JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

Comparative Study on Stabilization of Expansive Soil Using Waste Ceramic Dust and Limestone for Weak Subgrade Soil

A Thesis Submitted to the School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Highway Engineering Stream)

By:
Leta Jirata

March, 2020

Jimma, Ethiopia

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SCHOOL OF POST GRADUATE STUDIES

JIMMA UNIVERSITY

As members of the examining board of the final MSc open defense, we certify that we have read and evaluated the thesis prepared by **Leta Jirata** entitled: **“Comparative Study on Stabilization of Expansive Soil Using Waste Ceramic Dust and Limestone for Weak Subgrade Soil”** and recommended that it be accepted as fulfilling the thesis requirement for the degree of Master of Science in Highway Engineering.

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DECLARATION

I, the undersigned, declare that this thesis is my original work and has not been presented for a degree in any other university and that all sources of materials used for this thesis proposal have been duly acknowledged.

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Signature: ______________________________

Title of the thesis:

Comparative Study on Stabilization of Expansive Soil Using Waste Ceramic Dust and Limestone for Weak Subgrade Soil

I have submitted this thesis for examination with my approval as university Adviser for the above mentioned title.

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ABSTRACT

Soil stabilization is one of the ground improvement methods for treating weak soils. Unfit for engineering purpose and makes them suitable for construction purposes. However, there is a need to look toward different industrial waste materials that are being produced in huge quantities and it used for the stabilization works. The expansive soil has a serious threat as it possesses seasonal variations of moisture content. It leads to severe damages to the pavements and foundations of the structure. In order to minimize this problem, it needs to be stabilized.

Therefore, the objective of this study was a comparative study on the stabilization of expansive soil using waste ceramic dust and limestone for weak subgrade soil for road construction. For the achievement of this study different materials were collected from Ambo town areas and different laboratory tests were designed and conducted. The laboratory investigation has been made to study the suitability of waste ceramic dust, limestone, and ceramic dust-limestone dust to improve the engineering properties of soil. The experimental tests were conducted to determine moisture content, specific gravity, grain size analysis, atterberg limits, proctor test, free swell test and California bearing ratio tests. The test procedures based on AASHTO, ASTM and IS laboratory test standards.

In this investigation, the engineering properties of the soil was determined and the strength characteristics of the soil sample treated with waste ceramic dust & limestone was observed by various percentage of waste ceramic dust from 5 to 25% at an increment of 5% and limestone from 0 to 10% at an increment of 2% by different weight ratio was mixed with soil sample. The comparison limestone stabilized with soil samples gave higher CBR values than waste ceramic dust. For the combination, it was established that replacing optimum 20% WCD and 8% limestone has resulted in better results than the performance of each individual and the soil mixed with limestone -waste ceramic dust was more appropriate and suitable for weak subgrade construction. In generally, expansive soils causes a serious problems on structures, so that it should be avoided or treated properly. This review provides the disposal problem of waste ceramic can be effectively used in stabilization works for weak subgrade soil.

Keywords: Expansive soil, Stabilization, Subgrade strength, Waste ceramic dust, limestone.

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ACRONOMY

AASHTO	American State Highway and Transportation Officials
USCS	Unified Soil Classification System
ASTM	American Society for Testing and Materials
ERA	Ethiopian Road Authority
CBR	California Bearing Ratio
WCD	Waste Ceramic Dust
GSE	Ethiopian Geological Survey
OMC	Optimum Moisture Content
MDD	Maximum Dry Density
FS	Free Swell
FSI	Free Swell Index
IS	Indian Standard
BSI	British Standard
LL	Liquid Limit
PL	Plastic Limit
PI	Plastic Index
YD	Dry Density of the Soil
YB	Bulk Density of the Soil
VD	Volume of the Soil Specimen Containing Water
VK	Volume of the Soil Specimen Containing Kerosene

CHAPTER ONE
INTRODUCTION

1.1 Background

Subgrade soil is the natural material underneath a constructed road or pavement. It is the foundation of the pavement structure and called formation level. 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 qualities of subgrade will greatly influence pavement design, performance, and service life. Roads constructed on expansive soil areas are known as bad conditions and unpredictable behavior for which the nature of the soil contributes to some extent. The failures of pavement, in the form of heave, depression, cracking and unevenness are most likely to happen by the expansive soil in the subgrade (Magdi, 2013).

In the parts of the world, this soil is problematic that causes extensive damage to civil engineering structures. Documented evidence is available on the existence and problems associated with expansive soils having occurred in countries like India, Africa, Australia, USA and Canada (Bashir, A., 2016).

Expansive soils are widely spread in African continent, occurring in South Africa, Kenya, Mozambique, Morocco, Ghana, Nigeria, Ethiopia, etc. in Ethiopia the following the major trunk roads like Addis- Ambo, Addis- Woliso and Addis- Debrebrihan, and some part of the Mekele, Gondar, Bahir dar, Debrebrihan, and Gambela are also known to be covered by expansive soils (Bantayehu, 2007:Teklu, 2003).

These soils occurred in Ambo town and its surrounding areas. Such soils expand when subjected to moisture and shrink when they lose moisture. Thus, alternative expansions and shrinkages lead to structural failure or settlements to the road (ATAO, 2012).

To eliminate the danger from such of these soils, the properties of the subgrade may need to be improved, either mechanically, chemically, or both to provide a platform for the construction of subsequent layers and to provide adequate support for the pavement over its design life (David Jones, A. R. S. H., 2010). According to (Mandeep, P. D. 2017) on his study, such soils need to be stabilized by amending the natural soil characteristics with an additive. These additives may include other soils or materials such as ceramic dust, cement, lime, fly ash, asphalt cement, polymers, fibers, marble dust, limestone dust, and quarry dust many more industrial wastes which may be used.

In Ethiopia, ceramic production occurred in different areas like Dukem, Dire Dawa, Addis Ababa, Adama, and Tabor ceramic productions that occurred in Hawassa. From those Tabor ceramic products manufacturing share companies were established in Hawassa city with a design capacity of 6000 tons per year. The factory constitutes three production lines; sanitary wares line (1000 tons/year), tablewares line (2000 tons/year), tiles (3000 tons/year) (Bekele, A .2015). Based on researcher Upadhyay (2016) on his study this ceramic production has wastages and it can be conveniently used for soil stabilization and the problem of their disposal can be overcome in an environmentally safe way.

However, limestone has occurred in many parts of Ethiopia which is predominantly found within the east-central part of the country. The best exposure and the most interesting deposits of the Antalo natural lime are found in the central part of the Abay Valley, and side valleys such as the Jema, Wonchit and Muger valleys and large limestone deposits are also found in the eastern part of Ethiopia, in the Harar-Hakimgara areas (Wondaferash and Hailu, 1993).

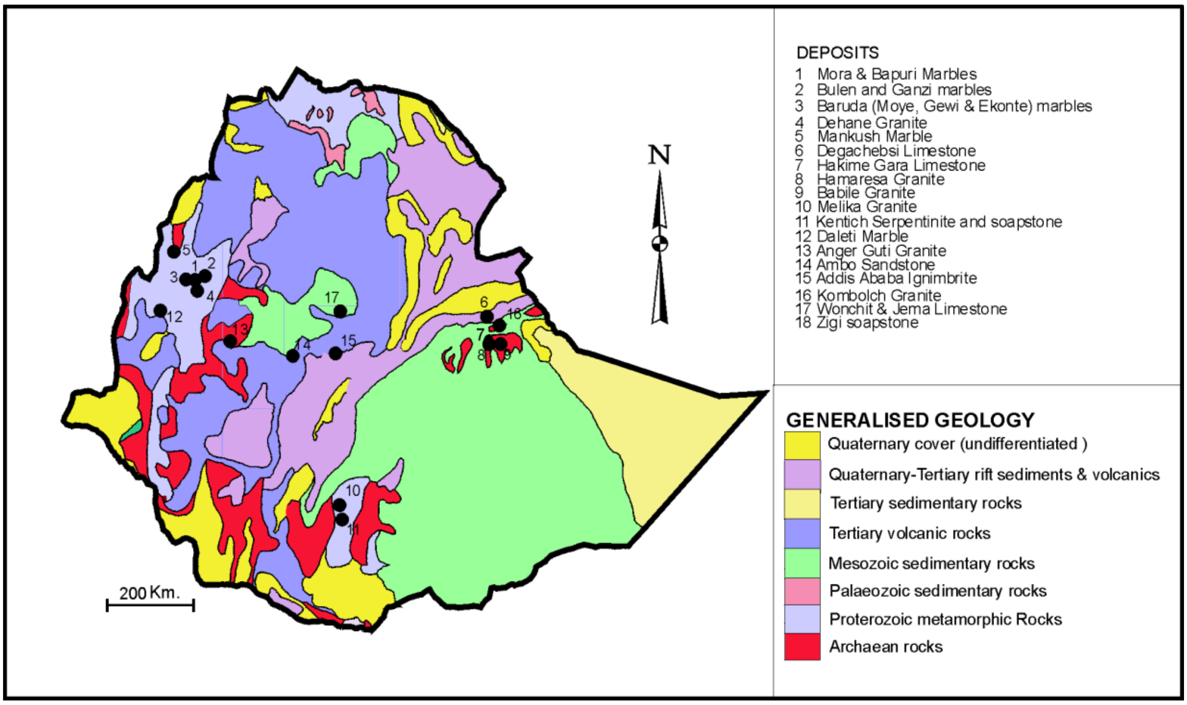

Figure 1-1: Geological map of Ethiopia with localities of building stone

The recent trends in research work in the field of geotechnical engineering and construction materials (Sabat et.al, 2011) focus on the examination for cheap and locally available materials such as waste marble dust, limestone dust, rise husk ash, etc. as stabilizing improvement agents for expansive soil.

In general, the possible use of industrial waste of ceramic dust and limestone would considerably be lessening the cost as well as using locally available materials to improve weak subgrade soil. Taking, these considerations, the objective of this study was stabilization weak subgrade soil using waste ceramic dust and limestone to reduce the cost of road construction as well as replacing the rather costly chemicals employed such as cement and lime.

1.2 Problem Statement

Expansive soils, occurring in arid and semi-arid climate regions of the world cause serious problems in civil engineering structures. Such soils swell when given access to water and shrink when they dry out. The swelling potential of expansive soil mainly depends upon the properties of soil and environmental factors and stress conditions (Masoumeh, M, 2012).

The expansive soils causes very serious geotechnical problems in various parts of Ethiopia. The problems encountered on these soils are mainly associated with excessive volume changes of the soil profiles when there is a change in moisture content. Those excessive volume changes can cause serious distress and damage to engineering structures such as buildings and roads built on them. Roads built on expansive soils prematurely primarily because of the highly variable properties, expansive clays due to moisture fluctuations throughout the year. Failures occur as a result of variations in strength and stiffness or subgrade volumetric change or both (Taye, 2015).

Relative to this Ambo town is dominated by expansive soils that have poor engineering properties hence is not used during infrastructure development. The most serious problem associated with these expansive soils in Ambo town is apparent in existing roads. The roads in Ambo town usually deteriorate quickly before their design periods reach. Hence, the influences of the soil erosion on town settlements like gully erosion, road undulations, cracks, and potholes, very sticky and plastic when wet that hinders driving vehicles and walking on the road (Debelo, N, 2015). However, there are many methods to avoid the problem that occurred due to expansive soil. One of the methods used to avoid problems of such soil is stabilization by using naturally occurring lime and waste ceramic dust. Thus, materials are cheap and locally available as stabilizing agents of expansive soil of the study area.

The above problems are attracting the researcher attention to solve the problems regarding to road pavement failure due to expansive soil by enhancing its engineering properties using limestone and waste ceramic dust used as stabilizers to evaluate the index properties, CBR test, proctor compaction, atterberg's limits, and free swell test of weak subgrades soil as stabilizing agents.

1.3 Research Questions

The main research questions to be answered by the researcher include the following:

1. What are the engineering properties of the expansive soil in Ambo town?
2. What are the potential effects of waste ceramic dust and Limestone as stabilizing agents for weak subgrade soil?
3. What are the optimum replacement ratio of waste ceramic dust and limestone mix as a stabilizing agent for weak subgrade soil as compared within specification?

1.4 Objective

1.4.1 General Objective

The main objective of this research was a comparative study on the stabilization of expansive soil using waste ceramic dust and limestone for weak subgrade soil.

1.4.2 Specific Objective

- ✓ To determine the engineering properties of expansive soil in Ambo town.
- ✓ To determine the effects of waste ceramic dust and limestone required to improve the engineering properties of subgrade soil.
- ✓ To determine the optimum replacement ratio of waste ceramic dust and limestone mix stabilizers for weak subgrade soil to meet the specification.

1.5 Significance of the Study

Ambo town has an abundance of available naturally occurring lime that can be used as a stabilizing agent for weak subgrade soil. Waste ceramic have a number of important propertries which were used for soil stabilization and problem of their disposal overcome as environmental in a safer way. Therefore, the aim of this research was conducted to study the effects of waste ceramic dust and limestone mixture for weak subgrade soil stabilization and to use the in-situ subgrade soil after treatment. Hence, study is useful for contractors, owners, managers and highway planners for using these materials as subgrade road embankment: also this finding is used for a researcher as secondary data.

1.6 Scope of the Study

The scope of this research was the stabilization of expansive soil using waste ceramic dust and limestone for weak subgrade soil stabilizer depending on the laboratory test. The laboratory test that was determine the effectiveness of waste ceramic dust and limestone mix as stabilizing agents for expansive soils are: grain size analysis, standard proctor compaction, CBR, free swell test, atterberg's limits (plastic limit and liquid limit). The test result was compared with the ERA design standard specification. The test was conducted in Jimma town and Jimma Institute of Technology (JiT).

CHAPTER TWO

LITERATURE REVIEW

2.1 Theoretical Review of Expansive Soil

Expansive soils exist all in different parts over worldwide and can cause damage to foundations infrastructures ranging from building structures to road structures (Seco et.al, 2011). Expansive soils undergo volumetric changes upon wetting and drying, thereby causing ground heave and settlement problems. This characteristic causes considerable construction defects if not adequately taken care off. The presence of montmorillonite clay mineral in expansive soils imparts them high swell-shrink potentials. Low rainfall has hindered the weathering of the active montmorillonite mineral into low active clay types such as illite and kaolinite (Amer, A. and Mattheus, F .A, 2006).

According to a researcher (Taye, 2015), on his study, there are several roads in Ethiopia whose failures were attributed to volumetric changes of expansive soil. However, the damage caused to the roads vary from the development of fine cracks on the road surface to premature pavement failures as the result of these: vehicle operating cost increases, traffic accident increases, travel time increases and a lot of money is usually spent on rectifying the damages to pavements built on expansive soil.

This soil has high plasticity and black or gray in color. They are characterized by their nature of expansion or shrinkage upon changes in moisture content. Foundations constructed on these soils are subjected to large uplifting forces caused by the swelling. These forces will induce heaving, cracking, and the breakup of different structures. Most of the structural damages due to expansive soils result from the differential rather than the total movements of the foundation soil as a result swell. Damages can occur within a few months following construction, may develop slowly over a period of a few years, or may not appear for many years until some activity occurs to disturb the soil equilibrium (Fikadu, A., 2015). According to (Bantayehu, 2017), on his study that construction of pavement on weak or soft subgrade soil is highly risky because such soil is susceptible to differential settlements, poor shearing strength, and high compressibility.

2.1.1 Clay Mineralogical Structure

1. Kaolinite

Kaolinite has one of clay mineralogical structure clay that consists of one silica sheet and one alumna sheet bonded together into a layer about 0.72mm thick and stacked repeatedly. The layers of these structures are held together by hydrogen bonds. Kaolinite has a few or no exchangeable cation, and the interlayer bonds are relatively strong preventing any hydrogen between layers and allowing many layers to build up. Kaolinite is relatively stable and the water is unable to penetrate between the layers. Consequently, kaolinite shows little swelling on wetting. Kaolinites are found in soils that have undergone considerable weathering in warm, moist climates. They have low liquid limits and low activity (Fasil, A., 2003).

2. Montmorillonite

This structure is also one of the clay mineralogical structures which are made up of sheet-like unit comprising an alumina octahedral sheet between two silica tetrahedral sheets. As the electrons rotate around the nucleus of an atom there will be times when there are more electrons on one side of the atom than the other, giving rise to a weak instantaneous dipole. Weak Vander Waals forces hold layers together and the bonding of these sheets is rather weak, resulting in a rather unstable mineral, especially when wet. In fact, montmorillonite display a significant affinity for water, with subsequent swelling and expansion. Its excessive swelling capacity may seriously endanger the stability of overlying structures and road pavements (Fasil, A., 2003).

3. Illite

Also, illites are slightly similar to montmorillonites in the structural units but are different in their chemical composition. In illite, the layers are separated by potassium ion, whereas in montmorillonite the layers are separated by loosely held water and exchangeable metallic ions. Unlike montmorillonites particles, which are extremely small and have a great affinity for water, the illite particles will normally aggregate and thereby develop less affinity for water than montmorillonites. Consistently, their expansion properties are less. The cation exchange capacity of illite is less than that of montmorillonite (Fasil, A., 2003).

2.1.2 Characteristics of Expansive Soils

The characteristics and nature of expansive soils are different. These weak soils which absorb water heavily, swell, become soft and lose strength. These soils are easily compressible when wet and possess a tendency to heave during the wet conditions and shrink in volume and develop cracks during dry seasons of the year. Also, expansive soils in relation to their free swell index (FSI) are called highly expansive when the free swell index exceeds 50% and such soils undergo volumetric changes leading to pavement distortion, cracking and general uneven due to seasonal wetting and drying (Rao, 2007).

Based on the researchers (Masoumeh, M. and M. Dehghani, 2012), studied that the swell-shrink potential of expansive soils is determined by its initial water content, dry density, void ratio, internal structure, and vertical stresses, as well as the type and amount of clay minerals in the soil. Generally, the larger the amount of these minerals presents in the soil, the greater the expansive potential. Fine-grained soils can absorb large quantities of water after a rainfall, becoming sticky and heavy.

2.1.3 Identification of Expansive Soils

The identification of expansive soils undertakes significant importance in checking the possible construction problems for the structures. Due to a steep increase in construction activities in recent times, there is a need for a quick and simple method to facilitate civil engineers in evaluating and identifying the expansiveness and swelling potential of soils. The identification of potential swelling or shrinking of subsoil problems is an important tool for the selection of suitable foundations (Chen, 1988).

2.1.3.1 Field Identification

Expansive soils are often like clay, becoming very sticky when wet and hard and brittle when dry. However, Some of the important field identification methods that indicate the potential for expansiveness of soil are: a shiny surface is easily obtained when a partially dry piece of the soil is polished with a smooth object such as the top of a fingernail, the wet sample of the soil is sticky and it is relatively difficult to clean the soil from the hands, the appearance of cracking in nearby structure, they usually have a color of black and or grey (Chen, 1988).

2.1.3.2 Experimental Identification

Generally, there are three different methods of identifying expansive soil in the laboratory experiment (Chen, 1988).

A) Direct Measurement

Chen (1988) recommended the direct method of expansion potential measurement to recognize expansive soils since the test is simple to perform and does not require any expensive laboratory equipment. According to him, x-ray diffraction is principally used in determining the proportions of various minerals present in colloidal clay and if supported by differential thermal analysis and skimming electron microscopic examination it provides good results.

B) Mineralogical Methods

The mineralogical composition of expansive soils has an important bearing on the swelling potential. There are a lot of factors that contribute to the swelling potential of the clay that has occurred like the negative electric charges on the surface of the clay mineral, the strength of the interlayer bonding, and the cation exchange capacity. Therefore, it is claimed by the clay mineralogists that the swelling potential of any clay can be evaluated by identifying the constituent mineral through the following methods: X-ray Diffraction, Differential Thermal Analysis, Dye Adsorption, Chemical Analysis, and Electron Microscopic Resolution. These different mineralogical identification methods are important in a research laboratory in exploring the basic properties of clays; however, they are impractical and uneconomical for practicing engineers.

C) Indirect Methods

This indirect method is used to investigate the swelling potential of soil by examining other parameters, which indirectly give information about the soil properties. These include the index property of the soil tests: CBR test, grain size analysis, Atterberg limit, free swells test, and potential volume change test (Chen, 1988).

2.2 Engineering Properties of Expansive Soil

2.2.1 Moisture content Test

Expansive soil has a higher affinity for water and with the higher the affinity the more swell it exhibits. Generally, the moisture content of the soil is the ratio between the mass of water in the sample and the mass of solid material. The water content of the material is used in expressing the phase relationship of air, water and solid in the given volume of material. The natural moisture content of the soil is affected by climate, vegetation cover of the area and other artificial factors. Hence, the same soil could have different moisture contents in different seasons of a year and at different times. Since such type of moisture content is likely to fluctuate any time it may not indicate the general property of the soil (Murthy, 2001).

According to a researcher (Forouzan, 2016), studied that fine-grained soils, the consistency which is a term used to indicate the degree of firmness of cohesive soils, of a given soil type depends on its water content and it can be very soft, soft, very stiff and hard. When the water content increases the consistency will be soft and as water decreases, it becomes hard the water content of the soil along with its liquid limit and plastic limits are used to express its relative consistency termed as liquidity index.

Table 2-1: Typical moisture content of soils (Terzaghi, K. and Peck, R. B., 1963)

Material	Moisture content
Gravel	2-10
Sand	5-15
Sits	5-40
Clays	10-50 (or more)
Organic (peat)	>50

2.2.2 Compaction of the Soil

Compaction is a process that brings an increase in soil density, accompanied by a decrease in air volume with no change in water content. The degree of compaction is measured by dry unit weight and depends on the water content and compaction effort (weight of the hammer, number of impacts, the weight of roller and number of passes). For a given compaction effort, the maximum dry density occurs at optimum water content. Mechanical compaction is one of the most common and cost-effective means in order to decrease the porosity (or voids ratio) of the soil and thus increase density. An extremely important task of geotechnical engineers is the performance and analysis of field control tests to assure that compacted fills are meeting the prescribed design specifications. Design specifications usually state the required density as a percentage of the maximum density measured in a standard laboratory test and the water content. In general, most engineering properties, such as strength, stiffness, resistance to shrinkage, and imperviousness of the soil, will improve by increasing the soil density (Reddy, 2002). Also the effects of compaction on the expansive soil: reduces the compressibility of the soil, thereby decreasing the tendency for settlement of structures founded on these soils, increase the dry density of the soil, thus increasing its shear strength and bearing capacity. Compaction of soil is measured in terms of the dry density of the soil, which is the weight of soil solids per unit volume of the soil by means of the following relation:

$$\gamma d = \frac{\gamma b}{1+W} \quad \text{.. Eq (1)}$$

Where γd = dry density of the soil

γb= wet or bulk density of the soil

w = water content expressed as a fraction

A) Dry density/ Water Content Relationship

The aim of the test is to establish the maximum dry density that may be attained for a given soil with a standard amount of compactive effort. When a series of samples of soil is compacted at different water content the plot usually shows a distinct peak. Compacting soil at water content higher than or wet of the OMC results irrelatively dispersed soil structure or parallel particle orientations. The soil compacted lower than or dry of the OMC typically results in a flocculated soil structure or random particle orientations (Reddy, 2002).

2.2.3 Sub-grade Strength

ERA (2013), suggested that the strength of road subgrade for flexible pavement is commonly assessed in terms of California bearing ratio (CBR) and this is dependent on the type of soil, its density, and its moisture content. Direct assessment of the likely strength or CBR of the subgrade soil under the completed road pavement is often difficult to make. Its value, however, can be inferred from an estimate of the density and equilibrium moisture content of the subgrade together with knowledge of the relationship between strength, density and moisture content for the soil in question. This relationship must be determined in the laboratory the density of subgrade soil can be controlled within limits by compaction at a suitable moisture content at the time of construction. The moisture content of subgrade soil is governed by the local climate and depth of the water table below the road surface. Hence, the strength of the sub-grade is classified into six subgrade strength classes.

Table 2-2: Sub grade strength classes (ERA, 2013)

Class	CBR Range (%)
S1	<3
S2	3,4
S3	5,6, 7
S4	8-14
S5	15-30
S6	>30

The structural catalog given in this manual requires that the subgrade strength for design be assigned to one of six strength classes reflecting the sensitivity of thickness design to subgrade strength. However, from the above table according to ERA (2013) for subgrade strength class with S1 needs special treatment. Also according to a researcher (Ayothiraman et.al, 2002), specified that the lower CBR values (less than 10), lead to the deflection of the subgrade material under heavy traffic loadings. Thus, it is very crucial for the engineers to develop a minimum of a CBR value of 10 for all subgrades.

2.2.3.1 Density- Moisture Content- Strength Relationships of the Subgrade

During road construction, the (dry) density of the Subgrade soil (and its moisture content) is modified from its original state by compaction at subgrade level (in cuts and by compaction of the excavated materials used in embankments. The moisture content is adjusted in order to make it easier to achieve a high level of compaction. Upon completion of the construction operations, the density of the compacted subgrade soil will remain approximately the same except for some residual compaction under traffic and possible volume variations of certain moisture-sensitive soils. However, the moisture content of the subgrade will change; depending on the climate, soil properties, depth of water table, rainfall and drainage. It is the knowledge of this condition of the subgrade that is required in the design process (ERA, 2013).

According to ERA (2013) illustrate the above discussion, the figure below shows the relationship between density, moisture content, and CBR. The figure indicates a likely level of compaction achieved during construction. Also, the moisture content increases at constant density (moving the right) the CBR decreases quite quickly. If the soil becomes saturated, i.e. the air voids become filled with water and decrease to zero, the soil becomes very weak indeed.

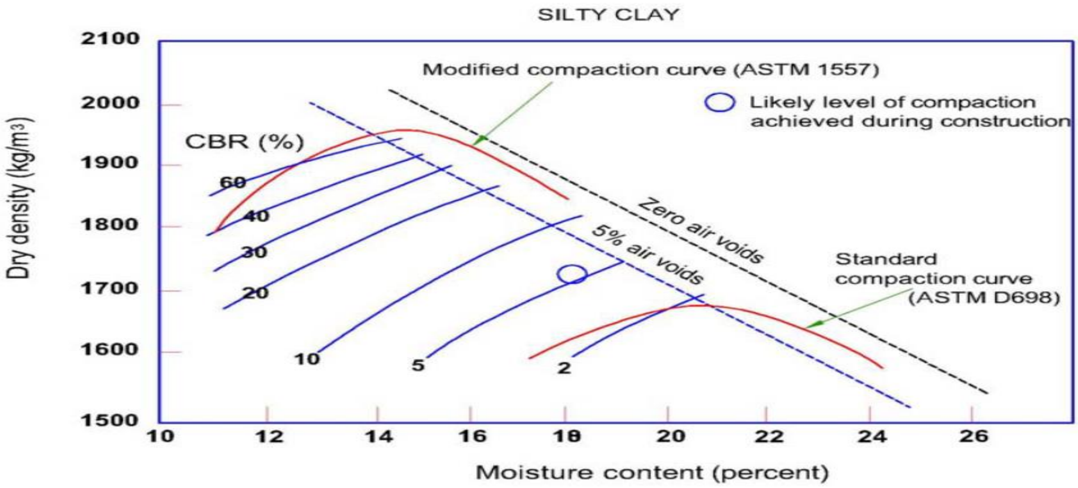

Figure 2-1: Dry densities, moisture content, and soil strength relationship for silt- clay

According to ERA design standard specification the subgrade strength class with the presence of a water table was discussed under the table blow.

Table 2-3: Estimated design subgrade strength under the presence of a water table

Depth of water table from formation level (m)	Subgrade strength class				
	Non plastic sand	Sandy clay PI = 10	Sandy clay PI = 20	silty clay PI = 30	Heavy clay PI = 40
0.5	S4	S4	S2	S2	S1
1	S5	S4	S3	S2	S1
2	S5	S5	S4	S3	S2
3	S6	S5	S4	S3	S2

Table 2.3 is not applicable for the silt, micaceous, organic or tropically weathered clays. Due to that, it must be treatment need and laboratory CBR tests should be undertaken for these soil.

2.2.4 Atterberg Limits

A fine-grained soil can exist in solid, semisolid, plastic and viscous of the fluid state depending on its water content. Swedish soil scientists Albert Atterberg originally defined seven “limits of consistency” to classify fine-grained soils, but in current engineering practice, only two of limits the liquid and plastic limits are commonly used. A third limit called the shrinkage limit is used occasionally. Wide varieties of soil engineering properties have been correlated to the liquid limit and plastic limits and these atterberg limits are also used to classify a fine-grained soil according to USCS and AASHTO system. However, the liquid limit and plastic limits are widely used for engineering classification of fine-grained soils or fine portion of coarse-grained soils (Fororuzan, 2016). The liquid limit and plasticity index of the soil are both used in determining the need for and type of subgrade stabilization. The liquid limit is used to classify the soil and the plasticity index is used as an indicator for the degree of stabilization that will be required and the most likely stabilization method that will be used. Soils with a plasticity index higher than 12% will typically require some form of modification or stabilization (David Jones et al, 2010).

A. Liquid Limits (LL)

LL of a soil is the boundary between plastic and liquid state. It is the minimum water content at which the soil mass flows like a liquid. LL is determined in the laboratory by the Casagrande apparatus test. In addition to being useful in identifying and classifying soils, the liquid limit can also be used to compute an approximate value of the compression index Cc for normally consolidated clays (Forouzan, 2016).

B. Plastic Limit (PL)

The plastic limit is a change in water content is accompanied by a change in volume of the soil mass. Soil mass can be deformed without cracking. The plastic limit is the boundary between plastic and semisolid state. According to the Casagrande apparatus, the soil begins to disintegrate when rolled into threads of a specified size (3mm) (forouzan, 2016).

C. Shrinkage Limit (SL)

It is the maximum water content at which there is no reduction in the volume of the soil mass accompanying reduction in water content. The concept of shrinkage limit can be used to evaluate the shrinkage potential or possibility of development, or both of cracks in earthworks involving cohesive soils (Forouzan, 2016).

D. Plastic Index (PI)

The range of water content between the liquid limit and plastic limit, which is an important measure of plastic behavior, is called the plasticity index. PI indicates the degree of plasticity of the soil. The greater the difference between the liquid limit and plastic limits, the greater is the plasticity of the soil. Cohesion less soil has zero plasticity indexes. Such soils are termed as non-plastic. Soils possessing large values of LL and PI are said to be highly plastic or fat. Those with low values are described as slightly plastic or lean. Organic clays possess liquid limits greater than 50. The plastic limits of such soils are equally higher. Hence soils with organic content have low plasticity indices corresponding to comparatively high liquid limits (Forouzan, 2016). According to the British standard (BSI: 1990), Atterberg's classification of soils based on the plasticity Index.

Table 2-4: Atterberg's classifications of soils based on plasticity index (BSI: 1990)

Plasticity index	Plasticity
0	Non-plastic
<7	Low
7-17	Medium
>17	High

2.2.5 Specific Gravity

The specific gravity (G) of the soil was expressed as the ratio of mass in air of a given volume of soil particles to the weight in air of an equal volume of distilled water at standard temperature. The specific gravity of the soil is used in calculating the phase relationships of soil water, and solids in a given volume of the soil. Also specific gravity of soils an important quantity that is frequently used in the calculation of percentage finer and diameter of the soil grains in hydrometer analysis (Murthy, 2001). The specific gravity is one of the parameters to identify types of soils. The specific gravity of the soil is ranged below the table.

Table 2-5: Specific gravity of the soil (Arora, 2004)

Type of soils	Specific gravity
Gravel	2.65-2.68
Sand	2.65-2.68
Silty sands	2.66-2.7
Inorganic clays	2.67-2.8
Organic soils variable	<2.0

2.2.6 Free Swell Tests

A free swell test of the soil is the increase in the volume of soil without any external constraints on submerged water (IS: 2720. 1977). Such soils have the possibility to damage the structure when the groundwater table reaches the influence zone. It is therefore always essential to investigate the swelling or expansive nature of these soils which are likely to posse undesirable expansion characteristics. Free swell ceases when the moisture reaches the plastic limit, swelling is caused mainly by repulsive forces that separate the clay particles causing volume increase.

A) Differential Free Swell (DFS)

The degree of the expansiveness of soils can be assessed more conveniently with its differential free swell. According to (IS: 2911 Part III - 1980) code practice the differential free swell is determined by noting the volume of soil in water and the volume of soil in kerosene after allowing the soil to get soaked in water and in kerosene separately for 24 hours.

The differential free swell is given by:

Differential Free Swell (DFS) = (Vd-Vk/Vk) x100………………………….. Eq (2)

Where: Vd = Volume of the soil specimen read from the graduated cylinder containing distilled water.

Vk = Volume of the specimen read from the graduated cylinder containing kerosene.

The differential swell has been adopted by the IS code of practice for directly assessing the degree of expansiveness or swelling potential of clayey soils as given in the table below.

Table 2-6: Degree of expansiveness on the bases of DFS (IS: 2911 Part III - 1980)

Differential Free Swell %	Soil Expansiveness
<20	Low
20-35	Moderate
35-50	High
>50	Very high

B) Free Swell Index (FSI)

The free swell index is increasing in the volume of the soil without any external constraint when subjected to submergence in water. IS-1498 states a criterion to predict the swell potential of soil. This approach based on the free swell ratio, defined as a ratio of sediment volume of soil in distilled water to that in kerosene or carbon tetrachloride. In some cases, for kaolinite-rich soil, this method results in the negative free swell index, subsequently, this technique may underestimate the swell potential of montmorillonite soil if the soils include a high amount of kaolinite clay material.

To work out this problem, the free swell index (FSI) was proposed by Sridharana (2008). This method is based on the ratio of the equilibrium soil volume to the dry weight soil. To ready, the sediment 10gm soil must be oven-dried and mixed thoroughly with the distilled water in 100 ml measuring jar then allow settling. This provides acceptable information about the soil expansiveness and constitution of soil type-expansive/combination of both.

The percent of free swell was calculated according to the following formula:

$$FS = (Vf\text{-}Vk/Vk)*100 \qquad \text{Eq. (3)}$$

Where: V_f: sediment volume of 10gm oven-dried soil passing sieve No. 40 placed in a 100ml graduated measuring jar containing distilled water;

Vk: sediment volume of 10 gm/cm3 of oven-dried soil passing sieve No.40 placed a 100ml graduate measuring jar containing kerosene.

2.2.7 Grain size Analysis

According to (AASHTO, 2006) was carried out soil consists mostly of different sized soil particles as a major constituent ingredient. The determination of the fraction of particles will help to identify the soil type as well as to estimate many other engineering properties such as strength and permeability and also to identify whether the soil is suitable for construction projects such as highways, dams or as black or for filter design. Two methods mostly used to determine grain size distribution are sieve analysis for a coarse-grained portion of soil the soil (size coarser than 0.075mm) and hydrometer analysis for a fine-grained portion whose size finer than 0.075mm.

Algebraic relationships have been established between grain size and significant soil properties. The suitability criteria for road airfield and embankment construction have been based on grain size distribution. The prediction of permeability can be done using grain size analysis. The proper gradation of filter material is established from particle size distribution. The grain size analysis usually used in engineering soil classifications (AASHTO, 2006).

2.3 Classification of Expansive Soils

The soils can be classified as AASHTO soil classification and Unified Soil Classification systems (USCS).

2.3.1 AASHTO Soil Classification System

According to the AASHTO soil classification system was developed in 1928 by the U.S Bureau of Public Roads, which is now called the American Association of State Highway and Transportation Officials (AASHTO). It is a textural-plasticity classification that uses sieved fractions and atterberg limits for assignment of soils to seven main groups and several subgroups.

The AASHTO system uses similar techniques but the driving lines have an equation of the form of PI = LL-30. It generally classifies a soil broadly into granular materials and silt-clay material. The soils classified under groups A-1, A-2 and A-3 are granular materials with 35% or less passing through a No.200 sieve but A-1 and A-3 non-plastic. Soils with more than 35% passing No.200 sieve are classified under groups A-4, A-5, A-6, and A-7. These soils are mostly silt and clay type materials. Group A-4 the typical material of this group is a non-plastic or moderately plastic silty soil usually having 75% or more passing a No.200 sieve. Group A-5 typical material of this group is similar to that described under group A-4, except that it may be highly elastic as indicated by the high liquid limit. Group A-6 the typical material of this group is a plastic clay soil usually having 75% or more passing a No.200 sieve Materials of this group usually have a high volume change between wet and dry states. Also A-7 group similar to the A-6 group except that it has the high liquid limits characteristic of group A-5 and may be elastic as well as subject to high-volume change. Subgroup A-7-5 includes those materials with moderate plasticity indexes in relation to the liquid limit and which may be highly elastic as well as subject to considerable volume change. Subgroup A-7-6 includes those materials with high plasticity indexes in relation to liquid limit and which are subject to extremely high volume change (AASHTO, 2006).

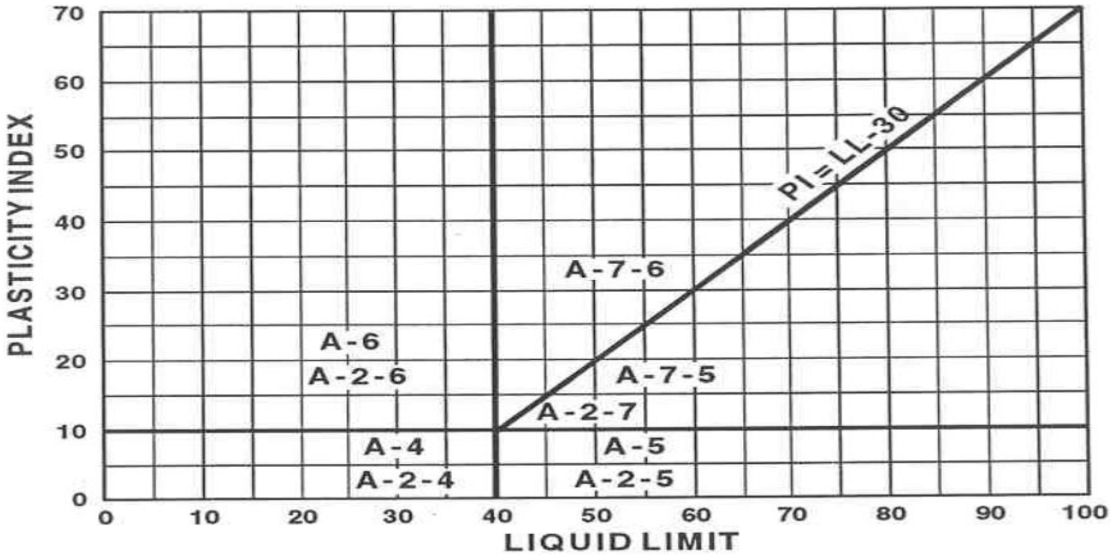

Figure 2-2: Liquid limit and plasticity index chart for AASHTO system

Fine-grained soils are further rated for their suitability for highways by the group index (GI) determined as follows:

GI = (F-35) [0.2+0.005(LL-40)] +0.01(F-15) (PI-10)……………………….…….....Eq (4)

Where: F = percentage by weight passing through sieve No. 200(sieve 0.075), expressed as the whole number; LL = liquid limit; and PI = Plasticity index.

While calculating GI from the above equation, if any term in the parentheses becomes negative, it is dropped; not given a negative value. The group index is rounded off to the nearest whole number. If the computed value is negative, the group is reported as zero. The group index is appended to the soil type determined from the classification table. For example, A-6(15) indicates the soil type A-6, having a group index of 15. The smaller the value of the group index, the better is the soil in the category. A GI of zero indicates a good subgrade, whereas a group index of 20 or greater shows a very poor subgrade. The GI must be mentioned even when it is zero to indicate that the soil has been classified as per the AASHTO system (AASHTO, 2006).

2.3.2 Unified Soil Classification System (USCS)

The Unified soil classification system was developed cooperatively by the U.S. Army Corps of Engineers (USA) and the U. S Bureau of Reclamation (USBR). The USC classification was published in 1953. It has since been adopted by the American Society for Testing and Materials (ASTM) as the standard classification of soils for engineering purposes. The success of the USC is indicated by its routine use worldwide and its acceptance for international geotechnical communication.

The USC system is a textural- plasticity classification scheme. Soils are divided into two major groups, coarse-grained and fine-grained soils, using the No.200 sieve as the size criterion. When more than half of the soil sample is larger than the No. 200sieve, it is classified as coarse-grained and is further subdivided by sieving and gradation. When more than half of the soil sample is smaller than the No.200 sieve, it is classified as fine-grained soil and is subdivide primarily based on the liquid limit values and degree of plasticity. The presence of organic material is an additional classification factor for fine-grained soils. Paired letter symbols are used for each soil group in the USC system. The first symbol refers to the predominant particle size (with the exception of organics). The second symbol for fine-grained soils refer to gradation for clean (little or no fines) soils and the presence of silt and clay-size particles for soils with appreciable amounts of fines. The second symbol for fine-grained soils subdivides on the basis of low (L) or high (H) plasticity (ASTM, 2002).

The USC system includes typical soil names with the classification system. Soils that are intermediate between two groups may be identified symbolically by a combined notation such as SM-ML and SC-CL.

The basis for the USCS system is the liquid limit and plasticity index of soil. The plasticity chart is a plot of PI inordinate and in abscissa that describes the properties of clay and silt soils in terms of atterberg limits. The figure 2.3 chart consists of two lines namely A-line and U- line as shown below. The A-line is assumed to be a boundary between clay and silt soils. This system is defined by an equation for A-line:

PI = 0.73 *(LL-20)………………………………………………………..Eq (5)

PI= 0.9*(LL-8)……………………………………………….……………Eq (6)

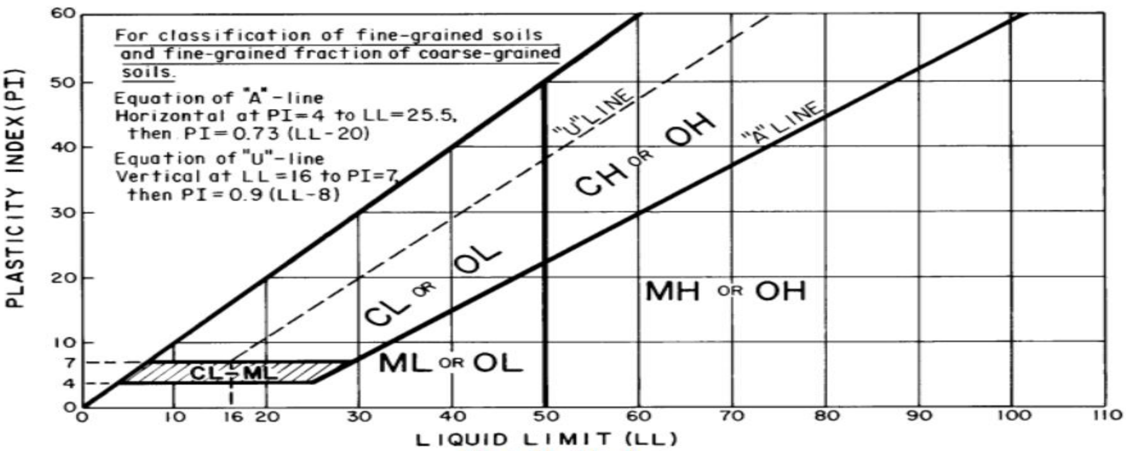

Figure 2-3; Liquid limit and plasticity index chart for USCS system (ASTM, 2002)

According to the Unified soil classification system (ASM D2487), the soils are classified as coarse-grained or fine-grained as follows: coarse-grained if more 50% of the soil sample is retained on the #200 (0.075mm) sieve. Coarse-grained soils are further classified as gravels if 50% or more of the coarse fraction is retained on the #4 (4.75mm) sieve, or sands if 50% or more of the coarse fraction passes the #4 (4.75mm) sieve. Fine-grained soil if 50% or more of the sample passes the #200 (0.075mm) sieve. Fine-grained soils are further classified according to whether their liquid limit is less than or greater than 50% (David Jones, et al, 2010).

2.4 Soil Stabilization

Expansive soils, due to their poor swell-shrink characteristics, pose a challenge to geotechnical engineers in handling them during the course of construction activities in or on them. In order to improve the engineering properties of such soils, they need to be stabilized (AI-Sharif & Attom, 2014). However, the term soil stabilization is generally restricted to the processes which alter the soil material itself for the improvement of its properties. Soil stabilization is used to reduce the permeability and compressibility of the soil mass in earth structures and to increase its shear strength. Also, it is required to increase the bearing capacity of foundation soils. However, soil stabilization means the improvement in the properties of poor soils by the use of controlled compaction; proportioning and the addition of suitable admixtures or stabilizers. Soil stabilization deals with mechanical, physic-chemical and chemical methods to make the stabilized soil serve its purpose. The stabilization process, essentially involves the excavation of the in-situ soil, treatment to the in-situ soil and compacting the treated soil (Mahiyar **, A, 2014).

2.4.1 Soil Stabilization Methods

The two usually used methods of stabilizing soils are stabilization by chemical or stabilization mechanical (Guyer, J., P, 2011).

2.4.1.1 Chemical Stabilization

Chemical stabilization is additive stabilization achieved by the addition of proper percentages of cement, lime; fly ash or combinations of these materials to the soil. The selection of type and determination of the percentages of additive to be is dependent upon the soil classification and the degree of improvement in soil quality desired. Generally, smaller amounts of additives are required when it is simply desired to modify soil properties such as gradation, workability, and plasticity (Guyer, J., P, 2011).

2.4.1.2 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 into compaction, mixing or blending of two or more gradations, applying geo-reinforcement and mechanical remediation (Guyer, J., P, 2011). However, the study was concerned with the strength stability of road subgrade material. In this study mechanical stabilization was applied with waste ceramic and limestone. The additives used in this study are limestone, waste ceramic dust and comprising of its effects with limestone –waste ceramic dust.

2.5 Waste Ceramic Dust (WCD)

Industrial waste as a result, recent years have witnessed rising social concern about the problem of waste management in general, and industrial waste and waste from the construction industry in particular. This problem is becoming increasingly critical due to the growing quantity of industrial, construction and demolition waste generated (Binici, 2007). However, this wastage or scrap material is an inorganic material and hazardous. Its disposal is a problem that can be removed with the idea of utilizing it as an admixture to stabilization (Jithin et.al, 2016).

Generally, the term “ceramics” (ceramic products) is used for inorganic materials (with possibly some organic content), made up of non-metallic compounds and made permanent by the firing process. The firing of ceramic bodies induces the time-temperature transformation of the constituent minerals, usually into a mixture of new minerals and glassy phases. Characteristic properties of ceramic products include high strength, long service life, chemical inertness and non-toxicity, resistance to heat and fire and sometimes also a specific porosity. Later ceramics were glazed and fired to create a colored, smooth surface. The potters used to make glazed tiles with clay; hence the tiles are called “ceramic tiles”. The raw materials to form tile consist of clay minerals mined from the earth’s crust, natural minerals such as feldspar that are used to lower the firing temperature, and chemical additives for the shaping process. A lot of ceramic tiles wastage is produced during formation, transportation and placing of ceramic tiles (Glass & Ceramic Division Micro, Small &Medium, 2011).

Table 2-7: Chemical composition of ceramic dust (Source: Tabor ceramic product factory)

R/m name	Fe_2O_3	MgO	CaO	TiO_2	Al_2O_3	SiO_2	K_2O	Na_2O
A/r/s/sand	1.76	0.34	0.68	0.18	9.37	84.34	2.98	1.96
A/Feldspar	3.0	0.72	0.01	0.5	14.84	71.66	4.02	3.10
H/kaolin	1.0	0.12	0.01	1.23	33.03	52.22	0.24	0.26
A/kaolin	1.75	0.01	0.97	0.24	40.92	60.21	-	-
Adam/kaolin	0.18	0.01	0.94	0.34	19.76	77.08	-	-
Ball clay	1.0	0.54	<0.01	1.98	32.94	47.5	2.68	0.58
Talc	8.05	14.5	14.16	0.04	1.7	61.53	-	-
B/kaolin	0.61	0.26	0.01	0.13	38.14	44.68	1.54	0.36
Pure quartz	0.80	0.34	0.56	0.01	13.01	74.40	2.46	5.30
Pure/feldspar	0.01	0.12	0.01	0.03	0.43	98.92	0.06	0.34
BS101	0.01	1.09	4.45	0.39	15.11	64.21	-	
Muger clay	10.22	1.54	0.72	0.01	25.21	59.19	-	
Basalt	8.97	0.12	1.88	0.07	20.75	76.41	-	
Bentonite	6.31	0.36	0.44	3.59	41.24	48.66	-	
Flanto F/spar	0.51	0.67	-	0.01	27.31	71.71	-	

2.5.1 Laboratory Studies on Expansive Soil Using Waste Ceramic Dust

Significance amounts of laboratory testing on expansive soil containing varying percentages of waste ceramic dust have been documented in the literature. Some researchers reported on the effects waste ceramic dust mixed with expansive soil to be amended for road subgrade material.

However, expansive soils have poor shearing strength and low bearing capacity. It is not easy to work with such soil, as it does not have enough strength to support the imposed load on them. For the satisfactory performance of the structure put in such soil, the properties of such soil need to be improved (Upadhyay1, 2016). An ideal solution lies in reducing cost, increasing longevity and reduce the accumulation of waste shall be through the utilization of industrial waste combined with weak soil for pavement construction. From the available literature, it is found that limited research has been done to study the effects of ceramic waste on different geotechnical properties of expansive soil. The present study has been undertaken to investigate the effects of waste ceramic dust on index properties, compaction properties, soaked California Bearing Ratio (CBR) and swelling pressure of expansive soil. Thus the use of ceramic waste not only improves the soil properties but the problem of their disposal can also be solved. In the present study, ceramic waste materials have been used to improve the properties of clayey soils and the effect of ceramic dust on various soil properties has been evaluated (Mandeep, 2017).

Torgal and Jalali (2010) examined the feasibility of using ceramic waste in stabilization, and test results show that expansive soil with 20% replacement has minor strength loss, but possess increased durability performance. While when soil mixes with waste ceramic dust show better results than the control mixtures concerning capillary water absorption, compressive strength and oxygen permeability thus leading to more durable in soil structure. However, from the results; liquid limit, plastic limit, plastic index, and OMC are decreased. The soaked CBR and MDD of the soil were increased.

According to a researcher (Krishna, 2016), the laboratory investigation was carried out to study the improvement in geotechnical properties of an expansive soil stabilized with waste ceramic dust in increments of 5% up to a maximum of 30%. The modified soil was tested for its liquid limit; plastic limit, optimum moisture content and swelling pressure go on decreasing as a percentage of waste ceramic dust increases from 0 to 30%. While as, the maximum dry density (MDD), and soaked CBR goes on increasing within the percentage of the addition of ceramic dust from 0 to 30%. From the economic analysis, it is found that ceramic dust can be utilized for strengthening the subgrade of flexible pavement with a substantial save in the cost of construction. The interaction behavior of waste, ceramic dust with soils can lead to a viable solution for its large scale utilization and disposal.

Also, another researcher (Sabat, 2012 and 2016), adopted waste ceramic dust for the stabilization of expansive soil. The expansive soil was treated with ceramic dust in increments of 5% up to a maximum of 30%. The modified expansive soil was tested for its atterberg limits, compaction characteristics, CBR, and swell pressure. The amended soil showed reduced plasticity characteristics, increased strength and bearing, increased dry density, optimum moisture, and swell pressure, such a trend indicates the behavior of waste ceramic dust was adsorption capacity in the soil. The swelling pressure of the soil is decreased as a percentage of waste ceramic dust increments from 5 to 30%, due to a decrease in clay content of expansive soil by replacement of ceramic dust, which is non-expansive in nature. As the attraction for water molecules decreases, the swelling nature of the soil decreases which results in the swelling pressure. The amendment of expansive soil with ceramic dust resulted in its classification changing from CH to CL. Economic analysis carried out by the author or concluded that up to 30% of ceramic dust can be used for strengthening of subgrade for flexible pavement.

The testing performed by Sabat waste ceramic dust can be effective for soil stabilizers. The soil modified with waste ceramic dust on consistency limits, compaction characteristics, California bearing ratio and swelling pressure of clayey soil was evaluated. From the results of tests, it was found that liquid limit decreases as 5% waste ceramic dust mixe with soil 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 18.1KN/m^3, OMC decreased from 20.4% to 17.6%. The soaked CBR values increased as the percentage of waste ceramic dust content increases.

According to the researcher (Geta Rani et al. 2014), were investigated the potential of ceramic waste in altering the strength of expansive soil. A test program consisting of Atterberg limits, California Bearing Ratio (CBR), and swell pressure tests were conducted to study the effect of amending expansive soil with ceramic dust. The ceramic waste was crushed to fine size by means of abrasion testing machine. The ceramic waste was amended in increments of 10% up to 30% in expansive soil. The effect of the addition resulted in a steady reduction in liquid limit, plastic limit, swell pressure, and optimum moisture content. The maximum dry density and California Bearing Ratio of the soil increased and reached a maximum at 20% amendment and reduced thereafter. Hence, according to researchers, 20% of the ceramic waste amendment was reported to be utilized for strengthening the expansive soil subgrade of flexible pavement with a substantial save in the cost of construction.

2.6 Limestone

The limestone is essentially calcareous, fossiliferous sandstone with poorly developed structure; color varies from brown to off-white. Joint spacing varies considerably in the area, where the more massive parts of the deposits form small hills and plateau. Generally, the extraction of commercial-sized blocks is possible in the thicker beds, exceeding one meter in thickness, where the spacing of vertical joints is wide. The natural lime is partly fossiliferous and contains abundant stylolites. The color varies between yellowish-brown and dark grey, the latter occurring in irregularly distributed reduction patterns. Hence, the natural lime forms hills and the area is considered to have a large potential for easily accessible deposits (Schlede, H., Walle, H. & Ayalew, S, 1990).

According to (Sabat, 2016) limestone mostly consists of $CaCO_3$ in its chemical composition. Approximately 20% of natural lime which is produced by the processing of limestone. Limestone is calcium-containing inorganic material in which carbonates, oxides, and hydroxides predominate. Strictly speaking, lime is calcium oxide or calcium hydroxide. The properties of good quality lime, which makes it suitable for use as an engineering material such as; easily workable, possesses good plasticity, offer good resistance to moisture, stiffens early, used for stabilizing the soils, used for plastering walls, ceiling, etc., and an excellent cement and adheres to masonry units perfectly (Md. Oliur Rahman, 2001).

2.6.1 Laboratory studies on Expansive Soil Using Limestone Stabilization

Laboratory examination was carried out to study the improvement of engineering properties of an expansive soil stabilized with limestone. According to (NLA, 2004), examined that most of the lime used for soil treatment is high calcium lime, which contains no more than 5 percent magnesium oxide or hydroxide. On some occasions, however, dolomitic lime is used. Dolomite lime contains 35 to 46 percent magnesium oxide or hydroxide. Dolomite lime can perform well in soil stabilization, although the magnesium fraction reacts more slowly than the calcium fraction. Soil stabilization significantly changes the characteristics of a soil to produce long term permanent strength and stability, particularly with respect to the action of water and frost. Lime, either alone or in combination with other materials, can be used to treat a range of soil types. The mineralogical properties of the soil will determine their degree of reactivity with lime and the ultimate strength that the stabilized layers will develop. In general, fine-grained clay soils (with a minimum of 25% passing the #200 sieve (75mm) and plasticity index greater than 10%) are considered to be good candidates for stabilization. Lime can permanently stabilize fine-grained soil employed as a subgrade or sub-base to create a layer with structural value in the pavement system. The treated soils may be in place (subgrade) or borrow materials. Subgrade stabilization usually involves in place road mixing and generally requires adding 3 to 6 percent lime by weight of the dry soil (NLA, 2004).

According to some researcher (Little, 1995), the chemical reactions in the lime involves immediate changes in soil texture and soil properties caused by cation exchange. The free calcium exchanges with the adsorbed cation of the clay mineral, resulting in a reduction in the size of the diffused water layer surrounding the clay particles. This reduction in the diffused water layer allows the clay particle to come into closer contact with one another, causing flocculation or agglomeration of the clay particles, which transforms the clay into a more silt-like material. Overall, flocculation and agglomeration of lime /limestone stabilization results in a soil that is more readily mixable, workable, and ultimately compatible.

However, some researchers reported on the effects limestone mixed with expansive soil to be modified for strengthening road subgrade soil. According to AI-Azzo (2009) had studied the stabilizing effect of crushed limestone on engineering properties of expansive clayey. Different percentages of crushed limestone dust added to were 2, 4, 6, 8, and 10%. The modified soil was tested for its atterberg limits, compaction, California bearing ratio, and swelling pressure. The reduction of swell was due to the replacement of soil by limestone, which had non-plasticity characteristics. The amended expansive soil showed reduced liquid limit, plastic index and maximum dry density at the optimum percentage of limestone. However, the California bearing ratio, optimum moisture content, and plastic limit were increased as well as its classification changing from CH to ML.

Ogila (2016) found a reduction in the swelling characteristics of high expansive soil when the limestone dust was mixed with the soil. As another researcher (Sabat, 2015), pointed out that when expansive soil blended with limestone. The expansive soil was blended with limestone in increments of 3% up to a maximum of 12%. The Liquid Limit, plastic index, and maximum dry density were decreased as the content of limestone increased up to 12%. The plastic index, maximum dry density (MDD) and free swelling was decreased and thereafter increased the percentage of limestone powder are powder is probably due to the considerable increase of matric suction caused by the reduction of the initial water content of the sample since the dust is to the wet soil. The optimum moisture content (OMC) and the soaked California bearing ratio of soil go on increasing with the increase in percentage addition of limestone.

Based on another researcher (Brooks, et.al, 2011), studied that the potential of limestone on stabilization of weak subgrade soil. The tests such as atterberg,s limits, compaction, California bearing ratio (CBR), and swell index test were used. A one-way analysis of variance tests performed on the generated data. Results showed that the plasticity and swell of the soils were reduced. A significant increase was observed in the strength of the soils for CBR when stabilized with this additive. The maximum dry density of the soil-with limestone was decreased and optimum moisture content of the mixture increased with an increase in additive content. Consequently, the ERA manual recommends that limestone stabilization (stabilizing by mixing lime into the expansive soil) is one of the countermeasures to prevent weak subgrade soil (Japan International Cooperation Agency, 2013).

2.7 Specification

The specification is one that controls the result in the laboratory. The specification was reviewed according to the ERA design standard and IS standard specification.

Table 2-8: Specification reviewed

Subgrade Material	ERA (2013 & 2002)	IS: 2911 Part III – 1980
Liquid limit	< 30	
Plastic index	<60	
CBR, %	>3	
CBR swell, %	< 2	
Free swell, %		< 20

CHAPTER THREE

MATERIAL AND METHODOLOGY

3.1 Study location and Topography

The geographic location of Ambo town is approximately between 8º 56'30" N- 8º 59'30" N latitude and between 37º 47'30" E- 37º 55'15" E longitude. Relatively Ambo town is located 114 km far away from the east of Addis Ababa, 60km from east woliso town and 12km west from Guder town. Ambo town is a zonal town with the second stage of administrative status. As information obtained from the municipality shows, the town previously had six kebeles such as Hora Ayetu, Ya'I Gada, Torban Kuttaye, Sanqalle Faris, Kisose Oddo Liban, Awaro Korra.

Figure 3-1: Location of sample site (Source: Google Earth Pro, @ 2019, image @ 2020)

3.1.1 Climate

The data obtained from the National Meteorological Agency of Ambo branch which is located in Ambo University 10 Years consecutive meteorological data was taken and the following result is observed on the temperature, rainfall, and humidity (ATAO, 2012).

Temperature: - the mean temperature, the mean annual maximum and mean annual minimum temperatures of the town are record to be 18.87°C, 19.63°C and 18.24°C respectively, which is the characteristic of a warm temperature climate.

Rainfall: - the mean annual rainfall is about 82.32mm. The highest rainfall concentration occurs from June to September. Thus low infiltration of rain water, storm water occurrence, and in the undulation of low gradient areas and incidence of sheet and gully erosion are some of the problems in the town and surrounding areas.

Humidity: - The mean monthly relative humidity of the town varies from 64.6% - in August to 35.8% in December, which is very comfortable for human life.

Table 3-1: Mean total rainfall and mean annual temperature for Ambo town over 30 years from 1981-2010 (Source: NMA of Ethiopia, 2013)

Variables	Minimum	Maximum	Mean	Std. Deviation
Total Rainfall in mm	474.20	1323.60	968.74	207
Mean Annual Temp ^{0}C	16.40	21.30	18.64	0.90

3.1.2 Geology and Soil

The town is located on the Shewa plateau. Most of the existing built-up areas of the town are an almost gentle slope and undulated while some hill slope and mountain are also seen in the town. Along the course of the rivers and streams, steep slope and gullies are also observed. Concerning the altitude of the town, the town's altitude ranges from 1872 meters above sea level to 2362m. As regards the proposed route expansion, most of the areas are characterized by flat, gentle slopes, higher slopes and undulation towards; Awaro kora, in the eastern direction, Kisose Oddo Liban in the Northern part and Sanqalle Faris in western direction.

The soil characteristics of Ambo town and its surrounding include dark grey, reddish-brown, grey, sandy silts, and silty clays. The dominant type of soil in Ambo town and its surrounding area is vertisol soil. pellic vertisol soils that are dark, usually occupying vast areas that are waterlogged during the rainy season and shrink and have deep cracks in the dry season. The vertisol soils cover the gently slopes in the southern, eastern and northern parts of the town. Dystric Nitosol soils are also observed in some parts of Ambo especially in the western part around Sanqalle Faris areas. These are deep brown clay soils have a uniform profile and porous. There are effects of the road distraction, settlements due to such soil in the town. Silty clay and sand silty mainly cover the central parts of the town. In general, the soil type of the study area is dominated and characterized as black to red soils. A wide range of variations in soil type can be observed in the entire road corridor (ATAO, 2012).

3.2 Study Design

The experimental study designs were used in this study. That means the experimental research method is always based on experimental work with description and analysis. For the accomplishment of this research objective, the secondary data of the related study were reviewed, and the primary data were a collection of the sample materials with different laboratory tests was conducted. The laboratory procedure was conducted according to AASHTO, ASTM, and IS standard testing procedures were performed for the accomplishment of this research objective. The test designed to accomplish the research objective was to evaluate the engineering properties of the soil and soil with different proportions of additives (waste ceramic dust, limestone). In particularly laboratory, sample was subjected to different tests such as natural moisture contents, CBR value, atterberg limits, specific gravity, grain size, and hydrometer analysis, free swell tests, and proctor compaction tests for subgrade soil with different proportion of limestone, waste ceramic dust and limestone-waste ceramic dust those laboratory tests were carried out. Finally, the results were compared with design standards. Figure 3.2 shows the overall research design for the achievement of the research objective.

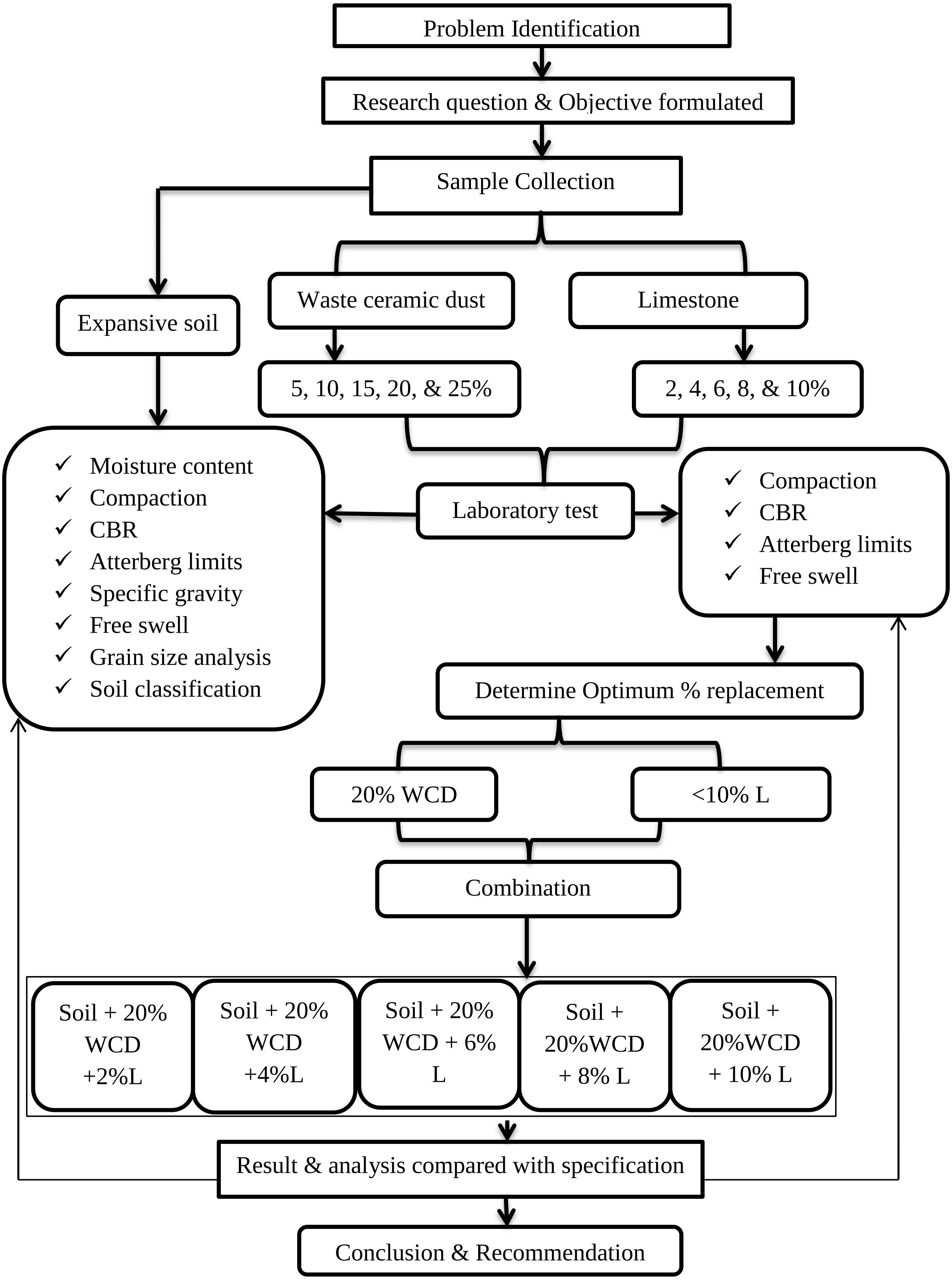

Figure 3-2: Study design of the research

3.3 Study Population

The study population for this study was including the materials used in this research works; those are expansive soil, waste ceramic dust, and limestone.

3.4 Data Collection

The data collection was carried out from two different data sources, primary and secondary data sources. The primary data was collected materials and experimental output was conducted and secondary data were different works of literature, scientific researches, and different specifications were reviewed to analyze the research.

3.5 Sample Size and Sample Technique

The sampling procedure for this study was purposive sampling techniques which are a non-probable method and accommodating sampling techniques is used for sample preparation. This sampling technique was proposed based on the target to perform the laboratory tests to investigate the performance of the waste ceramic dust and limestone for weak subgrade soil. The test was performed according to AASHTO, ASTM and IS standards.

3.6 Materials and Sample Collection

The materials used for this research was; expansive soil, waste ceramic dust, and limestone. Also, the soil samples were randomly collected from Ambo town around Sanqalle Faris along Guder road. The collected sample was disturbed and taken from1.5m depth. The sample soil from this place was structural damage and failure was taken as special attention. The excavation was made by using manual instruments like Shovel, Ironton digging bar for soil, and tape to measure the depth of the pit. Limestone was collected from Ambo around Sanqalle naturally occurring lime and waste ceramic dust also collected at a different construction site around Jimma town and taken to Jimma Institute of Technology laboratory test.

Expansive soil (July: 15/11/2011 4:15 by Daraje)

Limestone (July: 25/11/2011 9: 15 by Daraje) Waste Ceramic (July: 5/11/2011 3: 30 by Leta)

Figure 3-3: Materials used (Expansive soil, limestone, and Waste ceramic dust)

3.7 Sample Preparation and Mixing Ratio

The sample was taken from the study area and the moisture of the soil was placed into the inside plastic bags. The natural moisture of the soil was kept and come into the laboratory test inserted in oven-dried. The sample soil was air-dried and divided into a section for each laboratory test. However, the total sample soil used for this thesis was 145kg. Limestone and waste ceramic dust were crushed into powder form by using manually; those passing 425 μ sieves according to AASHTO M-145 were used in the experimental programmed. Different various proportions of limestone and waste ceramic dust with expansive soil were taken at 2 to 10% and 5 to 25% by weight, respectively. The combination of both waste ceramic dust and limestone with expansive soil was taken at optimum replacement 20% waste ceramic dust + 2 to 10% of limestone by weight to get the maximum effect.

Table 3-2: Sample preparation and mixing ratio

	% L. Stone	% WCD	% Soil	% L. stone + W.C.D	% Soil
	-	-	100		
	2	-	98	20 % WCD + 2% L	78%
	4	-	96		
	6	-	94	20 % WCD + 4% L	76%
Sample soil	8	-	92		
	10	-	90	20 % WCD + 6% L	74%
	-	5	95		
	-	10	90	20 % WCD + 8% L	72%
	-	15	85		
	-	20	80	20 % WCD + 10% L	70%
	-	25	75		

3.8 Study Variables

a) Dependent Variable

The dependent variable of the research is the engineering properties of stabilized weak subgrade soil.

b) Independent Variables

The physical and engineering properties of treated and untreated soil and dosage of waste ceramic dust dust - limestone.

3.9 Data Processing and Analysis

For the accomplishment of this research objective, the data was processed according to the following tasks. Those are: data handling and recording format were prepared for laboratory tests, all data were properly observed and recorded using the standard format, by arranging and wrote the results, and then the relationship was noted. The obtained results were presented using MS-Excels, a table, and different types of graphs presentation were carried out.

3.10 Laboratory Test performed and Test methodology

The laboratory tests in this study included the California bearing ratio as a strength property test, atterberg's limits, standard proctor compaction test, and grain size analysis as engineering properties tests. For comparison purposes, the primary experimental plan for the strength property test involved preparing and testing four broad categories of treatment types: untreated soil, soil with waste ceramic dust, limestone, and waste ceramic dust- limestone at variable percentages to examine their influence.

3.10.1 Natural Moisture Content (AASHTO T-265)

The natural moisture content of the laboratory test is performed to determine the water (moisture) content of soils. The oven-drying method was used to determine the moisture contents of the soil samples. The water content is the ratio, expressed as a percentage, of the mass of "pore" or free water in a given mass of soil to the mass of the dry soil solids. According to AASHTO T-265 Standard Test Method for laboratory determination, the moisture content of the soil was determined. Two sets of samples were dried to a constant temperature using oven-dry at a temperature of 110 $\pm$ 5 ºC and the average is taken. The moisture content of the soil was determined according to the following formula:

W = (Ww/Ws)*100…………………………………………………………….. (Eq.1)

Where: w = Moisture content (%)

Ws = Dry weight of solids (gm)

Ww = weight of water (gm)

3.10.2 Proctor Compaction Test (AASHTO T-99)

This laboratory test is performed to determine the relationship between the moisture content and the dry density of the soil in a specified compaction effort. There are two types of compaction tests: standard proctor and modified proctor compaction test. In this study, I performed a standard proctor compaction test. In general, most engineering properties, such as the strength, stiffness, resistance to shrinkage, and imperviousness of the soil, would be improved by increasing the soil density. The test was conducted according to AASHTO T-99 (Standard Proctor Test) procedures are employed to conduct the compaction test. This method employed for the particles of the soil retained on the sieve No.4 (4.75mm). Densities are calculated from unit weights measured from the laboratory divided by gravity due to the earth. Hence, this test was done on the soil, and then various percentages of waste ceramic dust, limestone and ceramic dust –limestone on the natural soil and MDD and OMC were determined.

Figure 3-4: Determinations of MDD and OMC (August 1, 2011, 3:15 AM by Diriba)

3.10.3 California Bearing Ratio (CBR) (AASHTO T-193-93)

The California Bearing Ratio (CBR) test is a penetration test that is used to evaluate the subgrade strength of roads and pavements. The results of these tests are used with the empirical curves to determine the thickness of the pavement and its component layers. However, CBR is expressed by force exerted by a plunger and the depth of penetration into specimen; it is aimed at determining the relationship between force and penetration. The method uses material passing 19 mm sieve size and provides the CBR value of the material at optimum moisture content. The samples are compacted in three layers with 56 blows from the 2.5kg rammer. The CBR test indirectly measures the shearing resistance of soil under controlled moisture and density conditions. To determine the strength and swelling potential of the subgrade soil sample test has been carried out by soaking 4- days and loaded swell testing procedure. Final CBR is determined by compaction level with 65 blows at 95% maximum dry density (MDD). The CBR swell of the soil was measured by placing the tripod with the dial indicator on the top of the soaked CBR mold in the bath. The initial dial reading of the dial indicator on the soaked CBR mold is taken just after the soaking the sample. At the end of 96 hours of the final dial reading of the dial indicator is taken hence the swell percentage of the initial sample is given by:

$$CBR\ swell = \frac{Change\ in\ length\ in\ \ mm\ \ during\ soaking\ \ x\ 100\%}{116.43\ mm} \qquad \text{(Eq.2)}$$

The CBR was calculated as the ratio of force per unit area required to penetrate a soil at the rate of 1.25mm/min, to that required for the corresponding penetration of a standard material.

$$CBR = \frac{\text{Test load on the sample x 100\%}}{\text{Standard load}} \qquad \text{(Eq.3)}$$

The standard loads adopted for different penetrations for standard material with a CBR value of 100%, at 2.54 (mm) standard loads is 1370 (Kg), and penetration at 5.08 (mm) is 2055 (Kg).

Figure 3-5: CBR determinations (August 9, 2011, 3:15 by Diriba)

3.10.4 Atterberg's Limits (AASHTO T-89 and 90 or ASTM 4318)

Atterberg's limits were determined for air-dried samples. It was done on standard reference: AASHTO T-89 and 90 or ASTM 4318 Standard Test Method for liquid limit, plastic limit, and plasticity index of the soils. The test was computed by using the Casagrande apparatus. The representative sample was air-dried and the sample soil of 200g passing through a No.40 (0.425mm) sieve was used for the preparation of the sample for this test. It also the same procedure was carried out for the treated soil with an increment of limestone and waste ceramic dust. The plasticity index was computed for each soil based on the liquid limit and plastic limit obtained. The plastic limit was calculated according to the following formula:

Plastic limit (PL) = (Mass of Water/ Mass of Oven – Dried Soil)*100……….….. (Eq.4)

Plastic Index (PI) = Liquid Limit (LL) - Plastic Limit (PL)……..……………….... (Eq.5)

Figure 3-6: Atterberg's determination for the soil (August 10, 2011, 3:15 AM by Aseffa)

3.10.5 Specific Gravity (AASHTO T 100-93)

The test was to determine the specific gravity of the soil by using a pycnometer. Based on the soil type-specific gravity may be unusually high or low. Specific gravity is the ratio of the mass of a unit volume of soil at a standard temperature of the mass of the same volume of gas-free distilled water at a stated temperature. The test was conducted according to AASHTO T 100-93 for specific gravity of soil solids by water pycnometer procedure. Since the specific gravity results which are to be used for determination of particle size of the hydrometric analysis portion of AASHTO T-88, which was intended that the specific gravity test is done on that portion of the soil which passes 2 mm (No. 10) Sieve size. The room temperature was about 19-24 ^{o}C. The specific gravity of the soil refers to the mass of solid matter of a given soil sample as compared to an equal volume of water.

Figure 3-7: Specific gravity determinations (August 16, 2011, 8:20 PM by Aseffa)

3.10.6 Free Swell Test

To study the swelling properties of the soils, the simplest test conducted is a free swell test. The free swell index is the increase in the volume of soil, without any external constraints, on submergence in water. According to standard IS 2720(Part 40)1977 the test was conducted by slowly pouring 10 grams of oven-dry soil, which passed through No.40 (0.425mm) sieve into a 100 ml graduated cylinder filled with distilled water and kerosene. Sufficient time, not less than 24 hours shall be allowed for soil samples to attain an equilibrium state of volume without any further change in the volume of the soils and final volume of suspension being recorded. Free swell test results for oven-dried samples at a temperature of 105 ^{o}C. The free swell test was calculated as:

$$FS\ (\%) = \frac{\text{Vd}-\text{Vk x 100\%}}{\text{Vk}} \quad \text{(Eq.6)}$$

Vd = Volume of the soil specimen containing distilled water

Vk = Volume of the specimen containing kerosene

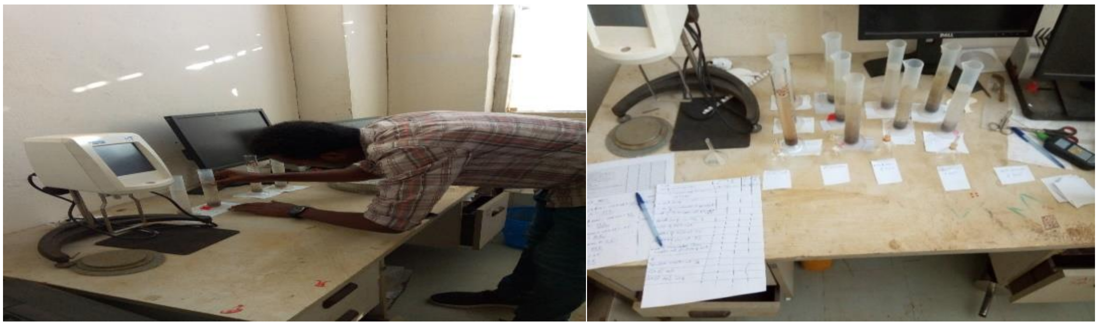

Figure 3-8: Free swell test determinations (August 22, 2011, 9:30 PM by Ahadu)

3.10.7 Grain size Analysis (ASTM D422- 63)

In this study wet sample preparation in accordance with ASTM D422-63 Standard Test Method for particle size analysis was applied. The mechanical analysis was used for the coarse sized soils by using a set of the sieve and whereas hydrometer analysis is used for fine-grained soils. Here sodium hexametaphosphate was used as a dispersing agent. For soils comprising coarser and finer sizes, both mechanical and hydrometer testing methods were performed. An air-dried sample was used during the laboratory tests.

Figure 3-9: Sieve and hydrometer analysis (August 20, 2011, 8:15 PM by Aseffa)

3.11 Soil Classification (AASHTO M 145-91)

The soil was classified according to the AASHTO soil classification system using particle size distribution and atterberg limits as well as according to the USCS classification system. Soil classification provides a method of identifying soils in a particular group that would likely exhibit similar characteristics. According to the ASHTO classification system classifies the soil into seven major groups: A-1 through A-7 the major groups divided into subgroups. USCS system also classifies the soil based on grain size, gradation, plasticity characteristics and classifying by group symbols and group names.

Table 3-3: AASHTO classification of soils & soil-aggregate mixtures (AASHTO M 145-91)

CLASSIFICATION OF SOILS AND SOIL-AGGREGATES MIXTURES											
General Classification	Granular Materials(35% or less passing 75 µm) [No.200]							Silty-Clay materials (more than 35% passing 75 µm) [No.200]			
Group Classification	A-1		A-3*	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis:											
% passing:											
2mm (No.10)	50 max	_	_	_	_	_	_	_	_	_	_
425 µm (No.40)	30 max.	50 Ma.	51 min.	_	_	_	_	_	_	_	_
75 µm (No.200)	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 0.425mm(No.40)											
Liquid Limit	_		_	40 max	41 min.	40 max	41 min	40 max	41 min	40 max	41 Min
Plasticity Index	6max		N.P	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 Min
Usual types of significant constituent Materials	Stone Fragments Gravel & Sand		Fine Sand	Silty or Clayey Gravel & Sand				Silty Soils		Clayey Soils	
General Rating as Subgrade	Excellent to Good							Fair to Poor			

3.12 Mixing soil and Stabilizer

If necessary to achieve the desired moisture content for the batch, additional water was first blended into the soil and mixed for three to five minutes. After water addition, the appropriate amounts of stabilizer were then added to the mixture and blended thoroughly for three to five minutes. The mixture was set at the lowest speed, and the water and stabilizer were each added slowly to promote uniform blending and to prevent clumping of the soil and /or stabilizer. It was sometimes necessary to stop the mixture and scrape unmixed portions from the sides and bottom of the bowl into the mixture and resume mixing (Christopher M, 2005).

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Introduction

The results are analyzed and discussed to give insight into the research in terms of engineering properties of expansive soil in relation to use. The analysis involved the evaluation of both natural and stabilized soil samples separately by performing the following tests: natural moisture content, atterberg limits, moisture density relationship (compaction), grain size analysis, CBR, free swell, and specific gravity test was discussed under the subsequent section.

4.1.1 Chemical Composition of Expansive Soil and Lime Stone

The chemical composition of soil and sanqalle limestone was studied in the Ethiopian Geological Survey Laboratory Test (GSE). The mineral content of both samples was discussed in Table 4.1. The photo of the results was attached under Appendix A-1.

Table 4-1: Chemical composition of expansive soil and limestone

Collector's code	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O	LOI
limestone	12.58	2.56	2.20	47.16	1.30	<0.01	0.32	0.16	0.07	0.09	6.79	26.80
soil	64.30	15.76	6.50	1.12	1.62	0.82	1.10	<0.01	0.08	0.36	<0.01	6.95

4.2 Engineering Properties of Expansive Soil

4.2.1 Natural Moisture Content of the Soil

The natural moisture content of the soil was observed in the oven- drying method used to determine the moisture contents of the soil samples. The water content of sample soil of the study area was 42.51% which indicates that the soil is fine-grain soil because the fine grain soil contains high moisture than coarse grain soils. So, according to Terzaghi, (1963), the typical moisture content of the soil was clayed which ranges between 10 to 50% or more. Also, another researcher (Forouzan, 2016) suggested that the consistency of the soil is very soft. So, the consistency of the soil of the study areas was high water content and degrees of firmness are very soft. Hence, the soil must need treatment. The laboratory test results discussed in Appendix A-2

4.2.2 Proctor Compaction Test

The compaction test has been conducted for soil samples under consideration to determine the maximum dry density (MDD) and optimum moisture content (OMC) of the soils. Figure 4.1 compaction curves which show the peak of moisture – density relationship of the soil samples. From the compaction curve graph, the MDD was 1.25g/cm^3 and OMC becomes 33.39%. The sample soil selected from the area was high moisture content. The laboratory test results discussed in Appendix A-3.

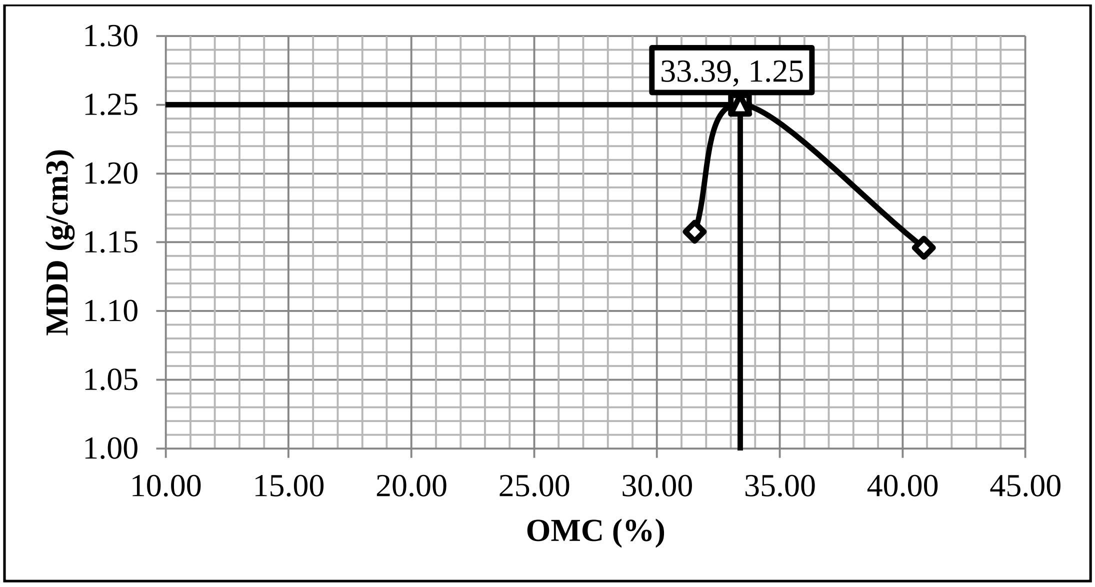

Figure 4-1: MDD and OMC curve of the soil

4.2.3 California Bearing Ratio (CBR)

The strength of subgrade soil samples has been determined using the CBR test and how it was performed when subjected to loading. It was determined by the relationship between force and penetration. The CBR penetration test was to evaluate the subgrade strength of roads and pavements. The MDD and OMC of the soil sample were used to prepare a CBR test for soaking 4 days and the CBR test at 95% maximum dry density (MDD) was determined. Table 4.2 indicates the CBR value @ 2.54 mm and @5.08 mm was 2.08% & 1.90%. The CBR at 2.54 mm penetration is generally used for assessing the quality of the materials. The soil had low bearing capacity when soaked and high plasticity index hence fell below the standard recommendations for most geotechnical construction works especially highway construction. Therefore, the soil has a need for initial modification and stabilization to improve its workability and engineering property. The summary of the CBR test result is presented in Appendix A-4.

Table 4-2: CBR test result for the soil

Penetration	CBR @(2.54)	CBR @(5.08)
Load (KN)	2.08	1.90
Standard load (KN)	13.344	20.016
Swell %	5.02	
ERA (2013)	Subgrade	>3

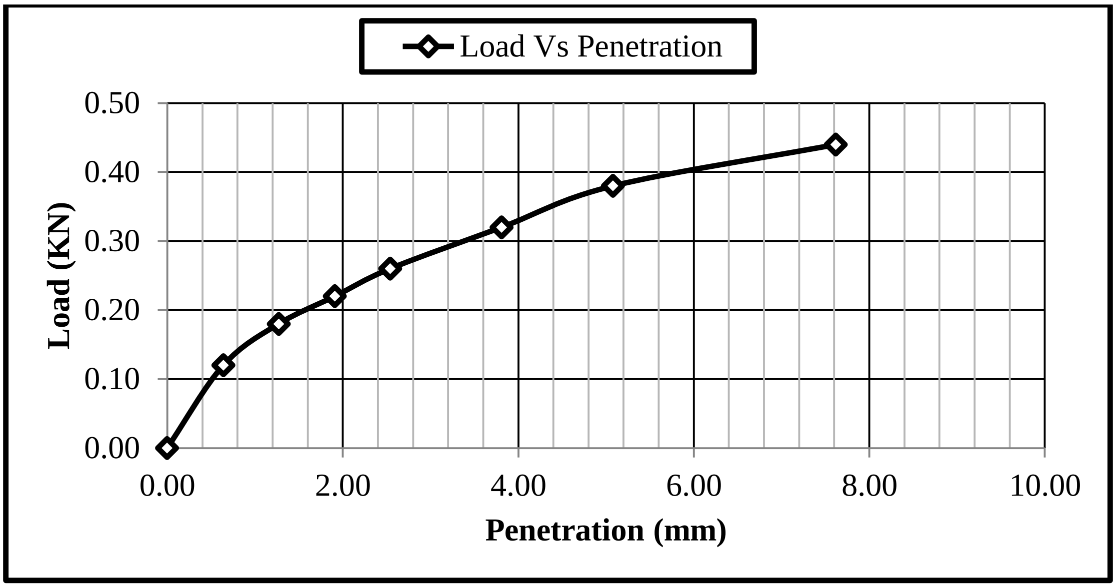

Figure 4-2: CBR test of the soil

Figure 4.2 shows load versus penetration for the soil sample. The test result showed that the subgrade soil has a low CBR value of 2.08%. This does not satisfy the minimum requirements as subgrade material. According to ERA standard specification, a CBR value of less than 3% special treatment is required. Since the CBR value was low and they need special treatment like stabilization by using waste ceramic dust and limestone to use the in-situ soil without excavation and reduce the cost of construction.

4.2.4 Atterberg's Limits

Atterberg's limits of the soils indicate the critical water contents of the fine-grained soil were high as observed from the test. Table 4.3 shows that the liquid limit and plastic limit of the soil samples were 81.1, and 28.9%, as well as a plastic index, was 52.1%. From this test result, the water content of the soil was high. Also according to ERA specification the liquid limit and plastic index of the sample soil were above 60% and 30%, respectively due to that the representative sample not fulfilled standard specification. Hence, the soil samples must take into consideration improvements to use as subgrade material. The summary of Atterberg's limits of the soil was discussed under Appendix A-5.

Table 4-3: Atterberg's limits test result for the soil

Atterberg's limits			ERA (2013) Requirement
LL (%)	PL (%)	PI (%)	PI < 30 max. LL< 60 max.
81	28.9	52.1	

4.2.5 Specific Gravity

It is the ratio of unit volume at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. Based on the test results the specific gravity of the soil samples was 2.68 which shows the soil are inorganic clays. Therefore, the soil was high specific gravity or high water content. According to Arora (2004), the soil is classified as inorganic clay soil because the specific gravity of the soil is greater than 2.67 and less than 2.8. Hence, in order to overcome the problem of such soil, the improvement must take place. The summary of the test results was presented in Appendix A-6.

4.2.6 Free Swell Test

The free swell test of the soil is the increase in the volume of soil without any external constraints on submerged water. This approach based on the free swell ratio, defined as the ratio of sediment volume of soil in distilled water to that in kerosene. Table 4.4 indicates the representative soil sample of the area was 70%. This result indicated that the degree of the expansiveness of the soil sample was very highly expansive. According to (IS: 2911 Part III – 1980), the soils are clay types that are swelling. The degree of the expansiveness of soil for the study area was very high. Due to that soil, the study area was need enhancement.

Table 4-4: Free swell tests result for soil sample

Sample soil	Vk	Vf	FS (%)
	41.0	70	70

4.2.7 Grain Size Analysis

The grain size analysis test which is used to determine the particle size distribution of the soil with applicable measurement constraints and which helps to determine the soil classification with together the Atterberg limits of the soil. Table 4.5 and Figure 4.3 shows the combined of grain size analysis and hydrometer analysis test results for soils. The textural classification of soil based on the particle size distribution of the percent of sand, silt, and clay size fractions in a given soil had no percent of gravel, but which have been percent of sand and % finer No.200 sieve. As shown in Figure 4.3 the sample soil on the particle size distribution curve almost 95.0% of the soil passing through No.200 sieve based on unified soil classification system; percentage of gravel (75mm to 4.75mm) = 0, percentage of sand (particle size 4.75mm to 0.075mm) = 5%, and percentage of fine particles = 100% - 5% = 95.0%. Also according to AASHTO M-145, the sample soil was silty-clay materials (more than 35% passing 75 μm No.200. And by using the ASTM D2487 soil classification system more than 50% of the soil passes the 0.075 mm (No.200) sieve the soil sample was under silty-clay materials. However, from a hydrometer analysis test results the soils have more clay. The soil under this class is generally classified as a material of poor engineering property to be used as subgrade material. The laboratory test examination was attached in Appendix A-6.

Table 4-5: Grain size analysis for soil samples

	Grain size %				% Finer than 0.075 mm
	Gravel	Sand	Silt	Clay	
Sample soil	0	5	28	67	95.0

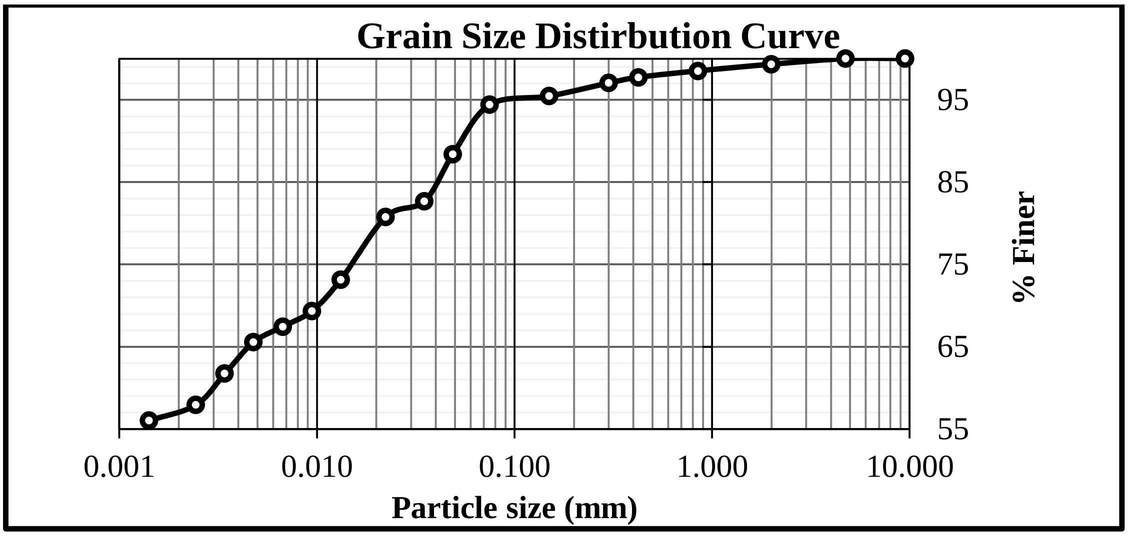

Figure 4-3: Combined of grain size curve from sieve analysis and hydrometer analysis

4.2.8 Soil Classification

4.2.8.1 AASHTO Soil classification

The soils were classified based on their index properties such as particle-size distribution and consistency limits (Atterberg limits). Table 4.6 and Figure 4.4 indicate the classification index of the soil were lies A-7-6 with group index 58%. This indicates that the soils are swelling soils and poor subgrade construction. Since the group index of the soils twenty or greater than twenty indicates the soils are very poor for road subgrade constructions as stated as by AASHTO M-145 standard the soil of the study area is fair too poor to be used as subgrade material.

Table 4-6: Classification of soils based on the AASHTO classification system

Sample soil	% Passing Sieve #200	LL	PI	Classification index & Group Index	Remark	Rating as Subgrade material
	95.0	81.1	52.1	A-7-6 (58)	Clay soil	Poor

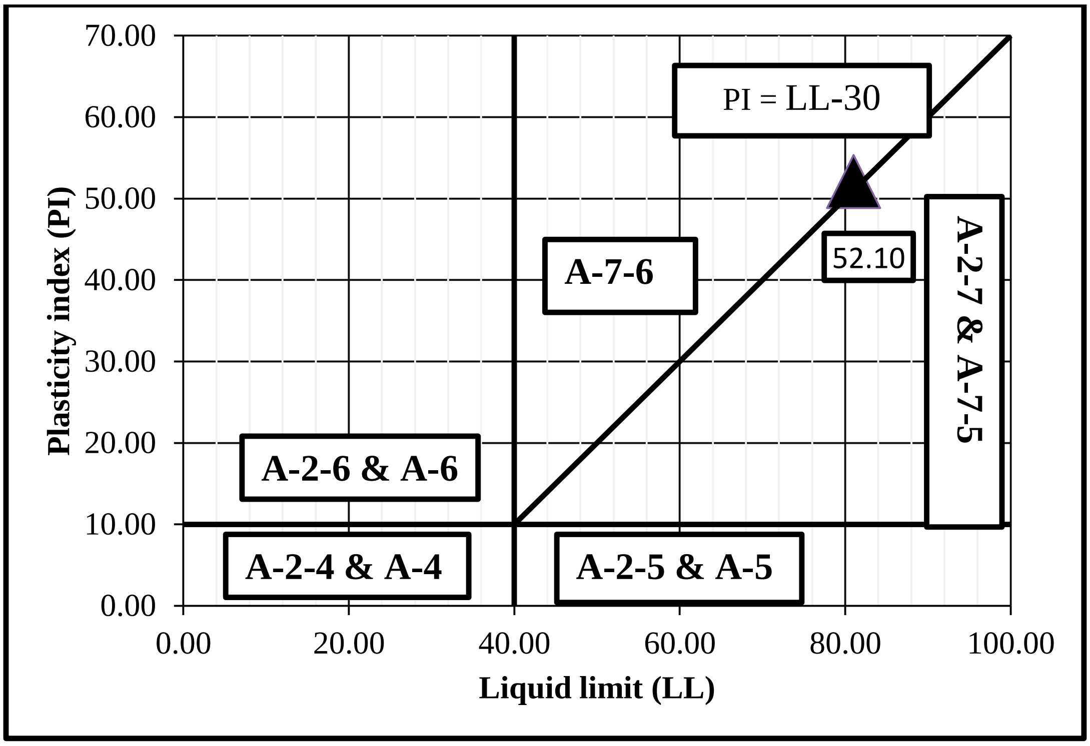

Figure 4-4: Plasticity chart of the soils according to AASHTO classification system

4.2.8.2 USCS Soil Classification

USCS soil classification the system depends on the Grain size, plasticity charts and classifying by groups symbol and group names. This system classifies the soil into two categories coarse-grained soils that are gravelly and sand with less than 35% passing through No. #200 sieve and fine grain soils that are silt, clay, and organic silt clays are with 35% or more passing through No. #200 sieve. Table 4.7 and Figure 4.5 show the soil sample was lies in (CH). Most of the soils along the road are highly inorganic clay. According to ASTM (2002), indicated that the USCS soil classification system of the soil of the study area fell in CH. These means the soil of the areas was inorganic clay with high plasticity and high compressibility which indicates the soils are poor.

Table 4-7: Classification of the soils based on USCS classification system

Sample soil	% passing sieve #200		LL	PI	Classification according to USCS
	95.0		81	52.1	CH

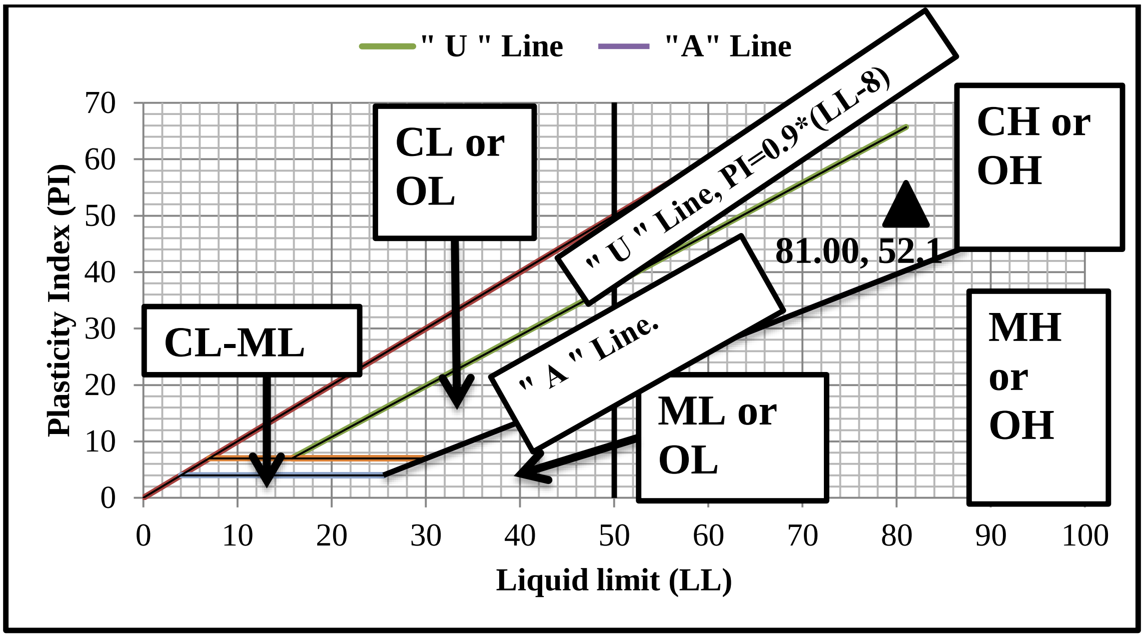

Figure 4-5: Plasticity chart according to the USCS system

Figure 4.5 indicates the relation between soil plasticity index and liquid limit lies on the graph. The test result of the soil sample based on the liquid limit and plasticity index the soil was highly expansive. Thus, such subgrade soil is unsuitable to be used in road construction and the proper remedial measure has to be taken before construction pavements.

Table 4-8: Summary of the engineering properties of expansive soil

Sr. No	Property of expansive soil	Observed value
1.	Classification	
	AASHTO (Group index)	A-7-5
	USCS group symbol	CH
	USCS group name	Inorganic clay soil
2.	Specific gravity	2.68
3.	Free swell, %	70
4.	Particle size analysis	
	Gravel content %(19mm to 4.75mm)	0
	Sand content % (4.75mm to0.75mm)	5
	Silty and clay content % (below 0.075mm)	95.0
5.	Atterberg limits %	
	Liquid limit	81
	Plastic limit	28.9
	Plasticity index	52.1
6.	Proctor test	
	Optimum moisture content (OMC), %	33.39
	Maximum dry density (MDD), g/cm^3	1.25
7.	California bearing ratio (CBR), % (soaked)	2.08

Results of the study on physical properties on a neat sample indicated that the sample belonged to expansive clay. Most of the properties required to be improved to meet the engineering standards.

4.3 Laboratory Test Results for Mix Design

The soil sample along the road section was collected to assess the effects of limestone and waste ceramic dust for weak subgrade soil stabilization. The most important parameters which are used to evaluate the effects of additives for this study were the CBR test, Atterberg's limits, and free swell test.

4.3.1 Effect of Waste Ceramic Dust on Engineering Properties of Expansive Soil

4.3.1.1 Effect of Waste Ceramic Dust (WCD) with Soil on Compaction Test

Standard proctor tests have been conducted to determine optimum moisture content (OMC) and maximum dry density (MDD) of soil treated with various percentages of waste of ceramic dust. Table 4.9 and Figure 4.6, Shows the values of maximum dry densities were noted to significantly increase with the addition of percentages of waste ceramic dust from 0 to 20% a neat value of 1.25 g/cm^3 to a maximum value of 1.53g/cm$^{3.}$ The OMC goes decreasing from 33.39% for parent soil to 15.37% for the inclusion of 25% waste ceramic dust. The reason for such behavior is, due to the replacement of waste ceramic dust particles with soil particles the attraction for water molecules decreases hence, OMC decreases. The summary of laboratory test results was discussed in detail under Appendix B-1.

Table 4-9: The MDD and OMC of stabilized soil with waste ceramic dust

Additive Content	OMC (%)	MDD (g/cm3)
0%	33.39	1.25
5% WCD	29.31	1.30
10% WCD	25.47	1.34
15% WCD	21.19	1.47
20% WCD	16.01	1.53
25% WCD	15.35	1.51

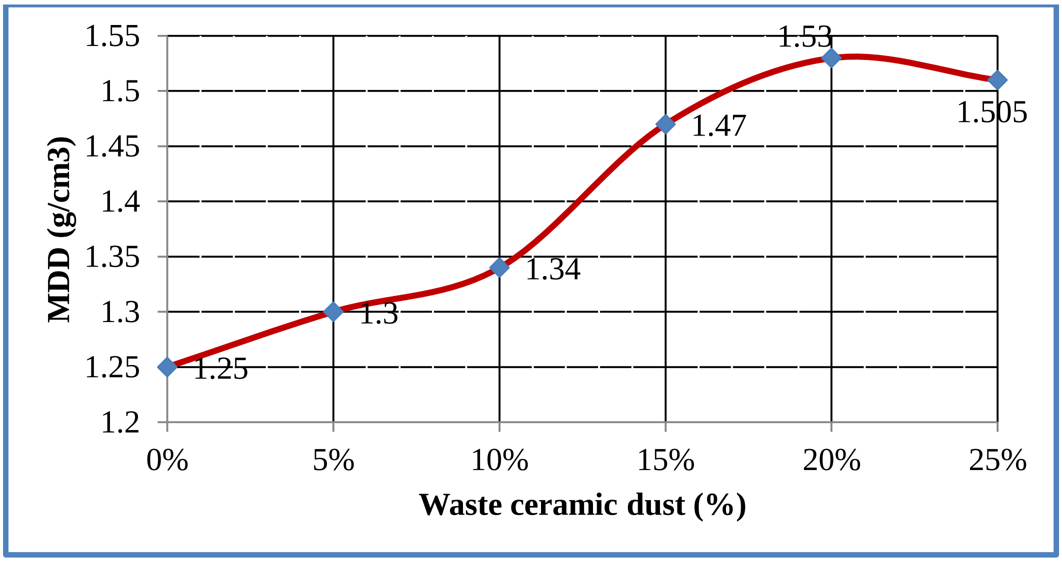

Figure 4-6: MDD and OMC of stabilized soil with waste ceramic dust

Figure 4.6 shows the variation of maximum dry density (MDD) with the inclusion of waste ceramic dust at various percentages. The MDD increases from 1.25g/cm^3 a neat value to a maximum value of 1.53g/cm^3 when 20% of waste ceramic dust added with the parent soil. There is a 22.4% increase in MDD of the soil at this percentage of ceramic dust as compared to untreated soil and thereafter it was reduced. The increase of MDD is due to the replacement of ceramic dust particles having high specific gravity (2.81) with soil particles having low specific gravity (2.68). In general, the maximum increase in MDD is by the addition of 20% of waste ceramic dust due to an increase in the density soil mix it leads to having more strength. According to the researcher (Geta Rani et al. 2014), found that the effect of waste ceramic dust resulted in a steady reduction in optimum moisture content. The maximum dry density and California Bearing Ratio of the soil increased and reached a maximum at 20% amendment and reduced thereafter.

4.3.1.2 Effect of Waste Ceramic Dust (WCD) on CBR Value

The California bearing ratio (CBR) was conducted to determine the performance of road subgrade of parent soils and soil treated with various percentages of waste ceramic dust. Table 4.10 indicates that the CBR value of parent soils and soil treated with waste ceramic dust. For the parent soil sample CBR, MDD and CBR swell value has been observed as 2.08, 1.2g/cm^3, and 5.02%, respectively. In the case of waste ceramic dust added to the soil, the values of MDD increase from 1.24 to 1.46g/cm^3 and CBR swell decreases from 4.76 to 1.9 % as its content increases from 5 to 25%, respectively. The laboratory test was summarized in Appendix B-2.

Table 4-10: Stabilization of waste ceramic dust with soil on CBR value

	Percentages of Waste Ceramic Dust							ERA (2013) (Specification
Sample soil		0%	5%	10%	15%	20%	25%	Subgrade
	MDD (g/cm3)	1.2	1.24	1.27	1.38	1.40	1.46	
	CBR %	2.08	3.30	4.65	5.7	6.6	6.15	CBR >3
	CBR Swell %	5.02	4.76	3.95	3.09	2.01	1.9	ERA (2002) 2 max

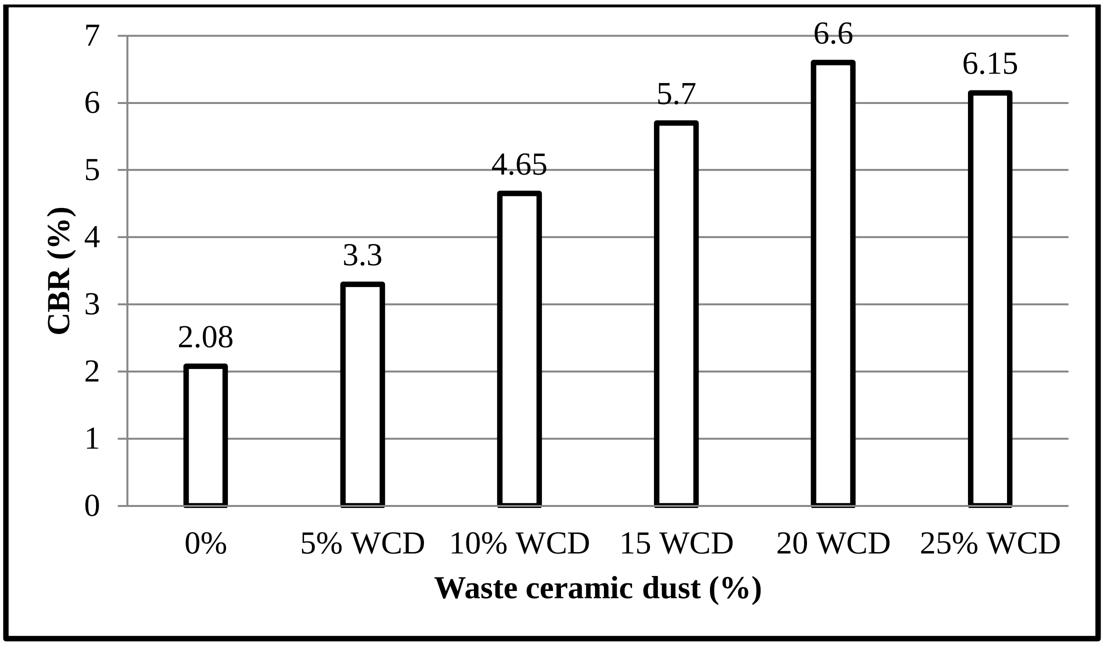

Figure 4-7: Waste ceramic dust with soil on CBR value

Figure 4.7, it can be seen that with an increase in the percentage of waste ceramic dust the soaked CBR value of soil goes on increasing. The soaked CBR increases from 2.08% to 6.6% when waste ceramic dust increased from 0 to 20% or at the blending of 20% WCD was reached.

There is a 217.3% increase in soaked CBR of the soil at 20% of ceramic dust as compared to untreated soil and thereafter it was reduced. The percentage change in CBR is very high in unstabilized soil when the sample is prepared at MDD (corresponding to OMC) but at water contents both dry and wet of optimum, whereas it is negligible in case of stabilized soil. As MDD increases with an increase in the percentage of ceramic dust, it results in an increase in soaked CBR values of the soil. The reason for such type of behavior of the stabilized soil is the reduction in clay content of soil by the replacement of soil by waste ceramic dust mix and due to chemical reaction between the soil and ceramic dust. It was another advantage of using ceramic dust stabilized expansive soil in place of unstabilized soil in pavement construction other than the advantage of increased CBR. Therefore depending on the given value, the result was fulfilling the design standard specification. According to ERA (2013) specification, the minimum value of CBR for road subgrade is >3. Sabat (2012) also found that with an increase in the percentage of waste ceramic dust, the soaked CBR of soil went on increasing. As MDD increases with an increase in the percentage of ceramic dust, it results in an increase in the soaked CBR value of the soil. Hence, waste ceramic dust was stabilized weak subgrade soil.

Figure 4.8 shows that a percentage of waste ceramic dust content increase, the swell in the soil due to expected moisture content decrease. The swell decreases from 5.02% to 1.9% as the content of waste ceramic dust increased from 5 to 25%. However, soaked CBR swells from 5 to 15% has not fulfilled the specification. According to ERA (2002) requirement, the CBR swell of the soil sample was not fulfilled which is greater than 2%. According to (Sabat, 2012) the amended soil showed reduced CBR swell, increased strength and bearing capacity for road subgrade soil. Such a trend indicates the behavior of waste ceramic dust was adsorption capacity in the soil. However, waste ceramic dust was little influence in the reduction of CBR swell due to give more strength additional stabilizer was needed.

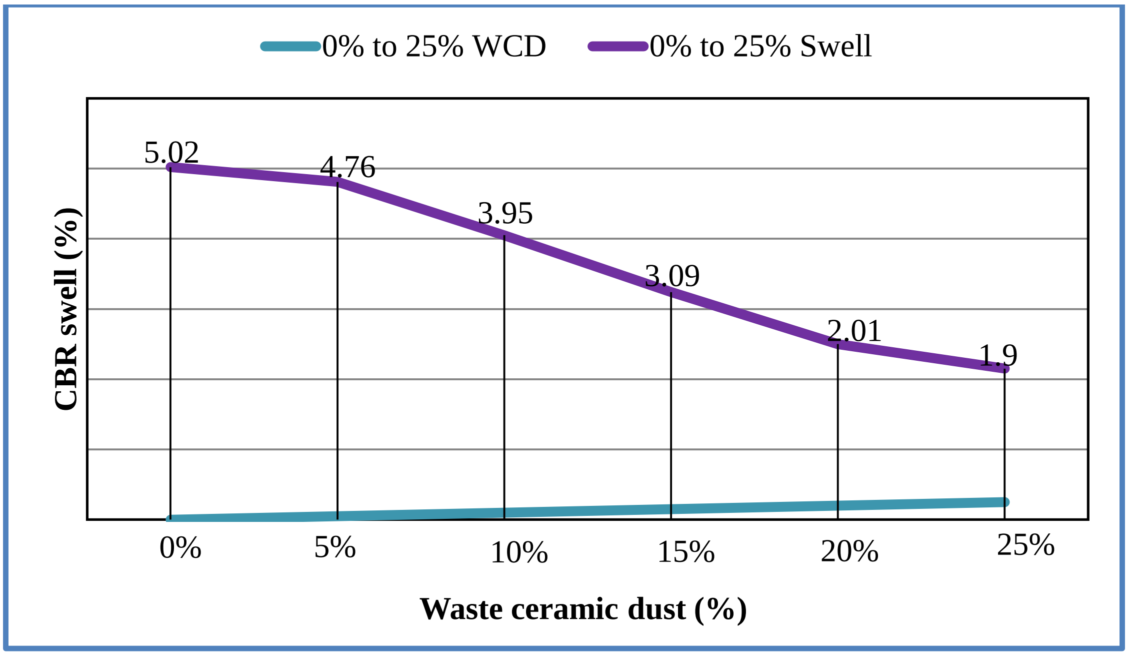

Figure 4-8: Waste ceramic dust with soil on the swell

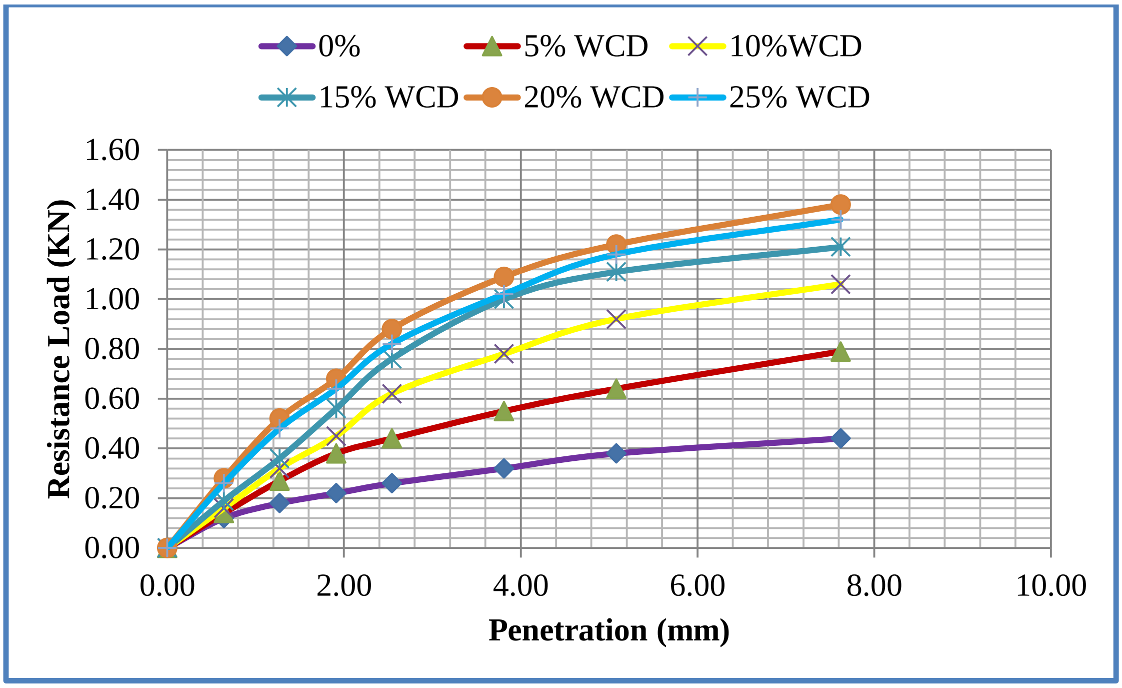

Figure 4-9: Graph of waste ceramic dust with soil

Figure 4.9 shows that resistance load and penetration; as percentage waste ceramic dust content increases the load-carrying capacity of the soil was increased. As ceramic dust content increases up to 20% the load-carrying capacity of the soil increases, however it decreases when 20% of ceramic dust content was reached.

4.3.1.3 Effect of Waste Ceramic Dust (WCD) with Soil on Atterberg's Limits

Atterberg's limit tests were conducted at different ratios of waste ceramic dust within varying proportions of soil samples. The main objective of this additive is to reduce the Plasticity Index of the soils. Table 4.11 indicates as a percentage of waste ceramic dust increases the liquid limit; plastic limit and plasticity index were decreased. For the parent soil the liquid limit, plastic limit, and Plasticity index value have been observed as 81%, 28.9%, and 52.1%, respectively. In the case of waste ceramic dust added to the soil sample, the values of the liquid limit of each soil decreased from 81 to 36% and plastic limit decrease from 28.9 to 11.1%, respectively as its content increases from 5 to 25%. The summary of test results was discussed under Appendix B-3.

Table 4-11: Atterberg's limits of WCD with soil samples

Additive content	Atterberg's limits			ERA (2013)
	LL (%)	PL (%)	PI (%)	LL< 60 PI < 30
0%	81	28.9	52.1	
5% WCD	74	27.1	46.9	
10% WCD	68	26.0	42.0	
15% WCD	58	21.6	36.4	
20% WCD	46	16.9	29.1	
25% WCD	36	11.1	23.4	

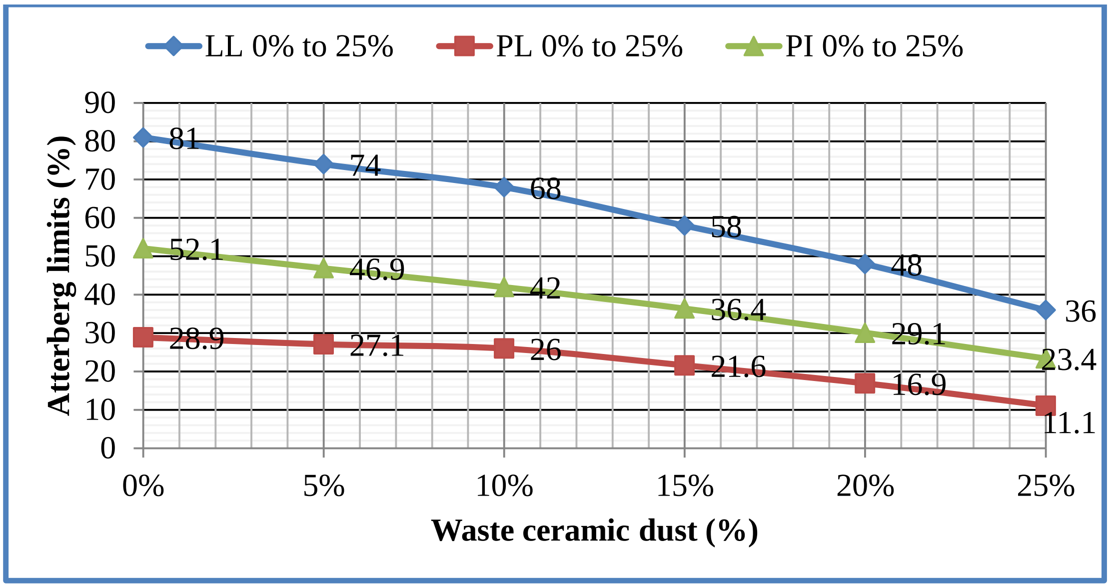

Figure 4-10: Effect of waste ceramic dust on atterberg's limits

Figure 4.10, indicates that the plasticity index of the soil samples goes on decreasing with the addition of waste ceramic dust. As the percentage of waste ceramic dust content increases from 0% to 25%, the value of the plasticity index was decreased from 52.1% to 23.4%. The decrease of plasticity index was in the order of 122.6% by the addition of 0 to 25% of waste ceramic dust as compared to untreated soil. The maximum reduction of the plasticity index has occurred at 25% of waste ceramic dust. This is clearly shown by the fact that the plasticity index of the treated soil was decreased with increasing additive content quantity. This has happened due to the replacement of fine-grained particles of expansive soil with coarse-grained particles of waste ceramic dust. The blending of soil with waste ceramic dust from 5 to 15% was not fulfilled the ERA specification but only satisfy at the mix of 20% and 25% of Waste ceramic dust. Hence, the soil needs additional stabilizers to improve the subgrade properties.

4.3.1.4 Stabilization of Waste Ceramic Dust (WCD) on Free Swell Test

The objective of this test is the stabilization of expansive soil with waste ceramic dust reduces the swelling potential of soil. Table 4.12 shows that as the content of waste ceramic dust increases the swell of the soil was decreased. The swelling pressure decreases from 70% to 18.6%, when ceramic dust is increased from 0 to 25%. The maximum attainment of the free swell of the soil with waste ceramic dust was at 25%. According to (IS: 2911 Part III - 1980), the degree of the expansiveness of the soil at 25% was less than 20%.

Table 4-12: Free swell test of WCD with soil samples

Additive content											
0%		5%		10%		15%		20%		25%	
Vf	Vk	Vf	Vk	Vf	Vk	Vf	Vk	Vf	Vk	Vf	Vk
17	41	55	34	49	33	44	31	35	27	21	17.7
70		61.8		48.5		41.9		29.6		18.6	

Based on Table 4.12 the test results which were obtained from the strength characteristics, it was concluded that the maximum free swell was attained at 25% of waste ceramic dust. According to (Krishna, 2016), examined that the modified soil with waste ceramic dust swelling pressure goes on decreasing as a percentage of waste ceramic dust increases. Sabat (2016), studied that, the free swelling of the soil was decreased as a percentage of waste ceramic dust increases, this happens due to a decrease in clay content of the expansive soil by replacement of ceramic dust, which is non-expansive in nature. As the attraction for water molecules decreases, the free swelling nature of the soil decreases which results in a decrease in the free swells. However, ceramic dust was little effect on free swelling. Hence, it needs additional stabilizer to reduce the swelling of the soil the weak subgrade soil.

4.3.2 Effect of Limestone on Engineering Properties of Expansive Soil

4.3.2.1 Effect of Limestone on Compaction Test

The maximum dry density and optimum moisture content of the parent soil and soil treated with various percentages limestone were determined using a compaction test. The variation of MDD of expansive soil with limestone (%) has been shown in Table 4.13 and Figure 4.11. The MDD of soil goes on decreasing with an increase in the percentage of limestone. The maximum dry density decreased to 1.2g/cm^3 from 1.25g/cm^3 when the limestone increased to 10%. The decreasing trends in maximum dry density can be attributed to the cationic exchange of the limestone which induces flocculation and agglomeration of clay particles. The OMC was found to increase from 33.39 % to 44.43%, due to the additional fine contents, which requires more water in addition to the free lime that needed more water for pozzolanic reactions. This occurs in spite of reduced surface area caused by flocculation and agglomeration. The summary of laboratory test results was discussed in detail in Appendix B-4.

Table4-13: Limestone with soil on compaction test

Additive content		
	OMC (%)	MDD (g/cm3)
0%	33.39	1.25
2% L. stone	34.7	1.24
4% L. stone	35.4	1.235
6% L. stone	37.75	1.22
8% L. stone	39.36	1.21
10% L. stone	44.43	1.20

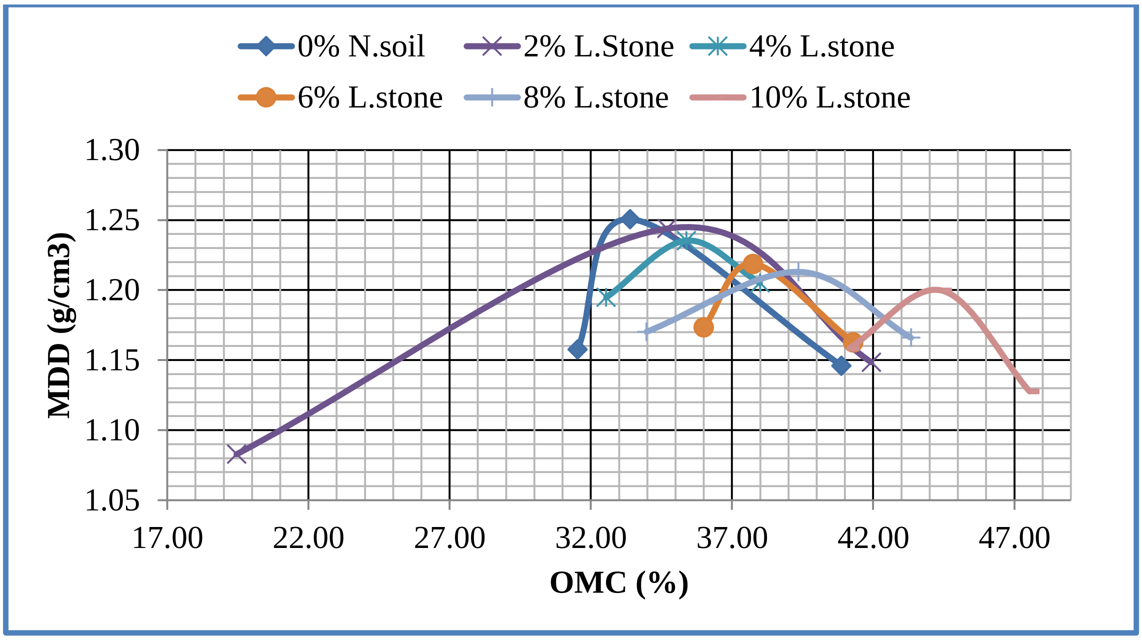

Figure 4-11: Graph of limestone with soil on the compaction test

Figure 4.11 shows that the percentage of limestone content increases the moisture content of the soil increases and the maximum dry density was decreased. Hence, the soil was improved with limestone by 2%, 4%, 6%, 8%, and 10% the value of MDD found to be decreased by 0.01, 0.8, 0.82, 0.83, and 0.84%, respectively. According to (Brooks et al. 2011) studied the potential of limestone to stabilize the expansive soil. The maximum dry density of the soil with limestone mixture decreased and the optimum moisture content of the mixture was increased with an increase in limestone content Hence the limestone was more effective than waste ceramic dust.

4.3.2.2 Effect of Limestone on CBR Value

The CBR was conducted to determine the strength of parent subgrade soil and soil treated with various percentages of limestone. Table 4.14 shows that the CBR value of parent soils and soil treated with limestone. For the parent soil sample CBR, MDD and CBR swell value has been observed as 2.08%, 1.2g/cm^3, and 5.02%, respectively. In the case of limestone added to the soil the values of MDD decrease from 1.18 to 1.14g/cm^3 and CBR swell decreases from 4.54 to 1.55% as its content increases from 2 to 10%, respectively. The CBR value of the soil was found to be increased from 3.75 to 10.9% as the content of limestone increases from 2 to 8%. The maximum attainment of CBR value was at 8% and thereafter it was decreased. Such performance has been observed due to the bond between the soil particles and limestone becomes strong and the load-bearing capacity has been increased. The laboratory test was summarized in Appendix B-5.

Table 4-14: Stabilization of limestone with soil on CBR value

Sample soil	Percentages of Limestone dust							ERA (2013) Specification
		0%	2%	4%	6%	8%	10%	Subgrade CBR >3
	MDD (g/cm3)	1.2	1.18	1.17	1.16	1.15	1.14	
	CBR %	2.08	3.75	5.7	7.95	10.9	9.9	
	Swell %	5.02	4.54	3.52	2.97	1.82	1.55	ERA (2002)

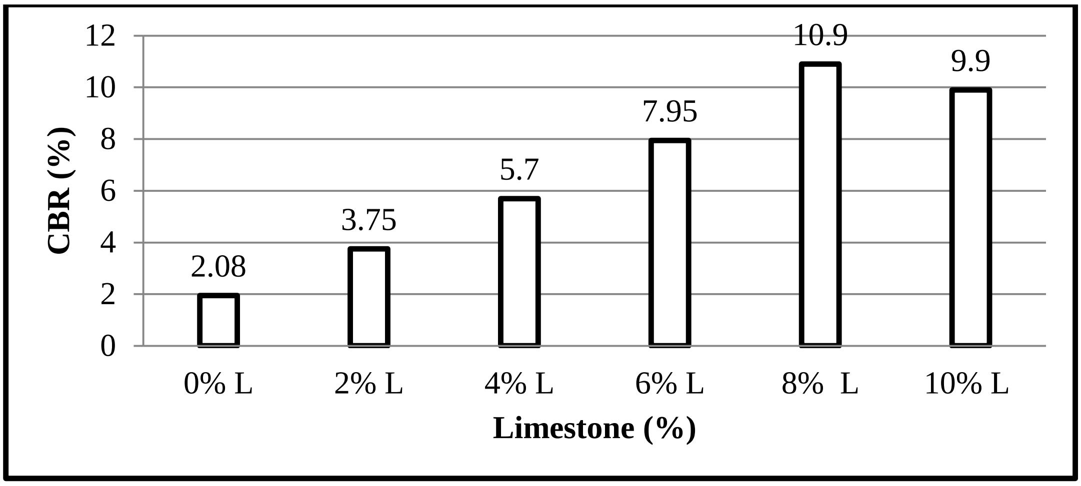

Figure 4-12: Limestone with soil on CBR value

The variation of soaked CBR of expansive soil with limestone (%) has been shown in Figure 4.12, with an increase in percentage addition of limestone the soaked CBR of goes on increasing. The soaked CBR increased to 10.9% from 2.08% when the percentage of limestone is 8%, thereafter it decreased. The maximum increase in the percentage of soaked CBR is 424% as compared to the soaked CBR of virgin expansive soil when the percentage addition of limestone is 8%. Such a trend indicates the behavior of limestone was fascinated capacity in the soil and gave strength. According to (Brooks et al. 2011) suggested that a significant increase was observed in the strength of the soils for CBR when stabilized with the limestone. However, based on the results limestone improves the soil which fulfilled the ERA specification for subgrade soil. Since the ERA specifies material to be used in subgrade soil have a minimum CBR value was 3%. According to ERA low pavement manual specification, it is not allowed to use CBR values less than 3%, because from both a technical and economical perspective it would normally be inappropriate to lay a pavement on soils of such bearing capacity, due to that the soil was stabilized with this additive to improve the properties of the soil. However, when comparing the result of the stabilization of limestone and waste ceramic dust with soil on CBR value limestone was more effective than waste ceramic dust.

Figure 4.13 indicates that the percentage of limestone content increase, the swell in the soil due to expected moisture content decrease. As limestone content increase from 2% to 10% the soaked CBR swell of the soil was decreased from 4.54% to 1.55%. The CBR swell of expansive soil decreased with all higher limestone dust contents. This shows that the swelling potential of the sample decreased with limestone stabilization. However, limestone dust stabilized expansive soil was non-swelling material. Hence, the blending of limestone with soil at 8% and 10% were satisfied ERA specification, but the blending at 2, 4 and 6% were not satisfied with the specification. According to ERA specification, the maximum of the soaked CBR swell for subgrade soil was 2%.

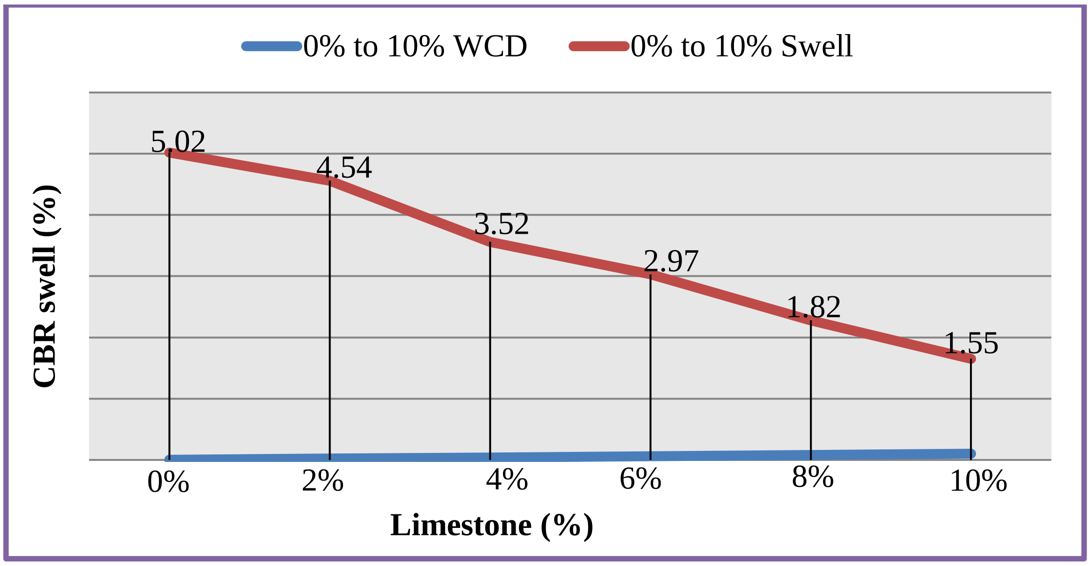

Figure 4-13: Graph of limestone with soil on the swell

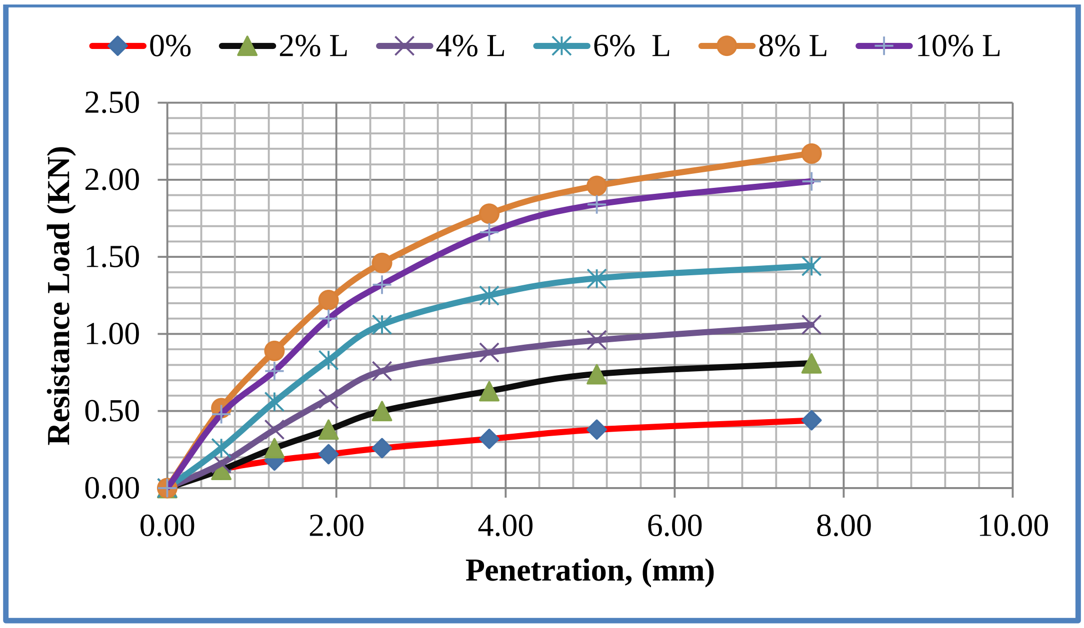

Figure 4-14: Resistance load and penetration on CBR value

Figure 4.14 shows that resistance load and penetration; the maximum attainment of the CBR value was at 8% and thereafter it is decreased. Therefore limestone can be used as a stabilizing agent. Hence, when comparing the results as limestone mixed with soil in an equal amount from 2% to 10% as waste ceramic dust, higher strength than those provided by waste ceramic dust.

4.3.2.3 Effect of Limestone on Atterberg limits

The Atterberg limit tests were conducted at different ratios of limestone within varying proportions of soil samples. The objective of this additive is to reduce the Plasticity Index of the soils. Table 4.15 indicates as a percentage of limestone increases the liquid limit and plasticity index was decreased as well as the plastic limit was increased. For the parent soil, the liquid limit and Plasticity index value have been observed as 81 and 52.1, respectively. As limestone added to the soil sample from 0 to 10% the liquid limit was decreased from 81 to 27%, and the plastic limit was increased from 28.9% to 40.6%, respectively. The summery of test results were discussed under Appendix B-6.

Table 4-15: Limestone with soil on Atterberg's limits

Additive content	Atterberg's limits			ERA (2013)
	LL (%)	PL (%)	PI (%)	
0%	81	28.9	52.1	Subgrade soil
2% limestone	68	31.5	36	
4% limestone	58	32.6	25	LL<60
6% limestone	47	35.3	10.7	
8% limestone	36	38.5	6.5	PI <30
10% limestone	27	40.6	3.2	

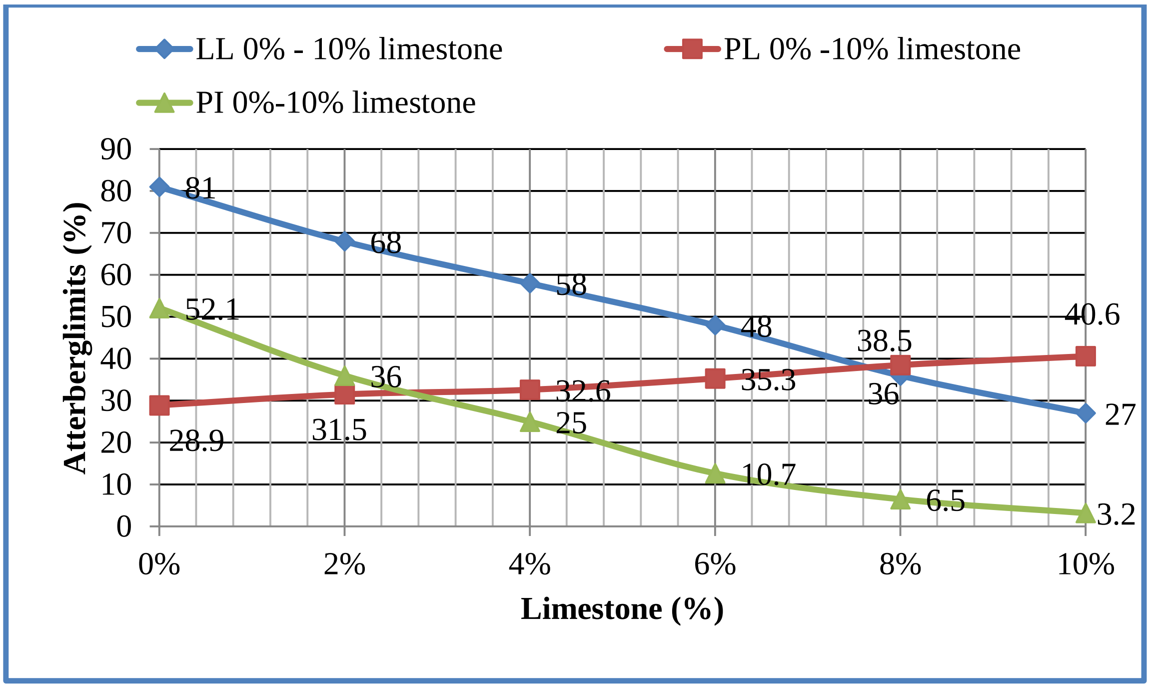

Figure 4-15: Graph of Atterberg's limits of expansive soil with limestone

The variation of the plasticity index of the expansive soil with limestone (%) has been shown in Figure 4.15. the plasticity index goes on decreasing with an increase in the percentage of limestone. The plasticity index decreased to 3.2% from 52.1% when the limestone increased to 10%. Hence, this mixes soil with limestone at 2% not fulfilled the ERA specification. The blending limestone with soil from 4% to 10%, which was fulfilled the specification. According to AI-Azzo (2009) had studied the stabilizing effect of limestone on engineering properties of expansive clay, there was a reduction in the plasticity index of the soil. However, limestone has more influence on the plasticity index. Such behavior has been observed due to the addition of limestone with soil has no plasticity. Based on the discussed results limestone was more effective and influence than waste ceramic dust.

4.3.2.4 Effect of Limestone Dust on Free Swell Test

The variation of swelling pressure of expansive soil with limestone (%) has been shown in Table 4.16 and Figure 4.16 shows with an increased percentage of the addition of limestone the swelling pressure of soil goes on decreasing. The swelling pressure decreased to 14.3% from 70% when the percentage of the addition of limestone was 10%. There is a 280% decrease in swelling pressure as compared to the swelling pressure of virgin expansive soil. Free swelling was reduced from 70% for natural soil to 57.1, 41.7, and 28.6% when 2%, 4% and 6% of limestone powder is added. For additions of higher than 6%, less reduction was achieved, obtaining values of 18.4% and 14.3% of free swelling for 8, and 10% limestone, respectively. Based on the results the maximum swell reduction was attained at 10% of limestone. According to (IS: 2911 Part III - 1980), the degree of the expansiveness of the soil at 8% and 10% was less than 20%; hence, limestone is more suitable than waste ceramic dust**.** According to AI-Azzo (2009) had studied the stabilizing effect of limestone on engineering properties of expansive soil, there was a significant decrease in the expansion of the soil as limestone content increased. This reduction swell was due to the replacement of soil by limestone, which had non-plasticity characteristics. Also according to Sabat (2015), the increase of the free swelling for 8% and 10% of limestone powder is probably due to the considerable increase of matric suction caused by the reduction of the initial water content of the sample, since the dust is to the wet soil. Hence, when compared the results limestone was more influence than waste ceramic dust.

Table 4-16: Limestone with soil on free swell test

Additive content											
0%		2%		4%		6%		8%		10%	
Vf	Vk	Vf	Vk	Vf	Vk	Vf	Vk	Vf	Vk	Vf	Vk
17	41	44	28	34	24	27	21	22.5	19	12	10.5
70		57.1		41.7		28.6		18.4		14.3	

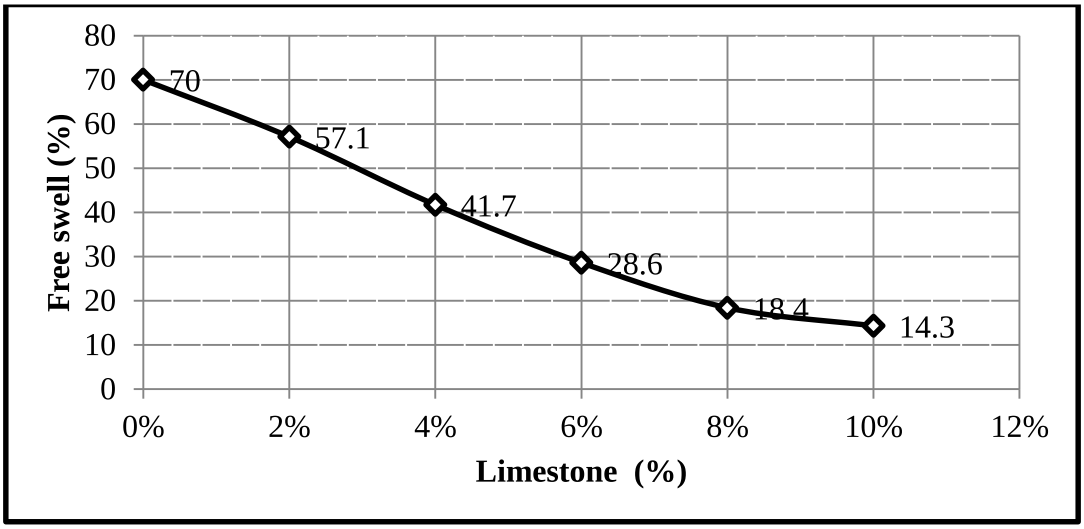

Figure 4-16: Effect of limestone content on the free swell of stabilized expansive soil

4.3.3 Effect of Limestone -Waste Ceramic dust on Engineering Properties of Expansive Soil

4.3.3.1 Effect of Limestone-Waste Ceramic Dust on Compaction Test

Stabilization of soil with waste ceramic dust - limestone was performed. The values for the maximum dry densities were significantly increased and the water content value was decreased with the addition of limestone-waste ceramic dust mixed with soils. Table 4.17 and Figure 4.17, indicate the OMC and MDD of parent soils and soil treated with limestone-waste ceramic dust. The values for the maximum dry densities were noted to significantly decrease with the addition of waste ceramic dust-limestone from a neat value of 1.25g/cm^3 to a maximum value of 1.56g/cm^{3}attained in the mix of 20% WCD + 8%L. The optimum moisture content (OMC) was found to decrease; due to the increase of coarser particles in the mix, the attraction for water molecule reduces, and hence OMC reduces. The ceramic waste-limestone content increases the MDD of the soil increases beyond 20% WCD + 10% limestone content and started reducing when 20% WCD +8% limestone content was reached. The increase of MDD is due to the fill-up of the void spaces of clayey soil by the limestone-waste ceramic dust particles. Also, it may be due to the chemical reaction between the soil and additives to be used for the increment of the MDD. Hence, waste ceramic dust- limestone dust great effect when compared to each individual's stabilization of limestone, waste ceramic dust with soils. Therefore, from our findings limestone-waste ceramic dust was suitable for the improvement of weak subgrade soil. The summary of the test results was discussed in Appendix B-7.

Table 4-17: The OMC and MDD of limestone-waste ceramic dust on compaction test

Additive Content		
	OMC (%)	MDD (g/cm3)
0%	33.39	1.25
20% WCD + 2L. stone	28.65	1.38
20% WCD + 4 L. stone	26.55	1.41
20% WCD + 6 L. stone	24.1	1.44
20% WCD + 8 L. stone	20.03	1.56
20% WCD + 10 L. stone	17.78	1.53

Table 4.17 shows, the MDD for the neat value of soil was 1.25 g/cm^3. Improvement with 20% of waste ceramic dust alone gave an MDD value of 1.53g/cm^3, while as a blend of 2% of limestone 1.24g/cm^3. A mix of 20% of waste ceramic dust + 2% limestone gave a value of 1.38 g/cm^3, while a mix of 20% waste ceramic dust + 8% limestone gave a value of 1.56g/cm^3. However, the maximum dry density of a mix of 20% waste ceramic dust + 10% limestone was reduced thereafter.

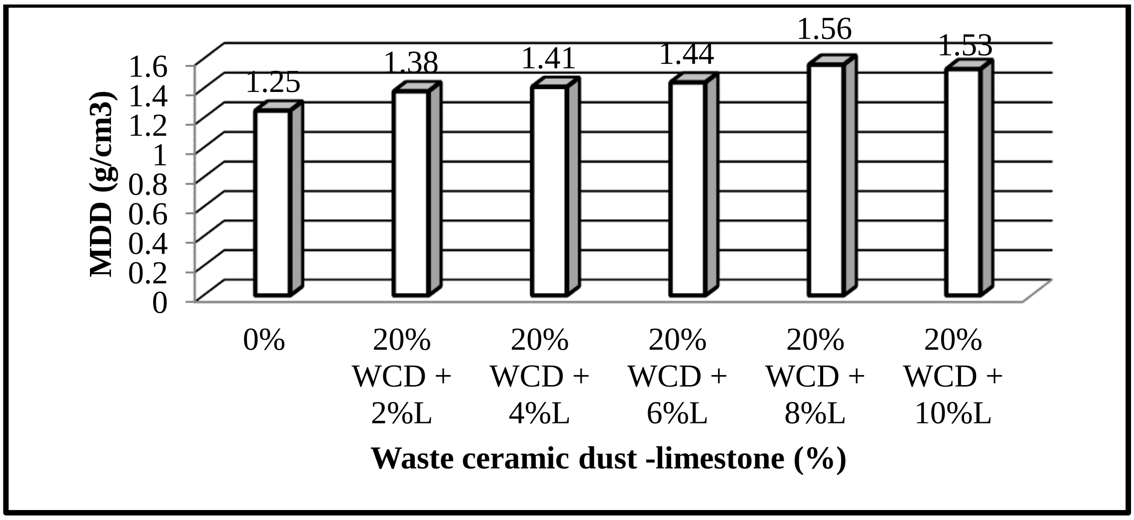

Figure 4-17: Effect of limestone -waste ceramic dust addition on compaction test

Figure 4.17 shows that the blend of limestone with waste ceramic dust improves the maximum dry density. A mix of 20% waste ceramic dust + 2% limestone improves maximum dry density by 10.4% and a blend of 8% limestone + 20% of waste ceramic dust improves the maximum dry density by 24.8% when compared to untreated soil. Therefore, the blending of 8% L. stone + 20% WCD provides a maximum dry density for the soil.

4.3.3.2 Effect of Limestone-Waste Ceramic Dust on CBR value

The CBR was conducted to determine the strength of parent subgrade soil and soil treated with various percentages of limestone-waste ceramic dust. Figure 4.18 indicates that the CBR values of parent soils and soil treated with limestone-waste ceramic dust. For the parent soil, the soaked CBR value has been observed as 2.08%. In the case of limestone-waste ceramic dust added to the soil, the values of CBR noted to significantly increase from 2.08 to 16.04% and thereafter it reduced. The CBR values of the soil sample were increased due to the bond between limestone-waste ceramic dust and soil particles become strong the load-bearing capacity has been increased. The laboratory test result was discussed in Appendix B-8.

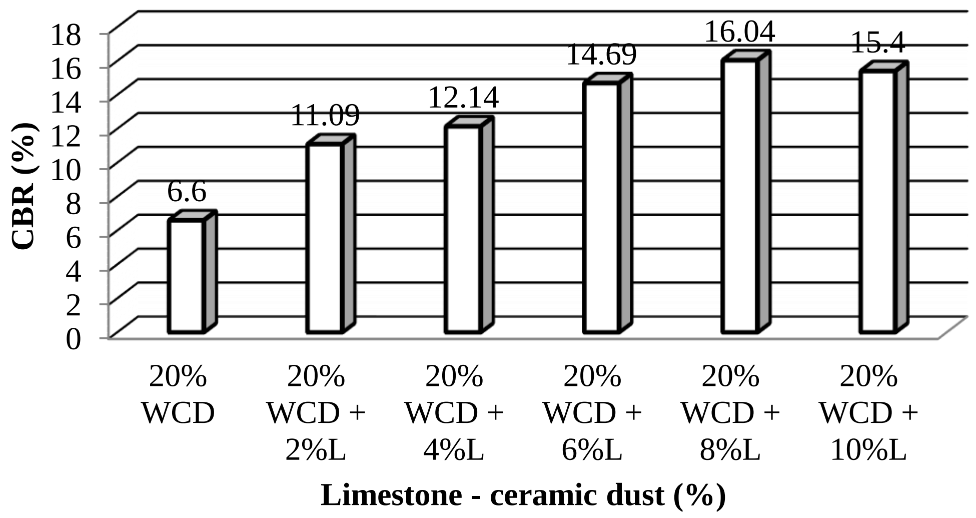

Figure 4-18: Waste ceramic dust - limestone with soil on CBR test

The California bearing ratio for natural soil was 2.08%. The improvement soil with 2% L. stone alone gave a CBR value of 3.75%, 4% L. stone gave a value of 5.7%, 6% L. stone gave a value of 7.95%, 8% L. stone gave a value of 10.9%, while as 20% WCD alone gave a value of 6.6%. A mix of 20 % waste ceramic dust+ 2% limestone gave 11.09%, a mix of 20% waste ceramic dust + 4% limestone gave a value of 12.04, while as a mix of

20% waste ceramic dust + 6% limestone gave a value of 14.69%, a mix of 20% waste ceramic dust + 8% limestone gave a value of 16.04% and thereafter a mix of 20% waste ceramic dust + 10% limestone was reduced.

Based on the result the addition of limestone- waste ceramic dust with soil which it was fulfilled the ERA specification for subgrade soil. According to (Ayothiraman et.al, 2002), specified that the lower CBR values (less than 10), lead to the deflection of the subgrade material under heavy traffic loadings. Thus, it is very crucial for the engineers to develop a minimum of a CBR value of 10 for all subgrades. Hence, depending on the findings waste ceramic dust-limestone CBR values were greater than each individual stabilizer so that the mixtures of both additives are good stabilizers for weak subgrade soil.

4.3.3.3 Stabilization of Limestone-Waste Ceramic Dust on Atterberg's limits

The Atterberg's limits were conducted to determine the plasticity index of soil and soil treated with limestone-waste ceramic dust. Table 4.18 and Figure 4.19 indicate as a percentage of limestone –waste ceramic dust increases the liquid limit and plasticity index was decreased as well as the plastic limit was increased. In the case of limestone-waste ceramic dust added to the soil, the values of liquid limit decrease from 81 to 42%, and plastic limit increase from 28.9 to 39.8%, at the optimum percentage of (8% L + 20%WCD) attained. The variation of the plasticity index with the percentage of limestone – ceramic dust is shown in Figure 4.19. As the figure indicates, it can be observed that the plasticity index goes on decreasing with the addition of limestone-ceramic dust. The plasticity index decreases from 21% to 2.7% when the additives are increased from 20% WCD + 2% L to 20% WCD + 10% L. Waste ceramic dust-limestone was cohesionless, it was expected that it would reduce plasticity index of soil and the result satisfies the expectation. The effects due to partial replacement of plastic soil which is non-plastic material and flocculation/ agglomeration of clay particles caused by cation exchange maybe the other cause. According to ERA specification the blending of 20% WCD + 2% L, 20% WCD + 4% L, 20% WCD + 6% L, 20% WCD + 8% L, and 20% WCD + 10% L was fulfilled the specification for subgrade construction. These effects are due to the partial replacement of plastic soil with limestone-waste ceramic dust which is non-plastic material and flocculation and agglomeration of clay particles caused by cation exchange maybe the other cause. The test result was discussed in Appendix B-9.

Table 4-18: Stabilization of limestone – ceramic dust with soil on Atterberg's limits

Additive content	Atterberg's limits			ERA (2013)
	LL (%)	PL (%)	PI (%)	
0%	81	28.9	52.1	Subgrade soil
20% WCD + 2% L	53	31.6	21	
20% WCD + 4% L	51	34.1	17	LL<60
20% WCD + 6% L	48	37.6	10.4	
20% WCD + 8% L	45	38.3	6.7	PI <30
20% WCD + 10% L	42	39.8	2.7	

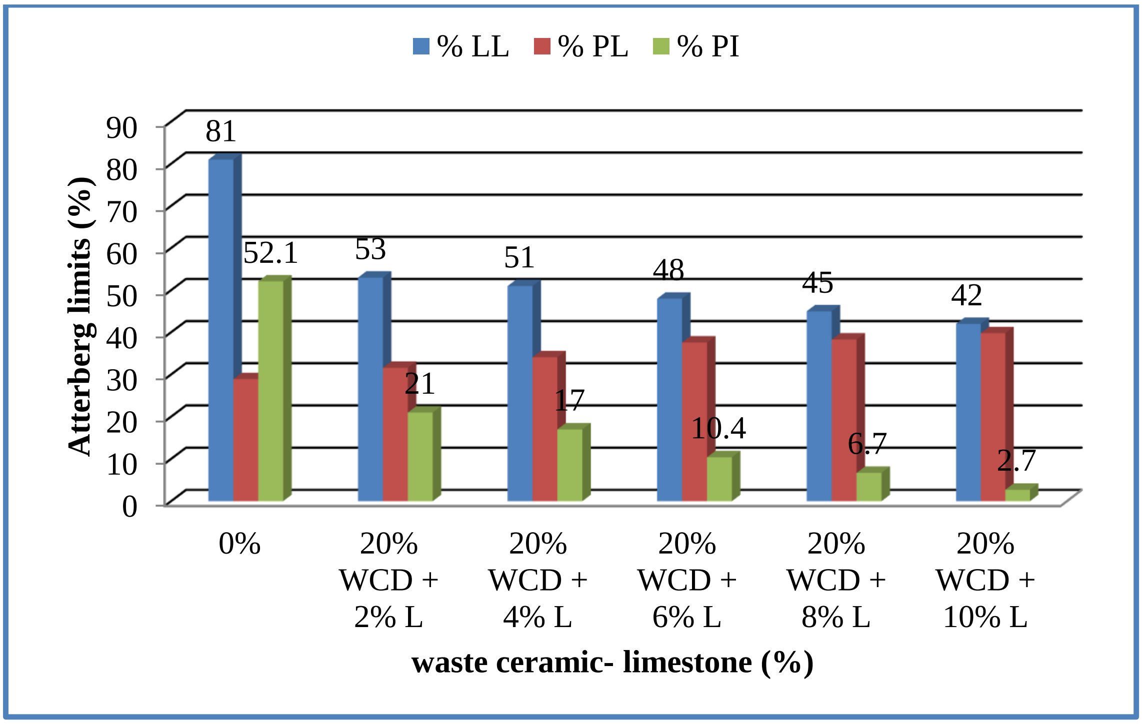

Figure 4-19: Waste ceramic dust - limestone with soil on Atterberg's limits

Summary of test results for subgrade soil in comparison with ERA (2013) and requirements depending on the California Bearing Ratio and plasticity index the suitability of blended material for road subgrade construction was discussed in Table 4.19 below.

Table 4-19: Summary of test results for subgrade soil with standard specification

No.	Additive Content	PI (%)	CBR (%)	Suitability for Subgrade ERA PI<30	Suitability for Subgrade ERA CBR>3%
	Soil	52.1	2.08	Not suitable	Not suitable
1	(WCD)				
	Soil + 5% WCD	46.9	3.3	Not suitable	Suitable
	Soil + 10% WCD	42	4.65	Not suitable	Suitable
	Soil + 15% WCD	36.4	5.7	Not suitable	Suitable
	Soil + 20% WCD	29.1	6.6	Suitable	Suitable
	Soil + 25% WCD	23.4	6.15	Suitable	Suitable
2	Limestone (L)				
	Soil + 2% L	36	3.75	Not suitable	Suitable
	Soil + 4% L	25	5.7	Suitable	Suitable
	Soil + 6% L	10.7	7.95	Suitable	Suitable
	Soil + 8% L	6.5	10.9	Suitable	Suitable
	Soil + 10% L	3.2	9.9	Suitable	Suitable
3	Combination				
	Soil + 20% WCD + 2% L	21	11.09	Suitable	Suitable
	Soil + 20% WCD + 4% L	17	12.14	Suitable	Suitable
	Soil + 20% WCD + 6% L	10.4	14.7	Suitable	Suitable
	Soil + 20% WCD + 8% L	6.7	16.1	Suitable	Suitable
	Soil + 20% WCD + 10% L	2.7	15.4	Suitable	Suitable

As per ERA specification, it is possible to use a material those have a maximum plasticity index of 30% and a minimum CBR of 3% for subgrade construction.

4.3.3.4 Stabilization of Limestone -Waste Ceramic Dust on Free Swell Test

The test was conducted to determine the improvement of limestone-waste ceramic dust with soil to reduce the free swell of material. Table 4.20 shows that as the content of limestone-waste ceramic dust added with the soil at a maximum percentage of (20%WCD + 10% L) the percentage of swell soil was decreased. As a percentage of limestone- waste ceramic dust added to the soil the swelling potential of soil was decreased. According to (IS: 2911 Part III - 1980), the degree of the expansiveness of the soil was less than 20%, but only 20% WCD + 2 L was greater than 20. Hence, limestone-waste ceramic dust was more suitable and effective than each individual stabilizer. Such a trend has been observed due to standby of soil by limestone-waste ceramic dust which had been non-plasticity. Hence, limestone-waste ceramic dust with soil was more influential than each individual stabilizer.

Table 4-20: Limestone-waste ceramic dust with soil on free swell test

Additive content	Initial reading by kerosene (Vk)	Final reading by water (Vw)	FS (%)
0%	41.0	70	70
20% WCD + 2% L	26	32	23.1
20% WCD + 4% L	25	29.5	18
20% WCD + 6% L	21	24.5	16.6
20% WCD + 8% L	15	17	13.3
20% WCD + 10% L	11	12	9.1

The Table 4.20 shows the free swell of the soil at 20% WCD + 4%L gave a value of 18%, 20% WCD + 6%L gave a value of 16.6%, 20% WCD + 8% L gave a value of 13.3% and 20% WCD gave a value of 9.1%, thus according to (IS: 2911 Part III - 1980) which was fulfilled or below 20%. Hence, the blending of 20%WCD + 2% L not fulfilled the specification. However, the combination of both additives was effective and influence on stabilization of the soil when compared to each individual stabilizer.

4.4 Properties of Crushed Limestone and Ceramic Waste

The locally collected waste ceramic from the construction sites is used in the experiment. The geotechnical properties of the ceramic dust and limestone used in the experimental program are given table. The test result was discussed in Appendix B-10.

Table 4-21: Properties of crushed waste ceramic dust and limestone

Properties	Waste ceramic	Limestone
Specific gravity at 21 ^oC	2.81	2.66
Liquid limit	None	30.5
Plastic limit	None	None
Plastic index	None	None
Free swell	0.5	1

Table 4.21 shows the specific gravity test result of ceramic waste and limestone. Both materials have specific gravity which it was 2.81, and 2.66, respectively. Since the specific gravity of ceramic waste was larger relative to limestone, thus additive has no water content.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

Expansive soils undergo volumetric changes upon wetting and drying, thereby causing ground heave and settlement problems. Therefore, these problematic soils when encountered as subgrade should be avoided or treated properly. The objective of this study is to compute the improvements achieved on the engineering properties of expansive soils due to waste ceramic dust and limestone dust. From the study the following findings are deduced:

- The subgrade soils are categorized as fine-grained soil from which more than 50% of the particle size passed through 75 micrometers and inorganic soils. According to AASHTO soil classification, expansive soil of the study area was A-7-6 and high plastic clay (CH), as per the UCS system. The soil of the study area was not suitable for road constructions which have a low bearing capacity and they did not satisfy ERA specification based on test results.
- The addition of waste ceramic dust with soil, the value of liquid limit, plastic limit and plastic index was decreased. Also, the values of MDD were noted to significantly increase while OMC, CBR swell, the free swell ratio of the soil was found to decrease. The soaked CBR increases from 2.08% to 6.6% when waste ceramic dust increased from 0 to 20% and thereafter it was reduced. The maximum increase in CBR is by the addition of 20% of waste ceramic dust by 217.3% as compared to untreated soil.
- Also as limestone mixed with soil, the value of liquid limit, plastic index, MDD, and CBR swell of the soil was decreased while the plastic limit of modified soil was increased. Hence, OMC, CBR of the soil samples was increased at optimum percentages of limestone increases from 2 to 8% and thereafter it was reduced. The maximum increase in CBR is by the addition of 8% of limestone.
- The combination of both waste ceramic dust and limestone blending with the soil at optimum replacement ratio 20 % of WCD and 2 to 8% of limestone: the value of liquid limit, plastic limit and plastic index of the improved soil was decreased. However, MDD was significantly increased while as OMC, free swell ratio, CBR

swell ratio of the soil was decreased as waste ceramic dust – limestone mixed with the soil.

- The blending of limestone - waste ceramic dust with soil at optimum percentage the values of CBR was 16.14%, which was a better result than individually.
- According to ERA design specification minimum, CBR for weak subgrade soil was 3%. According to some researchers suggests that the minimum CBR for weak subgrade soil is 10. From this study, the optimum replacement ratio of waste ceramic dust with limestone is more effective and influences the soil in which CBR values are greater than 10.

Generally, the stabilization of weak subgrade soil with waste ceramic dust, limestone dust, and ceramic dust-limestone was computed in the laboratory test, it was concluded that the stabilization of the soil with ceramic dust –limestone dust was a good stabilization for weak soil than each individual stabilizers. Based on the results and analysis, the maximum percentage of waste ceramic dust and limestone can be used for road subgrade soil.

5.2 RECOMMENDATIONS

Based on the presented study results the following recommendations were forwarded:

- The present study was conducted by taking limited parameters of atterberg limits, CBR, Free swell test, moisture-density relation, CBR swell potential on stabilization by waste ceramic dust and limestone.
- The effects of limestone and waste ceramic dust on engineering properties of the soil was satisfied ERA specification and used as a capping layer for soil stabilization.
- The optimum replacement ratio of 20% WCD and 8% limestone is suitable for weak subgrade soil.

Scope for further study

- Industrial waste like waste ceramic dust stabilization has well for weak soil; it should be planned for future construction.
- Also, it is recommended that additional parameters of unconfined compressive strength (UCS), Curing days and mineralogical tests should be also performed to have more faithful.
- This study was conducted by taking a limited sample. It is recommended to conduct stabilization by taking a large number of samples as the whole study area.
- Further studies should be carried out in order to identify the PH value of ceramic dust, limestone and ceramic dust-Limestone.
- For practical applicability of the stabilized soils further detail investigations including mechanical analysis of the treated soils will of supreme.

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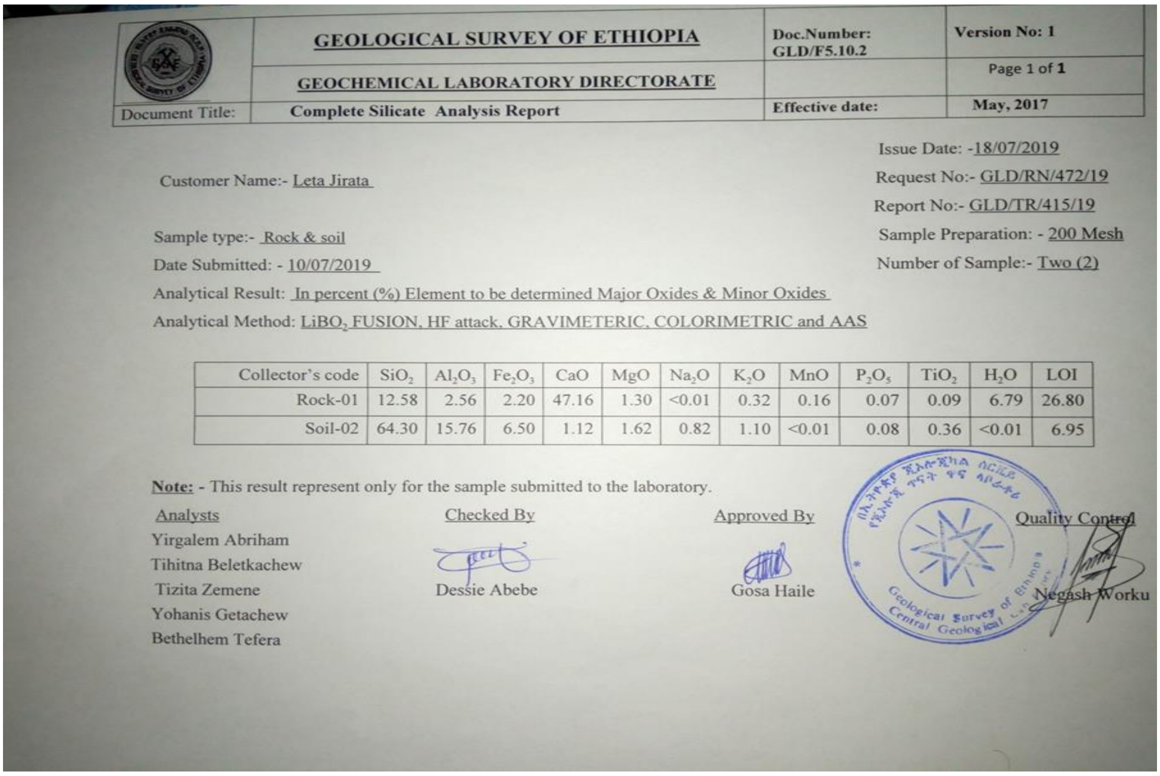
APPENDIX

Appendix A-1: Chemical composition of soil and limestone



GEOLOGICAL SURVEY OF ETHIOPIA		Doc.Number: GLD/F5.10.2	Version No: 1
GEOCHEMICAL LABORATORY DIRECTORATE			Page 1 of 1
Document Title:	Complete Silicate Analysis Report	Effective date:	May, 2017

Issue Date: -18/07/2019

Customer Name:- Leta Jirata

Request No:- GLD/RN/472/19

Report No:- GLD/TR/415/19

Sample type:- Rock & soil

Sample Preparation: - 200 Mesh

Date Submitted: - 10/07/2019

Number of Sample:- Two (2)

Analytical Result: In percent (%) Element to be determined Major Oxides & Minor Oxides

Analytical Method: $LiBO_2$ FUSION, HF attack, GRAVIMETERIC, COLORIMETRIC and AAS

Collector's code	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O	LOI
Rock-01	12.58	2.56	2.20	47.16	1.30	<0.01	0.32	0.16	0.07	0.09	6.79	26.80
Soil-02	64.30	15.76	6.50	1.12	1.62	0.82	1.10	<0.01	0.08	0.36	<0.01	6.95

Note: - This result represent only for the sample submitted to the laboratory.

Analysts: Yirgalem Abriham, Tihitna Beletkachew, Tizita Zemene, Yohanis Getachew, Bethelhem Tefera

Checked By: Dessie Abebe

Approved By: Gosa Haile

Quality Control: Negash Worku

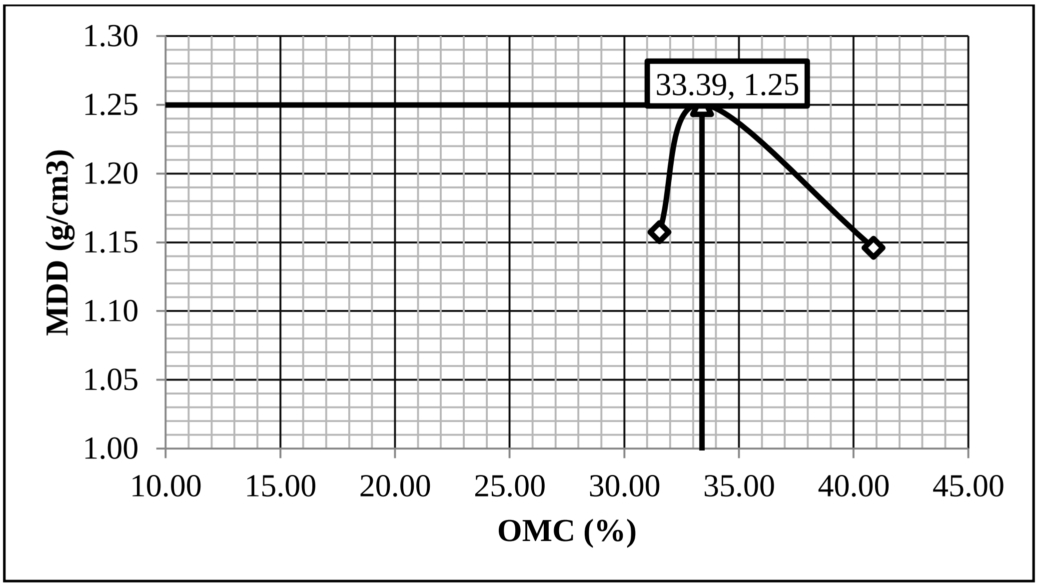
Geological Survey of Ethiopia Central Geological

Appendix A-2: Natural moisture content of the soil

Natural Moisture content of the soil		
Container Code.	D3	B1
Mass of Wet soil+Container(gm)(F)	101.5	96.56
Mass of dry soil+container(gm)(G)	76.2	74.56
Mass of container(gm)(H)	20.6	18.9
Mass of moisture(gm)F-G=(I)	25.3	22
Mass of Dry soil(gm)G-H=(J)	55.6	55.66
Moisture content % (I/J)*100=K	45.50	39.53
Average of moisture	42.51	

Appendix A-3: Compaction test result for soil

	Test No.	1	2	3
Sample soil	Mass of sample (gm)	4000	4000	4000
	Mass of sample (gm)	1800	1800	1800
	Water Added(cc)	300	380	460
	Mass of Mold+Wet soil(gm)(A)	3245.5	3389.6	3337.9
	Mass of Mold(gm)(B)	1808.2	1814.9	1813.9
	Mass of Wet Soil(gm)A-B=C	1437.3	1574.7	1524
	Volume of Mold cm^3(D)	944	944	944
	Container Code.	G63	D3	N3
	Mass of Wet soil+Container(gm)(F)	100.6	87.6	88.4
	Mass of dry soil+container(gm)(G)	81.9	68.9	67.8
	Mass of container(gm)(H)	22.6	12.9	17.4
	Mass of moisture(gm)F-G=(I)	18.7	18.7	20.6
	Mass of Dry soil(gm)G-H=(J)	59.3	56	50.4
	Moisture content % (I/J)*100=K	31.53	33.39	40.87
	Dry Density gm/cm^3 E/(100+K)*100	1.16	1.25	1.15

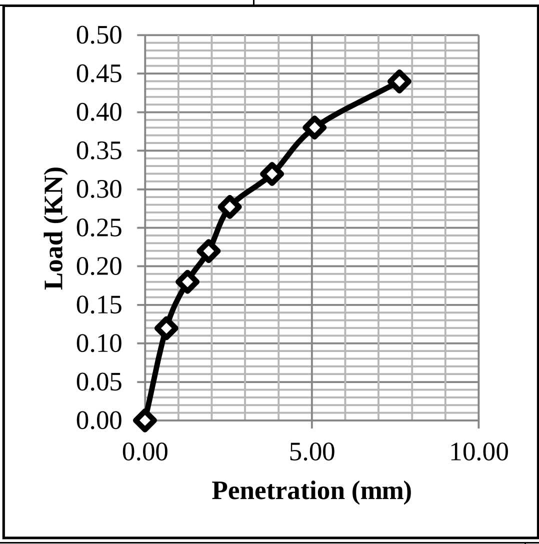


Appendix A-4: CBR test results for soil

Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		S-1-10	S-1-10
Mass of soil + Mould	g	10681.3	10719.1
Mass Mould	g	7118.2	7118.2
Mass of Soil	g	3563.1	3600.9
Volume of Mould	g	2124	2124
The wet density of soil	g/cc	1.678	1.695
The dry density of soil	g/cc	1.198	1.193

Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		65	65
Mass of wet soil + Container	138.36	187.51	187.51
Mass of dry soil + Container	109.48	143.04	143.04
Mass of container	37.42	37.39	37.39
Mass of water	28.9	44.5	44.5
Mass of dry soil	72.1	105.7	105.7
Moisture content	40.1	42.1	42.1
Max. Dry Density g/cc	1.045	OMC %	46.85

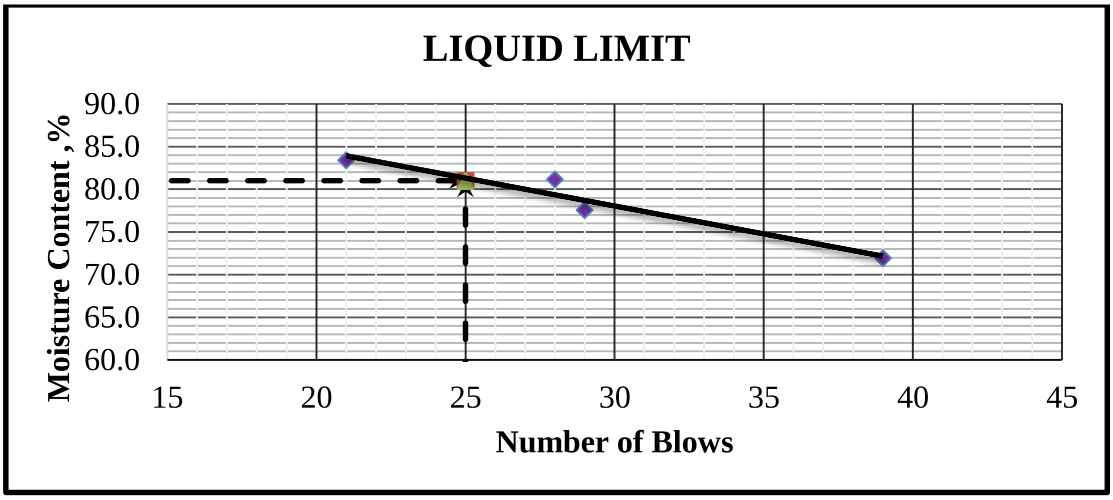
CBR Penetration Determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking Per.		
Pen. (mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.12	
1.27	0.18	
1.91	0.22	
2.54	0.28	2.08
3.81	0.32	
5.08	0.38	1.90
7.62	0.44	



Swell Determination				
Gauge rdg (mm)		Swell in %		
Initial	17.53			
Final	23.37	5.02		
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		0.3	2.0	
5.08mm		0.4	1.9	
Dry Density at 95% MDD:			1.045	
No. of blows	MCBS %	DBBS (g/cm3)	Corr. CBR %	% Compaction
56	40.1	1.198	2.0	100
CBR at 95% MDD	2.08	Swell	5.0	

Appendix A-5: Atterberg limits test result for soil

	Number of blows	39	29	28	21
Sample soil	Test	1	2	3	4
	Container	T8	C15	A20	B13
	Wt. of container + wet soil,g	23.64	37.23	31.9	30.2
	Wt. of container + dry soil,g	16.2	31.8	26.3	23.65
	Wt. of container,g	5.86	24.8	19.4	15.8
	Wt. of water,g	7.44	5.43	5.6	6.55
	Wt. of dry soil,g	10.34	7	6.9	7.85
	Moisture content,%	72.0	77.6	81.2	83.44
	Liquid limit	81			



Plastic Limit(U) pit 1		
Trial	1	2
Container	A4	C14
Wt. of container + wet soil,g	24.01	26.07
Wt. of container + dry soil,g	22.8	25.6
Wt. of a container,g	19.8	22.9
Wt. of water,g	1.2	0.47
Wt. of dry soil,g	3	2.7
Moisture content,%	40.3	17.4
Plastic limit	28.9	
Plasticity Index (PI)	52.1	

Appendix A-6: Specific gravity test result for sample soil

	Pycnometer No.	C1	C2	C3
Sample soil	Weight of dry, clean pycnometer, w_p (g)WP	32.1	31.3	30.9
	Weight of pycnometer + water, w_{pw} (g)	128.4	129.4	127.8
	The observed temperature of the water, T_i (OC)	20	20	20
	Determination No.	1	2	3
	Pycnometer No.	C1	C2	C3
	Weight of pycnometer + soil + water, W_{pws} (g) WPWS	143.5	143.8	143.6
	Temperature, T_x(°c)	21	21	21
	Weight of pycnometer + water at T_x ,W_{pw}(atT_x) (g)	127.90	127.90	127.68
	Weight of dry soil, w_e (gm)	25	25	25
	Conversion factor , K	0.9998	0.9998	0.9998
	The specific gravity of soil at 20°c.	2.66	2.69	2.7
	The average specific gravity of soil	**2.68**		

Appendix A-7: Sieve and hydrometer analysis for the soil

	Sieve size (mm)	Mass of retain on each sieve (g)	% retained	Cumm. % retained	% Finer
Sample soil	9.5	0.00	0.00	0.00	100.00
	4.75	0.26	0.03	0.03	99.97
	2	6.57	0.67	0.70	99.30
	0.85	7.08	0.72	1.42	98.58
	0.425	7.86	0.80	2.22	97.78
	0.3	8.22	0.84	3.06	96.94
	0.15	17.62	1.80	4.87	95.13
	0.075	12.98	1.33	6.19	93.81

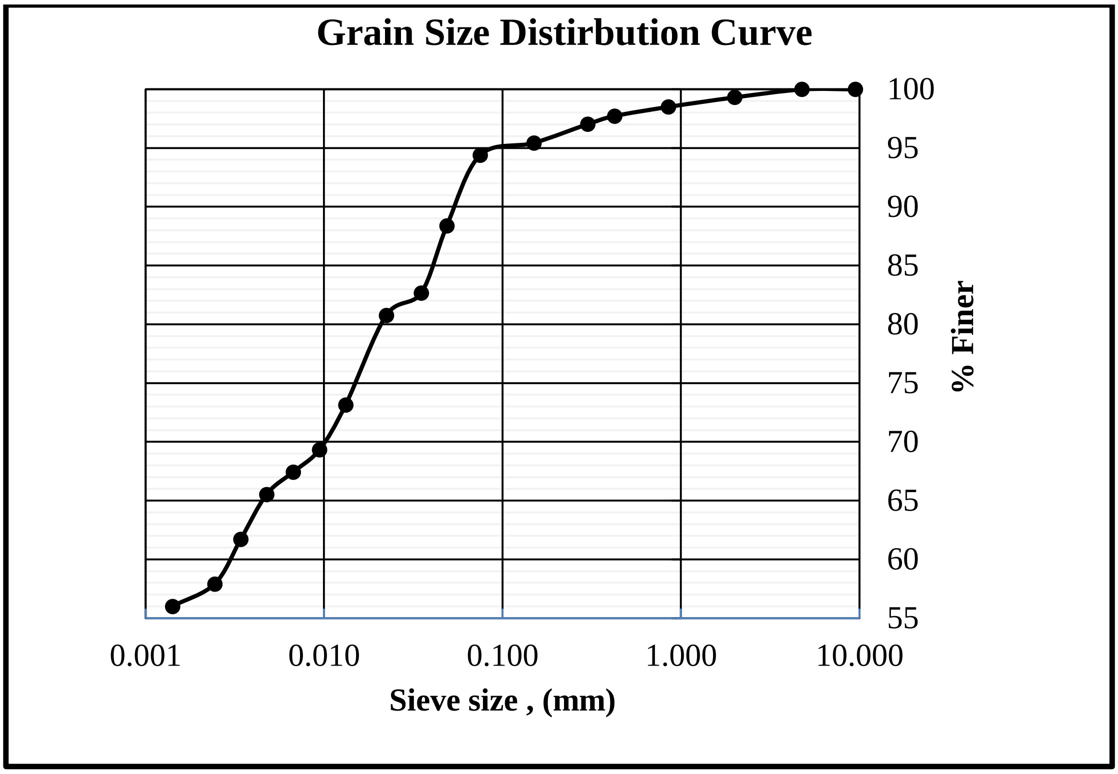
Hydrometer Analysis of ASTM 152-H

Gs= 2.69		
dry weight of soil, Ws= 50g	temperature of test=21	
Meniscus correction, Fm =1	Zero Correction, Fz =6	Temp. corr.= -4.85+0.25T
Tested by Leta		

Time min	T m p.	R= H. rdng	corr. For temp.	corr. H. reading	a= values	% finer in susp. p=(Ra/w)*100	corr. Hcl (H+Fm)	corr. L.(cm)	K	Diameter (D)	% finer
1	21	50	0.4	44.4	1	89.688	51	13.2	0.013585	0.049	84.32
2	21	47	0.4	41.4	1	83.628	48	13.75	0.013585	0.036	78.63
5	21	45	0.4	39.4	1	79.588	46	13.8	0.013585	0.023	74.83
15	21	44	0.4	38.4	1	77.568	45	14.2	0.013585	0.013	72.93
30	21	43	0.4	37.4	1	75.548	44	14.3	0.013585	0.009	71.03
60	21	41	0.4	35.4	1	71.508	42	14.7	0.013585	0.007	67.23
120	21	40	0.4	34.4	1	69.488	41	14.8	0.013585	0.005	65.33
240	21	39	0.4	33.4	1	67.468	40	15	0.013585	0.003	63.43
480	21	37	0.4	31.4	1	63.428	38	15.3	0.013585	0.002	28.01
1440	21	35	0.4	29.4	1	59.388	36	15.6	0.013585	0.001	67.0

Hydrometer analysis

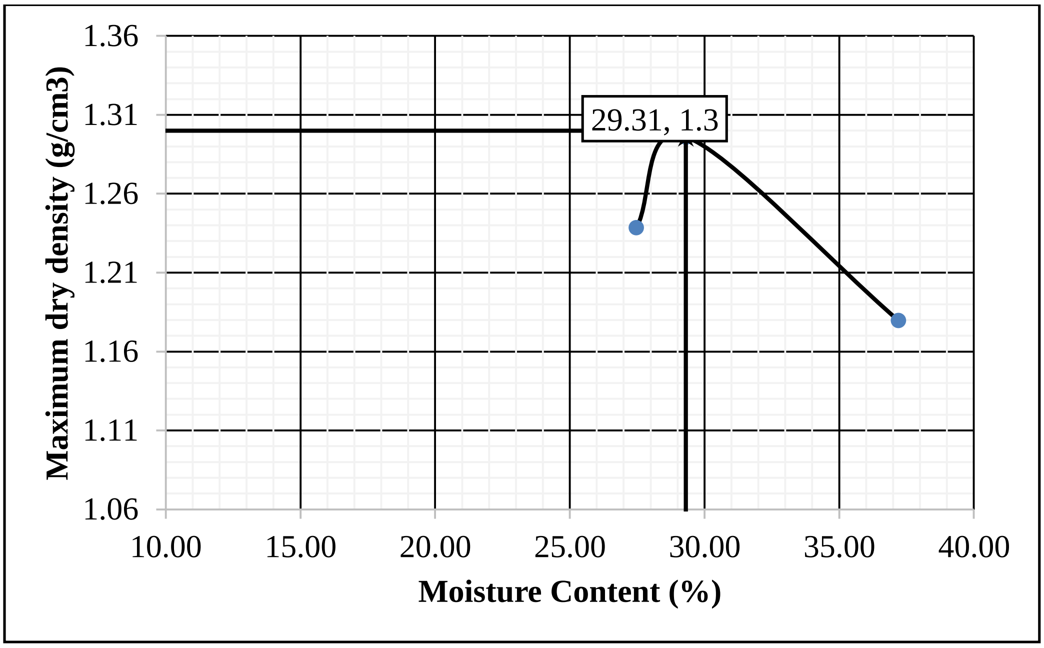
Opening (mm)	Percent of passing
0.075	95.001
0.002	28.001
0.001	67.0



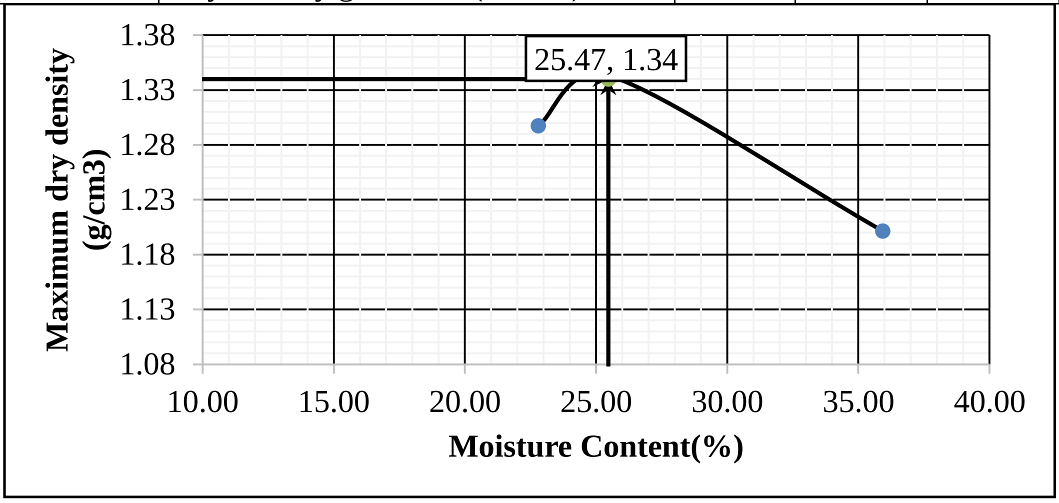
Laboratory Test for Mix Design

Appendix B-1: Compaction test soils with waste ceramic dust

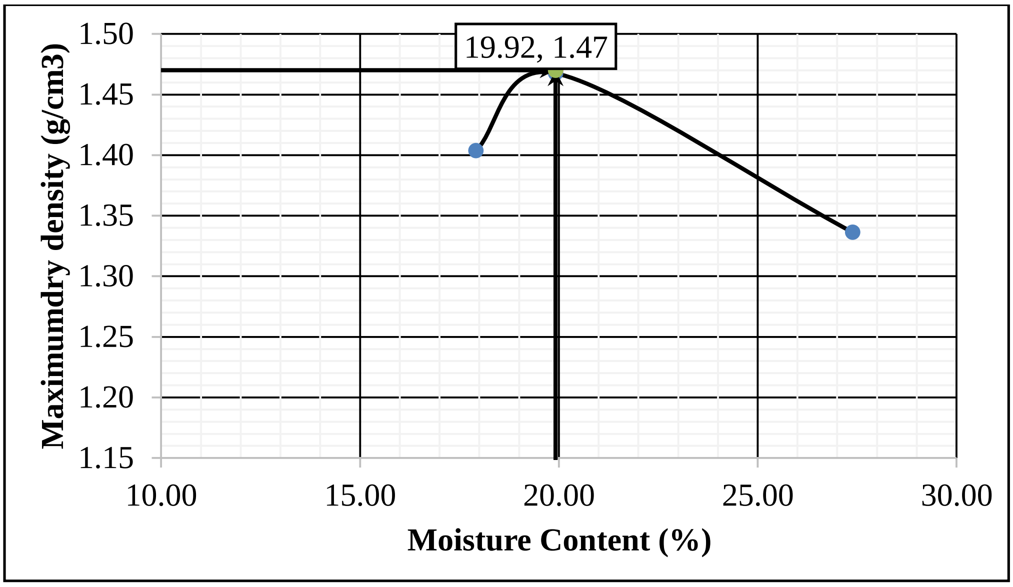
	Test No.	1	2	3
5% WCD	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	200	280	360
	Mass of Mold+Wet soil(gm)(A)	3296.5	3392.5	3339.4
	Mass of Mold(gm)(B)	1806.1	1810.4	1811.5
	Mass of Wet Soil(gm)A-B=C	1490.4	1582.1	1527.9
	Volume of Mold cm^3(D)	944	944	944
	Bulk Density gm/cm^3 C/D=(E)	1.58	1.68	1.62
	Container Code.	L1	L2	L3
	Mass of Wet soil+Container(gm)(F)	93.1	101.3	91.2
	Mass of dry soil+container(gm)(G)	75.6	80.9	69.4
	Mass of container(gm)(H)	11.9	11.3	10.8
	Mass of moisture(gm)F-G=(I)	17.5	20.4	21.8
	Mass of Dry soil(gm)G-H=(J)	63.7	69.6	58.6
	Moisture content % (I/J)*100=K	27.47	29.31	37.20
	Dry Density gm/cm^3 E/(100+K)*100	1.24	1.30	1.18



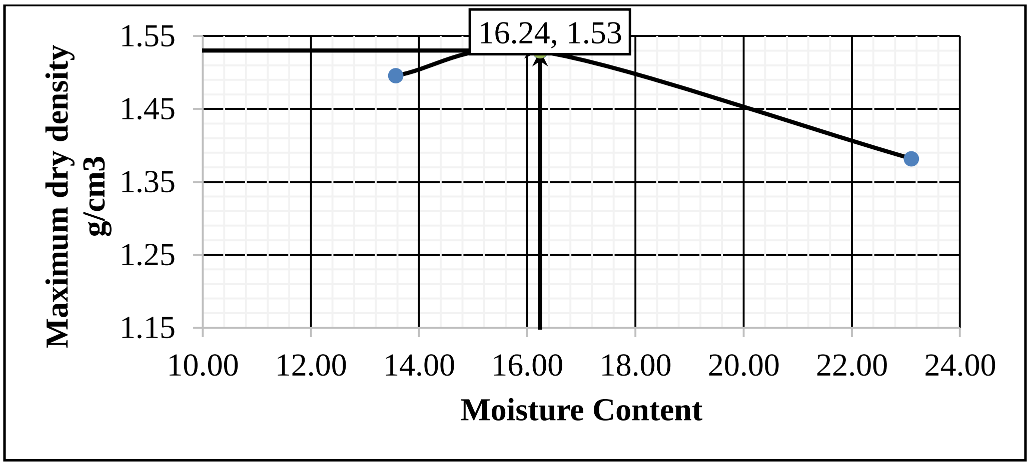
	Test No.	1	2	3
10% WCD	Test No.	1	2	3
	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	240	320	400
	Mass of Mold+Wet soil(gm)(A)	3316.9	3397.6	3356.3
	Mass of Mold(gm)(B)	1812.9	1806.3	1814.6
	Mass of Wet Soil(gm)A-B=C	1504	1591.3	1541.7
	Volume of Mold cm^3(D)	944	944	944
	Container Code.	A1	G2	G3
	Mass of Wet soil+Container(gm)(F)	110	123.1	92.8
	Mass of dry soil+container(gm)(G)	93.4	100.3	69.8
	Mass of container(gm)(H)	20.6	10.8	5.8
	Mass of moisture(gm)F-G=(I)	16.6	22.8	23
	Mass of Dry soil(gm)G-H=(J)	72.8	89.5	64
	Moisture content % (I/J)*100=K	22.80	25.47	35.94
	Dry Density gm/cm^3 E/(100+K)*100	1.30	1.34	1.20



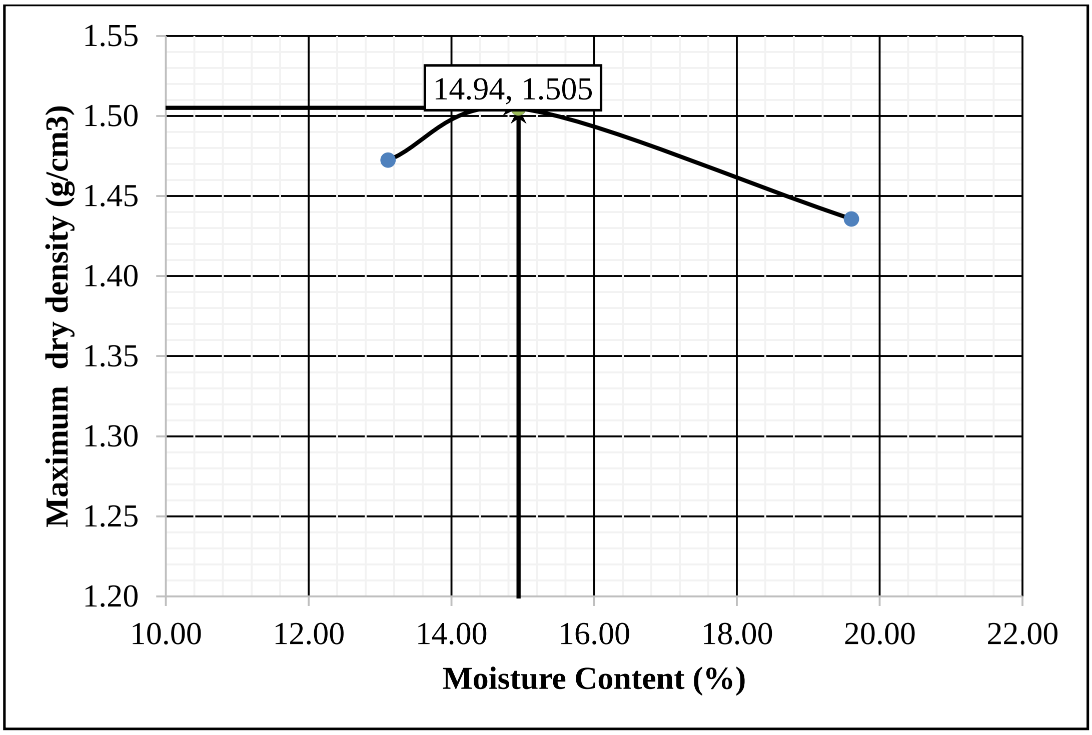
	Test No.	1	2	3
15% WCD	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	250	330	410
	Mass of Mold+Wet soil(gm)(A)	3377.9	3475.9	3415.6
	Mass of Mold(gm)(B)	1815.6	1814.6	1808.7
	Mass of Wet Soil(gm)A-B=C	1562.3	1661.3	1606.9
	Volume of Mold cm^3(D)	944	944	944
	Bulk Density gm/cm^3 C/D=(E)	1.65	1.76	1.70
	Container Code.	D1	B1	B2
	Mass of Wet soil+Container(gm)(F)	120.5	105.3	99.6
	Mass of dry soil+container(gm)(G)	103.5	89.6	78.6
	Mass of container(gm)(H)	8.6	10.8	10.7
	Mass of moisture(gm)F-G=(I)	17	15.7	18.6
	Mass of Dry soil(gm)G-H=(J)	94.9	78.8	67.9
	Moisture content % (I/J)*100=K	17.91	19.92	27.39
	Dry Density gm/cm^3 E/(100+K)*100	1.40	1.47	1.34



	Test No.	1	2	3
20% WCD	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	300	380	460
	Mass of Mold+Wet soil(gm)(A)	3420.4	3503.6	3415.6
	Mass of Mold(gm)(B)	1816.8	1825.6	1809.7
	Mass of Wet Soil(gm)A-B=C	1603.6	1678	1605.9
	Volume of Mold cm^3(D)	944	944	944
	Bulk Density gm/cm^3 C/D=(E)	1.70	1.78	1.70
	Container Code.	T1	T2	T3
	Mass of Wet soil+Container(gm)(F)	101.3	82.9	88.1
	Mass of dry soil+container(gm)(G)	91.3	72.1	73.8
	Mass of container(gm)(H)	17.6	5.6	11.9
	Mass of moisture(gm)F-G=(I)	10	10.8	14.3
	Mass of Dry soil(gm)G-H=(J)	73.7	66.5	61.9
	Moisture content % (I/J)*100=K	13.57	16.24	23.10
	Dry Density gm/cm^3 E/(100+K)*100	1.50	1.53	1.38



	Test No.	1	2	3
25% WCD	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	320	400	480
	Mass of Mold+Wet soil(gm)(A)	3389.6	3445.2	3435.6
	Mass of Mold(gm)(B)	1817.4	1812.4	1814.6
	Mass of Wet Soil(gm)A-B=C	1572.2	1632.8	1621
	Volume of Mold cm^3(D)	944	944	944
	Bulk Density gm/cm^3 C/D=(E)	1.67	1.73	1.72
	Container Code.	D1	D2	D3
	Mass of Wet soil+Container(gm)(F)	114.3	101	102.5
	Mass of dry soil+container(gm)(G)	103.9	89.9	88.6
	Mass of container(gm)(H)	24.6	15.6	17.7
	Mass of moisture(gm)F-G=(I)	10.4	11.1	13.9
	Mass of Dry soil(gm)G-H=(J)	79.3	74.3	70.9
	Moisture content % (I/J)*100=K	13.11	14.94	19.61
	Dry Density gm/cm^3 E/(100+K)*100	1.47	1.505	1.44



Appendix B-2: Stabilization of WCD with soil CBR values

5% WCD of CBR

Compaction Determination @ 56 blows

		Before soak	After soak
Mould No.		N 13	N 13
Mass of soil + Mould	g	10556.9	11044.1
Mass Mould	g	7143.6	7143.6
Mass of Soil	g	3413.3	3900.5
Volume of Mould	g	2124	2124
The wet density of soil	g/cc	1.607	1.836
The dry density of soil	g/cc	1.193	1.239

Moisture content determination data @ 56 blows

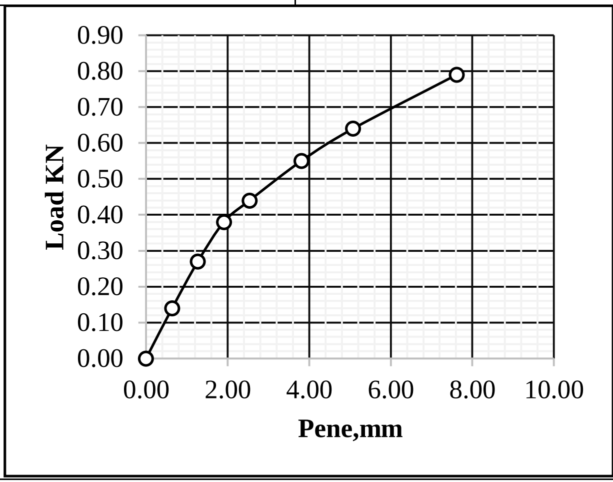
		Before soak	After soak
Container no.		E	T1
Mass of wet soil + Container	124.13	161.20	161.20
Mass of dry soil + Container	101.92	120.94	120.94
Mass of container	37.87	37.53	37.53
Mass of water	22.2	40.3	40.3
Mass of dry soil	64.1	83.4	83.4
Moisture content	34.7	48.3	48.3
Max. Dry Density g/cc	1.30	OMC %	29.31

CBR Penetration Determination

Surcharge Weight:-4.55 KG

Penetration after 96 hrs. Soaking Period

Pen. (mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.14	
1.27	0.27	
1.91	0.38	
2.54	0.44	3.30
3.81	0.55	
5.08	0.64	3.20
7.62	0.79	



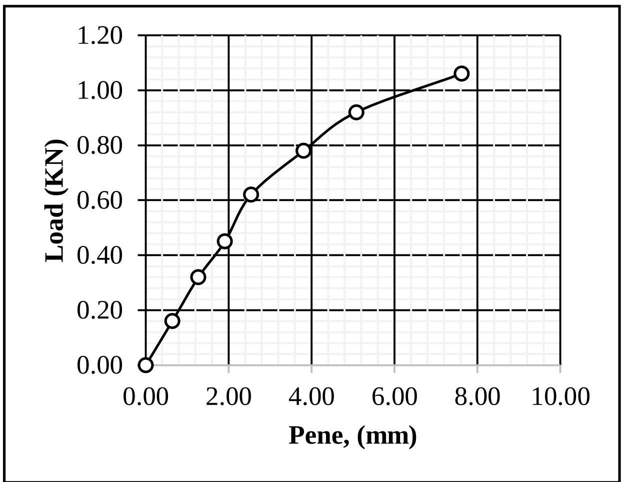
Swell Determination

56 Blows

Gauge rdg (mm)		Swell in %		D D @95% MDD
Initial	12.46			1.24
Final	18.00	4.76		
	Load KN			
Penetration(mm)	Top	Bottom	Corr. CBR %	
2.54mm		0.4	3.3	
5.08mm		0.6	3.2	
No. of blows	MCBS %	DBBS (g/cm3)	Corr. CBR %	% Compaction
56	34.68	1.14	3.3	95.01
CBR @ 95% MDD	3.30	swell	4.76	

10% WCD of CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		N14	N14
Mass of soil + Mould	g	10131.2	11095.2
Mass Mould	g	7189.6	7189.6
Mass of Soil	g	2941.6	3905.6
Volume of Mould	g	2124	2124
The wet density of soil	g/cc	1.385	1.839
The dry density of soil	g/cc	1.151	1.455
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		C1T1	T2C2
Mass of wet soil + Container		144.60	175.33
Mass of dry soil + Container	g	126.40	146.50
Mass of container	g	36.80	37.20
Mass of water	g	18.2	28.8
Mass of dry soil	g	89.6	109.3
Moisture content	%	20.3	26.4
Max. Dry Density g/cc	**1.34**	**OMC %**	**25.47**

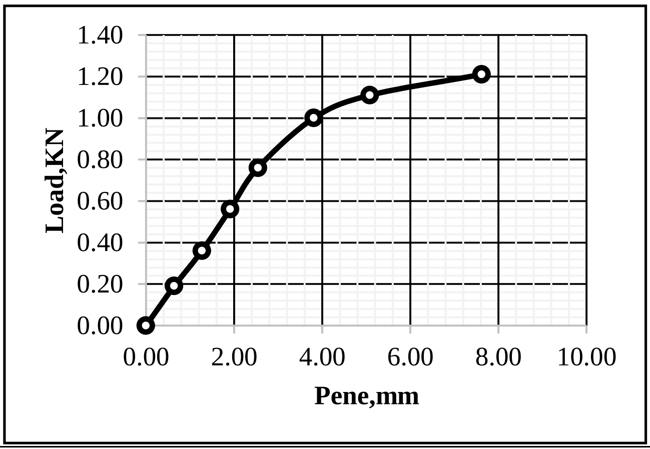
CBR Penetration Determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking Period		
Pen. (mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.16	
1.27	0.32	
1.91	0.45	
2.54	0.62	4.65
3.81	0.78	
5.08	0.92	4.60
7.62	1.06	



Swell Determination				
Gauge rdg (mm)		Swell in %		
Initial	16.20			
Final	20.80	3.95		
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		0.6	4.7	
5.08mm		0.9	4.6	
Dry Density at 95% MDD:			1.27	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	20.3	1.2	4.7	95
CBR at 95% MDD	4.65	Swell in %	3.95	

15% WCD of CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		N1	N1
Mass of soil + Mould	g	10232.6	11156.8
Mass Mould	g	7110.6	7110.6
Mass of Soil	g	3122	4046.2
Volume of Mould	g	2124	2124
The wet density of soil	g/cc	1.470	1.905
The dry density of soil	g/cc	1.221	1.515
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		G1	A2
Mass of wet soil + Container		164.30	182.60
Mass of dry soil + Container	g	142.40	152.50
Mass of container	g	34.80	35.60
Mass of water	g	21.9	30.1
Mass of dry soil	g	107.6	116.9
Moisture content	%	20.4	25.7
Max. Dry Density g/cc	1.45	OMC %	21.19

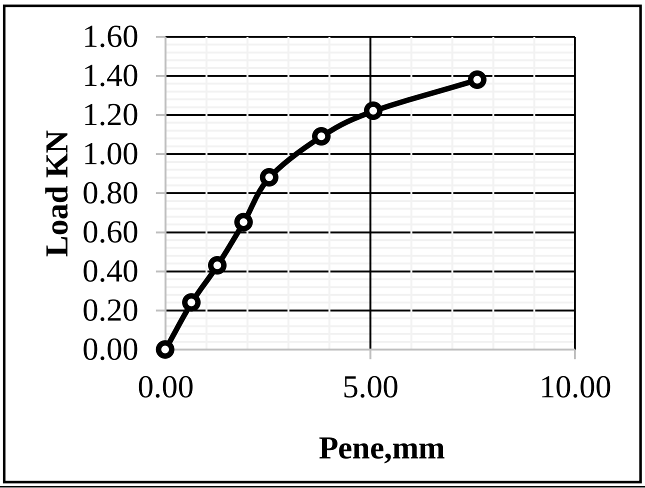
CBR Penetration determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking Period		
Pen. (mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.19	
1.27	0.36	
1.91	0.56	
2.54	0.76	5.70
3.81	1.00	
5.08	1.11	5.55
7.62	1.21	



Swell Determination				
56 Blows				
Gauge rdg (mm)		Swell in %		
Initial	19.2	3.09		
Final	22.8			
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		0.76	5.7	
5.08mm		1.11	5.6	
Dry Density at 95% MDD:			1.38	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	20.4	1.221	5.7	95
CBR at 95% MDD	5.7	Swell	3.09	

20% WCD of CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		10311.2	11196.5
Mass of soil + Mould	g	7196.3	7196.3
Mass Mould	g	3114.9	4000.2
Mass of Soil	g	2124	2124
Volume of Mould	g	1.467	1.883
The wet density of soil	g/cc	1.268	1.489
The dry density of soil	g/cc	10311.2	11196.5
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		A1	A2
Mass of wet soil + Container		167.60	193.60
Mass of dry soil + Container	g	149.50	160.50
Mass of container	g	34.2	35.4
Mass of water	g	18.1	33.1
Mass of dry soil	g	115.3	125.1
Moisture content	%	15.7	26.5
Max. Dry Density g/cc	1.47	OMC %	18.55

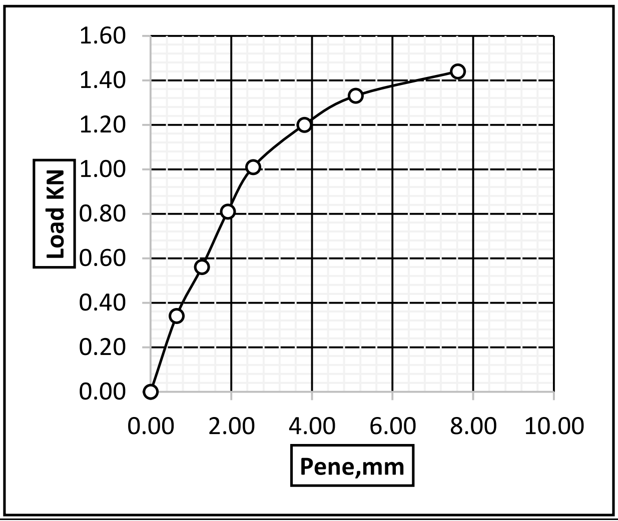
CBR Penetration determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking Period		
Pen. (mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.24	
1.27	0.43	
1.91	0.65	
2.54	0.81	6.6
3.81	1.09	
5.08	1.12	5.60
7.62	1.38	



Swell Determination				
56 Blows				
Gauge rdg (mm)		Swell in %		
Initial	18.50	2.30		
Final	21.3			
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		0.9	6.6	
5.08mm		1.12	5.6	
Dry Density at 95% MDD:			1.40	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	15.70	1.27	6.62	95
CBR at 95% MDD	6.6	Swell	2.30	

25% of CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		10723.7	11263.5
Mass of soil + Mould	g	6903.6	6903.6
Mass Mould	g	3820.1	4359.9
Mass of Soil	g	2124	2124
Volume of Mould	g	1.799	2.053
The wet density of soil	g/cc	1.319	1.411
The dry density of soil	g/cc	10723.7	11263.5
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		G 19	C3T2
Mass of wet soil + Container		140.48	195.86
Mass of dry soil + Container	g	105.90	146.42
Mass of container	g	10.9	37.8
Mass of water	g	34.6	49.4
Mass of dry soil	g	95.0	108.6
Moisture content	%	36.4	45.5
Max. Dry Density g/cc	1.54	OMC %	15.57

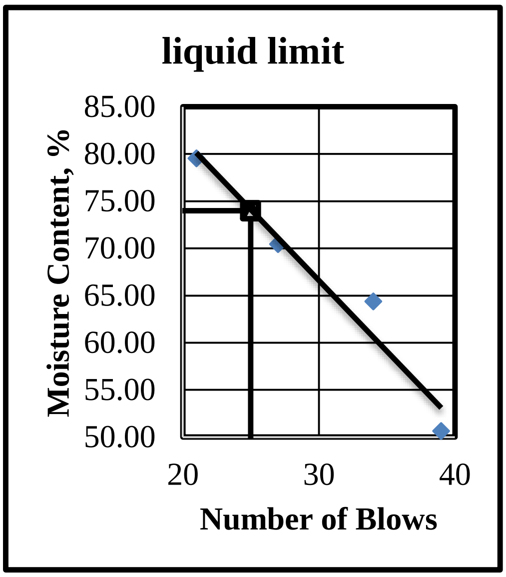
CBR Penetration determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking Period		
Pen. (mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.26	
1.27	0.48	
1.91	0.64	
2.54	0.82	6.15
3.81	1.08	
5.08	1.18	5.9
7.62	1.32	



Swell Determination				
Gauge rdg (mm)		Swell in %		
Initial	19.22	1.90		
Final	21.2			
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		1.0	6.2	
5.08mm		1.3	5.9	
Dry Density at 95% MDD:			1.46	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	36.40	1.32	7.59	95.01
CBR at 95%	6.15	Swell	1.9	

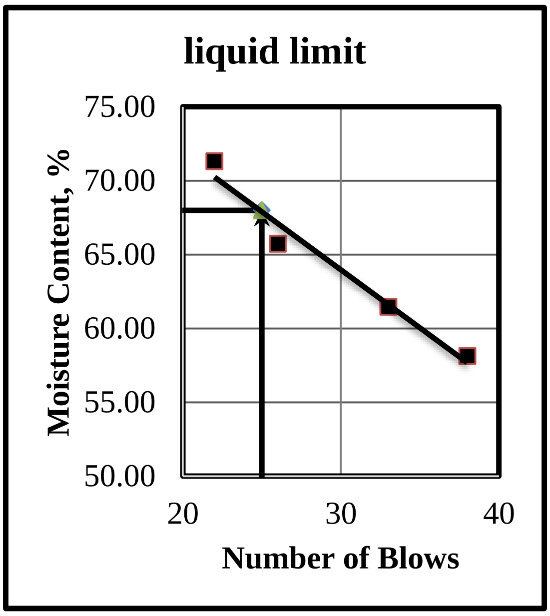
Appendix B-3: Stabilization of WCD with soil atterberg limits tests

	Liquid limit				
5% WCD	Number of blows	21	27	34	39
	Test	1	2	3	4
	Container	L1	L2	L3	L4
	Wt. of container + wet soil,g	25.1	23.4	22.3	24.6
	Wt. of container + dry soil,g	16.55	16.03	15.76	18.9
	Wt. of container,g	5.8	5.57	5.6	7.64
	Wt. of water,g	8.55	7.37	6.54	5.7
	Wt. of dry soil,g	10.75	10.46	10.16	11.26
	Moisture content,%	79.53	70.46	64.37	50.62
	LL	74			



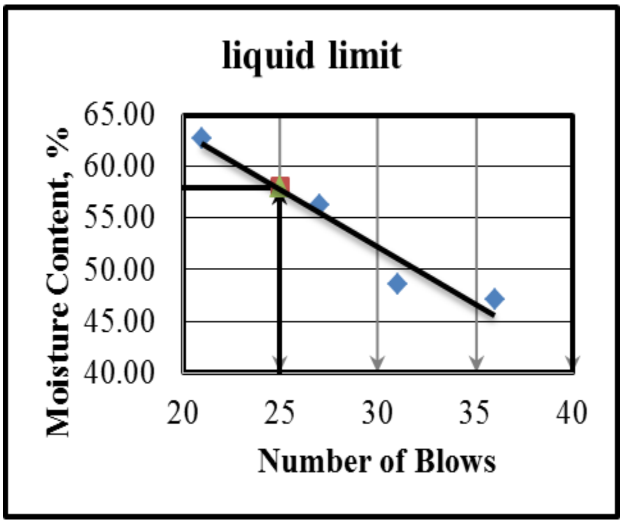
Plastic limit		
Trial	1	2
Container	P2	21
Wt. of container + wet soil,g	10.4	11.12
Wt. of container + dry soil,g	9.67	10.44
Wt. of container,g	6.6	8.2
Wt. of water,g	0.7	0.68
Wt. of dry soil,g	3.07	2.24
Moisture content,%	23.8	30.4
Average moisture Content(%)	**27.1**	
Plasticity Index (PI)	**46.9**	

	liqud limit				
10% WCD	Number of blows	22	26	33	38
	Test	1	2	3	4
	Container	SP1	SP2	SP3	A1
	Wt. of container + wet soil,g	25.4	30.56	35.7	29.6
	Wt. of container + dry soil,g	17.99	23.12	24.63	25.63
	Wt. of container,g	7.6	11.8	6.62	18.8
	Wt. of water,g	7.41	7.44	11.07	3.97
	Wt. of dry soil,g	10.39	11.32	18.01	6.83
	Moisture content,%	71.32	65.72	61.47	58.13
	LL	68			



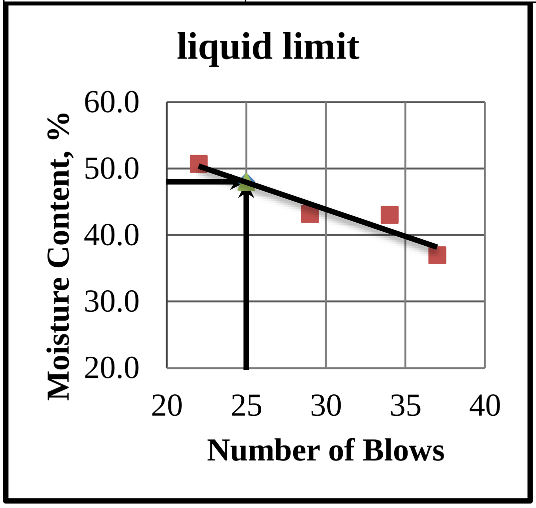
Plastic limit		
Trial	1	2
Container	P9	P10
Wt. of container + wet soil,g	11.2	12.2
Wt. of container + dry soil,g	10.57	11.62
Wt. of container,g	8.4	9.1
Wt. of water,g	0.6	0.58
Wt. of dry soil,g	2.17	2.52
Moisture content,%	29.0	23.0
Average moisture Content(%)	**26.0**	
Plasticity Index (PI)	**42.0**	

15% WCD	Liquid limit				
	Number of blows	21	27	31	36
	Test	1	2	3	4
	Container	E1	E2	E3	A2E2
	Wt. of container + wet soil,g	36.8	37.9	34.8	32.9
	Wt. of container + dry soil,g	32.98	32.72	30.22	28.42
	Wt. of container,g	26.89	23.5	20.8	18.9
	Wt. of water,g	3.82	5.18	4.58	4.48
	Wt. of dry soil,g	6.09	9.22	9.42	9.52
	Moisture content,%	62.73	56.18	48.62	47.06
	LL	58			



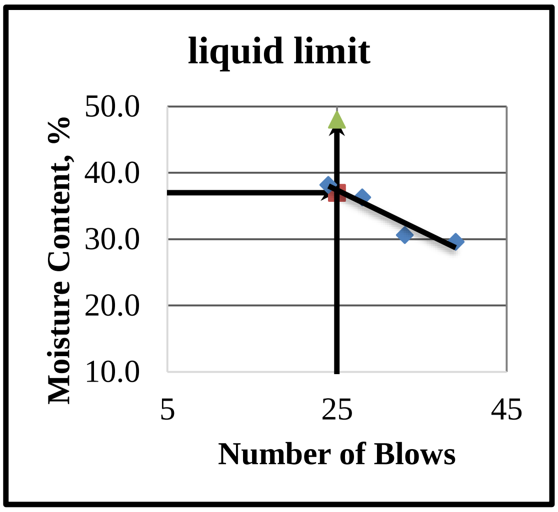
Plastic limit		
Trial	1	2
Container	H1	H2
Wt. of container + wet soil,g	11.7	19.6
Wt. of container + dry soil,g	11.2	19.42
Wt. of container,g	9.9	15.6
Wt. of water,g	0.5	0.18
Wt. of dry soil,g	1.3	3.82
Moisture content,%	38.5	4.7
Average moisture Content(%)	**21.6**	
Plasticity Index (PI)	**36.4**	

	liquid limit				
20% WCD	Number of blows	22	29	34	37
	Trial	1	2	3	4
	Container	G1	H2	A4	G20
	Wt. of container + wet soil,g	36.7	31.1	27.5	28.6
	Wt. of container + dry soil,g	32.63	27.39	23.92	25.66
	Wt. of container,g	24.6	18.8	15.6	17.7
	Wt. of water,g	4.07	3.71	3.58	2.94
	Wt. of dry soil,g	8.03	8.59	8.32	7.96
	Moisture content,%	50.7	43.2	43.0	36.9
	LL	48			



Plastic Limit		
Trial	1	2
Container	K1	K2
Wt. of container + wet soil,g	29	30.1
Wt. of container + dry soil,g	25.5	28.1
Wt. of container,g	8.1	13.4
Wt. of water,g	3.5	2
Wt. of dry soil,g	17.4	14.7
Moisture content,%	20.1	13.6
Average moisture Content(%)	**16.9**	
Plasticity Index (PI)	**29.1**	

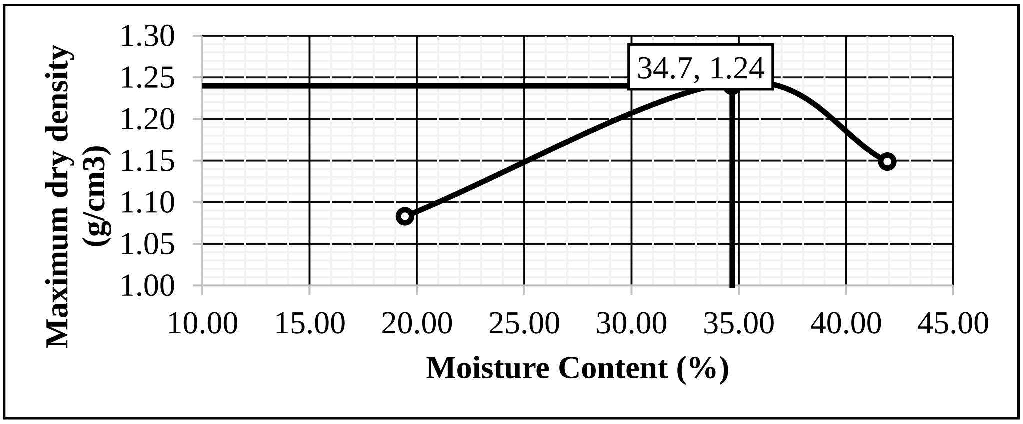
	liquid limit				
25% WCD	Number of blows	24	28	33	39
	Trial	1	2	3	4
	Container	G3	G2	C14	G1
	Wt. of container + wet soil,g	19.2	19.6	19.2	22.9
	Wt. of container + dry soil,g	15.3	15.9	16.1	18.9
	Wt. of container,g	5.87	5.71	5.97	5.4
	Wt. of water,g	3.9	3.7	3.1	4
	Wt. of dry soil,g	9.43	10.19	10.13	13.5
	Moisture content,%	41.4	36.3	30.6	29.6
	LL	36			



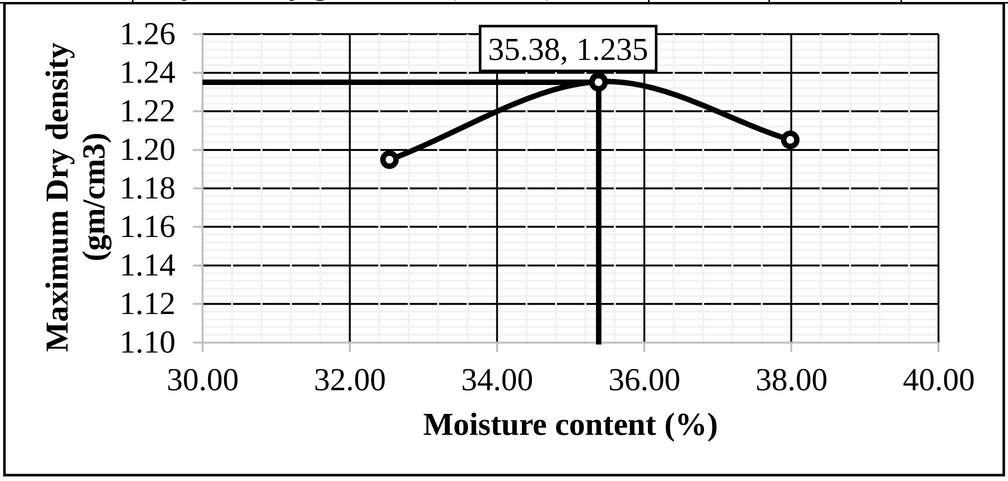
Plastic Limit		
Trial	1	2
Container	G14	G12
Wt. of container + wet soil,g	25.1	24.9
Wt. of container + dry soil,g	24.3	23.79
Wt. of container,g	5.6	17.6
Wt. of water,g	0.8	1.11
Wt. of dry soil,g	18.7	6.19
Moisture content,%	4.3	17.9
Average moisture Content(%)	**11.1**	
Plasticity Index (PI)	**23.4**	

Appendix B-4: Compaction test of limestone with soil

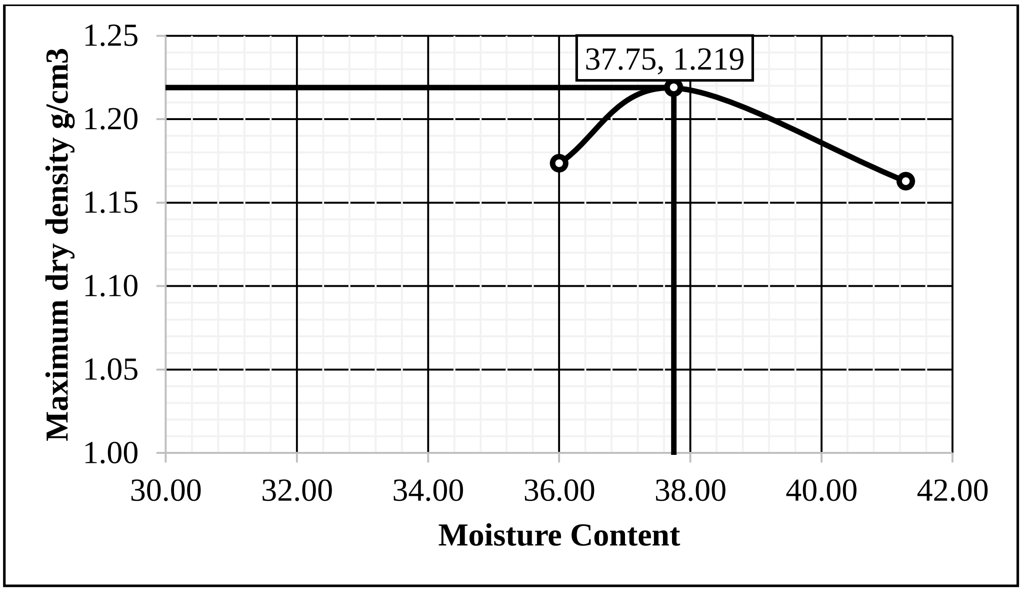
	Test No.	1	2	3
2% limestone	Test No.	1	2	3
	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	280	360	440
	Mass of Mold+Wet soil(gm)(A)	3326.9	3387.9	3356.2
	Mass of Mold(gm)(B)	1808.9	1806.3	1817.3
	Mass of Wet Soil(gm)A-B=C	1518	1581.6	1538.9
	Volume of Mold cm^3(D)	944	944	944
	Container Code .	T1	T2	T3
	Mass of Wet soil+Container(gm)(F)	102.2	124.1	110.8
	Mass of dry soil+container(gm)(G)	86.5	96.1	83.5
	Mass of container(gm)(H)	5.8	15.4	18.4
	Mass of moisture(gm)F-G=(I)	15.7	28	27.3
	Mass of Dry soil(gm)G-H=(J)	80.7	80.7	65.1
	Moisture content % (I/J)*100=K	19.45	34.70	41.94
	Dry Density gm/cm^3 E/(100+K)*100	1.08	1.24	1.15



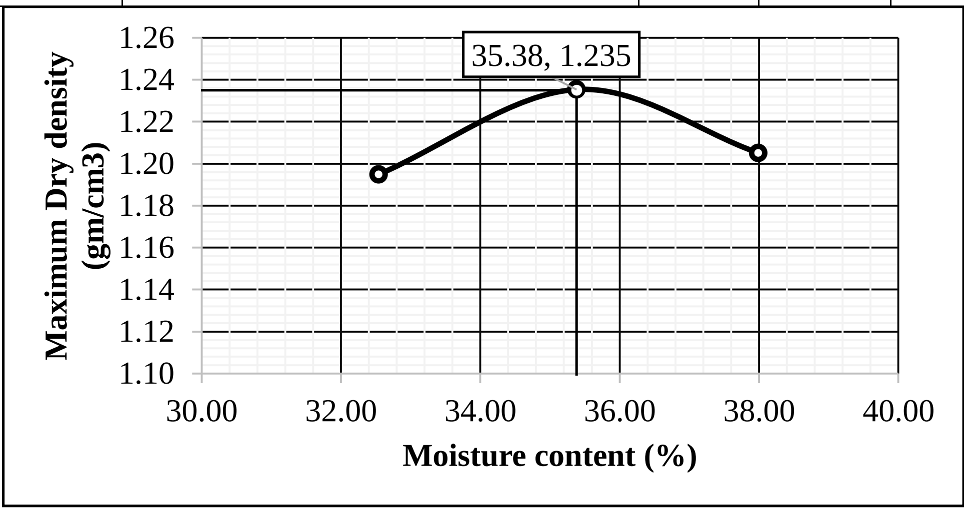
	Test No.	1	2	3
4% limestone	Test No.	1	2	3
	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	340	420	500
	Mass of Mold+Wet soil(gm)(A)	3315.6	3398.6	3389.6
	Mass of Mold(gm)(B)	1820.7	1819.8	1819.8
	Mass of Wet Soil(gm)A-B=C	1494.9	1578.8	1569.8
	Volume of Mold cm^3(D)	944	944	944
	Container Code .	N1	P62	A4
	Mass of Wet soil+Container(gm)(F)	179.8	184.2	189.8
	Mass of dry soil+container(gm)(G)	145.6	145.6	147.4
	Mass of container(gm)(H)	40.5	36.5	35.79
	Mass of moisture(gm)F-G=(I)	34.2	38.6	42.4
	Mass of Dry soil(gm)G-H=(J)	105.1	109.1	111.61
	Moisture content % (I/J)*100=K	32.54	35.38	37.99
	Dry Density gm/cm^3 E/(100+K)*100	1.19	1.235	1.21



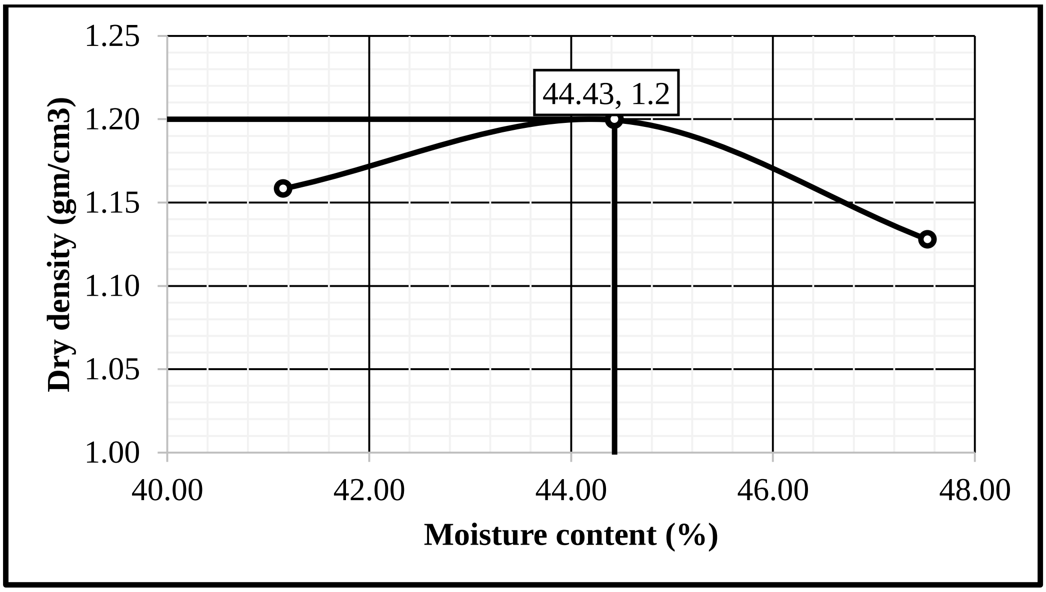
	Test No.	1	2	3
6% limestone	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	320	400	480
	Mass of Mold+Wet soil(gm)(A)	3326.3	3403.6	3366.9
	Mass of Mold(gm)(B)	1819.8	1818.9	1816.2
	Mass of Wet Soil(gm)A-B=C	1506.5	1584.7	1550.7
	Volume of Mold cm^3(D)	944	944	944
	Bulk Density gm/cm^3 C/D=(E)	1.60	1.68	1.64
	Container Code .	A1T1	T2C2	T3C3
	Mass of Wet soil+Container(gm)(F)	103.6	113.5	125.4
	Mass of dry soil+container(gm)(G)	85.6	90.4	97.2
	Mass of container(gm)(H)	35.6	29.2	28.9
	Mass of moisture(gm)F-G=(I)	18	23.1	28.2
	Mass of Dry soil(gm)G-H=(J)	50	61.2	68.3
	Moisture content % (I/J)*100=K	36.00	37.75	41.29
	Dry Density gm/cm^3 E/(100+K)*100	1.17	1.219	1.16



	Test No.	1	2	3
8% limestone	Test No.	1	2	3
	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	340	420	500
	Mass of Mold+Wet soil(gm)(A)	3315.6	3398.6	3389.6
	Mass of Mold(gm)(B)	1820.7	1819.8	1819.8
	Mass of Wet Soil(gm)A-B=C	1494.9	1578.8	1569.8
	Volume of Mold cm^3(D)	944	944	944
	Container Code .	N1	P62	A4
	Mass of Wet soil+Container(gm)(F)	179.8	184.2	189.8
	Mass of dry soil+container(gm)(G)	145.6	145.6	147.4
	Mass of container(gm)(H)	40.5	36.5	35.79
	Mass of moisture(gm)F-G=(I)	34.2	38.6	42.4
	Mass of Dry soil(gm)G-H=(J)	105.1	109.1	111.61
	Moisture content % (I/J)*100=K	32.54	35.38	37.99
	Dry Density gm/cm^3 E/(100+K)*100	1.19	1.235	1.21



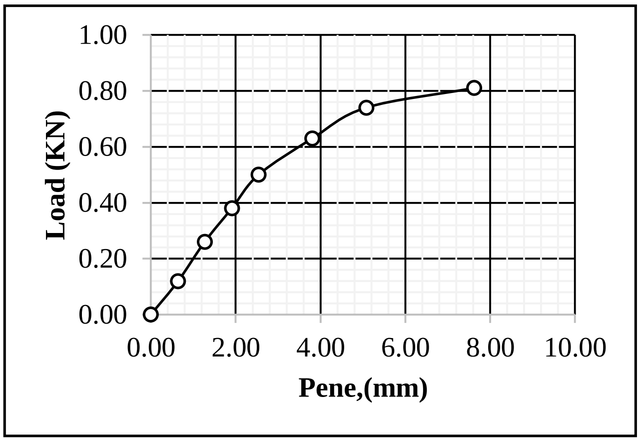
	Test No.	1	2	3
10% limestone	Test No.	1	2	3
	Mass of sample (gm)	1800	1800	1800
	Water Added(cc)	300	380	460
	Mass of Mold+Wet soil(gm)(A)	3362.3	3445.6	3386.2
	Mass of Mold(gm)(B)	1818.9	1810.1	1815.6
	Mass of Wet Soil(gm)A-B=C	1543.4	1635.5	1570.6
	Volume of Mold cm^3(D)	944	944	944
	Container Code .	B1	B12	B3
	Mass of Wet soil+Container(gm)(F)	136.6	135.8	134.4
	Mass of dry soil+container(gm)(G)	108.7	106.3	103.6
	Mass of container(gm)(H)	40.9	39.9	38.8
	Mass of moisture(gm)F-G=(I)	27.9	29.5	30.8
	Mass of Dry soil(gm)G-H=(J)	67.8	66.4	64.8
	Moisture content % (I/J)*100=K	41.15	44.43	47.53
	Dry Density gm/cm^3 E/(100+K)*100	1.16	1.20	1.13



Appendix B-5: Stabilization of limestone with soils on CBR values

2% L of CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		N 5	N 5
Mass of soil + Mould	g	10320.5	11022.2
Mass Mould	g	6984.3	6984.3
Mass of Soil	g	3336.2	4037.9
Volume of Mould	g	2124	2124
Wet density of soil	g/cc	1.571	1.901
Dry density of soil	g/cc	1.174	1.287
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		C 13 T2	G19
Mass of wet soil + Container		110.12	166.86
Mass of dry soil + Container	g	91.07	124.03
Mass of container	g	34.77	34.16
Mass of water	g	19.1	42.8
Mass of dry soil	g	56.3	89.9
Moisture content	%	33.8	47.7
Max. Dry Density g/cc	1.24	OMC %	34.7

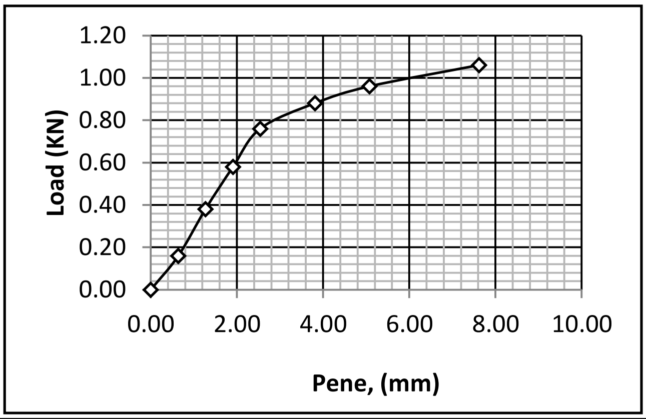
CBR Penetration Determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking		
Pen.(mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.12	
1.27	0.26	
1.91	0.38	
2.54	0.50	3.75
3.81	0.63	
5.08	0.74	3.70
7.62	0.81	



Swell Determination				
Gauge rdg (mm)		Swell in %		
Initial	16.12	4.54		
Final	21.40			
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		0.5	3.8	
5.08mm		0.7	3.7	
Dry Density at 95% MDD:			1.18	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	33.8	1.174	3.8	95
CBR at 95% MDD	3.75	Swell %	4.54	

4% L of CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		N3	N3
Mass of soil + Mould	g	10456.2	11052.2
Mass Mould	g	7110.3	7110.3
Mass of Soil	g	3345.9	3941.9
Volume of Mould	g	2124	2124
Wet density of soil	g/cc	1.575	1.856
Dry density of soil	g/cc	1.185	1.207
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		C2T3	G21
Mass of wet soil + Container		124.30	156.20
Mass of dry soil + Container	g	102.30	114.60
Mass of container	g	35.60	37.20
Mass of water	g	22.0	41.6
Mass of dry soil	g	66.7	77.4
Moisture content	%	33.0	53.7
Max. Dry Density g/cc	1.23	OMC %	35.4

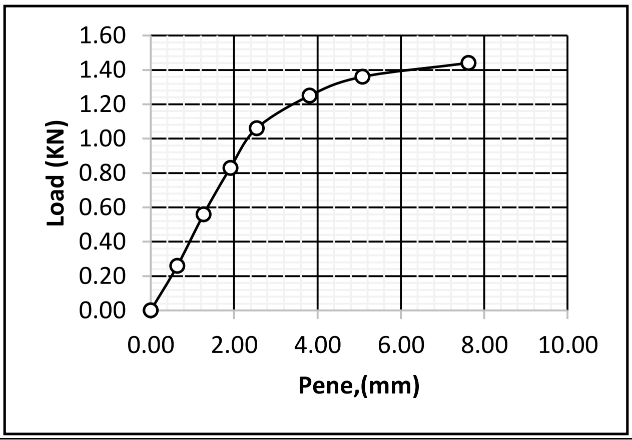
CBR Penetration Determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking		
Pen.(mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.16	
1.27	0.38	
1.91	0.58	
2.54	0.76	5.70
3.81	0.88	
5.08	0.96	4.80
7.62	1.06	



Swell Determination				
Gauge rdg (mm)		Swell in %		
Initial	17.20	3.52		
Final	21.30			
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		**0.8**	**5.7**	
5.08mm		**1.0**	**4.8**	
Dry Density at 95% MDD:			1.17	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	32.98	1.18	5.71	95
CBR at 95% MDD	5.70	swell	3.52	

6% L of CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		N 7	N 7
Mass of soil + Mould	g	10532.6	11089.8
Mass Mould	g	7135.6	7135.6
Mass of Soil	g	3397	3954.2
Volume of Mould	g	2124	2124
Wet density of soil	g/cc	1.599	1.862
Dry density of soil	g/cc	1.236	1.282
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		P 9	P6
Mass of wet soil + Container		131.73	167.75
Mass of dry soil + Container	g	109.64	127.20
Mass of container	g	34.42	37.60
Mass of water	g	22.1	40.6
Mass of dry soil	g	75.2	89.6
Moisture content	%	29.4	45.3
Max. Dry Density g/cc	1.22	OMC %	37.75

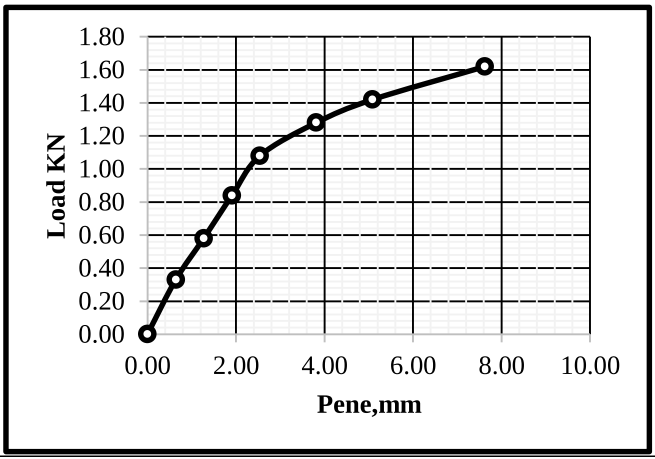
CBR Penetration Determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking		
Pen.(mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.26	
1.27	0.56	
1.91	0.83	
2.54	1.06	7.95
3.81	1.25	
5.08	1.36	6.80
7.62	1.44	



Swell Determination				
Gauge rdg (mm)		Swell in %		
Initial	18.94	2.97		
Final	22.4			
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		1.1	8.0	
5.08mm		1.4	6.8	
Dry Density at 95% MDD:			1.16	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	29.4	1.236	8.0	95
CBR at 95% MDD	7.95	Swell	2.97	

8% L of CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		N9	N9
Mass of soil + Mould	g	10656.6	11102.4
Mass Mould	g	7116.5	7116.5
Mass of Soil	g	3540.1	3985.9
Volume of Mould	g	2124	2124
The wet density of soil	g/cc	1.667	1.877
The dry density of soil	g/cc	1.303	1.285
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		P10	P12
Mass of wet soil + Container		140.60	162.30
Mass of dry soil + Container	g	117.60	122.50
Mass of container	g	35.20	36.00
Mass of water	g	23.0	39.8
Mass of dry soil	g	82.4	86.5
Moisture content	%	27.9	46.0
Max. Dry Density g/cc	1.21	OMC %	39.36

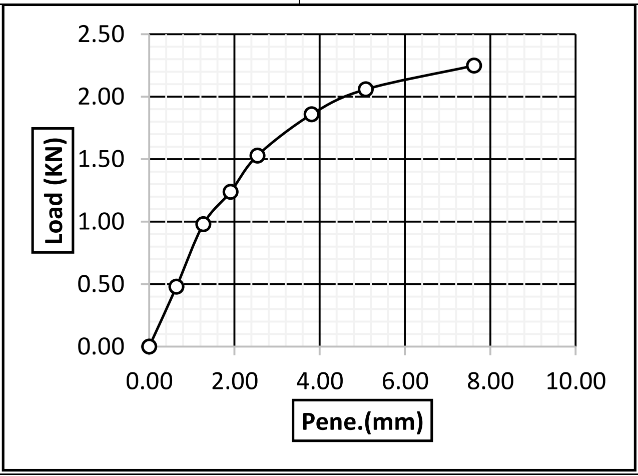
CBR Penetration Determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking		
Pen.(mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.52	
1.27	0.89	
1.91	1.22	
2.54	1.46	10.94
3.81	1.78	
5.08	1.96	9.80
7.62	2.17	



Swell Determination				
Gauge rdg (mm)		Swell in %		
Initial	18.94	1.82		
Final	22.05			
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		1.3	10.94	
5.08mm		1.7	9.80	
Dry Density at 95% MDD:			1.15	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	27.9	1.303	9.9	95
CBR at 95% MDD	10.94	Swell	1.82	

10% L of CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		N 8	N 8
Mass of soil + Mould	g	11236.2	11148.2
Mass Mould	g	7136.8	7136.8
Mass of Soil	g	4099.4	4011.4
Volume of Mould	g	2124	2124
Wet density of soil	g/cc	1.930	1.889
Dry density of soil	g/cc	1.520	1.317
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		165.14	158.52
Mass of wet soil + Container		137.58	121.41
Mass of dry soil + Container	g	35.4	35.8
Mass of container	g	27.6	37.1
Mass of water	g	102.2	85.6
Mass of dry soil	g	27.0	43.4
Moisture content	%	165.14	158.52
Max. Dry Density g/cc	1.20	OMC %	44.43

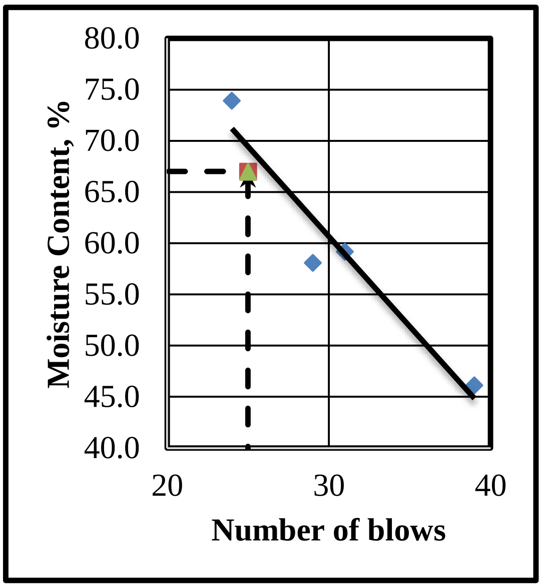
CBR Penetration Determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking		
Pen.(mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.48	
1.27	0.76	
1.91	1.10	
2.54	1.32	9.90
3.81	1.66	
5.08	1.90	9.50
7.62	2.09	



Swell Determination				
Gauge rdg (mm)		Swell in %		
Initial	19.23	1.55		
Final	21.04			
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		1.7	9.9	
5.08mm		2.1	9.5	
Dry Density at 95% MDD:			11.14	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	27.0	1.520	12.9	95
CBR at 95% MDD	9.9	Swell in %	1.55	

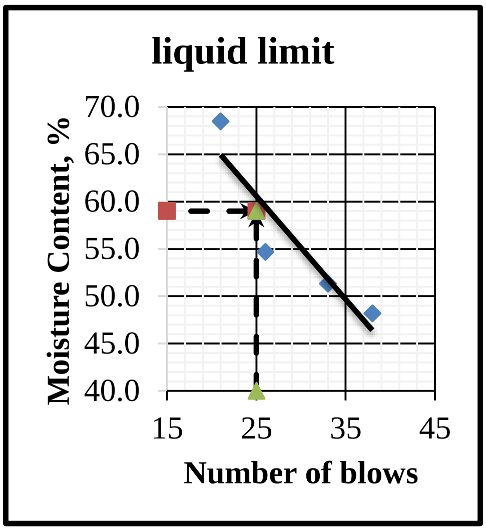
Appendix B-6: Atterberg limits test of limestone with soil

	liquid limit				
2% limestone	Number of blows	24	29	31	39
	Test	1	2	3	4
	Test	1	2	3	4
	Container	C1	A5	SP2	SP1
	Wt. of container + wet soil,g	34.6	27.66	29.7	27.9
	Wt. of container + dry soil,g	28.56	22.2	24.16	24.68
	Wt. of container,g	20.39	12.8	14.8	17.7
	Wt. of water,g	6.04	5.46	5.54	3.22
	Wt. of dry soil,g	8.17	9.4	9.36	6.98
	LL	68			



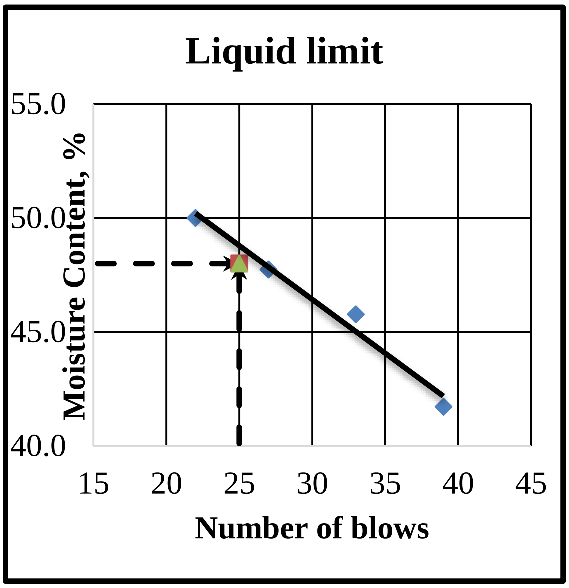
Plastic Limit		
Trial	1	2
Container	C8	C9
Wt. of container + wet soil,g	8.3	19.6
Wt. of container + dry soil,g	7.65	18.88
Wt. of container,g	5.8	16.3
Wt. of water,g	0.7	0.72
Wt. of dry soil,g	1.85	2.58
Moisture content,%	35.1	27.9
Average moisture Content(%)	**31.5**	
Plasticity Index (PI)	**36**	

	liquid limit				
4% limestone	Number of blows	21	26	33	38
	Test	1	2	3	4
	Container	B1	B2	D3	K1
	Wt. of container + wet soil,g	33.2	30.2	27.6	29.9
	Wt. of container + dry soil,g	26.9	26.1	24.31	25.9
	Wt. of container,g	17.7	18.6	17.9	17.6
	Wt. of water,g	6.3	4.1	3.29	4
	Wt. of dry soil,g	9.2	7.5	6.41	8.3
	Moisture content,%	**68.5**	**54.7**	**51.3**	**48.2**
	LL	58			



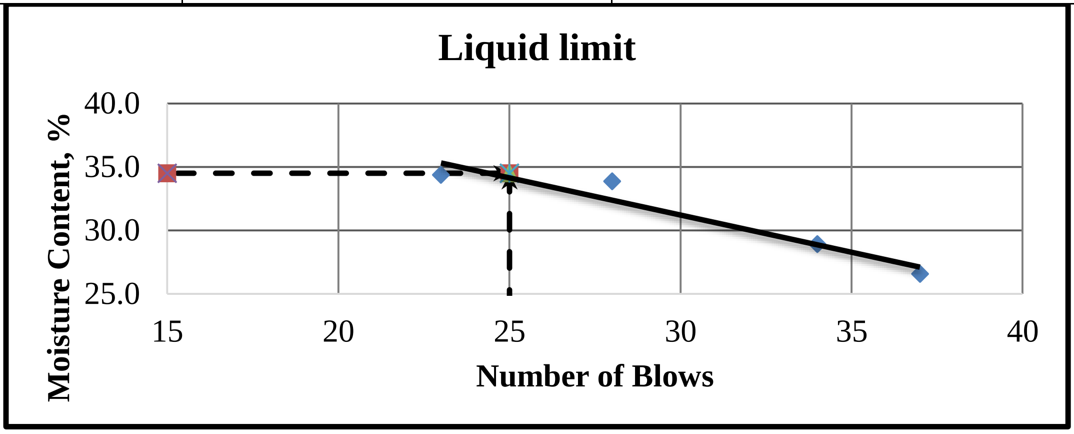
Plastic Limit		
Trial	1	2
Container	C5	C6
Wt. of container + wet soil,g	10.3	15.9
Wt. of container + dry soil,g	9.7	15.24
Wt. of container,g	8.21	12.6
Wt. of water,g	0.6	0.66
Wt. of dry soil,g	1.49	2.64
Moisture content,%	40.3	25.0
Average moisture Content(%)	**32.6**	
Plasticity Index (PI)	**25**	

	liquid limit				
6% limestone	Number of blows	22	27	33	39
	Trial	1	2	3	4
	Container	C13	2L	3L	A4
	Wt. of container + wet soil,g	17.2	29.3	33.5	33.2
	Wt. of container + dry soil,g	13.1	24.55	29.45	27.99
	Wt. of container,g	4.9	14.6	20.6	15.5
	Wt. of water,g	4.1	4.75	4.05	5.21
	Wt. of dry soil,g	8.2	9.95	8.85	12.49
	Moisture content,%	50.0	47.7	45.8	41.7
	LL	48			



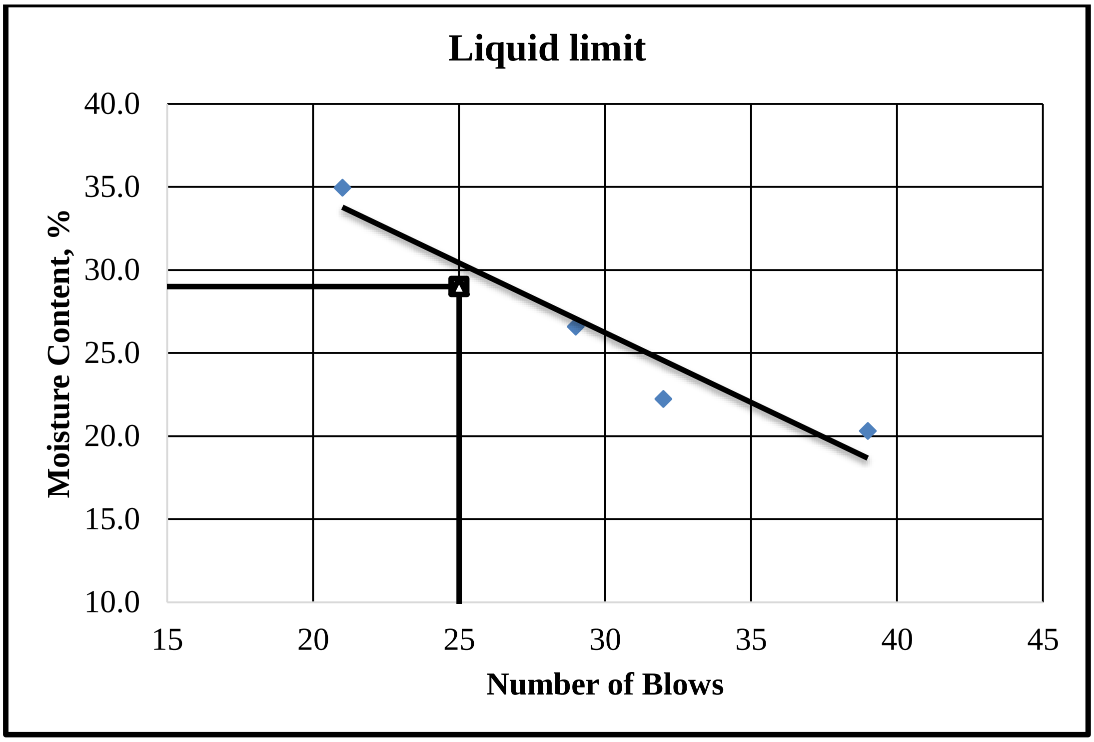
Plastic Limit		
Trial	1	2
Container	C8	C9
Wt. of container + wet soil,g	8.7	20.4
Wt. of container + dry soil,g	7.86	19.6
Wt. of container,g	5.6	17.2
Wt. of water,g	0.8	0.8
Wt. of dry soil,g	2.26	2.4
Moisture content,%	37.2	33.3
Average moisture Content(%)	**35.3**	
Plasticity Index (PI)	**10.7**	

	liquid limit				
8% limestone	Number of blows	23	28	34	37
	Trial	1	2	3	4
	Container	A5	A6	A7	A8
	Wt. of container + wet soil,g	20.8	28.6	30.4	27.9
	Wt. of container + dry soil,g	17.22	25.11	27.53	25.36
	Wt. of container,g	6.8	14.8	17.6	15.8
	Wt. of water,g	3.58	3.49	2.87	2.54
	Wt. of dry soil,g	10.42	10.31	9.93	9.56
	Moisture content,%	34.4	33.9	28.9	26.6
	LL	34.5			



Plastic Limit		
Trial	1	2
Container	G22	H10
Wt. of container + wet soil,g	10.2	19.5
Wt. of container + dry soil,g	9.02	19.38
Wt. of container,g	6.3	18.6
Wt. of water,g	1.2	0.12
Wt. of dry soil,g	2.72	0.78
Moisture content,%	43.4	15.4
Average moisture Content(%)	**38.5**	
Plasticity Index (PI)	**6.5**	

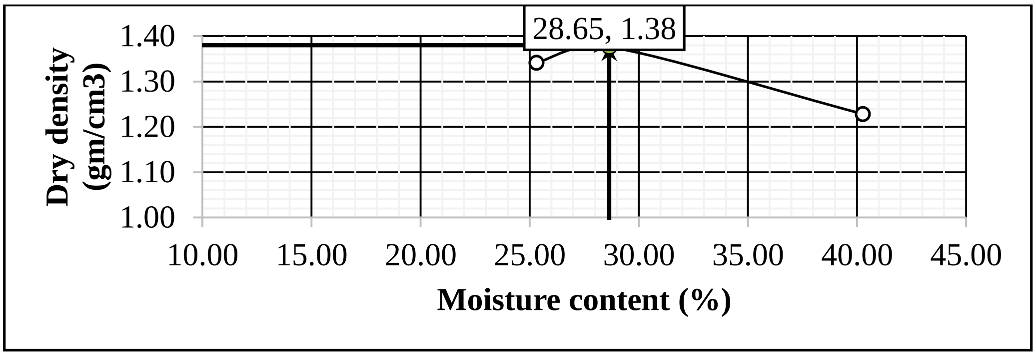
	liquid limit				
10% Limestone	Number of blows	21	29	32	39
	Test	1	2	3	4
	Container	W1	W2	T3	T5
	Wt. of container + wet soil,g	31.9	26.6	36.7	20.6
	Wt. of container + dry soil,g	28.43	24.29	34.5	19.25
	Wt. of container,g	18.5	15.6	24.6	12.6
	Wt. of water,g	3.47	2.31	2.2	1.35
	Wt. of dry soil,g	9.93	8.69	9.9	6.65
	Moisture content,%	34.9	26.6	22.2	20.3
LL		27.5			



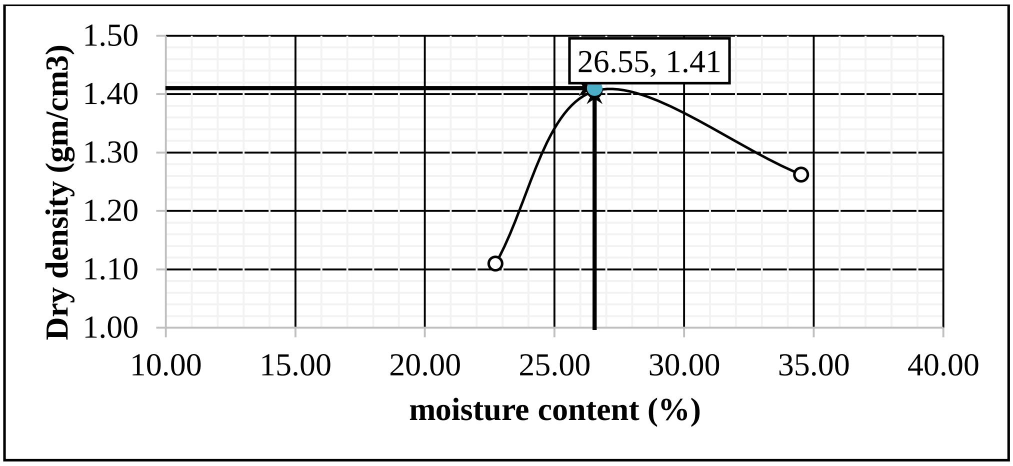
Plastic Limit		
Trial	1	2
Container	G8	G9
Wt. of container + wet soil,g	10.9	10.7
Wt. of container + dry soil,g	9.5	10.5
Wt. of container,g	5.6	9.4
Wt. of water,g	1.4	0.2
Wt. of dry soil,g	3.9	1.1
Moisture content,%	35.9	18.2
Average moisture Content(%)	**40.6**	
Plasticity Index (PI)	**3.2**	

Appendix B-7: Compaction test results of WCD – limestone

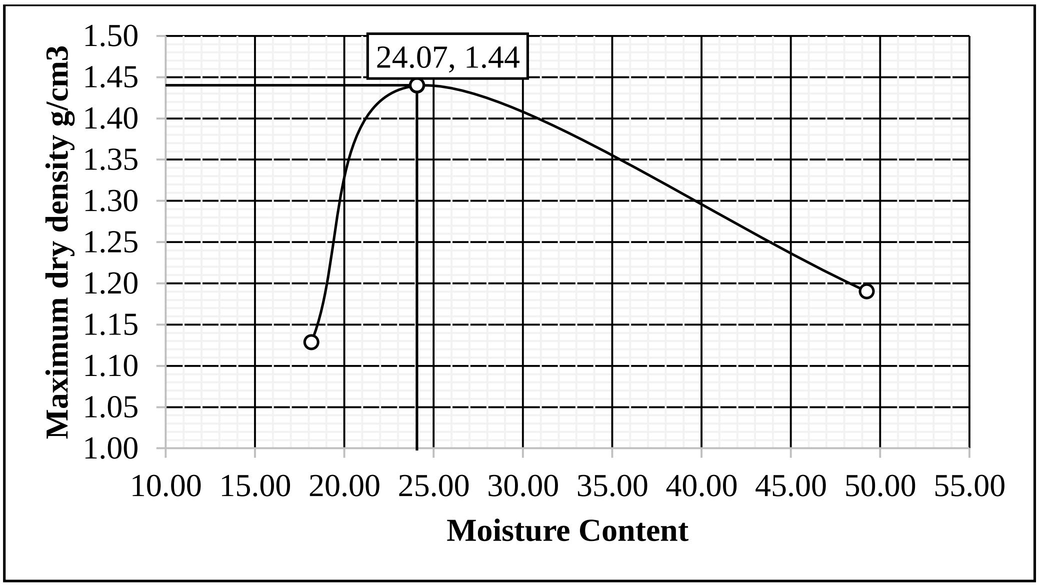
	Test No.	1	2	3
20% WCD + 2% L	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	420	500	580
	Mass of Mold+Wet soil(gm)(A)	3403.6	3486.3	3435.8
	Mass of Mold(gm)(B)	1816.8	1815.3	1809.5
	Mass of Wet Soil(gm)A-B=C	1586.8	1671	1626.3
	Volume of Mold cm^3(D)	944	944	944
	Bulk Density gm/cm^3 C/D=(E)	1.68	1.77	1.72
	Container Code .	K1	K2	H1
	Mass of Wet soil+Container(gm)(F)	86.4	100.7	103.6
	Mass of dry soil+container(gm)(G)	71.9	80.12	80.8
	Mass of container(gm)(H)	14.6	8.3	36.6
	Mass of moisture(gm)F-G=(I)	14.5	20.58	17.8
	Mass of Dry soil(gm)G-H=(J)	57.3	71.82	44.2
	Moisture content % (I/J)*100=K	25.31	28.65	40.27
	Dry Density gm/cm^3 E/(100+K)*100	1.34	1.38	1.23



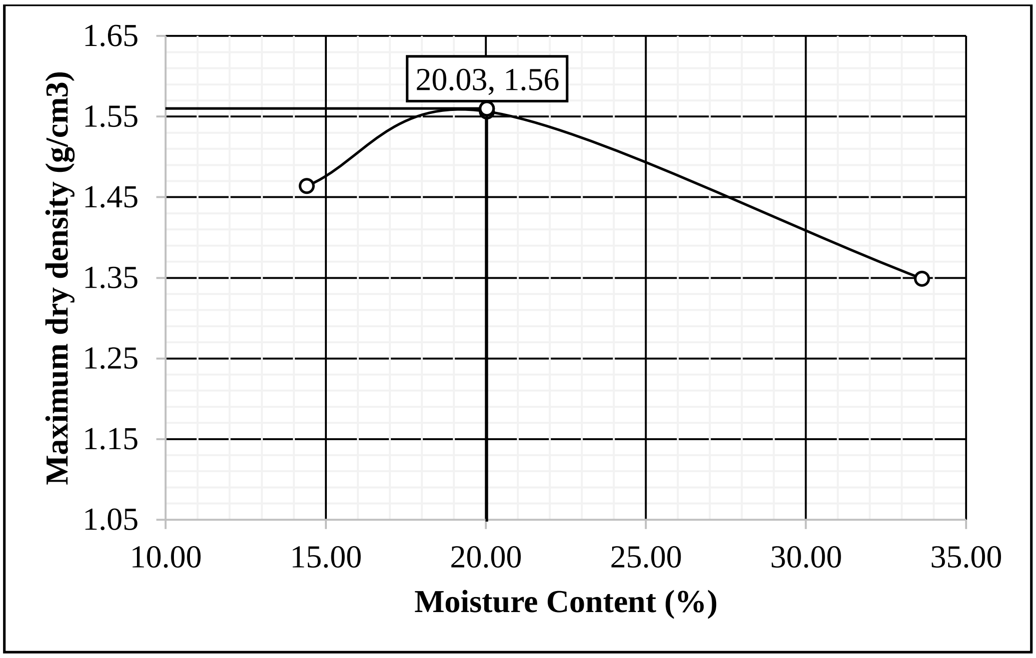
	Test No.		1	2
20% WCD + 4% L	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	400	480	560
	Mass of Mold+Wet soil(gm)(A)	3370.9	3485.6	3410.5
	Mass of Mold(gm)(B)	1815.7	1806.6	1807.8
	Mass of Wet Soil(gm)A-B=C	1555.2	1679	1602.7
	Volume of Mold cm^3(D)	944	944	944
	Bulk Density gm/cm^3 C/D=(E)	1.65	1.78	1.70
	Container Code .	T1	N2	C1T2
	Mass of Wet soil+Container(gm)(F)	91.3	85.1	78.6
	Mass of dry soil+container(gm)(G)	77.1	69.51	63.28
	Mass of container(gm)(H)	14.6	10.8	18.9
	Mass of moisture(gm)F-G=(I)	14.2	15.59	15.32
	Mass of Dry soil(gm)G-H=(J)	62.5	58.71	44.38
	Moisture content % (I/J)*100=K	22.72	26.55	34.52
	Dry Density gm/cm^3 E/(100+K)*100	1.11	1.41	1.26



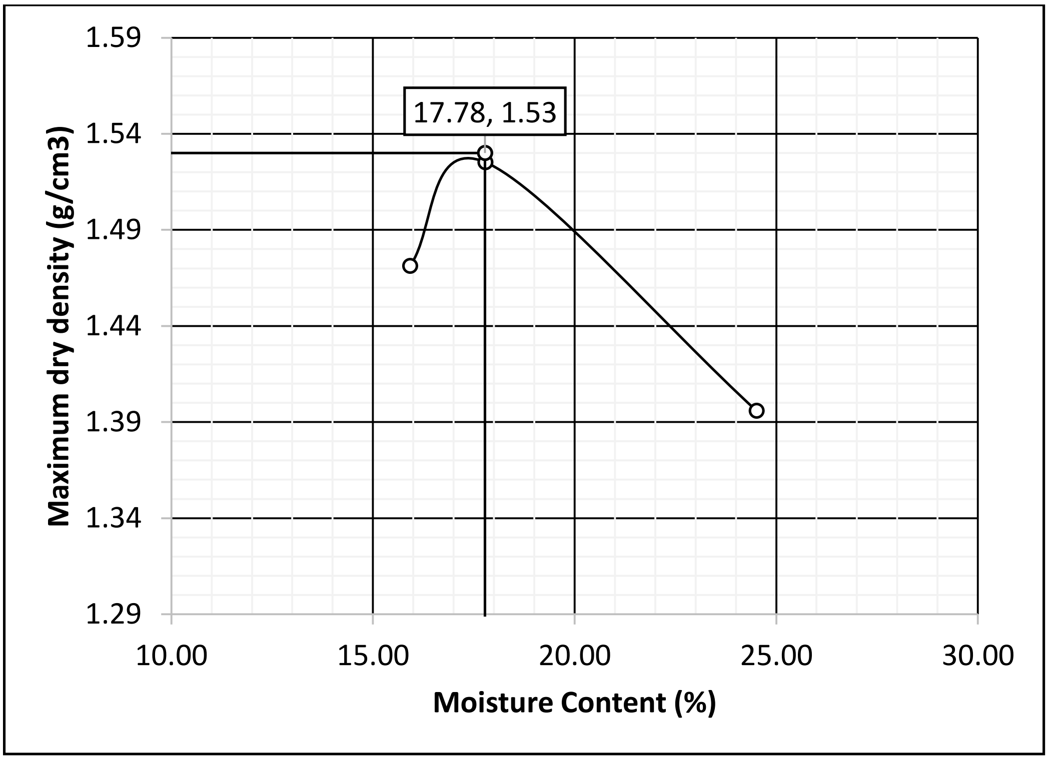
	Test No.	1	2	3
20% WCD + 6% L	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	380	460	540
	Mass of Mold+Wet soil(gm)(A)	3396.6	3502.6	3486.9
	Mass of Mold(gm)(B)	1814.56	1816.4	1809.9
	Mass of Wet Soil(gm)A-B=C	1582.04	1686.2	1677
	Volume of Mold cm^3(D)	944	944	944
	Bulk Density gm/cm^3 C/D=(E)	1.68	1.79	1.78
	Container Code .	T4	NB	T2C1
	Mass of Wet soil+Container(gm)(F)	81.1	93.2	86.09
	Mass of dry soil+container(gm)(G)	69.5	78.3	63.28
	Mass of container(gm)(H)	5.6	16.4	16.96
	Mass of moisture(gm)F-G=(I)	11.6	14.9	22.81
	Mass of Dry soil(gm)G-H=(J)	63.9	61.9	46.32
	Moisture content % (I/J)*100=K	18.15	24.07	49.24
	Dry Density gm/cm^3 E/(100+K)*100	1.13	1.44	1.19



	Test No.	1	2	3
20% WCD + 8 L	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	390	470	550
	Mass of Mold+Wet soil(gm)(A)	3388.9	3586.6	3510.3
	Mass of Mold(gm)(B)	1807.9	1822.9	1808.8
	Mass of Wet Soil(gm)A-B=C	1581	1763.7	1701.5
	Volume of Mold cm^3(D)	944	944	944
	Bulk Density gm/cm^3 C/D=(E)	1.67	1.87	1.80
	Container Code .	K50	T1C1	65
	Mass of Wet soil+Container(gm)(F)	82.1	81.4	130.7
	Mass of dry soil+container(gm)(G)	73.6	69.8	103.27
	Mass of container(gm)(H)	14.6	11.9	21.7
	Mass of moisture(gm)F-G=(I)	8.5	11.6	27.43
	Mass of Dry soil(gm)G-H=(J)	59	57.9	81.57
	Moisture content % (I/J)*100=K	14.41	20.03	33.63
	Dry Density gm/cm^3 E/(100+K)*100	1.46	1.56	1.35



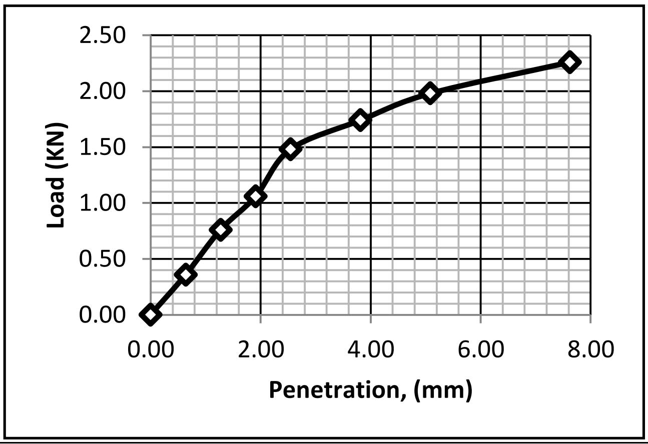
	Test No.	1	2	3
20% WCD + 10% L	Mass of sample (gm)	4000	4000	4000
	Water Added(cc)	400	480	560
	Mass of Mold+Wet soil(gm)(A)	3425.6	3510.3	3456.3
	Mass of Mold(gm)(B)	1815.6	1814.5	1815.6
	Mass of Wet Soil(gm)A-B=C	1610	1695.8	1640.7
	Volume of Mold cm^3(D)	944	944	944
	Bulk Density gm/cm^3 C/D=(E)	1.71	1.80	1.74
	Container Code .	A	HH	T3
	Mass of Wet soil+Container(gm)(F)	105.6	82.5	132.2
	Mass of dry soil+container(gm)(G)	94.2	73.2	113.4
	Mass of container(gm)(H)	22.6	20.9	36.7
	Mass of moisture(gm)F-G=(I)	11.4	9.3	18.8
	Mass of Dry soil(gm)G-H=(J)	71.6	52.3	76.7
	Moisture content % (I/J)*100=K	15.92	17.78	24.51
	Dry Density gm/cm^3 E/(100+K)*100	1.47	1.53	1.40



Appendix B-8: CBR test for waste ceramic dust-Limestone with soils

20% WCD + 2% L CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		N11	N11
Mass of soil + Mould	g	10687.9	11066.6
Mass Mould	g	7123.3	7123.3
Mass of Soil	g	3564.6	3943.3
Volume of Mould	g	2124	2124
Wet density of soil	g/cc	1.678	1.857
Dry density of soil	g/cc	1.399	1.484
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		B3	B4
Mass of wet soil + Container		140.90	154.80
Mass of dry soil + Container	g	118.90	129.70
Mass of container	g	8.80	29.80
Mass of water	g	22.0	25.1
Mass of dry soil	g	110.1	99.9
Moisture content	%	19.2	47.1
Max. Dry Density g/cc	1.38	OMC %	28.65

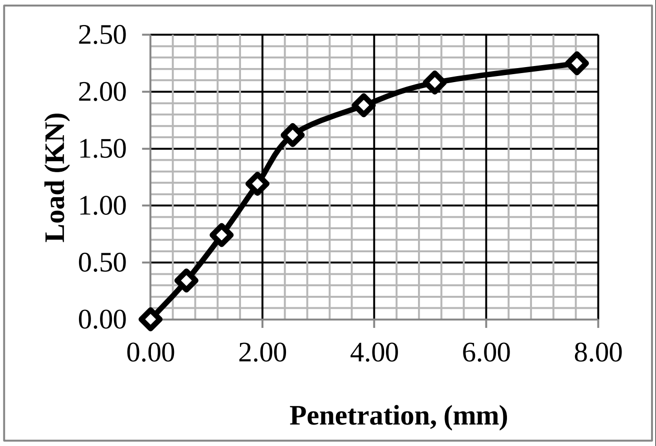
CBR Penetration Determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking		
Pen.(mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.36	
1.27	0.76	
1.91	1.06	
2.54	1.48	11.09
3.81	1.74	
5.08	1.98	9.90
7.62	2.26	



Swell Determination				
Gauge rdg (mm)		Swell in %		
Initial	16.80	1.72		
Final	18.80			
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		1.5	11.1	
5.08mm		2.0	9.9	
Dry Density at 95% MDD:			1.31	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	20.0	1.399	11.1	95
CBR at 95% MDD	11.1	Swell %	1.72	

20% WCD + 4% L CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		A2	A2
Mass of soil + Mould	g	10445.6	11021.1
Mass Mould	g	6947.6	6947.6
Mass of Soil	g	3498	4073.5
Volume of Mould	g	2124	2124
Wet density of soil	g/cc	1.647	1.918
Dry density of soil	g/cc	1.294	1.389
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		B1	B2
Mass of wet soil + Container		168.70	180.90
Mass of dry soil + Container	g	140.60	142.30
Mass of container	g	37.70	40.80
Mass of water	g	28.1	38.6
Mass of dry soil	g	102.9	101.5
Moisture content	%	27.3	38.0
Max. Dry Density g/cc	1.41	OMC %	26.5

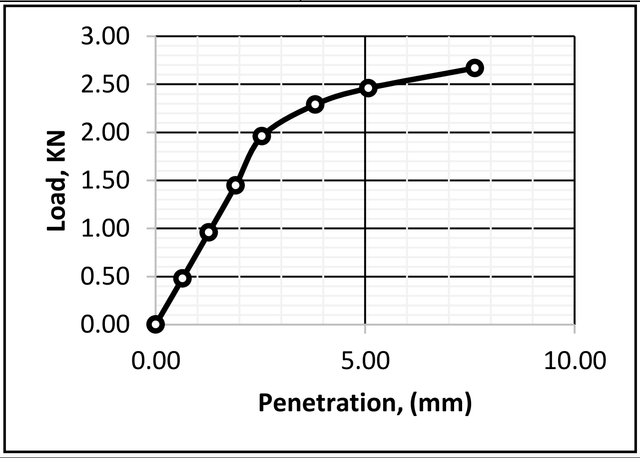
CBR Penetration Determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking		
Pen.(mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.34	
1.27	0.74	
1.91	1.19	
2.54	1.62	12.14
3.81	1.88	
5.08	2.08	10.40
7.62	2.25	



Swell Determination				
Gauge rdg (mm)		Swell in %		
Initial	19.10	1.63		
Final	21.00			
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		1.6	12.2	
5.08mm		2.1	10.4	
Dry Density at 95% MDD:			1.1.34	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	27.3	1.29	5.6	95.0
CBR at 95% MDD	12.2	Swell %	1.63	

20% WCD + 6% L CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		N7	N7
Mass of soil + Mould	g	10660.8	10988.9
Mass Mould	g	7198.9	7198.9
Mass of Soil	g	3461.9	3790
Volume of Mould	g	2124	2124
Wet density of soil	g/cc	1.630	1.784
Dry density of soil	g/cc	1.400	1.284
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		L1	L2
Mass of wet soil + Container		122.60	144.40
Mass of dry soil + Container	g	108.90	113.90
Mass of container	g	25.60	35.70
Mass of water	g	13.7	30.5
Mass of dry soil	g	83.3	78.2
Moisture content	%	16.4	39.0
Max. Dry Density g/cc	1.44	OMC %	24.07

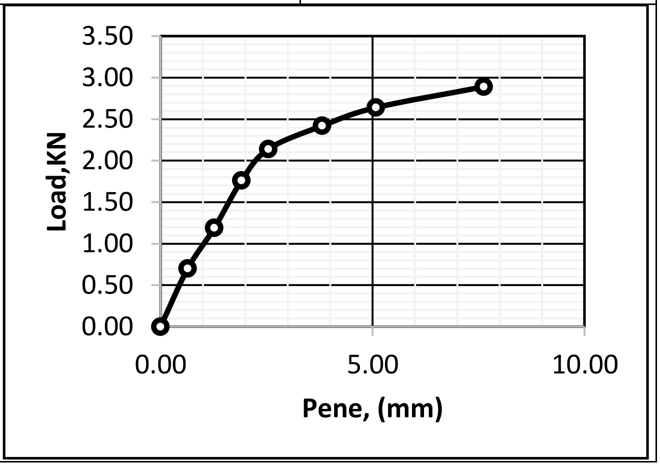
CBR Penetration Determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking		
Pen.(mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.48	
1.27	0.96	
1.91	1.45	
2.54	1.96	14.69
3.81	2.29	
5.08	2.46	12.30
7.62	2.67	



Swell Determination				
Gauge rdg (mm)		Swell in %		
Initial	19.60	1.46		
Final	21.30			
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		2.0	14.7	
5.08mm		2.5	12.3	
Dry Density at 95% MDD:			1.37	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	16.4	1.400	14.7	95.003
CBR at 95% MDD	14.7	Swell %	1.46	

20% WCD + 8% L CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		N13	N13
Mass of soil + Mould	g	10530.8	11024.8
Mass Mould	g	6999.9	6999.9
Mass of Soil	g	3530.9	4024.9
Volume of Mould	g	2124	2124
Wet density of soil	g/cc	1.662	1.895
Dry density of soil	g/cc	1.422	1.503
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		B1	B2
Mass of wet soil + Container		157.60	178.90
Mass of dry soil + Container	g	138.40	145.80
Mass of container	g	24.80	18.70
Mass of water	g	19.2	33.1
Mass of dry soil	g	113.6	127.1
Moisture content	%	16.9	26.0
Max. Dry Density g/cc	1.56	OMC %	20.03

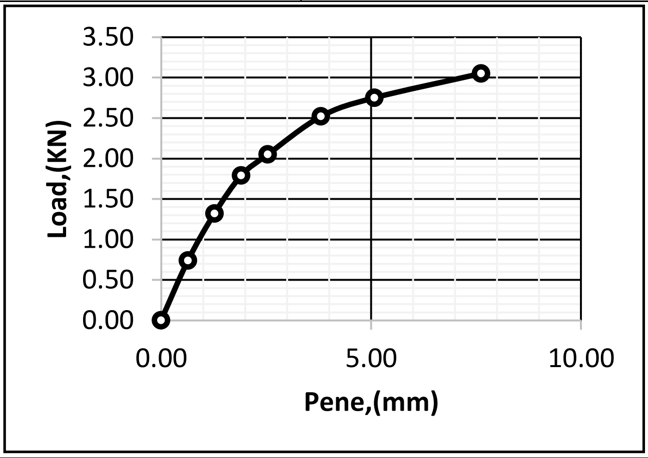
CBR Penetration Determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking		
Pen.(mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.70	
1.27	1.19	
1.91	1.76	
2.54	2.14	16.04
3.81	2.42	
5.08	2.64	13.20
7.62	2.89	



Swell Determination				
Gauge rdg (mm)		Swell in %		
Initial	19.60	1.37		
Final	21.30			
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		2.1	16.1	
5.08mm		2.6	13.2	
Dry Density at 95% MDD:			1.422	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	16.9	1.422	16.1	95.01
CBR at 95% MDD	16.1	Swell %	1.37	

20% WCD + 10% L CBR			
Compaction Determination @ 56 blows			
		Before soak	After soak
Mould No.		N2	N2
Mass of soil + Mould	g	10661.5	10958.7
Mass Mould	g	7101.6	7101.6
Mass of Soil	g	3559.9	3857.1
Volume of Mould	g	2124	2124
The wet density of soil	g/cc	1.676	1.816
The dry density of soil	g/cc	1.406	1.234
Moisture content determination data @ 56 blows			
		Before soak	After soak
Container no.		3	G19
Mass of wet soil + Container		109.83	156.05
Mass of dry soil + Container	g	98.70	117.60
Mass of container	g	40.68	36.02
Mass of water	g	11.1	38.5
Mass of dry soil	g	58.0	81.6
Moisture content	%	19.2	17.78
Max. Dry Density g/cc	1.53	OMC %	29.9

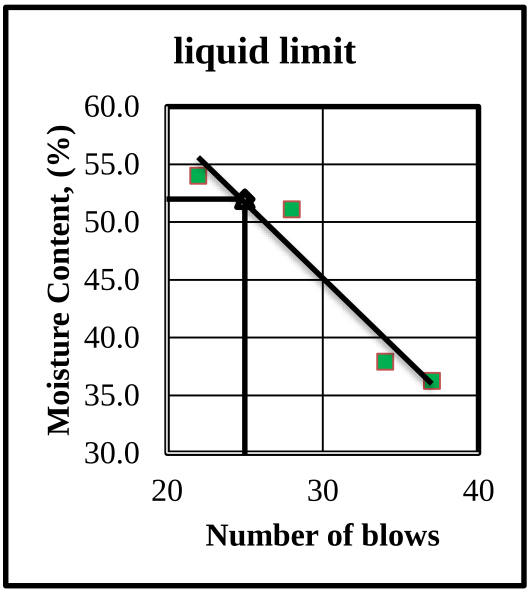
CBR Penetration Determination		
Surcharge Weight:-4.55 KG		
Penetration after 96 hrs. Soaking		
Pen. (mm)	Load, KN	CBR %
0.00	0.00	
0.64	0.74	
1.27	1.32	
1.91	1.79	
2.54	2.05	15.37
3.81	2.52	
5.08	2.75	13.75
7.62	3.05	



Swell Determination				
Gauge rdg (mm)		Swell in %		
Initial	17.21	1.19		
Final	18.60			
Penetration(mm)	Load KN			
	Top	Bottom	Corr. CBR %	
2.54mm		2.1	15.4	
5.08mm		2.8	13.8	
Dry Density at 95% MDD:			1.5	
No. of blows	MCBS %	DDBS (g/cm3)	Corr. CBR %	% Compaction
56	19.2	1.406	15.4	95.02
CBR at 95% MDD	15.4	Swell %	1.19	

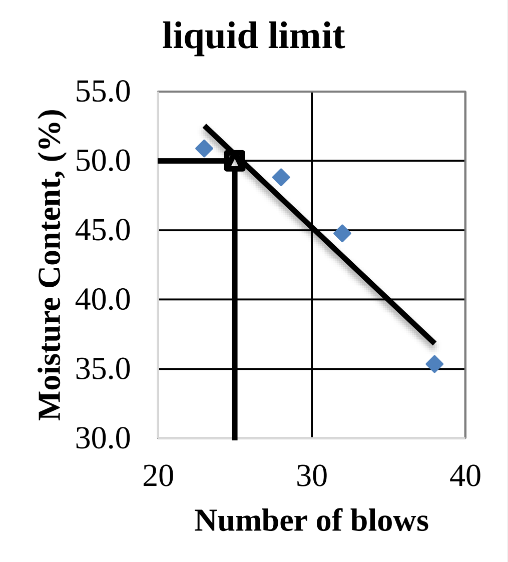
Appendix B-9: Atterberg limit tests for WCD- Limestone with soils

	liquid limit				
20% WCD + 2% L	Number of blows	22	28	34	37
	Test	1	2	3	4
	Container	D1	D3	SP1	SP3
	Wt. of container + wet soil,g	32.8	30.8	27.3	29.06
	Wt. of container + dry soil,g	27.89	25.32	24.44	26.33
	Wt. of container,g	18.8	14.6	16.9	18.8
	Wt. of water,g	4.91	5.48	2.86	2.73
	Wt. of dry soil,g	9.09	10.72	7.54	7.53
	Moisture content,%	54.0	51.1	37.9	36.3
	LL	53			



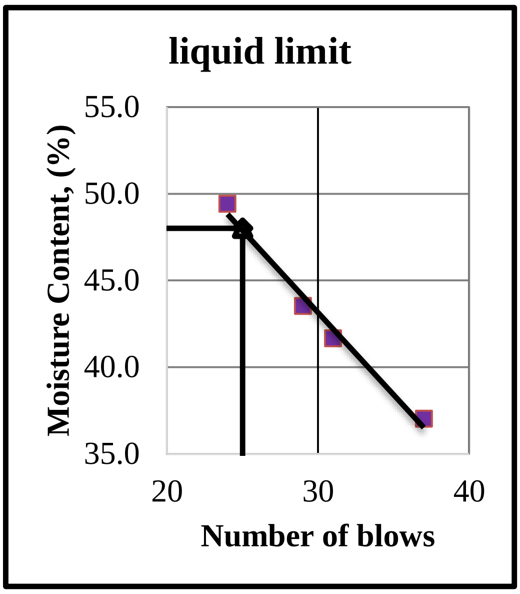
Plastic Limit		
Trial	1	2
Container	A1	A2
Wt. of container + wet soil,g	18.3	20.2
Wt. of container + dry soil,g	17.12	18.05
Wt. of container,g	11.6	12.9
Wt. of water,g	1.2	2.15
Wt. of dry soil,g	5.52	5.15
Moisture content,%	21.4	41.7
Average moisture Content(%)	**31.6**	
Plasticity Index (PI)	**21**	

	liquid limit				
20% WCD + 4% L	Number of blows	23	28	32	38
	Test	1	2	3	4
	Container	A3	A1T1	D3T2	K11
	Wt. of container + wet soil,g	29.1	33.1	31.8	37.4
	Wt. of container + dry soil,g	26.2	30.64	28.12	34.11
	Wt. of container,g	20.5	25.6	19.9	24.8
	Wt. of water,g	2.9	2.46	3.68	3.29
	Wt. of dry soil,g	5.7	5.04	8.22	9.31
	Moisture content,%	50.9	48.8	44.8	35.3
	LL	51			



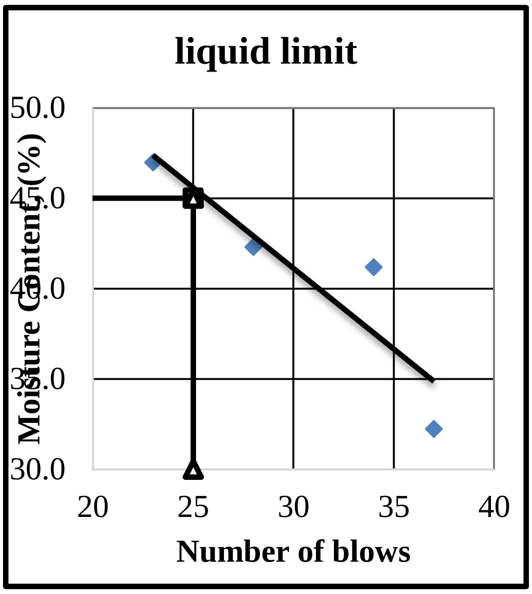
Plastic Limit		
Trial	1	2
Container	B3	C7
Wt. of container + wet soil,g	12.1	14.1
Wt. of container + dry soil,g	11.01	13.23
Wt. of container,g	8.8	8.62
Wt. of water,g	1.1	0.87
Wt. of dry soil,g	2.21	4.61
Moisture content,%	49.3	18.9
Average moisture Content(%)	**34.1**	
Plasticity Index (PI)	**17**	

	liquid limit				
20% WCD + 6% L	Number of blows	24	29	31	37
	Trial	1	2	3	4
	Container	J1	L3	L2	A1
	Wt. of container + wet soil,g	18.5	28.4	34.2	38.94
	Wt. of container + dry soil,g	14.3	24.7	30.2	36.2
	Wt. of container,g	5.8	16.2	20.6	28.8
	Wt. of water,g	4.2	3.7	4	2.74
	Wt. of dry soil,g	8.5	8.5	9.6	7.4
	Moisture content,%	49.4	43.5	41.7	37.0
	LL	48			



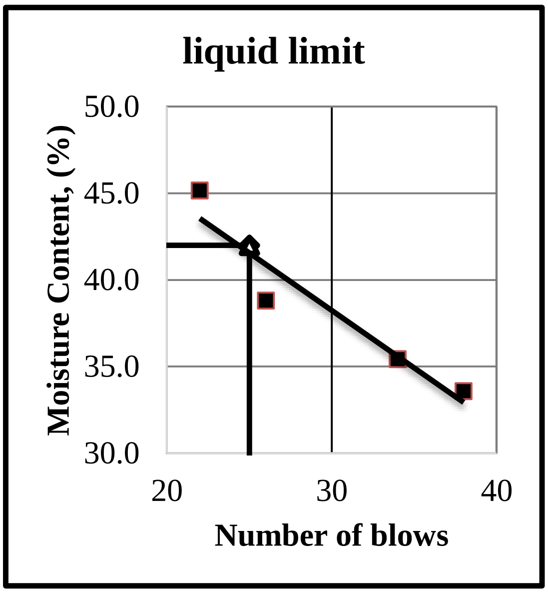
Plastic Limit		
Trial	1	2
Container	E1	E2
Wt. of container + wet soil,g	9.4	19.9
Wt. of container + dry soil,g	8.3	18.03
Wt. of container,g	5.6	12.6
Wt. of water,g	1.1	1.87
Wt. of dry soil,g	2.7	5.43
Moisture content,%	40.7	34.4
Average moisture Content(%)	**37.6**	
Plasticity Index (PI)	**10.4**	

	liquid limit				
20% WCD + 8% L	Number of blows	23	28	34	37
	Trial	1	2	3	4
	Container	T1	T2	T3	K2
	Wt. of container + wet soil,g	20.8	25.7	29.4	31.23
	Wt. of container + dry soil,g	16.9	21.2	26.6	28.2
	Wt. of container,g	8.6	10.56	19.8	18.8
	Wt. of water,g	3.9	4.5	2.8	3.03
	Wt. of dry soil,g	8.3	10.64	6.8	9.4
	Moisture content,%	47.0	42.3	41.2	32.2
	LL	45			



Plastic Limit		
Trial	1	2
Container	G1	G10
Wt. of container + wet soil,g	11.02	17.9
Wt. of container + dry soil,g	10.18	16.55
Wt. of container,g	6.9	13.9
Wt. of water,g	0.8	1.35
Wt. of dry soil,g	3.28	2.65
Moisture content,%	25.6	50.9
Average moisture Content(%)	38.3	
Plasticity Index (PI)	6.7	

	liquid limit				
20% WCD + 10% L	*Number of blows*	22	26	34	38
	Test	1	2	3	4
	Container	T1	T2	T3	T4
	Wt. of container + wet soil,g	22.3	19.4	35.8	20.54
	Wt. of container + dry soil,g	18.1	15.9	31.9	16.88
	Wt. of container,g	8.8	6.88	20.89	5.98
	Wt. of water,g	4.2	3.5	3.9	3.66
	Wt. of dry soil,g	9.3	9.02	11.01	10.9
	Moisture content,%	45.2	38.8	35.4	33.6
	LL	42			



Plastic Limit		
Trial	1	2
Container	T5	T6
Wt. of container + wet soil,g	9.56	10.1
Wt. of container + dry soil,g	8.37	9.14
Wt. of container,g	6.1	5.6
Wt. of water,g	1.2	0.96
Wt. of dry soil,g	2.27	3.54
Moisture content,%	52.4	27.1
Average moisture Content(%)	39.8	
Plasticity Index (PI)	2.7	

Appendix B-10: Specific gravity test results for ceramic waste & limestone

	Pycnometer No.	C1	C2	C3
WCD	Weight of dry, clean pycnometer, w_p (g)WP	30.05	31.31	30.17
	Weight of pycnometer + water, w_{pw} (g)	125.58	125.98	125.88
	The observed temperature of the water, T_i (OC)	19	19	19
	Determination No.	1	2	3
	Pycnometer No.	A1	A2	A3
	Weight of pycnometer + WC + water, W_{pws} (g) WPWS	141.6	140.6	141.03
	Temperature, T_x(^{o}c)	20	20	20
	Weight of pycnometer + water at T_x ,W_{pw}(atT_x) (g)	124.90	124.01	126.22
	Weight of dry WC, w_e (gm)	25	25	25
	Conversion factor , K	1.0000	1.0000	1.0000
	The specific gravity of soil at 20°c.	3.01	2.97	2.45
	The average specific gravity of soil	**2.81**		

	Pycnometer No.	K1	K2	K3
L. stone	Weight of dry, clean pycnometer, w_p (g)WP	31.32	30.88	31.75
	Weight of pycnometer + water, w_{pw} (g)	125.94	125.01	125.62
	The observed temperature of the water, T_i (OC)	20	20	20
	Determination No.	1	2	3
	Pycnometer No.	A1	A2	A3
	Weight of pycnometer + limestone + water, W_{pws} (g) WPWS	140.12	141.35	140.89
	Temperature, T_x(^{o}c)	21	21	21
	Weight of pycnometer + water at T_x ,W_{pw}(atT_x) (g)	124.90	125.31	125.40
	Weight of dry limestone, w_e (gm)	25	25	25
	Conversion factor , K	0.9998	0.9998	0.9998
	The specific gravity of soil at 20°c.	2.56	2.79	2.63
	The average specific gravity of soil	**2.66**		