ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

STABILIZATION OF EXPANSIVE SOIL USED AS SUBGRADE MATERIALUSING CEMENT KILN DUST

By:

ADEY GIRMA

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BY:
ADEY GIRMA

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

Advisor
Prof. Alemayehu Teferra

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This is to certify that the thesis prepared by Adey Girma entitled '*Stabilization of Expansive Soil Used as Subgrade Material Using Cement Kiln Dust*', submitted in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering complies with the regulations of the University and meets the accepted standards with respect to originality and Quality.

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ABSTRACT

Expansive soils are worldwide problems that pose several challenges to civil engineers. Such soils swell when given access to water and shrink when they dry out. The most common and economical method for stabilizing these soils is using admixtures that reduce volume changes. This research work is aimed at evaluating the suitability of cement kiln dust for stabilization of expansive clay soils. The preliminary investigated expansive soil shows that it belongs to A-7-5 class of soil according to the AASHTO soil classification system. Soils under this class are generally of poor engineering benefit. The soil was stabilized with cement kiln dust with the proportions of 5%, 10%, 15%, 20%, 25% and 30% by dry weight of the soil. Atterberg limits, free swell, swell –pressure, compaction, UCS, linear shrinkage and CBR tests were conducted to evaluate the properties of the stabilized soil. All stabilized soil samples were cured for 7 days for Atterberg limit and compaction tests. For CBR test, the samples were cured for 7, 14 and 28 days. For UCS test, however curing was done for 7 and 14 days.

Analysis of the results shows that slight improvement was achieved on the geotechnical properties of the cement kiln dust stabilized soil. Cement kiln dust reduces the plasticity index, swelling and MDD with increase in OMC and CBR with all higher cement kiln dust contents. Curing has significant effect on the geotechnical properties of cement kiln dust stabilized soil.

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LIST OF ABBREVIATIONS/ACRONYMS

AACRA	Addis Ababa City Roads Authority
AAIT	Addis Ababa Institute of Technology
AASHTO	American Association of Highway and Transportation Officials
ASTM	American Society for Testing and Materials
BCS	Black Cotton Soil
C	Cement
CBR	California Bearing Ratio
CKD	Cement Kiln Dust
CS	Clayey Sand
EiABC	Ethiopian Institute of Architectural and Building City
EMA	Ethiopia Meteorological Agency
ERA	Ethiopian Roads Authority
ET	Ethiopian Airlines
GSE	Geological Survey of Ethiopia
LL	Liquid Limit
MC	Moisture Content
MCE	Mugher Cement Enterprise
MDD	Maximum Dry Density
MRTD	Material and Testing Department
OMC	Optimum Moisture Content
OPC	Ordinary Portland cement
PI	Plastic Index
PL	Plastic Limit
PLC	Private Limited Company
SADC	Southern Africa Transport and Communications Commission
UCS	Unconfined Compressive Strength
USBR	United States Bureau of Reclamation
XRD	X-Ray Diffraction

CHAPTER ONE

INTRODUCTION

1.1.General Background

To Civil Engineers, soil is any uncemented accumulation of mineral particles formed by weathering of rocks. Soils are generally used as foundation or as construction materials. A class of soil, known as expansive soil, when used as foundation material, is usually affected by environmental conditions and undergoes detrimental volumetric and hydraulic conductivity changes because of the variation in moisture contents. Expansive soils are, therefore, soils with potential for shrinking or swelling under changing moisture condition (Fredlund and Rahardjo,1993). These soils cause damage to structures, particularly light buildings and pavements (Jones and Holtz, 1973).

Black cotton soil (BCS) is a type of expansive soil that principally occurs in semiarid and arid regions of the tropical/temperate zones marked with dry and wet seasons; and with low rainfall, poor drainage and exceedingly great heat. The climate condition is such that the annual evapo transpiration exceeds precipitation (Chen, 1988; Nelson and Miller, 1992; Warren and Kirby, 2004). Black cotton soils are black clays that are produced from the breakdown of basic igneous rocks where seasonal variation of weather is extreme.

Figure 1.1 and Figure 1.2 below clearly illustrate typical expansive soil (black cotton soil of Ethiopia) with polygonal pattern of cracks as a manifestation of considerable shrinkage in a dry season. These cracks are wide on the order of tens of centimeters and also deep on the order of meters (Fekerte, 2009).

Figure 1.1Typical Expansive Soil with Polygonal Cracks

Figure 1.2Exiting Apron Crack at Bole Site

Although poor and undesirable for engineering purposes, expansive soils could be improved to meet standard specification by modification/stabilization processes. Stabilization of the soil with chemical and /or mechanical stabilization technique is a common method of reducing theswell-shrink tendencies of the soil and also makes the soil less plastic (Balogun, 1991; Osinubi,1995). This urges the need for wider application of most effective and environmentally friendly technologies, such as chemical stabilization, to be customized and adopted to the current road construction trend in the country. Thus, locally available materials such as cement kiln dust (by-product of cement industries), fly ash, Sodium silicates, cement, lime, and sugar cane molassescan be applied to improve the Engineering properties of expansive soil.

This research is aimed at examining the potential of cement kiln dust to improve the problematic nature of expansive soils to be used as pavement bedding, subgrade material and to investigate the changes in engineering proprieties of the soil sample under varying percentages by weight of the cement kiln dust.

1.2. Statement of the problem

Relatively large areas in Ethiopia are covered with expansive clay soils. These soils have caused persistent difficulties in road, airfield construction. The conventional stabilizing agents commonly used in expansive soils and replacement of the inferior sub-grade soils by foreign soils may lead to increase in project cost and time as it needs excavation of the problematic soil, production and transportation of the selected material and compaction to the required specification are fairly expensive. Therefore, it is essential to modify the properties of locally available soil with cheaper stabilizer to the extent that it can be used in the construction of roads, airfields and to make best utilization of various industrial by products like cement kiln dust as a soil modifying/soil stabilizing agent.

1.3. Objective of the Study

1.3.1. General Objective

The main objective of this research work is to investigate the performance of cement kiln dust to improve the engineering proprieties of expansive soils to be used as subgrade material. This can be done by mixing the expansive soil with varying percentages by weight of cement kiln dust and to investigate the change in engineering properties of the soil sample. The specific objectives are described below.

1.3.2. Specific Objectives

The study has the specific objectives of investigating the response of the expansive clay soils through the application of cement kiln dust of varying proportion at different curing period. More specifically the work has the following objectives;

- To Assess the geotechnical engineering properties of the natural expansive clay soil by conducting index and classification tests, strength tests, volume and stability test.

- To investigate constituents of the cement kiln dust.
- To assess the strength and volume stability of the mixture of natural expansive clay soils and cement kiln dust by varying the proportions as reflected by California Bearing Ratio(CBR), Unconfined Compressive Strength (UCS) , swell test, proctor, pH, Atterberg limits , linear Shrinkage , volumetric shrinkage test and swell-pressure tests.
- To come up with a recommendable optimum proportion by weight of the cement kiln dust required to improve expansive soils to meet the specification requirement of subgrade material.

1.4. Research Methodology

In order to achieve the above objectives, the following methodologies were followed:

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

ii) Sampling and testing: material sampling and testing methods are undertaken as per ASTM and AASHTO standards. Soil samples were collected from Addis Ababa Bole International Airport site. The chemical additive, cement kiln dust taken (CKD) was collected from Mugher Cement Enterprise.

- Sample preparation in the experimental work involved air drying, pulverization and sieving of the natural soil sample to the required particle sizes.

The laboratory tests conducted on the natural soil samples and after blending with CKD include:

- Natural Moisture content determination test
- Free swell test
- Sieve and Hydrometer analysis test
- Specific gravity
- pH
- Atterberg limits
    - Liquid limit,
    - Plastic limit,
    - Plasticity index
- Linear Shrinkage

- California Bearing Ratio(CBR)
- Proctor test
- Unconfined compressive strength test
- Swell consolidation test

The laboratory tests conducted on the cement kiln dust additive include:

- Moisture content test
- Specific gravity
- X-ray diffraction
- PH test
- Dry Shrinkage test
- Atterberg limits
    - Liquid limit ,
    - Plastic limit,
    - Plasticity index
- Pozzolana test

The engineering properties of the soil samples after stabilization were tested at different percentage of cement kiln dust i.e., 5%, 10%, 15%, 20%, 25% and 30% by weight of dry soil.

Curing period was taken as another variable.

- Curing period of 7, 14 and 28 days for CBR test
- Curing period of 7, and 14 days for UCS tests
- 7days curing period for Atterberg limits
- 7days curing period for compaction

iii) Analysis and discussion of test results: the results obtained have been analyzed and discussed.

v) Conclusions were drawn and recommendations made based on the results obtained.

1.5.Scope of the Study and Limitations

- This study has been supported by literature survey and a series of laboratory tests. However, the findings of the research are limited only to one specific site and specific soil sample and limited number of tests considered in this research.
- The composition and characteristics of cement kiln dust is highly influenced by the following parameters: raw feed material, type of kiln operation, dust collection systems, and fuel type used for production. Since the properties of cement kiln dust can be significantly affected by the design, operation and materials used in a cement kiln, the chemical and physical characteristics of cement kiln dust must be evaluated on an individual plant basis.
- The results and finds of the present study must be limited to Mugher Cement Enterprise. For Cement kiln dust from other cement factories, further studies and additional tests are required before implementing these results.
- Therefore, findings should be considered indicative rather than definitive for field applications.

Figure 1.3Cement Kiln Dust Dispersion at Mugher Cement Enterprise Premise

1.6. Organization of the study

The presentation of this thesis work is organized in six chapters.

- The first chapter gives a brief description of the thesis background, study area location, problem statement, objectives, scope and limitations of the study and methodology employed.

- Chapter two and chapter three present a background material on expansive soils and soil stabilization respectively. Important findings from previous studies are also included in Chapter three.
- The fourth chapter briefly describes the materials used for the study and laboratory testing procedures followed.
- The fifth chapter presents the test results obtained; analysis of results and discussions are provided with respect to the theoretical background and findings of previous studies.
- Finally, conclusions are drawn and recommendations made from the research .These are presented in Chapter six.

CHAPTER TWO

REVIEW OF RELATED LITERATURE

2.1. REVIEW ON EXPANSIVE SOILS

2.1.1. Introduction

Expansive soil refers to a soil that has the potential for swelling and shrinking due to changing moisture condition. The major problem is that its deformations are significantly greater than elastic deformation, thereby preventing its prediction by simple classical elastic or plastic theories. The movement is usually in an uneven pattern and of such magnitude as to cause extensive damage to the structures and pavements resting on them (Nelson and Miller, 1987). Expansive soils cause more damage to structures particularly pavements and light buildings than any other natural hazard, including earthquakes and floods. It has been reported that the damage caused by these soils contribute significantly to the burden that natural hazards pose on the economy of countries where the occurrence of these soils is significant (Nelson and Miller, 1992). Ethiopia is amongst the list of countries where the occurrence and spatial distribution is recognized as significant.

Expansive soils can be found anywhere in the world but they are basically confined to semiarid and arid regions. These areas are naturally characterized by marked dry and wet seasons with low rainfall, and poor drainage. The climatic condition is such that the annual evapo-transpiration exceeds the precipitations (Chen, 1988).

Two groups of parent materials have been associated with the formation of expansive soils. The first group comprises sedimentary rocks of volcanic origin which can be found in North America, South Africa and Israel, while the second groups of parent materials are basic igneous rocks found in India and Southwestern USA (Chen, 1988). The most well-known example of expansive soils is the black cotton soil which is dark grey to black in color and the name originated from India where locations of these soils are favorable for growing cotton.

2.1.2. Origin of Expansive Soils

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

2.1.2.1. Parent Material

The constituents of the parent material during the early and intermediate stages of the weathering process determine the type of clay formed. The nature of the parent material is much more important during these stages than after intense weathering for long periods of time (Chen, F.H., 1988).

The parent materials that can be associated with expansive soils are classified into two groups. The first group comprises basic igneous rocks and the second group comprises the sedimentary rocks that contain montimorillonite as a constituent (Chen, 1988).

Basic igneous rocks are comparatively low in silica, generally about 45 to 52 percent. Rocks that are rich in metallic base such as the pyroxenes, amphiboles, biotite and olivine fall within this category. Such rocks include the gabbros, basalts and volcanic glasses (Chen,1988).

Sedimentary rocks that contain montimorillonite as constituent include shale and clay stones. Limestone and marls rich in magnesium can also weather to clay. These parent materials contain varying amounts of volcanic ash and glass, which can subsequently be weathered to montimorillonite. The volcanic eruptions sent up clouds of ash, which fell on the continents and sea. Some of fine grained sediments which accumulated to form these rocks also contain montimorillonite derived from weathering of continental igneous rocks and from ash, which fell on the continental areas (Chen, 1988).

2.1.2.2. Weathering and Climate

The weathering process by which clay is formed includes physical, biological and chemical process. The most important weathering process responsible for the formation of montmorillonite is the chemical weathering of parent rock mineral. The parent material generally consists of ferromagnesium mineral, calcite feldspars, volcanic glass, volcanic rocks and volcanic ash. The formation is aided in alkaline environment, presence of magnesium ion and lack of leaching. Such condition is favorable in semi-arid regions with relatively low rain fall or seasonal moderate rainfall particularly where evaporation exceeds precipitation. Under these conditions enough water is available for the alteration process but the accumulated cations will not be removed by rain water (Chen, 1988).

2.1.3. Clay Mineralogy

Clay minerals are classified as follows:

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

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

Illite, also a three-layer clay mineral, exhibits very minor interlayer swelling. This results from the presence of non-hydrated K+ ions in the interlayer positions within the hexagonal openings of the tetrahedral layer. The K+ satisfies charge deficiencies residing mainly on the tetrahedral layer and is thus tightly bonded. The clay mineral responsible for most of the damages incurred by expansive soils is montmorillonite. The mineral configurations of these clay types are shown in Figure 2.1 (Grim, 1962).

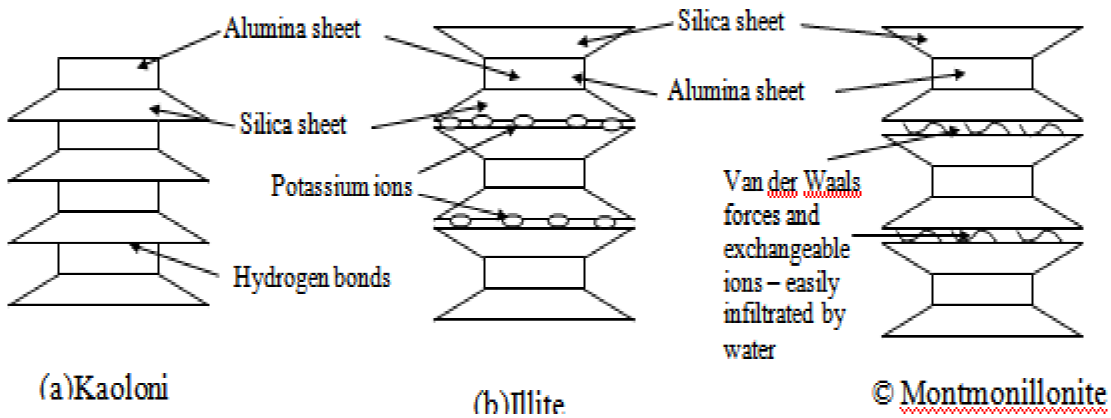


Figure 2.1 Typical structural configurations of clay minerals

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

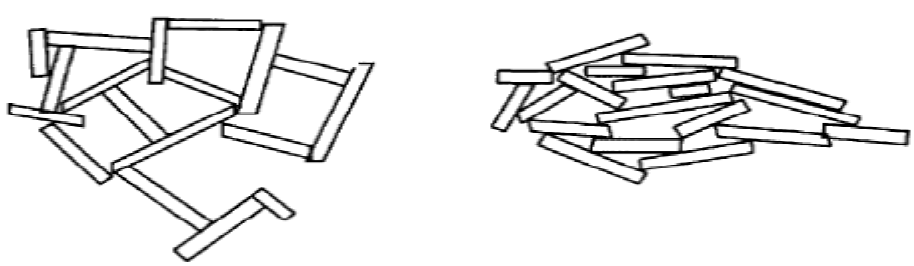


Figure 2.2 Flocculated and Dispersed Type Soil Structures

2.1.4. Clay Structure

Dispersed and flocculated structures are the two elementary structures of clay particles as shown on Figure 2.2. If the magnitude of the attractive force is lesser than the magnitude of the repulsive force, the final result will exactly be repulsion. The individual clay particles will settle and form a dense layer at the bottom; however, they will separately remain from their surroundings. This is referred to as the dispersed state of the soil. On the other hand, flocs will beformed and these flocs will settle to the bottom, if the net force between the particles is attraction, which is called flocculated clay (Das, 2008).

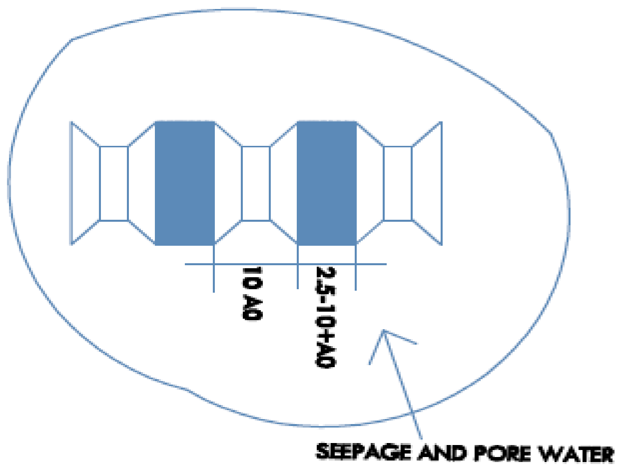


Figure 2.3Deflocculated clay mineral showing surface & interlayer water

2.1.5. Diffuse Double Layer

Every soil particle is found in nature surrounded by water. The molecules gaining the specialty of being dipole take place in the centers of positive and negative charges of water molecules and prevent matching up. As a natural result of that, hydrogen (positive end) of the water molecules is attracted by the negative charge on the surface of the soil particle. The definite pattern of the arrangement of the water molecules are achieved in immediate vicinity of the boundary between solid and water. A considerable and attractive force, which prevents increasing the distance of water molecule from the surface, helps more than one layer of water molecules to stick on the surface. Then the clay particle is surrounded by the electrically attracted water. That phenomenon is called as the diffused double-layer of water (Murthy, 2002) as shown on Figure 2.3.

2.1.6. Cation Exchange Capacity (CEC)

The cations can be arranged in a series in terms of their affinity for attraction as below that shows some cations are strongly more attracted than the others:

$Al^{3+} > Ca^{2+} > Mg^{2+} > NH^{4+} > K^{+} > H^{+} > Na^{+} > Li^{+}$, (Das, 2008).

This series illustrate that hydrogen clay can be transformed to sodium clay by a constant percolation of water including dissolved Na salts. Such changes can facilitate decreasing the permeability of a soil. However, all adsorbed cations are not exchangeable. The quantity of exchangeable cations in a soil determines exchange capacity (Murthy, 2002).

CEC is term that referred to the quantity of exchangeable cations required to balance the charge deficiency on the surface of the clay particles. Higher CEC, higher surface activity and consequently higher water absorption potential usually belong to clays with larger specific surface area. Cation exchange capacity (CEC) values have been used to explain the effect of the new pozzolanic reaction products on the particle size and the swell potential of the treated soils.Typical values of CEC for three basic clay minerals are tabulated in Table 2.1as follow (Nelson,D.J., and Miller,J.D., 1992)

Table 2.1Typical CEC values of basic clay minerals

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

2.1.7. Identification of Expansive Soils

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

2.1.7.1. Field Identification

Currently it is evident that soil deposits can be recognized in the field through visual inspection. Some of the important field identification method that indicates the potential for expansiveness of a soil is the following:

Polygonal pattern of surface cracks in the dry season. Crack 2.5 cm wide and over 1 m depth is not uncommon. The cracks close down after rainy season. 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 samples of the soil are sticky and it will be relatively difficult to clean the soil from the hands. They are very hard when dry.

2.1.7.2. Laboratory Identification

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

2.1.7.2.1. Mineralogical Identification

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

The various techniques under these methods are (Chen, F.H., 1988; Nelson, D.J. and Miller, J.D., 1992):

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

are usually used to study the amount and type of clay minerals, by which the swelling characteristics of clay can be identified. These methods are mostly used for academic or research purposes. They are time consuming, require expensive test equipment, and the results are interpreted by specially trained technicians. Thus, for ordinary engineering works these methods are generally not often used.

2.1.7.2.2. Indirect Methods

In this method simple soil property tests can be used for the evaluation of identifying expansive clay soils. Such tests are easy to perform and should be included as routine tests in the investigation of expansive soils. Such tests may include (Chen, F.H., 1988; Nelson, D.J., and Miller, J.D 1992): Atterberg limits test, and grain size distribution, free swell test, free swell index test, free swell ratio test.

2.1.7.2.3. Direct Methods

These methods offer the most useful data by direct measurement; and tests are simple to perform and do not require complicated equipment. Testing should be performed on a number of samples to avoid erroneous conclusions. Direct measurement of expansive soils can be achieved by the use of conventional one-dimensional consolidometer carried out on representative undisturbed samples. These methods are usually performed through actual measurement of swelling pressure and volume change of soil.

2.1.8. Classification of Expansive Soils

Parameters determined from expansive soil identification tests have been combined in a number of different classification schemes. The classification system used for expansive soils are based on indirect and direct prediction of swell potential determined by correlation with other test results of swell test data. There are a number of classification systems. The following are some of the common classification methods.

2.1.8.1. General Classification

Soils are classified in the general schemes; Unified Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials Method (AASHTO) according to index properties. Soils rated CL or CH by USCS, and A6 or A7 by AASHTO, may be considered potentially expansive (Nelson, and Miller, 1992).

i) Unified Soil Classification System

Table 2.2 Soil classification as per USCS

Category	Soil Classification System
Little or no expansion	GW,GP,GM,SW,SP,SM
Moderate expansion	GW,SC,ML,MH
High volume change	CL OL,CH,OH
No rating	PT

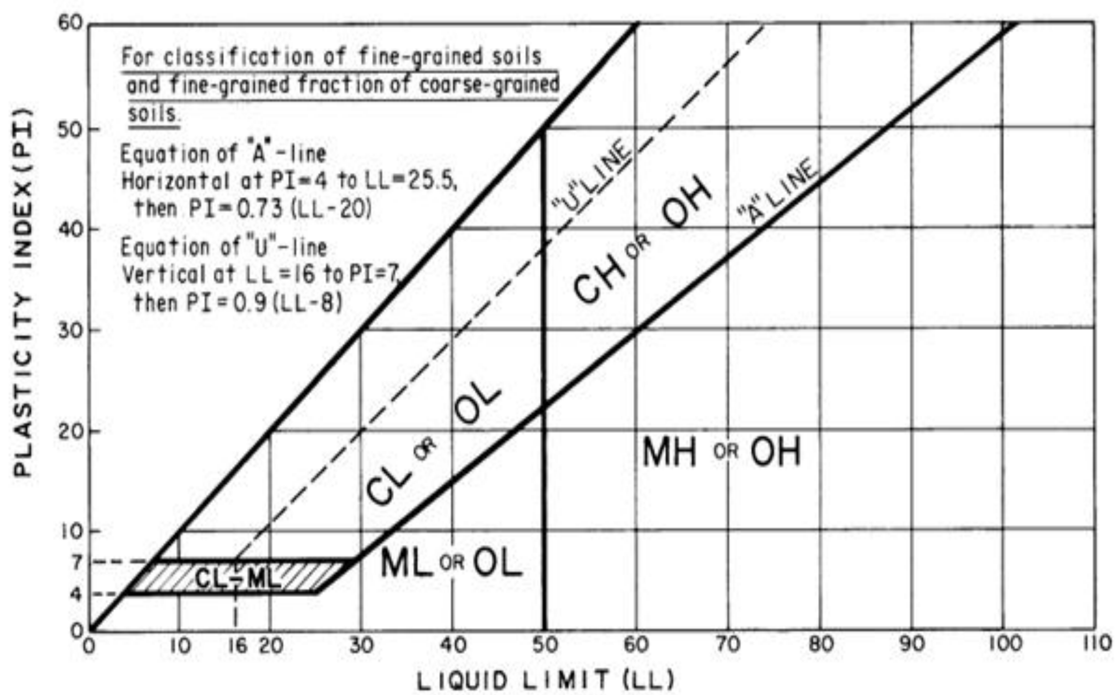


Figure 2.4. Plasticity Chart

ii) AASHTO Classification

This classification is most widely used for general classification system. Soils rated A-6 orA-7 can be considered potentially expansive (Nelson, D.J., and Miller, J.D 1992)

Table2.3 AASHTO Soil Classification

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

2.1.8.2. Specific Classification

i)Method of Chen

Chen (1988) relates swelling potential with plasticity index for identifying expansiveness.

Table 2.4 Relation between the swelling potential of clays and plasticity index

Swelling Potential	Plasticity index
Low	0-15
Medium	10-35
High	20-55
Very high	35 and above

ii) USBR Method

Holtz and Gibbs developed this method based on direct correlation of observed volume change with colloid content, plastic index and shrinkage limit. As shown in Table2.5 below (Chen, 1988; Ranjanand Rao, 2002).

Table 2.5 Classification based on bureau of reclamation method

Colloid Content,(%)	Plasticity Index,(%)	Shrinkage Limit,(%)	Probable Expansion,(%)	Degree of expansion
<15	<18	>15	<10	Low
13-23	15-28	10-16	10-20	Medium
20-31	25-41	7-12	20-30	High
>28	>35	<11	>30	Very high

iii) Method of Daksanamurthy and Raman (1973)

The identification of expansive soils is based on single index method using only liquid limit. The classification according to this method is presented in Table 2.6 below.

Table 2.6 Relation between the swelling potential of clay and the liquid limit

Swelling potential	**Liquid limit**
Low	20<LL≤35
Medium	35<LL≤50
High	50<LL≤70
Very high	LL>70

iv) Method of Skempton

The method is developed, by combining Atterberg limits and clay content into a single parameter that is Activity.

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

Table 2.7 Relation between clay activity and potential of expansion

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

v) Method of Seed et al

Seed et.al as cited in Chen,1988 has developed a chart based on activity and percent clay size .the activity here is defined as:

$$Ac=\frac{PI}{C-10}$$

Where: Ac=activity

C=percentage of clay size finer than 0.002mm

PI=plasticity index

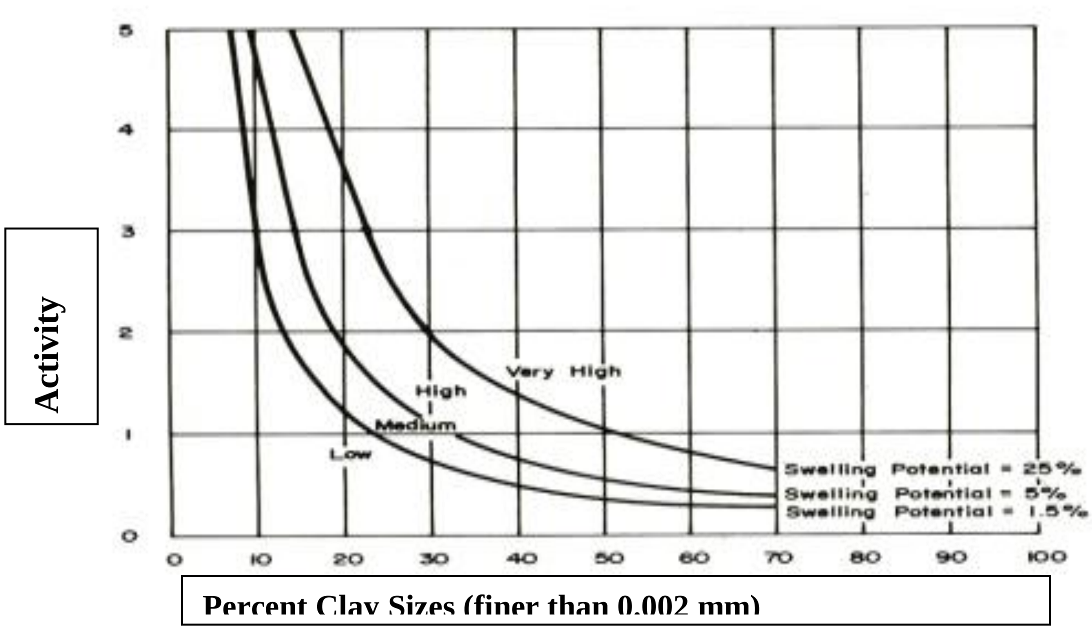


Figure 2.5 Classification chart for swelling potential

2.1.9. Distribution of Expansive Soil

Expansive soils are widespread in African continent, occurring in South Africa, Ethiopia, Kenya, Mozambique, Morocco, Ghana, Nigeria etc. In other parts of the world, expansive soils have been widely reported in countries like USA, Australia, Canada, India, Spain, Israel, Turkey, Argentina, Venezuela etc. (Teferra and Leikun, 1999).

The aerial coverage of expansive soils in Ethiopia is estimated to be 24.7 million acres (Lyon associates, 1971; as cited by Nebro, D., 2002). They are widely spread in central part of Ethiopia following the major truck roads like Addis-Ambo, Addis-Wolliso, Addis-Debrebirhan, Addis-Gohatsion, Addis-Modjo.Also areas like Mekele and Gambella are covered by expansive soil. The distributions are shown in Figure 2.6 below.

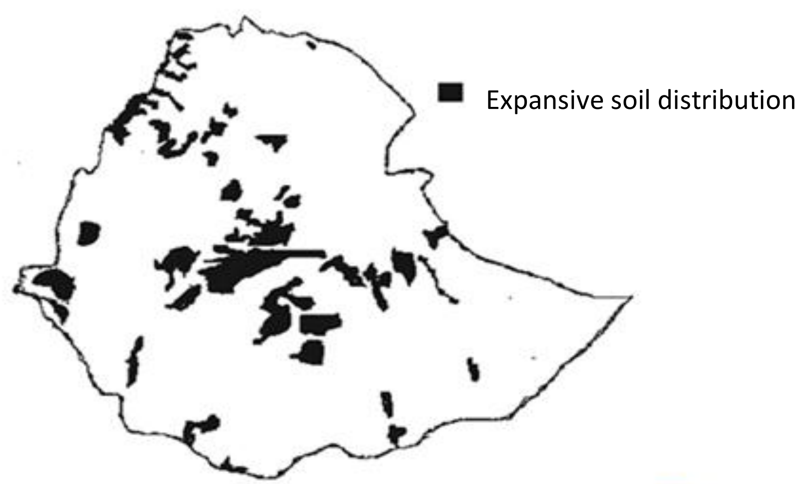


Figure 2.6 Distribution of Expansive Soil in Ethiopia (Tilahun, 2004; Teklu, 2003)

2.1.10.Problem Associated with Expansive Soils

The most vulnerable engineering structure susceptible to damage due to shrink-swell potential (volume change) of expansive soils are highway pavements because of their light weights and very shallow foundations. Expansive soils fail to support the loads transmitted from the pavement structure and cause excessive deformation beyond permissible limits.

A road pavement is expected to fulfill two basic functions: to remain structurally sound to satisfy the engineer's design; and to provide a satisfactory ride to the road user during its design life. Unfortunately and very frequently in practice, road pavements do not fulfill their basic functions especially when they are constructed on expansive soils as shown in Figure 2.7. There is, therefore, a need to incorporate an economical soil stabilization process before roads are constructed to forstall unfortunate results, because experience has clearly shown that the cost of repairs (of bad roads) is very much higher than the cost of a proper initial design, and the results are much less satisfactory (Krazynski, 1980).

Figure. 2.7 A View of Road Undergoing Volume Change

2.1.11. Treatment of Expansive Soil

Treatment of expansive soil consists of using, chemical additives, pre wetting, soil removal replacement, moisture control, compaction, and surcharge loading.

Chen (1988) also proposed different options that can be applied to treat expansive sub-grade soils either by replacing, modifying or protecting the soil moisture variation.

ERA Manual (2002) also suggests separate treatment methods based on the average annual daily traffic design and the removal of expansive soil will be based on the finished road level .EAR manual also recommends realignment if the area covered with expansive soils is limited in extent.

AACRA (2004) has also proposals for mitigation measures .Excavate and remove the potential expansive soil and replace with dry, dense unweathered shale and dry dense clays backfill materials or treat the sub-grade expansive soil. The depth of soil stabilization should be determined using the sub-grade information such as thickness and swelling potential of the swelling materials and these materials depends on the index (PI) of the sub-grade soil and the traffic class of the proposed road as shown in the Table 2.8.

Table 2.8 Treatment depths below the normal sub -grade level of high swelling potential soils (AACRA-2004).

Highly Trafficked (Primary Roads)		**Lightly Trafficked (Secondary Roads)**	
Plastic index (PI)	Depth of Treatment (m)	Plasticity index (PI)	Depth of Treatment (m)
10-20	0.6	10-30	0.6
20-30	0.9	30-50	0.9
30-40	1.2	*Above 50	1.2
40-50	1.5		
*Above 50	1.8		

**Excavation and replace by with selected impermeable material*

2.2. Review on Soil Stabilization

2.2.1. Introduction

Generally, the long-term performance of any geotechnical structures depends on the soundness of the underlying soils. Unstable/expansive soils can create significant problems for pavements or structures. The black cotton soil is an expansive soil with low bearing capacity when it is subjected to moisture, has the ability to absorb and dissipate water with subsequent change in volume. Construction of any structure on this type of soil requires either replacement of the soil by importing a better foreign one or by addition of chemical(s) that will improve the soil towards the desired property.

The successful construction of highways requires the construction of a structure that is capable of carrying the imposed traffic loads. One of the most important layers of the road is the actual foundation, or subgrade. The main function of the subgrade is to give adequate support to the pavement and for this; the subgrade should possess sufficient stability under adverse climate and loading condition. If these structures are founded on soil with low bearing capacity, they are likely to fail either during or after construction, with or without application of wheel load on them. Where the pavement is founded in an inherently weak soil, this material will be removed and replaced with a stronger granular material or improving the soil towards the desired property by addition of chemicals.

This removal and replacement technique can be both costly and time consuming. Where aggregates are scarce, the use of these non-renewable resources is viewed as non-sustainable, particularly if haulage distances are significant.

An alternative to the removal and replacement option is to chemically stabilize the host material. This eliminates the requirement to replace the material, and ensures the engineering characteristics and performance of the host material is enhanced to allow for its use within the pavement structure.

2.2.2. Soil Stabilization

When unsuitable materials are encountered measures like avoiding the route, redesigning the pavement with thicker sections or replacing the poor soil with good quality materials are viable but increasingly expensive options. With improved technological advances and concern for depletion of non-renewable resources, improving the properties of soil using chemical additives is gaining increased popularity (Caterpillar, 2006).

Soil stabilization is the improvement of the original soil properties to meet specific engineering requirements. It is aimed at the enhancement of the engineering properties of deficient soils to enable them perform and sustain their intended engineering use (Yoder and Witczak, 1975; Gillot, 1987; Osinubi, 1995; Nicholas and Lester, 1999). Its objectives are: improvement of the strength of the soil and bearing capacity, decreasing permeability and water absorption, and to increase the durability under varying moisture content. Soil stabilization can be taken as alternative to borrow selected materials and it has advantage that the effect to the environment is reduced and in areas where selected materials are scarce, stabilization have comparative economic advantage.

2.2.3. Method of Soil Stabilization

2.2.3.1. Mechanical Stabilization

The stabilization of soils through mechanical means includes compaction, vibration (of various techniques) and blasting. Mechanical stabilization by compaction is the densification of the soil by the application of mechanical energy. It involves the modification of the water content as well as the gradation of the soil.

Cohesionless soils are compacted by some means of confining the soil coupled with vibration energy. In the field, hand-operated vibrating plates and motorized vibratory rollers of various sizes are very efficient in compacting sand and gravely soils. Large falling weights have been used to dynamically compact loose granular fills (Markwick, 1944).

Fine grained cohesive soils are compacted in the field by using common compaction equipment like hand-operated tampers, sheeps foot rollers, rubber tyred rollers and other types of specialty equipment. Considerable compaction can also be obtained by proper routing of the hauling equipment over the loose soil. The objective of mechanical compaction is the improvement of the engineering properties of the soil mass, and the advantages which are obtained through compaction are as follows :

- Reduction in settlement due to reduced void ratio
- Increase in soil strength
- Reduction in shrinkage.

2.2.3.2.Chemical Stabilization

The transformation of soil index properties by adding chemicals such as cement, flyash lime, cement kiln dust, or a combination of these, often alters the physical and chemical properties of the soil including the cementation of the soil particles. There are the two primary mechanisms by which chemicals alter the soil into a stable subgrade:

- Increase in particle size by cementation, internal friction among the agglomerates, greater shear strength, reduction in the plasticity index, and reduced shrink/swell potential.
- Absorption and chemical binding of moisture that will facilitate compaction.

Generally, the addition of chemical compounds, to expansive soils increase the strength, bearing capacity and durability of the soil. These organic or inorganic chemical compounds perform as cementations and bonding agents or water proofers/repellants (Slate and Johnson, 1953).

Chemical stabilization are classified in to two (Slate and Johnson, 1953);

- Organic compounds including resinous and bituminous materials act as water proofers and sometimes behave similar to glue. These water proofing agents reduce the capacity for water intake and help the soil to retain its dry strength, even under wet conditions.
- Inorganic agents employed for soil stabilization include cement kiln dust, Portland cement, lime, slag, sodium silicate, marble dust, sugar cane molasses, fly ash, etc. Their functions are to reduce plasticity and facilitate densification (Balogun, 1991).

2.2.4. Mechanism of Stabilization

The soil stabilization mechanism can be portrayed a coating and/or binding of soil particle to form another output soil with improved characteristics (Texas DOT,2005).

The efficiency and effectiveness of the stabilizer depends upon the type of the soil to be stabilized, the type and properties of stabilizer, and the associated moisture content during compaction as well as the long-term moisture content. Furthermore, the effectiveness of the stabilizer can be measured by its ability to provide enough calcium to chemical reaction. Lime, Portland cement and fly ash materials are the most frequently used chemical stabilizers. Fly ash that possesses self-cementing property that can stabilize/treat soil without cement or lime are called class C fly ash, whereas that often used either with lime or cement in order to make it more reactive are called class F or non-cementing fly ash. The mechanism of stabilization for these stabilizers is almost similar regardless of few different processes. The overall stabilization process can be summarized into four different processes (Mallela et. al., 2004). All four processes mentioned below will occur in cement treated subgrade soils, whereas in case of lime treated soils cementitious hydration will be absent due to the lack of calcium aluminate hydrate

(C-A-H) after hydration of the stabilizer.

i) Cation exchange,
ii) Flocculation and agglomeration,
iii) Cementitious hydration, and
iv) Pozzolanic reaction.

i) Cation exchange

Cation exchange includes an immediate reaction of the clay with the stabilizer within few minutes of mixing, resulting in a soil with improved texture. The tetrahedral (T) and octahedral (O) combination of clay minerals in 1:1 (1T and 1O) or 2:1 (2T and 1O) have charge deficiency that results in the attraction of the cations or water molecule. Generally, sodium or potassium (Na+ or K+) are prevalent in clay minerals along with water. However, these cations can be replaced by the higher valance cations like Al+3, Ca+2, Mg+2 etc. so called cation exchange. During this process calcium rich chemical stabilizer provides enough cations to replace the monovalent cations resulting in a reduced thickness of diffused double layer (Geiman, 2005). The calcium is released in suspension of stabilizer-soil-water and will be available for the stabilization of soil. The general reaction of the cement with water that yields calcium is presented in Eq. (2.1) and (2.2).

$$C_3S + H \Longrightarrow C\text{-}S\text{-}H + Ca(OH)_2 \tag{2.1}$$

$$C_2S + H \Longrightarrow C\text{-}S\text{-}H + Ca(OH)_2 \tag{2.2}$$

Where, H= H_2O, C = Ca, S= SiO_2, C_3S = tri-calcium silicate, C_2S= di-calcium silicate and C-S-H = $C_3S_2H_3$.

ii) Flocculation and agglomeration

Flocculation and agglomeration is the rearrangement of the clay particles from face to face orientation to more compact edge-face orientation. The fine grained soil changes to the more coarse grained with much more improved strength/stiffness as well as workability

(Brooks, 2009). As cation exchange, flocculation and agglomeration is also a short-term process, which takes place within few hours of mixing the stabilizer and water with subgrade soil. The flocculation and agglomeration of clay particles is shown inFigure2.8.

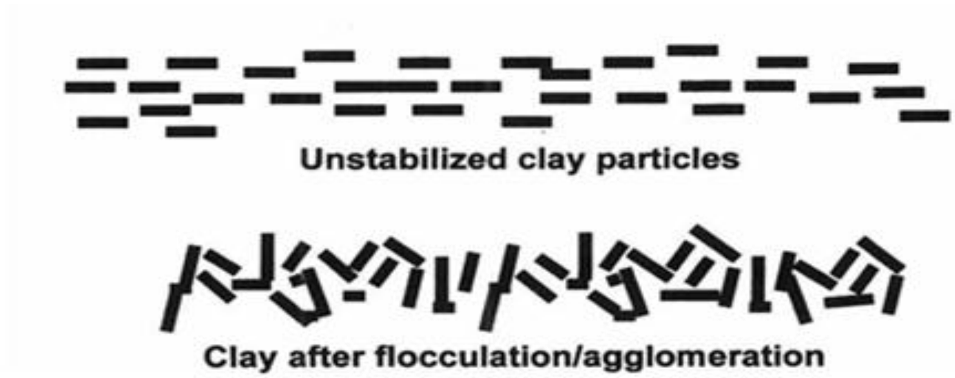


Figure 2.7 Flocculation and agglomeration of clay particles (Sanjay Kumar Dhakal, 2012)

iii) Cementation Hydration

In addition to lime, cement contains calcium-aluminate-hydrate (C-A-H) that takes part in the further stabilization of the flocculated clay particles by yielding the glue like structure with calcium-silicate-hydrate (C-S-H). The strength provided by cementitious hydration in cement treated soil is extra strong as compared to the lime treated soils that is the main reason why cementtreated/stabilized soil have better strength compared to any other treated/stabilized soils. The rapid gain in strength continues from the day of mixing till a month and may continue up to few years .Sometimes, carbonation occurs in lime treatedsoils that is undesirable reaction at the start of stabilization because of formation of relatively insoluble carbonates rather than hydrates (Mallela, 2004).

iv) Pozzolanic Reaction

Pozzolanic reaction is a long-term process that produces more stable hydrates and aluminates of calcium after few months of mixing of soil, stabilizer and water. The PH environment in the system initiates further reaction of the silica and alumina with the clay particles, hence proving extra strength to the stabilized soils .The minimum PH of 12.4 is necessary in order to maintain the pozzolanic reaction (Eades and Grim, 1960 cited in Harty, 1970). Fly ash stabilization of the soil is similar to cement; however the strength provided is less than the cement. Depending upon the reactivity, the fly ashes are classified as self-cementing (C-class) and/or non-self-cementing (F-class). Generally, C-class fly ash is applied with either cement or lime, whereas F-class does not includes any activating stabilizers.

2.2.5.Cement Kiln Dust Chemical Stabilizer

Cement Kiln dust is a by-product in the production of cement clinker .Disposal of cement kiln dust is an environmental problem. The utilization of this waste material has received increasing attention because it not only solves a potential soil waste problem but also provides an alternative stabilizing agent using in chemical stabilization of problematic soils.

The dust is a particulate mixture of partially calcined and unreacted raw feed, clinker dust and ash, enriched with alkali sulfates, halides and other volatiles (Wayne S. Adaska May2008).It is derived from the same raw materials as Portland cement but, as the cement kiln dust fraction has not been fully burnt, it differs chemically from the former.

It is a fine powdery material similar in appearance to Portland cement. There are two types of cement kiln processes; wet-process kilns, which accept feed materials in slurry form; and dry – process kilns, which accept feed materials in a dry, ground form. AtMugher Cement Factory the process is dry and large quantities of cement kiln dust are produced during the manufacture of cement clinker as shown in Figure 2.9.

Figure 2.8 Cement Kiln Dust at Mugher Cement Enterprise

Several factors influence the chemical and physical properties of CKD. Because plant operations differ considerably with respect to raw feed, type and operation, dust collection facility, and type of fuel used, the use of the terms typical or average CKD when comparing different plants can be misleading .The dust from each plant can vary markedly in chemical, mineralogical and physical composition (Klemm, 1993).

Ismaiel 2013, states that CKD having self-cementing characteristics reacts with soil in a manner similar to Portland cement. Typically, CKD has approximately one-third of the amount of cement oxides (CaO, Al_2O_3, SiO_2, and Fe_2O_3) present in Portland cement. The primary value of CKD is its cementitious property. Depending on the concentration of free lime (CaO), CKD can be highly cementitious. Therefore, it can be used as a replacement for cements. The formed cementitious compounds obtained as a result of the chemical reactions between the silica and the alumina existing in the soil and the additives reduced the volume of the void spaces and participate in the soil particles (Ismaiel, 2013)

2.2.6. Previous Similar Research

There is considerable information regarding the application of cement kiln dust. For the present work the studies of Ismaiel (2013) is reviewed.

Accordingly the addition of 14 % by total weight of cement kiln dust to expansive clay soil resulted in a reduction of the liquid limit, plastic limit and the plasticity index values from 91.60 %, 61 % and 30.60 % to 34.50 %, 28.20 % and 6.3 % respectively.

Similarly the addition of cement kiln dust results in the maximum dry density reduction from 1.79g/$cm^3$to 1.68 g/cm^3 and an increment of the optimum moisture content from 15.10 % to 25.00 %. Free swell reduction showed from 80.00 % to zero %.

Parsons, (2004) conducted a major laboratory tests for eight different soils with classifications of CH, CL, ML, SM, and SP blended with cement kiln dust. Effective option for improvement of soil properties, based on the test result strength and stiffness were improved and plasticity and swell potential were substantially reduced.

McCoy and Kriner (1971) studied soil stabilization with soil-CKD mixes containing 3,8, and 10 % CKD . Engineering properties ,such as the unconfined compressive strength, moisture-density relation, liquid limits (LL),plastic limit(PL),plasticity index(PI),and shrinkage limit were tested The study found that the result of CKD was potentially promising in stabilizing soils for sub-grade application.

Beside the above studies, the work of Ehitabezahu Nigussie was also reviewed. Significant improvement on strength due to stabilization was observed.

CHAPTER THREE

DESIGN AND METHOD OF RESEARCH

3.1. Introduction

Different types of soil formations are highly dominated by the surrounding environmental factors, such as climatic conditions and the geologic and physiographic set up of the specific area. These have impact on the formation of expansive soils. Accordingly a considerable part of Addis Ababa is covered by expansive soils.

3.2. Location of the study area

The study area is Addis Ababa, the capital city of Ethiopia, bounded approximately by the UTM coordinates 964000N, 1002000Nand 456000E, 496000E. The city overlies at the western margin of the Main Ethiopian Rift and is a part of the western highland of Ethiopia. The native soil sample is collected from Addis Ababa International Airport Hangar project.

3.3. Climate

Ethiopia is classified into five climatic zones (EMA, 1981). These include "Kur"(Alpine),above 3000m mean sea level; "Dega" (Temperate), 2300m to about 3000m; "Weina Dega" (Sub tropical), 1500m to about 2300m; "Kolla" (Tropical), 800m to about 1500m and "Bereha" (Desert), less than 800m. Most parts of Addis Ababa fall under the Weina Dega (Sub tropical) category.

3.4. Geology of the area

Addis Ababa is located on a plateau with an elevation ranging from 2000 to 2800m a.s.l on the shoulder of the western Main Ethiopian Rift (MER) escarpment. The morphology of Addis Ababa is a direct reflection of the different volcanic stratigraphic successions, tectonic activities and the action of erosion between successive lava flows (Tamiru et al., 2006). The physiographic map of Addis Ababa shows that the city is founded on an area with a well-developed morphology as shown in Figure 3.1. It is surrounded by high rising mountain systems in all directions and the center of the city lies on an undulating topography with some flat land areas. The urban area of the city is deeply dissected by numerous valleys formed by the river systems crossing the city from north to east.

Intoto mountain ridge forms the northern boundary of the city following the East-West trending Ambo-Kassam major fault system. The elevation of this ridge ranges from 2600 to 3200 m. The volcanic mountains; Mt. Wechecha in the west, Mt. Furi in the south-west and Mt. Yerer in the southeast are the high massive volcanic centers rising to elevations of 3385m, 2839m and 3100m a.s.l, respectively.

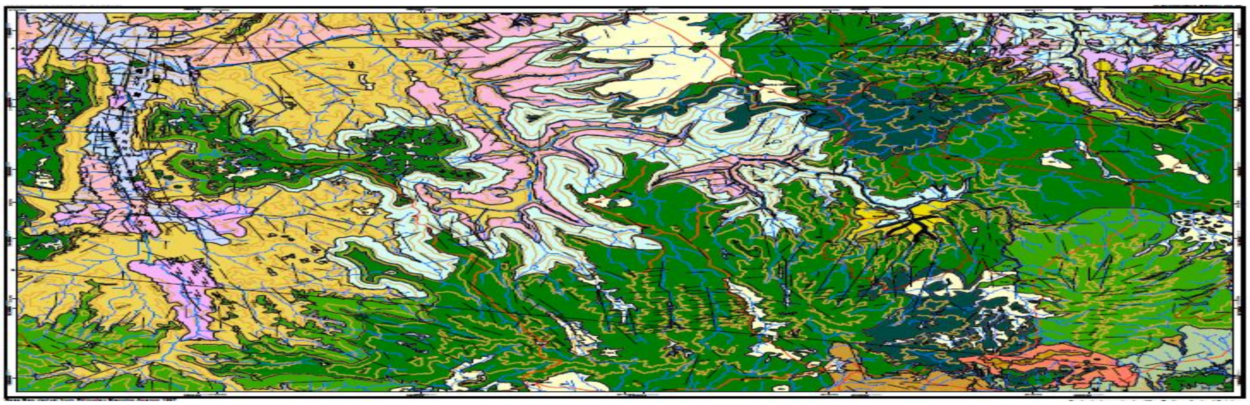
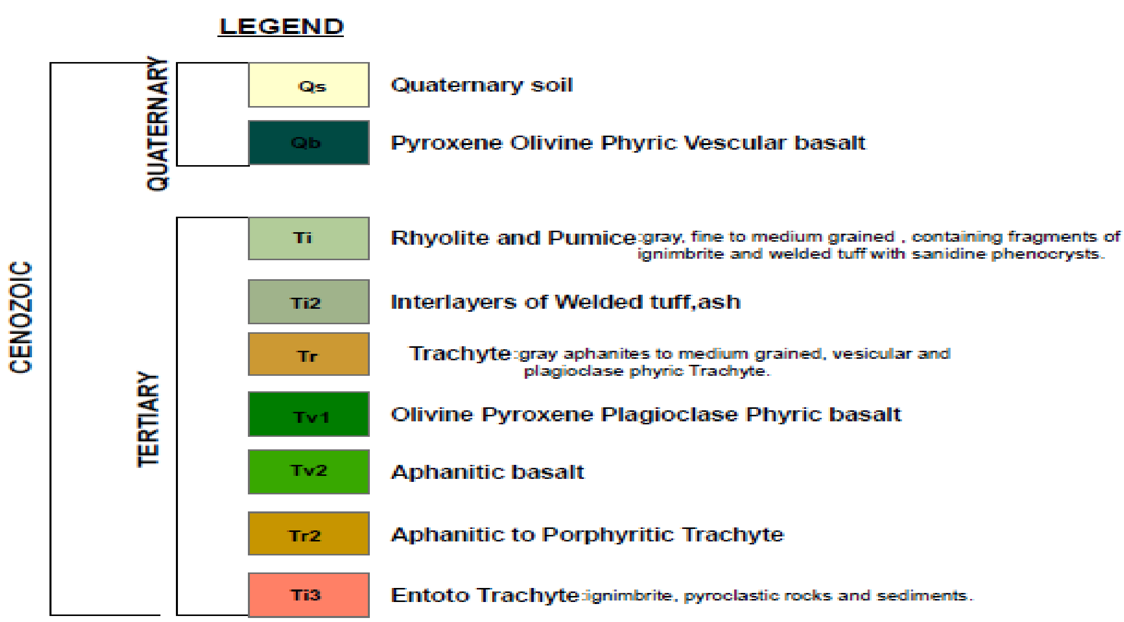
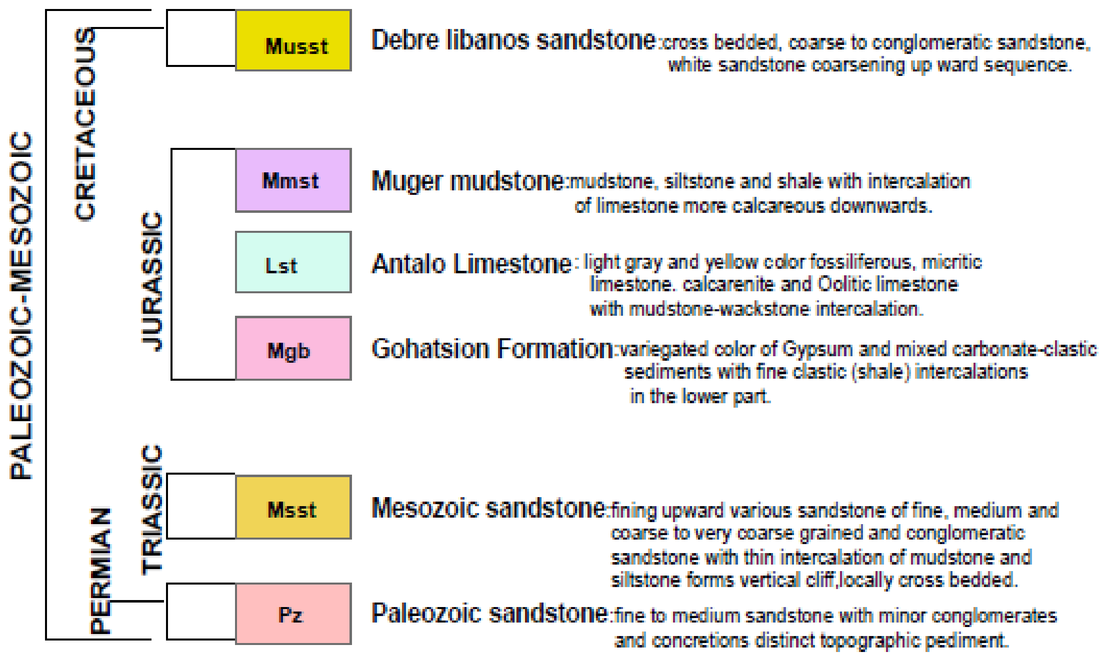


Figure 3.1 Geological Map of Addis Ababa (Assiged, 2007)





3.5.Engineering Properties of expansive soil in the study area

Recent researches on the engineering properties of expansive soils of Ethiopia revealed that, the expansive soils in most parts of the country are classified as inorganic clays of high plasticity. The clay content of the soil is found to be as high as 80% and the amount of montmorillonite for Addis Ababa expansive soil is 70-80% (Legesse, 2004) [Refer Figure 3.2 below].These soils have the ability to hold significant amount of water that affects the shear strength, as well as, shrinkage and swelling characteristics. The mineralogical composition of Addis Ababa Expansive soils is summarized in Table3.1 below (Luelseged, 1990).

Table 3.1.Mineralogical Composition of Addis Ababa Expansive Soils

Clay Mineral	**Mineral content (%)**	
	Black clays	Gray clays
Montimorillonite	70-80 %	70-75 %
Illite	10 – 15 %	25-30 %
Kaolinite	10-15 %	10 -15 %

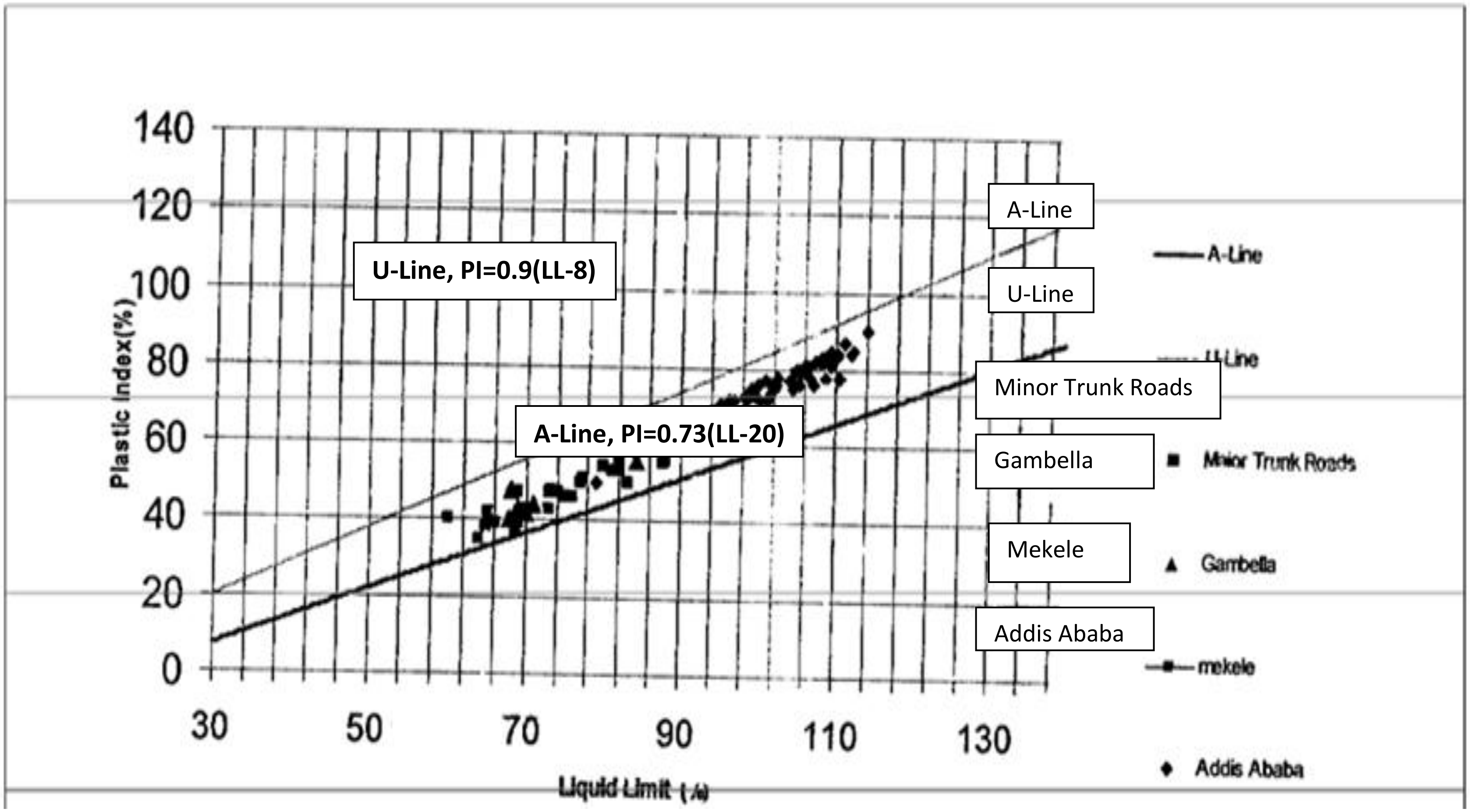


Figure 3.2 Plasticity chart of expansive soil of Ethiopia

CHAPTER FOUR

DATA PRESENTATION, AND INTERPRETATION

Experimental Methodology

In this section description and classification of materials used for the research, testing procedures, and results are presented. The testing of both native and blended soil laboratory has been undertaken at EiABC MRTD laboratory, and AAIT Highway Engineering laboratories. The chemical evaluation of cement kiln dust and natural soil were done at Mugher Cement Enterprise chemical laboratory and at Geological Survey of Ethiopia laboratory respectively. Relevant data of material characterization obtained from secondary resources is acknowledged.

4.1. Sample Collection

Field identification and laboratory tests were conducted to characterize the natural subgrade soil. The field investigation includes identification of soil based on observation of color and soil texture.

4.1.1. Native Soil

A typical expansive soil was collected from Addis Ababa International Bole Hanager Apron project at 2m depth below natural ground surface to execute detail laboratory test as shown in Figure 4.1 below.

Figure 4.1Natural Soil Sample Collection Process

4.1.2. Cement Kiln Dust Chemical Additives

Soil stabilizer chemical additives i.e., CKD used for this research was collected from Mugher Cement Enterprise as shown in Figure 4.2.

Figure 4.2Cement Kiln Dust Collection process at Mugher Cement Enterprise

The laboratory tests conducted on the natural subgrade soil includes;

i) Chemical analysis test

ii) Sieveand Hydrometer analysis

iii) Consistency test

iv) Swell test

- Swell-pressure test
- Percent swell test
- Free swell test

v) Shrinkage test

- Linear shrinkage test
- Volumetric shrinkage test

vi) Moisture-density test

- Strength test
- California bearing ratio
- Unconfined compressive strength

vii) pH test

The following laboratory tests were conducted for Cement Kiln Dust (CKD)

i) Physical Analysis

- Fineness
- Specific gravity
- Loss on ignition

ii) Chemical analysis using X-ray diffraction test

- Aluminum ratio
- Hydraulic modular
- Liquid phase
- Coating index

iii) Consistency test

iv) Dry shrinkage test

v) Moisture test

4.2.Standard Testing Procedures

ASTM and AASHTO standards are used for testing soils in the laboratory. Specific standards considered in the study are summarized in Table 4.1 below.

Table 4.1 Standard Testing Procedures

Test	Test description	ASTM	AASHTO
Grain size Analysis	Standard Test Method for Particle-Size Analysis of soil		AASHTO T-88
Atterberg limit test	Standard Test Methods for Liquid limit, plastic limit and Plasticity index of soils		AASHTO T-89
Soil Classification	Standard practice for classification of soils for engineering purposes		AASHTO
Specific Gravity	Standard specification for specific gravity of soils	ASTM D854-83	AASHTO T-100-95
pH stabilization	Standard test method for using pH to estimate the soil-lime proportion	ASTM D627	T-208
Moisture-Density Relationship	Standard test method for laboratory compaction characteristics of soil using standard effort	D698	T-180
Unconfined Compression Strength test	Standard method of test for unconfined compressive strength of cohesive soil	D 1633, 5102	T-208 D5102*
CBR test	Standard test method for the California bearing ratio		T 193
Swell –Pressure test	Standard test methods for one-dimensional swell or settlement potential of cohesive soils	D4546-96	
Free Swell			IS 2720
Linear shrinkage test		D4943	
Volumetric Shrinkage test	Standard test method for shrinkage factors of soils by the mercury method	D427-39	T160

4.3.Material Characterization

4.3.1.Natural Soil

Soil samples were prepared for test by drying, pulverizing with rubber covered mallet and sieving to the required size as per AASHTO T87.Grain size analysis of samples was determined according to AASHTO T-100. Index properties of soils were determined using Atterberg limit tests using AASHTO T-89. Soil classification was made according to AASHTO and USCS system and the result is summarized as shown in Table 4.2.

Table 4.2 Classification of the Natural Soil

Index and swelling Properties (%)		**AASHTO Soil Classification system**
LL	109%	A-7-5/A-7-6 with rating Fair -to-Poor
PL	37%	
PI	72 %	
More than 35 % passing #200 sieve		

4.3.2. Soil Classification

The most widely used soil classification systems for engineering purposes are American Association of State Highway and Transportation Officials (AASHTO) and Unified Soil Classification System (USCS). The AASHTO system of soil classification comprises seven groups of inorganic soils from A-l to A-7 with twelve subgroups in all. The system is based on particle-size distribution, liquid limit and plasticity index of the soil. Whereas unified Soil Classification System is based on the recognition of the type and predominance of the constituents considering grain-size, gradation, plasticity and compressibility. It divides soil into three major divisions: coarse-grained soils, fine grained soils and highly organic soils.

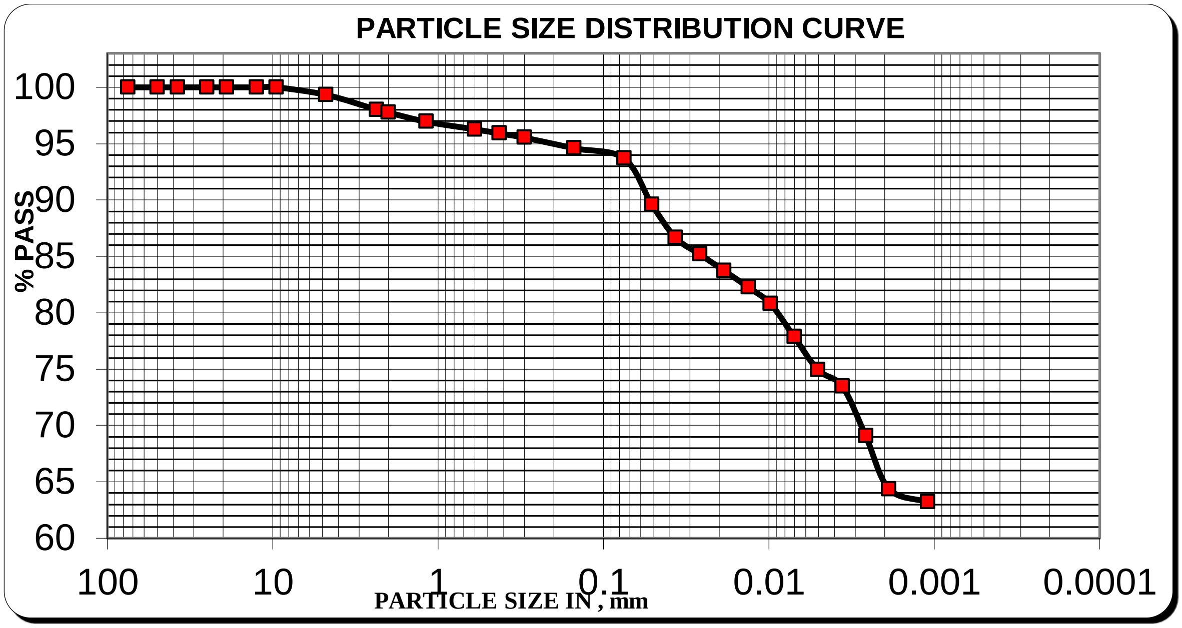


Figure 4.3 Grain Size Distribution of Natural Soil

4.3.3. Grain Size Distribution

Sieve analysis was carried out to determine the grain size distribution of sub-grade soil and used in the classification of the soil.

i) Wet Sieve Analysis

In Accordance with AASHTO T-88, to determine the distribution of coarser particles, 500 g of the natural soil was taken and washed on sieve size of 75 µm. The soil sample is socked in chemical solution (sodium hexa-meta phosphate) for 24 hours. The test results are shown on Table 4.3.

Table 4.3 Wet Sieve Analyses for the Natural Soil

Description		**Natural soil**	
Wt. Before wash (g)		**500**	
Sieve No(mm)	Wt. Retained (g)	% Retained	% Pass
4.75	0	0	100
2	0.7	0.14	99.48
0.425	1.1	0.22	98.66
0.075	1.5	0.30	97.90

ii) Hydrometer Analysis

To determine the distribution of fine particles , 50 gram of air dried sample passing sieve 75 µm is socked in mixture of water and 4 gm of sodium hexa-meta phosphate(dispersing agent)for 24 hrs.Test was undertaken with hydrometer number 151H. The test result for the combined wet sieve and hydrometer analysis are graphed in Figure 4.3. For detail test result refer Annex I, A and B.

4.3.4.Atterberg Limit Tests

Atterberg limits were determined according to ASTM D 4318 on particles passing No. 40 sieve. Curing durations were varied up to 7 days and the test results are shown in Figures 4.4 and 4.5 below.

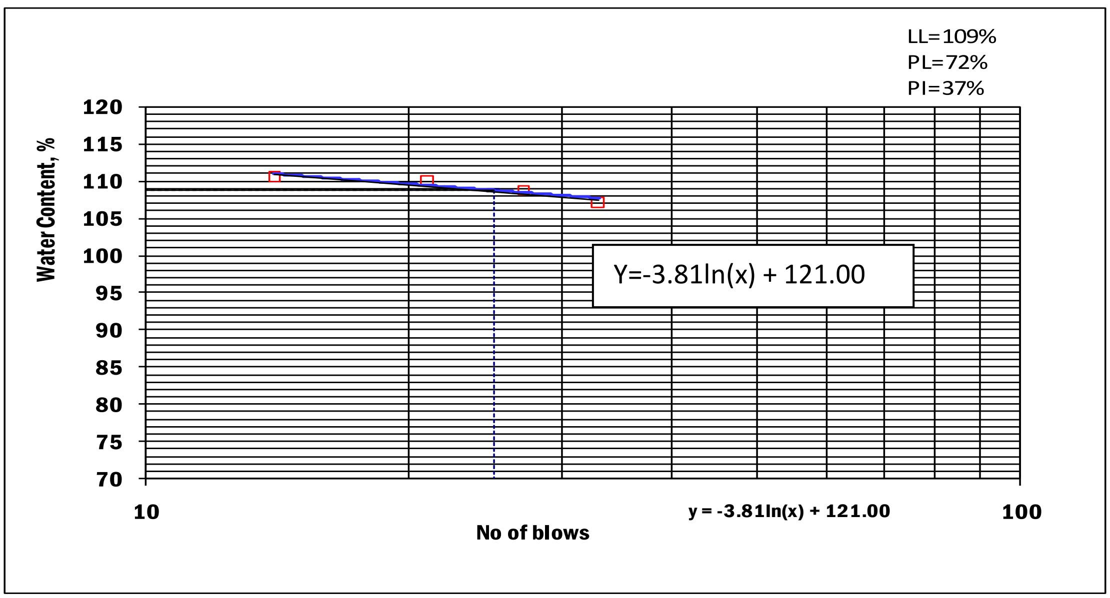


Figure 4.4 Plot of Liquid Limit for the Natural Soil with Zero-curing period

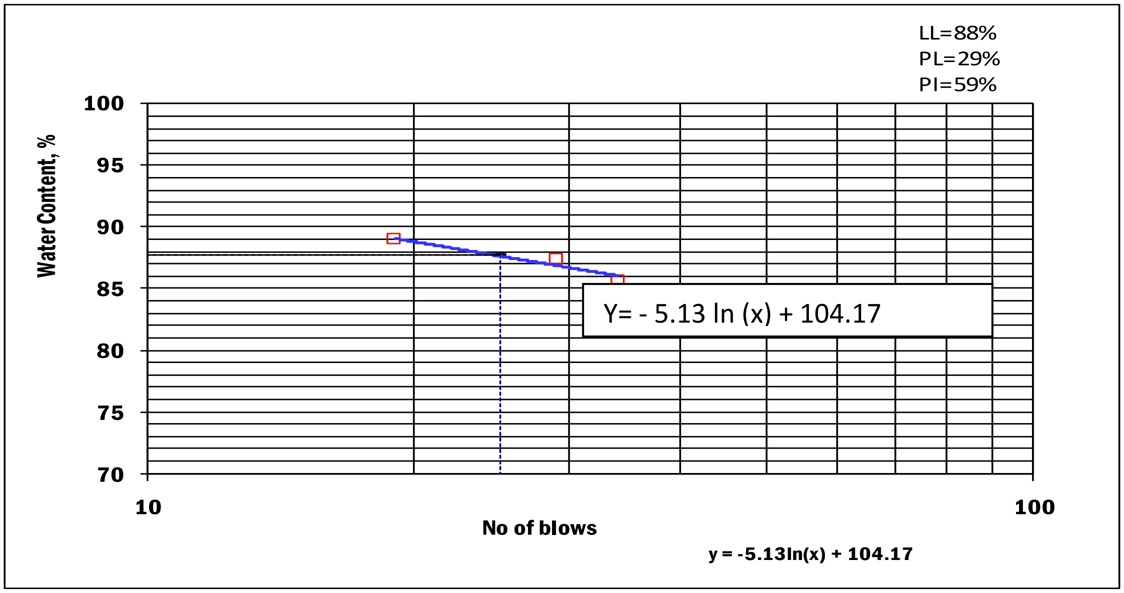


Figure 4.5 Plot of Liquid Limit for the Natural soil with 7 days curing period

4.3.5. Initial Moisture Content of the Soil

The oven-drying method was used to determine the moisture contents of the samples. Small, representative specimens obtained from large bulk samples were weighed as received, then oven-dried at 105^{o}C for 24 hours. The sample was then weighed, and the difference in weight was assumed to be the weight of the water driven off during drying. The difference in weight was divided by the weight of the dry soil, giving the water content on a dry weight basis. For this study the soil has initial moisture content of 38.61 %.All the necessary information is attached in the Annex.

4.3.6. Moisture-Density Relation

Standard proctor tests on the natural soil to determine the relationship between the moisture content and dry density with no curing and 7 days curing were conducted for a specific compaction effort as per AASHTO T99-94 procedure.

4.3.6.1. Maximum Dry Density

The maximum dry density is conducted for both the natural and soil - CKD mixture .The bulk density is then calculated for each compacted specimen using:

$$\gamma_b = \frac{(m1-m2)}{944} \tag{4.1}$$

$$\gamma_d = \frac{(\gamma b)}{(1+\frac{\omega}{100})} \tag{4.2}$$

Where; w is the moisture content of each compacted specimen. The values of the dry densities as obtained from equation above are plotted against their respective moisture contents and the dry densities; MDD is deduced as the maximum point on the resulting curves refer Figure 4.6, 4.7 and 4.8 below for natural soil at zero and 7-day curing period respectively.

From the standard compaction test the natural soil has MDD, 1.36 g/cm^3 with optimum moisture content of 35 .75 % for no-curing time and MDD, 1.18 g/cm^3 with optimum moisture content of 35.82 % for seven day curing period obtained.

Figure 4.6 Standard Proctor Test on progress

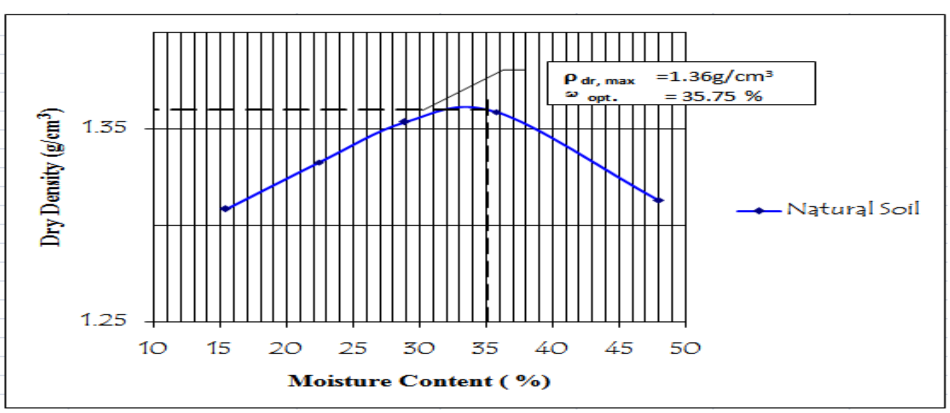


Figure 4.7 Moisture-Density relation for the Natural Soil at Zero-Curing Period

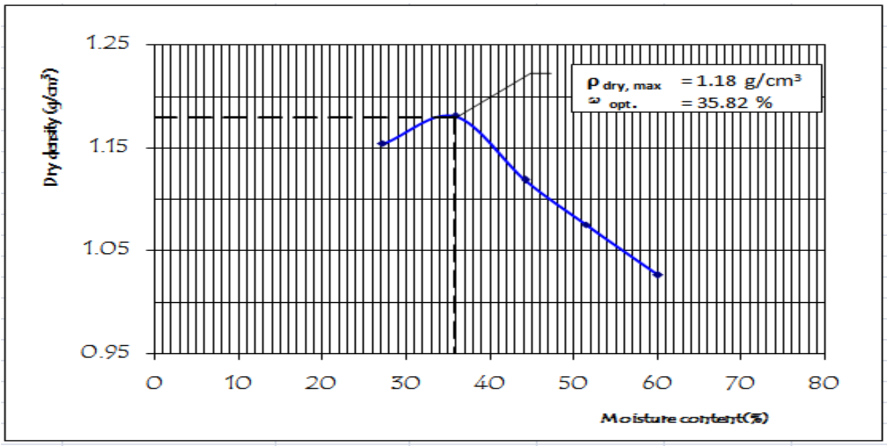


Figure 4.8 Moisture-Density relation for the Natural Soil at 7 daysCuring Period

4.3.6.2.California Bearing Ratio (CBR) Test

The CBR test measures the shearing resistance of a soil under controlled moisture and density conditions. It is a major laboratory test conducted on subgrade and other pavement layers of roads. CBR value is the ratio of load required to affect a certain depth of penetration in to a soil specimen compacted at given moisture content and dry density to the load required to obtain the same depth of penetration on a standard sample of crushed stone.

For this research one point CBR test is conducted. As per AASHTO T 193-93 test procedure, CBR test with curing durations of 7 & 14 days conducted. To make a general evaluation on the effect of applying cement kiln dust on strength development, CBR samples were prepared using soil passing No. 4 sieve and treated samples were compacted using moisture content at maximum dry density obtained from compaction results. And also swelling potential of soil sample was measured.

No surcharge loads have been applied to compacted samples during curing durations assuming no traffic flow is allowed during construction and hence curing process undertaken by using plastic bag and immerse into water bath to obtain uniform temperature inside and outside the plastic bag as shown in Figure 4.9 below for both the natural and blended soil sample. When the allocated curing period is finished, the compacted soil in CBR mold was then soaked in water for four days to simulate the saturated condition of the site. According to ASTM recommendation a surcharge load of approximately 9 kg is used.

Table 4.4. ERA manual-2002 rating of subgrade, sub-base & base-course materials based on CBR value

CBR (%)	General Rating	Uses
0-3	Very poor	Sub-grade
3-7	Poor to fair	Sub-grade
7-20	Fair	Sub-base
20-50	Good	Base Coarse/base
>50	Excellent	Base coarse

Figure 4.9 Sample Curing and CBR Test on progress

From the CBR test it was found that, the natural subgrade soil has very low CBR value 0.9 which is much less than the minimum requirement for a soil to be used as subgrade material.

4.3.6.3. CBR Swell Tests

The volume change/swell-shrink of expansive soils as a result of moisture change is one of the significant identification features. The potential swell of expansive soils is an important parameter to classify subgrade soils based on their expansiveness.

Figure 4.10 CBR Swell Dial Reading Process

For the soaked case of CBR test, the volume change of the compacted specimen is measured before and after soaking using dial gage reading using the following formula. The calculated swell value is summarized in Table 4.5.

$$\text{CBR Swell} = \left(\frac{\text{Change in height in mm during Soaking}}{116.3\text{mm}}\right) \times 100\ \% \qquad (4.3)$$

For which 116.4mm is the height of soil specimen;

Table 4.5 Swell from CBR Mold for the Natural Soil

Dial Reading	Before Socking (mm)	After Socking (mm)
	1.05	10.56
Swell (%)	**8.17**	

According to ERA site investigation manual (2002), a subgrade soil should have a maximum swell of 2 %.Hence, this material has high potential to swell and cannot be used as subgrade material.

4.3.6.4. Free Swell Test

The test was conducted in accordance with the United States Bureau of Reclamation (USBR) method (Holtz and Gibbs, 1956). About 10 g of soil passing BS No 4 sieve (425µm aperture) was oven dried and allowed to cool in a desiccators. The sample was slowly poured into a 100 cm^3 measuring cylinder to which water was added in order to fill the cylinder. The cylinder was then agitated in order to obtain a homogenous mixture of soil and water after which it was allowed to settle for 2 hours or more before the initial volume was recorded. The final volume recorded after 24 hrs. The Free Swell can be obtained using the following equation:

$$FS= \left(\frac{(V_F - V_I)}{V_I}\right) \times 100 \tag{4.4}$$

Where: V_F: Final Volume

V_I: Initial Volume

Table 4.6 Free Swell Determination for the Natural Soil

Initial Volume	Final Volume		Average	Free Swell Index
	Sample No.1	Sample No.2	Final Volume	
(cc)	(cc)	(cc)	(cc)	(%)
10.0	24.9	25.5	25.2	**152**

4.3.6.5. Shrinkage Limits Test

The shrinkage limit, expressed as moisture content in percent, represents the amount of water required just to fill all of the voids of a given cohesive soil at its minimum void ratio obtained by oven drying and used to evaluate the shrinkage potential, crack development potential, and swell potential of cohesive soil. A sample of soil passing through a 425 μm (No. 40) sieve was used for this test. Then the moisture-content loss to dry the soil to a constant volume is determined and subtracted from the initial moisture content to calculate the shrinkage limit. The volume of the dry soil pat is determined from its mass in air and its indicated mass when submerged in water. A coating of wax is used to prevent water absorption by the dry soil pat.

4.3.6.6.Linear Shrinkage Limits Test

Linear shrinkage can be defined as; the decrease in one dimension of the soil mass when the water content is reduced from a given percentage to the shrinkage limit. It is a measure of how a sample will reduce linearly upon drying expressed as a percentage of the original length. A linear shrinkage test was carried out to determine the linear shrinkage characteristics of the stabilized soil sample when completely dry and also the linear shrinkage characteristics of the soil when various percentages of CKD combination.

It is found by determining the change in length of semi-cylindrical bar sample of soil when it dried out, starting from near the liquid limit. The linear shrinkage value is a way of quantifying the amount of shrinkage likely to be experience by clayey material.

The method covers the determination of the total linear shrinkage from linear measurements on a bar of soil of the fraction of a soil sample passing a 425µm sieve. For highly plastic material even 3 days of air drying may be deemed necessary. The sample should not be placed too early in the oven. Linear shrinkage can be obtained by the following equation;

$$LS=\left(1-\frac{Ld}{Lo}\right)\times 100 \quad (4.5)$$

Where: L_d is the length of the oven-dry specimen (mm)

L_o is the original length of the specimen (mm)

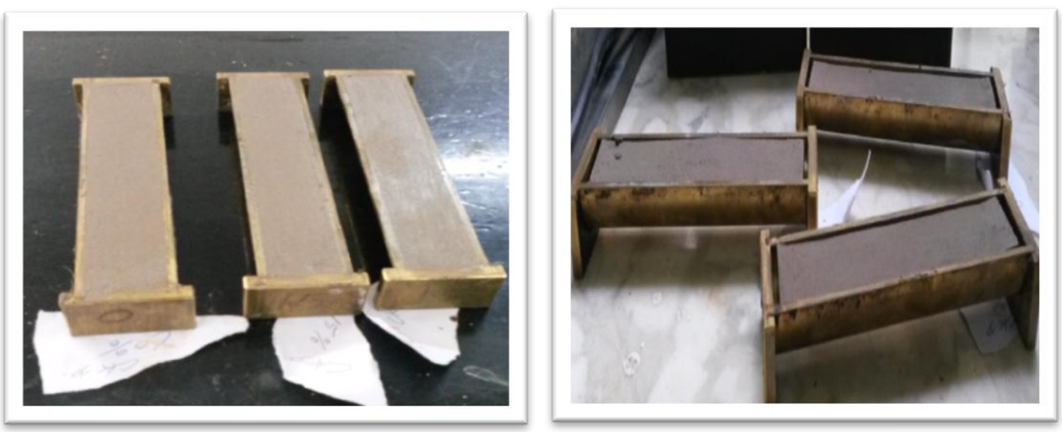


Figure 4.11Specimen on Air drying Process with varying percentage of CKD

4.3.6.7. Volumetric Shrinkage

Volumetric shrinkage can be defined as, change of a soil due to decrease in volume of the soil mass when the water content reduced from a given percentage to the shrinkage limit. This is a measure of the magnitude of shrinkage that the soil can undergo upon severe drying.

4.3.6.8. Specific Gravity

It is the measure of heaviness of the soil particles determined by the pychnometer method. Values for specific gravity of the soil solids were determined by placing a known weight of oven-dried soil passing No.10 sieve in a flask, then filling the flask with water. The weight of displaced water then calculated by comparing the weight of the soil and the displaced water. The specific gravity was then calculated by dividing the weight of the dry soil by the weight of the displaced water. The test includes the determination of the specific gravity for the natural soil conducted in accordance with AASHTO T100-93 testing procedure. For this study, the test result obtain for the natural soil has a specific gravity of 2.77.

4.3.6.9. Swelling Pressure Test

According to ASTM definition, the pressure which prevents the specimen from swelling or that pressure which is required to return the specimen to its original state (void ratio, height) after swell stress controlled (Chen, 1988). The swelling pressure of the soil is dependent on the surcharge pressure, the initial moisture content, degree of saturation, and the thickness of the soil stratum. It depends on initial density soil composition and soil fabrics.

Swelling pressure test result from a one-dimensional odometer instrument for the natural soil is 380 kPa.

4.3.6.10. Unconfined Compressive Strength

The unconfined compressive strength (qu) is defined as the compressive stress at which unconfined cylindrical specimen of soil will fail in a simple compression test. This test was conducted to determine the UCS of the natural soil, soil-CKD specimens prepared by mixing, compacting and curing. For stabilized subgrade, a minimum 30 psi (207kpa) increase from untreated natural soil is required.

The prepared specimens were molded in the standard compaction mold, extracted using Shelby tube samplers and cut to size with a height-to-diameter ratio of 2. And the extracted specimens placed in an airtight plastic bag and allowed to cure in bath to avoid any moisture loss from the sample. At the end of the curing period, the specimen was carefully placed in the compression device as shown in Figure 4.13.Records are taken simultaneously of the axial deformation and the axial force at regular interval until failure of the sample occurs. The UCS of the sample was determined at the point on the stress -strain curve at which failure occurred. The same was done for 7, 14 days curing and the UCS was calculated from the Eq. (4.6). And for this study UCS test result for the natural soil are summarized in Table 4.7 and Figure 4.12.

$$\text{Compressive Strength}=\frac{\text{Failure Load}}{\text{Surface Area of Specimen}} \tag{4.6}$$

Table 4.7 Test Result of UCS value for Native Soil

Native Soil UCS Value (Kpa)	With no-Curing Period	122.64
	7th Day Curing	136.95
	14th Day Curing	416.74

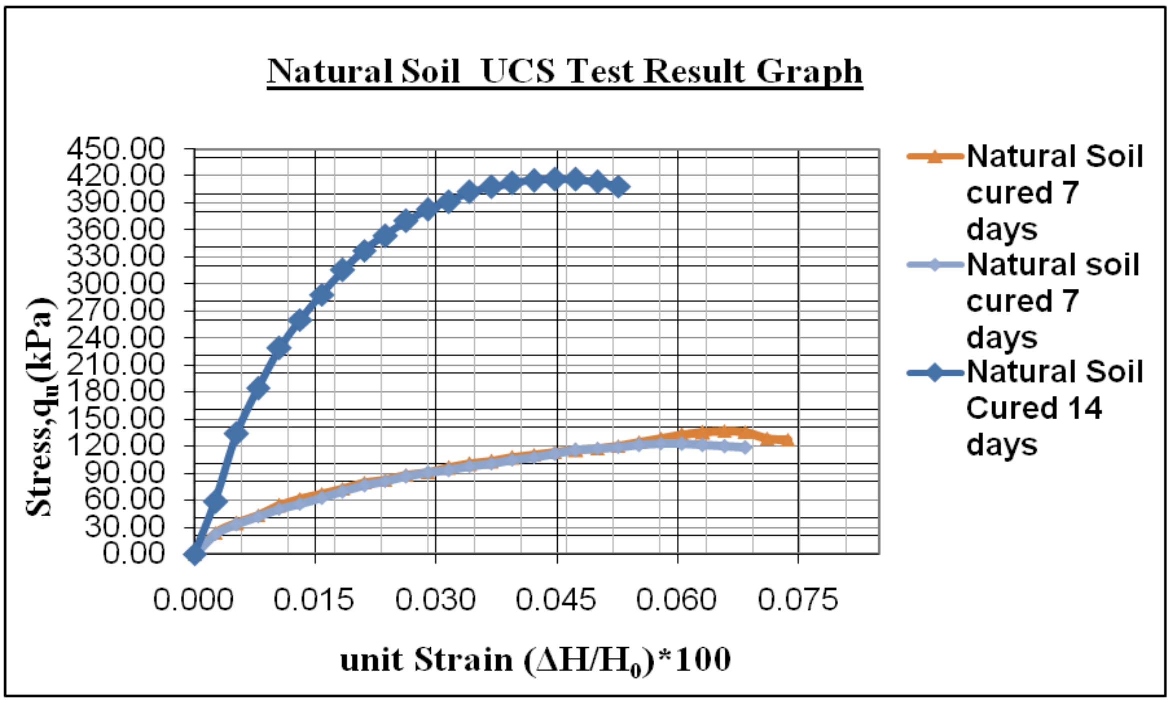


Figure 4.12 Stress-Strain diagram for the natural soil

Figure 4.13 Unconfined Compressive Strength Test Process

Unconfined compressive strength tests were conducted for the sample soils and results are given above and the un-drained shear strengths were calculated as one half of the UCS values as shown in Eq. (4.7) and the test result is shown on Table 4.8. The un-drained shear strength (Su) of a cohesive soil is equal to one-half the unconfined compression strength (qu) when the soil is under the Φ=0 condition (Φ is angle of internal friction).The most critical condition in the soil usually occurs immediately after construction, which represent un-drained conditions, when the un-drained shear strength is basically equal to the cohesion (C). This is expressed as:

$$S_u = C = \frac{qu}{2} \qquad (4.7)$$

Table 4.8UCS; Strain at Failure and Cohesion Values of the Natural Soil

Curing date	UCS (KN/m^2) =q_u	Cohesion (KN/m^2)q_u/2	Moisture Content (%)
No-curing	122.64	61.32	29.62
7days curing	136.95	68.475	32.72
14 days curing	416.74	208.37	27.92

4.3.6.11. Chemical Composition of Soil Samples

The chemical composition of complete silicate analysis and major oxides & minor oxides of the soil sample taken from Bole site were examined at the Geological Survey of Ethiopia. The results are shown in Table 4.9.

Table 4.9 Chemical Composition of Native soil

Constituent	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O	LOI	SO_3	CL
% age of Oxides	55.12	20.94	9.04	0.66	1.26	<0.01	0.32	0.17	0.06	1.36	0.23	11.00	0.55	<0.01

4.4. Overall Characterization of the Native Soil

Based on the field observation and soil laboratory test results the natural subgrade soil is found to be significantly/highly expansive. The primary indications for expansive soils such as dark brown color, widely spaced polygonal pattern cracks were observed in the selected study area, where currently a Hangar apron project is on progress.

The grain size analysis of the sub-grade soil samples shows that it has 2.1% sand, 37% silt and 61% clay. The sub-grade soil is found to be highly expansive with plasticity index of 59 %. Thus, as per AASHTO classification system, the subgrade soil can be classified as A-7-5 /A-7-6 rating Fair-to-Poor to be used as subgrade material.

The sub-grade soil has very low load bearing capacity (CBR value of 0.9%) and very high swelling potential (Free swell value 152%), CBR Swell 8.17 %, Swell –pressure value of 380kPa, linear shrinkage value of 27.14 %. Such soils are potentially problematic and can result in series damages to pavements constructed on them without appropriate treatment measures. Therefore, appropriate treatment methods should be conducted before any pavement structure is constructed over such poor sub-grade soils. Thus, one of the improvement methods mentioned in section 2.1.11 should be adopted before construction of overlaying pavement sections on such poor subgrade soils.

The next Chapters present the performance of Cement Kiln Dust (by produce of Cement Factories) to improve the poor strength of the native expansive soil based on laboratory test results are discussed.

4.4.1. Cement Kiln Dust (CKD) Stabilization

Introduction

Cement Kin Dust is a particulate matter that is collected from cement kiln exhaust gases and consists of entrained particles of clinker, unreacted and partially calcined raw materials, and fuel ash enriched with alkali sulfates, halides and other volatiles (KAĞAN, 2014).

The chemical and physical properties of CKD can be influenced by several factors, because, plan operations differ considerably with respect to raw feed, type of operation, dust collection facility, and type of fuel used.

The disposal of this fine dust is very difficult and poses an environmental threat. To overcome this problem, research is being carried out in different parts of the world to find out economical and efficient ways and means of using Cement kiln dust (CKD) in various applications. According to the research undertaken by Rahman (2011),this dust is applied for soil stabilization, waste treatment, and manufacture of bricks.

Another study KAĞAN, (2014) reported that CKD production worldwide amounts to about 30 million tons per year.

In another literature on the other hand, CKD can also be used for stabilized Nile Silt soil along the banks of the Nile.

Generally, CKD represents a mixture of raw feed, partly calcined cement clinker and condensed volatile salts. The chemical composition of CKD is influenced by the size of particles carried away by the kiln gases. It consists mainly of limestone as a main component, and minor amounts of quartz, together with $CaSo_4$, KCL , NACL, K_2So_4, KNa Cl_2, $2(2Cao. Sio_2).CaSo_4$ and Sulfospurite {$2(2CaO.Sio_2).CaCo_3$}.

In this research the cement kiln dust was obtained from Mugher Cement Enterprise which is located about 90 kms North West of Addis, at an altitude of about 2450mts above sea level. The mother plant of the enterprise has three production lines with a total production capacity of 1.83 million tons per annum as shown Figure 4.14.

The quarry site of this plant is at Derba, which is located north west of Addis at a distance of about 70 kms. The quarry material is rich in silica, sand, sandstone, gypsum and limestone(Source: MCE bulletin).

Figure 4.14. Clinker Production Lines at Mugher Cement Enterprise

Figure 4.15 View of Cement Kiln Dust used in this Study

4.4.2. Properties of Cement Kiln Dust

Cement Kiln Dust used for the present study is grey in color as shown in the Figure 4.15 above and is fine grained and non–plastic material.

4.4.2.1. Chemical Properties of CKD

The chemical composition was determined using X-ray diffraction machine at Mugher Cement Enterprise chemical laboratory testing department as shown in Figure 4.16 and Table 4.10.

Figure 4.16 View of X-Ray Diffraction Test at MCE Chemical Laboratory

Table 4.10.Chemical Composition of Mugher Cement Kiln Dust

Type of Chemicals	Percentage
Aluminum Oxide , Al_2O_3	3.564
Calcium Oxide, CaO	43.800
Calcium Carbonate, $CaCo_3$	78.215
Iron Oxide, Fe_2O_3	2.113
Magnesium Oxide, MgO	0.905
Silicon Dioxide, SiO_2	9.896
Sulfur Trioxide, SO_3	0.526
Sodium Oxide, Na_2O	0.248
Potassium oxide, K_2O	0.355
Silica ratio	1.7 43
Aluminum ratio	1.686
ASR	1.097
Hydraulic modular	2.813
Liquid phase	16.955
Coating index	10.936
Burning index	6.069
pH Value	11.49

4.4.2.2. Physical Properties of Cement Kiln Dust

Pertinent physical tests were conducted at Mugher Cement Enterprise physical analysis testing laboratory and Geological Survey of Ethiopia laboratory as shown in Figure 4.17.The test results are presented in Table 4.11.

Figure 4.17 View of CKD Physical Analysis Test at MEC Laboratory

Table 4.11Physical Properties of CKD

Fineness (retained on No.9 µm sieve (No. 325 sieve)	4.48 %
Specific Gravity	2.65 %
Loss on Ignition (LOI)	36.25 %
Water Content	0.53 %
Dry Shrinkage	5.71 %

4.4.2.3. Atterberg Limit test of MEC CKD

Atterberg limit tests were conducted and CKD found to be of low plasticity as summarized in Table 4.12.

Table 4.12 Consistency Test Result for CKD

Atterberg Limit Test Result	
Liquid Limit %	38
Plastic Limit,%	34
Plasticity index %	4

4.4.2.4.Pozzolanic Test of CKD

According to ASTM C618-98, Pozzolana can be defined as;″a siliceous or siliceous and aluminous material which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties.″

As per the classification of ASTM, CKD can be categorized as Class N; comply for the properties of: pumicites, calcined or uncalcined, clay and shales.The chemical and physical requirement of mineral admixtures to comply pozzolanic effect for class N described as presented in Table 4.13.

Table 4.13 Chemical requirement of Mineral Admixtures

Chemicals		**Physical**	
Requirement	Percentage	Requirement	Percentage
Silicon dioxide (SiO_2) Plus aluminum oxide (Al_2O_3) plus iron oxide (Fe_2O_3)	Minimum 70 %	Percent retained on 45 µm(o.325),	Maximum 5%
Sulfur trioxide (SO_3)	Maximum 4 %	Soundness	0.8
Moisture content	Maximum 3%	Water requirement	Maximum 115 %
Loss on ignition	Maximum 10 %	Strength activity index at 7 days and 28 days	Minimum 75 %

4.4.3 Percentage of Stabilizer (CKD)

For the present research the amount of CKD to be applied on the expansive soil is from 5 % to 30 % by weight of the native soil with a 5 % increment as per literature and related soil stabilization research works. It is noted that depending on the nature of the stabilizing agent and properties of the soil to be stabilized, the amount by percent (weight or volume) of the stabilizing agent varies.

4.5.Blending of CKD and Native Soil

4.5.1.pH Value

Using ASTM –C-977 appendix-XI test procedures, method for determining stabilization ability of additive was conducted as presented in Figure 4.18 below, and the result indicated on Figure 4.19. ASTM D6276 also discusses the estimation of pH test that can be utilize for the determination of optimum amount of chemical additives for the proportioning of lime-soil stabilization. It gives an indication whether the soil in question can be stabilized or not. Likewise the optimum amount of CKD can be determined in the same manner as that of lime to indicate the percentage of CKD at which stabilization may occur. The proportion of CKD combined with expansive clay was taken as 5%, 10%, 15 %, 20%,25 %, and 30 % by dry weight of the soil. From the pH test results, the optimum amount of CKD is estimated at 25 %. This test was conducted at Addis Ababa Water Works Authority chemical laboratory.

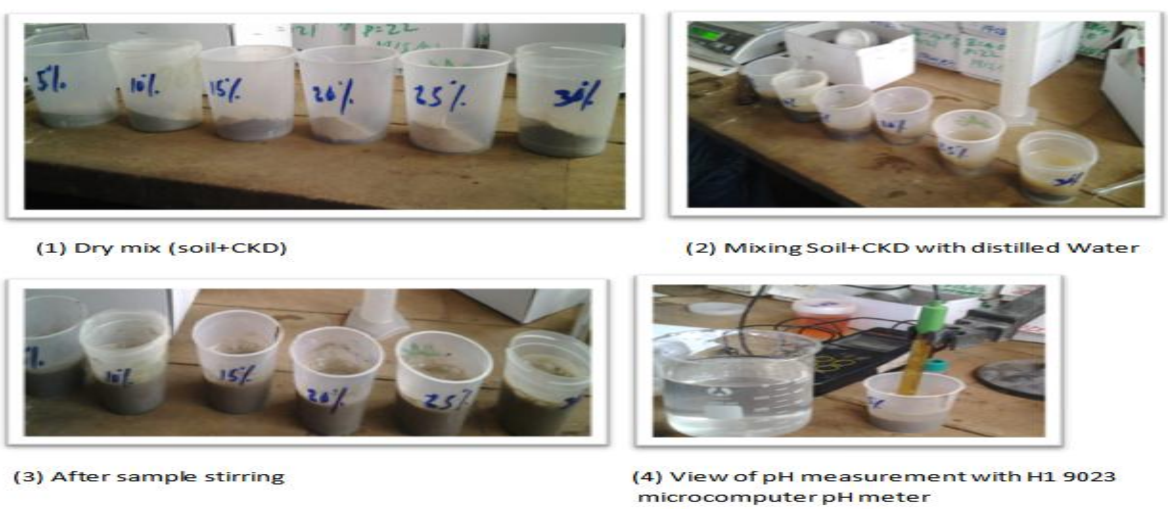


Figure 4.18. View of pH value Determination Lab Procedure

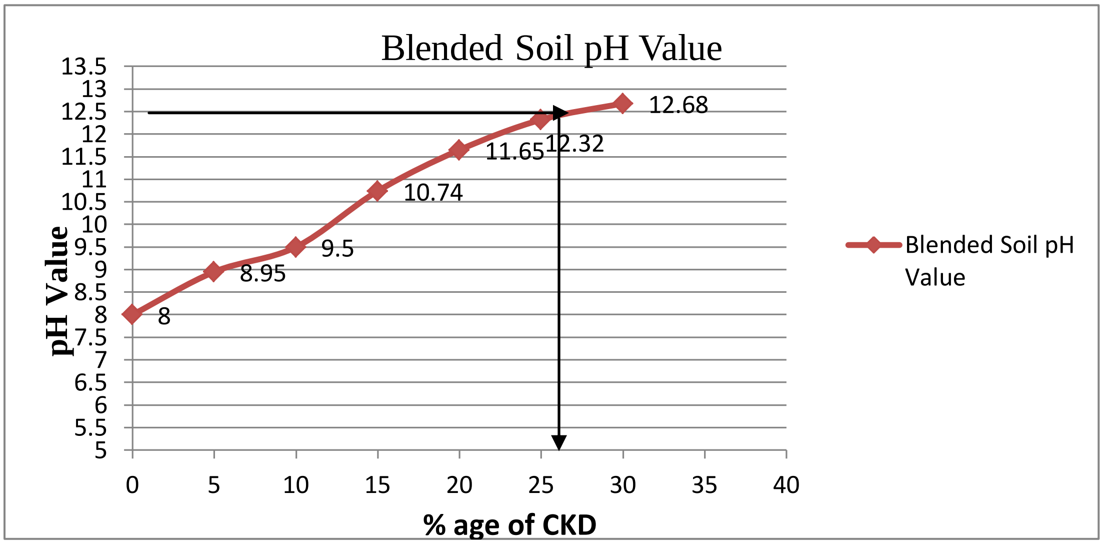


Figure 4.19. pH Value Determination Test Result

4.5.2. Mixing Procedures

The needed CKD was added to the prepared dry sample of soil and dry mixed with the soil with the specified percentage of CKD as shown in Figure 4.20 below.

(1) Native Soil

(2) Native soil + CKD

(3) Soil +CKD dry mix process

(4) determination of amount of Water


(5) Sprinkling water on (soil+CKD) dried mix tures (6) soil+CKD wet mix

(7) Preparation of wet mixture on plastic bag (8) samples (soil+CKD) ready for curing

Figure 4.20 Soil-CKD Mixing Procedure

4.5.3. Atterberg Limit

One of the important and principal aims of the present study is to evaluate the changes of liquid limits, plastic limits and plasticity index with addition of cement kiln dust to the selected soil samples. To achieve this objective, liquid limit and plastic limit tests were conducted on soil – CKD mixtures according to consistency test of AASHTO T89 and T90 respectively.

Soil passing no 40 sieve was mixed with different proportion of chemical additives at optimum moisture content and kept for curing packed in plastic bags to protect loss of moisture. The proportion of CKD used was 5 %, 10 %, 15 %, 20%, 25%, and 30 %. Atterberg limit tests of the mixtures were conducted after 7 days of curing to estimate the influence of curing on the limits. The test results of Atterberg limits of soil-CKD mix summarized in Table 4.14 below and for detailed test result refer Annex III,1.1.

Table4.14 Atterberg Limit Test Results for uncured and 7 days cured respectively

No-curing period				7 days curing period			
% of CKD	**LL(%)**	**PL(%)**	**PI(%)**	**% of CKD**	**LL (%)**	**PL(%)**	**PI(%)**
0	109	37	72	0	88	29	59
5	106	34	72	5	78	32	46
10	97	27	69	10	75	27	48
15	95	26	69	15	72	29	43
20	89	30	59	20	68	30	38
25	84	29	56	25	69	33	36
30	77	27	50	30	65	29	36

4.5.4. Moisture Density Relation

Air dried and pulverized soil passing no 4 sieve was used to determine moisture-density relation of the soil mixed with varying proportions of the CKD additives in accordance to AASHTO T180-97. Moisture content versus dry density graph is plotted and the optimum moisture content (OMC) and Maximum dry density (MDD) are determined from the graph in Figures 4.21 and4.22.

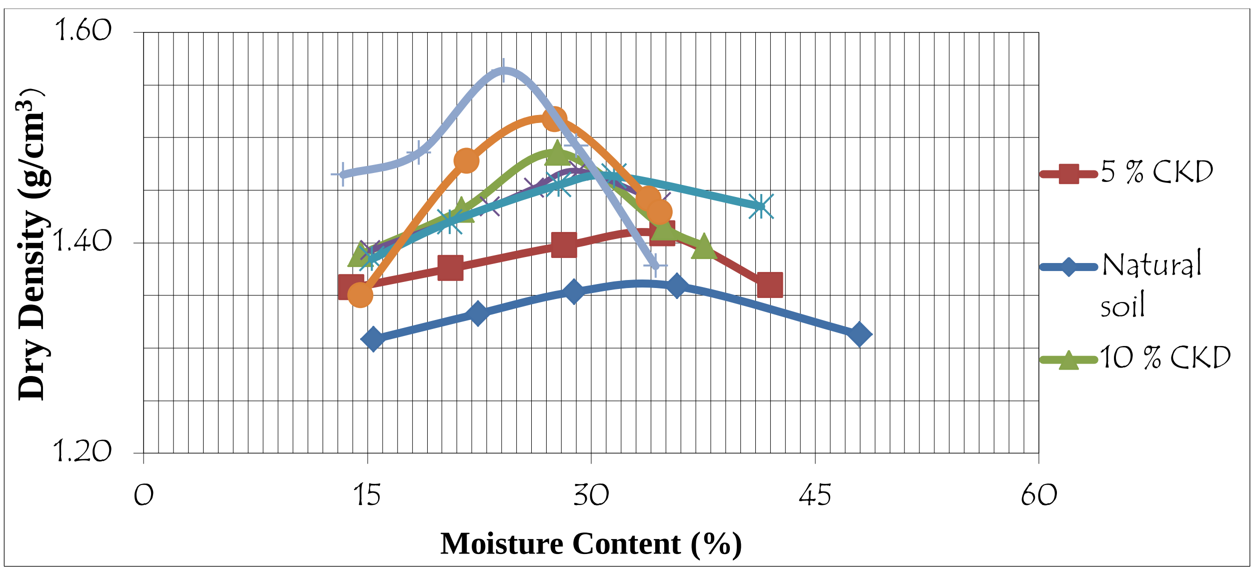


Figure 4.21 Moisture-Density Relations for Different Percentage of CKD with No-Curing

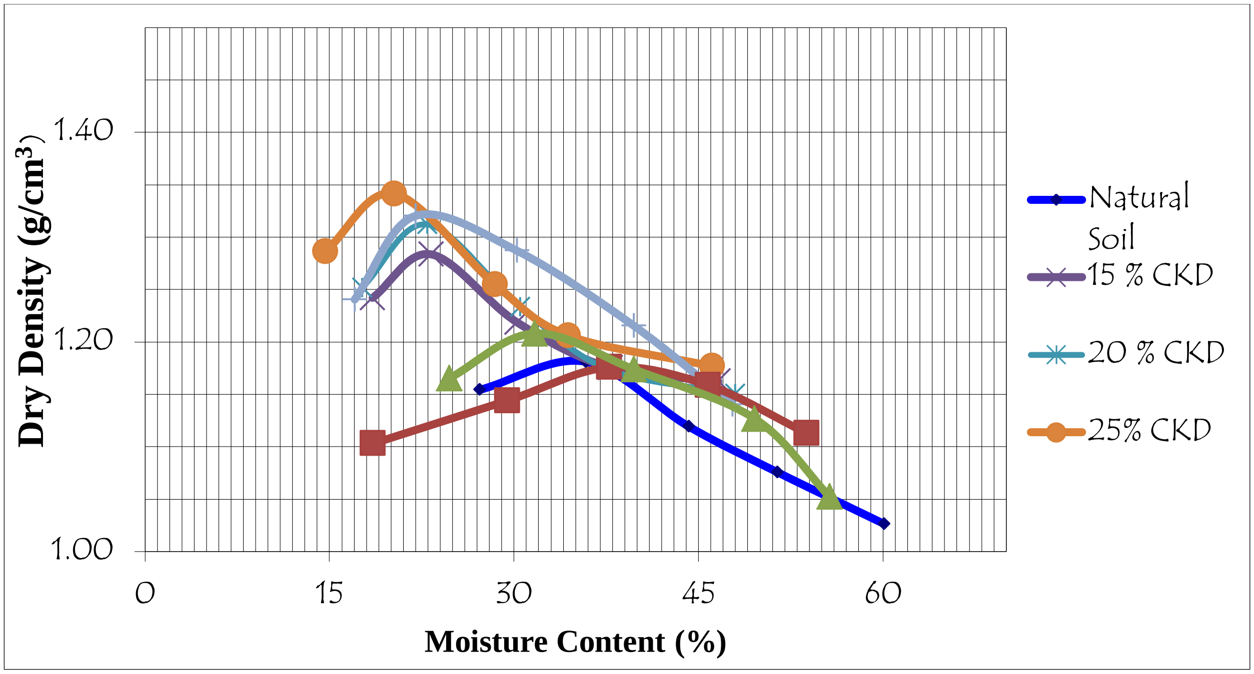


Figure 4.22. Moisture-Density relation for different percentage of CKD cured for 7 days

The Variations in optimum moisture content and maximum dry density with increasing percentage of CKD are presented in Table 4.15.

Table 4.15 OMC & MDD with Increasing Percentage of CKD

With No Curing time

	Native soil	% CKD					
		5	10	15	20	25	30
Water Content, %	35.75	34.80	27.75	29.17	31.53	27.56	24.13
Dry density, g/cm^3	1.36	1.41	1.49	1.47	1.46	1.52	1.56

With 7days Curing Time

	Native soil	% CKD					
		5	10	15	20	25	30
Water Content, %	35.82	37.74	31.67	23.26	22.95	20.27	22.01
Dry density, g/cm^3	1.18	1.18	1.21	1.28	1.31	1.34	1.32

The effect of curing on the engineering properties of the blended samples is observed for 7 days curing. The curing was done by placing the blended sample in plastic bags for the required curing period to reduce loss of moisture as shown in Figures 4.23 and 4.28 below.

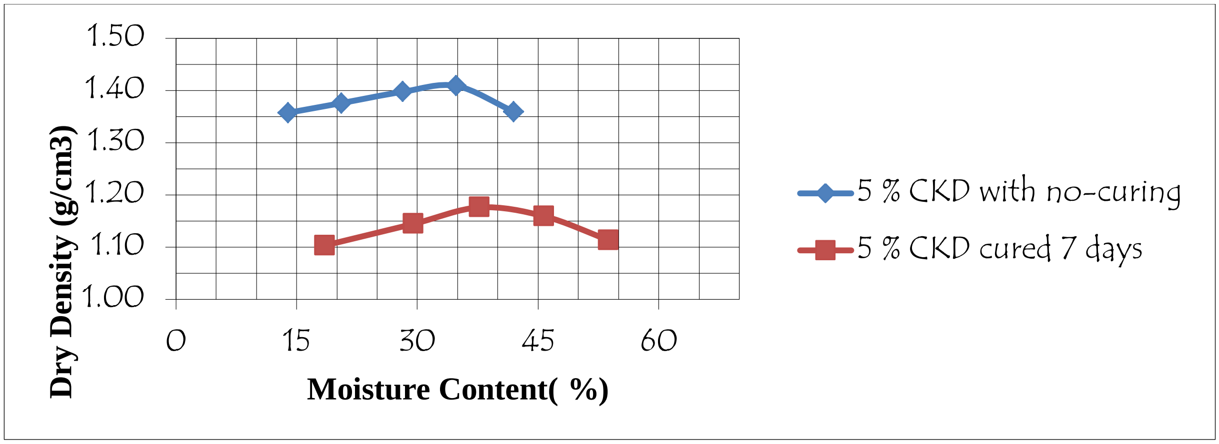


Figure 4.23Effect of Curing on 5% CKD Blended Soil

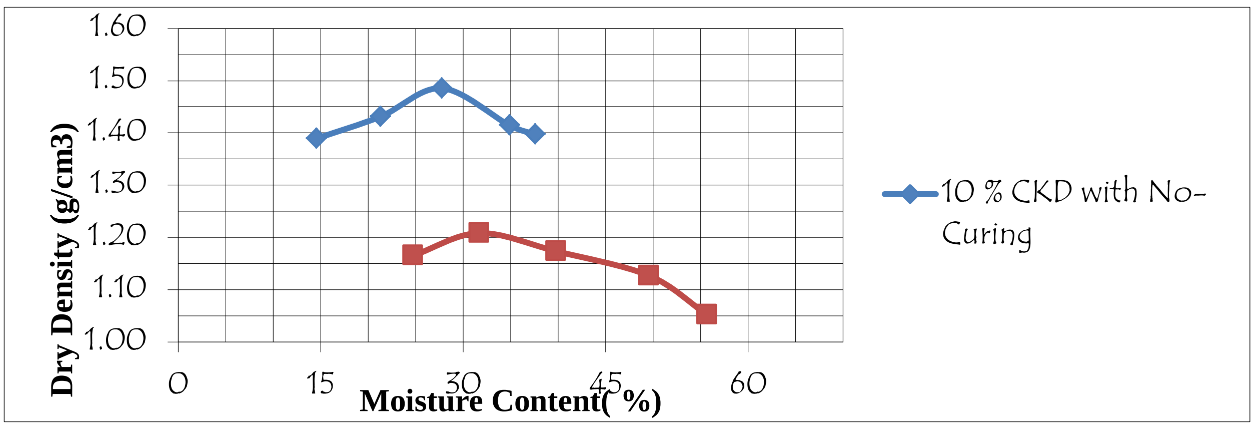


Figure 4.24 Effect of Curing on 10% CKD Blended Soil

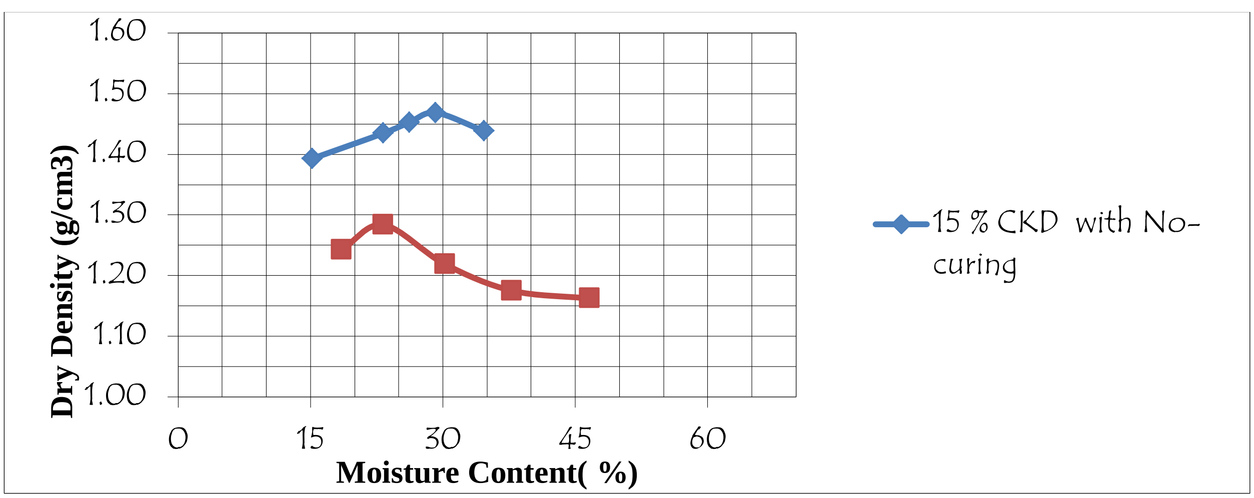


Figure 4.25 Effect of Curing on 15% CKD Blended Sample

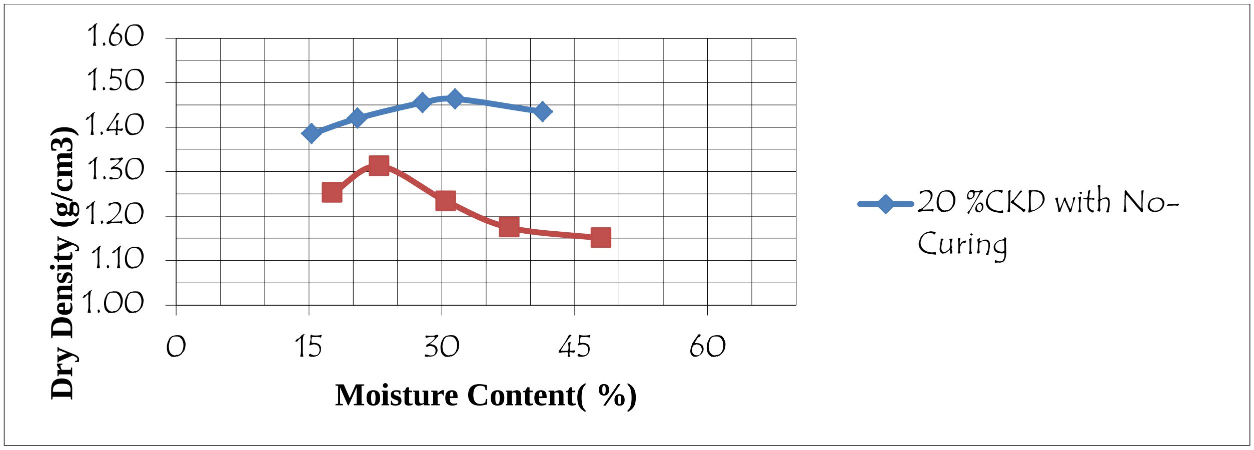


Figure 4.26 Effect of Curing on 20% CKD Blended Sample

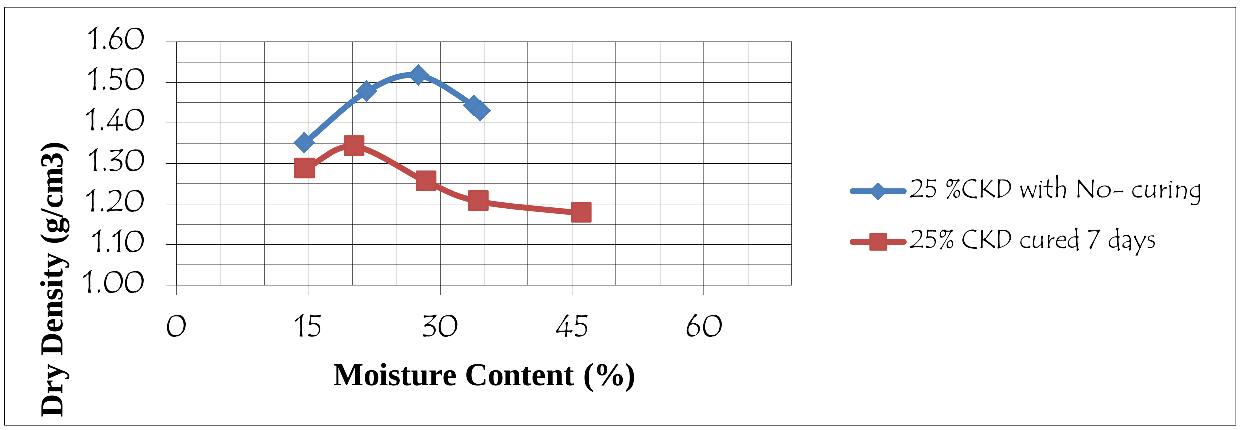


Figure 4.27Effect of Curing on 25% CKD Blended Sample

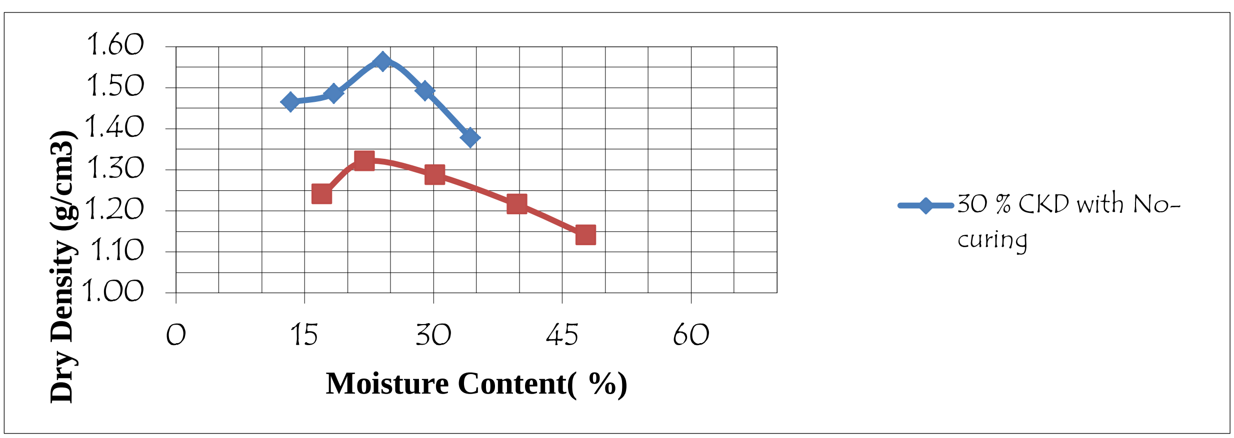


Figure 4.28Effect of Curing on 30% CKD Blended Sample

From the test result shown in the figures it can be observed that:

- With the increment of CKD the maximum dry density (MDD) increased and the optimum moisture content (OMC) decreased.
- Due to curing the maximum dry density (MDD) decreased and the optimum moisture content (OMC) for 5% and 10% CKD increased while for 15%, 20%, 25% and 30% CKD blended sample the value of OMC decreased. The details are presented in the Annex III, 1.2.

4.5.5. California Bearing Ratio (CBR)

The soil plus CKD mixture were kept compacted in CBR molds for 7,14 and 28 days of curing periods for 10 % ,20 %, and 30 % CKD mix to estimate the influence of curing period on CBR value. For this study, one point CBR with 56 blows and 4 days (96 hours) was used after curing periods to simulate worst condition of the site in rainy season. The CBR test results are shown in Table 4.16.For details, refer Annex III, iii.

Table 4.16 Soaked CBR values with increasing percentage of CKD

Soaked CBR on cured sample

Days of curing	7 days (Soaked)			14 days (Soaked)			28 days (Soaked)		
% of Cement kiln dust	10%	20%	30%	10%	20%	30%	10%	20%	30%
CBR at 2.54 mm penetration	2.45	6.98	10.97	7.12	9.24	11.17	6.16	7.12	7.51
CBR at 5.08 mm penetration	2.06	5.68	9.03	5.03	6.45	9.29	3.87	5.8	7.22

From the Table 4.16,the soaked CBR value has good performance with the increased percentage of CKD .The effect of curing on the engineering properties of the blended samples of CKD is observed for 7,14 and 28 days curing. The corresponding un-soaked results are presented in Table 4.17. The curing was done by placing the blended sample in plastic bags for the required curing period to reduce loss of moisture.

Table 4.17 Un-Soaked CBR with Increasing Percentage of CKD

Un-soaked CBR on cured sample

Days of curing	7 days (Unsoaked)			14 days (Unsoaked)			28 days (Unsoaked)		
% of Cement kiln dust	10%	20%	30%	10%	20%	30%	10%	20%	30%
CBR at 2.54 mm penetration	6.74	10.59	13.09	10.40	15.98	18.68	21.56	22.72	25.99
CBR at 5.08 mm penetration	5.68	8.00	11.95	9.03	13.03	16.51	15.74	16.12	16.25

From the Table 4.17, it can be observed that:

- CBR values increase with increasing percentage of cement kiln dust
- With increased period of curing, the CBR values increase
- The un-soaked CBR is greater than the soaked CBR value for the same blended specimen

4.5.6. Percent Swell of CBR

The potential swell of expansive soils is an important parameter to classify subgrade soils. The Soil –CKD mixtures compacted in CBR molds at optimum moisture content with maximum dry density were gauged for swelling characteristics before and after soaking to evaluate the percent swell. The test helps to evaluate the change in volume of the sub-grade soil after treatment with the CKD chemical additives determined from the CBR mold before and after soaking .The results for natural soil, 10 % 20 %, and 30 % CKD blended samples are summarized in Tables 4.18 and 4.19.

Table 4.18. Effect of Curing on the potential swell of blended sample for soaked CBR

% of CKD		10%		% Swell	20%		% Swell	30%		% Swell
Curing date		Before Socking	After Socking		Before Socking	After Socking		Before Socking	After Socking	
7days	Dial Reading (mm)	1.23	5.45	**3.625**	0	3.21	**2.758**	0.04	2.88	**2.440**
14 days		0.03	2.47	**2.096**	4.33	6.61	**1.959**	0.08	1.89	**1.555**
28 days		2.03	3.45	**1.220**	3.67	4.56	**0.765**	2.78	4.56	**1.529**

Table 4.19. Effect of curing on the potential swell of blended samples for un-soaked CBR

% of CKD		10%		% Swell	20%		% Swell	30%		% Swell
Curing date		Before Socking	After Socking		Before Socking	After Socking		Before Socking	After Socking	
7days	Dial Reading (mm)	0	0.32	**0.275**	0	0.3	**0.258**	0.12	0.25	**0.112**
14 days		0.03	0.12	**0.077**	0.08	0.14	**0.052**	0.2	0.25	**0.043**
28 days		0.16	0.21	**0.043**	0.23	0.25	**0.017**	0.24	0.25	**0.009**

From the results obtained it can be observed that:

- The potential swell decreases with increasing amount of cement kiln dust
- With increased period of curing ,the potential swell decreases

4.5.7. Free Swell

As discussed in section4.5.6, the test was conducted in accordance with USBR test procedure; the effect of CKD on the free swell index of the expansive soil is shown in Figure 4.29.From the analysis of test results the free swell index of expansive soil is 152 % and when it combined with CKD the free swell value decreases to 62 % with increased CKD content from 0 % to 30 %.

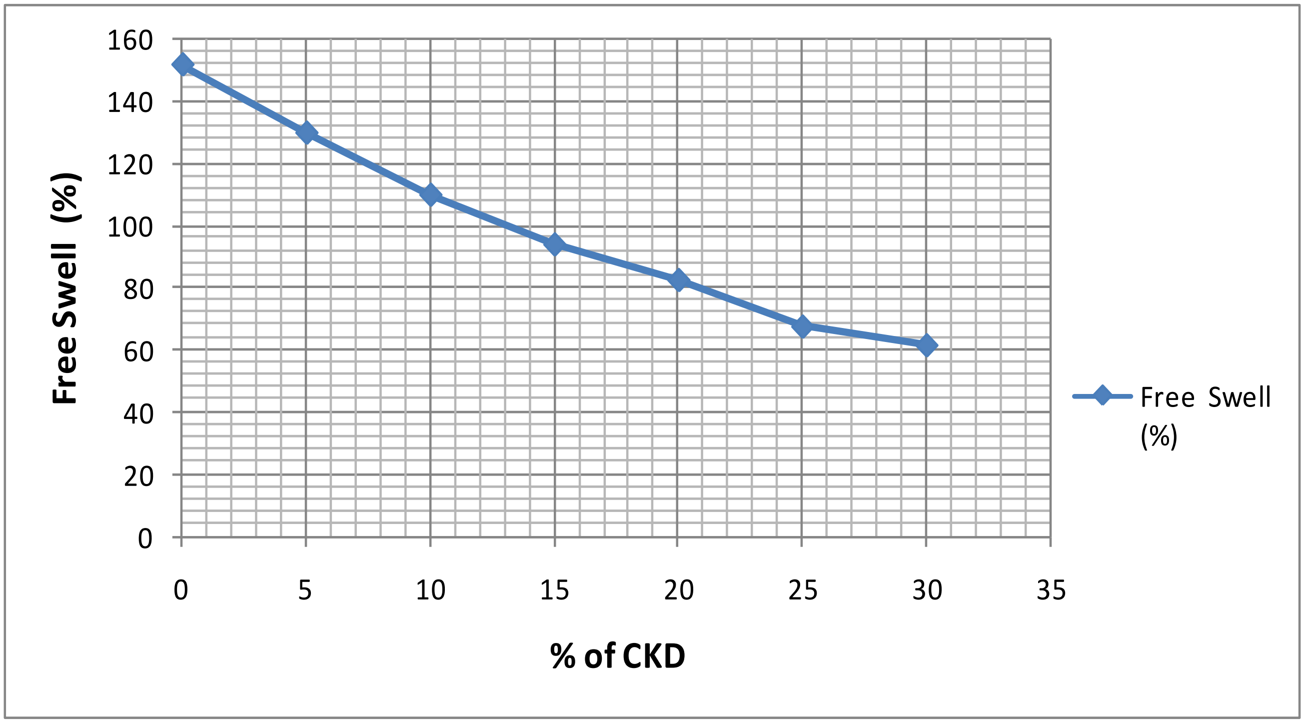


Figure 4.29.Changes in the Free Swell value with varying percentage of CKD

4.5.8. Unconfined Compressive Strength (UCS) Test

The UCS values determined for the natural soil used in the experimental work are discussed in 4.4.14. The UCS values for remolded blended specimens are summarized in Table 4.20. For details refer Annex III, Vi.

Table 4.20. Summary of UCS values with %age of CKD increment

Without curing day UCS test result			
Additive Content (% by Weight)	**UCS(kN/m^2) =q_u**	**Cohesion (kN/m^2)=q_u/2**	**Moisture Content (%)**
For No Curing UCS Test Result			
Natural soil (0 % CKD)	122.64	61.32	29.62
5 % CKD	128.83	64.415	31.27
10%	273.65	136.825	25.2
15%	479.09	239.545	16.79
20%	498.51	249.255	16.48
25%	507.2	253.6	13.8
30%	530.66	265.33	15.54
For 7 days Cured UCS Test Result			
Natural soil (0 % CKD)	136.95	68.475	32.72
5 % CKD	142.02	71.01	36.03
10%	281.35	140.675	34.47
15%	364.25	182.125	10.51
20%	490.9	245.45	25.45
25%	526.66	263.33	28.81
30%	613.9	306.95	30.65
For 14 days Cured UCS Test Result			
Natural soil (0 % CKD)	416.74	208.370	27.92
5 % CKD	445.26	222.630	29.84
10%	469.04	234.520	23.77
15%	471.09	235.545	15.36
20%	534.53	267.265	15.05
25%	535.53	267.765	12.37
30%	574.92	287.460	14.11

From the above observation, one can conclude that the increment of CKD enhances the strength of the natural soils. The effect of curing has also significant improvement on strength as shown in Table 4.21, and Figure 4.30 below.

Table 4.21. Summary of Effect of Curing Period on UCS Values

Curing day	% of CKD						
	0	5	10	15	20	25	30
	UCS (kN/m^2) Value						
W/out curing Day	122.64	128.83	247.14	325.94	380.46	453.05	502
7th day curing	136.95	142.02	281.35	453.04	472.59	526.66	574.01
14th day curing	416.74	445.26	469.56	495.39	526.66	564.17	613.9

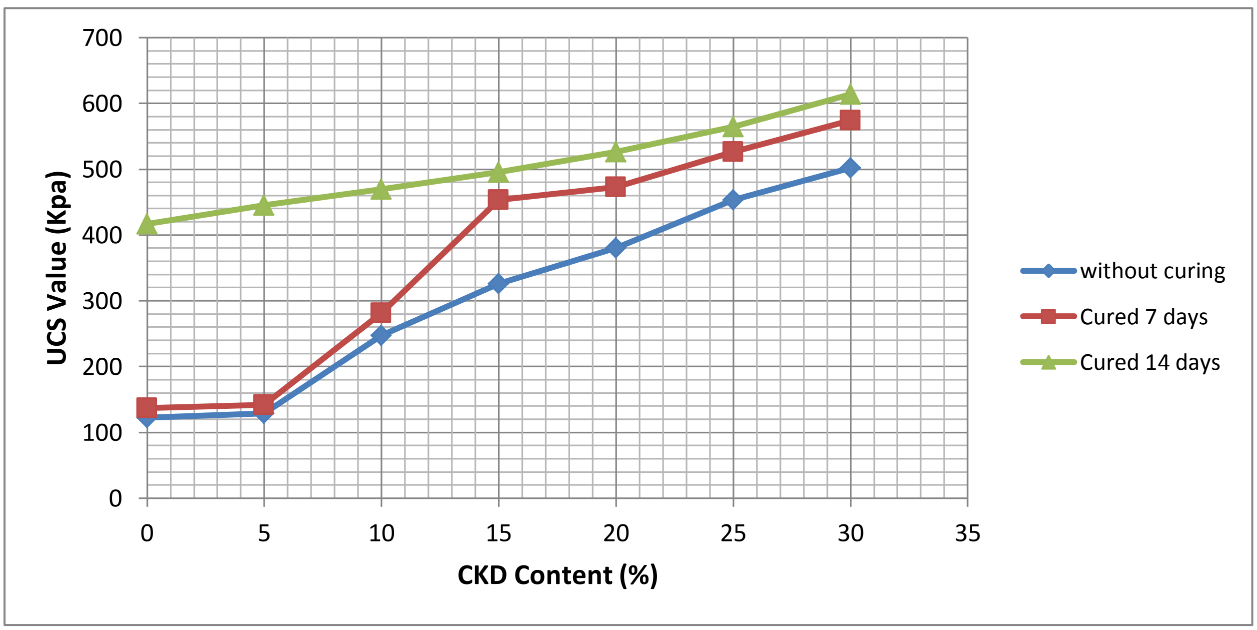


Figure 4.30. UCS as a function of curing time

UCS value of the treated soil increase with increasing percentage of the CKD chemical additives for no-curing, 7day and 14 day curing as referred below in Figures 4.31 up to 4.36.

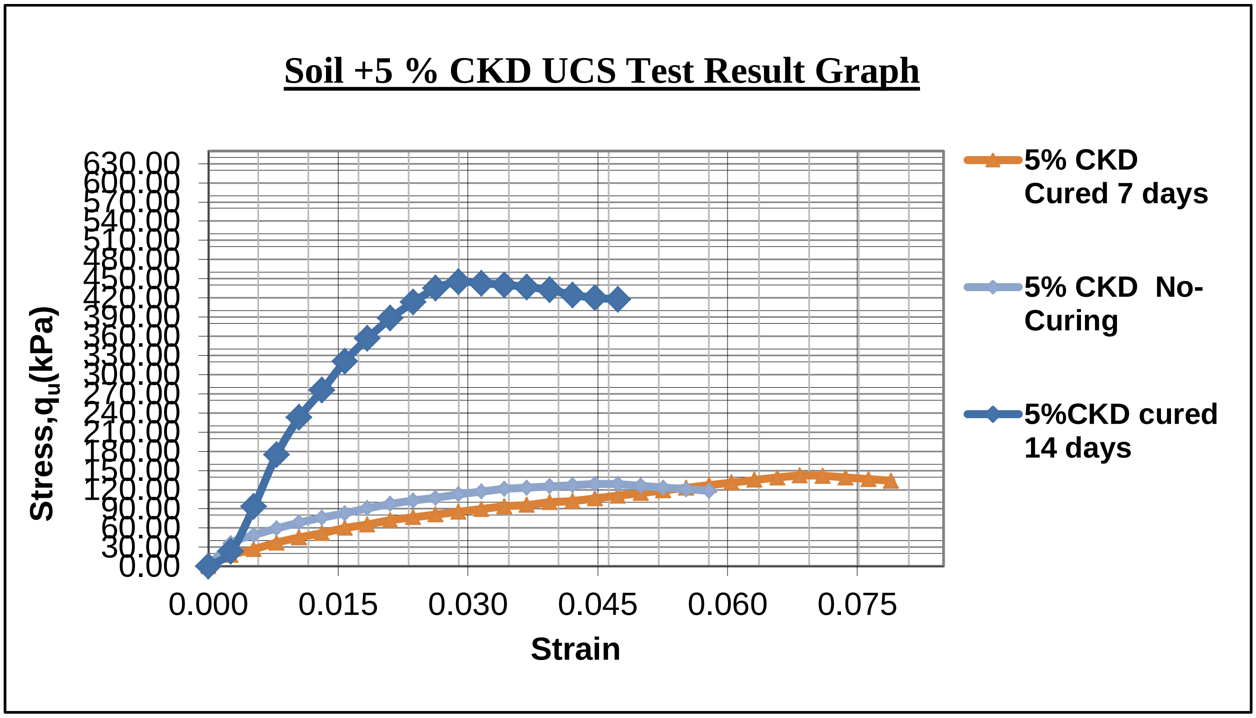


Figure 4.31.Stress-Strain curve for soil treated with 5%CKD

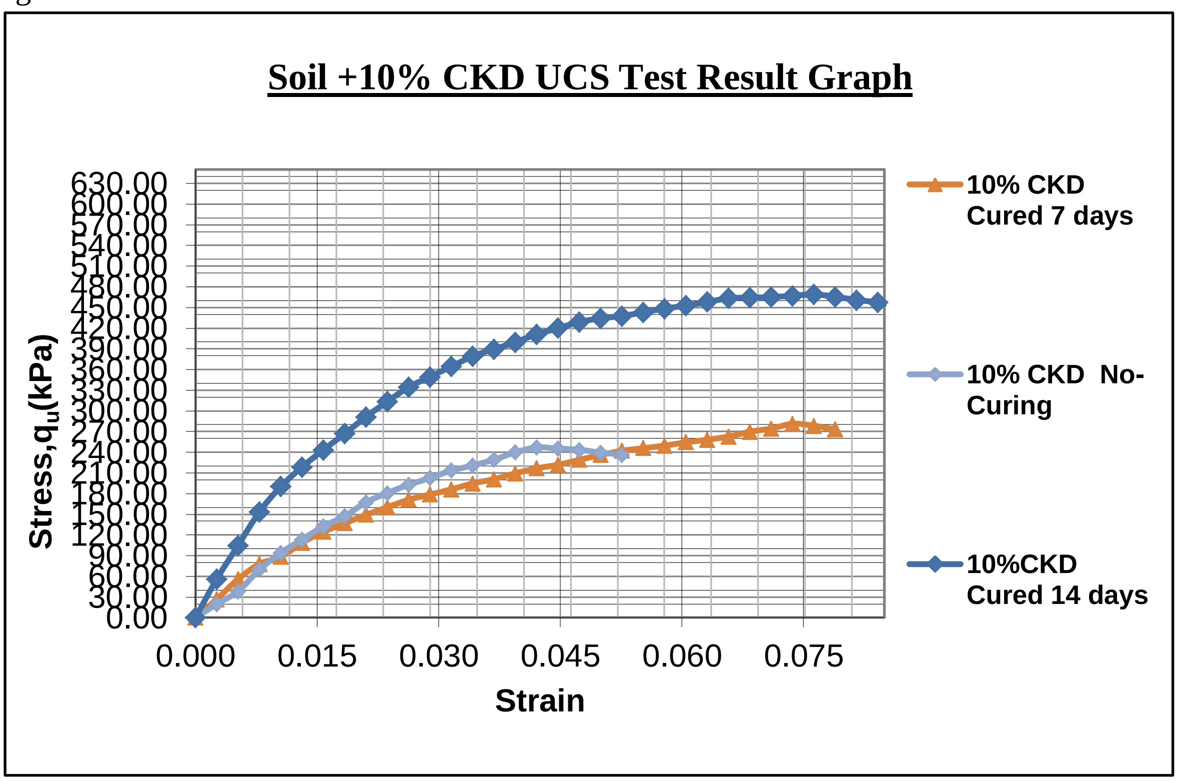


Figure 4.32.Stress-strain curve for soil treated with 10%CKD

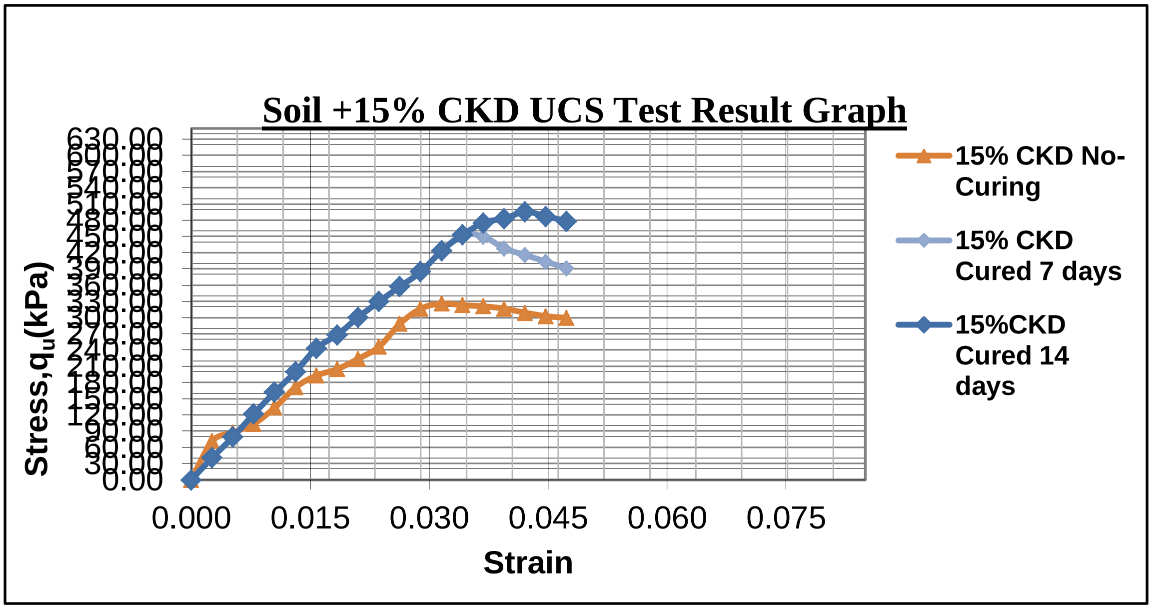


Figure 4.33. Stress -strain curve for soil treated with 15 % CKD

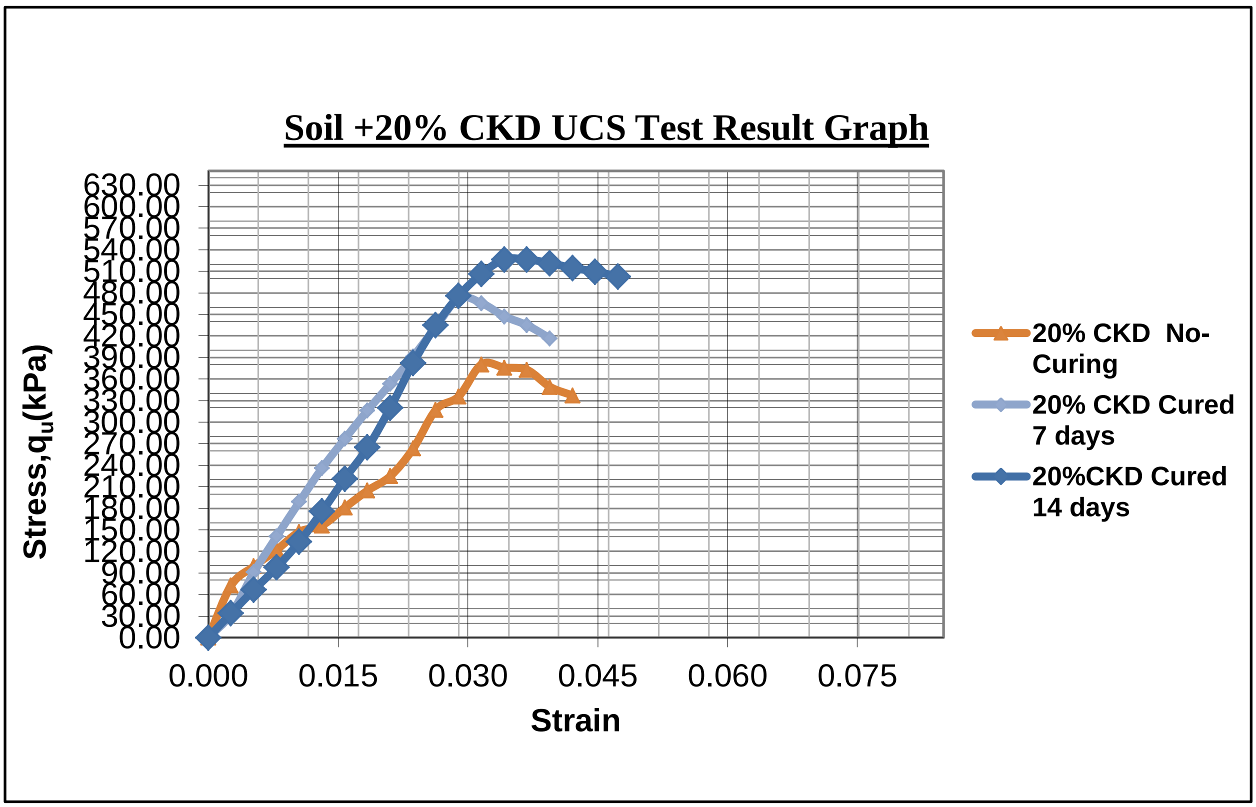


Figure 4.34. Stress-strain curve for soil treated with 20%CKD

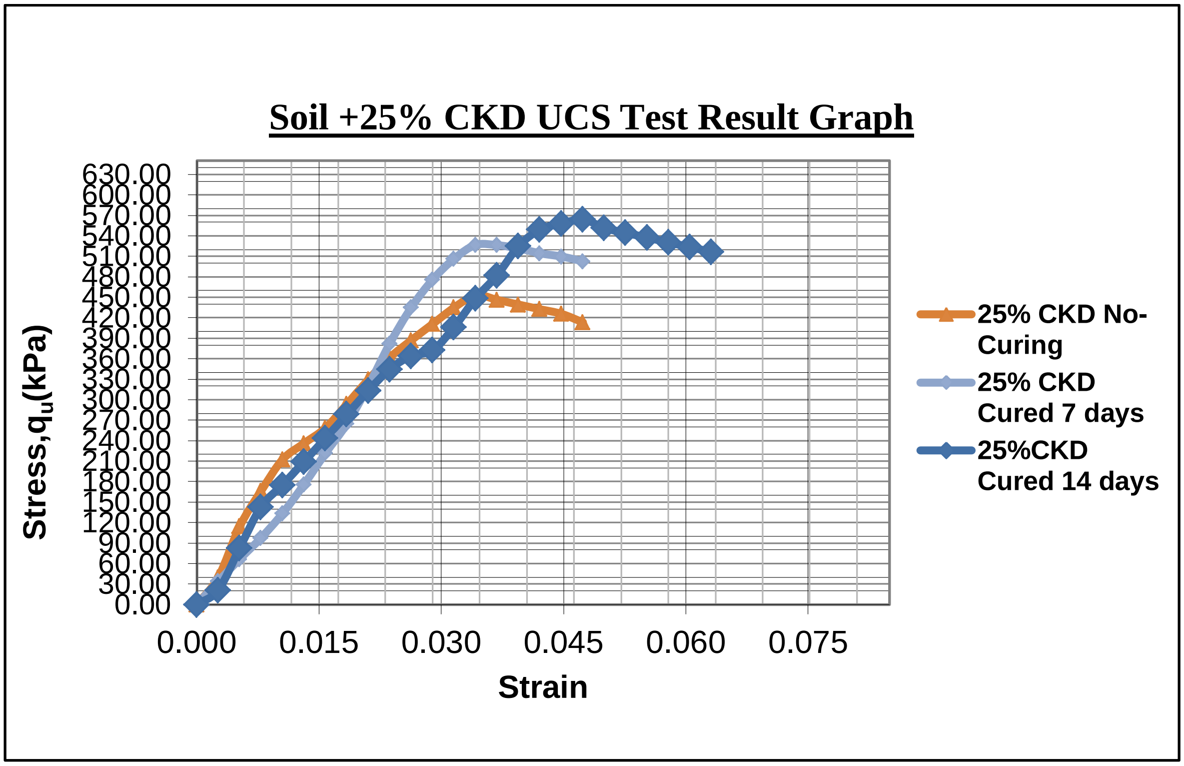


Figure 4.35.Stress-strain curve for soil treated with 25%CKD

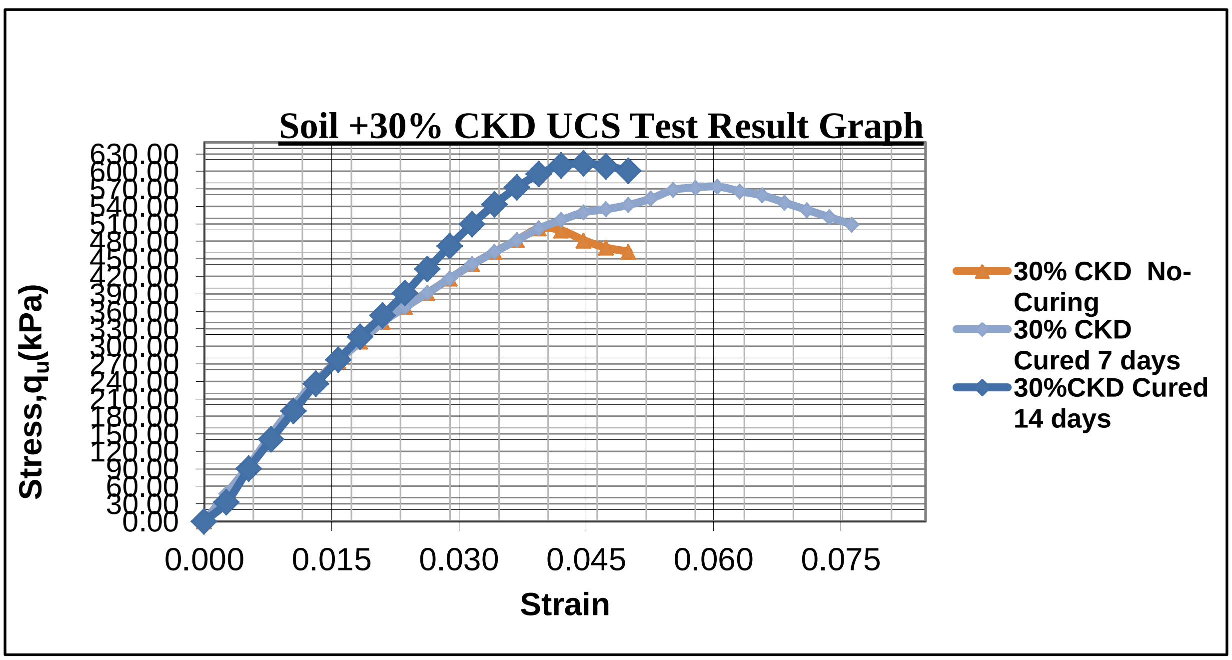


Figure 4.36. Stress-strain Curve for soil treated with 30% CKD

4.5.9. Swell Pressure Test

Swelling pressure is the pressure, which prevents the specimen from swelling or that pressure which is necessary to retain the specimen back to its original state (void ratio, height) after swelling (Chen, 1998).

The samples are placed in the consolidation ring of height of 20mm then they are subjected to a vertical pressure of 7 kPa. Upon completion of the consolidation, water is added to the sample. When swelling of the sample has ceased the vertical stress is increased in increments until it has compressed to its original height. The stress required to compress the sample to its original height is commonly termed the zero-volume-change swelling pressure.

The above testing procedure was followed for the evaluation of swelling pressure. The swelling pressure tests were conducted on both natural soil and blended soil and the test results are shown in Table 4.22. With increasing percentage of CKD the swelling pressure decreases.

Table 4.22. Result of Swelling Pressure for Blended Soil & Natural Soil

Blended Type	Swelling Pressure (kPa)
Natural Soil	380
Soil+ 20 % CKD	165
Soil + 25 % CKD	100
Soil + 30 % CKD	68

CHAPTER FIVE

DISCUSSION ON TEST RESULTS

The results of the testing program are explained/discussed in this chapter. And also the relevant engineering properties of the soil are evaluated both for the natural and stabilized soil samples.

5.1. Improvement on Atterberg Limits

5.1.1. Atterberg Limits with No-curing

The natural soil was found to have a liquid limit of 109%, plastic limit 37%, and plasticity index of 72%. Blending the soil with increasing percentage by weight of the cement kiln dust has given the following result in Table 5.1.

Table 5.1 Effect of CKD on Atterberg Limit

% of CKD	LL (%)	PL (%)	PI (%)
Natural Soil	109	37	72
5	106	34	72
10	97	27	69
15	95	26	69
20	89	30	59
25	84	29	56
30	77	27	50

From the result, it can be noted that the Atterberg limits (LL, PL, and PI) decrease with increasing percentage of CKD.

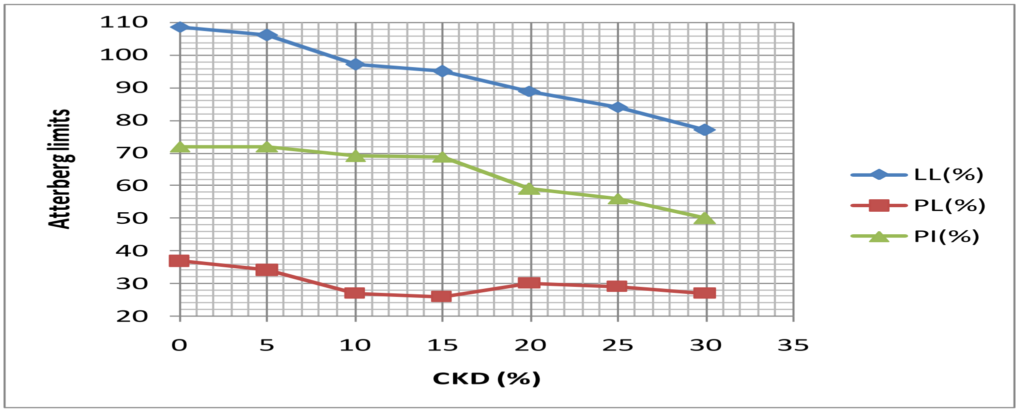


Figure 5.1. Variation of Atterberg Limit with increasing %age of CKD

The percentage decrease of PI value of the natural soil with every 5 % increment of CKD is shown in Table 5.2.

Table 5.2 Average decrease of PI value with increasing percentage of CKD

% of CKD	PI (%)	% Decrease in PI Value
0	72	0
5	72	0
10	69	4
15	69	4
20	59	18
25	56	22
30	50	30
Average		13

5.1.2. Atterberg Limits with Curing

The blended samples have been cured for 7days and tested for atterberg limits. Significant change in liquid limit and plasticity index is observed as shown in Table 5.3 and Figure 5.2.

Table 5.3 Effect of curing for blended soil on Atterberg Limit and Difference b/n cured and un-cured sample

% of CKD	LL (%)	PL(%)	PI(%)
0	88	29	59
5	78	32	46
10	75	27	48
15	72	29	43
20	68	30	38
25	69	33	36
30	65	29	36

% of CKD	With No-curing PI (%)	With 7days -curing PI (%)	Difference PI (%)
0	72	59	13
5	72	46	26
10	69	48	21
15	69	43	26
20	59	38	21
25	56	36	20
30	50	36	14

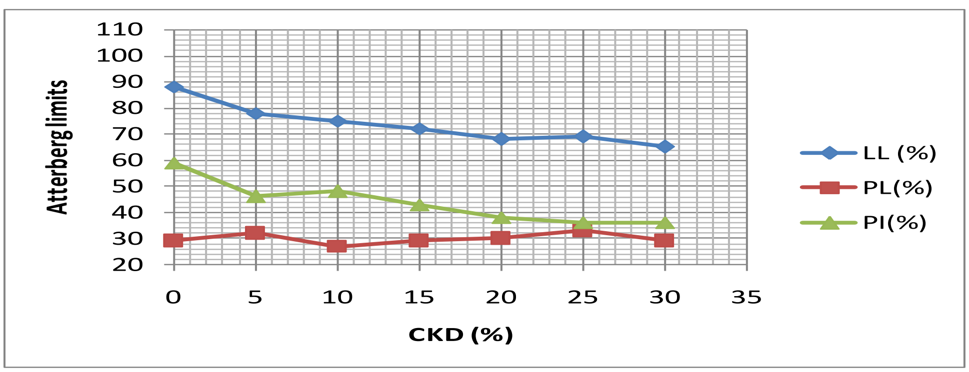


Figure 5.2. Improvement on Atterberg Limit for 7days-curing blended samples

5.2. Improvement on CBR Swelling

The swell of CKD blended expansive soil samples is measured and determined from the CBR mould before and after socking .The results for natural soil, 10 %, 20 %, and 30% CKD blended samples are summarized in Table 5.4 below.

Table 5.4 Potential swell of natural soil & blended samples

% of CKD	0% (Natural Soil)	10%	20%	30%
% Swell	8.17	3.63	2.76	2.44

Effect of Curing on Potential swell

Blended soil with 10%, 20%, and 30% CKD were cured for 7, 14 and 28 days .The potential swell from the CBR mold are measured and the results are summarized in Table 5.5 and Figure 5.3 below.

Table 5.5 Effect of curing on the potential swell of blended samples

Days of Curing	**7**			**14**			**28**		
% of CKD	**10%**	**20%**	**30%**	**10%**	**20%**	**30%**	**10%**	**20%**	**30%**
% Swell	**3.63**	**2.76**	**2.44**	**2.10**	**1.96**	**1.55**	**1.22**	**0.76**	**1.53**

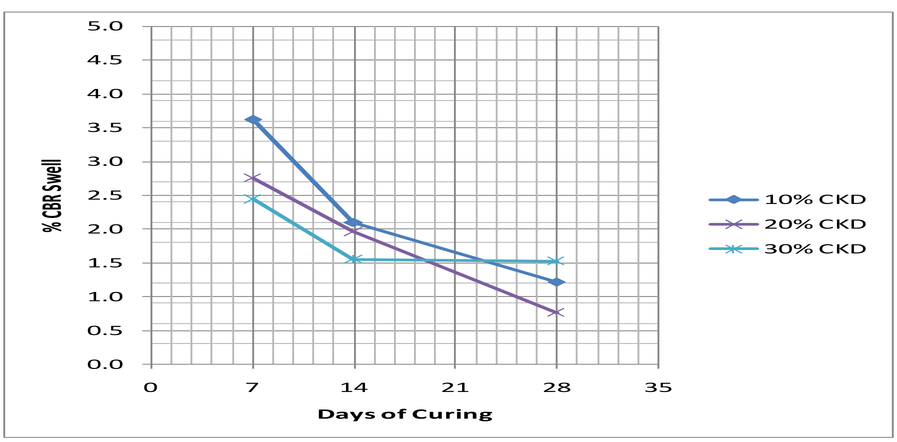


Figure 5.3. Effect of curing on the potential swell

From the results obtained it can be observed that:

- The potential swell decreases with increasing amount of CKD
- With increased period of curing, the potential swell decreases .However for 30% CKD after 14 days curing period does not show any improvement of the swelling.

5.3. Improvement on CBR

The CBR value of the natural soil has shown improvement when blended with increasing percentage of CKD as shown in Figure 5.4below.

From the result it can be observed that the CBR value for the blended samples increases with increasing percentage of CKD.

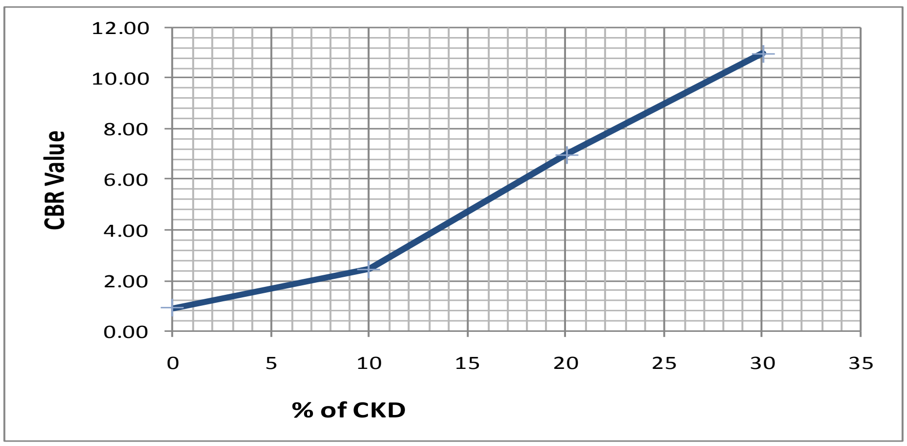


Figure 5.4. Improvement on CBR value

Effect of Curing on CBR value of CKD Blended Sample

To allow the blended sample to mellow and to observe if some kind of chemical reactions has taken place between the expansive soil and CKD, the 10 %, 20%, and 30% CKD blended samples were kept in plastic bags for the periods of 7, 14 and 28 days and the CBR values are investigated after the end of the respective curing period. The results are shown in Tables 5.6 and 5.7 below.

Table 5.6 Effect of Curing on CBR values for soaked case

Days of Curing	% of CKD	CBR Value
7	10	2.45
	20	6.98
	30	10.97
14	10	7.12
	20	9.24
	30	11.17
28	10	6.16
	20	7.12
	30	7.51

Table 5.7 Effect of curing on CBR values for un-soaked case

Days of Curing	% of CKD	CBR Value
7	10	6.74
	20	10.59
	30	13.09
14	10	10.40
	20	15.98
	30	18.68
28	10	21.56
	20	22.72
	30	25.99

5.4. Improvement on Free Swell

The effect of CKD on the free swell of the expansive soil is shown in Figure 5.5 below. As it is shown in the figure, the reduction in free swell is directly proportional to the quantity of CKD. Due to economic point of view the highest reduction in free swell is attained when the expansive soil is treated with 30% CKD which is 90.5 % reduction compared to untreated soil.

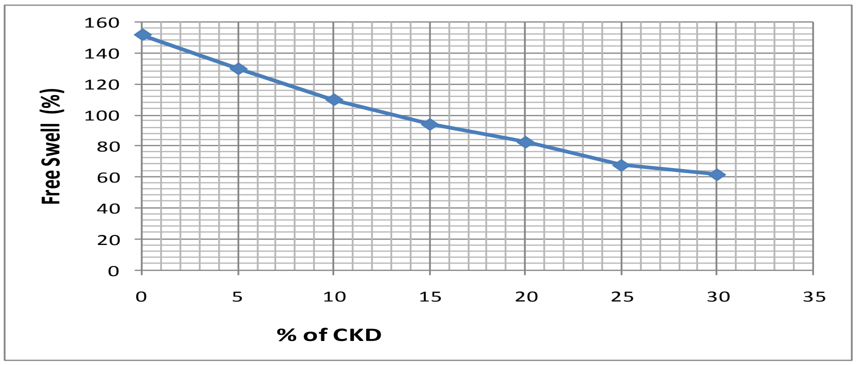


Figure 5.5. Changes in the Free Swell with varying percentage of CKD

5.5. Improvement on Swell –Pressure

The effect of CKD on the swell-pressure of the expansive soil is shown in Figure 5.6 below. As it is shown in the figure, the reduction in swell-pressure is directly proportional to the quantity of CKD .The highest reduction in swell-pressure is attained when the expansive soil is treated with 30% CKD.

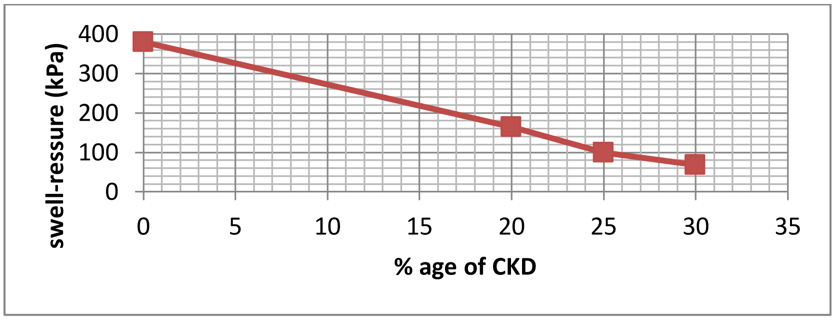


Figure 5.6. Changes in the swell-pressure with varying percentage of CKD

5.6. Improvement on Specific Gravity

The Effect of CKD on the specific gravity of the expansive soil is shown in Figure 5.7 below. For 5% CKD blended sample specific gravity decreased from 2.77 to 2.42 where as for the rest the specific gravity increased. For 30 % CKD mixture the specific gravity became 2.78 which have greater value from the natural soil.

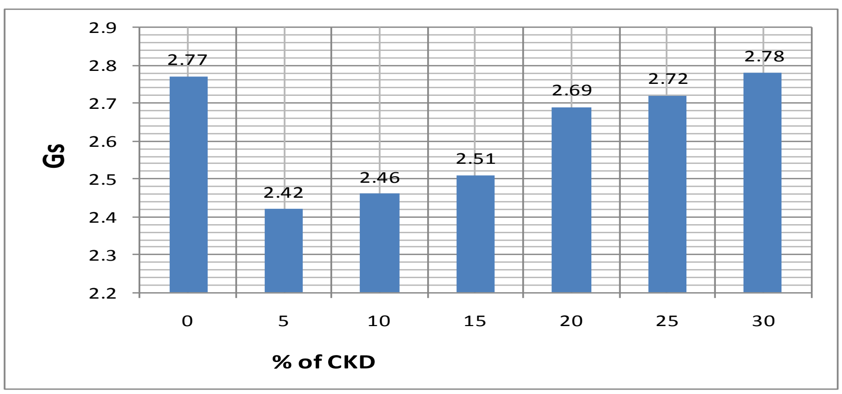


Figure 5.7. Variation of Specific Gravity of blended soil with CKD content

5.7. Improvement on Compaction Characteristics

5.7.1. Maximum Dry Density

The effect of CKD on the maximum dry density of the expansive soil is shown in Figure 5.8 below for uncured and 7 days cured soil sample. As shown in the Figure 5.8, maximum dry density increases from 1.36 g/cm^3 to 1.56 g/cm^3 for uncured and for 7 days cured from 1.18 g/cm^3 to 1.32 g/cm ^{3} with increased CKD content from 0% to 30%.

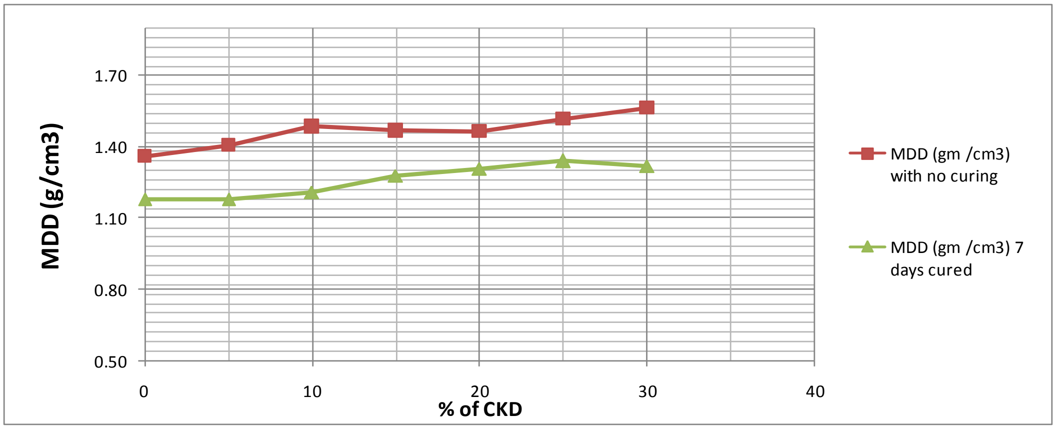


Figure 5.8. Variation of MDD with application of CKD contents

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

- Comparatively specific gravity of CKD and the natural soil are almost near to each other therefore the blended sample brings higher density so that MDD increased

5.7.2. Optimum Moisture Content

The effect of CKD on the optimum moisture content for the soil CKD mixtures are shown in Figure 5.9. The optimum moisture content decreases from 35.75% to 24.13% for uncured and 35.82% to 22.01% for 7 days cured soil samples with increased CKD content from 0% to 30%.

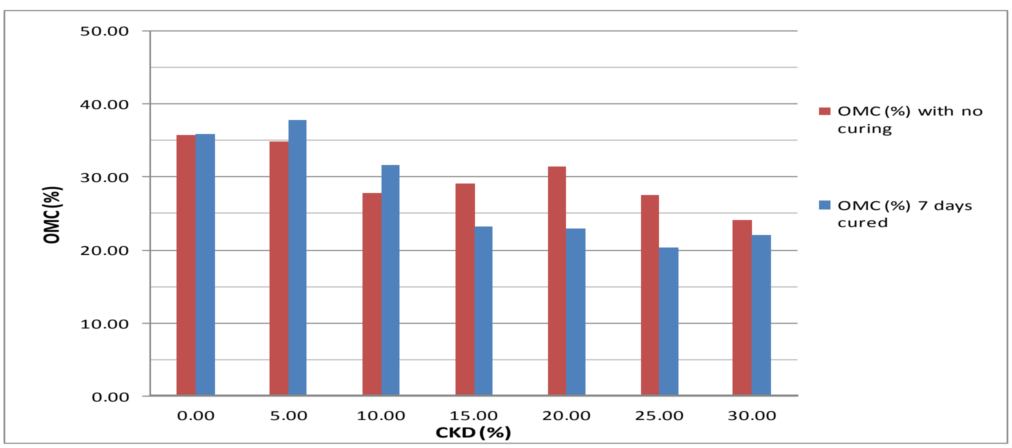


Figure 5.9. Variation of OMC with application of CKD contents

The optimum moisture content of soil decrease with an increase cement kiln dust, because, it has more water absorption behavior. The cement kiln dust forms coarser materials, which occupy larger spaces for retaining water. The decrease of water content may also be attributed by the pozzolanic reaction of cement kiln dust with the soil.

The summaries of compaction curves are shown in Figures 5.10 and 5.11 for uncured and 7 days cured samples respectively. Treating soil with cement kiln dust gives the bell shaped curves for both cured and uncured samples. These curves shifted to the right with respect to untreated soil sample, which also means addition of cement kiln dust decreased the maximum dry density and the optimum moisture content vary with percentage of cement kiln dust. The decrease in maximum dry density and the variation of the corresponding optimum moisture content of all treated soils is related to the additive quantities. Details of the maximum dry density and optimum moisture content results are shown in the Appendix III, 1.2.

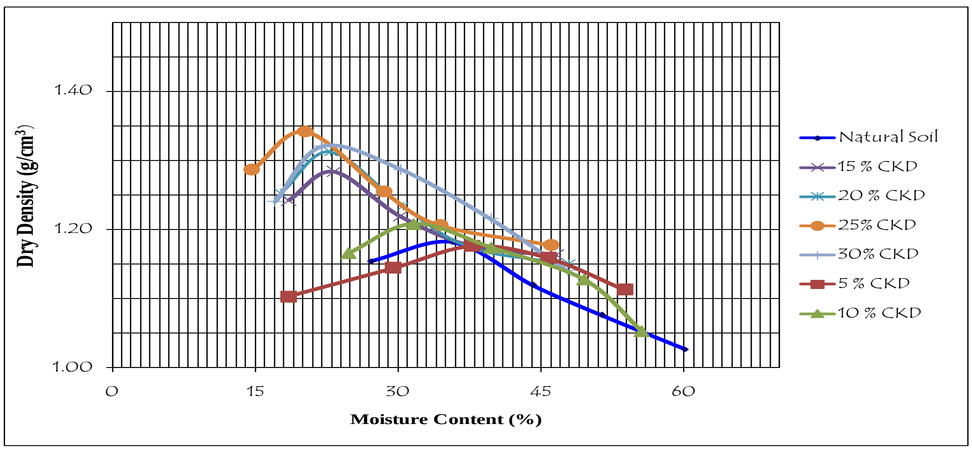


Figure 5.10. Summary of Compaction curves with application of CKD contents for un-cured samples

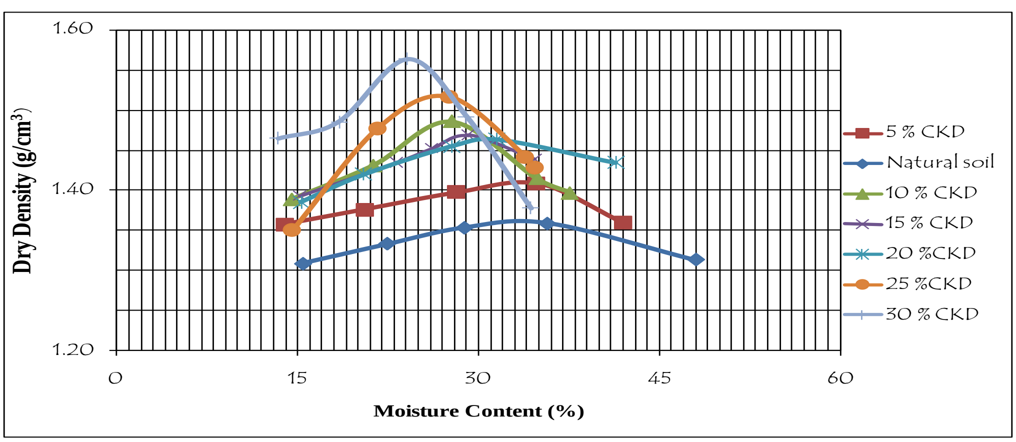


Figure 5.11. Summary of Compaction curves with application of CKD contents for 7 days - cured sample

5.8. Improvement on Linear Shrinkage

The effect of CKD on the linear shrinkage of the expansive soil is shown in Figure 5.12 below. Reduction trend was observed for shrinkage limit values of treated samples as shown in Table 39 below. Maximum decrease was seen for soil blended with 30% CKD that is 27.14 % shrinkage limit was seen on the natural soil for 30%. The CKD blended soil showed shrinkage limit of 19.29% which has a reduction of 7.86 %.Therefore increasing the proportion of CKD additive has more effect on reducing linear shrinkage.

Table 5.8 Summary of Linear Shrinkage test result

% of CKD	Linear Shrinkage LS,%
0	27.14
5	26.43
10	26.43
15	26.43
20	25.00
25	23.57
30	19.29

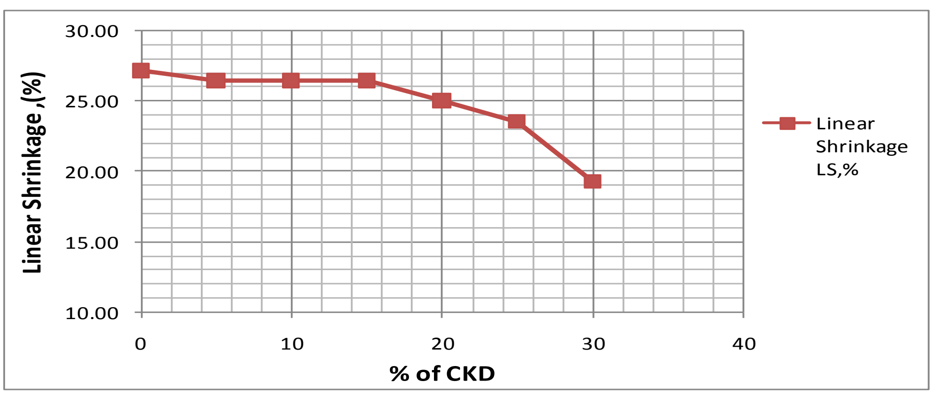


Figure 5.12. Summary of Linear shrinkage curves with the % CKD increment

5.9. Improvement on Unconfined Compressive Strength

Unconfined compressive strength at optimum moisture increased with increasing percentage of CKD, and also increases with curing period as shown in Figure 5.13 and Table 5.9 below.

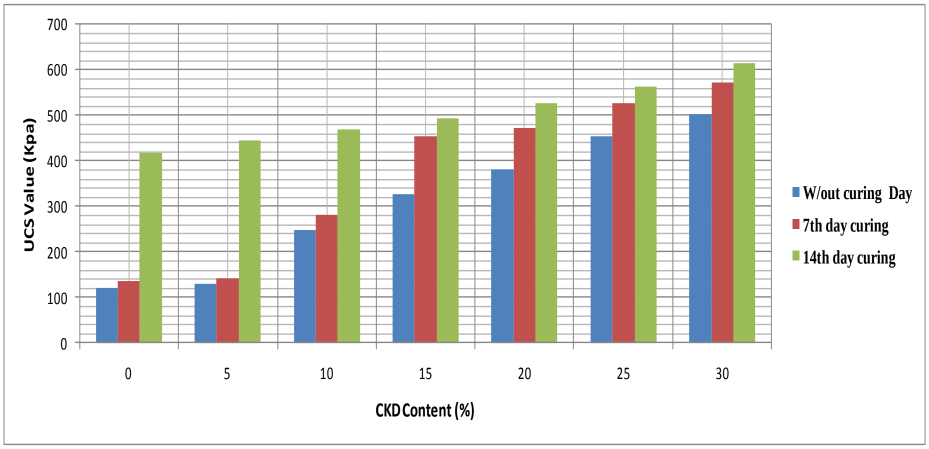


Figure 5.13. Summary of UCS chart with application of CKD contents for un-cured, 7days-cured and 14 days-cured blended samples

Table 5.9. Summary UCS with percentage increment of CKD and curing period

Curing day	% of CKD						
	0	5	10	15	20	25	30
	UCS Value						
W/out curing Day	122.64	128.83	247.14	325.94	380.46	453.05	502
7th day curing	136.95	142.02	281.35	453.04	472.59	526.66	574.01
14th day curing	416.74	445.26	469.56	495.39	526.66	564.17	613.9

- The increase in UCS of CKD treated soils may be attributes to cation exchange reaction and flocculation and agglomeration effect
- The addition of CKD enhances the strength of the untreated soils, while at the same time the soil improves its durability nature or cohesive nature and become more ductile as the axial strain increased considerably with increase in CKD content.
- The failure mode of the treated material exhibited a brittle type of failure mode.

Details of the unconfined Compressive strength test results are shown in the Appendix III, Vi.

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1. Conclusions

This study intends to investigate the suitability of using cement kiln dust as a stabilizer for expansive soils. The effects on Atterberg limits, density, grain size distribution California bearing ratio, pH value, unconfined compressive strength, swell consolidation and free swell percentage, were studied.

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

1. The expansive soil used in this study has liquid limit (LL= 109 %), plastic index(PI= %72) and more than 35 % passing #200 sieve .Thus, as per AASHTO soil classification system, the soil categorized as an A-7-5 /A-&-6 with rating of Fair-to Poor to be used as a sub-grade material.
2. As the combined percentage composition of Cement Kiln Dust, Al_2O_3,CaO, $CaCo_3$,and Fe_2O_3 is more than 70 %,it is a good pozzolana that could help to mobilize the calciumion with the combination of clay to form pozzolanc reaction.
3. The plasticity index is reduced by increasing the CKD content and curing has also significant effect on the plasticity of the expansive soil.
4. With increment of cement kiln dust content,the optimum moisture content decreased while the maximum dry density values increased.
5. Swell –pressure and CBR swell of the stabilized samples decreased with increasing cement kiln dust content.However, the influence of cement kiln dust stabilization on free swell properties of the expansive soil is not that much satisfactory.
6. CBR values and UCS values also increased with increase of cement kiln dust.
7. Linear shrinkage values slightly decreased with increasing percentage of CKD.
8. The improvement effect of curing period, namely, 7 days and 28 days on the CBR and CBR -swelling percentage of the samples was observed
9. It can be obviously said from this study that cement kiln dust can be used as a stabilizer for improvement of expansive soils.

10. Utilization of CKD in this manner also has the advantage of reusing an industrial waste by-product without adversely affecting the environment or potential land use.

6.2. Recommendation

- In this study, cement kiln dust was examined as a stabilizing agent for expansive soils and swell potential tests were performed. Although performed swell tests are reliable, additional tests (scanning electron microscope and X-Ray diffraction studies) can be done to examine the microfabric and mineralogical characteristics of the specimens in order to reach to the full scale characterization of the specimens.
- Mineralogy has an effect on the stabilization of soil with chemical additives. Knowing the mineralogy of soils, helps to understand the response when a chemical agent is mixed. Therefore, it is recommended to study mineralogy of representative soil samples of the different expansive soils of Addis Ababa.
- Long term durability of the stabilized material, wetting-drying and leaching effect need further study.
- Studies on effect of cement kiln dust fineness on the stabilization of expansive soil.
- Stabilizing soil with the combination of cement kiln dust with lime on different types of soils.
- Stabilizing soil with the combination of cement kiln dust with marble dust on different types of soil.

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ANNEX I- Laboratory Test Results of Expansive Clay Soil

1. Gradation test result of the natural soil

A. Sieve Analysis

Total mass of sample, 1347.9 g

Sieve Analysis

Sieve Opening	Mass of Sieve (g)	Mass of sieve + Retained soil (g)	Mass of Retained soil (g)	Percentage Retained (%)	Cum. Percent. Retained (%)	Percentage Passing (%)
4.75	1263.40	1272.4	9	0.67	0.67	99.33
2.36	990.49	1008.1	17.61	1.31	1.97	98.03
2	956.30	959.8	3.5	0.26	2.23	97.77
1.18	894.80	905.8	11	0.82	3.05	96.95
0.6	831.40	840.5	9.1	0.68	3.73	96.27
0.425	786.60	791.5	4.9	0.36	4.09	95.91
0.3	750.30	755.4	5.1	0.38	4.47	95.53
0.15	778.30	791	12.7	0.94	5.41	94.59
0.075	764.00	776.1	12.1	0.90	6.31	93.69
pan			1262.89	93.69	100.00	0.00
			85.01			
			1347.9			

B. Hydrometer Analysis

Gs=2.77

Elapsed Time (min)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Hydrometer Depth (L)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)	Test Temperature, deg.c
0.5	1.033	0.00245	1.03055	7.57084	0.01309	0.051	95.620	89.59	Avg.21^{o}c
1	1.032	0.00245	1.02955	7.83536	0.01309	0.037	92.490	86.66	
2	1.0315	0.00245	1.02905	7.96762	0.01309	0.026	90.925	85.19	
4	1.031	0.00245	1.02855	8.09988	0.01309	0.019	89.360	83.72	
8	1.0305	0.00245	1.02805	8.23214	0.01309	0.013	87.795	82.26	
15	1.03	0.00245	1.02755	8.3644	0.1309	0.098	86.230	80.79	
30	1.029	0.00245	1.02655	8.62892	0.01309	0.007	83.100	77.86	
60	1.028	0.00245	1.02555	8.89344	0.01309	0.005	79.970	74.93	
120	1.0275	0.00245	1.02505	9.0257	0.01309	0.004	78.405	73.46	
240	1.026	0.00245	1.02355	9.42248	0.01309	0.003	73.710	69.06	
480	1.0244	0.00245	1.02195	9.845712	0.01309	0.002	68.702	64.37	
1440	1.024	0.00245	1.02155	9.95152	0.01309	0.001	67.450	63.20	

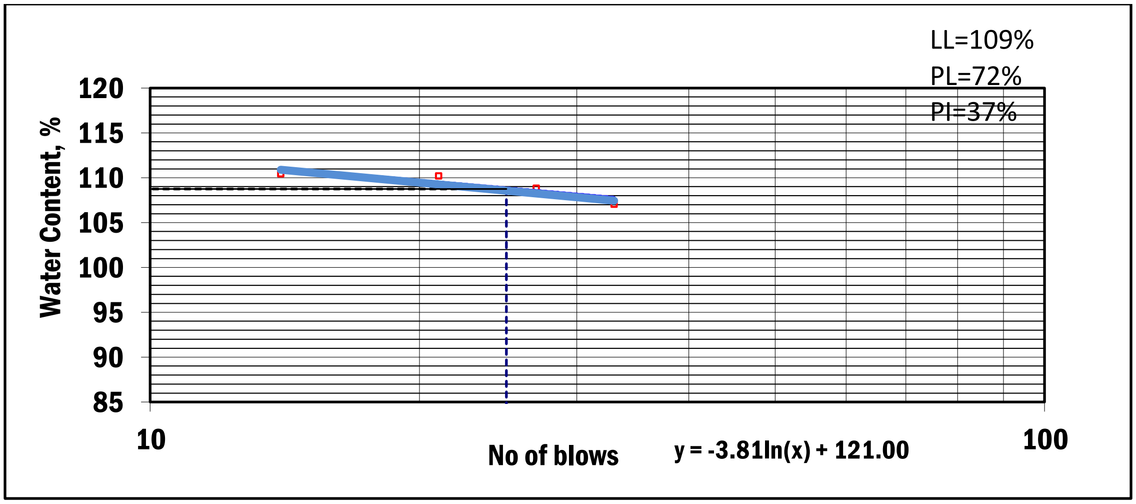
2. Atterberg Limits

2.1 Natural Soil with no-curing result

Natural Soil No-curing

	Liquid Limit				Plastic Limit	
Trial No	1	2	3		1	2
Container No	9	D	AA		H20	HD4
Mass of container, g	33.70	33.30	33.30	32.80	32.80	33.30
Mass of container + Wet soil, g	45.50	47.50	45.70	46.90	39.40	38.40
Mass of container + Dry soil, g	39.40	40.10	39.20	39.50	37.80	36.90
Mass of water, g	6.10	7.40	6.50	7.40	1.60	1.50
Mass of dry soil, g	5.70	6.80	5.90	6.70	5.00	3.60
Water content, %	107.02	108.82	110.17	110.45	32.00	41.67
No of blows	33	27	21	14	------	------

Liquid Limit, % = 109 Plastic Limit, % = 37 PI, %= 72

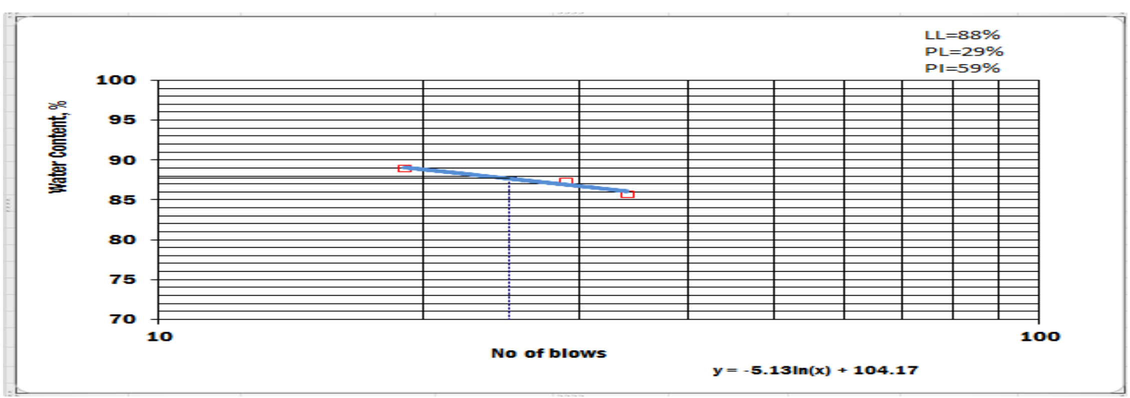


2.2 Natural Soil with 7-days cured result

Natural Soil 7days Cured

	Liquid Limit				Plastic Limit	
Trial No	1	2	3		1	2
Container No	9	D	AA		H20	HD4
Mass of container, g	17.74	17.80	17.91		17.76	17.88
Mass of container + Wet soil, g	47.50	47.40	45.10		23.04	22.94
Mass of container + Dry soil, g	33.76	33.60	32.30		21.82	21.85
Mass of water, g	13.74	13.80	12.80		1.22	1.09
Mass of dry soil, g	16.02	15.80	14.39		4.06	3.97
Water content, %	85.77	87.34	88.95		30.05	27.46
No of blows	34	29	19		------	------

Liquid Limit, % = 88 Plastic Limit, % = 29 PI, %= 59



3. Initial Moisture Content

Calculation of Initial Moisture Content of the Soil			
Trial No	1	2	3
Container No	C40	21	J45
Mass of container, g	15.60	15.50	15.40
Mass of container + Wet soil, g	32.90	34.70	39.70
Mass of container + Dry soil, g	28.10	29.20	33.10
Mass of water, g	4.80	5.50	6.60
Mass of dry soil, g	12.50	13.70	17.70
Water content, %	38.40	40.15	37.29
Ave. Initial moisture content,%	**38.61**		

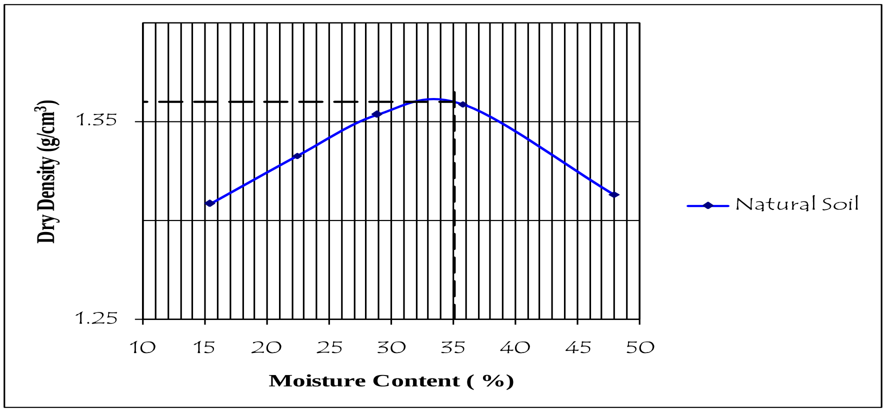
4. Moisture –Density Relationship

Natural Soil with No-curing

Moisture content Vs dry density computation table

Determination No.	1	2	3	4	5
Mass of Mold, g	5060	5160	5160	5160	5160
Mass of mold + Compacted Soil, g	6564	6785	6897	6997	7095
Mass of Compacted soil, g	1504	1625	1737	1837	1935
Volume of Mold, cm3	995.96	995.96	995.96	995.96	995.96
Bulk density, g/cm3	1.51	1.63	1.74	1.84	1.94
Water Contenet, %	15.42	22.43	28.86	35.75	47.98
Dry density, g/cm3	1.31	1.33	1.35	1.36	1.31

Water Content Calcuation					
Conatainer No	p14	q2	t2	ne18	n2
Mass of container, g	69	88.7	91.8	69.4	68.4
Mass of container + wet soil, g	200	200.6	200.3	200.4	200.4
Mass of ontainer + Dry soil, g	182.5	180.1	176	165.9	157.6
Mass of Water, g	17.5	20.5	24.3	34.5	42.8
Mass of Dry soil, g	113.5	91.4	84.2	96.5	89.2
Water content, %	15.42	22.43	28.86	35.75	47.98
Dry Unit Weight, g/cm3	1.31	1.33	1.35	1.36	1.31



MDD=1.36g/cm^3

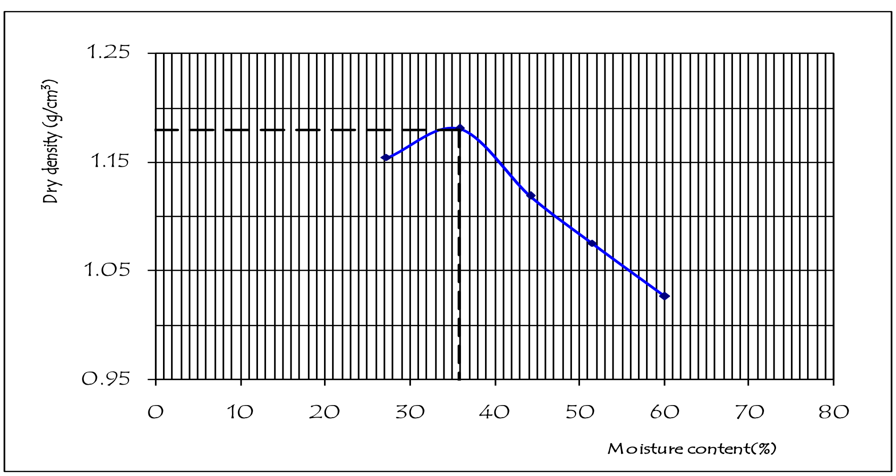
OMC=35.75 %

Natural Soil with 7day Cured

Moisture content Vs dry density computation table

Determination No.	1	2	3	4	5
Mass of Mold, g	4571.1	4571.1	4571.1	4571.1	4571.1
Mass of mold + Compacted Soil, g	5957.5	6085.5	6094.7	6110.2	6125.4
Mass of Compacted soil, g	1386.4	1514.4	1523.6	1539.1	1554.3
Volume of Mold, cm3	944	944	944	945	946
Bulk density, g/cm3	1.47	1.60	1.61	1.63	1.64
Water Contenet, %	27.23	35.82	44.19	51.40	60.07
Dry density, g/cm3	1.15	1.18	1.12	1.08	1.03

Water Content					
Conatainer No	135	128	156	37	116
Mass of container, g	32.6	32.9	33.6	33.7	33
Mass of container + wet soil, g	132.6	125.8	123	125.6	126.8
Mass of ontainer + Dry soil, g	111.2	101.3	95.6	94.4	91.6
Mass of Water, g	21.4	24.5	27.4	31.2	35.2
Mass of Dry soil, g	78.6	68.4	62	60.7	58.6
Water content, %	27.23	35.82	44.19	51.40	60.07
Dry Unit Weight, g/cm3	1.15	1.18	1.12	1.08	1.03



MDD=1.18g/cm^3

OMC=35.82%

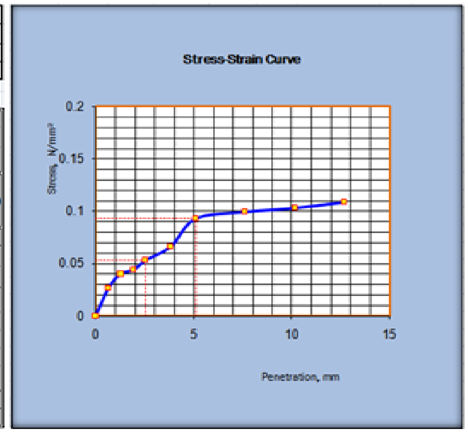
5. Californian Bearing Ratio (CBR) Tests

Natural Soil	Ring Calibration Factor,N/Div	25.707	Swell data	
4 days soaked	Plunger Area, mm^2	1935	Initial Reading	Final reading
	Rate of strain ,mm/min	1.27	1.05	10.56
	Rammer wt. (kg)	4.54	8.17	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	8.17
CBR Value, %	0.90

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0	0	0		
0.64	2	51	0.03		
1.27	3	77	0.04		
1.91	3	85	0.04		
2.54	4	103	0.05	6.9	0.77
3.81	5	129	0.07		
5.08	7	180	0.09	10.3	0.90
7.62	8	193	0.10		
10.16	9	200	0.10		
12.70	10	210	0.11		

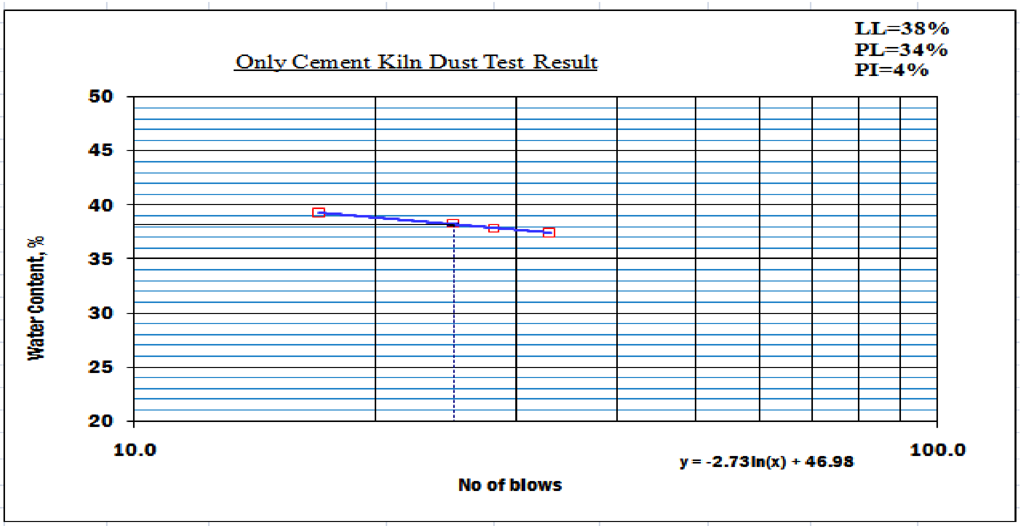


ANNEX II-Laboratory Test Results of Cement Kiln Dust

1. Atterberg Limit Test

Atterberg Limit test result for Cement Kiln Dust

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	6	F	FA	ZD	D4	HD4
Mass of container, g	15.140	14.860	14.990	14.660	15.580	14.990
Mass of container + Wet soil, g	32.200	33.470	34.580	35.760	27.390	27.360
Mass of container + Dry soil, g	27.560	28.360	29.150	29.820	24.380	24.290
Mass of water, g	4.64	5.11	5.43	5.94	3.01	3.07
Mass of dry soil, g	12.42	13.50	14.16	15.16	8.80	9.30
Water content, %	37.36	37.85	38.35	39.18	34.20	33.01
No of blows	33.0	28.0	25.0	17.0	------	------
Liquid Limit, % = 38	Plastic Limit, %	=	34	PI, %=	4	



ANNEX III- Laboratory Test Result of Blended Soil with Cement Kiln Dust

1) Blended soil plus CKD

1.1 Atterberg Limit Test Result

1.1.1) soil blended with 5% CKD

i) Soil + 5 % CKD with no-curing

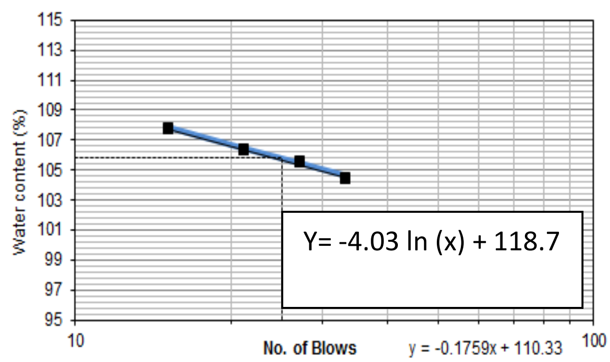
5 % CKD blending with No- curing

Set number	Tare mass (g) Wc	Tare with wet soil (g) Ww	Tare with dry soil (g) Wd	Blow count N	Water content (%) w	Water content fitted (%)
1	33.80	47.30	40.40	33	104.55	104.66
2	32.90	47.50	40.00	27	105.63	105.47
3	33.40	46.20	39.60	21	106.45	106.48
4	33.50	46.80	39.90	15	107.81	107.84
5	32.80	41.40	39.30		32.31	
6	33.60	41.80	39.90		30.16	

Slope of flow line = 0.038

Liquid limit (%) = 105.78 Plastic limit (%) = 31.23

plasticity index = 74.54

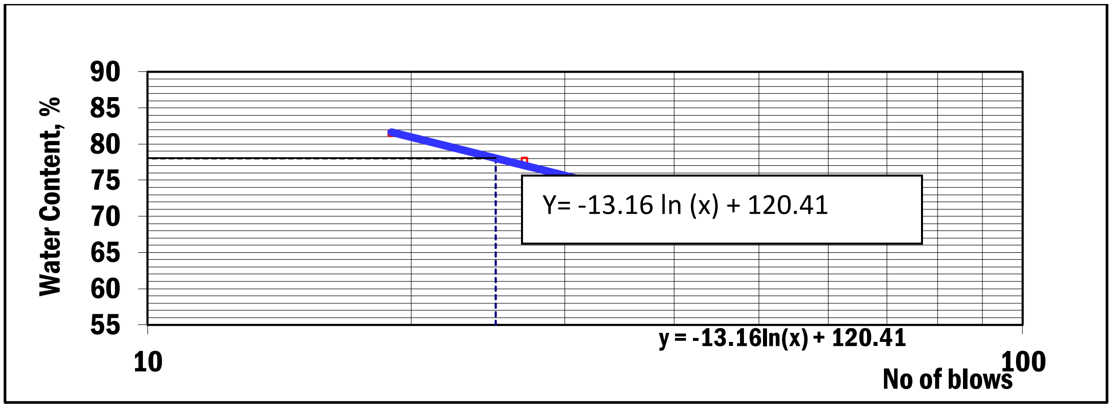


ii) Soil+ 5 % CKD with 7 days –Cured

5 % CKD blended soil 7days cured

	Liquid Limit				Plastic Limit	
Trial No	1	2	3		1	2
Container No	9	D	AA		H20	HD4
Mass of container, g	18.10	17.80	17.91		17.76	17.88
Mass of container + Wet soil, g	42.56	39.16	41.59		22.38	22.37
Mass of container + Dry soil, g	32.10	29.82	30.96		21.28	21.27
Mass of water, g	10.46	9.34	10.63		1.10	1.10
Mass of dry soil, g	14.00	12.02	13.05		3.52	3.39
Water content, %	74.71	77.70	81.46		31.25	32.45
No of blows	31	27	19		------	------

Liquid Limit, % = 78 Plastic Limit, % = 32 PI, %= 46

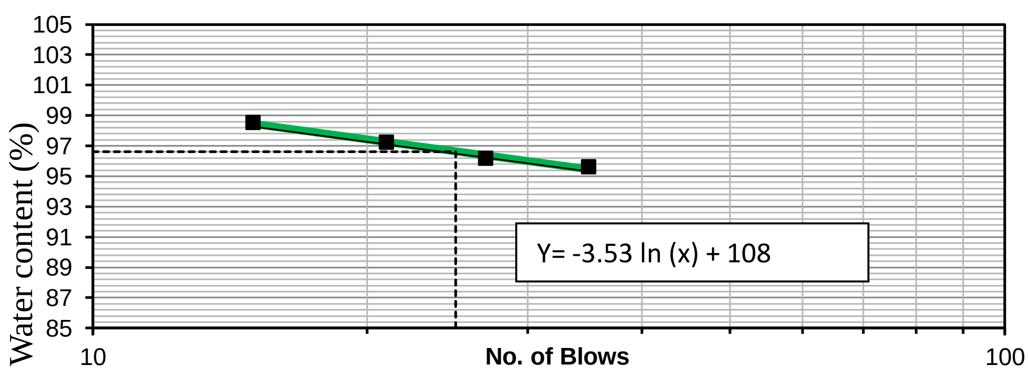


1.1.2. Soil blended with 10 % CKD

i) Soil + 10 % CKD with no-curing

Set number	Tare mass (g) Wc	Tare with wet soil (g) Ww	Tare with dry soil (g) Wd	Blow count N	Water content (%) w	Water content fitted (%)
1	33.30	46.60	40.10	35	95.59	95.44
2	33.20	43.40	38.40	27	96.15	96.36
3	33.10	47.30	40.30	21	97.22	97.24
4	32.90	46.20	39.60	15	98.51	98.43
5	33.20	41.90	40.10		26.09	
6	33.70	41.80	40.00		28.57	

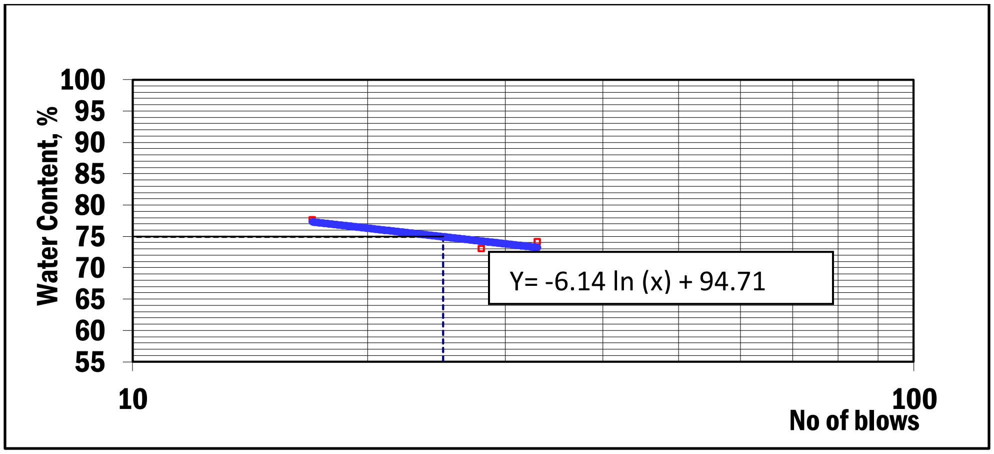
Slope of flow line = 0.036
Liquid limit (%) = 96.63 Plastic limit (%) = 27.33
plasticity index = 69.30



ii) Soil + 10%CKD with 7days cured

	Liquid Limit				Plastic Limit	
Trial No	1	2	3		1	2
Container No	9	D	AA		H20	HD4
Mass of container, g	17.94	17.83	17.81		17.76	17.88
Mass of container + Wet soil, g	43.03	38.58	41.59		22.28	22.29
Mass of container + Dry soil, g	32.35	29.82	31.20		21.38	21.27
Mass of water, g	10.68	8.76	10.39		0.90	1.02
Mass of dry soil, g	14.41	11.99	13.39		3.62	3.39
Water content, %	74.12	73.06	77.60		24.86	30.09
No of blows	33	28	17		——	——

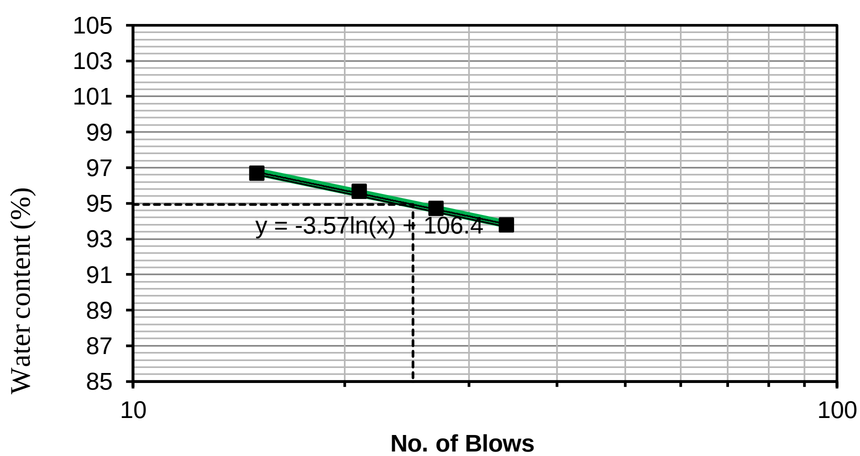
Liquid Limit, % = 75 Plastic Limit, % = 27 PI, %= 48



1.1.3 Soil blended with 15 % CKD

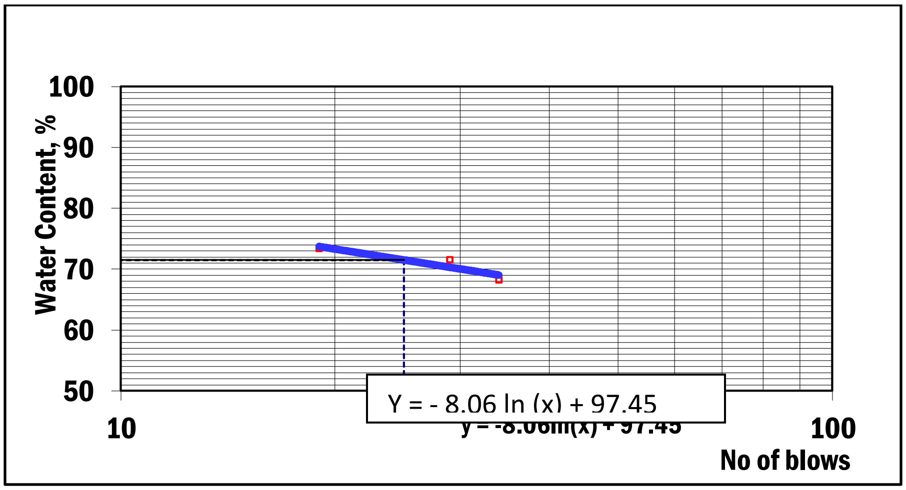
i) 15 % CKD with no-cured

15% CKD blending						
Set number	Tare mass (g) Wc	Tare with wet soil (g) Ww	Tare with dry soil (g) Wd	Blow count N	Water content (%) w	Water content fitted (%)
1	*33.60*	*46.00*	*40.00*	*34*	93.75	93.82
2	*32.90*	*48.30*	*40.81*	*27*	94.69	94.65
3	*33.60*	*47.10*	*40.50*	*21*	95.65	95.55
4	*33.30*	*47.50*	*40.52*	*15*	96.68	96.75
5	*33.40*	*40.70*	*39.20*		25.86	
6	*33.70*	*40.90*	*39.40*		26.32	
Slope of flow line = 0.038						
Liquid limit (%) = 94.92			Plastic limit (%) =		26.09	
plasticity index =		68.83				



ii) 15 % CKD with 7days cured

15 % CKD blended soil						
	Liquid Limit				Plastic Limit	
Trial No	1	2	3		1	2
Container No	9	D	AA		H20	HD4
Mass of container, g	17.75	17.77	17.96		17.76	18.36
Mass of container + Wet soil, g	46.02	47.49	46.64		23.35	23.57
Mass of container + Dry soil, g	34.56	35.10	34.50		22.13	22.38
Mass of water, g	11.46	12.39	12.14		1.22	1.19
Mass of dry soil, g	16.81	17.33	16.54		4.37	4.02
Water content, %	68.17	71.49	73.40		27.92	29.60
No of blows	34	29	19		——	——
Liquid Limit, % = 72	Plastic Limit, %	=	29		PI, %= 43	

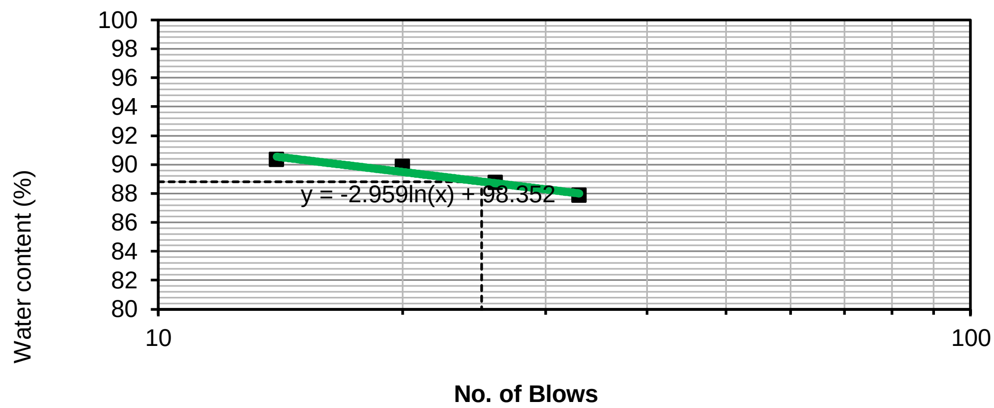


1.1.4 Soil blended with 20 % CKD

i) 20% CKD no-cured

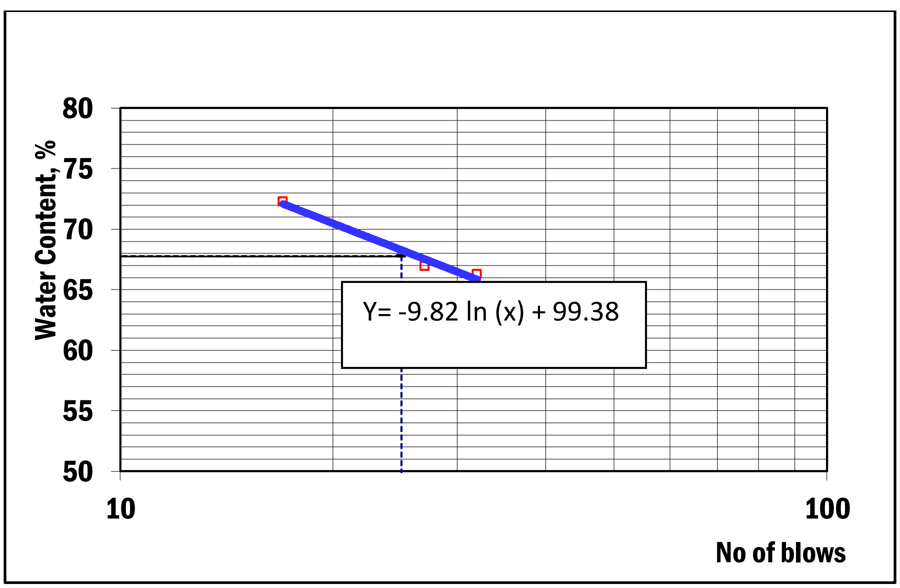
20% CKD blending						
Set number	Tare mass (g)	Tare with wet soil (g)	Tare with dry soil (g)	Blow count	Water content (%)	Water content fitted (%)
	Wc	Ww	Wd	N	w	
1	*32.90*	*46.80*	*40.30*	*33*	87.84	88.01
2	*33.60*	*47.00*	*40.70*	*26*	88.73	88.71
3	*34.00*	*47.10*	*40.90*	*20*	89.86	89.49
4	*32.90*	*44.70*	*39.10*	*14*	90.32	90.54
5	*33.30*	*40.30*	*38.30*		40.00	
6	*32.90*	*40.00*	*38.00*		39.22	

Slope of flow line = 0.033
Liquid limit (%) = 88.83 Plastic limit (%) = 39.61
plasticity index = 49.22



ii) 20% CKD with 7days cured

20 % CKD blended soil						
	Liquid Limit				Plastic Limit	
Trial No	1	2	3		1	2
Container No	9	D	AA		H20	HD4
Mass of container, g	17.79	17.88	18.33		17.63	17.55
Mass of container + Wet soil, g	50.13	50.88	49.92		22.81	22.18
Mass of container + Dry soil, g	37.24	37.65	36.67		21.58	21.14
Mass of water, g	12.89	13.23	13.25		1.23	1.04
Mass of dry soil, g	19.45	19.77	18.34		3.95	3.59
Water content, %	66.27	66.92	72.25		31.14	28.97
No of blows	32	27	17		------	------
Liquid Limit, % = 68	Plastic Limit, %	=	30		PI, %= 38	

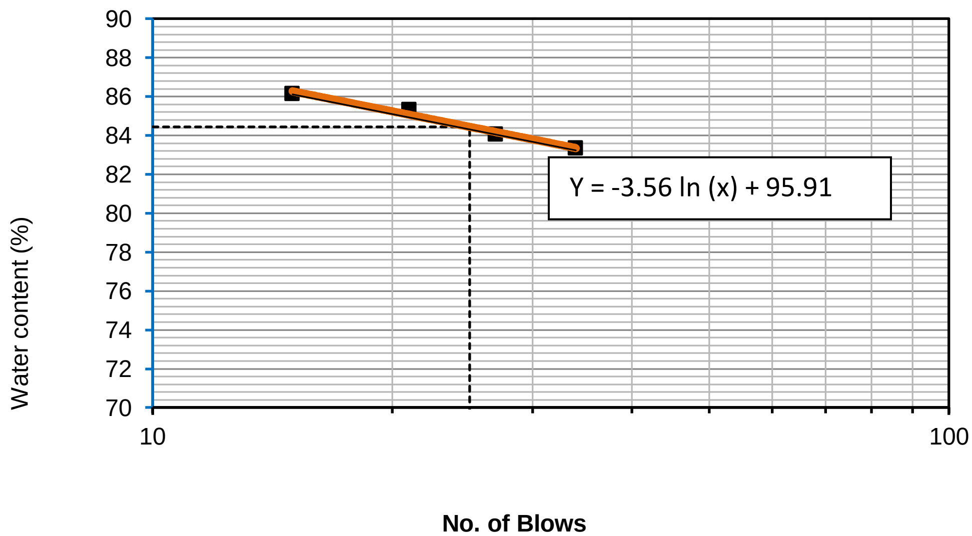


1.1.5 Soil blended with 25 % CKD

i) 25% CKD No-curing

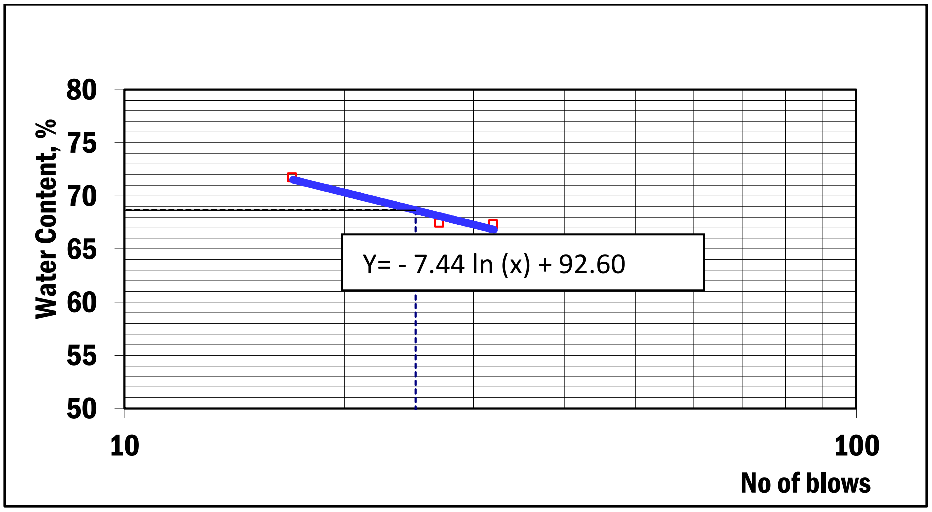
Set number	Tare mass (g) Wc	Tare with wet soil (g) Ww	Tare with dry soil (g) Wd	Blow count N	Water content (%) w	Water content fitted (%)
1	*33.20*	*47.50*	*41.00*	*34*	83.33	83.35
2	*32.20*	*44.90*	*39.10*	*27*	84.06	84.17
3	*33.40*	*44.00*	*39.12*	*21*	85.31	85.07
4	*33.70*	*45.80*	*40.20*	*15*	86.15	86.27
5	*33.60*	*41.60*	*39.70*		31.15	
6	*33.70*	*41.80*	*40.10*		26.56	

Slope of flow line = 0.042
Liquid limit (%) = 84.45 Plastic limit (%) = 28.86
plasticity index = 55.59



ii) 25 % CKD 7days cured

25 % CKD blended soil	Liquid Limit				Plastic Limit	
Trial No	1	2	3		1	2
Container No	9	D	AA		H20	HD4
Mass of container, g	17.79	17.88	18.33		17.63	17.55
Mass of container + Wet soil, g	50.33	50.98	49.82		22.92	22.28
Mass of container + Dry soil, g	37.24	37.65	36.67		21.58	21.14
Mass of water, g	13.09	13.33	13.15		1.34	1.14
Mass of dry soil, g	19.45	19.77	18.34		3.95	3.59
Water content, %	67.30	67.43	71.70		33.92	31.75
No of blows	32	27	17		——	——
Liquid Limit, % = 69	Plastic Limit, %	=	33		PI, %= 36	



1.1.6 Soil blended with 30 % CKD

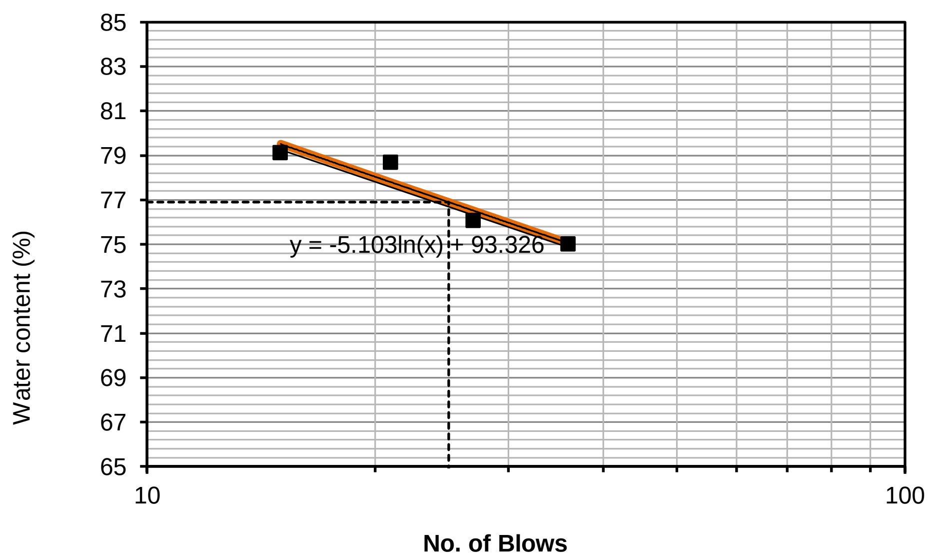
i) 30% CKD no-cured

Set number	Tare mass (g) Wc	Tare with wet soil (g) Ww	Tare with dry soil (g) Wd	Blow count N	Water content (%) w	Water content fitted (%)
1	*33.50*	*45.40*	*40.30*	*36*	75.00	75.04
2	*33.50*	*46.00*	*40.60*	*27*	76.06	76.51
3	*33.20*	*44.10*	*39.30*	*21*	78.69	77.79
4	*33.70*	*45.70*	*40.40*	*15*	79.10	79.51
5	*32.70*	*41.30*	*39.60*		24.64	
6	*33.30*	*41.40*	*39.60*		28.57	

Slope of flow line = 0.066

Liquid limit (%) = 76.90 Plastic limit (%) = 26.60

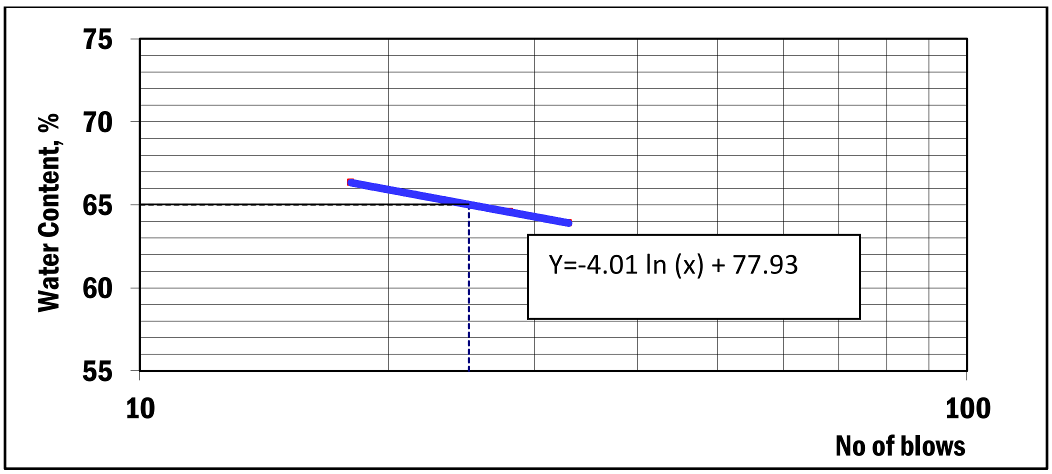
plasticity index = 50.30



ii) 30 % CKD with 7days cured

	Liquid Limit				Plastic Limit	
Trial No	1	2	3		1	2
Container No	9	D	AA		H20	HD4
Mass of container, g	17.78	18.32	17.59		17.76	17.88
Mass of container + Wet soil, g	50.02	50.00	51.34		23.04	22.94
Mass of container + Dry soil, g	37.45	37.57	37.88		21.82	21.85
Mass of water, g	12.57	12.43	13.46		1.22	1.09
Mass of dry soil, g	19.67	19.25	20.29		4.06	3.97
Water content, %	63.90	64.57	66.34		30.05	27.46
No of blows	33	28	18		------	------

Liquid Limit, % = 65 Plastic Limit, % = 29 PI, %= 36



1.2.**Moisture –Density (compaction) Test Result**

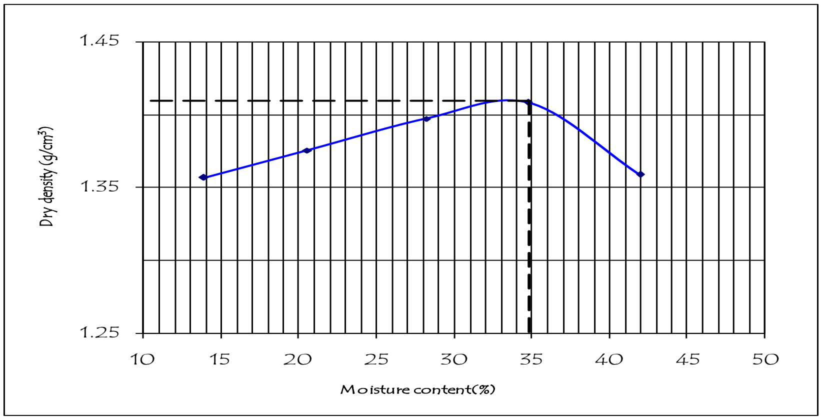
i) With No-curing period

Soil Blended with 5 % CKD

Moisture content Vs dry density computation table

Determination No.	1	2	3	4	5
Mass of Mold, g	5060	5160	5160	5160	5160
Mass of mold + Compacted Soil, g	6600	6812	6944	7051	7082
Mass of Compacted soil, g	1540	1652	1784	1891	1922
Volume of Mold, cm3	995.96	995.96	995.96	995.96	995.96
Bulk density, g/cm3	1.55	1.66	1.79	1.90	1.93
Water Contenet, %	13.94	20.58	28.20	34.80	41.99
Dry density, g/cm3	1.36	1.38	1.40	1.41	1.36

Water Content Calcuation					
Conatainer No	p14	q2	t2	ne18	n2
Mass of container, g	86.6	69.4	78.3	92.6	70.5
Mass of container + wet soil, g	190.4	190.7	190.6	190.6	190.2
Mass of ontainer + Dry soil, g	177.7	170	165.9	165.3	154.8
Mass of Water, g	12.7	20.7	24.7	25.3	35.4
Mass of Dry soil, g	91.1	100.6	87.6	72.7	84.3
Water content, %	13.94	20.58	28.20	34.80	41.99
Dry Unit Weight, g/cm^3	1.36	1.38	1.40	1.41	1.36



MDD=1.41g/cm^3

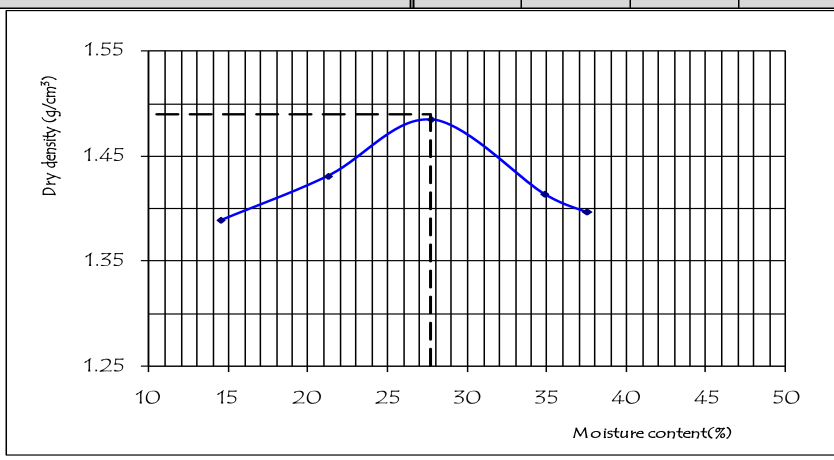
OMC=34.8 %

Soil blended with 10% CKD

Moisture content Vs dry density computation table

Determination No.	1	2	3	4	5
Mass of Mold, g	5060	5060	5060	5060	5060
Mass of mold + Compacted Soil, g	6645	6789	6950	6960	6960
Mass of Compacted soil, g	1585	1729	1890	1900	1914
Volume of Mold, cm3	995.96	995.96	995.96	995.96	995.96
Bulk density, g/cm3	1.59	1.74	1.90	1.91	1.92
Water Contenet, %	14.56	21.30	27.75	34.88	37.57
Dry density, g/cm3	1.39	1.43	1.49	1.41	1.40

Water Content Calculation					
Conatainer No	p14	q2	t2	ne18	et3
Mass of container, g	86	85.1	70.4	65.1	61.4
Mass of container + wet soil, g	209.5	204.7	206.2	184.6	183.88
Mass of ontainer + Dry soil, g	193.8	183.7	176.7	153.7	150.43
Mass of Water, g	15.7	21	29.5	30.9	33.45
Mass of Dry soil, g	107.8	98.6	106.3	88.6	89.03
Water content, %	14.56	21.30	27.75	34.88	37.57
Dry Unit Weight, g/cm3	1.39	1.43	1.49	1.41	1.40



MDD=1.49g/cm^3

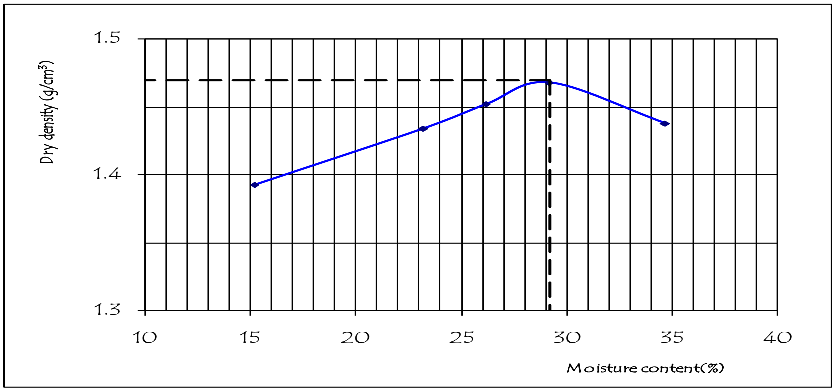
OMC=27.75 %

Soil Blended with 15 % CKD

Moisture content Vs dry density computation table

Determination No.	1	2	3	4	5
Mass of Mold, g	5060	5060	5060	5060	5060
Mass of mold + Compacted Soil, g	6658	6820	6885	6949	6989
Mass of Compacted soil, g	1598	1760	1825	1889	1929
Volume of Mold, cm3	995.96	995.96	995.96	995.96	995.96
Bulk density, g/cm3	1.60	1.77	1.83	1.90	1.94
Water Contenet, %	15.20	23.20	26.18	29.17	34.67
Dry density, g/cm3	1.39	1.43	1.45	1.47	1.44

Water Content Calcuation					
Conatainer No	p14	q2	t2	ne18	n2
Mass of container, g	67.4	111.5	82.4	68.9	66.9
Mass of container + wet soil, g	200	200.7	200	200.4	200.9
Mass of ontainer + Dry soil, g	182.5	183.9	175.6	170.7	166.4
Mass of Water, g	17.5	16.8	24.4	29.7	34.5
Mass of Dry soil, g	115.1	72.4	93.2	101.8	99.5
Water content, %	15.20	23.20	26.18	29.17	34.67
Dry Unit Weight, g/cm3	1.39	1.43	1.45	1.47	1.44



MDD=1.47g/cm^3

OMC=29.17 %

Soil Blended with 20% CKD

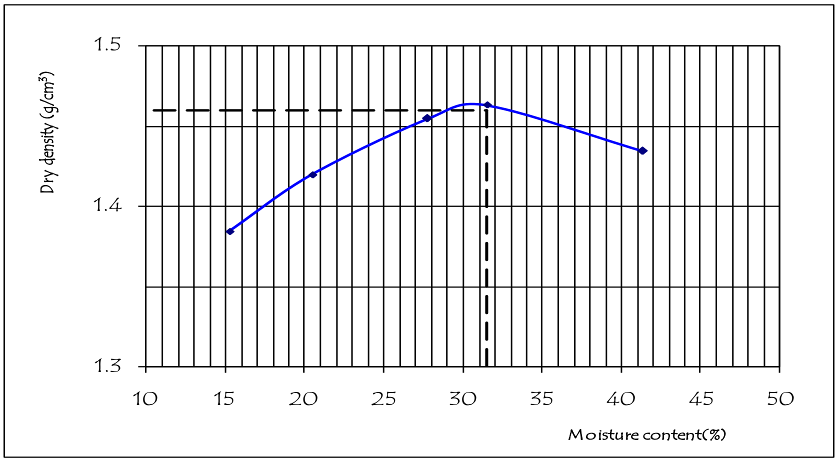
Moisture content Vs dry density computation table

Determination No.	1	2	3	4	5
Mass of Mold, g	5060	5060	5060	5060	5060
Mass of mold + Compacted Soil, g	6650	6764	6912	6977	7080
Mass of Compacted soil, g	1590	1704	1852	1917	2020
Volume of Mold, cm3	995.96	995.96	995.96	995.96	995.96
Bulk density, g/cm3	1.60	1.71	1.86	1.92	2.03
Water Contenet, %	15.30	20.50	27.81	31.53	41.39
Dry density, g/cm3	1.38	1.42	1.45	1.46	1.43

Water Content Calcuation					
Conatainer No	p14	q2	t2	ne18	n2
Mass of container, g	70.4	70.8	85.7	99.6	70
Mass of container + wet soil, g	200.8	200.1	200.6	206.4	200.5
Mass of ontainer + Dry soil, g	183.5	178.1	175.6	180.8	162.3
Mass of Water, g	17.3	22	25	25.6	38.2
Mass of Dry soil, g	113.1	107.3	89.9	81.2	92.3
Water content, %	15.30	20.50	27.81	31.53	41.39
Dry Unit Weight, g/cm3	1.38	1.42	1.45	1.46	1.43

MDD=1.45g/cm^3

OMC=27.81 %

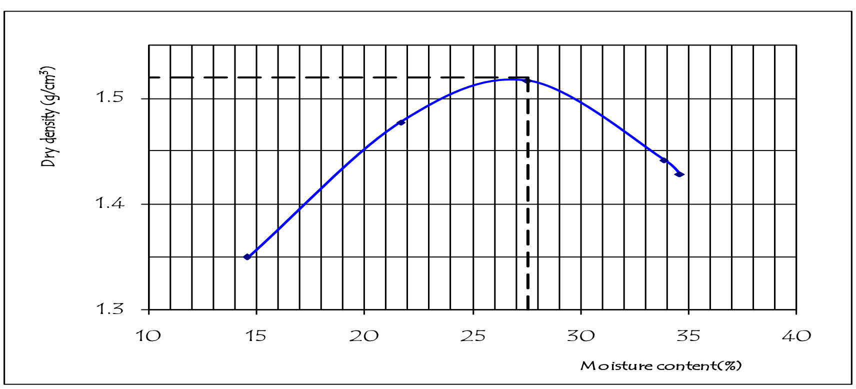


Soil Blended with 25% CKD

Moisture content Vs dry density computation table

Determination No.	1	2	3	4	5
Mass of Mold, g	5060	5060	5060	5060	5060
Mass of mold + Compacted Soil, g	6600	6850	6987	6982	6975
Mass of Compacted soil, g	1540	1790	1927	1922	1915
Volume of Mold, cm3	995.96	995.96	995.96	995.96	995.96
Bulk density, g/cm3	1.55	1.80	1.93	1.93	1.92
Water Contenet, %	14.56	21.66	27.56	33.87	34.60
Dry density, g/cm3	1.35	1.48	1.52	1.44	1.43

Water Content Calcuation					
Conatainer No	p14	q2	t2	ne18	et
Mass of container, g	91.2	86.8	87.3	85	86.7
Mass of container + wet soil, g	203.7	200.8	202.1	201.2	200.02
Mass of ontainer + Dry soil, g	189.4	180.5	177.3	171.8	170.89
Mass of Water, g	14.3	20.3	24.8	29.4	29.13
Mass of Dry soil, g	98.2	93.7	90	86.8	84.19
Water content, %	14.56	21.66	27.56	33.87	34.60
Dry Unit Weight, g/cm3	1.35	1.48	1.52	1.44	1.43



MDD=1.52g/cm^3

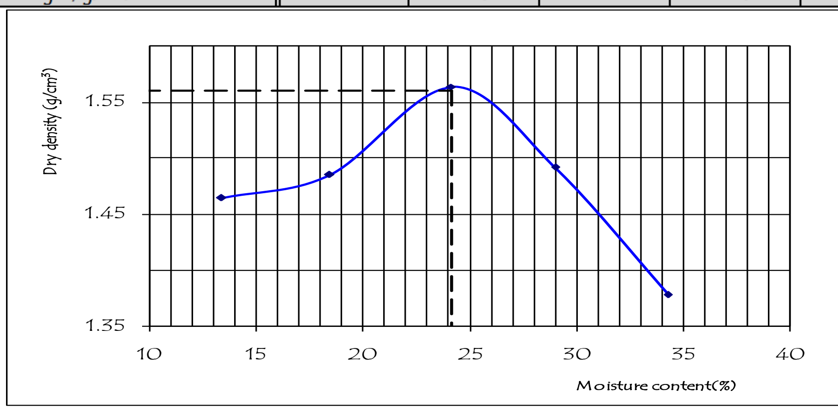
OMC=27.56 %

Soil Blended with 30% CKD

Moisture content Vs dry density computation table

Determination No.	1	2	3	4	5
Mass of Mold, g	5060	5060	5060	5060	5060
Mass of mold + Compacted Soil, g	6714.00	6812.00	6993.00	6977.00	6964.00
Mass of Compacted soil, g	1654	1752	1933	1917	1845
Volume of Mold, cm3	995.96	995.96	995.96	995.96	996.96
Bulk density, g/cm3	1.66	1.76	1.94	1.92	1.85
Water Contenet, %	13.38	18.44	24.13	29.01	34.31
Dry density, g/cm3	1.46	1.49	1.56	1.49	1.38

Water Content Calcuation					
Conatainer No	p14	q2	t2	ne18	f9
Mass of container, g	65.4	68.9	90.7	69.8	63.4
Mass of container + wet soil, g	200.10	200.60	200.80	200.10	220.00
Mass of ontainer + Dry soil, g	184.20	180.10	179.40	170.80	180.00
Mass of Water, g	15.9	20.5	21.4	29.3	40
Mass of Dry soil, g	118.8	111.2	88.7	101	116.6
Water content, %	13.38	18.44	24.13	29.01	34.31
Dry Unit Weight, g/cm3	1.46	1.49	1.56	1.49	1.38



MDD=1.56g/cm^3

OMC=24.13 %

ii) With 7 days -Curing Period

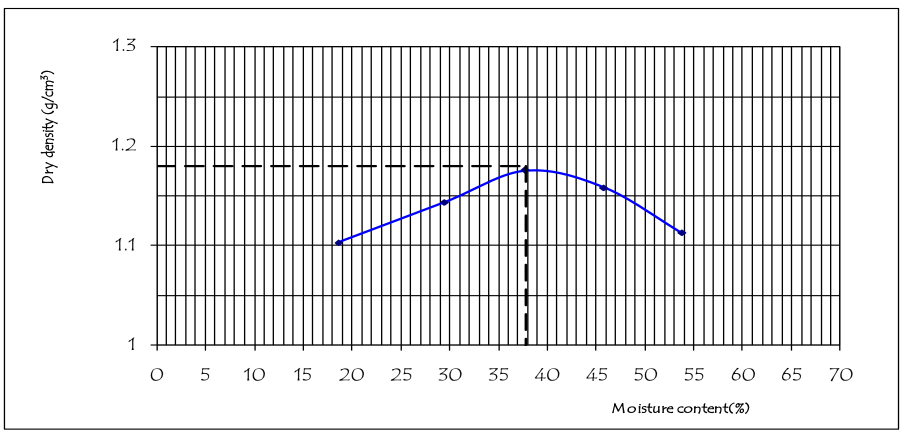
Soil Blended with 5 % CKD

Moisture content Vs dry density computation table

Determination No.	5	1	2	3	4
Mass of Mold, g	4571.1	4571.1	4571.1	4571.1	4571.1
Mass of mold + Compacted Soil, g	5807.8	5969.6	6099.9	6165.6	6188.5
Mass of Compacted soil, g	1236.7	1398.5	1528.8	1594.5	1617.4
Volume of Mold, cm3	946	944	944	944	945
Bulk density, g/cm3	1.31	1.48	1.62	1.69	1.71
Water Contenet, %	18.52	29.49	37.74	45.77	53.78
Dry density, g/cm3	1.10	1.14	1.18	1.16	1.11

Water Content Calcuation

Conatainer No	116	197	81	89	142
Mass of container, g	33.3	32.8	33	33.2	33.7
Mass of container + wet soil, g	124.8	146.1	140.3	131.3	129.2
Mass of ontainer + Dry soil, g	110.5	120.3	110.9	100.5	95.8
Mass of Water, g	14.3	25.8	29.4	30.8	33.4
Mass of Dry soil, g	77.2	87.5	77.9	67.3	62.1
Water content, %	18.52	29.49	37.74	45.77	53.78
Dry Unit Weight, g/cm3	1.10	1.14	1.18	1.16	1.11



MDD=1.18g/cm^3

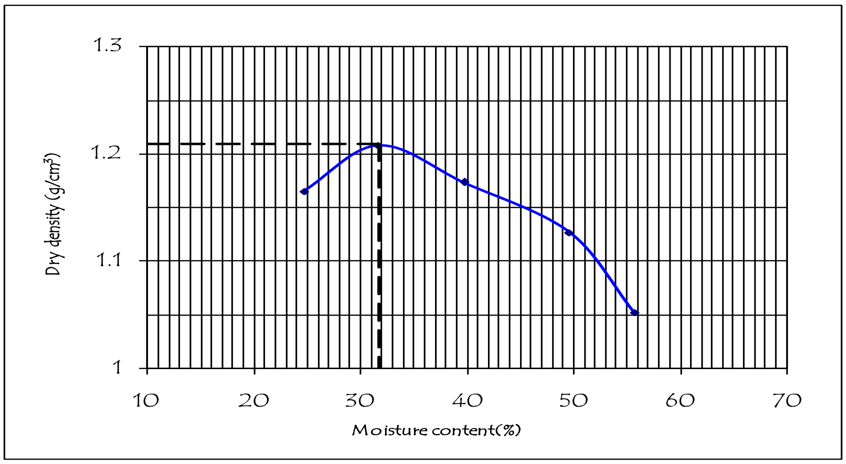
OMC=37.74 %

Soil Blended with 10 % CKD

Moisture content Vs dry density computation table

Determination No.	1	2	3	4	5
Mass of Mold, g	4571.1	4571.1	4571.1	4571.1	4571.1
Mass of mold + Compacted Soil, g	5943.2	6072.8	6119.7	6163.7	6120.6
Mass of Compacted soil, g	1372.1	1501.7	1548.6	1592.6	1549.5
Volume of Mold, cm3	944	944	944	945	946
Bulk density, g/cm3	1.45	1.59	1.64	1.69	1.64
Water Contenet, %	24.73	31.67	39.76	49.56	55.64
Dry density, g/cm3	1.17	1.21	1.17	1.13	1.05

Water Content					
Conatainer No	163	184	86	124	143
Mass of container, g	33.5	33.5	33.4	33.6	32.8
Mass of container + wet soil, g	149.5	149.5	139.9	135.3	130.7
Mass of ontainer + Dry soil, g	126.5	121.6	109.6	101.6	95.7
Mass of Water, g	23	27.9	30.3	33.7	35
Mass of Dry soil, g	93	88.1	76.2	68	62.9
Water content, %	24.73	31.67	39.76	49.56	55.64
Dry Unit Weight, g/cm3	1.17	1.21	1.17	1.13	1.05



MDD=1.21g/cm^3

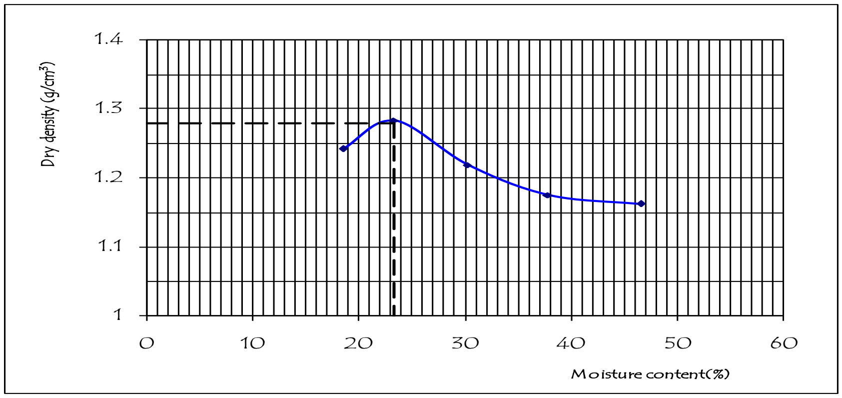
OMC=31.67 %

Soil Blended with 15 % CKD

Moisture content Vs dry density computation table

Determination No.	1	2	3	4	5
Mass of Mold, g	4569.1	4569.1	4569.1	4569.1	4569.1
Mass of mold + Compacted Soil, g	5958.6	6062.6	6067.5	6099.8	6181.2
Mass of Compacted soil, g	1389.5	1493.5	1498.4	1530.7	1612.1
Volume of Mold, cm3	944	944	944	945	946
Bulk density, g/cm3	1.47	1.58	1.59	1.62	1.70
Water Contenet, %	18.50	23.26	30.23	37.83	46.60
Dry density, g/cm3	1.24	1.28	1.22	1.18	1.16

Water Content					
Conatainer No	152	130	171	82	160
Mass of container, g	32.8	32.8	33.2	33.4	32.7
Mass of container + wet soil, g	186.5	186.5	164.6	150	157.6
Mass of ontainer + Dry soil, g	162.5	157.5	134.1	118	117.9
Mass of Water, g	24	29	30.5	32	39.7
Mass of Dry soil, g	129.7	124.7	100.9	84.6	85.2
Water content, %	18.50	23.26	30.23	37.83	46.60
Dry Unit Weight, g/cm3	1.24	1.28	1.22	1.18	1.16



MDD=1.28g/cm^3

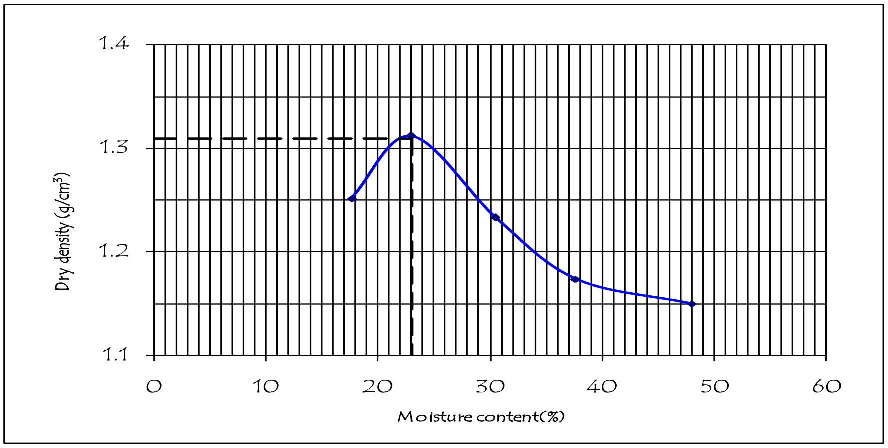
OMC=23.26 %

Soil Blended with 20 % CKD

Moisture content Vs dry density computation table

Determination No.	1	2	3	4	5
Mass of Mold, g	4568.3	4568.3	4568.3	4568.3	4568.3
Mass of mold + Compacted Soil, g	5958.6	6091.2	6087.6	6095.3	6178.9
Mass of Compacted soil, g	1390.3	1522.9	1519.3	1527	1610.6
Volume of Mold, cm3	944	944	944	945	946
Bulk density, g/cm3	1.47	1.61	1.61	1.62	1.70
Water Contenet, %	17.65	22.95	30.51	37.65	47.99
Dry density, g/cm3	1.25	1.31	1.23	1.17	1.15

Water Content					
Conatainer No	169	127	97	90	136
Mass of container, g	32.9	32.9	33.1	33.1	33.8
Mass of container + wet soil, g	172.2	172.2	178.1	136.2	144.5
Mass of ontainer + Dry soil, g	151.3	146.2	144.2	108	108.6
Mass of Water, g	20.9	26	33.9	28.2	35.9
Mass of Dry soil, g	118.4	113.3	111.1	74.9	74.8
Water content, %	17.65	22.95	30.51	37.65	47.99
Dry Unit Weight, g/cm3	1.25	1.31	1.23	1.17	1.15



MDD=1.31g/cm^3

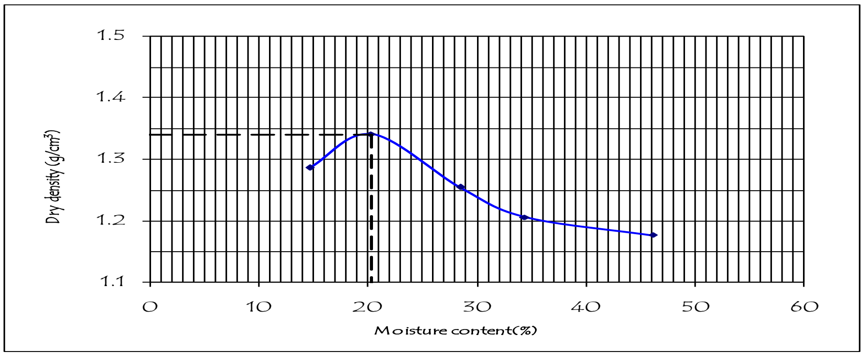
OMC=22.95 %

Soil Blended with 25 % CKD

Moisture content Vs dry density computation table

Determination No.	1	2	3	4	5
Mass of Mold, g	4567.9	4567.9	4567.9	4567.9	4567.9
Mass of mold + Compacted Soil, g	5960.3	6090.8	6089.5	6099.8	6194.6
Mass of Compacted soil, g	1392.4	1522.9	1521.6	1531.9	1626.7
Volume of Mold, cm3	944	944	944	945	946
Bulk density, g/cm3	1.48	1.61	1.61	1.62	1.72
Water Contenet, %	14.66	20.27	28.47	34.41	46.11
Dry density, g/cm3	1.29	1.34	1.25	1.21	1.18

Water Content					
Conatainer No	94	193	99	192	131
Mass of container, g	32.8	32.8	32.6	33.3	33.2
Mass of container + wet soil, g	168.1	168.1	164.8	158.3	147.9
Mass of ontainer + Dry soil, g	150.8	145.3	135.5	126.3	111.7
Mass of Water, g	17.3	22.8	29.3	32	36.2
Mass of Dry soil, g	118	112.5	102.9	93	78.5
Water content, %	14.66	20.27	28.47	34.41	46.11
Dry Unit Weight, g/cm3	1.29	1.34	1.25	1.21	1.18



MDD=1.34g/cm^3

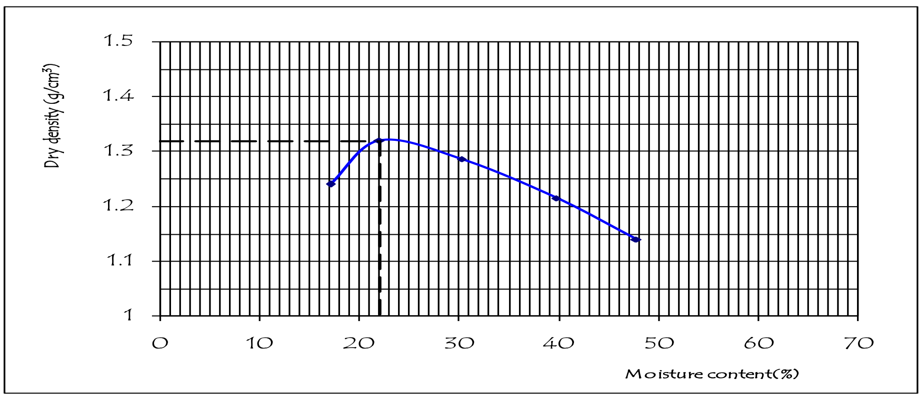
OMC=20.27 %

Soil Blended with 30 % CKD

Moisture content Vs dry density computation table

Determination No.	1	2	3	4	5
Mass of Mold, g	4567.9	4567.9	4567.9	4567.9	4567.9
Mass of mold + Compacted Soil, g	5938.6	6088.6	6150.2	6172.9	6162
Mass of Compacted soil, g	1370.7	1520.7	1582.3	1605	1594.1
Volume of Mold, cm3	944	944	944	945	946
Bulk density, g/cm3	1.45	1.61	1.68	1.70	1.69
Water Contenet, %	17.05	22.01	30.23	39.75	47.76
Dry density, g/cm3	1.24	1.32	1.29	1.22	1.14

Water Content					
Conatainer No	182	177	125	161	150
Mass of container, g	33.3	33.3	32.9	33.4	33
Mass of container + wet soil, g	165.8	165.8	158.7	154.7	128.6
Mass of ontainer + Dry soil, g	146.5	141.9	129.5	120.2	97.7
Mass of Water, g	19.3	23.9	29.2	34.5	30.9
Mass of Dry soil, g	113.2	108.6	96.6	86.8	64.7
Water content, %	17.05	22.01	30.23	39.75	47.76
Dry Unit Weight, g/cm3	1.24	1.32	1.29	1.22	1.14



MDD=1.32g/cm^3

OMC=22.01 %

iii) California Bearing Ratio Test Result

4.1 For 7 days curing soaked and un- soaked CBR test results

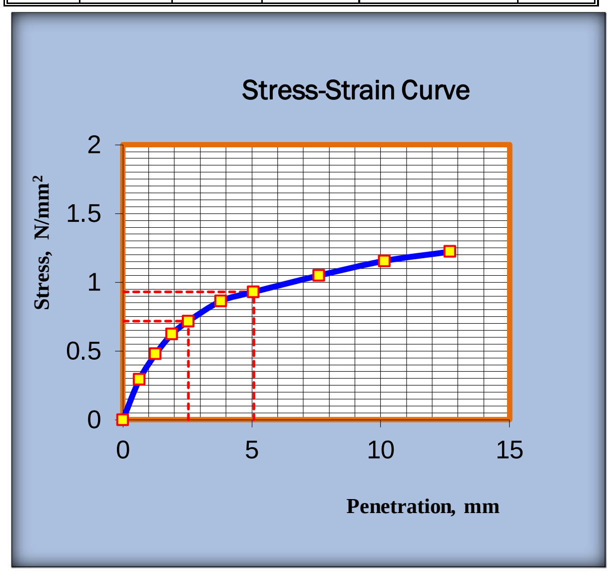
i) Soil with 10 % CKD blended

Soil Blended 10 % CKD (Unsoaked) 7 days cured and unsoaked	Ring Calibration Factor,N/Div	25.707	Swell data	
	Plunger Area, mm²	1935	Initial Reading	Final Reading
	Rate of strain ,mm/min	1.27	0	0.32
	Rammer wt. (kg)	4.54	0.275	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	0.27
CBR Value, %	6.74

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	10.00	257	0.13		
1.27	20.00	514	0.27		
1.91	28.00	720	0.37		
2.54	35.00	900	0.46	6.9	6.74
3.81	41.00	1054	0.54		
5.08	44.00	1131	0.58	10.3	5.68
7.62	47.00	1208	0.62		
10.16	50.00	1285	0.66		
12.70	52.00	1337	0.69		

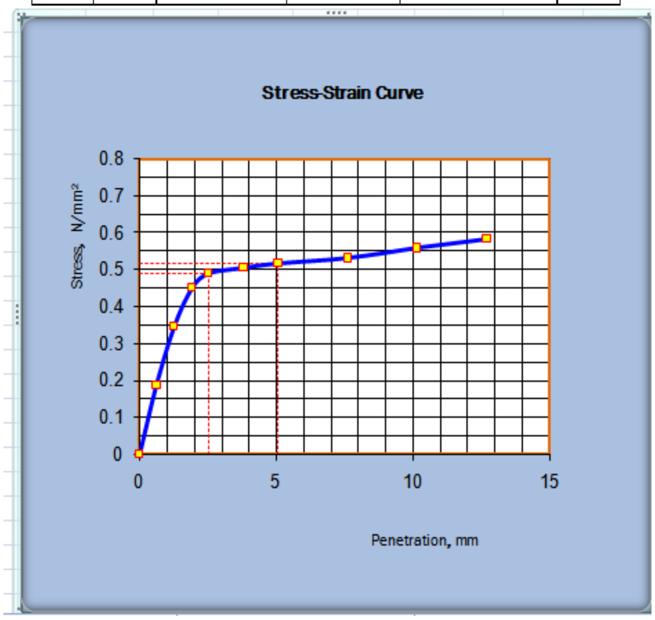


Soil Blended 10 % CKD (soaked) 7 days cured and soaked	Ring Calibration Factor,N/Div	25.707	Swell data	
	Plunger Area, mm^2	1935	Reading	Reading
	Rate of strain ,mm/min	1.27	1.23	5.45
	Rammer wt. (kg)	4.54	3.625	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	3.63
CBR Value, %	2.45

Penet. (mm)	Ring (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	5.00	129	0.07		
1.27	7.00	180	0.09		
1.91	10.00	257	0.13		
2.54	12.70	326	0.17	6.9	2.45
3.81	14.00	360	0.19		
5.08	16.00	411	0.21	10.3	2.06
7.62	18.00	463	0.24		
10.16	20.00	514	0.27		
12.70	23.00	591	0.31		



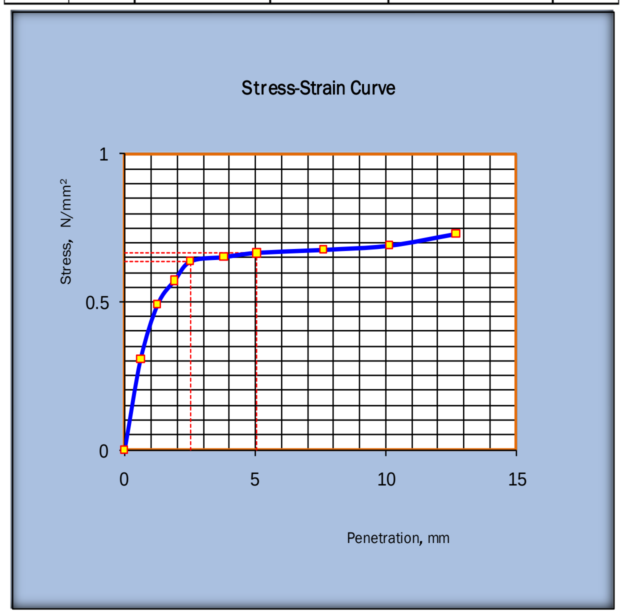
ii) Soil with 20 % CKD blended

Soil Blended 20 % CKD (soaked)	Ring Calibration Factor,N/Div	25.707	Swell data	
	Plunger Area, mm^2	1935	Reading	Reading
	Rate of strain ,mm/min	1.27	0	3.21
7 days cured and soaked	Rammer wt. (kg)	4.54	2.758	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	2.76
CBR Value, %	6.98

Penet. (mm)	Ring (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	20.00	514	0.27		
1.27	28.00	720	0.37		
1.91	33.00	848	0.44		
2.54	36.25	932	0.48	6.9	6.98
3.81	40.00	1028	0.53		
5.08	44.00	1131	0.58	10.3	5.68
7.62	50.00	1285	0.66		
10.16	58.00	1491	0.77		
12.70	64.00	1645	0.85		

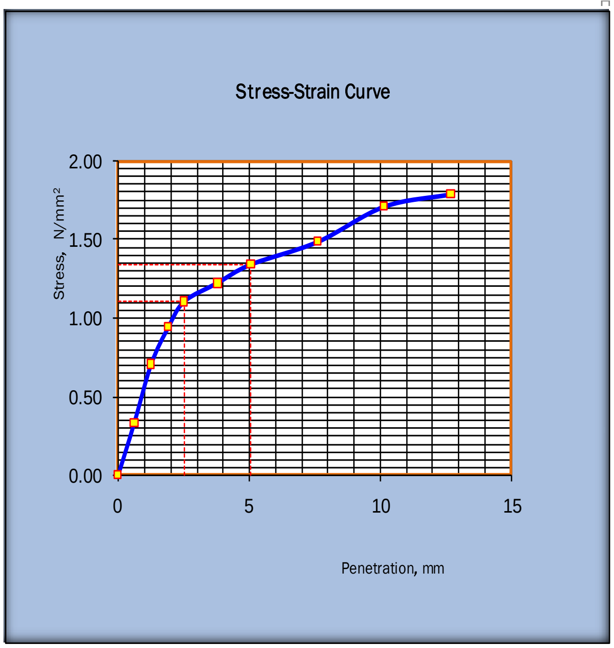


Soil Blended 20 % CKD (unsoaked) 7 days cured and unsoaked	Ring Calibration Factor,N/Div	25.707	Swell data	
	Plunger Area, mm²	1935	Initial Reading	Final Reading
	Rate of strain ,mm/min	1.27	0	0.3
	Rammer wt. (kg)	4.54	0.258	

CBR Computation Table

Blow/ Layer	**56/5**
Swell, %	**1.89**
CBR Value, %	**10.59**

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	22.00	566	0.29		
1.27	40.00	1028	0.53		
1.91	50.00	1285	0.66		
2.54	55.00	1414	0.73	6.9	10.59
3.81	59.00	1517	0.78		
5.08	62.00	1594	0.82	10.3	8.00
7.62	67.00	1722	0.89		
10.16	72.00	1851	0.96		
12.70	76.00	1954	1.01		



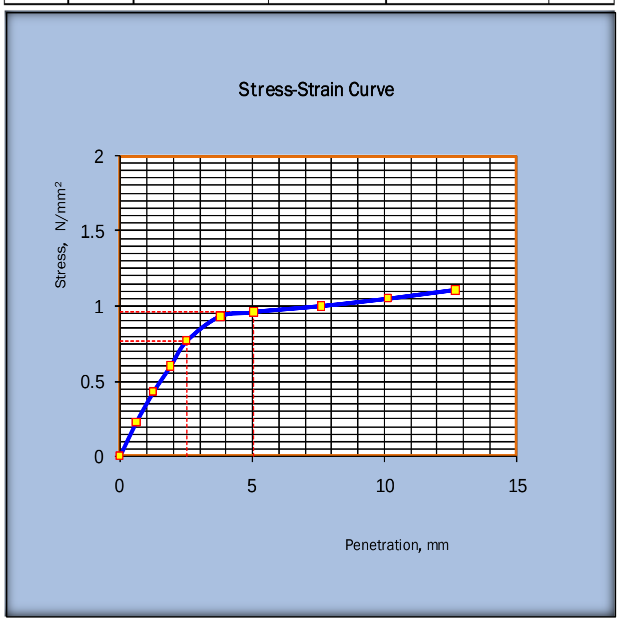
iii) Soil 30 % CKD blended

Soil Blended 30 % CKD (soaked) 7 days cured and soaked				
	Ring Calibration Factor,N/Div	25.707	Swell data	
	Plunger Area, mm^2	1935	Reading	Reading
	Rate of strain ,mm/min	1.27	0.04	2.88
	Rammer wt. (kg)	4.54	2.440	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	2.44
CBR Value, %	10.97

Penet. (mm)	Ring (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	20.00	514	0.27		
1.27	35.00	900	0.46		
1.91	45.00	1157	0.60		
2.54	57.00	1465	0.76	6.9	10.97
3.81	65.00	1671	0.86		
5.08	70.00	1799	0.93	10.3	9.03
7.62	80.00	2057	1.06		
10.16	90.00	2314	1.20		
12.70	100.00	2571	1.33		

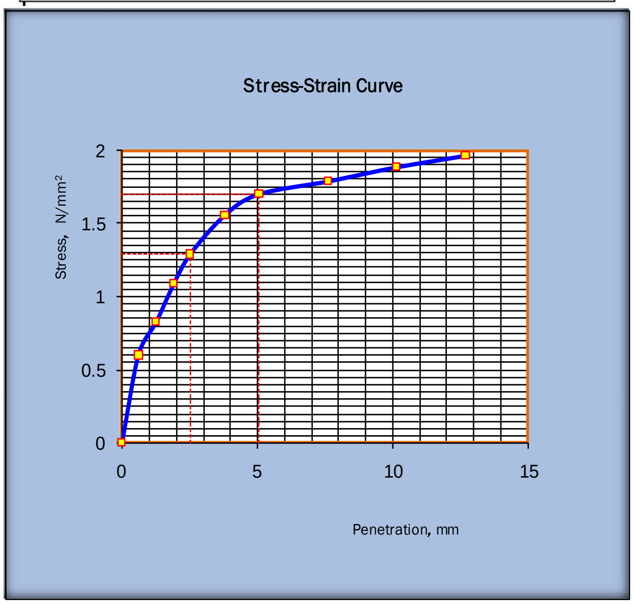


Soil Blended 30 % CKD (unsoaked) 7 days cured and unsoaked	Ring Calibration Factor,N/Div	25.707	Swell data	
			Initial Reading	Final Reading
	Plunger Area, mm²	1964		
	Rate of strain ,mm/min	1.3	0.12	0.33
	Rammer wt. (kg)	4.54	0.180	

CBR Computation Table

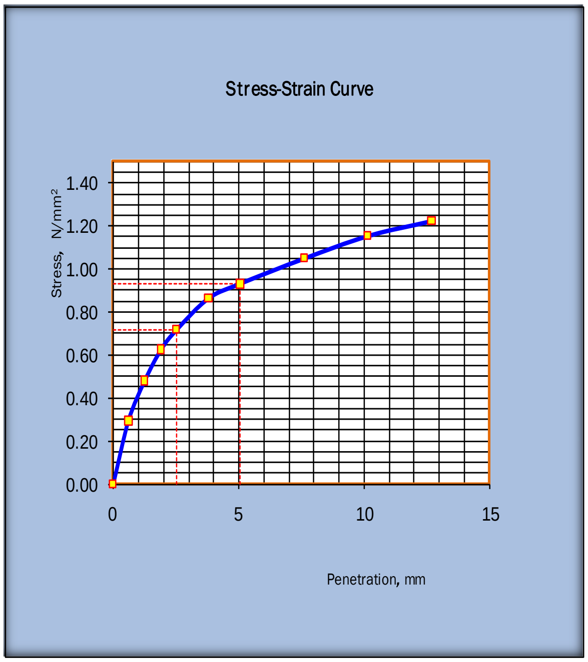
Blow/ Layer	56/5
Swell, %	0.18
CBR Value, %	13.09

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm²)	Standard stress (N/mm²)	CBR (%)
0.00	0.00	0	0.00		
0.64	21.00	540	0.27		
1.27	30.00	771	0.39		
1.91	52.00	1337	0.68		
2.54	69.00	1774	0.90	6.9	13.09
3.81	80.00	2057	1.05		
5.08	94.00	2416	1.23	10.3	11.95
7.62	110.00	2828	1.44		
10.16	115.00	2956	1.51		
12.70	118.00	3033	1.54		



4.2 For 14 days curing soaked and un- soaked CBR test results

i) Soil 10 % CKD blended



Soil Blended with 10 % CKD 14 days cured and unsoaked	Ring Calibration Factor,N/Div	25.707	Swell data	
			Initial Reading	Final Reading
	Plunger Area, mm²	1935		
	Rate of strain ,mm/min	1.27	0.03	0.12
	Rammer wt. (kg)	4.54	0.077	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	0.08
CBR Value, %	10.40

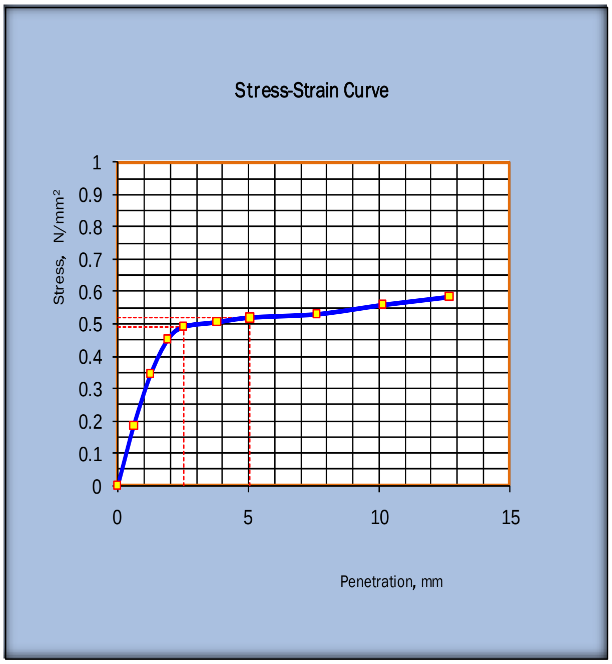
Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm²)	Standard stress (N/mm²)	CBR (%)
0.00	0.00	0	0.00		
0.64	22.00	566	0.29		
1.27	36.00	925	0.48		
1.91	47.00	1208	0.62		
2.54	54.00	1388	0.72	6.9	10.40
3.81	65.00	1671	0.86		
5.08	70.00	1799	0.93	10.3	9.03
7.62	79.00	2031	1.05		
10.16	87.00	2237	1.16		
12.70	92.00	2365	1.22		

Soil Blended with 10 % CKD unsoaked 14 days cured and unsoaked	Ring Calibration Factor,N/Div	25.707	Swell data	
	Plunger Area, mm²	1935	Initial Reading	Final Reading
	Rate of strain ,mm/min	1.27	0.03	2.47
	Rammer wt. (kg)	4.54	2.096	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	2.096
CBR Value, %	7.12

Penet. (mm)	Ring (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	14.00	360	0.19		
1.27	26.00	668	0.35		
1.91	34.00	874	0.45		
2.54	37.00	951	0.49	6.9	7.12
3.81	38.00	977	0.50		
5.08	39.00	1003	0.52	10.3	5.03
7.62	40.00	1028	0.53		
10.16	42.00	1080	0.56		
12.70	44.00	1131	0.58		



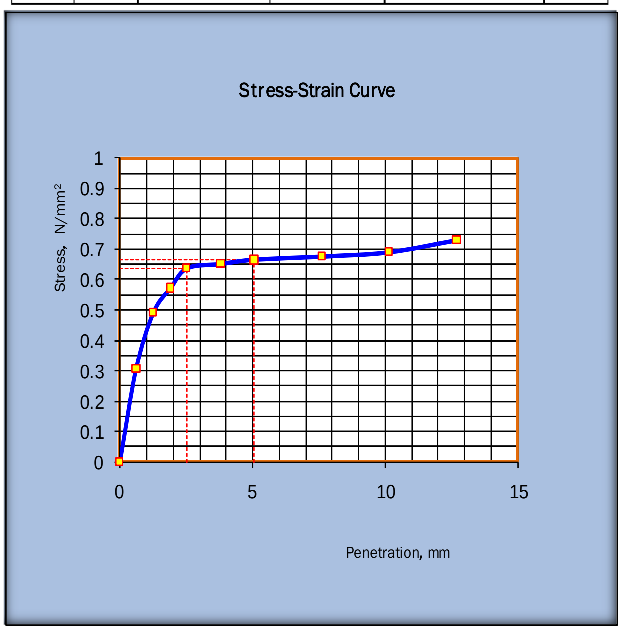
ii) Soil 20 % CKD blended

20 % CKD (Soaked) 14 days cured and 4 days soaked	Ring Calibration Factor,N/Div	25.707	Swell data	
			Initial Reading	Final Reading
	Plunger Area, mm^2	1935		
	Rate of strain ,mm/min	1.27	4.33	6.61
	Rammer wt. (kg)	4.54	1.959	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	1.96
CBR Value, %	9.24

Penet. (mm)	Ring (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	23.00	591	0.31		
1.27	37.00	951	0.49		
1.91	43.00	1105	0.57		
2.54	48.00	1234	0.64	6.9	9.24
3.81	49.00	1260	0.65		
5.08	50.00	1285	0.66	10.3	6.45
7.62	51.00	1311	0.68		
10.16	52.00	1337	0.69		
12.70	55.00	1414	0.73		

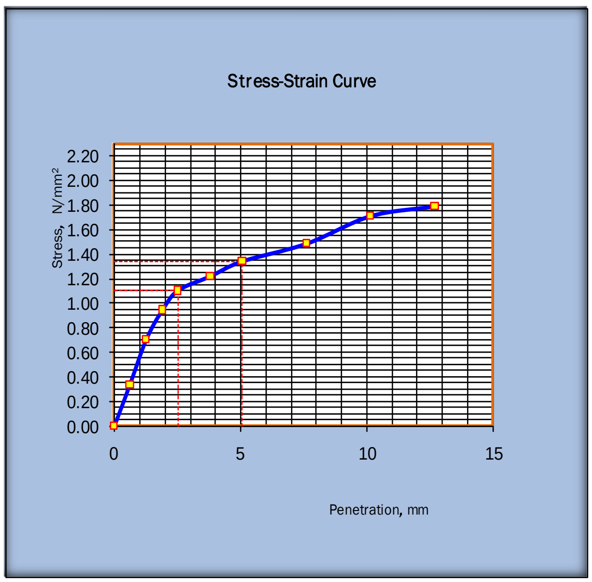


20 % CKD(unsoaked) 14 days cured & unsoaked	Ring Calibration Factor,N/Div	25.707	Swell data	
	Plunger Area, mm²	1935	Initial Reading	Final Reading
	Rate of strain ,mm/min	1.27	0.08	0.14
	Rammer wt. (kg)	4.54	0.052	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	0.01
CBR Value, %	**15.98**

Penet. (mm)	Ring Reading (Div.)	Load (N)	Stress (N/mm²)	Standard stress (N/mm²)	CBR (%)
0.00	0.00	0	0.00		
0.64	25.00	643	0.33		
1.27	53.00	1362	0.70		
1.91	71.00	1825	0.94		
2.54	83.00	2134	1.10	6.9	15.98
3.81	92.00	2365	1.22		
5.08	101.00	2596	1.34	10.3	13.03
7.62	112.00	2879	1.49		
10.16	129.00	3316	1.71		
12.70	135.00	3470	1.79		



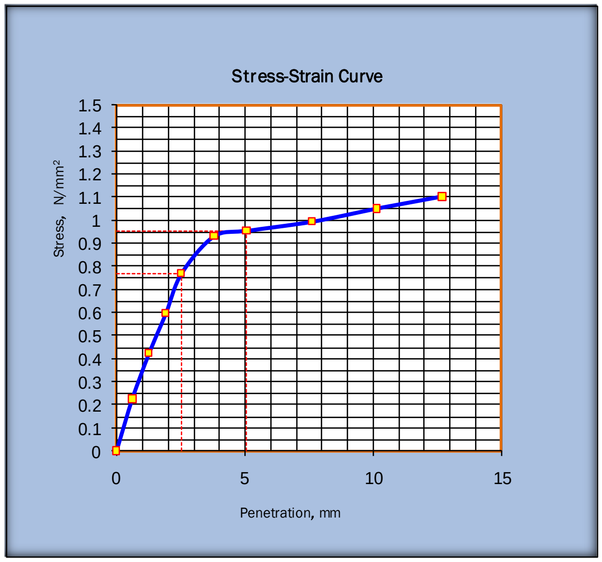
iii) Soil 30 % CKD blended

30 % CKD(soaked) 14 days cured & 4 days soaked	Ring Calibration Factor, N/Div	25.707	Swell data	
	Plunger Area, mm^2	1935	Initial Reading	Final Reading
	Rate of strain ,mm/min	1.27	0.08	1.89
	Rammer wt. (kg)	4.54	1.555	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	1.55
CBR Value, %	11.17

Penet. (mm)	Ring (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	17.00	437	0.23		
1.27	32.00	823	0.43		
1.91	45.00	1157	0.60		
2.54	58.00	1491	0.77	6.9	11.17
3.81	70.00	1799	0.93		
5.08	72.00	1851	0.96	10.3	9.29
7.62	75.00	1928	1.00		
10.16	79.00	2031	1.05		
12.70	83.00	2134	1.10		

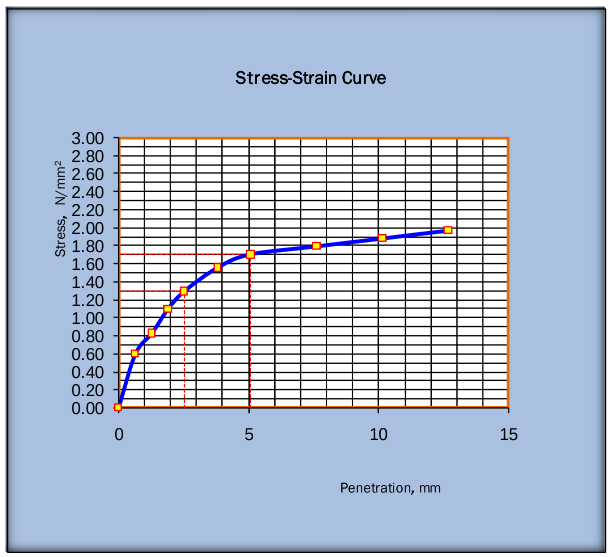


30 % CKD(un soaked) 14 days cured & un soaked	Ring Calibration Factor, N/Div	25.707	Swell data	
	Plunger Area, mm^2	1935	Initial Reading	Final Reading
	Rate of strain ,mm/min	1.27	0.44	1.28
	Rammer wt. (kg)	4.54	0.722	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	0.72
CBR Value, %	18.68

Penet. (mm)	Ring (Div.)	Load (N)	Stress (N/mm^2)	Standard load	CBR (%)
0.00	0.00	0	0.00		
0.64	45.00	1157	0.60		
1.27	62.00	1594	0.82		
1.91	82.00	2108	1.09		
2.54	97.00	2494	1.29	6.9	18.68
3.81	117.00	3008	1.55		
5.08	128.00	3290	1.70	10.3	16.51
7.62	135.00	3470	1.79		
10.16	142.00	3650	1.89		
12.70	148.00	3805	1.97		



4.3 For 28 days curing soaked and Unsoaked CBR test results

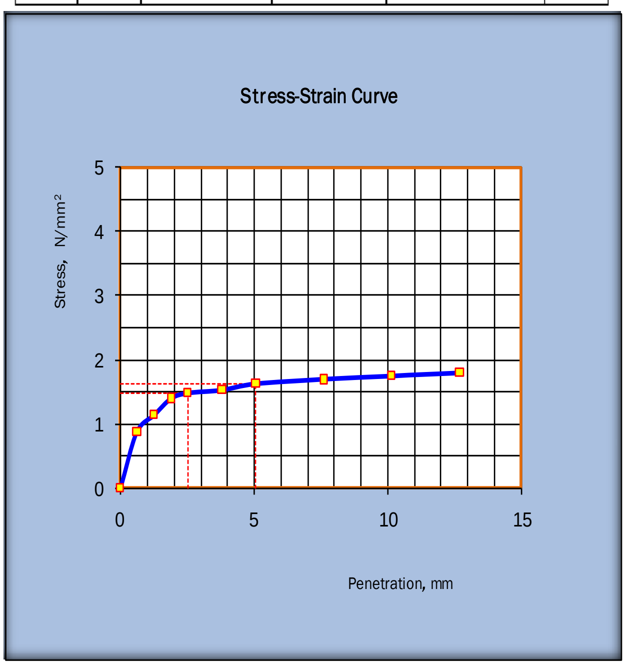
i) Soil 10% CKD Blended

Soil Blended 10 % CKD (unsoaked) 28 days cured and unsoaked	Ring Calibration Factor,N/Div	25.707	Swell data	
	Plunger Area, mm^2	1935	Reading	Reading
	Rate of strain ,mm/min	1.27	3.46	4.56
	Rammer wt. (kg)	4.54	0.945	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	0.95
CBR Value, %	21.56

Penet. (mm)	Ring (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	66.00	1697	0.88		
1.27	86.00	2211	1.14		
1.91	105.00	2699	1.39		
2.54	112.00	2879	1.49	6.9	21.56
3.81	115.00	2956	1.53		
5.08	122.00	3136	1.62	10.3	15.74
7.62	127.00	3265	1.69		
10.16	132.00	3393	1.75		
12.70	135.00	3470	1.79		

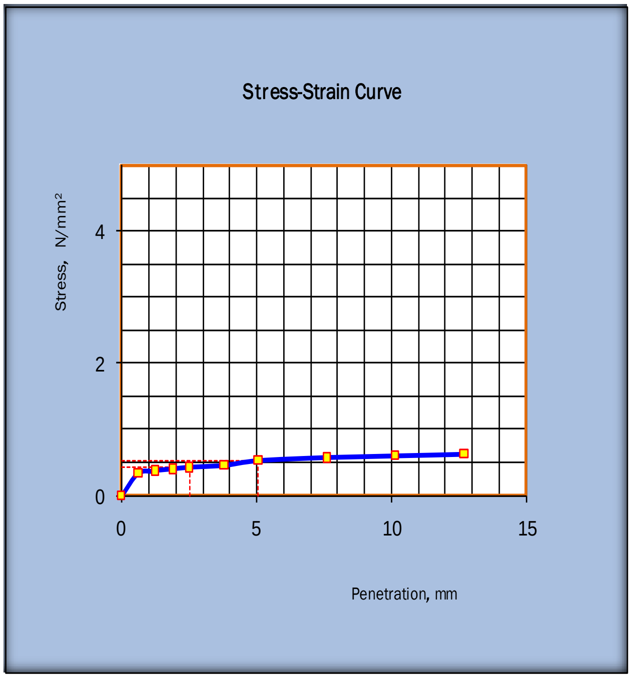


Soil Blended 10 % CKD (soaked) 28 days cured and soaked	Ring Calibration Factor,N/Div	25.707	Swell data	
	Plunger Area, mm^2	1935	Reading	Reading
	Rate of strain ,mm/min	1.27	2.03	3.45
	Rammer wt. (kg)	4.54	1.220	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	1.22
CBR Value, %	6.16

Penet. (mm)	Ring (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	26.00	668	0.35		
1.27	28.00	720	0.37		
1.91	30.00	771	0.40		
2.54	32.00	823	0.43	6.9	6.16
3.81	35.00	900	0.46		
5.08	40.00	1028	0.53	10.3	5.16
7.62	43.00	1105	0.57		
10.16	45.00	1157	0.60		
12.70	48.00	1234	0.64		



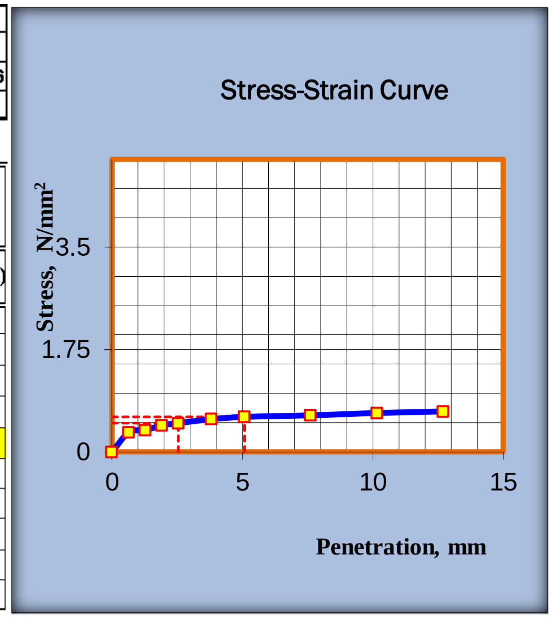
ii) Soil 20% CKD Blended

Soil Blended 20 % CKD (soaked) 28 days cured and soaked	Ring Calibration Factor,N/Div	25.707	Swell data	
	Plunger Area, mm^2	1935	Reading	Reading
	Rate of strain ,mm/min	1.27	3.67	4.56
	Rammer wt. (kg)	4.54	0.765	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	0.76
CBR Value, %	7.12

Penet. (mm)	Ring (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	25.00	643	0.33		
1.27	28.00	720	0.37		
1.91	34.00	874	0.45		
2.54	37.00	951	0.49	6.9	7.12
3.81	42.00	1080	0.56		
5.08	45.00	1157	0.60	10.3	5.80
7.62	47.00	1208	0.62		
10.16	50.00	1285	0.66		
12.70	52.00	1337	0.69		

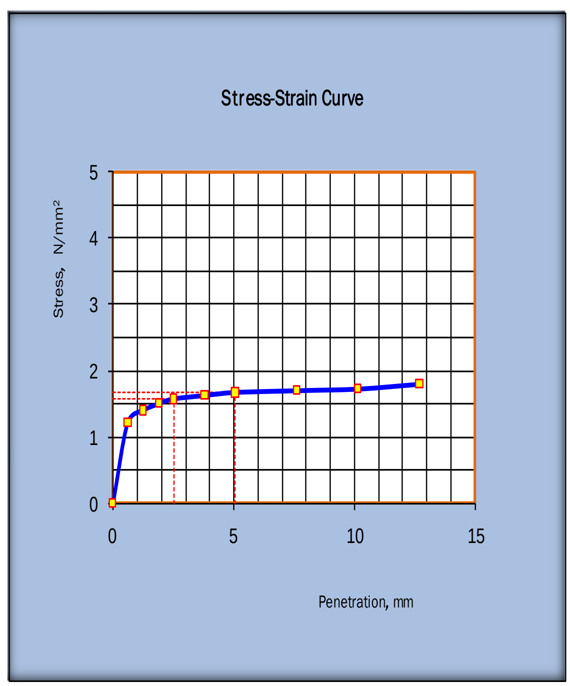


Soil Blended 20 % CKD (unsoaked) 28 days cured and unsoaked	Ring Calibration Factor,N/Div	25.707	Swell data	
	Plunger Area, mm^2	1935	Reading	Reading
	Rate of strain ,mm/min	1.27	3.67	4.26
	Rammer wt. (kg)	4.54	0.507	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	0.51
CBR Value, %	22.72

Penet. (mm)	Ring (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	91.00	2339	1.21		
1.27	105.00	2699	1.39		
1.91	113.00	2905	1.50		
2.54	118.00	3033	1.57	6.9	22.72
3.81	122.00	3136	1.62		
5.08	125.00	3213	1.66	10.3	16.12
7.62	128.00	3290	1.70		
10.16	130.00	3342	1.73		
12.70	135.00	3470	1.79		



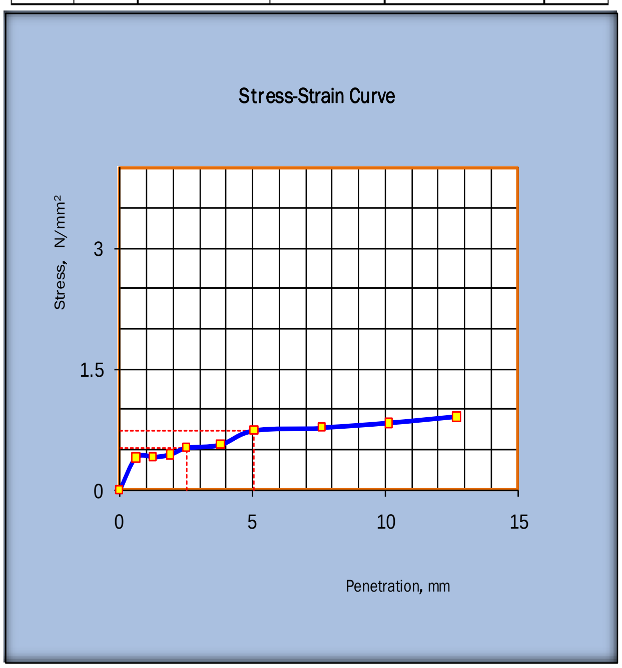
iii) Soil 30% CKD Blended

Soil Blended 30 % CKD (Unsoaked) 28 days cured and unsoaked	Ring Calibration Factor,N/Div	25.707	Swell data	
	Plunger Area, mm^2	1935	Reading	Reading
	Rate of strain ,mm/min	1.27	2.97	3.45
	Rammer wt. (kg)	4.54	0.412	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	0.41
CBR Value, %	25.99

Penet. (mm)	Ring (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	79.00	2031	1.05		
1.27	105.00	2699	1.39		
1.91	125.00	3213	1.66		
2.54	135.00	3470	1.79	6.9	25.99
3.81	139.00	3573	1.85		
5.08	142.00	3650	1.89	10.3	18.32
7.62	145.00	3728	1.93		
10.16	148.00	3805	1.97		
12.70	150.00	3856	1.99		

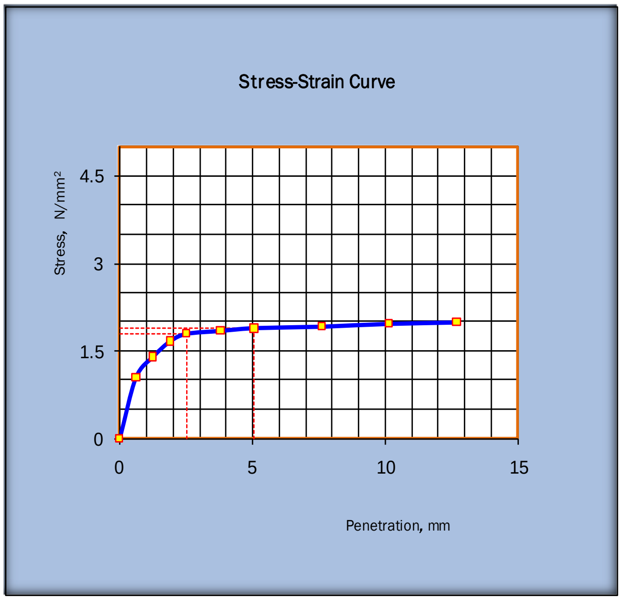


Soil Blended 30 % CKD (Unsoaked) 28 days cured and unsoaked	Ring Calibration Factor,N/Div	25.707	Swell data	
	Plunger Area, mm^2	1935	Reading	Reading
	Rate of strain ,mm/min	1.27	2.97	3.45
	Rammer wt. (kg)	4.54	0.412	

CBR Computation Table

Blow/ Layer	56/5
Swell, %	0.41
CBR Value, %	25.99

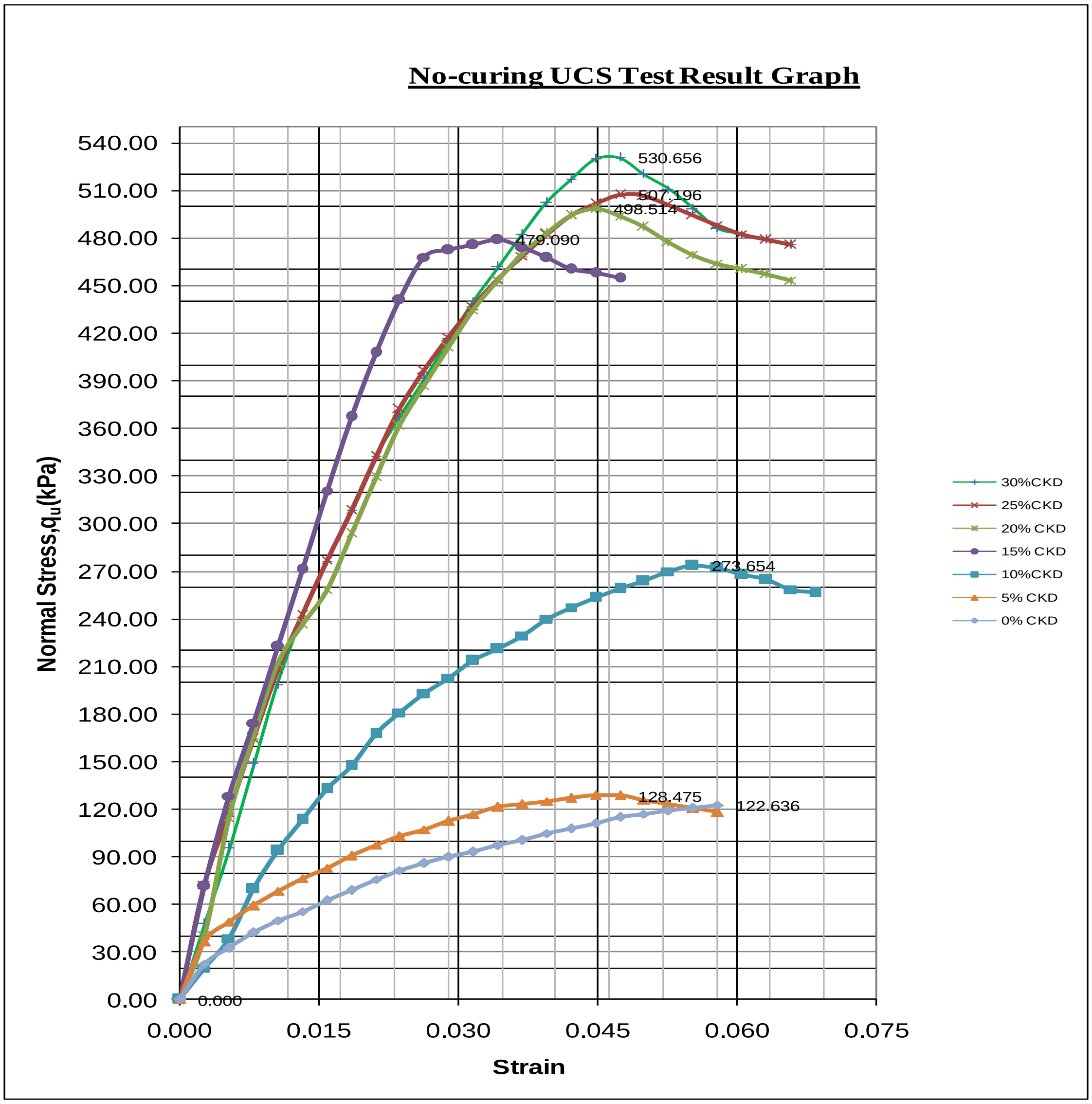
Penet. (mm)	Ring (Div.)	Load (N)	Stress (N/mm^2)	Standard stress (N/mm^2)	CBR (%)
0.00	0.00	0	0.00		
0.64	79.00	2031	1.05		
1.27	105.00	2699	1.39		
1.91	125.00	3213	1.66		
2.54	135.00	3470	1.79	6.9	25.99
3.81	139.00	3573	1.85		
5.08	142.00	3650	1.89	10.3	18.32
7.62	145.00	3728	1.93		
10.16	148.00	3805	1.97		
12.70	150.00	3856	1.99		



VI) Unconfined Compressive Strength Test Result

No Curing UCS Test Result Summary Table

serious-7		serious-6		serious-5		serious-4		serious-3		serious-2		serious-1	
0% CKD		**5% CKD**		**10%CKD**		**15% CKD**		**20% CKD**		**25%CKD**		**30%CKD**	
Unit strain (ΔL/Lo)	Sample stress (KPa)	**Unit strain (ΔL/Lo)**	**Sample stress (KPa)**	Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.003	22.223	0.003	36.259	0.003	19.884	0.003	71.348	0.003	39.768	0.003	72.517	0.003	47.955
0.005	32.663	0.005	48.995	0.005	38.496	0.005	128.320	0.005	114.322	0.005	117.821	0.005	95.657
0.008	41.885	0.008	59.337	0.008	69.808	0.008	174.519	0.008	166.375	0.008	164.048	0.008	148.923
0.011	49.896	0.011	68.462	0.011	93.990	0.011	222.792	0.011	212.349	0.011	207.707	0.011	198.424
0.013	55.550	0.013	76.381	0.013	113.414	0.013	270.806	0.013	236.087	0.013	243.031	0.013	240.716
0.016	62.327	0.016	83.103	0.016	132.733	0.016	320.869	0.016	258.542	0.016	277.009	0.016	275.855
0.018	69.067	0.018	90.938	0.018	147.343	0.018	367.207	0.018	293.535	0.018	308.500	0.018	308.500
0.021	75.770	0.021	97.583	0.021	167.613	0.021	407.551	0.021	329.485	0.021	342.113	0.021	342.113
0.024	81.291	0.024	103.045	0.024	180.901	0.024	440.804	0.024	361.803	0.024	372.107	0.024	367.528
0.026	85.639	0.026	107.335	0.026	192.974	0.026	468.162	0.026	387.090	0.026	396.225	0.026	391.658
0.029	89.963	0.029	112.739	0.029	202.702	0.029	472.591	0.029	411.097	0.029	417.930	0.029	416.791
0.032	93.126	0.032	116.976	0.032	213.509	0.032	475.853	0.032	434.968	0.032	437.240	0.032	440.647
0.034	97.404	0.034	121.188	0.034	220.857	**0.034**	**479.090**	0.034	453.040	0.034	454.173	0.034	462.101
0.037	100.527	0.037	123.117	0.037	229.292	0.037	474.396	0.037	469.878	0.037	468.749	0.037	482.303
0.039	104.758	0.039	125.034	0.039	239.929	0.039	467.468	0.039	483.238	0.039	482.112	0.039	502.387
0.042	107.841	0.042	126.938	0.042	247.135	0.042	460.571	0.042	494.271	0.042	494.271	0.042	516.738
0.045	110.905	**0.045**	**128.830**	0.045	253.178	0.045	458.185	**0.045**	**498.514**	0.045	501.875	0.045	529.881
0.047	115.069	0.047	128.475	0.047	259.184	0.047	454.688	0.047	493.789	**0.047**	**507.196**	**0.047**	**530.656**
0.050	116.979	0.050	125.892	0.050	264.038			0.050	486.855	0.050	506.908	0.050	520.278
0.053	118.877	0.053	123.321	0.053	269.973			0.053	477.729	0.053	501.060	0.053	511.059
0.055	120.762	0.055	120.762	**0.055**	**273.654**			0.055	469.755	0.055	494.129	0.055	498.561
0.058	**122.636**	0.058	118.216	0.058	271.787			0.058	464.027	0.058	488.333	0.058	486.124
0.061	122.293			0.061	267.723			0.061	460.527	0.061	482.562	0.061	482.562
0.063	120.852			0.063	264.776			0.063	457.040	0.063	479.013	0.063	479.013
0.066	119.417			0.066	258.554			0.066	453.565	0.066	475.476	0.066	475.476
0.068	117.988			0.068	256.733								

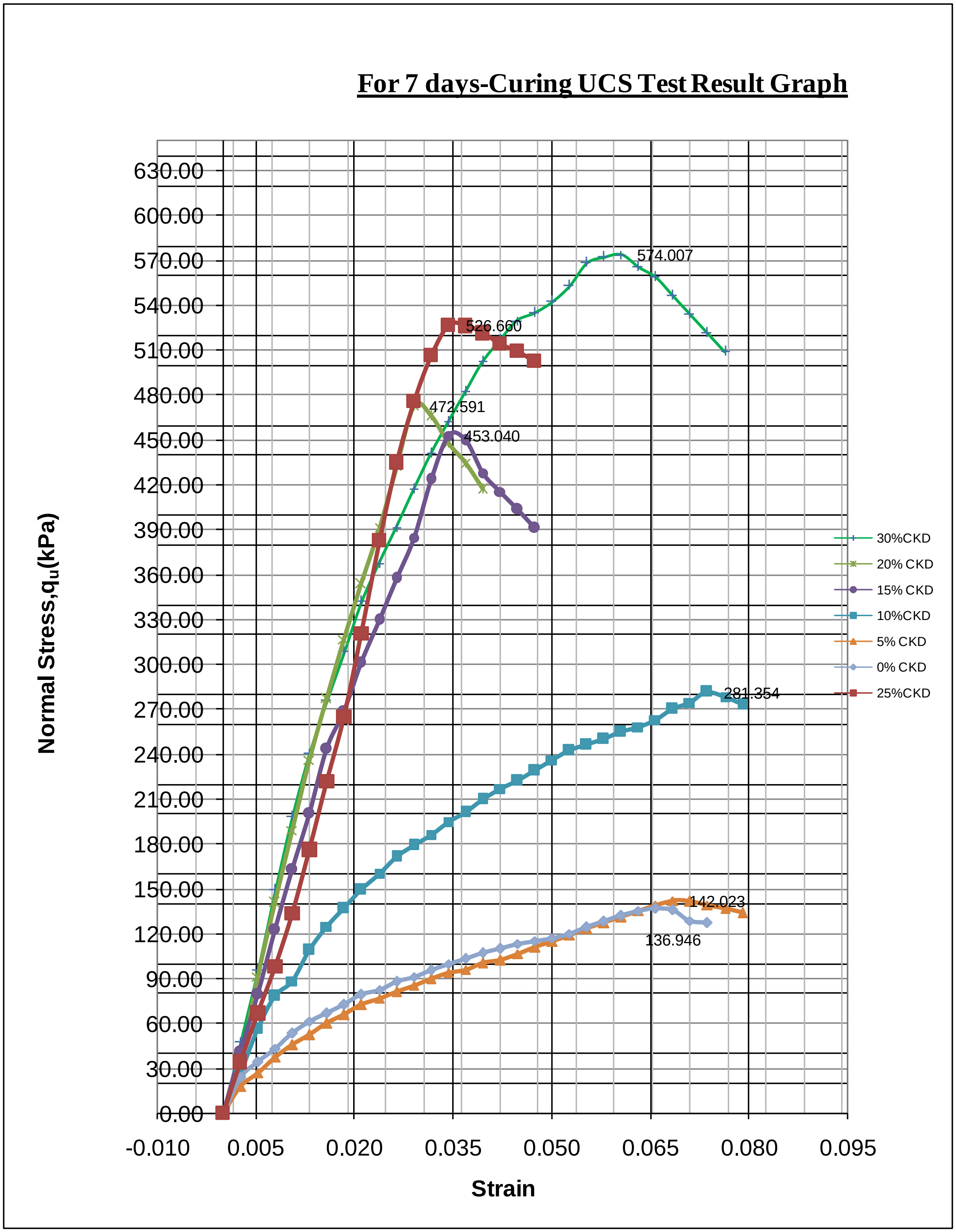


No-curing UCS Test Result Graph
Normal Stress,q_u(kPa)
540.00
510.00
480.00
450.00
420.00
390.00
360.00
330.00
300.00
270.00
240.00
210.00
180.00
150.00
120.00
90.00
60.00
30.00
0.00
0.000
0.015
0.030
0.045
0.060
0.075
Strain
530.656
507.196
498.514
479.090
273.654
128.475
122.636
0.000
30%CKD
25%CKD
20% CKD
15% CKD
10%CKD
5% CKD
0% CKD

strain

For 7 days- Curing UCS Test Result Summary Table

serious-7		serious-6		serious-5		serious-4		serious-3		serious-2		serious-1	
0% CKD		**5% CKD**		**10%CKD**		**15% CKD**		**20% CKD**		**25%CKD**		**30%CKD**	
Unit strain (ΔL/Lo)	Sample stress (KPa)	**Unit strain (ΔL/Lo)**	**Sample stress (KPa)**	Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.003	23.393	0.003	17.545	0.003	26.902	0.003	40.937	0.003	32.750	0.003	33.919	0.003	47.955
0.005	33.830	0.005	26.831	0.005	55.994	0.005	79.325	0.005	90.991	0.005	66.493	0.005	95.657
0.008	43.048	0.008	37.231	0.008	77.952	0.008	122.164	0.008	140.779	0.008	97.731	0.008	148.923
0.011	53.377	0.011	45.255	0.011	88.189	0.011	162.453	0.011	189.141	0.011	133.443	0.011	198.424
0.013	61.336	0.013	52.078	0.013	108.785	0.013	200.211	0.013	236.087	0.013	175.908	0.013	240.716
0.016	66.944	0.016	60.019	0.016	124.654	0.016	243.537	0.016	277.009	0.016	221.607	0.016	275.855
0.018	72.520	0.018	65.614	0.018	136.983	0.018	268.210	0.018	316.557	0.018	264.757	0.018	308.500
0.021	79.214	0.021	72.326	0.021	149.244	0.021	300.784	0.021	353.594	0.021	320.301	0.021	342.113
0.024	82.436	0.024	76.711	0.024	160.292	0.024	329.744	0.024	391.571	0.024	382.412	0.024	367.528
0.026	87.923	0.026	81.072	0.026	171.279	0.026	357.402	0.026	432.765	0.026	435.048	0.026	391.658
0.029	91.102	0.029	85.408	0.029	178.787	0.029	384.905	0.029	472.591	0.029	476.007	0.029	416.791
0.032	95.398	0.032	89.719	0.032	186.253	0.032	423.611	0.032	465.632	0.032	506.517	0.032	440.647
0.034	99.669	0.034	94.006	0.034	194.807	0.034	453.040	0.034	447.377	0.034	526.660	0.034	462.101
0.037	102.786	0.037	96.009	0.037	201.054	0.037	449.547	0.037	434.863	0.037	526.354	0.037	482.303
0.039	107.011	0.039	100.252	0.039	209.516	0.039	428.043	0.039	416.779	0.039	521.537	0.039	502.387
0.042	110.088	0.042	102.224	0.042	216.805	0.042	415.637			0.042	514.491	0.042	516.738
0.045	113.146	0.045	106.424	0.045	221.811	0.045	403.292			0.045	509.717	0.045	529.881
0.047	115.069	0.047	110.600	0.047	229.020	0.047	391.010			0.047	502.727	0.047	535.125
0.050	116.979	0.050	114.751	0.050	236.186							0.050	542.559
0.053	119.988	0.053	118.877	0.053	242.198							0.053	553.277
0.055	124.086	0.055	122.978	0.055	245.957							0.055	568.359
0.058	128.160	0.058	127.055	0.058	249.691							0.058	572.300
0.061	132.209	0.061	131.107	0.061	254.502							0.061	574.007
0.063	135.134	0.063	135.134	0.063	258.184							0.063	565.807
0.066	136.946	0.066	139.137	0.066	262.936							0.066	558.740
0.068	135.468	0.068	142.023	0.068	269.843							0.068	546.241
0.071	128.549	0.071	141.621	0.071	274.528							0.071053	533.8039
0.074	127.098	0.074	139.048	0.074	281.354							0.073684	521.4286
		0.076	136.486	0.076	278.388							0.076316	509.1151
		0.079	133.937	0.079	273.275								

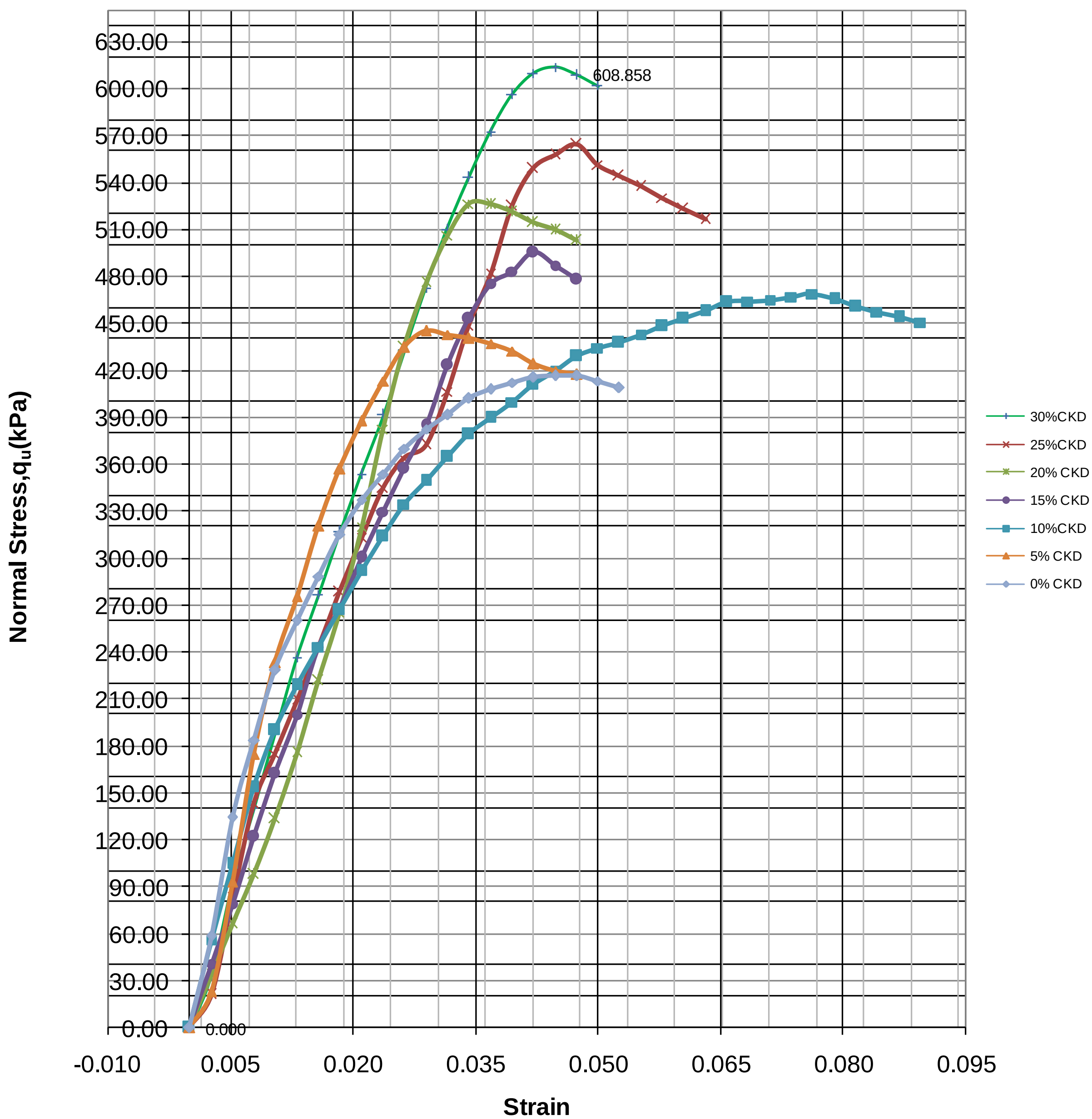


For 7 days-Curing UCS Test Result Graph
Normal Stress,qu(kPa)
Strain
630.00
600.00
570.00
540.00
510.00
480.00
450.00
420.00
390.00
360.00
330.00
300.00
270.00
240.00
210.00
180.00
150.00
120.00
90.00
60.00
30.00
0.00
-0.010
0.005
0.020
0.035
0.050
0.065
0.080
0.095
574.007
526.660
472.591
453.040
281.354
142.023
136.946
30%CKD
20% CKD
15% CKD
10%CKD
5% CKD
0% CKD
25%CKD

For 14 days- Curing UCS Test Result Summary Table

serious-7		serious-6		serious-5		serious-4		serious-3		serious-2		serious-1	
0% CKD		5% CKD		10%CKD		15% CKD		20% CKD		25%CKD		30%CKD	
Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)	Unit strain (ΔL/Lo)	Sample stress (KPa)
0.0000	0.000	0.000	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.0026	58.482	0.003	23.39	0.003	56.142	0.003	40.937	0.003	33.919	0.003	21.053	0.003	32.750
0.0053	134.153	0.005	93.32	0.005	104.989	0.005	79.325	0.005	66.493	0.005	82.825	0.005	90.991
0.0079	183.827	0.008	174.52	0.008	153.577	0.008	122.164	0.008	97.731	0.008	143.106	0.008	140.779
0.0105	228.594	0.011	233.24	0.011	190.302	0.011	162.453	0.011	133.443	0.011	175.217	0.011	189.141
0.0132	260.390	0.013	275.44	0.013	218.728	0.013	200.211	0.013	175.908	0.013	209.470	0.013	236.087
0.0158	288.551	0.016	320.87	0.016	243.537	0.016	243.537	0.016	221.607	0.016	243.537	0.016	277.009
0.0184	315.406	0.018	356.85	0.018	267.059	0.018	268.210	0.018	264.757	0.018	278.571	0.018	316.557
0.0211	337.521	0.021	388.03	0.021	291.600	0.021	300.784	0.021	320.301	0.021	313.413	0.021	353.594
0.0237	353.788	0.024	413.33	0.024	313.715	0.024	329.744	0.024	382.412	0.024	344.629	0.024	391.571
0.0263	369.962	0.026	435.05	0.026	334.565	0.026	357.402	0.026	435.048	0.026	364.253	0.026	432.765
0.0289	382.628	0.029	445.26	0.029	349.603	0.029	384.905	0.029	476.007	0.029	372.379	0.029	472.591
0.0316	391.812	0.032	442.92	0.032	364.556	0.032	423.611	0.032	506.517	0.032	406.576	0.032	509.924
0.0342	402.073	0.034	440.58	0.034	379.421	0.034	453.040	0.034	526.660	0.034	448.510	0.034	543.649
0.0368	407.755	0.037	437.12	0.037	389.683	0.037	475.526	0.037	526.354	0.037	482.303	0.037	572.664
0.0395	412.273	0.039	432.55	0.039	399.882	0.039	483.238	0.039	521.537	0.039	524.916	0.039	595.881
0.0421	415.637	0.042	424.62	0.042	411.144	0.042	495.394	0.042	514.491	0.042	549.315	0.042	609.975
0.0447	416.736	0.045	420.10	0.045	420.096	0.045	487.312	0.045	509.717	0.045	557.888	0.045	613.901
0.0474	416.705	0.047	417.82	0.047	428.994	0.047	478.149	0.047	502.727	0.047	564.171	0.047	608.858
0.0500	413.325			0.050	434.493					0.050	551.472	0.050	601.606
0.0526	408.847			0.053	437.733					0.053	544.389		
				0.055	443.165					0.055	537.338		
				0.058	448.559					0.058	530.317		
				0.061	452.815					0.061	523.327		
				0.063	458.139					0.063	516.367		
				0.066	463.425								
				0.068	464.305								
				0.0711	465.1720								
				0.0737	467.1131								
				0.0763	469.0358								
				0.0789	465.5392								
				0.0816	460.9780								
				0.0842	457.5092								
				0.0868	454.0527								
				0.0895	450.6086								

For 14 th days -Curing UCS Test Result Graph



VII) Linear Shrinkage Test Result

Natural Soil Linear Shrinkage Test Result

Determination No	1
Semi cylinderical trough No	A
initial wet length of soil L_0 (mm)	140
Dry length of soil L_d (mm)	102
Linear Shrinkage LS , %	**27.14**

Soil Blended with 10 %CKD Linear Shrinkage Test Result

Determination No	1
Semi cylinderical trough No	A
initial wet length of soil L_0 (mm)	140
Dry length of soil L_d (mm)	103
Linear Shrinkage LS , %	**26.43**

Soil Blended with 5 %CKD Linear Shrinkage Test Result

Determination No	1
Semi cylinderical trough No	A
initial wet length of soil L_0 (mm)	140
Dry length of soil L_d (mm)	103
Linear Shrinkage LS , %	**26.43**

Soil Blended with 15 %CKD Linear Shrinkage Test Result

Determination No	1
Semi cylinderical trough No	A
initial wet length of soil L_0 (mm)	140
Dry length of soil L_d (mm)	103
Linear Shrinkage LS , %	**26.43**

Soil Blended with 20 %CKD Linear Shrinkage Test Result

Determination No	1
Semi cylinderical trough No	A
initial wet length of soil L_0 (mm)	140
Dry length of soil L_d (mm)	105
Linear Shrinkage LS , %	**25.00**

Soil Blended with 25 %CKD Linear

Determination No	1
Semi cylinderical trough No	A
initial wet length of soil L_0 (mm)	140
Dry length of soil L_d (mm)	107
Linear Shrinkage LS , %	**23.57**

Soil Blended with 30%CKD Linear

Determination No	1
Semi cylinderical trough No	A
initial wet length of soil L_0 (mm)	140
Dry length of soil L_d (mm)	113
Linear Shrinkage LS , %	**19.29**

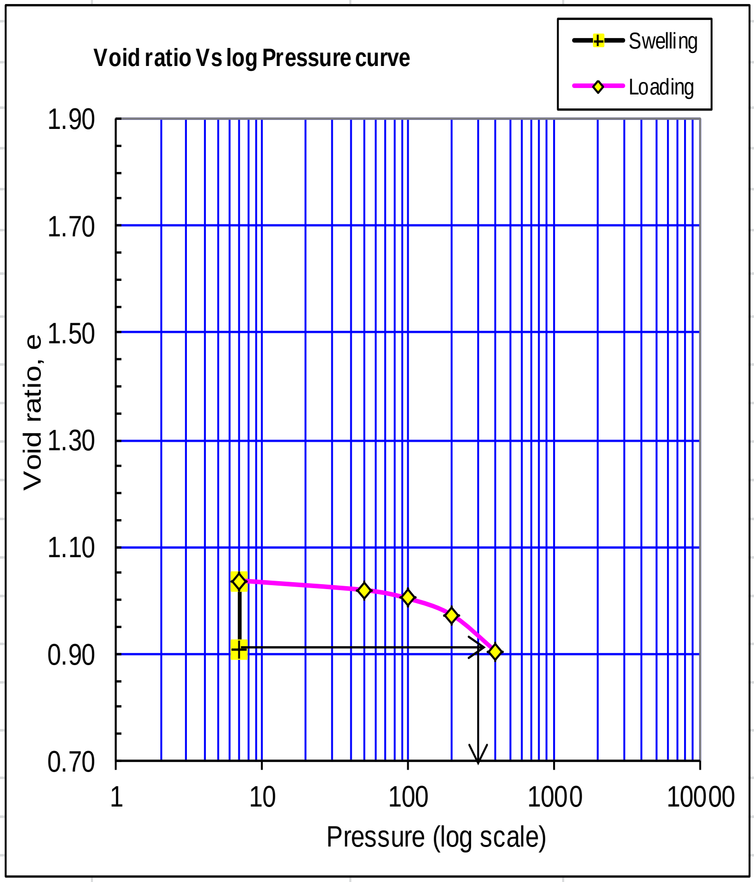
VIII) Volumetric Shrinkage Limit Determination

As per Test Method Test ASTM D427-39	0% CKD		5% CKD		10% CKD		15 % CKD		20% CKD		25 % CKD		30 % CKD	
	Tiral No		Tiral No		Tiral No		Tiral No		Tiral No		Tiral No		Tiral No	
Shrinkage Dish No	10	9	10	9	10	9	10	9	10	9	10	9	10	9
Mass of dish +wet soil (W1) ,gm	42.44	42.75	61.17	42.53	44.04	42.98	43.73	43.24	43.4	43.45	44.1	46.34	44.1	44.15
Mass of dish +dry soil (W3),gm	31.14	31.23	49.38	31.32	32.51	31.76	32.19	31.86	31.9	31.96	33.02	34.54	33.35	33.29
Mass of dish (W),gm	19.81	20.45	37.16	19.67	20.12	19.75	19.925	19.745	19.73	19.74	19.84	20.48	19.85	19.77
Mss of Water (W1-W3),gm	11.3	11.52	11.79	11.21	11.53	11.22	11.54	11.38	11.5	11.49	11.08	11.8	10.75	10.86
Mass of dry soil (W3-W),(W0) ,gm	11.33	10.78	12.22	11.65	12.39	12.01	12.265	12.115	12.17	12.22	13.18	14.06	13.5	13.52
Mass of wet soil (W1-W)=W2	11.3	11.52	11.79	11.21	11.53	11.22	11.54	11.38	11.5	11.49	11.08	11.8	10.75	10.86
Moisture content ((W1-W3)/W0)*100, (W	99.74	106.86	96.48	96.22	93.06	93.42	94.09	93.93	94.49	94.03	84.07	83.93	79.63	80.33
Volume of Dish (V)	18.5	18.7	19.3	18.4	19.3	18.98	18.7	18.99	18.1	19	19	19.8	18.8	18.7
Volume of dry soil (V0)	7.99	8	8.5	8	8.9	8.5	8.4	8.55	7.9	8.6	10.12	11.6	11.2	11.1
Volume change (V-V0),(ΔV)	10.51	10.7	10.8	10.4	10.4	10.48	10.3	10.44	10.2	10.4	8.88	8.2	7.6	7.6
Unit Volume change (ΔV/W0)*100 ,(ΔU)	92.76	99.26	88.38	89.27	83.94	87.26	83.98	86.17	83.81	85.11	67.37	58.32	56.30	56.21
Shrinkage Limit (W-ΔU)	**6.97**	**7.61**	**8.10**	**6.95**	**9.12**	**6.16**	**10.11**	**7.76**	**10.68**	**8.92**	**16.69**	**25.60**	**23.33**	**24.11**
Average	**7.290**		7.527		7.641		8.935		9.801		21.148		23.723	
Shrinkage Ratio (W0/V0)	**1.418**	**1.348**	**1.438**	**1.456**	**1.392**	**1.413**	**1.460**	**1.417**	**1.541**	**1.421**	**1.302**	**1.212**	**1.205**	**1.218**
Average	1.383		1.447		1.403		1.439		1.481		1.257		1.212	
Volumetric Shrinkage= SR*(W-SL)	**131.54**	**133.75**	**127.06**	**130.00**	**116.85**	**123.29**	**122.62**	**122.11**	**129.11**	**120.93**	**87.75**	**70.69**	**67.86**	**68.47**
Average	**132.64**		**128.53**		**120.07**		**122.36**		**125.02**		**79.22**		**68.16**	

IX) Swell-Pressure Test Result

i) Natural soil

Natural Soil



[A] In the beginning of the test

Sample type :	Remoulded
Ring Area,cm^2:	19.625
Height of sample,mm:	18
Seating Load,Kpa	7
Initial Void Ratio, e_o:	1.04
Initial moisture content,%	15.16
Specific Gravity:	2.77
Wet density,g/cm3	2.02

[B] In the end of the test

Final Moisture Content,%	17.97
Dry specimen wt (m_s), gm:	51.3
Dry density,g/cm3	1.38
Height of Solids(H_s), mm	9.44
Final Void Ratio, e_f:	0.90

[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	15.000	0.00	18.00	8.56	0.91
7	16.218	1.22	19.22	9.78	1.04
50	16.051	1.05	19.05	9.61	1.02
100	15.901	0.90	18.90	9.46	1.00
200	15.610	0.61	18.61	9.17	0.97
400	14.968	-0.03	17.97	8.53	0.90

Swell -Pressure= 380 Kpa

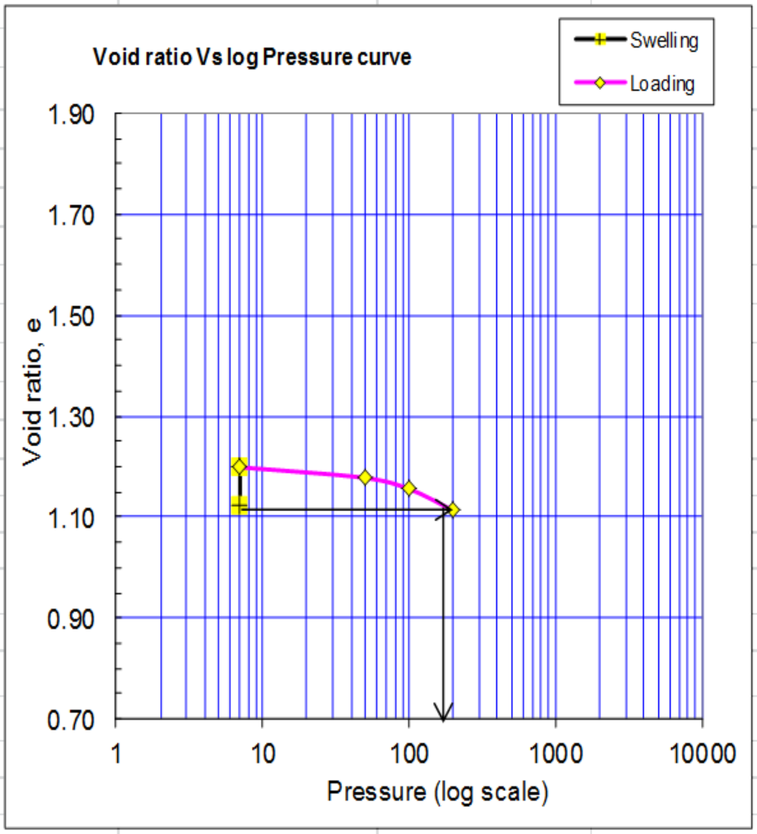
ii) Soil + 20 % CKD

[A] In the beginning of the test

Sample type :	Remould
Ring Area, cm^2:	19.625
Height of sample, mm:	18
Seating Load, Kpa	7
Initial Void Ratio, e_0:	1.20
Initial moisture content, %	22.01
Specific Gravity:	2.78
Wet density, g/cm3	1.90

[B] In the end of the test

Final Moisture Content, %	33.29
Dry specimen wt (m_s), gm:	46.3
Dry density, g/cm3	1.32
Height of Solids(H_s), mm	8.49
Final Void Ratio, e_f:	1.11



[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height, H_v (mm)	Void Ratio, E
Loading					
7	15.040	0.00	18.00	9.51	1.12
7	15.699	0.66	18.66	10.17	1.20
50	15.519	0.48	18.48	9.99	1.18
100	15.332	0.29	18.29	9.81	1.16
200	14.970	-0.07	17.93	9.44	1.11

Swell -Pressure= 165 Kpa

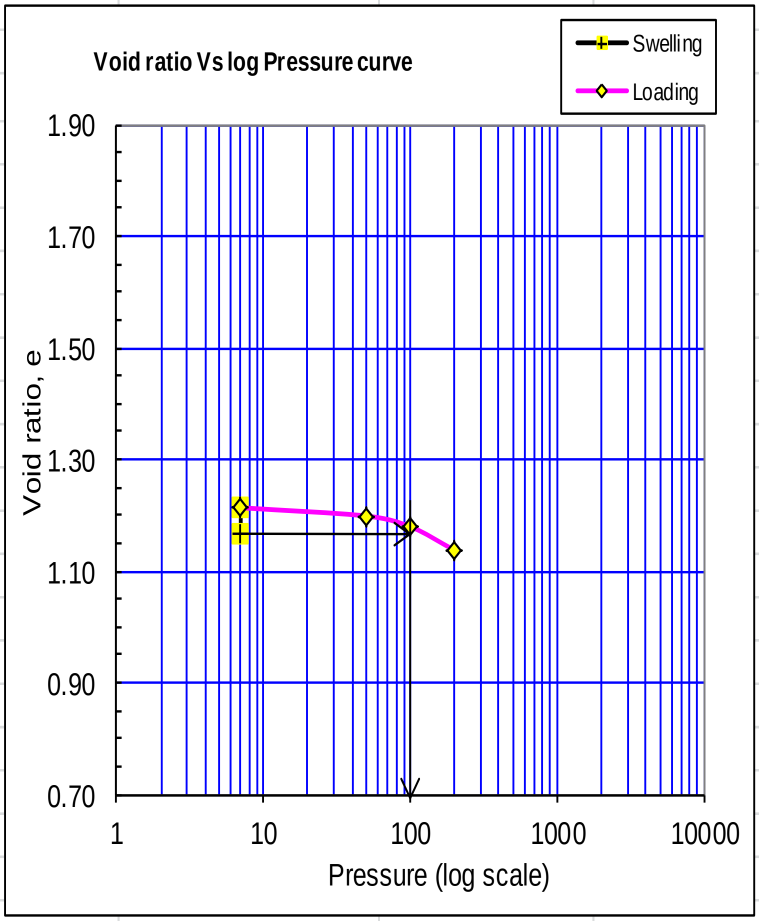
iii) Soil + 25 % CKD

[A] In the beginning of the test

Sample type :	Remould
Ring Area,cm^2:	19.625
Height of sample,mm:	18
Seating Load,Kpa	7
Initial Void Ratio, e_o:	1.21
Initial moisture content,%	30.47
Specific Gravity:	2.69
Wet density,g/cm3	1.78

[B] In the end of the test

Final Moisture Content,%	46.70
Dry specimen wt (m_s), gm:	43.9
Dry density,g/cm3	1.26
Height of Solids(H_s), mm	8.32
Final Void Ratio, e_f:	1.14



[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	13.820	0.00	18.00	9.68	1.16
7	14.225	0.40	18.41	10.09	1.21
50	14.102	0.28	18.28	9.97	1.20
100	13.942	0.12	18.12	9.81	1.18
200	13.591	-0.23	17.77	9.46	1.14

Swell -Pressure= 100 Kpa

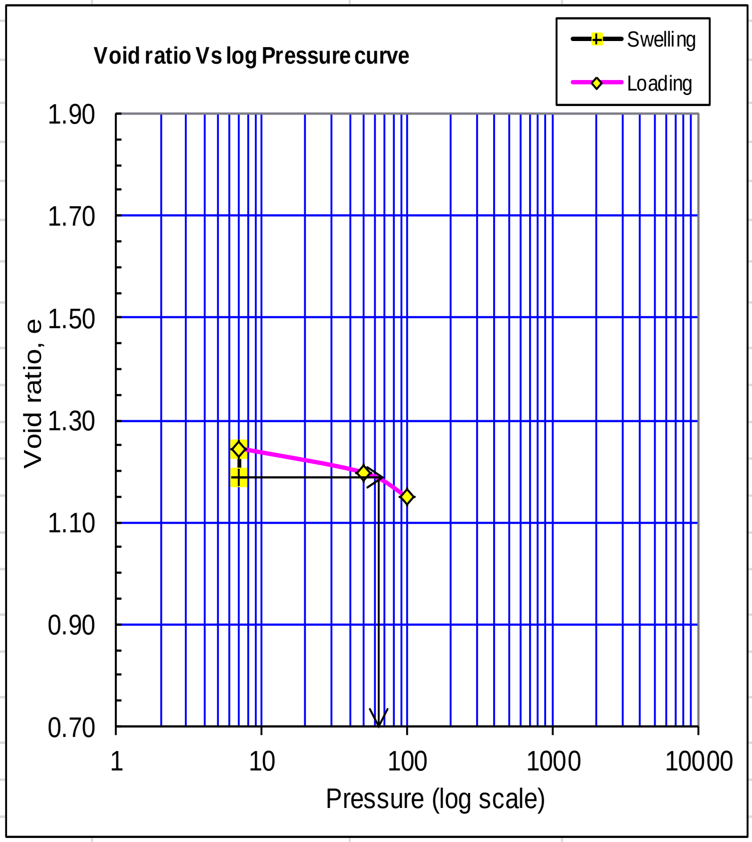
iv) Soil + 30 % CKD

[A] In the beginning of the test

Sample type :	Undisturbed
Ring Area,cm^2:	19.625
Height of sample,mm:	18
Seating Load,Kpa	7
Initial Void Ratio, e_o:	1.24
Initial moisture content,%	12.57
Specific Gravity:	2.72
Wet density,g/cm3	3.93

[B] In the end of the test

Final Moisture Content,%	18.33
Dry specimen wt (m_s), gm:	43.9
Dry density,g/cm3	1.27
Height of Solids(H_s), mm	8.22
Final Void Ratio, e_f:	1.15



[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	14.940	0.00	18.00	9.78	1.19
7	15.399	0.46	18.46	10.23	1.24
50	15.009	0.07	18.07	9.84	1.20
100	14.615	-0.32	17.68	9.45	1.15

Swell -Pressure= 68 Kpa

Specific Gravity Tests

A) Natural Soil

Specific Gravity of Soil		
Trial No	1	2
Dry mass of Soil (Ms),gm =	25	25
Wt of Pycnometer + Water + Soil (M1), gm	160.40	161.00
Wt of Pycnometer + Water (M2),gm	144.27	145.17
Test Temprature , ^{o}C	23.0	23.0
Correction Factor , K	0.9980	0.9983
Specific Gravity,G = [Ms/[Ms+(M2-M1)]x k	2.81	2.72
Average Specific Gravity of Soil	2.77	

B) 5 % CKD Plus Soil

Trial No	1	2
Dry mass of Soil (Ms),gm =	25	25
Wt of Pycnometer + Water + Soil (M1), gm	160.00	157.50
Wt of Pycnometer + Water (M2),gm	145.20	142.90
Test Temprature , ^{o}C	18.8	19.0
Correction Factor , K	0.9980	0.9983
Specific Gravity,G = [Ms/[Ms+(M2-M1)]x k	2.45	2.40
Average Specific Gravity of Soil	2.42	

C) 10 % CKD plus Soil

Trial No	1	2
Dry mass of Soil (Ms),gm =	25	25
Wt of Pycnometer + Water + Soil (M1), gm	165.30	161.90
Wt of Pycnometer + Water (M2),gm	150.40	147.10
Test Temprature , ^{o}C	23.0	23.0
Correction Factor , K	0.9993	0.9993
Specific Gravity,G = [Ms/[Ms+(M2-M1)]x k	2.47	2.45
Average Specific Gravity of Soil	2.46	

D) 15 % CKD Plus Soil

Trial No	1	2
Dry mass of Soil (Ms),gm =	25	25
Wt of Pycnometer + Water + Soil (M1), gm	164.10	163.60
Wt of Pycnometer + Water (M2),gm	149.00	148.60
Test Temprature , ^oC	23.0	23.0
Correction Factor , K	0.9993	0.9993
Specific Gravity,G = [Ms/[Ms+(M2-M1)]x k	2.52	2.50
Average Specific Gravity of Soil	2.51	

E) 20 % CKD Plus Soil

Trial No	1	2
Dry mass of Soil (Ms),gm =	25	25
Wt of Pycnometer + Water + Soil (M1), gm	159.00	161.05
Wt of Pycnometer + Water (M2),gm	144.20	144.60
Test Temprature , ^oC	23.0	23.0
Correction Factor , K	0.9993	0.9993
Specific Gravity,G = [Ms/[Ms+(M2-M1)]x k	2.45	2.92
Average Specific Gravity of Soil	2.69	

F) 25 % CKD Plus Soil

Trial No	1	2
Dry mass of Soil (Ms),gm =	25	25
Wt of Pycnometer + Water + Soil (M1), gm	157.65	161.86
Wt of Pycnometer + Water (M2),gm	142.90	145.20
Test Temprature , ^oC	23.0	23.0
Correction Factor , K	0.9993	0.9993
Specific Gravity,G = [Ms/[Ms+(M2-M1)]x k	2.44	3.00
Average Specific Gravity of Soil	2.72	

G) 30 % CKD Plus Soil

Trial No	1	2
Dry mass of Soil (Ms),gm =	25	25
Wt of Pycnometer + Water + Soil (M1), gm	165.89	163.56
Wt of Pycnometer + Water (M2),gm	150.40	147.10
Test Temprature , ^oC	23.0	23.0
Correction Factor , K	0.9993	0.9993
Specific Gravity,G = [Ms/[Ms+(M2-M1)]x k	2.63	2.93
Average Specific Gravity of Soil	2.78	

ANNEX IV - Laboratory Test Result From Geological Survey

Chemical tests of the natural soil i.e., major oxide and minor oxide and moisture content and shrinkage test of cement kiln dust attached here below.

Geological Survey of Ethiopia; Geoscience Laboratory Directorate Form G0004
Geochemical Laboratory Complete Silicate Analysis Report Format
FILE ID :- 6052/15 PVT Originator :- Adey Girma
Sample type:- soil(clay) Date submitted:- 13/3/2015
Preparation: -200 MESH **Element to be determined Major Oxides & Minor Oxides**
NUMBER OF SAMPLES: 1
Analytical Method: LIBO2 FUSION , HFattack,GRAVIMETERIC and AAS
Analytical Results in PERCENT

FIELD NO	Lab No	SiO2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O	LOI	SO3	CL
bole-01	6052/15	55.12	20.94	9.04	0.66	1.26	<0.01	0.32	0.17	0.06	1.36	0.23	11.00	0.55	<0.01

Analysts
Tizta Zemene
Getahun Bikila
Gosa Haile

Checked By

QUALITY CONTROL
WEREDE SAHILU

DATE REPORTED
1/6/2015

Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate ☐ **Gold & Base metal** ☐ **Water** ☐
Hydrocarbon ☐
Case Team: - Mineralogical: Lab section: - Mineralogy ☐ **Physical** [x]
Client /Originator Name:- Adey Girma
Client Category: - Survey ☐ **Gov.** ☐ **Pvt.** [x]
File name:- 6082/15 PVT Area Ref:- No of Samples:- Sample No :-
Sample Type :- Lab No :-
Type of Analysis :- Moisture content Preparation required:- Date Submitted:- 4/7/07

Coll. No.	Lab. No.	Weight of petridish with cover g	Weight of wet sample with petridish g	Weight of wet sample g	Weight of dried ample with petridish g	Weight of dried sample g	Water content Wn mass %	Average
		57.1304	78.1422	21.0118	78.0314	20.901	0.53	0.53
001	6082/15	56.6484	77.3608	20.7124	77.2513	20.6029	0.53	

Described By / Analysts Abayneh Begashaw
Checked by Misrak Tefera
Date Completed:- 19/3/15

Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate ☐ **Gold & Base metal** ☐ **Water** ☐
Hydrocarbon ☐

Case Team: - Mineralogical: Lab section: - Mineralogy ☐ **Physical** x
Client /Originator Name:- Adey Girma

Client Category: - Survey ☐ **Gov.** ☐ **Pvt.** x
File name:- 6084/15 GSE **Area Ref :- No of Samples: - Sample No.**
Sample Type :- Soil **Lab No:-**
Type of Analysis:- Shrinkage analysis **Preparation required: - Date Submitted:-** 4/7/07

Sample No.		Green Body	Dry Body	Fired Body	Shrinkage %	
Coll.No.	Lab.No.	Dimension in mm	Dimension in mm	Dimension in mm	Dry Shrinkage	Fired Shrinkage
003	6084/15	140	132		5.71	

Described By / Analysts Lakech Teferi
Checked by Misrak Tefera
Date Completed 20/3/15

Geological Survey of Ethiopia
Geosciences Laboratory Directorate
Result Form

Case Team: - Chemical: Lab Section: - Silicate ☐ **Gold & Base metal** ☐ **Water** ☐ **Hydrocarbon** ☐

Case Team: - Mineralogical: Lab section: - Mineralogy ☐ **Physical** [x]
Client /Originator Name:- Adey Girma

Client Category: - Survey ☐ **Gov.** ☐ **Pvt.** [x]
File name :- 6083/15 PVT **Area Ref:-** **No of Samples:-** **Sample No.**
Sample Type :- **Lab No: -**
Type of Analysis :- Specific gravity **Preparation required:-** **Date Submitted :-** 4/7/07

Coll.No.	Lab. No.	Pycnometer No.	m_2 Mass of picnometer in g	m_3 Mass of test solution in the picnometer without test sample in g	Q_2 Density of test solution in g/cm³	m_4 Mass of picnom.plus test sample in g	m_4-m_2 mass of test sample in g	m_5 mass of picnom. test sample and test solution in g	m_3+m_4 - m_5 volume of test sample in g/cm³	Specific Gravity in g/cm³	Average
002	6083/15	48/48	25.9856	75.8655	1 g/cm³	33.4097	7.4241	80.4829	2.8067	2.65	**2.65**
		63/63	26.5942	76.4686	1 g/cm³	33.2592	6.665	80.621	2.5126	2.65	

Described By / Analysts :- Meseret Desalegn **Checked by :-** Misrak Tefera **Date Completed** 18/3/15

DECLARATION

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

Name: Adey Girma ________Signature:____________ Date: September 2017

Contact address: Email: kirsura@yahoo.comTel: +251-911-657763

Place: School of Graduate Studies, Addis Ababa University

This is to certify that the thesis entitled '*Stabilization of Expansive Soil Used as Subgrade Material Using Cement Kiln Dust*'is the original work of Adey Girma, done under my close guidance and submitted for examination with my approval as a university advisor.

Name: Prof. Alemayehu Teferra (Adviser) Signature: _________Date: _______________