JIMMA UNIVERSITY

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JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

INVESTIGATING THE EFFECTS OF KAOLIN MIXED WITH CEMENT ON STRENGTH OF EXPANSIVE SUBGRADE SOIL

A final thesis submitted to the school of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Highway Engineering)

By:

Jemal Mohammed

February, 2019

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Co- Advisor: - Engr Yibas Mamuye (MSc.)

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DECLARATION

I, the undersigned, declare that this thesis entitled "Investigating the effects of kaolin mixed with cement on strength of expansive subgrade soil" is my original work, and has not been presented by any other person for an award of a degree in this or any other University, and all sources of material used for thesis have been duly acknowledged.

Candidate: **Jemal Mohammed**

Signature: ________________________

As Master research Advisors, we hereby certify that we have read and evaluated this MSc research prepared under our guidance, by **Jemal Mohammed** entitled: "Investigating the effects of kaolin mixed with cement on strength of expansive subgrade soil".

We recommend that it can be submitted as fulfilling the MSc Thesis requirements.

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ABSTRACT

Expansive soils have the tendency to behave volume change during wetting and drying scenario. This behavior of the soil cause large uplift pressure, differential settlement, and upheaval of structures built on them. Avoiding these types of unsuitable soils is mostly impractical, hence stabilization practices are considered to reduce the pre-specified behavior of the soils. The swell and shrinkage distinctiveness of expansive soil causes significant damage to structures such as buildings and pavements. The above problems are extensively occurring in Ethiopia. These clays are a consequence for expansive and unstable soil. As a result, they make pavement structure fail.

The general objective of this research was to Investigate the effect of kaolin mixed with cement on strength of expansive subgrade soil. In this study Atterberg Limits Tests, Particle Size Distribution, soil Classification, Free Swell Index, linear Shrinkage, Specific Gravity, Compaction (Moisture density relationships) tests, CBR and CBR-swell was determined. The research design was followed the experimental type of study which begins by collecting samples. The sampling technique used for this research was a purposive sampling which is non– probability method. Two expansive soil samples were taken for the study by observation and free swell index tests at a depth of 1.50 m to remove organic matter. Kaolin was taken from Tabor ceramic factory, in which the factory uses the material from Bombowha kaolin deposit located in Southern Ethiopia, Borena Zone, Bore district, at the locality called Bombowha.

Result of the chemical composition of kaolin shows that the total content of Silicon Dioxide (SiO_2), Aluminum Oxide (Al_2O_3) and Iron Oxide (Fe_2O_3) was 83.58% and physical properties of kaolin was conducted on the fineness (residue on 45 microns), specific gravity, PI, OMC and MDD were 30,2.61,10.85%,27.25%, and 1.42g/cm^3 respectively. Both the chemical and physical properties were fulfilling the requirements according to ASTM C-618.Wuhafisash-Yetebaberut road soil sample has plastic index 68 %, free swell index 122 %, linear shrinkage 23.7%, and CBR value 0.98 %. Similarly, Bosa Addis Kebele soil sample has plastic index 56 %, free swell index 64 %, linear shrinkage 19.36%, and CBR value 2.02%. Since both the given soil samples were found with high degree of expansion, stabilization was made with mix-ratio of 10% kaolin and 10% cement alone, 2% kaolin + 8% cement, 4% kaolin + 6% cement,6 % kaolin + 4% cement, and 8% kaolin + 2% cement.

The laboratory results obtained shows that 2% kaolin + 8% cement was an optimum ratio which achieved by most geotechnical parameters of the study. Moreover, Addition of 2%kaolin+8% cement yields almost the same result as addition of 10% cement content. Therefore, cement was partially replaced with 2% kaolin. All the laboratory result was compared with standard specifications. Since kaolin is not standalone stabilizer, it was recommended to study potential use of kaolin as admixture stabilizer. Additional curing time effect on all geotechnical laboratory tests should be performed. to have more accurate test results, additional test parameter like unconfined compressive strength, PH value test, volumetric shrinkage and mineralogical tests should also be performed.

Keywords: *Expansive Soils, Kaolin, Ordinary Portland Cement, Subgrade Strength, Stabilization*

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ABBREVIATIONS

AASHTO	American Association State Highway and Transportation Officials
ASR	Alkali-Silica Reaction
ASTM	American Society for Testing and Materials
BK	Bosa Adiss Ketema Kebele
CAH	Calcium Aluminate Hydrate
CBR	Californian bearing Ratio
CEC	Cation Exchange Capacity
CSA	Central Statistical Agency
CSH	Calcium Silicate Hydrates
ERA	Ethiopian Roads Authority
FSI	Free Swell Index
Gs	Specific Gravity
GSE	Geological Survey of Ethiopia
IS	Indian Standard
LL	Liquid Limit
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
OPC	Ordinary Portland cement
PI	Plastic Index
PL	Plastic Limit
UCS	Unified Compressive Strength
USCS	Unified Soil Classification System
WGS	World Geodetic System
WYR	Wuhafisash-Yetebaberut road

CHAPTER ONE

INTRODUCTION

1.1 Back ground of study

Expansive or reactive soil is a soil composed predominantly of clay. Clay undergoes significant volume change in response to changes in the soil moisture content. This volume change is realized by swelling upon wetting, and shrinkage upon drying. Being constructed on expansive soils, buildings are frequently prone to severe movement caused by non-uniform soil moisture changes with consequent cracking and damage related to the distortion. Rainfall and evaporation, garden watering, leaking water pipes, or tree root activity may trigger these moisture changes [1].

Expansive soils owe their expansive character mainly to the constituent clay mineral. Soils containing a considerable amount of montmorillonite minerals will exhibit high swelling and shrinkage characteristics [2]. The behavior of expansive soils varies from place to place depending upon the type of parent material, climate and topography [3].

Large areas of our country are covered with expansive soils such as dark and light grey clay soils. These clays have caused persistent difficulties in road construction and are common occurrences in Ethiopia [4]. To mitigate expansive soil problems several alternative solutions can be applied, of which stabilization is one of these alternatives. Stabilization techniques can also be applied to less expansive and non-expansive soils to improve important Engineering properties. Various organic and inorganic fractions of different soil types undergo modifications when catalyst agents are introduced into the soil. In turn the chemical reaction can convert inferior and formerly unstable materials to highly satisfactory road bed materials [5].

A siliceous or alumino-siliceous material that, in finely divided form and the presence of moisture, chemically reacts at ordinary room temperature with calcium hydroxide, released by the hydration of Portland cement, to form compounds possessing cementitious properties is called pozzolana [6]. Pozzolanic materials do not possess a cementing properties of their own, but they contain silica and alumina in reactive form. the chemical reaction of pozzolanic materials with calcium hydroxide in the presence of water forms compounds possessing cementitious properties [7]. The addition of pozzolanas reduces pore sizes and porosity leading to increased strength [8].

Kaolin is a subgroup of clay minerals having polytypes namely kaolinite, dickite and nacrite and a polymorph called halloysite [9]. It is commonly identified as white and soft clay that exhibits plasticity with the composition of fine-grained plate-like particles. It is formed from the alteration of anhydrous aluminate silicates in feldspar rich rocks like granite through weathering or hydrothermal processes. The clay is layered silica mineral, with one tetrahedral sheet linked through oxygen atoms to one octahedral sheet of alumina octahedral [10]. Such description implies that kaolin is suitable to be used as a natural pozzolan. Pozzolanic materials, of natural or artificial origin, contain a high percentage of amorphous silica and a high specific surface in order to generate a pozzolanic reaction [11]. This makes the clay capable of rendering the formation of secondary cementation bonds as a result of its high reactivity to calcium hydroxide generated from cement hydration process.

Geological works in the past indicated the presence of kaolin in many localities within Ethiopia, some of which namely, Kombolcha, near Harar, Debre tabor, kerker, Belesa and many occurrences in Tigray are worth mentioning. Of these the best studied and presently under mining is Bambowha deposit, in Sidamo. The Bambowha kaolin mining is supplying the main ceramic raw material to ceramic factory of Ethiopia known as Tabor ceramic factory, located in Hawassa, South Ethiopia [12].

1.2 Statement of the problem

Expansive soils occur in many parts of the world. However, the problem of expansion and shrinkage is associated with high moisture changes. Hence, it is restricted in areas where the seasonal variation in climatic condition is high. The large volume change with the periodic cycle of wetting and drying can cause extensive damages in civil engineering infrastructures; mainly on small buildings, shallow foundation and other lightly loaded structures including roads and airport pavements, pipelines and others. [13].

Expansive soil owes their expansive character mainly to the constituent clay mineral. The most important clay mineral, which is the cause for expansive nature is montmorillonite. Montmorillonite has an octahedral sheet sandwiched between two silica sheets. When this mineral is exposed to moisture, water is absorbed between interlay ring lattice structures and exerts an upward pressure. This upward pressure, known as swelling pressure, causes most of the damages associated with expansive soils [14]. During the last few decades, damages due to swelling action have been observed clearly in arid and semi-arid regions in the form of cracking and breakup of pavements, road ways, building foundations, slab on grade members, channel and reservoir linings, irrigation systems, water lines and sewer lines [15].

An appreciable part of Ethiopia is covered by expansive soil; big cities like Addis Ababa, Bahir-Dar, Mekelle and Jimma as well as main trunk roads are situated on this soil type. The aerial coverage of expansive soils in Ethiopia is estimated to be 24.7 million acres. Currently different construction activities are taking place in the road and building sector on this soil types. It has been noticed that construction on expansive soil face numerous problems and the causes of the problems are not investigated in depth in Ethiopia. Most of the roads constructed in Ethiopia on this type of soil fail before their expected design life, in some cases after few months of completion [16].

Since most soil which is found in Jimma Town have high plastic index and low CBR value. They are a consequence for expansive and unstable subgrade soil. As a result, they make pavement structure failure. The aim of this study was to investigate the effect of kaolin mixed with cement on strength of expansive soil for road subgrade construction.

1.3 Research questions

1. What are the physical properties of kaolin and properties of expansive soil?
2. What are the potential effects of kaolin on Engineering properties of expansive soil treated with varying dosage of kaolin and cement?
3. What optimum amount of stabilizing agents will be needed to attain the required properties of soils that can be used as subgrade material?

1.4 Objective of the study

1.4.1 General objective

The main objective of this research was to investigate the effect of kaolin mixed with cement on strength of expansive subgrade soil.

1.4.2 Specific objective

The specific objectives of this study are:

- To identify the physical properties of kaolin and properties of expansive soil.
- To investigate potential effects of kaolin on the Engineering properties of expansive soil treated with varying dosage of kaolin and cement.
- To determine optimum amount of Stabilizing Agents needed to attain the required properties of soils that can be used as subgrade material.

1.5 Significance of the study

For sub-grade and foundation preparation, particularly in the construction sector, stabilization minimizes cost of construction by reducing depletion of natural resources by improving properties of in situ soils to acceptable level. The production of hydraulic stabilizers (cement, lime, and fly ash) is environmental unfriendly and uneconomical processes. So it is crucial to find additional possibility of production of stabilizers which is environmentally friendly and Cost advantage. Natural pozzolans are the most alternative to replace part of cement. Natural pozzolans are typically both environmental friendly and cost effective than traditional stabilizers such as cement.

Furthermore, this research serves as a reference guide for practicing Civil Engineers and researchers that practice in the area of such study. This is useful in the sense that, it will cut down initial costs of new projects which are to commence and add our knowledge on the physical and Engineering behaviors of expansive soils and stabilizers.

1.6 Scope and Limitation of the Study

This study would have covered the stabilization of materials, which was not appropriate for road subgrade construction with naturally occurring expansive soil materials through using mechanical and chemical stabilization of different mix proportion of samples by conducting laboratory test. Two representative sample of expansive soil from different location was collected. The collected samples were disturbed and taken from 1.5 m depth. The appropriate laboratory tests resembling specific gravity, compaction test, CBR test, and sieve analysis test, linear shrinkage swelling test and Atterberg limit test were conducted to achieve the objective of the research. Correspondingly, the study has been compared the results with standard specifications similarly a recommendation was drawn and forwarded.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

Expansive soils are clay soils with high plasticity. As the name of the soil suggests, these soils are known for their peculiar nature of expanding or shrinking when exposed to moisture changes. Commonly, they are known as black clays or in some regions as “black cotton” soils. The name black cotton came from the fact that soils are found favorable in some regions for growing cotton [17]. Expansive soils have a complicated behavior and are generally characterized by detrimental volume changes when subjected to moisture fluctuations. At dry state, the expansive soils are very difficult to compact since their consistency varies from hard to very hard. At wet state, they are very sticking [18].

Expansive soils owe their expansive character mainly to the constituent clay mineral. Soils containing a considerable amount of montmorillonite minerals will exhibit high swelling and shrinkage characteristics [2]. The behavior of expansive soils varies from place to place depending upon the type of parent material, climate and topography [3]. Potentially expansive soils can be found almost anywhere in the world; it is widely spread throughout the five continents. Expansive soils cover large parts of the United States, South America, Africa, Australia and Asia. More than 30 countries have been reported for occurrence of expansive soils.

2.2 Origin of expansive soils

Parent materials that can be associated with expansive soils are either igneous rocks or sedimentary rocks. The basic igneous rocks comprise basalt, dolerite, sills, dykes, and gabbros. The second includes the sedimentary rocks that contain Montmorillonite as a constituent which breaks down physically to form expansive soils [19].

The Montmorillonite was probably formed from two separate origins. The product of weathering and erosion of rocks in the highlands were carried to the streams by coastal planes. The fine grained soil eventually accumulating in the ocean basin. The second case in which Montmorillonite is formed could be, volcanic eruption, sending up clouds of ash, fall on the plains and sees and through process thus ashes were altered to Montmorillonite [19].

2.3 Mineralogy of Expansive Soils

Expansiveness of soils is due to the presence of clay minerals. Clay particles have sizes of 0.002mm or less. However, according to Chain. 1988, the grain size alone does not determine clay minerals and the most important property of fine grained soils is their mineralogical composition. Clay minerals are crystalline hydrous alumino-silicates derived from parent rock by weathering. The basic building blocks of clay minerals are the silica tetrahedron and the alumina octahedron and combine into tetrahedral and octahedral sheets to form the various types of clays. Kaolinite, Illite and Montmorillonite are the common groups of clay minerals most important in Engineering studies. Montmorillonite is the clay mineral which causes most of the expansive soil problems. The name Montmorillonite is uses currently both as a group name for all clay minerals with high expansiveness potential [19].

I. Kaolinite

Kaolinite is a typical two layered mineral having a tetrahedral and an octahedral sheet joined to from 1 to 1-layer structure held by a relatively strong hydrogen bond. Kaolinite does not absorb water and hence does not expand when it comes in contact with water. The montmorillonite groups of clay minerals have 2 to 1-layer structure formed by an octahedron sandwiches between two tetra hedrons [13]. These clay groups have significant amount of magnesium and iron sandwiched into the octahedral layers. The most important aspect of the montmorilonite clay mineralogy group is the ability for water molecules to be absorbed between the layers, causing the volume of the minerals to increase when they come in contact with water. The Illite clay minerals have a structure similar to that of kaolinite, but are typically deficient in alkalis, with less aluminum substitution for silicon, magnesium and calcium can also sometimes substitute for potassium and illites are non-expanding type of clay minerals [20].

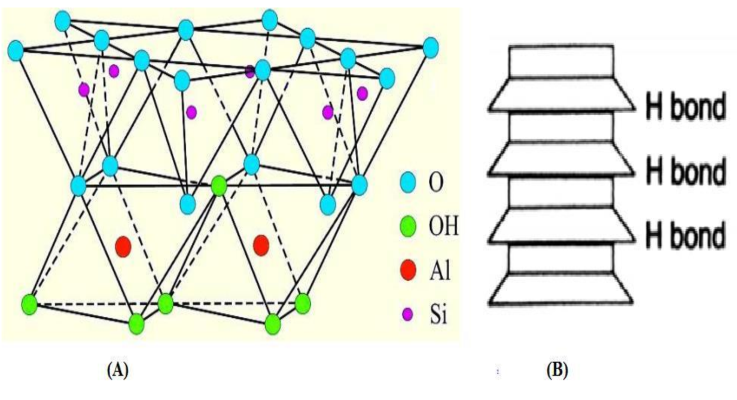

Figure 2.1: Diagrammatic and schematic representation of kaolinite [20]

II. Montmorillonite

Montmorillonite is a three-layer mineral having a single octahedral sheet sandwiched between two tetrahedral sheets [19]. The space between the combined sheets is occupied by water molecules and exchangeable cations. There is very weak bond between the combined sheets due to these ions. Considerable swelling of Montmorillonite being can occur due to additional water observed between the combined sheets [21].

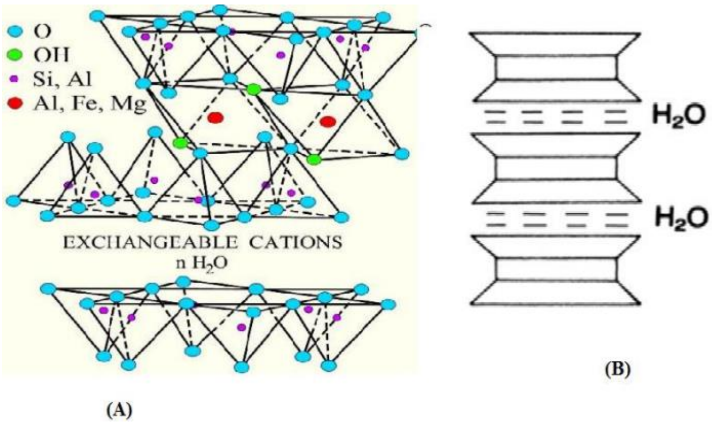

Figure 2.2: Diagrammatic and schematic representation of Montmorillonite [20]

III. Illite

Illite has similar structure with that of Montmorillonite, but some of the silicon atoms are replaced by aluminum and in addition potassium ions are present between the tetrahedral sheet and adjacent crystals [19]. The layers of Illite clay minerals are more strongly bonded together than the Montmorillonites. In terms of Cation exchange capacity, inability to absorb and retain water and in physical characteristics Illite is intermediate in activity between clays of kaolinite and Montmorillonite. The layers of Illite clay minerals are linked together by fairly weak bonding to potassium ions held between them [21].

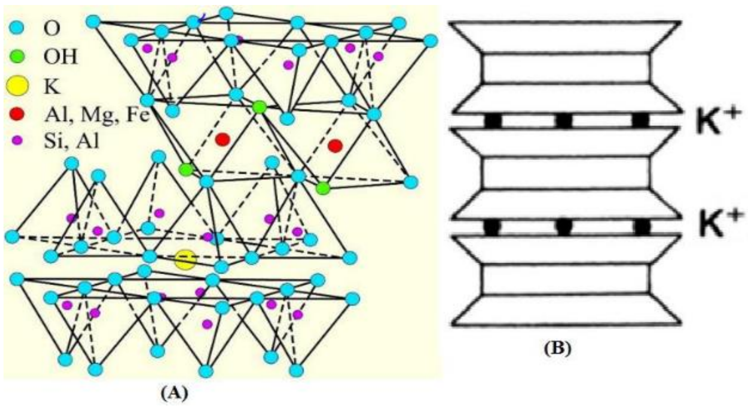

Figure 2.3: Diagrammatic and schematic representation of Illite [20]

2.4 Factors Affecting Expansive Soil Swelling and Shrinkage

According to Nelson and Miller (1992), the expansive soil’s swelling and shrinkage affecting factors are summarized in the table 2.1.

Table 2.1: Factors affecting expansive soil swelling [13]

Factors	Description
Clay mineralogy	Clay soils which have clay minerals with higher swelling potential like Montmorillonite have hinge swelling potential.as the amount of clay mineral with high swell potential increases the swelling potential of the soil increases.
Initial water content	Small amount of initial water content on the other hand indicates small degree of saturation. The tendency of soil to observe water will increase and this condition increases swelling potential.
Particle size	Fine particles in a soil exists densely, and the finer the particle the higher will be its expansion potential.
Dry density	The higher the value of initial dry density implies, a closer particle spacing have large swelling potential.
Concentration of pore fluid salts	Higher concentration of Cation in the pore field decreases expansion potential.
Pore field composition	Prevalence of monovalent Cation increases swelling potential while divalent inhabit shrinkage.
Climate	Arid climate courses desiccation of water content. This reduction of water content may lead to increase swelling potential of clay.
Location of water table	Fluctuating the location of water table causes variation of water content along the depth of the clay stratum, and the water content variation affects the soil swell-shrinkage property.
Thickness of clay stratum and confining pressure	High thickness of soil strata and large confining pressure reduces the soils swelling potential.
Field permeability	Joints and fissures in a soil allows to pass water through, and significantly affects swelling capacity

2.5 Identification of Expansive Soils

2.5.1 Field Identification

It is easy to recognize expansive soils in the field during either dry or wet seasons. Their color varies from dark grey to black. During dry seasons, shrinkage cracks are visible on the ground surface with the maximum width of these cracks reaching up to 20 mm or more and they travel deep into the ground. A lump of dry black cotton soil requires a hammer to break. A shiny surface is easily obtained when a partially dry piece of the soil is polished with a smooth object such as the top of a finger nail. During rainy seasons, these soils become very sticky and very difficult to traverse. Appearance of cracking in the nearby structures is also indicative [13] [2].

2.5.2 Laboratory Identification

there are a number of laboratory tests that are useful in identifying expansive soils. Generally, these can be categorized as mineralogical identification, direct and indirect methods [19].

I. Mineralogical identification

It is the nature of clay mineral in the soil, which is responsible for the swelling or shrinkage, and therefore, the methods that directly or indirectly enable to identify the types of minerals can be considered to be the most desirable ones [19]. For this purpose, tests like X-Ray diffraction, electron microscope, differential thermal analysis, dye absorption, determination of silica-sesquioxide ratio and Base Exchange capacity 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. As a result, for ordinary engineering works these methods are generally not often used.

II. 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 some samples to avoid erroneous conclusions. Direct measurement of expansive soils can be achieved by the use of conventional one-dimensional consolidometer.

III. Indirect Methods

This method is a valuable tool in evaluating the swelling potential of the soils. It is advisable not to use the indirect tests directly, instead direct tests are also important to avoid an error in conclusions. These methods are related to laboratory soil identification and are vital for the intended purposes [13].

A. Atterberg Limits

Atterberg limits define the moisture content boundaries between state of consistency of fine grained soils. Clay soil can exist in four distinct state of consistency depending on its water content. The water content at the boundaries between the different states are defined as the shrinkage, plastic and liquid limits. Two useful indices may be computed from the atterberg limits and the natural moisture content. These are the Plasticity Index and Liquidity Index. The Plasticity Index used extensively for classifying expansive soils and should always be determined during preliminary investigation [13].

Table 2.2: Soil Expansivity Predictions by Liquid Limit [19] [22]

Degree of Expansion	Liquid Limit (%)	
	Chen	IS 1498
Low	<30	20-35
Medium	30-40	35-50
High	40-60	50-70
Very High	>60	70-90

Table 2.3: Soil Expansivity Predictions by Plastic Limit [23] [19] [22]

Degree of Expansion	Plastic Limit (%)		
	Holtz and Gibbs	Chen	IS 1498
Low	<20	0-15	<12
Medium	12-20	10-35	12-23
High	20-35	20-55	23-32
Very High	>35	>35	>32

Table 2.4: Relation between the swelling potential of clays and the plasticity index [19]

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

While it may be true that high swelling soil will manifest high index property, the converse is not true [19].

B. Free Swell Index

The free swell index is also one of the most commonly used simple tests to estimate the swelling Potential of expansive clay. The procedure involves in taking two ovens dried soil samples Passing through the 425µm sieve, 10cc each was placed separately in two 100ml graduated soil Sample. Distilled water was filled with one cylinder and kerosene in the other cylinder up to 100ml mark. The final volume of soil is computed after 24 hours to calculate the free swell index [24].

The free swell index is then calculated using Equation below.

$$\text{Free Swell Index (\%)} = \frac{(Vw - Vk)}{Vk} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (2.1)$$

Where: V_w = final volume in water and Vk = final volume in kerosene

The soils having high free swell index value may show considerable volume Changes as compared to the soils having lower free swell index values. Mohan and Goel (1959) gave the following classification of degree of expansion based on the Free swell index values as given in the Table 2.5; and the same have been suggested by Indian standard IS1498.

Table 2.5:Classification of degree of expansion based on the Free swell index [22]

Free swell index	Degree of expansion	Degree of severity
>200	Very high	Severe
100 – 200	High	Critical
50-100	Medium	Marginal
<50	Low	Non-critical

C. Cation Exchange Capacity (CEC)

The CEC is the quantity of exchangeable cations required to balance the negative charge on the surface of the clay particles. CEC is expressed in milli equivalents per 100 grams of dry clay. CEC is related to clay mineralogy. High CEC values indicate a high surface activity. In general, swell potential increases as the CEC increases. Typical values of CEC for the three basic clay minerals are given in Table 2.6.

Table 2.6: Typical CEC values of basic clay minerals after Mitchell, 1976 [13]

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

D. Linear Shrinkage

The swell potential is presumed to be related to the opposite property of linear shrinkage measured in a very simple test. Theoretically it appears that the shrinkage characteristics of the clay should be a consistent and reliable index to the swelling potential [19]. Altmeyer in 1955 suggests a relationship between linear shrinkage, shrinkage limit and the potential of expansiveness as shown in the table 2.7.

Table 2.7: Shrinkage limit, linear shrinkage, and degree of expansion relationship [25]

Shrinkage limit (%)	Linear shrinkage (%)	Degree of expansion
< 10	>8	Critical
10-12	5-8	Marginal
> 12	0-5	Non-critical

2.6 Classification of Expansive Soils

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

2.6.1 General Classification

I. Unified Soil Classification Systems

This classification is based on plasticity chart and a correlation is made between swell potential and unified soil classification as follows.

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

The above classification system can be summarized as follow:

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

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

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

In the above classification soils rated as CL or OH may be considered as potentially expansive.

II. AASHTO Classification System

The AASHTO soil classification system is used to determine the suitability of soils for earthworks, embankments, and road bed materials such as subgrade, sub-base and base. According to this classification system, granular soils are soils in which 35% or less are finer than the No. 200 sieve (75 µm). Silt-clay soils are soils in which more than 35% are finer than the No. 200 sieve (75 µm). The system classifies soils into seven major groups, A-1 through A-7. The first three groups, A-1 through A-3 are granular (coarse-grained) soils, while the last four groups, A-4 through A-7 are silt-clay (fine-grained) soils [13].

2.6.2 Classification Specific to Expansive Soil

The general classification systems are found to be suitable for identification of expansive soils or for prediction of swelling characteristics or Expansion Potential, but it does not provide useful information. A parameter determined from the expansive soil identification tests have been combined in some different classification schemes to give the qualitative rating on the expansiveness of the soil. But the direct use of such classification systems as a basis for design may lead to an overly conservative construction in some places and inadequate construction in some areas [13]. Hence, it is very important to emphasize that design decision has to be based on predicting testing and analysis, which provide reliable information. An indirect prediction of swell potential includes correlations based on index properties, swell and a combination of them. Some of such classification systems are enumerated below.

a) Method of Chen

As Chen (1988) Presented a single index method for identifying expansive soils using only plasticity index. Chen suggested four classes of clays according to their plasticity indices shown in table 2.4.

b) Method of Daksanamurthy and Raman (1973)

Daksanamurthy and Raman (1973) presented a single index method for identifying expansive soils using only liquid limit. They suggested four classes of clays according to their liquid limits as shown in Table 2.8.

Table 2.8: Relation between the swelling potential of clays and the liquid limit [24]

Swelling potential	Liquid limit
Low	$20 < LL \leq 35$
Medium	$35 < LL \leq 50$
High	$50 < LL \leq 70$
Very high	$LL > 70$

c) USBR Method

This method is developed by Holtz and Gibbs; it is based on direct correlation of observed volume change with colloid content, plastic index and shrinkage limit. The classification is as given in Table 2.9.

Table 2.9: Classification based on the bureau of reclamation method [19] [26]

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

d) Method of Seed et al

After an extensive study on swelling characteristics of remolded, artificially prepared and compacted clays, Seed et al. Chen (1988) have developed a chart based on activity and percent clay sizes as shown in Figure 2.4. The activity here is defined as;

$$AC = \frac{PI}{C - 10} \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots (2.2)$$

Where; A = activity, C= percentage of clay-size finer than 0.002mm, PI= plasticity index

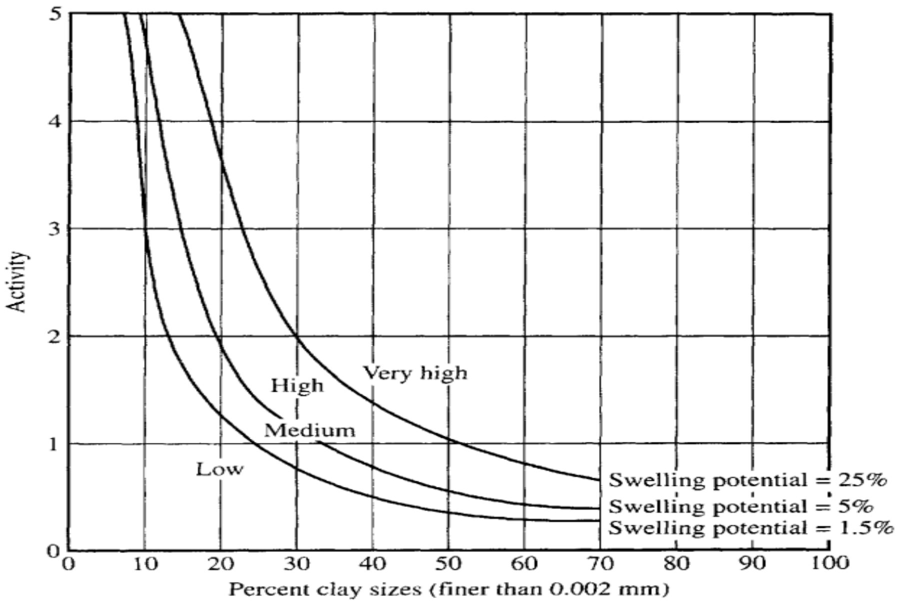

Figure 2.4: Classification chart for swelling potential according to Seed et al.

e) Method of Skempton

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

$$A = \frac{Ip}{c} \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots (2.3)$$

Where: Ip =plasticity Index, C=percentage of clay size finer than 0.002mm by weight

Skempton suggested that three classes of clays according to their activity shown in Table 2.10.

Table 2.10: Relation between clay activity and potential of expansion [19]

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

2.7 Stabilization

Soil stabilization is the process of improving the engineering property of the soil and thus making it more stable. The term stabilization is general restrict to the process which alters the soil material itself for improvement of its property. Soil stabilization is used to reduce the permeability and compressibility of the soil mass in earth structures and to increase its shear strength [27].

2.7.1 Methods of stabilization

Nowadays there are so many methods to stabilize soils; but all of the methods are restricted to juts two main groups which are mechanical and chemical.

2.7.1.1 Mechanical Method

Mechanical stabilization can be defined as a process of improving the stability and shear strength characteristics of the soil without altering the chemical properties of the soil. The main methods of mechanical stabilization can be categorized into compaction, mixing or blending of two or more gradations, applying geo-reinforcement and mechanical remediation [28].

2.7.1.2 Chemical Stabilization

Chemical stabilization involves mixing or injecting the soil with chemically active compounds such as Portland cement, lime, fly ash, calcium or sodium chloride or with viscoelastic materials such as bitumen. Dallas and Nair (2009) classifies chemical stabilizers in to three groups:

- **Traditional stabilizers** such as hydrated lime, Portland cement and Fly ash;
- **Non-traditional stabilizers** comprised of sulfonated oils, ammonium chloride, enzymes, polymers, and potassium compounds; and
- **By-product stabilizers** which include cement kiln dust, lime kiln dust etc.

Among these, the most widely used chemical additives are lime, Portland cement and fly ash. Soil improvement by means of chemical stabilization can be grouped into three chemical reactions; cation exchange, flocculation-agglomeration pozzolanic reactions.

I. Cation Exchange

Clay minerals have the property of absorbing certain anions and cations and retaining them in an exchangeable state. The exchangeable ions are held around the outside of the silica – alumina clay – mineral structural unit and the exchange reaction doesn't affect the structure of the silica – alumina pocket. In clay minerals, the most common exchangeable cations are Ca^{2+}, Mg^{2+} ,H^{+}, NH_4^{+}, Na^{+}, frequently in about that order of general relative abundance [29].

The existence of such charges is indicated by the ability of clay to absorb ions from the solution. Cations (positive ions) are more readily absorbed than anions (negative ions); hence, negative charges must be predominant on the clay surface. A cation, such as Na^{+}, is readily attracted from a salt solution and attached to a clay surface. However, the absorbed Na^{+} ion is not permanently attached; it can be replaced by K^{+} ions if the clay is placed in a solution of potassium chloride (KCl). The process of replacement by excess cation is called cation exchange. Some are more strongly attracted than others, and the cations can be arranged in a series in terms of their affinity for attraction as follows:

$Al^{3+} > Ca^{2+} > Mg^{2+} > NH_4^{+} > K^{+} > H^{+} > Na^{+} > Li^{+}$

This series indicates that, for example, Al_3+ ions can replace Ca_2+ ions, and Ca_2+ ions can replace Na+ ions. The exchangeable cations may be present in the surrounding water or be gained from the stabilizers. The process is called cation exchange [29].

An example of the cation exchange;

Na-clay +$CaCl_2$ → Ca-Clay + NaCl

II. Flocculation and Agglomeration

Flocculated structure occurs in clays. The clay particles have large surface area and, therefore, the electrical forces are important in such soils. The clay particles have a negative charge on the surface and a positive charge on the edges. Inter particle contact develops between the positively charged edges and the negatively charged faces. This results in a flocculated structure. Flocculent structure is formed when there is a net attractive force between particles. When clay particles settle in water, deposits formed have a flocculated structure. The degree if flocculation of a clay deposit depends upon the type and concentration of clay particles, and the presence of salts in water. Clays settling out in a salt water solution have a more flocculent structure than clays settling out in a fresh water solution. Salt water acts as an electrolyte and reduces the repulsive forces between the particles.

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

In general, the soils in a flocculated structure have a low compressibility, a high permeability and high shear strength.

III. Pozzolanic Reactions

The pozzolanic reaction process, which can either be modest or quite substantial depending on the mineralogy of the soil, is a long term process. This is because the process can continue as long as a sufficiently high pH is maintained to solubilize silicates and aluminates from the clay matrix, and in some cases from the fine silt soil. These solubilized silicates and aluminates then react with calcium from the free lime and water to form calcium-silicate-hydrates and calcium aluminate- hydrates, which are the same type of compounds that produce strength development in the hydration of Portland cement. However, the pozzolanic reaction process is not limited to long term effects. The pozzolanic reaction progresses relatively quickly in some soils depending on the rate of dissolution from the soil matrix [30]. Pozzolanic constituents produces calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH). Rate of the pozzolanic reactions depends on time and temperature.

$Ca^{2+} + 2(OH)^{-} + SiO_2$ (Clay Silica) $\rightarrow$ CSH………………………...……………… (2.4)

$Ca^{2+} + 2(OH)^{-} + Al_2O_3$ (Clay Alumina) $\rightarrow$ CAH ……………………………..….… (2.5)

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

2.8 Cement Stabilization

Traditional stabilizers generally rely on pozzolanic reactions and cation exchange to modify and/or stabilize. Among all traditional stabilizers, cement probably is the most routinely used. Portland cement is comprised of calcium-silicates and calcium-aluminates that hydrate to form cementitious products [31]. Portland cement stabilization, commonly referred to as soil cement, is a mixture of Portland cement, water and soil compacted to a high density. Soil cement is sometimes referred to as a cement treated subgrade or cement stabilized subbase. When cured, the soil cement mixture becomes a hard, rigid base material. A flexible or rigid pavement surface is placed on top of the soil cement to complete the pavement structure.

Cement stabilization differs from other forms of chemical stabilization in such a way that structural strength is primarily obtained from the cementing action rather than from internal friction, cohesion, chemical ion exchange and/or waterproofing of the materials. Almost all types of soils can be used for cement stabilization except highly organic soils and heavy clay soils.

The four fundamental control factors for the design and construction of soil cement are moisture content, curing procedure and duration, compaction and cement content. Cement stabilization is generally considered to be too expensive for workability improvements alone [32].

2.8.1 Mechanisms of Cement Stabilization

Hydration reaction is the primary mode of strength gain in soil cement. Free lime, $Ca(OH)_2$, produced during the hydration process can comprise up to about 25 percent of the cement and water mix on a weight basis. This free lime can produce pozzolanic reaction between the lime and soil, which can continue as long as the pH is high enough to solubilize the soil minerals [31].

The hydration product obtained from cement stabilization occurs through the same type of pozzolanic reactions as lime stabilization. It is the origin of silica required for pozzolanic reaction that differs. With cement stabilization, the cement already contains the silica, unlike lime stabilization where silica needs to be broken down from clay. Therefore, unlike lime stabilization, cement stabilization is fairly independent of soil properties [33]. According to The tensar Corporation (1998), mechanisms of cement stabilization are classified in to four major groups.

Hydration of cement (highest importance): continuous skeleton of hard, strong material forms and encloses a matrix of unaltered soil. Strengthening of treated material and filling some of the voids occurs. Permeability and shrink/ swell tendencies are reduced and resistance to changes in moisture content is increased.

Cation Exchange (high importance): Cation exchange alters electric charge, reducing plasticity and resulting in flocculation and aggregation of soil particles.

Carbonation (minor): Lime generated during hydration of cement reacts with carbon dioxide in air to from cementing agents.

Pozzolanic Reactions (minor): Free lime liberated during hydration reacts with silica or alumina from clay particles in the presence of moisture to form cementing agents.

Cement hydration is rapid and causes immediate strength gain in stabilized layers. Therefore, a mellowing period is not typically allowed between mixing and compaction. The general practice is to compact soil cement before or shortly after initial set, preferably within 2 hours of mixing [32].

2.8.2 Mix Design and Strength Characteristics

The goal of mixture design using cement stabilization is to find the lowest cement content that will produce the desired strength. The cement content determines whether the characteristics of the mixtures are dominated by the properties of original soil or hydration products. The strength of soil cement increases linearly with the quantity of cement added to the soil [4].

Cement stabilization is ideally suited for well graded aggregates with sufficient amount of fines to effectively fill void space and float the coarse aggregate particles. General guidelines for stabilization are that the plasticity index should be less than 30 for sandy materials. For fine-grained soils, soils with more than 50 percent by weight passing No.200, the general consistency guidelines are that the plasticity index should be less than 20 and the liquid limit should be less than 40 in order to ensure proper mixing. A more specific guideline based on the fines content is given in the equation (2.6) defining the upper limit Plasticity Index (PI).

$$\mathrm{PI} \leq \left(\frac{20 + 50 - (\%\text{smaller than } 0.075\text{mm})}{4}\right) \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots.. (2.6)$$

Cement content requirements vary depending on the soil type and desired properties of the soil cement. For most soil cement applications, Type I or Type II Portland cement conforming to ASTM C150 is used. Generally, as the clay content of the soil increases, the quantity of cement required increases. AASHTO soil classification system is used for pavement soil classification in Ethiopia. AASHTO's cement requirement for different types of soils is summarized in Table 2.11.

Table 2.11: Cement requirement for AASHTO soil Groups

AASHTO Soil Group	Usual Range in Cement Requirement in percent by		Estimated Cement Content, Percent by Weight
	Volume	Weight	
A-1-a	5-7	3-5	5
A-1-b	7-9	5-8	6
A-2	7-10	5-9	7
A-3	8-12	7-11	9
A-4	8-12	7-12	10
A-5	8-12	8-13	10
A-6	10-14	9-15	12
A-7	10-14	10-16	13

Water is necessary in soil cement to help obtain maximum compaction and cement hydration. Moisture contents of soil cement usually range from 10 to 13 percent by weight of oven dry soil cement. Water should be potable or relatively clean, free from harmful amount of alkalis, acids or organic matter matrix [30].

2.8.3 Moisture Density Relations of Soil-Cement Mixtures

Changes in optimum moisture content and dry density with addition of cement are not always predictable [34]. Flocculation of clay particles by cement can cause an increase in optimum moisture content and decrease in maximum dry density for cement-soil mixes whereas the higher density of cement relative to soil can result in a higher density for mixes. Therefore, it is appropriate to use the median cement content as estimated in Table 2.11 for determination of moisture density relationships as the maximum dry density varies only slightly with modest changes in percent cement content [35]. However, as previously discussed, if it is expected that acceptable treatment can be achieved with considerably lower cement contents than those in Table 2.11, then that cement content should be used to determine the moisture-density relationship. After the required amount of cement is added to the soil, the blend should be mixed thoroughly until the color of the mixture is uniform. Fabrication and testing of samples for moisture density relationship should be done in accordance with AASHTO T 134.

2.9 Kaolin

Kaolin is both a rock term and a mineral term. From the rock point of view, kaolin means that the rock is comprised predominantly of kaolinite and or one of the other kaolin minerals.

Mineral wise, it represents the group name for the minerals kaolinite, dickite, nacrite, and halloysite [36]. kaolin is also defined as a rock mass containing principally kaolinitic clays that are low in iron, and usually white or nearly white in color comprising naturally occurring kaolin group minerals. It can be contained in a variety of kaolinitic rock types. The primary kaolin explains a kaolin which is altered from an igneous or metamorphic rock that was kaolinized in situ by hydrothermal or weathering processes. Secondary kaolin is sedimentary kaolin comprising transported mineral particles. Kaolin is among the major industrial clays including Smectites, and Palygorskite–Sepiolite [37].The main Kaolin minerals include kaolinite, dickite, nacrite, and halloysite. These minerals are dioctahedral 1:1 phyllosilicates having a sheet of silicon atoms in tetrahedral coordination with four oxygen atoms and a sheet of aluminum atoms in octahedral coordination with two oxygen atoms and four hydroxide molecules. In general, the basic kaolin mineral structure constitutes a layer of a single tetrahedral sheet and a single octahedral sheet. Among the kaolin minerals, Kaolinite ($Al_2Si_2O_5 (OH)_4$) is the most common mineral and has great industrial importance.

Primarily, kaolin is used as (i) a pigment to improve the appearance and functionality of paper and paint, (ii) a functional filler for rubber and plastic, (iii) a ceramic raw material, and (iv) a component for refractory, brick, and fiberglass products. Other less significant uses for kaolin include chemical manufacture, civil engineering, agricultural applications, and some pharmaceuticals.

2.9.1 Kaolin deposits of Ethiopia

Economic Kaolin resources of Ethiopia are mostly associated with acidic intrusive rocks (granites and pegmatites) and gneissic rocks. Kaolin hosted by sedimentary rocks are reported in Blue Nile river basin, Ogaden basin and Mekelle Outlier [38].

Geological works in the past indicated the presence of kaolin in many localities within Ethiopia, some of which namely, Kombolcha, near Harar, and many occurrences in Tigray are worth mentioning. Of these the best studied and presently under mining is Bombowha deposit, in Sidamo. The Bombowha kaolin mining is supplying the main ceramic raw material to ceramic factory of Ethiopia known as Tabor ceramic factory, located in Hawasa, South Ethiopia [12]. These and the other kaolin which have been investigated in the different parts of the country are summarized in Table 2.12 and Figure 2.5.

Table 2.12: Kaolin occurrence in different parts of Ethiopia [39]

Occurrence Name	Longitude	Latitude	Region
Bombowha kaolin	38°46' 30" E	06°05' 20" N	Oromia
Belesa kaolin	37° 58'E	7° 35'N	SPNNRS
Awzet kaolin	38°07' 48" E	11°45' 00" N	Amhara
Debre Tabor kaolin	38°00' 36" E	11°50' 02" N	Amhara
Gypsite- Mariam kaolin	37°35’24” E	11°45’36” N	Amhara
Kerker kaolin	37°24' 43" E	12°42' 40"N	Amhara
Kombelcha kaolin	42°08' 50" E	09°27' 58" N	Oromia
Ansho Kaolin	37° 38' 28" E	7°20' 6" N	SPNNRS

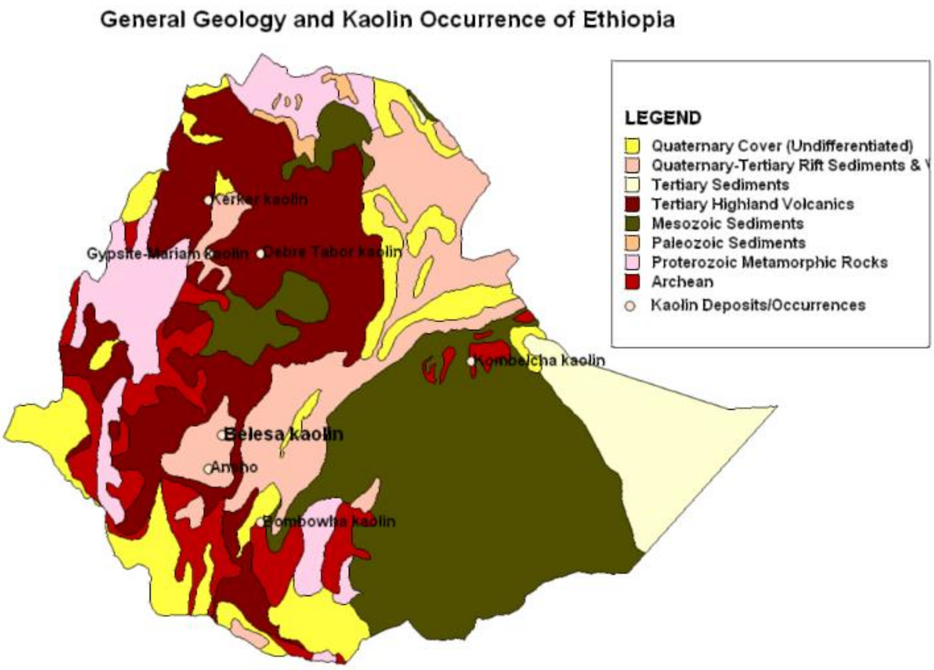

Figure 2.5: General Geology and Kaolin Occurrences of Ethiopia [39]

Exploration in Bombowha areas proved about 300, 000 tons of kaolin. Proven reserves of 150,000 ton of kaolin had been identified in 1992, at which time open-pit mining operations began. Moreover, recently it is reported that several hundred thousand tons of good quality kaolin is known to be hosted by the kaolinized granites of Bombowha area [40].

Table 2.13: Annual Kaolin Production in Ethiopia (metric tons) [41]

Year	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011
production	3,534	3088	4251	4,300	1641	1400	1,275	3,534	3,600	4,000

2.9.1.1 Bombowha Kaolin Deposit

Bombowha kaolin deposit is located in Southern Ethiopia, Borena Zone, Bore District, at the locality called Bombowha. It is found on the map sheet NB 37-6 in the northern extreme part of the Adola Gold Field. Geographically the area is bounded between 6° 04' 38''N to 6 05' 20''N latitude and 38° 45' 25''E to 38°46' 10''E longitude. The deposit is 430km from Addis Ababa in the south and can be reached by an all-weather road through Hawassa, on the way to Negele Borena. The road is asphalted for about 315 km from Addis Ababa and turns to the left to all weather gravely road. Bore and Kiberemengist towns are about 40km north and south of the area respectively. Bombowoha kaolin is a product of in-situ weathering of pegmatites and granites. The upper parts of the pegmatites and granites have been kaolinzed to a depth of 20mt. The main body of the deposit lies within the big intrusion in a zone extending outward towards west and south of the area. These granitic intrusions are highly weathered, kaolinized, pinkish near the surface and brownish yellow, and grey to white with depth [40].

Table 2.14: Average chemical composition of Bombowha I and III Kaolin deposits [40]

Oxides	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5
Value (%)	47.74	35.00	0.84	0.16	0.10	0.47	1.28	0.03	0.17

CHAPTER THREE

RESEARCH METHODOLOGY

3.1 Study Area

Jimma is located at about 354 Km in Southwest of Addis Ababa [42]. According to WGS 84 coordinate reference system which is the latest revision of the World Geodetic System, Jimma is geographically located between 7° 38’52” and 7° 43’ 14” N latitude, and between 36° 48’ 00” and 36° 53’ 24” E longitude. The town is found in an area of the altitude of 1718-2000m above sea level. It lies in the climatic zone locally known as Woyna Daga which is considered ideal for agriculture as well as human settlement [42].

The main geological formation of Jimma town is the Cenozoic tertiary volcanic rock of Nazareth series and Jimma volcanic that were formed by lava and debris ejected from fissure eruptions. Basalts, Trachyte, Rhyolite, and Ignimbrite are the major rock types that belong to the trap series formation [43]. Tropical Residual fine-grained soils, like clays and silt-clays, developed mainly on basaltic bedrock represent the soils found in Jimma town [44].

Figure 3.1: Map of the study area (source: Google earth 2018)

3.2 Study design

This research was designed to answer the research questions and meet its objectives based on experimental findings. The first step in the research work was sample collection. The next step was laboratory tests on the treated and untreated expansive soil. The laboratory test data was analyzed and interpreted so that the properties of expansive soil and its performances on additives requirement was addressed. Finally, the research findings and recommendations was expressed based on the laboratory test results.

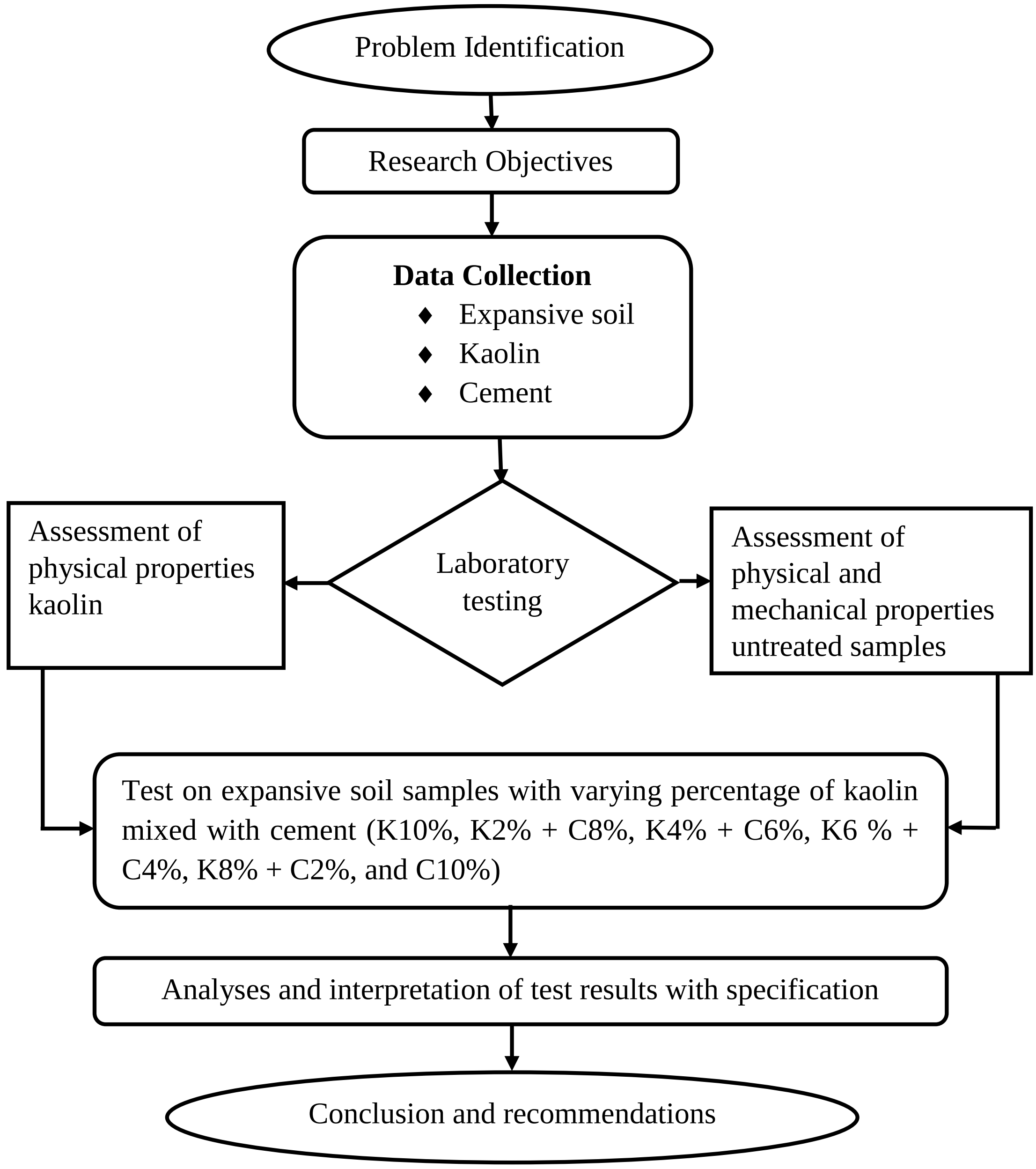

Figure 3.2: Research design flow chart

3.3 Sampling Techniques

The sampling technique used for this research was a purposive sampling which is non – probability method. This sampling technique was proposed based on goal of the researcher to be achieved and based on the information that to determine the strength of the expansive soil.

3.4 Study Variables

The dependent variables are more related with general objective of the study. Strength of expansive soil treated using kaolin mixed cement was a dependent variable. The independent variables were physical and mechanical properties of untreated and treated soil samples, and Dosage of cement-kaolin contents.

3.5 Sources of Data

Both primary and secondary data sources were used. The Primary sources of data for this study were a laboratory experimental outputs and Secondary data were collected from different standards, journals, book, website and others.

3.6 Materials for Laboratory Tests

Kaolin: for this study kaolin were taken from Tabor ceramic factory, in which the factory uses the material from Bombowha kaolin deposit located in Southern Ethiopia, Borena Zone, Bore District, at the locality called Bombowha.

Figure 3.3: Pulverized dry and wet kaolin (Source: Jemal M., 02/08/2018)

Cement: Dangote- Ordinary Portland cement (OPC) whose Cement Grade 42.5R and specific gravity of 3.15 which was commercially available and used as stabilizer mixed with kaolin. There are different types of cement however for this study OPC was used, because it has a high sulfate resistance capacity [45].

Table 3.1: oxide content of ordinary Portland cement [46]

Oxides	CaO	SiO_2	AL_2O_3	Fe_2O_3	SO_3	MgO	Na_2O	K_2O
Ranges (%)	60-70	17-25	3-8	0.5-6	1-3	0.1-4	0.5-1.3	0.5-1.3

Expansive soil: For this study two samples were collected from Jimma Town around wuhafisash (kebele 05) along yetebaberut road segment which was under construction and the second sample were collected from Bosa Adiss ketema kebele. The collected samples for this study were disturbed samples.

3.7 Sample collection

Generally, in this study five soil samples were taken from different locations in Jimma town. The selection of soil samples for laboratory evaluation was carried out based on information obtained from previous researches about the quality of predominant soil types in the area [47].Moreover, Based on observation and free swell test, two expansive soil samples were selected from, kebele 05, wuhafisash-yetebaberut road segment which was under construction with high degree of expansion properties and the second sample was collected from Bosa Adiss ketema kebele with medium degree of expansion properties. The excavation was made using excavator and shovel. According to ERA manual, the collected samples for this study were disturbed samples at a depth of below 1.50 m to remove organic matter.

Figure 3.4: Photo of soil sample collection (Source: Jemal M., 21/06/2018)

3.8 Laboratory tests

Tests for soil classification which included grain size distribution, free swell, specific gravity, and Atterberg limits. These are indicative tests that are usually used for identifying whether the soil is expansive or not. The conducted tests however included hydrometer analysis, Atterberg limits, wet sieve analysis, specific gravity, moisture density relation, free swell index, linear shrinkage, CBR, and CBR swell to fully characterize and attain the objective of the research.

3.8.1 Expansive soil

3.8.1.1 Grain Size Analysis

This test was performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis was performed to determine the distribution of the coarser, larger-sized particles, and hydrometer method was used to determine the distribution of finer particles. For this study both wet sieve analysis and hydrometer analysis was done according to AASHTO T 088-93.

3.8.1.2 Atterberg Limit test

The test procedure adapted for the determination of Liquid limit, Plastic Limit and plasticity index for both untreated and treated soil sample was in accordance with AASHTO T89-94 and T90-94 respectively. A sample weighting about 250gm was taken from the mixture prepared for liquid limit and plastic limit test for each samples. Soil samples were first air dried and pulverized and then sieved with number 40 sieve. Soil passing number 40 sieve was mixed with different proportion of kaolin-cement content at optimum water content and sealed with plastic for 24 hours in order to give sufficient time for chemical reaction before test. Hand mixing in a porcelain pan was the method of mixing. The liquid limit of each soil had been determined by using casagrande apparatus. The plastic limit of each soil was determined by using soil passing through a 475 µm sieve and rolling 3-mm diameter threads of soil until they began to crack. The plasticity index was then computed for each soil based on the liquid and plastic limit obtained.

Figure 3.5: Atterberg limit determination (Source: Tariku A., 02/08/2018)

3.8.1.3 Free Swell Index Test

The test includes the determination of the free swell for the natural soil and soil-cement-kaolin mixture. This test has not yet been standardized by AASHTO and ASTM. The method was suggested by Holtz and Gibbs (1956) and Indian standard IS 2720 (part XL) to measure the expansive potential of cohesive soils. But, in this research Indian standard IS 2720 (part XL) was used. The free swell test gives a fair approximation of the degree of expansiveness of the soil sample. The procedure involves in taking two oven dried soil samples passing through the 425µm sieve, 10g each was placed separately in two 100ml graduated soil sample. Distilled water was filled with one cylinder and kerosene in the other cylinder up to

100ml mark. The final volume of soil is computed after 24 hours to calculate the free swell index.

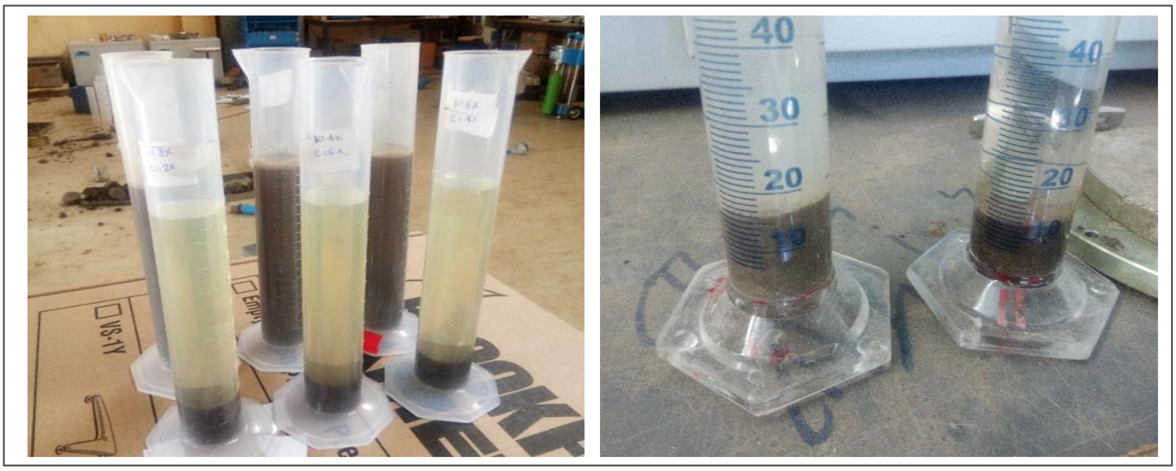

Figure 3.6: Free Swell Index test (Source: Jemal M., 25/07/2018)

3.8.1.4 Linear Shrinkage

Linear shrinkage test followed a British standard (BS1377: Part 2:1990), and covers the determination of total linear shrinkage from linear measurement on a standard bar of length 140 mm with a semicircular section of diameter 25 mm, the grove filled by a soil of the fraction passing 0.425 mm test sieve, originally having the moisture content of the liquid limit.

$$\text{Linear shrinkage} = \frac{\text{Initial length} - \text{oven dried length of specimen}}{\text{Initial length}} \times 100 \ldots\ldots\ldots..(3.1)$$

The linear shrinkage value is the way of quantifying the amount of shrinkage likely to be experienced by clayey material.

Figure 3.7: Linear Shrinkage Test (Source: Haile T., 05/08/2018)

3.8.1.5 Specific Gravity Test

Specific gravity measures of heaviness of the soil particles. The test includes the determination of the specific gravity for the natural soil. The importance of determining the specific gravity in this study was to determine particle sizes in hydrometer analysis. The specific gravity test was conducted on the soil in accordance with ASTM D 854-98 testing procedure.

Figure 3.8: Specific gravity Test (Source: Tariku A., 28/07/2018)

3.8.1.6 Soil Classification

The most widely used soil classification systems are AASHTO and USCS systems. The AASHTO Classification system is useful for classifying soils for high way. On this research each Soil was classified using the AASHTO Soil Classification System using particle size distribution and Atterberg limits. Soil classification is the arrangement of soils into different group in order that the soils in a particular group would have similar behavior. The method of classification used in this study was the AASHTO M-145 System.

3.8.1.7 Compaction Test

This laboratory test was conducted to determine optimum water content at maximum dry density of soil. Compaction is when mechanical loads applied to soil result in expulsion of air, increase in bulk density and resistance to penetration. The laboratory standard proctor test was performed as per AASHTO T 99-95. The test was performed on disturbed samples of soil passing sieve sizes 4.75mm or 19mm mixed with water to form samples at various moisture contents ranging from the dry state to wet state. These samples were compacted in three layers at 25 blows per layer in accordance with the specified nominal compaction energy of standard proctor test. Dry density was determined based on the moisture content and the unit weight of compacted soil. The corresponding water content at which the maximum dry density occurs is termed as the optimum moisture content.

Figure 3.9: Compaction test and procedures (Source: Molla A., 16/08/2018)

3.8.1.8 California Bearing Ratio Test (AASHTO T-193)

The CBR is expressed by force exerted by the plunger and the depth of its penetration into the specimen; it is aimed at determining the relationship between force and penetration. A three point CBR test at 10, 30 and 65 blows were conducted according to AASHTO T193 and the CBR values at 95% MDD was determined. The CBR test indirectly measures the shearing resistance of a soil under controlled moisture and density conditions. The CBR is obtained as the ratio of load required to affect a certain depth of penetration of a standard penetration piston into a compacted specimen of the soil at some water content and density to the standard load required to obtain the same depth of penetration on a standard sample of crushed stone. The equation to be computing the CBR value is as follows.

$$\mathrm{CBR}(\%) = \frac{\text{Applied load on sample}}{\text{standard load on the crushed stone}} \times 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.\,.\ldots\ldots\ldots.\,.\,(3.2)$$

For 2.54 mm Penetration = 6.9 MPa and for 5.08mm penetration = 10.3 MPa. The required quantity of soil, kaolin, cement and water for one specimen were calculated using bulk density and moisture content determined from Proctor Test and the total quantity of each needed to prepare the required number of test specimens at each prescribed stabilizers percentage of maximum dry unit weight and water content was known.

Figure 3.10: CBR test procedures (Source: Haile T., 23/08/2018)

3.8.1.9 California Bearing Ratio (CBR)Swell Test

CBR tests were conducted on the compacted specimens at the optimum moisture content using standard compaction test. A three point CBR test at 10, 30 and 65 blows were conducted according to AASHTO T193 and the CBR values at 95% MDD was determined. The compacted soil samples of the CBR mold were soaked for 96 hours in a water bath to get the soaked CBR value and the CBR swell of the soil. The CBR swell of the soil was measured by placing the tripod with the dial indicator on the top of the soaked CBR mold. The initial dial reading of the dial indicator on the soaked CBR mold was taken just after soaking the sample. At the end of 96 hours the final dial reading of the dial indicator was taken hence the swell percentage of the initial sample length is given by:

$$\text{CBR swell}(\%) = \frac{\text{change in length in mm during soaking}}{116.30\text{mm}} \times 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (3.3)$$

3.8.2 Kaolin

Investigations were made on kaolin obtained from Tabor Ceramics factory in which the factory uses the material from Bombowha kaolin deposit. It was tested for physical properties per AASHTO and fineness properties as per ASTM C 618.The chemical and physical properties of the kaolin used in this research were listed in Table 4.1 and Table 4.2 respectively.

3.9 Symbolization

For this study sample collected from Wuhafisash- Yetebaberut road segment was abbreviated as WYR and soil sample from Bosa Adiss ketema Kebele was abbreviated as BK. Additionally, kaolin and cement were also abbreviated as K and C respectively.

CHAPTER FOUR

RESULT AND DISCUSSION

4.1 Introduction

This chapter presents test results, discussion and analysis of all experimental work that were performed on untreated and treated soil samples with cement and Kaolin combination mixtures. Primarily, properties of materials (untreated soil, and kaolin) were examined, then the effect of stabilizers on Atterberg limits, moisture-density relation, linear shrinkage, free swell index, CBR, and CBR swell values were investigated by varying percentage of stabilizers from 2% to 10% by 2% increment and compared with native soil/untreated soil engineering properties. Then effect of stabilizers on the properties of treated soil was compared and contrasted with standard specification and manuals.

4.2 Properties of Materials

4.2.1 Kaolin

4.2.1.1 Chemical properties

The chemical composition carried out on kaolin was shown in Table 4.1. The results indicate pozzolanity of the kaolin. The combined percent composition of Al_2O_3, SiO_2 and Fe_2O_3 was more than 70%. This was adequate to meet the requirement of ASTM C618 standard for pozzolanic materials.

Table 4.1: Oxide composition of kaolin

Chemical Composition	Test Results (%)	Requirement ASTM C-618 (%)	Result status
SiO_2	47.74	35 and above	In range
Al_2O_3	35.00		
Fe_2O_3	0.84		
(SiO_2 + AI_2O_3 + Fe_2O_3)	83.58	70 and above	In range
MgO	0.10	5 and below	In range
Moisture Content (H_2O)	0.84	3 and below	In range
CaO	0.16		
Na_2O	0.47		
K_2O	1.28		

4.2.1.2 Physical properties

Kaolin is a soft, lightweight and often chalk-like sedimentary rock. It has an earthy odor and feel when touched with plate-like crystal morphology. The atterberg limits, moisture density relationship, and specific gravity of Kaolin were determined in the laboratory. The physical properties of Kaolin clay were summarized in Table 4.2.

Table 4.2: Physical properties test results of kaolin

Properties	**Symbol**	**Test result**
Liquid Limit, (%)	LL	38.05
Plastic Limit, (%)	PL	27.20
Plasticity index, (%)	PI	10.85
Maximum Dry Density, (g/cm^3)	MDD	1.42
Optimum Moisture Content, (%)	OMC	27.25
Specific Gravity	Gs	2.61
Residue on 45 micron	-	30

4.2.2 Properties of untreated soils

In order to determine the quality of the materials, laboratory tests were carried out on both WYR and BK untreated soil samples. The results of the laboratory tests conducted for identification and determination of the engineering properties of the untreated soil before mixing with cement and kaolin were presented in table 4.3.

Table 4.3: General Geotechnical properties of both soil samples

Parameters	**Laboratory Results (%)**	
	WYR soil sample	**BK soil sample**
Percentage of passing No.200sieve	95.95	96.41
Liquid limit (%)	109	93
Plastic limit (%)	41	37
Plasticity index (%)	68	56
Linear Shrinkage (%)	23.07	19.36
AASHTO classification system	A-7-5(81)	A-7-5(65)
USCS	CH	CH
Specific Gravity	2.72	2.70
Free swell index, (%)	122	64
Maximum dry density, (g/cm3)	1.276	1.313
Optimum moisture content, (%)	34.931	33.176
Soaked CBR value, (%)	0.98	2.02
CBR-swell, (%)	5.11	3.21
Color	Black	Dark Gray

4.2.2.1 Grain size analysis

The distribution of different grain sizes affects the engineering properties of the given soil. Grain size analysis provides the grain size distributions, and it was required in classifying the soil. In this study, for coarse-grained soils wet sieve analysis and for fine-grained soils hydrometer analysis was used. Wet sieve analysis and hydrometer analysis, using sodium hexametaphosphate as dispersing agent, were performed on both samples and a plot of percent finer against soil grain size (sieve size) in millimeter on a semi-logarithm scale was plotted. The results were given in Figure 4.1, Figure 4.2, and table 4.3. The detailed grain size analysis test results are attached in Appendix E.

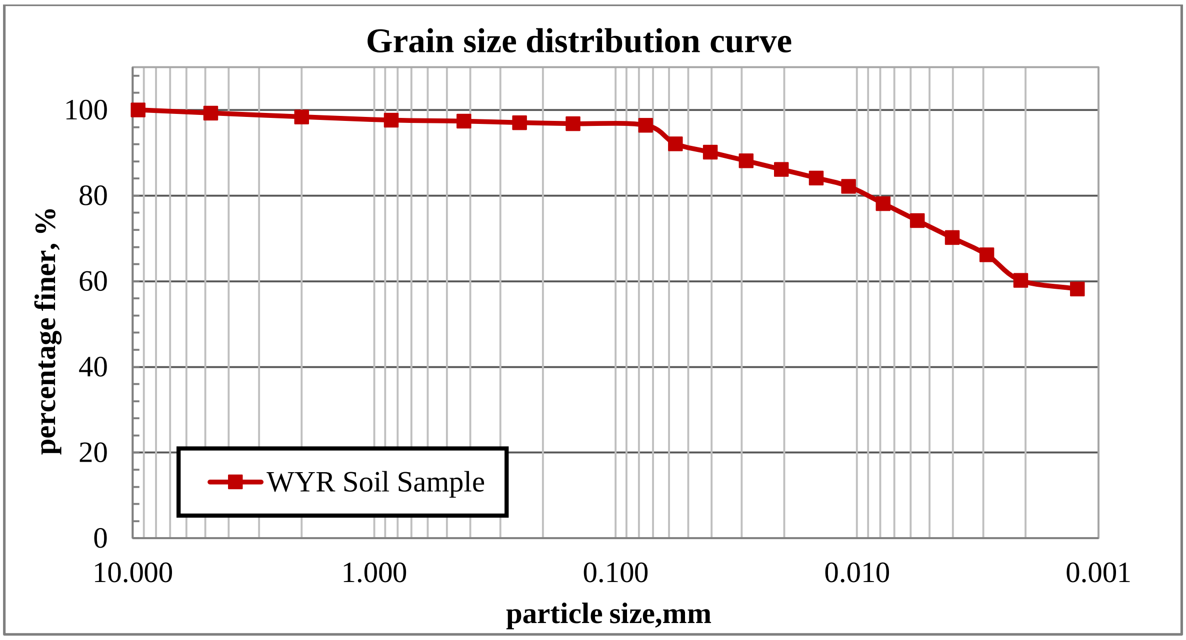

Figure 4.1: Grain size distribution curve of WYR untreated soil samples

The soil sample from WYR was black in color, and 95.95 % of the soil was passing through No.200 sieve(75µm), this indicates that, almost the given soil sample was clay soil.

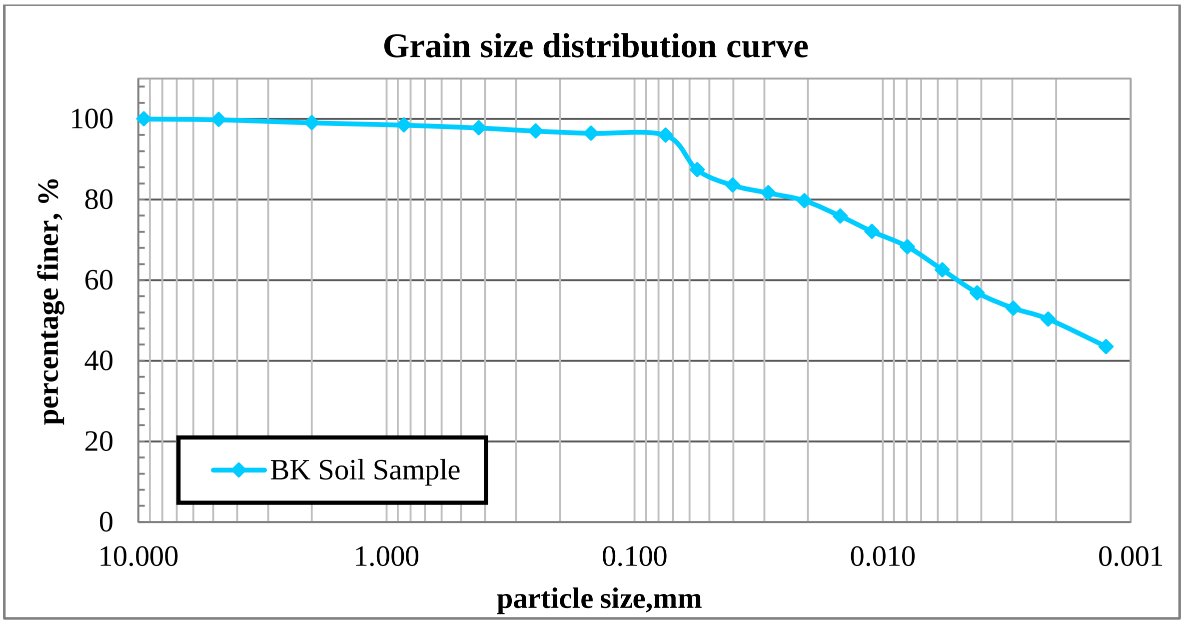

Figure 4. 2: grain size distribution curve of BK untreated soil samples

The soil sample from BK was dark gray in color, and 96.41 % of the soil is passing through No.200 sieve(75µm), this indicates that, almost the given soil sample was a clay soil.

4.2.2.2 Atterberg's Limits

The nature and response of soil upon change to moisture content is determined by Atterberg limit tests. Following the AASHTO procedure, designation AASHTO T89-96 and T90-00, the soil samples obtained from WYR and BK were subjected to varying water content and as a result the liquid limit, plastic limit and plastic index of the untreated sample as recorded in Table 4.4 were determined. The laboratory data analysis was attached in Appendix A.

Table 4.4: Atterberg's Limit test result for untreated samples

Sample location	Atterberg limits		
	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)
WYR	109	41	68
BK	93	37	56

Plasticity index represents the range in water content through which a soil is in plastic state. A high numerical value of plasticity index is an indication of the presence of high percentage of clay in the soil sample.

4.2.2.3 Soil Classification

According to unified soil classification system as shown in table 4.5, and figure 4.3, both WYR and BK soil sample lie above the A- line in CH region, which means clayey soil with high plasticity.

Table 4.5: Classification of soils based on USCS classification system

Sample location	Min. Sampling depth (m)	Quantity of grain size (%)				LL (%)	PI (%)	USCS Classification
		Gravel	Sand	Silt	Clay			
WYR	1.50	0.73	2.86	36.21	60.20	109	68	CH
BK	1.50	0.19	3.86	45.65	50.30	63	56	CH

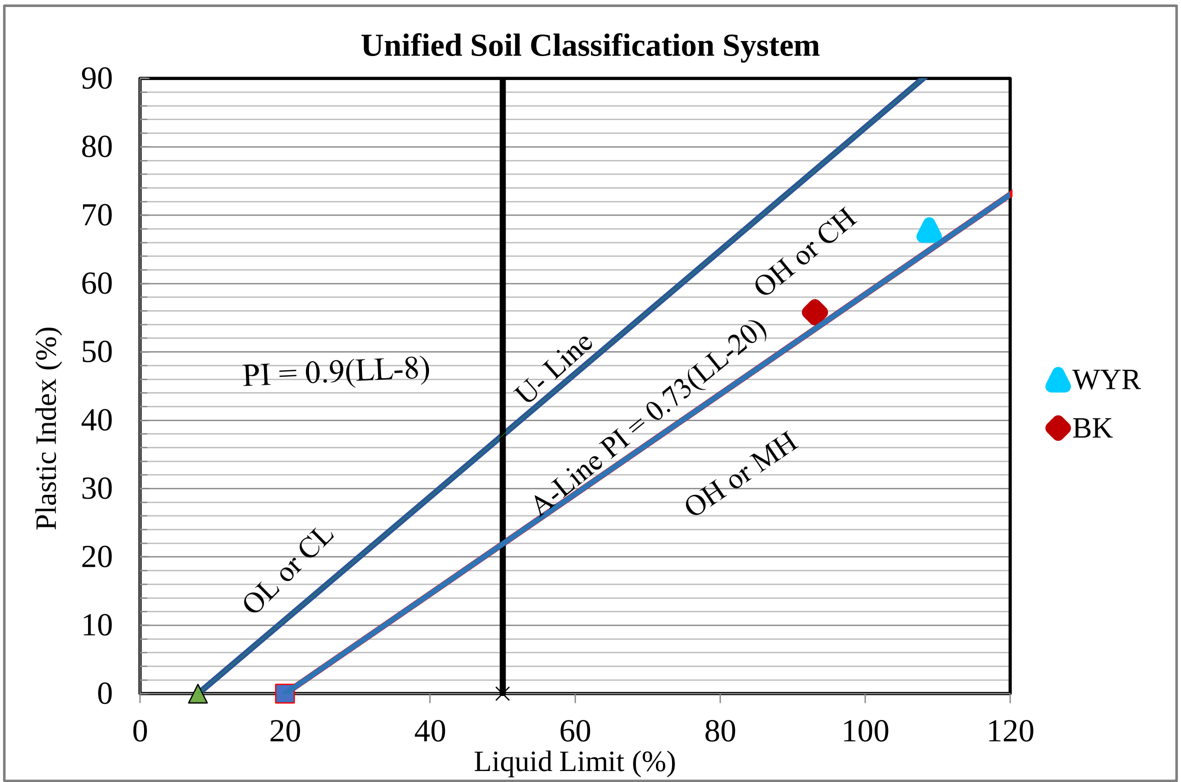

Figure 4.3: Plasticity chart of untreated soil samples according to USCS

From AASHTO Classification system results shown in table 4.6 and Figure 4.4 it can be concluded that both WYR and BK soil samples fall under A-7-5, which were clayey soils with group index of 81 and 65 respectively. The group index results indicate that generally the soils of the study area were very poor for subgrade material.

Table 4.6: Classification of soils based on AASHTO classification system

Sample Name	Sieve Analysis Percentage of Passing			LL (%)	PI (%)	LL-30	Group Index	Soil Group	Material Type
	No.10	No.40	No.200						
WYR	98.38	97..36	96.41	109	68	79	81	A-7-5	Clay
BK	99.05	97.78	95.95	93	56	63	65	A-7-5	Clay

the lower the GI value of a soil, the better as sub grade material. As GI value goes up to 20 and above it is not suitable as sub grade material [69].

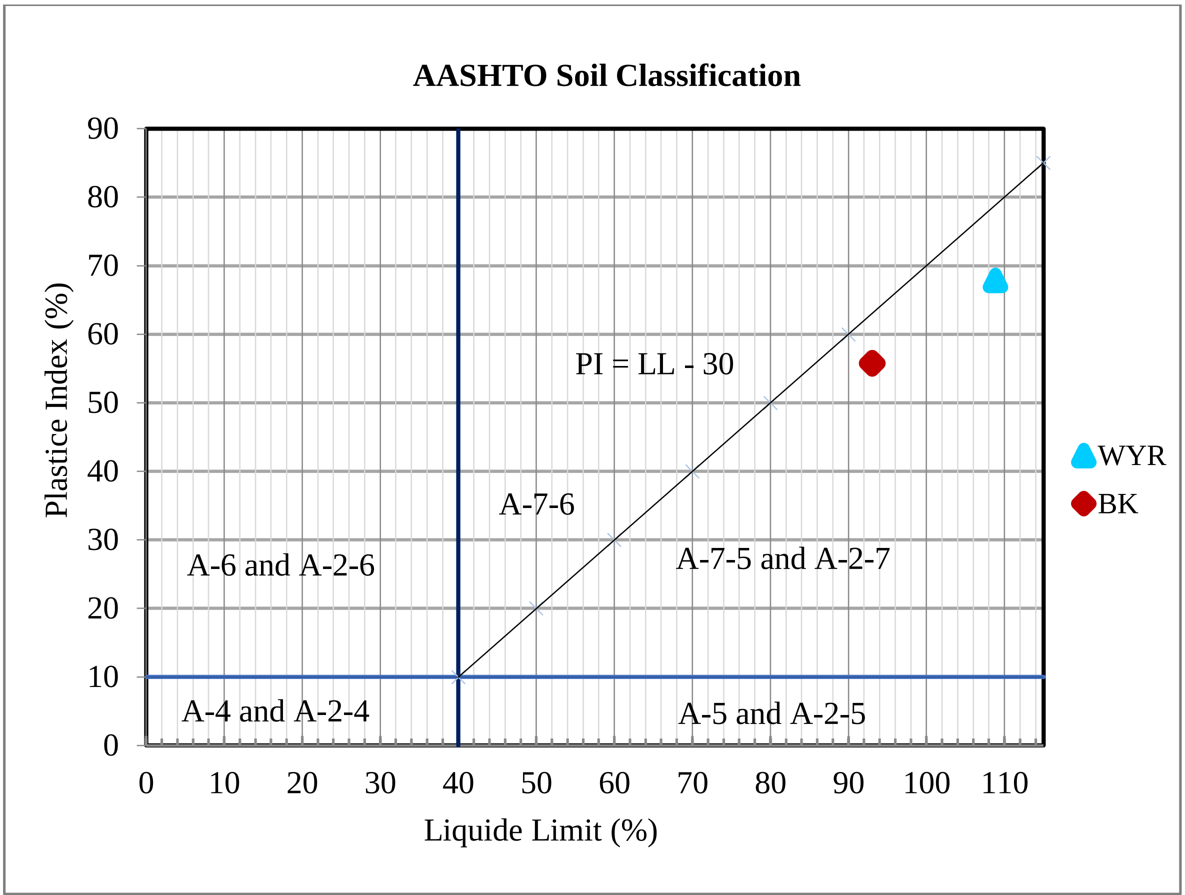

Figure 4.4: Plasticity chart of untreated soil samples according to AASHTO

4.2.2.4 Specific Gravity

Specific gravity which is the measure of heaviness of the soil particles was determined by the method of pycnometer method using a soil sample passing number 10 sieve and oven dried at 105^{o}C. The test was conducted in accordance with AASHTO T100-95 testing procedure. The laboratory test results of both soil samples were summarized in table 4.7 and The laboratory data analysis was attached in Appendix D.

Table 4.7: Specific Gravity of untreated Soil Samples

Sample Location	WYR			BK		
Trial Number	1	2	3	1	2	3
Pycnometer Code	C	B	A	A	B	C
Specific gravity at 20oc	2.71	2.75	2.69	2.71	2.72	2.68
Average Specific gravity at 20oc, Gs	2.72			2.70		

4.2.2.5 Linear Shrinkage Test

This test was conducted to determine the linear shrinkage of the drying soil. Linear shrinkage is the reduction in the length of the sample when completely dries. The linear shrinkage test was conducted on the treated and untreated soil. Results of the Linear Shrinkage Test of the untreated soil sample was given in Table 4.8 and appendix B.

Table 4.8: Linear Shrinkage test results of the study area

Sample Location	Linear Shrinkage (%)
WYR	23.07
BK	19.36

4.2.2.6 Free swell index test

The free swell test is one of the most commonly used simple tests for estimating soil swelling potential. Results of the free swell tests of the soil was given in Table 4.9.

Table 4.9: Free swell test results of the study area

Sample Location	FSI (%)
WYR	122
BK	64

The free swell index value of both soil samples exceeds 50%, and such soils undergo volumetric changes leading to pavement distortion, cracking and general unevenness due to seasonal wetting and drying.

4.2.2.7 Compaction Test

To determine the maximum dry density and optimum moisture content of the untreated soil samples, standard Proctor compaction test has been conducted according to AASHTO T-99.

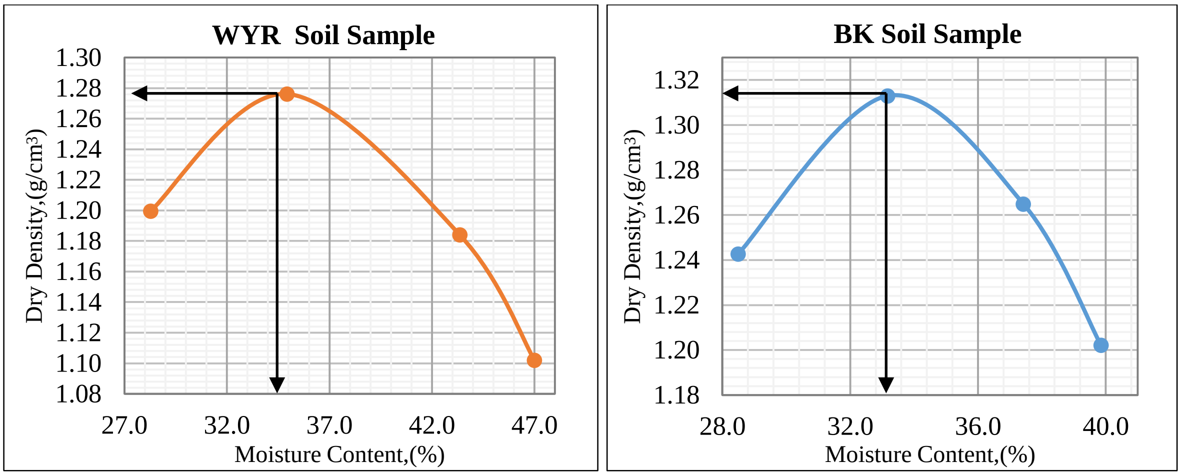

Figure 4.5: Density-Moisture Content Relationship for untreated soil samples

The WYR soil sample has a maximum dry density and optimum moisture content of 1.276 g/cm^3 and 34.931 % respectively. Similarly, The BK soil sample has a maximum dry density and optimum moisture content of 1.313 g/cm^3 and 33.176 %. detailed laboratory data was attached as an Appendix C.

4.2.2.8 Soaked California Bearing Ratio (CBR) and CBR swell Tests

The CBR value of both WYR and BK untreated soil samples were shown in the figure 4.6.

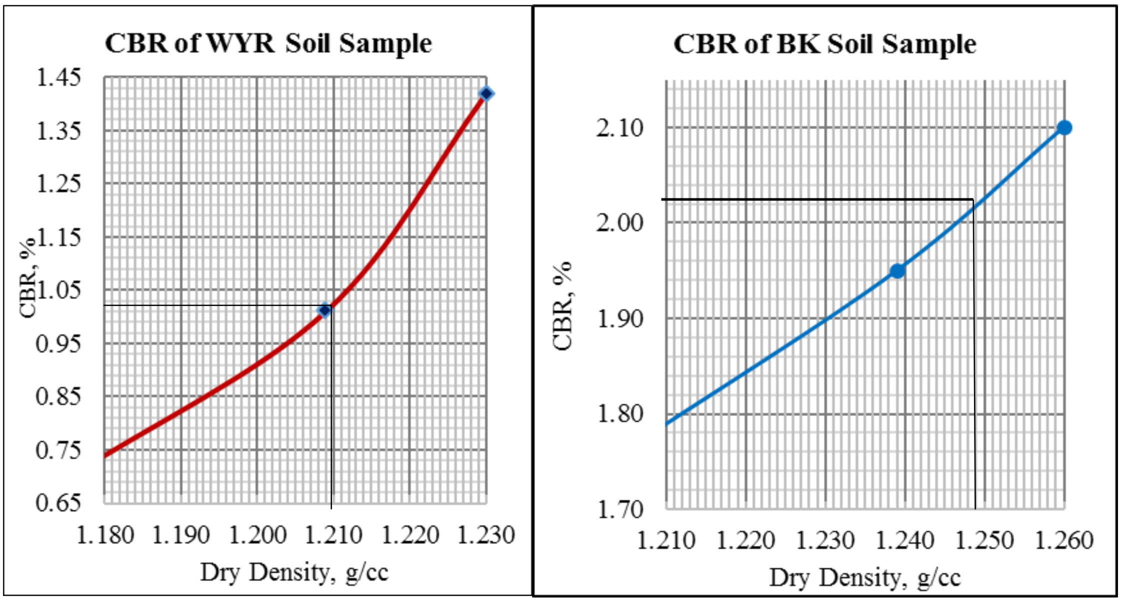

Figure 4.6: CBR test result of the WYR and BK soil samples

As shown in figure 4.6, WYR soil sample had 0.98% CBR value at maximum dry density with 5.11% CBR swell and BK soil sample had 2.02% CBR value with 3.21% CBR swell. The test result showed that both soil samples has low CBR value, which does not satisfy the minimum requirements as sub-grade material. According to ERA standard specification a CBR value of less than 3% special treatment is required.

4.3 Laboratory test results of stabilized expansive soil

4.3.1 The effect of addition of Kaolin-Cement on Atterberg's limit

The effect of kaolin-cement addition in varying proportion with natural expansive soil had been studied and the variation in consistency limit for various additive mix-ratio were presented in Table 4.10. It was found that as the percentage of additive content increases the liquid limit decreases on the other hand the plastic limit increases. As a result, the plasticity index also decreased followed with increase in additives content. The summary of the laboratory test result was analyzed and given in table 4.10.

Table 4.10: Effect of Kaolin-cement content addition on Atterberg's limit

Sample Location	Mix-Proportion of additives (%)		Atterberg's Limit (%)		
	Kaolin	Cement	Liquid Limit	Plastic Limit	Plasticity Index
WYR	0	0	109	41	68
	10	0	107	54	53
	8	2	98	58	40
	6	4	92	63	29
	4	6	87	66	21
	2	8	84	67	16
	0	10	81	66	15
BK	0	0	93	37	56
	10	0	86	42	44
	8	2	80	48	33
	6	4	76	52	24
	4	6	74	57	18
	2	8	74	60	13
	0	10	73	61	12

As shown in Table 4.10, The Liquid limit decreases from control value 109%-81% and 93%-73% for WYR and BK soil sample respectively. The Atterberg limit depends on the type of predominant clay mineral available in the soil mass. If the predominant clay is montmorillonite the liquid limit can reach or even exceed 100%. It is also expected that the Atterberg limit is less for illite dominated soil and even lesser for kaolinite dominated soils. However, the additives not shown significant change on liquid limit of the soil because the dispersing effect of the additive doesn't affect the liquidity natures of the soil but its plastic limit only.

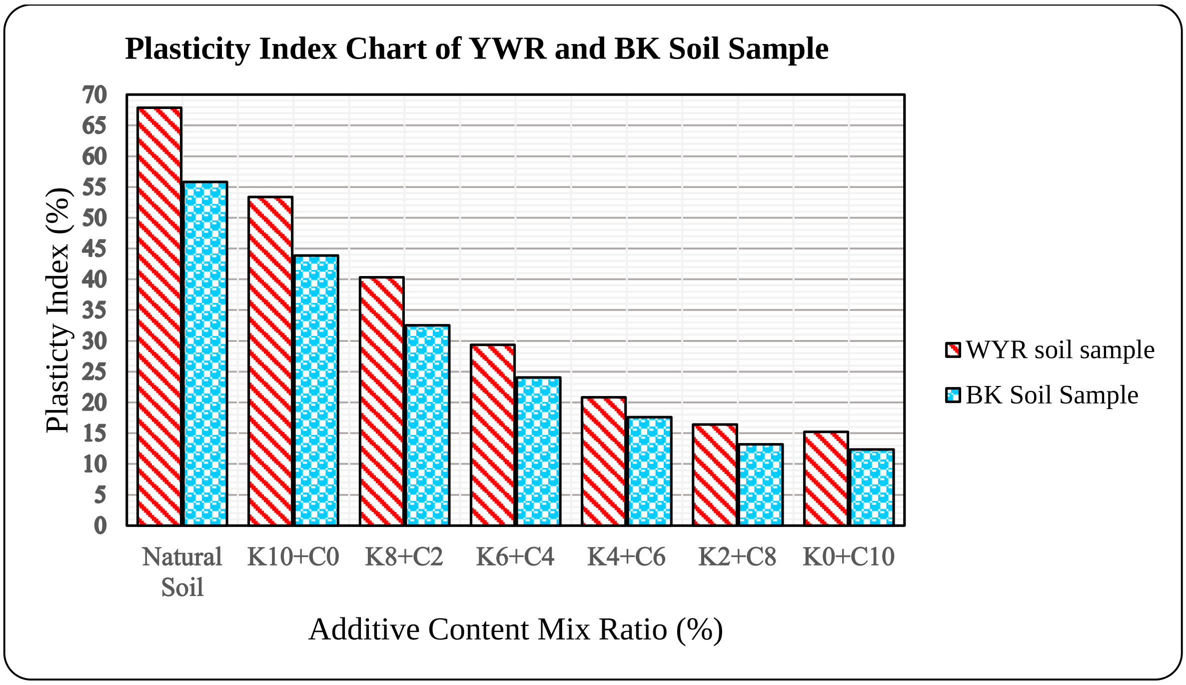

Figure 4.7: Effect of addition of Kaolin-Cement on PI of WYR and BK soil samples

From Figure 4.7, it indicated that highest reduction in plasticity index was observed when adding the maximum ratio of cement and minimum ratio of kaolin. Sodium in comparison with calcium as the exchangeable cation would be expected to reduce particle attractions resulting in lower values for the liquid limit. Due to this reason, the content of calcium ions in kaolin was not enough to replace the sodium ion in soil particle montmorillonite, therefore it is necessary confident amount of calcium ion from cement in anticipation of replacing the sodium montmorillonite.

The Plastic limits increased with additive contents of both Mix-ratio increased. However, significantly increment was shown when the mix-ratio of cement was higher rather kaolin. The plasticity index also decreased with additives content of both Mix-ratio increases, though, the percent of reduction was higher when the percentage of cement increases rather than kaolin.

Generally, Addition kaolin-cement have shown significant reduction in plasticity index and modest change in liquid limit of both soil samples. According to chen (1988), degree of expansion is not critical when value of PI and LL lower than 35% and 60% respectively and can be as a subgrade material.

4.3.2 The effect of addition of Kaolin-Cement on Linear Shrinkage

According to Altmeyer (1955), Soils having linear shrinkage value above 8%, between 5% and 8%, and less than 5% possess critical, marginal, and non-critical degree of expansion respectively. The laboratory result of linear shrinkage was presented on table 4.11 and figure 4.8. Increment of additive content percentage, especially when the ratio of cement was higher than kaolin, the linear shrinkage value has reduced significantly. So the additive contents were effective to reduce the volume change when exposed to variable humidity and whether condition.

Table 4.11: Effect of addition of cement-kaolin on linear shrinkage

Sample Location	**Mix-Proportion of additives (%)**		**Linear Shrinkage (%)**	**Degree of Expansion according to Altmeyer (1955)**
	Kaolin	**Cement**		
WYR	0	0	23.07	Critical
	10	0	21.36	Critical
	8	2	17.14	Critical
	6	4	10.00	Critical
	4	6	8.29	Critical
	2	8	4.64	Non-critical
	0	10	4.29	Non-critical
BK	0	0	19.36	Critical
	10	0	18.36	Critical
	8	2	11.21	Critical
	6	4	8.29	Critical
	4	6	6.00	Marginal
	2	8	2.50	Non-critical
	0	10	2.36	Non-critical

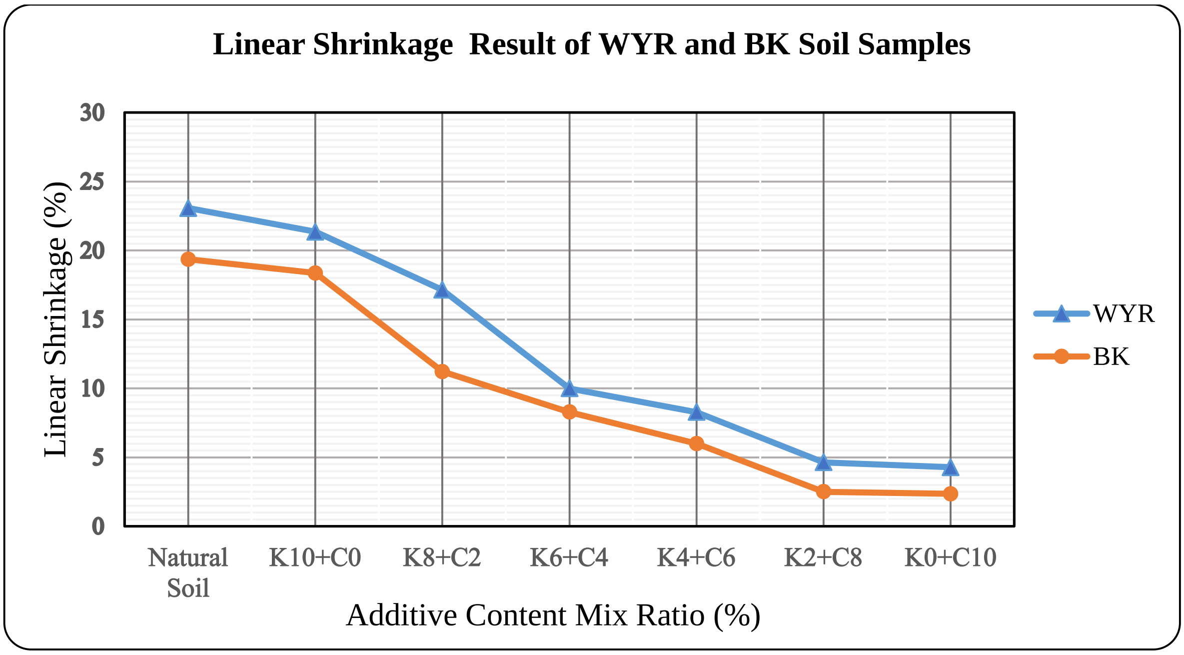

Figure 4.8: Effect of addition of kaolin-cement content on linear shrinkage

As shown in Figure 4.8 and table 4.11, The average linear shrinkage for both WYR and BK native soils were under critical degree of expansion with 23.07% and 19.36% respectively. For WYR soil sample, both 10% cement alone and K2%+C8% has significantly improved the native soil sample into non critical stage degree of expansion. The rest mix-proportion were not effective to arrest the shrinkage behavior of the native soil which was under critical degree of Expansion. Similarly, 10% cement alone, K2%+C8%, and K4%+C6% reduced the critical degree of expansion of BK soil sample in to non-critical and marginal degree of expansion. The linear shrinkage has been decreased with increase in kaolin-cement ration for both samples.

4.3.3 The effect of addition of Kaolin-Cement on Free Swell Index

According to Indian Standard (IS 1498), Soils having a free swell value above 100 can cause damage whereas free swell as low as 100 percent can cause considerable damage to light loaded structures and soils having a free swell value below 50 percent seldom exhibits appreciable volume change even under light loads. The effect of kaolin-cement on the free swell index of the treated soil sample was tabulated in the table 4.12 and plotted in figure 4.9.

Table 4.12: Effect of addition of cement-kaolin on free swell index

Sample Location	Mix-Proportion of additives (%)		FSI (%)	Percentage of reduction (%)	IS 1498 requirement	Test Result Status
	Kaolin	Cement				
WYR	0	0	122.00	0.00	FSI < 50%	Control
	10	0	111.10	10.90		Poor
	8	2	96.20	25.80		Poor
	6	4	73.10	48.90		Poor
	4	6	49.80	72.20		In range
	2	8	34.90	87.10		Satisfied
	0	10	35.00	87.00		Satisfied
BK	0	0	64.00	0.00	FSI < 50%	Control
	10	0	57.20	6.80		In range
	8	2	46.90	17.10		In range
	6	4	34.10	29.90		Satisfied
	4	6	22.30	41.70		Satisfied
	2	8	16.00	48.00		Satisfied
	0	10	16.20	47.80		Satisfied

For WYR soil sample, the highest reduction was attained when the sample was treated with K2%+ C8%, that means 87.10% reduction was observed from its natural state which was 122%. After the addition of 10% cement ratio, the result remains equivalent with the test result of K2%+C8%. Similarly, for BK soil sample the maximum reduction of 47% was observed when K2%+C8% was added and 47.80% reduction was observed after addition of 10% cement. But, further addition of cement ratio increases the free swell index. This indicates that, K2% +C8% was the optimum ration of additive content to achieve remarkable free swell index value.

Figure 4.9 and table 4.12 shows that both WYR and BK soil samples had reduced their swelling properties, due to chemical reaction and Cation exchange between the soil, water, kaolin and cement.

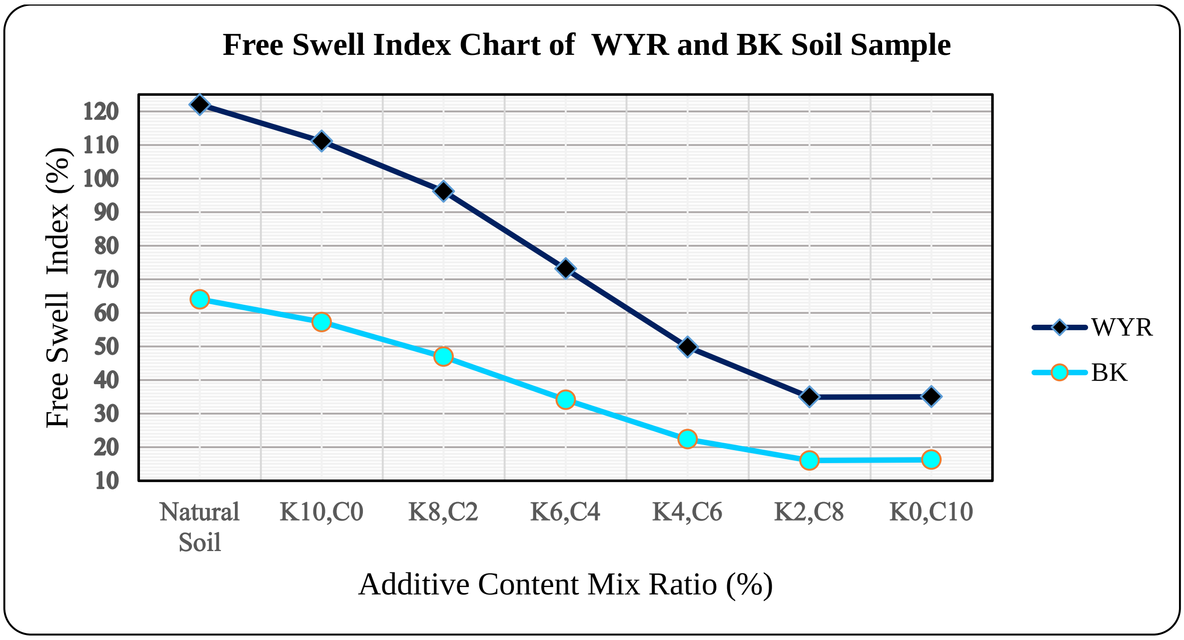

Figure 4.9: Effect of addition of kaolin-cement content on free swell index

4.3.4 The effect of addition of kaolin-cement on Moisture density relationships

The detail process of standard Procter compaction test of both soil samples was presented and plotted in table 4.13, figure 4.10, and figure 4.11. moreover, further laboratory test analysis data were illustrated in Appendix-C.

Table 4.13: Effect of Kaolin-cement content addition on Moisture Density Relation

WYR Soil Sample				BK Soil Sample			
Mix-Proportion of additives (%)		OMC (%)	MDD (gm/cm^3)	Mix-Proportion of additives (%)		OMC (%)	MDD (g/cm^3)
Kaolin	Cement			Kaolin	Cement		
0	0	34.931	1.276	0	0	33.176	1.313
10	0	36.784	1.253	10	0	33.978	1.300
8	2	37.784	1.233	8	2	35.176	1.283
6	4	39.612	1.223	6	4	36.400	1.270
4	6	43.176	1.213	4	6	37.253	1.259
2	8	46.707	1.199	2	8	39.253	1.229
0	10	47.824	1.194	0	10	40.400	1.210

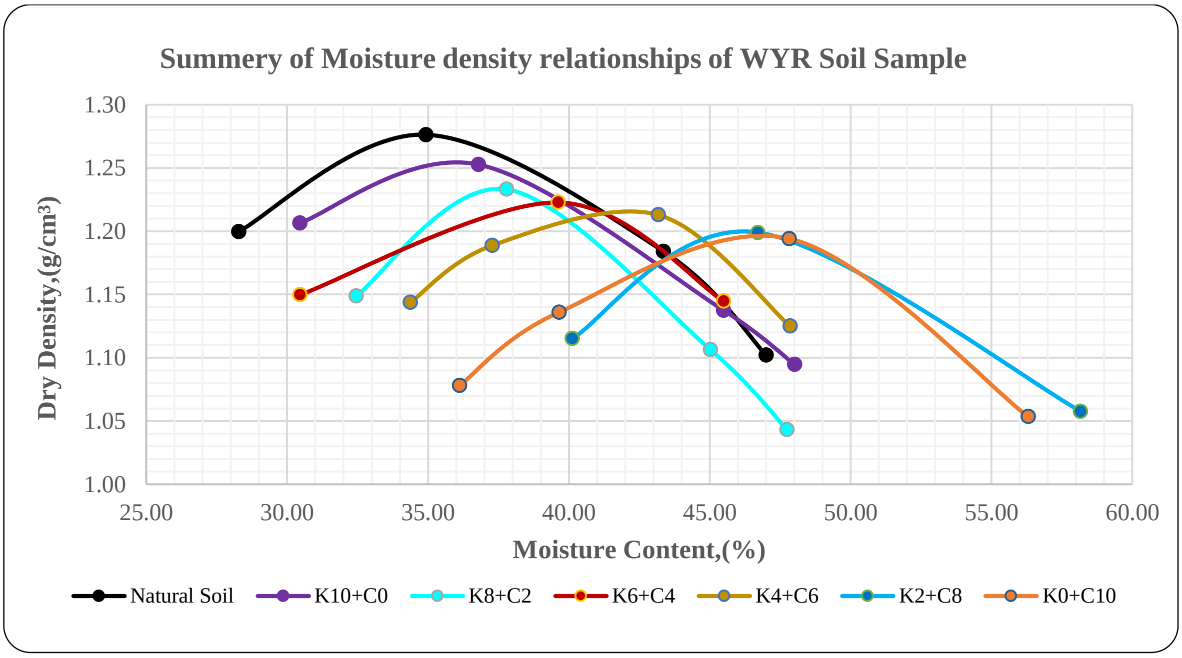

Figure 4.10: Summary of OMC and MDD of treated WYR soil sample

As shown in table 4.13, the MDD shows a slight reduction and OMC shows an increment in the treatment of WYR soil sample. The MDD decreases from 1.276 g/cm^3 to 1.194g/cm^3 and OMC increases from 34.93% to 47.82%. similarly, for BK soil sample, as shown in table 4.13 reduction in MDD from 1.313g/cm^3 to 1.210g/cm^3 and rise in OMC from 33.176% to 40.400% was observed.

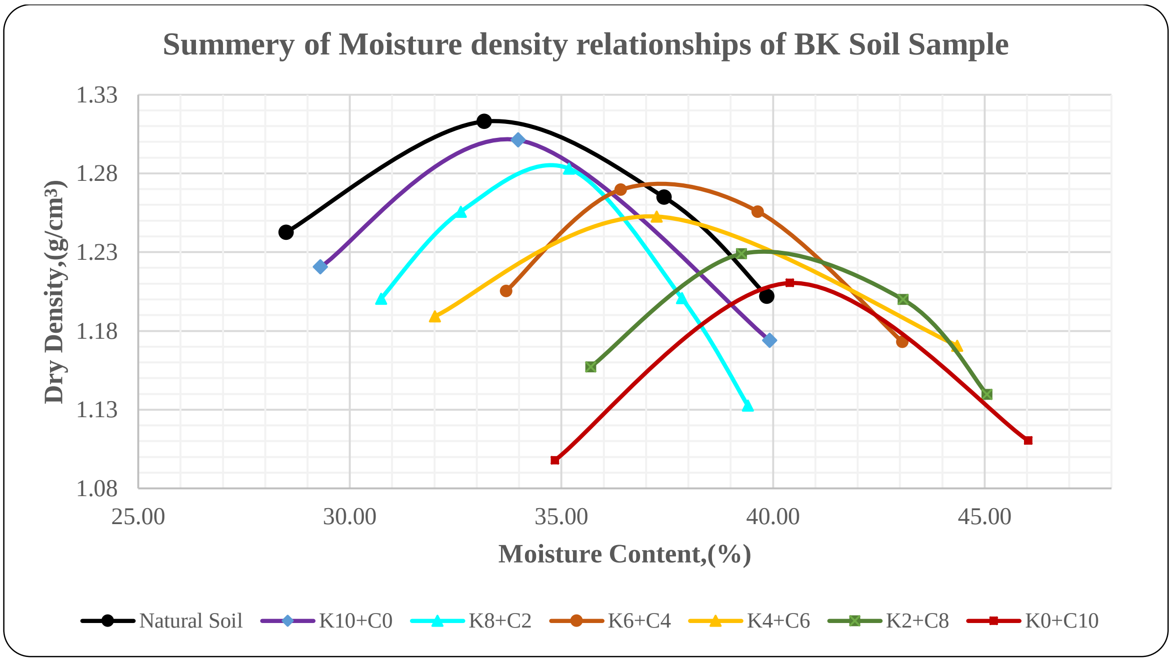

Figure 4.11: Summary of OMC and MDD of treated BK soil sample

The addition of kaolin-cement changes the optimum moisture content and maximum dry density of expansive soils because the effects of cation exchange and short-term pozzolanic reactions between cement and the soil results in flocculation and agglomeration of clay particles leading to texture changes. In addition to this, the decrease in density of all treated soils is mainly due to the partial replacement of comparatively heavy soil particles with the light weight and fine particle size of kaolin.

The advantage of the increase in OMC and corresponding decrease in MDD of the soil is that it allowed compaction to be easily achieved with wet soil. Any adverse effect on strength due to reduction in density is unlikely to occur due to the expected substantial gain in strength of treated soils due to the pozzolanic properties of cement and kaolin.

4.3.5 The effect of addition of kaolin-cement on CBR value

CBR is a parameter which is used to measure the strength of subgrade soil. The CBR value was determined after soil samples have been soaked in water for 96 hours. That means if soil is stabilized using sufficient amount of stabilizer and hardening occurs, the soaking acts as an efficient means of curing providing hydration and preventing carbonation resulting in higher strength than can be achieved in the field. The soaked CBR Test Result of both soil samples alongside ERA requirement was presented in figure 4.12, table 4.14, and 4.15 Detailed test results are given in Appendix-F.

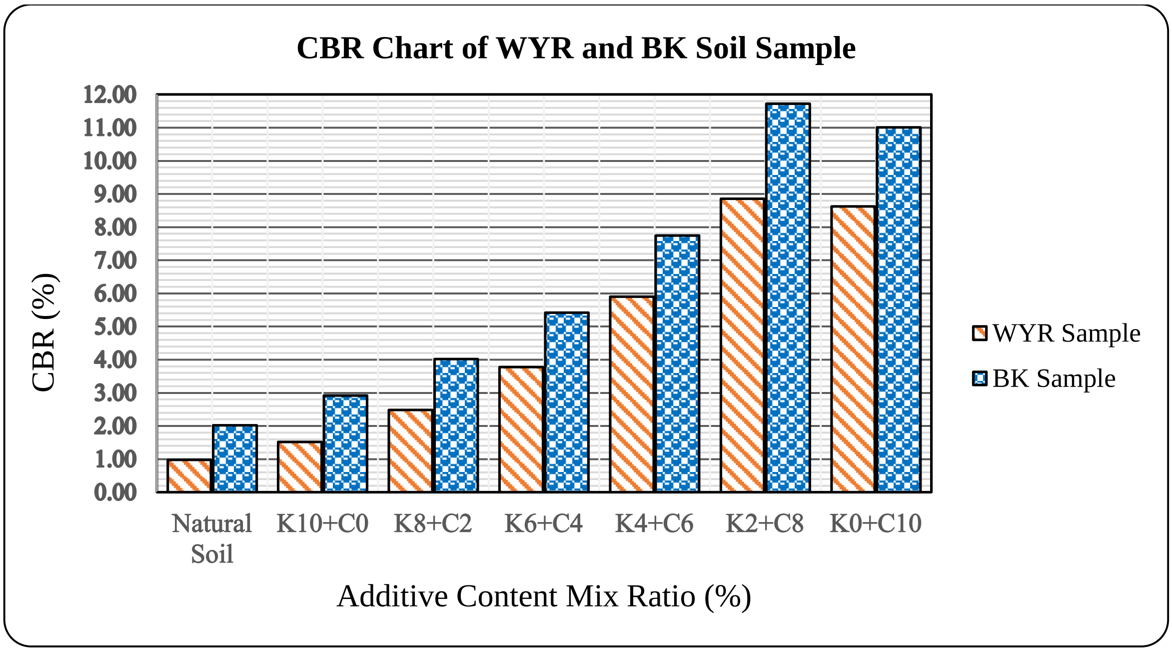

Figure 4.12: Summary of CBR test results of WYR and BK soil Samples

As shown from figure 4.12, WYR soil sample treated by 10% cement alone and K2%+C8% showed more improvement than kaolin alone and kaolin mixed with 2%,4%, and 6% of cement. Similarly, BK soil sample was significantly improved with additive content of 10% cement, and with K2%+C4%. The CBR increased with increasing cement content rather than kaolin content. The addition of kaolin and cement together led to a more increase of the CBR value compared to the addition of cement and kaolin separately. The combination of kaolin and cement can strongly improve the strength of expansive soils.

The increase in the CBR with the addition of cement and kaolin could be due to the presence of adequate amount of calcium required for the formation of Calcium Silicate Hydrate (CSH) and Calcium Aluminate Hydrate (CAH), which are the major compounds responsible for strength gain due to pozzolanic reaction, cation exchange reaction and adhesive properties of kaolin and cement.

Table 4.14: Summary of CBR Test results for WYR treated soil sample

Mix-ratio of additives(%)		CBR Value (%)						CBR@ 95% MDD	CBR Swell (%)	ERA Requirement	Subgrade Class
		10 blows		30 blows		65 blows					
Kaolin	Cement	2.54mm	5.08mm	2.54mm	5.08mm	2.54mm	5.08mm				
0	0	0.69	0.68	1.01	1.00	1.42	1.26	0.98	5.11	CBR > 3%	Control
10	0	1.09	1.02	1.41	1.26	1.80	1.50	1.52	4.35		S1
8	2	2.24	2.01	2.61	2.22	2.81	2.47	2.48	2.03		S1
6	4	3.28	2.40	3.86	2.80	4.24	3.36	3.78	1.65		S2
4	6	5.73	4.86	5.87	5.56	6.14	6.10	5.90	0.96		S3
2	8	8.06	7.37	8.64	8.00	9.22	8.93	8.86	0.62		S4
0	10	7.44	6.41	8.42	6.91	8.80	7.61	8.62	0.59		S4

Table 4.15 :Summary of CBR Test results for WYR treated soil sample

Mix-ratio of additives(%)		CBR Value (%)						CBR@ 95% MDD	CBR Swell (%)	ERA Requirement	Subgrade Class
		10 blows		30 blows		65 blows					
Kaolin	Cement	2.54mm	5.08mm	2.54mm	5.08mm	2.54mm	5.08mm				
0	0	1.76	1.46	1.95	1.56	2.10	1.79	2.02	3.21	CBR > 3%	Control
10	0	2.60	1.91	2.85	2.18	3.10	2.41	2.92	2.78		S1
8	2	3.36	3.01	3.91	3.46	4.44	4.10	4.02	1.38		S2
6	4	4.68	4.01	5.14	4.61	5.76	5.03	5.42	0.87		S3
4	6	7.33	5.20	7.61	5.75	8.00	6.24	7.75	0.45		S3
2	8	10.77	8.87	11.36	9.370	12.22	10.00	11.72	0.34		S4
0	10	10.42	8.91	10.88	9.41	11.39	10.01	11.01	0.31		S4

4.3.6 The effect of addition of kaolin-cement on CBR Swell Test

The kaolin-cement additive mixtures compacted in CBR molds at Optimum moisture content and maximum dry density gauged for swelling properties before and after soaking for four days to evaluate the percent of swell. The test result at different mix-ratio for both WYR and BK soil samples was plotted in table 4.14, table 4.15, and figure 4.13.

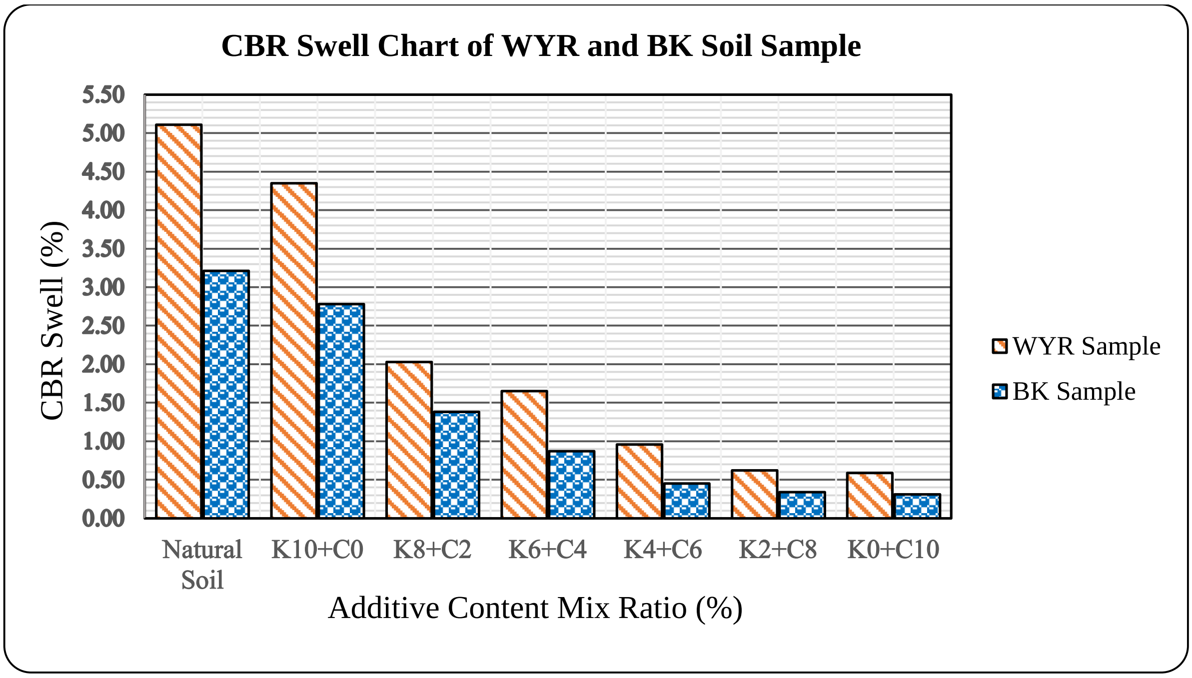

Figure 4.13: Summary of CBR Swell results of WYR and BK soil Samples

Figure 4.13, shows both WYR and BK untreated soil samples have the properties of swelling and potentially expansive soil. Though, when kaolin and cement was added with different mix-ratio the CBR swell value reduces. The reduction in CBR Swell was due to cation exchange and flocculation and agglomeration of the soil particles and variation in clay mineralogy of the expansive soils. This was happened due to replacement of some the volume that was previously occupied by expansive clay minerals (montomorillite and illite clay minerals) by kaolin. Using both the stabilizers improve the stability and strength of the subgrade soils.

The strength of subgrade is the principle factor in determining the thickness of the pavement, but deterioration due to frost action must also be taken into account.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

On the basis of the results obtained in the experimental investigation, the following conclusions have been drawn.

- The soil types were highly expansive and had high degree of expansion, high plastic index and poor strength. According to USCS and AASHTO classification system, WYR soil sample was categorized as CH and A-7-5 (81) and BK soil sample was categorized as CH and A-7-5 (65) respectively. Thus, the natural soil was very poor in strength to be used as a subgrade material as per ERA (2013) specification. The engineering properties of the studied expansive soil revealed that it was not suitable to use as a sub-grade material unless its undesirable properties are improved.
- The physical properties of kaolin were investigated and were found suitable for stabilization. Moreover, Kaolin satisfies the minimum requirement of Natural Pozzolan materials for use as a Mineral Admixture specified by ASTM (C 618-00) having the combined percentage chemical composition of main oxides (SiO_2 + Al_2O_3 + Fe_2O_3) of 83.58% which is satisfactory to encounter as pozzolanic material.
- The liquid limit, Plastic limit, and plastic index were Significantly improved to be in the range of subgrade material. The increment of cement-kaolin content in natural soil, the value of liquid limit and plastic limit was increased and plasticity index of treated natural soil was reduced satisfactorily. Plastic limits of both soil samples significantly increased when the percentage of cement was higher than kaolin in the mix-ratio. The plasticity index of both WYR and BK soil samples reduced from 67.87% to 15.22% and from 56.79% to 12.37% respectively at mix- ratio of K2% +C8%.
- Soil treated with kaolin has plastic nature while soil treated with cement has brittle nature. Addition of kaolin to cement reduced brittle nature of the soil.
- The values for the maximum dry density were noted to decrease with the addition of kaolin-cement content and the OMC was found to increase. However, the values for the maximum dry density was noted to decrease with higher cement percentage rather than kaolin in the mix-ratio.
- The addition of kaolin-cement additive content improved the CBR values of both WYR and BK soil samples. The improvement is more significant when the sample was cured because curing allows pozzolanic reactions. Hence, combination of kaolin and cement can strongly improve the strength of the expansive soil. As observed from the test result

performed under this study, the maximum value of CBR for both WYR and BK soil sample were achieved at K2% +C8% (with CBR value of 8.86% and 11.72%), and 10% cement (with CBR value of 8.62% and 11.01%) respectively.

- The results obtained during this investigation as discussed in the previous sections shows that, stabilized expansive soil with kaolin content alone does not bring significant change for using it as a sub-grade material. Therefore, kaolin is not an effective standalone stabilizer for highly plastic expansive soils because of lower calcium amount in the kaolin. However, kaolin in combination with cement can effectively improve the expansive soil. Therefore, expansive soil treated with kaolin combined with cement can be used as a good sub- grade material.

Generally, the most parameters of ERA (2013) specification requirement were and achieved the physical and Engineering properties of expansive soil were improved by kaolin combined with cement in different mix-proportion. The optimum amount for adequate stabilization was determined to be K2%+C8%. Furthermore, Addition of K2%+C8% yields almost the same result as 10% cement addition. Therefore, it was deduced that cement was partially replaced with 2% kaolin. Input cement was saved due to partial replacement of cement with 2% kaolin.

5.2 Recommendations

Based on the findings of this research, the following recommendations were forwarded:

- As investigated in this research work kaolin was not an effective standalone stabilizer for highly plastic expansive soils. Therefore, it is recommended to study potential use of kaolin as admixture stabilizer.
- As stabilization of expansive soil with cement and kaolin mixture is a relatively new concept and are scanty in the literature, chemical interactions and mechanisms involved in cement, kaolin, water and expansive soil shall be studied.
- Effect of curing period on soils treated with cement and kaolin combination shall be studied.
- The current study was conducted by taking limited parameter such as consistency limit, free swell index, linear shrinkage, moisture density relation, CBR and CBR swell potential on expansive soil sample using cement as hydraulic stabilizer and that of kaolin as non-hydraulic stabilizer. additional test parameter like unconfined compressive strength, PH value test, volumetric shrinkage and mineralogical tests should also be performed to have more accurate test results.

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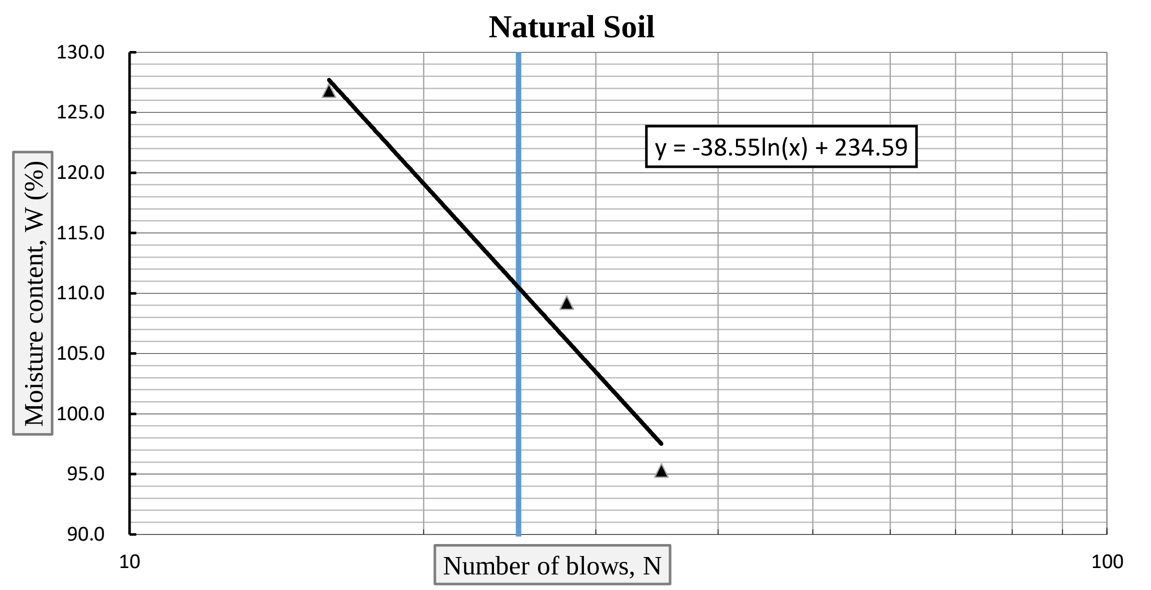
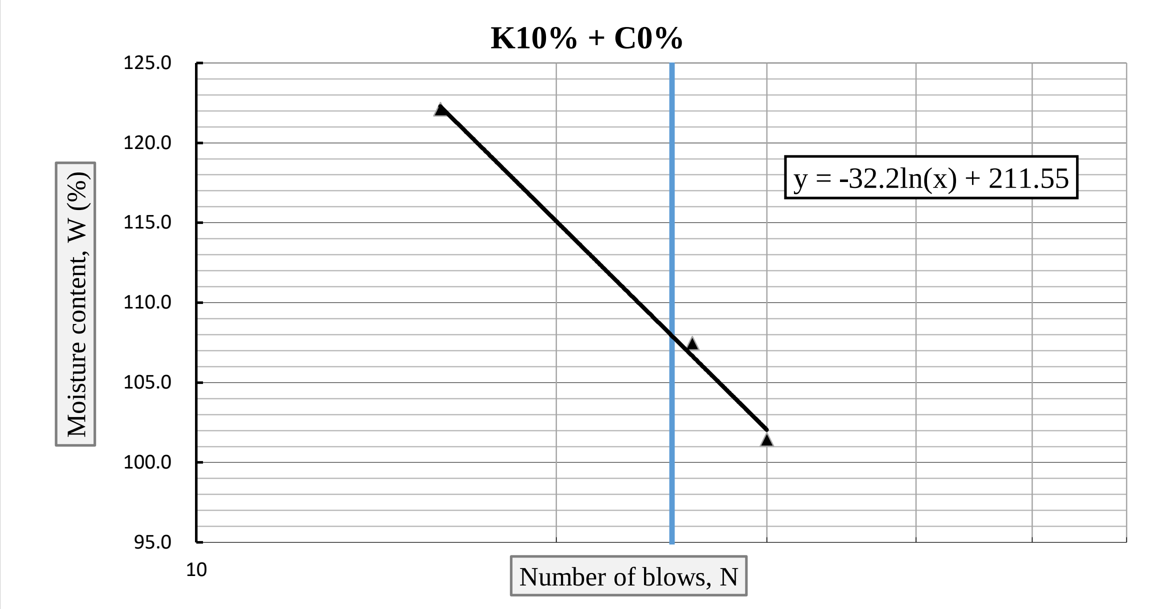
APPENDIX

APPENDIX A: Atterberg's Limit Test Analysis Data

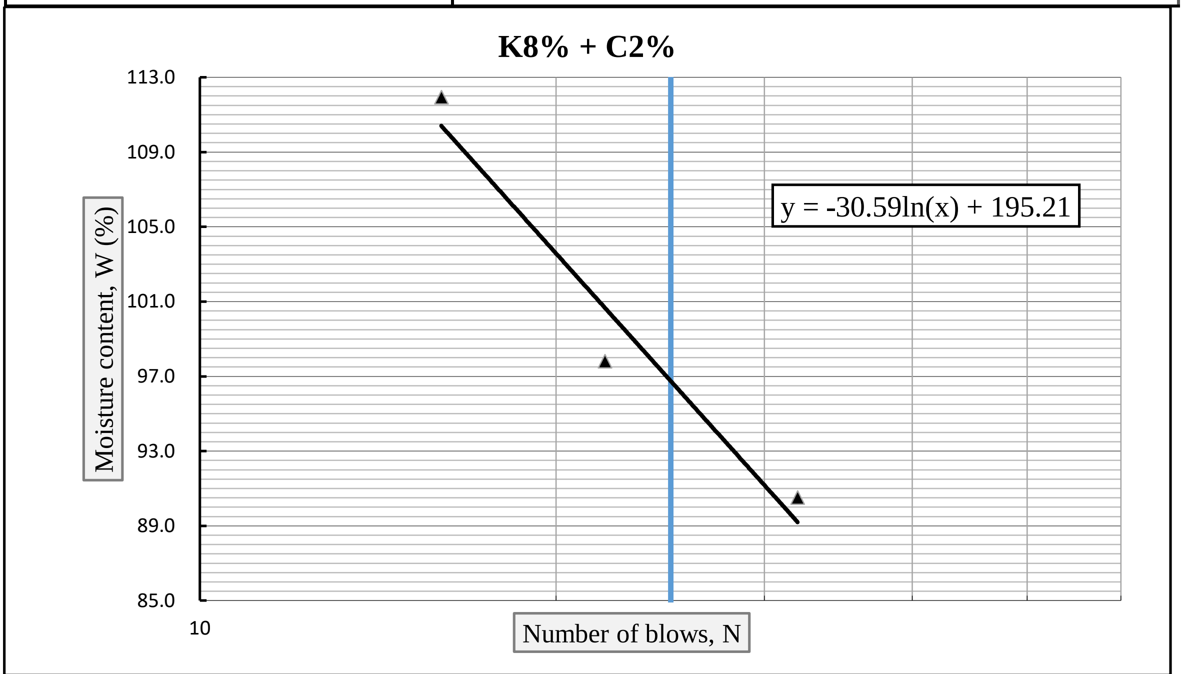
Sample Location: WYR	Natural soil				
Determination	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	—	—	35	28	16
Mass of Empty Can, M_C (g)	23.25	16.88	35.20	31.90	36.65
Mass Can + Wet Soil,M_c (g)	31.93	26.30	54.32	47.80	53.57
Mass Can +Dry Soil, M_{CDS} (g)	29.40	23.57	44.99	39.50	44.11
Mass of Dry Soil, M_{DS} (g)	6.15	6.69	9.79	7.60	7.46
Mass of Water, M_W (g)	2.53	2.73	9.33	8.30	9.46
Water Content, w (%)	41.14	40.81	95.30	109.21	126.81
Liquid Limit (LL) (%):	108.85				
Plastic Limit (PL) (%):	40.97				
Plasticity Index (PI) (%):	67.87				



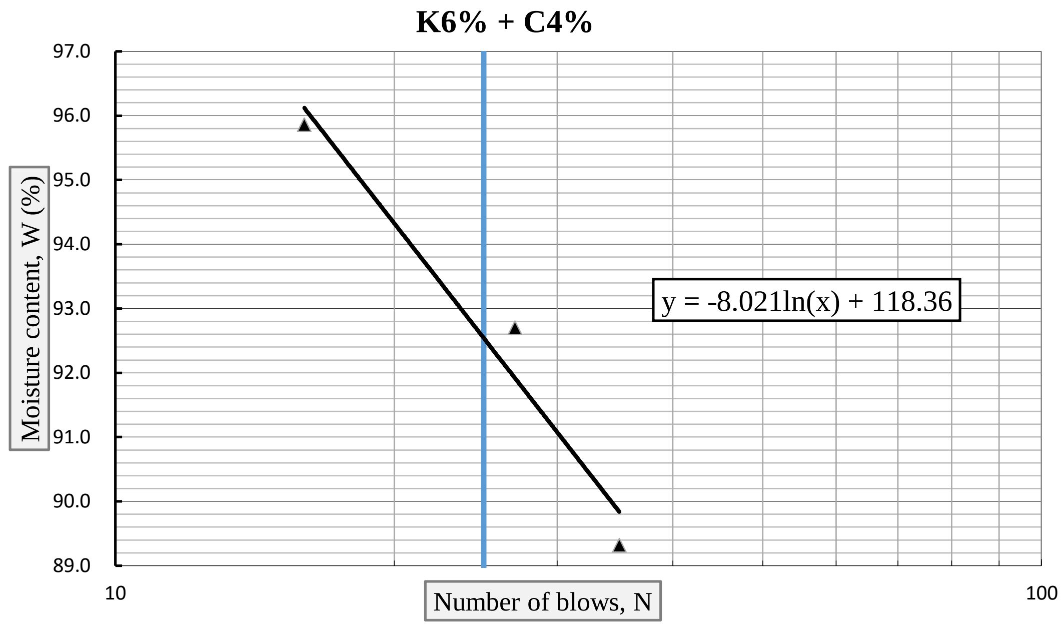
Sample Location: WYR	Additive Content: K10%+C0%				
Determination	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	—	30	26	16
Mass of Empty Can, M_C (g)	16.95	14.87	17.67	18.64	17.63
Mass Can + Wet Soil,M_c (g)	25.00	21.02	33.24	29.76	32.00
Mass Can +Dry Soil,M_{CDS} (g)	21.95	19.05	25.40	24.00	24.10
Mass of Dry Soil,M_{DS} (g)	5.00	4.18	7.73	5.36	6.47
Mass of Water,M_W (g)	3.05	1.97	7.84	5.76	7.90
Water Content,w (%)	61.00	47.13	101.42	107.46	122.10
Liquid Limit (LL) (%):	107.42				
Plastic Limit (PL) (%):	54.06				
Plasticity Index (PI) (%):	53.36				



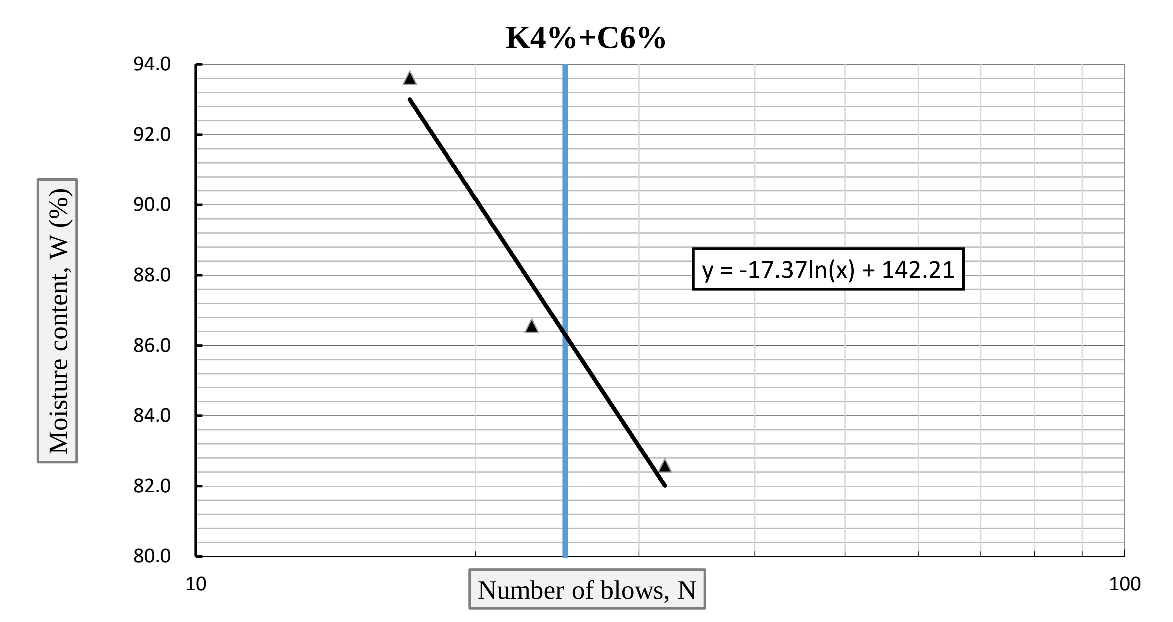
Sample Location: WYR	Additive Content: K8%+C2%				
Determination	**Plastic Limit**		**Liquid Limit**		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	–	32	22	16
Mass of Empty Can, M_C (g)	17.90	18.13	36.97	34.79	28.16
Mass Can + Wet Soil,M_c (g)	24.92	26.43	56.65	58.25	47.32
Mass Can +Dry Soil,M_{CDS} (g)	22.40	23.33	47.30	46.65	37.20
Mass of Dry Soil,M_{DS} (g)	4.50	5.20	10.33	11.86	9.04
Mass of Water,M_W (g)	2.52	3.10	9.35	11.60	10.12
Water Content,w (%)	56.00	59.62	90.51	97.81	111.95
Liquid Limit (LL) (%):	98.15				
Plastic Limit (PL) (%):	57.81				
Plasticity Index (PI) (%):	40.34				



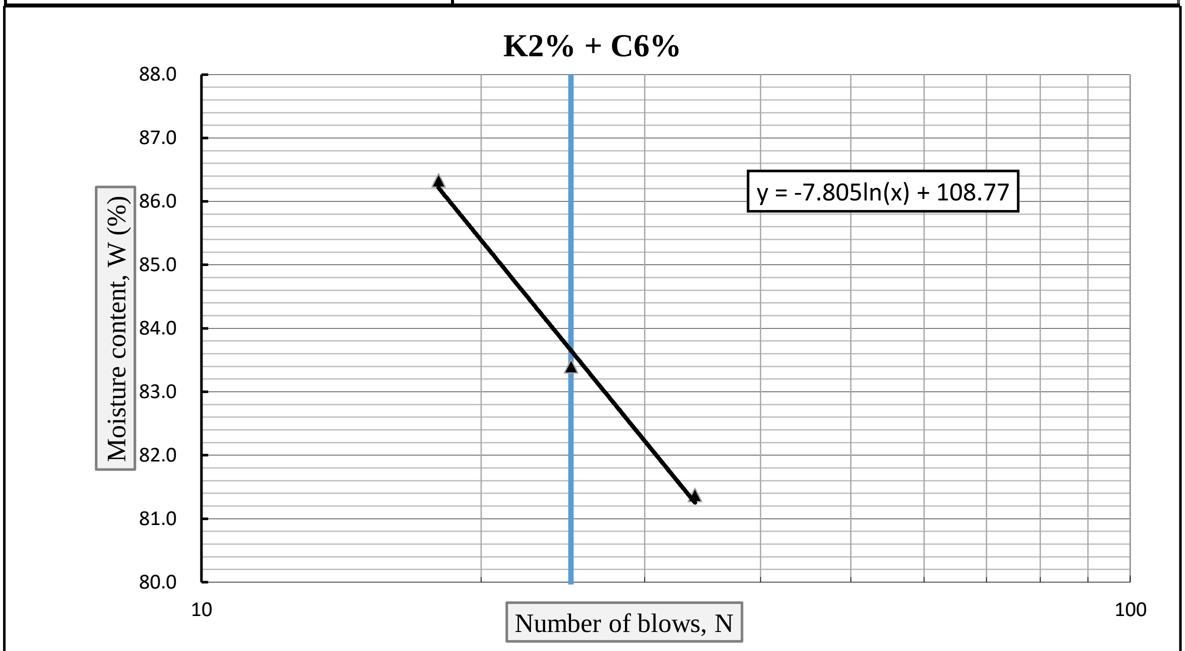
Sample Location: WYR	**Additive Content: K6%+C4%**				
Determination	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	–	35	27	16
Mass of Empty Can, M_C (g)	17.88	19.59	35.01	37.68	17.63
Mass Can + Wet Soil, M_c (g)	25.55	27.60	56.10	59.07	33.24
Mass Can +Dry Soil, M_{CDS} (g)	22.50	24.61	46.15	48.78	25.60
Mass of Dry Soil, M_{DS} (g)	4.62	5.02	11.14	11.10	7.97
Mass of Water, M_W (g)	3.05	2.99	9.95	10.29	7.64
Water Content, w (%)	66.02	59.56	89.32	92.70	95.86
Liquid Limit (LL) (%):	92.13				
Plastic Limit (PL) (%):	62.79				
Plasticity Index (PI) (%):	29.34				



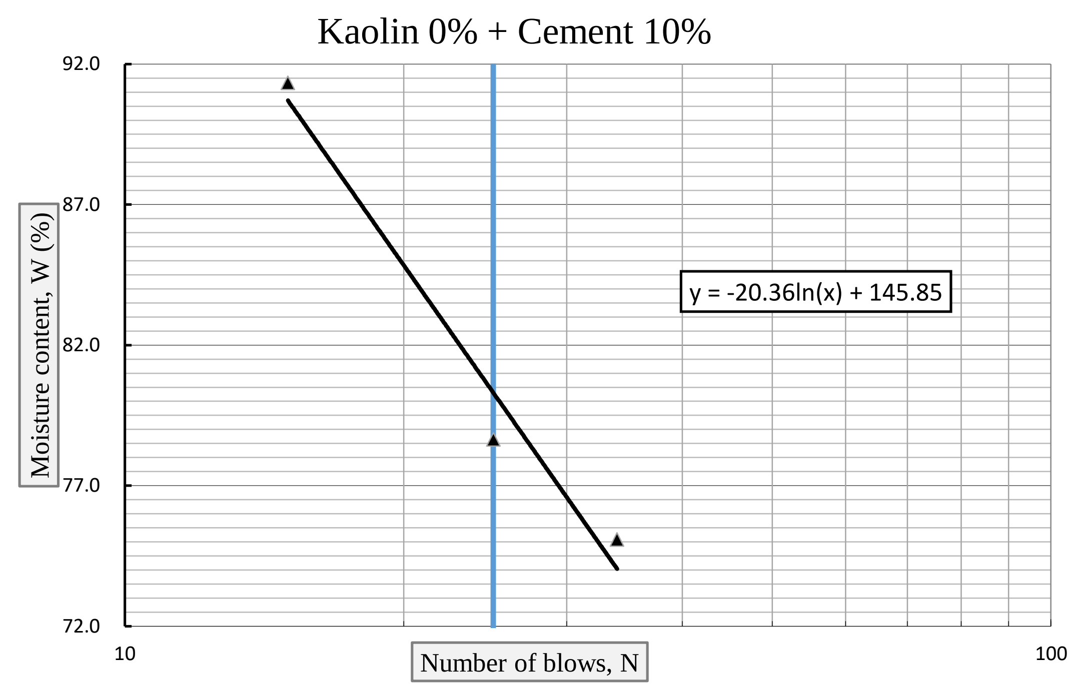
Sample Location: WYR	**Additive Content: K4%+C6%**				
Determination	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	–	32	23	17
Mass of Empty Can, M_C (g)	16.01	18.14	37.80	37.63	32.56
Mass Can + Wet Soil, M_c (g)	26.00	26.87	54.36	55.54	50.16
Mass Can +Dry Soil, M_{CDS} (g)	21.89	23.52	46.87	47.23	41.65
Mass of Dry Soil, M_{DS} (g)	5.88	5.38	9.07	9.60	9.09
Mass of Water, M_W (g)	4.11	3.35	7.49	8.31	8.51
Water Content, w (%)	69.90	62.27	82.58	86.56	93.62
Liquid Limit (LL) (%):	86.89				
Plastic Limit (PL) (%):	66.08				
Plasticity Index (PI) (%):	20.81				



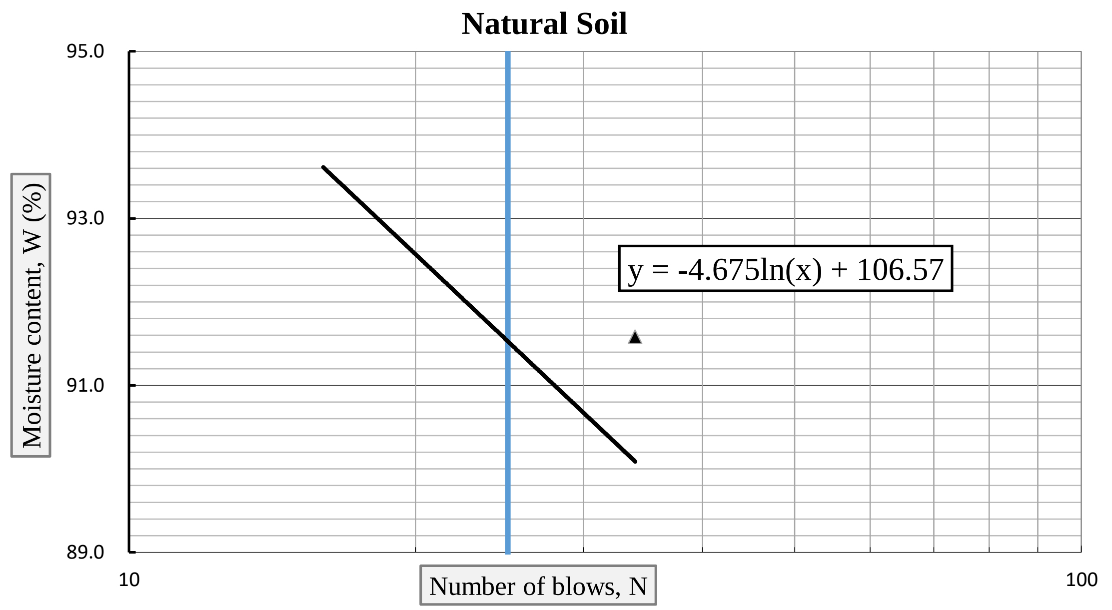
Sample Location: WYR	**Additive Content: K2%+C8%**				
Determination	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	–	34	25	18
Mass of Empty Can, M_C (g)	18.95	18.71	53.46	37.50	36.74
Mass Can + Wet Soil,M_c (g)	27.11	27.00	72.94	59.05	56.51
Mass Can +Dry Soil,M_{CDS} (g)	23.84	23.65	64.20	49.25	47.35
Mass of Dry Soil,M_{DS} (g)	4.89	4.94	10.74	11.75	10.61
Mass of Water,M_W (g)	3.27	3.35	8.74	9.80	9.16
Water Content,w (%)	66.87	67.81	81.38	83.40	86.33
Liquid Limit (LL) (%):	83.77				
Plastic Limit (PL) (%):	67.34				
Plasticity Index (PI) (%):	16.43				



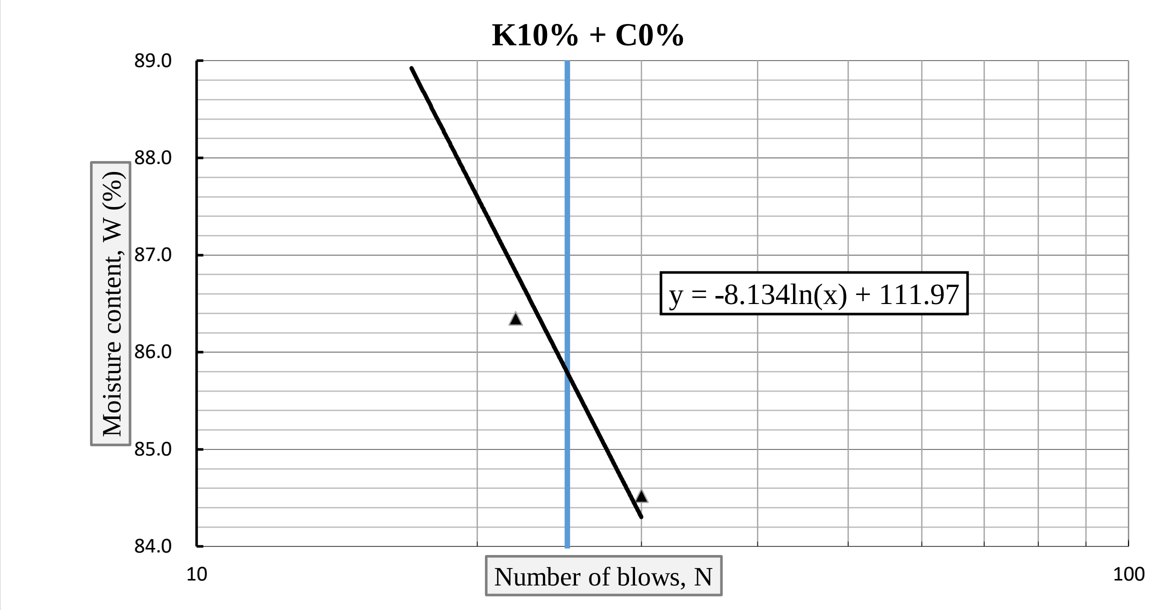
Sample Location: WYR	**Additive Content: K =0%, C =10%**				
Determination	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	–	34	25	15
Mass of Empty Can, M_C (g)	19.43	19.72	51.31	32.20	29.42
Mass Can + Wet Soil, M_c (g)	28.20	27.90	68.10	50.60	48.84
Mass Can +Dry Soil, M_{CDS} (g)	24.64	24.72	60.90	42.50	39.57
Mass of Dry Soil, M_{DS} (g)	5.21	5.00	9.59	10.30	10.15
Mass of Water, M_W (g)	3.56	3.18	7.20	8.10	9.27
Water Content, w (%)	68.33	63.60	75.08	78.64	91.33
Liquid Limit (LL) (%):	81.18				
Plastic Limit (PL) (%):	65.97				
Plasticity Index (PI) (%):	15.22				



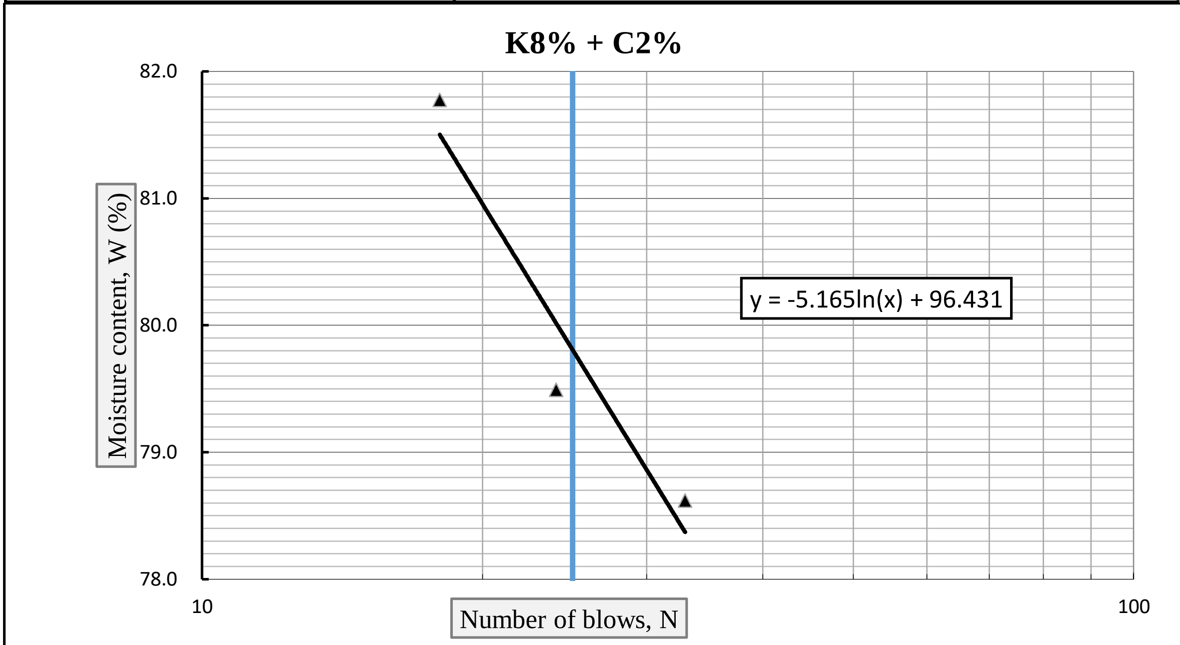
Sample Location: BK	**Natural soil**				
Determination	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	—	34	23	16
Mass of Empty Can, M_C (g)	16.42	18.64	36.96	36.62	29.42
Mass Can + Wet Soil,M_c (g)	24.30	26.30	59.49	56.37	50.66
Mass Can +Dry Soil,M_{CDS} (g)	22.12	24.26	48.72	47.08	40.30
Mass of Dry Soil,M_{DS} (g)	5.70	5.62	11.76	10.46	10.88
Mass of Water,M_W (g)	2.18	2.04	10.77	9.29	10.36
Water Content,w (%)	38.25	36.30	91.58	88.81	95.22
Liquid Limit (LL) (%):	93.07				
Plastic Limit (PL) (%):	37.27				
Plasticity Index (PI) (%):	55.79				



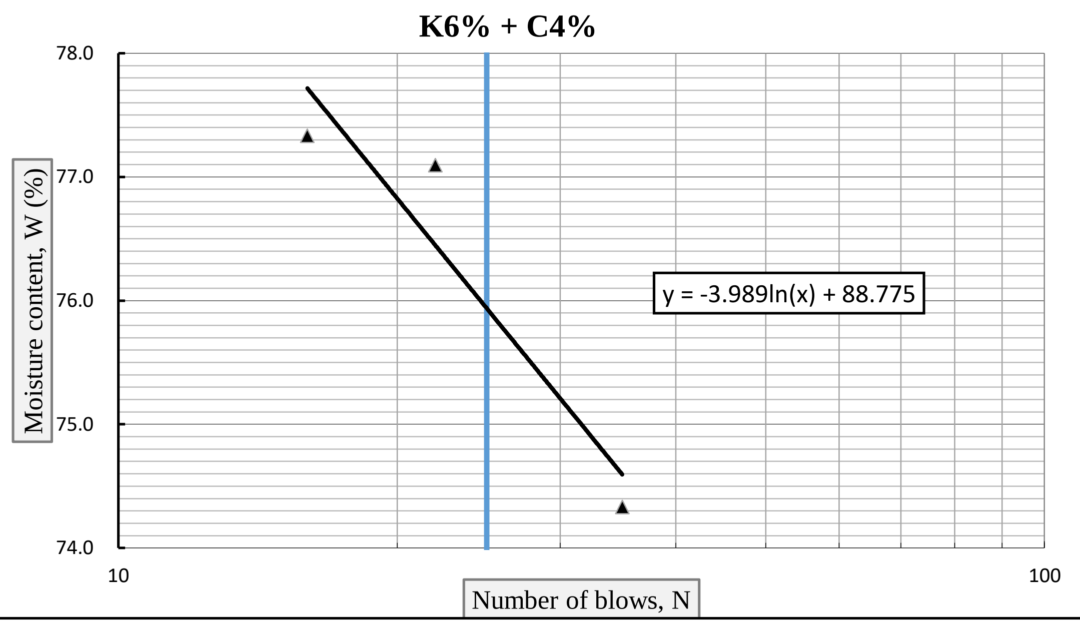
Sample Location: BK	**Additive Content: K10%+C0%**				
Determination	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	—	30	22	17
Mass of Empty Can, M_C (g)	19.59	18.70	17.54	17.57	17.80
Mass Can + Wet Soil,M_c (g)	26.50	26.15	35.90	40.36	38.97
Mass Can +Dry Soil,M_{CDS} (g)	24.51	23.88	27.49	29.80	28.99
Mass of Dry Soil,M_{DS} (g)	4.92	5.18	9.95	12.23	11.19
Mass of Water,M_W (g)	1.99	2.27	8.41	10.56	9.98
Water Content,w (%)	40.45	43.82	84.52	86.35	89.19
Liquid Limit (LL) (%):	86.02				
Plastic Limit (PL) (%):	42.13				
Plasticity Index (PI) (%):	43.89				



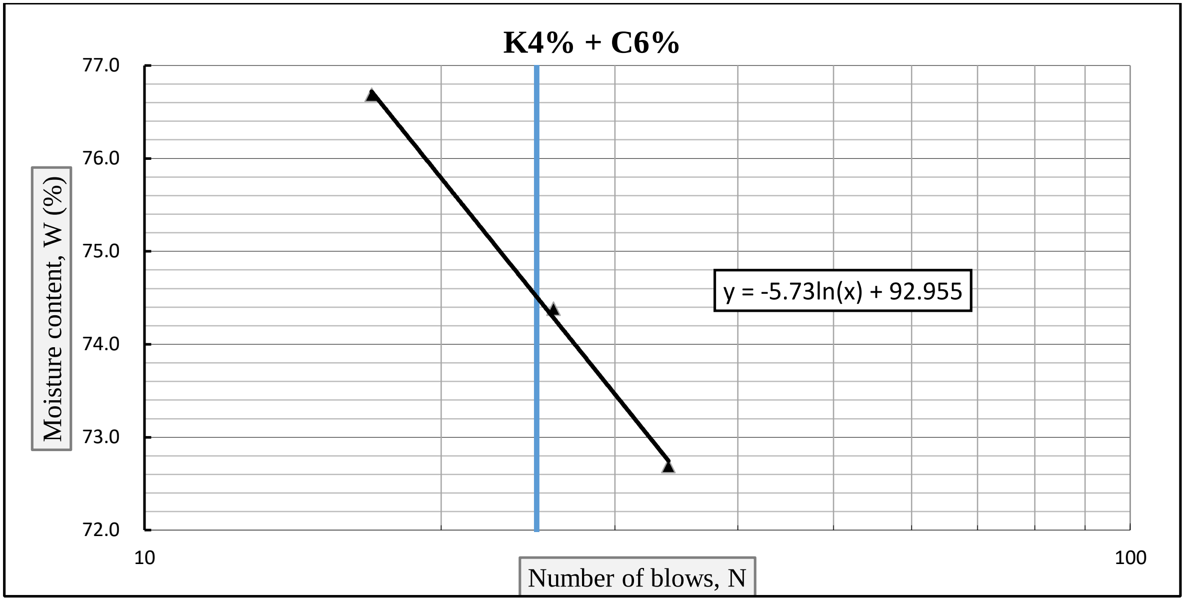
Sample Location: BK	Additive Content: K8%+C2%				
Determination	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	–	33	24	18
Mass of Empty Can, M_C (g)	18.74	19.58	34.71	53.36	32.55
Mass Can + Wet Soil, M_c (g)	26.30	27.35	58.27	73.84	56.89
Mass Can +Dry Soil, M_{CDS} (g)	23.88	24.83	47.90	64.77	45.94
Mass of Dry Soil, M_{DS} (g)	5.14	5.25	13.19	11.41	13.39
Mass of Water, M_W (g)	2.42	2.52	10.37	9.07	10.95
Water Content, w (%)	47.08	48.00	78.62	79.49	81.78
Liquid Limit (LL) (%):	80.07				
Plastic Limit (PL) (%):	47.54				
Plasticity Index (PI) (%):	32.53				



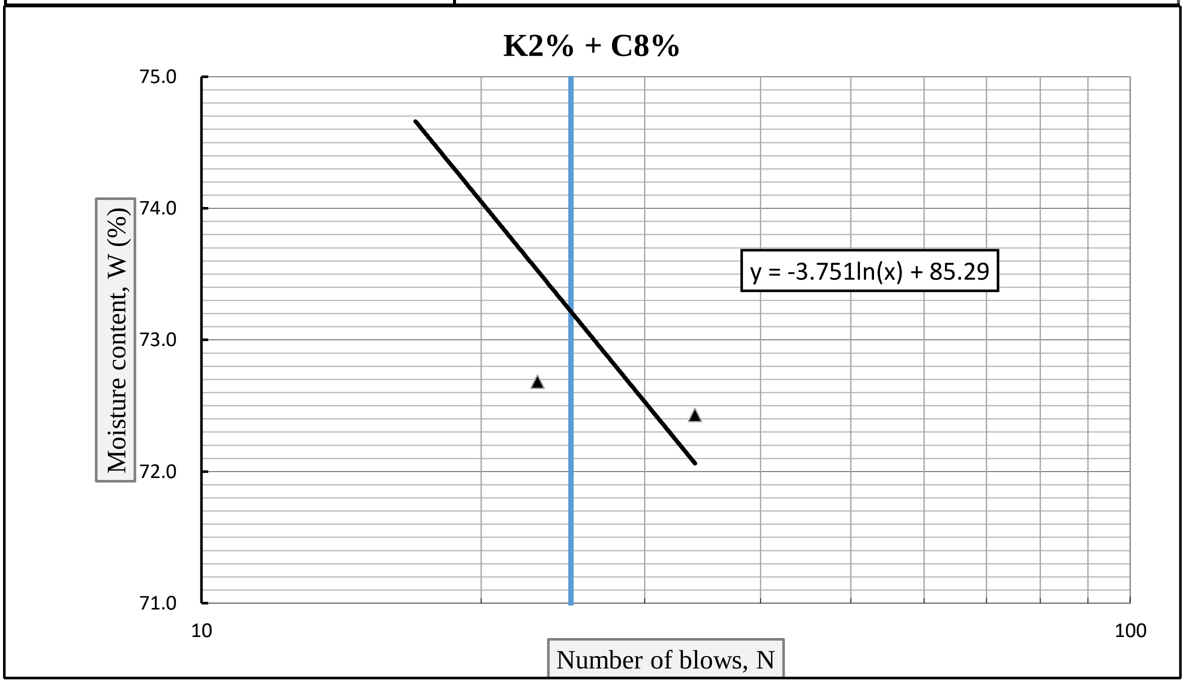
Sample Location: BK	**Additive Content =K6%+C4%**				
Determination	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	—	35	22	16
Mass of Empty Can, M_C (g)	16.98	16.58	18.56	17.54	17.56
Mass Can + Wet Soil,M_c (g)	28.10	26.00	39.41	40.58	43.22
Mass Can +Dry Soil,M_{CDS} (g)	24.37	22.75	30.52	30.55	32.03
Mass of Dry Soil,M_{DS} (g)	7.39	6.17	11.96	13.01	14.47
Mass of Water,M_W (g)	3.73	3.25	8.89	10.03	11.19
Water Content,w (%)	50.47	52.67	74.33	77.09	77.33
Liquid Limit (LL) (%):	75.62				
Plastic Limit (PL) (%):	51.57				
Plasticity Index (PI) (%):	24.05				



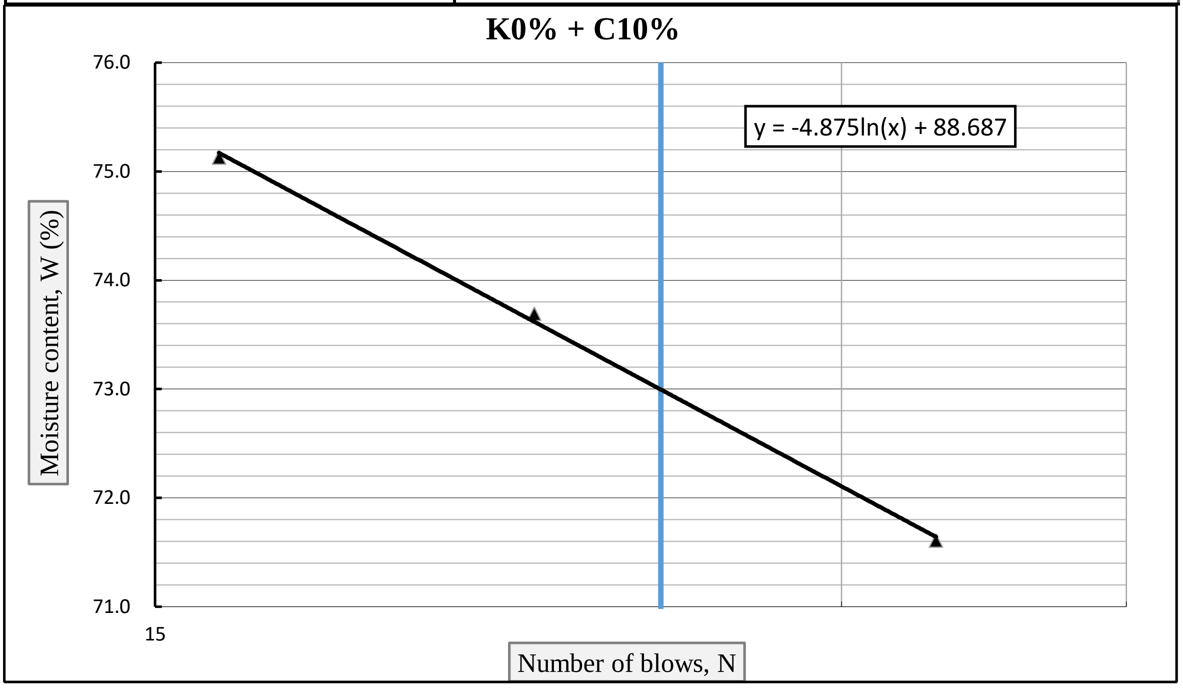
Sample Location: BK	**Additive Content: K4%+C6%**				
Determinantion	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	—	34	26	17
Mass of Empty Can, M_C (g)	17.88	19.59	37.39	37.62	32.23
Mass Can + Wet Soil,M_c (g)	26.00	27.28	61.67	65.12	58.75
Mass Can +Dry Soil,M_{CDS} (g)	22.93	24.62	51.45	53.39	47.24
Mass of Dry Soil,M_{DS} (g)	5.05	5.03	14.06	15.77	15.01
Mass of Water,M_W (g)	3.07	2.66	10.22	11.73	11.51
Water Content,w (%)	60.79	52.88	72.69	74.38	76.68
Liquid Limit (LL) (%):	74.46				
Plastic Limit (PL) (%):	56.84				
Plasticity Index (PI) (%):	17.62				



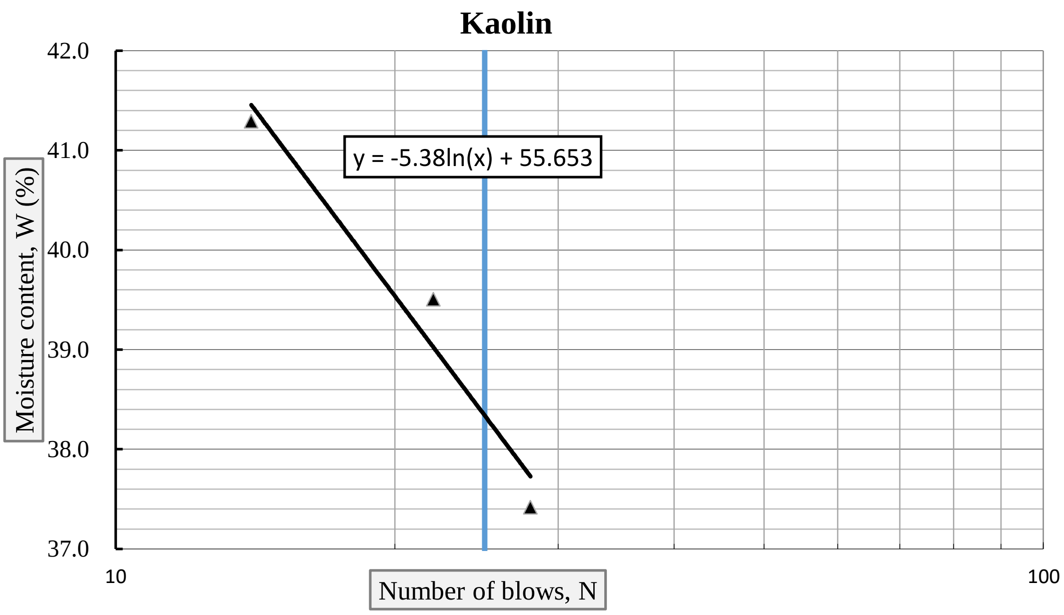
Sample Location: BK	Additive Content: K2%+C8%				
Determination	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	–	34	23	17
Mass of Empty Can, M_C (g)	16.46	16.99	49.51	36.26	40.01
Mass Can + Wet Soil, M_c (g)	24.25	29.20	74.84	57.12	62.13
Mass Can +Dry Soil, M_{CDS} (g)	21.23	24.74	64.20	48.34	52.64
Mass of Dry Soil, M_{DS} (g)	4.77	7.75	14.69	12.08	12.63
Mass of Water, M_W (g)	3.02	4.46	10.64	8.78	9.49
Water Content, w (%)	63.31	57.55	72.43	72.68	75.14
Liquid Limit (LL) (%):	73.63				
Plastic Limit (PL) (%):	60.43				
Plasticity Index (PI) (%):	13.20				



Sample Location: BK	**Additive Content: K =0%, C =10%**				
Determination	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	–	33	22	16
Mass of Empty Can, M_C (g)	19.43	19.71	37.79	35.03	31.27
Mass Can + Wet Soil,M_c (g)	26.70	26.68	63.72	56.95	55.14
Mass Can +Dry Soil,M_{CDS} (g)	24.15	23.88	52.90	47.65	44.90
Mass of Dry Soil,M_{DS} (g)	4.72	4.17	15.11	12.62	13.63
Mass of Water,M_W (g)	2.55	2.80	10.82	9.30	10.24
Water Content,w (%)	54.03	67.15	71.61	73.69	75.13
Liquid Limit (LL) (%):	72.96				
Plastic Limit (PL) (%):	60.59				
Plasticity Index (PI) (%):	12.37				



Kaolin					
Determination	Plastic Limit		Liquid Limit		
Sample Trial Number	1	2	1	2	3
Number of Blows	–	—	28	22	14
Mass of Empty Can, M_C (g)	6.03	6.28	29.55	17.15	17.69
Mass Can + Wet Soil,M_c (g)	18.22	19.49	47.84	36.82	36.75
Mass Can +Dry Soil,M_{CDS} (g)	15.59	16.69	42.86	31.25	31.18
Mass of Dry Soil,M_{DS} (g)	9.56	10.41	13.31	14.10	13.49
Mass of Water,M_W (g)	2.63	2.80	4.98	5.57	5.57
Water Content,w (%)	27.51	26.90	37.42	39.50	41.29
Liquid Limit (LL) (%):	38.05				
Plastic Limit (PL) (%):	27.20				
Plasticity Index (PI) (%):	10.85				



APPENDIX B: Linear Shrinkage Analysis Data

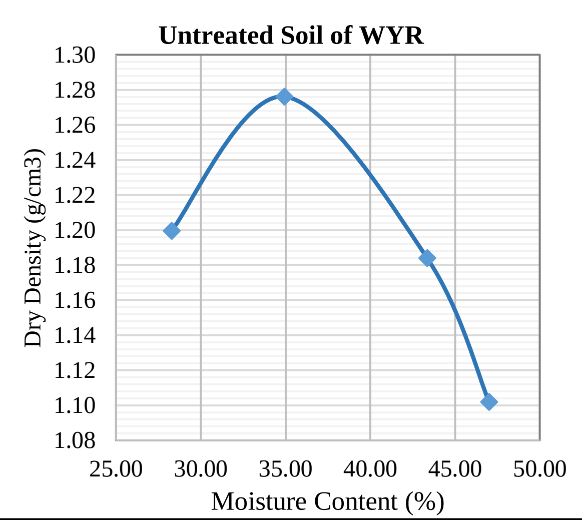
Sample Location:	**WYR**		
Additives	**Length of Mold (cm)**	**length of dry specimen(cm)**	**Linear Shrinkage (%)**
Natural Soil	14.00	10.77	23.07
K10+C0	14.00	11.01	21.36
K8+C2	14.00	11.60	17.14
K6+C4	14.00	12.60	10.00
K4+C6	14.00	12.84	8.29
K2+C8	14.00	13.35	4.64
K0+C10	14.00	13.40	4.29

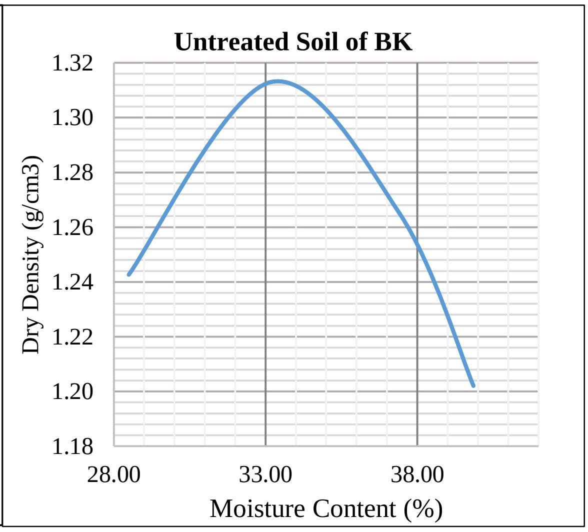
Sample Location:	**BK**		
Additives	**Length of mold(cm)**	**length of dry specimen(cm)**	**Linear Shrinkage (%)**
Natural Soil	14.00	10.77	23.07
K10+C0	14.00	11.01	21.36
K8+C2	14.00	11.60	17.14
K6+C4	14.00	12.60	10.00
K4+C6	14.00	12.84	8.29
K2+C8	14.00	13.35	4.64
K0+C10	14.00	13.40	4.29

APPENDIX C: Compaction Test Analysis Data

I. Compaction test results of WYR and BK untreated soil samples

Sample Location: WYR	**Untreated Soil of WYR**							
Trial No.	1		2		3		4	
Mold + wet soil(g)	5495.01		5667.9		5660.3		5587.225	
Mold(g)	4042.4		4042.4		4058		4058	
Wet Soil(g)	1452.61		1625.5		1602.3		1529.225	
Wet Density, (g/cm^3)	1.539		1.722		1.697		1.620	
Moisture Content Determination								
Can wt.(g)	18.05	18.45	17.04	17.81	37.67	34.32	36.46	41.21
Wet soil + can(g)	110.00	95.78	95.68	96.29	128.50	130.29	127.91	124.66
Dry soil + can(g)	88.95	79.41	75.65	75.65	99.81	102.61	98.31	98.31
Mass of moisture(g),	21.05	16.37	20.03	20.64	28.69	27.68	29.60	26.35
Dry soil(g)	70.90	60.96	58.61	57.84	62.14	68.29	61.85	57.10
Moisture content (%)	29.69	26.86	34.18	35.69	46.17	40.53	47.86	46.15
Av.moisture content (%)	28.274		34.931		43.354		47.003	
Dry Density(g/cm^3)	1.200		1.276		1.184		1.102	

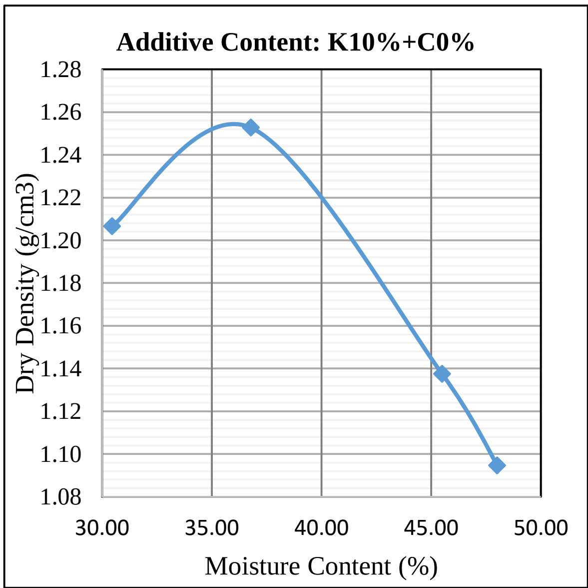


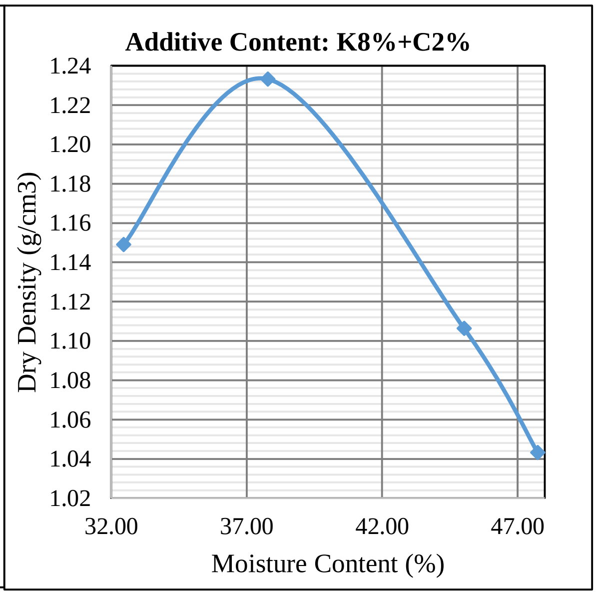


Sample Location: BK	**Untreated Soil of BK**							
Trial No.	1		2		3		4	
Mold + wet soil(g)	5495.01		5667.9		5660.3		5587.225	
Mold(g)	4042.4		4042.4		4058		4058	
Wet Soil(g)	1452.61		1625.5		1602.3		1529.225	
Wet Density, (g/cm^3)	1.539		1.722		1.697		1.620	
Moisture Content Determination								
Can wt.(g)	18.05	18.45	17.04	17.81	37.67	34.32	36.46	41.21
Wet soil + can(g)	110.00	95.78	95.68	96.29	128.50	130.29	127.91	124.66
Dry soil + can(g)	88.95	79.41	75.65	75.65	99.81	102.61	98.31	98.31
Mass of moisture(g),	21.05	16.37	20.03	20.64	28.69	27.68	29.60	26.35
Dry soil(g)	70.90	60.96	58.61	57.84	62.14	68.29	61.85	57.10
Moisture content (%)	29.69	26.86	34.18	35.69	46.17	40.53	47.86	46.15
Av.moisture content (%)	28.274		34.931		43.354		47.003	
Dry Density(g/cm^3)	1.200		1.276		1.184		1.102	

II. Compaction test results of WYR treated soil sample

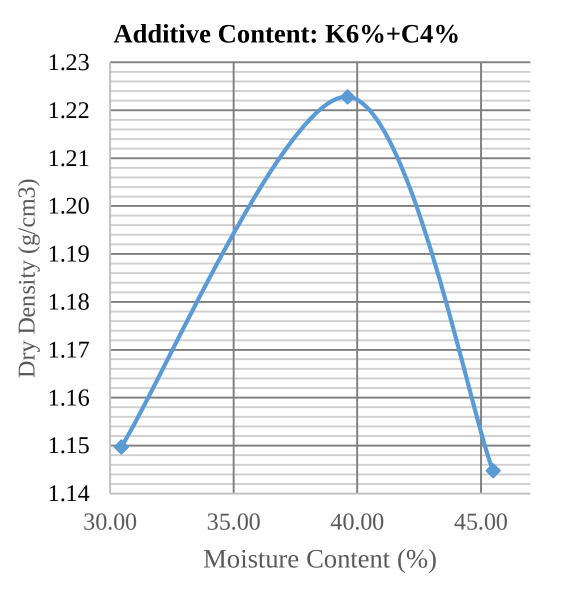
Sample Location: WYR	**Additive Content: K10%+C0%**							
Trial No.	**1**		**2**		**3**		**4**	
Mold + wet soil(g)	5528.261		5659.977		5620.321		5587.52	
Mold(g)	4042.4		4042.4		4058		4058	
Wet Soil(g)	1485.861		1617.577		1562.321		1529.52	
Wet Density, (g/cm^3)	1.574		1.714		1.655		1.620	
Moisture Content Determination								
Can wt.(g)	17.50	18.55	18.40	17.18	36.76	35.24	37.65	40.12
Wet soil + can(g)	111.25	97.48	94.97	98.94	129.40	132.30	127.16	126.51
Dry soil + can(g)	88.95	79.41	75.65	75.65	99.81	102.6	98.31	98.31
Mass of moisture(g),	22.30	18.07	19.32	23.29	29.59	29.69	28.85	28.20
Dry soil(g)	71.45	60.86	57.25	58.47	63.05	67.38	60.67	58.19
Moisture content (%)	31.21	29.69	33.74	39.83	46.93	44.06	47.55	48.46
Av.moisture content (%)	30.451		36.784		45.495		48.007	
Dry Density(g/cm^3)	1.207		1.253		1.137		1.095	

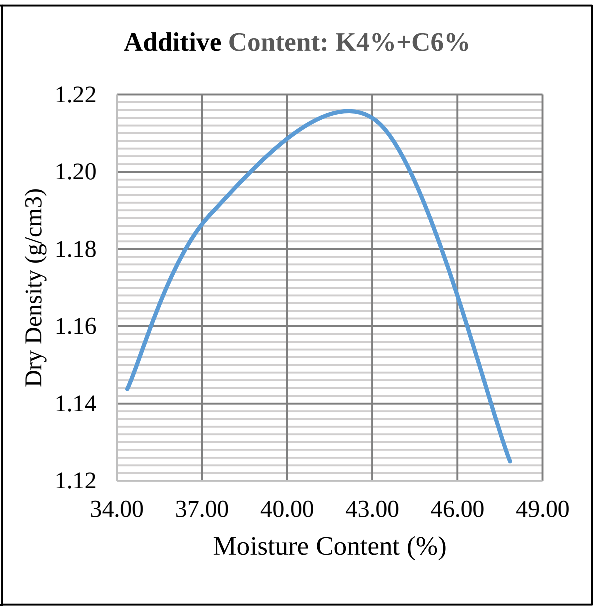




Sample Location: WYR	**Additive Content: K8%+C2%**							
Trial No.	**1**		**2**		**3**		**4**	
Mold + wet soil(g)	5485.07		5655.04		5563.10		5503.42	
Mold(g)	4048.50		4051.10		4048.50		4048.50	
Wet Soil(g)	1436.57		1603.94		1514.60		1454.92	
Wet Density, (g/cm^3)	1.52		1.70		1.60		1.54	
Moisture Content Determination								
Can wt.(g)	32.01	35.25	37.20	40.02	29.05	36.28	29.05	36.28
Wet soil + can(g)	131.8	126.75	123.2	132.18	128.35	127.42	133.35	126.42
Dry soil + can(g)	107.7	104.00	98.91	107.7	99.74	97.21	99.74	97.21
Mass of moisture(g),	24.08	22.75	24.30	24.49	28.61	30.21	33.61	29.21
Dry soil(g)	75.68	68.75	61.71	67.67	70.69	60.93	70.69	60.93
Moisture content (%)	31.82	33.09	39.38	36.19	40.47	49.58	47.54	47.94
Av.moisture content (%)	32.451		37.784		45.025		47.741	
Dry Density(g/cm^3)	1.149		1.233		1.106		1.043	

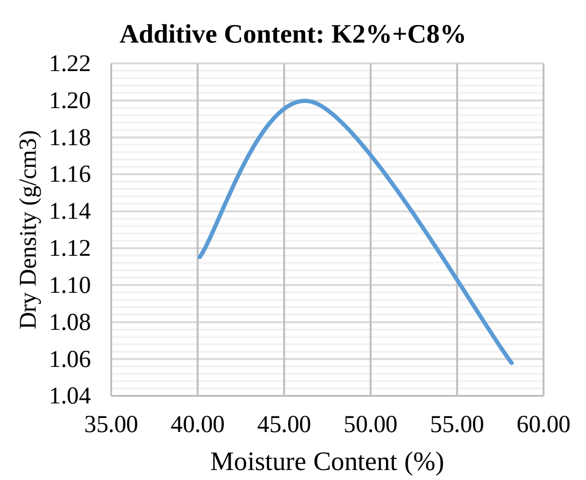
Sample Location: WYR	Additive Content: K6%+C4%					
Trial No.	1		2		3	
Mold + wet soil(g)	5458.261		5653.977		5630.321	
Mold(g)	4042.4		4042.4		4058	
Wet Soil(g)	1415.861		1611.577		1572.321	
Wet Density, (g/cm^3)	1.500		1.707		1.666	
Moisture Content Determination						
Can wt.(g)	17.50	18.55	18.40	17.18	36.76	35.24
Wet soil + can(g)	111.25	97.48	98.20	98.94	129.40	132.30
Dry soil + can(g)	88.95	79.41	75.65	75.65	99.81	102.61
Mass of moisture(g),	22.30	18.07	22.55	23.29	29.59	29.69
Dry soil(g)	71.45	60.86	57.25	58.47	63.05	67.38
Moisture content (%)	31.21	29.69	39.39	39.83	46.93	44.06
Av.moisture content (%)	30.451		39.612		45.495	
Dry Density(g/cm^3)	1.150		1.223		1.145	

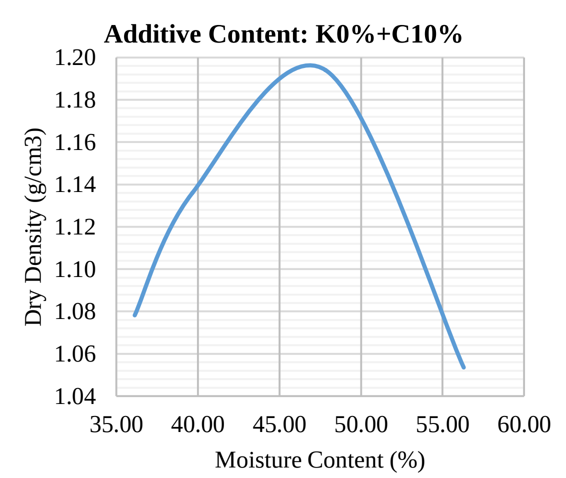




Sample Location: WYR	Additive Content: K4%+C6%							
Trial No.	**1**		**2**		**3**		**4**	
Mold + wet soil(g)	5501.19		5589.5		5688.5		5619.5	
Mold(g)	4050.4		4049.03		4049.03		4049.3	
Wet Soil(g)	1450.79		1540.47		1639.47		1570.2	
Wet Density, (g/cm^3)	1.537		1.632		1.737		1.663	
Moisture Content Determination								
Can wt.(g)	32.04	30.25	31.93	17.78	32.89	30.13	35.36	37.00
Wet soil + can(g)	102.9	102.37	100.97	97.42	108.89	105.37	138.78	134.38
Dry soil + can(g)	85.21	83.45	82.09	75.95	85.21	83.45	104.08	104.06
Mass of moisture(g),	17.64	18.92	18.88	21.47	23.68	21.92	34.70	30.32
Dry soil(g)	53.17	53.20	50.16	58.17	52.33	53.33	68.72	67.06
Moisture content (%)	33.18	35.56	37.64	36.91	45.25	41.11	50.49	45.21
Av.moisture content (%)	34.371		37.272		43.176		47.852	
Dry Density(g/cm^3)	1.144		1.189		1.213		1.125	

Sample Location: WYR	Additive Content: K2%+C8%					
Trial No.	1		2		3	
Mold + wet soil(g)	5524.12		5711.51		5628.25	
Mold(g)	4049.03		4051		4049.03	
Wet Soil(g)	1475.09		1660.51		1579.22	
Wet Density, (g/cm^3)	1.563		1.759		1.673	
Moisture Content Determination						
Can wt.(g)	37.02	34.25	37.47	33.40	34.82	36.99
Wet soil + can(g)	119.50	105.70	119.27	120.90	154.17	146.12
Dry soil + can(g)	92.54	88.51	94.79	91.46	110.48	105.81
Mass of moisture(g),	26.96	17.19	24.48	29.44	43.69	40.31
Dry soil(g)	55.52	54.26	57.32	58.06	75.66	68.82
Moisture content (%)	48.56	31.68	42.71	50.71	57.74	58.57
Av.moisture content (%)	40.121		46.707		58.157	
Dry Density(g/cm^3)	1.115		1.199		1.058	

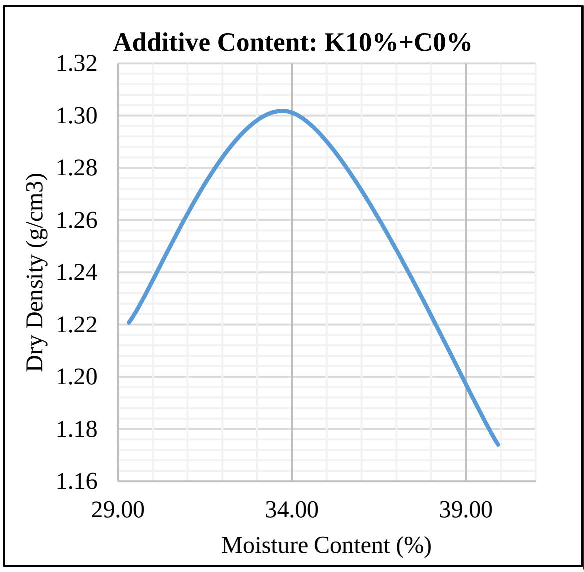


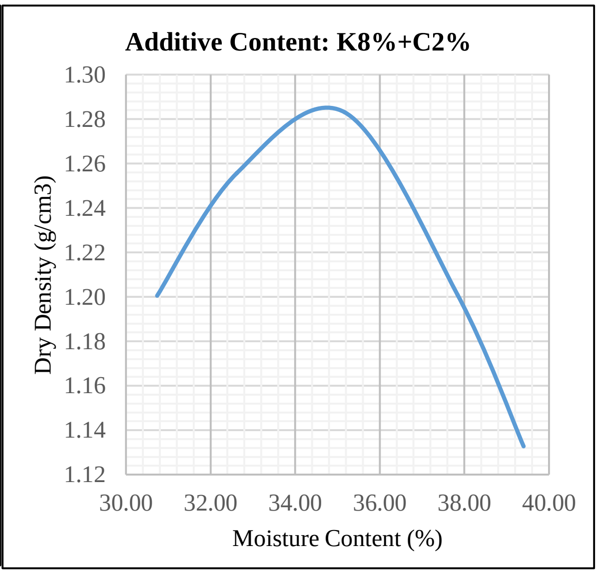


Sample Location: WYR	Additive Content: K0%+C10%							
Trial No.	1		2		3		4	
Mold + wet soil(g)	5437.5		5542.9		5718.4		5603.1	
Mold(g)	4052.1		4045.5		4052.1		4048.7	
Wet Soil(g)	1385.4		1497.4		1666.3		1554.4	
Wet Density, (g/cm^3)	1.468		1.586		1.765		1.647	
Moisture Content Determination								
Can wt.(g)	17.45	17.52	16.99	17.07	17.67	17.66	18.35	17.57
Wet soil + can(g)	101.22	104.54	94.61	89.25	117.95	119.39	105.03	119.09
Dry soil + can(g)	78.92	81.52	71.03	70.28	87.18	84.86	73.39	83.02
Mass of moisture(g),	22.30	23.02	23.58	18.97	30.77	34.53	31.64	36.07
Dry soil(g)	61.47	64.00	54.04	53.21	69.51	67.20	55.04	65.45
Moisture content (%)	36.27	35.97	43.64	35.65	44.27	51.38	57.49	55.12
Av.moisture content (%)	36.121		39.646		47.824		56.301	
Dry Density(g/cm^3)	1.078		1.136		1.194		1.053	

III. Compaction test results of BK treated soil sample

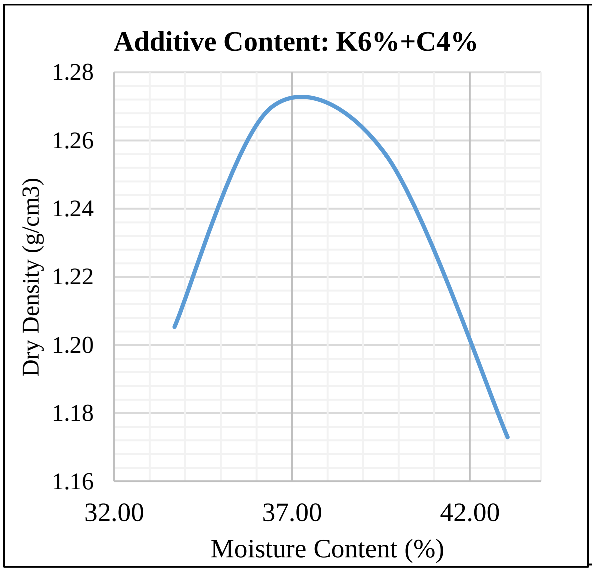
Sample Location: BK	**Additive Content: K10%+C0%**					
Trial No.	1		2		3	
Mold + wet soil(g)	5524.12		5711.51		5628.25	
Mold(g)	4049.03		4051		4049.03	
Wet Soil(g)	1475.09		1660.51		1579.22	
Wet Density, (g/cm^3)	1.563		1.759		1.673	
Moisture Content Determination						
Can wt.(g)	37.02	34.25	37.47	33.40	34.82	36.99
Wet soil + can(g)	119.50	105.70	119.27	120.90	154.17	146.12
Dry soil + can(g)	92.54	88.51	94.79	91.46	110.48	105.81
Mass of moisture(g),	26.96	17.19	24.48	29.44	43.69	40.31
Dry soil(g)	55.52	54.26	57.32	58.06	75.66	68.82
Moisture content (%)	48.56	31.68	42.71	50.71	57.74	58.57
Av.moisture content (%)	40.121		46.707		58.157	
Dry Density(g/cm^3)	1.115		1.199		1.058	

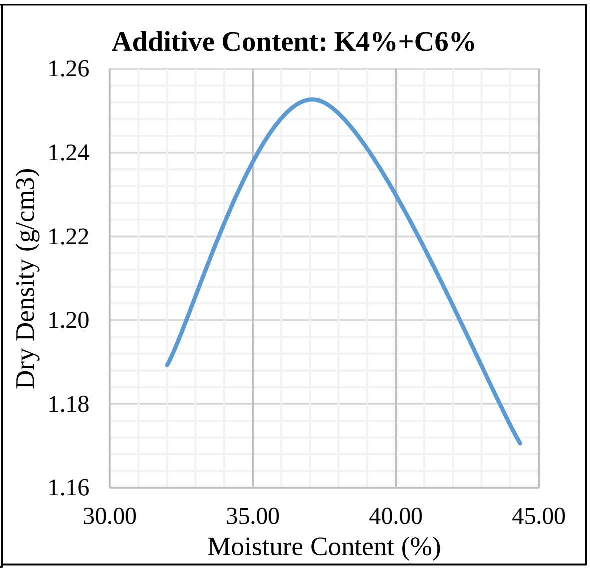




Sample Location: BK	**Additive Content: K8%+C2%**									
Trial No.	1		2		3		4		5	
Mold + wet soil(g)	5521.5		5611.9		5691.75		5602.51		5530.5	
Mold(g)	4039.9		4039.9		4054.5		4039.9		4039.9	
Wet Soil(g)	1481.6		1572		1637.25		1562.61		1481.6	
Wet Density, (g/cm^3)	1.569		1.665		1.734		1.655		1.569	
Moisture Content Determination										
Can wt.(g)	36.9	35.4	37.2	37.6	34.3	35.2	37.0	36.4	37.0	36.4
Wet soil + can(g)	144.8	156.0	139.0	154.6	172.4	173.5	171.8	168.2	172.4	170.6
Dry soil + can(g)	119.1	128.1	115.2	124.4	137.3	136.6	134.7	132.2	134.7	132.2
Mass of moisture(g),	25.8	27.9	23.7	30.3	35.1	36.8	37.2	36.1	37.8	38.5
Dry soil(g)	82.1	92.7	78.1	86.8	103.0	101.4	97.7	95.8	97.7	95.8
Moisture content (%)	31.4	30.1	30.4	34.9	34.1	36.3	38.0	37.7	38.6	40.2
Av.moisture content (%)	30.739		32.621		35.176		37.845		39.405	
Dry Density(g/cm^3)	1.200		1.256		1.283		1.201		1.133	

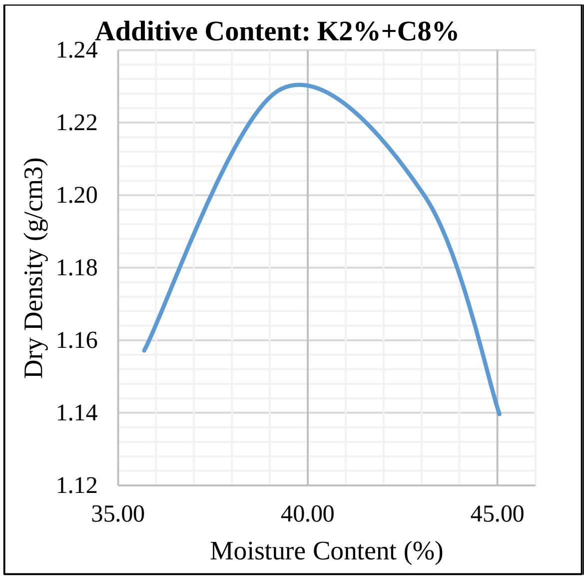
Sample Location: BK	**Additive Content: K6%+C4%**							
Trial No.	1		2		3		4	
Mold + wet soil(g)	5569.021		5682.574		5703.145		5631.865	
Mold(g)	4047.8		4047.8		4047.8		4047.8	
Wet Soil(g)	1521.221		1634.774		1655.345		1584.065	
Wet Density, (g/cm^3)	1.611		1.732		1.754		1.678	
Moisture Content Determination								
Can wt.(g)	36.92	35.35	37.15	37.59	34.32	35.21	36.99	36.40
Wet soil + can(g)	142.5	154.87	142.1	158.7	133.01	166.03	153.50	152.56
Dry soil + can(g)	115.9	124.66	114.3	126.1	105.47	128.27	118.13	117.90
Mass of moisture(g),	26.53	30.21	27.78	32.57	27.54	37.76	35.37	34.66
Dry soil(g)	79.06	89.31	77.17	88.49	71.15	93.06	81.14	81.50
Moisture content (%)	33.56	33.83	36.00	36.80	38.71	40.58	43.59	42.53
Av.moisture content (%)	33.693		36.400		39.644		43.059	
Dry Density(g/cm^3)	1.205		1.270		1.256		1.173	

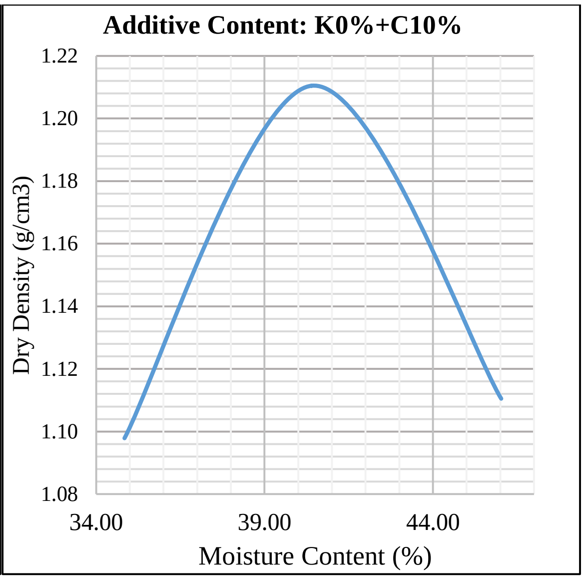




Sample Location: BK	**Additive Content: K4%+C6%**					
Trial No.	1		2		3	
Mold + wet soil(g)	4472.025		4612.94		4585.12	
Mold(g)	2990		2990		2990	
Wet Soil(g)	1482.025		1622.94		1595.12	
Wet Density, (g/cm^3)	1.570		1.719		1.690	
Moisture Content Determination						
Can wt.(g)	17.78	17.14	17.64	18.36	18.01	17.22
Wet soil + can(g)	110.63	83.59	96.34	98.53	102.23	97.96
Dry soil + can(g)	87.37	68.03	75.02	76.73	76.06	73.44
Mass of moisture(g),	23.27	15.56	21.32	21.80	26.17	24.52
Dry soil(g)	69.59	50.89	57.38	58.37	58.04	56.22
Moisture content (%)	33.44	30.58	37.16	37.35	45.09	43.61
Av.moisture content (%)	32.010		37.253		44.352	
Dry Density(g/cm^3)	1.189		1.253		1.171	

Sample Location: BK	Additive Content: K2%+C8%							
Trial No.	1		2		3		4	
Mold + wet soil(g)	4472.2		4605.57		4610.68		4550.54	
Mold(g)	2990		2990		2990		2990	
Wet Soil(g)	1482.2		1615.57		1620.68		1560.54	
Wet Density, (g/cm^3)	1.570		1.711		1.717		1.653	
Moisture Content Determination								
Can wt.(g)	18.36	18.01	17.43	18.85	17.22	18.64	17.60	17.14
Wet soil + can(g)	107.5	93.84	89.08	90.08	102.80	97.72	100.41	90.99
Dry soil + can(g)	80.99	76.78	68.98	69.90	77.06	73.90	74.56	68.17
Mass of moisture(g),	26.52	17.06	20.10	20.18	25.75	23.82	25.85	22.82
Dry soil(g)	62.63	58.76	51.56	51.05	59.84	55.25	56.96	51.02
Moisture content (%)	42.35	29.03	38.98	39.53	43.03	43.11	45.39	44.73
Av.moisture content (%)	35.689		39.253		43.072		45.056	
Dry Density(g/cm^3)	1.157		1.229		1.200		1.140	





Sample Location: BK	Additive Content: K0%+C10%					
Trial No.	1		2		3	
Mold + wet soil(g)	4387.5		4594.32		4520.85	
Mold(g)	2990		2990		2990	
Wet Soil(g)	1397.5		1604.32		1530.85	
Wet Density, (g/cm^3)	1.480		1.699		1.622	
Moisture Content Determination						
Can wt.(g)	17.81	5.79	6.69	7.64	5.58	6.58
Wet soil + can(g)	94.59	82.57	71.25	99.49	116.65	101.26
Dry soil + can(g)	74.52	62.97	52.56	73.22	81.02	71.95
Mass of moisture(g),	20.07	19.61	18.69	26.26	35.62	29.31
Dry soil(g)	56.71	57.18	45.86	65.58	75.44	65.37
Moisture content (%)	35.39	34.29	40.75	40.05	47.22	44.84
Av.moisture content (%)	34.842		40.400		46.029	
Dry Density(g/cm^3)	1.098		1.210		1.111	

APPENDIX D: Specific Gravity Test Analysis Data

Sample Location	**WYR**		
Trial Number	1	2	3
Weight of dry, clean pycnometer, Wp (g)	19.11	19.41	18.53
Weight of dry, clean pycnometer + soil, Wps (g)	29.10	29.53	28.51
Weight of oven dry soil , Ws (g)	10.00	10.00	10.00
Weight of pycnometer + water, Wpw (g)	45.67	46.30	43.68
Weight of pycnometer + soil + water, Wpws (g)	51.98	52.67	49.96
Observed temperature of water, Ti (oC)	23.00	23.00	22.00
Temperature, Tx (oC)	24.00	24.00	24.00
Temperature Correction factor, K	0.9991	0.9991	0.9991
Specific gravity at 20oc	2.71	2.75	2.69
Average Specific gravity at 20oc, Gs	2.72		

Sample Location	**BK**		
Trial Number	1	2	3
Weight of dry, clean pycnometer, Wp (g)	18.53	19.41	19.11
Weight of dry, clean pycnometer + soil, Wps (g)	28.92	29.40	29.10
Weight of oven dry soil , Ws (g)	10.00	10.00	10.00
Weight of pycnometer + water, Wpw (g)	43.59	46.27	45.68
Weight of pycnometer + soil + water, Wpws (g)	49.90	52.60	51.95
Observed temperature of water, Ti (oC)	22.00	23.00	23.00
Temperature, Tx (oC)	24.00	24.00	24.00
Temperature Correction factor, K	0.9991	0.9991	0.9991
Specific gravity at 20oc	2.71	2.72	2.68
Average Specific gravity at 20oc, Gs	2.70		

Kaolin			
Trial Number	1	2	4
Weight of dry, clean pycnometer, Wp (g)	16.17	19.10	16.12
Weight of dry, clean pycnometer + soil, Wps (g)	26.35	29.95	26.91
Weight of oven dry soil , Ws (g)	10.00	10.00	10.00
Weight of pycnometer + water, Wpw (g)	44.32	45.54	44.21
Weight of pycnometer + soil + water, Wpws (g)	50.56	51.70	50.32
Observed temperature of water, Ti (oC)	23.00	23.00	23.00
Temperature, Tx (oC)	24.00	25.00	24.00
Temperature Correction factor, K	0.9991	0.9988	0.9991
Specific gravity at 20oc	2.66	2.60	2.57
Average Specific gravity at 20oc, Gs	2.61		

APPENDIX E: Grain Size Distribution Test Analysis Data

I. Wet Sieve analysis

WYR Soil Sample					
sieve number	**sieve size (mm)**	**mass of retained (g)**	**percentage of retained %**	**percentage of commutative retained (%)**	**percentage of finer particle (%)**
	9.500	0.00	0.00	0.00	100.00
4	4.750	3.67	0.73	0.73	99.27
10	2.000	4.42	0.88	1.62	98.38
20	0.850	3.88	0.78	2.39	97.61
40	0.425	1.21	0.24	2.64	97.36
60	0.25	1.71	0.34	2.98	97.02
140	0.150	1.22	0.24	3.22	96.78
200	0.075	1.84	0.37	3.59	96.41
	pan	482.05	96.41	100.00	0.00
	total	500	998.76		100.00

BK Soil Sample					
sieve number	**sieve size (mm)**	**mass of retained (g)**	**percentage of retained %**	**percentage of commutative retained (%)**	**percentage of finer particle (%)**
	9.500	0.00	0.00	0.00	100.00
4	4.750	0.95	0.19	0.19	99.81
10	2.000	3.79	0.76	0.95	99.05
20	0.850	2.91	0.58	1.53	98.47
40	0.425	3.47	0.69	2.22	97.78
60	0.250	3.88	0.78	3.00	97.00
140	0.150	2.83	0.57	3.57	96.43
200	0.075	2.44	0.49	4.05	95.95
	pan	479.73	95.95	100.00	0.00
	total	500	998.76		100.00

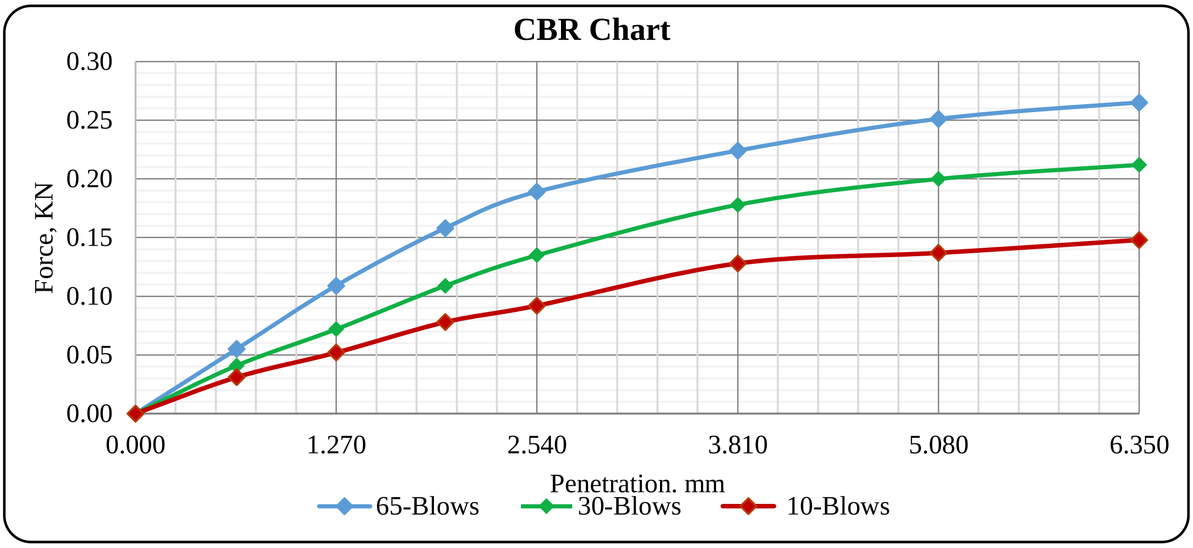
II. Combined wet sieve analysis and hydrometer analysis

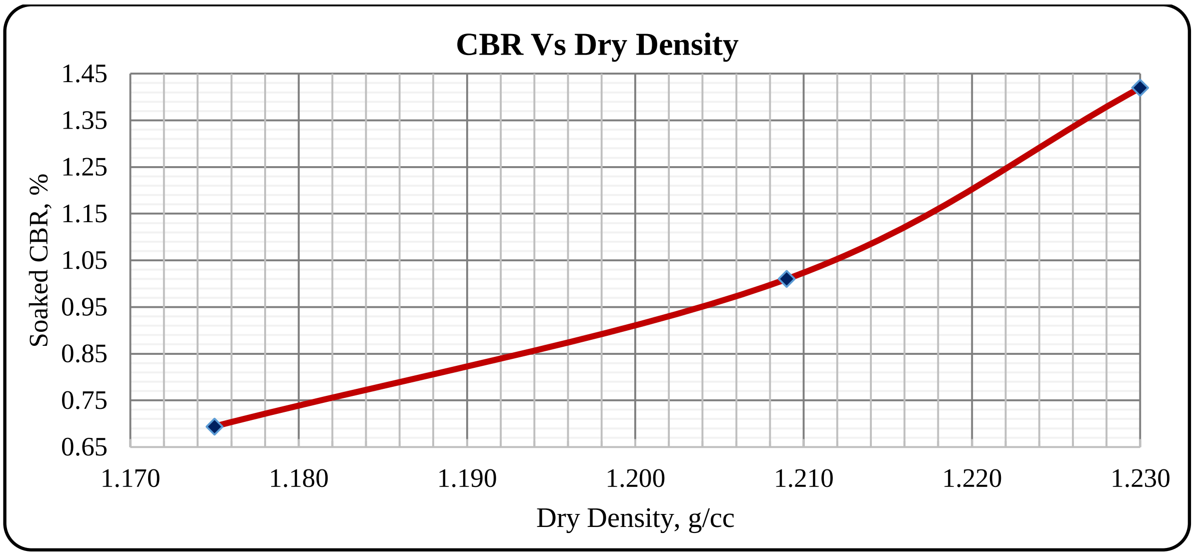
	WYR	BK
Sieve opening	Percent passing	Percent passing
9.5	100	100
4.750	99.266	99.810
2.000	98.382	99.052
0.850	97.606	98.470
0.425	97.364	97.776
0.250	97.022	97.000
0.150	96.778	96.434
0.075	96.410	95.946
0.057	92.101	87.350
0.040	90.107	83.535
0.029	88.114	81.628
0.021	86.120	79.721
0.015	84.127	75.907
0.011	82.133	72.092
0.008	78.146	68.278
0.006	74.159	62.556
0.004	70.172	56.835
0.003	66.185	53.020
0.002	60.205	50.299
0.001	58.211	43.484

APPENDIX F: California Bearing Ratio (CBR) Test Analysis Data

PENETRATION AND LOAD DETERMINATION OF UNTREATED SOIL @ WYR						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	0.189	1.419	0.135	1.014	0.092	0.691
5.080	0.251	1.255	0.200	1.000	0.137	0.685

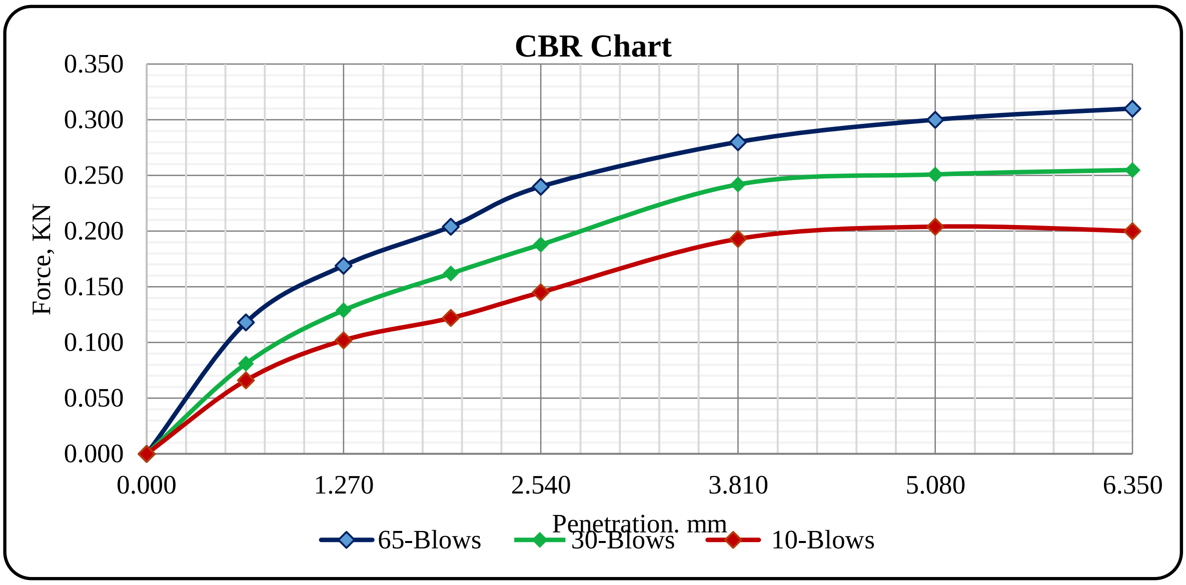
CBR RESULT SUMMARY OF UNTREATED SOIL @ WYR			
MMDD	1.276		
Dry Density at 95% of MDD	1.21		
No of Blows	65	30	10
CBR Values (%)	1.42	1.01	0.69
DDBS g/cc	1.230	1.209	1.175
CBR at 95% MDD	**0.98 %**		

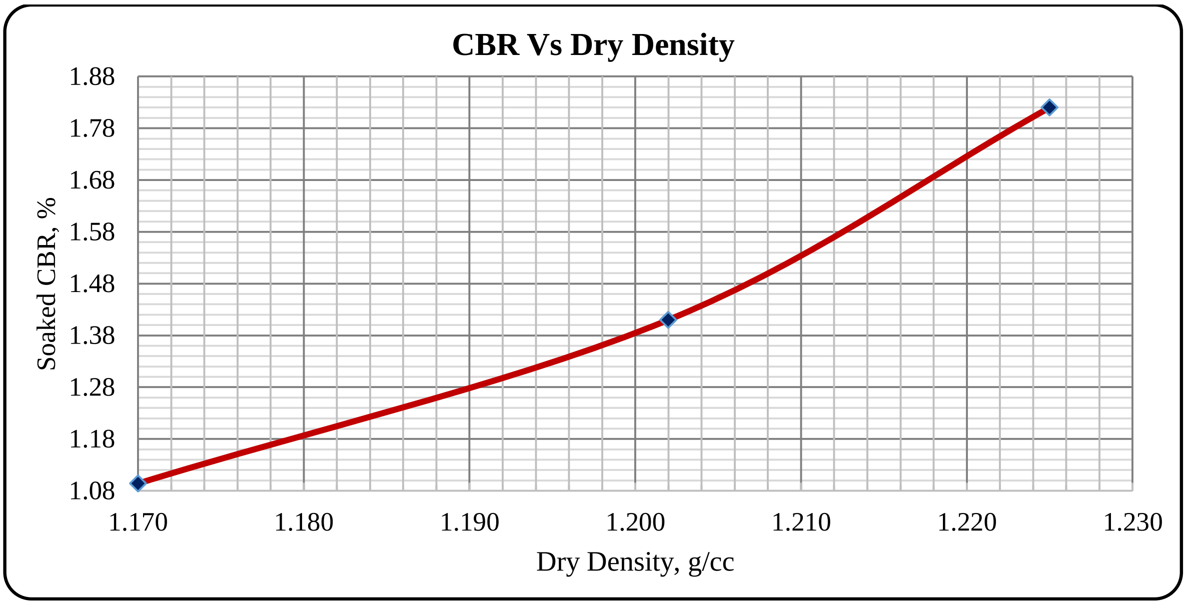




PENETRATION AND LOAD DETERMINATION OF K10%+C0% @ WYR						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	0.240	1.802	0.188	1.411	0.145	1.089
5.080	0.300	1.500	0.251	1.255	0.204	1.020

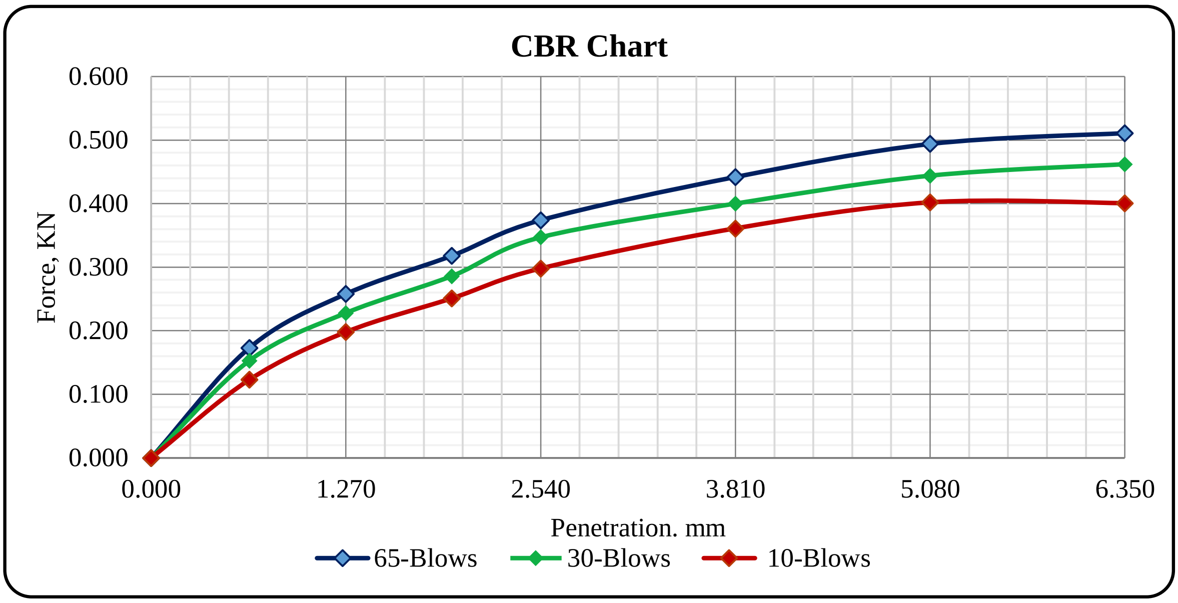
CBR RESULT SUMMARY OF K10%+C0% @ WYR			
MMDD	1.253		
Dry Density at 95% of MDD	1.190		
No of Blows	65	30	10
CBR Values (%)	1.82	1.41	1.09
DDBS g/cc	1.225	1.202	1.170
CBR at 95% MDD	**1.52 %**		

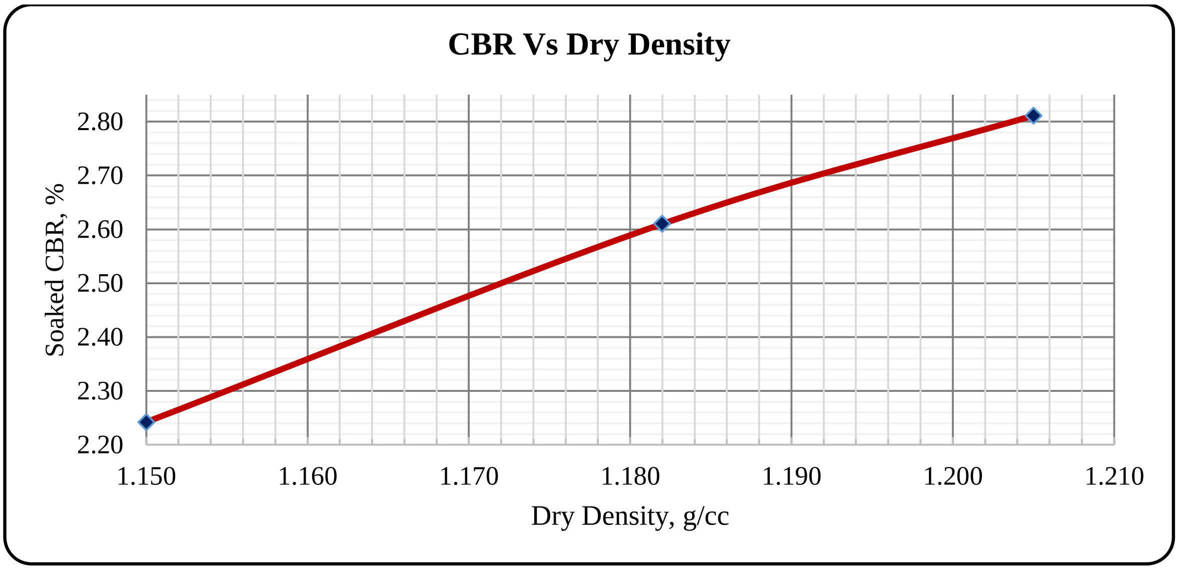




PENETRATION AND LOAD DETERMINATION OF K8%+C2% @ WYR						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	0.374	2.808	0.347	2.605	0.298	2.237
5.080	0.494	2.470	0.444	2.220	0.402	2.010

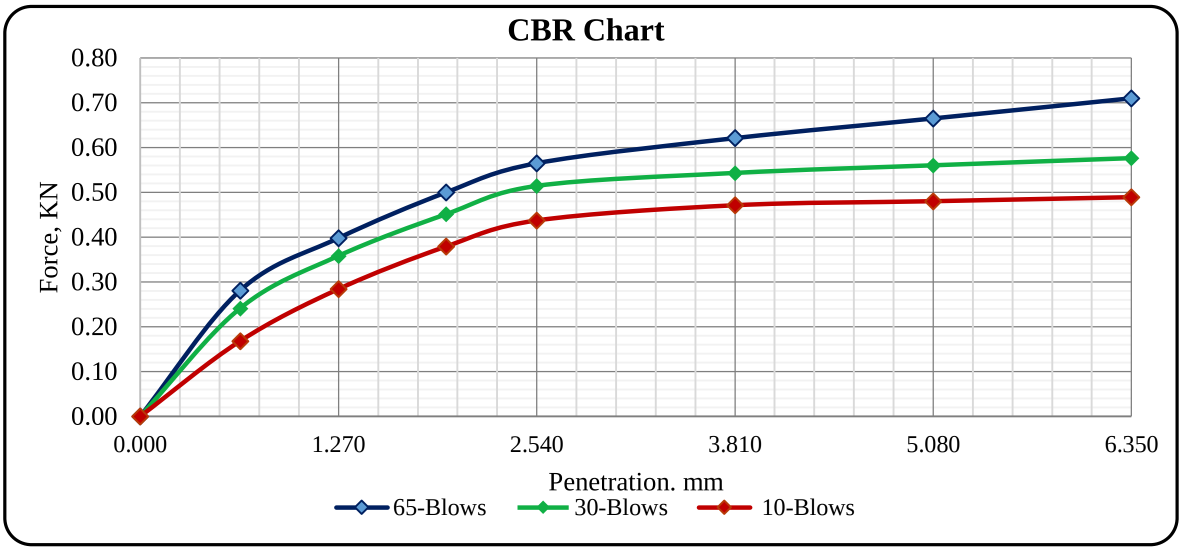
CBR RESULT SUMMARY OF K8%+C2% @ WYR			
MMDD	1.233		
Dry Density at 95% of MDD	1.171		
No of Blows	65	30	10
CBR Values (%)	2.81	2.61	2.24
DDBS g/cc	1.205	1.182	1.150
CBR at 95% MDD	**2.48 %**		

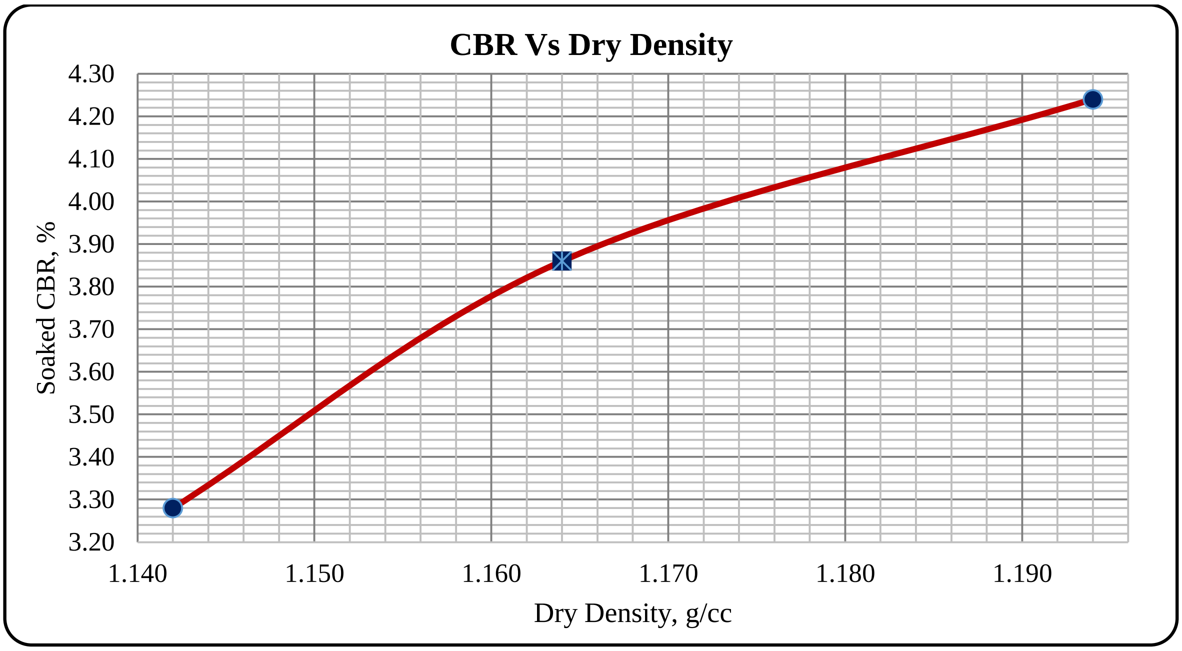




PENETRATION AND LOAD DETERMINATION OF K6%+C4% @ WYR						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	0.565	4.242	0.514	3.859	0.437	3.281
5.080	0.665	3.325	0.560	2.800	0.480	2.400

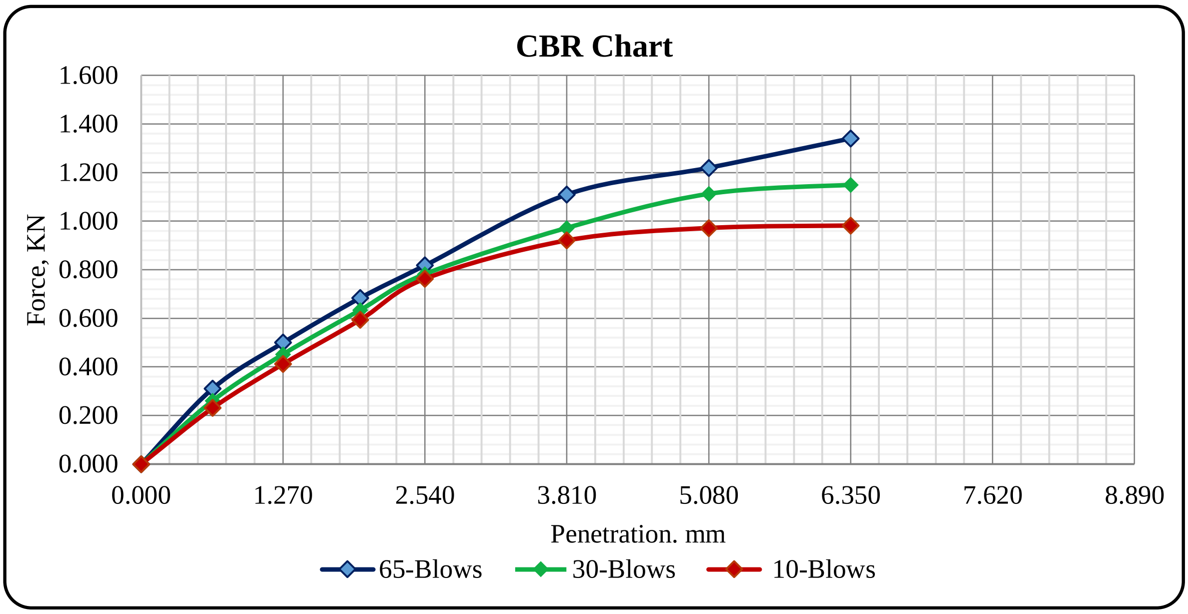
CBR RESULT SUMMARY OF K6%+C4% @ WYR			
MMDD	1.223		
Dry Density at 95% of MDD	1.162		
No of Blows	65	30	10
CBR Values (%)	4.24	3.86	3.28
DDBS g/cc	1.194	1.164	1.142
CBR at 95% MDD	**3.78%**		

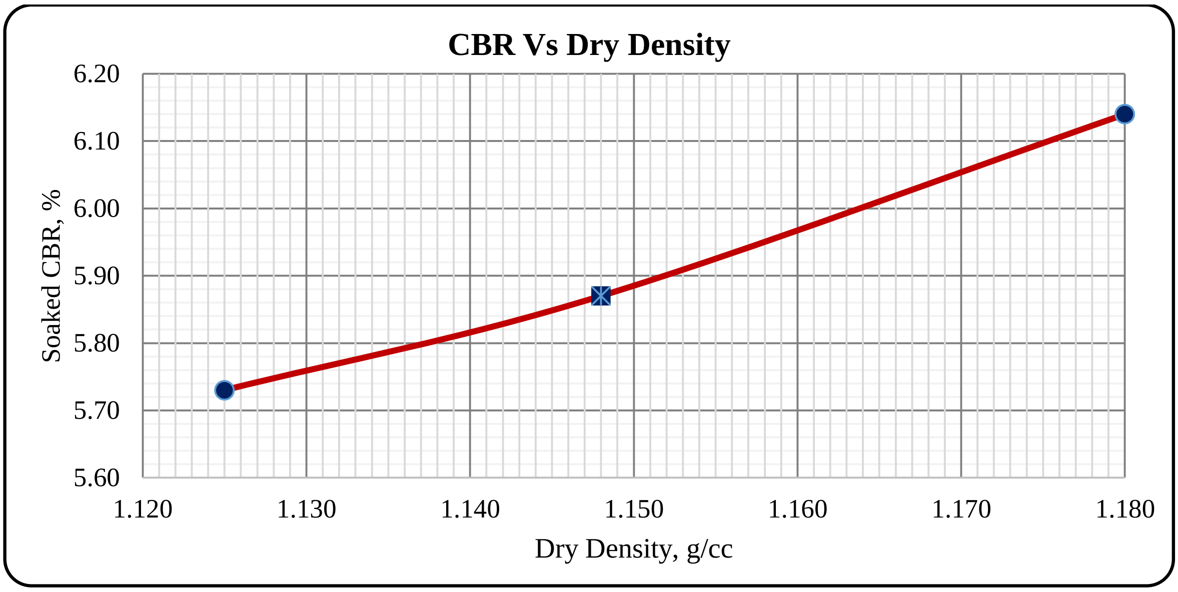




PENETRATION AND LOAD DETERMINATION OF K4%+C6% @ WYR						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	0.818	6.141	0.782	5.871	0.763	5.728
5.080	1.220	6.100	1.112	5.560	0.972	4.860

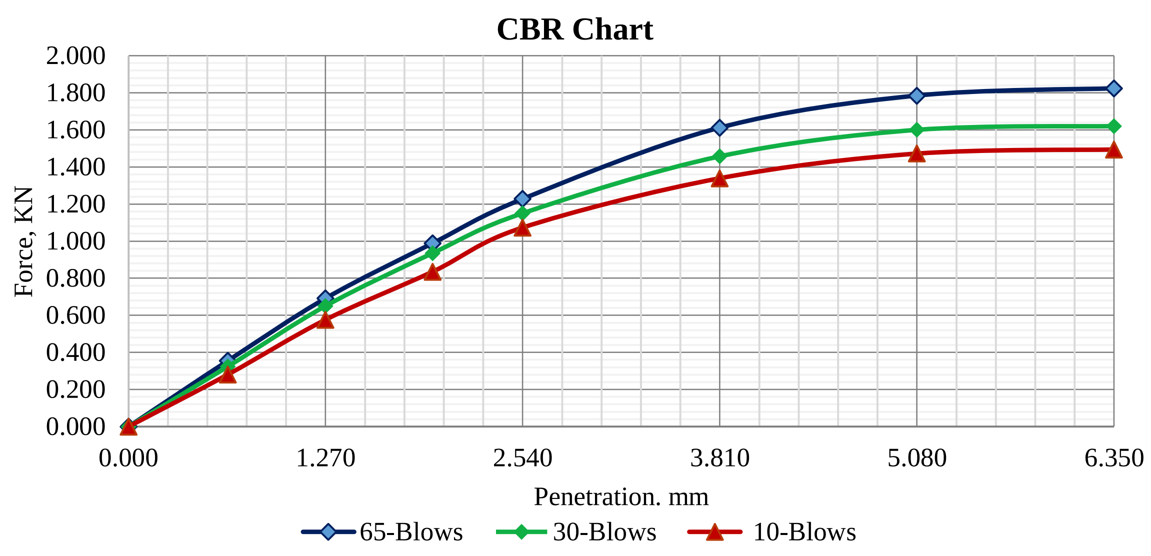
CBR RESULT SUMMARY OF K4%+C6% @ WYR			
MMDD	1.213		
Dry Density at 95% of MDD	1.152		
No of Blows	65	30	10
CBR Values (%)	6.14	5.87	5.73
DDBS g/cc	1.180	1.148	1.125
CBR at 95% MDD	**5.90%**		

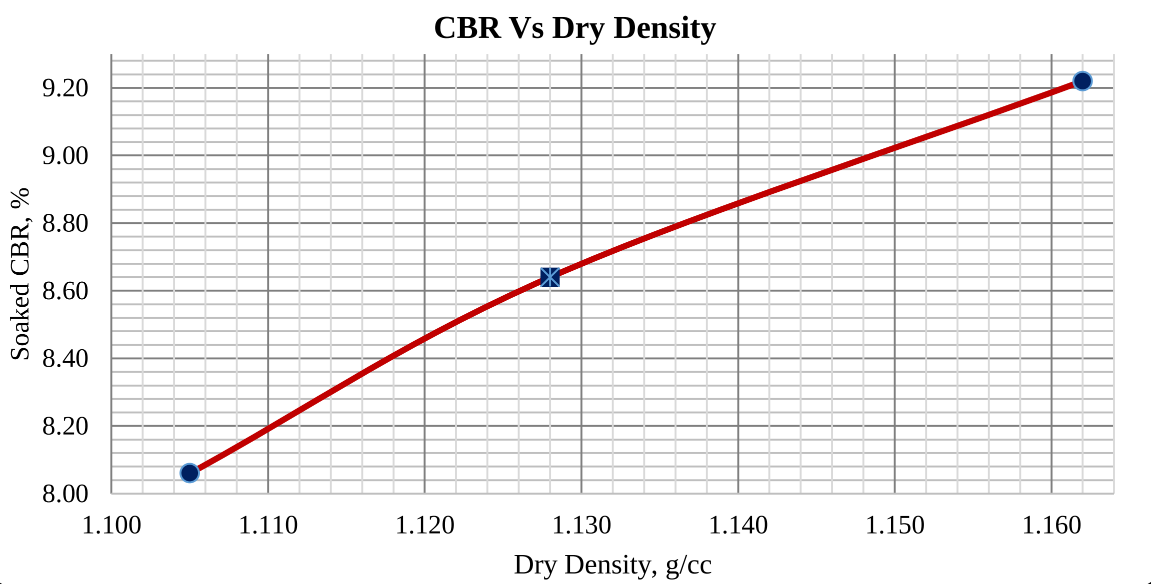




PENETRATION AND LOAD DETERMINATION OF K2%+C8% @ WYR						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	1.228	9.219	1.151	8.641	1.074	8.063
5.080	1.785	8.925	1.601	8.005	1.473	7.365

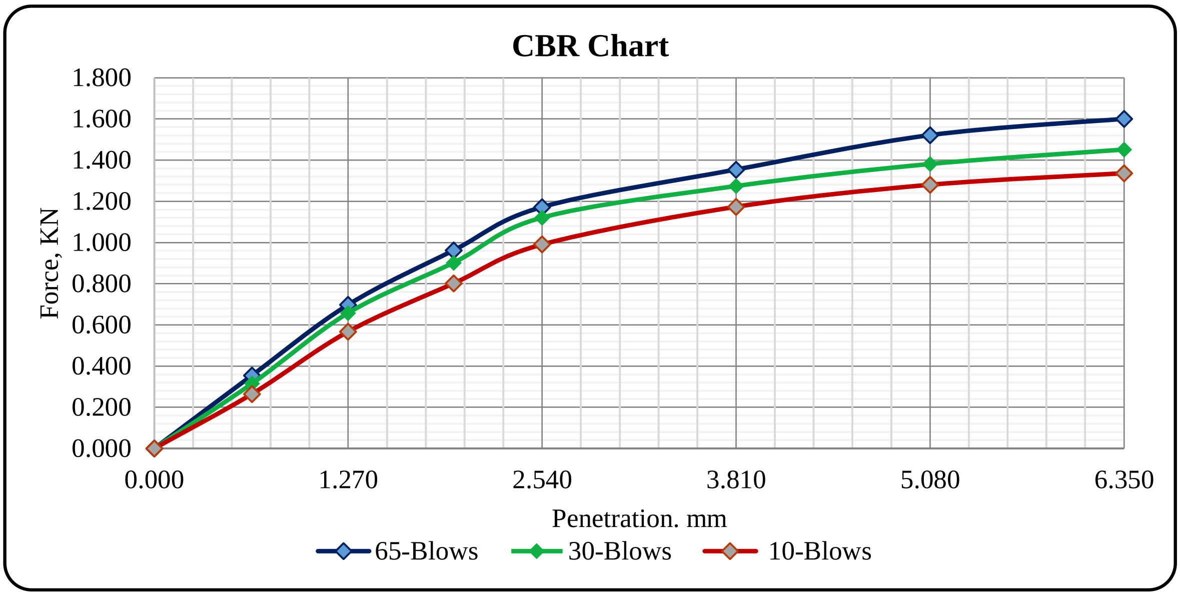
CBR RESULT SUMMARY OF K2%+C8% @ WYR			
MMDD	1.199		
Dry Density at 95% of MDD	1.14		
No of Blows	65	30	10
CBR Values (%)	9.22	8.64	8.06
DDBS g/cc	1.162	1.128	1.105
CBR at 95% MDD	**8.86%**		

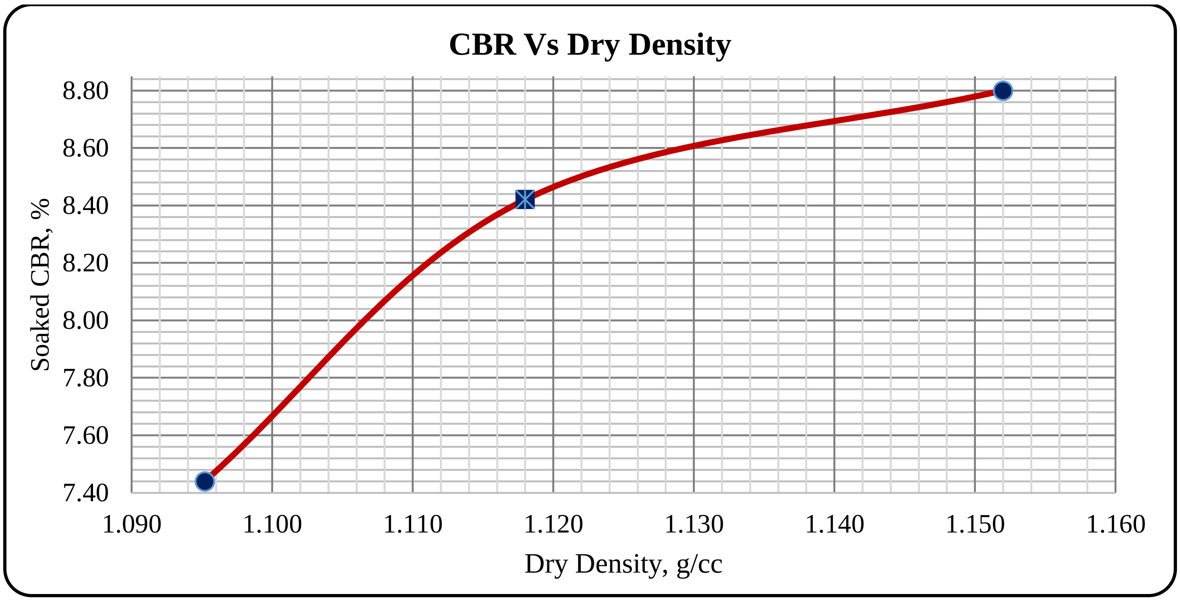




PENETRATION AND LOAD DETERMINATION OF K0%+C10% @ WYR						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	1.172	8.799	1.121	8.416	0.991	7.440
5.080	1.521	7.605	1.381	6.905	1.281	6.405

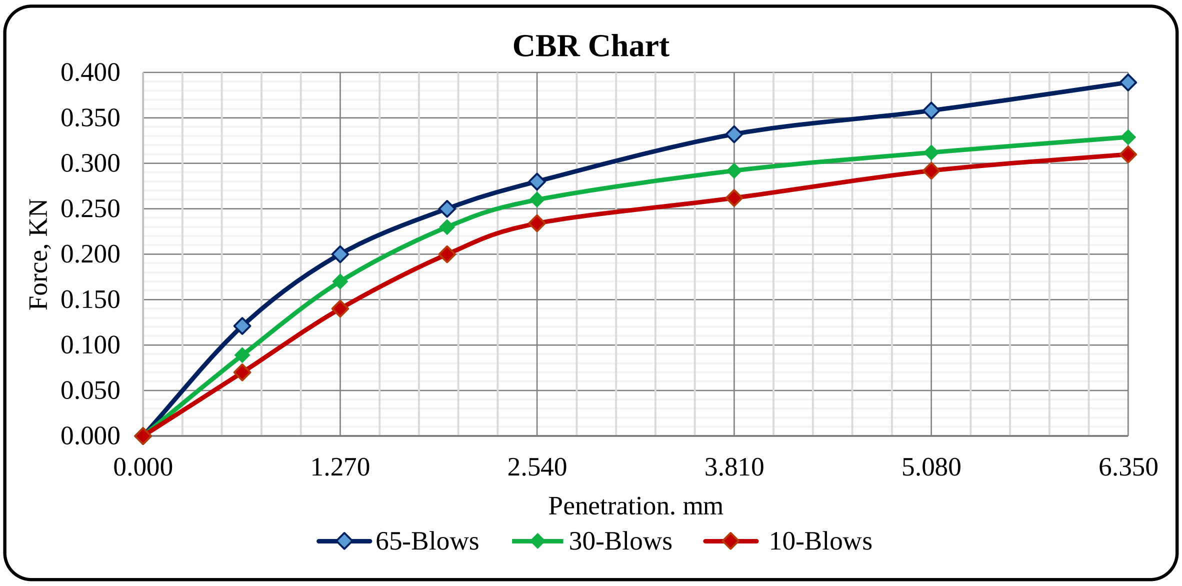
CBR RESULT SUMMARY OF K0%+C10% @ WYR			
MMDD	1.194		
Dry Density at 95% of MDD	1.13		
No of Blows	65	30	10
CBR Values (%)	8.80	8.42	7.44
DDBS g/cc	1.152	1.118	1.095
CBR at 95% MDD	**8.62%**		

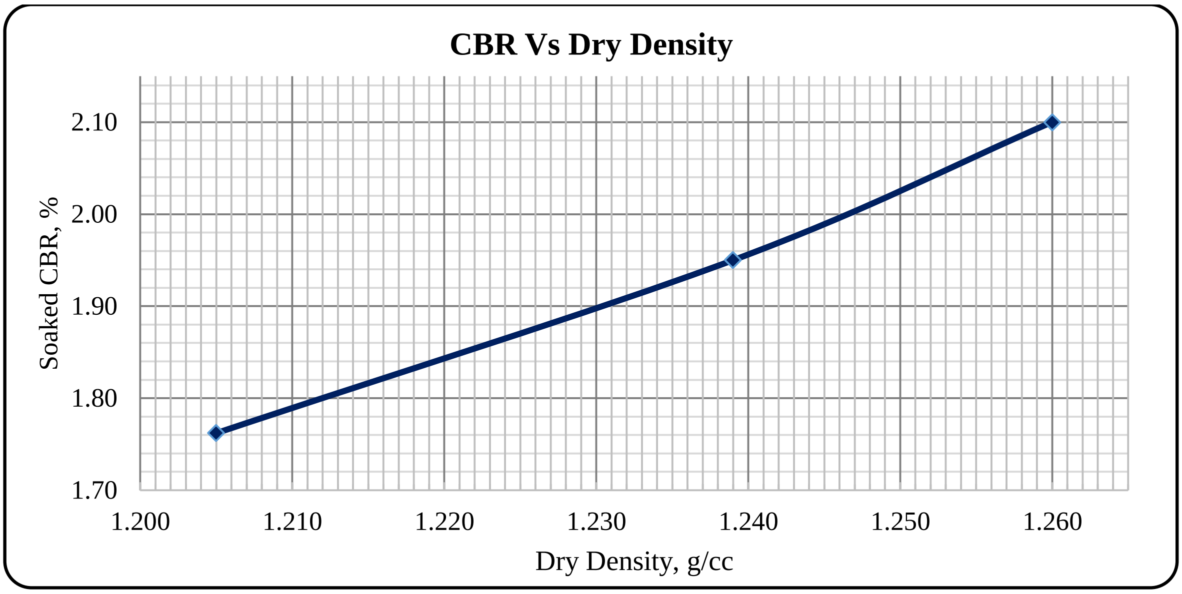




PENETRATION AND LOAD DETERMINATION OF UNTREATED SOIL @ BK						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	0.280	2.10	0.260	1.95	0.234	1.76
5.080	0.358	1.79	0.312	1.56	0.292	1.46

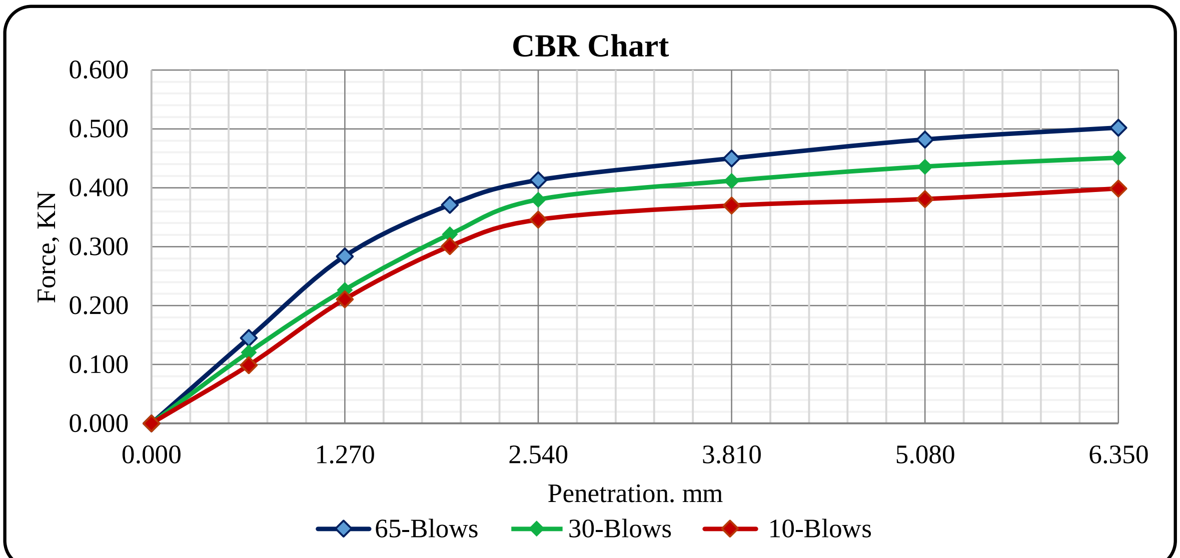
CBR RESULT SUMMARY OF K0%+C10% @ BK			
MMDD	1.313		
Dry Density at 95% of MDD	1.25		
No of Blows	65	30	10
CBR Values (%)	2.10	1.95	1.76
DDBS g/cc	1.260	1.239	1.205
CBR at 95% MDD	**2.02%**		

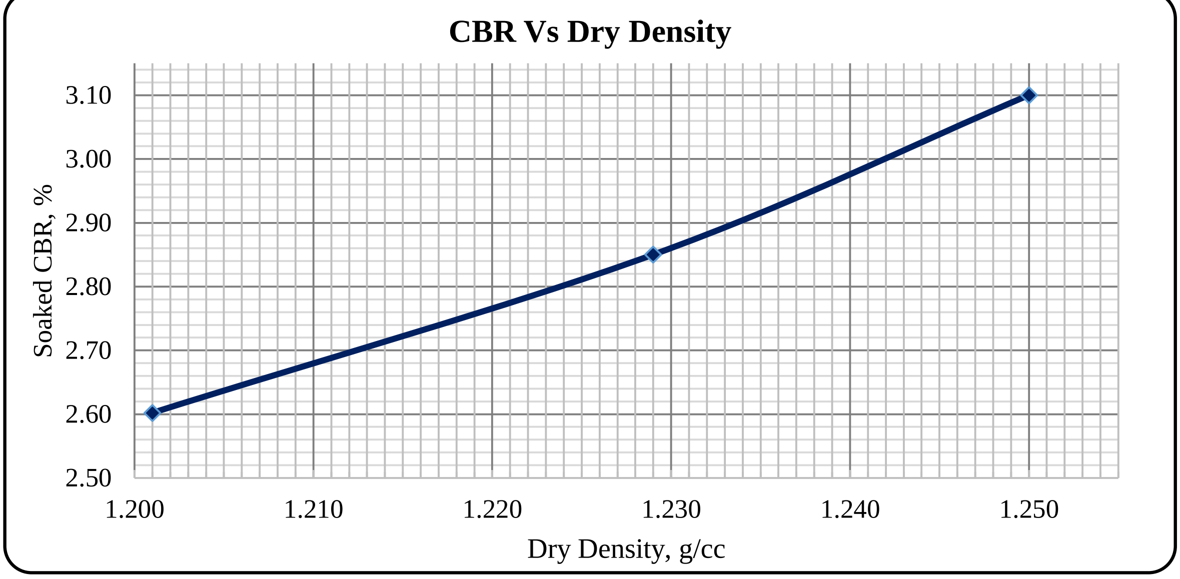




PENETRATION AND LOAD DETERMINATION OF K0%+C10% @ BK						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	0.413	3.101	0.380	2.853	0.346	2.60
5.080	0.482	2.410	0.436	2.180	0.381	1.905

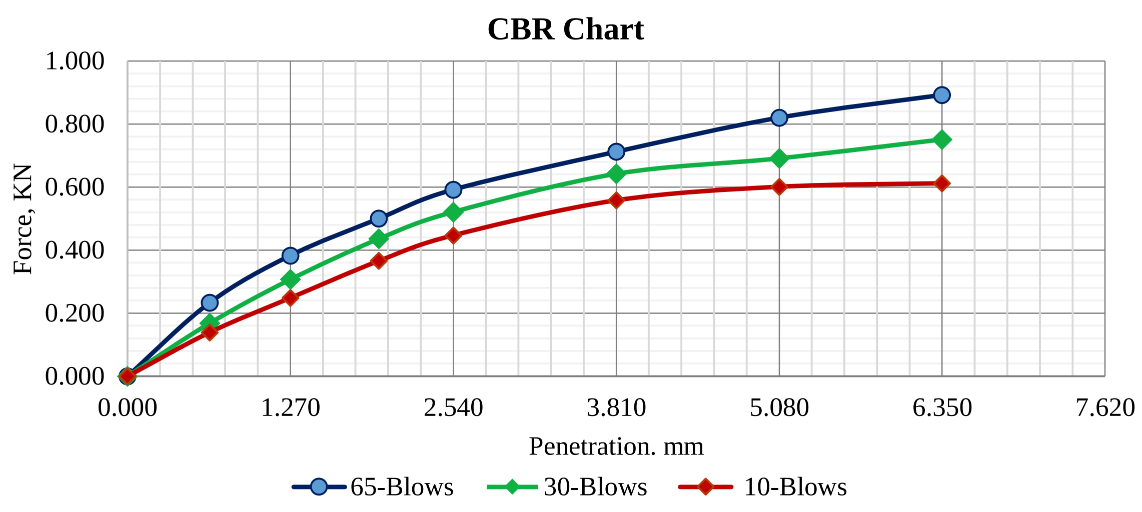
CBR RESULT SUMMARY OF K0%+C10% @ BK			
MMDD	1.3		
Dry Density at 95% of MDD	1.24		
No of Blows	65	30	10
CBR Values (%)	3.10	2.85	2.60
DDBS g/cc	1.250	1.229	1.201
CBR at 95% MDD	**2.92%**		

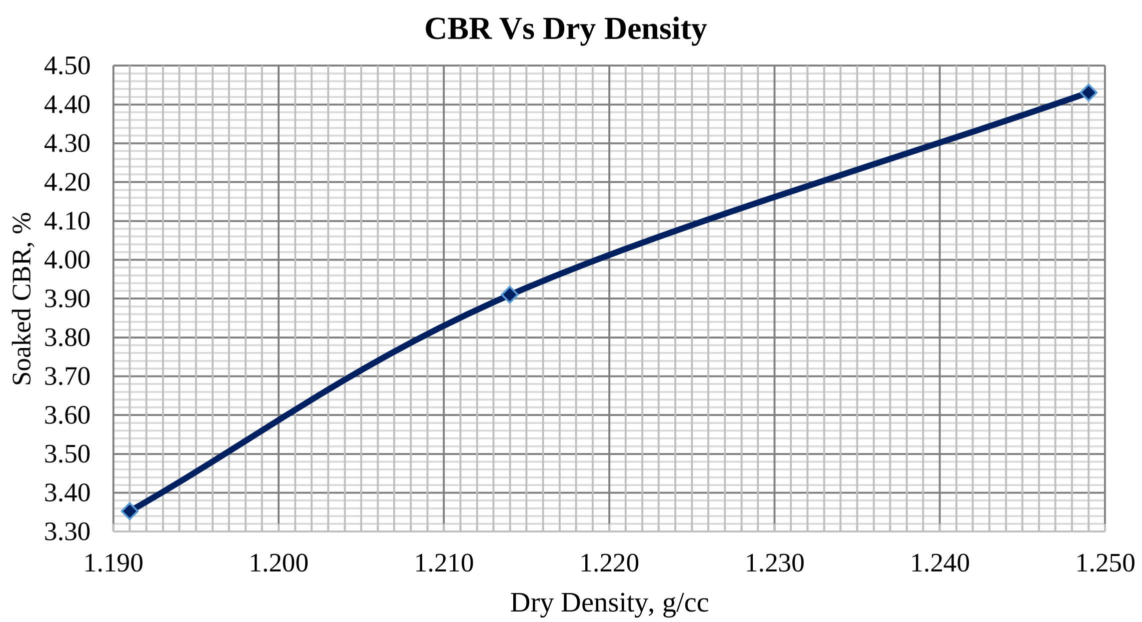




PENETRATION AND LOAD DETERMINATION OF K8%+C2% @ BK						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	0.592	4.44	0.521	3.91	0.447	3.36
5.080	0.820	4.100	0.691	3.455	0.601	3.005

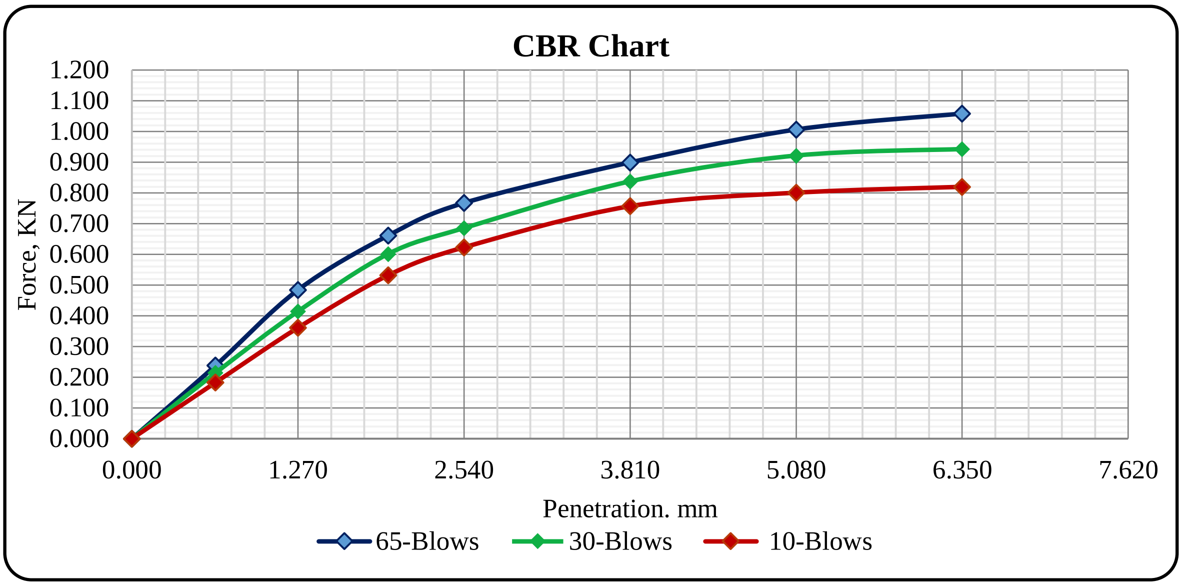
CBR RESULT SUMMARY OF K8%+C2% @ BK			
MMDD	1.283		
Dry Density at 95% of MDD	1.22		
No of Blows	65	30	10
CBR Values (%)	4.43	3.91	3.35
DDBS g/cc	1.249	1.214	1.191
CBR at 95% MDD	**4.02%**		

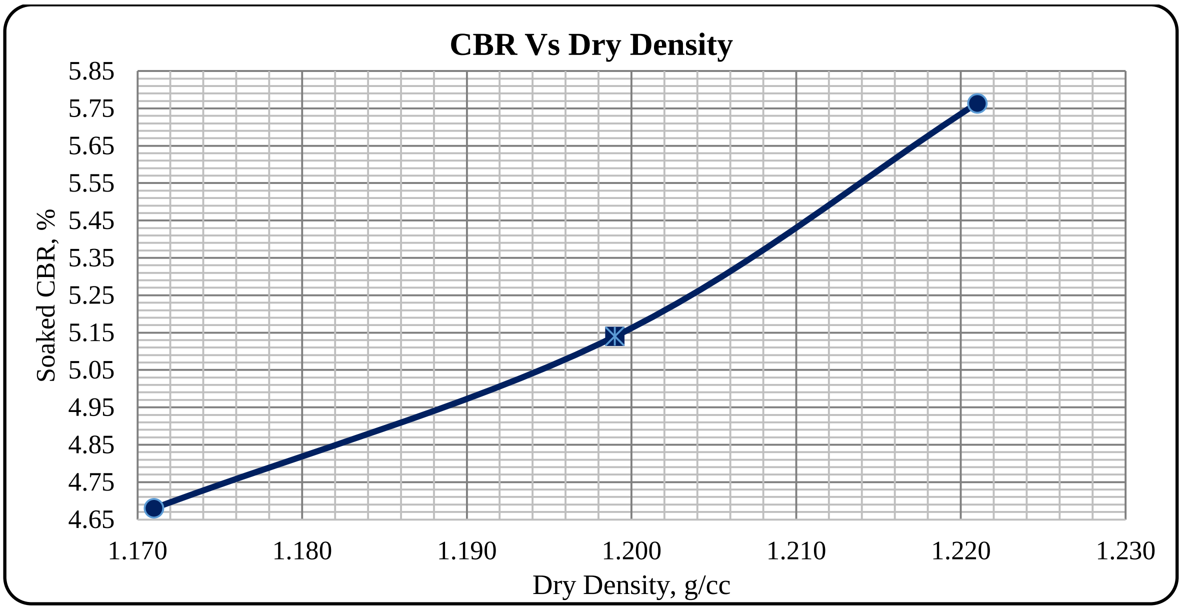




PENETRATION AND LOAD DETERMINATION OF K6%+C4% @ BK						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	0.767	5.758	0.685	5.143	0.623	4.677
5.080	1.006	5.030	0.921	4.605	0.801	4.005

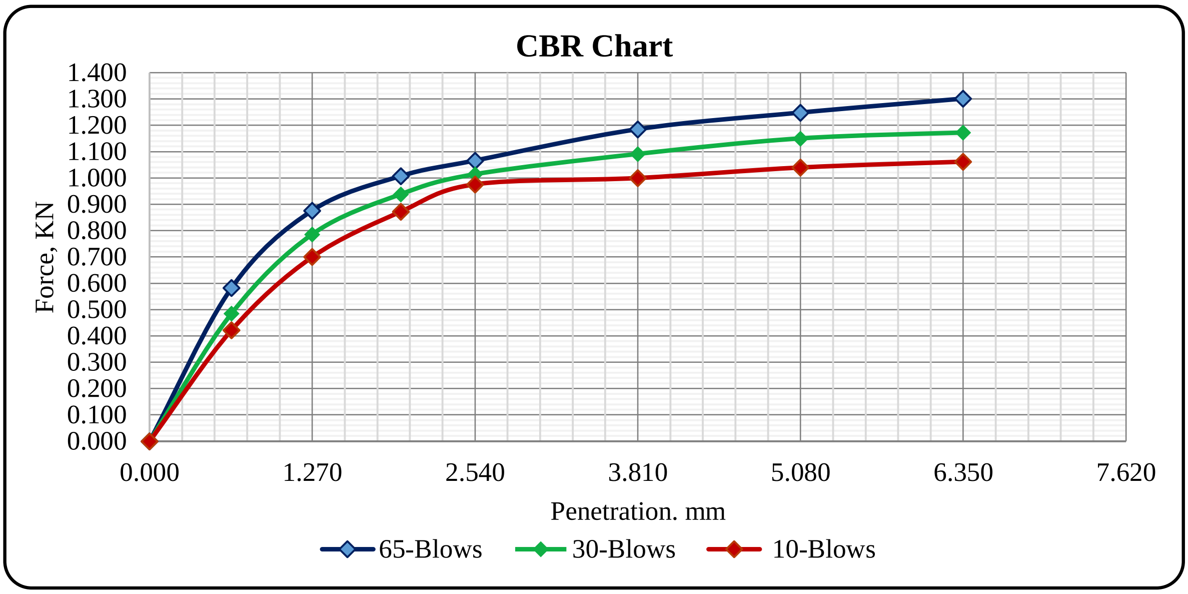
CBR RESULT SUMMARY OF K6%+C4% @ BK			
MMDD	1.27		
Dry Density at 95% of MDD	1.21		
No of Blows	65	30	10
CBR Values (%)	5.76	5.14	4.68
DDBS g/cc	1.221	1.199	1.171
CBR at 95% MDD	**5.42%**		

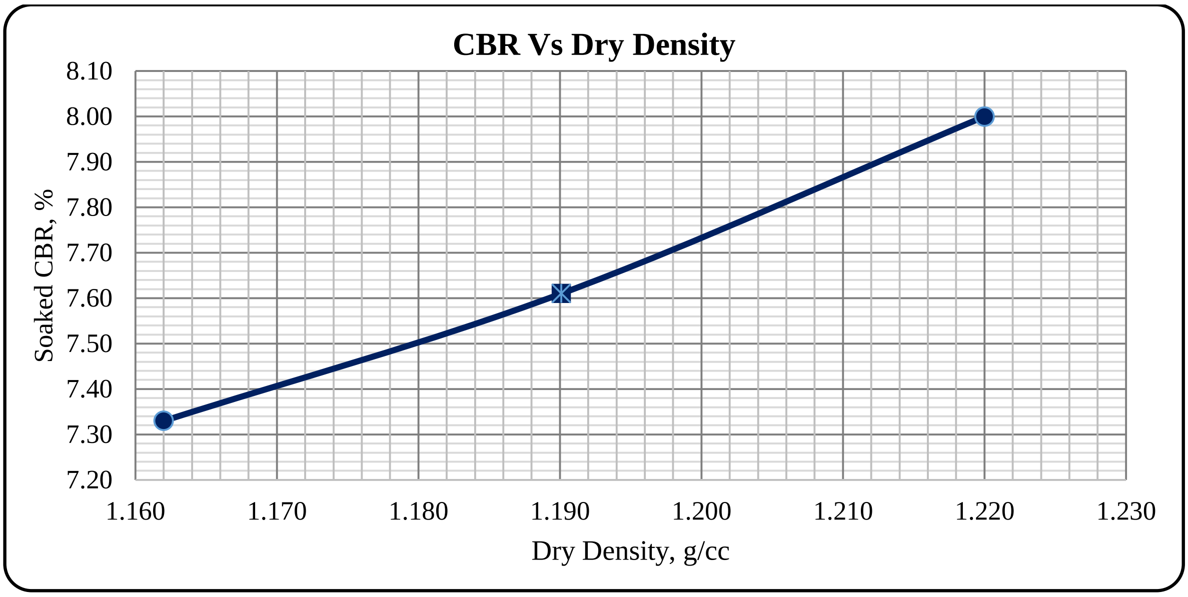




PENETRATION AND LOAD DETERMINATION OF K4%+C6% @ BK						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	1.066	8.00	1.014	7.613	0.976	7.327
5.080	1.248	6.240	1.150	5.750	1.040	5.200

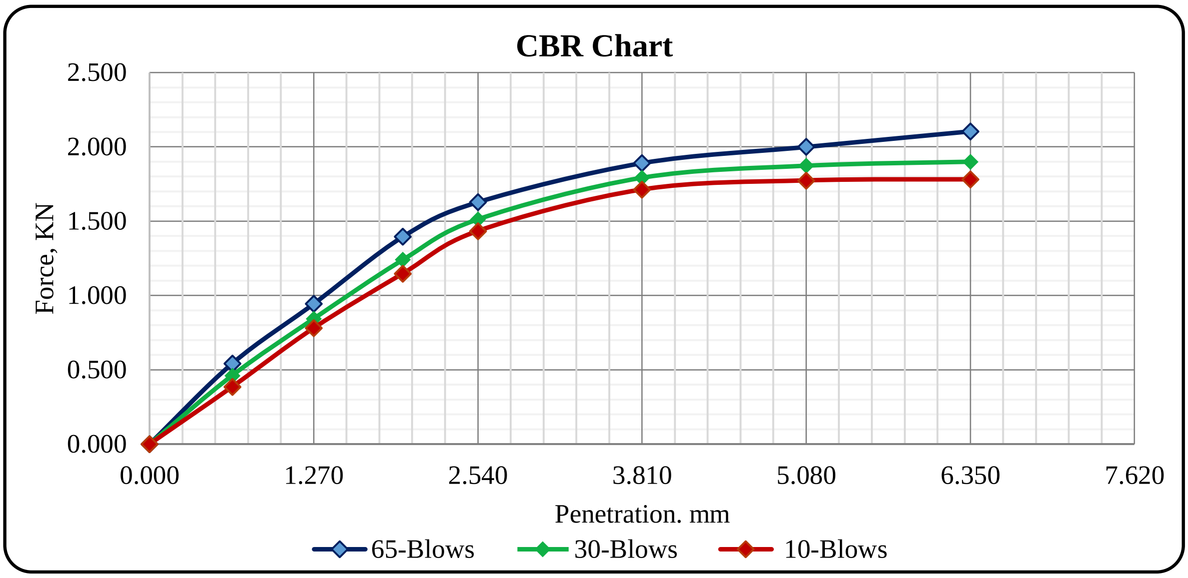
CBR RESULT SUMMARY OF K4%+C6% @ BK			
MMDD	1.26		
Dry Density at 95% of MDD	1.20		
No of Blows	65	30	10
CBR Values (%)	8.00	7.61	7.33
DDBS g/cc	1.220	1.190	1.162
CBR at 95% MDD	**7.75%**		

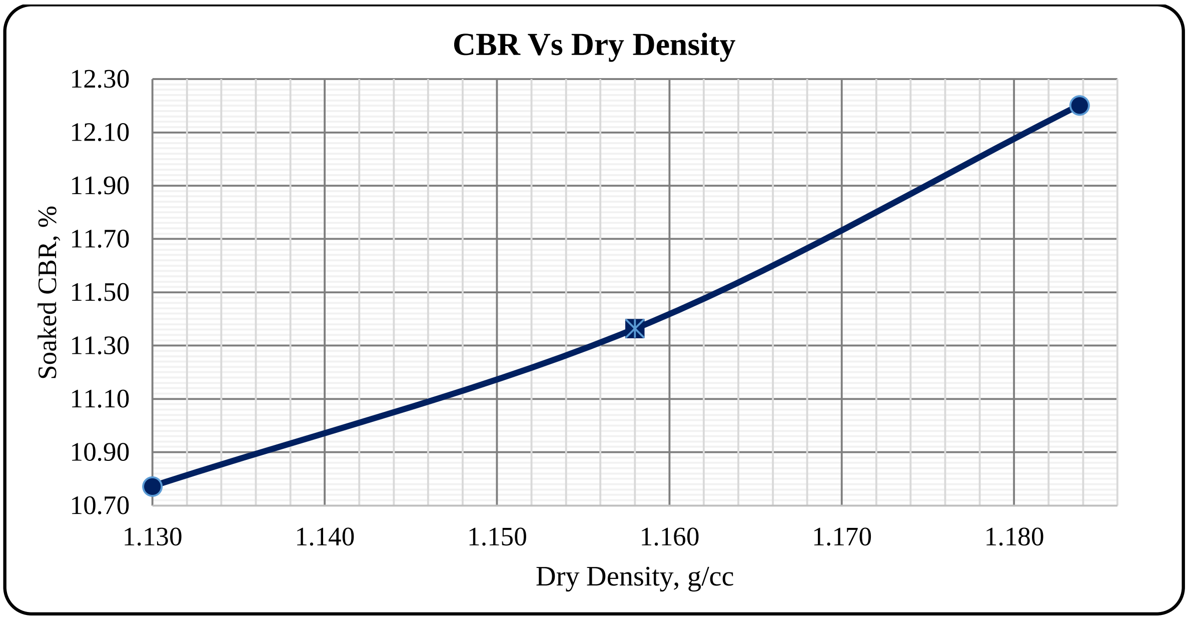




PENETRATION AND LOAD DETERMINATION OF K2%+C8% @ BK						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	1.628	12.22	1.513	11.359	1.434	10.766
5.080	2.000	10.000	1.874	9.370	1.774	8.870

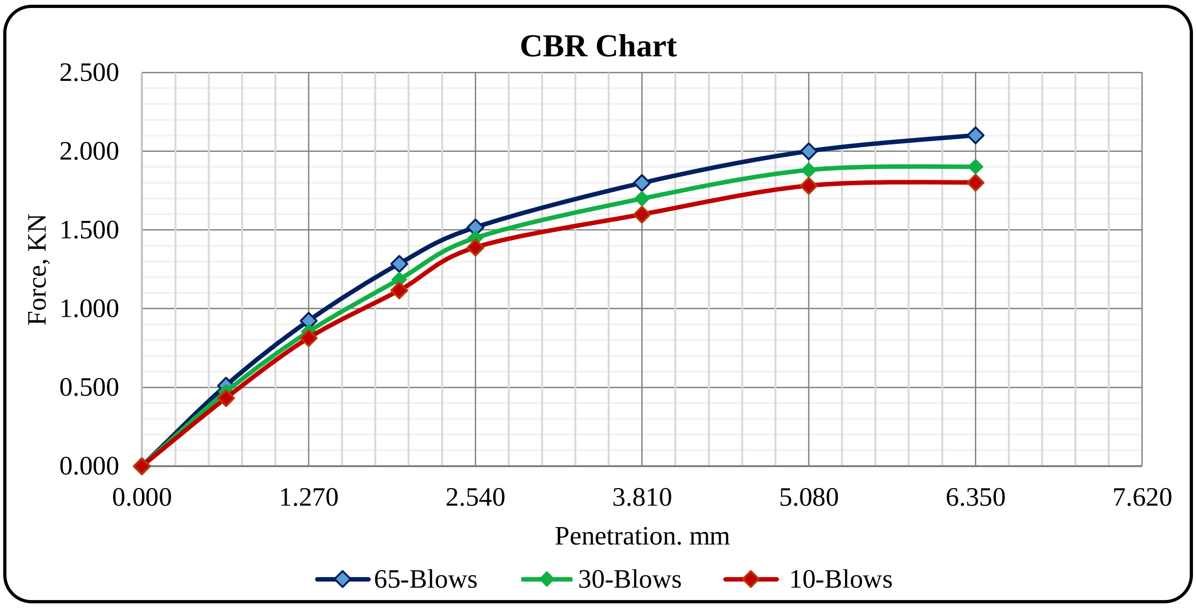
CBR RESULT SUMMARY OF K2%+C8% @ BK			
MMDD	1.229		
Dry Density at 95% of MDD	1.17		
No of Blows	65	30	10
CBR Values (%)	12.20	11.36	10.77
DDBS g/cc	1.184	1.158	1.130
CBR at 95% MDD	**11.72%**		

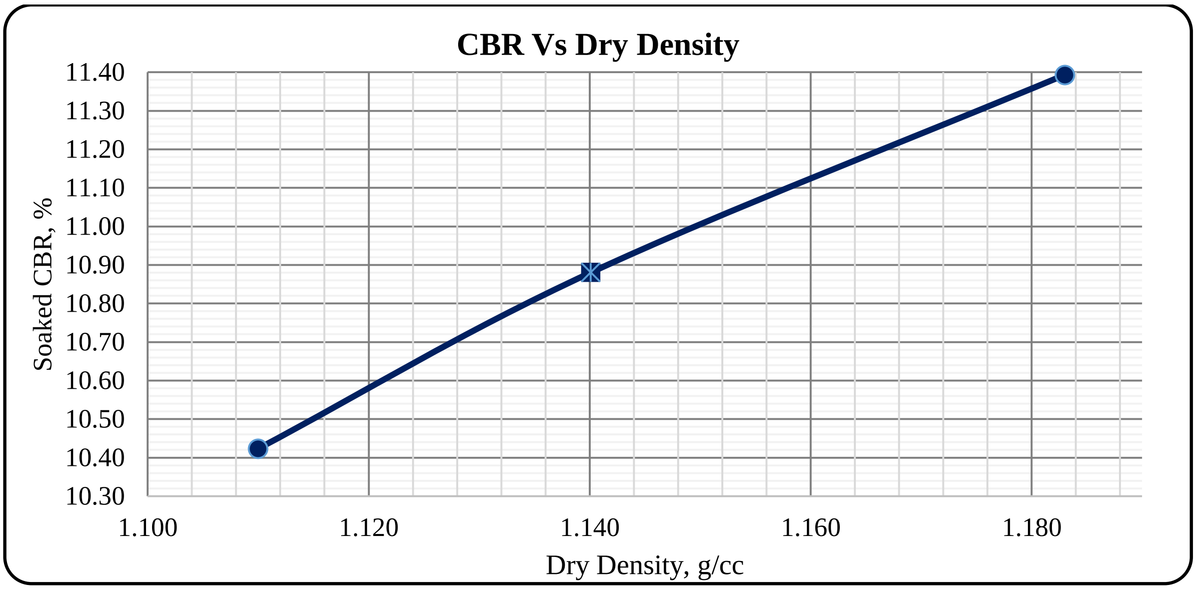




PENETRATION AND LOAD DETERMINATION OF K0%+C10% @ BK						
Penetration Data After 96-hours Soaking						
Penetration (mm)	65-Blows		30-Blows		10-Blows	
	Load (KN)	CBR (%)	Load (KN)	CBR (%)	Load (KN)	CBR (%)
2.540	1.517	11.389	1.449	10.878	1.389	10.428
5.080	2.001	10.005	1.881	9.405	1.781	8.905

CBR RESULT SUMMARY OF K0%+C10% @ BK			
MMDD	1.21		
Dry Density at 95% of MDD	1.150		
No of Blows	65	30	10
CBR Values (%)	11.39	10.88	10.42
DDBS g/cc	1.183	1.140	1.110
CBR at 95% MDD	**11.01%**		





APPENDIX G: Natural Moisture Content

Sample Location	WYR		
Trial Number	**1**	**2**	**3**
can number	F11	P61	P01
mass of can(Mc), g	17.21	17.05	16.99
mass of can + moist soil (Mcms), g	82.07	100.03	88.62
Mass of can + mass of oven dried soil(Mcds), g	61.00	74.17	65.99
Mass of water (Mw), g	21.07	25.86	22.63
Mass of dry soil(Ms), g	43.79	57.12	49.00
Water Content(w), %	48.12	45.27	46.18
Average water content(w), %	46.52		

Sample Location	BK		
Trial Number	**1**	**2**	**3**
can number	ZE	35	2
mass of can(Mc), g	17.34	17.29	17.41
mass of can + moist soil (Mcms), g	76.97	95.01	86.65
Mass of can + mass of oven dried soil(Mcds), g	59.99	74.08	66.67
Mass of water (Mw), g	16.98	20.93	19.98
Mass of dry soil(Ms), g	42.65	56.79	49.26
Water Content(w), %	39.81	36.86	40.56
Average water content(w), %	39.08		