IMPROVEMENT OF GEOTECHNICAL PROPERTIES OF EXPANSIVE SOIL USING STONE DUST AND COAL ASH (THE CASE OF OLONKOMI TOWN, AMBO AREA)

BY:

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A Thesis Submitted to The Department of Civil Engineering

School of Civil Engineering and Architecture

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Approval of Board of examiner

We, the undersigned members of the board of examiner of the final open defense by Birhane Mamo Urgessa Have read and evaluated her thesis entitled "Improvement of Geotechnical Properties of Expansive Soil Using Stone Dust and Coal Ash (The case of Olonkomi town, Ambo area)" and examine the candidate. This is therefore, to certify that the thesis has been accepted in partial fulfillment of the requirement of the degree of Mater of Science in Civil Engineering with specialization in Geotechnical engineering.

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DECLARATION

I, hereby declare that this thesis entitled improvement of Geotechnical properties of Expansive soil using Stone dust and Coal ash (the case of Olonkomi Town, Ambo area) is my original work and has not been presented for as degree in any other university. All sources of materials used for this thesis have also been duly acknowledged.

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This MSc thesis has been submitted for examination with my approval as thesis advisor.

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Submission date ……………………….

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LIST OF ABBREVIATIONS AND SYMBOLS

AASHTO	American Association of states high ways and transport Official
ASTM	American Society for testing material
CA	Coal ash
ES	Expansive Soil
IS	Indian Standard
LL	Liquid Limit
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plastic Limit
SD	Stone Dust
SL	Shrinkage Limit
UCS	Unconfined Compressive Strength
USCS	Unified Soil Classification System
Cu	Cohesion
gm/cm^3	Gram per Centimeter Cube
%	Percent
Kpa	Kilopascal
KN/m^2	Kilo newton per Meter Square

ABSTRACT

Nearly 40% of land area in Ethiopia is covered with Expansive soil. This soil possesses low shear strength and high plasticity causing damage to foundation of light weight buildings and pavements. To improve the shear strength, plasticity and swelling properties of expansive soil, stabilization technique is one of the known techniques used worldwide in construction Industry. In the present study expansive soil (ES) and Stone dust (SD) were collected from Olonkomi Town, Oromia Region. The coal ash (CA) was collected from Ayka Addis textile and investment group, Alemgena, Oromia Region to meet the desired objectives. The experimental program is carried out with a batch of three material mix (Expansive Soil-Stone Dust-Coal ash). The laboratory tests have been carried out for Index and Engineering tests. The moisture-density relationship is studied for different moisture contents. The maximum dry density for optimum mix decreases from 13.4 kN/m^3 to 12.3kN/m^3 to that of the expansive clay soil. The optimum moisture content of the optimum mix increase from 31% to 35% to that of the clay soil. Based on the maximum UCS value the percent of the optimum mix of clay soil, Stone dust and coal ash are 85% ,5% and 10% respectively (ES+5SD+10CA). The UCS value for expansive soil and optimum mix are 158 kN/m^2 and 237 kN/m^2 respectively. Based on free swell index test result the optimum percentages of blended soil changed from highly expansive to medium or marginal. The results of CBR test shows that the CBR value increases from 3.54% to 5.31% from clay soil to stabilized soil. Finally, the cost comparison was made between stabilized soil and selected material and it was concluded that the expansive soil stabilized with stone dust and coal ash are better than replacing the soil with selected material.

Key words: Expansive soil, stabilization, stone dust, coal ash, Unconfined compressive strength

CHAPTER ONE

INTRODUCTION

1.1 Background of the study

Expansive soils of clayey nature are also called swelling soil. When these soils are partially saturated, they increase in volume with the addition of water and they shrink greatly on drying and develop cracks on the surface, these soils possess a high plasticity index .The clay mineral that mostly responsible for expansiveness belongs to Montmorillonite group and the problem of expansive soils is widespread throughout the world, the countries facing the problems with expansive soils are Australia, united states, Canada, India, Israel, China, and Egypt (Murthy, 2002). Ethiopia, covering nearly 40% surface area of the country with expansive soils are observed in area such as central Ethiopia, following the major trunk road, like Addis Ababa – Ambo, Addis Ababa- Weliso, Addis Ababa – Mojo etc.

Expansive soils are difficult to use in the construction of highways and light weight structures, because such light structures cannot exert the necessary counter load to overcome the swelling. Substantial damage occurs on buildings and road that are constructed on expansive soil can cause economic consequences.

Types of structures most often damaged from swelling soil include foundation and wall of residential and light weight (one or two story) buildings , highways and retaining walls etc. (Mesfun, Quezon, & Geremew, 2019).

Shear strength of soil is necessary for many stability problems of geotechnical engineering including prediction of stability of slopes and embankments, design of foundations and earth pressure calculation against retaining structures. Expansive soil has low shear strength and high plasticity. To improve shear strength, and plasticity of clayey soil, stabilization is important. Soil stabilization is the process of improving the engineering properties of the soil, it is also required when the soil is not suitable for the intended purpose. However, the term stabilization is generally restricted to the process of amending the soil itself for improvement of its properties (Arora, 1992) .Therefore, stabilization of expansive soil by using different additives could be one solution to solve the problematic nature of Expansive soils.

However, most of the conventional stabilizing agents are relatively expansive to be implemented, hence it becomes essential to alter the properties of locally available soil to the extent that it can be used in the civil engineering construction works and to make best utilizing of industrial by- products like coal ash and stone dust as soil stabilizing agent. Hence the goal of this research was to establish the effect of stone dust and coal ash combination on expansive soil to amend geotechnical properties of clay soil.

1.2 Statement of the problem

Expansive soils in construction site have a significant influence on planning structural design, construction and maintenance cost, especially for light weight structures such as one or two story buildings, retaining walls, basement floors and foundations, and also it can damage pavements (Mesfun *et al.*, 2019).Due to rapid urbanization on Olonkomi town , the capital of Ejersa Lafo district, there are high construction rate of residential houses , roads, and other commercial infrastructures. The town faces serious problems related to Expansive soils such as, settlement of small houses, cracking of floors and walls, sticking of doors and windows also high construction cost to avoid and replace expansive soils .Therefore a remedial action made thinking about the quality, cost and time utilization of the project by utilizing efficient soil adjustment technique.

1.3 Objective of the study

1.3.1 General objective

The main aim of this research is to study the geotechnical properties of Expansive soil stabilized with stone dust and coal ash.

1.3.2 Specific objective

1. To assess the geotechnical properties of the expansive soil.
2. To investigate the influence of stone dust and coal ash on the nature of expansive soil.
3. To make a cost comparison of the optimum mix with the expansive soil for building foundations.

1.4 Research question

1. What are geotechnical properties of expansive soil found in Olonkomi town?
2. Does the combination of stone dust and coal ash improve geotechnical properties of expansive soil?
3. Is the stabilizing foundation soil economical than replacement of existing soil with other material?

1.5 Significance of the study

Expansive soil formed over the world mainly in India, Israel, USA, Ethiopia etc. The significant damage to foundation for light buildings caused by expansive soils are common problems in Ethiopia. The expansive soil can damage the foundation by uplift as they swell with increasing moisture. The geotechnical properties of expansive soil have been improved by different stabilization techniques. The waste materials such as coal are available as a large coal reserves in yayo worada alone which is about 229 million tons alone and used as energy sources in many factories. The coal ash produced from coal is the waste material and that causes a problem for disposal and air pollution, on the other hand stone dust is by product of stone and is available in Ethiopia. Therefore, reusing industrial wastes provide proper waste management system and also develops a well understanding of locally available industrial wastes as soil stabilizing agents.

1.6 Scope and limitation the study

The research addresses the general and specific objectives and investigate the degree of stabilization of expansive clay soil with the addition of stone dust and coal ash materials based on the existing theories and principles of stabilizations. The study includes different tests conducted before and after the addition of stone dust and coal ash materials to the expansive soil and a comparison has been done to assess the improvements found after stabilization.

The present study has limitations such as, the clay minerals of the soil in the study and the chemical combustion of stabilizers were not determined due to the X-Ray diffraction soil test machine at geological survey of Ethiopia was not working by these reasons only the physical properties of soil and the stabilizers were studied.

1.7 Organization of the thesis

The thesis consists of six chapters the first chapter is the introduction part and it discusses the definition of expansive soil, Objective, Statement of the problem, scope & limitation and significance of the study in general. The second chapter is literature review about expansive soil, method of stabilization and previous works in detail. Chapter three deals with materials and methods used in the study in detail, and chapter four comprises laboratory data collection and analysis of the laboratory result of various blended soils with stone dust and coal ash materials by different ratio. The fifth chapter describes the cost comparison between the replacement of expansive soil by non-expansive soil and stabilized expansive soil for subgrade of G+1 building in detail. Lastly the six chapter consists of conclusion and recommendation of the thesis work.

CHAPTER TWO

LITERATURE REVIEW

2.1 Origin of expansive soil

The parent material of expansive soil classified into two groups, these are igneous rocks and sedimentary rocks. Expansive soils can be found anywhere in the world where climatic, geological and topographical conditions favored their formation. these soils are abundant in semi-arid regions of tropical and temperate zones which the annual evaporation exceeds the precipitation, climatic condition coupled with environmental factors facilitates weathering of the parent rock and formation of residual soil. Weathering process by which clay is formed includes physical, biological and chemical process. The most important weathering process responsible for the formation of montmorillonite is chemical weathering, which include hydrolysis, hydration, oxidation, carbonation and solution of parent rock mineral which generally consists of ferromagnesium mineral, feldspars, volcanic glass, volcanic rocks and volcanic ash. (Teferra & Leikun, 1999).

Expansive soil consists clay minerals and one of the most dominant clay minerals is Montmorillonite group which is responsible for shrink- swell characteristics of expansive soil when moisture fluctuation in the soil mass is prevalent. Foundation constructed on such clays are subjected to large uplifting forces caused by swelling these forces induced heaving , cracking and breakup of both building foundations and slabs on grade members (R. D. Holtz, Kovacs, & Sheahan, 1981).

2.2 Clay minerals

Clay minerals are very tiny crystalline substance involved primary from chemical weathering of certain rock -forming minerals. Chemically they are hydrous aluminosilicates plus other metallic ions they can be divided into three general groups on the basis of their crystalline arrangement these are namely: Kaolinite group, Montmorillonite group and Illite group (R. D. Holtz *et al.*, 1981).

- Kaolinite Minerals

Its basic structural unit consists of an alumina sheet combined with silica sheet tips the silica sheet and one base of the alumina sheet from a common interface, the structural units joined together by hydrogen bond, which develops between the oxygen of silica sheet and the hydroxyls of alumina sheet as the bond is fairly strong, the mineral is stable moreover, water cannot easily enter between the structural unit and cause expansion.(R. D. Holtz *et al.*, 1981)

- Montmorillonite Minerals

Montmorillonite minerals is the most common mineral of the montmorillonite group of minerals the basic structural unit consists of an alumina sheet sandwiched between two silica sheet, successive structural units are stacked one over another. The two successive structural units are joined together by a link between oxygen ions of the two-silica sheet the link is due to natural attraction for the cations in the intervening space and due to Vander Waal forces. the negatively charged surfaces of silica sheet attract water in the space two structural units this results in an expansion of the mineral the soil montmorillonite exhibits high shrinkage and high swelling characteristics. (Nelson & Miller, 1997)

- Illite Minerals

The basic structural unit is similar to that of Montmorillonite. However, the basic illite units are bonded together by potassium ions which are non -exchangeable. Because of this, the illite units are reasonably stable and so that minerals swell much hence, illite moderate degree of expansive.

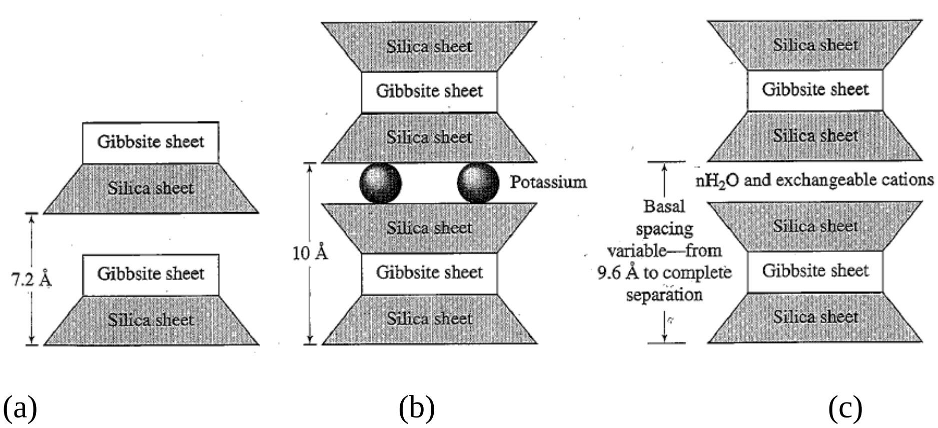


Figure 2. 1 Clay minerals (a) kaolinite, (b) Illite, (c) Montmorillonite (Das, 2015)

2.3 Identification of Expansive soil

2.3.1 Field identification

Expansive deposits can be recognized in the field through visual inspections. The method is simple and easy to use. Some of the important field identification method that indicate the potential for expansive soil is the following.

- The wet samples of the soil are sticky and it will be relatively difficult to clean the soil form the hands.
- Cracking appear in nearby structures (mainly on walls, foundations and grade beams of buildings).
- They usually have a color of black or gray.
- Open or closed fissure (a joint or similar discontinuity).

2.3.2 Laboratory identification

In general, there are three different methods of laboratorial identification of expansive soil namely; mineralogical identification, indirect and direct methods (Nelson & Miller, 1997).

A) Mineralogical identification

This method is used for identifying the mineralogy of clay particles such as characteristics crystal dimensions, characteristics reaction heat treatment, size and shape of clay particles and change deficiency and surface activity of clay particle. These properties are fundamental factors controlling expansive soil behavior. The various techniques under these methods are X-ray diffraction, Differential thermal analysis, optical property determination and electron microscope but these methods are not suitable for routine laboratory test equipment and the results are interpreted by specially trained technicians.

B) Indirect method

The indirect method in which one or more of the related intrinsic properties are measured and complemented with experience to provide indicator of potential volume change. These tests are :

- **Atterberg limit test**: The Atterberg limit define moisture content boundaries between state of consistency of fine-grained soils.

A clay soil can exist in four distinct state of consistency depending on its water content. The water content at the boundaries between the different state are defined as shrinkage limit, plastic limit and liquid limit.

- **Free swell test**: Free swell test consisting of placing a known volume of dry soil passing through the no 40 sieve into graduated cylinder filled with water and measuring the swells volume after it has completely settled, the free swell of a soil is determined as:

$$\text{Free swell} = \frac{(\text{final volume}) - (\text{initial volume})}{\text{initial volume}} \ldots\ldots\quad (2.1)$$

Soils with free swell of 100% may cause damage to light structures when they become wet; soils with free swells less than 50% have been found to exhibit only small volume changes .

- **Colloidal content test:** this test is used to determine the quantity of material in a soil sample that is smaller than a selected size expressed as a percentage by weight of the total sample. Sizes used are 0.002 mm and 0.001mm. The test usually required hydrometer analysis.

C) Direct method

These methods present the most useful data by direct measurement; which involves 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.

2.4 Soil classification Method

2. 4.1 General Soil Classification

Soil classification is the arrangement of soil into different groups such that the soil in a particular group have similar behavior. It divided soils into groups and sub groups based on common engineering properties such as grain size distribution, liquid limit and plastic limit. The two major system presently in use are American Association of state Highway and Transportation official (AASHTO) and the unified soil classification system (USCS). The AASHTO system is used mainly for the classification of highway subgrade, it is not used in foundation construction. AASHTO classification system.

This system was originally developed by Hogentogler and Terzaghi in 1929 as the public roads classification system. After wards there are several revisions, the present AASHTO (1978) system is primarily based on the version in 1945.The original purpose of this classification system is used for road construction (sub grade rating), there are eight major groups in AASHTO classification system A1- A7 with several sub groups and organic soils A8.The required testes are sieve analysis and Atterberg limit. (Das, 2015)

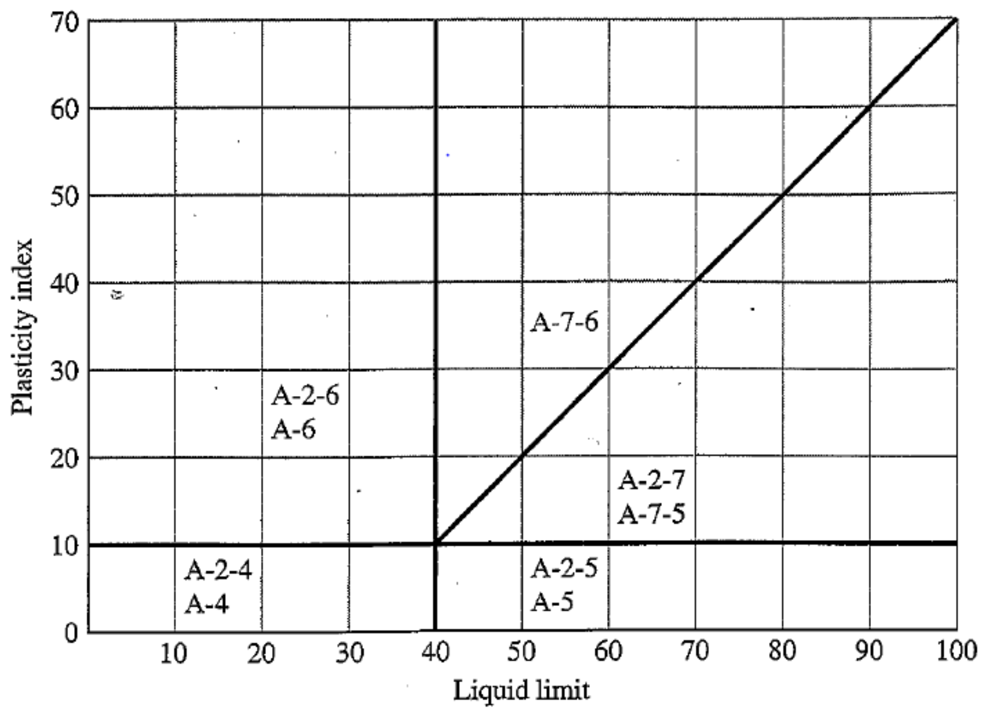


Figure 2. 2 Range of LL & PI for soils in group A-2, A-4, A-5,A-6 & A-7 (Das, 2015)

To evaluate the quality of a soil as a high way subgrade material group index (GI) is an important parameter. it is given by the equation

$$GI = (F_{200} - 35)[0.2 + 0.005(LL - 40)] + 0.01(F_{200} - 35)(PI - 10) \qquad (2.2)$$

Where F_{200} is percentages passing through sieve number 200 and LL & PI are liquid limit and plasticity index respectively.

Table 2. 1 AASHTO classification system (Das, 2015)

General classification	Granular materials (35% or less of total sample passing No. 200 sieve)						
	A-1			A-2			
Group classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7
Sieve analysis (% passing)							
No. 10 sieve	50 max						
No. 40 sieve	30 max	50 max	51 min				
No. 200 sieve	15 max	25 max	10 max	35 max	35 max	35 max	35 max
For fraction passing No. 40 sieve							
Liquid limit (LL)				40 max	41 min	40 max	41 min
Plasticity index (PI)	6 max		Nonplastic	10 max	10 max	11 min	11 min
Usual type of material	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand			
Subgrade rating	Excellent to good						

General classification	Silt–clay materials (More than 35% of total sample passing No. 200 sieve)			
Group classification	A-4	A-5	A-6	A-7
				A-7-5[a] A-7-6[b]
Sieve analysis (% passing)				
No. 10 sieve				
No. 40 sieve				
No. 200 sieve	36 min	36 min	36 min	36 min
For fraction passing No. 40 sieve				
Liquid limit (LL)	40 max	41 min	40 max	41 min
Plasticity index (PI)	10 max	10 max	11 min	11 min
Usual types of material	Mostly silty soils		Mostly clayey soils	
Subgrade rating	Fair to poor			

[a]If PI $\leq$ LL $-$ 30, the classification is A-7-5.
[b]If PI $>$ LL $-$ 30, the classification is A-7-6.

A) Unified soil classification system

According to this system the soils are divided into three major groups: coarse -grained, fine -grained and highly organic (peat). The soils are represented by symbols such as, GW or SP, border line materials are represented by a double symbol as GW-GP.

The fine -grained soils are divided into three groups these are inorganic sits (M), inorganic clays (C), organic slits and clay (O). The soils are further divided into those having liquid limits lower than 50 % (L), or higher (H). the distinction between the inorganic clay and the inorganic silt and organic is made by the basis of modified plasticity chart. The unified soil classification system the most common system among geotechnical Engineers.

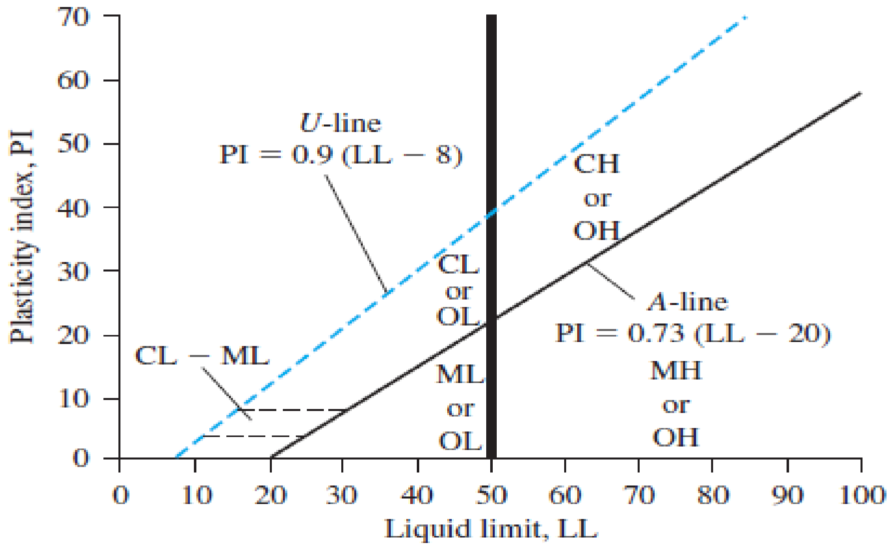


Figure 2.3 Plasticity chart (Das, 2015)

2.4.2 Classification specific to expansive soil

The parameters determined from expansive soil identification tests have been combined in a number of different classification scheme to provide a qualitative assessment of the degree of expansion. Unfortunately, there has no type evolved a standard classification procedure and a different scheme is used in practically every different location (Nelson & Miller, 1997) . some of the classification methods discussed in literature is given in the following section.

I. Classification based on indirect prediction of swell potential

a) Skempton Method

Skempton (1953) observed plasticity index of a soil increase linearly with percentages of clay -size fraction (percent finer than 2 microns by weight) present. It is used as an index for identifying the swell potential of clay soil and expressed as:

$$\text{Activity(A)} = \frac{\text{plasticiy index}}{\text{\%of clay size fraction} - \text{by weight}} \ldots\ldots\ldots\ldots 2.3$$

Typical values of activities for clay minerals are given in the table below

Table 2.2 Activity of clay minerals (Nelson & Miller, 1997)

Activity	Types of clay minerals
<0.75	Non active
0.75-1.25	Normally active
1.25-2.0	Active

b) Method of Chen

This method was proposed by Chen (1988) and presented a single index method for identifying expansive soil using only plasticity indices shown in Table 2.3.

Table 2. 3 Relation between swelling potential and plasticity

Plasticity index (%)	Swelling potential
0-15	Low
10-35	Medium
20-55	High
35 & above	Very high

c) USBR classification method

This method developed by W. Holtz (1956) which is based on the simultaneous consideration of colloid content < 0.001mm, plasticity index and shrinkage limit with observed volume change. The typical relationships of these properties with swelling potential as shown in Table 2.4

Table 2. 4 Classification of expansive soil based on USBR method (Murthy, 2002)

Colloid content < 0.001mm	PI	SL	Probable expansion % 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

II. Classification based on the oedometer swell potential value

Based on the odometer swell potential values, seed, et, al (1962) and Holtz (1956) have classified the relative expansivity of the swelling soils. The expansivity categories proposed by these workers are shown in Table 2.5 (Nelson & Miller, 1997).

Table 2.5 Classification based on Odometer swell potential value

Holtz and Gibbs (1956) Classification percent swell	Seed, et,al (1962) Classification of % 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

2.5 Soil stabilization

Soil stabilization is referring to the procedure employed with a view to improving one or more engineering properties of a soil that are used for base course, sub base, and subgrade by the use of additive which are mixed into the soil to gain the desired improvement. The main aim of soil stabilization is to increase shear strength, reduce permeability and compressibility, to enhance bearing capacity of a soil, most importantly to improve natural soil for the construction (Venkatramaiah, 1995).

2.5.1 Method of stabilization

Soil stabilization method may be brought under the two main categories these are Mechanical stabilization and chemical or additive stabilization (Venkatramaiah, 1995).

A) Mechanical Stabilization

Mechanical stabilization is a process of improving the quality soil by mixing different types of soil without the addition of any chemicals. It is performed by mixing or blending soils of two or more gradations to obtain a material meeting the required specification.

Mechanical stabilization is a process, in which the grading of the soil is improved by the combination of another material which affect only the physical properties of the soil. It is the most widely used method where the material is made more stable by adjusting the particle size distribution. It can be achieved by compaction or by mixing a different soil with the existing soil.

Changing of gradation

Changing of gradation is the process of addition and removal of soil particles to obtain suitable grading, which involves mixing coarse material or gravel, sand, silt and clay in proper proportion so that the mixture when compacted attains maximum dry density and strength, mechanical stabilization of this type has been used in the construction of low-cost road.

Compaction

It is the densification of soil by the application of mechanical energy. It is the simplest and the most commonly used method, which increase the performance of a natural material. in the stabilization by compaction method the soil density is increased by the application of short - term external mechanical forces to carry the loads that will be applied. It interlocks soil particles and provides a significant effect on soils property, such as strength and stress- strain characteristics, permeability, compression, swelling and water absorption.

Mechanical stability of blended soils depends up on the following factors (Arora, 1992).

a. Mechanical strength of aggregate: Aggregates which have high strength will provide a mechanically stable mix.
b. Mechanical composition: The minerals in the mixed soil showed be weather resistance
c. Gradation: as much as possible the blended soil should be well graded resulting in higher density.
d. Compaction: The mechanical stability of the mixed soil depends on the degree of compaction attained on site.

B) Chemical / Additives stabilization

Additives are manufactured commercial products that, when added to the soil in the proper quantities, improved engineering characteristic of the soil such as, texture, workability and plasticity. Chemical stabilization is the alteration of soils index property by adding chemicals.

It includes mixing or injecting soil with chemically active compounds such as cement, lime, fly ash and bitumen.

It results a change in both physical and chemical properties of the soil. Under this type, soil stabilization depends mainly on chemical relation between the stabilizers and soil minerals to achieve the desired effect (Lim, Wijeyesekera, Lim, & Bakar, 2014).

some additives widely used for soil stabilization are listed below.

i) Cement stabilization

Cement stabilization is the most common method of stabilization with additives, cement is mixed with soil to produce mixture name soil-cement. Soil cement has been used as a base material as an adoption of improved measuring in many projects, such as slope protection of dams and embankments, pavements of high way, building pad etc. The soil – cement method of stabilization had been performed over 100 years, it serves to amend mechanical and engineering properties of soil.

ii) Bitumen stabilization

Bituminous soil stabilization refers to a process by which a controlled amount bituminous material is thoroughly mixed with an existing soil or aggregate material to form a stable base or wearing surface. Bitumen increases the load bearing capacity of the soil and renders it resistance to the action of water. Bitumen stabilization performed by using Asphalt cement and asphalt emulsions. Asphalt and tars are bituminous materials which are used for stabilization of soil. Any in organic soil which can be mixed with asphalt is suitable for bituminous stabilization, in cohesionless soils, asphalt binds the soil particles together and thus serves as a bonding or cementing agent. In cohesive soils, asphalt protects the soil by plugging its voids and water proofing it helps the cohesive soil to maintain low moisture content and to increase the bearing capacity (Arora, 1992).

iii) Lime stabilization

Three forms of lime are formed when lime stone is broken to produce lime. These are quick lime (Cao), hydrated lime ($CA[OH]_2$) and hydrated lime slurry; all of them can be used to treat the soils. Lime has been known to reduce the swelling potential, liquid limit, plasticity index and it improves the workability and compaction of sub grade soils. In addition to being used alone lime is also used with different admixtures some of these are lime- fly ash, lime-Portland cement, and lime- bitumen.

iv) Fly ash stabilization

This method of stabilization is gaining more important recent times since it has wide spread availability. It is inexpensive and takes less than any other methods and it also have a long history of use as an engineering material and has successfully employed in geotechnical application. Fly ash can be reduced liquid limit and plasticity index and enhanced the CBR and UCS.

2.6 Coal Resource

Coal is a sedimentary rock comprised primarily of carbon, hydrogen, oxygen, sulfur, and nitrogen it is used worldwide in the production of energy and heat in power plants. According to world coal association it is estimated that there are over 850 Giga tones of proven coal reserves worldwide which is enough to last more than 130 years at current rate production.

Coal reserves are available in almost every country worldwide, china and India are the leading countries in the coal production and estimated about 341.1 and 692.4 million ton respectively. Also, in Africa 92% of production coal was produced in south Africa and quantitatively about 251.3 million tons of coal produced annually (Heidrich, Feuerborn, & Weir, 2013).

Ethiopia has as significant potential of coal resource at difference places in the country. The nation should work on the mining sector to unleash its potential of coal there by save foreign currency expenditure the coal mining provides multifaceted jobs. Besides extract services as much needed input to fertilizer factory, textile factory the project helps to generate thermal power, cement factory coal resource appears to be quite wide spread in Ethiopia.

The most promising coal resource areas are found at Yayu, DebreBirhan, DelbiMoye,Chilga, Mush valley and Wuchale. Yayo woreda alone endowed it over 229 million tons of coal production provided that is possible to produce 100000 tons of coal per day. Additionally, it is believed that there would be 32 million tons of coal potential in Achebo area of the total on the average 70 tons have been extracted and supplied to Ayka Addis textile and investment group plc annually (Birhane, 2017; Dawit, 2018).

2.6.1. Coal combustion products

Coal combustion products (CCPS) include fly ash, bottom ash, boiler slop, fluidized, bed combustion (FBC) ash, or flue gas desulfurization (FGD) the term coal ash is used inter change able for different ash type. Fly ash is the fine ash produced in a coal – fired power station, which is collected using electro-static precipitators. Bottom ash is the coarse ash that fouls to the bottom of a furnace (Heidrich *et al.*, 2013).

Coal ash is reused to create many environmental benefits such as reduced need for disposing landfills, to gain economic benefit to reduce cost associated with coal ash disposal, increase revenue from sealing coal ash and saving from using coal ash in place of costly materials and lastly to improved strength, durability and workability of a materials.

Fly ash also known as “pulverized fuel ash” and the particles are generally spherical in size, this property make it easy for them to blend. The chemical composition of fly ash influences its color, the color ranges from light brown or gray (Mohanty, 2015).

According to ASTMC 618 -99 fly ash classified in two classes, class F and class C, based on the chemical composition of the fly ash. The chief difference between these classes is the amount of calcium, silica, aluminum and iron content in the ash.

Class F fly ash is produced from burning of anthracite and bituminous coals. This fly ash is pozzolanic in nature and contains less than 7 % lime (Cao), it requires cementing agent such as Portland cement, quicklime or hydrated lime mixed with water to react and produced cementitious compound. Class C fly ash produced from lignite and sub -bituminous coals and usually contains more than 20% lime (Cao), it has pozzolanic properties and does not require an activator.

2.7 Stone Dust

stone dust is a solid waste produced as by -product during crushing of large size of stone in crusher unit during production of coarse aggregates (Indiramma & Sudharani, 2016). Type of stones used are like lime stone, basalt stone and many other stones. The basalt stone is used for a wide variety of purpose, it is most commonly crushed for use as in aggregate in construction projects such as road base, concrete, aggregate, asphalt pavement, railroad ballast, filter stone in drain fields and many other purposes. Stone dust is low cost -material that possesses pozzolanic property and coarser content in it.

2.8 Previous related research works

Ali and Koranne (2011) studied the Performance analysis of expansive soil treated with stone dust and fly ash. The prime aim of this study was to assess the usefulness of the stone dust and fly ash as soil reinforcement, for this study the material used were lime stone dust, expansive soil, and fly ash. The laboratory test conducted to attain the objective of the study were standard proctor test, free swell test, Atterberg limit test, and CBR test by using Indian standard codes.

As mentioned in the research work the percentage of Fly ash and stone dust to the expansive soil were prepared from10% - 40% (by an increment of 10%) by dry weight of the total quantity taken also only their individual effect on the expansive soil was examined. From the test result MDD was increased from 1.540 to 1.85 kg/m^3 by the addition of 30% stone dust at OMC 15.75 % and 30% of fly ash yield MDD 1.75 kg/m^3 at OMC of 20.05%.
After conducting soaked and un-soaked CBR, the value increased for stabilized soil. The value of soaked CBR varied from 2.84 % for un stabilized soil to 4.50 % for stabilized soil. The improvement in CBR value is due to the better compaction achieved. Finally, they conclude that as addition of stone dust and fly ash increased Atterberg limits, free swell index, and OMC were decreased, MDD and CBR values were increased .and also the optimum percentage of fly ash and stone dust were 25 % and 30 % respectively for improving the properties of expansive soil.

The Properties of materials used for the research work are shown in the Table 2.4.

Table 2.6 Index and engineering properties of material used (Ali & Koranne, 2011)

Index properties	Expansive soil	Limestone dust	Fly ash
Liquid limit (%)	51	NIL	31
Plastic limit (%)	27.4	NP	NP
Plasticity index	23.6	NP	NP
Specific gravity	2.24	2.58	*1.9*
Free swell (%)	86	-	-
OMC (%)	20.3	11.5	21
MDD (kN/m^3)	1.54	15.4	12.50
CBR (%)	1.79	7.8	-

Mohanty (2015), reported the "stabilization of expansive soil by using fly ash. The result of this test indicated that the expansive soil combined with altering percentage of fly ash from 0 to 50 % intervals in multiple of 10 by weight. The highest MDD was observed being at fly ash content of 30 % by weight, while the highest UCS was obtained with the mixture of soil and 20 % fly ash content by weight. And with the increasing fly ash content in the soil -fly ash mixture, the decrease in value of free swell ratio was remarked. This decrease was also reciprocated by the plasticity index value, plasticity index values are directly proportional to percent swell in an expansive soil, thus affecting the swelling behavior of the soil fly ash mixture. Finally, he concludes that fly ash an additive decreases the swelling and increases the strength of the black cotton soil

Indiramma and Sudharani (2016), examined the "Use of Quarry Dust for Stabilizing Expansive Soil". The study was used quarry dust to improve different geotechnical properties of expansive soil, a series of laboratory tests were conducted to study the effect of quarry dust on the plasticity characteristics, compaction characteristics, differential free swell index and compressive strength of expansive soil. All tests were conducted according to Indian standard codes.

The quarry dust was added to the expansive soil from 0 % to 25 % at an increment of 5%. The plastic limit of soil decreased from 20 % to 16 %, plasticity index also decreased from 24 % to 18 % maximum dry density was increased from19.14 KN/m^3 to 19.58 KN/m^3 and optimum moisture content decreased from 12 % to 9.5% with the increase percentage of stone dust from 0 to 25 %. Finally, reported that by increasing the percentage of quarry dust MDD, UCS were increased also LL, PL, DFSI, OMC were decreased by increasing the percentage of quarry dust. Therefore, the quarry dust can be used as stabilizing admixture for expansive soil.

Jemal, Agon, and Geremew (2019), studied on “utilization of crushed stone dust as stabilizer” For the study area Jimma town and the main aim of the study was utilization of crushed stone dust as stabilizer for subgrade soil, also determine the optimum crusher dust percentage to be added. The crushed stone dust was added from 5% to 50 % to the natural soil. From the test result the subgrade soil were A-7-5 as per AASHTO soil classification system and CH as USCS. The CBR of soil increased from 1.86% to 5.561% with the increasing percentage of crusher dust 0 to 35% it was also identified that the addition of 30 % &35 % crusher dust yields high CBR values, MDD was increased and plasticity decreased with the addition of crusher dust. The final result indicted that crusher dust can be used as an embankment material, backfill material for lower layer of sub base.

The summary of articles reviewed is shown in table 2.5

Table 2. 7 summery of pervious related works

Year	Authors name	Title of the study	Mix percentages	Findings
2011	Ali & Koranne	Performance analysis of expansive soil treated with stone dust and fly ash	0-40 % by dry weight	Stone dust is more effective than fly ash to improve expansive soil
2015	Mohanty	Stabilization of expansive soil by using fly ash	0-50%	Fly ash decreased swelling and increase strength of expansive soil
2016	Indiramma &sudharani	Use of quarry dust for stabilizing expansive soil	0-25%	Quarry dust can be used as stabilizing agent for expansive soil
2019	Jemal & Geremew	Utilization of crushed stone dust as stabilizer	5-50%	Crushed SD used as an embankment and backfill material

In the previous section a review of available literature is presented and the following conclusion was made from reviews.

so far, in Ethiopia the combination of stone dust and coal ash to improve the expansive soil is not available in the literature. Similarly, the effect of basaltic stone dust on the nature of expansive soil is limited. Also, the cost comparison for stabilized soil and replacement of expansive soil by selected material is limited in the literature.

CHAPTER THREE

MATERIALS AND METHODS

The aim of this chapter is to discuss the materials, equipment's and testing method used in stabilization of expansive soil with Stone dust and Coal ash. All tests have been conducted in accordance with ASTM or IS.

The experimental work (Geotechnical tests and Engineering tests) were performed in Soil mechanics laboratory at Ethiopian construction design and supervision works corporation.

3.1 Description of the Study Area

Olonkomi town is located in, Ejersa Lafo district, West Shewa zone of Oromia reginal state of Ethiopia. It is suited in western part of Ethiopia at a distance of 60 km from Addis Ababa; On the asphalt road that leads to Ambo town and the Olonkomi town has about 112 km^{2}size with the of Altitude 2147.00 latitude 9North, longitude 38.25East the average temperature in between 11.54^{o}C-25.28 ^{o}C.

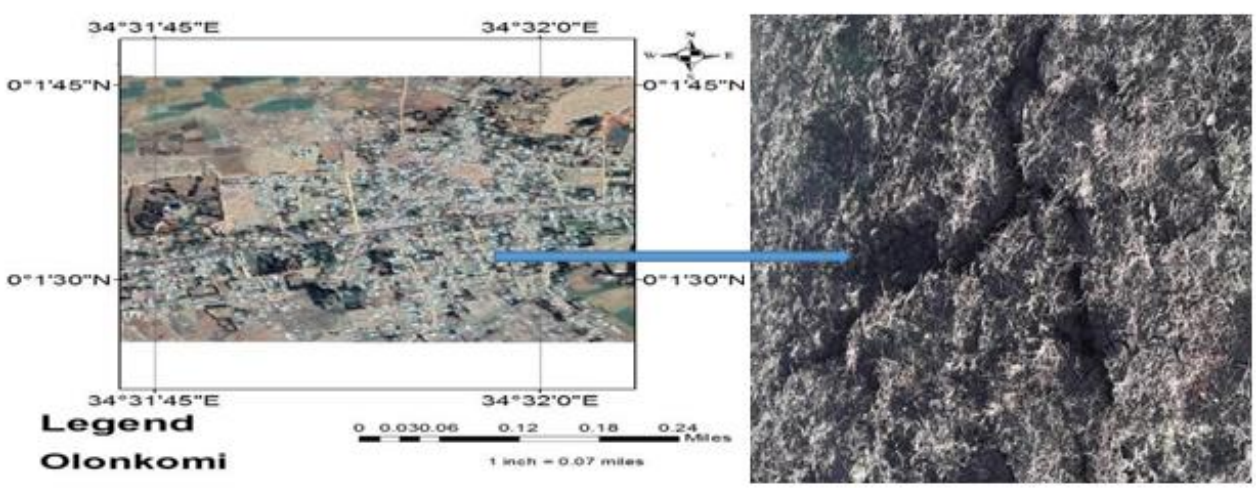


Figure 3. 1 location map of study area

3.2. Materials

The three materials used in present study are shown in Figure 3.2 and explained in detail in the following sections.

Expansive soil

Test Pits

Basaltic stone dust

Coal ash

Figure 3. 2 Photographic view of expansive clay soil, stone dust and coal ash used in present study

3.2.1 Expansive Soil

The expansive soil was collected from Olonkomi town, which is found in west shewa zone (Ambo zone) at a distance of 60 Km from addis Ababa.

The depth of investigation is 1.5m below the natural ground level from one test pit. The geotechnical properties of expansive clay soil are shown in Table 3.1. The grain size distribution of clay soil is shown in Figure 3.3.

The soil was classified as CH soil and A-7-5 hence this value indicate that the soil is highly plastic clay with poor shear strength and workability also it has poor subgrade material property.

Table 3.1 Physical properties of expansive soil used in present study

Properties	Value
Natural moisture content (%)	33.74
Specific gravity	2.77
Free swell (%)	125
Liquid limit (%)	81.76
Plastic limit (%)	36.04
Plastic index Ip)	45.72
Linear shrinkage (%)	20

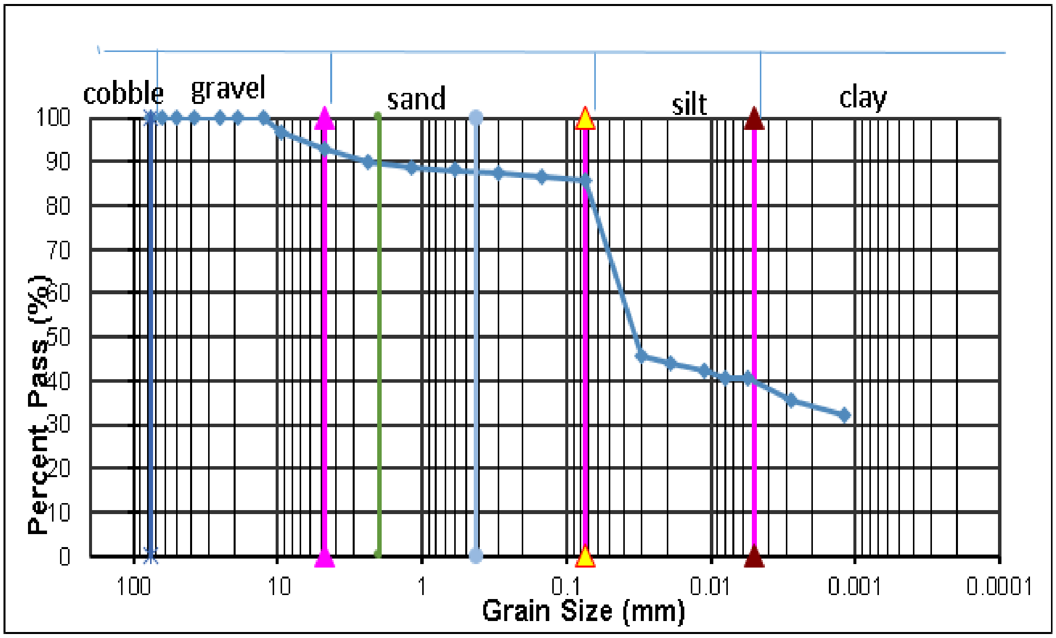


Figure 3. 3 Grain size distribution curve for ES

3.2.2 Stone Dust

The basaltic stone dust was collected from stone crusher plant which is found in Olonkomi town. The Physical properties of stone dust shown in Table 3.2 and the grain size distribution of basaltic stone dust is shown in Figure 3.4.

Table 3.2 Physical properties of stone dust used in present study

Properties	Value
Natural moisture content (%)	6.60
Specific gravity	2.47
Free swell	5
Liquid limit	27.5
Plastic limit	NP
Plastic index	-

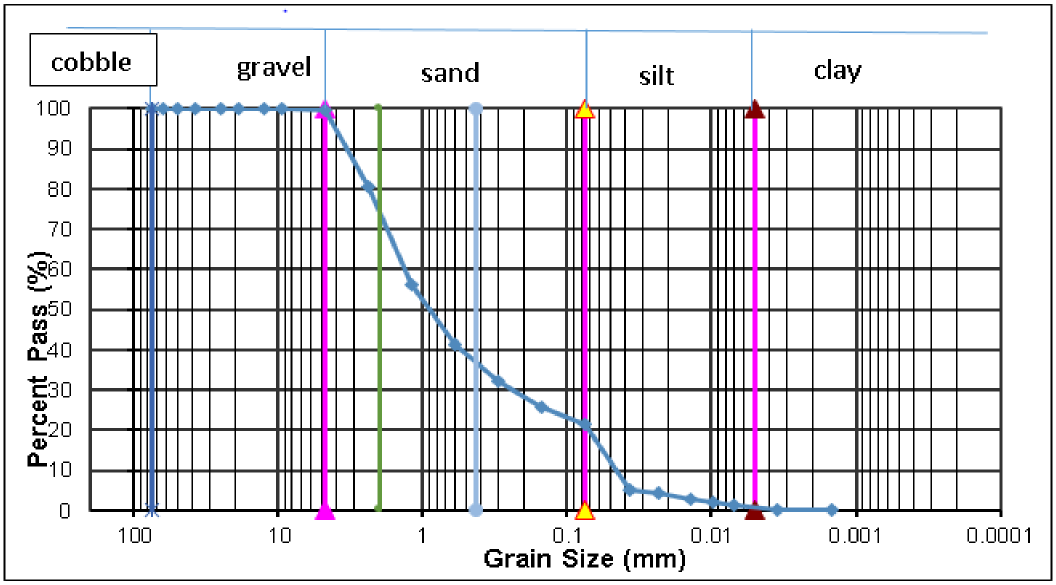


Figure 3. 4 Grain size distribution curve for SD

3.2.3 Coal ash

The type of coal ash was used for this study is called fly ash, which is one of the coal composition products and it was collected from Ayka Addis textile and investment. The Physical properties of coal ash shown in Table 3.3 also the grain size distribution of clay soil is shown in Figure 3.5. The classification of fly ash collected from Ayka addis is categorized under class F fly ash because the sum of silicon oxide, aluminum oxide and iron oxide are 88 % (kasshaun, 2019).

Table 3.3 Physical properties of coal ash used in present study

Properties	Value
Natural moisture content (%)	5.66
Specific gravity	2.47
Free swell	10
Liquid limit	35.72
Plastic limit	NP
Plastic index	-

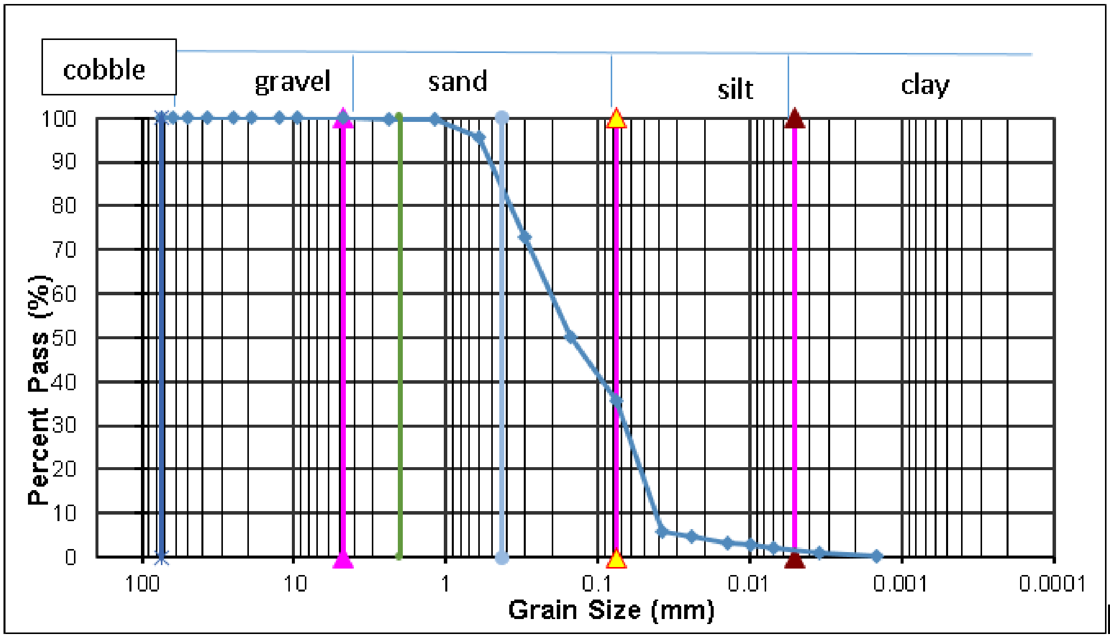


Figure 3.5 Grain size distribution curve for CA

3.3. Experimental Program

In this section, the experimental program was divided in to two parts. In the first part, the geotechnical properties of the soil, stone dust and coal ash were characterized. In the second part, stabilization of the soil was carried out and optimum mix was determined and the detailed experimental program of the present study is given in Table 3.4.

Table 3.4 Testing Mechanism for Clay-Stone Dust-Coal ash

MATERIALS USED Mix Proportion (%)			Particle size distribution	Atterb erg limit	Proctor compaction	UCS	CBR
Clay	SD	CA					
85	5	10	x	x	X	x	x
80	10	10	x	x	X	x	
75	15	10	x	x	X	x	
75	5	20	x	x	X	x	
70	10	20	x	x	X	x	
65	15	20	x	x	X	x	
65	5	30	x	x	X	x	
60	10	30	x	x	X	x	
55	15	30	x	x	X	x	

3. 4. Methods

3.4.1 Sample Preparation

3.4.1.1 Clay-Stone Dust-Coal ash

The specimen in Clay-stone dust-coal ash mix has been prepared with clay soil containing 5 to 15 percent of stone dust in increments of 5 percent for each combination of 10,20 and 30 percent of coal ash by dry weight mixture. Then water, equal to optimum mixture content was added to mixture and a uniform wet mix was obtained by proper mixing.

The main purpose of the present study is to determine the strength of the Clay-stone dust-coal ash mixture. The optimum mix was determined by the following techniques:

The optimum mix was determined from the unconfined compressive strength test results which will discussed in chapter 4. Based on the maximum unconfined compressive strength value the optimum mix was found.

3.5. Test Procedures

All tests have been conducted in accordance with ASTM or IS

In this section methods used for this study are discussed briefly.

- Visualizing expansive soil in the area was done and then locations of the test pit was selected, the excavation work conducted by daily laborers using local digging equipment the excavation was continued up to 1.5 m depth below natural ground level then required amount of undisturbed soil sample taken by shell and disturbed soil samples carried to the laboratory in sack.
- To evaluate the effect of stone dust and coal ash on expansive soil series of the laboratory tests, were conducted by varying the percentage of coal ash and stone dust. In the mixing process the effect of stabilizer on Atterberg limit, USC, CBR and proctor compaction were done by combing both stabilizers but on Atterberg limits and free swell index test it was observed that their individual effect to decide the percentage of stabilizers And also the test which are performed in the laboratory are mentioned below.

3.5.1. Grain size analysis

This test was performed to determine the percentage of different grain sizes contained within a soil. The grain size was obtained from the mechanical sieve shaker and the hydrometer analysis, all the samples were passed through 9.5,4.75,2.,1.18,0.6,0.3,0.15,0.075mm mechanical sieve and soil particle size smaller than 0.075mm are determined by hydrometer test. The photographic view of mechanical sieve is shown in Figure 3.6.

Equipment used to perform this test were; Balance, set of sieves, Cleaning brush, Sieve shaker, Mixer (blender), 152H Hydrometer, Sedimentation cylinder, Control cylinder, Thermometer, Beaker, Timing device. By using ASTM D 422 standard test method for particle – size analysis.

Figure 3.6 The photographic view of mechanical sieve

3.5.2. Natural moisture content test

This test was performed to determine the water (moisture) content of soils. The water content is the ratio, expressed as a percentage, of the mass of "pore" or "free" water in a given mass of soil to the mass of the dry soil solids. Equipment used for this test were mentioned as follows; Drying oven, Balance, Moisture can, Gloves, Spatula. By using ASTM D2216 standard test methods for laboratory determination of water (moisture) content of soil.

3.5.3. Specific gravity test

The specific gravity determination of a sample of soil ,stone dust and coal ash were conducted by displacement in water using pycnometer, in this test a 10 gram of oven dried soil samples carefully put in the 100 ml pycnometer which is then half filled with distilled water .The air entrapped in the soil sample is removed by heating the bottle is then topped up with distilled water up to a calibration mark and brought up to a constant temperature after carefully wiped and dried it is weighed .It was performed according to ASTM D 854 test method.

3.5.4. Atterberg limit test

The test is a consistency Limit identification test on the basis of moisture content. It includes, the determination of; the liquid limits, plastic limits, shrinkage limit for the natural soil and the soil-stone dust and coal ash mixture. The tests were conducted accordance with ASTMD4318 testing procedures. Soil sample used for liquid limit, plastic limit and shrinkage limit determination was air dried passed through No. 40 sieve (425μm) pass. about 100 gm of soil was mixed with distilled water into a uniform paste and placed on Casagrande cup and leveled , by standard grooving tool the soil sample was cut at the center, the handle was next turned at a rate of about two revolution per second then number of blows counted between 10 to 40 blows to close the groove and the water content near the closed groove is find out up to four trials the procedure was continued .Then After the graph of water content versus no of blows was plotted , the water content corresponding to 25 blows determined as liquid limit. For coal ash and stone dust the liquid limit was determined by cone penetration method. The same procedure is also carried out for the soil treated with varied contents of stone dust and coal ash. And the plastic limit was determined by mixing the soil with water and rolling the soil up to 3mm diameter, the procedure of mixing and rolling is repeated till the soil signs of crumbling when the diameter is 3 mm. The average water content of the crumbled portion of the thread is determined this is called plastic limit. The photographic view of liquid limit test is shown in Figure 3.7.

For determination of shrinkage limit, a container of 50mlvolume is filled with an expansive soil, soil- stone dust and coal ash mixture in saturated state and the weight of the saturated soil was determined The specimen was dried gradually first in air and then in an oven at a constant temperature of 105 °c after oven drying the specimen weighed .

Figure 3. 7 The photographic view of liquid limit test

3.5.5. Compaction test

This laboratory test was performed to determine the relationship between the moisture content and the dry density of a soil for a specified compactive effort. The standard Procter test was done for both original and mixed soils. All samples used for this study were an air -dried passed through sieve number 4 (4.75mm) sieve and mixed with a measured quantity of water and was compacted in the mould in there equal layer and each layer was compacted in 25 blows after compacted the representative soil sample was taken to determine its water content. after the compaction process was completed the MDD and OMC obtained from dry density versus water content graph. Equipment used are;

Molds, Manual rammer, Extruder, Balance, drying oven, Mixing pan, Trowel, #4 sieve, Moisture cans, Graduated cylinder, Straight Edge. By using ASTM standard D 698-12e compaction characteristics of soil using standard effort. From this test maximum dry density and optimum moisture content were determined.

3.5.6. Free swell index test

Free swell test is used to determine the free swell index of soil. The expansive soil, stone dust and coal ash which used for free swell test were passed through 452 µm, and all the samples were added to 100ml cylinder in which one was filled with distilled water and the other one was filled with kerosene up to 100 ml marked. Then letting the content for 24 hrs until all the sample completely settled to the bottom of cylinder, after 24 hrs the final volume of samples in each cylinder was read out and the free swell index value was the average value of the two readings. Equipment used

Oven105 ^{o}C to 110 ^{o}C, balance, sieve 425-micron, graduated glass cylinder 100mlcapacity, referring to IS:2720 (part40)19985- method of test for determination free swell index of soil.

3.5.7. Unconfined compressive strength test

The primary purpose of this test is to determine the unconfined compressive strength, which is then used to calculate the unconsolidated undrained shear strength of the clay under unconfined conditions. According to the ASTM D 2166, the unconfined compressive strength (qu) is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test.

All samples used for this test were prepared passing through 4.75mm sieve and the test specimen shells have a diameter of 38mm with length 76 mm. The photographic view of Unconfined compressive strength test is shown in Figure 3.8.

Equipment used: Compression device, sample extruder, deformation indicators, dial comparator, timer, balance. By using ASTM D 2166-13 standard test method for unconfined compressive strength of cohesive soil.

Figure 3. 8 The photographic view of Unconfined compressive strength test

3.5.8. California Bearing Ratio test

The California Bearing Ration test was performed according to ASTM D1883. Equipment used: Moulds, a separate base plate, an extension collar and a spacer disk, mechanical compaction rammer, surcharge weight to simulate the effect of overlaying pavement weight, CBR machine.

CHAPTER FOUR

RESULT AND DISCUSSION

In this chapter, laboratory test results are presented and their analysis have been briefly discussed. The relevant engineering properties of the soil evaluated both for natural and blended soil sample separately and also for the stabilizer the laboratory test was conducted to determine their engineering properties. The test result includes Atterberg limits, grain size analysis, Free swell, Proctor compaction, linear shrinkage, unconfined compressive strength, specific gravity and natural moisture content.

4.1 Laboratory Test Results

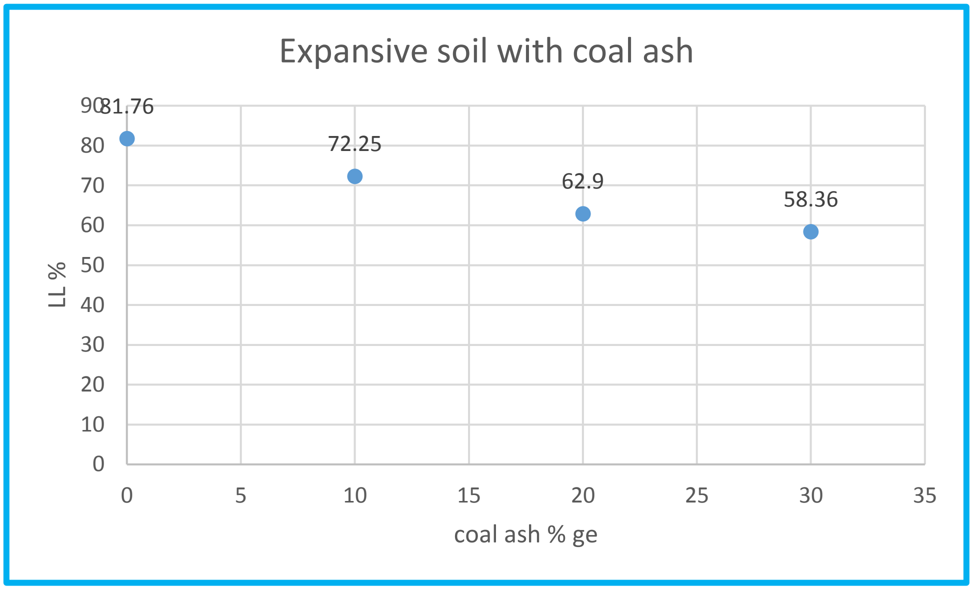
In this section the laboratory tests have been discussed on the effect of coal ash and stone dust, in the first section their individual effect on Atterberg limit and free swell tests and in the second section their combined effect on Atterberg limit, Free swell, grain size, Compaction, UCS and CBR laboratory tests.

4.1.1 Effect of Coal ash on Atterberg limits

For each percentage of coal ash, Atterberg limit tests have been conducted according to ASTM D 4318 as the percentage of coal ash increase LL and PI are decreased. This is due to non- plastic properties of coal ash and the expansive soil is improved from CH to MH class according to USCS. The summaries of Atterberg limits of soil with coal ash is presented in the Table 4.1 and effect of coal ash on LL and PI of the soil is shown in Figure 4.1.

Table 4.1 Summary of Atterberg limit for different combination of CA with ES

Percent of CA	LL	PL	PI
0	81.76	36.04	45.72
10	72.25	36.56	35.69
20	62.90	39.59	22.90
30	58.36	35.56	22.79



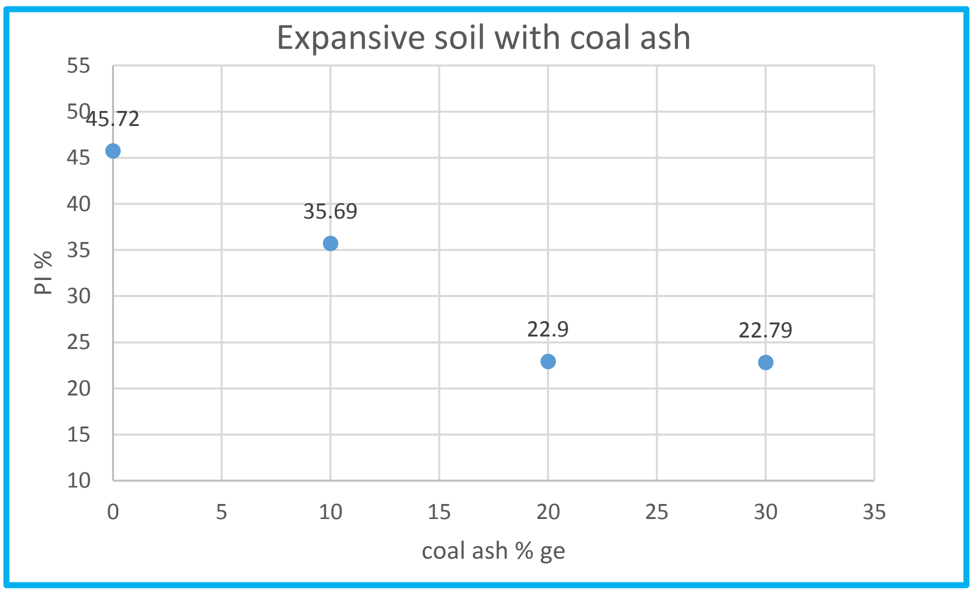


Figure 4.1 Effect of coal ash on LL and PI

4.1.2 Impact of stone dust on Atterberg limit

AS shown in the table below as the stone dust increase LL reduced from the LL of original soil and the impact of stone dust on plasticity index of expansive soil was indicated that as the stone dust increased the plasticity index were decreased from the original soil. This is due to the fine particle of clay soil are decreased. The test result summary for Atterberg limits of soil with stone dust sown in Tale 4.2.

Table 4.2 Summary of various combinations of SD with ES

Percent of SD	LL (%)	PL (%)	PI (%)
0	81.76	36.04	45.72
5	71.00	32.96	38.04
10	70.00	33.00	37.00
15	69.76	32.92	36.84

4.1.3 Impact of Coal Ash and stone dust on Free swell of Expansive soil

Free swell test helps to know the expansiveness of a soil sample that is by measuring the volume of a soil sample that is soaked and settled down in water for 24 hours. The free swell for each percentage of coal ash and stone dust blended are presented in figure 4.2 and figure 4.3. From test results for 10 % of coal ash, the value of free swell shows improvement and 5 % of SD observed that significant improvement, therefore the blended soil changed from highly expansive to medium or marginal.

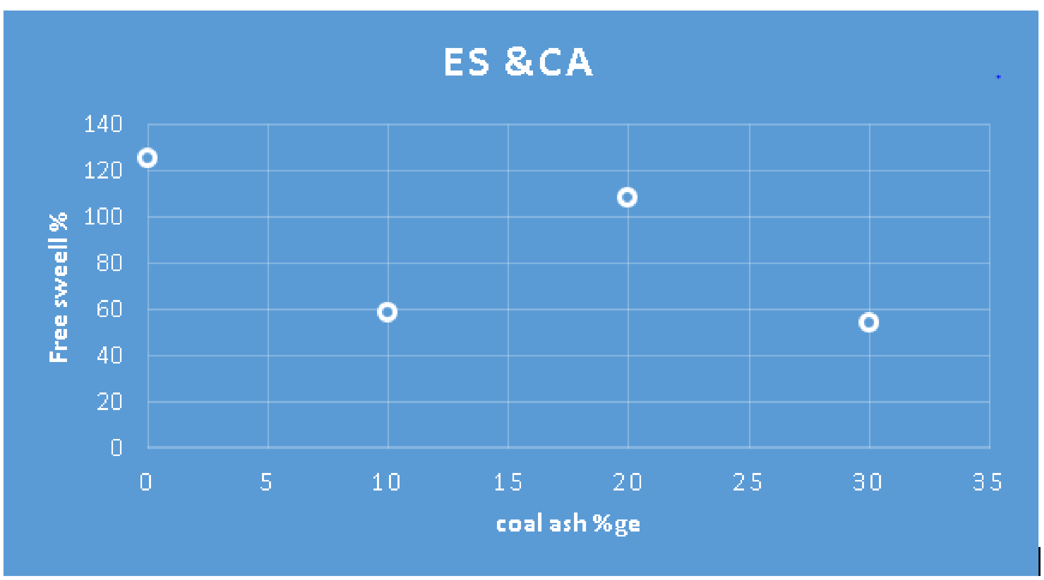


Figure 4. 2 Impact of coal ash on free swell of Expansive soil

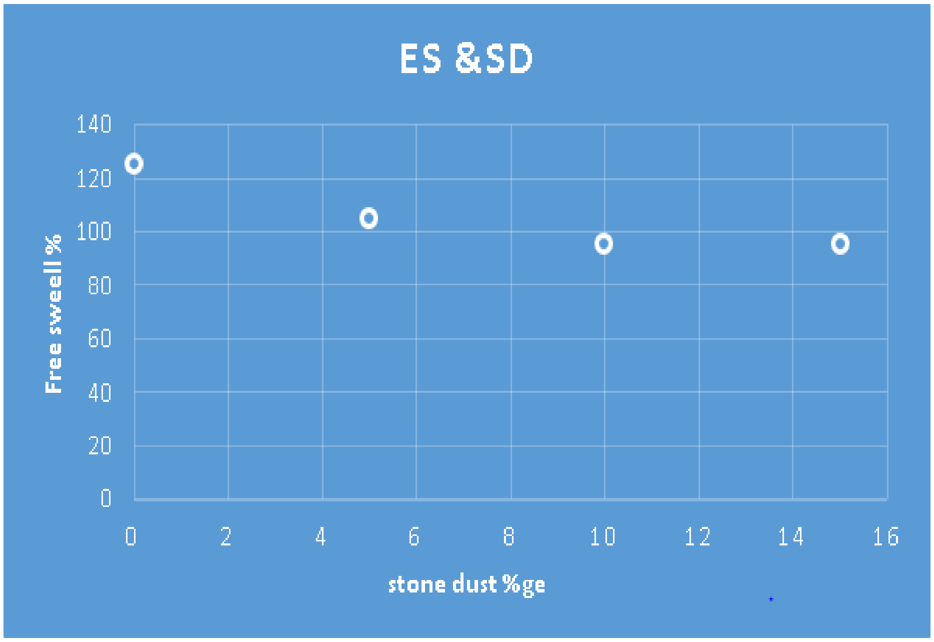


Figure 4.3 Impact of stone dust on Free swell of soil

4.1.4 Influence of Stone dust and Coal ash on grain size analysis

The test was conducted according to ASTM D422 in order to determine the particle size distribution of fine and coarse materials by sieving and hydrometer analysis and the summary of results obtained are plotted in figure4.6 below and the laboratory test analysis are given in Appendix -A.

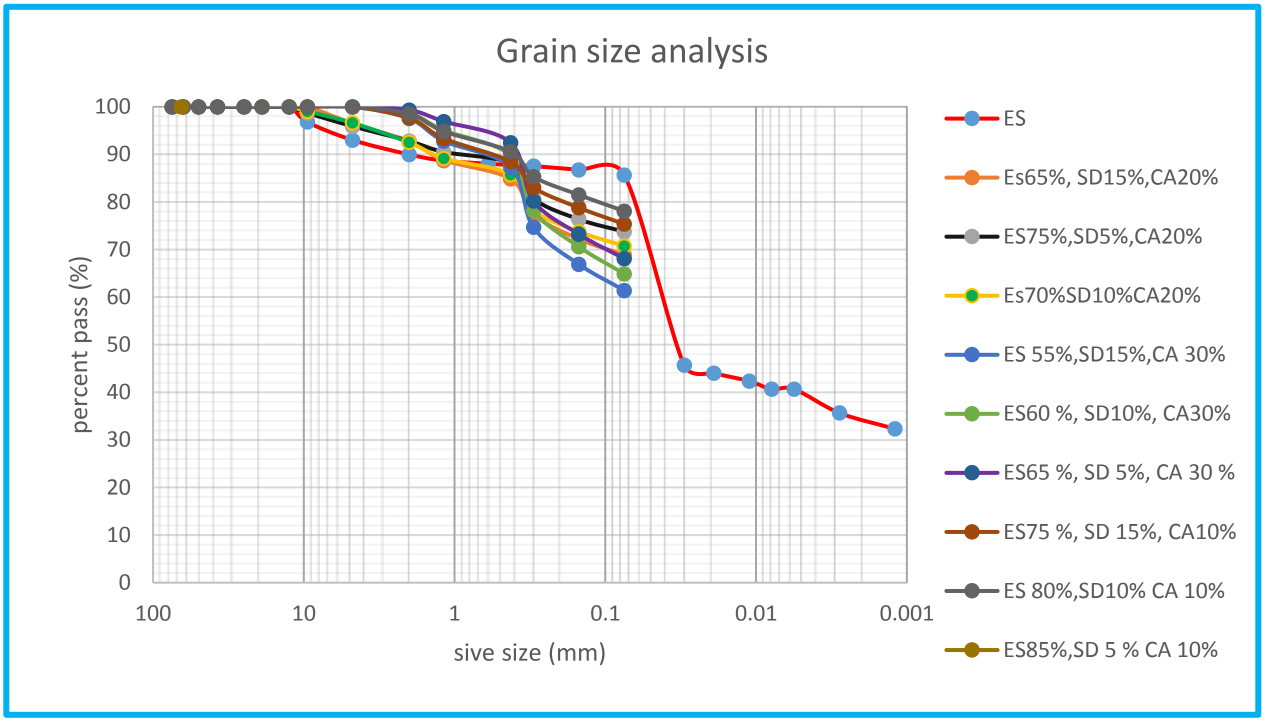


Figure 4.4 Combined effect of CA and SD on gradation of ES

From the grain size distribution curve of various combination of SD &CA with expansive soil it is observed that the curve moves from right to left which imply that the percent of particles size finer than a specific size is reduced so that the particle size distribution of the blended soils are improved. Generally, the number of fine particles presented in the expansive soil that are passing sieve No 200, are decreased after blending with the SD&CA. Thus, the plasticity of soil is reduced.

4.1.5 Influence of Stone dust and Coal ash on Atterberg limit

The Atterberg limit tests were conducted according to ASTM D 4318 for soil, and soil mixed stone dust and coal ash. The summary of the test results is shown in Table 4.3. The liquid limit value reduced by 38.69% for a mixture of 15% Stone dust and 30% of Coal ash. Reduction in plastic limit value for 5,10,15% of Stone dust with 10% Coal ash each are respectively1.04, 5.56, 7.77% respectively. As same reduction in plasticity index for 5,10,15% of Stone dust with 10 % Coal ash each are respectively 45.93,61.04,64.46% respectively. The linear shrinkage reduction from 20% to 17 % from 85% ES+5%SD+10CA to that of Clay soil.

The reduction in LL, PL and PI is due to the non-plastic properties of stone dust and coal ash and the number of fine particles present in clay soil decreased by the addition of SD and CA.

Table 4.3 Summaries of influence of CA and SD on Atterberg limit

Percent of ES, SD& CA	Liquid limit	Plastic limit	Plasticity index	Linear shrinkage
Pure ES	81.76	36.04	45.72	20
85Es+5SD+10CA	67	35.67	31.33	17
80Es+10SD+10CA	62.54	34.14	28.39	-
75Es+ 15SD+10CA	61.24	33.44	27.80	
70Es+10SD+20CA	68.87	30.37	38.50	
75Es+5SD+20CA	68.21	39.13	29.07	19
65Es+15SD+20CA	61.08	34.58	26.58	
65Es+5SD+30CA	73	34.02	38.98	
60Es+10SD+30CA	59.63	27.63	31.76	
55Es+15SD+30CA	58.95	32.76	26.18	12

4.1.6 The effect of Stone dust and Coal ash on Free swell

The Free swell tests were conducted for soil, soil mixed with stone dust and coal ash in accordance with IS 2720. Each test repeated twice. The result for average Soil, Soil mixed with stone dust and coal ash are shown in Table 4.4. The application of 15% SD and 30% CA on the expansive soil results in reduction of free swell of the soil to 212.5% to that of soil alone and this was due cementing effect of Stone dust.

Table 4.4 Effect of SD and CA on free swell of ES

Percent of ES, SD & CA	Free swell (%)
100% ES	125
85Es+5SD+10CA	82
80Es+10SD+10CA	54
75Es+ 15SD+10CA	50
70Es+10SD+20CA	49
75Es+5SD+20CA	52
65Es+15SD+20CA	46
65Es+5SD+30CA	46
60Es+10SD+30CA	43
55Es+15SD+30CA	40

4.1.7 Influence of Stone dust and Coal ah on moisture density relationship

The standard proctor tests were conducted for soil, soil mixed with stone dust and coal ash in the proctor mould with 944cm^3 in three equal layers. Each test repeated four times. The moisture density relationship for Soil alone shown in Figure 4.7. and Figure 4.8 shows the moisture density relationship for soil- stone dust- coal ash mixtures. The optimum moisture content found to be 35% and the maximum dry density are 12.3kg/m^3. The result for average soil, soil mixed with stone dust and coal ash are shown in Figure 4.8. It is observed that the optimum moisture content has decreased by 11% and maximum dry density has increased from 12.3kg /m^3 to 13.7kg/m^3 for 15 % SD and 10% CA.

The application of SD and CA increases the MDD as well increase from the original soil and the OMC decreases this is due to the clay particles acts as binding material between in the SD

& CA particles. Additionally, SD & CA needs less moisture content to achieve MDD since their surface area are less than that of clay particles.

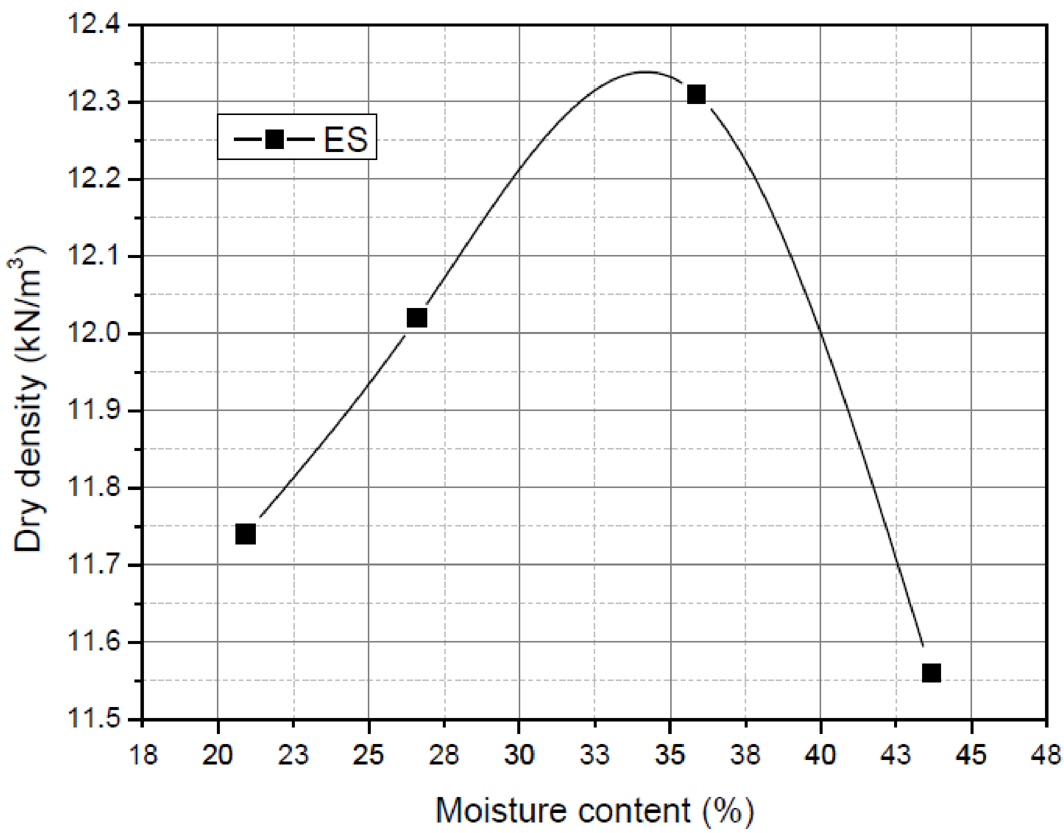


Figure 4.5 Moisture – density relation for soil alone

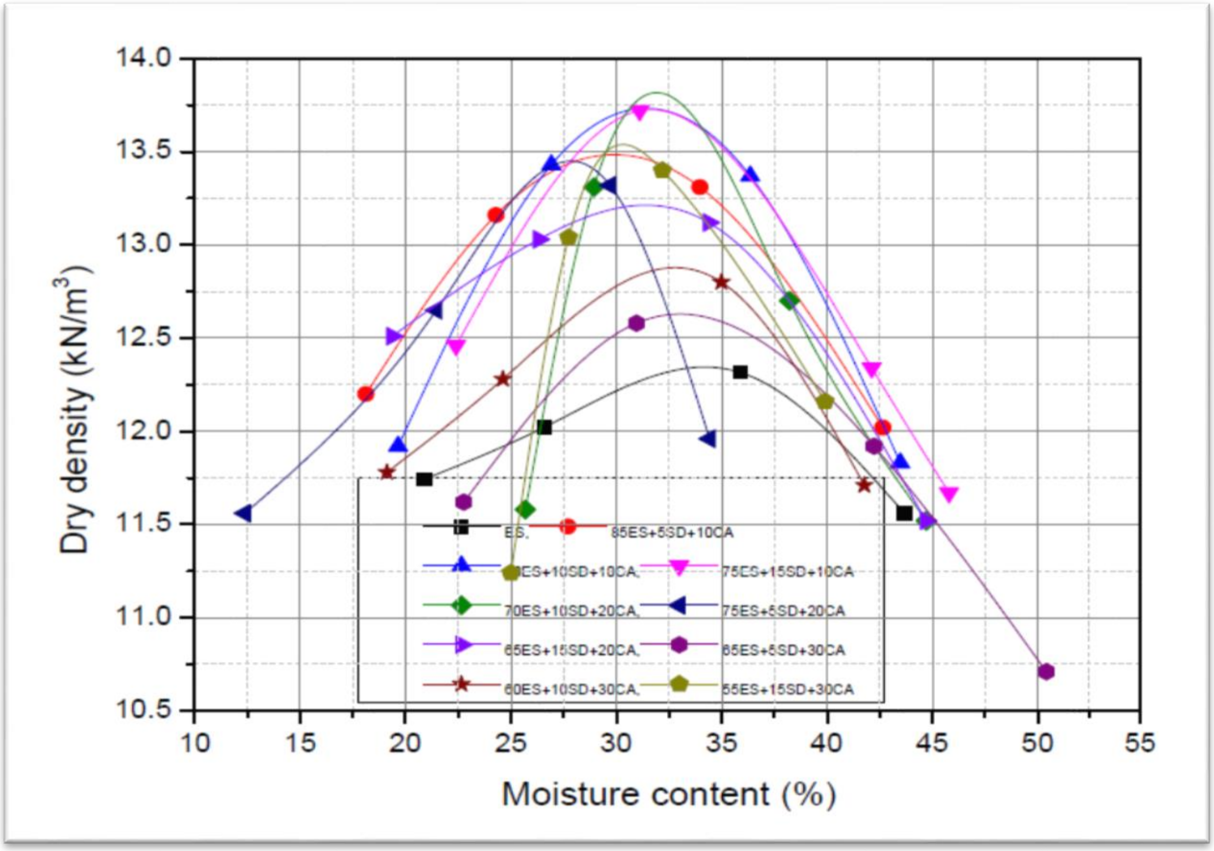


Figure 4. 6 Moisture density relationship for Mixture of soil- stone dust – coal ash

4.1.8 Influence of Stone dust and Coal ash on Unconfined Compressive strength

In this test the soil is subjected to axial load without a confining pressure. It is a special type of triaxial compression. As the percentages of both type of stabilizers increase, the unconfined compressive strength also increased but it was decreased from the UCS value of the original soil on 30 % of CA and the maximum value of UCS is observed on 85% ES, 5% SD & 10% CA. The mixture of a samples (ES, SD and CA) are prepared at their respective MDD and OMC for unconfined compression test. The test result summary for soil and soil- stone dust – coal ash mixtures are shown in Table 4.5,The UCS results goes to in decreasing after 10% of CA in all mix proportions as shows in Figure 4.8, 4.9 and 4.10 this is due to lack of cohesion or the addition of more cohesionless material to clay soil reduced its natural cohesive forces between particle of clay soil.

Table 4.5 Test result summaries of UCS of soil mixed with CA & SD

Percent of ES, SD& CA	UCS (KN/m^2)	C_U (KN/m^2)
100% ES	158	79
85Es+5SD+10CA	237	118.5
80Es+10SD+10CA	164	82
75Es+ 15SD+10CA	174	87
70Es+10SD+20CA	162	81
75Es+5SD+20CA	202	101
65Es+15SD+20CA	191	95.5
65Es+5SD+30CA	94	47
60Es+10SD+30CA	113	56.5
55Es+15SD+30CA	148	74

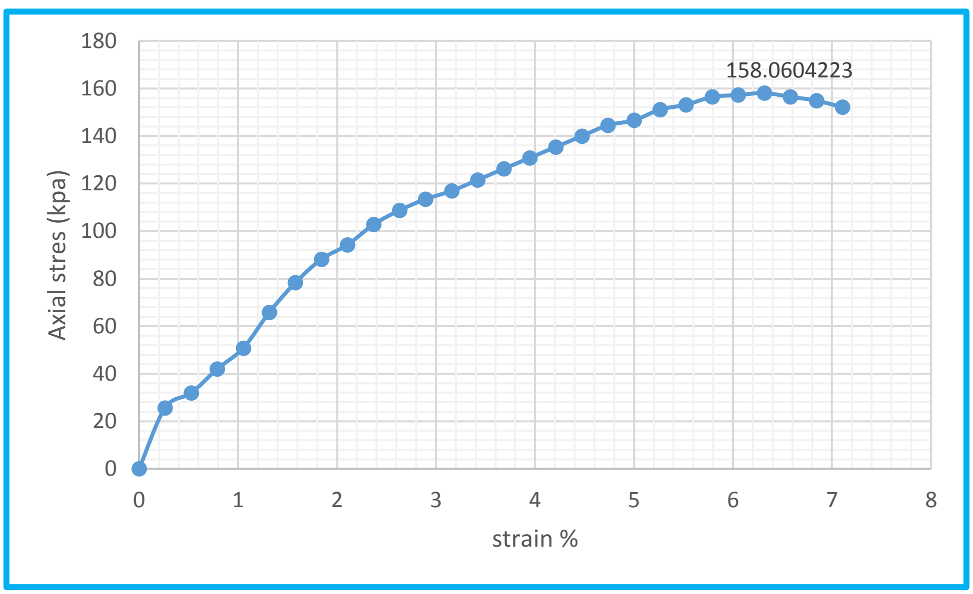


Figure 4. 7 UCS test result for expansive soil

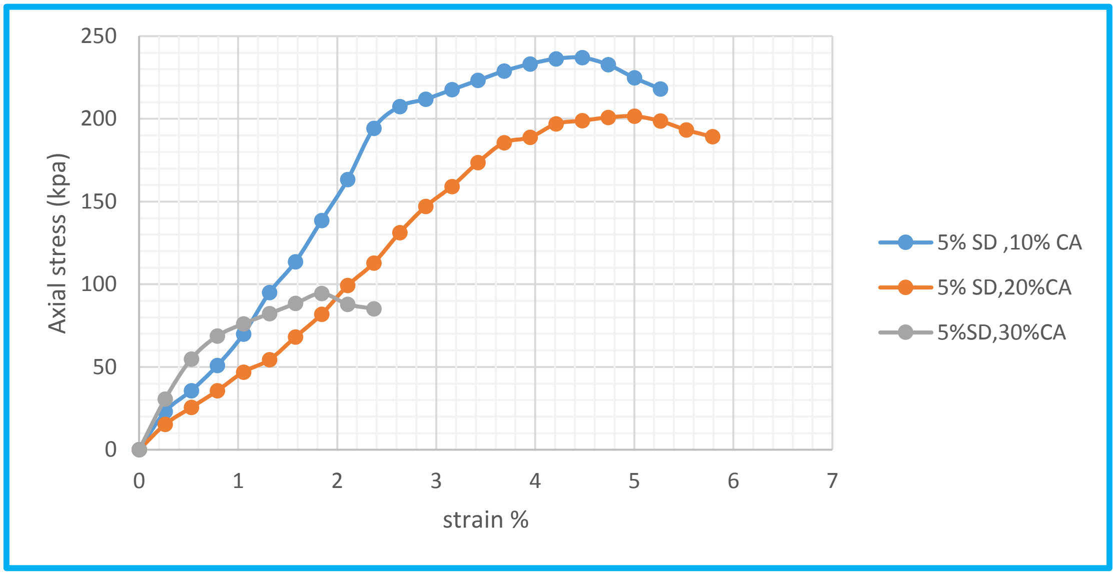


Figure 4. 8 UCS test result for the mixture of 5 % SD and 10,20,30 % of CA

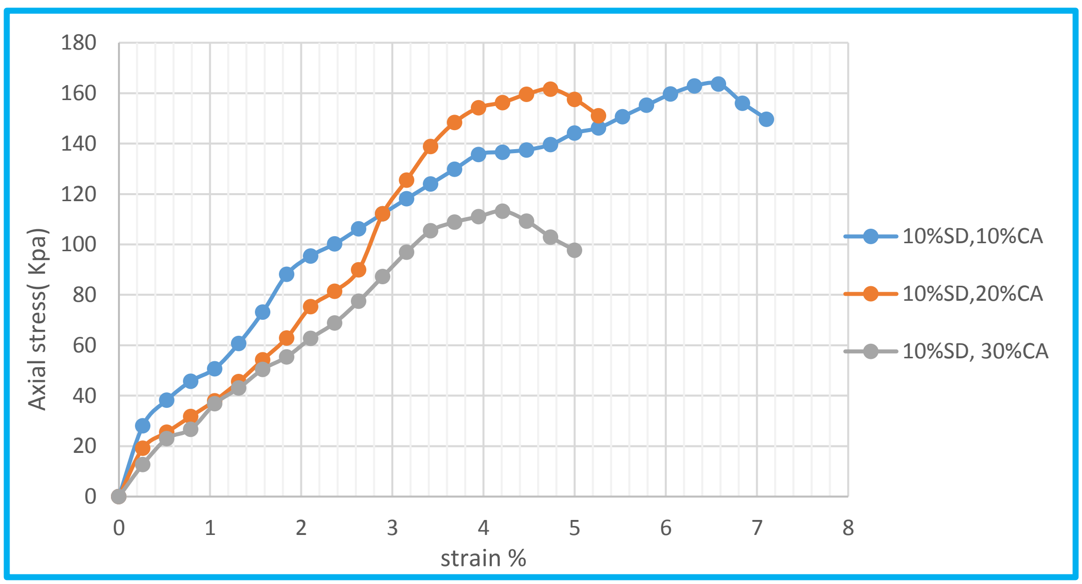


Figure 4. 9 UCS test result for the mixture of 10 % SD and 10,20,30 % of CA

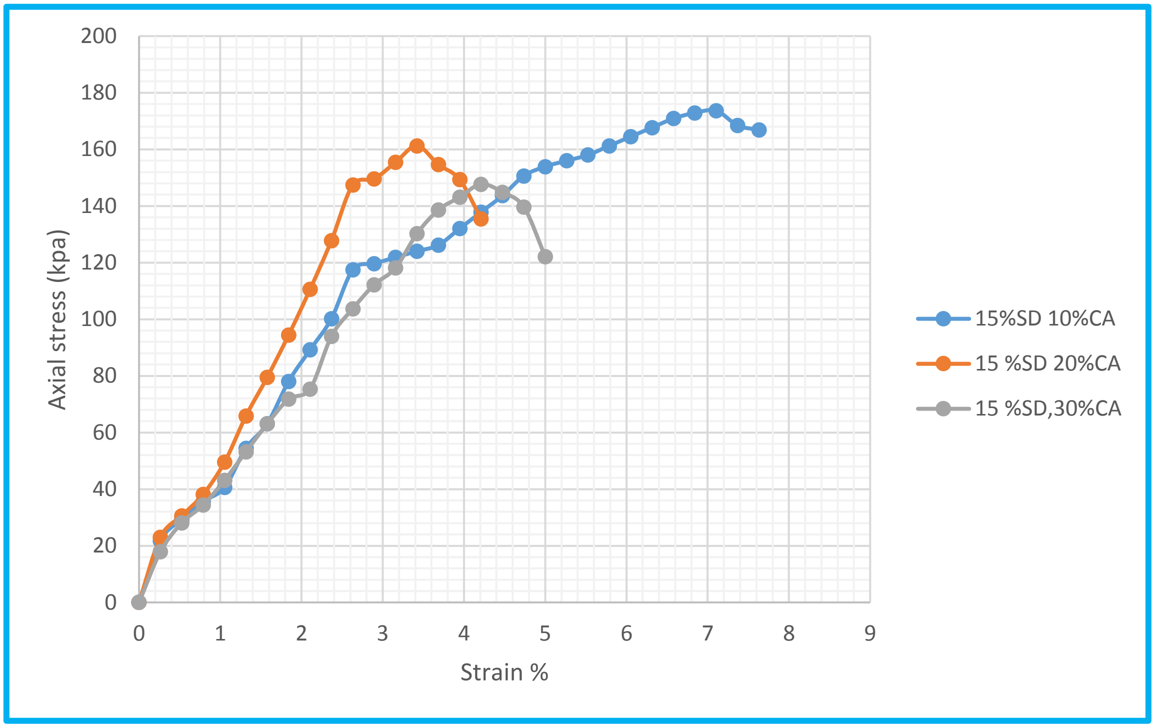


Figure 4. 10 UCS test result for the mixture of 15 % SD and 10,20,30 % of CA

4.1.9 The effect of CA and SD on soaked CBR of ES

The CBR test were carried out for ES and ES, SD & CA combinations for their respective optimum moisture content and maximum dry density after determining by standard compaction. one-point CBR were conducted on selected combination of blended soil (85 ES, 5 SD & 10 CA), the CBR test result for the expansive soil and the optimum mix is shown in figure 4.11 and figure 4.12 respectively.

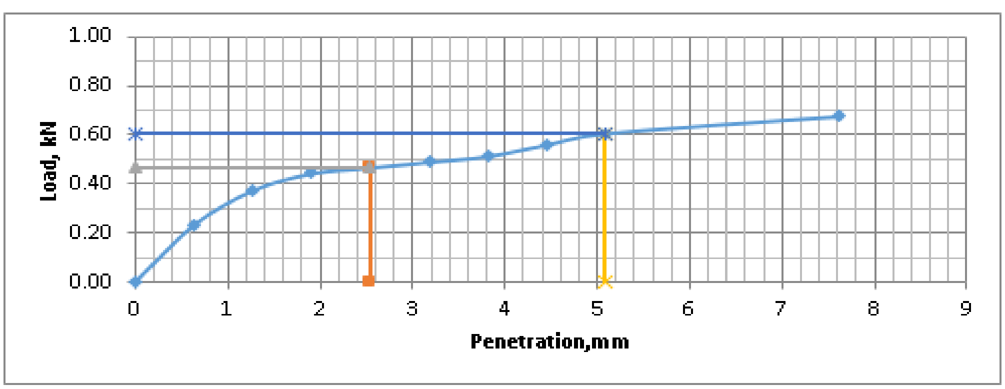


Figure 4.11 CBR test result for ES

Table 4.6 CBR test result

Penetration, mm	Load, KN	Standard load, KN	CBR%	% Swell
2.54	0.47	13.2	3.54	7.02
5.08	0.61	20.0	3.04	

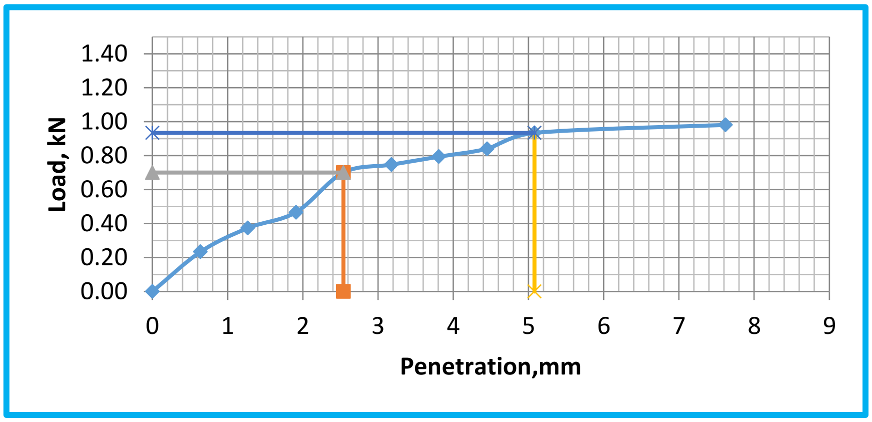


Figure 4. 12 CBR test result for 85 ES, 5SD & 10 CA

Table 4.7 CBR test result for blended soil

Penetration(mm)	Load (KN)	Standard load (KN)	CBR %	% swell
2.54	0.70	13.2	5.31	5.17
5.08	0.93	20.0	4.67	

4.2 Discussion on laboratory test result of original soil and blended soil

The laboratory tests were conducted on disturbed and undisturbed soil samples. Tests such as Atterberg limits, particle size analysis, linear shrinkage, free swell index, proctor compaction, CBR, and UCS of a soil was performed by both undisturbed and remolded soil samples in order to know the sensitivity of clay.

As the test result indicated that the original soil has a natural moisture content 33.74 and the specific gravity of soil is 2.77. On the particle size distribution curve almost 88.88 % of the soil is passing through No.200 sieve; it shows a liquid limit of 81.76 %, plastic limit 36.04 % and plasticity index of 45.72%. Based on unified soil classification system the soil is CH, hence this value indicate that the soil is highly plastic clay with poor shear strength and workability. According to AASHTO the soil fails under the A-7-5 soil class which have poor subgrade material property. Result that is related to swelling characteristics of the soil also indicated that the soil is highly expansive with the free swell value of 125 %.The soil has a maximum dry density of 1.23gm/cc ,optimum moisture content of 35 % and also the unconfined compressive strength for remolded soil sample is 158KN/m^2 with cohesion of 79 KN/m^2 ,were as for undisturbed soil sample the USC value is 55 KN/m2 has cohesion of 27.5KN/m^2 so that according to K. Arora 1987 soil is in sensitive and stiff clay. And the soaked CBR value is 3.54 % and the CBR % swell is 7.02%.

From the test result of blended soil, As the percentage of SD and CA increased LL, PI, SL are decreased this is due to the non-plastic properties of stone dust and coal ash and the number of fine particles present in clay soil decreased by the addition of SD and CA. On the individual SD and ES mix the highest reduction on LL and PI gained on 5 % of SD, also the lowest value of LL & PI observed on 30 % CA &70 ES mix. On the combination of SD, CA and ES the lowest value of LL and PI gained by 15 % SD and 30 % CA.

The application of SD &CA on the expansive soil results in reduction in free swell of the soil, this is due to SD and CA controls swelling of soil by cementing SD& CA mix.

10% SD & 30%CA has result 79% decreases in FSI. Thus, the blended soil is changes from highly expansive to low expansive group.

From the grain size distribution curve of various combination of SD &CA with expansive soil it is observed that the curve moves from right to left which imply that the percent of particles size finer than a specific size is reduced so that the particle size distribution of the blended soils are improved. Generally, the number of fine particles presented in the expansive soil that are passing sieve No 200, are decreased after blending with the SD&CA. Thus, the plasticity of soil is reduced.

Based on the Unified soil classification system all combination of CA with ES has fallen under the group MH, also 10SD combination fall under MH but the rest combination of SD is under CL group. For the combination of SD& CA (10SD+20CA, 5 SD+30CA,10SD+30CA) has fallen under CL group but the other combinations are fallen under MH group. The result was showed in the figure 4.13 below.

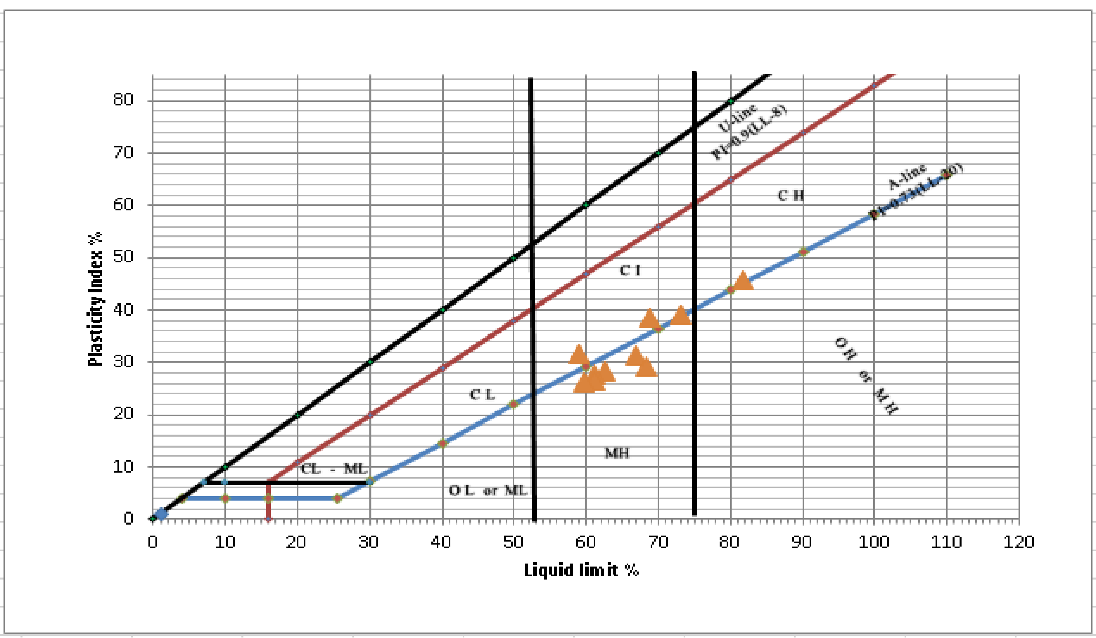


Figure 4.13 Classification of soil based on Unified soil classification system

On the other hand, using ASSHTO classification system all combination of ES, SD &CA are under A-7-5 class the results are showed in the figure 4.14 below.

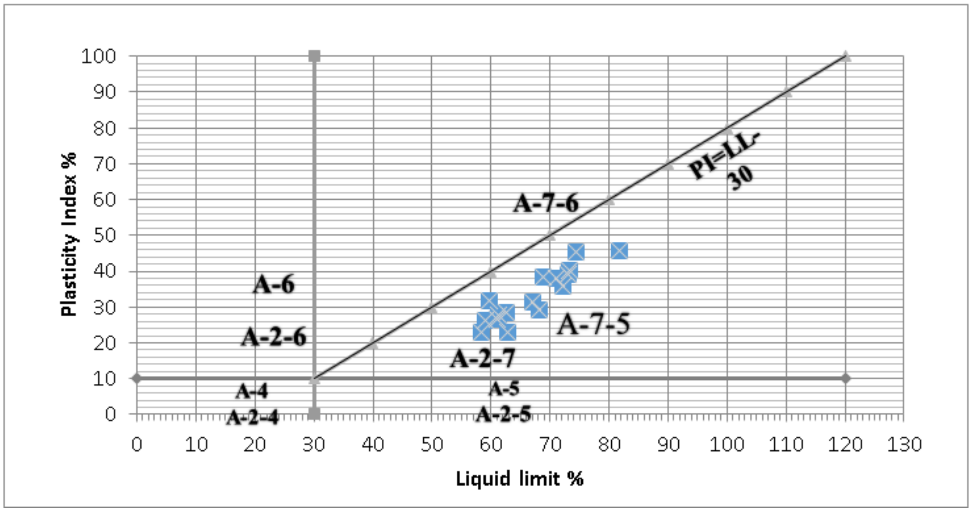


Figure 4.14 Soil classification based on AASHTO classification system

The application of SD and CA increases the MDD as well increase from the original soil and the OMC decreases this is due to the clay particles acts as binding material between in the SD& CA particles .Additionally SD&CA needs less moisture content to achieve MDD since their surface area are less than that of clay particles .

The increment in unconfined compressive strength of the new mix is greater for small percentage of SD &CA (5 SD &10 CA) blended.

CHAPTER FIVE

COST COMPARISONS

The cost comparison was done for typical G+1 residential building based on two options ,the first one is the conventional construction methods that is replacement of the expansive soil by non-expansive soil and the second option is calculated based on substructure work constructed using stabilized expansive soil of 5 % stone dust and 10% coal ash by dry weight of the expansive soil. During collection of materials costs, the cost of stone dust is actual in the market of Olonkomi town and the coal ash has only transportation and labor cost.

The sample calculation used for stabilization works are listed below and in Appendix F table F-1 shows the cost of excavation and earth work for G+1 residential building by replacing with non-expansive soil and table F-2 shows cost of excavation and earth works by using an expansive soil stabilized by 5% stone dust and 10% coal ash .

Sample calculation

In 100 % of sample the amount of stone dust and coal ash needed to stabilize are 5% stone dust and 10% coal ash and the expansive soil is 85 %.

5 % of stone dust and 10 % of coal ash by weight was defined as: from 100 kg of expansive soil we need 5kg of stone dust and 10 kg of coal ash to stabilize 85kg of soil.

Specific gravity of soil $= \frac{\text{unit weight of soil}}{\text{unit weight of water}}$.........5.1

Unit weight of soil = specific gravity * unit weight of water

Where SG of soil = 2.77 and unit wight of water = 1000kg/m^3

Unit weight of soil = 2.77* 1000kg/m3= 2770kg/m3

1m^3 of soil =2770kg

So that 1kg of soil = 0.000361m^3

For SD and CA their SG = 2.47

Unit weight of SD and CA = 2470kg/m^3 , 1 m^3= 2470kg

1kg of CA and SD = 0.00040m^3

But we have 85 kg of soil and we need 10 kg of CA and 5 Kg of SD

i.e. 1 kg of soil = $3*10^{-4}m^3$

85 kg = 0.03$m^3$of soil

1kg of CA = $4*10^{-4}m^3$ then 10kg of CA will be $4*10^{-3}m^3$

1kg of SD = $4*10^{-4}m^3$ then 5 kg of SD = $2*10^{-4}m^3$

So, 85 kg of soil = 0.03m^3 …. then 10kg of CA = $4*10^{-3}m^3$, 5 kg of SD=$2*10^{-4}m^3$

Therefor 0.03m^3 of soil = $4*10^{-3}m^3$of coal ash

<u>For 1m^3 of soil = 0.1m^3 of coal ash</u>

0.03m^3 of soil = $2*10^{-4}m^3$ of stone dust

<u>For 1m^3 of soil = 0.06m^3 of stone dust</u>

The detail work for cost comparisons is shown in the table 5.1 and table 5.2 below.

Table 5.1 cost of sub structure work for G+1 residential building by replacing with non-expansive soil

Item No.	Description	Unit	Quantity	Rate	Amount
1.1	**A. SUB STRUCTURE** **1. Excavation and Earth work** Site clearing to remove top soil to an average				
	depth of 20cm	m^2	90.95	9.25	841.29
1.2	Bulk excavation over the site to reduce ground level up to 1000mm	m^3	45.50	99	4,504.50
1.3	Excavation for footing foundation to a depth not exceeding 300 cm.	m3	186.63	60	11,197.80
1.4	Excavation for trench foundation Ordinary soil to a depth not exceeding 3.00mfrom G. L	m3	10.55	105	1,107.75
1.5	Back fill with selected material around foundation and under hard core		203.40	260	52,884.00
1.6	Cart away and spread excavated material to a distance not exceeding 2km from site.	m3	261.00	43	11,223.00
1.7	25cm thick basaltic or equivalent stone hard well rolled, consolidated and blinded with crushed stone.	m^2	75.30	100	7,530.00
	Total				**89,288.34**

Table 5. 2 Cost of substructure work for G+1 residential building by fill using an expansive soil stabilized by 5% SD and 10% CA

Item No.	Description	Unit	Quantity	Rate	Amount
1.1	**A. SUB STRUCTURE** **1. Excavation and Earth work** Site clearing to remove top soil to an average depth of 20cm	m^2	90.95	9.25	841.29
1.2	Bulk excavation over the site to reduce ground level up to the requited level	m^3	45.50	99	4,504.50
1.3	Excavation for footing foundation to a depth not exceeding 300 cm.	m3	186.63	60	11,197.80
1.4	Excavation for trench foundation ordinary soil to a depth not exceeding 3.00m from G. L	m3	10.55	105	1,107.75
1.5	Back fill with stabilized expansive soil around foundation, under hard core 1.5.1 soil	m3	170.55	0	- -
	1.5.2 stone Dust	m3	12.18	400	4,872.00
	1.5.3 Coal ash (only transport and labor cost)	m3	20.30	200	4,060.00
1.7	Cart away and spread excavated material to a distance not exceeding 2km from site.	m3	90.45	43	3,889.35
1.8	25cm thick basaltic or equivalent stone hard well rolled, consolidated and blinded with crushed stone.	m^2	75.30	100	7,530.00
	Sub Total				**38,002.69**

CHAPTER SIX

CONCLUSION AND RECOMMENDATION

6.1 CONCLUSION

The following conclusions are drawn on the basis of the test result and Analysis of the research work with the scope of the study.

- As it can be deduced from the laboratory test result of natural soil, the engineering properties of Expansive soil under the study are not suitable to be used as subgrade material. Unless its problematic properties are improved.
- The results obtained from Atterberg limits indicated that reduction of liquid limit and plasticity index gained on 5% of stone dust, further addition of stone dust increases LL and PI but FSI goes in decreasing. While addition of coal ash in soil reduced LL, PI and FSI.
- The plasticity index is reduced with increasing percentages of stone dust and coal ash.
- The MDD is increased for the combination of stone dust and coal ash but reduced beyond 15% SD & 10% CA.
- In the soil treated with 5 % SD and 10 % CA by dry weight of a soil was found to be optimum stabilizer content. The pick values for the USC test was found at this percentages.
- Finally, after cost comparison it is recommended that 5% stone dust and 10 % coal ash can be used for construction of light weight buildings.

As the improvement of geotechnical properties of expansive soil using stone dust and coal ash with its cost comparison was performed , stone dust and coal ash can improve the engineering properties of expansive soil and the stabilized soil is more economical than the expansive soil replacement by other material .Therefore stone dust and coal ash can be used as stabilizing agent for expansive soil.

6.2 RECOMMENDATIONS

Based on the finding of the research the following are the main recommendations

- The effect of coal ash and stone dust on chemical combustion of expansive soil shall be studied.
- This study was conducted on the Expansive soil combination with stone dust and coal ash in the laboratory therefore their real performance in the field shall be studied.

SPECIAL ACKNOWLEDGEMENT

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APPENDICES

Appendix A- Atterberg limit test results

Table A-1Atterberg limit for ES

Type of test	Liquid limit				Plastic limit	
	Trial 1	Trial 2	Trial 3	Trial 4	Trial1	Trial 2
No. of Blows N	36	29	23	18	-	-
Mass of moist sample +can M_1	31.18	32.60	36.83	42.92	27.68	28.77
Mass of dry sample +can M_2	24.02	24.72	26.79	29.86	24.10	25.10
Mass of can M_c	14.74	14.81	14.68	14.84	14.37	14.70
Mass of moist $M_m = M_1 - M_2$	7.16	7.88	10.04	13.06	3.58	3.67
Mass of dry $M_d = M_2 - M_c$	9.28	9.91	12.11	15.02	9.73	10.40
Moisture content M_m/M_d*100	77.16	79.52	82.91	86.95	36.79	35.29

LL = 81.76 % PL = 36.04 % PI = 45.72%

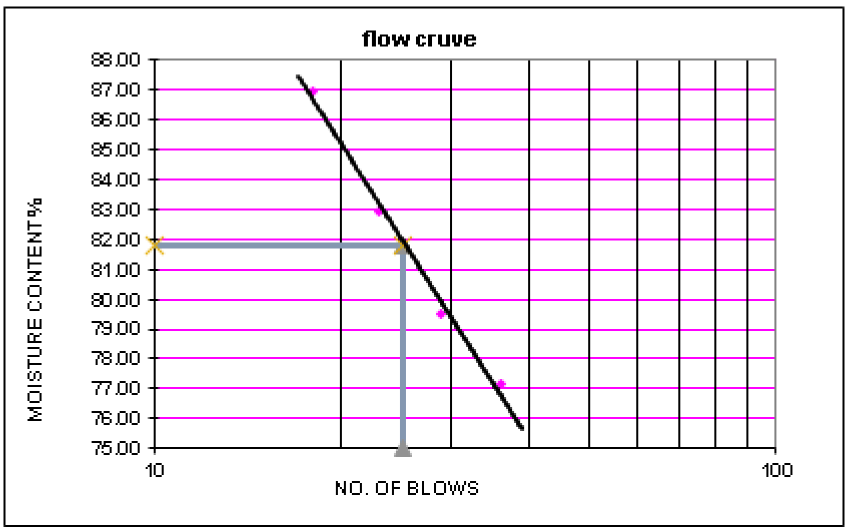

Figure A-1 flow curve for ES

Table A-2 Atterberg limit test result for 55 ES, 15 SD, 30CA

Type of test	Liquid limit				Plastic limit	
	Trial 1	Trial 2	Trial 3	Trial 4	Trial1	Trial 2
No. of Blows N	38	28	24	18	-	-
Mass of moist sample +can M_1	37.33	39.52	41.86	43.44	21.87	21.82
Mass of dry sample +can M_2	29.28	30.43	31.77	32.22	20.15	20.11
Mass of can M_c	14.82	14.53	14.82	14.33	14.97	14.82
Mass of moist $M_m= M_1-M_2$	8.05	9.09	10.09	11.22	1.72	1.71
Mass of dry $M_d= M_2-M_c$	14.46	15.90	16.95	17.89	5.18	5.29
Moisture content M_m/M_d*100	55.67	57.17	59.53	62.72	33.20	32.33

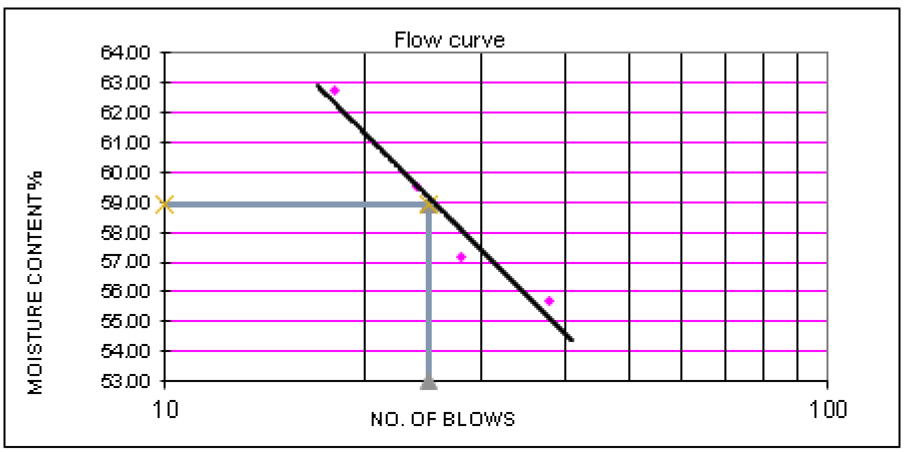

Figure A-2 Flow curves for 55 Es, 15 SD, 30CA

Appendix B Moisture – Density relationship test result

Table B -1 Moisture density relationship for ES

	Trial 1	Trial 2	Trial 3	Trial 4
Water in cc	-	-	-	-
Weight of mould + wet sample (g)	4925.70	5022.10	5165.00	5153.30
Weight of mould (g)	3585.30	3585.10	3585.30	3585.30
Weight of wet soil (g)	1340.40	1436.80	1579.70	1568.00
Volume of mould cm^3	944.00	944.00	944.00	944.00
Wet density gm/cm^3	1.420	1.522	1.673	1.661
Moisture content				
Wet soil + tin (g)	80.50	106.70	126.60	122.10
Dry soil + tin (g)	69.50	87.60	97.50	90.00
Weight of tin (g)	16.90	15.80	16.40	16.50
Weight of water (g)	11.00	19.10	29.10	32.10
Weight of dry soil (g)	52.60	71.80	81.10	73.50
Moistures content %	20.91	26.60	35.88	43.67
Dry density gm /cm3	1.174	1.202	1.232	1.156s

MDD = 1.23 gm/cc   OMC = 35.00 %

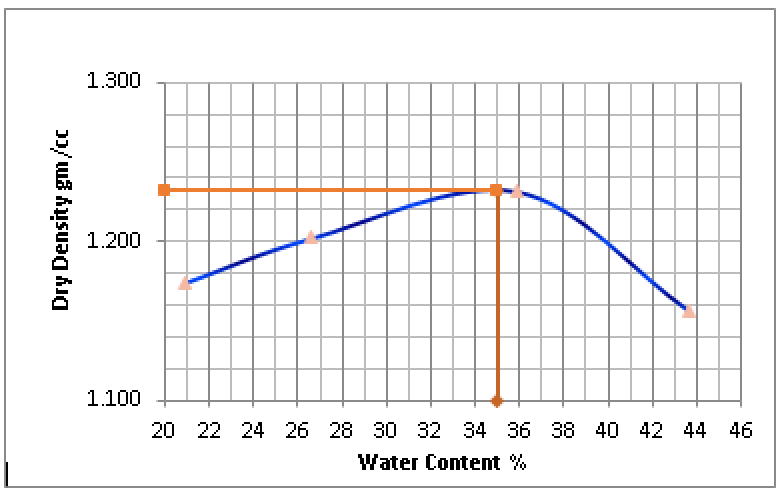

Figure B - 1 Moisture density relationship for ES

Table B – 2 Moisture density relationship for 85 ES, 5 SD & 10 CA

	Trial 1	Trial 2	Trial 3	Trial 4
Water in cc	-	-	-	-
Weight of mould + wet sample (g)	5489.60	5673.20	5811.70	5748.00
Weight of mould (g)	4129.00	4129.00	4129.00	4129.00
Weight of wet soil (g)	1360.60	1544.20	1682.70	1619.00
Volume of mould cm^3	944.00	944.00	944.00	944.00
Wet density gm/cm^3	1.441	1.636	1.783	1.715
Moisture content				
Wet soil + tin (g)	120.90	126.70	135.00	136.00
Dry soil + tin (g)	104.90	105.20	104.80	100.30
Weight of tin (g)	16.60	16.70	15.90	16.60
Weight of water (g)	16.00	21.50	30.20	35.70
Weight of dry soil (g)	88.30	88.50	88.90	83.70
Moistures content %	18.12	24.29	33.97	42.60
Dry density gm /cm3	1.220	1.316	1.331	1.202

MDD = 1.34 gm/cc

OMC = 31.00 %

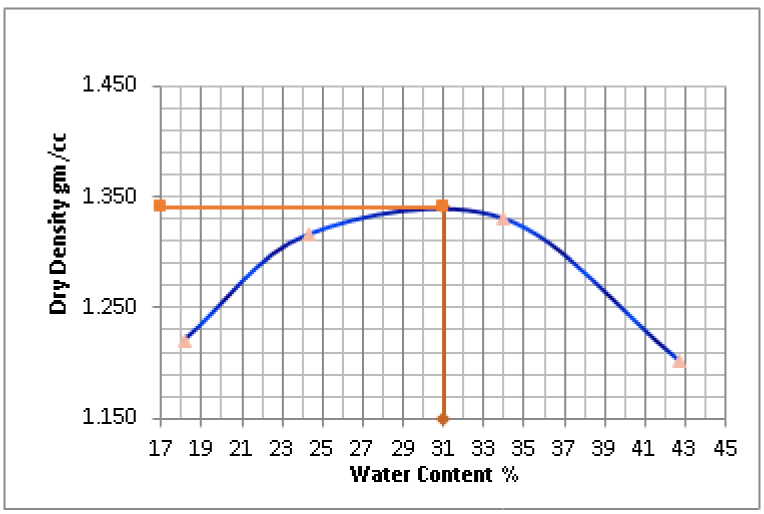

Figure B-2 Moisture density relationship for 85 ES, 5 SD & 10 CA

Appendix C Unconfined Compressive Strength test result

Table C-1 Unconfined compressive strength test result for disturbed ES sample

A) Moisture content determination

Container No	
Wt. of Wet Sample + Container(gm)	154.55
Wt. of Dry Sample + Container(gm)	114.60
Wt. Of Water(gm)	39.95
Wt. Of Container (gm)	0.00
Wt. of Dry Sample (gm)	114.60
Water Content (%)	34.86

Specimen data:

Diameter (mm) = 38.00 Length (mm) = 76.00 Area, A_o (mm^2) = 1134.26

Volume (mm3) = 86203.

Bulk density (gm/cm^3) = 1.793 Dry density (gm/cm^3) = 1.329

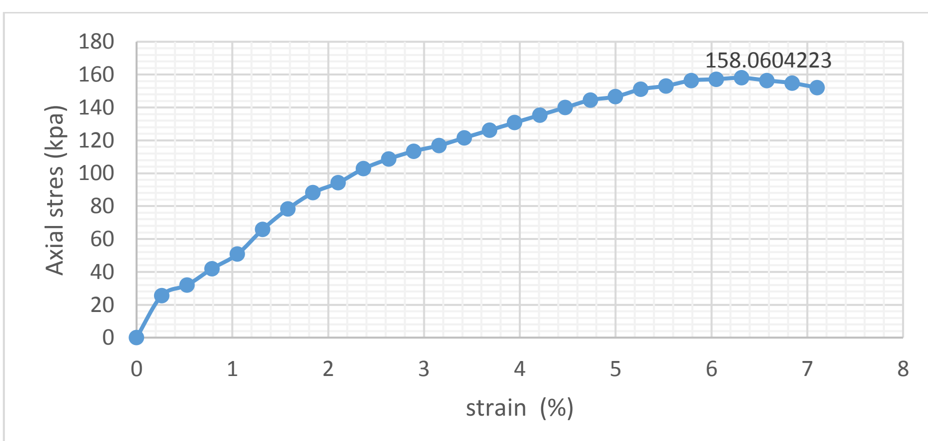

Figure C-1 unconfined compressive strength curve for disturbed ES

B) Unconfined compressive strength test data

Deformation Dial Reading	Load Dial Reading	Sample Deformation,Δ L(mm)	Strain(ε)	**% Strain**	Corrected Area A'	Load (kN)	Stress (kPa)
0	0.0	0.00	0.00	0.000	1134.262	0.000	0.00
20	20.0	0.20	0.003	0.263	1137.247	0.029	25.59
40	25.0	0.40	0.005	0.526	1140.232	0.036	31.90
60	33.0	0.60	0.008	0.789	1143.217	0.048	42.00
80	40.0	0.80	0.011	1.053	1146.202	0.058	50.78
100	52.0	1.00	0.013	1.316	1149.187	0.076	65.84
120	62.0	1.20	0.016	1.579	1152.171	0.090	78.30
140	70.0	1.40	0.018	1.842	1155.156	0.102	88.17
160	75.0	1.60	0.021	2.105	1158.141	0.109	94.22
180	82.0	1.80	0.024	2.368	1161.126	0.119	102.75
200	87.0	2.00	0.026	2.632	1164.111	0.127	108.74
220	91.0	2.20	0.029	2.895	1167.096	0.132	113.45
240	94.0	2.40	0.032	3.158	1170.081	0.137	116.89
260	98.0	2.60	0.034	3.421	1173.066	0.143	121.55
280	102.0	2.80	0.037	3.684	1176.051	0.148	126.19
300	106.0	3.00	0.039	3.947	1179.036	0.154	130.81
320	110.0	3.20	0.042	4.211	1182.020	0.160	135.40
340	114.0	3.40	0.045	4.474	1185.005	0.166	139.97
360	118.0	3.60	0.047	4.737	1187.990	0.172	144.52
380	120.0	3.80	0.050	5.000	1190.975	0.175	146.60
400	124.0	4.00	0.053	5.263	1193.960	0.180	151.11
420	126.0	4.20	0.055	5.526	1196.945	0.183	153.16
440	129.0	4.40	0.058	5.789	1199.930	0.188	156.42
460	130.0	4.60	0.061	6.053	1202.915	0.189	157.24
480	131.0	4.80	0.063	6.316	1205.900	0.191	158.06
500	130.0	5.00	0.066	6.579	1208.885	0.189	156.47
520	129.0	5.20	0.068	6.842	1211.869	0.188	154.88
540	127.0	5.40	0.071	7.105	1214.854	0.185	152.10

qu = 158.06 Kpa   cu $=\frac{qu}{2}$ = 79.03 kpa

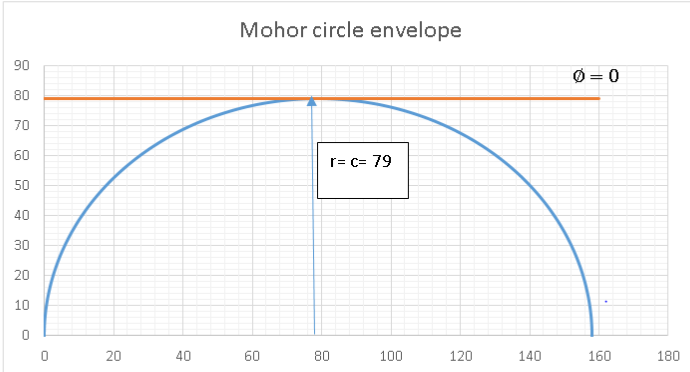

Figure C-1 B Mohr circle for disturbed ES

Table C -2 unconfined compressive strength test result for 85ES,5 SD&10CA

A) Moisture content determination

Container No	
Wt. of Wet Sample + Container(gm)	149.62
Wt. of Dry Sample + Container(gm)	114.20
Wt. Of Water(gm)	35.42
Wt. Of Container (gm)	0.00
Wt. of Dry Sample (gm)	114.20
Water Content (%)	31.02

dry density (gm/cm^3) = 1.325 Rate (mm/min) = 1.50 (KN/div) = 0.001455

B) Unconfined compressive strength test data

Deformation Dial Reading	Load Dial Reading	Sample Deformation,ΔL(mm)	Strain(ε)	**% Strain**	Corrected Area A'	Load (kN)	Stress (kPa)
0	0.0	0.00	0.00	0.000	1134.262	0.000	0.00
20	18.0	0.20	0.003	0.263	1137.247	0.026	23.03
40	28.0	0.40	0.005	0.526	1140.232	0.041	35.73
60	40.0	0.60	0.008	0.789	1143.217	0.058	50.91
80	55.0	0.80	0.011	1.053	1146.202	0.080	69.82
100	75.0	1.00	0.013	1.316	1149.187	0.109	94.96
120	90.0	1.20	0.016	1.579	1152.171	0.131	113.65
140	110.0	1.40	0.018	1.842	1155.156	0.160	138.55
160	130.0	1.60	0.021	2.105	1158.141	0.189	163.32
180	155.0	1.80	0.024	2.368	1161.126	0.226	194.23
200	166.0	2.00	0.026	2.632	1164.111	0.242	207.48
220	170.0	2.20	0.029	2.895	1167.096	0.247	211.94
240	175.0	2.40	0.032	3.158	1170.081	0.255	217.61
260	180.0	2.60	0.034	3.421	1173.066	0.262	223.26
280	185.0	2.80	0.037	3.684	1176.051	0.269	228.88
300	189.0	3.00	0.039	3.947	1179.036	0.275	233.24
320	192.0	3.20	0.042	4.211	1182.020	0.279	236.34
340	193.0	3.40	0.045	4.474	1185.005	0.281	236.97
360	190.0	3.60	0.047	4.737	1187.990	0.276	232.70
380	184.0	3.80	0.050	5.000	1190.975	0.268	224.79
400	179.0	4.00	0.053	5.263	1193.960	0.260	218.14

qu = 236.97 Kpa $\quad$ cu $= \frac{qu}{2}$ = 118.48 Kpa

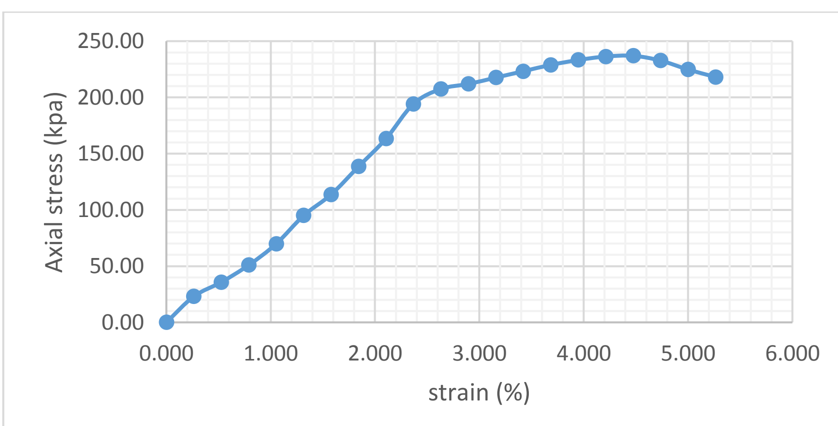

Figure C-2 A unconfined compressive strength test result for 85ES,5 SD&10CA

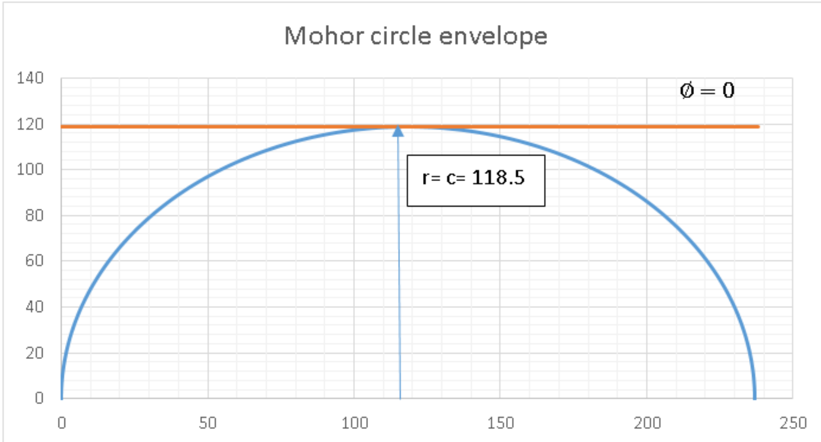

Figure C-2 B Mohr circle for 85ES,5 SD&10CA

Table C-3 unconfined compressive strength test result for 70 ES,10 SD&20CA

A) Moisture content determination

Container No	
Wt. of Wet Sample + Container(gm)	143.82
Wt. of Dry Sample + Container(gm)	110.20
Wt. Of Water(gm)	33.62
Wt. Of Container (gm)	0.00
Wt. of Dry Sample (gm)	110.20
Water Content (%)	30.51

Bulk density (gm/cm^3) = 1.668 Dry density (gm/cm^3) = 1.278

B) Unconfined compressive strength test data

Deformation Dial Reading	Load Dial Reading	Sample Deformation, ΔL(mm)	Strain(ε)	**% Strain**	Corrected Area A'	Load (kN)	Stress (kPa)
0	0.0	0.00	0.00	0.000	1134.262	0.000	0.00
20	15.0	0.20	0.003	0.263	1137.247	0.022	19.19
40	20.0	0.40	0.005	0.526	1140.232	0.029	25.52
60	25.0	0.60	0.008	0.789	1143.217	0.036	31.82
80	30.0	0.80	0.011	1.053	1146.202	0.044	38.08
100	36.0	1.00	0.013	1.316	1149.187	0.052	45.58
120	43.0	1.20	0.016	1.579	1152.171	0.063	54.30
140	50.0	1.40	0.018	1.842	1155.156	0.073	62.98
160	60.0	1.60	0.021	2.105	1158.141	0.087	75.38
180	65.0	1.80	0.024	2.368	1161.126	0.095	81.45
200	72.0	2.00	0.026	2.632	1164.111	0.105	89.99
220	90.0	2.20	0.029	2.895	1167.096	0.131	112.20
240	101.0	2.40	0.032	3.158	1170.081	0.147	125.59
260	112.0	2.60	0.034	3.421	1173.066	0.163	138.92
280	120.0	2.80	0.037	3.684	1176.051	0.175	148.46
300	125.0	3.00	0.039	3.947	1179.036	0.182	154.26
320	127.0	3.20	0.042	4.211	1182.020	0.185	156.33
340	130.0	3.40	0.045	4.474	1185.005	0.189	159.62
360	132.0	3.60	0.047	4.737	1187.990	0.192	161.67
380	129.0	3.80	0.050	5.000	1190.975	0.188	157.60
400	124.0	4.00	0.053	5.263	1193.960	0.180	151.11

qu = 161.67 Kpa cu $=\frac{\boldsymbol{qu}}{\mathbf{2}}$ = 80.835 Kpa

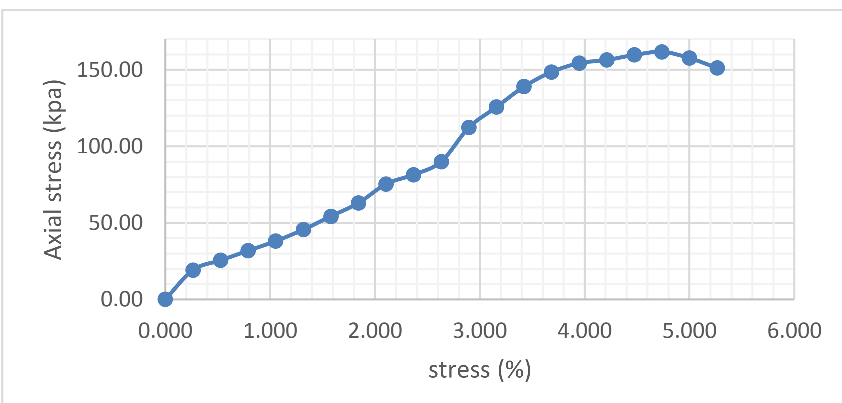

Figure C-3 A unconfined compressive strength test result for 70 ES,10 SD&20CA

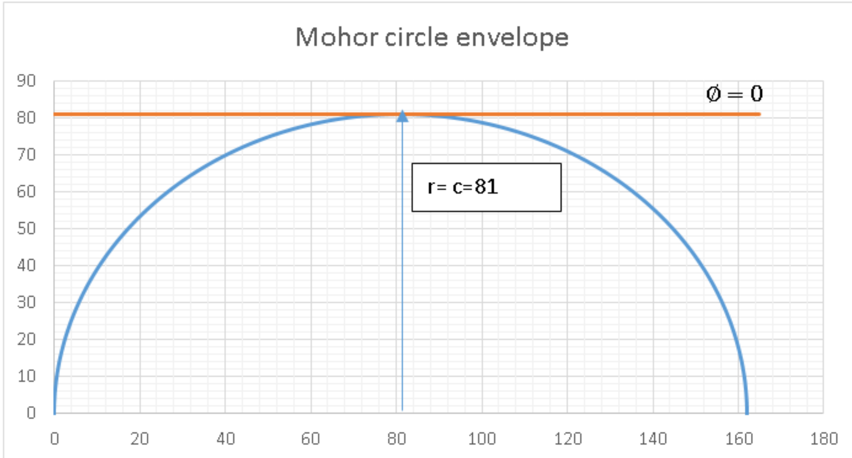

Figure C-3B Mohr circle for 70 ES,10 SD&20CA

Table C-4 unconfined compressive strength test result for 75 ES,5 SD&20CA

A) Moisture content determination

Container No	
Wt of Wet Sample + Container(gm)	144.38
Wt of Dry Sample + Container(gm)	111.30
Wt. Of Water(gm)	33.08
Wt. Of Container (gm)	0.00
Wt of Dry Sample (gm)	111.30
Water Content (%)	29.72

Bulk density (gm/cm^3) = 1.675 Dry density (gm/cm^3) = 1.291

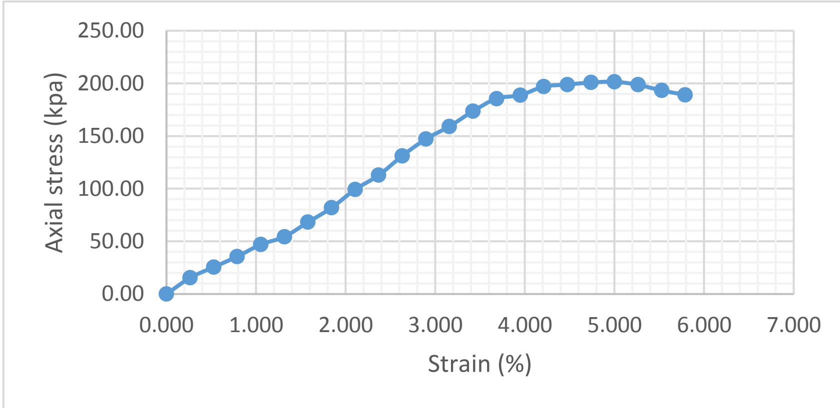

Figure C-4 A unconfined compressive strength test result for 75 ES,5 SD&20CA

B) Unconfined compressive strength test data

Deformation Dial Reading	Load Dial Reading	Sample Deformation,ΔL(mm)	Strain(ε)	% Strain	Corrected Area A'	Load (kN)	Stress (kPa)
0	0.0	0.00	0.00	0.000	1134.262	0.000	0.00
20	12.0	0.20	0.003	0.263	1137.247	0.017	15.35
40	20.0	0.40	0.005	0.526	1140.232	0.029	25.52
60	28.0	0.60	0.008	0.789	1143.217	0.041	35.64
80	37.0	0.80	0.011	1.053	1146.202	0.054	46.97
100	43.0	1.00	0.013	1.316	1149.187	0.063	54.44
120	54.0	1.20	0.016	1.579	1152.171	0.079	68.19
140	65.0	1.40	0.018	1.842	1155.156	0.095	81.87
160	79.0	1.60	0.021	2.105	1158.141	0.115	99.25
180	90.0	1.80	0.024	2.368	1161.126	0.131	112.78
200	105.0	2.00	0.026	2.632	1164.111	0.153	131.24
220	118.0	2.20	0.029	2.895	1167.096	0.172	147.11
240	128.0	2.40	0.032	3.158	1170.081	0.186	159.17
260	140.0	2.60	0.034	3.421	1173.066	0.204	173.65
280	150.0	2.80	0.037	3.684	1176.051	0.218	185.58
300	152.0	3.00	0.039	3.947	1179.036	0.221	186.34
320	160.0	3.20	0.042	4.211	1182.020	0.233	196.95
340	162.0	3.40	0.045	4.474	1185.005	0.236	198.91
360	164.0	3.60	0.047	4.737	1187.990	0.239	200.86
380	165.0	3.80	0.050	5.000	1190.975	0.240	201.58
400	163.0	4.00	0.053	5.263	1193.960	0.237	198.64
420	159.0	4.20	0.055	5.526	1196.945	0.231	193.28
440	156.0	4.40	0.058	5.789	1199.930	0.227	189.16

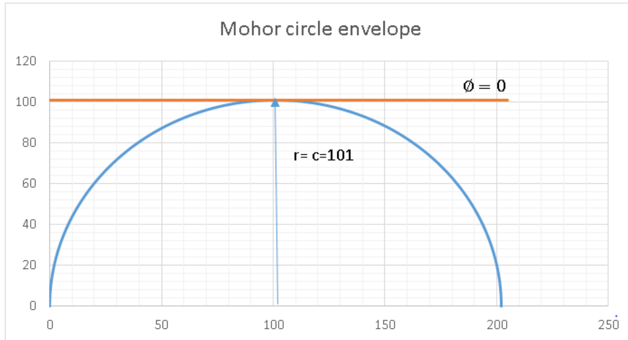

Figure C-4 B Mohr circle for 75 ES,5 SD&20CA

Table C-5 unconfined compressive strength test result for 65 ES,15 SD&20CA

A) Moisture content determination

Container No	
Wt of Wet Sample + Container(gm)	144.68
Wt of Dry Sample + Container(gm)	109.90
Wt. Of Water(gm)	34.78
Wt. Of Container (gm)	0.00
Wt of Dry Sample (gm)	109.90
Water Content (%)	31.65

Bulk density (gm/cm^3) = 1.678 dry density (gm/cm^3) = 1.275

B) Unconfined compressive strength test data

Deformation Dial Reading	Load Dial Reading	Sample Deformation,ΔL(mm)	Strain(ε)	**% Strain**	Corrected Area A'	Load (kN)	Stress (kPa)
0	0.0	0.00	0.00	0.000	1134.262	0.000	0.00
20	18.0	0.20	0.003	0.263	1137.247	0.026	23.03
40	24.0	0.40	0.005	0.526	1140.232	0.035	30.63
60	30.0	0.60	0.008	0.789	1143.217	0.044	38.18
80	39.0	0.80	0.011	1.053	1146.202	0.057	49.51
100	52.0	1.00	0.013	1.316	1149.187	0.076	65.84
120	63.0	1.20	0.016	1.579	1152.171	0.092	79.56
140	75.0	1.40	0.018	1.842	1155.156	0.109	94.47
160	88.0	1.60	0.021	2.105	1158.141	0.128	110.56
180	102.0	1.80	0.024	2.368	1161.126	0.148	127.82
200	118.0	2.00	0.026	2.632	1164.111	0.172	147.49
220	120.0	2.20	0.029	2.895	1167.096	0.175	149.60
240	125.0	2.40	0.032	3.158	1170.081	0.182	155.44
260	130.0	2.60	0.034	3.421	1173.066	0.189	161.24
280	125.0	2.80	0.037	3.684	1176.051	0.182	154.65
300	121.0	3.00	0.039	3.947	1179.036	0.176	149.32
320	110.0	3.20	0.042	4.211	1182.020	0.160	135.40

qu =161.24 kpa   cu = 80.62

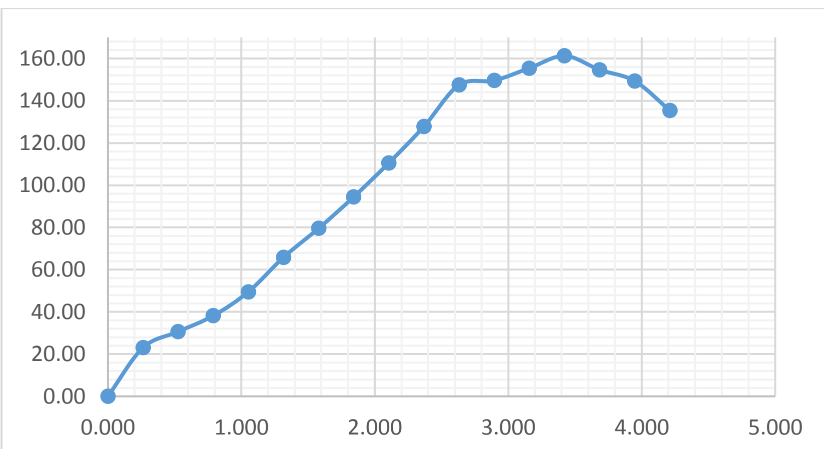

Figure D-5 A unconfined compressive strength test result for 65 ES,15 SD&20CA

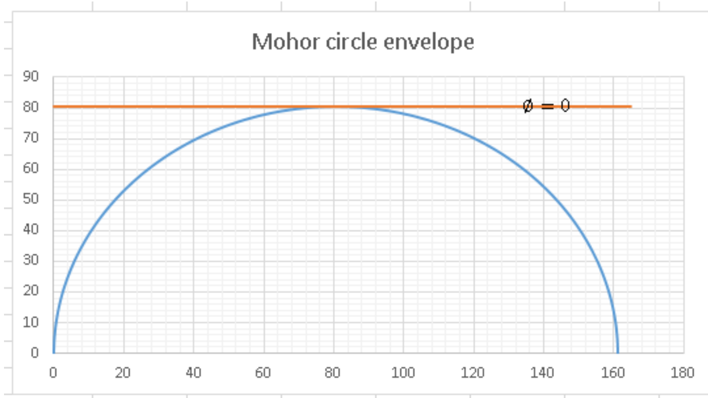

Figure C-5 B Mohr circle for 65 ES,15 SD&20CA

Table C-6 unconfined compressive strength test result for 65 ES,5 SD&30CA

A) Moisture content determination

Container No	
Wt of Wet Sample + Container(gm)	144.75
Wt of Dry Sample + Container(gm)	110.00
Wt. Of Water(gm)	34.75
Wt. Of Container (gm)	0.00
Wt of Dry Sample (gm)	110.00
Water Content (%)	31.59

Bulk density (gm/cm^3) = 1.699 dry density (gm/cm^3) = 1.290

B) Unconfined compressive strength test data

Deformation Dial Reading	Load Dial Reading	Sample Deformation,ΔL(mm)	Strain(ε)	% Strain	Corrected Area A'	Load (kN)	Stress (kPa)
0	0.0	0.00	0.00	0.000	1134.262	0.000	0.00
20	24.0	0.20	0.003	0.263	1137.247	0.035	30.71
40	43.0	0.40	0.005	0.526	1140.232	0.063	54.87
60	54.0	0.60	0.008	0.789	1143.217	0.079	68.73
80	60.0	0.80	0.011	1.053	1146.202	0.087	76.16
100	65.0	1.00	0.013	1.316	1149.187	0.095	82.30
120	70.0	1.20	0.016	1.579	1152.171	0.102	88.40
140	75.0	1.40	0.018	1.842	1155.156	0.109	94.47
160	70.0	1.60	0.021	2.105	1158.141	0.102	87.94
180	68.0	1.80	0.024	2.368	1161.126	0.099	85.21

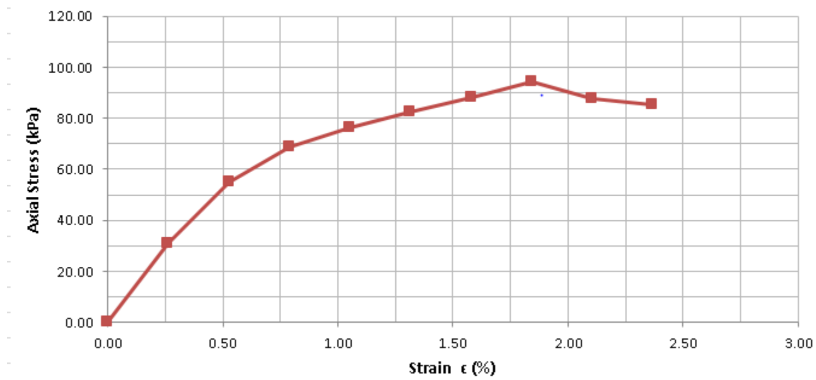

Figure C-6 A unconfined compressive strength test result for 65 ES,5 SD&30CA

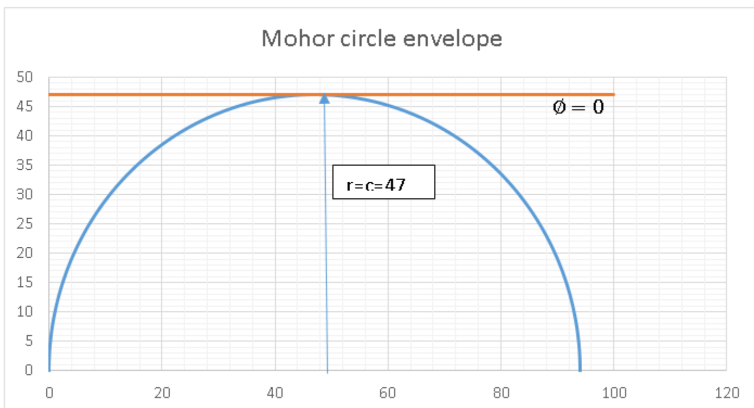

Figure C-6 B Mohr circle for 65 ES,5 SD&30CA

Table C-7 unconfined compressive strength test result for 60 ES,10 SD&30CA

A) Moisture content determination

Container No	
Wt of Wet Sample + Container(gm)	135.96
Wt of Dry Sample + Container(gm)	101.20
Wt. Of Water(gm)	34.76
Wt. Of Container (gm)	0.00
Wt of Dry Sample (gm)	101.20
Water Content (%)	34.35

Bulk density (gm/cm^3) = 1.699

dry density (gm/cm^3) = 1.290 Rate (mm/min) = 1.50 (KN/div) = 0.001455

B) Unconfined compressive strength test data

Deformation Dial Reading	Load Dial Reading	Sample Deformation, ΔL(mm)	Strain(ε)	% Strain	Corrected Area A'	Load (kN)	Stress (kPa)
0	0.0	0.00	0.00	0.000	1134.262	0.000	0.00
20	10.0	0.20	0.003	0.263	1137.247	0.015	12.79
40	18.0	0.40	0.005	0.526	1140.232	0.026	22.97
60	21.0	0.60	0.008	0.789	1143.217	0.031	26.73
80	29.0	0.80	0.011	1.053	1146.202	0.042	36.81
100	34.0	1.00	0.013	1.316	1149.187	0.049	43.05
120	40.0	1.20	0.016	1.579	1152.171	0.058	50.51
140	44.0	1.40	0.018	1.842	1155.156	0.064	55.42
160	50.0	1.60	0.021	2.105	1158.141	0.073	62.82
180	55.0	1.80	0.024	2.368	1161.126	0.080	68.92
200	62.0	2.00	0.026	2.632	1164.111	0.090	77.49
220	70.0	2.20	0.029	2.895	1167.096	0.102	87.27
240	78.0	2.40	0.032	3.158	1170.081	0.113	96.99
260	85.0	2.60	0.034	3.421	1173.066	0.124	105.43
280	88.0	2.80	0.037	3.684	1176.051	0.128	108.87
300	90.0	3.00	0.039	3.947	1179.036	0.131	111.07
320	92.0	3.20	0.042	4.211	1182.020	0.134	113.25
340	89.0	3.40	0.045	4.474	1185.005	0.129	109.28
360	84.0	3.60	0.047	4.737	1187.990	0.122	102.88
380	80.0	3.80	0.050	5.000	1190.975	0.116	97.74

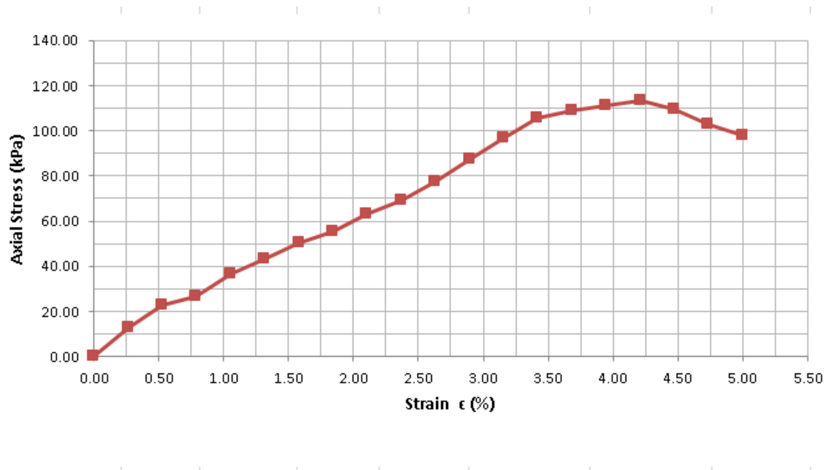

Figure C-7A unconfined compressive strength test result for 60ES,10 SD&30CA

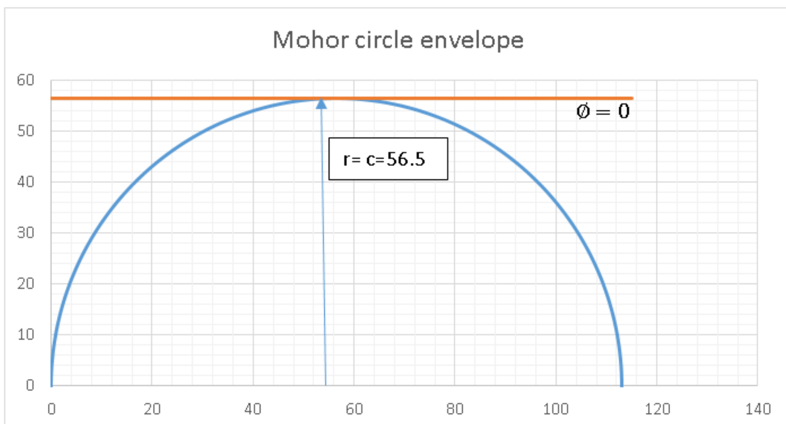

Figure C-7 B Mohr circle for 60ES,10 SD&30CA

Table C-8 unconfined compressive strength test result for 55 ES,15 SD&30CA

A) Moisture content determination

Container No	
Wt of Wet Sample + Container(gm)	142.41
Wt of Dry Sample + Container(gm)	108.80
Wt. Of Water(gm)	33.61
Wt. Of Container (gm)	0.00
Wt of Dry Sample (gm)	108.80
Water Content (%)	30.89

Bulk density (gm/cm^3) = 1.699 dry density (gm/cm^3) = 1.290

B) Unconfined compressive strength test data

Deformation Dial Reading	Load Dial Reading	Sample Deformation, ΔL(mm)	Strain(ε)	% Strain	Corrected Area A'	Load (kN)	Stress (kPa)
0	0.0	0.00	0.00	0.000	1134.262	0.000	0.00
20	14.0	0.20	0.003	0.263	1137.247	0.020	17.91
40	22.0	0.40	0.005	0.526	1140.232	0.032	28.07
60	27.0	0.60	0.008	0.789	1143.217	0.039	34.36
80	34.0	0.80	0.011	1.053	1146.202	0.049	43.16
100	42.0	1.00	0.013	1.316	1149.187	0.061	53.18
120	50.0	1.20	0.016	1.579	1152.171	0.073	63.14
140	57.0	1.40	0.018	1.842	1155.156	0.083	71.80
160	60.0	1.60	0.021	2.105	1158.141	0.087	75.38
180	75.0	1.80	0.024	2.368	1161.126	0.109	93.98
200	83.0	2.00	0.026	2.632	1164.111	0.121	103.74
220	90.0	2.20	0.029	2.895	1167.096	0.131	112.20
240	95.0	2.40	0.032	3.158	1170.081	0.138	118.13
260	105.0	2.60	0.034	3.421	1173.066	0.153	130.24
280	112.0	2.80	0.037	3.684	1176.051	0.163	138.57
300	116.0	3.00	0.039	3.947	1179.036	0.169	143.15
320	120.0	3.20	0.042	4.211	1182.020	0.175	147.71
340	118.0	3.40	0.045	4.474	1185.005	0.172	144.89
360	114.0	3.60	0.047	4.737	1187.990	0.166	139.62
380	100.0	3.80	0.050	5.000	1190.975	0.146	122.17

qu = 147 kpa cu = 73.5

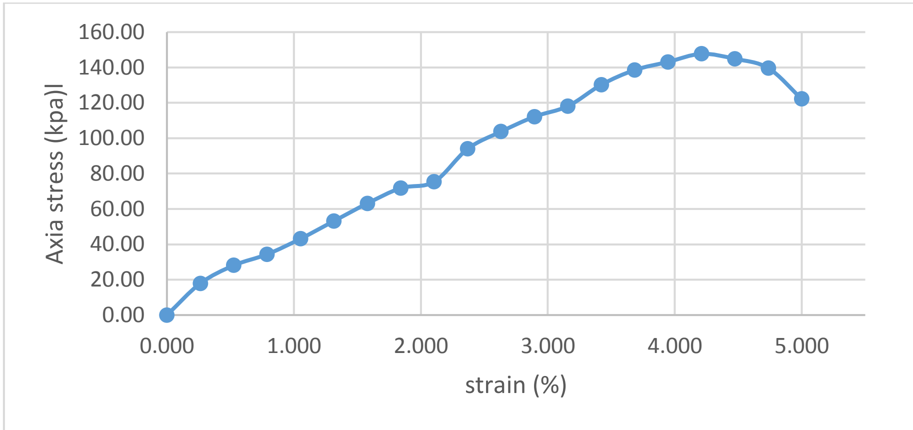

Figure C-8 A unconfined compressive strength test result for 55ES,15 SD&30CA

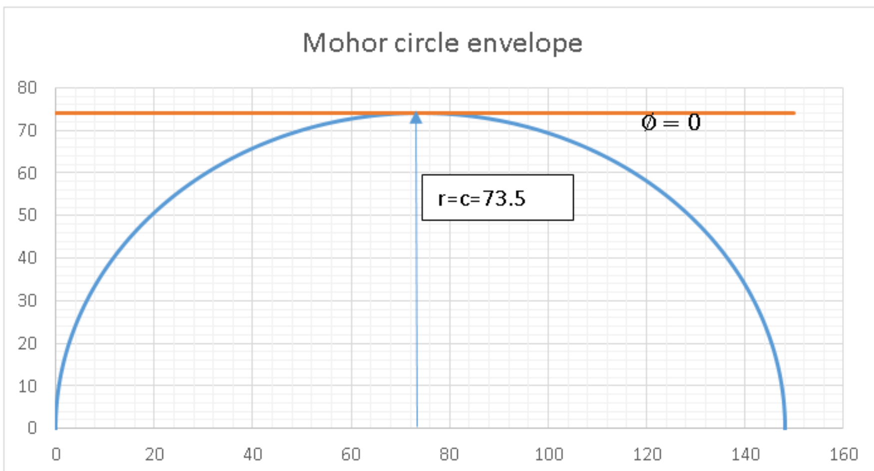

Figure C-8B Mohr circle for 55ES,15 SD&30CA