ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

STABILIZATION OF EXPANSIVE CLAY SOIL WITH SUGAR CANE MOLASESS AND CEMENT

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

Master of Science in Civil Engineering

(Road and Transport Engineering)

By: Bizualem Taye

Advisor: Alemgena Alene (PhD)

May, 2015

Addis Ababa, Ethiopia

ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUTE STUDIES

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

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

By

Bizualem Taye

Approved by Board of Examiners

1.________________________	____________________	__________________
Advisor	**Signature**	**Date**
2.________________________	____________________	__________________
External Examiner	**Signature**	**Date**
3.________________________	____________________	__________________
Internal Examiner	**Signature**	**Date**
4.________________________	____________________	__________________
Chairman	**Signature**	**Date**

DECLARATION

I hereby declare that the thesis entitled ***STABILIZATION OF EXPANSIVE CLAY SOIL WITH SUGAR CANE MOLASESS AND CEMENT*** has been carried out by me under the supervision of Doctor Alemgena Alene, Department of Addis Ababa University during the year 2014-15 as part of Master of Science Program in Road and Transport Engineering. I further declare that this work has not been submitted to any other University or institution for the award of any degree or diploma.

Bizualem Taye

Signature: __________

Place and date of submission: School of Graduate Studies, Addis Ababa University

May 2015

Contact Address:

- Email: bizualemtaye@gamil.com
- Tel: +251912-08-52-60

ACKNOWLEDGEMENT

First of all I would like to thank my advisor Dr. Alemgena Alene for his invaluable advices, his kind encouragement and his consistent support throughout the study.

I want to express my sincere gratitude to Ethiopian Roads Authority (ERA) for giving me this prestigious scholarship chance.

My special thanks also go to the following companies who allowed me to use their laboratory for conducting tests: Ethiopian Road Construction Corporation (ERCC) Laboratory at Chancho-Derba-Becho Road Construction Project, CORE Consulting Engineer PLC and Transport Construction and Design (TCD) Share Company. My heartfelt gratitude also goes to the laboratory technicians who have offered their technical skills throughout the labor intensive test procedures.

Finally, I am greatly indebted to the love and support of my family. Thus, I would like to dedicate this thesis work to my wife W/ro Selamawit Fentahun, my brother Habtamu Taye and my son Kidus and say **"Thank You"**.

Bizualem Taye

ABSTRACT

Soil stabilization with the application of Portland cement is one of the most popular methods of soil stabilization. Research showed that molasses can modify properties of expansive soil. Former research has proved that sugar containing molasses improves the quality of reaction between cement and aggregate. Molasses is a local material easily obtained from sugar factories and cheaper than cement. Therefore, a research must be carried on to explore the effect of molasses addition to soil stabilization with Ordinary Portland cement on geotechnical properties.

Research was done on expansive clay soil sample from Modjo-Ejere Road, with addition of molasses alone, cement alone, and combination of cement and molasses by varying content of stabilizers in steeped concentration of 0, 4, 8 and 12% each by dry weight of the soil, was used to treat the soil. Furthermore, the sample soil was treated with combination of cement and molasses by keeping 4% molasses constant and varying cement content to 4, 8 and 12% by dry weight of the soil .i.e. at 1:1, 2:1 and 3:1 cement to molasses ratio. For the analysis of the effect of the stabilizer on soil, comparison was made on geotechnical properties of the native soil and stabilized soil. The comparison includes by carrying out: pH test, compaction test, Atterberg's limit test, linear shrinkage test, California Bearing Ration (CBR) test, Unconfined Compressive Strength (UCS) test and swelling test on both the native soil and stabilized soil.

Sample soils with molasses application had a relatively higher strength, lower PI (plasticity index) and swelling potentials than those of the native soil. The addition of cement on the sample soil gave significant improvement in strength, eliminated swelling properties and were more effective in improving properties of the natural soil than molasses. However, shrinkage cracks were observed during shrinkage tests and stress-strain curve of the UCS test showed that the soil stabilized with cement had brittle nature. Molasses application on soil-cement mixture gave higher strength, lower PI and negligible swelling potential than those of cement stabilized or molasses stabilized soils.

Molasses application improves soil-cement reaction with water and cause larger size grains than cement /molasses treated soils. Application of molasses on soil-cement mixture has been proven to increases soil strength, to eliminate shrinkage cracks and to reduce brittleness nature of cement stabilized soils. Addition of 4% molasses to 4% cement increased CBR value of 1% of the native soil to 64%, reduced 53% PI value of the native soil to 19%, and reduced 10.4% CBR swell value of the native soil to negligible values. Therefore; soil stabilized with 4% molasses and 4% cement combination satisfied all specification requirements for stabilized sub-grade soil and selected as optimum content of stabilizer for sub-grade soils.

Key Words: Cement, Expansive Clay Soil, Modjo-Ejere, Sugar Cane Molasses, Soil Modification, Soil Stabilization.

Table of Contents

LIST OF ABBREVIATIONS /ACRONYMS

- **AACRA**……… Addis Ababa City Roads Authority
- **ASTM**……… American Society for Testing of Materials
- **AASHTO**….. American Association of States Highways and Transport Officials
- **C**…………… Cement
- **C+ M**……… Cement and Molasses
- **CBR**………. California Bearing Ratio
- **CMS**……… Cement Modified Soil
- **ERA**……… Ethiopian Roads Authority
- **GSE**……… Geological Survey of Ethiopia
- **ICC**………….Initial Cement Consumption
- **ICL**………….Initial Consumption of Lime
- **IS**…………Indian Standard
- **LL**………... Liquid Limit
- **M**………….. Molasses
- **MDD**……… Maximum Dry Density
- **OMC**……... Optimum Moisture Content
- **ORN**……... Oversea Road Note
- **OPC**……… Ordinary Portland Cement
- **PI**…………. Plasticity Index
- **PL**…………. Plastic Limit
- **SC**………. Soil Cement
- **TRL**………. Transport Research Laboratory
- **TCD**……… Ton of Cane Per Day
- **UCS**……… Unconfined Compressive Strength
- **USCS**…….. Unified Soil Classification System
- **USBR**…….. United States Bureau of Reclamation
- **XRD**…….. X-Ray Diffraction

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

1.1 Back Ground of the Study

Expansive clay soils have a world-wide distribution; their occurrence is not climatic specific though they are particularly widespread in arid to semi-arid climate and are problematic to engineering structures because of their tendency to heave during wet season and shrink during dry season. Although the extent and range of distribution of this problematic soil have not been studied thoroughly, expansive soil is known to be widely spread in Ethiopia (ERA, 2013).

Expansive soils are a worldwide problem that possesses several challenges for civil engineers. They are considered a potential natural hazard, which can cause extensive damage to structures if not adequately treated. Expansive soils cause more damage to structures, particularly lighter buildings and pavements, than any other natural hazard, including earthquakes and floods (Jones and Holtz, 1973). In Ethiopia, there are several roads, whose premature failures attributed to the volumetric changes of expansive clay soil; Modjo-Ejerie-Areti Road and Addis-Jimma Road could be examples of such failures.

With the increased global demand for energy and increasing local demand for aggregates it has become expensive from a material point of view to remove inferior soils and replace them with foreign soils. Stabilization of the expansive clay soil by different additives could be one solution to alleviate the problem. However, most of the conventional stabilizing agents are relatively expensive to be implemented. Hence, it becomes essential to modify the properties of locally available soil to the extent that it can be used in the construction of roads and to make best utilization of various industrial by-products like molasses as a soil modifying/soil stabilizing agent.

In Ethiopia sugar production amount was 21.7 million tons in the year of 2010; however, in 2017 this amount will be increased to 51 million tons (Ethiopia Sugar Development Corporation, 2014). Increasing demand for sugar raises the generation of cane molasses material which constitute about 30% - 40% of sugar volume. Molasses contains a resinous and some inorganic constituents that render it unfit for human consumption. This liquid is mildly discomforting and adhesive when it gets into contact with a person’s skin. It is slippery when spilt and could be a cause of road accident if a major spill takes place on the road. Molasses could cause environmental pollution through aesthetic degradation if spills are not properly cleaned. It can also cause water pollution if major spills or factory effluents enter river streams. It is therefore important to consider critically the handling and disposal of molasses particularly in situations where supply exceeds demand. This can arise, especially where industrial use of molasses is not diversified.

Sugar factories usually use cane molasses as a dust palliative on roads inside their compounds and few researches were done to evaluate the soil stabilization process of expansive soils using molasses. In this context, an extensive research is needed to understand the mechanism and geo-engineering properties of expansive soil stabilized with sugar cane molasses.

Cement is used to improve the expansive clay soil, but it is very expensive. Moreover, expansive clay soil treated with cement is prone to shrinkage cracks and rapid setting time of cement makes compaction difficult. Research conducted on the effect of sugar containing molasses on concrete have proved that sugar improves the quality of reaction between the cement and the aggregates and reduces setting time of concrete (Hasan and Baris, 2012; and Akogu, 2011). Therefore, a research must be conducted to explore the effect of cement and molasses combination on stabilization of expansive clay soil.

Taking these into consideration, the aim of this research was to establish the effects of cement and molasses combination on expansive clay soils to reduce the cost of road construction as well as reducing the environmental hazard molasses causes.

1.2 Study Area

The Modjo – Ejere – Arerti – Sembo Road Project is located in the Central Ethiopia and forms the principal artery for the development of the project areas. The road project is shared between two Regional States: approximately the first 65 km is found in the Oromiya Region, while the remaining 126 km is found in the Amhara Region. The project benefits directly involve three Weredas (H/mariamna Kesem, Minjarna Shenkora and Lome) in two zones (North Shewa and East Shewa in Amahara and Oromiya Regional States respectively).

The study area of this research covers road segment from Modjo to Ejere (km 0+000 to km 35+000) which falls dominantly in flat terrain condition with some interception of rolling terrain. The road segment is located in the central part of the country and lies in Oromiya regional states. The road starts from the Addis Abeba – Nazret trunk road in Modjo town, at the rotary junction, where the road to Awasa also joins the Addis-Nazaret road and ends at Ejere Town. The project area is shown in Figure 1.1 and 1.2.

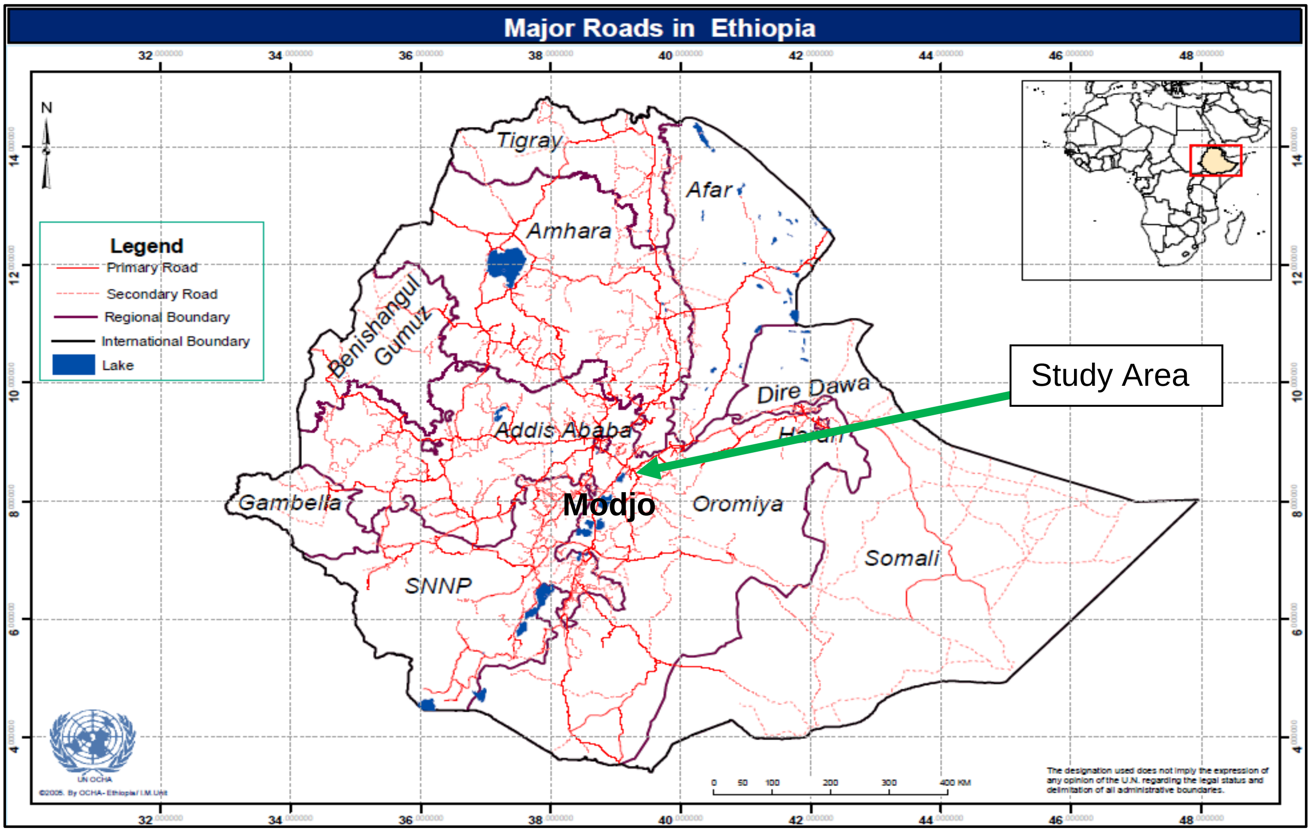


Figure 1.1: Location Map of the Study Area (Source: ERA (2011)).

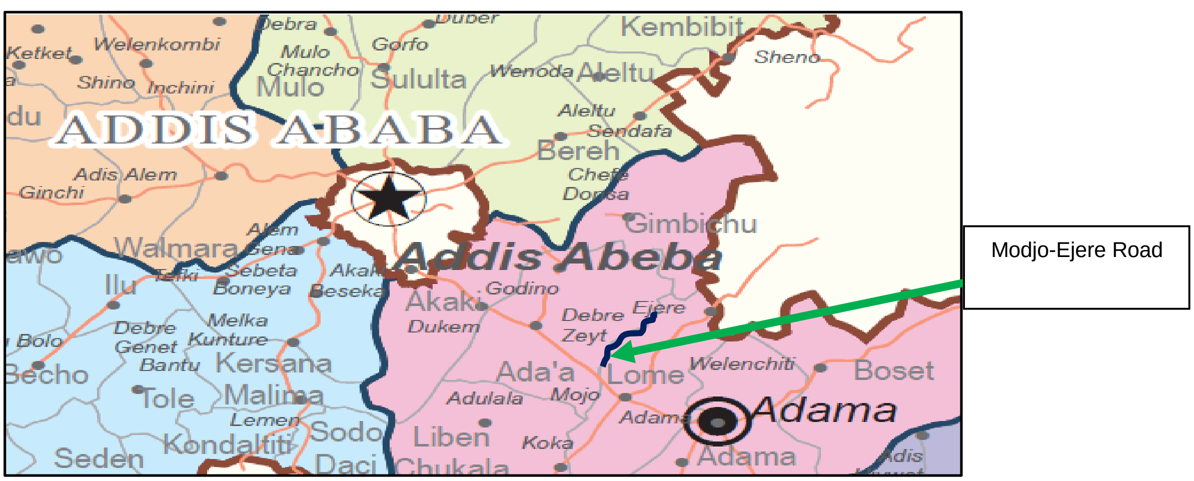


Figure 1.2: Location Map of Modjo-Ejire Road segment (Source: ERA (2011))

The terrain upon which the Modjo - Ejere road traverses is mostly flat to gently rolling. The starting point at Modjo is at an altitude of 1790 m. The altitude increases to 2294 m near km 35 at Ejere. The main geological formation of the project area is Alluvial and Lacustrine deposits which include sand, silt, clay, diatomite, limestone. The road from km 7 to the end of the road passes through the geological fault zone.

The climate in the project area is tropical to warm temperate and all along the Project Road, there is a similar trend throughout the seasons. The mean annual minimum and maximum temperature is 12.7 and 28.3^oC respectively. The highest observed air temperature is in May. The monthly rainfall pattern along the project road reflects that the main rainfall season is June to September.

1.3 Statement of the Problem

Relatively large areas in Ethiopia are covered with expansive clay soils. These soils have caused persistent difficulties in road construction and are a relatively common problem in the country. The conventional stabilizing agents commonly used in expansive soils and replacement of the inferior sub-grade soils by foreign soils are fairly expensive. As a result, such roads are not adequately constructed and therefore frequently require close attention. Therefore, it becomes essential to modify the properties of locally available soil with cheaper stabilizer to the extent that it can be used in the construction of roads and to make best utilization of various industrial by products like molasses as a soil modifying/soil stabilizing agent.

1.4 Research Questions

Can sugar cane molasses and ordinary Portland cement (OPC) mixtures be adequately used as a stabilizing agent for roads passing on expansive clay soils?

1.5 Objectives of the Study

1.5.1 General Objectives

The objective of this thesis work was to study the suitability of sugar cane molasses and cement (OPC) combination for expansive clay soil stabilization, unsuitable for pavement sub-grade, by increasing their bearing capacity and decreasing their swelling potential. In addition to this, it aims to make comparison on the effect of stabilizers on soils treated with molasses alone, cement alone and cement and molasses combinations.

1.5.2 Specific Objectives

The study had the specific objectives of investigating the response of the expansive clay soils through the application of the sugar cane molasses alone, cement alone and cement and molasses combination at various contents and at different curing durations.

- To assess geotechnical engineering properties of the natural expansive clay soil by conducting routine laboratory tests, strength tests, volume and stability tests.
- To investigate constituents of the sample sugar cane molasses.
- To assess the strength and volume stability effect of the natural expansive clay soils and expansive soil mixed with cane molasses only, cement only and cement and molasses combination by varying their percentages as reflected by California Bearing Ratio (CBR), Unconfined Compressive Strength (UCS) and swell tests.
- To analyze the effect of treatment with molasses alone, cement alone and cement and molasses combination on Proctor, pH, Atterberg Limits, Linear Shrinkage, CBR, UCS and swell tests.

1.6 Outcome of the Study

This research was conducted to study the suitability of molasses and cement combination for expansive clay soil stabilization and to use the in-situ sub-grade soil as it is after treatment.

The research will fix optimum molasses and cement content for adequate stabilization of the existing expansive sub-grade soil for road construction. It also gives a cost comparison of replacement of the existing expansive sub-grade soil with foreign suitable material and stabilization of the existing sub-grade soils with selected optimum stabilizer content.

1.7 Scope and Limitations

The scope of the research was to evaluate laboratory performance of expansive clay soil from Modjo-Ejere Road with the addition of sugar cane molasses alone from Wenji/Shoa sugar factory, cement alone from Derba OPC and cement and molasses combination of various content of the additives.

In the present study, there were limitations on;

- To precisely identify clay mineralogy of the soil under study due to capacity limitations of X-Ray Diffraction (XRD) soil test machine at Geological Survey of Ethiopia (GSE),
- The composition of molasses is influenced by the soil where the cane is grown, climatic conditions, variety and maturity of the cane and the processing conditions at the factory, hence sugar cane molasses from Wenji/Shoa sugar factory is not compared with other sugar factories molasses composition in the country, and
- Finally mechanism of soil stabilization with cement and molasses combination is relatively new concept and literatures are scanty in the area.

Therefore, it is strongly recommended that the results and findings of the present study must be considered as a complete only for sugar cane molasses form Wenji/Shoa sugar factory, for OPC cement form Derba Cement Factory and sample soil form Modjo-Ejere Area. However, further studies and additional tests are required before implementing these results or finding for filed applications, hence shall be considered indicative only.

1.8 Organization of the thesis

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

- The first chapter gives a brief description of the thesis background, study area location, problem statement, objectives, scope and limitations of the study.
- The second chapter reviews related literatures on weak sub-grade and treatment, identification and classification of expansive soils, composition of sugar cane molasses and its production and use in Ethiopia, mechanisms involved in soil stabilization with sugar cane molasses and OPC.
- The third chapter briefly discusses research methods followed and materials used to conduct the research. In addition, the results of the native soil and stabilizers were given and their natures were characterized in this chapter.
- Sample preparation methods and experimental procedures for stabilizing soil are included in the fourth chapter.
- The fifth chapter reports the test results obtained for stabilizing soils; analysis of results and discussion of results with respect to the theoretical background and with respect to findings of previous studies.
- Finally, conclusions and recommendations drawn from the study are presented in chapter six.

2. REVIEW OF RELATED LITERATURES

2.1 Practical Treatment and Design Alternatives for Highway Construction on Expansive Clay Soil

Expansive soils cause very serious geotechnical problems in various parts of the world including Ethiopia and relatively large areas (Central, Western and South-Western region) in the country are covered with expansive soils. The problems encountered on these soils are mainly associated with excessive volume changes of the soil profiles when there is a change in moisture content. Those excessive volume changes cause serious distress and damage to engineering structures such as buildings and roads built on them. Pavements are particularly susceptible to damage made by expansive soils because they are lightweight and extended over large areas. There are several roads in Ethiopia whose failures were attributed to the volumetric changes of expansive clay soil. The damage caused to the roads varied from development of fine cracks on the road surface to premature pavement failures. As a result of these; vehicle operating cost increases, traffic accident increases, travel time increases and a lot of money is usually spent on rectifying the damages to pavements built on expansive soil.

Problems associated with construction over expansive soils are usually the seasonal moisture changes in sub-grade soils rather than the low bearing strength, as expansive soils are often relatively strong at equilibrium moisture content. Generally for road construction over expansive soils, it is essential to address the influence of the expansive soils both as naturally occurring undisturbed soils beneath the road and as compacted soil in the road formation.

The practical treatment and design alternatives for highway construction on expansive clay soil are separated into the following broad categories.

I. Re-route alignment / choose alternative location to avoid the problem.
II. Pre-wet, expansive soils to achieve post-construction equilibrium moisture contents
III. Remove / dig out expansive soil and replace with non-expansive fill either entirely or a proportion (generally about 1 meter).
IV. Prevent moisture changes in expansive soils by means of barriers, both horizontal and vertical.

V. Improve the expansive soil by stabilization.

In general, Options 1 is not practical for highway use because the problem soils tend to occur over broad areas and option 2 tends to be impractical as well, while Option 3 and 4 has been the most commonly used methods. However, due to improvement in technology coupled with increased transportation costs, Option 5 is being used more often today and is expected to dramatically increase in the future.

2.2 Identification and Classification of Expansive Soils

2.2.1 Field Identification of Expansive Soils

Some of the important field identification methods that indicate the potential for expansiveness of a soil during reconnaissance and preliminary stages are summarized in Table 2.1.

Table 2.1: Field identification methods for expansive soil

FIELD ASSESMENT	
Field Description & Identification	Detailed soil profiling required.
Consistency with respect to moisture content	High shear strength when dry. Soft and sticky when wet.
Structure	Shrinkage fissures and cracks, Shear surfaces have a glazed or shiny appearance
Color	May be of value on a regional or local level.
Suction	Expansive soils have a high suction towards water when partly dry
Local Knowledge	Local authority engineers and builders may be a valuable source of information.

2.2.2 Laboratory Identification and Testing Techniques of Expansive Soils

Generally there are three different methods of identifying expansive soil in the laboratory. These are mineralogical identification, indirect methods and direct methods.

I. Mineralogical Identification

This method claims that the swelling potential of any clay can be evaluated by identification of the constituent mineral of this clay. The various techniques under these methods are: X-ray diffraction, differential thermal analysis, methylene blue, electron microscope, cation exchange capacity, etc. But these methods are not suitable for routine laboratory tests because of the following reasons; they are time consuming, require expensive test equipment and, the results are interpreted by specially trained technicians (Chen, 1988).

Table 2.2: Mineralogical identification of expansive soil

Test	Properties Investigated	Parameters Determined
X-ray Diffraction	Characteristics crystal dimension	Proportion of various minerals present in a colloidal clay
Differential Thermal Analysis	Characteristic reactions to heat treatment	Area and amplitude of reaction peaks on thermos-grams
Electron Microscopy	Size and shape of clay particles	Visual record of particles
Cation Exchange Capacity	Charge deficiency and surface activity of clay particles	CEC (Meg/100gm)

II. Indirect Methods

Indirect methods in which one or more of the related intrinsic properties are measured and complemented with experience to provide indicators of potential volume change of expansive clay soil. These methods include, such as the index property, potential volume change (PVC) method, and activity method, etc. Chen (1988) strongly states that erroneous conclusion can be drawn if the indirect methods are used independently.

Table 2.3: Indirect Methods of Expansive Soil Identification

Test	Properties Investigated	Parameters Determined
Atterberg's Limits	Liquid limit (LL), Plastic Limit (PL) Shrinkage Limit (SL)	PI=LL-PL L_s=Linear Shrinkage
Clay Content	Distribution of fine-grained particle sizes	Percent finer than $2\mu m$

Test	Properties Investigated	Parameters Determined
Activity method	Plasticity Index (PI) Percent by weight finer than 0.002	$\text{Activity}(A_c) = \frac{\text{Plasticity Index}}{(\text{\% by weight finer than } 0.002\text{mm})}$
Potential volume change method	One dimensional swell and pressure of compacted, remolded sample under semi-strained controlled conditions	Swelling Pressure (lb/ft^2) PVC (potential volume change)

III. Direct Method

The direct methods which involve actual measurement of volume change in an odometer-type testing apparatus are generally grouped into swell or swell pressure tests. These testing methods are necessary to obtain measurable properties for predicting or estimating the magnitude of volume change the material will experience in order to ascertain approximate treatment and/or design alternatives (Senthen, et al., 1975).

2.2.3 Classification of Expansive Soils

The parameters determined form expansive soil identification tests have been combined in a number of different classification schemes. But before using any classification system, it should be understood the database from which it was derived and establish its limitations; otherwise, poor reliability and lack of confidence in the system may result. The different classification systems are categorized into two:

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

2.2.3.1 General Classification

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

2.2.3.2 Classification Specific to Expansive soils

The parameters determined from expansive soil identification tests have been combined in a number of different classification schemes to give a qualitative assessment of the degree of probability expansion. Unfortunately, there has not yet evolved a standard classification procedure, and a different scheme is used in practically every different location (Nelson and Miller, 1992). Some of the classification methods discussed in the literatures are given in the following section:

I. Classification based on indirect predictions of swell potential

a) Skempton Method

This method classifies clays according to their activities which is developed by Skempton (1953) by combing Atterberg limits and clay content (% percent by weight finer than 2µ m) into a single parameter called activity. Skempton classified clays into three classes according to their activities as indicated in Table 2.4. Activity is defined as:

$$\textbf{Activity} = \frac{\textbf{PI}}{(\textbf{Percent of clay} < 0.002mm)} \ldots\ldots\ldots\ldots\ldots\ldots..(\mathbf{2.1})$$

Table 2.4: Classification of Expansive soil based on Skempton Method

Degree of activity	**Activity**
Inactive Clay	<0.75
Normal clay	0.75-1.25
Active Clay	>1.25

Following this classification, montmorillonitic clay (Expansive Clay) is defined as active, illitic clay as normal and Kaolinitic clay as inactive (Chen, 1988; Nelson and Miller, 1992).

b) USBR Classification Method

This method developed by Holtz and Gibbs (1956) is based on the simultaneous consideration of colloid content < 0.001mm, Plasticity index and Shrinkage limit with observed volume changes. Relation of volume change with index properties was developed based on actual expansion tests for only 45 undisturbed and remoulded samples (air-dry to saturated under a load of 1 pound per square inch). The classification is presented in Table 2.5.

This method accepts the accompanying drawbacks:

I. Data limitation to form an accurate empirical relationship between measured expansion and three tests,
II. Differentiations of soil behavior between undisturbed and remoulded samples were not considered.
III. Chen (1988) observed that no conclusive evidence of the correlation between swelling potential and Shrinkage limit, and
IV. Sridharan and Prakash (2000) have also shown that the shrinkage limit cannot be satisfactorily used to predict the swell potential of a soil and that the mechanisms governing the shrinkage and swelling are entirely different.

Table 2.5 Classification of Expansive Soils based on USBR Method

Colloid content < 0.001mm	Plasticity index	Shrinkage limit	Probable expansion, percent total volume change	Degree of expansion
>28	>35	>11	>30	Very high
20-13	25-41	7-12	20-30	High
13-23	15-28	10-16	10-30	Medium
>15	<18	>15	<10	Low

c) Activity Method

This method was proposed by Seed, Woodward, and Lundgren (1962) based on remoulded, artificially prepared samples. The expansion was measured as percentage of swell on soaking from 100% maximum density and optimum moisture content in a standard AASHTO compaction test under a surcharge of 1 psi. The activity was defined as:

$$\boldsymbol{Activity} = \frac{\boldsymbol{PI}}{(\boldsymbol{Percent\ of\ clay} < 0.002\boldsymbol{mm}) - \mathbf{10}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.\mathbf{2.2}$$

This method is more preferable than USBR method due to:

I. Shrinkage limit did not enter into the evaluation of swell potential

II. Differentiate between undisturbed and remolded samples

d) Chen Classification Method

Chen (1988) correlates the expansive properties with the percentage of silt & clay passing No. 200 sieve, and Liquid Limit. The guide developed by Chen (1988) for estimating the probable volume changes of expansive soil using a vertical load of 1, 000 psf to gauge the swelling potential is presented in Table 2.6.

II. Classification based on the Oedometer Swell Potential Values

Based on the oedometer swell potential values, Seed et al. (1962) and Holtz and Gibbs (1956) have classified the relative expansivity of the swelling soils. The expansivity categories proposed by these workers are shown in Table 2.7 (Nelson and Miller, 1992; and Senthen et al, 1975).

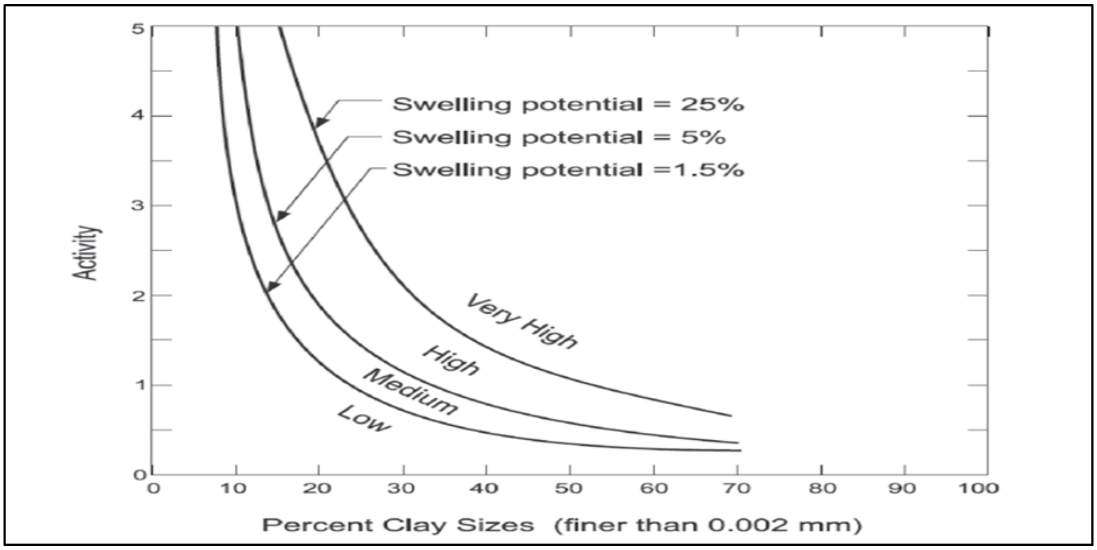


Figure 2.1: Classification Chart for Swelling Potential (after Seed, et.al, 1962)

Table 2.6: Classification of Expansive soils according to Chen

The percentage passing No.200 sieve	Liquid limit, percent	Percentage of swell	Swelling Pressure, ksf	Degree of expansion
>95	>60	>10	>20	Very high
60-95	40-60	3-10	5-20	High
30-60	30-40	1-5	3-5	Medium
<30	<30	<1	1	Low

Table 2.7: classification based on Oedometer Swell Potential Values

Holtz and Gibbs (1956) classification of percent swell*	Seed et al's (1962) classification of percent swell**	Degree of Expansion
0-10	0-1.5	Low
10-20	1.5-5	Medium
20-35	5-25	High
>35	>25	Very High

** Holtz and Gibbs (1956) classified degree of expansion from the volume change on undisturbed and remoulade samples (air-dry to saturated under a load of 1 pound per square inch).*

*** Seed et al's (1962) classified degree of expansion from volume change measured as percentage of swell on soaking from 100% maximum density and optimum moisture content in a standard AASHTO compaction test under a surcharge of 1 psi on remolded, artificially prepared samples.*

2.2.3.3 Comparison of Classification Schemes

The classification procedures discussed above provide only qualitative ratings such as high, medium or low swell potential. Therefore, the classification should be used only to indicate potentially hazardous areas and a need for predictive testing. The direct use of such classification systems as a basis for design in areas outside the region where the ratings were established may lead to overly conservative construction in some places and inadequate construction in others (Nelson and Miller, 1992). Hence, it is very important to emphasize that design decision has to be based on predicting testing and analysis, which provide reliable information.

2.3 Soil Modification/Stabilization

Soil stabilization or modification refers to the improvement of the soil physically or chemically by using various techniques including mechanical compaction and the use of various calcium rich chemicals. The selection of proper stabilization technique depends on the soil type and its condition.

Mechanical stabilization of a material is usually achieved by adding a different material in order to improve the grading or decrease the plasticity of the original material. The physical properties of the original material will be changed, but no chemical reaction is involved. The main methods of mechanical stabilization can be categorized in to compaction, mixing or blending of two or more gradations, applying geo-reinforcement and mechanical remediation (Caterpillar, 2006).

Chemical stabilization is mixing of soil with one of or a combination of chemically active compounds for the general objectives of improving or controlling its volume stability, strength and stress-strain behavior, permeability, and durability (Winterkorn & Pamucku, 1990).

Mechanical stabilization is best suited for coarse grained soils or aggregates at optimum or below optimum moisture contents. However, clayey soils are more effective under chemical stabilization. If the clayey soil is mixed with the specific stabilizer just enough to make it workable, better in texture and compactibility regardless the strength and durability, then it is referred to as modification ; modification is restricted to the soil having AASTHO designation A-4, A-5, A-6 and A-7 (Sanjay, 2012).

On the other hand, stabilization refers to the selection of the stabilizer in order to achieve certain target strength/stiffness values in addition to modification. In conclusion, creating working platform for construction purpose only is part of modification/treatment; whereas stabilization is essential if we are dealing with construction of subbase in pavements.

Dallas and Nair (2009) classify 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, potassium compounds and
- ❖ **By-product stabilizers:** which include cement kiln dust, lime kiln dust etc.

This research evaluates the suitability of sugar cane molasses and its combination with traditional stabilizer (cement) for soil stabilization. Accordingly, the respective soil additives and their mechanisms of stabilization are briefly discussed in section 2.3.1and 2.3.2 of this research.

2.3.1 Portland cement Stabilization

When stabilization of soil is done by mixing of pulverized soil and measured amount of cement and water it is known as soil cement stabilization. Cement has been found to be effective in stabilizing a wide variety of soils and waste materials such as pulverized bituminous pavements and crushed concrete. Cement-stabilized materials generally fall into two classes: soil-cement and cement modified soil.

- ❖ **Soil-Cement(S-C)** is a mixture of pulverized soil material and/or aggregates, measured amounts of portland cement, and water that is compacted to a high density to serve as the primary structural base layer in a flexible pavement or as a sub-base for rigid pavements (*Dallas and Nair, 2009*).

- ❖ **Cement-Modified Soil (CMS**) is a soil or aggregate material that has been treated with a relatively small proportion of Portland cement with the objective of altering undesirable properties of soils or other materials so they are suitable for use in construction. CMS is typically used to improve subgrade soils or to amend local aggregates for use as base in lieu of more costly transported aggregates (*Dallas and Nair, 2009*).

The improvement of soils/aggregates containing clay through the addition of Portland cement involves four distinct processes discussed in the order of their occurrence:

- ❖ Cation exchange,
- ❖ Flocculation and agglomeration/Particle restructuring,
- ❖ Cementitious hydration, and
- ❖ Pozzolanic reaction.

I. Cation Exchange

The first reaction in calcium reach stabilizers is hydration reaction which occurs between the reaction of lime and water. This reaction is an exothermic reaction which produces calcium ions and hydrogen ions. The calcium ions produced from the hydration process and other elements / compounds which are the constituents of the stabilizer are responsible for cation exchange.

$$\mathbf{CaO\ (lime) + H_2O \rightarrow Ca^{++} + 2(OH)^{-}} \quad \text{…………..} \quad \mathbf{(2.3)}$$

Cation exchange includes an immediate reaction of the clay with the stabilizer within few minutes of mixing, resulting in a soil with improved texture. The tetrahedral (T) and octahedral (O) combination of clay minerals in 1:1 (1T and 1O) or 2:1 (2T and 1O) have charge deficiency that results in the attraction of the cations or water molecule. Generally, sodium or potassium ($Na+$ or $K+$) are prevalent in clay minerals along with water. However, these cations can be replaced by the higher valance cations like Al^{+3}, Ca^{+2}, Mg^{+2} etc. so called cation exchange. During this process calcium rich chemical stabilizer provides enough cations to replace the monovalent cations resulting in a reduced thickness of diffused double layer (Sanjay, 2012).

II. Flocculation and Agglomeration

Flocculation and agglomeration, which is made possible through cation exchange (Herzog and Mitchell, 1966), is the process of clay particles altering their arrangement from a flat, parallel structure to a more random edge-to-face orientation (Figure 2.3). The restructuring of modified soil/aggregate particles changes the texture of the material from that of a plastic, fine-grained material to one more resembling a friable, granular soil/aggregate.

The reduced size of the double layer due to cation exchange, as well as the increased internal friction of clay particles due to flocculation and agglomeration, result in a reduction in plasticity, an increase in shear strength, and an improvement in texture. As with cation exchange, the particle restructuring process happens rapidly.

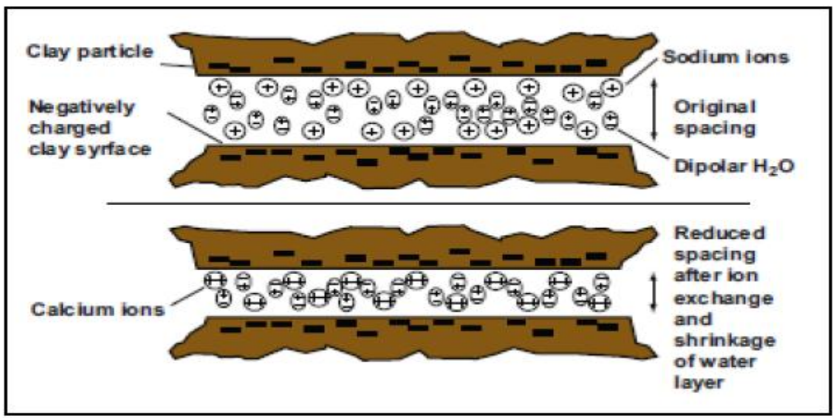


Figure 2.2: Cation Exchange

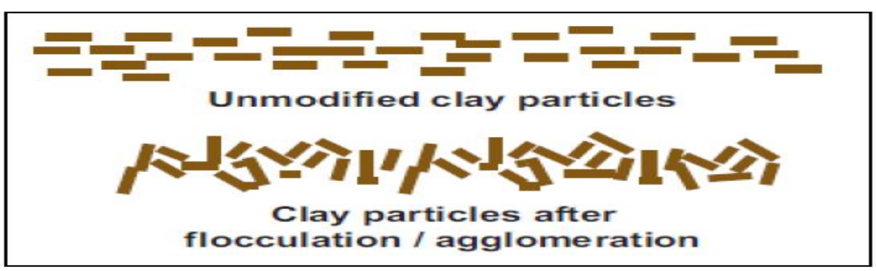


Figure 2.3: Particle Restructuring

III. Cementitious Hydration

Cementitious hydration (Figure 2.4) is a process that is unique to cement, and produces cement hydration products referred to in cement chemistry as calcium-silicate-hydrate (CSH) and calcium-aluminum-hydrate (CAH). CSH and CAH act as the “glue” that provides structure in a cement-modified soil/aggregate by stabilizing flocculated clay particles through the formation of clay-cement bonds. This bonding between the hydrating cement and the clay particles improves the gradation of the modified clay by forming larger aggregates from fine-grained particles. This process happens between one day and one month after mixing.

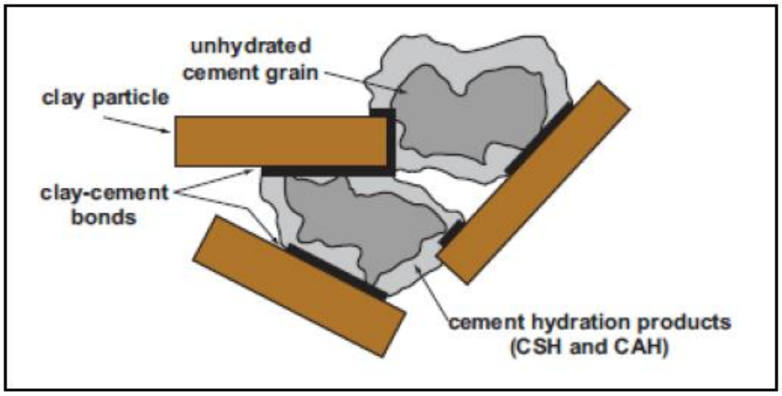


Figure 2.4: Cementitious hydration

IV. Pozzolanic Reaction

In addition to CSH and CAH, hydrated portland cement also forms calcium hydroxide, or Ca(OH)2, which enters into a pozzolanic reaction. The calcium which is released during hydration process is in suspension of stabilizer-soil-water and will be available for the stabilization of soil. The general reaction of the cement with water that yields calcium is presented in equations 2.4 and 2.5. This phenomenon is illustrated in Figure 2.5.

$$\mathbf{C_3S + H \text{ ------------------------ } C\text{-}S\text{-}H + Ca(OH)_2 \ldots\ldots\ldots\ldots\ldots\ldots (2.4)}$$

$$\mathbf{C_2S + H \text{ ------------------------ } C\text{-}S\text{-}H + Ca(OH)_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.5)}$$

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

This secondary soil modification process takes the calcium ions supplied by the incorporation of portland cement and combines them with the silica and alumina dissolved from the clay structure (at highly alkaline solution pH $\geq$12.4) to form additional CSH and CAH (Figure 2.5) (Harty, 1970). The pozzolanic reactions take place slowly, over months and years, and can further strengthen a modified soil/aggregate as well as reduce its plasticity and improve its gradation. The pH environment in the system initiates further reaction of the silica and alumina with the clay particles, hence proving extra strength to the stabilized soils (Harty, 1970). The minimum PH of 12.4 is necessary in order to maintain the pozzolanic reaction (Eades and Grim, 1960 cited in Harty, 1970).

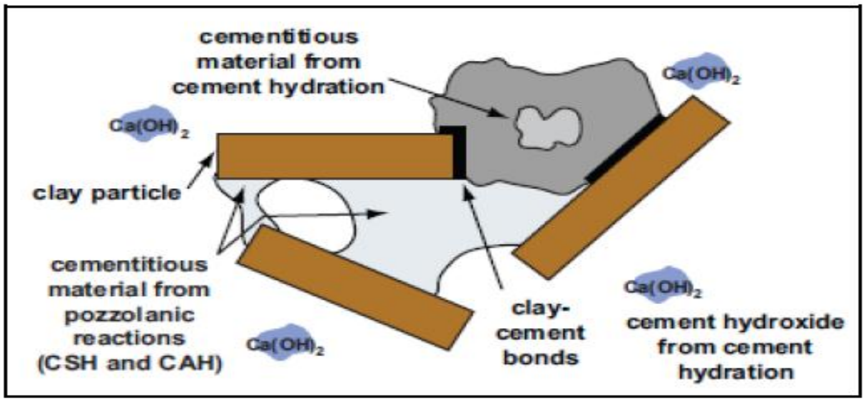


Figure 2.5: Pozzolanic Reaction

2.3.2 Soil Stabilization Using Cane Molasses

Molasses stabilization is a relatively new concept that is scanty in the literature and the chemical reactions that take place between soil molasses mixtures are not fully understood.

During sugar processing, some materials are added into the process as clarification agents and evaporator decadents. These materials include lime and sulphur dioxide among others. During crystallization of the sugar juice, those elements remain in molasses and are then included in the natural molasses ingredients. Those elements plus others imbibed from the soil by the sugar cane as nutrients to support growth are the ones, which probably interacted with expansive soil to change its characteristics during stabilization (Ndegwa, 2011).

Therefore, the explanation of chemical reactions of soil stabilization with molasses is based on the results of the foregoing studies and on findings of other researchers on related subjects .i.e. cation exchange and flocculation-agglomeration are the two chemical reactions that play role for molasses stabilization of expansive soil (Ndegwa and Shitote, 2012; Ndegwa, 2011).

Ndegwa and Shiota (2012) and Ndegwa (2011) briefly described mechanisms of stabilization when molasses is mixed with expansive clay soil. They stated that *cation exchange reaction* and *adhesive property of molasses* are responsible for stabilization of expansive soil by cane molasses.

In nutshell they stated that geotechnical properties improvements of stabilized expansive clay soils by cane molasses come from:

- Cation exchange reaction in soil molasses mixture which plays a role in enhancement of flocculation and soil aggregate stability, and reduction in water affinity of the clay soil; and
- The electrostatic attraction between aggregated soil particles emanates from adhesivity of molasses even further increases the size of soil particles.

2. 4 Effect of Stabilizers on Soil Properties

2.4.1 Effect of Stabilizers on Atterberg's Limits

When highly plastic soil (LL>50-70) is treated with calcium rich additive, the chemical reaction between clay particles and additive results in reduction of the size of the diffused double layer and increase in the inter-particle contact. Consequently, the liquid limit (LL) of the soil will be decreased associated with the increase in plastic limit (PL); hence decreasing the plasticity index (PI) of the stabilized soil. As a result, strength/stiffness of the stabilized soil will be improved (Amu, et al, (2005).

Amu, et al, (2005) investigated the effect of cement addition by varying cement content on plasticity of expansive clay soil and found that LL decreased and PL increased with increasing cement content, thus PI decreased with addition of cement content up to 14% cement content and then increased with further addition of cement, see Table 2.8.

Table 2.8: Atterberg's limit values for cement stabilized soil samples (Amu, O.O et al., (2005)).

Cement by weight (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
0	58.70	16.48	42.22
2	56.80	20.28	36.52
4	56.60	23.16	33.44
6	54.40	24.03	29.37
8	51.40	25.37	26.37
10	47.85	29.32	18.53
12	47.30	30.13	17.17

Cement by weight (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
14	47.80	28.11	19.64
16	48.00	27.50	20.50

However, Balasingam and Farid (2008) and Grytan et al., (2012) found somewhat different results on their investigation of geotechnical properties of cement treated Soil with LL=54% and PI=11%and Bangladesh soil with LL=46% and PL=19% respectively. The former researchers observed that LL increased slightly (initially) and decreased with increasing in cement content, while PL remained relatively constant. Consequently the PI increased initially followed by a decrease with increase in cement content. While, the latter researchers investigated that LL and PL of the soil increases gradually with increases in percentage of cement and PI slightly decreases.

Shirsavkar and Koranne (2010) studied the effect of molasses addition in varying proportion up 7.5% on plasticity of soft sandy soil and they found that as the percentage of molasses increased Plasticity Index (PI) decreased. Furthermore, Ndegwa and Shitote (2012) studied influence of cane molasses on plasticity of expansive soil up to 20% and they found that it can reduce the PI of expansive clay soil if not more than 8% of it is added to the soil, see Figure 2.6.

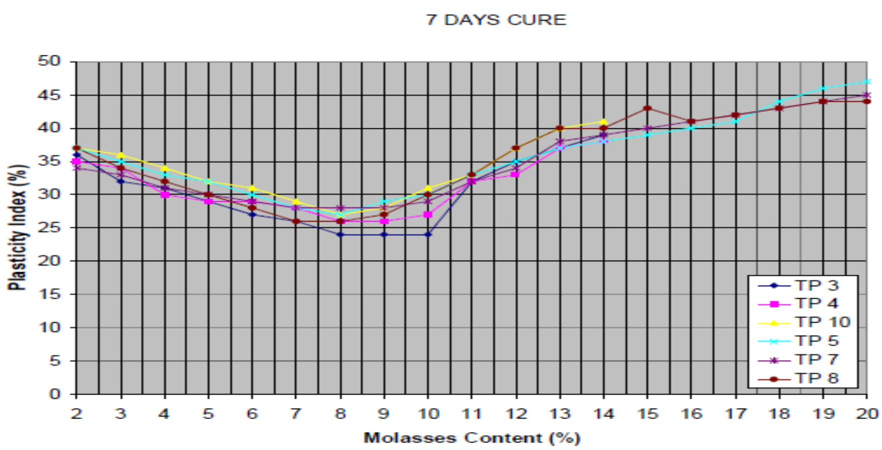


Figure 2.6: Plasticity Index versus Molasses Content (Ndegwa and Shitote (2012)).

2.4.2 Effect of Stabilizer on Moisture Density Relation

The change in chemical composition of the soil can be noticed by decrease in the maximum dry density (MDD) and increase in the optimum moisture content (OMC) of the soil-stabilizer mixture. However; from compaction test result of cement treated soil, it was found that a slightly different observed from MDD and OMC due to increment of stabilizer content. In general, the changes in the MDD and OMC were considered too small due to addition of cement content is small and hydration process is not occurred within a short period of time (Amu et al. 2005, Grytan et al. 2012, and Bello, 2012).

Amu, et al, (2005) reported that with addition of 12% cement by weight for a highly plastic soil with LL=59% and PI=43%, the maximum dry density reduced from 1641 Kg/m3 to 1601 Kg/m3 and the optimum moisture content increased from 17% to 19%.

Additionally, Grytan et al. (2012) investigated the effect of cement (0, 5, 7.5, 10 and 12.5% by weight) on compaction characteristics of cement stabilized soil with LL=46% and PI=19% and results showed that the MDD of the soil decreases gradually with an increase of cement content. On the other hand, the OMC of soil increases with an in cement content. Nur et al. (2014) also reported similar results and reported that addition of cement on kaolin soil increases OMC while decreasing MDD values.

However, Gautreau et al. (2009) and Horpibulsuk et al. (2010) found different types of results for cement stabilized soils. For heavy clay soil with LL=74% and PI=46%, Horpibulsuk et al. (2010) reported an increase in maximum dry density with no significant change in optimum moisture content; while Gautreau et al. (2009) found out that for soil with LL=34% and PI=12% there is no significant difference in optimum moisture content and maximum dry density with cement content.

Bello (2011) looked at the effect of compaction delay and compaction effort on Nigerian lateritic soil and reported that the MDD increased with an increase in compaction effort, while the OMC decreases with increase in compaction effort. Furthermore, OMC values increase with increase in cement content while MDD decrease with increase in cement content irrespective of applied compaction effort.

Shirsavkar and Koranne (2010) investigated that the effect of addition of cane molasses to sandy soil with LL=29% and PI=11% and reported that the addition of cane molasses on the soils increased maximum dry density and slightly reduced optimum moisture content.

2.4.3 Effect of Stabilizers on Strength Characteristics

2.4.3.1 California Bearing Ratio

The California Bearing Ratio (CBR) value of high PI soils increases significantly with the increase of the stabilizer content and curing periods, and soaked CBR values are greater than un-soaked CBR values (Osinubi et al. (2012), Nur et al. (2014) and Bello (2011)).

Nur et al. (2014) studied the effect of cement content (0, 7, 13% by weight) on Kaolin clay soil with LL=54% and PI=24.8% treated with cement and cure for 7 days and found that the CBR value increased with the increasing of percentage of cement. CBR value equal to 16, 76 and 110% for 0, 7 and 13% cement were attained respectively.

Osinubi et al. (2012) investigated effect on addition of cement on black cotton soil which belongs to A-7-6(13) in the AASHTO classification system and reported that, generally, CBR values increased with higher OPC content.

Bello (2011) investigated influence of compaction delay (0, 1, 2, 3 h) on CBR values of cement stabilized soil (0, 3, 5, 7, 9 % by weight of soil) with LL=48% and PI=18% and cure for 7 days, 14 days and 28days. The results showed that CBR values generally increased with higher cement content at no compaction delay, CBR values decreased with higher elapse times up to 3 h irrespective of the cement content and compaction effort.

Generally, cement stabilized soils resulted in higher CBR values (both un-soaked and soaked) than the soils treated with lime for the same stabilizer-moisture ratio (Bello (2011).

The effect of increasing quantities of cement by weight on strength of some lateritic soils with LL=40.91% and PI=17.31% investigated by Oyediran and Kalejaiye (2011) and results showed that stabilization of the soils with cement increase CBR value up to 10% cement content by weight of the soil for the soaked and un-soaked options while further addition of more than 10% by weight of cement was observed to cause reduction in CBR values.

Effect of cane molasses on CBR values of expansive clay soil was investigated by Ndegwa (2011) on both un-soaked and soaked samples for different curing periods and it was found that molasses increased the CBR values of expansive clay soil and thus the load bearing ability of the soil. It was also observed that cane molasses mixed with expansive clay soil could reduce swelling tendencies of the soil.

Furthermore, Ravi et al. (2015) studied the effectiveness of molasses for improving the shear strength and CBR value of two compressible fine grained sample soils (intermediate compressible clay with PI values of 28.57% and highly compressible clay with PI values of 33.20%) and results showed that there were increment ratio in range of 1.57-2.01 and 2-3.5 in unconfined compressive strength and CBR values respectively for both soils.

2.4.3.2 Unconfined Compressive Strength

The unconfined compressive strength (UCS) of the soil increases drastically with the increase of the stabilizer content and soaked samples exhibited significant greater unconfined compressive strength compared with un-soaked samples for cement content greater than 7.5% (Grytan et al. (2012), Osinubi et al. (2011), Nur et al. (2014), Bello (2011), Muhunthan .B. and Sariosseiri .F (2008)). Generally, cement stabilized soils possesses higher UCS than the soils treated with other stabilizers for the same stabilizer-moisture ratio (Bhattacharja and Bhatty (2003), and Hossain and Mol (2011)).

Grytan et al. (2012) investigated the effect of cement on UCS of soil with LL=46% and PI=19% at different percent of cement content for 0, 7 and 28 days curing and found that the value of UCS gradually increased for both un-soaked and soaked samples. They also observed that the value of compressive strength increases with the increases of soaking days for cement treated soil, whereas the untreated soil shows the compressive strength decreases with the increasing of soaking days.

Additionally, Osinubi et al. (2011) studied the effect of cement addition on UCS of black cotton soil which belongs to A-7-6(13) in the AASHTO classification system and reported that there was a general increase in the UCS values with OPC content and curing period.

Nur et al.(2014) explored the effect of cement with 0, 7 and 13% by dry weight of Kaolin clay soil with LL=54% and PI=24.8% and the fining of the study showed that cement effectively increase the strength of the clay soil and UCS values increased with increasing percentage of cement.

Bello (2011) investigated influence of compaction delay (0, 1, 2, 3 h) on UCS values of cement stabilized soil (0, 3, 5, 7, 9 % by weight of soil) with LL=48% and PI=18% and cure for 7 days, 14 days and 28days. The results showed that UCS values generally increased with higher cement content at no compaction delay, UCS values decreased with higher elapse times up to 3 h irrespective of the cement content and compaction effort.

Furthermore, Bhattacharja and Bhatty (2003) compared the performance of lime and cement on three different types of soils in Texas with PI of 25%, 37% and 42%, and found that for all soils, better performance was observed from cement stabilizer. However, there was great decrease in the strength (by more than 50%) of the cement treated soils with delay compaction of 24 hour.

Muhunthan and Sariosseiri (2008) looked at effect of compacion delay on UCS values and reported similar results and suggested that the achievement of stronger material with immediate compaction can be attributed to the physicochemical phenomenon resulting from the hydration of Portland cement. Whit in a short time after mixing Portland cement with soil, the mixture becomes granular due to agglomeration, which primarily results from the hydration of the cement grains and helps form a network. If the compaction is delayed, the network is broken during compaction and never re-established, leading to a weaker mass. However, compaction prior to such granulation is more efficient and provides a stable network with superior engineering properties.

Bhattacharja and Bhatty (2003) conducted comparative study on performance of lime and Portland cement treated highly plasticity soils (Soil 1 and Soil 2 with PI values of 42 and 37% respectively) results showed that Portland cement is about equivalent to lime at low dosages (3%) but at higher dosages of 6% to 9% cement addition produces a much stronger stabilized product at all ages (See Table 2.9). Additionally, Hossain and Mol (2011) used natural pozzolans and industrial waste to stabilized the clay soil (A-6) having LL 39% and PI of 19% and reported almost double strength gain with cement kiln dust (CKD) compared to volcanic ash (VA) under identical condition.

The effect of cement treatment on unconfined stress-strain behavior of Soil with LL=54% and PI=11% for un-soaked and soaked samples were studied by Muhunthan and Sariosseiri (2008) it is observed that the peak axial stress increased significantly due to cement treatment, but corresponding strain to peak axial stress decreased from approximately 4% to slightly greater than 1% as shown in Figure 2.7. Thus, cement-treated soils exhibited much more brittle behavior than non-treated soils. Additionally, they studied the effect of addition of cement on UCS of Soil with LL=54% and PI=11% for un-soaked and soaked samples. It is reported that soaked samples with 7.5% and 10% cement content exhibited greater unconfined compressive strength compared with un-soaked samples. See Figure 2.8.

Table 2.9: UCS of highly plasticity soil stabilized with three dosages of Portland cement and Lime and Compacted at Respective OMC (Bhattacharja and Bhatty (2003)).

Dosage	**3%**		**6%**		**9%**	
Stabilizer	**Cement**	**Lime**	**Cement**	**Lime**	**Cement**	**Lime**
	Soil 1					
Age 1 day	80 psi	50 psi	160 psi	58 psi	210 psi	60 psi
7	90	115	190	110	250	90
28	110	150	240	200	330	170
91	110	150	280	320	360	320
	Soil 2					
Age 1 day	100 psi	60 psi	180 psi	60 psi	250 psi	65 psi
7	110	95	200	75	300	100
28	110	120	280	135	320	130
91	110	135	310	180	365	190

2.5 Minimum Requirements for Pavement Stabilization in Ethiopia

ERA pavement design manual (2013) states subgrade materials with CBR values <4% and swelling potential > 2% need to be replaced or treated with stabilizing agents. The manual also set minimum requirements for stabilized sub-grade and sub-materials respectively which are presented in Table 2.10 and Table 2.11.

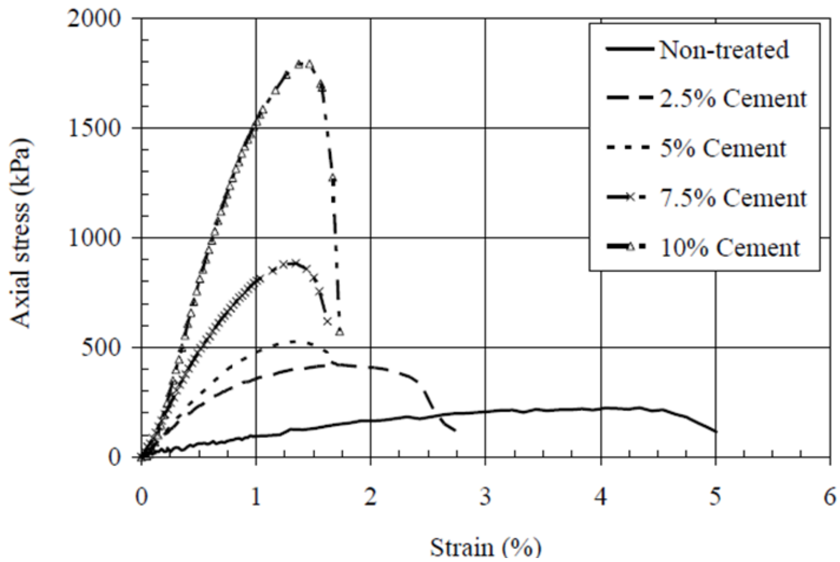


Figure 2.7: Effect of cement treatment on unconfined stress-strain behavior un-soaked samples. Muhunthan and Sariosseiri (2008)

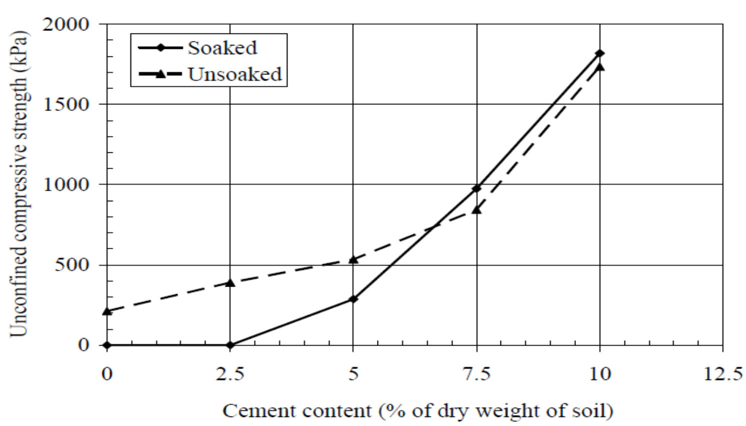


Figure 2.8: Effect of cement treatment on unconfined compressive strength of soaked and un-soaked samples (Muhunthan and Sariosseiri (2008)).

Table 2.10: Stabilized sub-grade requirement (ERA Site Investigation Manula 2002)

No.	**Test**	**Stabilized Sub-grade Requirement**
1	Atterberg Limits	Reduction of the plasticity index to less than 20.
2	Shrinkage Limits	Considerable increase of the shrinkage limit.
3	Swell	Reduction of the swell to negligible values.
4	CBR	Increase of the CBR to a minimum of 10

No.	Test	Stabilized Sub-grade Requirement
		(after 7 days cure) and 15 (after 28 days cure), with corresponding improvement of the subgrade strength class.
5	Particle size distribution	Modification of the particle size distribution (by agglomeration of the clay particles), the final grading being similar to that of a silt.

2.6 Sugar Cane Molasses

Sugar cane molasses is a viscous byproduct of the processing of sugar cane into sugar. During the sugar making process, the juice extracted from sugar cane is boiled down until the sugars crystallize and precipitate out. The syrup left over after crystallization is referred to as molasses (Olbrich, 1963).

There are three grades of molasses: *light molasses*, also known as *first molasses*; *dark, or second molasses*; and *blackstrap* (Peter, 2007).

Light Molasses-The result of this first boiling and of the sugar crystals is first molasses, which has the highest sugar content and the least viscous texture because comparatively little sugar has been extracted from the source.

Second molasses are created from a second boiling and sugar extraction. This molasses is darker and more viscous than light molasses, and contains less sugar.

Blackstrap molasses- final byproduct of the third boiling cycle in the sugar making process makes blackstrap molasses. This variety of molasses contains the least sugar and has the highest concentration of vitamins and minerals.

2.6.1 Chemical Composition of Molasses

Molasses is not just one chemical compound, but many. The main content is sugar (sucrose) ($C_{12}H_{22}O_{11}$). The composition of molasses is influenced by the soil where the cane is grown, climatic conditions, variety and maturity of the cane and the processing conditions at the factory (Olbrich, 1963).

Ndegwa (2011) has reported the mean constituent values for cane molasses as shown in Table 2.11 from four sugar factories in Kenya. Composition of Indian molasses and the world average of molasses composition have been reported by KBK (2009) and Olbrich (1963) are given in Table 2.11.

Table 2.111 Mean Constituent values for Cane Molasses from Kenya, India and World average. (Source: (Ndegwa, 2011; KBK, 2009 and Olbrich, 1963).

Constituents	**Kenya**	**India**	**World Average**
pH	5.7	4.2	-
Specific gravity	1.46	-	-
Moisture (%)	21.9	19.1	20
Total Sugar	57.0	55.96	62
Sucrose (%)	37.1	22.93	32
Invert Sugar (glucose & fructose) (%)	19.9	-	30
Ash (%)	8.1	19.75	-
Major mineral elements found in Cane Molasses			
Ca (%)	1.09	2.27	1.5
Mg (%)	0.15	-	0.1
Na (%)	0.02	-	-
K (%)	2.97	-	3.5
Si (%)	0.30	-	0.5

During sugar processing, some materials are added into the process as clarifying agents and evaporator decadents. These materials include lime and sulfur dioxide among others. During crystallization of the sugar juice, those elements remain in molasses and are then included in the natural molasses ingredients. Those elements plus others imbibed from the soil by the sugar cane as nutrients to support growth are the ones, which probably interacted with expansive soil to change its characteristics during stabilization (Ndegwa, 2011).

2.6.2 Molasses Production in Ethiopia

At present there are four operating sugar factories in Ethiopia. These are Wonji/Shoa, Metahara, Fincha and Tendaho sugar factory which have a total daily crushing capacity of 42, 250.00(Tone of Cane per Day). The current molasses production in Ethiopia is 1,260 ton of molasses per day. The Federal Democratic Republic of Ethiopia has launched sugar development program to undertake new and expansion projects across the country with a clear objective of boosting sugar production to satisfy the domestic sugar demand as well as to become one of the top 10 exporters in the coming 15 years. The new sugar factories which are being built are: Beles, Wolkait, Kesem and Omo Kuraz. After the construction and expansion of the new and existing sugar factories, the current 44,250.00 TCD crushing capacity will increase to 1,041,000.00 TCD almost by 2,253 % in the coming 15 years with molasses production about 30% of this tonnage. The molasses produced in Ethiopia is mainly used in production of ethanol, production of alcoholic & soft drinks, for baker's yeast and also used as animal feeds (Ethiopian Sugar Development Corporation, 2014).

CHAPTER 3

3. RESEARCH DESIGN, METHODS AND MATERIALS

3.1 RESEARCH DESIGN

Significance of Soil Stabilization in the Ethiopian Context

The road network provides the principal mode of freight and passenger transport in Ethiopia. Efficient transport plays a huge role in promoting development by lowering transport costs, reducing vehicle operating costs, cutting travel time and improving the quality of transport services. Therefore, good road network is a key of development to the country.

However, only 30% of Ethiopia's area is served by a modern road transport system with road density of 44.4 km per 1000 square kilometers which is lower than the average road density of 54 km per 1000 square kilometer for Sub-Saharan African countries. This limitation of the road network in Ethiopia often causes remoteness and isolation of communities. Remoteness leads to lack of services and severely constrains citizens' ability to contribute to the economy and development of the country (ERA, 2011).

With the increased global demand for energy and increasing local demand for aggregates, it has become expensive from a material cost and energy use standpoint to remove inferior soils and replace them with choice, well-graded aggregates. On the other hand, the extraction of substantial amounts of non-renewable natural resources for road construction creates significant damaging impacts on the local environment and its inhabitants (Caterpillar, 2006). Therefore, construction techniques implemented to solve these socioeconomic problems need to be not only time and cost effective but also environmentally friendly.

Sugar cane molasses is a viscous byproduct of the processing of sugar cane into sugar. In Ethiopia sugar production amount was 21.7 million tons in the year of 2010; however, in 2017 this amount will be increased to 51 million tons (Ethiopia Sugar Development Corporation, 2014). Increasing demand for sugar raises the generation of cane molasses material which constitute about 30% - 40 % of sugar volume. Molasses contains a resinous and some inorganic constituents that render it unfit for human consumption. Molasses could cause environmental pollution if spills are not properly cleaned. It can also cause water pollution if major spills or factory effluents enter river streams. It is therefore important to consider critically the handling and disposal of molasses particularly in situations where supply exceeds demand. This can arise, especially where industrial use of molasses is not diversified.

Chemical stabilization minimizes cost of pavement construction by reducing thickness of pavement layers and reduces depletion of natural resources by improving properties of in situ soil to acceptable levels. A stabilizer is more often required to improve the properties of weak and very weak soils (Alex and Jones, 2010). Such kind of weak soils can be referred to as problematic soils.

Chemical stabilization in pavement construction has been widely used in many parts of the world. For example, in South Africa, stabilization has been extensively used that one or more of the pavement layers in every sealed road in Gauteng (one of the provinces) is stabilized (Gautrans, 2004). In Ethiopia, the primary use of cement and lime stabilization has so far been with gravelly soils to produce road bases (ERA, 2002). Non-traditional stabilizers that have been introduced to the country include RBI Grade 81, CON-AID, SS44/LS40, Pure Crete, AnyWay Natural Soil Stabilizer and Zym-Tec Enzyme (Nigussie, 2011).

In Ethiopia, pavement soil stabilization is mainly limited to mechanical stabilization particularly blending soils of two or more gradations. It is a recommended practice to replace sub-standard or problematic soils in the country instead of treating them with traditional stabilizers for economic reasons (AACRA, 2004). However, the use of imported material that has to be borrowed at a specific site and transported over significant distances to the construction site is also expensive not only due to production and transportation costs, but also heavy haul of large quantities that damages the existing infrastructure resulting in increased maintenance costs. Currently the conventional stabilizing agents commonly used on expansive soils, lime and cement, which are fairly expensive; particularly in areas where suitable replacement materials are abundant and therefore rarely used in construction of roads passing on expansive soils in such areas. *Therefore, there is a need to investigate an alternative cheap technique for stabilizing in-situ expansive clay soil in to acceptable construction materials*. This can be achieved through application of various industrial by-products / locally available materials alone such as sugar cane molasses, fly ash, geo-fiber, Locust bean waste ash, etc. or blending with the conventional stabilizers as a soil modifying / soil stabilizing agent.

Sugar factories usually use cane molasses as a dust palliative on roads inside their compounds and few researches were done to evaluate the soil stabilization process of expansive soils using molasses. In this context, an extensive research is needed to understand the mechanism and geo-engineering properties of expansive soil stabilized with sugar cane molasses.

Cement is used to improve the expansive clay soil, but it is very expensive. Moreover, expansive clay soil treated with cement is prone to shrinkage cracks and rapid setting time of cement makes compaction difficult. Research conducted on the effect of sugar containing molasses on concrete have proved that sugar improves the quality of reaction between the cement and the aggregates and reduces setting time of concrete (Hasan and Baris, 2012; and Akogu, 2011). *Therefore, a research must be conducted to explore the effect of cement and molasses combination on stabilization of expansive clay soil*. Taking these into consideration, the aim of this research was to establish the effects of cement and molasses combination on expansive clay soils to reduce the cost of road construction as well as reducing the environmental hazard molasses causes.

The broad objective of this thesis work was to study the suitability sugar cane molasses and cement (OPC) combination for expansive clay soil stabilization by increasing their bearing capacity and decreasing their swelling potential. The specific objectives were investigating the response of the expansive clay soils through the application of the sugar cane molasses alone, cement alone and cement and molasses combination at various contents (0, 4, 8, and 12%) and at different curing durations (7 and 14 days) as well as to make comparisons on effect of stabilizers on soils treated with prescribed stabilizers.

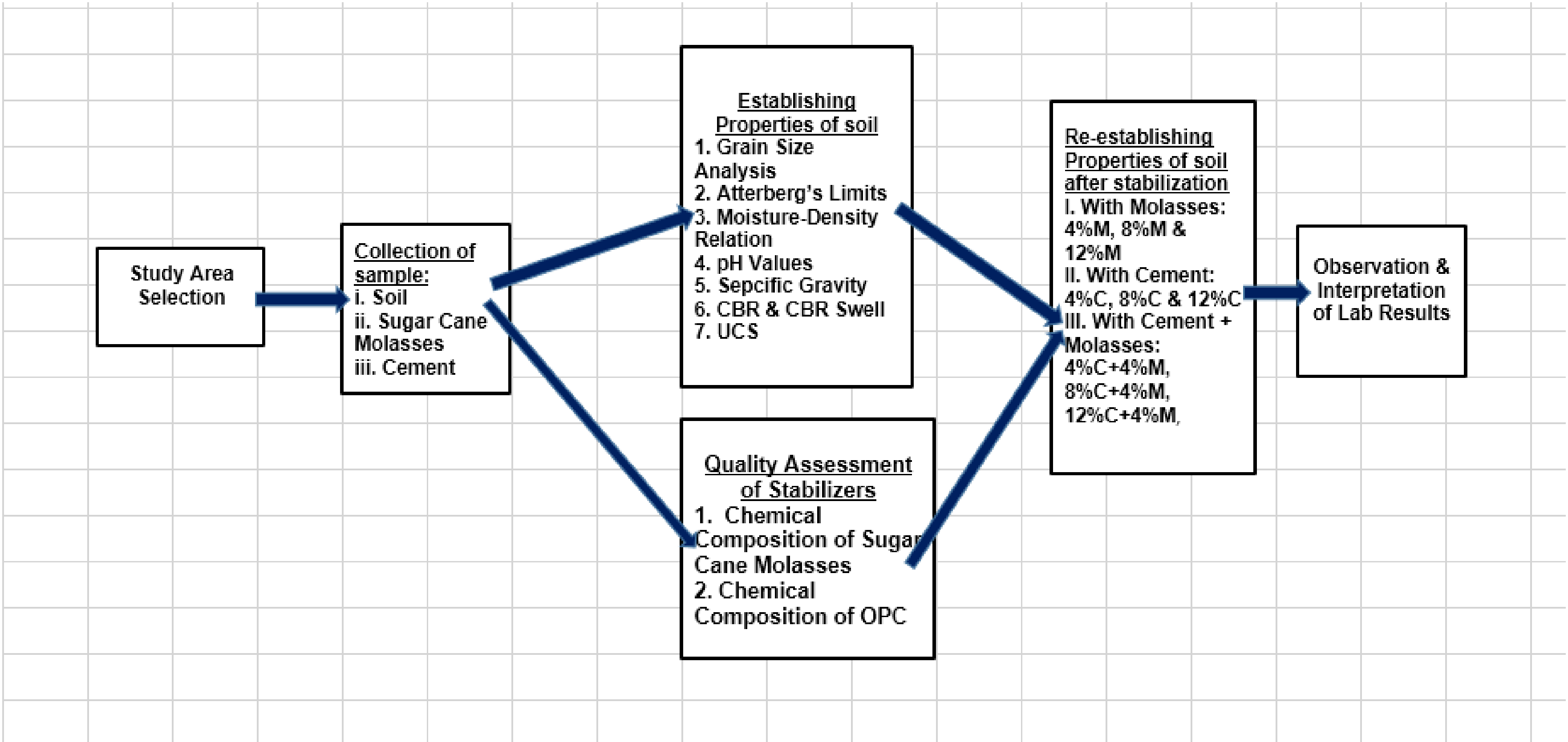
3.2 Research Methods

Based on the research objectives and scope as well as information from the literature review, research methods used to conduct this study were organized into the following six components (See Figure 3.1): study area selection, sample collection, quality assessment of stabilizers, establishing property of native soil, re-establishing property of stabilized soil, result analysis and interpretation, cost comparison of stabilization and replacement. Detail descriptions of each components are given from section 3.2.1 through section 3.2.6.

3.2.1 Study Area Selection

It is well known that expansive soils are found in the central, northwestern and eastern highlands of Ethiopia, in western lowlands around Gambella, and in some parts of the rift valley as shown in Figure 3.2. In this research expansive soils in the central part of Ethiopia were used for conducting the study. Among roads in this area whose premature pavement failures attributed to the volumetric changes of expansive soils; Modjio-Ejere Road segment was selected as study area for this research.

A visual site condition survey was carried out on the whole 35 km length of Modjio-Ejerie Road Segment to assess the performance of the road pavement and it was classified into three sections: good pavement section, moderately damaged pavement section and severely damaged pavement sections. Severely damaged pavement sections, in areas where expansive soil was prevalent, were identified. To identify pavement failures which were associated with shrinkage and swell property of expansive soil, ERA (2002) Site Investigation Manual was used. It is stated in the manual that pavement failures associated with shrink and swell property of expansive soil, first occurring in the shoulder area, and subsequently developing in the carriageway, as indicated in Figure 3.3. Accordingly, pavement failures around km 14+000 on Modjio-Ejere road segment was rated as severely damaged sections and expansive soils are prevalent in the area, see Figure 3.4 and 3.5.



Note: M=molasses, C=Cement and C+M= Cement and Molasses combination

Figure 3.1: Research Method adopted for conducting this research

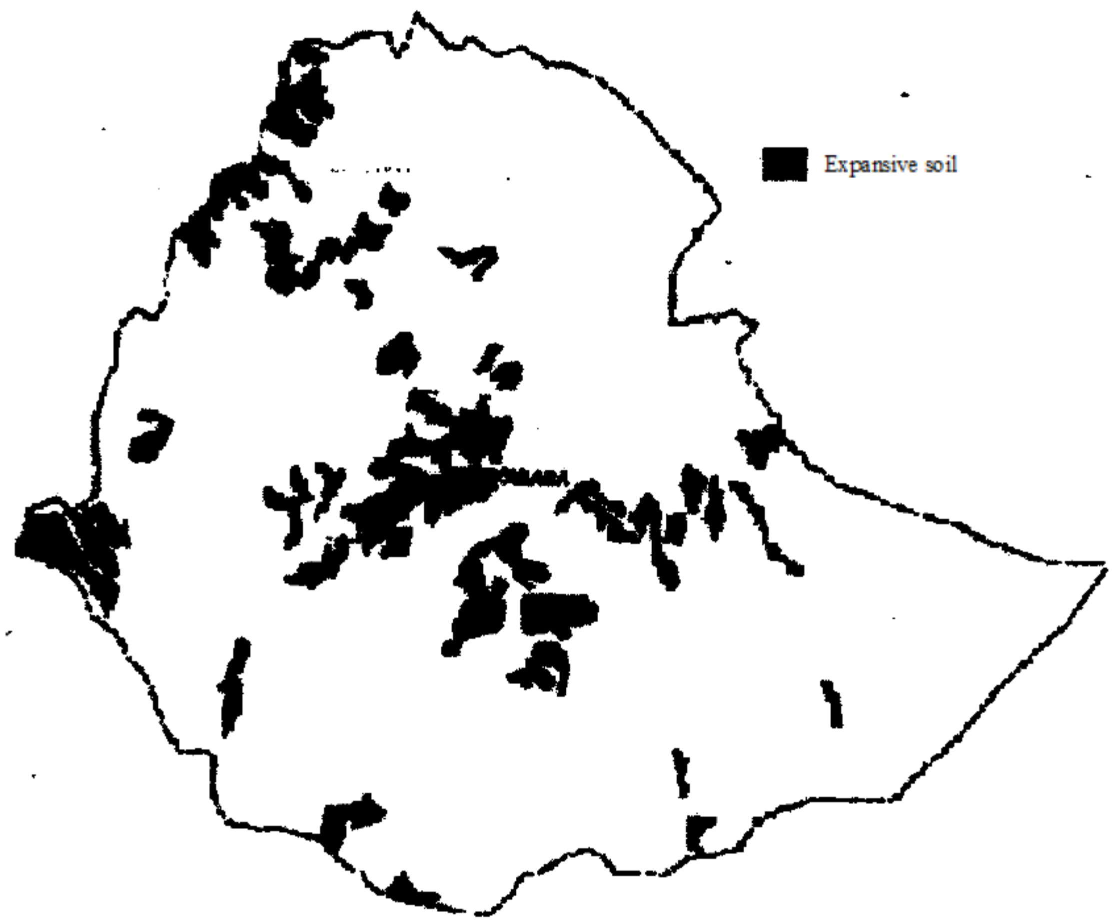


Figure 3.2: Distribution of Expansive Soil in Ethiopia (Source: Nigussie (2011)).

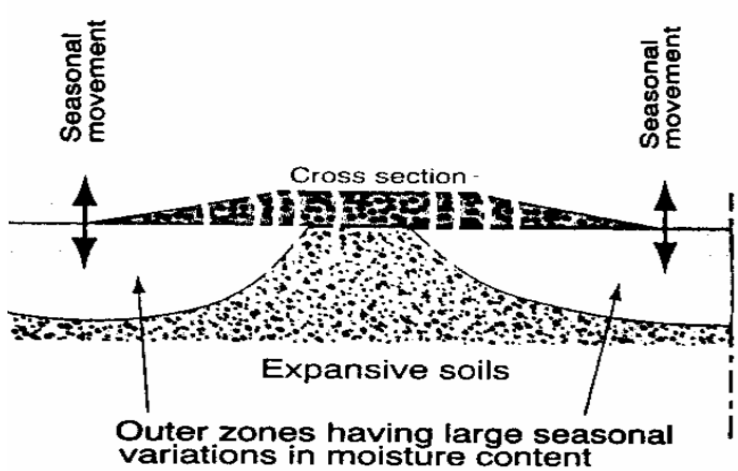


Figure 3.3: Moisture content in expansive soils (Source: ERA (2002))

Figure 3.4: Expansive Soils in Modjio-Ejerie Road around km 14+000 where pavement failure observed.

Figure 3.5: Road Failures near to Pavement Edge on shoulders (Severely damaged section around km 14 +000 in Modjo-Ejere Road segment)

3.2.2 Sample Collection

Expansive clay soil samples from three test pits from Modjo-Ejere road at 14 km distance from Modjo Town and 10m of the road center where pavement failures observed due to expansive clay soils were collected for use in soil stabilization. The samples were excavated manually using picks and spades to a depth 0.2m to 0.65m from natural the subgrade level. The soil from the surface, to a depth of 0.2m, was excluded from the sample because it is not normally used during road construction. The sample soils were transported to laboratories for conducting tests.

Figure 3.6: Sample soil collection from Modjo-Ejere Road at km 14+000

3.2.3 Quality Assessment of the Stabilizers

Black strap sugar cane molasses and Ordinary Portland Cement (OPC) which are proposed stabilizers for this study were brought from **Wenji/Shoa** Sugar Factory and Derba Cement Factory respectively. Quality assessments of the molasses were done at Wenji/Shoa Sugar Factory Research Center while quality test result of the OPC was obtained from Derba Cement Factory Laboratory.

I. Quality assessment of the molasses

Black strap molasses sample was purchased from Wenji/Shoa sugar factory molasses storage tanker on 07 August 2014, see Figure 3.7. The sample was given to Wenji/Shoa Sugar Factory Research Center for quality assessment. Physical properties of the molasses such as color, pH, specific gravity, and viscosity as well as the chemical composition of the molasses which are brix, moisture (%), total Sugar (%), invert sugar (%) and Ca (%) were assessed at Wenji/Shoa Sugar Factory Research Center. Once the molasses brought from the factory, it was stored in a cool and dry place and sealed to protect contact with the atmospheric air. The exact production date of the molasses was unknown during sampling since it was stored in bulk quantity. However, it is understood that the molasses had an average duration of 3 months during sampling.

II. Quality assessment of the OPC

Derba Ordinary Portland Cement (OPC) grade 42.5N type I cement, see Figure 3.8, was purchased for use in this research. Laboratory test results of physical and chemical properties of the OPC particularly it's as oxide composition such as SiO_2 (silica)%, Al_2O_3

(Alumina)%, Fe_2O_3 (Iron Oxide)%, CaO (Lime)%, MgO (Magnesia)%, SO_3 (Sulphur Trioxide)%, C_3S (of Clinker) – tricalcium silicate %, C_2S (of Clinker) dicalcium silicate %, C_3A (of Clinker) tricalcium aluminate %, C_4AF (of Clinker) – tetracalcium alumina-ferrite %, and free Lime % were obtained at Derba Cement Factory Laboratory.

Figure 3.7: Sample sugar cane molasses collection from Wenji/Shoa Sugar Factory

Figure 3.8: Derba OPC used in this research

3.2.4 Establishing Properties of Native Soil

To establish engineering property of the native/natural soil, the following tests were conducted for the three soil samples from the three test pits: natural moisture content, grain size analysis, Atterberg limits, shrinkage limits, pH test, specific gravity, free swell, modified Proctor, California Bearing Ratio (CBR), CBR Swell, and Unconfined Compressive Strength (UCS). Laboratory tests for the native soil were done in Chancho-Derba-Becho Road Construction Project Laboratory and CORE Consulting Engineers PLC Laboratory. Table 3.2 summarizes all conducted tests on native soil and their frequencies.

I. Initial Moisture Content of the Soil (AASHTO T-265)

The oven-drying method was used to determine the moisture contents of the samples. Small, representative specimens obtained from large bulk samples were weighed as received, then oven-dried at 105°C for 24 hours. The sample was then weighed, and the difference in weight was assumed to be the weight of the water driven off during drying. The difference in weight was divided by the weight of the dry soil, giving the water content on a dry weight basis.

II. Grain Size Distribution (AASHTO T-88)

The sieve and hydrometer analysis tests were conducted 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 the hydrometer method was used to determine the distribution of the finer particles. The sample was then washed through a series of sieves (No.4 (4.75 mm) sieve at top and No. 200 sieve at bottom) with progressively smaller screen sizes to determine the percentage of sand-sized particles in the specimens. Approximately 50 grams of dry soil, which was the fine soil from the bottom of the pan of sieve test was treated with a 125 mls dispersing agent (Sodium hexametaphosphate (40gm/L) solution) for 18 hours. A hydrometer analysis was then performed using 152H Hydrometer to measure the amount of silt and clay size particles.

III. Atterberg Limits (AASHTO T-89)

Representative samples of each soil were subjected to Atterberg limits testing to determine the plasticity of the soils. An Atterberg limits device was used to determine the liquid limit of each soil using the material passing through a 425 μm (No. 40) sieve. The liquid limit was determined as the water content, at which a pat of soil in a standard cup and cut by a groove of standard dimensions flowed together at the base of the groove for a distance of 13 mm (1/2 inch) when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. The plastic limit of each soil was determined by using soil passing through a 425 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. The liquid limit and plasticity index were then used to classify each soil.

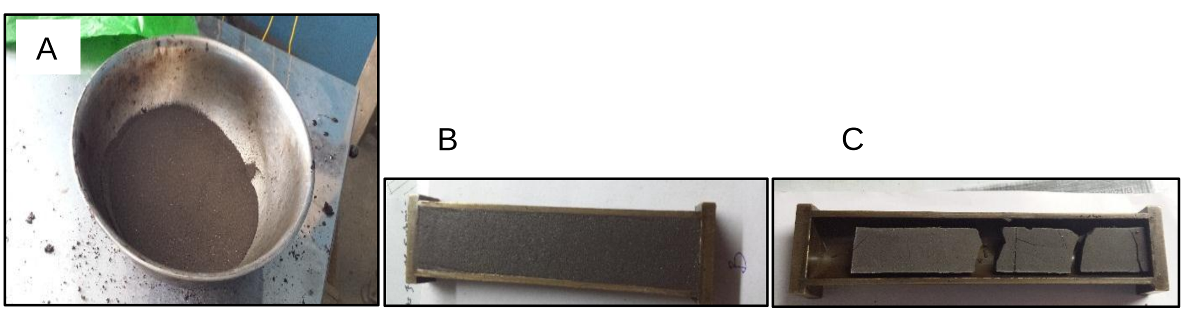


Figure 3.9: (A) Soil sample prepared for linear shrinkage, (b) & (C) specimens for linear shrinkage test before and after oven drying respectively.

IV. Shrinkage Limits (ASTM D 4943)

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

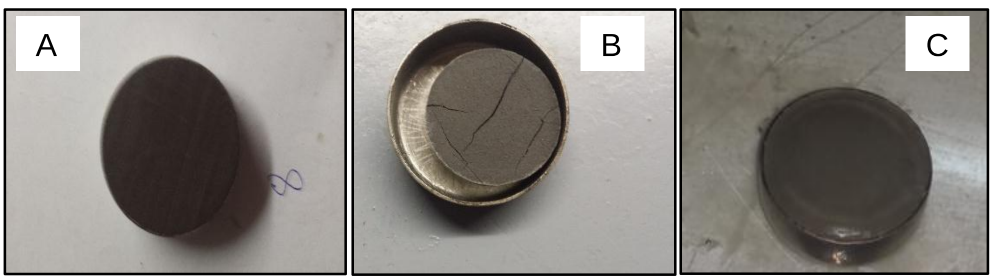


Figure 3.10: (a) & (b) specimen for Shrinkage limit test before & after oven drying, (c) wax coated after oven drying

V. Soil Classification (AASHTO M-145)

Each soil from the three test pits was classified using the AASHTO classification system. Using the particle size distribution and the Atterberg limits, the AASHTO classification system divides soils into groups and subgroups based on grain-size distribution, liquid limit, and plastic limit. A visual-manual procedure can also be used to identify soils easily in the field; however, all classifications provided in this research are based on the laboratory testing-based procedure.

VI. pH Values (ASTM D-6276)

The pH values of the native soils were determined on soil slurry prepared by mixing 50 gm of air dried soil passing through a 425 μm (No. 40) sieve and 200 ml of distilled water in dry plastic bottles. The plastic bottle cap was tightly sealed and shaken the soil-water mixtures for a minimum of 30 second or until the specimens are thoroughly mixed for 30 second every 10 minutes for 1 hour. Then the pH value was measured by bringing the mixtures at 25 $\pm$ 1^0 C and inserting calibrated pH probes inside the plastic bottle which contained the slurry.

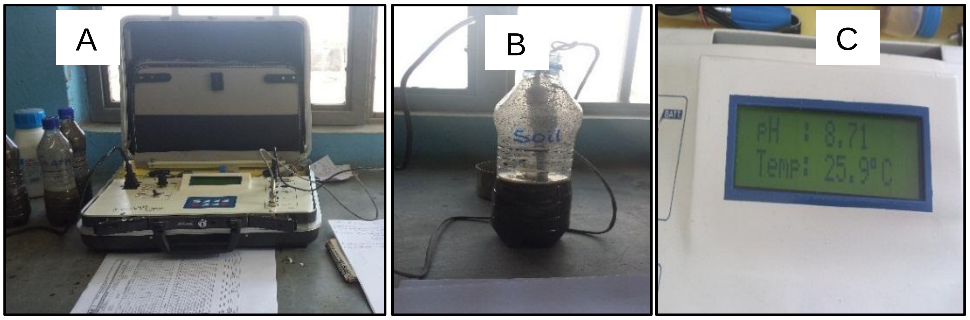


Figure 3. 11: Sample soil pH Value measurement. (a) pH measuring kit, (b) soil-specimen, and (c) pH value of soil

VII. Specific Gravity (AASHTO T-100)

Values for specific gravity of the soil solids were determined by placing a known weight of oven-dried soil in a flask, then filling the flask with water. The weight of displaced water was then calculated by comparing the weight of the soil and water in the flask with the weight of flask containing only water. The specific gravity was then calculated by dividing the weight of the dry soil by the weight of the displaced water.

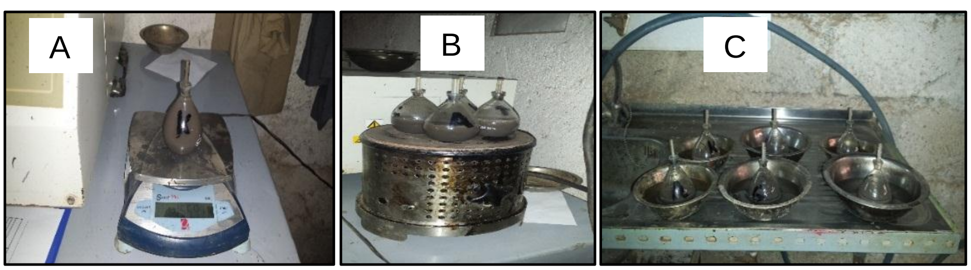


Figure 3.12: Soil specific gravity measurement test photos. (a) Measuring soil & water, (b) boiling soil & water to remove air bubbles, and (c) cooling with water

VIII. Moisture Density Relations of the subgrade Soils (AASHTO T-180)

Modified Proctor Test was done to determine the maximum dry density (MDD) and optimum moisture content (OMC) of the natural soil according to AASHTO T-180. A sufficient quantity of air dried soil were obtained in large mixing pan and pulverized the soil and run it through the No. 4 (4.75mm) sieve and prepare 5 representative samples each about 6000gm for a single Proctor Test using 6 inch mould. Compaction for each portion were done with 4.5kg hammer falling a distance of 18 inches, and used five equal layers by giving each layer 56 blows. For each proctor test five runs were conducted by increasing water content 2% of the preceding tests using 8% water content an initial run. These series of determinations were continued until there was either a decrease or no change in the wet unit mass (g/cm3) of the compacted soil.

Figure 3.13: Proctor test. (a) Soil-water mixing, (b) apparatus for compaction test, and (c) weight measuring after compaction

IX. California Bearing Ratio (CBR) Values and CBR Percent Swell Values (AASHTO T-193)

Three point CBR tests were done according to AASHTO T-193 to determine the strength of the sample soil and how it will behave when subjected to loading. About 20 kg quantity of air dried soil and passed through #4 (4.75mm) sieve were mixed at optimum moisture content in large mixing pan. The sample was divided to obtain three representative portions having a mass of approximately 6.0 kg each to be compacted about 10, 30, and 65 blows per layer for each equal five layers for one CBR mold. Moisture content of samples was collected before and after compaction. The compacted soil in CBR mold was then soaked in water for four days with 4.5 kg surcharge load applied to it. After the end of soaking period, the CBR mold was removed from water and drained for 15 minutes before CBR penetration. The CBR was performed on a mechanical press manufactured by ELE, 28 KN, which is shown in Fig. 3-14.

CBR samples penetration were performed at a strain rate of 1mm per minute (at a plunger force of 250N) and the readings of the force were taken at intervals for 0.5mm to a total of penetration not exceeding 7.5 mm. The CBR value was calculated at penetration of 2.5 and 5.0 mm and the higher value was taken. A graph was drawn for penetrations against dry densities in mould and CBR value at 95% of MDD was taken as the design CBR value. The design CBR value is used as the index measurement of soil strength. Table 3.1 presents the general sub-grade strength classes corresponding to ranges of CBR Value. Generally S1 & S2 class sub-grade are rated poor sub-grade soil and sub-grade soil whose CBR swell percent greater than 2 is considered to expansive sub-grade soil according ERA design manual (ERA, 2013).

Table 3. 1: Sub-grade Strength Classes (ERA, 2013)

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

Figure 3.14: CBR Test Procedure: (a) & (b) CBR specimen preparation, and (c) CBR penetration.

X. Free Swell Test (IS (Indian Standard) 2720)

For one free swell test, two 10 gm ovens dry soil passing through a 425 μm (No. 40) sieve was taken. Each soil specimen was poured into each of two glass graduated cylinder of 100 ml mark. One cylinder was filled with kerosene oil and the other with distilled water up to 100 ml mark. After removal of entrapped air (by gently shaking and stirring with a glass road), the soils in both the cylinders were allowed to settle for 24 hours. The final volume of the soils in each cylinder was read. The level of the soil in the kerosene graduated cylinder was read as the original volume of the soil. The level of the soil in the distilled water cylinder was read as the free swell level. The free swell index of the soils was calculated using Equation 3.1:

$$\textbf{\textit{Free swell index, Percent}} = \frac{V_d - V_K}{V_K} \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.\ 1)$$

Where: V_d= the volume of soil specimen read from the graduated cylinder containing distilled water, and

V_k= the volume of soil specimen read from the graduated cylinder containing kerosene.

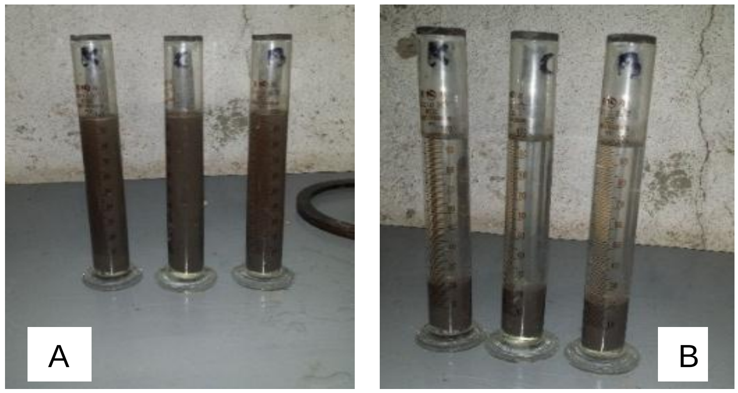


Figure 3.15: Free swell test: (a) initial volume, and (b) final volume after 24 hours

XI. Unconfined Compressive Strength (AASHTO T-208)

Unconfined Compressive Strength (UCS) testing was done according to AASHTO T-208. Figure 3-16 provides an illustration of the Unconfined Compression apparatus used for testing, HUMBOLDT, HM-3000. Samples were compacted in a 20 cm height by 6 cm diameter cylinder mold using modified proctor compaction. The compaction procedure was using 4.5 kg rammer with a drop of 450 mm, applying 25 blows evenly distributed blows to each of the 3 equal thick layers. After the specimen was formed, it was extruded from the Shelby tube sampler and cut height-to-diameter ratio of 2. The mass of the specimen, the length of specimen, and diameter of the specimen at mid height were determined and recorded. Having determined the mass and dimension of the specimens, then it was placed in the loading device. A strain rate of 2 percent per minute was used with measurements taken every 20 divisions on deformation until the load values decreased with increasing strain. The specimen was removed from the compression device and a sample for water content determination was taken. The UCS value obtained for cylindrical specimen was converted to the equivalent cube strength of a 150mm*150mm cube.

Figure 3.16: UCS test photo: (a) UCS compaction apparatus (b) UCS compression test machine.

XII. Potential Swell of Sub-grade Soil

One of the predominant properties of the sub-grade soils is a measure of the potential swell and this parameter is important to classify sub-grade soils based on their degree of expansion. Potential swell can be measured either directly or indirectly. The indirect methods involve the use of soil properties and classification schemes to estimate swell potential, whereas the direct methods provide actual physical measurement of swelling (Chen, 1988). Hence, in this research swelling potential of the soil was determined by index properties of the soil using indirect methods developed by Van Der Marwe's chart and Seed et al, 1962 model. The Van Der Marwe's chart as seen in Figure 3.17 measures swelling potential using plasticity index and percent of the clay fraction in the soil. The Seed et al, 1962 model calculates swelling potential as shown in Figure 3.18 using the activity of clay which is calculated as shown in equation 3.2 and percent of the clay fraction in the soil.

$$\boldsymbol{Activity(A_c)} = \frac{\boldsymbol{Plasticity\ Index}}{(\%\ \boldsymbol{by\ weight\ finer\ than\ 0.002mm}) - \boldsymbol{10}} \quad 3.2$$

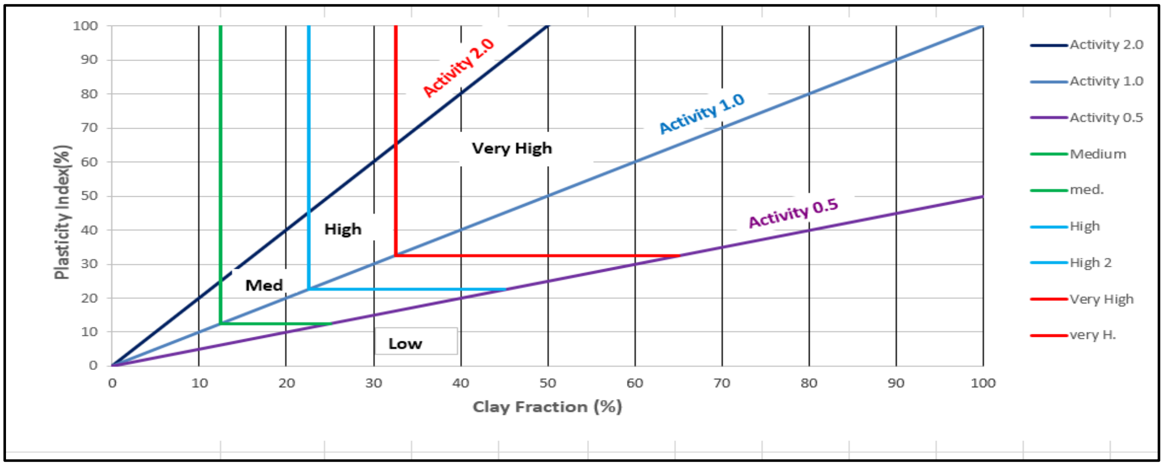


Figure 3.17: Swelling potential classification chart proposed by Van Der Marwe (1964)

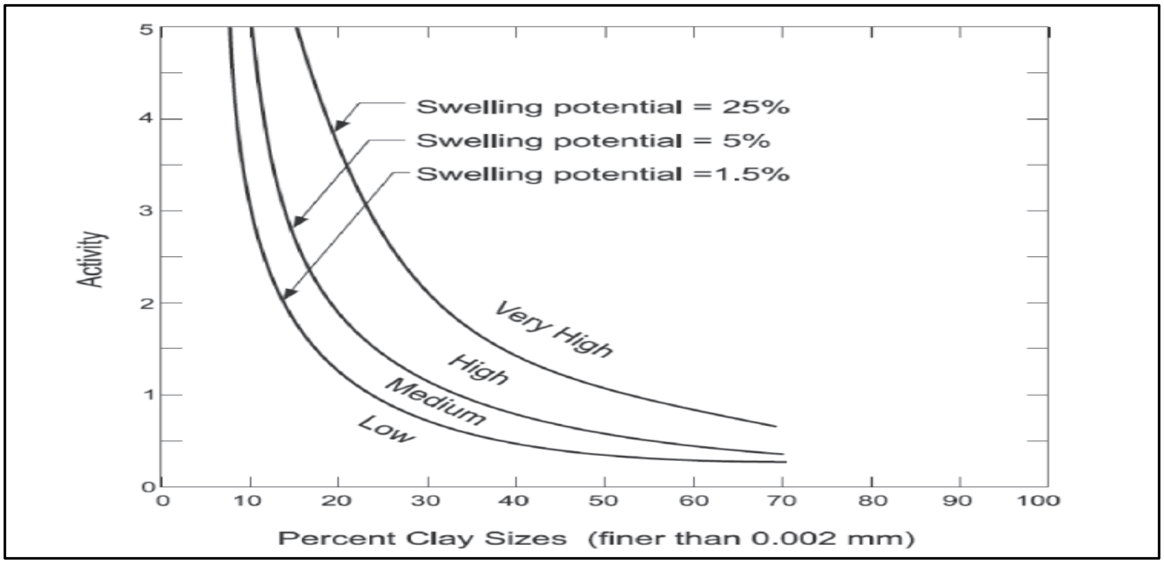


Figure 3.18: Classification Chart for Swelling Potential (after Seed, et.al, 1962)

3.2.5 Establishing Properties of Stabilized Soils

To establish engineering property of the stabilized soil, the following tests were conducted for soils treated with molasses only, cement only and cement and molasses combinations: pH test, Atterberg limits, Linear shrinkage, modified Proctor, California Bearing Ratio (CBR) and CBR Swell, and Unconfined Compressive Strength (UCS). Treated soil laboratory tests were done in Chancho-Derba-Becho Road Construction Project Laboratory, CORE Consulting Engineers PLC Laboratory, and Transport and Construction Design (TCD) Share Company Laboratory.

In this research, standardized test procedure synthesizes ASTM/AASHTO procedures for mixing specimens using traditional stabilizers (cement) with other procedures used for mixing specimens for non-traditional stabilizers (molasses only and cement and molasses combinations) were used. Applied curing periods and test frequencies for treated soil are presented in Table 3.3. The following is a brief discussion and an explanation of the procedures used for preparing and testing the specimens. The complete step-by-step test procedures for treated soils are included in Chapter 4.

3.2.5.1 Sample Care and Pre-treatment Storage

In order to prevent moisture loss, care was taken to protect the bulk soil samples. The samples soil was placed inside plastic bags under a cool and dry place. The sample sugar cane molasses which were brought form Wenji/Shoa Sugar Factor were stored in a dry plastic container and sealed tightly to prevent its contact with atmospheric air. Similarly, appropriate measures were given for the OPC cement to prevent its contact with atmospheric air to avoid the effect of carbonation and dampness. The OPC cement bag was not opened till conducting tests and immediately closed during the course of the test after taking the required amount of cement for tests from the bag.

3.2.5.2 Mixing of Soil and Stabilizers

I. Dosage Rates

Dosage rates can be specified in many different ways, but the most common way to define the dosage rate is based on the dry weight of soil to be treated. For the stabilizers used in this research dosage rates are given as a percentage of the dry weight of the untreated soil. Accordingly, the amount of stabilizer to be used was found from the following formula:

$$\boldsymbol{C} = \frac{\boldsymbol{YW(100-NMC)}}{\boldsymbol{100}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \textbf{\textit{(3. 3)}}$$

Where;C=mass of stabilizer required in gm

Y=percentage of stabilizers required

W=mass of air-dry material in gm

NMC=Natural moisture Content of the soil

Table 3. 2: Summary of test frequencies for native soils and treated soils

*ASTM D 5102-For treated soils

No.	Test Description	Test Method	Native Soil	Molasses Treated Soil			Cement Treated Soil			Cement and molasses Treated Soil			Total
				4% M	8% M	12% M	4% C	8% C	12% C	4% C+ 4% M	8% C+ 4% M	12% C+ 4% M	
1	Sieve and Hydrometer Analysis	AASHTO T-88	3										3
2	Specific Gravity	AASHTO T-100	3										3
3	Free Swell	IS 2720	3										3
4	pH	ASTM D6276	3	1	1	1	1	1	1	1	1	1	12
5	Atterberg Limits	AASHTO T-89	3	3	3	3	1	1	1	1	1	1	18
6	Linear Shrinkage	AASHTO T-90	3	3	3	3	1	1	1	1	1	1	18
7	Modified Procoter	AASHTO T-180	3	1	1	1	1	1	1	1	1	1	12
8	CBR	AASHTO T-193	3	2	2	2	1	1	1	2	2	2	18
9	CBR Swell	AASHTO T-193	3	2	2	2	1	1	1	2	2	2	18
10	UCS	AASHTO T-208/ ASTM D 5102*	3	4	4	4	1	1	1	1	1	1	21
	TOTAL												126

Table 3.3: Summary of applied curing duration for native soils and treated soils

No.	Test Description	Native Soil	Molasses Treated Soil			Cement Treated Soil			Cement and molasses Treate Soil		
			4% M	8% M	12% M	4% C	8% C	12% C	4% C+ 4% M	8% C+ 4% M	12% C+ 4% M
1	Sieve and Hydrometer Analysis	No curing									
2	Specific Gravity	No curing									
3	Free Swell	No curing									
4	pH	1hr	1hr	1hr	1hr	1hr	1hr	1hr	1hr	1hr	1hr
5	Atterberg Limits	30 mins	14 days	14 days	14 days	7days	7days	7days	7days	7days	7days
6	Linear Shrinkage	30 mins	14 days	14 days	14 days	7days	7days	7days	7days	7days	7days
7	Modified Procoter	10 min	10 min	10 min	10 min	10 min	10 min	10 min	10 min	10 min	10 min
8	CBR	2hr + 4day soaked	14day cure + 4day soaked*			7day cure + 7day soaked*			7day cure + 7day soaked*		
9	CBR Swell	4day soaked	4day soaked			7day soaked			7day soaked		
10	UCS	No curing	14 day cure & 64 day cure*			7 Day cure*			7 Day cure*		

*Cure at room temperature

Since sugar cane molasses are a non-conventional stabilizer, the amount of molasses required to improve the quality of the expansive clay soil through modification/stabilization was determined by trial and error approach using CBR values and historical data from previously conducted research. As shown in Figure 3.19 the highest CBR was determined to be 8% by dry weight of the soil. The proportion of molasses combined with expansive clay soil was taken as 4%, 8% and12% by dry weight of the soil.

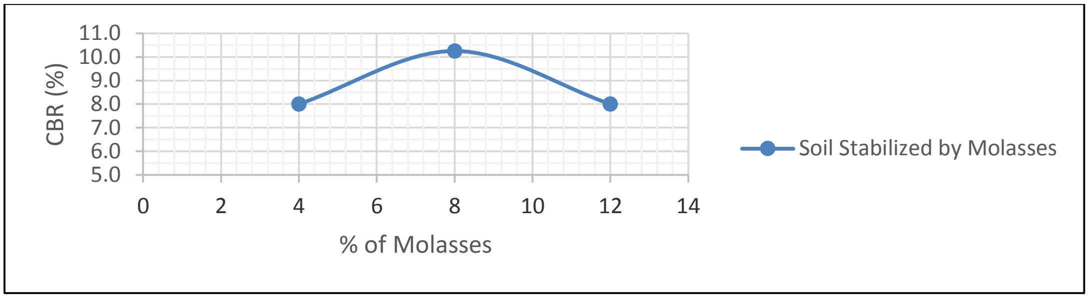


Figure 3.19: Variation of Soaked CBR values of Expansive soil mixed with various proportion of molasses

The minimum amount of cement added to the sample soil was determined according to AASHTO requirement for soil groups given in Table 3.4. Since the natural soil was classified as A-7, the quantity of cement that is required to stabilize the soil is from 10% to 16% by weight of the soil. Hence, the quantity of cement added to the sample soil was taken at 12% by dry weight of the soil and additional mixes were prepared at 4% and 8% cement by dry weight of the soil.

Table 3.4: Cement requirement for AASHTO soil Groups

AASHTO Soil Group	Usual Range in Cement Requirement in percent by		Typical 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

Molasses and cement combinations were mixed with the soil with varying the cement content by 4%, while keeping 4% molasses content constant, i.e. 1:1, 1:2 and 1:3 ratio of molasses and cement combinations were chosen randomly. Hence, sample soil was mixed with 4% cement +4% molasses, 8% cement +4% molasses and 12% cement +4% molasses by dry weight of the soil.

The amount of water required to provide the desired water content by considering the amount of extra water required for evaporation that occurred during mixing were measured into a suitable container. Once the Optimum Moisture Content had been determined from Proctor test, the quantity of water required for mixing was determined from the following relation given by equation 3.4.

$$\boldsymbol{V = \frac{Q(P+C)}{100} - (M-P)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; \textbf{\textit{(3. 4)}}$$

Where,

V=volume of water (ml)

Q=optimum moisture content

P=mass of material (Oven-dry) to be used (gm)

C=mass of stabilizer to be added (gm)

M=mass of material (air-dry) (gm)

II. Mixing Procedures

For cement stabilized soils, it is recommended dry mixing with the soil first with cement and then adding a measured amount of water. In this research for cement treated soil, the aforementioned method of mixing was followed. For soil treated with molasses only and cement and molasses combinations wet mixing method were followed. For soil treated with molasses only, first calculated amount of molasses based on the dry weight soil was diluted with measured amount of water, then diluted molasses with water added to the pre-determined amount of soil to test.

For soil treated with cement and molasses combination, first calculated amount of molasses based on the dry weight soil was diluted with measured amount of water. Then measured amount of cement based on dry weight of soil and pre-determined soil for the test were dry mixed. Finally, diluted molasses with water was added to soil-cement dry mixture.

A small mechanical mixture shown in Figure 3.20 (a) was tried for mixing of the soil and stabilizer. However, it was found inefficient for blending soil and molasses together due to an adhesive property of molasses as shown in Figure 3.20(b), soil and molasses mixtures was not uniformly mixed rather clumping of the soil and/or stabilizer was formed. Therefore, hand mixing was used in this research for mixing soil and stabilizers. To achieve the desired moisture content for each batch of soil and stabilizer mixtures, hand mixing continued until the soil and stabilizers were uniformly mixed, see Figure 3.21 and 3.22.

Figure 3.20: Soil and stabilizer mixing with mechanical mixer. (a) Mechanical mixer and (b) clumping of soils and stabilizers

Figure 3.21: Measuring soil (left), molasses (middle) and Water (right)

Figure 3.22 Mixed molasses with water (left) and mixing soil with molasses and water mixture (right)

III.Mellowing Time

Some published test procedures mention a “mellowing time,” which is a rest time between mixing the amendment with the soil and compacting the mixture into molds to form specimens. For cement-amended soils, a mellowing time is generally not specified in AASHTO T 134-95 and ASTM D-558-82 for cement treated soils, hencethe mellowing period for mixture preparations of soil and stabilizer had not been applied in this research but rather an absorption period.

Potable water, which was diluted with molasses for molasses treated soils and pure water for cement treated soils, just 3% above optimum moisture content was added to the measured amount of soil or soil-cement mixture and mixed thoroughly with hands, till tightly squeezed in the palm of the hand. Covered, and allowed to stand for not less than 5 minutes, but not more than 10 minutes to aid the dispersion of the moisture and to permit more complete absorption by the soil-cement, soil-molasses or soil-cement-molasses mixtures. After the absorption period, thoroughly break up the mixture, without reducing the natural size of individual particles, until it will pass a No.4 (4.75-mm) sieve and then re-mix before making specimens.

3.2.5.3 Specimen Molding Procedures

Strength testing, CBR and UCS, were performed on samples molded and cure in the following ways.

I. CBR Specimen Molding Procedures

The specimen was prepared by compacting soil-stabilizer mixtures into 6 inch CBR mould. The mould height was divided into equal five thicknesses and soil-stabilizes mixtures for one mould were divided into equal five parts by mass. Each equal five layers were compacted into the molds by giving 10, 30 and 65 blows for three point CBR tests for each prescribed stabilizers content. Each layer should occupy about or a little more than one-fifth of the height of the mould. Ensure that the blows are evenly distributed over the surface. The final level of the soil surface should be about 5-10 mm above the top of the mould body. The extension collar was removed, and using a straightedge, the compacted soil was trimmed even with the top of the mold. Surface irregularities were patched with small-sized material. The spacer disk was removed and a coarse filter paper was placed on the perforated base plate. The compacted specimen was ready for curing.

II. UCS Specimen Molding Procedures

UCS test was performed on samples casted and cure in 60 mm diameter by 200 mm tall metal cylindrical molds at optimum moisture content and density found from Proctor Test. Soil-stabilizers mixtures for one specimen based on the volume of the metal cylindrical mold were divided into three equal mass and one third of the height of the mold was determined. Specimens were molded to the desired unit by kneading and tamping each three layer until the accummulated mass of the soil-stabilizer mixtures placed in the mold was compacted to a known volume (one third volume of each layer).

During preliminary testing, it was noticed that there were weak joints between each layers. Hence, to remove weak joints between each layer, the top of each layer was scarified prior to the addition of material for the next layer. This technique eliminated the initially encountered problems and helped to get uniform specimen. Compaction of all specimens from a single batch was completed within 30 minutes of completion of mixing. After the specimen was formed, it was extruded from the Shelby tube sampler, and then it was ready for the curing process.

3.2.5.4 Curing

I. Curing Temperature and Humidity

Since this research did not involve an investigation of variations of curing temperature, all samples were cure at room temperature (approximately 20°C). Generally, two types of curing process were applied for this research, according to standards and/or data from previously conducted research. In the first curing method, the tightly sealed samples were submerged in a water bath to provide a curing environment of 100% relative humidity, while in the second curing method the samples were sealed in plastic bags and stored at room temperature to prevent the effect of carbonation and loss of moisture during the curing period.

II. Curing Time

Curing times of 7, 14, and 64 days were used in this research. Two samples for each curing time were prepared in order to provide an indication of reproducibility as well as to provide sufficient data for accurate interpolation of the results. Additional curing times beyond 64 days for UCS test of soils treated with molasses was desired in some cases to investigate longer term changes in strength. Table 3.3 summarizes applied curing durations and methods and Table 3.4 gives a summary of test frequency used in this research.

3.2.6 Result analysis and Interpretation

After treatment of the expansive clay soil with molasses only, cement only and cement and molasses combination of prescribed stabilizer contents, engineering properties of the stabilized soil were established and *qualitatively* compared to stabilized subgrade requirement, see Table 3.5, given in the ERA (2002) Pavement Design Manual. Then, the lowest stabilizer content which met all the criteria given in Table 3.5 for stabilized sub-grade was chosen as optimum stabilizer content for stabilizing the in-situ expansive sub-grade clay soil.

Table 3. 5: Stabilized sub-grade requirement (ERA 2002)

No.	Test	Stabilized Sub-grade Requirement
1	Atterberg Limits	Reduction of the plasticity index of less than 20.
2	Shrinkage Limits	Considerable increase of the shrinkage limit.
3	Swell	Reduction of the swell to negligible values.
4	CBR	Increase of the CBR to a minimum of 10 (after 7 days cure) and 15 (after 28 days cure), with corresponding improvement of the subgrade strength class.
5	Particle size distribution	Modification of the particle size distribution (by agglomeration of the clay particles), the final grading being similar to that of a silt.

3.3 Materials Characterization

3.3.1 Soil

I. Grain Size Distribution

The sieve and hydrometer analysis test results showed that on average the native soil contained 20% sand, 25% silt and 55% clay by weight. The grain size distribution of the soil samples is presented in Table 3.7 and Figure 3.23.

II. Atterberg Limits and Shrinkage Limits

Atterberg limits and shrinkage limits tests were conducted as described in section 3.2.2.1 according to ASTM D 4943 and results are presented in Table 3.6. Based on the grain size analysis tests and Atterberg limit tests, the soils were classified as A-7-5 (20) according to the AASHTO Classification System. Further, the soil samples tested to Atterberg Limits were plotted over the Casagranede's plasticity chart, see Figure 3.24. As it may be seen in the chart all sample soils tested are plotted above "**A"** line and are inorganic clays of high plasticity. This soil could undergo large volumetric changes if its moisture content changed.

Table 3.6: Atterberg Limits, Shrinkage limits and Group Index (GI) for the Soil

Sample No.	Depth of Sample (cm)	Visual description of Material	Atterberg Limits			GI	Shrinkage Limit (%)
			LL (%)	PL (%)	PI (%)		
Test Pit #1	20-60	Black Cotton Soil	90	37	53	20	7
Test Pit #2	20-60	Black Cotton Soil	90	35	55	20	5.1
Test Pit #3	20-60	Black Cotton Soil	87	36	51	20	6.4

The sample materials doesn't fulfill the minimum requirement set by ERA Specification of fill materials which requires materials shall have a liquid limit not exceeding 60% and a plasticity index not exceeding 30 when determined in accordance with the requirements of AASHTO T-89 and T- 90 (ERA, 2013).

III. pH Values

The pH values of the native soils were 8.18, 8.2 and 8.21 for test pits one, two and three respectively. These results show that the soils were neutral implying that it was vertic soil (Fitzpatric, 1986 cited in Ndegwa, 2012).

IV. Specific Gravity

The specific gravity values ranged from 2.54 to 2.66 which fell within the range for clay soil.

V. Moisture Density Relations of the subgrade Soils

Modified Proctor tests were conducted on the sub-grade soils to determine the relationship between the moisture content and dry densities. The results showed optimum moisture contents were 26.9%, 28.8% and 27.1% and the maximum dry densities were 1.48 gm/cm3, 1.50 gm/cm3 and 1.51 gm/cm3 for test pits one, two and three respectively.

Table 3.7: Grain size distribution of the sample soils

Test Pit 1		Test Pit 2		Test Pit 3	
Sieve Size (mm)	% Passing	Sieve Size (mm)	% Passing	Sieve Size (mm)	% Passing
9.5	100	9.5	100.00	9.5	100
4.75	99.96	4.75	99.97	4.75	99.97
2.36	99.67	2.36	99.64	2.36	99.62
2.00	99.60	2.00	99.56	2.00	99.51
1.18	98.97	1.18	98.83	1.18	98.70
0.600	97.22	0.600	96.92	0.600	96.61
0.425	94.71	0.425	94.17	0.425	93.64
0.300	91.47	0.300	90.85	0.300	90.23
0.150	86.55	0.150	86.09	0.150	85.63
0.075	79.97	0.075	79.96	0.075	79.94
0.028	67.85	0.028	67.03	0.028	66.21
0.018	63.00	0.018	62.99	0.018	62.98
0.011	61.39	0.011	61.37	0.011	61.36
0.008	59.77	0.008	58.95	0.008	58.13
0.005	58.16	0.005	57.34	0.005	56.52
0.003	53.31	0.003	52.49	0.003	51.67
0.001	48.46	0.001	48.45	0.001	48.44

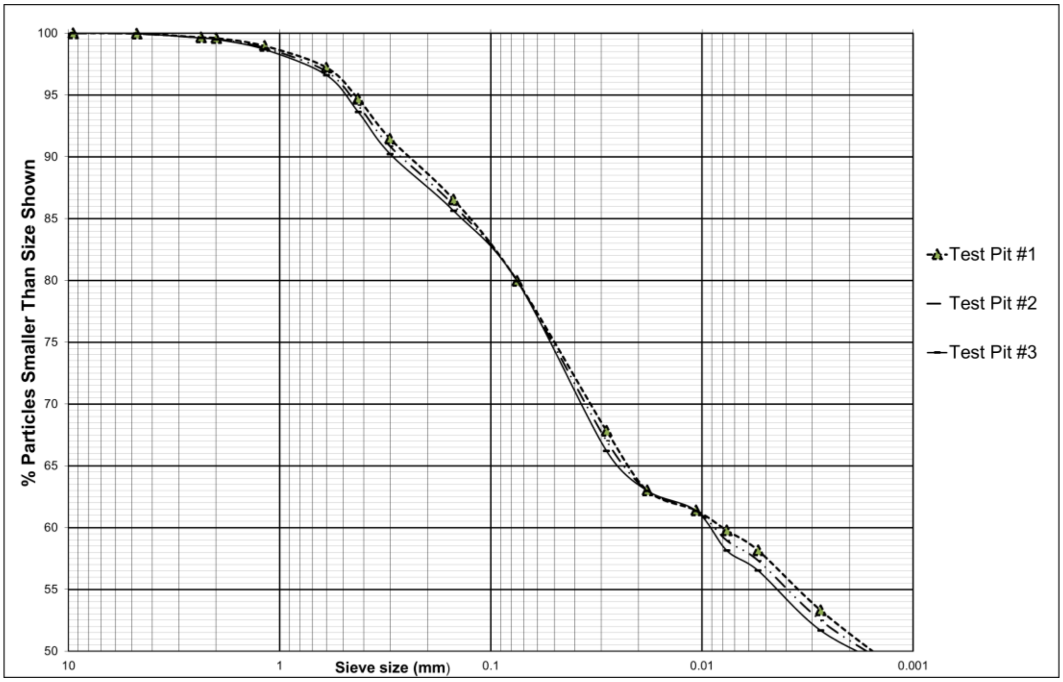


Figure 3.23: Grain Size Distribution Curves for the Soil Samples Tested

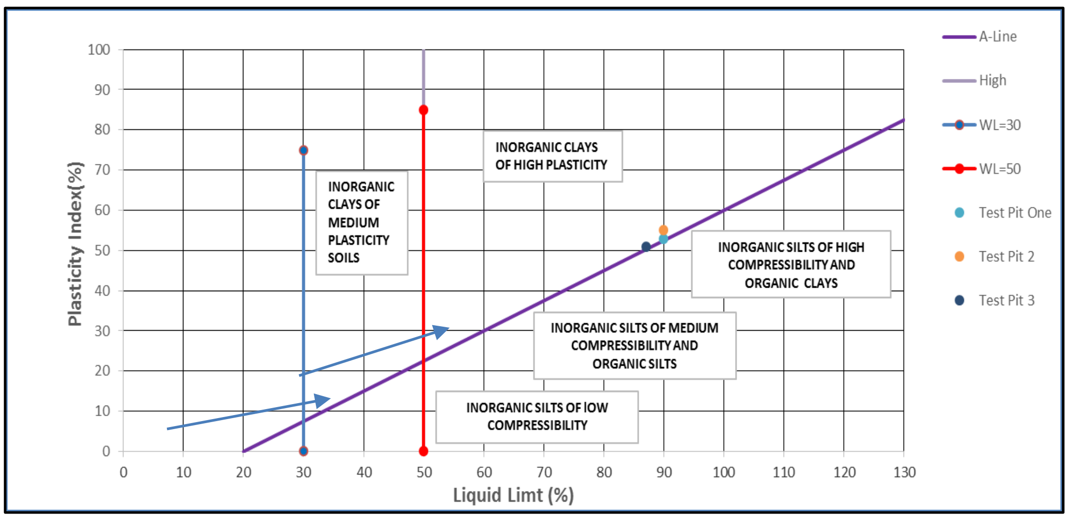


Figure 3.24: Casagranede's plasticity chart

VI. California Bearing Ration (CBR) Values and CBR Percent Swell Values

The CBR test result values were determined at a density of 95% of the maximum dry density and showed that the sub-grade soils had very low CBR value of 1%, which does not satisfy the minimum requirements stipulated as sub-grade materials and embankment material in ERA specification. Hence, it can be concluded that the sub-grade soil under investigation cannot be used as sub-grade and/or selected fill material for embankment construction unless otherwise its undesirable properties are rectified (ERA, 2013).

The CBR percent swell tests were determined with 4.5 Kg surcharge ring in accordance with AASHTO-T193. Readings of swelling values were taken before soaking and after 4 days soaking of CBR molds for 10 blows, 30 blows and 65 blows and the swell percent value were taken as the average of the three molds. The results showed that 9.3%, 11.6% and 10.2% swell of CBR for test pits one, test pit two and test pit three respectively. These results are found to be comparable to values obtained from laboratory tests of expansive soils found in different parts of Ethiopia. The CBR percent swell values are well above the permissible limits of 2.0 % for sub-grade and select fill material requirement of ERA specification (ERA, 2013).

VII. Free Swell Test

Free swell tests were conducted according to IS 2720 as described in section 3.2.2.1 and free swell values of 110%, 120% and 110% were observed for sample soil.

VIII. Unconfined Compressive Strength (UCS)

Unconfined compressive strength tests were conducted for the sample soils and results are given in Table 3.8 and Figure 3.25 and the un-drained shear strengths were calculated as one half of the UCS values as shown in Equation 3.5. The un-drained shear strength (Su) of clays is commonly determined from unconfined compression test. The un-drained shear strength (Su) of a cohesive soil is equal to one-half the unconfined compression strength (qu) when the soil is under the f=0 condition (f=the angle of internal friction). The most critical condition in the soil usually occurs immediately after construction, which represent un-drained conditions, when the un-drained shear strength is basically equal to the cohesion (C). This is expressed as:

$$S_u = C = \frac{q_u}{2} \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.5)$$

Table 3.8: UCS, strain at failure and Cohesion Values for the three test pits

Test Pits	**UCS (KN/m2) =qu**	**Cohesion (KN/m2) = qu/2**	**Moisture Content (%)**
Test Pit One	141.03	70.52	27.3%
Test Pit Two	138.76	69.38	27.9%
Test Pit Three	135.83	67.92	27.1%

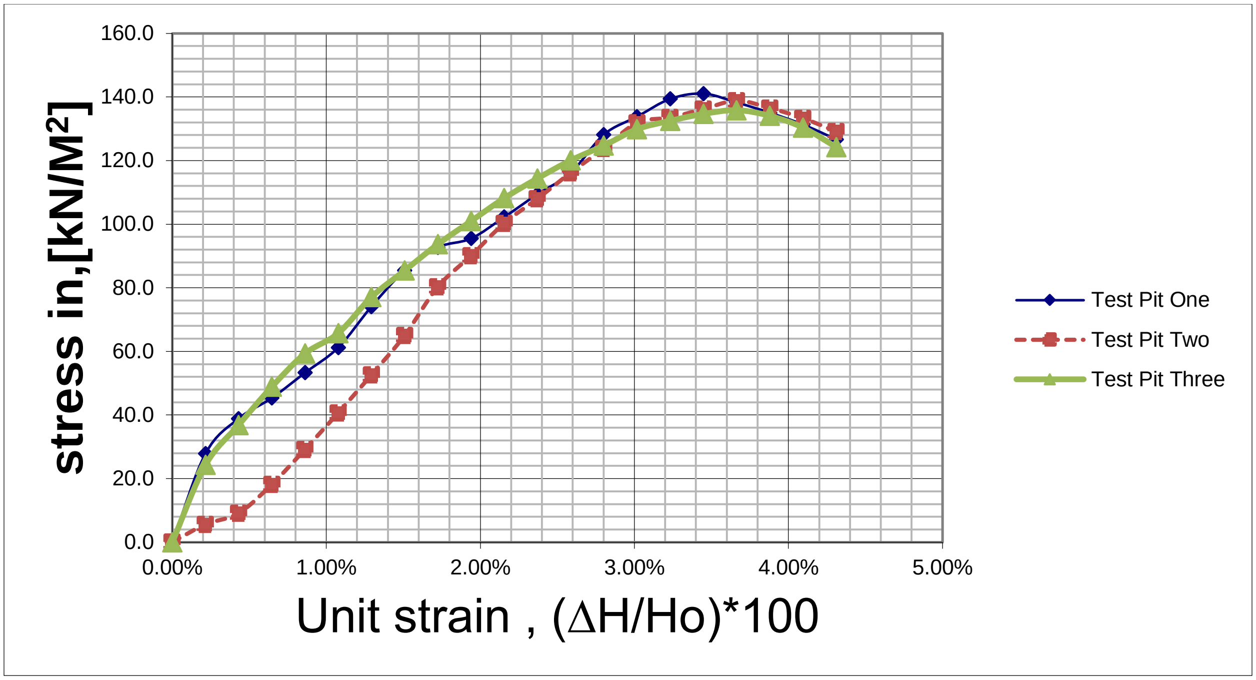


Figure 3.25: Stress –Strain Diagram for the Sample Soils

IX. Indirect Estimation of Potential Swell

a) Potential Swell based on Clay Fraction

The sub-grade soil samples have been plotted on the Van Der Marwe's chart as seen in Figure 3.26 and found to be highly expansive soil (Van Der Marwe, 1964).

b) Potential Swell based on Activity

Activities of the clay soil samples collected for this research were calculated by empirical relations developed by Seed et al, 1962 model and given in Table 3.9 and plotted on clay content against activity chart as shown on Figure 3.27. All the samples were plotted to have very high swelling potential which is above 25%.

Table 3.9: Activity result calculated according to Seed et al model

Test Pit	PI	C	Activity
1	54	49.00	1.38
2	55	49.00	1.41
3	51	50.00	1.28

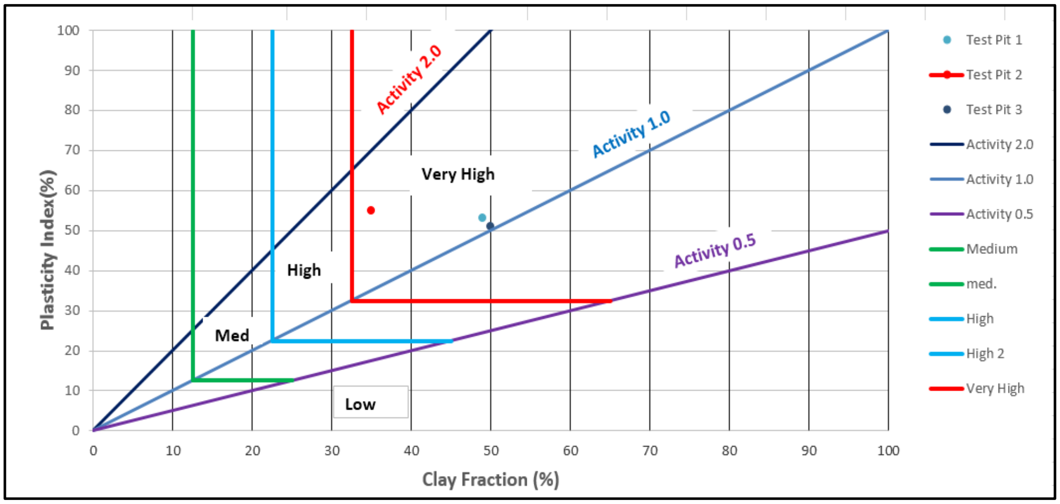


Figure 3.26: Plot using swelling potential classification chart proposed by Van Der Marwe (1964)

3.3.2 Sugar Cane Molasses

Constituent values of the Cane Molasses obtained from Laboratory Test Results are shown in Table 3.10. The results have shown that the sample molasses is high grade black strap molasses and its chemical composition are within the range of the world average molasses composition briefly discussed in the literature review parts and very close to the mean constituent values for cane molasses from sugar factories in Kenya and India (Hubert, 2006; Ndegwa, 2012 and Peter, 2007).

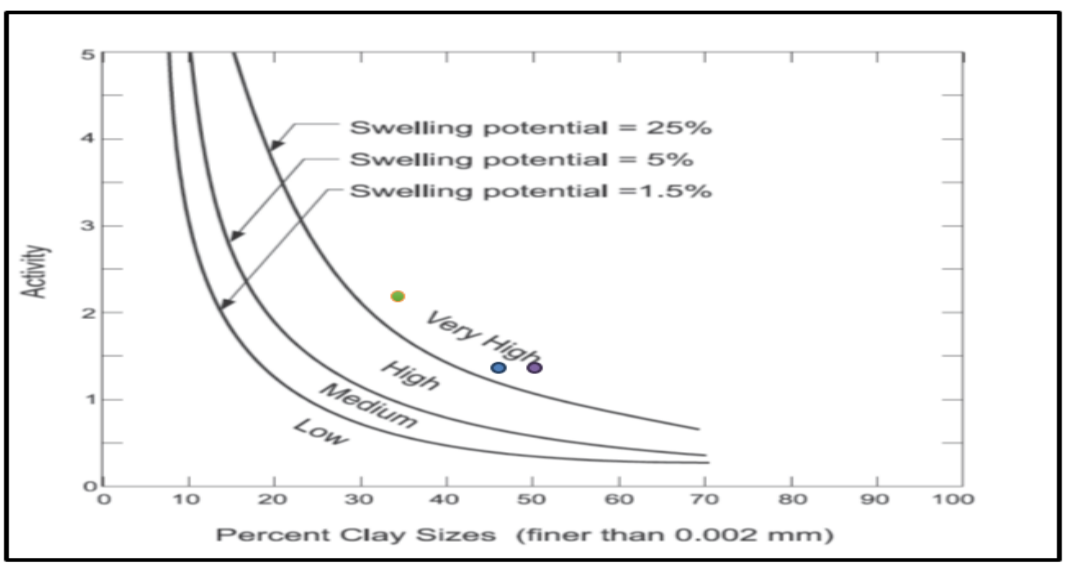


Figure 3.27: Classification Chart for Swelling Potential (after Seed, et.al, 1962)

From the major mineral elements which could be found in cane molasses only Ca composition was identified in the sample molasses and other minerals such as Mg, Na, K and Si were not known due to laboratory capacity limitation in Wenji/Shoa Sugar Factory Research Center. Since the sample molasses is high grade black strap molasses and its constitutes are very close to the world standard, other major minerals like Mg, Na, K and Si may exist in the sample molasses.

Table 3.10: Constituents values for Cane Molasses obtained in this study

Constituents	**Wonji/Shoa Molasses Result**
Brix	84.9
pH(1:1 at 20^0C)	5.5
Specific gravity at $20/4^0$C	1.43181
Viscosity @ 30^Oc (mPa. s)	18400
Viscosity @ 60^Oc (mPa. s)	6600
Moisture (%)	23.78
Total Sugar (%)	48.46
Invert Sugar (%)	10.53
Sulphated ash (%)	14.52
Major mineral elements found in Cane Molasses	
Ca (%)	1.60

From laboratory test results it can be concluded that the sample molasses contained elements/compounds which were active in causing a chemical reaction involving cation exchange with the expansive clay soil (Ndegwa, 2012).

3.3.3 Ordinary Portland Cement (OPC)

Laboratory test results of physical and chemical Properties of Derba Ordinary Portland cement (OPC) were obtained from Derba MIDROC Cement Factory on December 12th 2014 and are presented Table 3.11. The oxide composition result of Derba OPC showed that the presence of lime (CaO) more than 60% and other elements/compounds which were active in causing a chemical reaction involving pozzolanic reaction, cation exchange and hydration reaction with the expansive clay soil.

Table 3.11: Oxide Composition of Derba OPC

No.	Specification Items	Unit	Measured
1	Loss on ignition	%	2.05
2	SiO_2 (silica)	%	21.63
3	Al_2O_3 (Alumina)	%	5.52
4	Fe_2O_3(Iron Oxide)	%	3.42
5	CaO(Lime)	%	61.45
6	MgO(Magnesia)	%	1.86
7	SO_3(Sulphur Trioxide)	%	2.71
8	C_3S (of Clinker) – tricalcium silicate	%	$\geq$55.0
9	C_2S (of Clinker) – dicalcium silicate	%	$\geq$14
10	C_3A (of Clinker) – tricalcium aluminate	%	$\geq$8
11	C_4AF (of Clinker) – tetracalcium alumina-ferrite	%	$\geq$10
12	Free Lime	%	0.45-0.80
13	Liter Weight	G/Lt	$\geq$1310

CHAPTER 4

4. Sample Preparation and Experimental Procedures for Stabilized Soils

4.1 pH Test

The method developed by Eades and Grimm is essentially the measurement of the pH of a soil stabilized with various percentages of lime. The word “lime” may be substituted by cement, or any other pozzolanic stabilizer, as they all behave in a similar manner. The quantity of cement necessary to maintain a pH of 12.4 in a cement-soil-water mix after 1 hour is considered to be the Initial Consumption of Cement (ICC) of the material. For adequate stabilization using pozzolanic stabilizers, sufficient stabilizer should be added to ensure an excess after the reactions are complete, i.e., initial consumption of cement (ICC) of the soil should be satisfied and an excess provided. The lime demand of the soil is the primary reaction and is responsible for the consumption of a portion of lime [Ca (OH) 2] released during the setting and hardening process of cement. In such a scenario, the ICL (initial consumption of lime) could seriously interfere with the gain in strength since only the lime that is left after the lime demand of the soil has been satisfied is available for the binding and hardening process. A stabilizer content of ICC plus an additional 1 percent is recommended.

A series of specimens were prepared as shown in Figure 4.1 containing a range of percentages of molasses, cement and cement and molasses content in soil. Measurements of pH were made on slurries of the specimens according to ASTM D 6276. The following apparatus was used for the tests: balance, sieve (No. 40), plastic bottles, thermometer, water bath heater and pH meter.

pH Test Procedure

i. 50gm mass of air-dried soil and corresponding amount of stabilizers by dry weight of the soil were determined as given in Table 4.1.
ii. 100 ml of water was added to each soil-stabilizer mixtures in the plastic bottle, see Figure 4.1.
iii. The bottles were shaken until the specimens were thoroughly mixed and continued shaking the specimens for every 10 minutes for 1 hour.

iv. As shown in Figure 4.2 specimens were put in a water bath heater to bring the temperature of the specimen to 25 $\pm$ 1°C during the test.

v. Within 15 minutes by the end of the 1-hour shaking period, the pH of each slurries were determined while maintaining its temperature at 25 $\pm$1°C. See Figure 4.3.

Table 4.1: Mix Design for pH Test

Test	Mass of air-dry material in gram (passing through sieve No.40 (0.425mm)	NMC (%)	Mass of oven-dry material in gram (passing through sieve No.4(4.75mm)	OMC(%)	Percentage of Stabilizer Required		Mass of Stabilizers Required in gm		Volume of Water to be admixed with the material (ml)
					Molasses	Cement	Molasses	Cement	
4% Molasses	50.00	8.00	46.00	25.40	4.00%	-	1.84	-	200.00
8% Molasses	50.00	8.00	46.00	24.70	8.00%	-	3.68	-	200.00
12% Molasses	50.00	8.00	46.00	24.6	12.00%	-	5.52	-	200.00
4% Cement	50.00	8.00	46.00	25.7		4.00%	-	1.84	200.00
8% Cement	50.00	8.00	46.00	26.5		8.00%	-	3.68	200.00
12% Cement	50.00	8.00	46.00	26.7		12.00%	-	5.52	200.00
4% Cement & 4% Molasses	50.00	8.00	46.00	25.1	4.00%	4.00%	1.84	1.84	200.00
8% Cement & 4% Molasses	50.00	8.00	46.00	22.7	4.00%	8.00%	1.84	3.68	200.00
12% Cement & 4% Molasses	50.00	8.00	46.00	22.8	4.00%	12.00%	1.84	5.52	200.00

Figure 4.1: Specimens prepared for pH tests

Figure 4.2: Preparing water bath at 25 $\pm$1°C (Left) and soaking specimen in the water bath for 1 hour (Right).

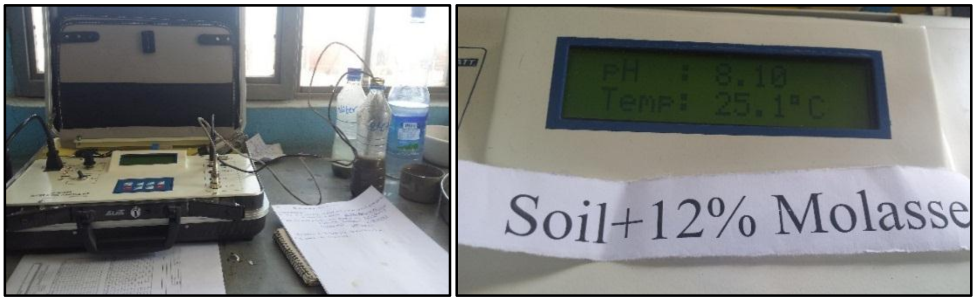


Figure 4.3: Measuring pH values

4.2 Atterberg Limits

These tests were performed to determine the plastic and liquid limits and plasticity index of a fine grained soil. The liquid limit (LL) is arbitrarily defined as the water content, in percent, at which pat of soil in Cassagrande's cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (1/2 in.) when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. The plastic limit (PL) is the water content, in percent, at which a soil can no longer be deformed by rolling into 3.2 mm (1/8 in.) diameter threads without crumbling.

The Atteberg limits are based on the moisture content of the soil. The plastic limit is the moisture content that defines where the soil changes from a semi-solid to a plastic (flexible state) state. The liquid limit is the moisture content that defines where the soil changes from a plastic to a viscous fluid state. A wide variety of soil engineering properties have been related to the liquid and plastic limits, and these Atterberg limits are also used to classify a fine-grained soil according to the Unified Soil Classification system or AASHTO system.

The Atterberg Limits of treated soils were determined according to AASHTO T-89 (Method A) and AASHTO T- 90. The following apparatus was used for conducting the tests: liquid limit device, grooving tools with gage, moisture cans, glass plate, spatula, wash bottle filled with water, dry oven set at 105^0 C.

The calculated amount of stabilizers was added to the dry weight of the raw or untreated material and then water added and mixed thoroughly to a uniform consistency just above the plastic limit (at 3% above the optimum moisture content of the soil - stabilizer mixture). See Table 4.2 for Atterberg Limits mix design. The specimen was immediately covered with plastic sheets in order to prevent loss of moisture and allowed to cure for the required number of days as shown in Figure 4.4.

After the curing period, the sample dried in oven (see figure 3.5) and thoroughly broken up the mixture, without reducing the natural size of individual particles, until it passed a No.40 (0.425mm) sieve and the Atterberg Limits of treated soils were determined according to AASHTO T- 89 (Method A) and AASHTO T- 90.

Figure 4.4: Specimens prepared for Atterberg's Limits (left) and curing (right)

Table 4.2: Mix-Design for Atterberg's Limit Tests

Test	Mass of air-dry material in gram (passing through sieve No.40 (0.425mm)	NMC (%)	Mass of oven-dry material in gram (passing through sieve No.40(0.425mm)	OMC(%)	Percentage of Stabilizer Required		Mass of Stabilizers Required in gm		Volume of Water to be admixed with the material (ml)
					Molasses	Cement	Molasses	Cement	
4% Molasses	500.00	8.00	460.00	25.40	4.00%	-	18.40	-	81.51
8% Molasses	500.00	8.00	460.00	24.70	8.00%	-	36.80	-	82.71
12% Molasses	500.00	8.00	460.00	24.6	12.00%	-	55.20	-	86.48
4% Cement	500.00	8.00	460.00	25.7		4.00%	-	18.40	83.00
8% Cement	500.00	8.00	460.00	26.5		8.00%	-	36.80	91.50
12% Cement	500.00	8.00	460.00	26.7		12.00%	-	55.20	97.61
4% Cement & 4% Molasses	500.00	8.00	460.00	25.1	4.00%	4.00%	18.40	18.40	84.85
8%Cement & 4% Molasses	500.00	8.00	460.00	22.7	4.00%	8.00%	18.40	36.80	76.95
12% Cement & 4% Molasses	500.00	8.00	460.00	22.8	4.00%	12.00%	18.40	55.20	81.71

Figure 4.5: Oven dried specimen for Atterberg Limits after curing period

Test Procedures for Liquid Limits:

i. 100 gm of the treated soil sample was taken and placed into the porcelain dish. Thoroughly mixed the soil with a small amount of distilled water until it appeared as a smooth uniform paste. Cover the dish with cellophane to prevent moisture from escaping.
ii. The crank of the apparatus was turned at a rate of approximately two drops per second and the number of drops, N, was counted it took to make the two halves of the soil pat came into contact at the bottom of the groove along a distance of 13 mm (1/2 in.).
iii. A sample was taken, using the spatula, from edge to edge of the treated soil pat and placed into a moisture can. Immediately weighed the moisture can which contained the treated soil, record its mass and place the can into the oven.
iv. The entire soil specimen in the porcelain dish was remixed. A small amount of distilled water was added to increase the water content so that the number of drops required to close the groove decreases.
v. The above steps were repeated for two additional trials producing successively lower numbers of drops to close the groove. One of the trials should be for a closure requiring 25 to 35 drops, one for closure between 20 and 30 drops, and one trial for a closure requiring 15 to 25 drops.
vi. The water content of each the liquid limit moisture cans after they have been in the oven for at least 16 hours were calculated.
vii. The number of drops, N, (on the log scale) versus the water content (w) were plotted and the best fit straight line through the plotted points were drawn and the liquid limit (LL) was determined as the water content at 25 drops.

Test Procedures for Plastic Limit:

i. 50 gm treated soil sample was taken and distilled water added to the soil until it was at a consistency where it could be rolled without sticking to the hands.

ii. The sample formed into an ellipsoidal mass. Rolled the mass between the palm or the fingers and the glass plate. Sufficient pressure used to roll the mass into a thread of uniform diameter. The tread should be deformed so that its diameter reached 3.2 mm (1/8 in.), taking no more than two minutes.

iii. When the diameter of the tread reached the correct diameter, broke the thread in to several pieces. Kneaded and reformed the pieces into ellipsoidal masses and re-rolled them. This alternate rolling, gathering together, kneading and re-rolling continued until the thread crumbled under the pressure required for rolling and could no longer be rolled into a 3.2 mm diameter thread.

iv. Gathered the portions of the crumbled thread (at least 6 grams) together and placed the sample into a moisture can, then covered it. Immediately weighed the moisture can which contained the soil, and placed the can into the oven. The moisture can left in the oven for at least 16 hours.

v. The above procedures repeated for two more times. The water content from each trial by using the same method used in the first laboratory was determined after they have been in the oven for at least 16 hours.

vi. The average of the water contents to were determined and computed as the plastic limit, PL.

vii. The plasticity index (PI) was calculated as follows, PI=LL-PL.

4.3 Linear Shrinkages

Linear shrinkage is a measure of how a sample will reduce in length upon drying expressed as a percentage of the original length. A linear shrinkage test was carried out to determine the linear shrinkage characteristics of the stabilized soil sample when completely dry and also the linear shrinkage characteristics of the soil when various percentages of molasses, cement and cement and molasses combination were added.

Linear Shrinkage Testing Procedures:

i. Shrinkage moulds were cleaned thoroughly and a thin layer of oil applied to its inner wall in order to prevent the soil from adhering to the mould.

ii. Part of the sample used during liquid limit determination was applied on the shrinkage mould for linear shrinkage determination.

iii. Excess soil was struck off to give a smooth surface and soil adhering to the rim of the mould was removed by wiping with damp cloth, see Figure 4.6.

iv. The soil pat was allowed to dry in air until the color of the pat changed.

v. Mould was placed in the oven at 1050C to complete dryness and remove carefully from the mould, see Figure 4.6.

vi. Mould and soil were cooled and mean length of soil bar measured.

vii. The linear shrinkage (LS) was calculated as a percentage of the original specimen from the equation,

$$\boldsymbol{L_s = \frac{L_o - L_D}{L_D} * 100} \ldots\ldots\ldots\ldots\ldots\ldots \boldsymbol{Equation\ (4.1)}.$$

Where; L_o=Original length of the mould and L_D=length of dry specimen

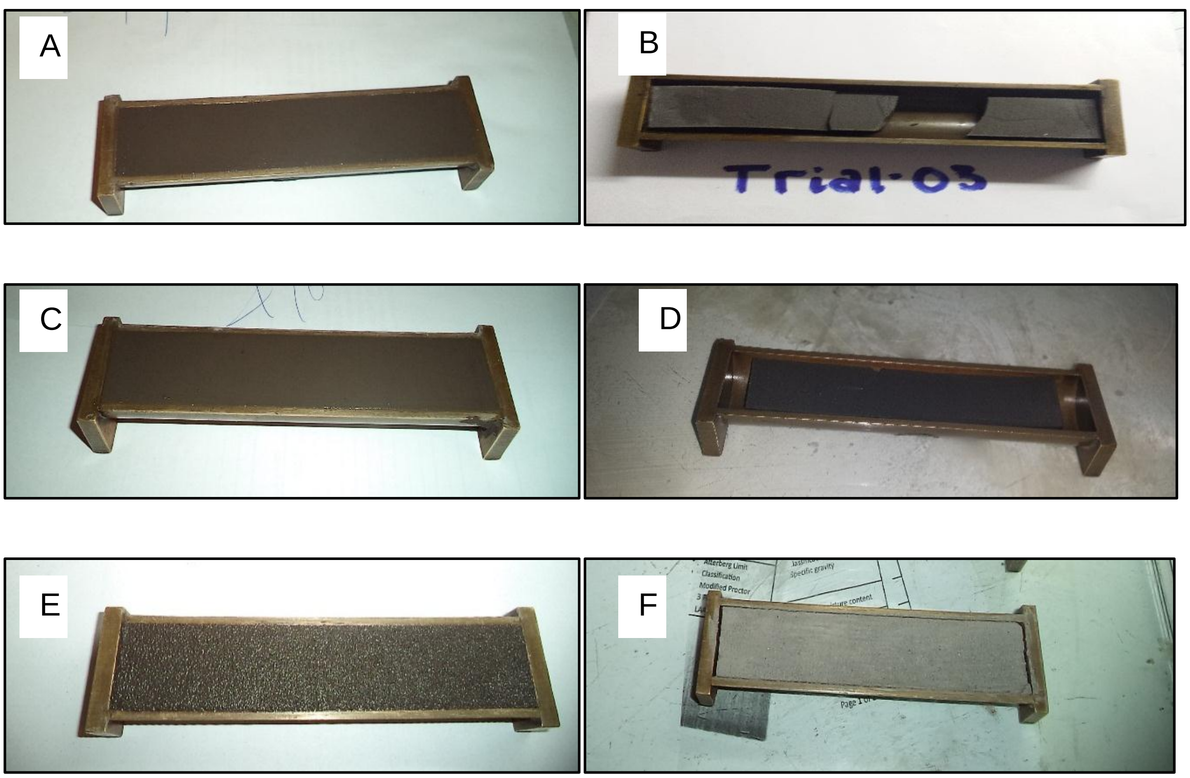


Figure 4.6: Specimens prepared for Linear Shrinkage tests before oven drying (left side) and after oven drying (right side). (A & B) native soil, (C & D) molasses treated soils, and (E & F) cement and molasses treated soils.

4.4 Moisture-Density Relations

This test was done to determine the maximum dry density (MDD) and optimum moisture content (OMC) of the soil treated with molasses, cement and cement and molasses combinations. It was done on the soil sample and then various percentages of stabilizers were added to the soil and MDD and OMC determined according to AASHTO-T 134 Standard Test Methods for Moisture-Density Relations of Soil-Cement Mixtures, Method A. the following apparatus was used for conducting the tests: Mold (6'' inch), Manual rammer (4.5kg), Extruder, Balance, Drying oven, Mixing pan, Trowel, No.4 (4.75 mm) sieve, Moisture cans, Graduated cylinder, Straight Edge and Spatula.

Proctor Test (Moisture Density Relations) Procedures

i. A sufficient quantity of air dried soil were obtained in large mixing pan and pulverized the soil and run it through the #4 sieve and prepare 5 representative samples each about 6000gm for a single Proctor Test using 6 inch mould.

ii. Amount of stabilizers which are 4%, 8% and 12% by dry weight of the soil were measured and placed in air tight container to avoid the effect of carbonation and atmospheric moisture.

iii. The amount of initial water added to the soil-stabilizer mixture were computed by the following method:

$$\boldsymbol{Water\ to\ add\ in\ ml} = \frac{(\boldsymbol{Oven\ dry\ Soil + molasses\ mixtures\ in\ grams}) * \mathbf{8}}{\mathbf{100}} \ldots\ldots\ldots..\boldsymbol{Equation}\ (\mathbf{4.2})$$

iv. Two methods of mixing were followed, i.e. wets mixing and dry mixing based on the nature of the stabilizers as discussed in section 3.2.4.2.

v. After the end of absorption period, the samples were weighed into five 6000 gm portions and put in different trays/basins.

vi. Compaction for each portion was done with 4.5kg hammer in five layers each layer 56 blows.

vii. For each proctor test five runs were conducted by increasing water content 2% of the preceding tests using 8% water content an initial run. This series of determinations was continued until there was either a decrease or no change in the wet unit mass (g/cm3) of the compacted soil-stabilizer mixture.

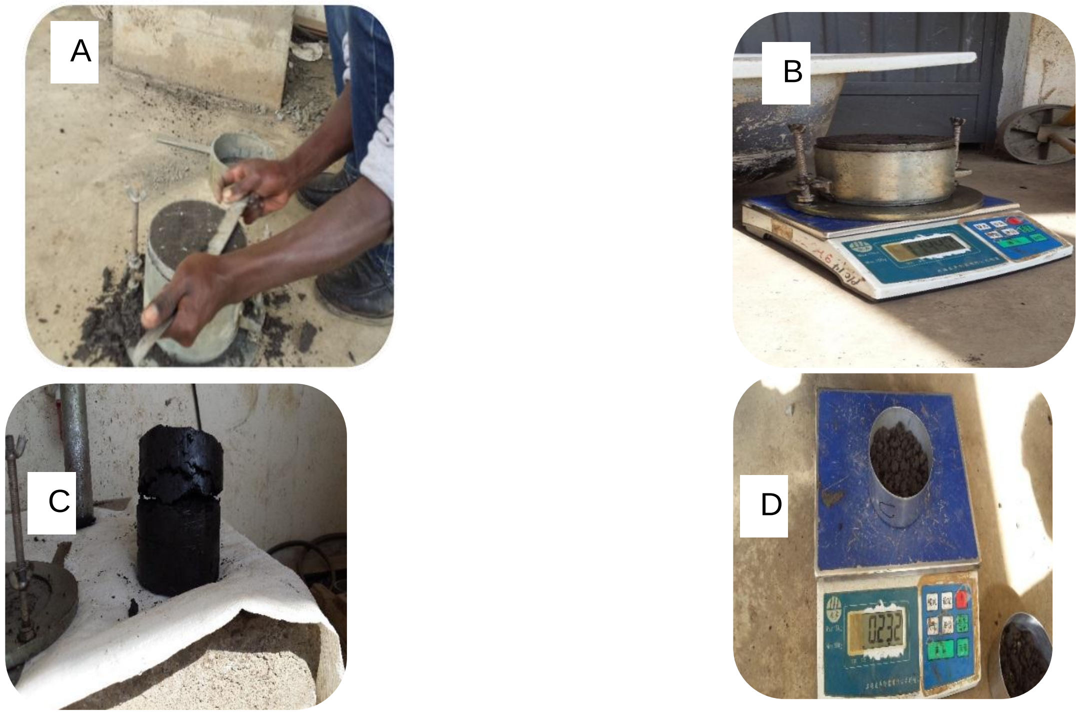


Figure 4.7: Porctor Test Procedures. (A) Trimming Proctor Sample. (B) Weighing weight of mold and wet sample. (C)Compacted Proctor Sample-Soil Molasses Mixture. (D) Weight measuring for moisture content determination.

4.5 The California Bearing Ratio Test

CBR test was done to determine the strength of a given material and how it will behave when subjected to loading. According to ERA (2013) Design Manual for stabilized sub-grade, an increase of the CBR to a minimum of 10 (after 7 days cure) and 15 (after 28 days cure), with corresponding improvement of the subgrade strength class are required.

Three point CBR values were determined according to AASHTO T-193. The following apparatus were used for conducting the tests: Mold (6''), Spacer Disk, Manual rammer (2.5kg), Apparatus for Measuring Expansion, Surcharge weights having a mass of 4.5kg, Penetration Piston, Loading Device, Soaking Tank, Drying Oven, Moisture Content Containers, Extruder, Balance, Drying oven, Mixing pan, Trowel, #4 (4.75 mm) sieve, Graduated cylinder, Straight Edge, and Filter paper.

The required quantity of soil, molasses, 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. Accordingly mixture design for CBR values were determined and presented in Table 4.5.

CBR and CBR Swell Test Procedures

i. Samples for the CBR tests were mixed as per the mix design in Table 4.3 and mixed according to section3.2.4.2, left for 5 -10minutes (to account for absorption period) before being compacted into CBR molds.
ii. The sealed soil-stabilizers mixtures were opened after an absorption period and just before the compaction amount of water that weighs 3% of the oven dry soil and stabilizer mix was added to the sample and then remixed well.
iii. Each equal five layers were compacted into the molds by giving 10, 30 and 65 blows for three point CBR tests for each prescribed stabilizers content. Each layer should occupy about or a little more than one-fifth of the height of the mould. Ensure that the blows are evenly distributed over the surface. The final level of the soil surface should be about 5-10 mm above the top of the mould body.
iv. The extension collar was removed, and using a straightedge, the compacted soil was trimmed even with the top of the mold. Surface irregularities were patched with small-sized material. The spacer disk was removed and a coarse filter paper was placed on the perforated base plate.
v. The compacted samples treated with only cement and cement molasses combinations were then moist cure for 7 days and soaked for 7 days in accordance with BS 1924 as shown in Figure 4.8 whereas samples treated with only molasses were cure at room temperature for 14 days and soaked for 4 days. See figure 4.8. A surcharge weight of 4.5 kg was applied during curing and soaking period for each specimen. CBR swell readings were taken before soaking and after soaking period, see Figure 4.9.
vi. When the soaking phases were completed, CBR moulds were drained for 15 minutes before CBR penetrations.
vii. Each mould was placed on CBR machine with plate in position as shown in Figure 3.15 and the samples were penetrated, their strengths measured.
viii. The CBR values were calculated at penetration of 2.5 and 5.0 mm and the higher value was taken.

Figure 4.8: Curing CBR test specimen at room temperature (left) and wet drying (right).

Table 4.3: Mix-Design of Soil-Molasses Mixtures for CBR tests

Mass of air-dry material in gram (passing through sieve No.4 (4.75mm)	NMC (%)	Mass of oven-dry material in gram (passing through sieve No.4(4.75mm)	OMC(%)	Percentage of Stabilizer Required		Mass of Stabilizers Required in gm		Volume of Water to be admixed with the material (ml)
				Molasses	Cement	Molasses	Cement	
6,000.00	8.00	5,520.00	25.40	4.00%	-	220.80	-	978.16
6,000.00	8.00	5,520.00	24.70	8.00%	-	441.60	-	992.52
6,000.00	8.00	5,520.00	24.6	12.00%	-	662.40	-	1,037.78
6,000.00	8.00	5,520.00	25.7		4.00%	-	220.80	995.96
6,000.00	8.00	5,520.00	26.5		8.00%	-	441.60	1,098.04
6,000.00	8.00	5,520.00	26.7		12.00%	-	662.40	1,171.32
6,000.00	8.00	5,520.00	25.1	4.00%	4.00%	220.80	220.80	1,018.15
6,000.00	8.00	5,520.00	22.7	4.00%	8.00%	220.80	441.60	923.40
6,000.00	8.00	5,520.00	22.8	4.00%	12.00%	220.80	662.40	980.57

Figure 4.9: Measuring swell value at the beginning & end of soaking (left) and Soaking CBR Test Specimen (Right)

Figure 4.10: Penetrating CBR Specimen

4.6 Unconfined Compressive Strength (UCS) Tests

The unconfined compressive strength (q_u) is defined as the compressive stress at which unconfined cylindrical specimen of soil will fail in a simple compression test. This test was conducted to determine the UCS of soil-molasses, soil-cement and soil-cement and molasses specimens prepared by mixing, compacting and curing to an increasing load until failure. The primary purpose of this test is to determine the UCS of cure soil-molasses, soil-cement and soil-cement and molasses specimens to determine the suitability of the mixture for uses such as sub-grades. For stabilized sub-grade, a minimum 30 psi (207 Kpa) increase from untreated is required.

UCS values were determined according to ASTM D 5102-96, Procedure A. The following apparatus was used for conducting the tests: compaction device, Shelby tube, Load & deformation dial gauges, sample trimming equipment, balance, moisture can, desiccator for curing and drying oven.

The required quantity of soil, molasses, 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. Accordingly mixture design for UCS values were determined and presented in Table 4.4.

Figure 4.11: UCS Test Procedures. (A) Specimens curing (B) specimen ready for compression (C) specimen compression, and (D) crushed specimen.

Table 4.4: Mix-Design for Unconfined Compressive Strength Tests

NMC (%)	Mass of oven-dry material in gram (passing through sieve No.4(4.75mm)	OMC(%)	Percentage of Stabilizer Required		Mass of Stabilizers Required in gm		Volume of Water to be admixed with the material (ml)
			Molasses	Cement	Molasses	Cement	
8.00	460.00	25.40	4.00%	-	18.40	-	81.51
8.00	460.00	24.70	8.00%	-	36.80	-	82.71
8.00	460.00	24.6	12.00%	-	55.20	-	86.48
8.00	460.00	25.7		4.00%	-	18.40	83.00
8.00	460.00	26.5		8.00%	-	36.80	91.50
8.00	460.00	26.7		12.00%	-	55.20	97.61
8.00	460.00	25.1	4.00%	4.00%	18.40	18.40	84.85
8.00	460.00	22.7	4.00%	8.00%	18.40	36.80	76.95
8.00	460.00	22.8	4.00%	12.00%	18.40	55.20	81.71

Test Procedures

i. Calculated amount of soil was thoroughly mixed with prescribed amount of stabilizers at optimum moisture content and sealed and stored for 2 hours before compacting into the mould.

ii. Soil-stabilizers mixtures for one specimen were divided into three equal mass and one third of the height of the mould was determined.

iii. Specimens were moulded to the desired unit by kneading and tamping each three layer until the accumulative mass of the soil-stabilizer mixtures placed in the mould is compacted to a known volume (one third volume for each layer).

iv. The top of each layer was scarified prior to the addition of material for the next layer.

v. After the specimen was formed, it was extruded from the Shelby tube sampler and cut height-to-diameter ratio of 2.

vi. The mass of the specimen, the length of specimen, and diameter of the specimen at mid height were determined and recorded.

vii. Having determined the mass and dimension of the specimens, then it was placed in airtight, moisture proof container and allowed to cure at room temperature for specified curing periods. The test specimens were wrapped and sealed in plastic to reduce carbonation, see Figure 4.11 (A).

viii. At the end of the curing period, the specimen was carefully placed in the compression device and centered it on the bottom of the plate. The device adjusted so that the upper plate just made contact with the specimen and set the load deformation dials to zero, see Figure 4.11 (B and C).

ix. The load was applied so that the device produces an axial strain of 0.5% to 2.0% per minute, and then the load deformation dial readings were recorded on the data sheet at every 20 to 50 divisions on deformation the dial.

x. Applying the load was kept until:

xi. The load (load dial) decreased on the specimen significantly, or

xii. The load held constant for at least four deformation dial readings

xiii. A sketch to depict the sample failure was drawn.

xiv. The specimen was removed from the compression device and a sample for water content determination was taken, see Figure 4.11 (D).

xv. The UCS value obtained for cylindrical specimen was converted to the equivalent cube strength of a 150mm*150mm cube.

CHAPTER 5

5. Results and Discussion

5.1 Introduction

This chapter presents test results, discussion and analysis of all experimental work that were performed on treated/ stabilized soils with molasses only, cement only and cement and molasses combination mixtures. Generally, first the effect of each stabilizers on pH value, Atterberg limits, linear shrinkage, moisture-density relation, CBR values and UCS values were established by varying percent of stabilizers from 4% to 12% by 4% increment and compared with native soil/untreated soil engineering property. Then effect of each stabilizer on the property of treated soil was compared and contrasted.

5.2 pH Values

A series of specimens were prepared in soil containing a range of percentages of molasses, cement and cement and molasses content and measurements of pH were made in slurries as described in section 4.1. Figure 5.1 shows that the pH value of soil-cement and soil-cement and molasses mixtures increase sharply at small cement content and reach values of 13.19 and 12.75 at 4% cement and 4% cement + 4% molasses content respectively. After 8% cement and 8% cement + 4% molasses content, pH value shows insignificant change and almost becomes constant. More over Figure 5.1 also shows that pH values of soil-molasses mixture decreases with the addition of molasses up to 4% then increases up 8% molasses, after which pH values gets decreasing with extra molasses content.

The quantity of cement necessary to maintain a pH of 12.4 in a cement-soil-water mix after 1 hour is considered to be the Initial Consumption of Cement (ICC) of the material. For adequate stabilization using pozzolanic stabilizers, sufficient stabilizer should be added to ensure an excess after the reactions are complete, i.e., initial consumption of cement (ICC) of the soil should be satisfied and an excess provided. Therefore, as it may be shown in Figure 5.1, pH values results except soil-molasses mixtures all soils-cement and soil-cement and molasses mixtures satisfy ICC demand.

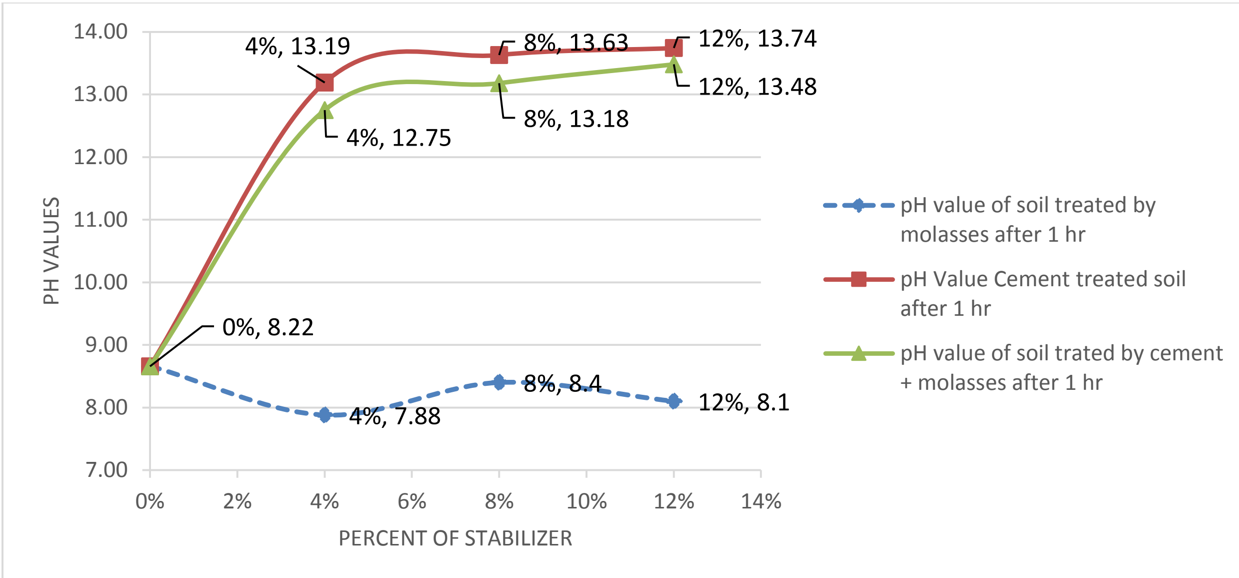


Figure 5.1: pH values of soil treated by molasses, cement and cement and molasses

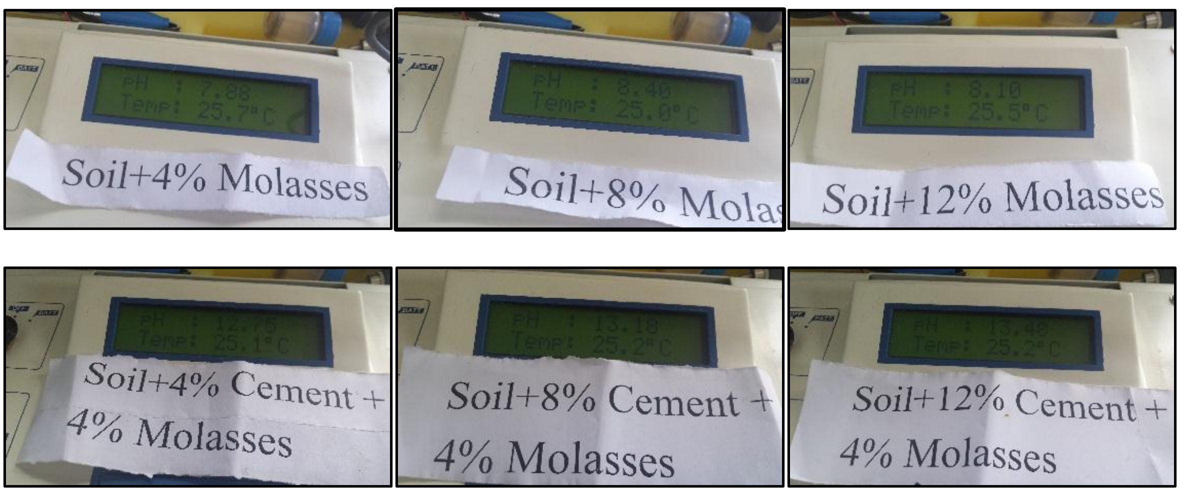


Figure 5.2: pH values of soils treated with molasses (top) and cement and molasses (bottom)

5.3 Consistency Limits

Consistency limits were conducted for soils treated with molasses alone, cement alone and cement and molasses combination for different curing periods as described in chapter 4.

5.3.1 Effect of Molasses on Atterberg Limits

The effect of molasses addition in varying proportion with soil was studied and the variation in consistency limits for various mixes is presented in Table 5.1 and Figure 5.3. Soil was treated with various contents of molasses from 4% to 12% molasses content by 4% increment and cure for 14 days.

The liquid limit (LL) of untreated soil was determined as 89%, whereas it varied from 67% to 72% after molasses was added. The LL of the soil decreased with increase in molasses content up to 8% after that it increased with increasing in molasses content.

Table 5. 1 Atterberg limit values for soils treated with molasses

Limits	4% Molasses			Average
	Test #1	Test #2	Test #3	
LL	69	63	68	66.67
PL	38	33	39	36.67
PI	31	30	29	30.00
Limits	8% Molasses			Average
	Test #1	Test #2	Test #3	
LL	60	69	64	64.33
PL	32	40	37	36.33
PI	28	29	27	28.00
Limits	12% Molasses			Average
	Test #1	Test #2	Test #3	
LL	73	72	72	72.33
PL	39	38	40	39.00
PI	34	34	32	33.33

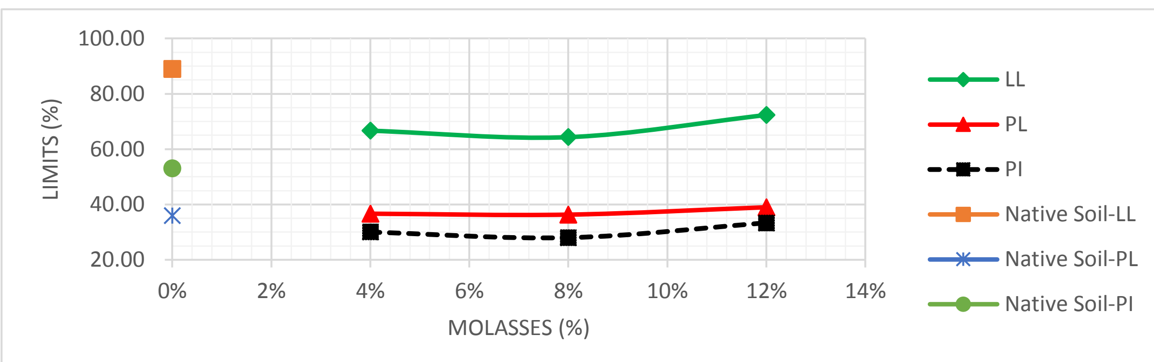


Figure 5.3: Effect of addition of molasses on Atterberg's Limit for Soils

The plastic limit (PL) of untreated soil was determined as 36%, while it didn't change distinctly (ranged between 36 % and 39%) with an increase in molasses content. See Figure 5.3. Plasticity index (PI) values of soils treated with molasses decreased with increasing in molasses content up 8% content after then it increased with increasing in molasses content. The PI varied from 28% to about 33% for soils treated with molasses. These results are in harmony with Ndegwa and Shitote (2012).

The initial decrease in LL even though it is minimal is attributed to cation exchange and adhesive property of molasses which are caused by the addition of molasses to the expansive clay soil. The cation exchange process between the cations of the soil and those of molasses led to reduction in double layer thickness which caused flocculation of clay particles and eventually aggregation (Haddinott and Lamb, 1990 cited in Ndegwa and Shitote (2012)). They also explained that the multivalent cations that replaced the monovalent ones at the clay particle surfaces, reduced also the amount of adsorbed water in clay and consequently caused reduction of water in clay and consequently caused reduction of water content of liquid limit of the clay soil and increased the water content of plastic limit. The cation exchange processes in the soil-molasses mixture is minimal because of the amount of inorganic compounds which causes a cation exchange process in molasses as shown in Table 3.10 are below 2.0% and on the top of that the pH value of clay-molasses mixture does not show significant change up to 12% molasses content as shown in Figure 5.1.

In addition to this electrochemical attraction between aggregated soil particles was enhanced by molasses adhesivity which bound the particles together. Adhesivity properties of molasses were derived from Hydrogen bounds attributed to Hydroxyl group found in sucrose of molasses as given in Table 3.10. Aggregation /agglomeration changed textural condition of clay soil and reduced the specific surface of soil particles which consequently decrease the liquid limit and plasticity index, and increase plastic limit.

On the other hand, the increase in LL for molasses content of above 8% is due to the increase molasses content. Molasses increase in clay soil beyond a certain limit leading to decrease in size of individual aggregates and finally the soil reverted to its fine condition which leads to increases the water holding capacity of the soil and thus increases the LL and PI of the soil (Ndegwa and Shitote, 2012). Consistency limit results are well matched with pH values of soil molasses mixtures given in Figure 5.1.

5.3.2 Effect of Cement on Atterberg Limits

The Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) of soil treated with cement and cure for seven days were determined (Table 5.2) and plotted (Figure 5.4) against cement content.

The LL values of the samples significantly decreased with increasing cement percentages from 0% to 8%, then increased with the addition of cement up 12% contained. The addition of 4% cement, 8% cement and 12% cement reduced the LL of untreated soils by 31%, 39% and 36% respectively. The LL exhibits a minimum value of at an optimum cement content of about 8% in value of 50%. Unexpectedly, PL values of samples decreased with increasing stabilizer percentages up to 8%, then after it increased with the addition of cement up 12% content.

Table 5. 2 Atterberg limit values for soils treated with cement

Average Limits	Cement		
	4%	8%	12%
LL	58.00	50.00	53.00
PL	34.00	30.00	32.00
PI	24.00	20.00	21.00

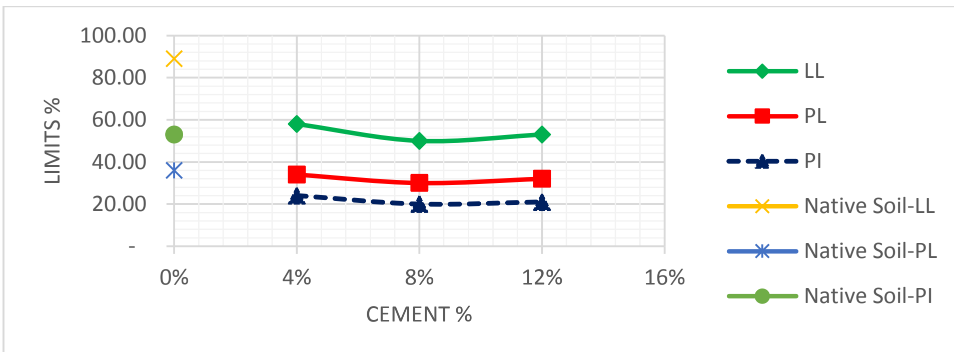


Figure 5.4: Effect of addition of cement on Atterberg's Limit for Soils

PI of the samples decreased significantly with increasing stabilizers percentages up 8% of cement content and then after increases with the addition of cement. The addition of 8 % cement meaningfully reduced PI of the natural soil from 53% to 20%. While, the addition of 12% cement produced very close result to 8% cement in terms of PI reduction with PI values of 21%. Similarly, 4% cement addition reduces PI of natural soil from 53% to 24%. Generally, it is observed that rate of changes in consistency limits between 8% cement and 12% cement is not appreciable when compared to the extra amount of stabilizers exists between the two proportions of stabilizers. For example, extra addition, 4% cement from 8% of cement only produced 1% change in PI values from soils treated with 8% cement.

The initial decrease in water holding capacity of the soil (LL) is attributed to the cation exchange process between the cations of the soil and those of cement that suppresses the double layer thickness due to the increase in the cation concentration. This cation exchange process at small cement content is associated with minimal, if any, pozzolanic activities as indicated by the pH value, depicted in Figure 5.1, which reflects the hydration of the lime content present in cement; the pozzolanic reaction is induced by the $Ca(OH)_2$ produced from the hydration process. On the other hand, the increase in the liquid limit of cement content of 12% is due to the increase in pozzolanic reaction attributed to the presence of high amount of Ca $(OH)_2$ as indicated by the large pH values for cement content greater than 4%. This pozzolanic reaction increases the water holding capacity of the soil and thus increases the liquid limit and plastic limit of the soil.

5.3.3 Effect of Cement and molasses on Atterberg Limits

The Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) of soil treated with cement and molasses combination were determined and plotted against cement and molasses content. (Table 5.3 and Figure 5.5).

The LL values of the samples decreased with increasing stabilizer percentages. The addition of 4% cement + 4% molasses, 8% cement + 4% molasses and 12% cement + 4% molasses diminished the LL of untreated soils by 41%, 45% and 49% respectively. While, PL values of samples didn't show distinct change with increasing stabilizer percentages and unexpectedly increased with addition stabilizer content.

Table 5.3: Atterberg Limit values for soils treated with cement and molasses

Average Limits	Cement + 4% Molasses		
	4%	8%	12%
LL	48.00	44.00	40.00
PL	29.00	27.00	25.00
PI	19.00	17.00	15.00

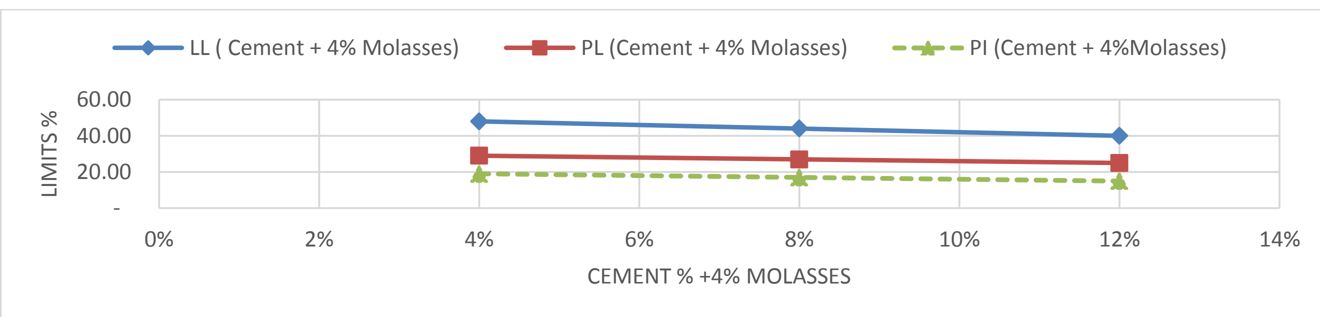


Figure 5.5: Effect of addition of cement and molasses on Atterberg's Limit for Soils

PI of the samples decreased significantly with increasing stabilizers percentages. The addition of 12% cement + 4% molasses reduced PI of the natural soil significantly from 53% to 15%. The addition of 8% cement + 4% molasses and 4% cement + 4% molasses reduced the PI of the natural soil sample from 53% to 17% and from 53% to 19% respectively.

The significant decrease in water holding capacity of the soil (Liquid Limit) may be attributed to reactions between cement, molasses, clay soil and water. As already discussed above cation exchange reaction and adhesive property of molasses are responsible in reducing water holding capacity of the soil in addition to these hydration of cement and pozzolanic reaction is responsible for the same in soil-cement mixture. Stabilization of expansive clay soil with cement and molasses combination is a new concept that is the reaction of cement, molasses, clay soil and water is scanty in the literature. Therefore; one or more the following mechanisms cation exchange, hydration reaction, pozzolanic reaction, adhesive property of molasses may be responsible for the significant reduction of Atterberg Limits.

5.3.4 Comparison of Effect of Stabilizers on Atterberg Limits

The summary of the consistency limit results of treated soils as compared with the raw/untreated soils is presented in Figure 5.6. As it may be seen from the Figure, soil treated with cement and cement and molasses showed highest reduction in PI values while soils treated with molasses showed the least reduction of PI values. Soils treated with cement showed higher PI values than cement and molasses treated soils, but lower PI values than molasses treated soils. Soil treated with the cement and molasses combination were more folliculated and agglomerated than soil treated with cement only at the end of the curing period. See Figure 5.7.

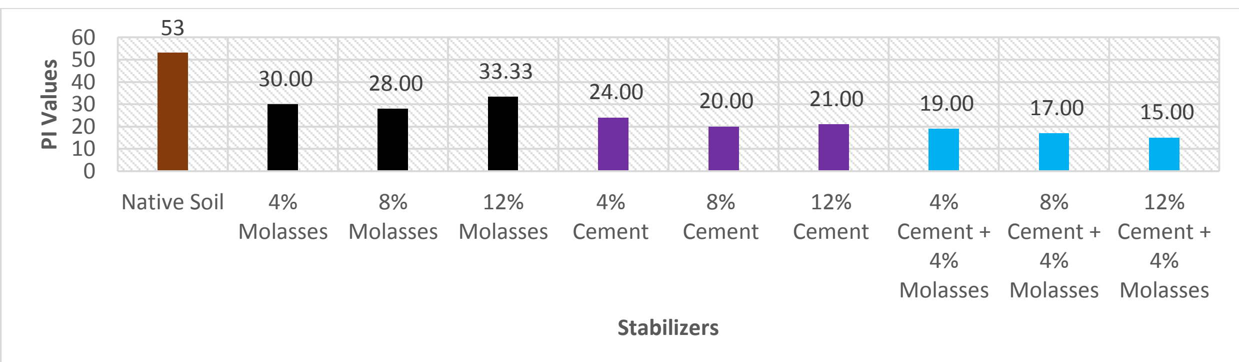


Figure 5.6: Comparison of Effect of Stabilizes on Atterberg Limits

Figure 5.7: Soil treated with cement and molasses (front) and soils treated with cement (back) after curing period

Moreover, the maximum change in PI values was observed to be 38% of soil treated by 12 % cement and 4% molasses from a value of 53% for untreated soils to 15% for treated soils. While the least change in PI values was observed to be 20% for treated soil by 12 % molasses from a value of 53% for untreated soils to 33 % for treated soils. On the top of that PI value for soils treated with cement and molasses decrease with increase in stabilizer content, whereas 8% is the molasses content which gives highest reduction in PI values.

5.3.5 Effect of Stabilizers on Linear Shrinkage

The shrinkage curves for the sample soils from the linear shrinkage tests with the three chemical additives, molasses, cement and cement and molasses are presented in Table 5.4 and Figure 5.8. The average linear shrinkage for native soils without additives was around 21%. It can be seen that the shrinkage insignificantly decreased with the addition, molasses; however cement and cement and molasses were more effective at arresting shrinkage than the molasses for the same percentage by weight added. On the top of that it was observed that addition of cement to molasses significantly reduced shrinkage property of the expansive soil as shown in Figure 5.9. Furthermore, as shown in Figure 5.9 treated soil with cement reduced linear shrinkage but it shows shrinkage cracks. But these shrinkage cracks were not observed in soils treated with the cement and molasses combination and molasses only.

Table 5.4: Linear Shrinkage Values for Untreated and Treated Soils

LINEAR SHRINKAGE TESTS		
Stabilizer %	**Test #**	**Linear Shrinkage in Percent**
Native Soil	1	20.70%
Native Soil	2	20.70%
Native Soil	3	21.40%
4% Molasses	1	22.14%
8% Molasses	1	18.57%
12% Molasses	1	20.00%
4% Molasses	2	20.00%

LINEAR SHRINKAGE TESTS		
Stabilizer %	**Test #**	**Linear Shrinkage in Percent**
8% Molasses	2	18.57%
12% Molasses	2	21.43%
4% Molasses	3	15.71%
8% Molasses	3	17.86%
12% Molasses	3	15.71%
4% Cement	1	7.10%
8% Cement	1	5.70%
12% Cement	1	4.00%
4% Cement + 12% Molasses	1	10.00%
8% Cement + 12% Molasses	1	5.60%
12% Cement + 12% Molasses	1	3.60%

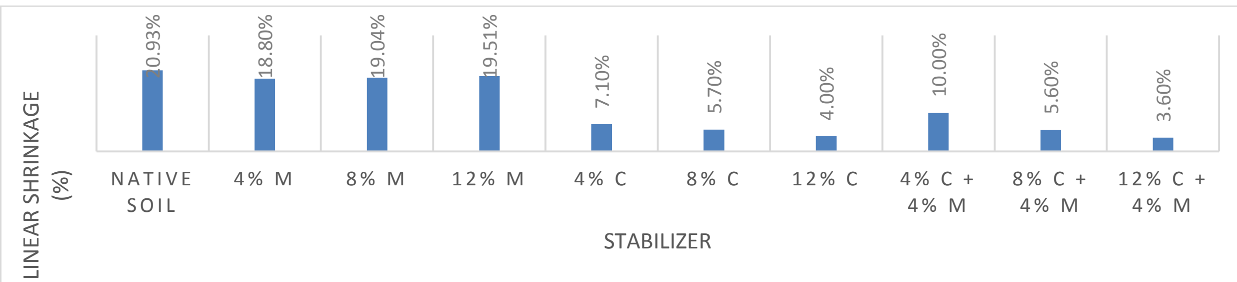


Figure 5.8: Reduction in Shrinkage due to Stabilizer Addition.

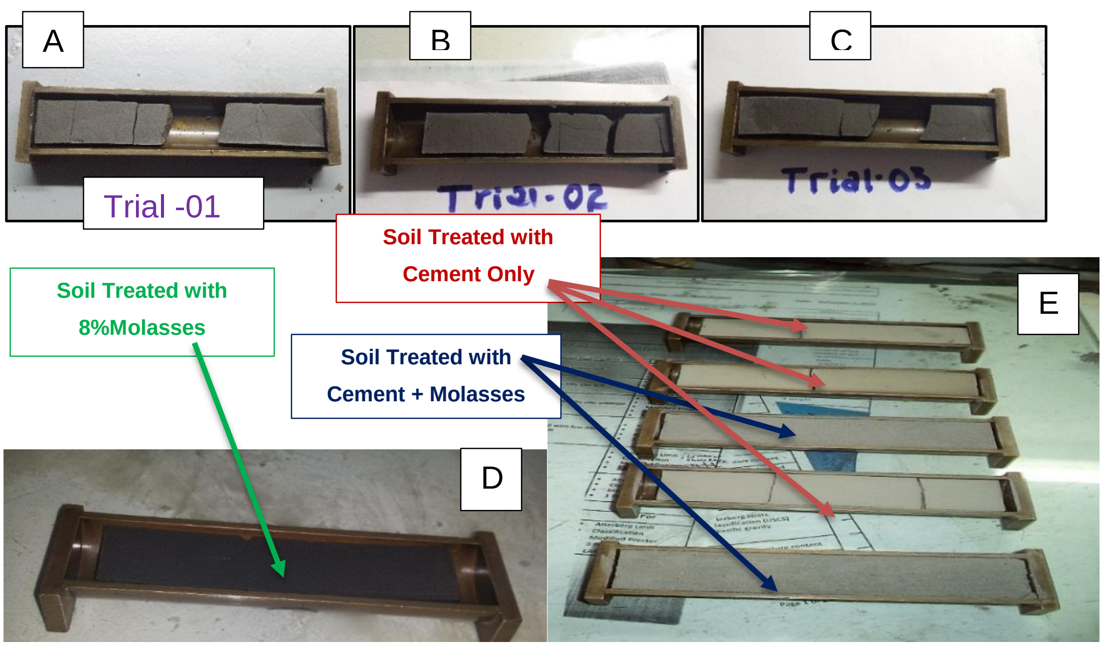


Figure 5.9: Linear shrinkage tests photos :(A,B,C) Native Soil, (D) molasses treated soil & (E) cement and cement and molasses treated soils.

5. 4 Compaction Characteristics

This section presents the compaction characteristic curves determined for sample soils used in the experimental work. Modified Proctor tests were performed on the raw subgrade soils as well as the treated /stabilized soils as described in chapter 3. The compaction curves obtained for raw soil subgrade and treated /stabilized soils by molasses, cement and cement and molasses are presented in Figure 5.10 through Figure 5.12; while Figure 5.13 and Table 5.5 show comparison of compaction curves obtained by the aforementioned stabilizers and the raw subgrade soils.

5.4.1 Compaction Characteristics of Soils stabilized by Molasses

Sample soils treated with molasses gave the typical bell shaped curve for moisture density relationship characteristics with MDD values of 1.52gm/cm^3, 1.55gm/cm^3 and 1.52gm/cm^3 for 4%, 8% and 12% molasses content by dry weight respectively, see Figure 5.10. Theses curves shifted to the left with respect to untreated soil samples which means the addition of molasses increased MDD values while reducing OMC. This result is in harmony with the findings of Shirsavkar and Koranne (2010).

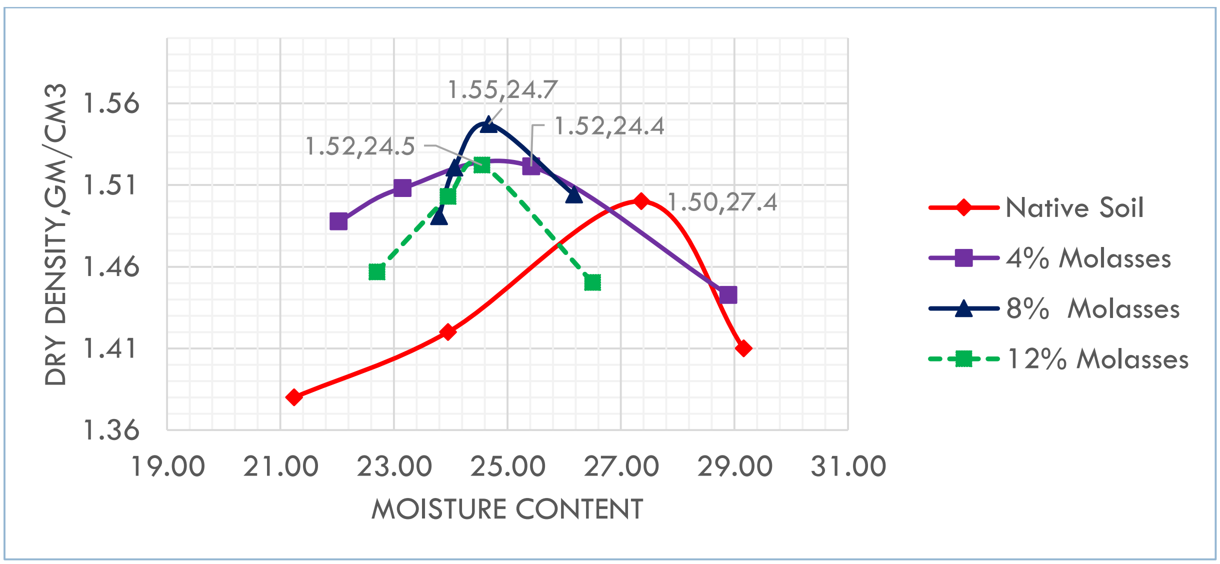


Figure 5.10: Compactions curve for stabilized soil with molasses

The maximum change in MDD and OMC values were observed to be +0.05% and -2.85% of soil treated by 8% and 12% molasses respectively. Furthermore, it is observed that with the increase in molasses content MDD values increases up to 8% molasses content and with further increase in molasses content the MDD decreases.

The increase in density may be associated to as molasses is positively charged and easily attracted to the surface of clay mineral particles as they were negatively charged the attraction of molasses to soil particles was enhanced by its adhesive properties and bound soil particles. Suriadi et al, (2002) observed that addition of cane molasses to clay soil reduced the clay content. Furthermore, the increased size of clay particles turned the fine soil to a coarse soil which may be attributed to decreased OMC.

5.4.2 Compaction Characteristics of Soils stabilized by Cement

Sample soils treated with cement gave the typical bell shaped curve for moisture density relationship characteristics, but theses curves shifted to the left with respect to untreated soil samples. In Figure 5.11 It is observed that with the increase in water content, the dry density increases up to 23-27% moisture content and with further increase in water content the dry density decreases gradually.

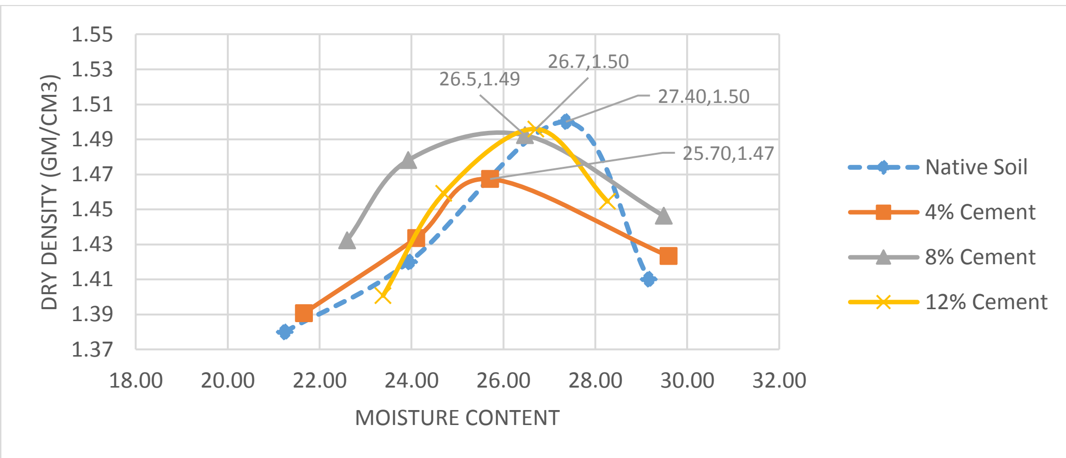


Figure 5.11: Compaction characteristics curve for stabilized soil with cement

The maximum dry density was in the range of 1.50 gm/cm^3 at 26.7% moisture content for soil treated by 12% cement and lowest maximum dry density was in the range of 1.47 gm/cm^3 at 25.7% moisture content for soil treated by 4% cement. It could be seen that the natural clay soil sample had a MDD of 1.50 gm/cm^3 and OMC of 27.4%, the addition of cement reduced the maximum dry density by 0.03%, 0.01% and 0.0% and decreased the moisture content by 1.69%, 0.93% and 0.69% in soil treated by 4%, 8% and 16% cement by dry weight of the sample respectively. Amu. O. O et al, (2005) were also observed similar results for this research on compaction characteristics of expansive clay soil treated with cement.

The decrease in density may be related to the flocculated and agglomerated clay particles occupying larger spaces leading to a corresponding decrease in dry density, and the effect of cement addition to soil sample on the specific gravity of soil mixed with different concentration of cement. (Hayder A. H., et al. (2012).)

5.4.3 Compaction Characteristics of Soils stabilized by Cement and molasses

For soils treated with cement and molasses mixtures ,Figure 5.12, it is observed that with the increase in water content the dry density increases up to 22-25 % moisture content and with further increase in water content the dry density decreases gradually. In addition to this, sample soils treated with cement and molasses gave the typical bell shaped curve for moisture density relationship characteristics, but theses curves shifted to the left with respect to untreated soil samples.

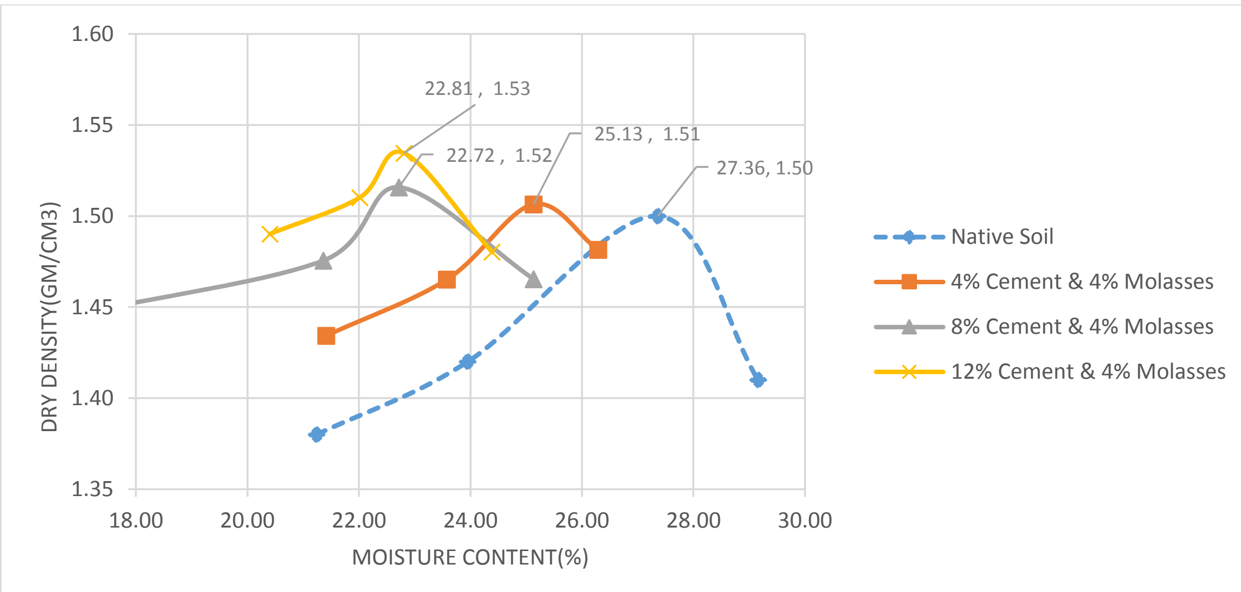


Figure 5.12: Compaction characteristics curve for stabilized soil with cement and molasses

The maximum dry density was in the range of 1.53 gm/cm^3 at 22.8% moisture content for soil treated by 12% cement + 4% molasses and lowest maximum dry density was in the range of 1.51 gm/cm^3 at 25.1% moisture content for soil treated by 4% cement + 4% molasses. It could be seen that the natural clay soil sample had a MDD of 1.50 gm/cm^3 and OMC of 27.4%, the addition of cement and molasses affected on dry density characteristics; however, relatively reduction in moisture contents were observed.

The increase in density may be associated with rearrangement of the flocculated and agglomerated clay particles in smaller space due to the presence of more plastic paste in soil-cement-molasses mixtures than soil-cement mixture. Furthermore, the increased size of clay particles turned the fine soil to a coarse soil which may be attributed to decreased OMC.

5.4.4 Comparison of Effect of Stabilizers on Compaction Characteristics of Soils

The summary of the modified Proctor test results of treated soils as compared with the raw soils is presented in Table 5.5 and Figure 5.13. As it may be seen from Figure 5.13 and Table 5.5, soil treated with molasses and cement and molasses showed improvement in MDD values at lower optimum moisture content but a decrease in MDD value was observed in soils treated by cement.

Table 5.5: Change in Compaction characteristics of soil with additives

Additive Types and Content (% by Weight)	**OMC (%)**	**MDD(gm/cm3)**	**% Change**	
			OMC	**MMD**
Native Soil	27.4	1.50		
4% Molasses	25.4	1.52	(1.98)	0.02
8% Molasses	24.7	1.55	(2.73)	0.05
12% Molasses	24.6	1.52	(2.85)	0.02
4% Cement	25.7	1.47	(1.69)	(0.03)
8% Cement	26.5	1.49	(0.93)	(0.01)
12% Cement	26.7	1.50	(0.69)	-
4% Cement & 4% Molasses	25.1	1.51	(2.27)	0.01
8% Cement & 4% Molasses	22.7	1.52	(4.68)	0.02
12% Cement & 4% Molasses	22.8	1.53	(4.59)	0.03

Moreover, the maximum change in optimum moisture content (OMC) was observed to be -4.68 % for soil treated by 8% cement and 4% molasses while the least change in OMC was noted to be 0.69 % of soil treated by 12% cement. The highest change in maximum dry density (MDD) was measured to be 0.05 % for treated soil by 8 % molasses while soil treated by 12% cement did not show any change in MDD.

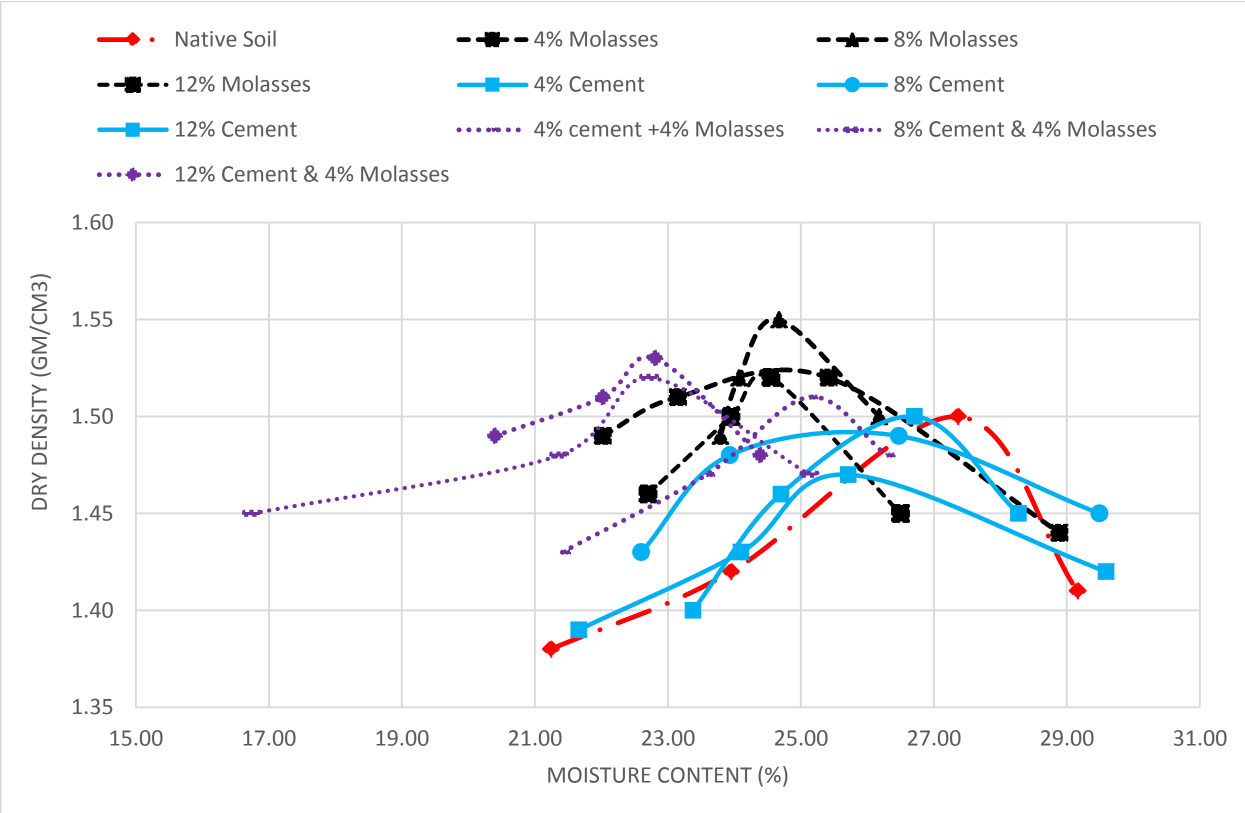


Figure 5.13: Compaction characteristics curve for stabilized soil with different additives

5. 5 California Bearing Ratio (CBR) and CBR Swell Values

This section presents the CBR values determined for sample soils used in the experimental work. CBR tests were conducted on the untreated soil samples as well as treated/stabilized soils by molasses, cement and cement and molasses. The tests were conducted by varying stabilizers contents up to 12% and by applying different soaking periods and curing methods and the results are presented in Figure 5.13 through Figure 5.15 while Figure 5.16 and 5.17 and Table 5.9 and 5.10 show comparison of CBR values obtained by the three stabilizers and the untreated sample soils.

5.5.1 Effect of Molasses on CBR Values

CBR results obtained from all the samples treated by molasses cure for 14 days and soaked afterwards for four days by 4.5 kg surcharge load as described in section 4.6 are summarized in Table 5.6.

Table 5.6: Soaked CBR values for Soils treated with Molasses

Soil Stabilized by Molasses			
Molasses %	Test #1	Test #2	Average CBR value
4	8.0	8.0	8.0
8	11.0	9.5	10.3
12	8.5	7.5	8.0

Soil treated with molasses using the optimum moisture content of the compaction test results showed considerable improvement in strength when compared to untreated soil sample and CBR values for the two tests are given in Table 5.6. A peak CBR value of 10.3 % was recorded for 8% treatment of the soil from a value of 1.3 % for natural soil. Results presented in Figure 5.14 show that the CBR of treated soil increases up to 8% molasses content then decreases with increasing molasses content. The highest CBR was observed at about 8 % molasses content. These results are in general agreement with Ndegwa (2011).

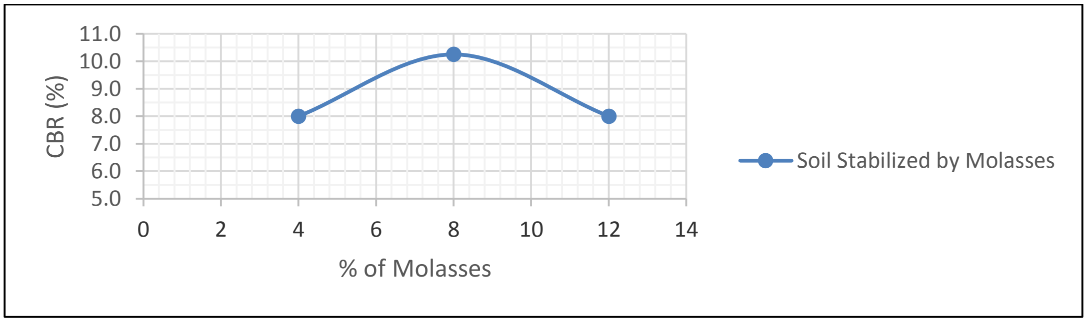


Figure 5.14: Variation of Soaked CBR values of Expansive soils with various proportion of molasses and cure for 14 days

The increase in CBR values of molasses treated soils is attributed to cation exchange reaction and flocculation and agglomeration effect. When molasses were intimately mixed with the sample clay soil, the pH of the soil was reduced from 8.66 to 7.88 (Table 5.1). The pH of the molasses, which was used in the test, was 5.5 (Table 3.10). Honddinott and Lamb (1990) cited in Ndegwa (2011) explained that a decrease in pH of soil was due to calcium and magnesium cations from cane molasses supplementing those already attached to clay and then replacing the weaker monovalent sodium ions at the surface of the clay particles. Ndegwa (2011) stated that molasses played a role in enhancement of flocculation and soil aggregate stability due to electrostatic attraction between aggregated soil particles which enhanced by adhesivity of molasses. That led to the formation of a strong cementing bond between soil particles caused by cane molasses which increased resistance to penetration during the CBR test.

The reduction of CBR values with increasing molasses content beyond a certain limit can also be attributed to coating of individual soil grains with molasses. As molasses coated the soil grains, its thickness around each grain increased with increase of molasses content in the soil and led to increase in the distances between individual soil grains (Ndegwa, 2011). Beyond certain molasses content in the soil the distances between individual grains reaches an extent that electrostatic attraction forces which keep the soil particles together due to relatively short distances between them become ineffective. The bond caused by adhesivity of molasses, then act alone, but it is not strong enough to offer high resistance due to deformation caused by the load applied to the compacted soil during CBR penetration (Ndegwa, 2011).

The recorded peak CBR value of 10.3% for 8% molasses which were cure for 14 days and soaked for four days marginally satisfied stabilized sub-grade requirement of an increase of the CBR to a minimum of 10 (after 7 days cure) and with corresponding improvement of the subgrade strength class. After stabilization, the sub-grade soil class was improved from S1 to S3 according to ERA design manual classification (ERA, 2013).

5.5.2 Effect of Cement on CBR Values

As soil stabilization by cement is one of the conventional stabilizer agent and well documented in the literature, one trial CBR tests were conducted on soils treated with cement cure for 7 days and soaked for the same 7 days by 4.5 kg surcharge and results are given in Table 5.7.

Table 5.7: Soaked CBR values for Soils treated with Molasses and cure for 14 days

Soil Stabilized by Cement	
Stabilizer %	CBR Value
4% Cement	27.3
8% Cement	82.3
12% Cement	123.0

As it may be seen from Table 5.7 CBR results showed significant improvement in strength compared to untreated soil sample. Results presented in Figure 5.15 show that CBR value of treated soils with cement increases as the quantity of cement increases. The formation of secondary cementitious materials that resulted from the reaction between the free lime, $Ca(OH)_2$, produced during the hydration of cement and pozzolanic reaction between the lime and soil could be responsible for the increase (AASHTO, 2008). See Table 3.11 for the Oxide composition of the OPC.

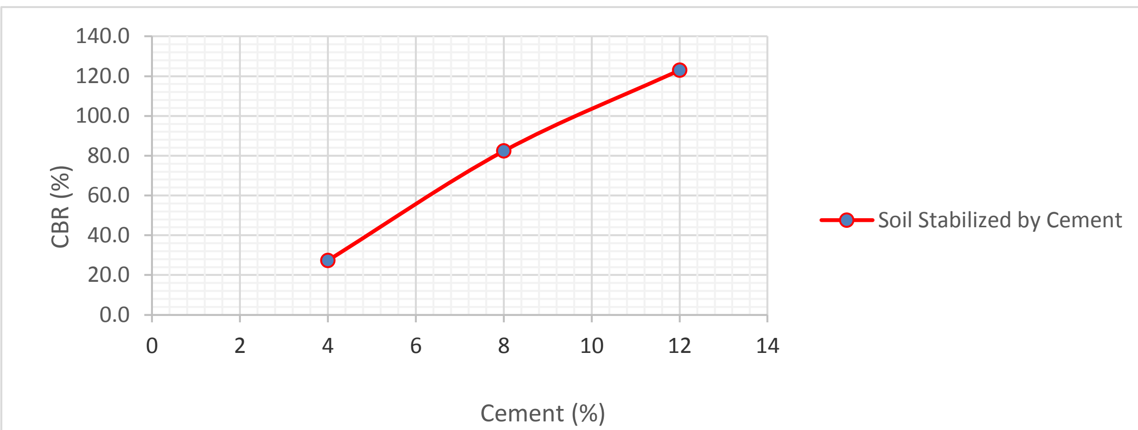


Figure 5.15: Variation of soaked CBR values of expansive soil mixed with various proportion of cement and cured for 14 days

5.5.3 Effect of Cement and molasses on CBR Values

Big difference in CBR values between untreated and treated soils with cement and molasses combination was obtained when compared to the 1.3 % CBR value of untreated soil for all the samples treated by cement and molasses cure for 7 days and soaked for the same days by 4.5 kg surcharge load. Results as given in Table 5.8 and Figure 5.16 show that CBR values increased with higher cement and molasses content.

Table 5.8: Soaked CBR values for Soils treated with Molasses

Soil Stabilized by Cement and molasses			
Stabilizer %	Test #1	Test #2	CBR Average Value
4% Cement + 4% Molasses	65.0	62.0	63.5
8% Cement + 4% Molasses	99.0	95.0	97.0
12% Cement + 4% Molasses	130.0	125.0	127.5

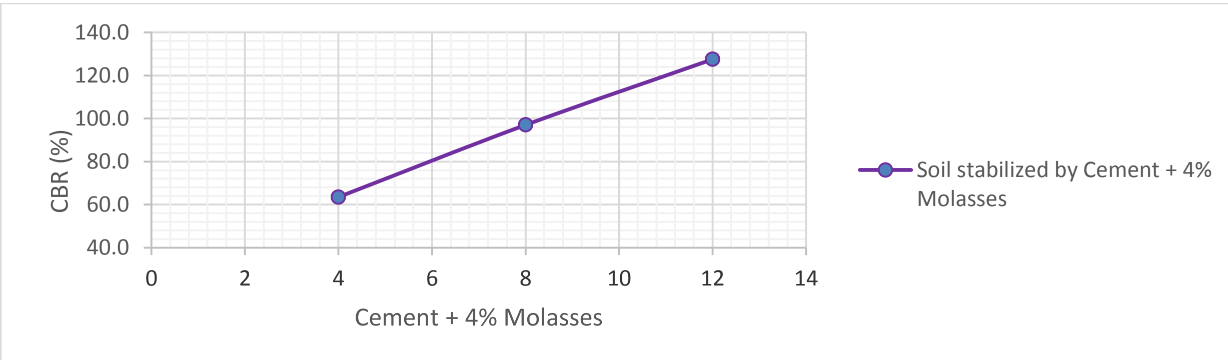


Figure 5.16: Variation of Soaked CBR values of Expansive soil mixed with various proportion of cement and molasses and cure for 14 days

The significant increase in CBR value may attributed to reactions between cement, molasses, clay soil and water. As already discussed above cation exchange reaction and adhesive property of molasses are responsible in the increasing load bearing capacity of the soil in addition to these hydration of cement and pozzolanic reaction are responsible for the same soil-cement mixture. Stabilization of expansive clay soil with cement and molasses combination is a new concept that is the reaction of cement, molasses, clay soil and water is scanty in the literature. Therefore; one or more the following mechanisms cation exchange, hydration reaction, pozzolanic reaction, adhesive property of molasses may be responsible for significant increase in CBR values.

5.5.4 Comparison of Effect of Stabilizers on CBR values

Figure 5.17 presents the summary of CBR values obtained from the CBR tests conducted with untreated soil as well as treated soils with molasses, cement and cement and molasses at various stabilizer contents. It can be seen from these figures that there is marked influence of the presence of cement and cement and molasses with in a test specimen on its load bearing capacity. Improvement in strength for soil treated with molasses when compared to soils treated with cement and cement and molasses is very low. Furthermore, it is observed that piston load at a given penetration was higher in all cases of treated specimens as compared to that of untreated specimen and the amount of increase in load bearing capacity depends on the percent of stabilizers within the specimen as well as type of stabilizers.

From the load penetration curves CBR values for each case was calculated for penetration of 2.5 mm and 5.0 mm and it was observed in all the cases considered in this research that CBR values corresponding to 2.5 mm penetration was higher than that obtained for 5.0 mm penetration. Therefore, CBR values reported in this research are those of 2.5 mm penetration.

CBR results as presented in Table 5.9 shows there is significant improvement in strength of soil as a result of cement and molasses addition when compared to 1.3% of untreated soils. Soils treated with 4%, 8% and 12% cement cure for seven days and soaked for the same period yielded CBR values of 27.3%, 82.3% and 123% respectively. Even higher CBR values of 63.5%, 97.0% and 127.5% were attained for 4%, 8% and 12% cement mixed with 4% molasses for the same curing and the soaking period as of cement treated samples.

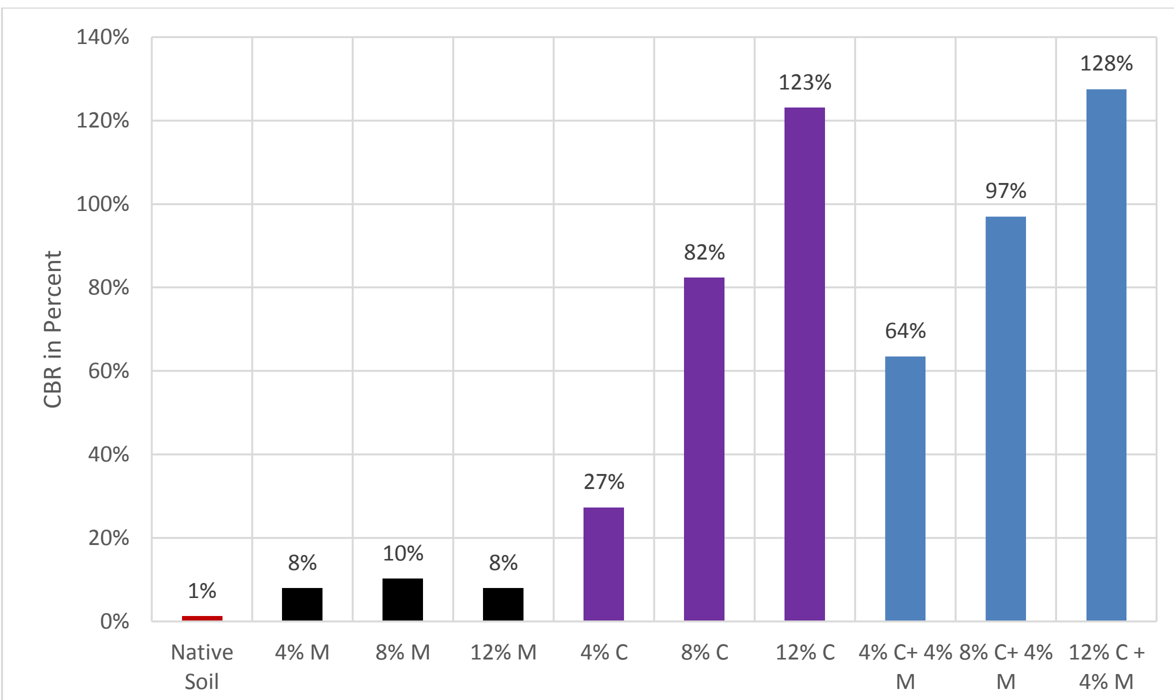


Figure 5.17: Summary of CBR values of treated and untreated soils

Table 5.9: CBR values of Native soils, Soils treated by Cement and Cement and molasses

	Type of Additive	Penetration (mm)	10 blows	30 blows	65 blows	CBR @ 95%MDD
Native Soil	Test #1	2.5	0.6	1.3	2	1.3
		5.04	0.4	0.8	1.6	
	Test #2	2.5	0.6	0.9	2.2	1.25
		5.04	0.5	0.7	1.8	
	Test #3	2.5	0.7	1.1	2	1.25
		5.04	0.6	0.8	1.7	
Test #1	4% Cement & 4% Molasses	2.5	42	61	71	65
		5.04	41	52	62	
	8% Cement & 4% Molasses	2.5	54	93	110	99
		5.04	52	78	89	
	12% Cement & 4% Molasses	2.5	77	129	142	130
		5.04	67	102	116	
Test #2	4% Cement & 4% Molasses	2.5	41	63	72	62
		5.04	40	55	62	
	8% Cement & 4% Molasses	2.5	57	94	115	95
		5.04	57	79	93	
	12% Cement & 4% Molasses	2.5	92	123	139	125
		5.04	75	96	114	
Test #1	4% Cement	2.5	14	28	40	27.3
		5.04	11	25	40	
	8% Cement	2.5	48	89	110	82.3
		5.04	7	77	87	
	12% Cement	2.5	99	127	143	123
		5.04	82	97	116	

However, for soils treated with molasses cure for 14 days and soaked for 4 days showed a maximum value of 10.3 % CBR values for soils treated with 8% molasses content. It was observed that soils treated with molasses at higher curing period and lesser soaking duration attained a very low CBR value when compared to the same quantity of cement and cement and molasses treatments. Table 5.10 summarizes CBR results of soil treated with molasses.

Table 5.10: CBR values of Soil treated by Molasses

Cure for 14 days and soaked for 4 days						
	Type of Additive	**Penetration (mm)**	**10 blows**	**30 blows**	**65 blows**	**CBR @ 95%MDD**
Test #1	4% Molasses	2.5	5	7	12	8
		5.04	4	5	9	
	8% Molasses	2.5	7	10	13	11
		5.04	5	8	9	
	12% Molasses	2.5	6	8	9	8.5
		5.04	4	6	7	
Test #2	4% Molasses	2.5	3	8	10	8
		5.04	2	5	7	
	8% Molasses	2.5	6	8	13	9.5
		5.04	4	6	9	
	12% Molasses	2.5	6	7	8	7.5
		5.04	4	5	6	

In addition to these, soils treated with molasses + cement showed higher CBR values than cement treatment alone at lower stabilizers content, i.e. 4% and 8% and very close CBR values attained at 12% cement and 12% cement + 4% molasses treatment as shown in Figure 5.18.

5.5.5 CBR Swell of Expansive Clay Soils

Expansive soil treated with molasses showed reduction in CBR swell when compared to 10.4 % of untreated soil. Soil treated with 4.0%, 8.0% and 12% molasses yielded average CBR swell of 2.62%, 2.32% and 2.46% respectively. Soil treated with 8.0 % molasses showed the largest reduction in CBR swell which is 78% smaller than untreated soil, see Figure 5.19. Moreover, it was observed that swelling potential decreases with increasing in molasses content up to 8.0% and then further increases beyond 8% molasses content resulted in increasing of swelling potential. Ndegwa (2011) in his study on effect of cane molasses on the strength of expansive soil found similar trend with the relationship of molasses content and swelling potential in terms CBR swell but lesser CBR swell value than this research findings. This could be due to variation in clay mineralogy of the expansive soils.

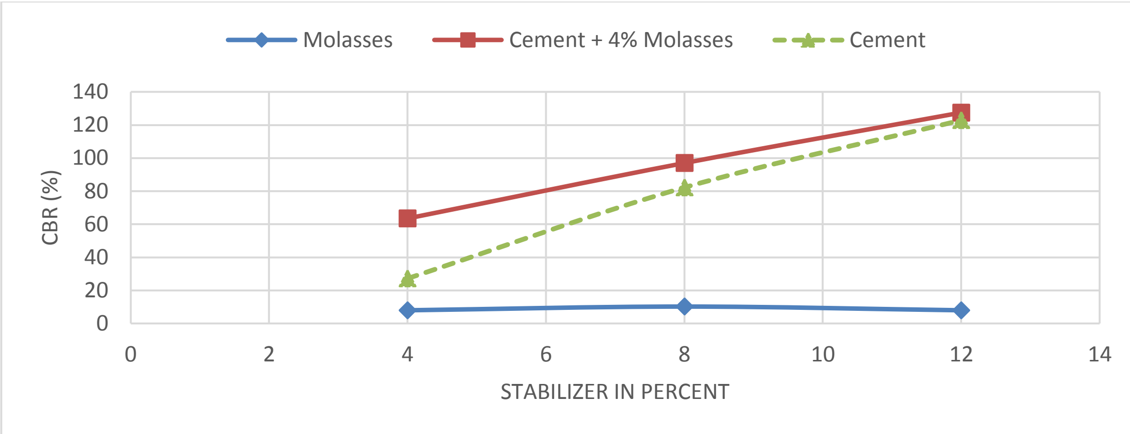


Figure 5.18: Soaked CBR test results of expansive soil mixed with various proportion the three Stabilizers

Although, the CBR swell values increased with molasses content up to 8% after then it decreases, the recorded peak values of CBR swell value of 2.32% fell short of 1.5% specified by the ERA /TRL as criterion for sub-grade/embankment fill material.

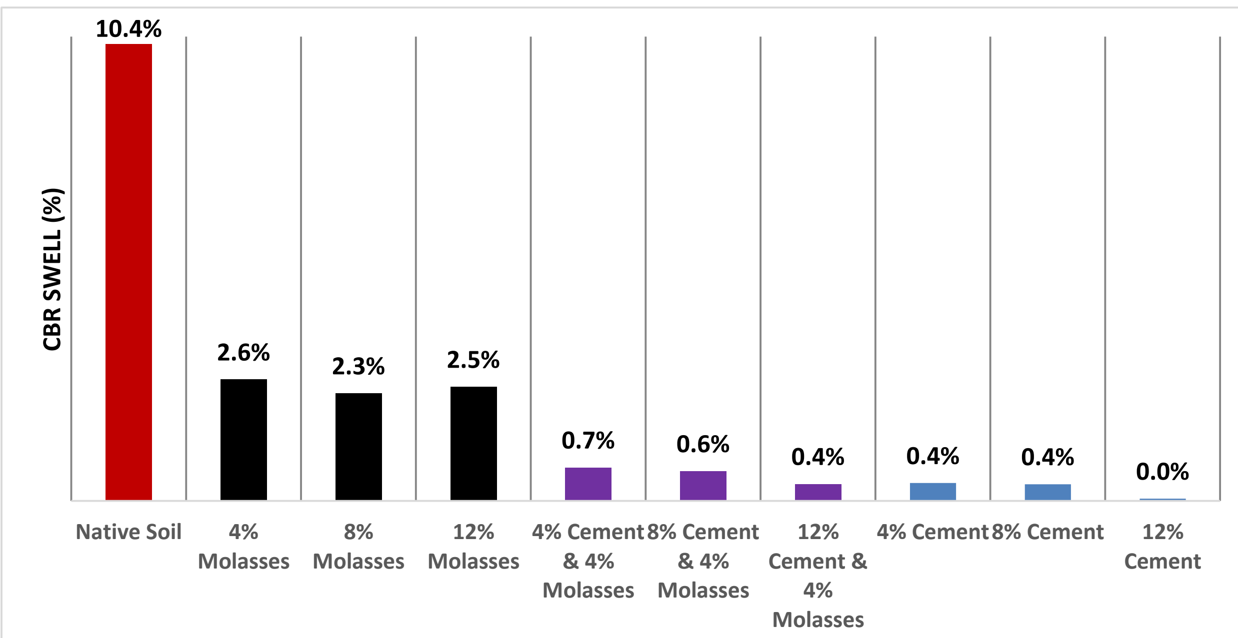


Figure 5.19: Summary of CBR swells for treated and untreated soils

CBR swell showed significant reduction with the addition of cement and cement and molasses. Soils treated with the two stabilizers, as shown in Table 5.11, gave CBR Swell of less than 1.0%. In addition to this, it is observed that swelling potential of expansive soils treated with cement and cement and molasses shows reduction in swelling potential as a percent of stabilizers increase. For example, soil treated with 12% cement yielded almost nil swelling. These reduced swell characteristics are generally attributed to decreased affinity for water of the calcium saturated clay and the formation of a cementious matrix that resists volumetric expansion (Siddique and Alogir, 2011). Soil treated with various percentages of cement alone and combination of cement and molasses as shown in Table 5.11 met the requirement specified by the ERA /TRL as criterion for suitable material.

Table 5.11: CBR Swell for Expansive Clay Soil

	Type of Additive	CBR Swell in %
Native Soil	Test #1	10.4
	Test #2	10.79
	Test #3	9.36
Test #1	4% Molasses	2.61
	8% Molasses	2.54
	12% Molasses	2.70
Test #2	4% Molasses	2.62
	8% Molasses	2.09
	12% Molasses	2.21
Test #1	4% Cement & 4% Molasses	1.08
	8% Cement & 4% Molasses	1.00
	12% Cement & 4% Molasses	0.57
Test #2	4% Cement & 4% Molasses	0.36
	8% Cement & 4% Molasses	0.28
	12% Cement & 4% Molasses	0.16
Test #1	4% Cement	0.39
	8% Cement	0.36
	12% Cement	0.05

5. 6 Unconfined Compressive Strength (UCS) Values

This section presents the UCS values determined for sample soils used in the experimental work. UCS tests were performed on treated and untreated remolded specimen following procedures in ASTM D-5102 Procedure A, as briefly discussed in Chapter Four. The stress-strain curves obtained from untreated and treated /stabilized soils by molasses, cement and cement and molasses are presented in Figure 5.20 through Figure 5.27; while Figure 5.28 and Table 5.13 show comparison of UCS values obtained by the aforementioned stabilizers and the untreated subgrade soils.

5.6.1 Effect of Molasses on UCS Values

Results of UCS tests performed on remolded, untreated, specimens and remolded specimen treated with different molasses contents and curing periods are summarized in Table 5.12. The UCS values of molasses stabilized soil found to increase in molasses content up to 8.0% and then after it decreases.

Table 5.12: Summary of UCS values of Untreated and Molasses Treated Specimens

Tests	Stabilizer Percent	Unconfined Compressive Strength in kN/m^2	Remark
Test #1	Native Soil	141	14 days Curing
Test #2		138.8	
Test #3		135.8	
Test #1	4% Molasses	140.3	14 days Curing
	8% Molasses	138.5	
	12% Molasses	120.3	
Test #2	4% Molasses	138.3	14 days Curing
	8% Molasses	191.8	
	12% Molasses	140.9	
Test #3	4% Molasses	199.9	14 days Curing
	8% Molasses	260.2	
	12% Molasses	140.9	
Test #1	4% Molasses	199.3	64 days Curing
	8% Molasses	254.4	
	12% Molasses	178.2	

Peak failure stress of treated specimens with 4%, 8% and 12% molasses and cure for 14 days yielded average stress of 159.50 KN/M^2, 196.83 KN/M^2 and 134.03 KN/M^2 respectively. In addition to these, peak stress of failure on treated specimens with the same molasses content as stated above and cure for 64 days showed improved strength with curing time irrespective of molasses content. Peak failure of stress of treated specimens as a function of time are plotted and shown in Figure 5.20.

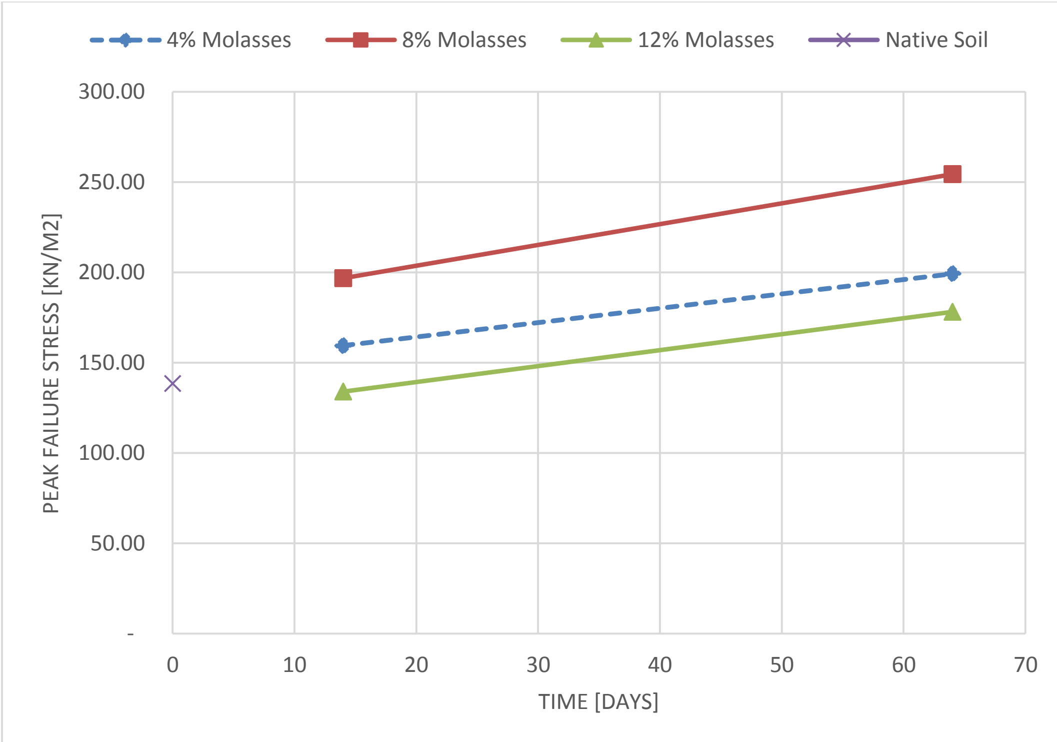


Figure 5.20: Peak Failure Stresses as a Function of Curing Time.

The addition of molasses enhances the strength of the untreated soils; while at the same time the soil improves its durability nature or cohesive nature and become more ductile as the axial strain increased considerably with increase in molasses content. Failure strains of the untreated specimen averaged about 3.67 percent, while the treated specimen cure 14 days ranged from 3.88% to 4.31%. Even higher strains for treated specimen and cure for 64 days yielded extended from 4.09% to 8.62%. Typical stress-strain curves of soil specimens with different molasses content as presented in Figure 5.21 through 5.23 clearly indicated that the stress-strain curves shift towards the right hand side as the strain at failure increased with addition of stabilizers; hence, molasses makes the soil more ductile than untreated/un-stabilized soils. Additionally, the failure mode of the treated material exhibited a plastic type of failure mode as shown below.

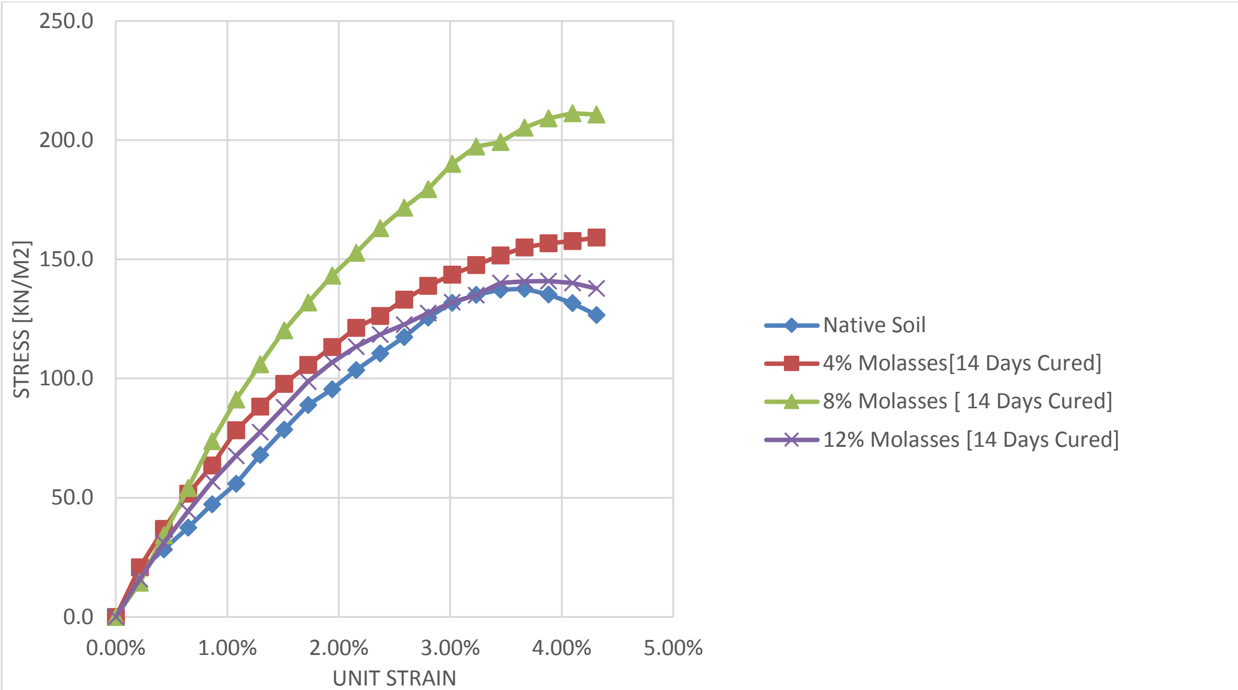


Figure 5.21: Stress-Strain relation for Molasses treated soils and Cure for 14 Days

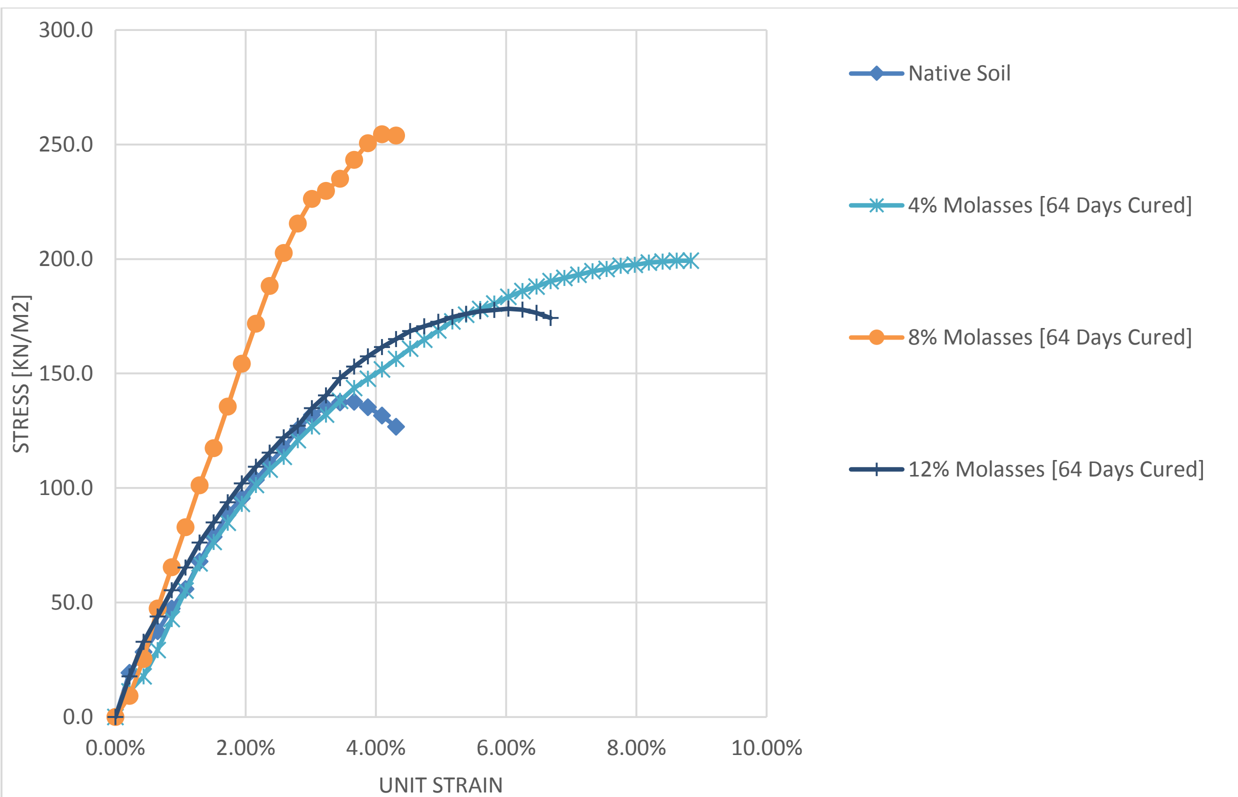


Figure 5.22: Stress-Strain relation for Molasses treated soils and Cure for 64 Days

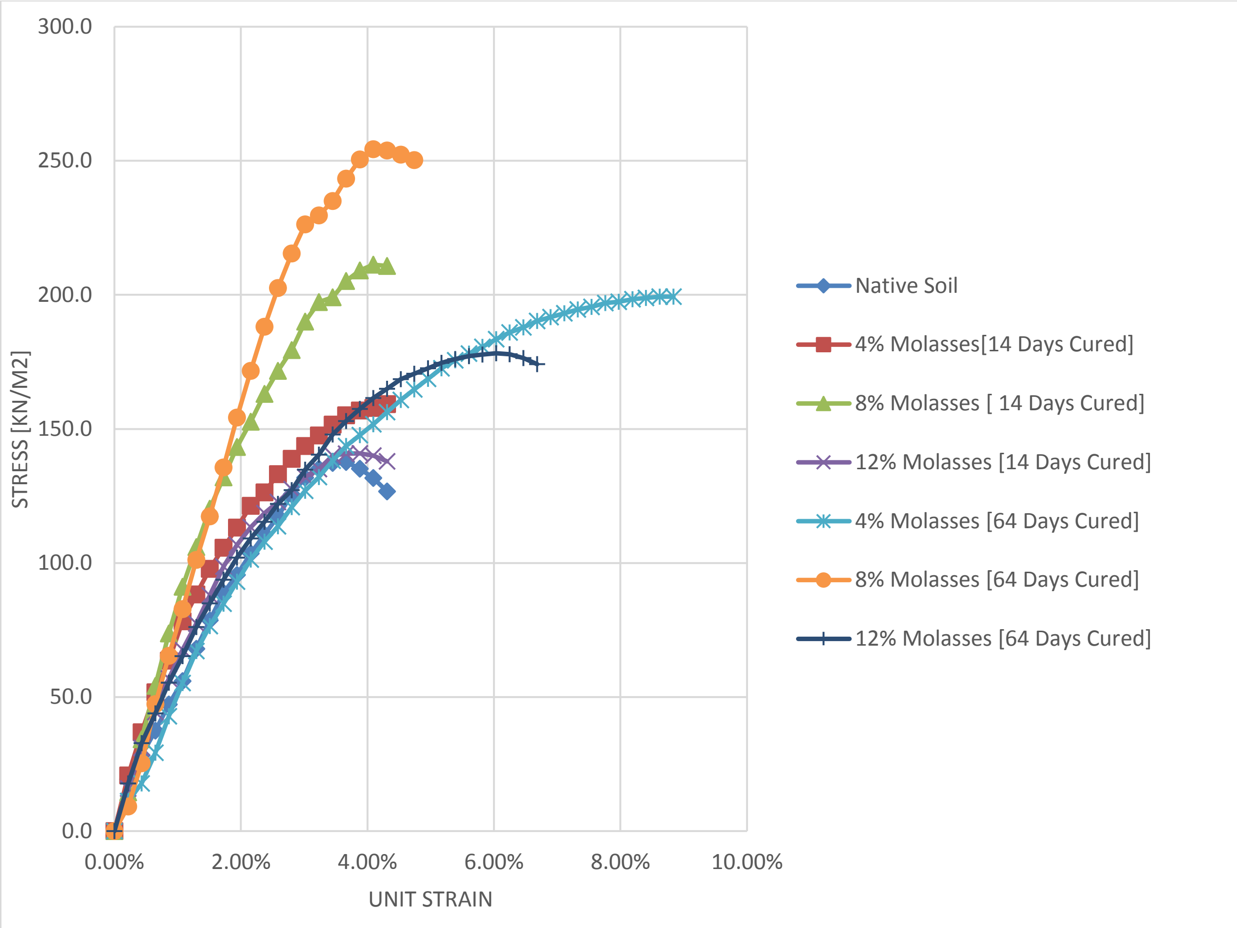


Figure 5.23: Stress-Strain relation for Untreated Soils and Molasses treated soils of Different Curing Days

The increase in UCS values of molasses treated soils may be attributed to cation exchange reaction and flocculation and agglomeration effect and the reduction of UCS values with increasing molasses content beyond a certain limit can also be attributed to coating of individual soil grains with molasses as discussed in section 5.5.1. The strength of 254.4 KN/M^2 at 64 days showed that the strength development of molasses treated soil is a slow process and it can be concluded that curing has a little effect on the strength of expansive clay soil treated with cane molasses.

5.6.2 Effect of Cement on UCS Values

Results of unconfined compressive strength tests performed on remolded, untreated, specimens and remolded specimens treated with 4%, 8% and 12% cement are summarized and compared in Figure 5.24. The specimens as discussed in chapter 4 were remolded to optimum moisture content and maximum dry density by Shelby tube and cure for 7 days.

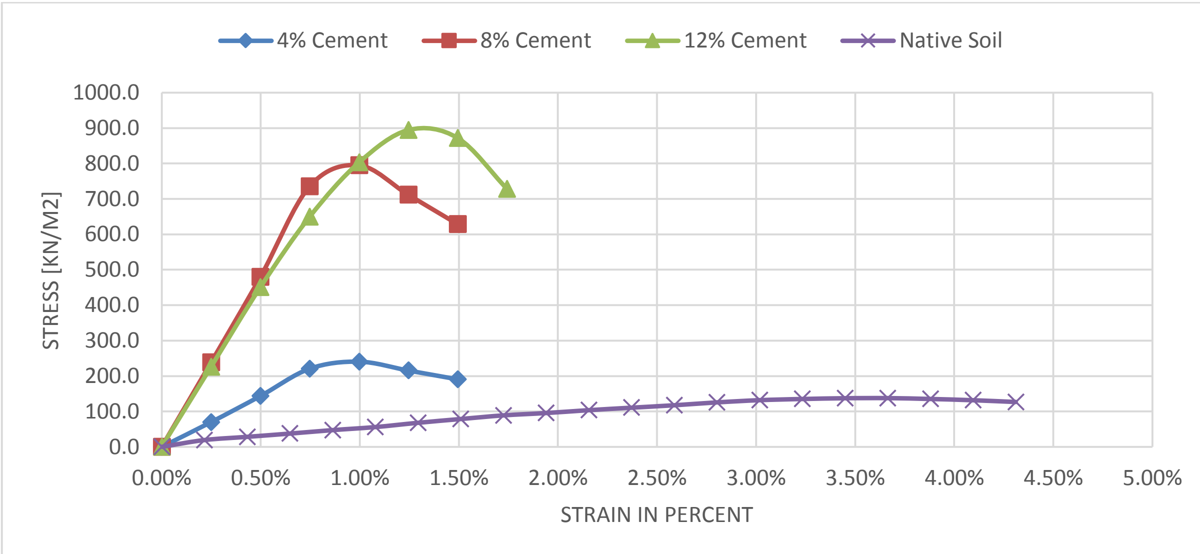


Figure 5.24: stress-strain curve for untreated and soils treated with cement

Peak failure stress of the cement treated specimens for 4%, 8% and 12% cement were 235, 780 and 877 KN/M^2 respectively. Average peak failure stress of the untreated specimens were 138.53 KN/M^2. Peak failure stress of treated specimens for 12% cement and 4% cement was about 6.3 times and 1.7 times larger than the peak stress failure of untreated specimen respectively. Generally it was observed that UCS values increased with increasing cement content. See Figure 5.25.

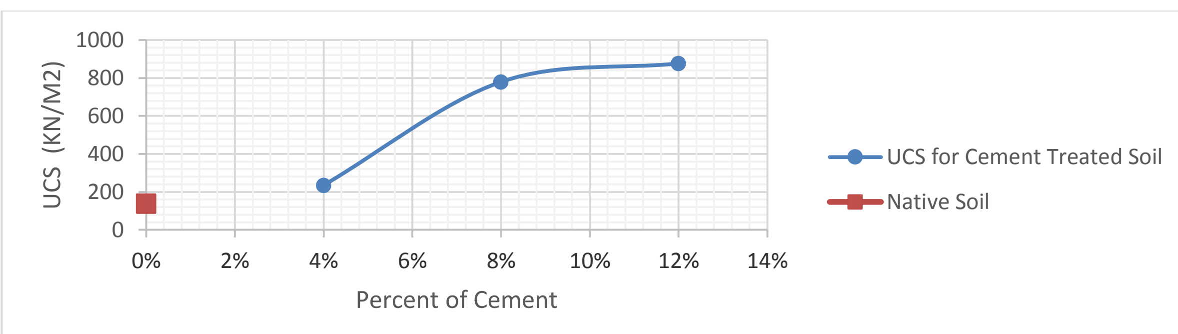


Figure 5.25:UCS test results for untreated and treated soils with cement

Failure strains of the untreated specimen averaged about 3.67 percent, while the treated specimen cure 7 days around 1.0% to 1.2% for cement. Typical stress-strain curves of soil specimens with different cement content as presented in Figure 5.24 clearly indicated that the stress-strain curves shift towards the left hand side as the strain at failure decreased with the addition of stabilizers; hence, treatment of soil with cement produced material having failure strains that were 30 percent of the failure strain of the untreated soil. Additionally, the failure mode of the treated material exhibited a brittle type of failure mode. See Figure 5.26.

Figure 5.26: Specimen treated by 4% cement (left) and 8% cement (right) after UCS test

5.6.3 Effect of Cement and molasses on UCS Values

The UCS values for on remolded specimens treated with 4% cement + 4% molasses, 8% cement + 4% molasses and 12% cement + 4% molasses and cure for 7 days were found 248, 999 and 859 KN/M^2 respectively. Generally, as shown in Figure 5.25, strength of the expansive clay soil increased with higher cement and molasses combination. Peak failure stress of treated specimens for 8% cement and 4% cement was about 7.2 times larger than the peak stress failure of untreated specimen. However, 12% cement +4% molasses combination yielded lower UCS value than 8% cement +4% molasses combination for the same curing period and condition; probably because of insufficient water needed to bring the pozzolanic reaction to completion.

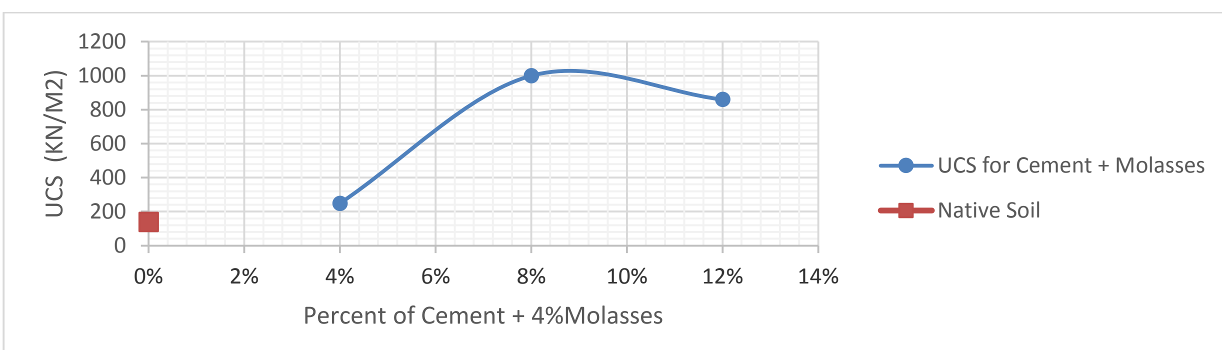


Figure 5.27: UCS results of untreated and treated soils with cement and molasses combination

Failure strains of the untreated specimen averaged about 3.67 percent, while the treated specimen cure 7 days ranged from 1.5% to 2.0%. Typical stress-strain curves of soil specimens with different cement and molasses content as presented in Figure 5.28 clearly indicated that the stress-strain curves shift towards the left hand side as the strain at failure decreased with the addition of stabilizers; hence, treatment of soil with cement and molasses produced material having failure strains that were 47 percent of the failure strain of the untreated soil. Additionally, the failure mode of the treated material exhibited a brittle type of failure mode. See Figure 5.29.

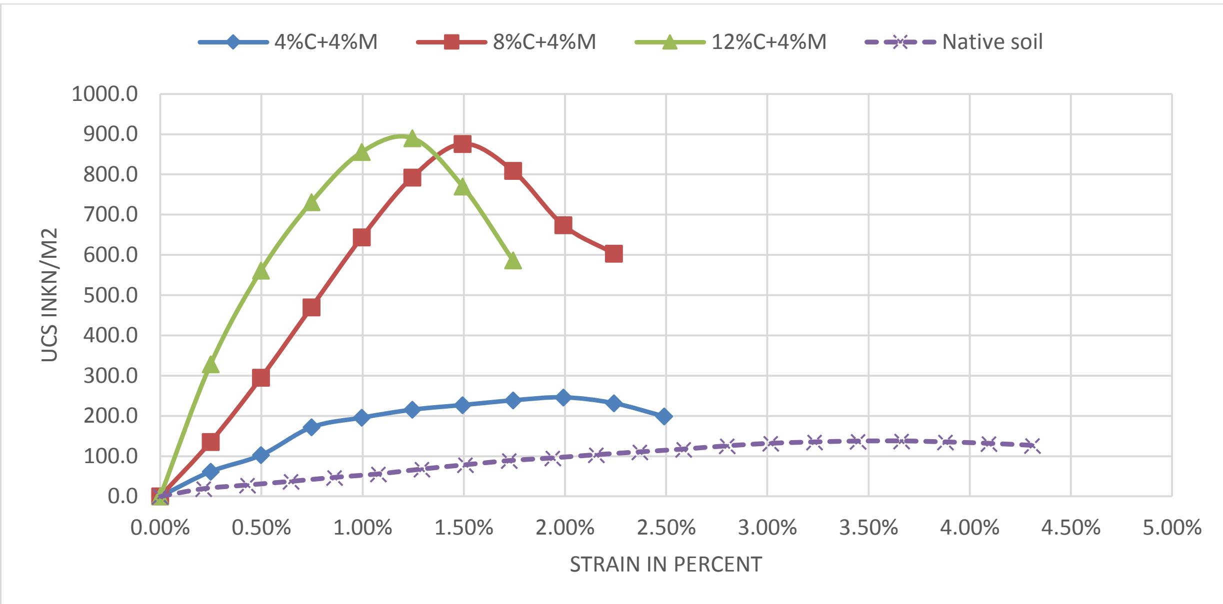


Figure 5.28: Stress-strain curve for untreated soils and treated soils with cement and molasses combination.

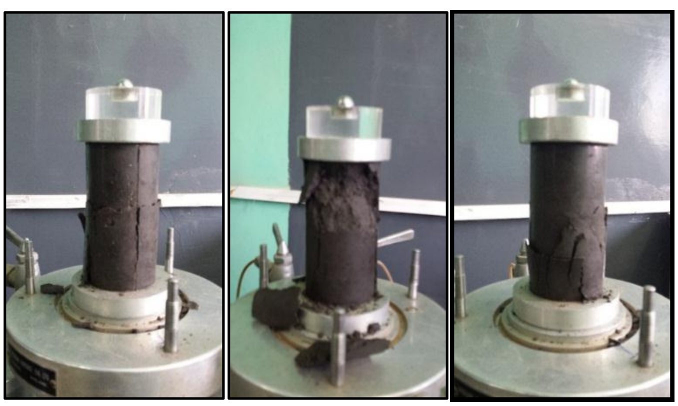


Figure 5.29: specimens treated by 8% cement + 4% molasses after UCS test

5.6.4 Comparison of Effect of Stabilizers on UCS Values

Unconfined compressive strength tests results performed on remolded, untreated, specimens and remolded specimens treated with molasses, cement and cement and molasses combination with various content of stabilizers and different curing period are summarized and compared in Figure 5.30.

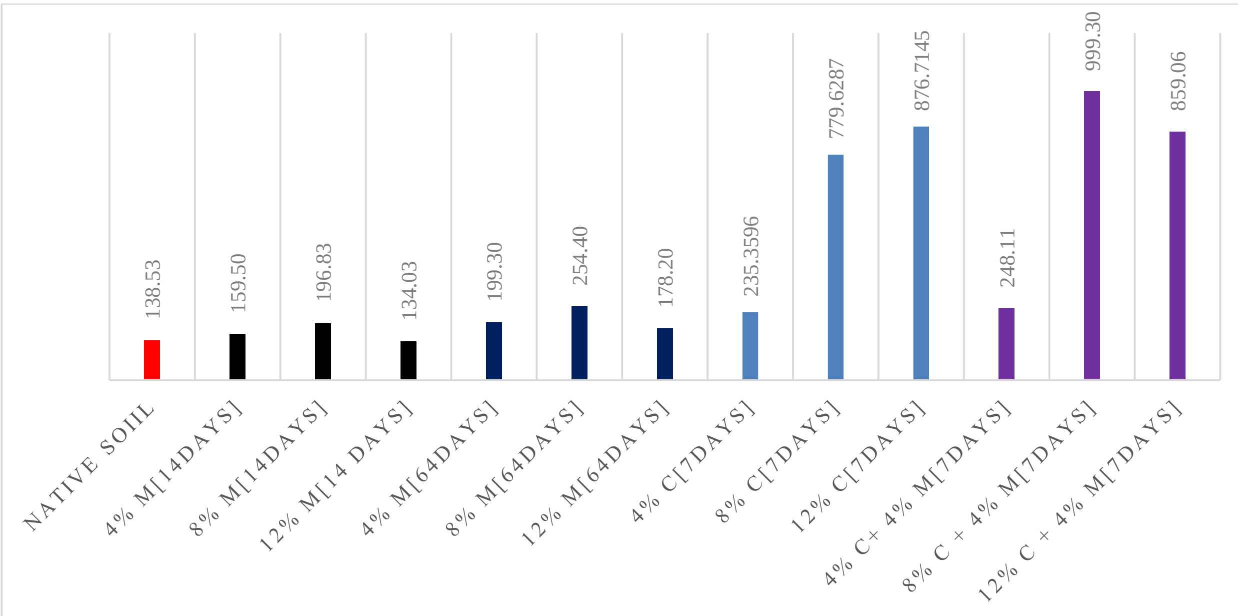


Figure 5.30: Summary of UCS results for untreated and treated soils with different stabilizers

The highest peak failure of stress with value of 999.30KN/M2 attained with the addition of 8% cement and 4% molasses cure for 7 days. However, soils treated 12% molasses and cure for 14 days yielded lowest stress failure with a value of 134.03 KN/M^2 even less than that of 138.53 KN/M^2 value of the untreated soil. Generally, soils treated with different content of molasses and for short (14 days) curing and longer (64 days) curing duration did not give appreciable improvement in UCS values. In addition to this, it is observed that the strength of expansive soils treated with cement and cement and molasses shows significant improvement, even at lower (7days) curing period than molasses treated soils at 14 days and 64 days.

Typical stress-strain curves of soil specimens with different molasses, cement and cement and molasses content are presented and compared with untreated soil in Figure 5.31. As it may be seen in Figure 5.31 the stress-strain curves shift towards the left hand side for soil treated with molasses whereas it shifts to right hand side for soils treated with cement alone and cement and molasses combination. Hence, treatment of soil with cement and cement and molasses produced material having a brittle type of failure mode and addition of molasses with soil improves ductility of the soils and gives a plastic type of failure mode. In addition to this it is observed that addition of molasses to cement shifts the stress-strain curve towards the right with respect to soil treated with cement alone; hence, addition of molasses to cement improves the ductility of the expansive soil than that of the expansive clay soil treated with cement alone.

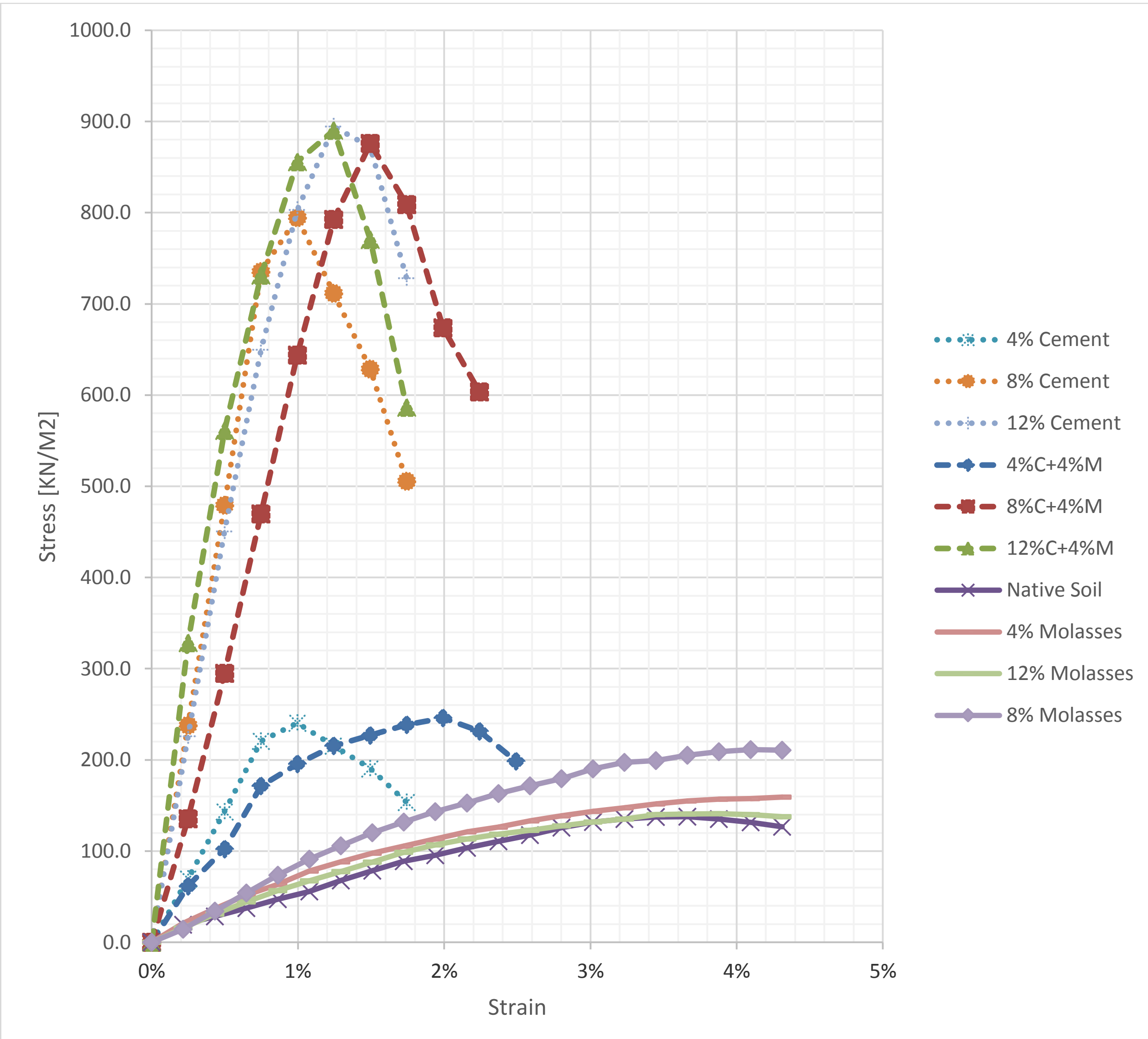


Figure 5.31: Summary of Stress-Strain curves for untreated soil and treated soil with different stabilizers

Chapter 6

6. Conclusions and Recommendations

6.1 Conclusions

Based on the results of the investigation the following conclusions and recommendations may be drawn;

- The laboratory test result of the sample sugar cane molasses test showed that it contained a small quantity of inorganic elements/ compounds which are active in causing a chemical reaction involving cation exchange with the expansive clay soil during stabilization.

- The engineering property of the studied expansive clay soil revealed that it is not suitable to use as a sub-grade and/or embankment fill material unless its undesirable properties are rectified.

- Soil treated with molasses showed substantial improvement. Soil treated with 8%molasses, gave CBR value of 10.4% from a value of 1.3% of the native soil and the CBR swell of 2.3% from 9.8% of the natural soil showed that strength of the expansive soil substantially improved and swelling potential decreased considerably.

- Molasses easily washed by water and soil particles that were flocculated and agglomerated due to an adhesive property of molasses got disintegrated, which resulted losses in strength and decrease in the clay particle size reduction.

- Soil treated with cement by 4%, 8% and 12% by dry weight of the soil showed significant improvement in strength and satisfactorily minimized swelling property of the native soil. Generally it was observed that with increasing cement content, strength improved and swelling potential decreased. Even though; addition of cement reduced PI and linear shrinkage values, shrinkage cracks were observed at all the three stabilizers contents.
- Soil treated with cement and molasses combination gave a significant improvement in strength and meaningfully reduced swelling property of the expansive soil. Cement and

molasses combination were effective in arresting linear shrinkages and eliminated shrinkage cracks that were observed in soil treated with cement alone.

- Soil treated with 4% cement and 12% cement yielded a CBR value of 27.3 and 123 % respectively, while soil treated with 4% cement + 4% molasses and 12% cement and 4% molasses gave a CBR value of 63.5% and 127.5%.

- Soil treated with cement and molasses combination showed higher strength values when cement and molasses contents are equal (at 1:1 ratio).

- Soil treated with molasses has plastic nature while soil treated with cement has brittle nature. Addition of molasses to cement reduced brittle nature of the soil.

- Stabilization of the existing sub-grade expansive clay soil with 4% cement + 4% molasses combination was selected as the optimum stabilizer content.

Even though significant improvements in the soil properties after treatment with molasses were observed and it falls short to meet ERA (2013) specification requirement to use as subgrade and/or embankment fill material.

The addition of cement to molasses has significantly improved strength and eliminated swelling potential of the expansive soil. Furthermore addition of cement to molasses avoids undesirable effects of molasses stabilization (molasses washing by water) and cement stabilization (shrinkage cracks). Therefore, it is concluded that the addition of sugarcane molasses and cement to expansive soil can adequately stabilize it. Furthermore, 4% sugar cane molasses and 4%cement was found to be optimum stabilizer content for sub-grade stabilization and meet all ERA (2013) specification requirement for stabilized.

6.2 Recommendations

Based on the findings of the research the following are the main recommendations;

- As this investigation was done for sugar cane molasses of Wonji/Shoa Sugar Factory and molasses composition is affected by the soil where the sugar cane grow and sugar production process, it is recommended to conduct more investigation on sugar cane molasses composition of sugar factories in different part of the country.

- Effect of sugar cane molasses duration on stabilization of expansive soil shall be studied.

- As stabilization of expansive soil with cement and molasses mixture is a relatively new concept and are scanty in the literature, chemical interactions and mechanisms involved in cement, sugar cane molasses, water and expansive clay soil shall be studied.

- Effect of curing period on soils treated with cement and molasses combination shall be studied.

And finally, as this study was conducted on expansive clay soils treated with sugar cane molasses alone, cement alone and cement and molasses combination on laboratory basis, their actual performance on the field shall be studied.

References

1. AACRA (2004). Addis Ababa City Roads Authority, (November, 2004). Pavement design Manual. Addis Ababa, Ethiopia.
2. AASHTO (2001). American Association of State Highways and Transport Officials. Standard Specifications for Transportation Materials and Methods of Sampling and Testing. Part 1-Specification.Tewenty First Edition. Washington, D.C.
3. AASHTO (2001). American Association of State Highways and Transport Officials. Standard Specifications for Transportation Materials and Methods of Sampling and Testing. Part 2-Tests.Tewenty First Edition. Washington, D.C.
4. AASHTO (2008). American Association of State Highways and Transport Officials. Standard Recommended Practice for Stabilization of Sub-grade Soils and Base Materials. Washington, D.C.20001.
5. Akogu, E. (2011). Effect of Sugar on physical properties of Ordinary Portland Cement Paste and Concrete. AU Journal of Technology. Volume 14 (3): p.225-228.
6. Alex E. and David J., (March, 2010). Soil Stabilization in Low-Volume Roads; Obstacles to Product Implementation from the Additive Supplier's Standpoint. AnyWay Solid Environmental Solutions, Toronto, Ontario, Canada and University of California Pavement Research Center, Davis, California.
7. Amu, O. O., Fajobi, A.B. and Afekhuai, S.O. (2005) Stabilizing Potential of Cement and Fly Ash Mixtures on Expansive Clay Soil. Journal of Applied Sciences. 5 (9), 2005. P.1669-1673.
8. ASTM (2003). American Society for Testing and Materials. Annual of Book of ASTM Standards. Volume 04.08. Soil and Rock (I): D 420-D-5611.100 Barr Harbor Drive. West Conshohocken, PA 19428-2959.
9. Balasingum,M. and Farid, S., (2008). Interpretation of Geotechnical Properties of Cement Treated Soils. Washington State University Department of Civil and Environmental Engineering. Pullman, WA 99164-2910.
10. Bello, A.A (2011). Influence of Compaction Delay on CBR and UCS of Cement Stabilized Lateritic Soil. The Pacific Journal of Science and Technology. Volume 12. Number 2. November 2011 (Fall).
11. Bhattacharja. S. and Bhatty J.I. (2003). Comparative Performance of Portland Cement and Lime Stabilization of Moderate to High Plasticity Clay Soils. Research and

Development Bulletin RD125. Portland Cement Association. 5420 Old Orchard Road. Skokie, Illinois 60077-1083. USA.

12. Caterpillar, (2006). Introduction to Soil Stabilization. QEDQ 1229. www.cat.com
13. Chen, F.H. (1988) Foundations on Expansive Soils. Chen and Associates: 96 South Zuni, Denver, CO80223, U.S.A.
14. Dalls, N. and Nair, S. (2009) Recommended Practice for Stabilization of Subgrade Soils and Base Materials. NCHP, Texas Transportation Institute and Texas A&M University, Texas.
15. ERA (2013).Ethiopian Roads Authority. Pavement Design Manual. Volume I-Flexible Pavements. Addis Ababa, Ethiopia.
16. ERA (2002). Ethiopian Roads Authority. Site Investigation Manual. Addis Ababa, Ethiopia.
17. ERA (2011).Ethiopian Roads Authority, (January, 2011). Road Sector Development Program (RSDP) 13 years Performance and Phase IV. Addis Ababa, Ethiopia.
18. ERA (2013).Ethiopian Roads Authority. Site Investigation Manual. Addis Ababa, Ethiopia.
19. Ethiopian Sugar Development Corporation (2014). [Online]. Available from: www.etsugar.gov.et/en.[Accessed: 20th June 2014].
20. Gautrans, (March, 2004). Manual L 2/04, Stabilization Manual. Department of Public Transport, Gauteng Provincial Government, Pretoria.
21. Gautreau G., Zhang, Z., Wu,Z., Mohammad, L.N. King,B. (2009). Laboratory and accelerated evaluation of sub-base layers in pavement performance. LTRC Project Number: 03-2GT.
22. Grytan S., Islam R., Alamgir M., and Rokonuzzaman (2012). Study on the Geotechnical Properties of Cement based Composite Fine-grained Soil. International Journal of Advanced Structures and Geotechnical Engineering. ISSN 2319-5347, Vol.01, No.02, October 2012.
23. Harty, J.R., (1970) Factors Influencing the Lime Reactivity of Tropically and Subtropical Weathered Soils. Ph.D. Thesis. Department of Civil Engineering, University of Illinois, Urbana, IL.
24. Hasan, Y. and Baris, A. (2012). Usage of Molasses in concrete as a water reducing & retarding admixture. Indian Journal of Engineering & Material Science. Vol.19, December 2012, pp.421-426.

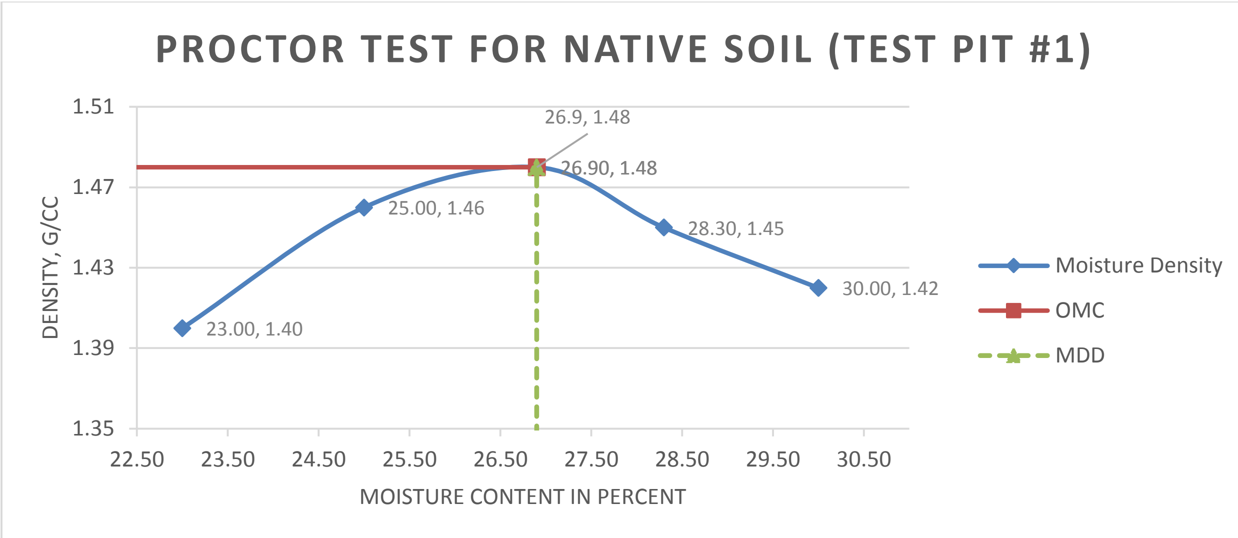
25. Hayder, A. H. (2012). Effect of Fly Ash on Geotechnical Properties of Expansive Soil. Journal of Engineering and Development. Vol.16, June 2012 ISSN 1813-7822.
26. Herzog, A., and Mitchell, J.K. (1966) Reactions Accompanying Stabilization of Clay with Cement. Highway Research Record 36, Highway Research Board, National Research Council, Washington, D.C.
27. Holtz, W.G. and Gibbs, H.J. (1956) Engineering Properties of Expansive Clays. American Society of Civil Engineers. Vol.121, pp.641-677.
28. Horpibulsuk,S., Rachan,R., Chinkulkijniwat, A., Raksachon, Y., Suddeepong, A. (2010). Analysis of strength development in cement-stabilized silty clay from microstructural considerations. Construction and building materials.
29. Jones, D.E J., and Holtz, W.G. (1973). Expansive Soils-The hidden Disaster. Civil Engineering. Vol.43, Nov.8.
30. KBK (2009). KBK Chemical Engineering PVT. LTD. Maharashtra, India.
31. Mohammed, S. (2008) Undrained shear strength and swelling characteristics of cement treated soil. Jordan Journal of Civil Engineering. Volume 2, No.1, 2008.
32. Muhunthan .B. and Sariosseiri .F (2008). Interpretation of Geotechnical Properties of Cement Treated Soils. Washington State University Department of Civil & Environmental Engineering Pullman, WA 99164-2910.
33. Ndegwa J. K and Shitote, S.M. (2012) Influence of Cane Molasses on Plasticity of Expansive Clay Soil. International Journal of Current Research, Volume 4, Issue, 01, pp. 136-141.
34. Ndegwa, J.K. (2011). The effect of Cane Molasses on Strength of Expansive Clay Soil. Journal of Emerging Trends in Engineering and Applied Sciences. Vol.2, No. 6. PP., 1034-1041.
35. Nelson, J.D. and Miller, D.J. (1992). Expansive Soils: Problems and Practice in Foundation and Pavement Engineering. John Wile & Sons, INC. New York.
36. Nigussie, E. (2011). Evaluation of Sodium Silicate and Its Combination with Cement/Lime for Soil Stabilization. MSC Thesis in Civil Engineering. Addis Ababa University, Ethiopia.
37. Nur A.A., Ahmed S.A., Norhazilan M.N and Haryati Y. (2014). Effect of Cement Stabilized Kaolin Subgrade on Strength Properties. Journal of Applied Sciences. 14(9): 842-845, 2014.

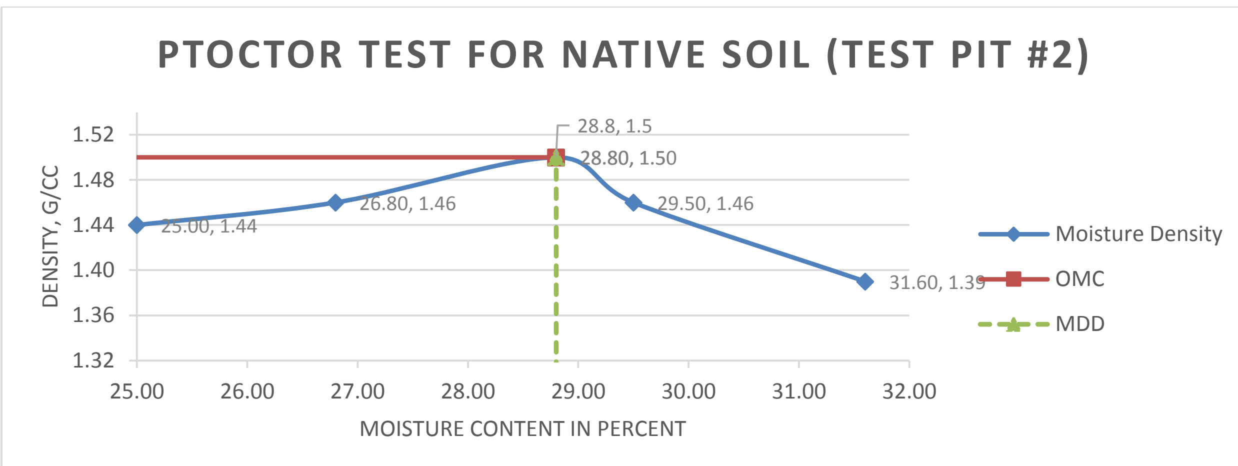
38. Olbrich, H. (1963). The Molasses. Fermentation Technologist, Institut für Zucker industrie, Berlin (Germany).
39. Osinubi, K.J., Oyelakin M.A. and Eberemu A.O. (2011).Improvement of Black Cotton Soil with Ordinary Portland Cement-Locust Bean Waste Ash Blend. Electronic Journal of Geotechnical Engineering (EJGE). Vol.16 [2011].
40. Oyediran I.A. and Kalehaiye M. (2011). Effect of Increasing Cement Content on Strength and Compaction Parameters of some Lateritic Soils from Southwestern Nigeria. Electronic Journal of Geotechnical Engineering (EJGE). Vol.16 [2011].
41. Ravi, E., Sharma A., Manikandan A.T, Kaarthick G., and Jameel A.A. (2015). Study on Effect of Molasses on Strength of Soil. International Journal of Advanced Research Trends in Engineering and Technology (IJARTET). Vol.2, Issue 2, February 2015.
42. Peter, R. (2007). Cane Sugar Engineering. Verlas Dr. Albert Bartens KG - Berlin 2007.
43. Sanjay, K. D., (2012). Stabilization of Very Weak Subgrade Soil with Cementitious Stabilizers. Master Thesis. Louisiana State University andAgricultural and Mechanical College.
44. Seed, H.B., Woodward, R.G. and Lundgren, R., (1962) Prediction of Swelling Potential for compacted clays. Proceedings American Society of Civil Engineers Journal of Soil Mechanics and Foundation Engineering. Volume 88, No. SM3, pp.53-87.
45. Senthen D.R., Towsend F.C., Johnson L.D., Patrick D.M., & Vedros P.J. (1975). A Review of Engineering Experience with Expansive Soils in Highway Subgrades. Washington, D.C.20590.
46. Shirsavkar, S.S and Koranne, S.S (2010). Innovation in Road Construction Using Natural Polymer. Electronic Journal of Geotechnical Engineering (EJGE). Volume 15 (2010). Page 1614-1624.
47. Siddique, A. and Alogir, M. (2011) Effect of Lime Stabilization on Engineering Properties of an Expansive Soil for Use in Road Construction. Journal of Society of Transportation and Traffic Studies (JSTS). Volume.2 No.4.
48. Sridharan, A. and Prakash, K. (2000) Classification Procedures for Expansive Soils. Proc. Instn. Civ. Engrs. Geotech. Engng. 2000, 143, Oct., 235-240.
49. Suriadi, A., Murray, R. S., Grant, C.D. and Nelson, P.N. (2002). Structural stability of sodic soils in sugarcane production as influenced by gypsum and molasses. Australian Journal of Experimental Agriculture. 42; 315-322.

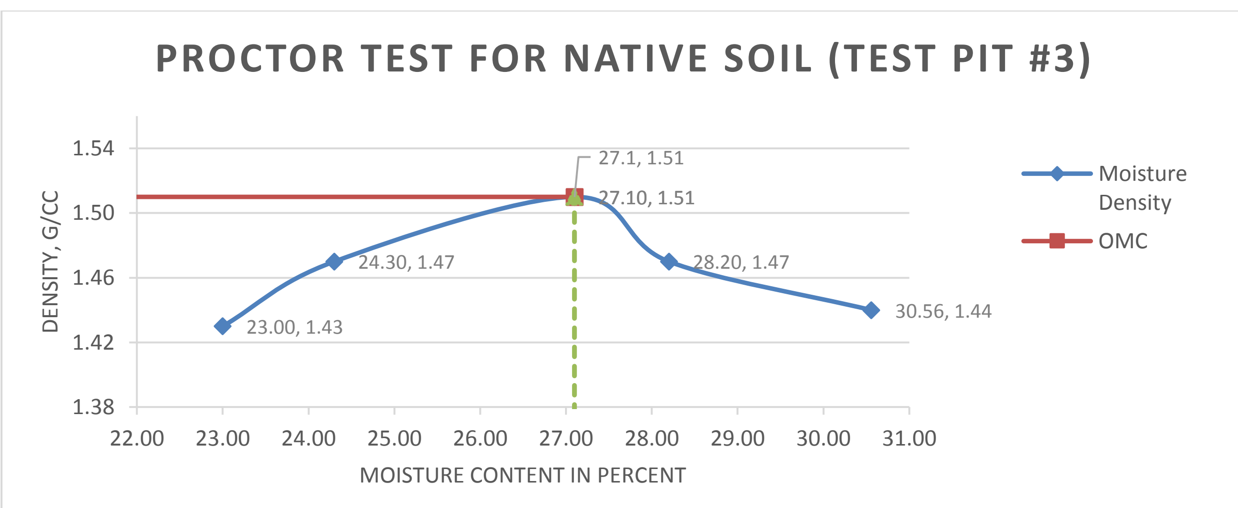
50. TRL. (1993). Oversea Road Note 31. A Guide to the Structural Design of Bitumen Surfaced Roads in Tropical and Sub-Tropical Countries. Transport Research Laboratory, UK, 1993.
51. Van Der Merwe, D.H. (1964) .The prediction of heave from the plasticity index and percentage clay fraction of soil. South African Institute of Civil Engineers. Vol. 6: pp103-107.
52. Winterkorn, H., F., and Pamukçu, S., (1990) Soil Stabilization and Grouting, Foundation Engineering Handbook. Edited by Hsai-Yang Fang, 2nd Edition. Van Nostrand Reinhold: New York.

Appendices

Appendix-1: Laboratory Test Results of Native Soils


PROCTOR TEST FOR NATIVE SOIL (TEST PIT #1)
DENSITY, G/CC
1.51
1.47
1.43
1.39
1.35
22.50
23.50
24.50
25.50
26.50
27.50
28.50
29.50
30.50
MOISTURE CONTENT IN PERCENT
26.9, 1.48
26.90, 1.48
25.00, 1.46
28.30, 1.45
30.00, 1.42
23.00, 1.40
Moisture Density
OMC
MDD

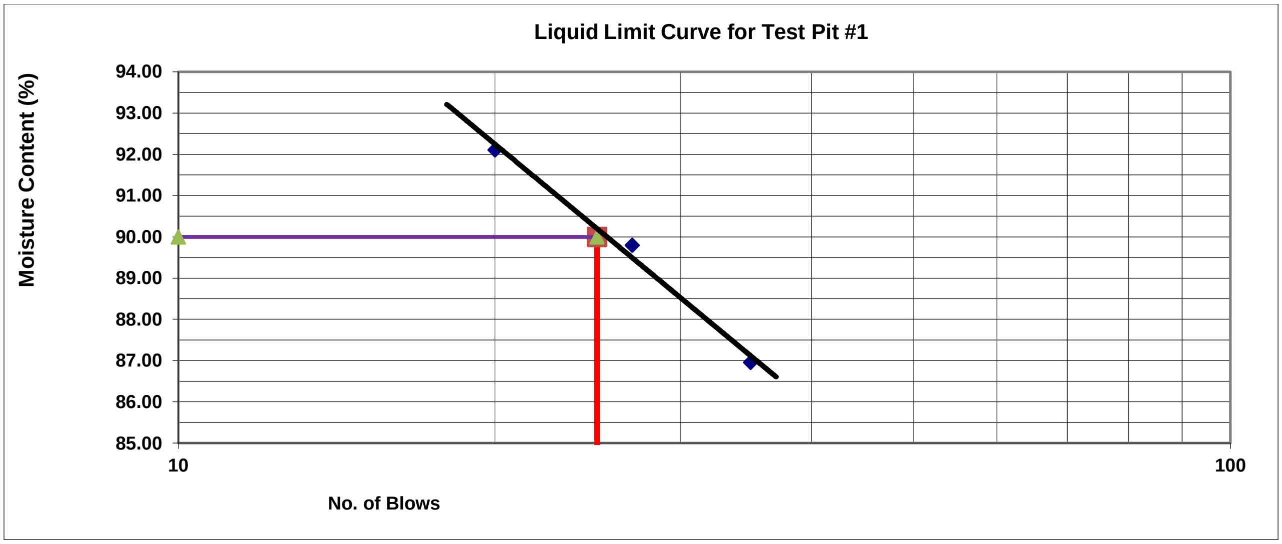

PTOCTOR TEST FOR NATIVE SOIL (TEST PIT #2)
DENSITY, G/CC
1.52
1.48
1.44
1.40
1.36
1.32
25.00
26.00
27.00
28.00
29.00
30.00
31.00
32.00
MOISTURE CONTENT IN PERCENT
28.8, 1.5
28.80, 1.50
26.80, 1.46
29.50, 1.46
25.00, 1.44
31.60, 1.39
Moisture Density
OMC
MDD

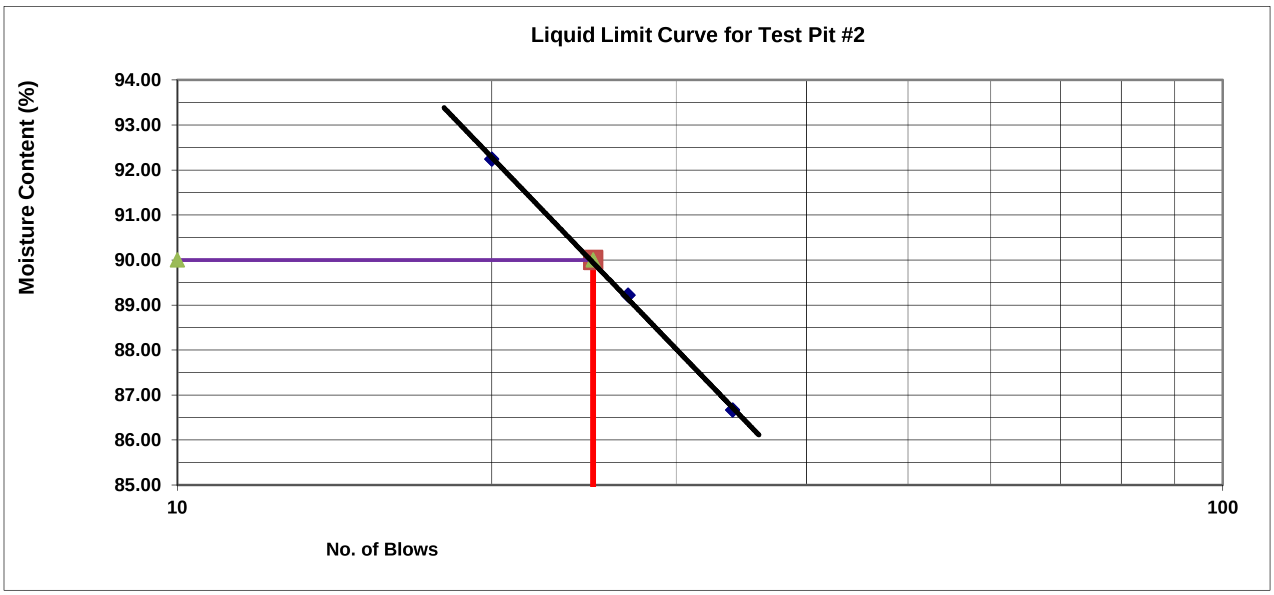

PROCTOR TEST FOR NATIVE SOIL (TEST PIT #3)
DENSITY, G/CC
1.54
1.50
1.46
1.42
1.38
22.00
23.00
24.00
25.00
26.00
27.00
28.00
29.00
30.00
31.00
MOISTURE CONTENT IN PERCENT
27.1, 1.51
27.10, 1.51
24.30, 1.47
28.20, 1.47
30.56, 1.44
23.00, 1.43
Moisture Density
OMC

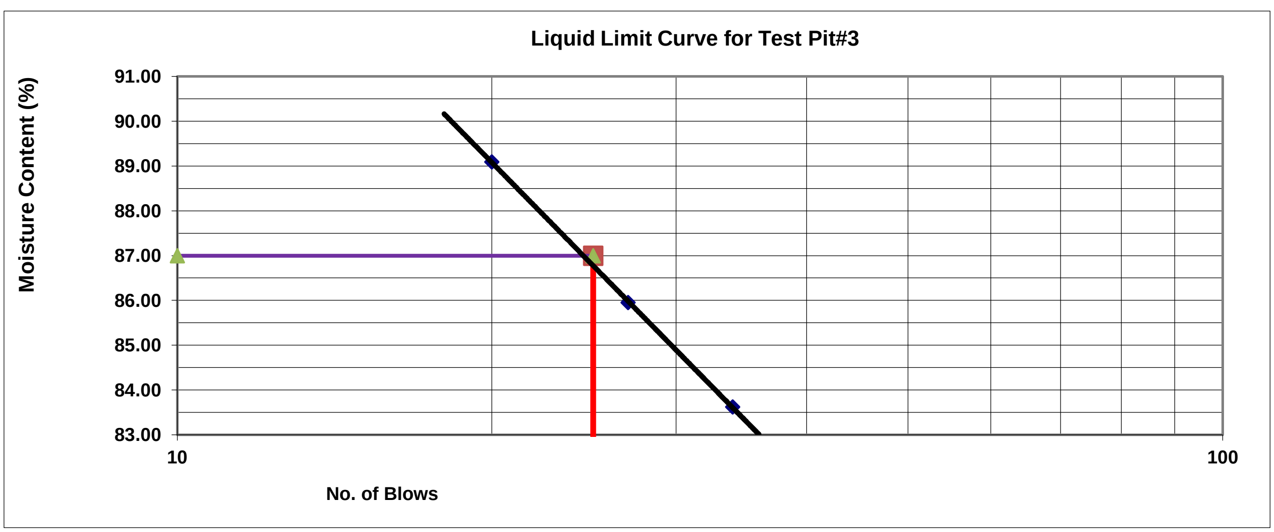
NATIVE SOIL-TEST PIT #1					
	Liquid Limit			**Plastic Limit**	
No. of Blows	35	27	20		
Wt. of cont. + wet soil (g) = (w_1)	25.50	26.60	24.20	16.05	15.40
Wt. of cont. + dry soil (g.) = (w_2)	17.50	17.80	17.20	14.20	13.60
Wt. of container (g.) = (w_3)	8.30	8.00	9.60	9.30	8.70
Mass of moisture (g.) (w_1-w_2) = x	8.00	8.80	7.00	1.85	1.80
Wt. of dry soil (g.) (w_2-w_3) = y	9.20	9.80	7.60	4.90	4.90
Moisture Content (%) = (100x/y)	86.96	89.80	92.11	37.76	36.73
	90			**37**	
	Plasticity Index			**53**	

NATIVE SOIL-TEST PIT #2					
	Liquid Limit			**Plastic Limit**	
No. of Blows	34	27	20		
Wt. of cont. + wet soil (g) = (w_1)	27.80	28.90	31.00	15.50	14.60
Wt. of cont. + dry soil (g.) = (w_2)	18.70	19.80	20.30	14.00	13.30
Wt. of container (g.) = (w_3)	8.20	9.60	8.70	9.80	9.60
Mass of moisture (g.) (w_1-w_2) = x	9.10	9.10	10.70	1.50	1.30
Wt. of dry soil (g.) (w_2-w_3) = y	10.50	10.20	11.60	4.20	3.70
Moisture Content (%) = (100x/y)	86.67	89.22	92.24	35.71	35.14
	90			**35**	
	Plasticity Index			**55**	

NATIVE SOIL-TEST PIT #3					
	Liquid Limit			**Plastic Limit**	
No. of Blows	34	27	20		
Wt. of cont. + wet soil (g) = (w_1)	29.30	30.40	30.00	13.50	14.30
Wt. of cont. + dry soil (g.) = (w_2)	19.60	20.00	20.20	11.90	12.80
Wt. of container (g.) = (w_3)	8.00	7.90	9.20	7.40	8.60
Mass of moisture (g.) (w_1-w_2) = x	9.70	10.40	9.80	1.60	1.50
Wt. of dry soil (g.) (w_2-w_3) = y	11.60	12.10	11.00	4.50	4.20
Moisture Content (%) = (100x/y)	83.62	85.95	89.09	35.56	35.71
	87			**36**	
	Plasticity Index			**51**	

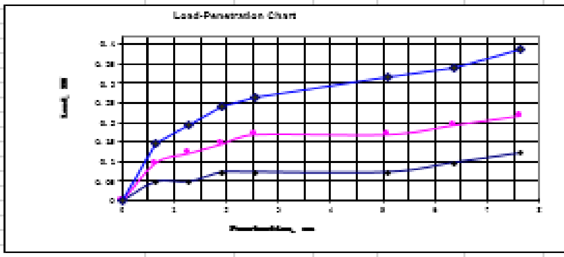

Liquid Limit Curve for Test Pit #1
Moisture Content (%)
94.00
93.00
92.00
91.00
90.00
89.00
88.00
87.00
86.00
85.00
10
100
No. of Blows

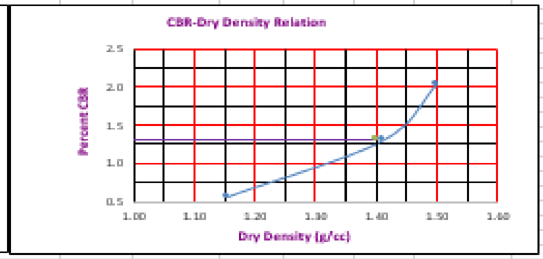

Liquid Limit Curve for Test Pit #2
Moisture Content (%)
94.00
93.00
92.00
91.00
90.00
89.00
88.00
87.00
86.00
85.00
10
100
No. of Blows


Liquid Limit Curve for Test Pit#3
Moisture Content (%)
91.00
90.00
89.00
88.00
87.00
86.00
85.00
84.00
83.00
10
100
No. of Blows

SWELL DATA (Surcharge Weight 4.5kg)												
No. of Blows	10				30				65			
Initial Height of Sample: 116mm	Gauge reading		Swell		Gauge reading		Swell		Gauge		Swell	
	Initial	Final	mm	%	Initial	Final	mm	%	Initial	Final	mm	%
	414	1600	11.86	10.22	74	1118	10.44	9.00	84	1113	10.29	8.87

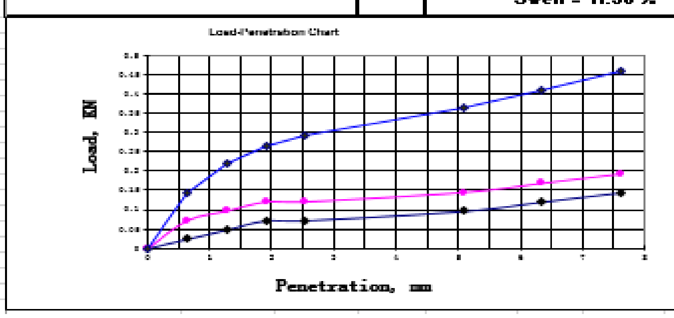
CBR DATA FOR NATIVE SOIL (TEST PIT #1)													
No. of Blows		10				30				65			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR	
			KN	KN	%		KN	KN	%		KN	KN	%
0		0	0			0	0			0	0		
0.64		2	0.05			4	0.10			6	0.14		
1.27		2	0.05			5	0.12			8	0.19		
1.91		3	0.07			6	0.14			10	0.24		
2.54	13	3	0.07	0.07	0.6	7	0.17	0.17	1.3	11	0.27	0.27	2.0
5.08	20	3	0.07	0.07	0.4	7	0.17	0.17	0.8	13	0.31	0.31	1.6
6.35		4	0.10			8	0.19			14	0.34		
7.62		5	0.12			9	0.22			16	0.39		
				Swell =10.22%						CBR			1.30

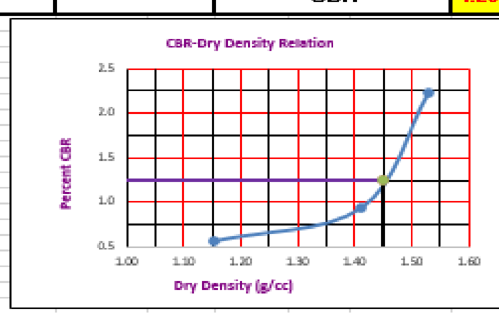




SWELL DATA (Surcharge Weight 4.5 kg)												
No. of Blows	10				30				65			
Initial Height of Sample: 116mm	Gauge reading		Swell		Gauge reading		Swell		Gauge		Swell	
	Initial	Final	mm	%	Initial	Final	mm	%	Initial	Final	mm	%
	745	2088	13.43	11.58	911	2185	12.74	10.98	1212	2350	11.38	9.81

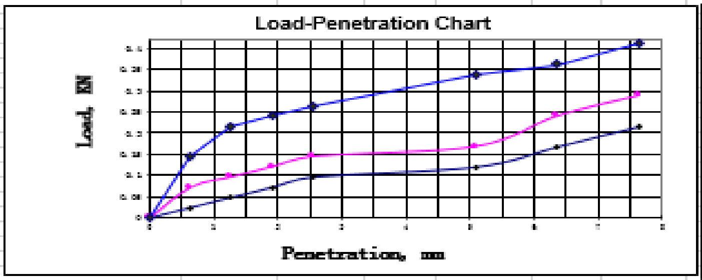
CBR DATA FOR NATIVE SOIL (TEST PIT #2)													
No. of Blows		10				30				65			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR	
			KN	KN	%		KN	KN	%		KN	KN	%
0		0	0			0	0			0	0		
0.64		1	0.02			3	0.07			6	0.14		
1.27		2	0.05			4	0.10			9	0.22		
1.91		3	0.07			5	0.12			11	0.27		
2.54	13	3	0.07	0.07	0.6	5	0.12	0.12	0.9	12	0.29	0.29	2.2
5.08	20	4	0.10	0.10	0.5	6	0.14	0.14	0.7	15	0.36	0.36	1.8
6.35		5	0.12			7	0.17			17	0.41		
7.62		6	0.14			8	0.19			19	0.46		
				Swell = 11.58 %						CBR			1.25

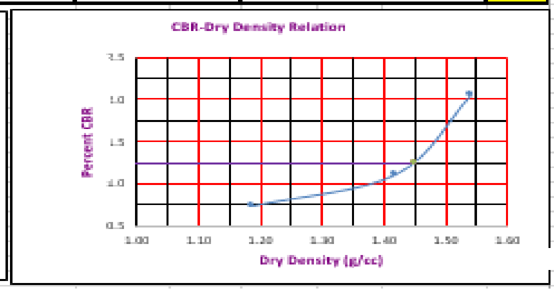




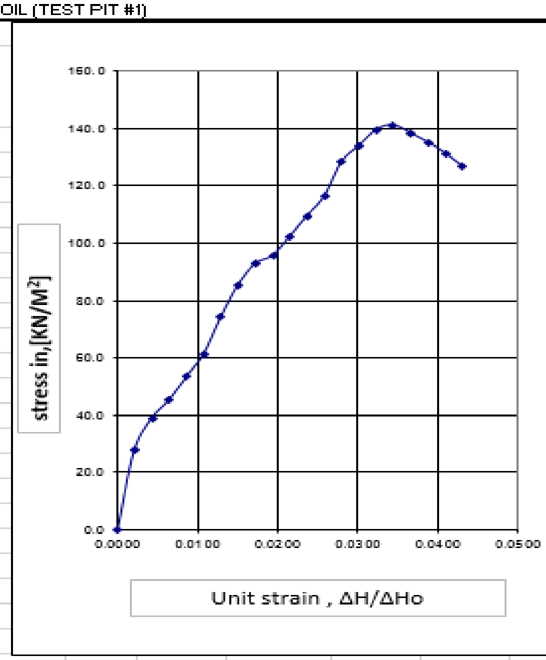
SWELL DATA (Surcharge Weight 4.5kg)												
No. of Blows	10				30				65			
Initial Height of Sample: 116mm	Gauge reading		Swell		Gauge reading		Swell		Gauge		Swell	
	Initial	Final	mm	%	Initial	Final	mm	%	Initial	Final	mm	%
	629	1872	12.43	10.72	785	1861	10.76	9.28	671	1611	9.40	8.10

CBR DATA FOR NATIVE SOIL (TEST PIT #3)													
No. of Blows		10				30				65			
Penetration (mm)	Std load (KN)	Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR		Gauge reading	Load	Corrected CBR	
			KN	KN	%		KN	KN	%		KN	KN	%
0		0	0			0	0			0	0		
0.64		1	0.02			3	0.07			6	0.14		
1.27		2	0.05			4	0.10			9	0.22		
1.91		3	0.07			5	0.12			10	0.24		
2.54	13	4	0.10	0.10	0.7	6	0.14	0.14	1.1	11	0.27	0.27	2.0
5.08	20	5	0.12	0.12	0.6	7	0.17	0.17	0.8	14	0.34	0.34	1.7
6.35		7	0.17			10	0.24			15	0.36		
7.62		9	0.22			12	0.29			17	0.41		
				Swell=9.28 %						CBR			1.25





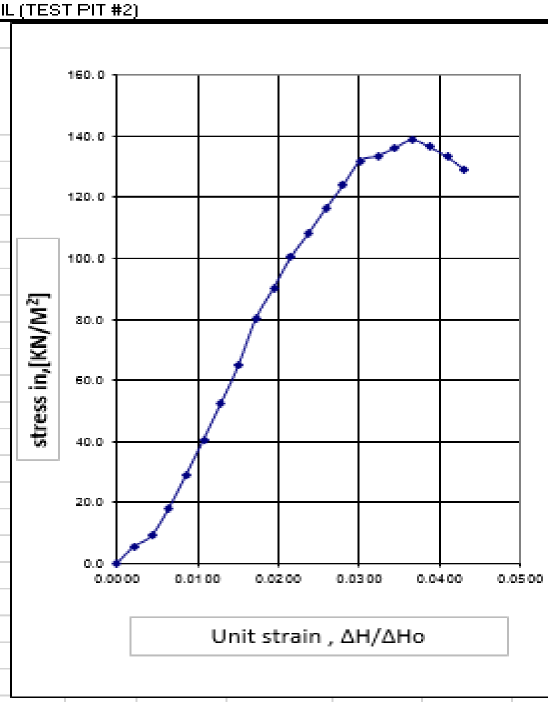
UCS TEST FOR NATIVE SOIL (TEST PIT #1)					
Deformation dial in [mm] ΔH	Unit strain ε = ΔH/ΔHo	Proving Ring reading	Load in [KN]	Cross - sectional Area A = A₀/1-ε	stress in [KN/M²]
0.00	0.0000	0.0	0.0000	0.00264	0.0
0.25	0.0022	55	0.0737	0.00265	27.8
0.50	0.0043	77	0.1032	0.00265	38.9
0.75	0.0065	90	0.1206	0.00266	45.4
1.00	0.0086	106	0.1420	0.00266	53.3
1.25	0.0108	122	0.1635	0.00267	61.2
1.50	0.0129	148	0.1983	0.00268	74.1
1.75	0.0151	171	0.2291	0.00268	85.4
2.00	0.0172	186	0.2492	0.00269	92.7
2.25	0.0194	192	0.2573	0.00269	95.5
2.50	0.0216	206	0.2760	0.00270	102.2
2.75	0.0237	221	0.2961	0.00271	109.4
3.00	0.0259	235	0.3149	0.00271	116.1
3.25	0.0280	260	0.3484	0.00272	128.2
3.50	0.0302	272	0.3645	0.00272	133.8
3.75	0.0323	284	0.3806	0.00273	139.4
4.00	0.0345	288	0.3859	0.00274	141.0
4.25	0.0366	283	0.3792	0.00274	138.3
4.50	0.0388	277	0.3712	0.00275	135.0
4.75	0.0409	270	0.3618	0.00275	131.3
5.00	0.0431	261	0.3497	0.00276	126.7



Unconfined compressive strength, q_u = 141.0 KN/m2

UCS TEST FOR NATIVE SOIL (TEST PIT #2)

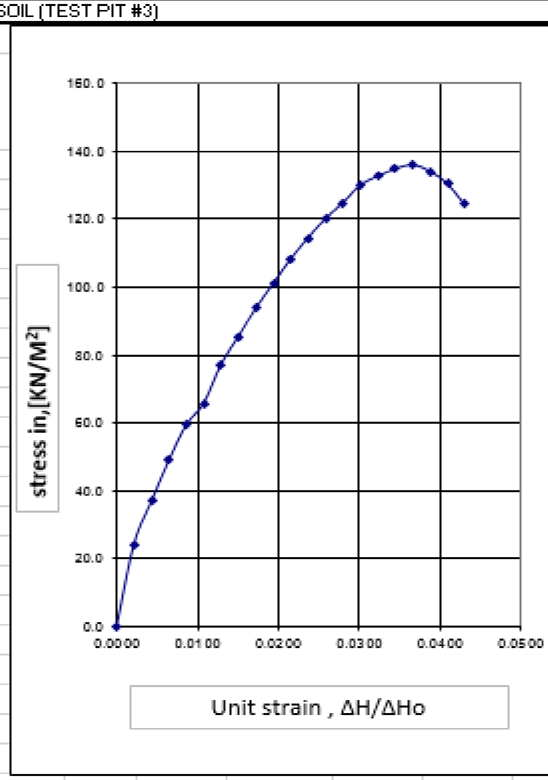
Deformation dial in [mm] ΔH	Unit strain ε = ΔH/ΔHo	Proving Ring reading	Load in [KN]	Cross - sectional Area $A = A_o/1-\varepsilon$	stress in [KN/M²]
0.00	0.0000	0.0	0.0000	0.00264	0.0
0.25	0.0022	11	0.0147	0.00265	5.6
0.50	0.0043	18	0.0241	0.00265	9.1
0.75	0.0065	36	0.0482	0.00266	18.1
1.00	0.0086	58	0.0777	0.00266	29.2
1.25	0.0108	81	0.1085	0.00267	40.6
1.50	0.0129	105	0.1407	0.00268	52.6
1.75	0.0151	130	0.1742	0.00268	64.9
2.00	0.0172	161	0.2157	0.00269	80.2
2.25	0.0194	181	0.2425	0.00269	90.0
2.50	0.0216	202	0.2707	0.00270	100.2
2.75	0.0237	218	0.2921	0.00271	107.9
3.00	0.0259	235	0.3149	0.00271	116.1
3.25	0.0280	251	0.3363	0.00272	123.7
3.50	0.0302	268	0.3591	0.00272	131.8
3.75	0.0323	272	0.3645	0.00273	133.5
4.00	0.0345	278	0.3725	0.00274	136.1
4.25	0.0366	284	0.3806	0.00274	138.8
4.50	0.0388	280	0.3752	0.00275	136.5
4.75	0.0409	274	0.3672	0.00275	133.3
5.00	0.0431	266	0.3564	0.00276	129.1



Unconfined compressive strength, q_u = 138.8 KN/m2

UCS TEST FOR NATIVE SOIL (TEST PIT #3)

Deformation dial in [mm] ΔH	Unit strain ε = ΔH/ΔHo	Proving Ring reading	Load in [KN]	Cross - sectional Area $A = A_o/1-\varepsilon$	stress in [KN/M²]
0.00	0.0000	0.0	0.0000	0.00264	0.0
0.25	0.0022	48	0.0643	0.00265	24.3
0.50	0.0043	73	0.0978	0.00265	36.9
0.75	0.0065	97	0.1300	0.00266	48.9
1.00	0.0086	118	0.1581	0.00266	59.3
1.25	0.0108	131	0.1755	0.00267	65.7
1.50	0.0129	154	0.2064	0.00268	77.1
1.75	0.0151	171	0.2291	0.00268	85.4
2.00	0.0172	188	0.2519	0.00269	93.7
2.25	0.0194	203	0.2720	0.00269	101.0
2.50	0.0216	218	0.2921	0.00270	108.2
2.75	0.0237	231	0.3095	0.00271	114.4
3.00	0.0259	243	0.3256	0.00271	120.1
3.25	0.0280	253	0.3390	0.00272	124.7
3.50	0.0302	264	0.3538	0.00272	129.9
3.75	0.0323	270	0.3618	0.00273	132.5
4.00	0.0345	275	0.3685	0.00274	134.7
4.25	0.0366	278	0.3725	0.00274	135.8
4.50	0.0388	275	0.3685	0.00275	134.1
4.75	0.0409	268	0.3591	0.00275	130.4
5.00	0.0431	256	0.3430	0.00276	124.2



Unconfined compressive strength, q_u = 135.8 KN/m2

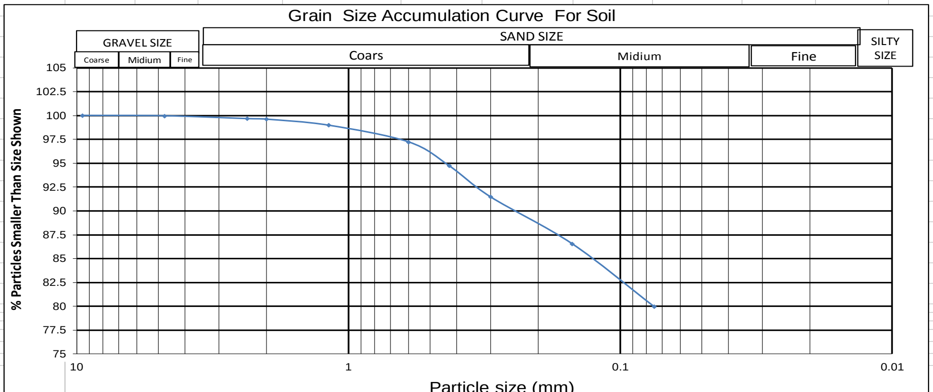
Grain size analysis test result for Native Soil (Test Pit #1)

Befor wash Wt. dry sample= **1612.2** g
After wash Wt. dry sample= **106** g

SIEVE SIZE Alternate (Number)	SIEVE SIZE Standard (mm)	Weight of Sieve in (g)	Weight of Sieve + Weight Retained Sample (g)	Weight Retained Sample (g)	Commulative %Retained	Commulative % Coarser	Commulative % Passing
	9.5						100
4	4.75	1125.6	1126.2	0.6	0.04	0.04	99.96
10	2.36	1025.5	1030.3	4.8	0.30	0.30	99.67
	2.00	990.6	991.6	1.0	0.06	0.06	99.60
20	1.18	1013.2	1018.6	5.4	0.33	0.63	98.97
40	0.600	990.5	1007.5	17.0	1.05	1.75	97.22
60	0.425	855.5	867.8	12.3	0.76	2.51	94.71
140	0.300	755.5	767.2	11.7	0.73	3.24	91.47
100	0.150	555.5	582.7	27.2	1.69	4.92	86.55
200	0.075	435.3	461.9	26.6	1.65	6.57	79.97
			Pan	1506.2	93.43	100.00	0.00
			Total	1612.2			

Gravel,%	0.04
Sand,%	19.99
Fine,%	79.97

Grain Size Accumulation Curve For Soil



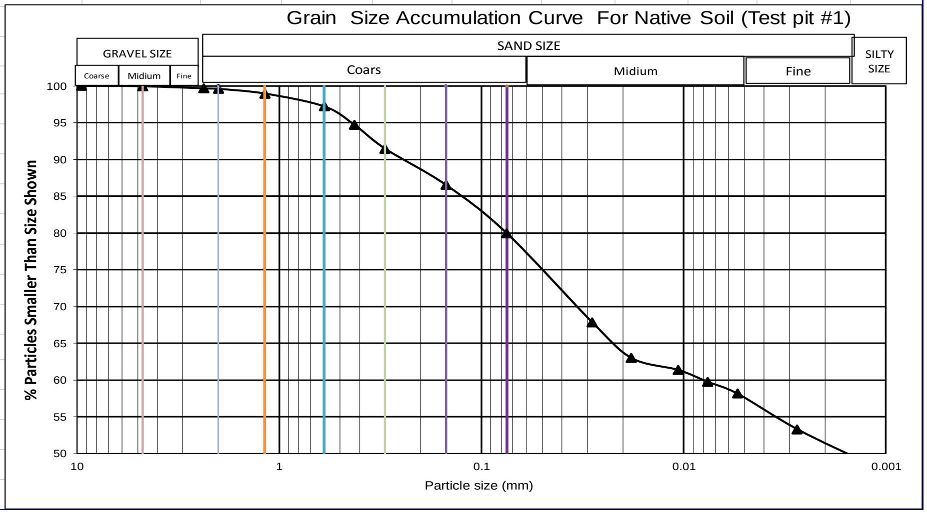
HYDROMETER ANALYSIS

Weight of soil sample gm= 50 Zero Correction = 3 Meniscus Correction = 1

Time	Elapsed time (min)	Temp .(°C)	Actual hydro. Rdg. R_a	Hydro. Corr. for Meniscus	L from Table 1(cm)	K from Table 2	D(mm)	CT from Table 3	a from Table 4	Corr. Hydr. Rdg. R_c	% finer,P	% Adjusted finer P_A
2:07pm	2	20.0	45	46	8.1	0.01408	0.028	0.00	1.01	42	84.84	67.85
	5	20.0	42	43	8.3	0.01408	0.018	0.00	1.01	39	78.78	63.00
	15	20.0	41	42	8.6	0.01408	0.011	0.00	1.01	38	76.76	61.39
	30	20.0	40	41	8.8	0.01408	0.008	0.00	1.01	37	74.74	59.77
	60	20.0	39	40	8.9	0.01408	0.005	0.00	1.01	36	72.72	58.16
	240	20.0	36	37	9.1	0.01408	0.003	0.00	1.01	33	66.66	53.31
2:07pm	1440	20.0	33	34	10.2	0.01408	0.001	0.00	1.01	30	60.6	48.46

9.5	100
4.75	99.96
2.36	99.67
2.00	99.60
1.18	98.97
0.600	97.22
0.425	94.71
0.300	91.47
0.150	86.55
0.075	79.97
0.028	67.85
0.018	63.00
0.011	61.39
0.008	59.77
0.005	58.16
0.003	53.31
0.001	48.46

Grain Size Accumulation Curve For Native Soil (Test pit #1)



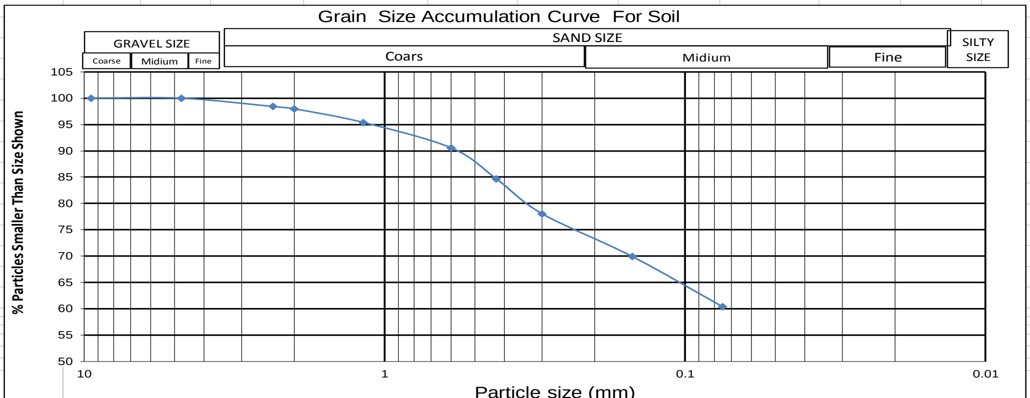
Grain size analysis test result for Test Pit #2

Befor wash Wt. dry sample= **1805.6** g

After wash Wt. dry sample= **172.6** g

SIEVE SIZE Alternate (Number)	SIEVE SIZE Standard (mm)	Weight of Sieve in (g)	Weight of Sieve + Weight Retained Sample (g)	Weight Retained Sample (g)	Commulative %Retained	Commulative % Coarser	Commulative % Passing
	9.5						100
4	4.75	1125.6	1126.1	0.5	0.03	0.03	99.97
10	2.36	1025.5	1053.9	28.4	1.57	1.57	98.40
	2.00	990.6	998.8	8.2	0.45	0.45	97.95
20	1.18	1013.2	1031.3	18.1	1.00	2.58	95.37
40	0.600	990.5	1023.4	32.9	1.82	4.85	90.52
60	0.425	855.5	874.0	18.5	1.02	5.88	84.64
140	0.300	755.5	768.6	13.1	0.73	6.60	78.04
100	0.150	555.5	583.5	28	1.55	8.15	69.89
200	0.075	435.3	460.7	25.4	1.41	9.56	60.33
			Pan	1633	90.44	100.00	0.00
			Total	1805.6			

Gravel,%	0.03
Sand,%	39.64
Fine,%	60.33

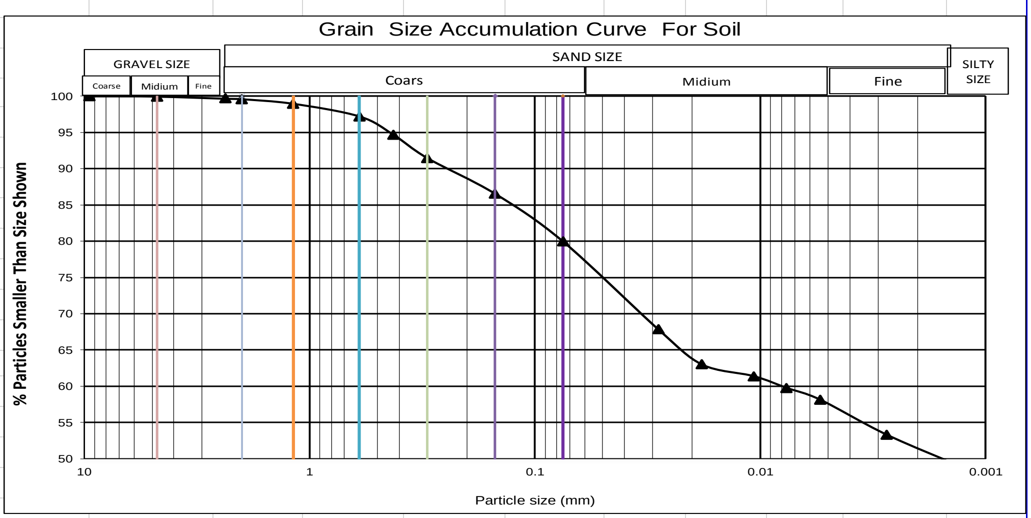


HYDROMETER ANALYSIS

Weight of soil sample gm= 50 Zero Correction = 3 Meniscus Correction =

Time	Elapsed time (min)	Temp .(°C)	Actual hydro. Rdg. R_a	Hydro. Corr. for Meniscus	L from Table 1(cm)	K from Table 2	D(mm)	CT from Table 3	a from Table 4	Corr. Hydr. Rdg. R_c	% finer,P	% Adjusted finer P_A
2:56pm	2	20.0	41	42	9.6	0.01408	0.031	0.00	1.01	38	76.76	46.31
	5	20.0	40	41	9.7	0.01408	0.020	0.00	1.01	37	74.74	45.09
	15	20.0	39	40	9.9	0.01408	0.011	0.00	1.01	36	72.72	43.87
	30	20.0	37	38	10.2	0.01408	0.008	0.00	1.01	34	68.68	41.43
	60	20.0	36	37	10.4	0.01408	0.006	0.00	1.01	33	66.66	40.22
	240	20.0	33	34	10.9	0.01408	0.003	0.00	1.01	30	60.6	36.56
2:56pm	1440	20.0	31	32	11.2	0.01408	0.001	0.00	1.01	28	56.56	34.12

9.5	100
4.75	99.966
2.36	99.6446
2.00	99.556
1.18	98.8331
0.600	96.9152
0.425	94.1746
0.300	90.8526
0.150	86.0887
0.075	79.955
0.031	67.0265
0.020	62.9886
0.011	61.3735
0.008	58.951
0.006	57.3359
0.003	52.4907
0.001	48.4528



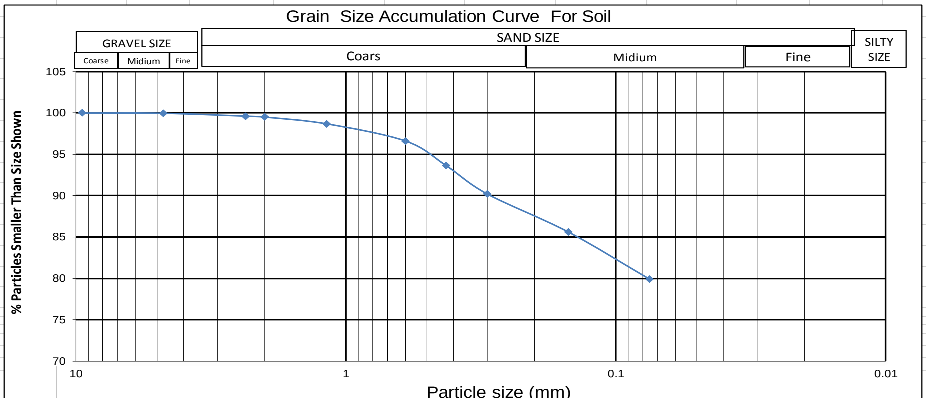
Grain size analysis test result for Test Pit #3

Befor wash Wt. dry sample= 1303.5 g

After wash Wt. dry sample= 74.2 g

SIEVE SIZE Alternate (Number)	SIEVE SIZE Standard (mm)	Weight of Sieve in (g)	Weight of Sieve + Weight Retained Sample (g)	Weight Retained Sample (g)	Commulative %Retained	Commulative % Coarser	Commulative % Passing
	9.5						100
4	4.75	1125.6	1126.0	0.4	0.03	0.03	99.97
10	2.36	1025.5	1030.0	4.5	0.35	0.35	99.62
	2.00	990.6	992.1	1.5	0.12	0.12	99.51
20	1.18	1013.2	1019.3	6.1	0.47	0.81	98.70
40	0.600	990.5	1005.6	15.1	1.16	2.09	96.61
60	0.425	855.5	867.0	11.5	0.88	2.97	93.64
140	0.300	755.5	761.2	5.7	0.44	3.41	90.23
100	0.150	555.5	571.1	15.6	1.20	4.60	85.63
200	0.075	435.3	449.5	14.2	1.09	5.69	79.94
			Pan	1229.3	94.31	100.00	0.00
			Total	1303.5			

Gravel,%	0.03
Sand,%	20.03
Fine,%	79.94

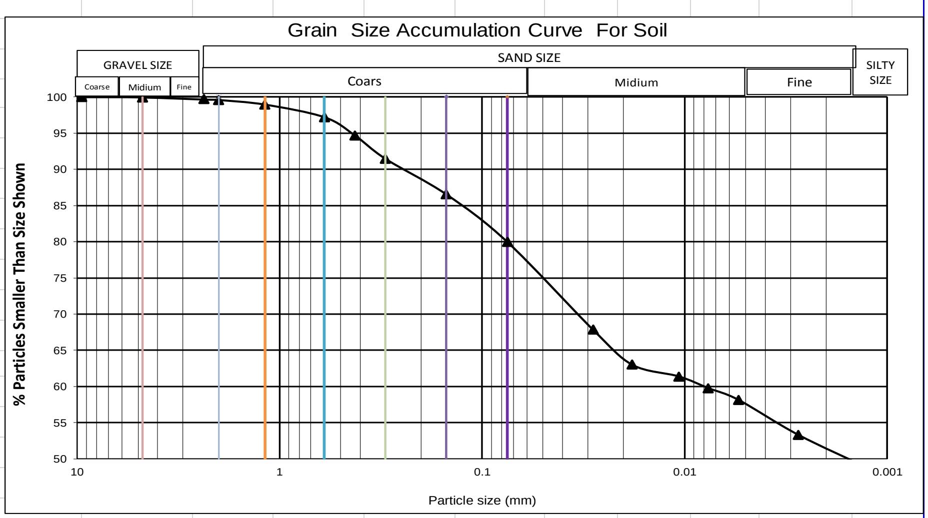


HYDROMETER ANALYSIS

Weight of soil sample gm= 50 Zero Correction = 3 Meniscus Correction = 1

Time	Elapsed time (min)	Temp .(°C)	Actual hydro. Rdg. R_a	Hydro. Corr. for Meniscus	L from Table 1(cm)	K from Table 2	D(mm)	CT from Table 3	a from Table 4	Corr. Hydr. Rdg. R_c	% finer,P	% Adjusted finer P_A
2.56pm	2	20.0	44	45	8.9	0.01408	0.030	0.00	1.01	41	82.82	66.21
	5	20.0	42	43	9.2	0.01408	0.019	0.00	1.01	39	78.78	62.98
	15	20.0	41	42	9.4	0.01408	0.011	0.00	1.01	38	76.76	61.36
	30	20.0	39	40	9.7	0.01408	0.008	0.00	1.01	36	72.72	58.13
	60	20.0	38	39	9.9	0.01408	0.006	0.00	1.01	35	70.7	56.52
	240	20.0	35	36	10.4	0.01408	0.003	0.00	1.01	32	64.64	51.67
2:56pm	1440	20.0	33	33	10.7	0.01408	0.001	0.00	1.01	30	60.6	48.44

9.5	100
4.75	99.97
2.36	99.62
2.00	99.51
1.18	98.70
0.600	96.61
0.425	93.64
0.300	90.23
0.150	85.63
0.075	79.94
0.030	66.21
0.019	62.98
0.011	61.36
0.008	58.13
0.006	56.52
0.003	51.67
0.001	48.44



SHRINKAGE LIMIT DETERMINATION DATA SHEET

Depth(m)	0.2-0.6	0.2-0.6	0.2-0.6
Trial No.			
Container i.d.	7	8	9
Mass of empty can (g) = A	19.8	19.7	19.9
Mass of can + wet soil pat (g) = B	44.6	45.1	44.7
Mass of can + dry soil pat (g) = C	33.4	33.8	33.2
Volume of wet soil pat (cm^3) = D	16.4	16.96	16.53
Mass of dry soil pat (g) = E = C - A	**13.6**	**14.1**	**13.3**
Initial moisture content (%) = F = (B-C)*100/E	**82.4**	**80.1**	**86.5**
Determination of Density of Water			
Mass of water at testing temperature (25.1°c) (g)=P	24.76	24.77	24.8
Volume of water at testing temperature (26.4°c) (cm^3)=Q	25	25	25
Density of water, ρ_w, at 25.5°c (g/cm^3)=R=P/Q	**0.9904**	**0.9908**	**0.992**
Average Density (g/cm^3)	**0.9911**		
Determination of Dry Soil Volume by Wax Method			
Specific gravity of wax = G	0.88	0.88	0.88
Mass of wax-coated dry soil pat in air (g) = H	23.3	23.7	23.6
Mass of wax-coated dry soil pat in water (g) = I	6.2	6.6	6.2
Mass of wax (g) = J = H-E	9.7	9.6	10.3
Volume of dry soil pat and wax (cm^3) = K = (H-I)/ρ_w	17.20	17.3	17.6
Volume of wax (cm^3) = L = J/(Gρ_w)	11.10	11	11.8
Volume of dry soil pat (cm^3) = M = K-L	6.10	6.3	5.8
Shrinkage Limit (%) = SL=F-[(D-M)ρ_w/E]*100	**7**	**5.1**	**6.4**
Average Shrinkage Limit (%)			
Shrinkage Ratio=N=E/(M*ρ_w)	**2.20**	**2.3**	**2.3**
Volumetric Shrinkage = N*(F-SL)	**169**	**170**	**186**

SPECIFIC GRAVITY OF SOIL TEST DATA SHEET

		Test Pit #1		Test Pit #2		Test Pit #3	
		1	2	1	2	1	2
1	Bottle Number	F	T	A	G	K	T
2	Weight of Bottle	47.7	42.4	44.1	40.6	45.1	39.9
3	Weight of sample	20.1	20.1	20.1	20.1	20.1	20.1
4	Weight of Bottle + Sample	67.8	62.5	64.2	60.7	65.2	60
5	Wieght of Bottle with full of Water	147.4	142.6	144.2	140.7	139.7	144.6
6	Wieght of Bottle +Sample + Water	159.6	154.8	156.4	153	152.3	157.1
7	Volume of Sample (3+5-6)	7.9	7.9	7.9	7.8	7.5	7.6
8	Specific Gravity 3/7)	2.544	2.544	2.544	2.577	2.680	2.645
	Average Specific Gravity ((A+B)/2)	2.54		2.56		2.66	

FREE SWELL TEST METHOD DATA SHEET	
Free Swell Test Data for Test Pit #1	
Original Volume of dry sample (Vi) ml	**10**
Final Volume (Vf) ml	**21**
Free Swell (%)=(Vf-Vi)*100/Vi	**110**
Free Swell Test Data for Test Pit #2	
Original Volume of dry sample (Vi) ml	**10**
Final Volume (Vf) ml	**22**
Free Swell (%)=(Vf-Vi)*100/Vi	**120**
Free Swell Test Data for Test Pit #3	
Original Volume of dry sample (Vi) ml	**10**
Final Volume (Vf) ml	**21**
Free Swell (%)=(Vf-Vi)*100/Vi	**110**

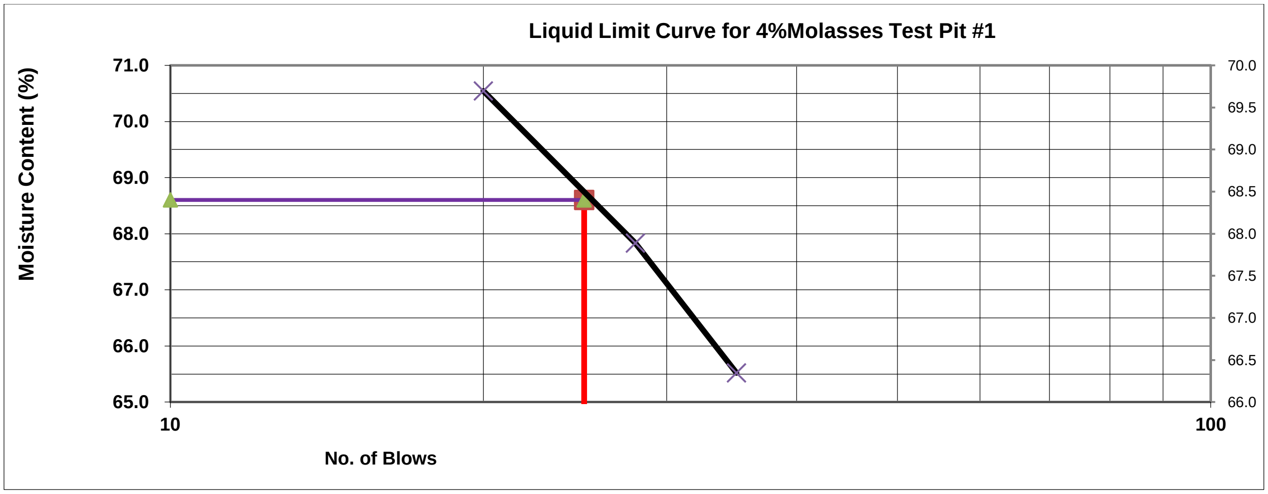
pH Test Measurement Data Sheet				
Type of test	**Test Pit #1**			**Average**
	1	**2**	**3**	
pH	**8.17**	**8.18**	**8.19**	**8.18**
Type of test	**Test Pit #2**			**Average**
	1	**2**	**3**	
pH	**8.21**	**8.20**	**8.19**	**8.20**
Type of test	**Test Pit #3**			**Average**
	1	**2**	**3**	
pH	**8.2**	**8.21**	**8.22**	**8.21**

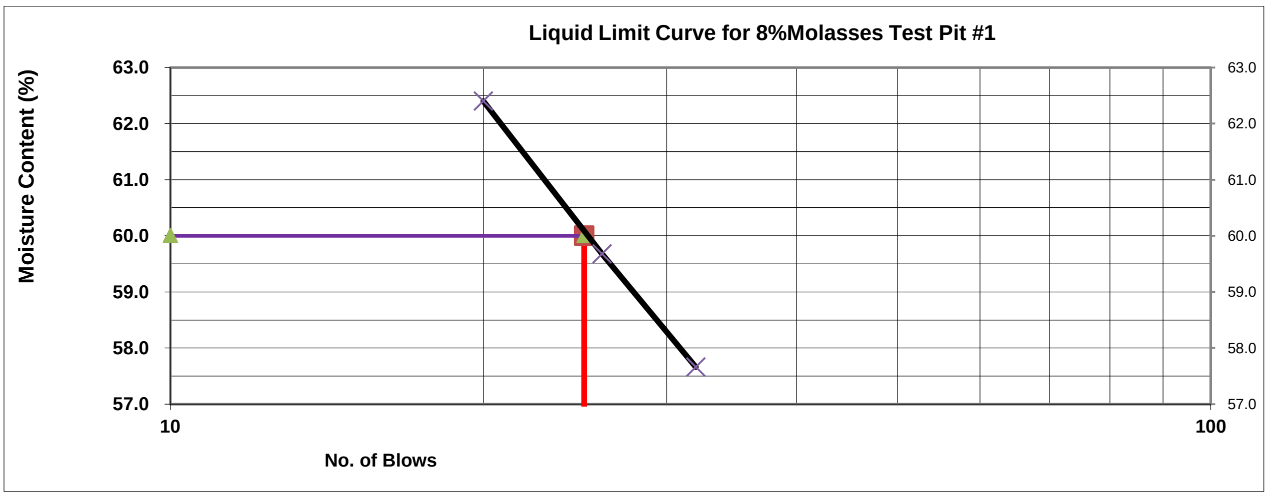
Appendix-2: Laboratory Test Results of Stabilized Soils

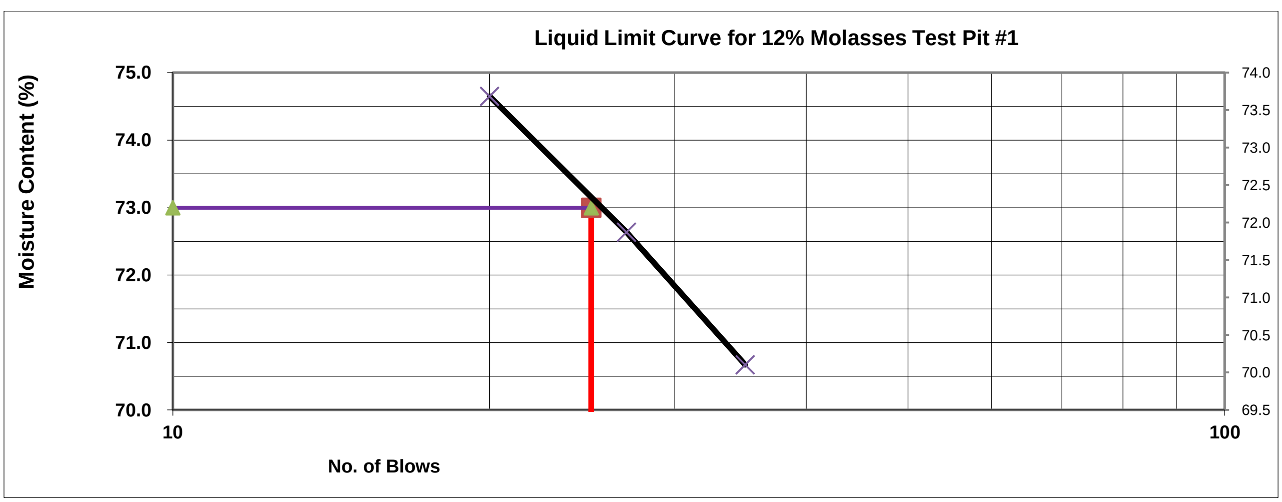
4% Molasses 14 days curing (Test #1)					
	Liquid Limit			**Plastic Limit**	
No. of Blows	35	28	20		
Wt. of cont. + wet soil (g) = (w_1)	24.50	26.50	25.20	15.00	15.40
Wt. of cont. + dry soil (g.) = (w_2)	17.60	19.10	18.30	12.90	13.10
Wt. of container (g.) = (w_3)	7.20	8.20	8.40	7.40	7.00
Mass of moisture (g.) (w_1-w_2) = x	6.90	7.40	6.90	2.10	2.30
Wt. of dry soil (g.) (w_2-w_3) = y	10.40	10.90	9.90	5.50	6.10
Moisture Content (%) = (100x/y)	66.3	67.9	69.7	38.2	37.7
	69			**38**	
	Plasticity Index			**31**	

8% Molasses 14 days curing (Test #1)					
	Liquid Limit			**Plastic Limit**	
No. of Blows	32	26	20		
Wt. of cont. + wet soil (g) = (w_1)	26.80	28.10	29.20	18.7	17.5
Wt. of cont. + dry soil (g.) = (w_2)	20.40	20.70	21.40	16.30	15.40
Wt. of container (g.) = (w_3)	9.30	8.30	8.90	8.90	8.90
Mass of moisture (g.) (w_1-w_2) = x	6.40	7.40	7.80	2.40	2.10
Wt. of dry soil (g.) (w_2-w_3) = y	11.10	12.40	12.50	7.40	6.50
Moisture Content (%) = (100x/y)	57.7	59.7	62.4	32.4	32.3
	60			**32**	
	Plasticity Index			**28**	

12% Molasses 14 days curing (Test #1)					
	Liquid Limit			**Plastic Limit**	
No. of Blows	35	27	20		
Wt. of cont. + wet soil (g) = (w_1)	23.10	25.10	24.60	16.2	14.5
Wt. of cont. + dry soil (g.) = (w_2)	16.30	18.20	17.60	14.20	12.40
Wt. of container (g.) = (w_3)	6.60	8.60	8.10	9.00	7.00
Mass of moisture (g.) (w_1-w_2) = x	6.80	6.90	7.00	2.00	2.10
Wt. of dry soil (g.) (w_2-w_3) = y	9.70	9.60	9.50	5.20	5.40
Moisture Content (%) = (100x/y)	70.1	71.9	73.7	38.5	38.9
	73			**39**	
	Plasticity Index			**34**	


Liquid Limit Curve for 4%Molasses Test Pit #1
Moisture Content (%)
71.0
70.0
69.0
68.0
67.0
66.0
65.0
70.0
69.5
69.0
68.5
68.0
67.5
67.0
66.5
66.0
10
100
No. of Blows

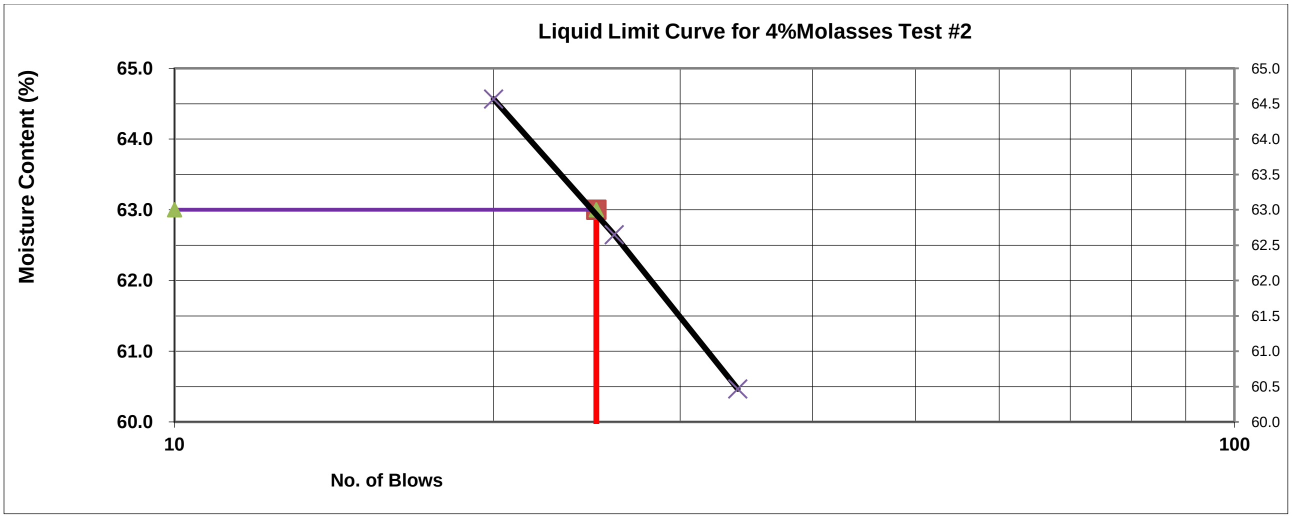

Liquid Limit Curve for 8%Molasses Test Pit #1
Moisture Content (%)
63.0
62.0
61.0
60.0
59.0
58.0
57.0
63.0
62.0
61.0
60.0
59.0
58.0
57.0
10
100
No. of Blows

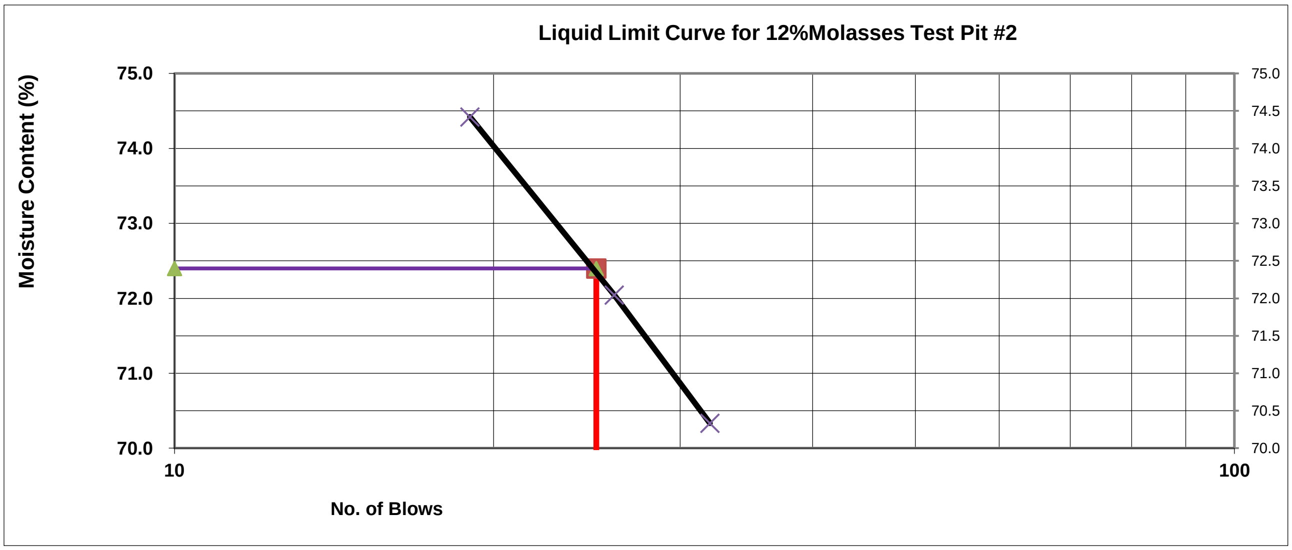

Liquid Limit Curve for 12% Molasses Test Pit #1
Moisture Content (%)
75.0
74.0
73.0
72.0
71.0
70.0
74.0
73.5
73.0
72.5
72.0
71.5
71.0
70.5
70.0
69.5
10
100
No. of Blows

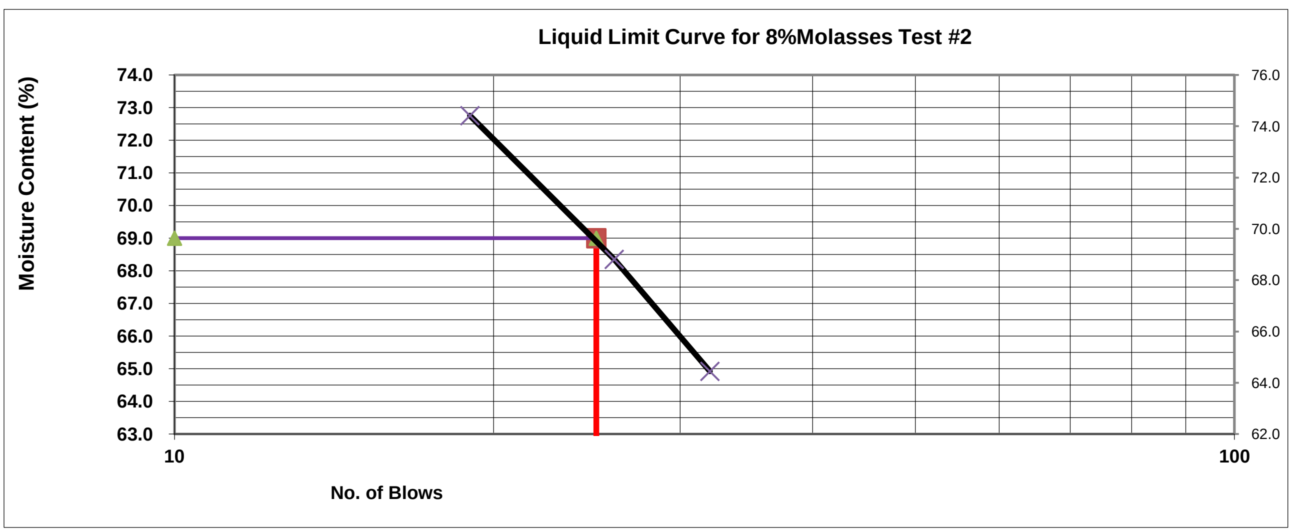
4% Molasses 14 days curing (Test #2)					
	Liquid Limit			**Plastic Limit**	
No. of Blows	34	26	20		
Wt. of cont. + wet soil (g) = (w_1)	20.60	21.70	28.20	15.30	15.50
Wt. of cont. + dry soil (g.) = (w_2)	15.40	16.50	20.00	13.70	13.80
Wt. of container (g.) = (w_3)	6.80	8.20	7.30	8.80	8.80
Mass of moisture (g.) (w_1-w_2) = x	5.20	5.20	8.20	1.60	1.70
Wt. of dry soil (g.) (w_2-w_3) = y	8.60	8.30	12.70	4.90	5.00
Moisture Content (%) = (100x/y)	60.5	62.7	64.6	32.7	34.0
	63			**33**	
	Plasticity Index			**30**	

8% Molasses 14 days curing (Test #2)					
	Liquid Limit			**Plastic Limit**	
No. of Blows	32	26	19		
Wt. of cont. + wet soil (g) = (w_1)	23.00	25.00	21.60	14	14
Wt. of cont. + dry soil (g.) = (w_2)	17.20	18.60	15.20	12.50	12.50
Wt. of container (g.) = (w_3)	8.20	9.30	6.60	8.80	8.70
Mass of moisture (g.) (w_1-w_2) = x	5.80	6.40	6.40	1.50	1.50
Wt. of dry soil (g.) (w_2-w_3) = y	9.00	9.30	8.60	3.70	3.80
Moisture Content (%) = (100x/y)	64.4	68.8	74.4	40.5	39.5
	69			**40**	
	Plasticity Index			**29**	

12% Molasses 14 days curing (Test #2)					
	Liquid Limit			**Plastic Limit**	
No. of Container					
No. of Blows	32	26	19		
Wt. of cont. + wet soil (g) = (w_1)	23.53	25.30	21.60	14	14
Wt. of cont. + dry soil (g.) = (w_2)	17.20	18.60	15.20	12.55	12.60
Wt. of container (g.) = (w_3)	8.20	9.30	6.60	8.90	8.90
Mass of moisture (g.) (w_1-w_2) = x	6.33	6.70	6.40	1.45	1.40
Wt. of dry soil (g.) (w_2-w_3) = y	9.00	9.30	8.60	3.65	3.70
Moisture Content (%) = (100x/y)	70.3	72.0	74.4	39.7	37.8
	72			**39**	
	Plasticity Index			**34**	


Liquid Limit Curve for 4%Molasses Test #2
Moisture Content (%)
65.0
64.0
63.0
62.0
61.0
60.0
64.5
63.5
62.5
61.5
60.5
10
100
No. of Blows

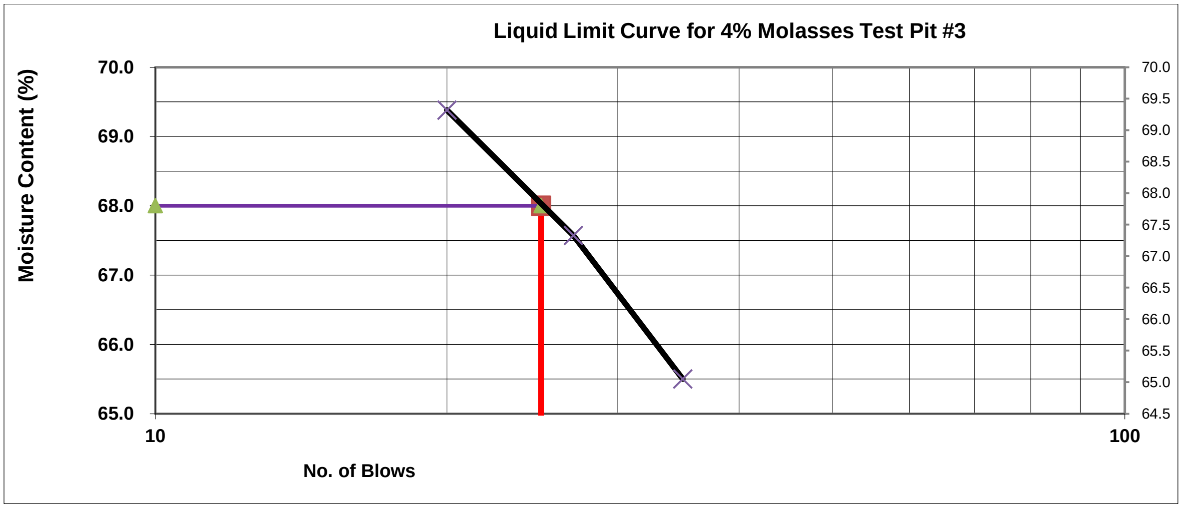

Liquid Limit Curve for 12%Molasses Test Pit #2
Moisture Content (%)
75.0
74.0
73.0
72.0
71.0
70.0
74.5
73.5
72.5
71.5
70.5
10
100
No. of Blows

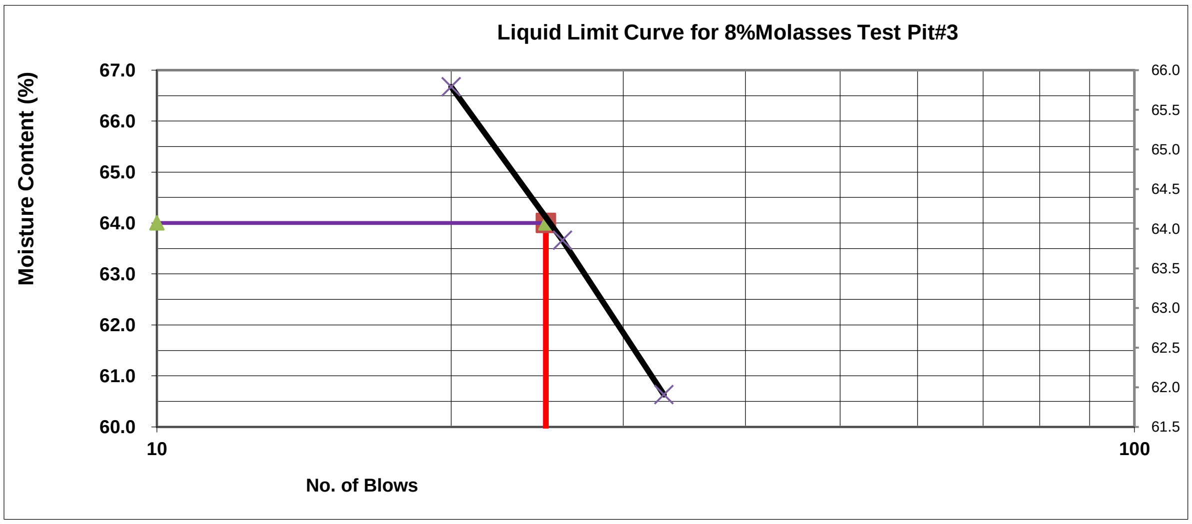

Liquid Limit Curve for 8%Molasses Test #2
Moisture Content (%)
74.0
73.0
72.0
71.0
70.0
69.0
68.0
67.0
66.0
65.0
64.0
63.0
76.0
74.0
72.0
70.0
68.0
66.0
64.0
62.0
10
100
No. of Blows

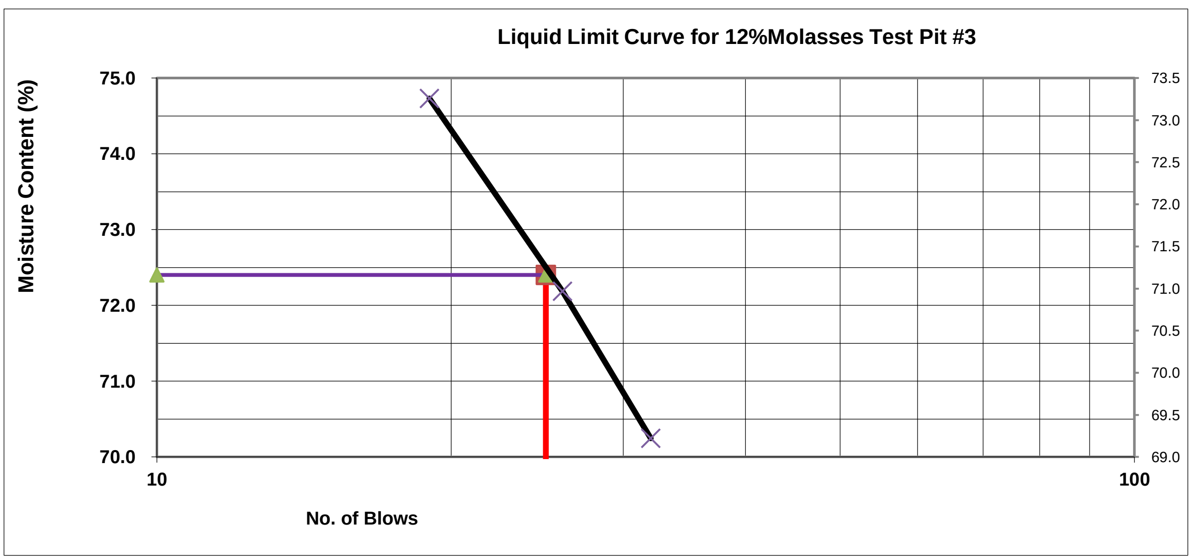
4% Molasses 14 days curing (Test #3)					
	Liquid Limit			**Plastic Limit**	
No. of Blows	35	27	20		
Wt. of cont. + wet soil (g) = (w_1)	25.60	25.70	23.80	12.90	14.00
Wt. of cont. + dry soil (g.) = (w_2)	18.90	18.90	17.70	11.20	12.60
Wt. of container (g.) = (w_3)	8.60	8.80	8.90	6.80	9.00
Mass of moisture (g.) (w_1-w_2) = x	6.70	6.80	6.10	1.70	1.40
Wt. of dry soil (g.) (w_2-w_3) = y	10.30	10.10	8.80	4.40	3.60
Moisture Content (%) = (100x/y)	65.0	67.3	69.3	38.6	38.9
	68			**39**	
	Plasticity Index			**29**	

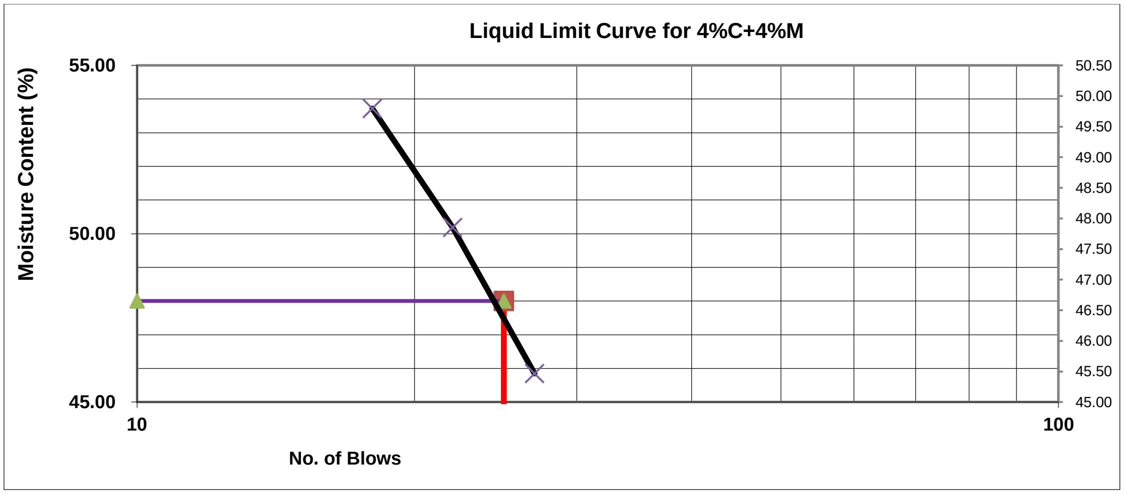
8% Molasses 14 days curing (Test #3)					
	Liquid Limit			**Plastic Limit**	
No. of Blows	33	26	20		
Wt. of cont. + wet soil (g) = (w_1)	22.80	21.80	21.10	13.7	14.4
Wt. of cont. + dry soil (g.) = (w_2)	17.60	16.50	16.10	12.40	13.00
Wt. of container (g.) = (w_3)	9.20	8.20	8.50	8.90	9.30
Mass of moisture (g.) (w_1-w_2) = x	5.20	5.30	5.00	1.30	1.40
Wt. of dry soil (g.) (w_2-w_3) = y	8.40	8.30	7.60	3.50	3.70
Moisture Content (%) = (100x/y)	61.9	63.9	65.8	37.1	37.8
	64			**37**	
	Plasticity Index			**27**	

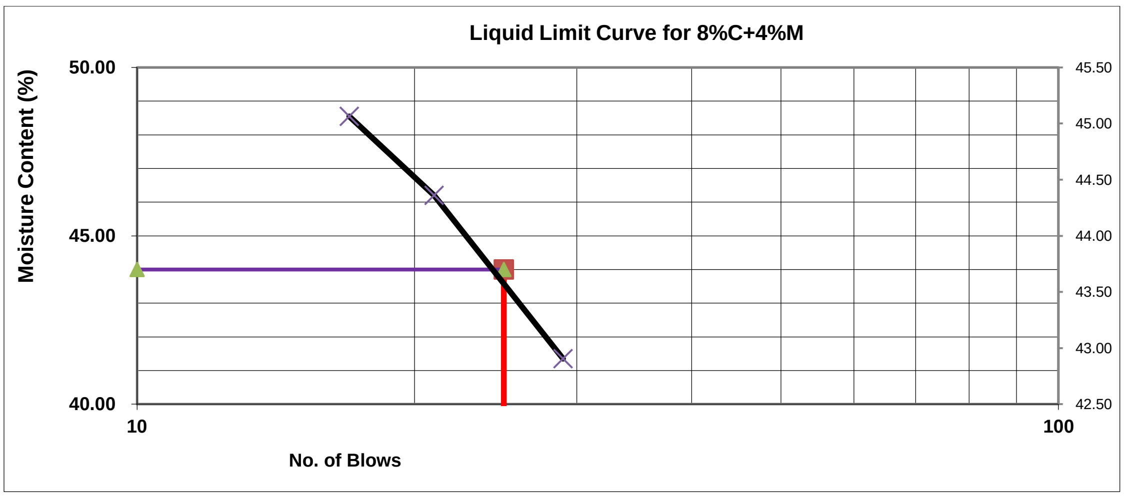
12% Molasses 14 days curing (Test #3)					
	Liquid Limit			**Plastic Limit**	
No. of Container					
No. of Blows	32	26	19		
Wt. of cont. + wet soil (g) = (w_1)	23.43	25.20	21.50	14	14
Wt. of cont. + dry soil (g.) = (w_2)	17.20	18.60	15.20	12.50	12.50
Wt. of container (g.) = (w_3)	8.20	9.30	6.60	8.80	8.70
Mass of moisture (g.) (w_1-w_2) = x	6.23	6.60	6.30	1.50	1.50
Wt. of dry soil (g.) (w_2-w_3) = y	9.00	9.30	8.60	3.70	3.80
Moisture Content (%) = (100x/y)	69.2	71.0	73.3	40.5	39.5
	72			**40**	
	Plasticity Index			**32**	

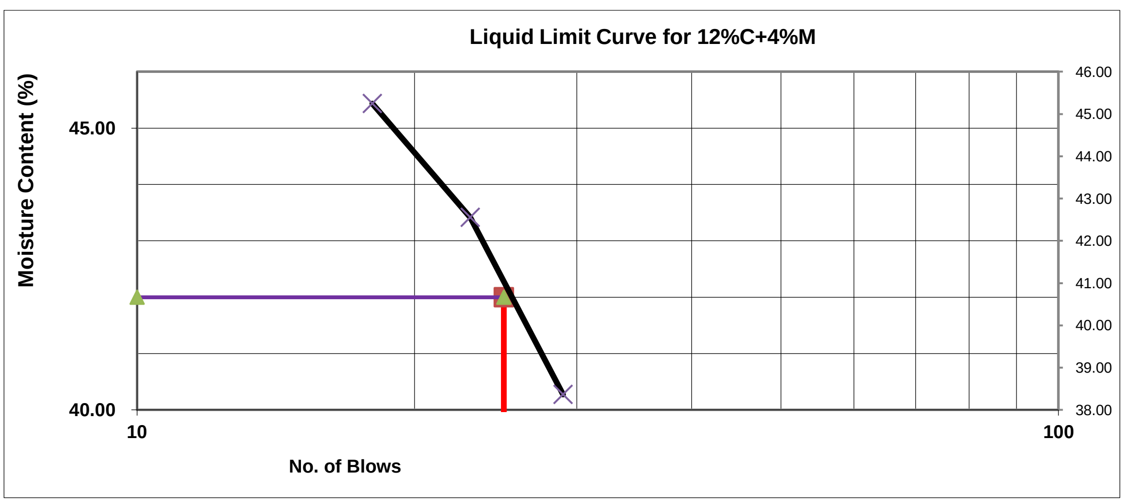

Liquid Limit Curve for 4% Molasses Test Pit #3
Moisture Content (%)
70.0
69.0
68.0
67.0
66.0
65.0
10
100
No. of Blows
70.0
69.5
69.0
68.5
68.0
67.5
67.0
66.5
66.0
65.5
65.0
64.5


Liquid Limit Curve for 8%Molasses Test Pit#3
Moisture Content (%)
67.0
66.0
65.0
64.0
63.0
62.0
61.0
60.0
10
100
No. of Blows
66.0
65.5
65.0
64.5
64.0
63.5
63.0
62.5
62.0
61.5


Liquid Limit Curve for 12%Molasses Test Pit #3
Moisture Content (%)
75.0
74.0
73.0
72.0
71.0
70.0
10
100
No. of Blows
73.5
73.0
72.5
72.0
71.5
71.0
70.5
70.0
69.5
69.0


Liquid Limit Curve for 4%C+4%M
Moisture Content (%)
55.00
50.00
45.00
50.50
50.00
49.50
49.00
48.50
48.00
47.50
47.00
46.50
46.00
45.50
45.00
10
100
No. of Blows


Liquid Limit Curve for 8%C+4%M
Moisture Content (%)
50.00
45.00
40.00
45.50
45.00
44.50
44.00
43.50
43.00
42.50
10
100
No. of Blows


Liquid Limit Curve for 12%C+4%M
Moisture Content (%)
45.00
40.00
46.00
45.00
44.00
43.00
42.00
41.00
40.00
39.00
38.00
10
100
No. of Blows

4 %Cement + 4% Molasses , 7days curing					
	Liquid Limit			**Plastic Limit**	
No. of Container	B1	F	Q	W4	B3
No. of Blows	34	26	20		
Wt. of cont. + wet soil (g) = (w_1)	56.71	51.45	49.49	15.11	15.22
Wt. of cont. + dry soil (g.) = (w_2)	45.48	41.32	39.72	13.63	13.71
Wt. of container (g.) = (w_3)	20.78	20.15	20.10	8.43	8.51
Mass of moisture (g.) (w_1-w_2) = x	11.23	10.13	9.77	1.48	1.51
Wt. of dry soil (g.) (w_2-w_3) = y	24.70	21.17	19.62	5.20	5.20
Moisture Content (%) = (100x/y)	45.47	47.85	49.80	28.5	29.0
	48			**29**	
	Plasticity Index			**19**	

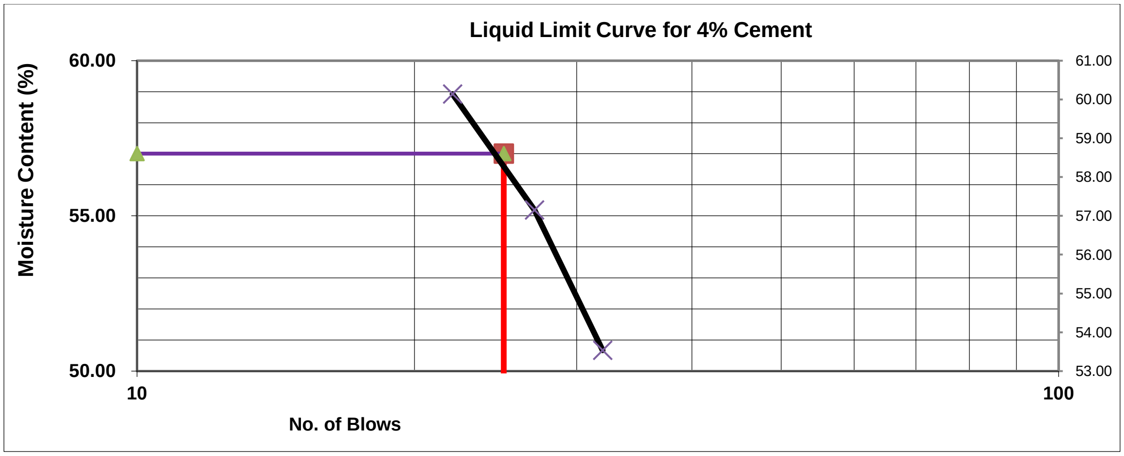
8 %Cement + 4% Molasses , 7days curing					
	Liquid Limit			**Plastic Limit**	
No. of Container	A	W9	D3	A5	W3
No. of Blows	29	21	17		
Wt. of cont. + wet soil (g) = (w_1)	50.36	45.41	49.35	11.88	12.03
Wt. of cont. + dry soil (g.) = (w_2)	40.26	36.48	39.08	11.05	11.15
Wt. of container (g.) = (w_3)	16.72	16.35	16.29	8.11	7.79
Mass of moisture (g.) (w_1-w_2) = x	10.10	8.93	10.27	0.83	0.88
Wt. of dry soil (g.) (w_2-w_3) = y	23.54	20.13	22.79	2.94	3.36
Moisture Content (%) = (100x/y)	42.91	44.36	45.06	28.2	26.2
	44			**27**	
	Plasticity Index			**17**	

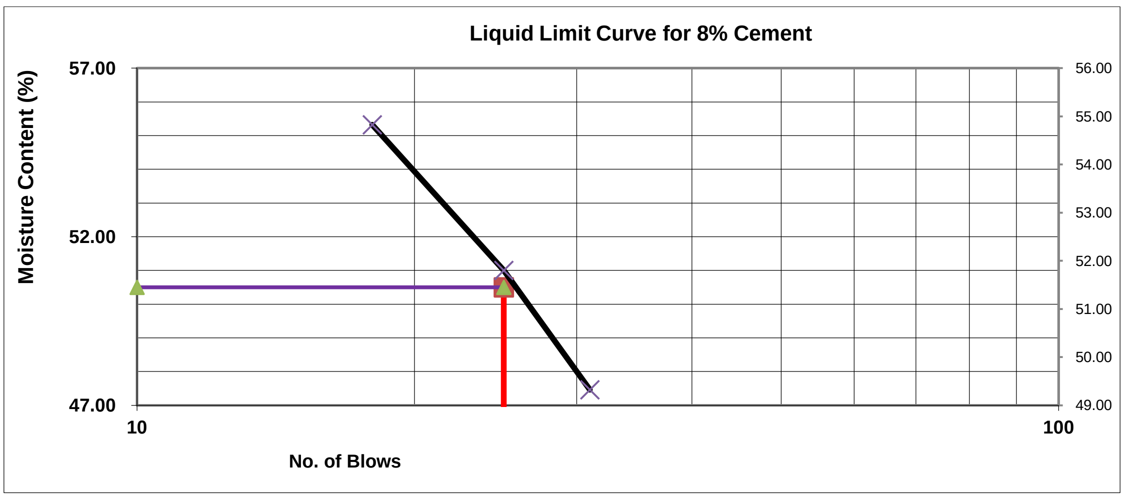
12 %Cement + 4% Molasses , 7days curing					
	Liquid Limit			**Plastic Limit**	
No. of Container	W	H	B	X	C
No. of Blows	31	23	18		
Wt. of cont. + wet soil (g) = (w_1)	50.12	48.59	53.40	13.06	14.02
Wt. of cont. + dry soil (g.) = (w_2)	41.66	40.10	43.45	12.00	12.90
Wt. of container (g.) = (w_3)	19.61	20.15	21.46	7.85	8.28
Mass of moisture (g.) (w_1-w_2) = x	8.46	8.49	9.95	1.06	1.12
Wt. of dry soil (g.) (w_2-w_3) = y	22.05	19.95	21.99	4.15	4.62
Moisture Content (%) = (100x/y)	38.37	42.56	45.25	25.54	24.24
	40			**25**	
	Plasticity Index			**15**	

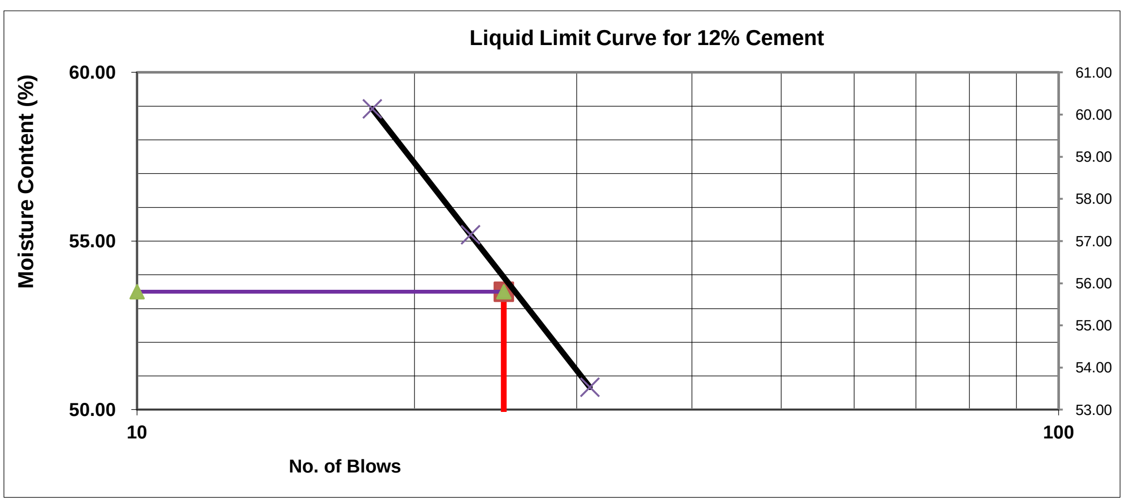
4 %Cement , 7days curing					
	Liquid Limit			**Plastic Limit**	
No. of Container	B1	F	Q	W4	B3
No. of Blows	32	27	22		
Wt. of cont. + wet soil (g) = (w_1)	51.14	41.91	45.00	15.39	15.21
Wt. of cont. + dry soil (g.) = (w_2)	40.32	34.00	36.16	13.75	13.09
Wt. of container (g.) = (w_3)	20.11	20.16	21.46	7.79	7.86
Mass of moisture (g.) (w_1-w_2) = x	10.82	7.91	8.84	1.64	2.12
Wt. of dry soil (g.) (w_2-w_3) = y	20.21	13.84	14.70	5.96	5.23
Moisture Content (%) = (100x/y)	53.54	57.15	60.14	27.5	40.5
	58			**34.0**	
	Plasticity Index			**24**	

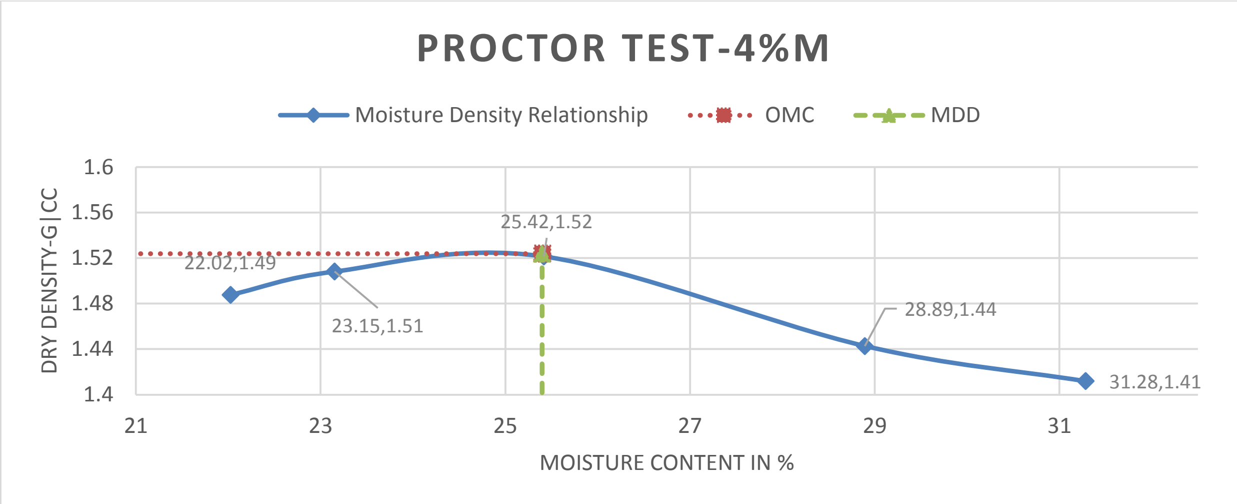
8 %Cement , 7days curing					
	Liquid Limit			**Plastic Limit**	
No. of Container	A	W9	D3	A5	W3
No. of Blows	33	29	20		
Wt. of cont. + wet soil (g) = (w_1)	50.14	40.91	44.00	15.09	15.00
Wt. of cont. + dry soil (g.) = (w_2)	40.32	34.00	36.16	13.75	13.09
Wt. of container (g.) = (w_3)	20.41	20.66	21.86	7.89	7.96
Mass of moisture (g.) (w_1-w_2) = x	9.82	6.91	7.84	1.34	1.91
Wt. of dry soil (g.) (w_2-w_3) = y	19.91	13.34	14.30	5.86	5.13
Moisture Content (%) = (100x/y)	49.32	51.80	54.83	22.9	37.2
	50			**30**	
	Plasticity Index			**20**	

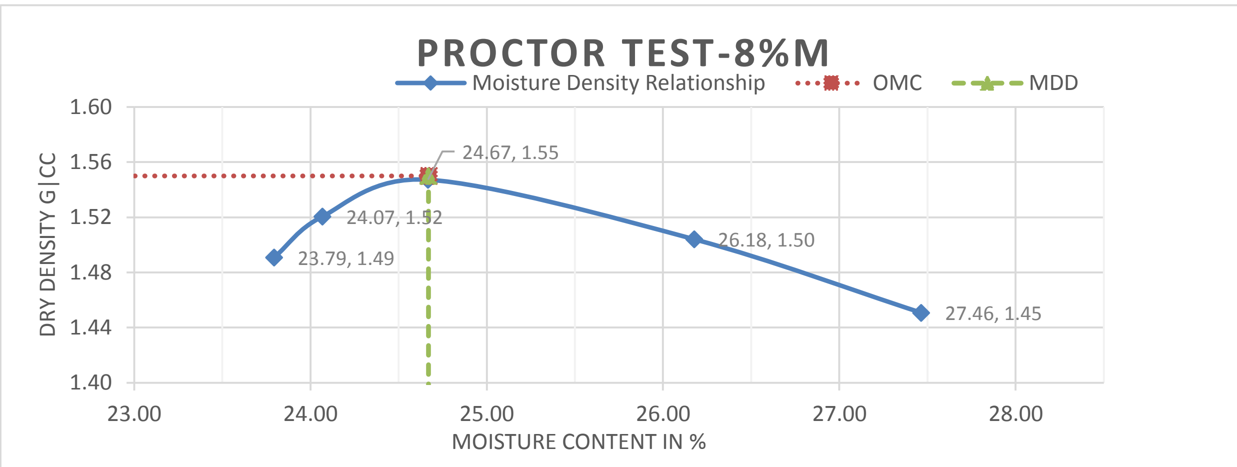
12 %Cement , 7days curing					
	Liquid Limit			**Plastic Limit**	
No. of Container	W	H	B	X	C
No. of Blows	33	25	19		
Wt. of cont. + wet soil (g) = (w_1)	51.14	41.91	45.00	15.29	15.26
Wt. of cont. + dry soil (g.) = (w_2)	40.32	34.00	36.16	13.65	13.29
Wt. of container (g.) = (w_3)	20.11	20.16	21.46	7.79	7.86
Mass of moisture (g.) (w_1-w_2) = x	10.82	7.91	8.84	1.64	1.97
Wt. of dry soil (g.) (w_2-w_3) = y	20.21	13.84	14.70	5.86	5.43
Moisture Content (%) = (100x/y)	53.54	57.15	60.14	27.99	36.28
	53			**32**	
	Plasticity Index			**21**	

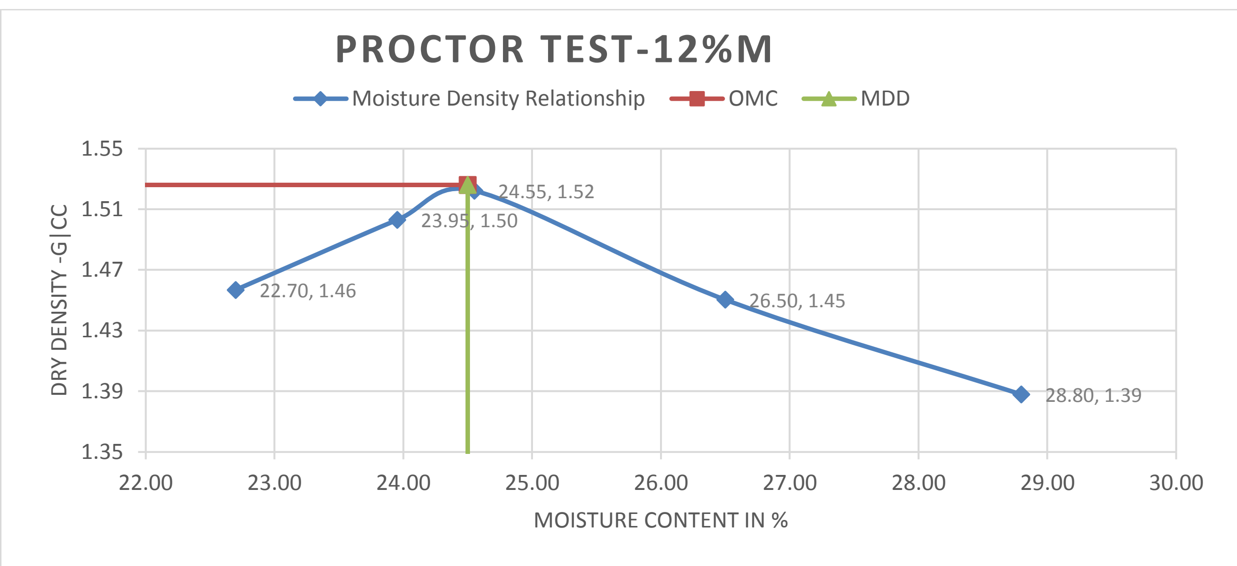

Liquid Limit Curve for 4% Cement
Moisture Content (%)
60.00
55.00
50.00
61.00
60.00
59.00
58.00
57.00
56.00
55.00
54.00
53.00
10
100
No. of Blows

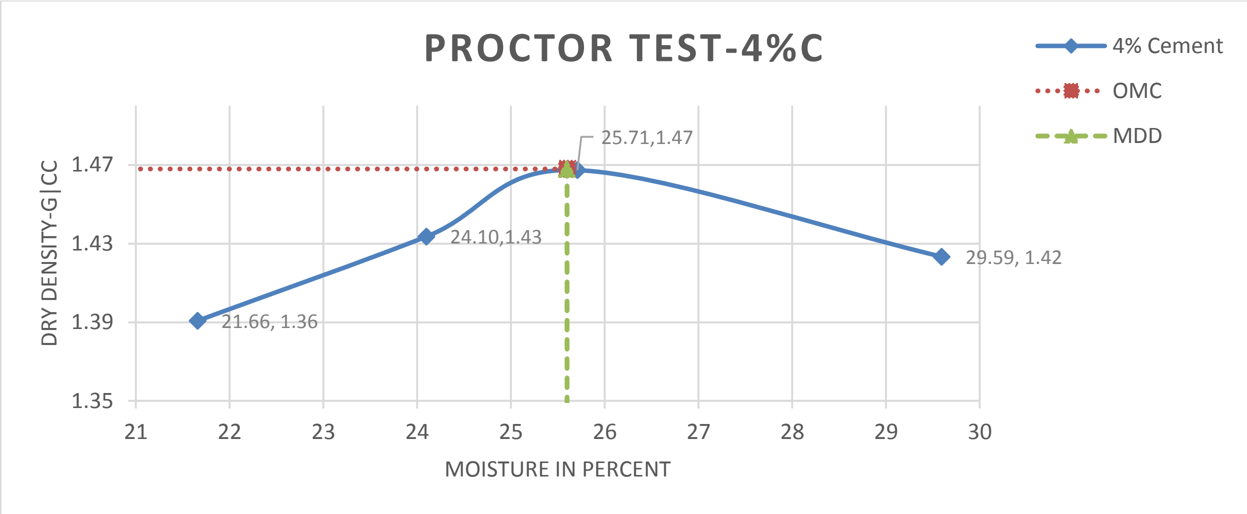

Liquid Limit Curve for 8% Cement
Moisture Content (%)
57.00
52.00
47.00
56.00
55.00
54.00
53.00
52.00
51.00
50.00
49.00
10
100
No. of Blows

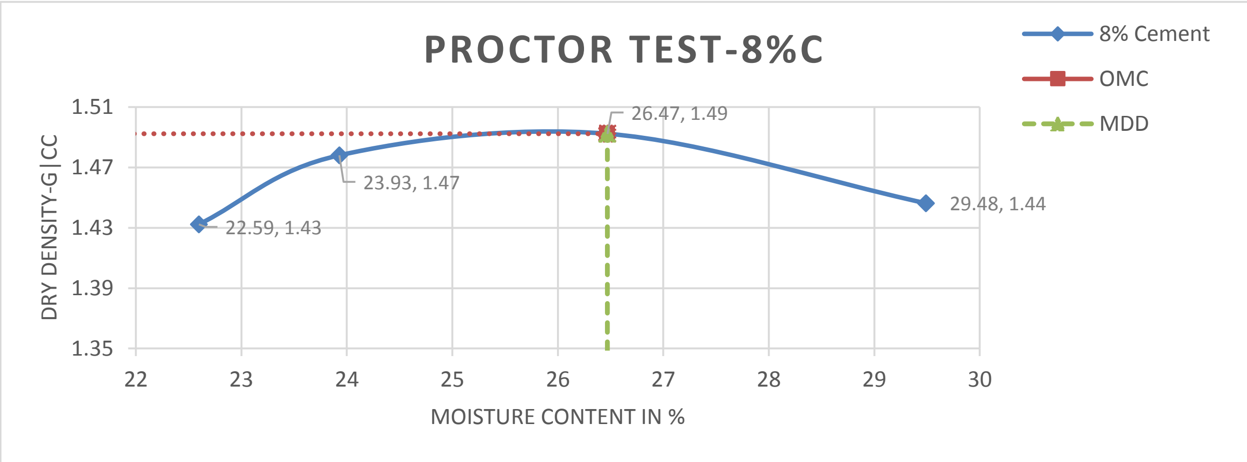

Liquid Limit Curve for 12% Cement
Moisture Content (%)
60.00
55.00
50.00
61.00
60.00
59.00
58.00
57.00
56.00
55.00
54.00
53.00
10
100
No. of Blows

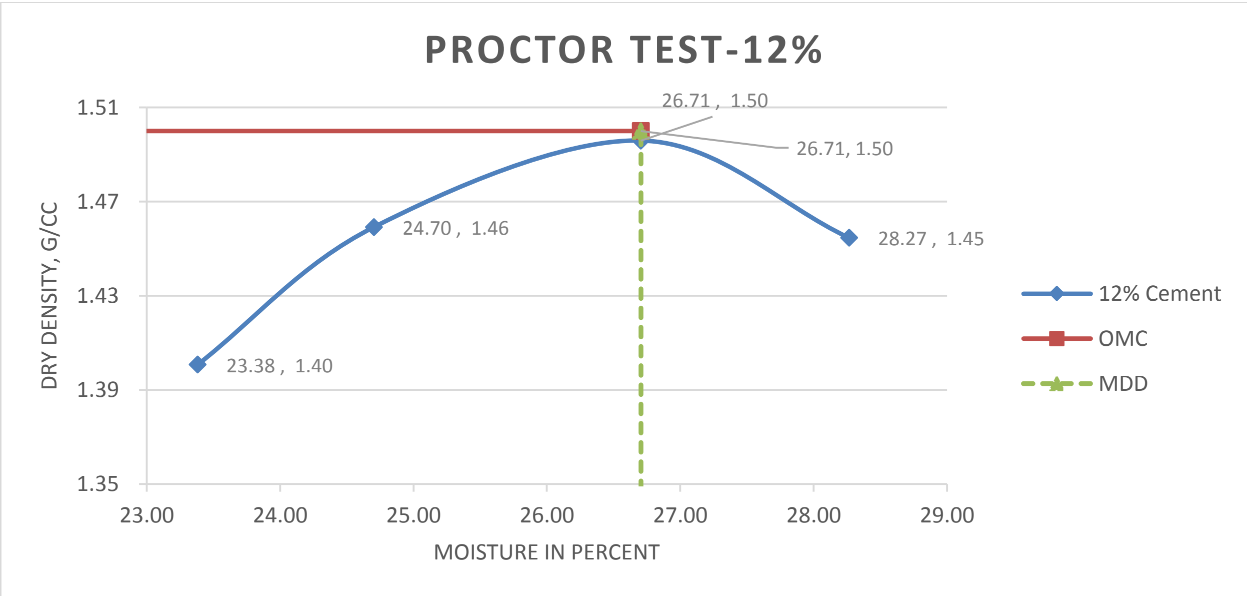

PROCTOR TEST-4%M
Moisture Density Relationship
OMC
MDD
DRY DENSITY-G|CC
1.6
1.56
1.52
1.48
1.44
1.4
25.42,1.52
22.02,1.49
23.15,1.51
28.89,1.44
31.28,1.41
21
23
25
27
29
31
MOISTURE CONTENT IN %

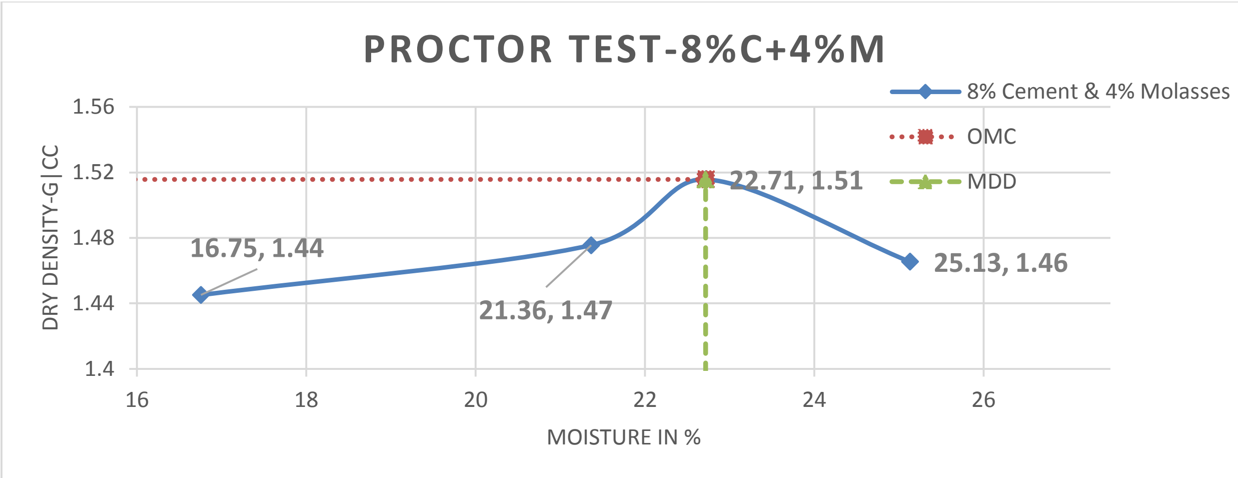

PROCTOR TEST-8%M
Moisture Density Relationship
OMC
MDD
DRY DENSITY G|CC
1.60
1.56
1.52
1.48
1.44
1.40
24.67, 1.55
24.07, 1.52
23.79, 1.49
26.18, 1.50
27.46, 1.45
23.00
24.00
25.00
26.00
27.00
28.00
MOISTURE CONTENT IN %

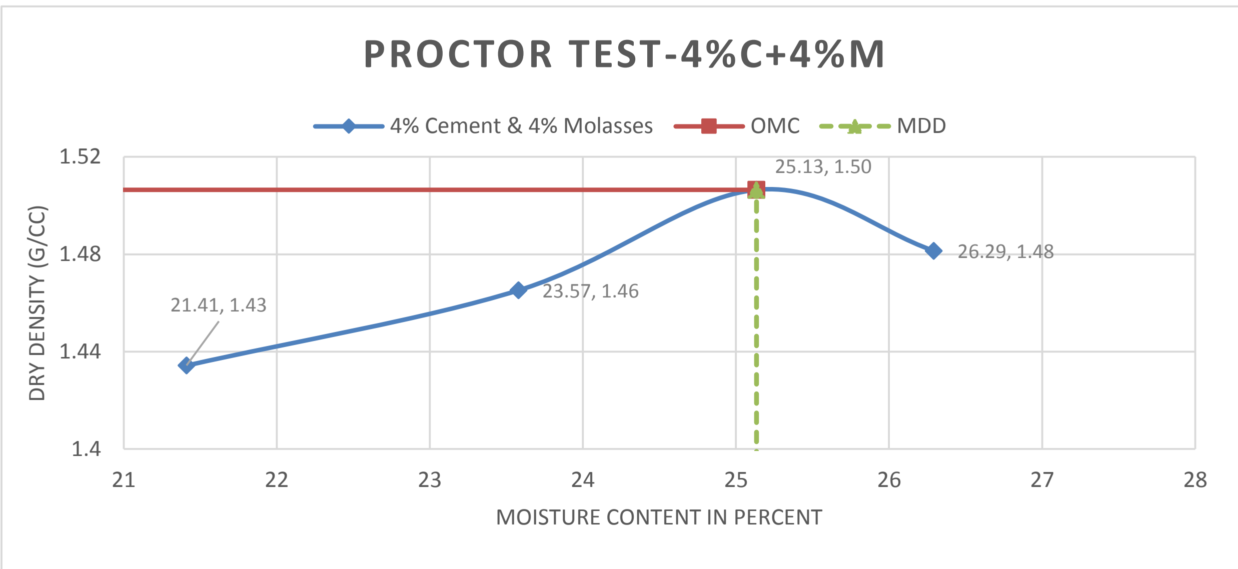

PROCTOR TEST-12%M
Moisture Density Relationship
OMC
MDD
DRY DENSITY -G|CC
1.55
1.51
1.47
1.43
1.39
1.35
24.55, 1.52
23.95, 1.50
22.70, 1.46
26.50, 1.45
28.80, 1.39
22.00
23.00
24.00
25.00
26.00
27.00
28.00
29.00
30.00
MOISTURE CONTENT IN %

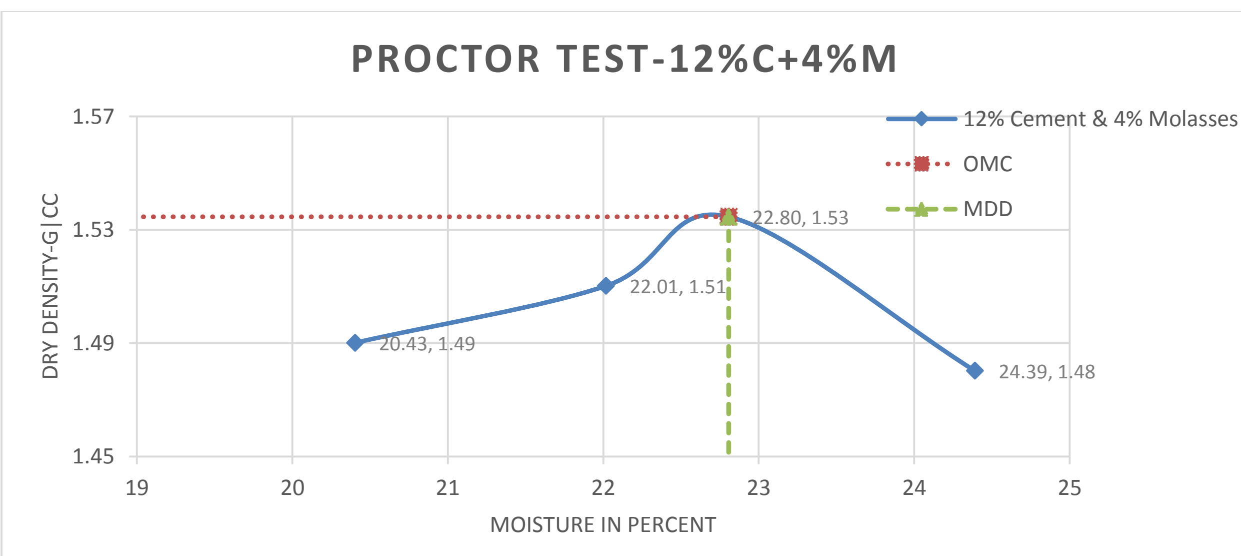

PROCTOR TEST-4%C
4% Cement
OMC
MDD
DRY DENSITY-G|CC
1.47
1.43
1.39
1.35
25.71,1.47
24.10,1.43
21.66, 1.36
29.59, 1.42
21
22
23
24
25
26
27
28
29
30
MOISTURE IN PERCENT


PROCTOR TEST-8%C
8% Cement
OMC
MDD
DRY DENSITY-G|CC
1.51
1.47
1.43
1.39
1.35
26.47, 1.49
23.93, 1.47
22.59, 1.43
29.48, 1.44
22
23
24
25
26
27
28
29
30
MOISTURE CONTENT IN %


PROCTOR TEST-12%
DRY DENSITY, G/CC
1.51
1.47
1.43
1.39
1.35
26.71 , 1.50
26.71, 1.50
24.70 , 1.46
28.27 , 1.45
23.38 , 1.40
12% Cement
OMC
MDD
23.00
24.00
25.00
26.00
27.00
28.00
29.00
MOISTURE IN PERCENT


PROCTOR TEST-8%C+4%M
8% Cement & 4% Molasses
OMC
MDD
DRY DENSITY-G|CC
MOISTURE IN %
1.56
1.52
1.48
1.44
1.4
16
18
20
22
24
26
16.75, 1.44
21.36, 1.47
22.71, 1.51
25.13, 1.46

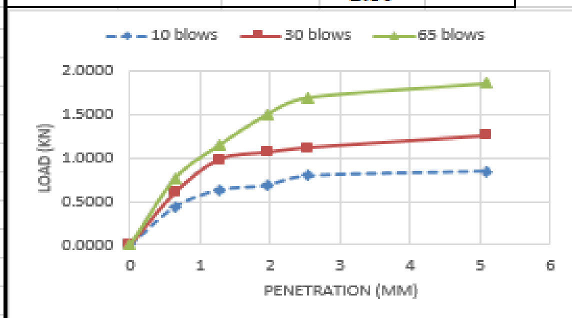

PROCTOR TEST-4%C+4%M
4% Cement & 4% Molasses
OMC
MDD
DRY DENSITY (G/CC)
MOISTURE CONTENT IN PERCENT
1.52
1.48
1.44
1.4
21
22
23
24
25
26
27
28
21.41, 1.43
23.57, 1.46
25.13, 1.50
26.29, 1.48

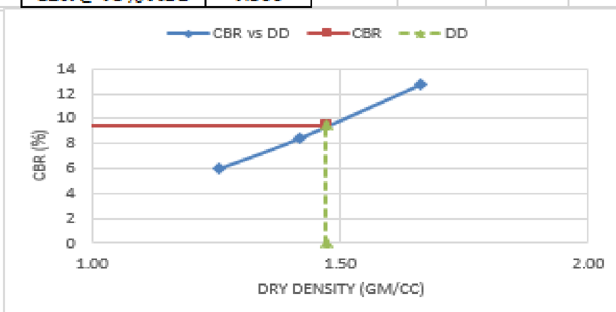

PROCTOR TEST-12%C+4%M
12% Cement & 4% Molasses
OMC
MDD
DRY DENSITY-G|CC
MOISTURE IN PERCENT
1.57
1.53
1.49
1.45
19
20
21
22
23
24
25
20.43, 1.49
22.01, 1.51
22.80, 1.53
24.39, 1.48

CBR PENETRATION TEST DATA-8%MOLASSES CURED FOR 14 DAYS AND SOAKED FOR 4 DAYS												
PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn	CBR %
0	0	0.0000			0	0			0	0		
0.64	16	0.4370			22	0.6008			28	0.7647		
1.27	23	0.6281			36	0.9832			42	1.1470		
1.96	25	0.6828			39	1.0651			55	1.5021		
2.54	29	0.7920	1	6	41	1.1197	1	8	62	1.6932	2	13
5.08	31	0.8466	1	4	46	1.2563	1	6	68	1.8571	2	9
7.62		0.0000				0.0000				0.0000		

SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	1165	1451	1335
RDG (AFTER) SOAKING)	1168	1454	1337
PERCENT SWELL	2.49	2.23	1.55
AVERAGE PERCENT SWELL:		2.09	

Dry Density	1.26	1.42	1.66
CBR	6	8	13
MDD	1.55		
95%MDD	1.4725		
CBR @ 95% MDD	9.500		

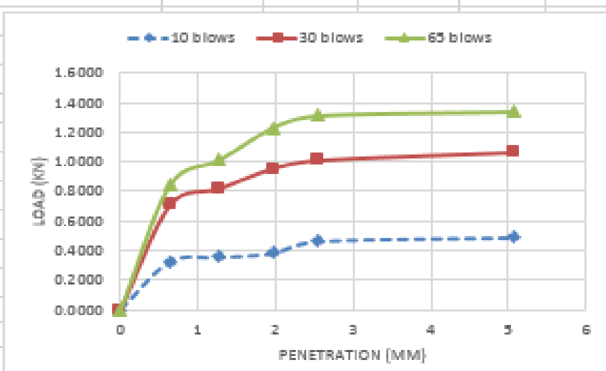


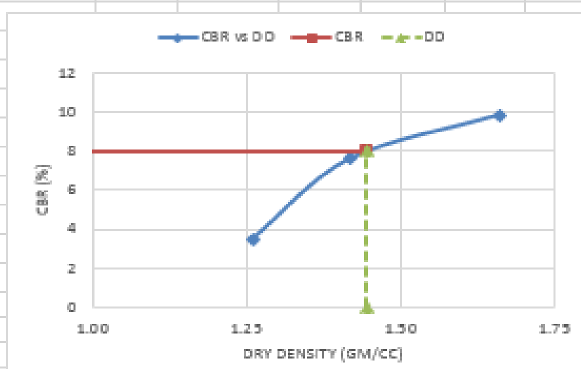


CBR PENETRATION TEST DATA-4% MOLASSES (14DAYS CURED AND 4 DAYS SOAKED)												
PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn)	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (k	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (k	CBR %
0	0	0.0000			0	0			0	0		
0.64	12	0.3277			26	0.7101			31	0.8466		
1.27	13	0.3550			30	0.8193			37	1.0105		
1.96	14	0.3823			35	0.9559			45	1.2290		
2.54	17	0.4643	0	3	37	1.0105	1	8	48	1.3109	1	10
5.08	18	0.4916	0	2	39	1.0651	1	5	49	1.3382	1	7
7.62		0.0000				0.0000				0.0000		

CBR SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	1165	1451	1335
RDG (AFTER) SOAKING)	1169	1454	1337
PERCENT SWELL	3.22	2.73	1.85
AVERAGE PERCENT SWELL:		2.62	

Dry Density	1.26	1.42	1.66
CBR	3	8	10
MDD	1.52		
95%MDD	1.444		
CBR @ 95% MDD	8.000		





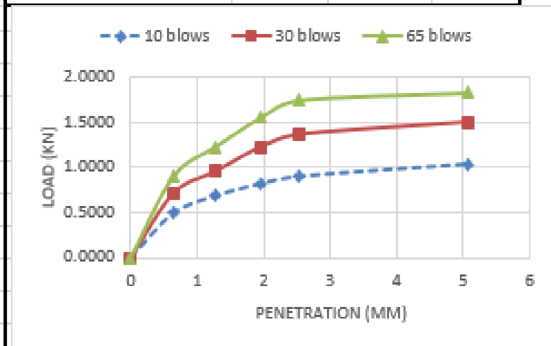
CBR P E N E T R A T I O N T E S T D A T A-8%MOLASSES CURED FOR 14 DAYS AND SOAKED FOR 4 DAYS

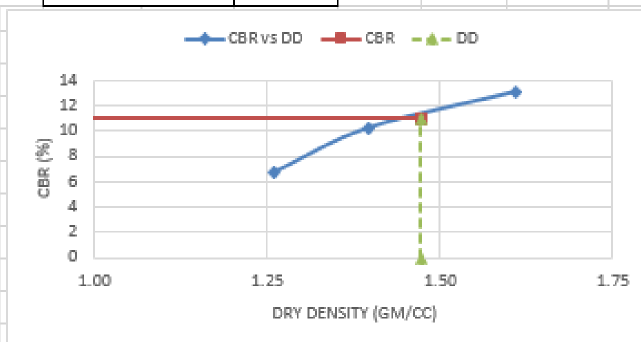
PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn)	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn)	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn)	CBR %
0	0	0.0000			0	0			0	0		
0.64	18	0.4916			26	0.7101			33	0.9012		
1.27	25	0.6828			35	0.9559			45	1.2290		
1.96	30	0.8193			45	1.2290			57	1.5567		
2.54	33	0.9012	1	7	50	1.3655	1	10	64	1.7478	2	13
5.08	38	1.0378	1	5	55	1.5021	2	8	67	1.8298	2	9
7.62		0.0000				0.0000				0.0000		

CBR SWELL

No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	1261	1651	1235
RDG (AFTER) SOAKING)	1265	1654	1237
PERCENT SWELL	3.22	2.58	1.83
AVERAGE PERCENT SWELL :		2.54	

Dry Density	1.26	1.40	1.61
CBR	7	10	13
MDD	1.55		
95%MDD	1.4725		
CBR @ 95% MDD	11.000		





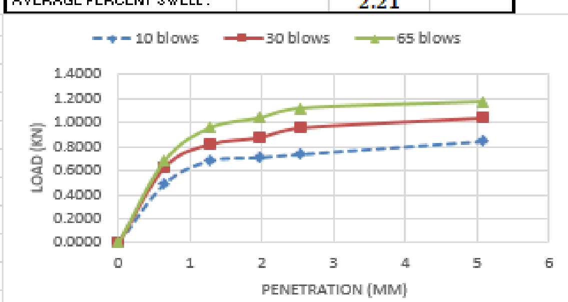
CBR P E N E T R A T I O N T E S T D A T A- 12% MOLASSES CURED FOR 14 DAYS & SOAKED FOR 4 DAYS

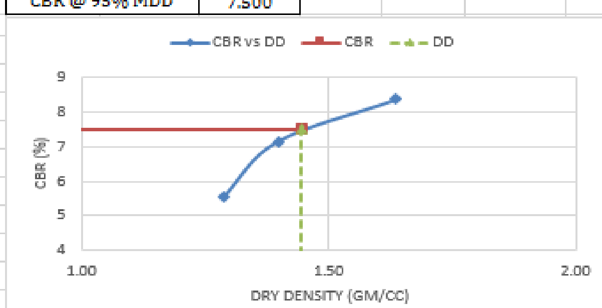
PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn)	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn)	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn)	CBR %
0	0	0.0000			0	0			0	0		
0.64	18	0.4916			23	0.6281			25	0.6828		
1.27	25	0.6828			30	0.8193			35	0.9559		
1.96	26	0.7101			32	0.8739			38	1.0378		
2.54	27	0.7374	1	6	35	0.9559	1	7	41	1.1197	1	8
5.08	31	0.8466	1	4	38	1.0378	1	5	43	1.1743	1	6
7.62		0.0000				0.0000				0.0000		

CBR SWELL

No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	1165	1451	1335
RDG (AFTER) SOAKING)	1168	1453	1337
PERCENT SWELL	2.92	1.98	1.72
AVERAGE PERCENT SWELL :		2.21	

Dry Density	1.29	1.40	1.64
CBR	6	7	8
MDD	1.52		
95%MDD	1.444		
CBR @ 95% MDD	7.500		



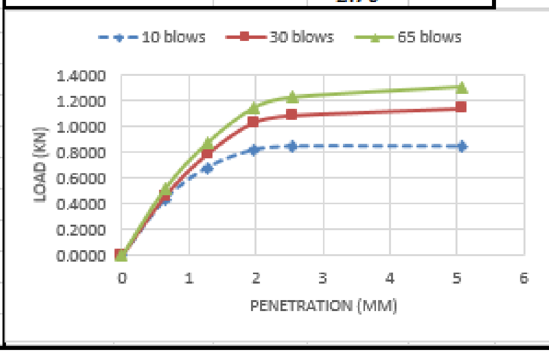


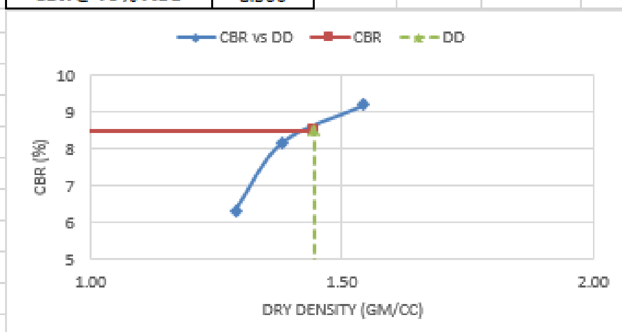
CBR PENETRATION TEST DATA- 12% MOLASSES CURED FOR 14 DAYS & SOAKED FOR 4 DAYS

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn))	CBR %
0	0	0.0000			0	0			0	0		
0.64	16	0.4370			17	0.4643			19	0.5189		
1.27	25	0.6828			29	0.7920			32	0.8739		
1.96	30	0.8193			38	1.0378			42	1.1470		
2.54	31	0.8466	1	6	40	1.0924	1	8	45	1.2290	1	9
5.08	31	0.8466	1	4	42	1.1470	1	6	48	1.3109	1	7
7.62		0.0000				0.0000				0.0000		

CBR SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	1165	1451	1335
RDG (AFTER) SOAKING)	1169	1454	1337
PERCENT SWELL	3.35	2.90	1.85
AVERAGE PERCENT SWELL:		2.70	

Dry Density	1.29	1.38	1.54
CBR	6	8	9
MDD	1.52		
95%MDD	1.444		
CBR @ 95% MDD	8.500		



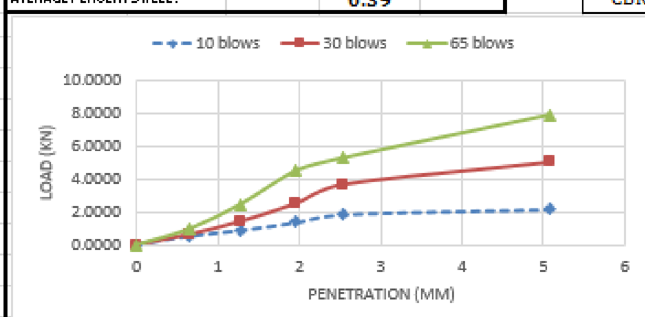


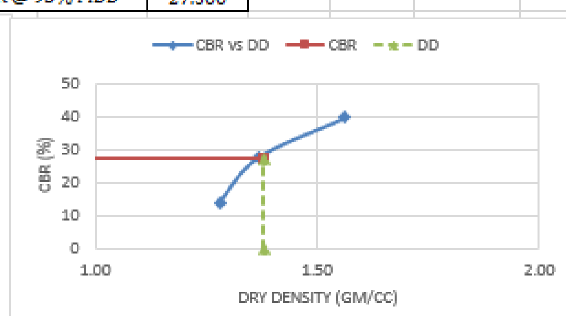
CBR PENETRATION TEST DATA-4% CEMENT 7 DAYS CURED & 7 DAYS SOAKED

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn))	CBR %
0	0	0.0000			0	0			0	0		
0.64	19	0.5189			23	0.6281			35	0.9559		
1.27	32	0.8739			52	1.4201			90	2.4579		
1.96	50	1.3655			93	2.5398			166	4.5335		
2.54	68	1.8571	2	14	135	3.6869	4	28	194	5.2981	5	40
5.08	80	2.1848	2	11	185	5.0524	5	25	290	7.9199	8	40
7.62		0.0000				0.0000				0.0000		

CBR SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	1065	1051	1035
RDG (AFTER) SOAKING)	1066	1051	1035
PERCENT SWELL	0.64	0.33	0.20
AVERAGE PERCENT SWELL:		0.39	

Dry Density	1.28	1.37	1.56
CBR	14	28	40
MDD	1.47		
95%MDD	1.3965		
CBR @ 95% MDD	27.300		



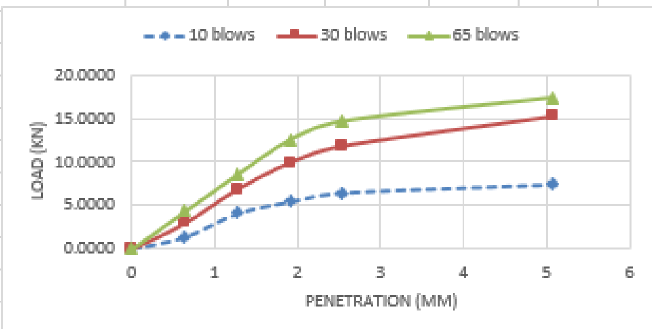


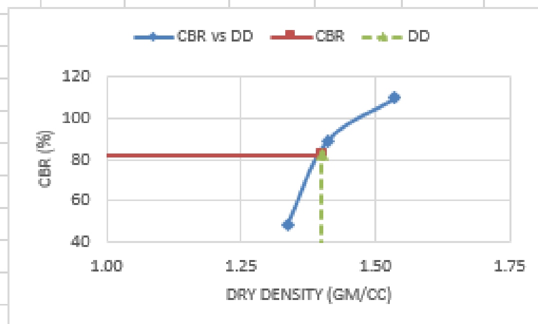
CBR PENETRATION TEST DATA- 8% CEMENT CURED FOR 7 DAYS AND SOAKED FOR 7 DAYS

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn	CBR %
0	0	0.0000			0	0			0	0		
0.64	51	1.3928			110	3.0041			158	4.3150		
1.27	149	4.0692			253	6.9094			312	8.5207		
1.91	199	5.4347			365	9.9682			459	12.5353		
2.54	236	6.4452	6	48	435	11.8799	12	89	536	14.6382	15	110
5.08	271	7.4010	7	37	560	15.2936	15	77	635	17.3419	17	87
7.62		0.0000				0.0000				0.0000		

SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	1165	1151	1135
RDG (AFTER) SOAKING)	1166	1151	1135
PERCENT SWELL	0.56	0.33	0.20
AVERAGE PERCENT SWELL :		0.36	

Dry Density	1.34	1.41	1.54
CBR	48	89	110
MDD	1.49		
95%MDD	1.4155		
CBR @ 95% MDD	82.300		



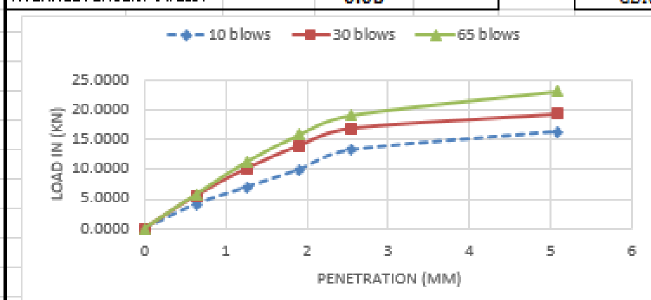


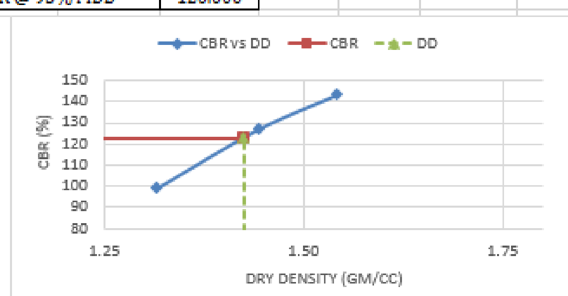
CBR PENETRATION TEST DATA- 12% CEMENT CURED FOR 7 DAYS & SOAKED FOR 7 DAYS

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn)	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn))	CBR %
0	0	0.0000			0	0			0	0		
0.64	151	4.1238			205	5.5986			212	5.7897		
1.27	257.5	7.0323			375	10.2413			415	11.3337		
1.91	365	9.9682			515	14.0647			580	15.8398		
2.54	485	13.2454	13	99	620	16.9322	17	127	700	19.1170	19	143
5.08	600	16.3860	16	82	710	19.3901	19	97	850	23.2135	23	116
7.62		0.0000				0.0000				0.0000		

SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	1065	1051	1035
RDG (AFTER) SOAKING)	1065	1051	1035
PERCENT SWELL	0.04	0.07	0.03
AVERAGE PERCENT SWELL :		0.05	

Dry Density	1.32	1.44	1.54
CBR	99	127	143
MDD	1.5		
95%MDD	1.425		
CBR @ 95% MDD	123.000		



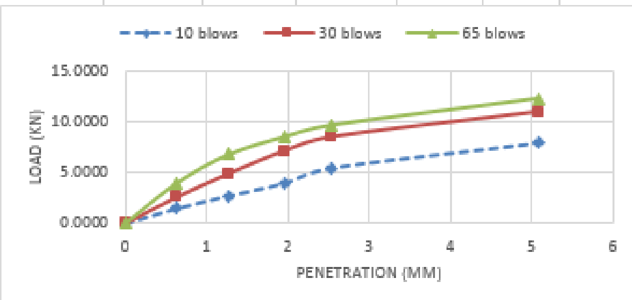


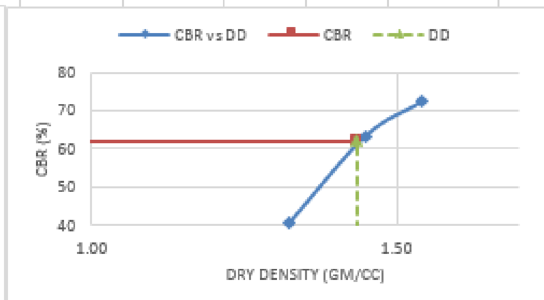
CBR PENETRATION TEST DATA-4% CEMENT + 4% MOLASSES-CURED FOR 7 DAYS & SOAKED FOR 7 DAYS

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn))	CBR %
0	0	0.0000			0	0			0	0		
0.64	54	1.4747			95	2.5945			146	3.9873		
1.27	99	2.7037			178	4.8612			248	6.7729		
1.96	145	3.9600			260	7.1006			312	8.5207		
2.54	200	5.4620	5	41	310	8.4661	8	63	354	9.6677	10	72
5.08	290	7.9199	8	40	400	10.9240	11	55	450	12.2895	12	62
7.62		0.0000				0.0000				0.0000		

SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	1465	1451	1435
RDG (AFTER) SOAKING)	1466	1451	1435
PERCENT SWELL	0.77	0.26	0.04
AVERAGE PERCENT SWELL :		0.36	

Dry Density	1.32	1.45	1.54
CBR	41	63	72
MDD	1.51		
95%MDD	1.4345		
CBR @ 95% MDD	62.000		



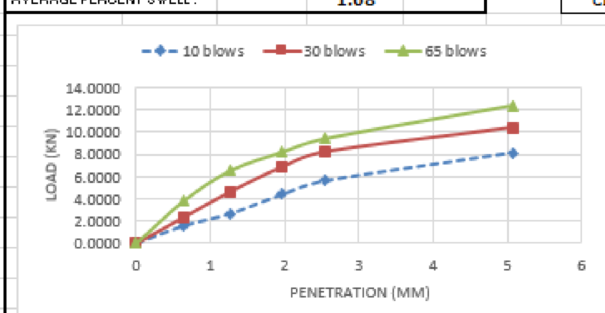


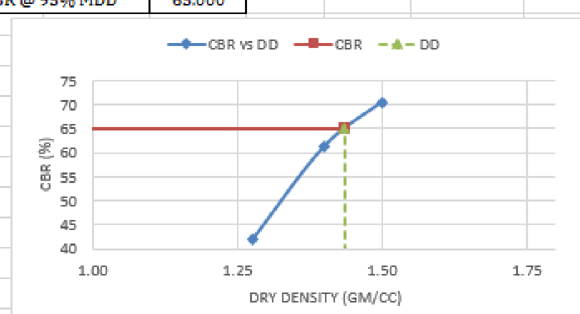
CBR PENETRATION TEST DATA-4% CEMENT + 4% MOLASSES-CURED FOR 7 DAYS & SOAKED FOR 7 DAYS

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn))	CBR %
0	0	0.0000			0	0			0	0		
0.64	58	1.5840			86	2.3487			140	3.8234		
1.27	98	2.6764			170	4.6427			240	6.5544		
1.96	160	4.3696			250	6.8275			300	8.1930		
2.54	205	5.5986	6	42	300	8.1930	8	61	345	9.4220	9	71
5.08	298	8.1384	8	41	380	10.3778	10	52	455	12.4261	12	62
7.62		0.0000				0.0000				0.0000		

SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	1065	1051	1035
RDG (AFTER) SOAKING)	1066	1052	1036
PERCENT SWELL	1.25	1.10	0.88
AVERAGE PERCENT SWELL :		1.08	

Dry Density	1.27	1.40	1.50
CBR	42	61	71
MDD	1.51		
95%MDD	1.4345		
CBR @ 95% MDD	65.000		



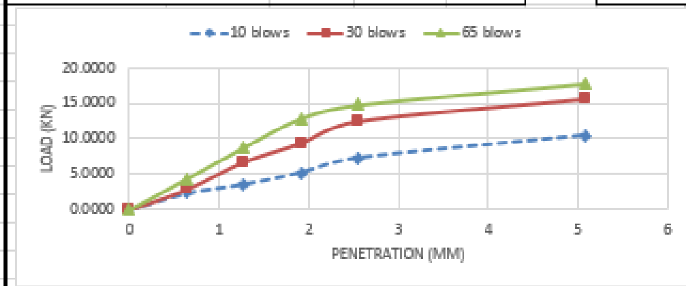


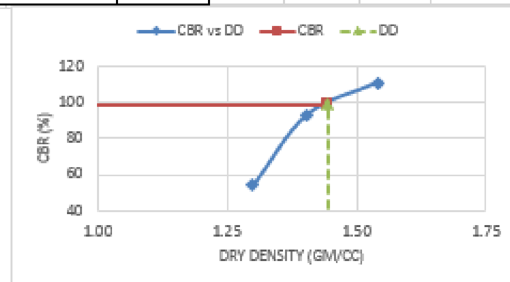
CBR PENETRATION TEST DATA- 8% CEMENT + 4% MOLASSES CURED FOR 7 DAYS & SOAKED FOR 7 DAYS

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (k	CBR %
0	0	0.0000			0	0			0	0		
0.64	85	2.3214			105	2.8676			160	4.3696		
1.27	130	3.5503			245	6.6910			320	8.7392		
1.91	190	5.1889			342	9.3400			471	12.8630		
2.54	265	7.2372	7	54	455	12.4261	12	93	540	14.7474	15	110
5.08	380	10.3778	10	52	570	15.5667	16	78	650	17.7515	18	89
7.62		0.0000				0.0000				0.0000		

SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	1465	1451	1435
RDG (AFTER) SOAKING)	1467	1452	1436
PERCENT SWELL	1.29	0.95	0.77
AVERAGE PERCENT SWELL :		1.00	

Dry Density	1.30	1.40	1.54
CBR	54	93	110
MDD	1.52		
95%MDD	1.444		
CBR @ 95% MDD		99.000	



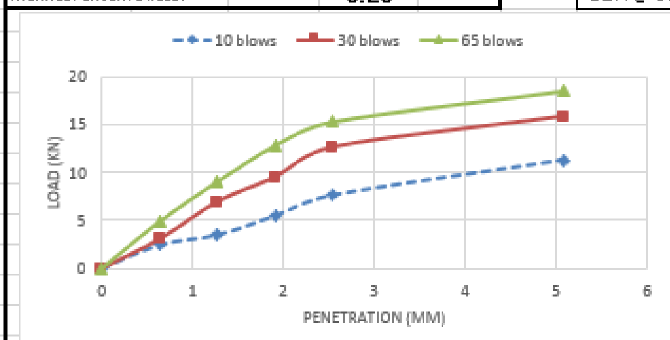


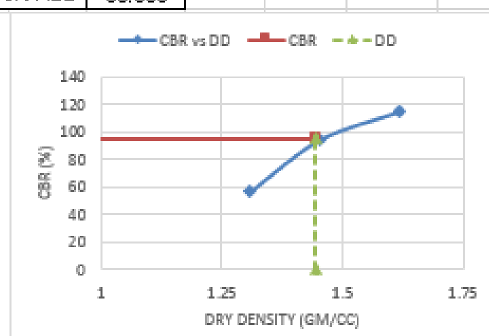
CBR PENETRATION TEST DATA- 8% CEMENT + 4% MOLASSES CURED FOR 7 DAYS & SOAKED FOR 7 DAYS

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (k	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn))	CBR %
0	0	0.0000			0	0			0	0		
0.64	89	2.4306			112	3.0587			180	4.9158		
1.27	126	3.4411			253	6.9094			331	9.0396		
1.91	200	5.4620			347	9.4766			470	12.8357		
2.54	278	7.5922	8	57	461	12.5899	13	94	560	15.2936	15	115
5.08	413	11.2790	11	57	578	15.7852	16	79	678	18.5162	19	93
7.62		0.0000				0.0000				0.0000		

SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	865	851	835
RDG (AFTER) SOAKING)	866	851	835
PERCENT SWELL	0.43	0.26	0.16
AVERAGE PERCENT SWELL :		0.28	

Dry Density	1.31	1.45	1.62
CBR	57	94	115
MDD	1.52		
95%MDD	1.444		
CBR @ 95% MDD		95.000	



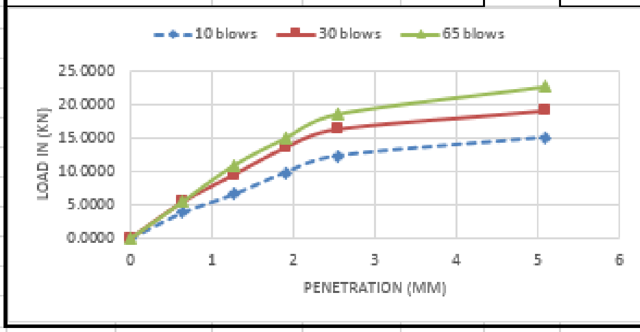


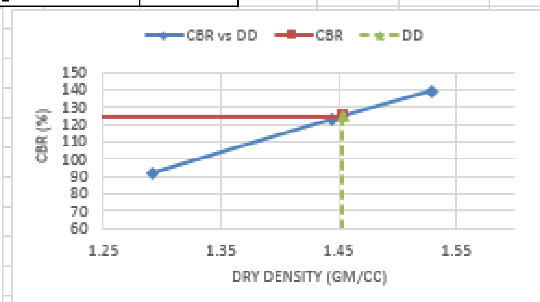
CBR PENETRATION TEST DATA-12% CEMENT + 4% MOLASSES CURED FOR 7 DAYS AND SOAKED FOR 7 DAYS

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn))	CBR %
0	0	0.0000			0	0			0	0		
0.64	140	3.8234			200	5.4620			200	5.4620		
1.27	240	6.5544			350	9.5585			400	10.9240		
1.91	360	9.8316			500	13.6550			550	15.0205		
2.54	450	12.2895	12	92	600	16.3860	16	123	680	18.5708	19	139
5.08	550	15.0205	15	75	700	19.1170	19	96	830	22.6673	23	114
7.62		0.0000				0.0000				0.0000		

SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	116.00	13.00	191.00
RDG (AFTER) SOAKING)	117.20	13.48	191.30
PERCENT SWELL	1.03	0.41	0.26
AVERAGE PERCENT SWELL:		0.57	

Dry Density	1.29	1.44	1.53
CBR	92	123	139
MDD	1.53		
95%MDD	1.4535		
CBR @ 95% MDD		125.000	



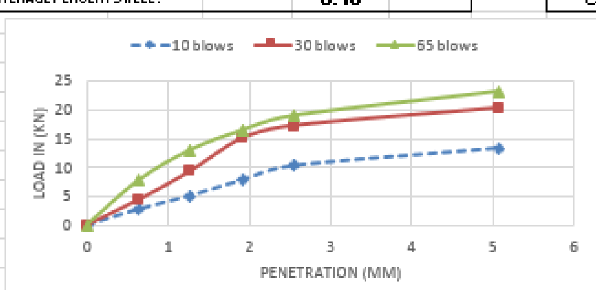


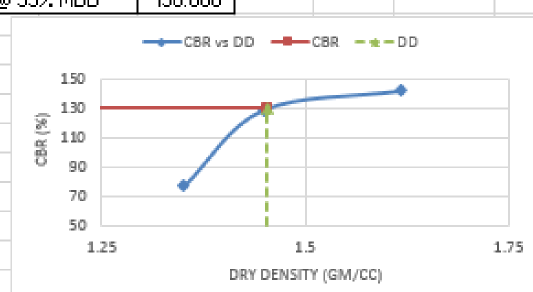
CBR PENETRATION TEST DATA-12% CEMENT + 4% MOLASSES CURED FOR 7 DAYS AND SOAKED FOR 7 DAYS

PENETRATION	10 Blows				30 Blows				65 Blows			
(mm)	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR. LOAD (kn))	CBR %	DIAL RDG	LOAD (kn)	COR LOAD (kn))	CBR %
0	0	0.0000			0	0			0	0		
0.64	100	2.7310			160	4.3696			285	7.7834		
1.27	185	5.0524			340	9.2854			476	12.9996		
1.91	284	7.7560			550	15.0205			602	16.4406		
2.54	375	10.2413	10	77	630	17.2053	17	129	695	18.9805	19	142
5.08	487	13.3000	13	67	743	20.2913	20	102	850	23.2135	23	116
7.62		0.0000				0.0000				0.0000		

SWELL			
No. OF BLOWS	10	30	65
RDG (BEFORE SOAKING)	216.00	233.00	291.00
RDG (AFTER) SOAKING)	216.40	233.10	291.05
PERCENT SWELL	0.34	0.09	0.04
AVERAGE PERCENT SWELL:		0.16	

Dry Density	1.35	1.45	1.62
CBR	77	129	142
MDD	1.53		
95%MDD	1.4535		
CBR @ 95% MDD		130.000	





UCS TEST-12 % MOLASSES- 14 DAYS CURED					
Deformation dial in [mm] ΔH	**Unit strain $\epsilon = \Delta H/\Delta Ho$**	**Proving Ring reading**	**Load in [KN]**	**Cross - sectional Area $A = A_o/1-\epsilon$**	**stress in [KN/M^2]**
0.00	0.0000	0.0	0.0000	0.00264	0.0
0.25	0.0022	31.3	0.0420	0.00265	15.9
0.50	0.0043	61.3	0.0822	0.00265	31.0
0.75	0.0065	88.0	0.1179	0.00266	44.3
1.00	0.0086	113.0	0.1514	0.00266	56.8
1.25	0.0108	134.7	0.1805	0.00267	67.6
1.50	0.0129	154.7	0.2073	0.00268	77.4
1.75	0.0151	176.0	0.2358	0.00268	87.9
2.00	0.0172	198.0	0.2653	0.00269	98.7
2.25	0.0194	214.7	0.2877	0.00269	106.8
2.50	0.0216	228.3	0.3060	0.00270	113.3
2.75	0.0237	239.3	0.3207	0.00271	118.5
3.00	0.0259	248.3	0.3328	0.00271	122.7
3.25	0.0280	258.7	0.3466	0.00272	127.5
3.50	0.0302	270.0	0.3511	0.00272	132.0
3.75	0.0323	275.0	0.3564	0.00273	135.0
4.00	0.0345	286.0	0.3832	0.00274	140.1
4.25	0.0366	288.0	0.3859	0.00274	140.7
4.50	0.0388	289.0	0.3873	0.00275	140.9
4.75	0.0409	288.0	0.3859	0.00275	140.1
5.00	0.0431	284.0	0.3806	0.00276	137.8

UCS TEST-12 % MOLASSES- 64 DAYS CURED					
Deformation dial in [mm] ΔH	**Unit strain $\epsilon = \Delta H/\Delta Ho$**	**Proving Ring reading**	**Load in [KN]**	**Cross - sectional Area $A = A_o/1\text{-}\epsilon$**	**stress in [KN/M^2]**
0.00	0.0000	0.0	0.0000	0.00264	0.0
0.25	92.1781	35	0.0469	0.00265	17.7
0.50	184.3562	65	0.0871	0.00265	32.8
0.75	276.5342	87	0.1166	0.00266	43.8
1.00	368.7123	110	0.1474	0.00266	55.3
1.25	460.8904	130	0.1742	0.00267	65.2
1.50	553.0685	152	0.2037	0.00268	76.1
1.75	645.2465	170	0.2278	0.00268	84.9
2.00	737.4246	188	0.2519	0.00269	93.7
2.25	829.6027	205	0.2747	0.00269	102.0
2.50	921.7808	220	0.2948	0.00270	109.2
2.75	1013.9589	233	0.3122	0.00271	115.4
3.00	1106.1369	247	0.3310	0.00271	122.0
3.25	1198.3150	258	0.3457	0.00272	127.2
3.50	1290.4931	274	0.3672	0.00272	134.8
3.75	1382.6712	286	0.3832	0.00273	140.4
4.00	1474.8493	302	0.4047	0.00274	147.9
4.25	1567.0273	313	0.4194	0.00274	152.9
4.50	1659.2054	323	0.4328	0.00275	157.5
4.75	1751.3835	332	0.4449	0.00275	161.5
5.00	1843.5616	340	0.4556	0.00276	165.0
5.25	1935.7396	348	0.4663	0.00277	168.5
5.50	2027.9177	353	0.4730	0.00277	170.5
5.75	2120.0958	358	0.4797	0.00278	172.6
6.00	2212.2739	363	0.4864	0.00279	174.6
6.25	2304.4520	367	0.4918	0.00279	176.1
6.50	2396.6300	370	0.4958	0.00280	177.1
6.75	2488.8081	372	0.4985	0.00281	177.7
7.00	2580.9862	374	0.5012	0.00281	178.2
7.25	2673.1643	374	0.5012	0.00282	177.8
7.50	2765.3423	372	0.4985	0.00282	176.5
7.75	2857.5204	368	0.4931	0.00283	174.2

UCS TEST-8 % MOLASSES- 14 DAYS CURED					
Deformation dial in [mm] ΔH	**Unit strain $\epsilon = \Delta H/\Delta Ho$**	**Proving Ring reading**	**Load in [KN]**	**Cross - sectional Area $A = A_o/1-\epsilon$**	**stress in $[KN/M^2]$**
0.00	0.0000	0.0	0.0000	0.00264	0.0
0.25	0.0022	28.3	0.0380	0.00265	14.3
0.50	0.0043	67.7	0.0907	0.00265	34.2
0.75	0.0065	107.3	0.1438	0.00266	54.1
1.00	0.0086	146.7	0.1965	0.00266	73.7
1.25	0.0108	181.7	0.2434	0.00267	91.1
1.50	0.0129	211.7	0.2836	0.00268	106.0
1.75	0.0151	240.7	0.3225	0.00268	120.2
2.00	0.0172	264.7	0.3547	0.00269	131.9
2.25	0.0194	288.0	0.3859	0.00269	143.2
2.50	0.0216	307.7	0.4123	0.00270	152.7
2.75	0.0237	329.3	0.4413	0.00271	163.1
3.00	0.0259	347.3	0.4654	0.00271	171.6
3.25	0.0280	364.0	0.4878	0.00272	179.4
3.50	0.0302	386.3	0.5177	0.00272	190.0
3.75	0.0323	402.0	0.5387	0.00273	197.3
4.00	0.0345	406.7	0.5449	0.00274	199.1
4.25	0.0366	420.0	0.5628	0.00274	205.2
4.50	0.0388	429.0	0.5749	0.00275	209.1
4.75	0.0409	434.3	0.5820	0.00275	211.3
5.00	0.0431	434.3	0.5820	0.00276	210.8

UCS TEST-8 % MOLASSES- 64 DAYS CURED					
Deformation dial in [mm] ΔH	**Unit strain $\epsilon = \Delta H/\Delta Ho$**	**Proving Ring reading**	**Load in [KN]**	**Cross - sectional Area $A = A_o/1\text{-}\epsilon$**	**stress in [KN/M^2]**
0.00	0.0000	0.0	0.0000	0.00264	0.0
0.25	92.1781	18	0.0241	0.00265	9.1
0.50	184.3562	50	0.0670	0.00265	25.3
0.75	276.5342	94	0.1260	0.00266	47.4
1.00	368.7123	130	0.1742	0.00266	65.4
1.25	460.8904	165	0.2211	0.00267	82.8
1.50	553.0685	202	0.2707	0.00268	101.1
1.75	645.2465	235	0.3149	0.00268	117.4
2.00	737.4246	272	0.3645	0.00269	135.6
2.25	829.6027	310	0.4154	0.00269	154.2
2.50	921.7808	346	0.4636	0.00270	171.7
2.75	1013.9589	380	0.5092	0.00271	188.2
3.00	1106.1369	410	0.5494	0.00271	202.6
3.25	1198.3150	437	0.5856	0.00272	215.4
3.50	1290.4931	460	0.6164	0.00272	226.3
3.75	1382.6712	468	0.6271	0.00273	229.7
4.00	1474.8493	480	0.6432	0.00274	235.1
4.25	1567.0273	498	0.6673	0.00274	243.3
4.50	1659.2054	514	0.6888	0.00275	250.6
4.75	1751.3835	523	0.7008	0.00275	254.4
5.00	1843.5616	523	0.7008	0.00276	253.8
5.25	1935.7396	521	0.6981	0.00277	252.3
5.50	2027.9177	518	0.6941	0.00277	250.3

UCS TEST-4 % MOLASSES- 14 DAYS CURED					
Deformation dial in [mm] ΔH	**Unit strain $\epsilon = \Delta H/\Delta Ho$**	**Proving Ring reading**	**Load in [KN]**	**Cross - sectional Area $A = A_o/1-\epsilon$**	**stress in [KN/M^2]**
0.00	0.0000	0.0	0.0000	0.00264	0.0
0.25	0.0022	41.0	0.0549	0.00265	20.8
0.50	0.0043	73.0	0.0978	0.00265	36.9
0.75	0.0065	102.7	0.1376	0.00266	51.7
1.00	0.0086	126.3	0.1693	0.00266	63.5
1.25	0.0108	156.0	0.2090	0.00267	78.3
1.50	0.0129	176.3	0.2363	0.00268	88.3
1.75	0.0151	195.7	0.2622	0.00268	97.7
2.00	0.0172	212.0	0.2841	0.00269	105.7
2.25	0.0194	227.7	0.3051	0.00269	113.2
2.50	0.0216	244.3	0.3274	0.00270	121.3
2.75	0.0237	255.0	0.3417	0.00271	126.3
3.00	0.0259	269.3	0.3609	0.00271	133.1
3.25	0.0280	281.7	0.3774	0.00272	138.9
3.50	0.0302	292.0	0.3913	0.00272	143.6
3.75	0.0323	300.7	0.4029	0.00273	147.6
4.00	0.0345	309.7	0.4150	0.00274	151.6
4.25	0.0366	317.3	0.4252	0.00274	155.1
4.50	0.0388	321.7	0.4310	0.00275	156.8
4.75	0.0409	324.3	0.4346	0.00275	157.8
5.00	0.0431	328.0	0.4395	0.00276	159.2

UCS TEST-4 % MOLASSES- 64 DAYS CURED					
Deformation dial in [mm] ΔH	**Unit strain $\epsilon = \Delta H/\Delta Ho$**	**Proving Ring reading**	**Load in [KN]**	**Cross - sectional Area $A = A_o/1-\epsilon$**	**stress in [KN/M^2]**
0.00	0.0000	0.0	0.0000	0.00264	0.0
0.25	92.1781	22	0.0295	0.00265	11.1
0.50	184.3562	35	0.0469	0.00265	17.7
0.75	276.5342	58	0.0777	0.00266	29.2
1.00	368.7123	85	0.1139	0.00266	42.7
1.25	460.8904	110	0.1474	0.00267	55.2
1.50	553.0685	134	0.1796	0.00268	67.1
1.75	645.2465	153	0.2050	0.00268	76.4
2.00	737.4246	170	0.2278	0.00269	84.7
2.25	829.6027	187	0.2506	0.00269	93.0
2.50	921.7808	204	0.2734	0.00270	101.2
2.75	1013.9589	218	0.2921	0.00271	107.9
3.00	1106.1369	230	0.3082	0.00271	113.6
3.25	1198.3150	245	0.3283	0.00272	120.8
3.50	1290.4931	258	0.3457	0.00272	126.9
3.75	1382.6712	269	0.3605	0.00273	132.0
4.00	1474.8493	282	0.3779	0.00274	138.1
4.25	1567.0273	294	0.3940	0.00274	143.7
4.50	1659.2054	303	0.4060	0.00275	147.7
4.75	1751.3835	312	0.4181	0.00275	151.8
5.00	1843.5616	322	0.4315	0.00276	156.3
5.25	1935.7396	332	0.4449	0.00277	160.8
5.50	2027.9177	341	0.4569	0.00277	164.8
5.75	2120.0958	350	0.4690	0.00278	168.7
6.00	2212.2739	359	0.4811	0.00279	172.7
6.25	2304.4520	366	0.4904	0.00279	175.6
6.50	2396.6300	372	0.4985	0.00280	178.1
6.75	2488.8081	378	0.5065	0.00281	180.6
7.00	2580.9862	385	0.5159	0.00281	183.5
7.25	2673.1643	391	0.5239	0.00282	185.9
7.50	2765.3423	396	0.5306	0.00282	187.9
7.75	2857.5204	402	0.5387	0.00283	190.3
8.00	2949.6985	406	0.5440	0.00284	191.7
8.25	3041.8766	410	0.5494	0.00284	193.2

8.50	3134.0547	414	0.5548	0.00285	194.6
8.75	3226.2327	417	0.5588	0.00286	195.5
9.00	3318.4108	421	0.5641	0.00286	197.0
9.25	3410.5889	423	0.5668	0.00287	197.4
9.50	3502.7670	426	0.5708	0.00288	198.4
9.75	3594.9451	428	0.5735	0.00288	198.8
10.00	3687.1231	430	0.5762	0.00289	199.3
10.25	3779.3012	431	0.5775	0.00290	199.3
10.50	3871.4793	432	0.5789	0.00290	199.3
10.75	3963.6574	432	0.5789	0.00291	198.8
11.00	4055.8354	432	0.5789	0.00292	198.3
11.25	4148.0135	432	0.5789	0.00293	197.9
11.50	4240.1916	432	0.5789	0.00293	197.4
11.75	4332.3697	431	0.5775	0.00294	196.5
12.75	4701.0820	429	0.5749	0.00297	193.7

UCS TEST-8 % CEMENT- 7 DAYS CURED							
DDR	**LDR**	ΔL=DDR * 0.00254	LDR * 1.85 /2.2	%ε = ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	**stress(kg/cm2)**
0	0	0	0.0000	0.0000	20.4	20.42821	0.00000
10	58	0.0254	48.7727	0.2490	20.4	20.47920	2.38157
20	117	0.0508	98.3864	0.4980	20.4	20.53046	4.79222
30	180	0.0762	151.3636	0.7471	20.4	20.58197	7.35419
40	195	0.1016	163.9773	0.9961	20.4	20.63373	7.94705
50	175	0.127	147.1591	1.2451	20.4	20.68576	7.11403
60	155	0.1524	130.3409	1.4941	20.4	20.73806	6.28511
70	125	0.1778	105.1136	1.7431	20.4	20.79062	5.05582
80		0.2032	0.0000	1.9922	20.4	20.84344	0.00000
90		0.2286	0.0000	2.2412	20.4	20.89653	0.00000
100		0.254	0.0000	2.4902	20.4	20.94990	0.00000

UCS TEST-4 % CEMENT- 7 DAYS CURED							
DDR	**LDR**	ΔL=DDR * 0.00254	LDR * 1.85 /2.2	%ε = ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	**stress(kg/cm2)**
0	0	0	0.0000	0.0000	20.4282	20.4282	0.0000
10	17	0.0254	14.2955	0.2490	20.4282	20.4792	0.6980
20	35	0.0508	29.4318	0.4980	20.4282	20.5305	1.4336
30	0	0.0762	0.0000	0.7471	20.4282	20.5820	2.2063
40	59	0.1016	49.6136	0.9961	20.4282	20.6337	2.4045
50	53	0.127	44.5682	1.2451	20.4282	20.6858	2.1545
60	47	0.1524	39.5227	1.4941	20.4282	20.7381	1.9058
70	38	0.1778	31.9545	1.7431	20.4282	20.7906	1.5370
80		0.2032	0.0000	1.9922	20.4282	20.8434	0.0000
90		0.2286	0.0000	2.2412	20.4282	20.8965	0.0000
100		0.254	0.0000	2.4902	20.4282	20.9499	0.0000

UCS TEST-12 % CEMENT- 7 DAYS CURED							
DDR	**LDR**	ΔL=DDR * 0.00254	LDR * 1.85 /2.2	%ε = ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	**stress(kg/cm2)**
0	0	0.0000	0.0000	0.0000	20.4282	20.4282	0.0000
10	55	0.0254	46.2500	0.2490	20.4282	20.4792	2.2584
20	110	0.0508	92.5000	0.4980	20.4282	20.5305	4.5055
30	159	0.0762	133.7045	0.7471	20.4282	20.5820	6.4962
40	197	0.1016	165.6591	0.9961	20.4282	20.6337	8.0286
50	220	0.1270	185.0000	1.2451	20.4282	20.6858	8.9433
60	215	0.1524	180.7955	1.4941	20.4282	20.7381	8.7181
70	180	0.1778	151.3636	1.7431	20.4282	20.7906	7.2804
80	80	0.2032	67.2727	1.9922	20.4282	20.8434	3.2275
90		0.2286	0.0000	2.2412	20.4282	20.8965	0.0000
100		0.2540	0.0000	2.4902	20.4282	20.9499	0.0000

UCS TEST-4 % CEMENT + 4% MOLASSES- 7 DAYS CURED							
DDR	**LDR**	ΔL=DDR * 0.00254	LDR * 1.85 /2.2	%ε = ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	**stress(kg/cm2)**
0	0	0.00000	0.00000	0.00000	20.42821	20.42821	0.00000
10	15	0.02540	12.61364	0.24902	20.42821	20.47920	0.61592
20	25	0.05080	21.02273	0.49804	20.42821	20.53046	1.02398
30	42	0.07620	35.31818	0.74706	20.42821	20.58197	1.71598
40	48	0.10160	40.36364	0.99608	20.42821	20.63373	1.95620
50	53	0.12700	44.56818	1.24510	20.42821	20.68576	2.15453
60	56	0.15240	47.09091	1.49412	20.42821	20.73806	2.27075
70	59	0.17780	49.61364	1.74314	20.42821	20.79062	2.38635
80	61	0.20320	51.29545	1.99216	20.42821	20.84344	2.46099
90	62.5	0.22860	52.55682	2.24118	20.42821	20.89653	2.51510
100	63	0.25400	52.97727	2.49020	20.42821	20.94990	2.52876

UCS TEST-8% CEMENT + 4% MOLASSES- 7 DAYS CURED

DDR	**LDR**	ΔL=DDR * 0.00254	LDR * 1.85 /2.2	%ε = ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	**stress(kg/cm2)**
0	0	0	0	0	20.4	20.42820623	0
10	45	0.02540	37.84091	0.24902	20.42821	20.47920	1.84777
20	168	0.05080	141.27273	0.49804	20.42821	20.53046	6.88113
30	237	0.07620	199.29545	0.74706	20.42821	20.58197	9.68301
40	250	0.10160	210.22727	0.99608	20.42821	20.63373	10.18852
50	205	0.12700	172.38636	1.24510	20.42821	20.68576	8.33357
60	166	0.15240	139.59091	1.49412	20.42821	20.73806	6.73115
70	131	0.17780	110.15909	1.74314	20.42821	20.79062	5.29850
80		0.20320	0.00000	1.99216	20.42821	20.84344	0.00000
90		0.22860	0.00000	2.24118	20.42821	20.89653	0.00000
100		0.25400	0.00000	2.49020	20.42821	20.94990	0.00000

UCS TEST-12% CEMENT + 4% MOLASSES- 7 DAYS CURED

DDR	**LDR**	ΔL=DDR * 0.00254	LDR * 1.85 /2.2	%ε = ΔL/Lo * 100	Ao	Ac=Ao/(1-ε)	**stress(kg/cm2)**
0	0	0.00000	0.00000	0.00000	20.42821	20.42821	0.00000
10	33	0.02540	27.75000	0.24902	20.42821	20.47920	1.35503
20	72	0.05080	60.54545	0.49804	20.42821	20.53046	2.94906
30	115	0.07620	96.70455	0.74706	20.42821	20.58197	4.69851
40	158	0.10160	132.86364	0.99608	20.42821	20.63373	6.43915
50	195	0.12700	163.97727	1.24510	20.42821	20.68576	7.92706
60	216	0.15240	181.63636	1.49412	20.42821	20.73806	8.75860
70	200	0.17780	168.18182	1.74314	20.42821	20.79062	8.08931
80	167	0.20320	140.43182	1.99216	20.42821	20.84344	6.73746
90	150	0.22860	126.13636	2.24118	20.42821	20.89653	6.03623
100		0.25400	0.00000	2.49020	20.42821	20.94990	0.00000

THE END!