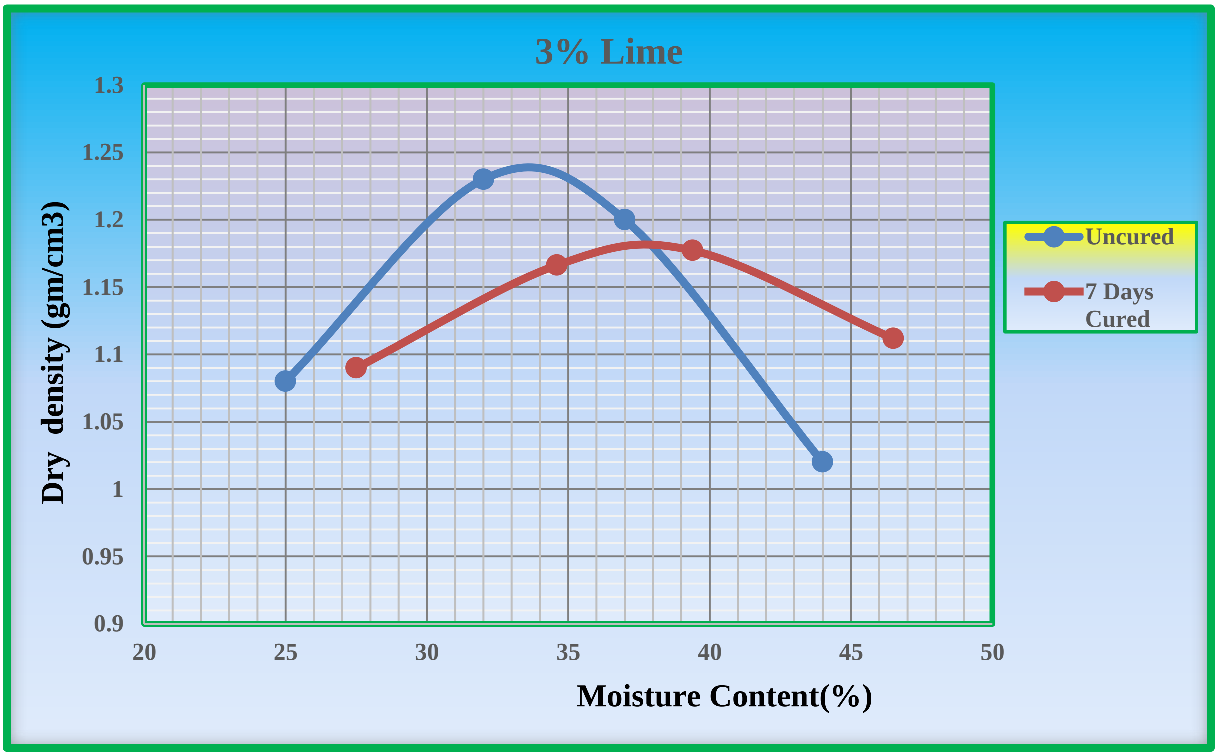
h) Waste Ceramic Powder Content =30%



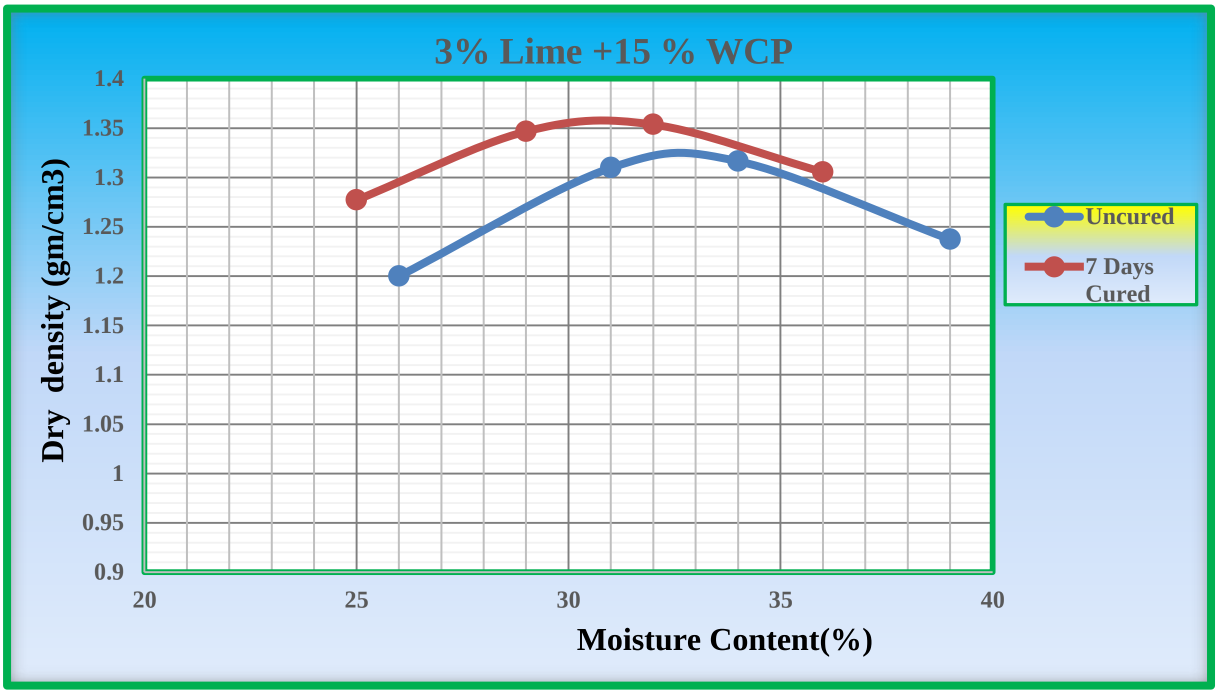
i) Waste Ceramic Powder Content =30%



j) 3% Lime



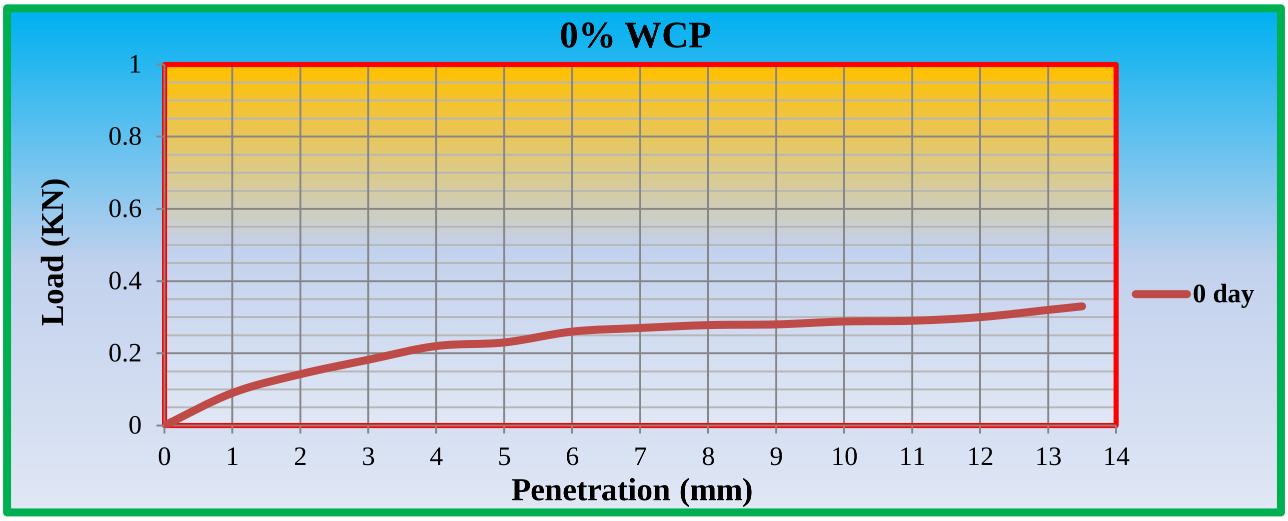
k) 3% Lime +15% Waste Ceramic Powder



Appendix 5: CBR Curves with varying percentage of Waste Ceramic Powder

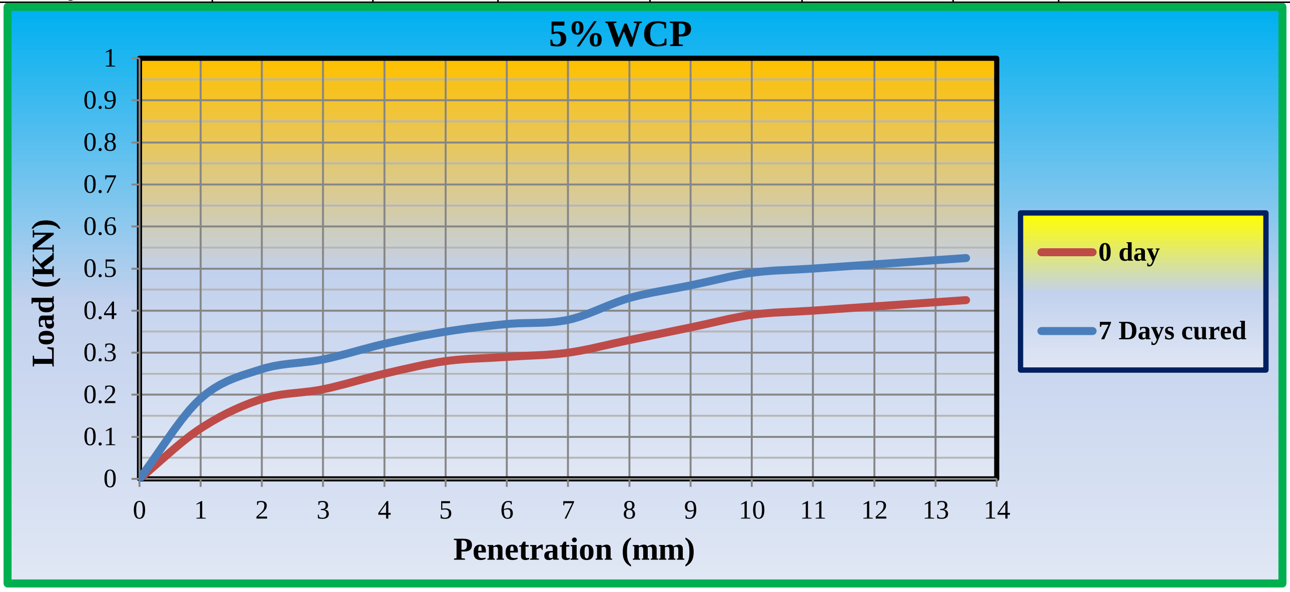
a) Waste Ceramic Powder=0%

Curing Period	Standard Load(KN)		Load (KN)		CBR(%)		CBR-Swell (%)
	2.54	**5.08**	**2.54**	**5.08**	**2.54**	**5.08**	
0 day	13.24	20	0.15	0.24	1.13293	1.2	9.2



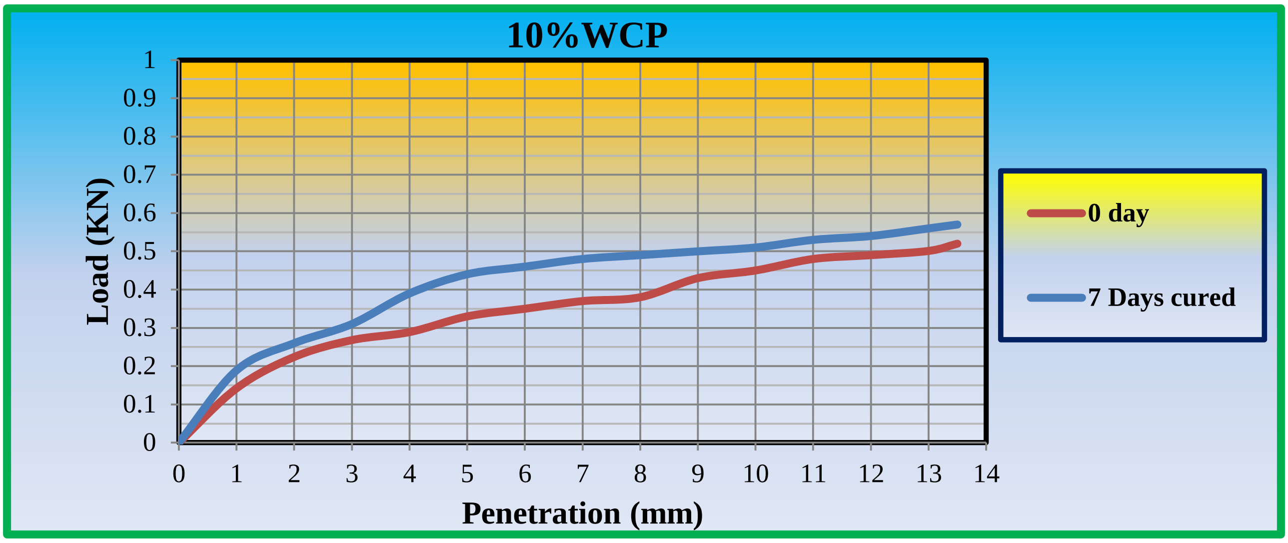
b) Waste Ceramic Powder=5%

Curing Period	Standard Load(KN)		Load (KN)		CBR(%)		CBR-Swell (%)
	2.54	**5.08**	**2.54**	**5.08**	**2.54**	**5.08**	
0 day	13.24	20	0.205	0.287	1.54834	1.435	8.05
7 days	13.24	20	0.287	0.356	2.16767	1.78	7.5



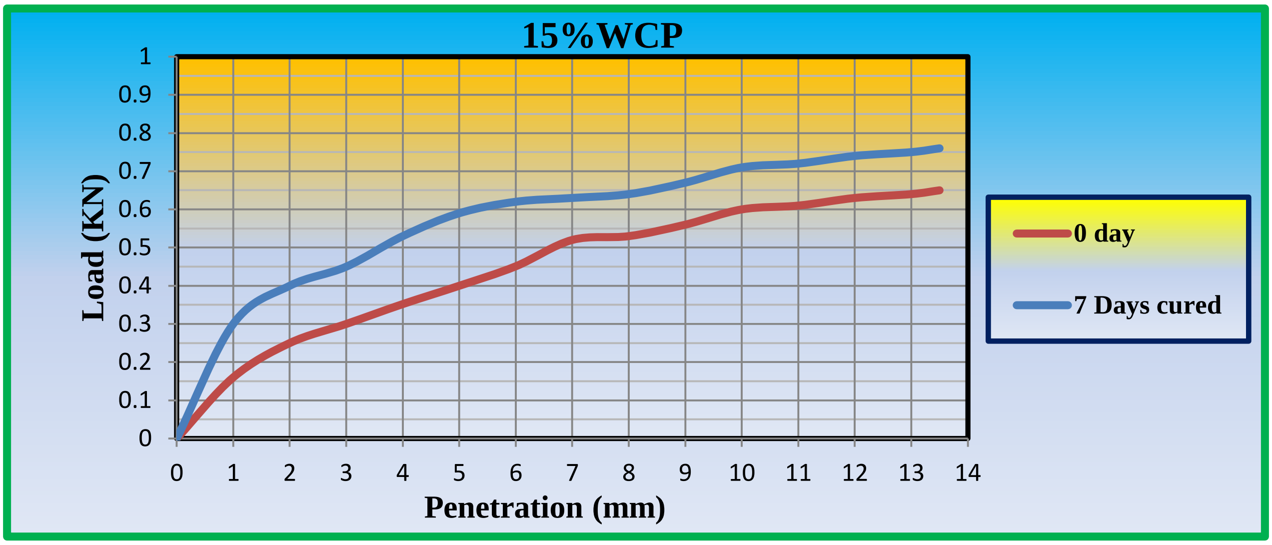
c) Waste Ceramic Powder=10%

Curing Period	Standard Load(KN)		Load (KN)		CBR(%)		CBR-Swell (%)
	2.54	**5.08**	**2.54**	**5.08**	**2.54**	**5.08**	
o day	13.24	20	0.277	0.335	2.09215	1.675	7.12
7 days	13.24	20	0.315	0.45	2.37915	2.25	6.2



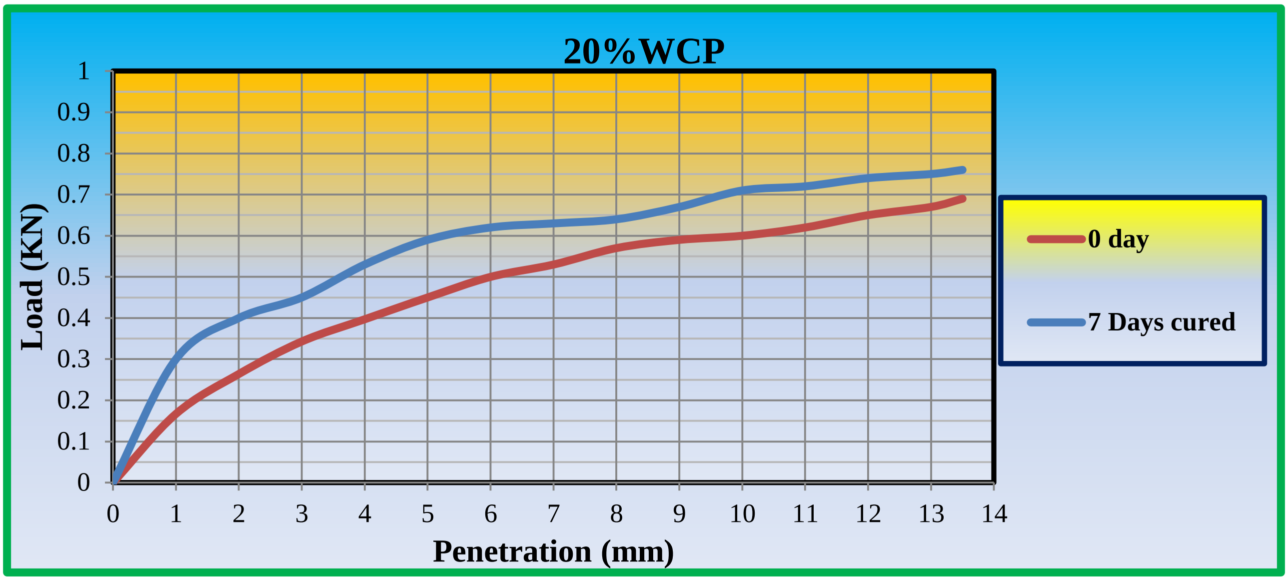
d) Waste Ceramic Powder=15%

Curing Period	Standard Load(KN)		Load (KN)		CBR(%)		CBR-Swell (%)
	2.54	**5.08**	**2.54**	**5.08**	**2.54**	**5.08**	
o day	13.24	20	0.312	0.461	2.3565	2.305	6.34
7 days	13.24	20	0.423	0.604	3.27	3.02	5.67



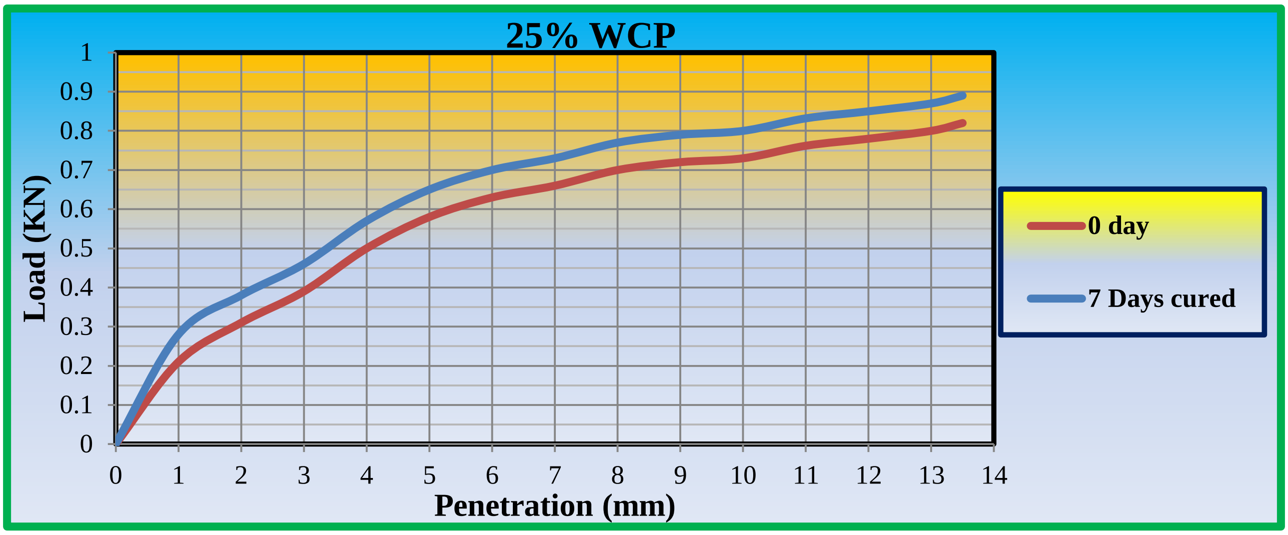
e) Waste Ceramic Powder=20%

Curing Period	**Standard Load(KN)**		**Load (KN)**		**CBR(%)**		**CBR-Swell (%)**
	2.54	**5.08**	**2.54**	**5.08**	**2.54**	**5.08**	
o day	13.24	20	0.325	0.486	2.45468	2.43	5.24
7 days	13.24	20	0.461	0.67	3.48187	3.35	4.54



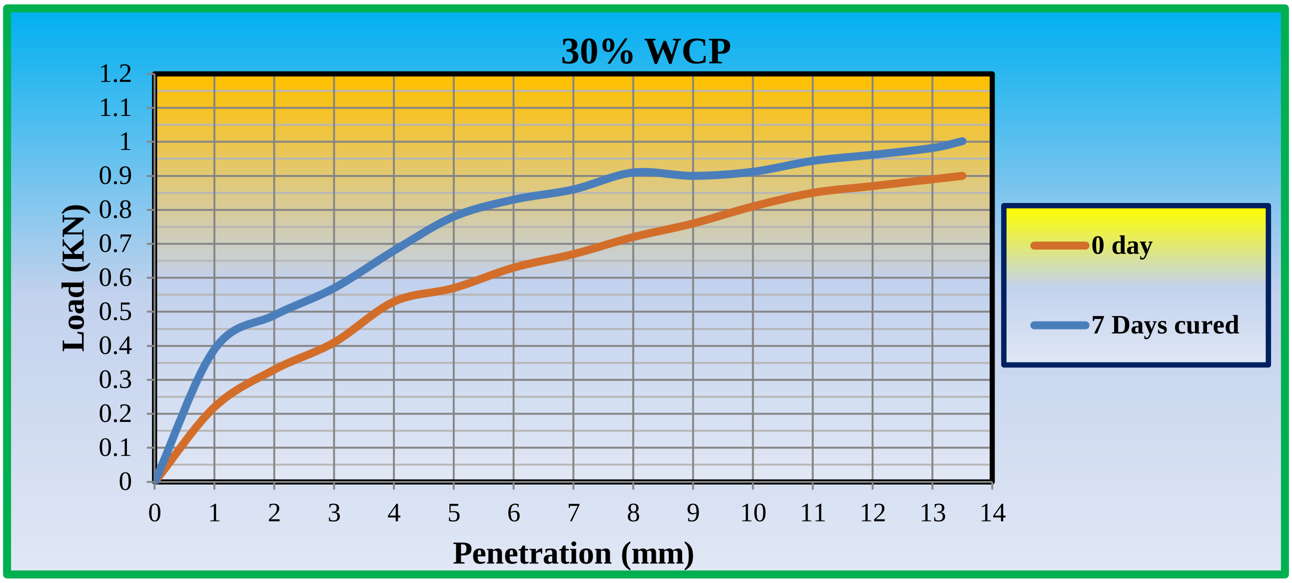
f) Waste Ceramic Powder=25%

Curing Period	**Standard Load(KN)**		**Load (KN)**		**CBR(%)**		**CBR-Swell (%)**
	2.54	**5.08**	**2.54**	**5.08**	**2.54**	**5.08**	
o day	13.24	20	0.35	0.562	2.6435	2.81	4.13
7 days	13.24	20	0.43	0.699	3.24773	3.495	3.34



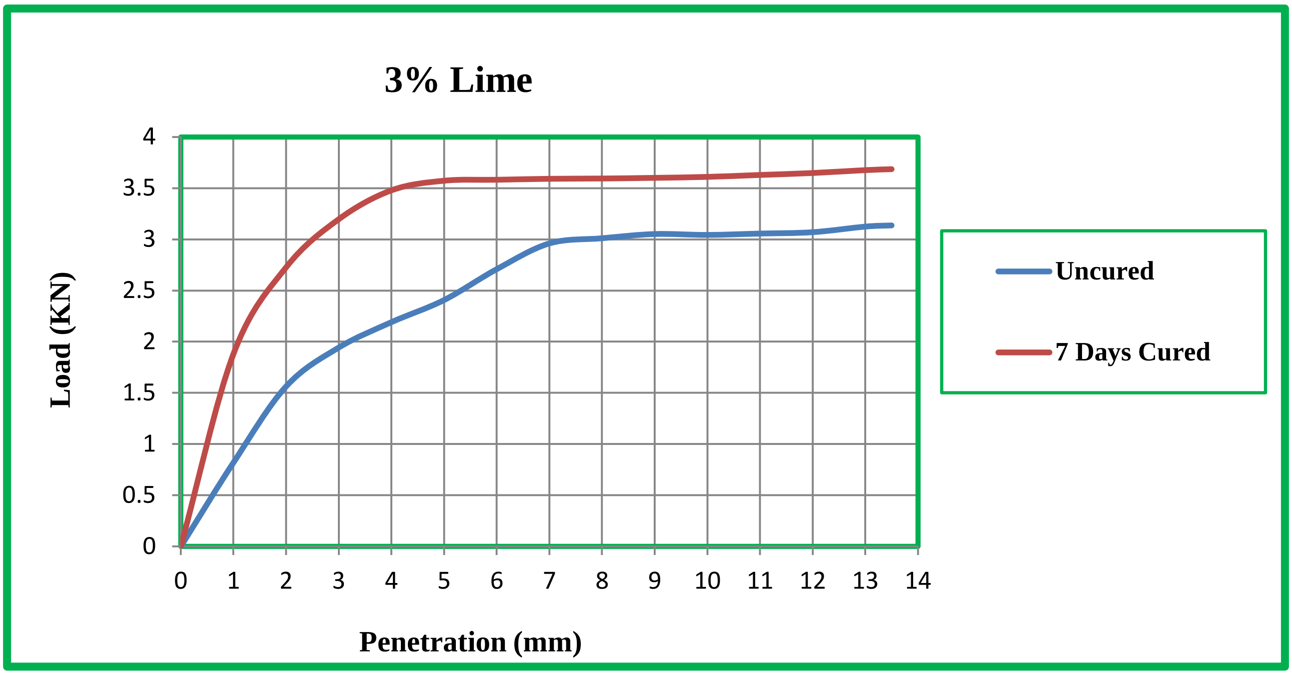
g) Waste Ceramic Powder=30%

Curing Period	**Standard Load(KN)**		**Load (KN)**		**CBR(%)**		**CBR-Swell (%)**
	2.54	**5.08**	**2.54**	**5.08**	**2.54**	**5.08**	
o day	13.24	20	0.42	0.592	3.17221	2.96	3.29
7 days	13.24	20	0.54	0.73	4.37855	3.65	2.08



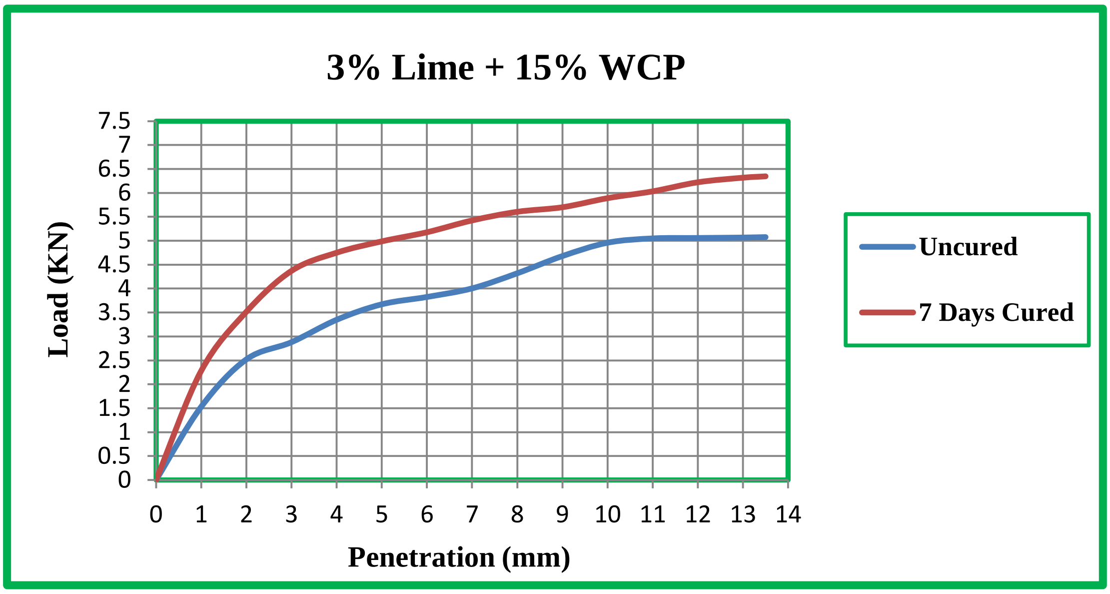
h) 3% Lime

Curing Period	**Standard Load(KN)**		**Load (KN)**		**CBR(%)**		**CBR-Swell (%)**
	2.54	**5.08**	**2.54**	**5.08**	**2.54**	**5.08**	
o day	13.24	20	1.95	2.5	14.7281	12.5	7.12
7 days	13.24	20	3.01	3.6	22.7341	18	6.2



i) 3% Lime+15% WCP

Curing Period	Standard Load(KN)		Load (KN)		CBR(%)		CBR-Swell (%)
	2.54	5.08	2.54	5.08	2.54	5.08	
o day	13.24	20	2.7	4	20.3927	20	2.09
7 days	13.24	20	3.8	5.3	28.7009	26.5	0.82



j) Calculation of CBR and Swelling.

	Uncured		7 Days Cured	
%WCP	**CBR**	**Swelling**	**CBR**	**Swelling**
0	1.2	9.2	1.2	9.2
5	1.55	8.05	2.17	7.5
10	2.1	7.12	2.402	6.2
15	2.36	6.34	3.27	5.67
20	2.452	5.24	3.4	4.54
25	2.82	4.13	3.52	3.34
30	3.17	3.28	4.08	2.08

$$\text{CBR}=\frac{\text{Load reading from penetration point at 2.54 and 5.04*100}}{\text{Standard Load at 2.54 and 5.04}}$$

CBR Swell = (Change in Length in mm during Soaking/116.3mm)*100 %

Appendix 6: ASTM C 618 – 03 chemical requirements for pozzolana (ASTM standard 2003) And CBR Laboratory test pictures

Chemicals	**Pozzolan Class**		
	N	**F**	**C**
$SiO_2+Al_2O_3+Fe_2O_3$ (Min %)	70	70	50
SO_3 (Max%)	4	5	5
Moisture content (Max %)	3	3	3
Loss on ignition (Max %)	10	12	6

Preparation of Mold for CBR test


Fix lighting