STABILIZATION OF EXPANSIVE SOIL WITH WASTE PLASTICS

M. Sc. THESIS

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ADVISORS' APPROVAL SHEET

This is to certify that the thesis entitled **"Stabilization of Expansive Soil with Waste Plastics"** submitted in partial fulfillment of the requirements for the degree of Masters of Science in Civil Engineering with specialization in **Road and Transport Engineering**, the Graduate Program of the School of **Civil Engineering**, and has been carried out by **Tezeta Moges** Id. No **PGRo/041/08**, under my supervision. Therefore, I recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

Dr. Alemgena Alene (PhD)	____________	____________
Name of major advisor	Signature	Date

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DECLARATION

I hereby declare that this M.Sc. thesis is my original work and has not been presented for a degree in any other university, and all sources of material used for this thesis have been duly acknowledged.

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I dedicate this thesis to my beloved family who helped me to grow by everything.

LIST OF ACRONYMS AND SYMBOLS

ε_f	Failure Strain
ASTM	American Society for Testing and Materials
BS	British Standard
C	Cohesion Value
CBR	California Bearing Ratio
CEC	Cat ion Exchange Capacity

D_{fs}	Differential Free Swell
ERA	Ethiopian Road Authority
FM	Farm-to-Market Roadways
HMA	Hot Mix Asphalt
IS	Indian Standard
LL	Liquid Limit
MDD	Maximum Dry Density
NCSS	Number Crunching Statistical System
OMC	Optimum Moisture Content
PET	Polyethylene Terephthalate
PI/Ip	Plasticity Index
PL	Plastic Limit
PPF	Polypropylene Fiber
r	Correlation Coefficient
R^2	Squared Correlation Coefficient
S	Swelling Potential
S_f	Free Swell
S_u	Un-drained Shear Strength
UCS/q_u	Unconfined Compressive Strength
USCS	Unified Soil Classification System
Φ	Friction Angle Value

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ABSTRACT

Expansive soils have high amounts of clay particles and have a special mineralogical characteristic. The presence of considerable amount of montmorillonite clay mineral causes them to swell and shrink when amount of water is changed. Due to successive shrink and swell, this type of soil causes some asphalt distresses like upheaval, roughness, rutting and longitudinal cracking. To other extent, black cotton soils in construction sites do have *significant influence on planning, structural design, construction & maintenance costs, performance and engineering life of roads. There are many techniques to improve poor engineering properties of expansive soils. One of this is stabilization of expansive soil with waste plastic materials in the form of flexi which was introduced in this specific study. The stabilization of black cotton soil with optimum amount of flexi material decreased plastic index (PI) value by 13.6% and optimum moisture content (OMC) value by 18.5%, increased maximum dry density (MDD) value by 1.9%, increased California bearing ratio (CBR) value by 50.9%, increased Cohesion(C) value by 83.3% and increased both unconfined compressive strength (q_u) and un-drained shear strength (S_u) value by 10.1%. The flexi material gave better strength and stability by means of replacement of black cotton soil by flexi. Sand also used as a stabilizer material to magnify the effect of flexi stabilizer. In addition, the correlation and regression model was done by the result of the laboratory test results of OMC, MDD, CBR, C, and q_u as dependent variables with the independent variables of percent (%) of flexi and PI in a single regression analysis. A good correlation and regression models were developed between percent of flexi and CBR, q_u & S_u. Coefficient of determination (R^2 = 0.79) indicates that CBR has strong relation with % of flexi by having equation of* $CBR = (2.9909) - (0.3465 * \%\ of flexi)$.

Key words
Expansive Soil, Montmorillonite, Flexi, Sand, Regression

1. INTRODUCTION

1.1. Background

Soil is highly complex, heterogeneous and unpredictable material which has been subjected to vagaries of nature, without any control. The properties of soil change not only from one place to other but also at the place with depth and with a change in the environmental, loading and drainage conditions (Pal, et al., 2015). The properties of soil depend not only on its type but also on the conditions under which it exists.

Soil is used as construction materials for highway construction as fill, capping layer, sub-base, and sub- grade materials. The sub-grade constitutes the foundation material for the pavement structure as highway pavements ultimately rest on the native soil. Geotechnical soils and material characterization are important components for the design and construction of road projects. During characterization visual descriptions, sampling, testing of natural sub-grade soil and embankment material along the project route are carried out. In the process of characterization, different techniques and procedures are applied for interpretation of sub-grade soil condition.

One of soil type which can be used for sub-grade construction is expansive soils. Expansive soils are known to occur in many parts of the world. Some of these countries are: Argentina, Australia, Burma, Canada, Cuba, Ethiopia, Ghana, India, Iran, Israel, Mexico, Morocco, Poland, South Africa, Spain, Turkey, U.S.A., U.S.S.R., Venezuela and Zimbabwe (Aswathy and Greeshma, 2016). Expansive soils are typically clay which has characteristics of 'shrink' and 'swell' with response to fluctuation in the moisture contents and ground water table following seasonal climate variation. The degree of expansion of expansive soil is classified in to four categories; very high, high, medium and low depending on their percentage of colloid content, potential expansion, Shrinkage limit and plasticity index (PI) values. Potentially expansive soils can typically be recognized in the laboratory by their plastic properties. Inorganic clays of high plasticity, generally those with liquid limits (LL) exceeding 50 percent and PI over 30 usually have high inherent swelling capacity (ERA, 2013). Expansion of soils can also be measured in the laboratory directly, by immersing a remolded soil sample and measuring its volume change. One of the major pavement defects due to the swelling potential of expansive soil is Upheaval. It is a localized upward movement in a pavement due to swelling of the sub-grade. This swell is due to moisture or frost heave. Fatigue and edge cracking's sometimes caused by weak material of sub-grade layer. Therefore; deep investigation should be done in every road alignments, which have expansive soil as a sub-grade construction material.

Therefore; to mitigate the problem of expansive soils in road construction, there are many solutions such as sub-excavation & removal of expansive soils to replace with non-expansive soils or selected materials, application of heavy applied load or compacted very well to balance the swelling pressure, preventing access of water to the soil by encapsulation, mechanical stabilization and stabilization by addition of chemical admixtures. In this study, the possible use of waste plastic for expansive soil stabilization was conducted to improve their engineering properties. This waste plastic is advanced technique to improve the bearing capacity, shear strength, Plastic index and other poor engineering properties of expansive soils besides the reduction of wastes by changing plastic wastes in to resource. The need to bring down the growing cost of soil stabilizers and the cost of waste disposal has lead to intense global research towards economic utilization of wastes for engineering purposes. The safe disposals of industrial & commercial plastic waste product demand are urgent and cost effective solutions.

The performance of plastic stabilizer in the improvement of expansive soil behavior is studied in this paper. Another mechanical stabilization method, which is the addition of sand in to the expansive soil, is also done and the improvements of the expansive soils were measured. Finally, each result that can be done by the above two different stabilizer methods were compared and judged which one is preferable in terms of improvement of engineering properties, high availability and high environmental safety. Stabilizing of the sub-grade can also reduce the section of the base material and asphalt paving thus, reducing cost. Hence, it becomes necessary to move forward with suitable method of low capital construction. This should also be followed strictly to meet up the rising demands of road traffic especially.

1.2. Statement of the Problem

The origin of expansive clay soils are related to a complex combination of condition and process that results in the formation of weak minerals having a particular makeup. Consequently, these types of clay soils have potential to shrink or swell under different moisture condition. There are several types of failures that caused by these expansive soils such as upheaval, roughness, rutting, longitudinal cracking and the roads become so slippery because of poor shear strength of expansive soils. Therefore, expansive soils need improvement by alteration of chemicals or physically which is one way for the reduction of their poor engineering properties. So, deep investigation and best management during the construction stage must be done.

There are a lot of ordinary stabilization materials such as lime, fly ash, cement, dusts and others. But in this paper, waste plastic stabilizer for the effective work of expansive soil as a sub-grade material in road construction is introduced. Stabilization using polyethylene terephthalate (PET) plastic material in the form of flexi was used. The production of these lime and cement traditional stabilizers is environmental unfriendly processes as well as the extraction of substantial amounts of non-renewable natural resources create significant impacts on the local environment and its inhabitants. In addition, this flexi material is unwanted waste for the recycle companies.

It is necessary to reduce the volume changes of expansive soils and strengthen them to the point where they can carry the imposed load even the soils are saturated. Using this flexi material especially for developing country like Ethiopia which have high demand of plastic have two basic roles, these are changing waste plastic into resource for stabilization of soils and minimizing the unwanted waste of the country in order to reduce environmental pollution. Non-biodegradable pollutants such as plastic wastes are materials which take thousands of years to decompose. These pollutants become extremely difficult to remove once released into the environment. Therefore; using of waste plastic materials as a stabilizer, reduces the amount of solid wastes as well as protects the environment from pollution. Ethiopian municipality and waste generation (2010) thought that the total number of plastic waste material that is discarded within a day in Ethiopia is1,385.50 kg/day in terms of dries weight. In addition, now a day’s half of the solid waste of Ethiopian cities is plastic materials.

1.3. Objectives

1.3.1. General Objectives

The main objective of this research is to study the engineering properties of expansive soil in road construction using waste plastic as a soil stabilizer.

1.3.2. Specific Objectives

- To find out flexi material potential for the improvement of expansive soil problems.
- To determine the optimum amount of flexi material to stabilize an expansive soil.
- To assess the effect of heavy compaction in flexi stabilized expansive soil.
- To establish correlation and develop equations between % of flexi and laboratory results of flexi stabilized expansive soil.

1.4. Research Questions

- What are the poor characteristics of expansive soils that cause damage in road construction?
- How can we best manage expansive soil engineering properties through stabilization using waste plastic re-cycled material?
- Why is this waste plastic material best for stabilization of expansive soils in our country Ethiopia?

1.5. Scope of the Study

The failures of expansive soil as a sub-grade material were assessed by using literature reviews. To deal with the objectives of the study, disturbed samples were taken for different laboratory tests. The study also addresses how the waste plastic and sand as a stabilizer work to adjust the engineering properties of expansive soil. In this specific study, the stabilization of soil by sand is included only to magnify the potential of flexi to stabilize expansive soil. In addition, correlation and regression analysis were done by using Number Crunching Statistical System (NCSS).

1.6. Limitation of the Study

The soil samples were taken limited from one representative location. In addition, the sample sizes, which were used for modeling of the laboratory results through mathematical equations, were small. The percentage of sand used during investigation of stabilization was small.

1.7. Organization of the Thesis

The thesis is structured in six different chapters. The first chapter is the introduction part which includes the general idea of the soil type and stabilization method, statement of problem, general and specific objectives with the scope & limitation of the study work. The second chapter is examined the different researcher reviews that talks about the characteristics of expansive soil and waste plastic materials with its improvement after stabilization. Different methods and materials, which were used to achieve all the specific objectives, are expressed in chapter three. Chapter four presents the analysis of results obtained from the laboratory tests with respect to the theoretical background & findings of previous studies and correlation & Regression analysis. The last chapter expresses the conclusion and recommendation of the study work.

2. LITERATURE REVIEW

2.1. Expansive Soil

Problematic soil is the soil that causes additional problems from engineering point of view as a result of circumstances of its composition or a change in environmental conditions. There are many types of problematic soil in the world. One of these types of soil is expansive soils. Expansive soils undergo slow volume changes when they change water content which occur independently of loading and were attributable to swelling or shrinkage (SAAD, 1962).The volume changes could give rise to ground movement which can cause damage to low-rise buildings that they don't have sufficient weight to resist and also represent a problem when they are encountered in road construction (Rezaei, et al., 2012). Shrinkage settlement of embankments composed of such clays could lead to cracking and breakup of the roads they support (Yenes, et al., 2010).

In the last few decades, the swelling potential of soils has been linked to parameters like void ratio, moisture content, liquid limit and activity through empirical methods (JALEH, 2016). However, studies have been performed and proved the incompleteness of the soil behavior characterization when correlations based on such parameters are applied. Swelling behavior of expansive clays is highly influenced by their moisture/suction state, mineralogy, pore distribution and in-situ stress state. Several semi empirical models based on matric suction have been proposed including volume change constitutive relationships or statistical correlations. Pino, et al. (2016) explained that lightly-loaded engineering structures such as pavements, single story buildings, railways and walkways may experience severe damages when they are founded on such soils. Determination of expansive soils by quantifying their swelling potential and pressure caused by their expansion are essential in geotechnical engineering. Therefore; it is necessary to develop models to predict swelling pressure and swelling potential of expansive soils (JALEH, 2016).

The significant engineering properties of expansive soils of Addis Ababa have been studied in the laboratory. These are clay soils which may be black or grey in color. In addition, damages on structures due to heaving and shrinkage of the expansive soils have been assessed. (Teferra and Yohannes, 1986).

To upgrade the poor characteristics of expansive soil, stabilization by using different materials have been used. Now days, stabilization using waste plastic material is an economic method since the stabilizer used here is easily available. A plastic material is any of a wide range of synthetic or semi-synthetic organic solids that are moldable. Plastics are typically organic plastics of high molecular mass, but they often contain other substances. They are usually synthetic, most commonly derived from petrochemicals, but many are partially natural. Some of researchers concluded that the improved properties of soil by using plastic stabilizer are California bearing ratio (CBR) value, shear strength, consolidation settlement, reduction in cracks & swelling and this method also avoids disposal problems of plastic wastes (Pal, et al.,2015).

2.2. Origin and Mineralogical Composition of Expansive Soils

The parent materials of expansive soils may be classified into two groups. The first group comprises the basic igneous rocks such as basalt, dolerite sills, dykes, gabbros and others where feldspar and pyroxene minerals of the parent rocks decompose to form montmorillonite which is the predominant mineral of expansive soil including other secondary minerals. The second group comprises sedimentary rocks that contain montmorillonite and break down physically to form expansive soils. There are indications that confirm the expansive soils of Ethiopia are derived from both groups (Teferra, 1965).

It is a known fact that the three most important groups of clay minerals are montmorillonite, illite and kaolinite, which are crystalline hydrous alumino silicates. Of these groups it is the clay mineral montmorillonite that presents most of the foundation problems. Essentially, montmorillonite is a three layered mineral having a single octahedral alumina sheet sandwiched between two silica sheets. The units are stacked one above the other like leaves of book. The bonds are comparatively weak and water can enter between the sheets & causing them to expand readily. When water is removed from the boundary, the sheets become contract. Thus, soil containing substantial amount of montmorillonite will exhibit high shrinkage and swelling characteristics. Experience shows that swelling problems arise when soils contain more than 20% montmorillonite mineral (Ravina, 1983). Different types of montmorillonite minerals including their arrangement are shown in figure 2.1.

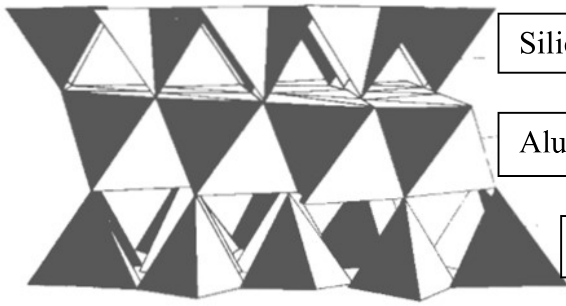


Figure2. 1 Model of a montmorillonite layer after Philip law (Assefa, 1979)

2.3. Swelling Potential of Expansive Soils

Some of the parameters which can be used for the identification of expansive soils potential are listed below;

I. Al-Rawas and Mattheus (2006) presented a single index method for identifying expansive soils using only liquid limit. They suggested four classes of clays on their swelling potential (S) according to their liquid limits (LL) after Daksanamurthy and Raman (1973) as shown in table 2.1.

Table2. 1 Relation among S and LL of clays (Al-Rawas and Mattheus, 2006)

Swelling Potential (S)	**Liquid limit(LL)**
Low	20< LL≤35
Medium	35< LL ≤50
High	50< LL ≤ 70
Very high	LL> 70

II. Chen (1988) offered a single index method for identifying expansive soils using only plasticity index (PI). Chen suggested four classes of clays according to their PI values shown in table 2.2.

Table2. 2 Relation between S and the PI values of clays (Chen, 1988)

Swelling potential (S)	Plasticity index (PI)
Low	0-15
Medium	10-35
High	20-55
Very high	35 and above

III. This method is developed by Holtz and Gibbs; it is based on direct correlation of observed volume change with colloid content, plastic index and shrinkage limit. The expansion level classification of expansive soil based on bureau of reclamation method is given in table 2.3.

Table2. 3 Expansive soil classification by reclamation method (Ranjan and Rao, 2002)

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

IV. By combining Atterberg limits & clay content into a single parameter called Activity, classification of expansion potential of clay soils are determined. Activity is defined by using equation 2.1.

$$Ac = \frac{PI}{\text{percentageby weight finer than2}\mu\text{m}} \qquad 2.1$$

Where;

Ac= activity

PI= plasticity index

Al-Rawas and Mattheus (2006) suggested three classes of clays according to their activity after Skempton that are shown in table 2.4.

Table2. 4 Relation on clay activity and expansion potential (Al-Rawas and Mattheus, 2006)

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

Soil with high amount of clay particles tend to hold a lot of moisture, which causes them to swell when water is absorbed and shrink water is lost. This swelling and shrinkage causes damage to road and building foundations that cost billions of dollars to repair in every year. All the above researchers concluded that expansive soils are found in Arid & Semi-Arid areas. The most significant group of expansive clay minerals is the smectite of which montmorillonite is the most common occurring. These minerals result from the chemical weathering or hydrothermal alteration of basic & intermediate igneous and metamorphic rocks containing feldspar and ferromagnesian minerals. The stability of expansive minerals produced depends on environmental conditions, particularly the drainage, oxidation region and seasonality of the rainfall of the area (TRL, 1993).

2.4. Characteristics and Availability of Expansive Soils in Ethiopia

ERA (2013) expressed that expansive soils are typically clayey soils that undergo large volume changes in direct response to moisture changes in the soil. Unlike collapsible soils, expansive soils tend to increase in volume that is swell as the moisture content of the soil increases and decreases in volume that is shrinkage as the moisture content decreases.

Expansive soils are found in the central, north-western and eastern highlands, in western lowlands around Gambella and in some parts of the rift valley. Local deposits of these soils are also presents throughout the country near rivers; water logged areas and in drainage restricted localities (ERA, 2013). Damage caused by expansive clays is particularly prevalent around Addis Ababa. The aerial coverage of expansive soils in Ethiopia is estimated to be 24.7 million acres that is shown below in figure 2.2.

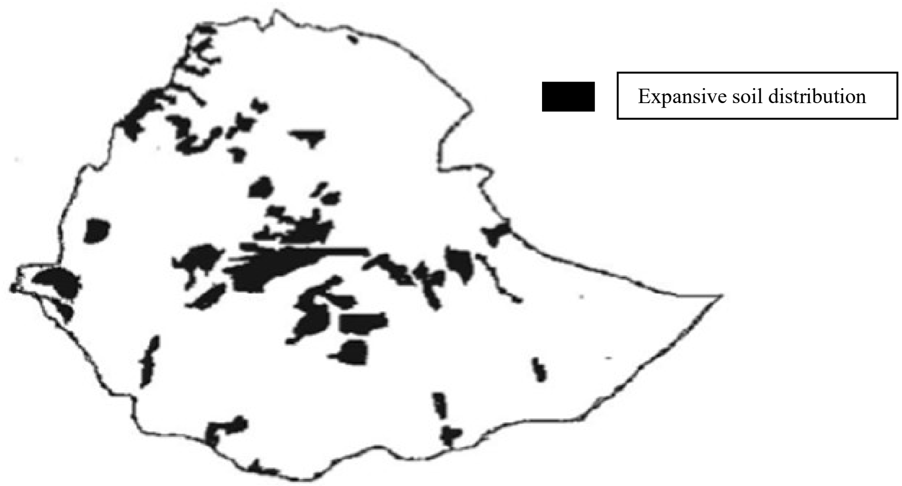


Figure2. 2 Distribution of Expansive Soils in Ethiopia after Tsegaw, 2003 (Assefa, 2016)

2.5. Identification of Expansive Soils

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

2.5.1. Field Identification

Soils that can exhibit high swelling potential can be identified by field observations, mainly during reconnaissance and preliminary investigation stages. Important observations include

- Usually have a color of black or grey
- Wide or deep shrinkage cracks
- High dry strength and low wet strength

- ❖ Stickiness and low trafficability when wet
- ❖ Cut surfaces have a shiny appearance
- ❖ Appearance of cracks in nearby structures

Arid and semiarid areas are particular trouble spots because of large variations in rainfall and temperature (Chen, 1988).

2.5.2. Laboratory Identification

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

2.5.2.1. Mineralogical Identification

Clay mineralogy is a fundamental factor to control expansive soil behavior. Clay minerals can be identified using a variety of techniques. The techniques that can be used are as follow; X-ray diffraction, differential thermal analysis, dye adsorption, chemical analysis and Electron microscope resolution. But these methods are not suitable for usual tests because of the following reasons (Nelson and Miller, 1992; Chen, 1988).

- ❖ They are time consuming
- ❖ They require expensive test equipment and
- ❖ The results can only interpreted by specially trained technicians

2.5.2.2. Indirect Methods

In this method simple soil property tests can be used for the evaluation of swelling potential of expansive soils. Such tests are easy to perform and should be included as usual tests in the investigation of expansive soils. Such tests may include the following tests that presented below.

1. **Atterberg Limits**

In this method, measurements of the atterberg limits of the soil is conducted for identification of all soils and provide a wide acceptable means of rating. Especially when they are combined with other tests, they can be used to classify expansive soils. The relation between the swelling potential of clay soils and their plasticity index is shown in the above table 2.2. While it may be true that high swelling soil will manifest high index property (Chen, 1988).

2. **Free Swell Tests**

The free swell test may be considered as a measurement of volume change in clay upon saturation and is one of the most commonly used simple tests to estimate the swelling potential of expansive clay.

Experiments indicated that a good grade of high swelling commercial bentonite will have a free swell from 1200 to 2000 percent. Soils having a free swell value as low as 100 percent can cause considerable damage to lightly loaded structures and soils having a free swell value below 50 percent seldom exhibit appreciable volume change even under very light loadings. The free swell percentage can be computed by using equation 2.3 from the relationship between initial and swelled volume (Nelson and Miller, 1992).

$$\text{Free swell (\%)} = \frac{\mathrm{Vf} - \mathrm{Vi}}{\mathrm{Vi}} \qquad 2.3$$

Where;

V_i =intial volume

V_f=final volume

3. **Free Swell Index**

Free swell index is also one of the most commonly used simple tests to estimate the swelling potential of expansive clay. The procedure involves in taking two oven dried soil samples passing through 425µm sieve and pour 10gm of sieved soil in to two 100ml graduated cylinders. Distilled water is filled in one cylinder and kerosene in the other cylinder up to 100ml mark. The final volume of soil is computed after 24hours to calculate free swell index. The free swell index is then calculated by using Equation 2.4.

$$\text{Free swell Index} = \left(\frac{\mathrm{Vw} - \mathrm{Vk}}{\mathrm{Vk}}\right) * 100 \qquad 2.4$$

Where;

V_w= final volume in water

V_k= final volume in kerosene

The relation between the degree of expansion and differential free swell index is shown in table 2.5. It is normal to quantify 10cc as the volume occupied by 10gm of soil. This does not account for variations of density (Al-Rawas and Mattheus, 2006).

Table2. 5 Degree of expansion and differential free swell index (Ranjan and Rao, 2002)

Free swell index (%)	Degree of expansion
< 20	Low
20 – 35	Moderate
35 – 50	High
>50	Very high

4. **Cat ion Exchange Capacity**

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

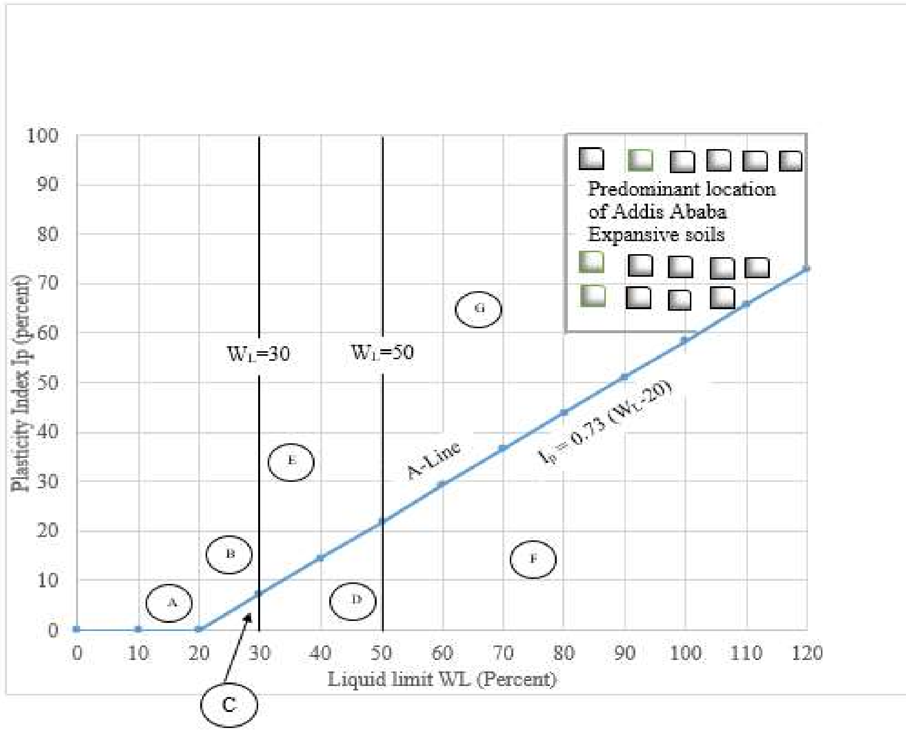
Table2. 6 CEC values of clay minerals after Mitchell, 1976 (Nelson and Miller, 1992)

Clay Minerals	CEC(meq/100gm)
Kaolinite	3-15
Illinite	10-40
Montmorillonite	80-150

2.5.2.3. Direct Methods

This method offers the most useful data by direct measurement and tests. Direct measurement of expansive soils can be achieved by the use of conventional one-dimensional consolidometer to get the swelling potential of the saturated expansive soils. In addition, Double Odometer and Restrained Swell test are used for the determination of expansive soil. Testing should be performed on a number of samples to avoid erroneous conclusions.

To identify the expansive soil of Addis Ababa, both visual identification and laboratory tests are necessary in order to assess the swelling potential of the clay. The expansive clay soils prevalent in Ethiopia are either black (black cotton soil) or dark grey soil. The direct test which provides information on the amount of heaving is swelling pressure tests (Teferra, 1965). Apart from these direct tests, soil mechanics practice for determining the engineering characteristics of expansive soils is usually based on the Atterberg limits and sometimes in conjunction with grain size analysis. One way of using these test results was developed by casagrande, who plotted the liquid limit against the plasticity index that illustrated in figure 2.4 (Cassagrande, 1947). By employing this plasticity chart, many authors found that the A-line in the chart generally defines the expansive soil. Since the great majority of plots representing soils known to be expansive that fall above this line. The location of the expansive soils of Addis Ababa is indicated in figure 2.3.



(A) =Cohesion less soil

(B) = Inorganic clays of low plasticity

(C) =Inorganic silts of low compressibility

(D) =Inorganic silts of Medium compressibility and organic silts

(E) =Inorganic clays of Medium compressibility

(F) =Inorganic silts of high compressibility and organic clays

(G) =Inorganic clays of high plasticity

Figure2. 3 Plasticity chart of soils after Cassagrande, 1947 (Teferra, 1992)

In addition to this, there was a detail investigation on the expansive soils of Addis Ababa which investigates in all over sub cities of Addis Ababa and laboratory test results of Bole road including their investigated depth are listed in table 2.7 (Teferra and Yohannes, 1986).

Table2. 7 Laboratory test results on expansive soil of bole road (Teferra and Yohannes, 1986)

Depth	Color	Gs	Natural moisture	LL	PL	PI	S_f	Clay fraction	Activity
M			W %	%	%	%	%	%	
1.00	Black	2.65	32.4	110	35	75	150	60	1.3
2.00	Grey	2.64	34.8	108	22	76	140	55	1.4

2.6. Effect of Expansive Soils in Asphalt Distress

Some of the most prevalant cases of pavement distress that needs maintenance forces to encounter in expansive soil environments are roughness, longitudinal cracking, and structural deterioration or fatigue cracking. For moderate cases of roughness, simply smoothing the section with a blade-on patch was by far the most frequently used treatment method. In general, roughness will reoccur unless action is taken to prevent moisture fluctuations in the expansive clay sub-grade. Research activities indicate crack filling and sealing is just as effective as conventional full-depth patching for sections with longitudinal cracking. As with roughness, the cracking generally reoccurs within a short time frame from 6 months to 2 years after patching because the sub-grade is not addressed with the patching activity. However, methods currently being used in the Bryan District is geogrid reinforcement to prevent dry-land cracks from propagating through the pavement surface that show promise. If a full-depth patch is to be performed on a section with longitudinal cracking in highly plastic soils, this method of performing the repair should be considered (Sebesta, 2002).

For structural repairs, the research activities indicate cement treatment of either the existing or replacement base with nominal levels of cement from 2 to 4 percent results in good performance. In general, the district survey, field observations and laboratory testing indicate that cement-treated base is preferred to using black bases for repairs. In fact, TxDOT area offices reported that they specifically try to avoid using black base if possible. Regardless of the base used if the repair is on expansive soils, the section will still be subject to environment-related distresses such as swelling of the soil resulting in roughness and longitudinal cracking from dry spells (Sebesta, 2002).

Expansive soil is considered one of the most common causes of pavement distresses in FM roadways. Depending upon the moisture level, expansive soils will experience changes in volume due to moisture fluctuations with in seasonal variations. Those roadways experienced failures in the form of fatigue and rutting in the wheel path and longitudinal or faulted cracking that include edge cracking. The causes of those failures were mainly linked to high PI expansive soil and narrow pavement (Dessouky, et al., 2012).

The major issues with pavements over expansive soils are dealing with the following pavement distresses (Sebesta, 2002).

- **Longitudinal Cracking :-** is a crack which existed in the highly plastic sub-grade soil and reflects through the pavement structure.
- **Alligator Cracking:-** is a crack due to structural deficiency that is available on weak or wet base or sub-grade, excessive voids in HMA, asphalt cement properties such as burnt binder, stripping in lower HMA layers, layer debonding, construction deficiencies such as poor joints and segregation.
- **Roughness:-** Possible Causes of this distress is the presence of other physical distresses, volume change in lower layers, non uniform construction and lack of bonding between pavement layers.

In developing countries like Ethiopia, transportation facilities are very important for sustainable development. However, a better performance of the agricultural sector in particular as Ethiopia's economic growth is highly dependent on it and the sustainable economic growth of the country at large would be achieved through an improvement of the basic infrastructure. Consequently, the road network has been identified as a serious bottleneck for the economic development of the country. Construction of pavement on weak or soft soil is highly risky because such soil is susceptible to swell, have poor shear strength and high compressibility. An appreciable part of Ethiopia is covered by this type of expansive soil (Uge, 2017).

One of the causes of pavement distress is climatic environmental conditions. This may cause surface irregularities and structural weaknesses on the pavement. For example volume change of soil due to wetting and drying resulting from improper drainage may be the prime cause of pavement distress. The increment of moisture content cause wetting of soil and consequently swelling (heave) appears. This is often localized movement that can distort the shoulders, causing settlement & failures. Estimation of the expected heave along the investigation of different road sections show values ranging between 24 mm and 70 mm (Uge, 2017). Figure 2.4 shows how the increment of moisture in the sub-grade layer up lifts the pavement layer upward.

Figure2. 4 Swelling- upward displacement of a pavement

Visual inspection and passable criteria to be the preferred performance evaluation has been used. The bearing capacity of the road material is reduced by moisture to such an extent that it becomes too low to support the vertical loads imposed on the road by a vehicle, resulting in shearing. In addition, the surface of the road becomes so slippery when wet that the wheels of the vehicles lose traction and the vehicle starts slipping to the extent that it becomes dangerous or impossible to make progress. This is a result of the lack of shear strength in the upper parts of the wearing course or inadequate friction between the vehicle tyre and the wearing course material. Slipperiness can be a problem, even with soils with a high bearing capacity but with little friction-generating aggregate.

Figure2. 5 Slippage cracks (Uge, 2017)

The pavement condition survey at Addis Ababa –Weldia road project could reveals that the roughness, rutting, coring action were mainly result of the expansive nature of the sub-grade soil (Gebrehiwot, 2003). When the sub-grade soil expands, it creates tensile stresses in the pavement layers. Recalling that the elastic modulus of a material is simply stress divided by the resultant strain and has the value when the material either cracks or snaps that means when it can no longer accommodate the strain. Hence, regardless of the pavement material, as the stress due to the sub-grade expansion will be identical, the modulus value can be regarded as a parameter for cracking potential (Pavement Information Note, 2011). Unbound materials as in flexible pavements have a very low tensile strength and hence also a low modulus of rupture. Current situation at Addis-Jimma road project also reveals that a series of longitudinal cracks were noticed in the completed shoulder and embankment (Jimilu, 2006). The distress of the road in Ethiopia consisted of longitudinal cracking of the shoulders and of the asphalt in the outer portion of the roadway blamed on the foundation soils. Such phenomena on the pavement includes a vertical crack visible at corner and edge of the asphalt, main cracks extending through sub-base to original shoulder and into the firm material below, deformation of shoulder and multiple cracking from edge to toe, coring activities & highlighted depression in right hand & left lane. There is also evidence from the investigation that the movement can be severe enough to send crack to the base of embankment, effectively splitting it from the main carriage way and the swelling has resulted in the entire embankment moving vertically upward. These movements began at toe of the embankment and progresses along the base to the existing road. They occur in longitudinal parallel series by decreasing in depth and width (Gebrehiwot, 2003). Other different pavement distress types such as rutting occurred on the surface on the urban trunk road and vertical rutting of pavement edge on the rural trunk road that were raised by the effect of the black cotton expansive soil are illustrated respectively in the figures 2.6 (a) and (b).

(a)

(b)

Figure2. 6 (a) Rutting occurred on the surface; (b) vertical rutting of pavement edge

Source: (Dessouky, et al., 2012 and Uba, 2017)

2.7. Improvement Methods of Expansive Soils as a Sub-grade Material

Control of expansive sub-grade materials to ensure uniform support through wet and dry seasons is a key consideration to the long-term performance of concrete pavement structures. Therefore; if an expansive soil is present in the confines of a new roadway project, the expansive potential of the sub-grade must be evaluated and accounted for the design phase (ACPA, 2008). They decided that expansive soils can be mitigated by compact the sub-grade at the proper moisture content, selectively grading the sub-grade material or chemically modifying the sub-grade. Ardani (1992) described that the following treatment methods in alleviating detrimental volume change of expansive soils.

- Sub-excavation & removal of expansive soil and replacement with non-expansive soil
- Application of heavy load to balance the swelling pressure
- Preventing access of water to the soil by encapsulation
- Stabilization by means of chemical admixtures
- Mechanical stabilization
- Explosive treatment to correct swelling shales
- Pre-wetting the soil

❖ Avoiding the expansive soil

Amu, et al. (2005) recommended that stabilization of soil to obtain the desired properties would be the most probable solution in situations where suitable alternative sites are not available, where land disputes are likely to occur over relocation and cost of borrow material is high. They mentioned that expansive soil, mostly clay often requires treatment by stabilization. Expansive soils are characterized by the phenomenon of swelling when wet; this makes them highly problematic as foundation soils. Lea (1990) believed that stabilization is aimed at improving the engineering properties of soil, which may involve increasing the soil density, increase in cohesion, frictional resistance and reduction of plasticity index. Some of the materials that have been used in soil stabilization include lime, cement, fly ash and others. They have been used solely and in combinations with other stabilizing materials to effect a chemical change in the soil. Previous researches have revealed that soil could be stabilized effectively using lime, cement, lime and cement, lime and fly ash combined together in proportion by mass or by weight.

There are different materials in utilization for the stabilization of black cotton soils. Depending on the internal factor which describes the bonding between the soil and the stabilizer utilized, the methods are broadly classified into two types. They are mechanical and chemical stabilization, now a day's plastics also recommended as one of stabilization material (Patle, et al., 2017).

1. **Mechanical Stabilization**: - It is based on the principle of friction that is, when the admixtures are added to soil and compacted the strength is enhanced due to the friction between the soil and the material added. Examples for the materials which increase the strength by this principle are sand, plastic, geo textiles and others.
2. **Chemical Stabilization**: - It is based on the chemical reaction between the material added and the minerals in soil. Examples for this type of stabilizers are lime, fly ash, bituminous materials, cement and others.
3. **Plastics: -** are considered as one of the important invention which has remarkably assisted in different aspects of life whether it might be in scientific field or others.

2.8. Plastic Wastes

2.8.1. Characteristics of Plastic Waste Materials

The term plastic applies to a wide range of material that at some stage in manufacture are capable of flow such that they can be extruded, molded, cast, spun or applied as a coating. Synthetic polymers are typically prepared by polymerization of monomers derived from oil or gas and plastics are usually made from these by addition of various chemical additives. There are currently some 20 different groups of plastics, each with numerous grades and varieties (APME, 2006). Plastics are incredibly versatile materials; they are inexpensive, lightweight, strong, durable, and corrosion resistant with high thermal and electrical insulation properties. The diversity of polymers and the versatility of their properties facilitate the production of a vast array of plastic products that bring technological advances, energy savings and numerous other societal benefits (Andrady and Neal, 2009). One plastic bottle will take more than 450 years to completely break down that's 25 generations (Schmitt, 2017). Sustainable consumption & production, and the circular economy require minimizing use of virgin materials and greenhouse gasses emissions, while delivering clean material cycles (Woldegiorgis, 2017). Therefore; this waste plastic material is the best for recycling purpose in terms of less environment pollution and economy.

2.8.2. Plastic Waste Management and Availability

The communities continually look at new ways to reduce, re-use, recycle and recover packaging and waste as we move towards becoming a zero waste business. The business case is clear. Reducing waste creates efficiencies and lowers costs. Re-using materials extends their life and helping to use less of the earth's precious resources. Recycling allows us to repurpose valuable materials that would otherwise have been wasted. The more we reduce, re-use, recycle and recover our packaging, the greater the cost savings in materials, energy, transport and disposal. The more we can design in a circular way, the more value we can create for our country. With the expansion of Addis Ababa city and the exponential growth of population, a huge increase has been observed on the waste generation from various sources in the city. According to Addis Ababa Sanitation, Beautification and Park's Development Agency (SBPDA), the daily waste generation of the city is about 0.252 kg per capital per day. Applying a population of 2.8 million to get the annual solid waste generation equates to 257,544 tones. Furthermore, about 72.4% of the generated solid waste is collected; of which about 90% available in land filled, about 5% is recycled and 5% is composted according to the SBPDA. The remaining 17.6% is disposed in open spaces, ditches, rivers or it is burned on the streets or in back-yards. In this case, the waste plastics have a great place in the disposal of the solid waste materials and the recycled system of this different waste plastic materials are analyzed by different plastic recycled company in Addis Ababa (Addis Ababa City administration, 2003). Now days, the plastic recycle technology is spread out in our country Ethiopia with a good management system. PET water bottles generate more than 121 million tons of waste each year only in united state of America. In addition, from the total trash in the ocean, 90% is plastic waste (Schmitt, 2017).

The usage and disposal process of plastic materials starts at the household level and with the help of different actors it reaches the recycling plants passing through various stages. One of the major benefits that every participant acquires is the economic benefit which varies according to the role everyone plays that is shown below (Woldegiorgis, 2017).

- ❖ **Sales of used plastic materials at household level**

In most cases the type of plastic waste disposed from residential areas is related with food Items such as Jerry cans, plastic bottles, PP bags for collecting grain & shipping bags come with food and drinking items. On the other hand these materials are seriously needed by the collectors. The next table 2.8 illustrates their demand and value.

Table2. 8 Types of Plastic waste collected from residential quarters (Woldegiorgis, 2017)

Types of plastic waste	**No. of respondents in Gotera**	**No. of respondents in Bole Bulbula**	**Unit price**	**In %**
5 lit. size plastic jerry cans/HDPE/	96	123	5 liter in 2 ETB	73
Plastic home appliances/HDPE/	134	112	Negotiable	82
1Kg of plastic bottles for mineral water	73	111	(6-7)ETB	61

/PET/				
Plastics shopping bags/LDPE/	135	142	Negotiable	92
One piece plastic bags for cement & cereals /PP/	13	8	1.10 ETB	7

Plastic bottles and plastic bags are not usually available at a household level because there is no interest to store the materials due to the small amount paid to these types of plastic waste. In all the above cases residents are encouraged to keep the plastic waste that may be disposed after giving service; hence this contributes in protecting the environment. There is also another way for the collection of these plastic waste materials, we call them middlemen. The activities performed at this stage are serving as a transfer station until the materials reach their final destination (the recyclers). According to the field observation; these group of collectors are better organized than the primary collectors both in their performance and financial capacity. The plastic bottle wastes are compacted in larger plastic bags which originally served to contain cereals or sugar and deliver to the buyers. The measurement of transaction in this case is usually in Kilogram (Kg). Therefore, the middlemen offer 6ETB for one kg plastic wastes. The plastic wastes were collected by the private collectors is shown in figure 2.7.

Figure2. 7 private collectors delivering plastic bottles to middle men (Woldegiorgis, 2017)

❖ **The transaction at the Recyclers level**

This group is well organized and mostly owned by foreign investors who came up with technology which can convert various types of plastic waste into raw materials like granules. These granules could be consumed by other factories to produce appliances for household consumptions or other services. Table 2.9 clearly shows the steady growth of supply of different types of plastic waste to meet the demands of the recycling factories of Ethiopia which is located in Addis Ababa.

Table2. 9 Annual plastic waste consumption of recycling factories (Woldegiorgis, 2017)

Year	**Name of Companies**			
	Weidong Jia plastic Recycling Co.	**Aisai Chemical fiber PLC.**	**Great Wall Packing Materials Plc.**	**AISAI recycled plastic manufacturing plc.**
2012	40 ton	-------	105 ton	-------
2013	84 ton	-------	170 ton	-------
2014	1078 ton	175 ton	2010 ton	-------
2015	1112 ton	178 ton	2030 ton	240 ton
2016	1213 ton	930 ton	2080 ton	360 ton

According to the information gathered during interview with the owners or the General Managers of the factories, they don't face any problem in receiving continuous supply throughout the year. Unlike in the case of the primary collectors or the middle men, they are operating in well-constructed shades where they rented from individuals. However, due to lack of space they are obliged to store the raw materials in open space without any protection from the direct sun. In this case, whenever the waste plastics stay for a long time its strength could be diminished (Woldegiorgis, 2017).

Figure2. 8 Plastic waste stored outside the recycling factories (Woldegiorgis, 2017)

2.9. Improvements of Expansive Soils with Waste Plastic Stabilizer

Soil stabilization is the process which improves the physical and engineering properties of a given problematic soil by adding of suitable admixtures like cement, lime, sand, fly ash or by providing geo textiles and geo synthetics. The new technique of soil stabilization can be effectively used to meet the challenges of society by reducing the quantities of waste and producing useful material from non-useful waste materials. Since the use of plastic in diversified forms such as chairs, bottles, polythene bags and others has been advancing speedily and its disposal has been a problem all the time regarding on the environmental concern. Using plastic as soil stabilizer would reduce the problem of disposing the plastic as well as increases the density and California Bearing Ratio of soil in an economical way **(**Patle, et al., 2017). Soil stabilization means the improvement of stability or bearing capacity of the soil by the use of controlled compaction, proportioning the addition of suitable admixture or stabilizers. The basic principles of soil stabilization are listed as follow **(**Patle, et al., 2017).

- Evaluating the properties of given soil
- Deciding the lacking property of soil and choose effective with less economic cost of stabilizer
- Designing the stabilized soil mix for intended stability and durability values

Plastic increases the shear strength, tensile strength and California bearing ratio of the soil. It can significantly enhance the properties of the soil that used in the construction of road infrastructure and it is also available abundantly in the world. The use of plastic has been enormously increasing these days. But now, plastic has become the significant pollutant of environment because of the use & throw mechanism and everyone should think about this in the present scenario. The use of plastic has to be limited by now otherwise; there would be harshly circumstance that human and environment has to face in near future. Since Plastic is a non-decomposable material, the necessity for recycling or reusing it is also increasing thereby reducing its wastage. Utilizing this Plastic waste for a positive purpose assists in reducing its effect on environment also (Schmitt, 2017). Stabilization was coined as to make anything in a stable condition which itself is a challenging task. Various researches are going on incorporating the waste to the soil and stabilizing it so that it can be utilized for different purposes. Thus, using plastic as stabilizer will help in two ways, reducing the problem of disposing the plastic waste as well as using them as a stabilizer of available black cotton soils wherever possible. The present study deals with the stabilization of Black cotton soils in the Capital Region of newly formed Andhra Pradesh State that is Amaravathi by utilizing plastic strips produced from used plastic chairs. The proper proportion in soil helps in controlling the compaction factor and also makes it very useful. This study indicates that Plastic wastes can be utilized for stabilization of soil which is concluded from different tests performed on soil in various percentages of plastic content. Investigation on the effect of waste polypropylene fiber in shear strength of unsaturated soil samples was done. Here, the percentage of specific gravity (G_s) of the soil increases 0.3% by using 0.5% of fiber (PPF). Generally, the specific gravity, liquid limit (LL), plastic limit (PL) and shrinkage limit values of the three different types of clay soil samples were improved by the inserted waste plastic stabilizer (Singh and Dixit, 2017).

Singh and Dixit (2017) observed that the replacement of 0.5 % waste plastic fiber to the expansive clayey soil reduced its optimum moisture content (OMC) and increased maximum dry density (MDD) but unconfined compressive strength (UCS) of the soil was found to be increased. Further increase in the plastic replacement showed decrease in the MDD and UCS. The test results also showed that with 1% replacement, MDD and UCC were less than the 0.5 % replacement but greater than the untreated soil. The increase in the MDD of the soil with 1% replacement is due to the decrease in the number of voids with the addition of plastic which leads to effective compaction and also increase in the cohesion. Thus authors concluded that optimum percentage of plastic was 0.5 % for best results. Additionally, some of the researchers investigated the effect of plastic granules on weak soil sample with plastic and without plastic granules in varying percentage. The percentage of waste plastic was taken as

0.25%, 0.5 % and 0.75%. MDD was obtained when 0.25 % plastic was added and OMC was less than the soil without plastic for this percentage of soil. Nsaif (2013) in different perspective concluded that the plastic pieces decrease the MDD of the soil due to their low specific gravity. This point can be beneficial in the construction of embankments of lightweight materials. Also, it can be noticed that the addition of plastic pieces decreases the OMC, this may be attributed to that the plastic pieces does not absorb water that is different to clay behavior but they have a tension surface water. Additionally, California bearing ratio (CBR) test is performed on the samples with varying percentages of plastic strips that are 2%, 4%, 6% and 8%. In this regard, the CBR value has been increasing up to 4% plastic content and thereon it started to decrease. From this, it can be inferred that 4% plastic content is the optimum content of utilization of waste plastic in the soil. Further CBR value decreases when 0.25 % plastic is added but it was found to be increased for 0.75 % of plastic. Authors also observed that for the same percentage of plastic, shear stress was increased **(**Patle, et al., 2017).

Aswathy and Greeshma (2016) expressed that Soil stabilization can be done in many ways. But, the stabilization using waste plastic fibers is an economic method, since the stabilizer used here is easily available. A plastic material is any of a wide range of synthetic or semi-synthetic organic solids that are moldable. Plastics are typically organic plastics of high molecular mass but they often contain other substances. They are usually synthetic, which most commonly derived from petrochemicals, but many are partially natural. The researchers think that the addition of plastic strips into the soil will be an innovative technique to improve the shear strength, tensile strength and CBR values of the soil in an economic way. Nsaif (2013) told that wastes such as plastic waste mixed with soil behave similar to fiber-reinforced soils and several researchers presented techniques of using discrete fibers to enhance the strength of soil. The results indicated that the addition of the fibers increases the peak and ultimate strength of the soil. Investigated the effects of plastic waste from waste water bottles as reinforcing material mixed with soil and Series of triaxial compression (UCC & CU) and one dimensional compression tests have been performed with various percentages of plastic waste. The experimental results are presented in the form of stress–strain pore water pressure response and compression paths. The experimental results showed that there is a significant improvement in the strength of soil with inclusion of plastic waste and significant reduction in compression parameters. Finally, the researchers concluded that the waste plastic materials has been chosen as the reinforcement material and it was randomly included in to the clayey soils with different plasticity indexes(PI) at five different percentages of fiber content (0%, 1%,2%,3% and 4%) by weight of raw soil. The use of plastic fibers in unreinforced soil tremendously increases the CBR value, Shear Strength, Resistance to desiccation cracking, reduces Consolidation & Swelling and avoids disposal problems of plastics.

When the polyethylene type of waste plastic material with specific gravity of 0.93 in the form of circular pieces of (1-2) mm diameter and 5mm thickness was used. The researchers stated that effects of random fiber inclusion on consolidation settlement of soil samples were evaluated as function of fiber length, content and consolidation pressure. Prior to the fiber inclusion, consolidation settlement of unreinforced soil sample was determined. A Constant pressure, increasing the fiber contents from 1 to 8% resulted in reducing consolidation settlement of the samples. This is a common trend with all fiber lengths examined. Maximum and minimum consolidation settlements of 7.5 and 2.6 mm were respectively measured for the unreinforced sample and the sample reinforced by 8% fibers that is having 5mm length .They stated that the result showed a reduction in consolidation settlement of approximately 25%. They also did on another experiment, which the work has been performed to investigate the influence of PI and percentage of waste plastic materials on the shear strength of unsaturated clayey soils. For this purpose, clayey soils with different PI were used and mixed with different percentage of waste materials to investigate the shear strength parameters of unreinforced and reinforced samples in terms of direct shear test. In order to determine the shear strength parameters (Cohesion, C and friction angle, φ) of unreinforced and reinforced samples, a series of shear box tests at vertical normal stresses of 100-300KPa and strain rate of 0.2% mm/min were carried out in accordance with ASTM D3080. They examined that shear stresses were recorded as a function of horizontal displacement up to total displacement of 17 mm to observe the post failure behavior as well. Verification tests were also performed in order to examine the repeatability of the experiments. The result of C (KPa) and φ (degrees) values due to the addition of different amount of plastic wastes in sandy and clayey soils are improved slightly and a little bit larger respectively (pal, et al., 2015).

2.10. Improvements of Expansive Soils with Sand

In this study, sand was also used as a stabilizer to magnify the effect of flexi stabilizer. Sand can be occurred from river beds and these are costly now days. The amount of sand for stabilization was taken in the proportion of 10%, 20%, 30%, and 40% by dry weight of soil and a set of laboratory tests were performed to determine the index properties and CBR values of mixed proportion samples. The study reveals that the CBR value increases with the increase in sand content and reaches to a desirable CBR value for sub grade of pavement for MH (Silt of High Compressibility) soil type (Ramesh, et al., 2016). Also, higher amount of sand was used to increase the strength and durability of the black cotton soil blocks. The compressive strength of soil with various mix proportions of sand was drawn. The results showed that the compressive strength of black cotton soil block increased by increasing the sand content up to 50% increased by 12% more than compressive strength of natural black cotton soil block and decreasing the black cotton soil quantity of 40% along with the combination of cement and Metakaolin quantity maintained as 5% and 5% respectively (Kandhasamy, 2015). In other way, the black cotton soil stabilized with sand and cement were studied and some of the laboratory test results are presented as follow (Ramesh, et al., 2016).

- ❖ LL and PL of stabilized soil definitely decreased with increased in sand and cement.
- ❖ There was a decreased in OMC and increased in MDD when addition of sand and cement for about 30%.
- ❖ There was a decreased of CBR value for 20% and increased value for 30%.
- ❖ There was a decreased in shear strength in 20% and increased in 30%.
- ❖ However, there was a gradual increased in 20% for UCS and decreased in 30%.

2.11. Summary

Expansive soil with high amount of clay particles tend to hold a lot of moisture that causes them to swell and shrink water is varied. This swelling and shrinkage causes damage to road and building foundations that cost billions of dollars to repair in every year. Different asphalt distresses in road construction are caused by this type of soils within the design period of the road. Some of asphalt distresses that sometimes caused by expansive soils are Upheaval, Longitudinal Cracking, Alligator Cracking and Roughness. Therefore; to mitigate this asphalt distresses, there are a lot of techniquies including stabilization by using waste plastics.

There is a need to utilize the waste plastic of PET as a stabilizer material which directly improve the poor engineering properties of expansive soils and help in decreasing the requirement of the valuable land for their disposal as well as declining the hazardous environmental impacts. The main properties which improve by this plastics are listed as follow; LL & PL is reduced by mixing of plastics with natural soil mechanically, OMC decreases & MDD increases with the increase of plastic waste, the CBR value improved due to the addition of plastic strips, significant improvement of shear strength parameters (c and Φ) of the treated soils is visualized as well as the UCS value also increase with the use of plastic more than the other stabilizer. But the proper proportion of plastic must be there, which helps in increasing bearing capacity or strength values and decreases the swelling potential of expansive soils. Because of the light weight of plastic; it can be beneficial in the construction of embankments of lightweight materials. It is cost effective stabilizer or inexpensive, it also helps to minimize use of virgin materials & green house gasses emissions in the protection of environment. Producing useful product from non- useful waste materials is leading to the foundation of sustainable economy as well as society.

All of the researchers concluded that the plastic waste materials are the perfect and cost-effective stabilizer, which enhance poor engineering properties of expansive soils in road constructions. The gaps of the researchers on the stabilization of the expansive soils are using of chips material of the plastic waste which has a lot of usage in the other economic dimension of the world. The flexi material, which is a waste of waste for the waste plastic recycled companies, was not used. In addition, they didn't use any modeling system to analyze the results.

3. MATERIALS AND METHODS

3.1. Introduction

The materials that used for this study were expansive soil, waste plastic and sand. Waste plastic in the form of polyethylene terephthalate (PET) and sand used to stabilize expansive soil. These materials help to check the stability point of expansive soil after stabilized in mechanical method. The sand helps to reflect the potential of waste plastic for the stabilization of expansive soils. Not only by looking their potential but also by their availability, low cost, during its recycle period; they are environmental friendly materials as well as using wastes as a resource reduces the renewable source of the country.

Methods that used for the accomplishment of this study are listed as follow; review the characteristics of expansive soil including the problems that existed in the constructed road, their strength & bearing capacity after stabilized by waste PET and the economic & environmental advantages of waste plastic as a stabilizer from different literatures. The other basic method which governed the result of this study was laboratory tests and statically analysis software for analyzing the laboratory test results.

3.2. Description of the Sampling Site

The expansive soil sample of this study was taken from the capital city of Ethiopia, Addis Ababa in Bole sub city at the backside of Bole medhanialem church at 8º 59’ 25.32’’N and 38º 47’ 38.78’’E position from one test pit. The area is located in altitude of 2350m.

- **Climate**

The summers are much rainier than the winters. The average annual temperature in this area is 16.3 °C with an average of 18.0 °C, May is the warmest month. And the lowest average temperatures in the year occur in November when it is around 14.8 °C. The variation in temperatures throughout the year is 3.2 °C. Precipitation here averages 1143 mm with an average of 263 mm greatest amount of precipitation occurs in August. And the driest month is November with 8 mm precipitation. The precipitation varies 255 mm between the driest month and the wettest month. The location of the study area is illustrated in figure 3.1.

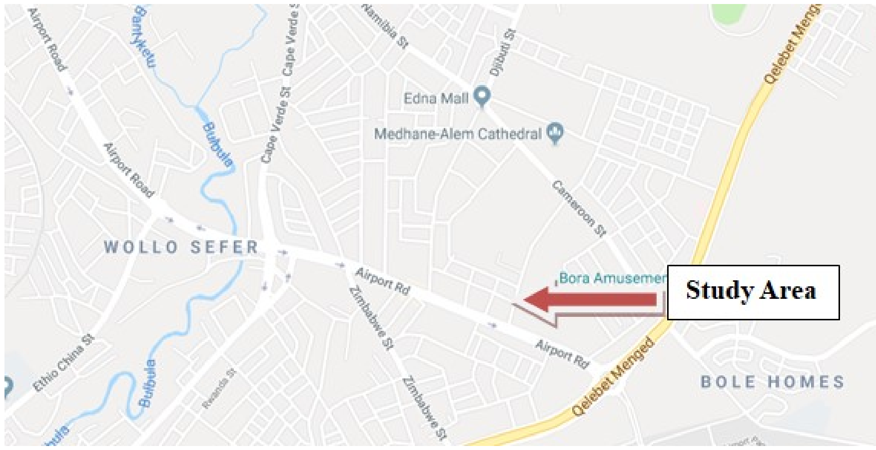


Figure3.1 Expansive soil sample location

3.3. Materials

3.3.1. Expansive Soils

Expansive soils are a type of problematic soil that causes swell and shrinkage due to the variation of moisture content from different sources. The expansive soil that used for this study was categories in to highly expansive soils by using different classification methods. The engineering properties by using different laboratory test results and color of the soil classify the expansive soils as black cotton soil. The black cotton soil has high clay content with high plastic and free swell index values. Both the disturbed and undisturbed soil samples of 400kg were collected at a depth of 3m to protect the soil sample from organic material. The quantity of the material required was determined by laboratory tests that are conducted as well as the number repetition of the tests. The expansive soil sample at the air dry time after taking out from the test pit is illustrated in figure 3.2.

Figure3. 2 Expansive soil samples at air dry time

3.3.2. Flexi Material

The waste plastics are taken from different waste collection areas of bole sub-city in Addis Ababa and took it in to COBA IMPACT plastic recycler company which is located at Kality sub-city in the opposite side of Saint Gebrieal church with 10m far away from the main road to get the main material that are called flexi. Flexi is a material after grinding, washing & drying the waste plastics. It has a size of 2 up to 3mm which is passed by sieve No. 4 or 4.75mm sieve size. Mostly these flexi materials are wastes for the company. Hence, it becomes necessary to move forward with suitable method of low capital construction by using this waste for stabilization purpose. The waste plastic bottles before & after processing the raw or wanted output of the plastic that is called flexi are illustrated in figure 3.3.

Figure3. 3 Waste plastic bottles before and after processing

- **Composition and Properties**

Flexi has a molecular formula of $C_{10}H_8O_4$. It is semi-crystalline thermoplastic material. It is inexpensive, lightweight, strong, durable, and has corrosion resistant with high thermal & electrical insulation properties. It has excellent wear resistance, low coefficient of friction, high flexural modulus and superior dimensional stability. Its density & specific gravity is low. In addition, its solubility in water is negligible and it can't decompose easily in earth.

Flexi material can't have any chemical reaction with the soil; it has only physical interaction due to roughness behavior of the small piece plastics within the interface between plastic and soil.

3.3.3. Sands

The main target of the sand in this study is used as a stabilizer material the same as flexi material for all types of laboratory tests. Then, compare the two stabilizers in different perspectives of improving the expansive soil strength as well as stability by using less percentage of stabilizers, less economic cost and protect environmental pollution. Sands were taken from Langanoo city and it had eight percent original silt content. After washing the sand to clean out the fine grained soil, the silt content became three point five percent. Therefore, it is possible to use it for construction purpose after washing. Because, the standard said that the silt content in the sand should be less than six percent. Add properties of sand

3.4. Methods

Several methods and tools, which have been used for achieving all the specific objectives of this study, are illustrated as follow.

3.4.1. Secondary Data Collection

One of the first specific objectives of the study is to identify the asphalt failure types that caused by expansive soil materials for the construction of sub-grade layer. To identify this failure types; different literature reviewer ideas about parental material, characteristics of expansive soils and their problems on the construction of roadways has been assessed in detail in the next chapter of result and discussion part.

3.4.2. Laboratory Test

There were basically two kinds of laboratory tests that have been done in this study. These tests are tests for classification of the soil and analyzing the strength and bearing capacity of black cotton soils after stabilized by flexi & sand material. The sample preparation before going through the laboratory tests and different laboratory tests are discussed below.

3.4.2.1. Sample Preparation

The natural soil sample of this study was taken from Addis Ababa, Ethiopia in bole sub city at the back side of bole medhanialem church. The soil sample was classified by using the results of natural moisture content, particle-size distribution, Atterberg limit, specific gravity and free swell index tests which also helps to determine the swelling potential of the soil sample.

Before starting the laboratory works, the sample preparation processes were done depending on ASTM standards as follow;

- ❖ Air -dried all the available soil samples
- ❖ Pulverized the air-dried soil as much as possible it becomes fines and sieve it by using different types of sieve for different laboratory tests such as;
    1. From Sieve No. 4 up to 200 for wet sieving particle size distribution,
    2. Sieve No. 40 or 425 micrometer for Atterberg limit and Free swell test,
    3. Sieve No. 10 or 2.0 millimeter for specific gravity test and
    4. Sieve No. 4 or 4.75 millimeter for Compaction, CBR, direct shear and UCS tests.
- ❖ Mixed the black cotton soil that passed through the given sieve No.s with the different percentages of flexi materials randomly until to get the uniform paste and used for different laboratory tests that listed below.

3.4.2.2. Moisture Content

The test was conducted in accordance with ASTM D 2216. The moisture content of the soil is defined as the ratio of the weight of water into the weight of solid material that expressed in percentage. Moisture content is one of the most significance index properties that used in establishing a relation between soil behavior and properties. This test was done for two basic cases; when the natural moisture content of this soil needed to calculate and for every laboratory tests to know their target values such as; for Atterberg test to know the liquid limit & the plastic limit used to express its relative consistency termed as 'plastic index' and for compaction test, it helps to know the optimum moisture content (OMC) as well as maximum dry density (MDD).

3.4.2.3. Particle Size Distribution Test

This test is conducted in accordance with ASTM D 422 testing procedure. The test was used for the classification of natural soil by using both the sieve analysis test for coarse grained soils and hydrometer test for fine grained soils. For Sieve analysis test; wet sieving process was used. For the case of fine grained soils, a hydrometer analysis was performed to measure the amount of silt and clay size particles.

3.4.2.4. Atterberg Limit Test

The atterberg limit test was used to determine the liquid limits (LL), plastic limits (PL) & the plastic indexes (PI) of the natural soil and disturbed soils mixed with different percentages of flexi & sand to realize the reduction of moisture contents after stabilization.

3.4.2.4.1. Liquid Limit Test by Cassa Grande Cup Method

The test standard is based on ASTM D 4318 for the determination of LL, PL and PI. The liquid limit (LL) is a boundary between the plastic and liquid states of the black cotton soils. It is quantified for a soil as specific water content and from physical standpoint; it is the water content at which the shear strength of the soil becomes so small that the soil 'flows' to close standard groove cut. The black cotton soil sample and percentage of flexi during randomly mix time and their mixture at the time of the grooving sample contact to each other on Cassa Grande cup equipment are illustrated in figure 3.4.

Figure3. 4 Random mix of black cotton soil with flexi and the jointed sample at the cup

3.4.2.4.2. Liquid Limit by Cone Penetration Method

The aim of this test is the same with the above one but this is based on the measurement of penetration depth of the standard cone in to the natural & flexi stabilized soil. The test standard is based on BS: 1377 for the determination of LL by cone penetrometer. The 300g air-dried soil sample which was passed The mixture sample of black cotton soil with percentage of flexi and the random mix sample after penetration are illustrated in figure 3.5.

Figure3. 5 Random mixture of soil with flexi and the penetration depth in the mix

3.4.2.4.3. Plastic Limit Test

The plastic limit (PL) of a soil is the water content at the boundary between the semisolid and plastic state. The water content at this boundary was arbitrarily defined as the water content at which soil began to crumble when rolled into threads of specified size of 3mm. The test standard is based on ASTM D 4318 for the determination of PL. A portion of the natural soil and the soil mixed with different percentage of flexi & sand stabilizers that used for the LL test was taken for the determination of PL with less moisture content.

3.4.2.4.4. Plastic Index

The plastic index (PI) of the soil samples which were the natural soil and disturbed soil stabilized with flexi and sand were calculated by using the PL values and their corresponding LL values. It could be calculated based on equation 3.1.

$$PI=LL-PL \qquad 3.1$$

Where;

PI= is the plasticity index of the soil sample

LL= is the liquid limit of the soil sample

PL= is the plastic limit of the soil sample

3.4.2.5. Specific Gravity

The specific gravity of a given soil tells how much the material is heavier or lighter than water. The test was conducted in accordance with ASTM D 854 testing procedure. It was determined by pycnometer method using a soil sample that passed through sieve No. 10. The test includes the determination of the specific gravity for the natural and flexi stabilized black cotton soil. It is defined as the ratio of the weight of a given volume of soil material to the weight an equal volume of water.

$$Gs = \frac{KWs}{Ws+Wpw-Wpw} \qquad 3.2$$

Where;

Gs= Specific gravity of soil based on water at 20^0c

K= Conversion factor used to report specific gravity based on water at 20^0c, can be determined from table

W_S= weight of oven dried soil sample, g

W_{pw}= weight of pcynometer filled with water, g

W_{pws}= weight of pcynometer plus water and soil, g

3.4.2.6. Free Swell Index

The free swell index of black cotton soil is determined by using IS: 2720-40. Free swell or differential free swell, also termed as free swell index. It is the increase in volume of soil without any external constraint when subjected to submergence in water and kerosene. For the case of free swell and differential free swell, the soil was submerged in both water & kerosene and calculated based on equation 3.3 & 3.4 respectively.

- **Free swell by using water (free swell index);**

$$Sf = \frac{\mathrm{Vf-Vi}}{\mathrm{Vi}} * 100\% \qquad 3.3$$

Where;

V_f= final volume of soil specimen read from the graduated cylinder containing distilled water

V_i= initial volume of soil specimen read from the graduated cylinder

- **Free swell by using both water and kerosene (Differential free swell index);**

$$Sdf = \frac{Vd-Vk}{Vk} * 100\% \qquad 3.4$$

Where;

V_d = volume of soil specimen read from the graduated cylinder containing distilled water
V_k = volume of soil specimen read from the graduated cylinder containing kerosene

Table 3.1 shows that how the expansiveness degree of the soil sample can be expressed by using free swell index, LL and PL.

Table3. 1 representation of degree of expansiveness by free swell index, LL and PL(Holtz and Gibbs)

Free swell index	Degree of expansiveness	LL	PL
< 20	Low	0.50	0-35%
20-35	Moderate	40-60%	25-50%
35-50	High	50-75%	35-65%
>50	Very high	> 60%	>45%

About the expansive level of the clay soil, there is also another parameter which is called Swelling Potential. Swelling potential is manifested by swelling pressure. Since swelling pressure is the actual pressure required to keep the volume of swelling soil constant, its magnitude indicates the danger that is to be anticipated. The swelling potential of the samples is easily calculated by the Anderson empirical formula that is presented in equation 4.3 (Teferra and Yohannes, 1986).

$$S = 0.23\ Ip - 3.12 \qquad 4.3$$

Where;

S = Swelling potential &

I_p = Plasticity index

Anderson has correlated the swelling potential (S) and plasticity index (I_P) with the degree of expansion as given in table 4.2.

Table3. 2 Relationship of degree of expansion with Ip and S according to Anderson (Teferra and Yohannes, 1986)

Degree of expansion	Plasticity Index (I_P)	Swelling potential (S)

Low	20	1.5
Medium	20-31	1.5 – 4.0
High	31-39	4.0 - 6.0
Very high	>39	> 6.0

3.4.2.7. Soil Classification

The unified soil classification system (USCS) was used to classify the soil type by the help of the above two laboratory tests such as particle size analysis and Atterberg limit test. In this type of classification system, name and symbols were used to distinguish between the typical and boundary soil groups. It is based on recognition of the type and predominance of the constituents considering grain size, gradation and plasticity. In addition; natural moisture content, Specific gravity and free swell index were the basic tests for the determination of the classified soil relative properties like degree of expansiveness and swelling potential.

3.4.2.8. Compaction or Proctor Test

Many types of earth construction like embankment and highway run ways need soil fill which is placed in layers and compacted very well. The tests were conducted for natural soil and different percentages of flexi & sand stabilized soil samples in accordance with ASTM D 698 and ASTM D 1557 testing procedures for standard & modified compaction test respectively. In this study, both type of proctor test that are standard and modified were conducted to compare the two test results of stabilized expansive soil samples. The main target of this test is to get the maximum dry density (MDD) and optimum moisture content (OMC) of the given soil samples with and without the addition of stabilizers.

3.4.2.8.1. Standard Compaction Test

In this case, the hammer weight that used for the compaction is 2.5kg with the hammer drop height of 0.3m. It was compacted with a three layer system for each layer 25 blow was used with a 590kg/cm^3 of compaction energy. This proctor method is adequate for most applications like highway embankments earth dams and retaining back fill.

3.4.2.8.2. Modified Compaction Test

For this type of compaction, the hammer weight is 4.5kg with the hammer drop height of 0.45m. It was compacted with a five layer system with blow of 25 for each layer and 2700kg/cm^3 of compaction energy was used. A well compacted soil is mechanically more stable, has a high compressive strength & high deformation resistance than a loose soil. This type of compaction is usually best for heavier load application like airport runway.

- **Maximum Dry Density (MDD)**

The MDD was conducted for both natural and different percentages of flexi & sand stabilized soil by varying the moisture content. The same procedure was repeated until minimum of four sets of sample were taken for moisture content determination. The blows were uniformly distributed over the surface of each layer. The dry density was calculated depending on equation 3.5.

$$\gamma_{dry} = \frac{\gamma wet}{(1+W)} \qquad 3.5$$

Where;

γ_{dry} = dry unit weight of the soil

γ_{wet} = wet unit weight of the soil

W = water content

- **Optimum Moisture Content**

From the available moisture content & dry density data of the natural & stabilized soil, plot dry density versus moisture content graph. Additionally, draw a curve termed the 100% saturation curve or zero air voids on that plot. Finally, obtain the peak value of MDD and the corresponding value of OMC from the plotted graph.

3.4.2.9. California Bearing Ratio Test

The CBR test is conducted in accordance with ASTM D 1883. In this case, CBR test was done by using both standard and modified methods in different layer with different load application to assess the effect of heavy compaction in expansive soils. Determination of CBR in remolded soil sample for both natural and different percentages of flexi & sand stabilized soil were done.

In this study, both un-soaked and soaked CBR methods were also used. If it is proposed to carry out the CBR test on un-soaked specimen, the moisture content for remolding sample should be the same as the equilibrium moisture content which the soil is likely to reach subsequent to the construction of the road. While it is proposed to carry out the CBR test on a soaked specimen, the moisture content for remolding sample should be at the optimum and soaked under water for four days or 96 hours. By using corrected and standard load values of the two desired penetration depths, the CBR values were calculated depending on equation 3.6.

$$\boldsymbol{CBR = \frac{PT}{PS} * 100} \qquad 3.6$$

Where;

CBR = California bearing ratio (%)

P_T = Corrected unit or total test load corresponding to the chosen penetration curve, and

P_S = Unit or total standard load for the same depth of penetration

There is also swelling measurement due to soaking of specimens for four days that is called percent swell of the soil. The initial dial reading of the dial indicator on the soaked CBR mold before soaking the sample and the final dial reading of the dial indicator at the end of 96 hours soaking period were taken and calculated as percent swell value.

3.4.2.10. Shear Strength Test

In this case, unconsolidated un-drained shear strength of expansive soils which is without variation of moisture content was done. This is performed by using ASTM D 2850 for both natural and different percentages of flexi & sand stabilized soil. Plot a graph by using the three different normal loads that were applied on the soil sample and their maximum shear stresses that get from each normal stresses. Finally, Cohesion (C) and internal friction angle (Φ) values for each soil samples were calculated from the plot.

3.4.2.11. Unconfined Compressive Strength Test

The test is performed by using ASTM D2166 test standard. The test was used to determine for the natural and different percentages of flexi & sand stabilized soil. To perform UCS test, remolded soil sample at a given OMC was used and extruded it from the sampling tube. And then a cylindrical sample of soil is trimmed such that the ends are reasonably smooth or leveled equally up to the length-to-diameter ratio equals to two. In this study, the UCS test was done by both methods of modified & standard compaction for the preparation of remolded soil samples. This is to compare the compressive resistance of soil by using different OMC and MDD. Soils also Categorized depending on their unconfined compressive Strength (UCS) value that are illustrated in table 3.3.

Table3. 3 Category of soil in terms of UCS value (Braja, 1997)

Unconfined compressive Strength, q_u(Kg/cm^2)	**Consistency of Soil**
< 0.25	Very soft
0.25-0.50	Soft
0.50-1.00	Medium
1.00-2.00	Stiff
2.00-4.00	Very Stiff
>4.00	Hard

3.4.3. Statically Analysis Method

To generalize the results of different laboratory tests which identify the properties of the soil sample, correlation and regression analysis were used. Correlation & regression analysis of the given independent and dependent variables were done by using number crunching statistical system (NCSS) software. This software is the appropriate, available and easy to analyze the simple model of single linear regression and correlation of the laboratory test results of this study.

Materials and methods that have been used for the accomplishment of this study by controlling the factors, which affect the strength of flexi stabilized black cotton soil such as compaction level, amount of moisture content, variation of temperature and dry-wet effect are generalized in figure 3.6.

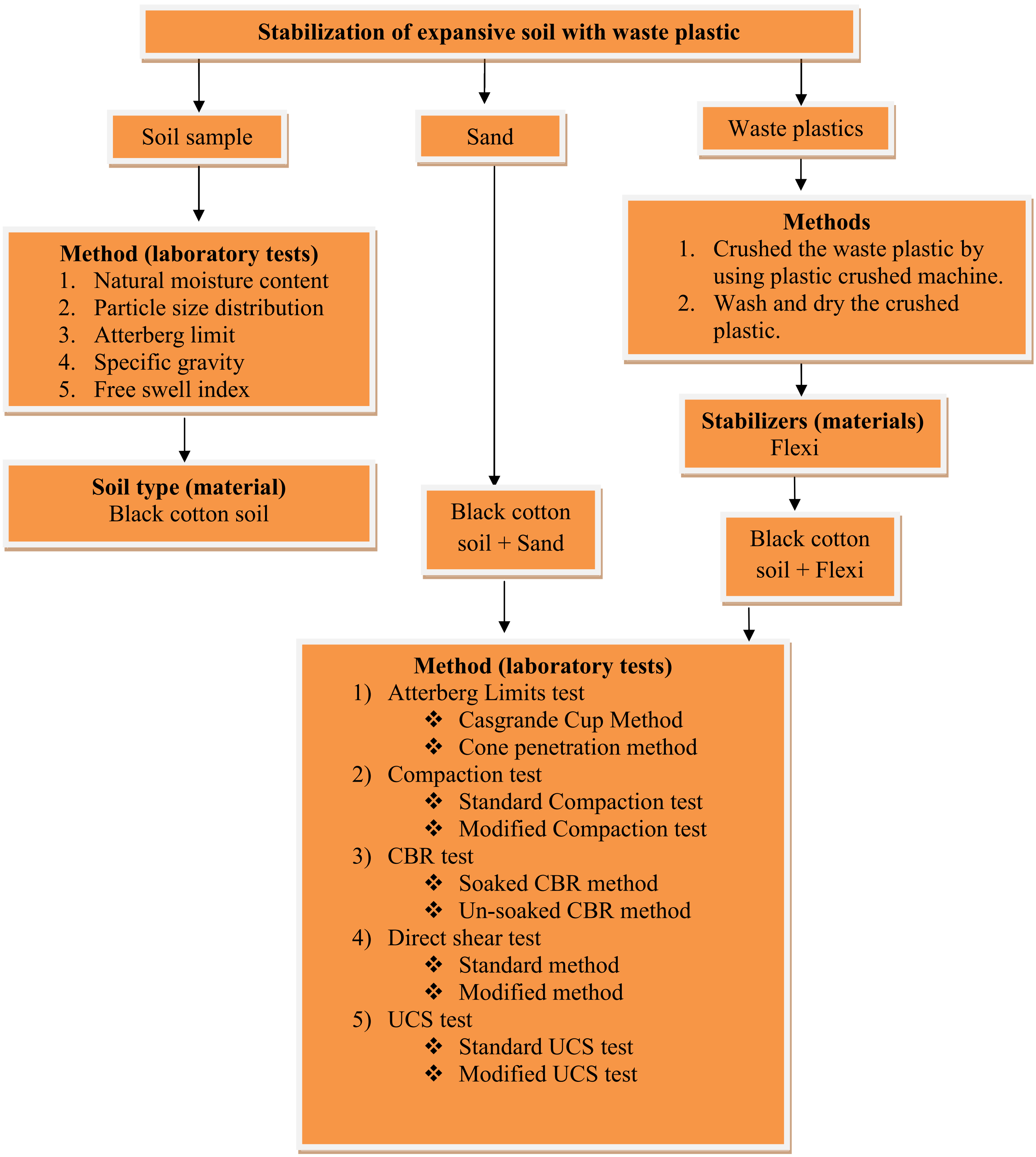


Figure3. 6 Schematics of the research

4. RESULTS AND DISCUSSION

4.1. Classification of Natural Soil

During sub-grade characterization, samples are necessary for further visual description and laboratory tests. The number of test specimens depends on the number of soil layers identified from pits as well as their engineering behavior. Most of the sub-grade test specimens should be taken from as close to the top of the sub-grade as possible, extending down to a depth of 50cm below the planned sub-grade elevation or take a depth up to 1.5m and take 2m depth for the construction of new alignment. However, some tests should be performed on soils encountered at a greater depth, especially if those deeper soils are soft and highly compressible. Likewise, the extent of the laboratory programmed depends on the criticality of the design and on the complexity of soil conditions. The primary test to assess the strength of the sub-grade is the California Bearing Ratio (CBR) test. Where possible, CBR tests should be performed on undisturbed specimens that represent the natural conditions of the sub-grade (ERA, 2013).

Particle size distribution test was used to determine the amount of different particle sizes. It helps to differentiate the amount of gravel and sand from the fine-grained soils by using sieve analysis test and hydrometer test was used to differentiate the amount of silt from clay. Totally from the sample on the particle size distribution curve almost 79.38% of the soil were passing through No. 200 sieve and the rest 20.62% were retained on different sieve sizes above No.200 sieve ; from this the amount of gravel, sand, silt and clay was 0%, 20.62%, 26.46% & 52.92% respectively. The total distributions of the above particle sizes are presented in the logarithmic scale graph in figure 4.1.

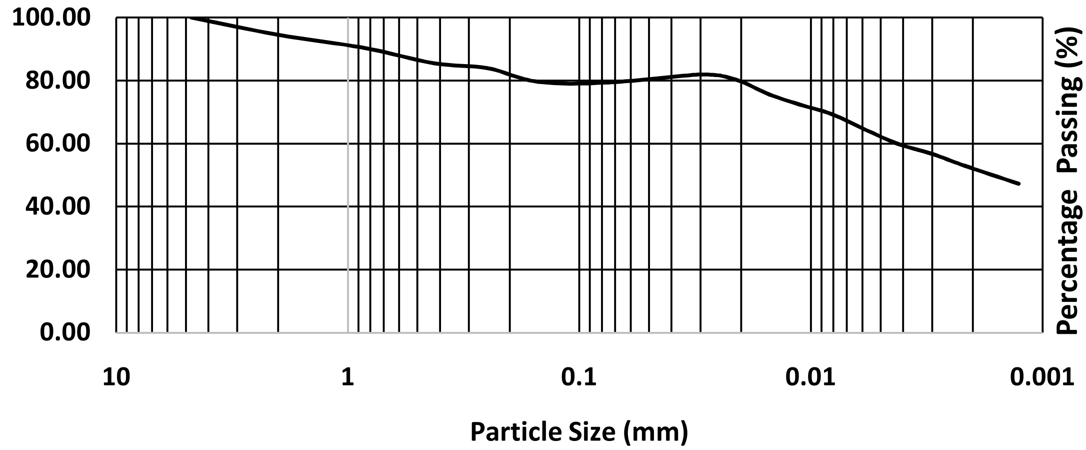


Figure4. 1 Particle-size distribution curve of the soil sample

The atterberg limit for the case of Cassa Grande cup method in terms of liquid limit (LL), plastic limit (PL) & plastic index (PI) values were 102%, 36% & 66% respectively. By using the particle size distribution & atterberg limit tests, the soil sample could be categorized in to CH or highly plastic clay based on the Unified soil classification system (USCS). The USCS helps in identifying soils according to their texture and plastic characteristics. Soils seldom exist in nature separately as sand, gravel or any other single component. They are usually found as mixtures with varying proportions of particle sizes. Each component part contributes its characteristics to the soil mixture. The USCS is based on those characteristics that control how the soil behaves as an engineering material. In the USCS,the soil is given a descriptive name and a letter symbol indicating its principal characterstics based on the followings (ERA, 2013).

- ❖ Percentages of gravel, sand, and fines such as silt and clay (fraction passing the 75 micron sieve)
- ❖ Shape of the grain-size distribution curve and
- ❖ Plasticity and compressibility characteristics

By using graph which can be drawn by PI (%) and LL (%), classify the soil depending on USCS "A" Line and "U" Line equations. The graph of PI and LL for CH classification of the taken soil sample is drawn in figure 4.2.

"A" Line: PI = 0.73(LL-20) 4.1

and

"U" Line: PI = 0.9(LL-8) 4.2

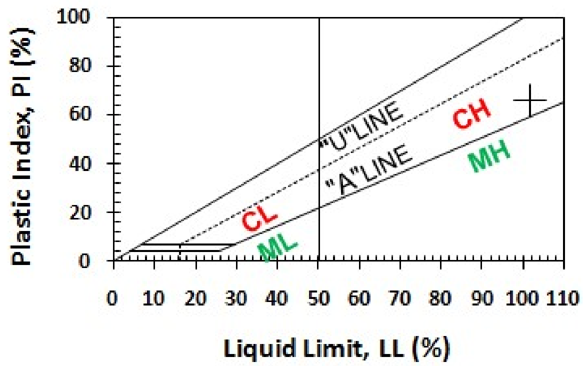


Figure4. 2 Plastic index and liquid limit graph of soil classification

The PI is the main parameter that governs the engineering performance of fine-grained soils. In addition to PI, liquid limit (LL) is also useful indicators of the engineering performance of fine-grained soils. The PI represents the range of water content over which the soil remains plastic and normally the higher the PI, the higher the percentage of clay particles in the soil. Also the more plastic soil, the more likely it is to be compressible. This type of soil will have a greater potential to shrink & swell and it will be less permeable. Totally, the engineering characteristics of inorganic clays are the following;

- ❖ Generally possess low shear strength
- ❖ Plastic and compressible
- ❖ Can lose part of shear strength up on wetting
- ❖ Can lose part of shear strength up on disturbance
- ❖ Can shrink up on drying and expand up on wetting
- ❖ Generally very poor materials for back fill
- ❖ Generally poor material for embankments
- ❖ Can be practically impervious
- ❖ Clay slopes are prone to landslides

In addition, the natural soil sample average natural moisture content, W (%) was 38.23%, specific gravity (G_s) was 2.66 and free swell index in terms of free swell (S_f) & differential free swell (S_{df}) had values of 110% & 90.9% respectively. Therefore, by looking the above five test results, the natural soil sample has been categorized in to expansive soil by the name of black cotton soil depending on the color & behavior. Why it became expansive soil, because expansive soils have a behavior of swell due to the addition of water and depending on ERA manual, the soil which has PI value of greater than 15% (PI > 15%) is classified as expansive soils. Potentially problematic expansive soils are located near the ground surface. These soils have natural moisture contents of equal or less than the plastic limit of the soil (ERA, 2013).

By using equation 4.3, swelling potential (S) was 12.06 and PI value of this black cotton soil was 66% from the laboratory test result. Therefore; depending on the above Anderson table 4.2, the classification of soil based on the expansion level is very high expansive soil type.

In addition to the above tests, the tests which determine the strength of the soil sample and both standard & modified compaction test, which was used for finding OMC & MDD of the remolded soil sample, were done. The standard and modified compaction tests of the available black cotton soil in terms of OMC had values of 37.78% & 26.11% with 1.13g/cm^3 & 1.368g/cm^3 values of MDD respectively. The soaked California bearing ratio (CBR), Unconfined compressive strength (q_u) and failure strain (ε_f) of the remolded soil sample had values of 2.14%, 246.54Kpa and 8.68% respectively for the case of standard compaction while in the case of modified compaction; the CBR had very less value with high percent swell, q_u had 1,373.1Kpa value with 6.053% value of ε_f of the natural soil. For standard and modified compaction methods; the un-drained shear strength (S_u) of this soil sample had 123.27Kpa & 686.55Kpa with 0.06KN/m^2& 0.05KN/m^2 values of C respectively. The angle of internal friction (Φ) for both compaction methods had zero values.

From the above test results of the standard and modified remolded soil samples; it can be concluded that the OMC has high value and the MDD has less value during the standard compaction method than the modified because of the light and heavy compaction effort respectively. The soaked CBR has high value during the standard than modified method because of the heavy compaction energy on the expansive soil during the soaked time or the expansive soils can't resist the heavy load at the soaked time. The q_u and S_u have less value during the standard time than the modified because of the heavy compaction load during the un-drained time. The C & Φ values of shear strength test had high and constant zero during standard method than modified respectively. The standard compaction is better than modified compaction for expansive soil. When the heavy load applied on expansive soil which has less moisture content and the compacted soil was soaked in water, it become highly swell. This swell is because of the concentration of montmorillonite mineral in one place. Therefore, the above strength tests also revealed that the soil sample is categorized into highly expansive black cotton soil for the construction of sub-grade layer of the pavement.

Depending on ERA manual (2013) site investigation for preliminary and final design focuses on soil condition & the situation of the sub-grade to support the pavement and earthworks operations. An assessment of the sub-grade strength is required during site investigation for pavement design, together with the conditions of compaction and reuse of excavated materials. The performance of road is significantly affected by the characteristics of the sub-grade. Desirable properties of the sub-grade include strength, stiffness, drainage, ease of construction and low compressibility. These properties can have a significant influence on road performance and long-term maintenance. The sub-grade must be strong enough to resist shear failure and have adequate stiffness to minimize vertical deflection. It should also form a suitable platform to achieve the required compaction of pavement layers above sub-grade level. Stronger and stiffer materials provide a more effective foundation for the riding surface and will be more resistant to stresses from repeated loadings and environmental conditions (ERA, 2013). One of the important laboratory test which is used to determine the bearing capacity of the soil as a sub-grade material is CBR. The general relationship between CBR values and the quality of the sub-grade soils used in pavement applications is described as follow in table 4.1 (Bowles, 1992).

Table4. 1 Quality of sub-grade depending on CBR values (Bowles, 1992) use ERA

CBR values	Quality of sub-grade
0-3%	Very poor sub-grade

3-7%	Poor to fair sub-grade
7-20%	Fair sub-grade
20-50%	Good sub-grade
>50	Excellent sub-grade

The expansive soil sample of this study doesn't have good bearing capacity for road construction of sub-grade layer. Therefore, the soil requires modification or stabilization to improve its workability and poor engineering properties. The general discovery of the soil sample by using the above different laboratory test results is illustrated in table 4.2.

Table4. 2 Different laboratory test results of natural soil sample

Test categories	**Laboratory tests**	**List of description**	**Results**
Index properties	Particle-size distribution(PSD)	Gravel	0%
		Sand	20.62%
		Silt	26.46%
		Clay	52.92%
	Atterberg limit (casagrande cup method)	LL	102%
		PL	36%
		PI	66%
Test categories	**Laboratory tests**	**List of description**	**Results**
Index properties	Natural moisture content	W	38.23%
	Specific gravity	G_S	2.66

Swelling properties	Free swell index	Free swell(S_f)	110%
		Differential free swell(S_{df})	90.9%
	Swelling potential (from empirical formula)	S	12.06
Compaction tests	Standard compaction	MDD	1.13g/cm^3
		OMC	37.78%
	Modified compaction	MDD	1.368g/cm^3
		OMC	26.11%
Strength Tests	Standard Soaked CBR	CBR	2.14%
	Standard shear strength	C	0.06KN/m^2
		Φ	0
	Modified shear strength	C	0.05KN/m^2
		Φ	0
	Standard Unconfined compression strength(UCS)	q_u	246.54KPa
		ε_f	8.68%
		S_u	123.27Kpa
	Modified Unconfined compression strength(UCS)	q_u	1,373.1Kpa
		ε_f	6.053%
		S_u	686.55Kpa
Unified soil classification system (USCS)		**CH (highly plastic clay)**	
Soil color-Black		**Highly expansive black cotton soil**	

4.2. Atterberg Limit Results of Flexi Stabilized Soil

Atterberg limit test results can be expressed in terms of liquid limit(LL), plastic limit(PL) & plastic index(PI) of the randomly mixture of flexi materials with natural black cotton soil in order to decrease the moisture content of natural black cotton soil. Not only the flexi material but also sand was used as one of a stabilizer agent in mechanical stabilization to compare or magnify the effect of flexi materials in terms of stable engineering properties, less economical cost as well as good environmental aspects. The LL results have been done by using two methods that were Cassa Grande cup and Cone penetration methods. The LL, PL and PI results of improved soil with different percents of flexi are presented in table 4.3.

Table4. 3 Atterberg limit results of flexi stabilized soil

% of flexi	LL (%)		PL (%)	PI (%)	Variation (%)	PI (%)	Variation (%)
	Cassa Grande cup	Cone penetration		Cassa Grande cup		Cone penetration	
0	102	83	36	66	0	47	0.00
1	102	83	36	66	0	47	0.00
2	99	82	35	64	3.03	47	0.00
3	93	81	34	59	10.61	47	0.00
5	91	80	34	57	13.64	46	2.13
7	93	77	33	60	9.09	44	6.38
9	96	74	32	64	3.03	42	10.64

The above table 4.5 for the case of Cassa Grande method shows when there is the addition of flexi by percent in a black cotton soil started from 1% up to 5%, the LL and PI are decreasing while with the addition of flexi above 5%, the LL and PI are increasing but still it is less than the LL and PI values of natural black cotton soil. The PL of the stabilized soil also decreases slightly up to 9% of flexi stabilizer. Depending on table 4.5, the LL and PI values for cone penetration method are decreased up to 9% of flexi stabilizer. This method is more advisable for the non plastic soils than the clay soils. In this case, the flexi stabilizer has 2mm size that is why the cone penetration was used to assure the potential of flexi stabilizer by reducing the moisture content of natural black cotton soil. The difference values of LL for both Cassa Grande cup and Cone penetration methods of different percentages of flexi stabilized black cotton soils are illustrated in figure 4.3.

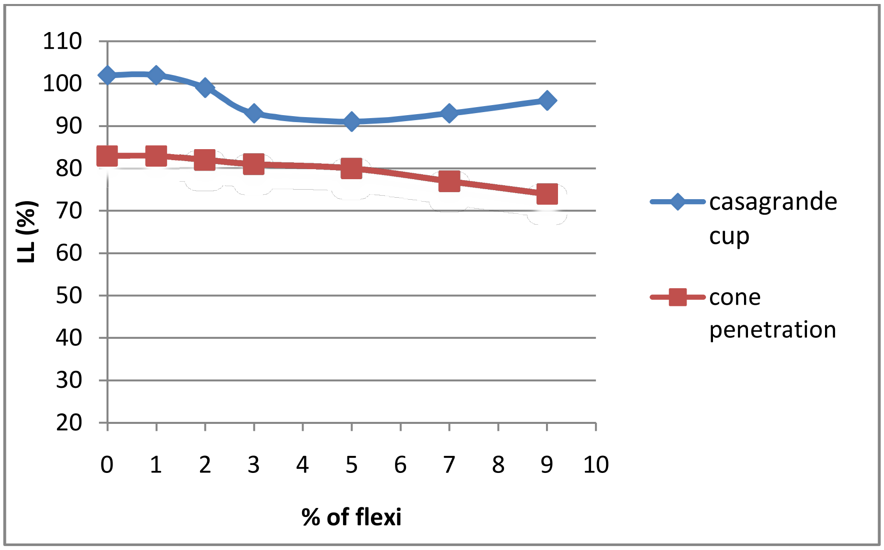


Figure4. 3 Variation of LL as a function of flexi for Casa Grande cup and cone penetration

Depending on table 4.5 and figure 4.6, the LL of 5% flexi stabilized soil is decreased by 10.8% than the natural black cotton soil for the Casa Grande cup method while the LL value of 9% flexi stabilized soil for the Cone penetration method is decreased by 10.8% than natural black cotton soil. Therefore, 5% of flexi material is the optimum stabilizer content for Casa Grande cup method of mechanical stabilization of this study. So, the flexi stabilizer helps the black cotton soil to reduce the moisture content amount in both methods.

When the waste plastic is crushed into small pieces, the flexi material brings a roughness characteristic. Consequently, figure 4.4 show that the difficult of rolling the plastic limit at 3mm thread diameter.

Figure4. 4 plastic limit of flexi stabilized soil

The PL of 9% flexi stabilized soil is decreased by 11.1% than the natural black cotton soil. The optimum content of flexi stabilizer for PL in this study is 9%. The different values of PI for flexi stabilizer in both methods of Cassa Grande and Cone penetration are illustrated in figure 4.5.

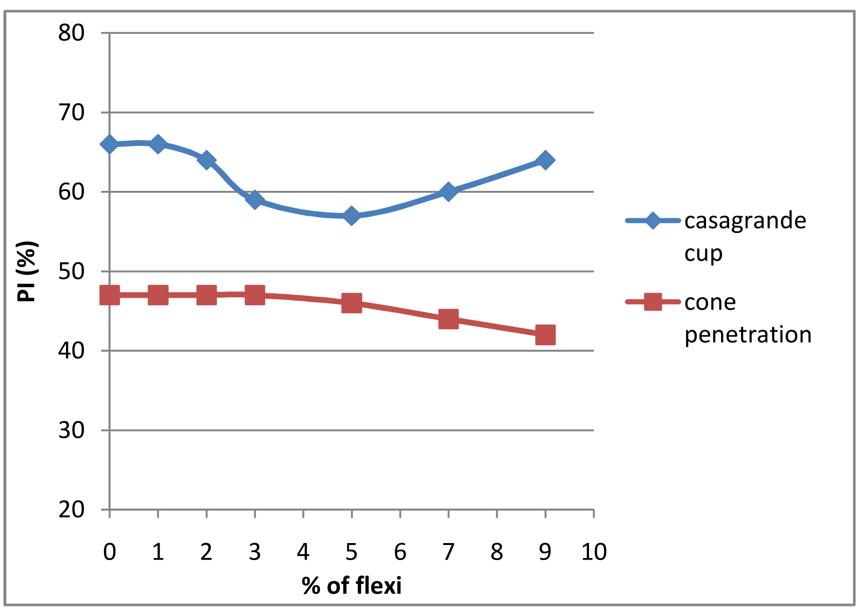


Figure4. 5 Variation of PI as a function of flexi for Cassa Grande cup and cone penetration

The above figure 4.8 presents that the difference of PI values between the Cassa Grande cup and cone penetration methods. Generally, the PI value of the natural soil is decreased by 13.6% when stabilized by 5% flexi material for Cassa Grande cup method while the PI value of the natural soil is decreased by 10.6% when stabilized by 9% flexi material for Cone penetration method. Therefore; 5% flexi stabilizer is the optimum content for the Cassa Grande cup method of mechanical stabilization.

Another stabilizer material which is called sand was used to compare with the flexi stabilizer in a mechanical method. The table 4.4 shows that the values of LL (%), PL (%) and PI (%) values of stabilized soil with different percentages of sand stabilizer in a Cassa Grande cup method.

Table4. 4 Atterberg limit results of sand stabilizer

% of sand	LL (%)	PL (%)	PI (%)	PI Variation (%)
0	102	36	66	0.00
1	102	34	68	3.03
2	97	33	64	3.03
3	92	31	61	7.58
7	97	31	66	0.00

From the above test results of black cotton soil that stabilized with flexi and sand in different percentages; the LL value is decreased more in sand stabilizer by 1% difference in 3% stabilizer content and the PL value of sand stabilizer at 3% stabilizer content is also decreased by 3% than flexi stabilizer. The sand stabilizer has more potential to decrease the LL and PL for the optimum stabilizer content of 3% of the two stabilizers. Because of sand has no plasticity property and has a power to share the moisture content of black cotton soils. The PI value of the soil that stabilized with flexi at 3% stabilizer content is less by 2% than the soil stabilized with sand. In general, black cotton soil stabilized with flexi has better PI value more than soil stabilized with sand in mechanical stabilization method. The PET has a chemical composition of $C_{10}H_80_4$ which can't be react with water. But due to the replacement of black cotton soil by some amount of flexi materials in terms of percentages of the dry soil mass, the moisture content is decreased due to the decrease of void spaces in the soil samples. This is clearly shown by the fact that PI of treated soil decreased with increasing flexi stabilizer amount up to the optimum limit of 5%.

4.3. Compaction Test Results of Flexi Stabilized Soil

Compaction is one of sub-grade soil test which determine the optimum moisture content (OMC) and maximum dry density (MDD) of the black cotton soil which is stabilized with different percentages of flexi and sand stabilizers. In this case, both types of compaction methods that are standard and modified proctor test were done to analyze the effect of light and heavy compaction effort on the stabilized black cotton soil with different percentages of flexi in mechanical stabilization method. The OMC and MDD results of flexi stabilized black cotton soil in both standard and modified compaction methods are listed below in table 4.5.

Table4. 5 The OMC and MDD results of flexi stabilized soil in both compaction methods

% of flexi	**Standard compaction test**		**Modified compaction test**	
	OMC (%)	**MDD (g/cm^3)**	**OMC (%)**	**MDD (g/cm^3)**
0	37.78	1.130	26.11	1.368
1	36.1	1.146	21.49	1.421
2	30.8	1.151	24.97	1.397
3	36	1.144	25.13	1.374
5	37.47	1.143	25.9	1.324
7	38	1.112	26.23	1.32
9	38	1.092	26.68	1.32

From the above results; the OMC is decreased up to 2% and increased started from 7% up to 9% of flexi stabilizer even as MDD is increased up to 2% and decreased started from 3% up to 9% of flexi stabilizer for standard compaction test. In addition, the OMC of black cotton soils for the case of modified compaction is decreased up to 1% and increased started from 2% up to 9% of flexi stabilizer while the MDD is increased up to 1% and decreased started from 2% up to 9% of flexi stabilizer. The optimum stabilizer content of the standard and modified compactions has been 2% and 1% flexi stabilizer respectively. The different values of OMC in flexi stabilized black cotton soils for both standard and modified compaction methods are illustrated in figure 4.6.

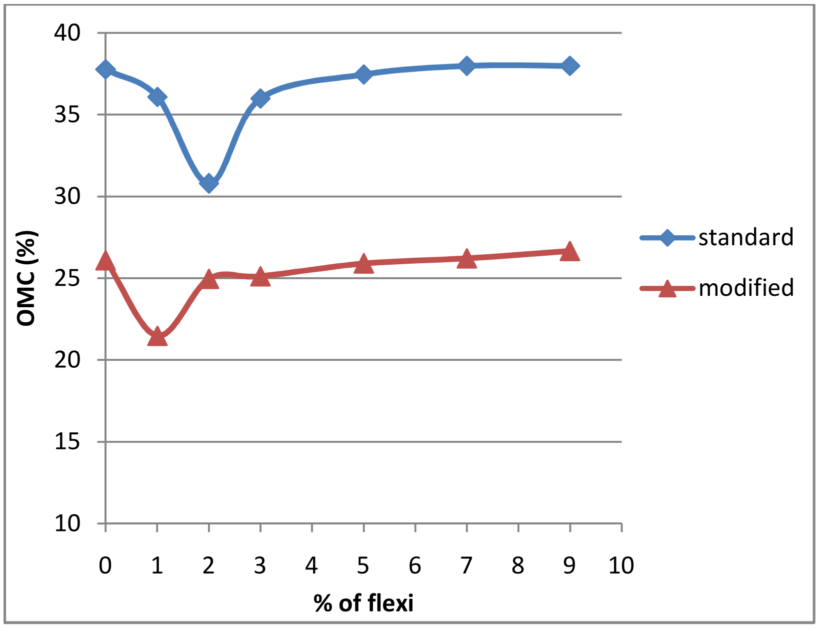


Figure4. 6 Variation of OMC as a function of flexi for both compaction methods

The above figure 4.10 shows that the difference values of OMC on the flexi stabilized black cotton soils for standard and modified compaction methods. The values of OMC at standard compaction of 2% and modified compaction of 1% of flexi stabilizer are smaller by 18.5% and 17.7% than the natural black cotton soil OMC values respectively. Therefore; the improvement of the OMC in optimum flexi stabilizer for standard compaction is better by 0.8% than the modified compaction method. The MDD values of flexi stabilized soil for both standard and modified compaction methods are illustrated in figure 4.7.

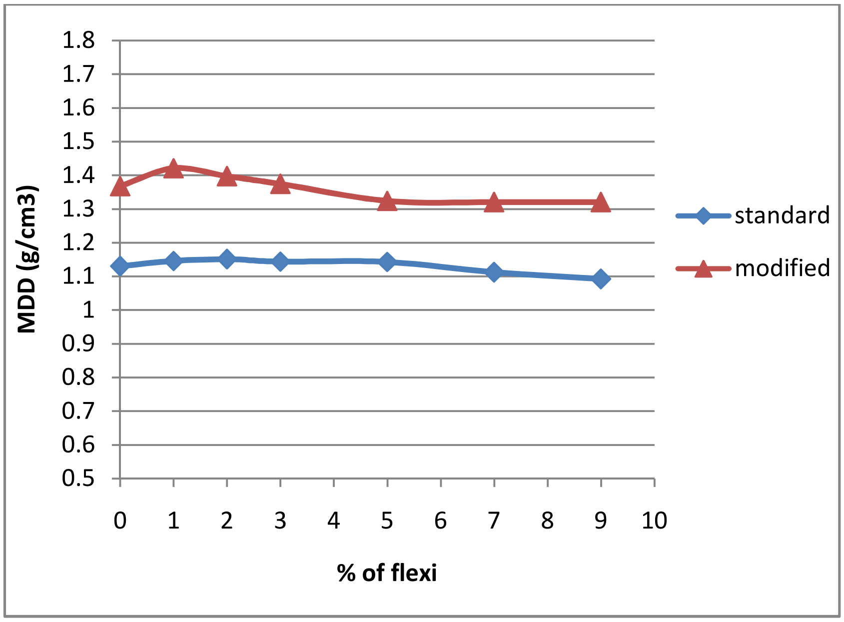


Figure4. 7 Variation of MDD as a function of flexi for both compaction methods

The above figure 4.11 revealed that how much is the difference between the MDD values of standard and modified compaction tests. The improved values of MDD at the standard 2% and modified 1% of flexi stabilized soil are larger by 1.9% and 3.9% than the natural black cotton soil respectively. Therefore; the MDD value of the effective flexi stabilizer for modified compaction is larger by 2% than standard compaction method.

Generally, the difference between the OMC and MDD values of the modified and standard methods are; the OMC values of standard compaction are higher than the modified compaction started from natural up to stabilized black cotton soils and inversely with OMC values, the MDD values of natural up to stabilized black cotton soil are higher on the modified compaction than the standard compaction. This difference is due to the applied compaction load of 2.5Kg and 4.5Kg for standard and modified compaction respectively. Figure 4.8 illustrates that the optimum percentage of flexi stabilized soil samples in both standard and modified compaction methods.

Figure4. 8 Optimum flexi stabilized soil sample of standard and modified compaction

When the MDD of the flexi stabilized black cotton soil increased, the OMC decreased and vice versa for both compaction methods. This means that the stabilized black cotton soils lose their air voids during the time of compaction. Consequently; the solid particles of the black cotton soil become close to each other means getting of denser state of soil. The sand stabilizer results in different percentages for modified compaction method are presented in table 4.6.

Table4. 6 The results of sand stabilized soil in modified compaction method

% of sand	OMC (%)	MDD(g/cm^3)
0	26.11	1.368
1	24.23	1.39
2	23.15	1.42
3	22	1.44
7	21.23	1.47

From the above test results of black cotton soil that stabilized by flexi and sand in a mechanical stabilization method ; the OMC of sand stabilized soil is increased than the flexi stabilized soil up to 1% stabilizer content and decreased started from 2% up to 7% stabilizer content while the MDD of soil stabilized with flexi is increase up to 1% more than soil stabilized by sand and for the rest of the stabilizer percentages that started from 2% up to 7% the soil stabilized with sand is increased relatively than the soil stabilized with flexi. Therefore, the flexi stabilizer has better performance by increasing the MDD in 0.03g/cm^3 value and decreasing the OMC in 2.74% value by using 1% optimum stabilizer content. In addition, by considering in the aspect of economical stabilizer and getting clean environment system with better recycling of wastes as a resource, the flexi material is the best stabilizer than sand. The flexi stabilizer brings the black cotton soil sample into the denser state. The flexi material can't reduce the moisture content of black cotton soil by sharing their moisture content but it replaces the black cotton soil in terms of percentages of the dry mass of soil. In addition if there is the application of compaction load, the stabilized soil become interlocked each other due to roughness characteristics of flexi materials.

4.4. California Bearing Ratio Results of Flexi Stabilized Soil

The California bearing ratio (CBR) is one of the basic sub-grade soil test that determines stability and measures strength of the black cotton soil. In this study, both type of compaction methods for the preparation of remolded soil sample, which is compacted by using the OMC of standard & modified compaction to attain MDD, were used. Additionally, the CBR test was conducted into two ways; un-soaked and soaked CBR test. The soaked CBR has been saturated into water for four days due to environmental and climatic conditions for both flexi and sand stabilizers. The un-soaked CBR is a process of taking load penetration reading of the remolded sample without saturated in water. The following table 4.7 presents the values of CBR and percent swell in different percentages of flexi stabilizer for standard CBR method.

Table4. 7 Standard CBR & percent swell values of flexi stabilized black cotton soil

% of flexi	CBR (%)	Swell (%)
0 (soaked)	2.14	4.3
1(soaked)	2.89	1.27
2(soaked)	3.23	1.22
3(soaked)	1.88	5.17
5(soaked)	1.44	8.78
7(soaked)	0.07	15.77
9(soaked)	0.04	17.71
2(Un-soaked)	12	-
3(Un-soaked)	9.31	-

The standard CBR values of black cotton soil that stabilized with flexi is increased up to 2% of the stabilizer and decreased started from 3% up to 9% that has a value of less CBR due to the addition of more flexi materials. The improved amount of CBR value at the optimum 2% of flexi content with respect to the natural black cotton soil for the standard method is 50.9%. This means that the flexi stabilizer at the optimum level enhance the CBR value by half times of the natural CBR value of black cotton soil. In addition, the percent swell value of 2% flexi stabilized soil is decreased by 71.6% than the percent swell value of natural black cotton soil while for stabilizer contents of 3% up to 9%, percent swell become increase highly. When the amount of flexi material is dominated by the black cotton soil, there would be high swell of mixture of soil & flexi after the soaked period that is presented in figure 4.9.

Figure4. 9 Percent swell for high % of flexi in standard method

Due to the plastic material's light weight, the samples of 7% & above flexi stabilized soils swell in water. Therefore, the optimum stabilizer content is 2% which has high standard CBR result means gives better stability & strength than the other stabilizer amount. For the case of un-soaked standard CBR value; the optimum stabilizer is 2% than 3% stabilizer without saturation in water. This improvement is due to the friction action between flexi and black cotton soil with the help of stable OMC to attain the needed MDD.

As indicated by different books, soils that have high clay content (particularly expansive clays) may expand in detrimental amounts if compacted to a high density at low moisture content and then exposed to water. The modified soaked CBR values of black cotton soil that stabilized with both flexi and sand materials were minimum values for different percentages of stabilizer content. The following figure 4.10 shows that percent swell is very high during four days of soaking period that means it has very less value of CBR.

Figure4. 10 Modified CBR soil sample for all stabilizer content during soaked period

Even if sometimes it is impossible to read the CBR penetration values because of high swelling properties of the soaked mixture of a lot of flexi materials with the natural soil and using of heavy loads on the black cotton soil minerals.

When the standard CBR is compared with the modified CBR values of black cotton soils with or without flexi stabilizer after four days soaking period, the standard method is suitable because of its light compaction load with limited moisture content. In general, the black cotton soil doesn't need heavy load compaction effort, it is better use the standard CBR method during soaked time. The percentage of swell is become high during the modified compaction of each percentage of flexi and sand stabilized black cotton soils.

According to ERA (2013) design manual, the Sub-grade Strength Classes of the 2% flexi stabilized black cotton soil and natural black cotton soil are S_2 and S_1 respectively. While in the un-soaked CBR of 2% flexi stabilizer, the Sub-grade Strength Class is S_5. This means that the stabilized soil in both soaked and un-soaked CBR methods have greater sub-grade strength than the untreated natural soil. Therefore; the flexi stabilizer meets the black cotton soil's minimum criteria as a sub-grade material as well as effective stabilizer in the aspects of economy and environmental safety.

4.5. Shear Strength Results of Flexi Stabilized Soil

In this study, unconsolidated un-drained shear strength test was used to determine the angle of internal friction (Φ) and cohesion value(C) of black cotton soil that stabilized with different percentages of flexi and sand materials. In addition, the un-drained shear strength (S_u) was determined by using the UCS test in terms of equation 4.2.

$$Su = \frac{qu}{2} \qquad 4.2$$

This test was conducted by using two compaction methods of standard & modified to satisfy the different values of OMC & MDD for the preparation of remolded samples. For remolded sample preparation, the soil sample & flexi material that passed through sieve No.4 was used. Finally, the remolded sample in the shear box was inserted and took the failure point. The soil sample that passed through sieve No.4 randomly mix with % of flexi and the shear failure point at 2% flexi stabilizer are illustrated in figure 4.11.

Figure4. 11 The mixture of soil sample with flexi and the shear failure point

The laboratory results for both standard and modified compactions of cohesion value (C), angle of internal friction (Φ) and un-drained shear strength (S_u) that obtained from shear and UCS tests respectively are listed in table 4.8.

Table4. 8 Results of C, Φ and S_u of flexi stabilized soil in both methods

% of flexi	Standard			Modified		
	C(KN/m^2)	Φ	S_u (Kpa)	C(KN/m^2)	Φ	S_u (Kpa)
0	0.06	0	123.268	0.05	0	686.550
1	0.07	0	129.221	0.06	0	1130.359
2	0.11	0	135.715	0.15	0	1158.95
3	0.08	0	99.881	0.1	0	1146.925
5	0.07	0	79.25	0.07	0	830.062
7	0.07	0	69.891	0.05	0	550.838
9	0.04	0	65.25	0.035	0	508.92

From the above results, both C and S_u values are increased up to 2% and start decreased from 3% up to 9% of flexi stabilizer for both standard and modified methods. But, the angle of internal friction has zero value throughout the different percentages of flexi stabilizer including the natural soil. Because, as per different researches idea, purely cohesive soils exhibit cohesion but the angle of shearing resistance is zero. Additionally, some of the researchers revealed that strength reduction due to swelling helps to reduce the cohesion rather than the friction angle values. In this case, the flexi material helps to reduce the swelling potential up to the optimum flexi content. That is why the value of friction angle is constant throughout the different percentages of flexi stabilizer up to 9% stabilizer content. The different values of C for both standard and modified methods are illustrated in figure 4.12.

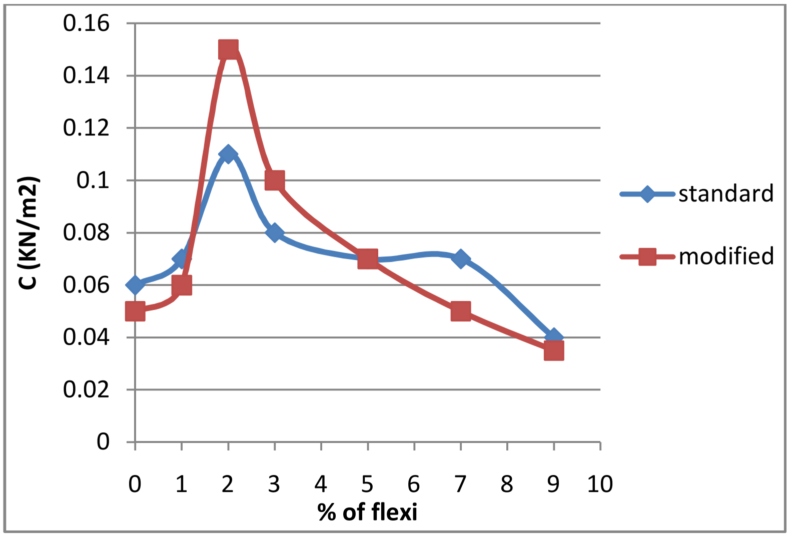


Figure4. 12 Variation of C as a function of flexi for both compaction methods

The values of cohesion at 2% effective flexi stabilizer are 83.3% and 200% greater than the cohesion value of the natural black cotton soil for standard and modified compaction methods respectively. The different S_u values of standard and modified compaction methods are illustrated in figure 4.13.

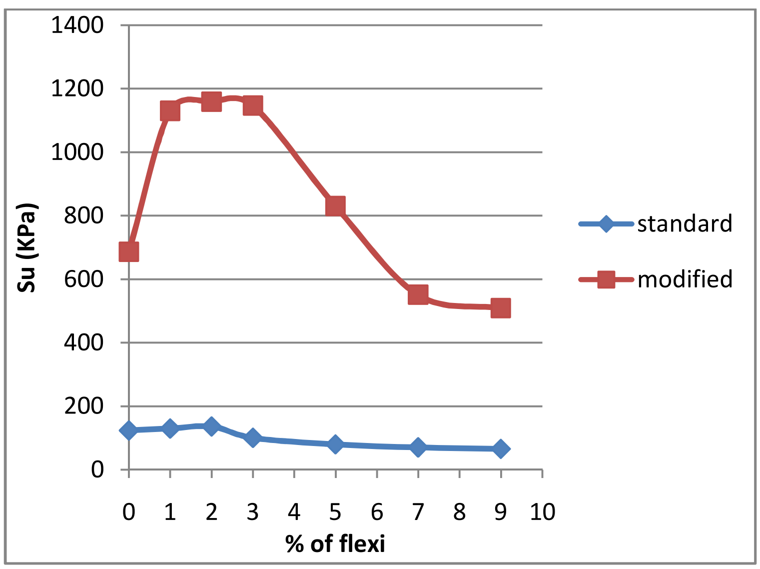


Figure4. 13 Variation of S_u as a function of flexi for both compaction methods

The difference values of S_u at 2% optimum amount of flexi stabilizer are 10.1% & 68.8% greater than the S_u values of natural black cotton soil for standard and modified compaction methods respectively. The S_u value of the modified method is five up to seven times of the S_u

value of the standard method while the C value in the standard method is greater up to 1% of flexi stabilizer, less value from 2% up to 5% than the modified method & after 5% stabilizer the results aren't constant. Consequently, by using the optimum stabilizer content of 2%, the modified values of S_u and C are better performance than the standard values. This is because of the MDD and OMC of the remolded sample of different percentages of flexi stabilizer. The results of C, Φ and S_u values of sand stabilized black cotton soil by using modified compaction method for the preparation of remolded sample are illustrated in table 4.9.

Table4. 9 Results of C, Φ and S_u in sand stabilized black cotton soil

% of sand	Modified		
	C(KN/m^2)	**(Φ)**	**S_u (Kpa)**
0	0.05	0	686.550
1	0.08	0	541.736
2	0.15	0	577.4
3	0.21	0	623.305
7	0.3	0	660.257

The value of C in flexi stabilized black cotton soil at 2% optimum stabilizer content is equal with C value of sand stabilized black cotton soil. The S_u value of flexi stabilized soil at 2% optimum stabilizer content is greater by 581.55Kpa than the Su value of soil stabilized with sand stabilizer. The internal angle of friction has the same value for both stabilizer types of black cotton soil. Therefore, the optimum stabilizer content of flexi is 2%. The flexi stabilizer gives the natural soil greater shear strength potential with a minimum amount of it than the sand stabilizer. The optimum content of flexi stabilizer improves C & S_u values, because of the replacement of some percentage of black cotton soil by flexi materials. Consequently, bond strength between the flexi material and black cotton soil is high due to the roughness surface of flexi material.

4.6. Unconfined Compressive Strength Results of Flexi Stabilized Soil

The unconfined compressive strength (UCS) of black cotton soil sample was prepared by using both standard and modified OMC to attain the MDD for different percentages of flexi & sand stabilized soil. This is done to see how the difference of compressive strength within the two different maximum densities. The unconfined compressive strength (q_u) results of soil in different percentages of flexi stabilizer for standard and modified compaction methods are presented in table 4.10.

Table4. 10 Results of q_u in flexi stabilized soil for both compaction methods

% of flexi	standard q_u(Kpa)	Modified q_u (Kpa)
0	246.536	1373.1
1	258.441	2260.72
2	271.43	2317.89
3	199.761	2293.85
5	158.5	1660.123
7	139.781	1101.675
9	130.5	1053.957

From the above test results, q_u value is increased up to 2% flexi stabilizer and become decreased starting from 3% up to 9% flexi stabilizer for both case of standard and modified methods. The shear failure (ε_f) values of different percentage of flexi stabilized soil are between 6.7% and 9.5% for standard UCS method whereas ε_f values of flexi stabilized soil for modified UCS method are between 3.8% and 9.3%. The difference q_u values of flexi stabilized black cotton soil for standard and modified compaction methods are illustrated in figure 4.14.

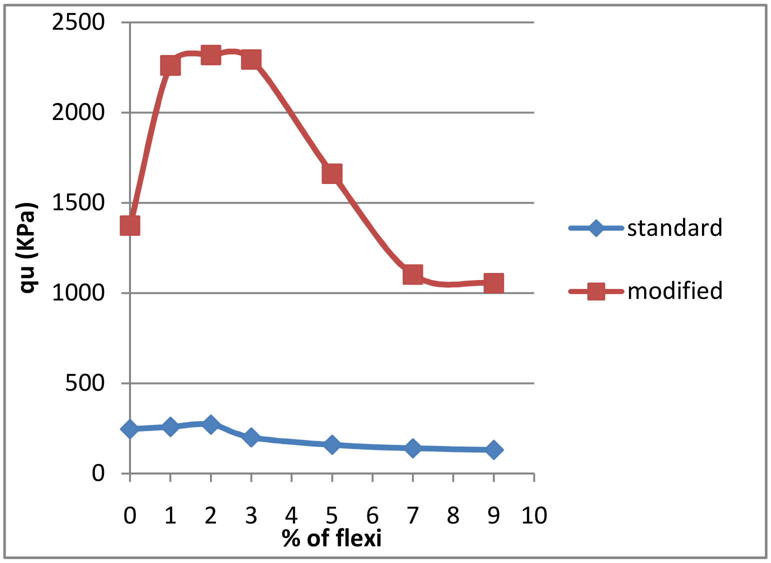


Figure4. 14 Variation of q_u as a function of flexi for both compaction methods

The q_u values of standard & modified UCS methods at optimum percentage of flexi stabilizer are 271.43Kpa & 2,317.89Kpa respectively. As a result, the values of q_u at 2% optimum flexi stabilizer are greater by 10.1% and 68.8% than the q_u values of natural black cotton soils for standard and modified UCS respectively. The above figure 4.17 shows that the difference of q_u values of the modified compaction method for the preparation of the remolded samples are eight times larger than standard compaction method. Therefore; due to heavy load application of the remolded black cotton soil that can be stabilized by different percentage of flexi stabilizer, the modified q_u has greater strength with a limited failure strain values. The following figure 4.15 shows that the remolded soil sample preparation and inclined failure line of the unconfined compressive strength after the application of compressive load.

Figure4. 15 UCS remolded sample and failure of 2% flexi stabilized black cotton soil

The q_u is also done by using sand stabilizer for modified UCS method in mechanical stabilization method. The results of modified q_u by using sand stabilizer for black cotton soil are illustrated below in table 4.11.

Table4. 11 Results of modified q_u for sand stabilized soil

% of sand	**modified q_u (Kpa)**
0	1373.1
1	1083.472
2	1154.8
3	1246.61
7	1320.515

The q_u of the black cotton soil that stabilized by sand stabilizer is reduced up to 1% & increased slightly up to 7% but not reach still in the q_u value of natural black cotton soil . The stabilized q_u values of the sand stabilizer are totally below the q_u value of natural black cotton soil. To compare the sand and flexi stabilizer for q_u values; the black cotton soil stabilized with flexi has high UCS value by 1,163.1Kpa more than UCS value of black cotton soil stabilized by sand. Therefore, the flexi stabilizer gives strength for the problematic black cotton soil. Because of the replacement of flexi materials in the black cotton soil in terms of percentages of dry soil mass and application of appropriate compaction energy by using OMC gives more strength to resist the coming load. In addition, the effective contact area of the interface and flexi surface roughness gives more strength for resisting the compression load.

4.7. Correlation and Regression Analysis

Correlation is one of statistical analysis tool which used to quantify the association between two continuous variables; between an independent and a dependent variable or between two independent variables. The two variables are relating each other if the change in one of the variables results in a change in the other variable. To identify the relationship of the two variables, there is a coefficient which is called correlation coefficient, r. The coefficient, r indicating both whether the relationship between x and y is positive or negative. When r is positive, there is a trend that one variable goes up as the other one goes up. When r is negative, there is a trend that one variable goes up as the other one goes down. The coefficient also tells there is no relation between the two specific variables. Generally, Correlation analysis clearly identifies the strength of the relationship of two variables (Douglas and George, 2003).

Regression analysis is a related technique to assess the relationship between outcome or dependent variable and one or more risk factors or independent variables. The term “independent” can be misleading if it is interpreted as the ability to predict even beyond the limits of the data. Also, the term “independent variable" might give an impression of a causal effect in a situation in which inferences should be limited to identifying associations. The terms "independent" and "dependent" variables are less subject to these interpretations as they do not strongly imply cause and effect. Regression model containing one independent variable is termed as simple regression model. While, a regression model that contains more than one independent variable is called multiple regression model. The regression equation depends upon which variable we choose as the explanatory variable, and which as the variable we wish to predict (Kothari, 2004).

In correlation analysis; the emphasis is on the degree to which a linear model may describe the relationship between two variables while, in regression the emphasis is on predicting one variable from the other. In carrying out the statistical analysis of the correlation & regression, one of the statistical software program such as SPSS, NCSS or MS-excel spreadsheet are possible to use for getting Scatter and linear regression plots to identify the best fit curve and analyzing the regression equations. In this study work, the Number Crunching Statistical

System (NCSS) statistical software program was used for analyzing of different laboratory test results. NCSS is easy for analyzing simple linear regression as well as correlation analysis.

4.8. Correlation Analysis

In correlation analysis; a sample correlation coefficient is estimated more specifically the Pearson Product Moment correlation coefficient is determined. The sample correlation coefficient, r, which ranges between -1 and +1, quantifies the direction and strength of the linear association between the two variables. The correlation between two variables can be positive that is higher levels of one variable are associated with higher levels of the other or negative that is higher levels of one variable are associated with lower levels of the other and it can also be zero means there is no relation between the two variables. The sign of the correlation coefficient indicates the direction of the association and the magnitude of the correlation coefficient indicates the strength of the association (Douglas and George, 2003).

4.8.1. Coefficient Determination

There are several ways for the determination of correlation coefficient especially Pearson Product Moment correlation coefficient; one of these is NCSS software. This software is used to determine the correlation coefficient for all dependent and independent variables of the laboratory test results that are listed in chapter four. The value of r gives a measure of how close the points are to lying on a straight line. The values of the correlation coefficients between two independent (% of flexi & PI) and different dependent variables of standard OMC, MDD, CBR, C, S_u and q_u are presented as follow. The first correlation are between % of flexi as an independent variable and various dependent variables like PI in Cassa Grande cup method, OMC, MDD, CBR, C, S_u and q_u as a dependent variables are presented in table 4.12.

Table4. 12 The correlation coefficient between % of flexi & laboratory test results

independent v_s dependent variables	**r**
% of flexi v_s PI (%)	-0.4193
% of flexi v_s OMC (%)	0.4090
% of flexi v_s MDD (g/cm^3)	-0.7805
% of flexi v_s CBR (%)	-0.8908
% of flexi v_s C (KN/m^2)	-0.4758
% of flexi v_s S_u (Kpa)	-0.9226
% of flexi v_s q_u (Kpa)	-0.9226

The second correlation type is between the index property test of the black cotton soil that is PI value and compaction & strength laboratory test results. This type of correlation express how the simple but important test results of black cotton soil relates with the other best sub-grade strength determination tests. The correlation coefficient values of PI (%) and the dependent variables are presented in table 4.13.

Table4. 13 The correlation coefficient between PI & other laboratory test results

independent v_s dependent variables	**r**
PI (%) v_s OMC (%)	-0.1955
PI (%) v_s MDD (g/cm^3)	-0.0926
PI (%) v_s CBR (%)	0.3784
PI (%) v_s C (KN/m^2)	-0.1159
PI (%) v_s S_u (Kpa)	0.5910
PI (%) v_s q_u (Kpa)	0.5910

4.9. Discussion on Correlation Analysis

The above table 5.1 correlation coefficient results revealed that degree of correlation between % of flexi and maximum dry density (MDD), California bearing ratio (CBR), un-drained shear strength (S_u) and unconfined compressive strength (q_u) values have good negative relationships means they have strong opposite side relationships. The % of flexi with plastic index (PI) and optimum moisture content (OMC) values have weak negative & positive relationships respectively while the % of flexi with cohesion value has moderate negative relationships. But correlations of 'PI' value with 'OMC', 'MDD', 'C' and 'CBR' have poor negative and positive relationships respectively. For the correlation coefficient of 'PI' with 'q_u' and 'S_u' have moderate positive degree of relationships.

4.10. Regression Analysis

In the case of linear regression, the systematic effect refers to the linear relationship between X and Y. A measure of effect size should get at how important or how strong the given relationship is. The fact of strength of relationship should be a hint that effect size will have something to do with correlation coefficient, r. The strength of the relationship between x and y is reflected in the extent to which knowing x reduces your uncertainty about y. Now suppose that the value of x is told and again asked you to predict y. This would be somewhat easier, because you could use regression to predict a good guess for y by the given x. Recall that regression attempts to fit the best line through the average y for each x (Kothari, 2004).

An important aspect of regression analysis is to estimate the parameters β_0 and β_1 that is estimate the so-called regression coefficients. Suppose the estimates of b_0 for β_0 and b_1 for β_1 is denoted. Then the estimated or fitted regression line is presented by using equation 5.1.

$$\tilde{y} = bo + b1x \qquad 5.1$$

Where; $\tilde{y}$ is the predicted or fitted value

Obviously, the fitted line is an estimate of the true regression line. It is important to introduce the concept of a residual. A residual is essentially an error in the fit of the above model that is presented in equation 5.1. It is calculated for a given set of regression data $\{(x_i, y_i); i = 1, 2, ..., n\}$ and a fitted model of the above equation 5.1.

Then, the i^{th} residual e_i is calculated by using equation 5.2.

$ei = yi - \tilde{y}i$ 5.2

When a set of n residuals is large, then the fit of the model is not good while Small residual is a sign of a good fit (Walpole, et al., 2012).

4.10.1. Linear Regression Plot Section

Before carrying out the regression analysis of different types of laboratory test results, a linear regression diagram have been generated by NCSS. The software helps to study how the cause of the independent variables expresses the effect in the dependent variables and determine the model that best suits the test results relationship. In this study, there are two types of regression analysis model namely; % of flexi and PI values as an independent variables with the dependent variables of the other laboratory test results which are optimum moisture content (OMC), maximum dry density (MDD), California bearing ratio (CBR), un-drained cohesion value(C), shear strength (S_u) and unconfined compressive strength (q_u). The linear regression plots of % of flexi as an independent variable & the above all mentioned laboratory test results include the plastic index (PI) value as dependent variables are listed below in the following figures between 4.16 and 4.22.

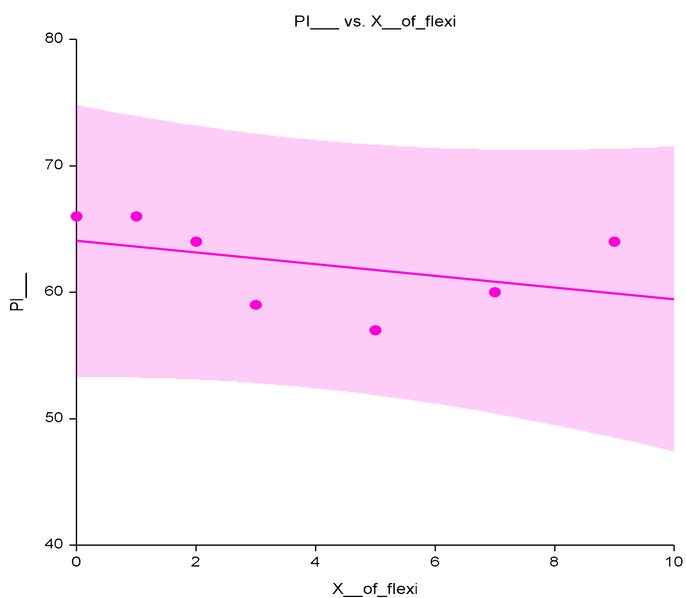


Figure4. 16 Linear regression plot of PI (%) v_s % of flexi

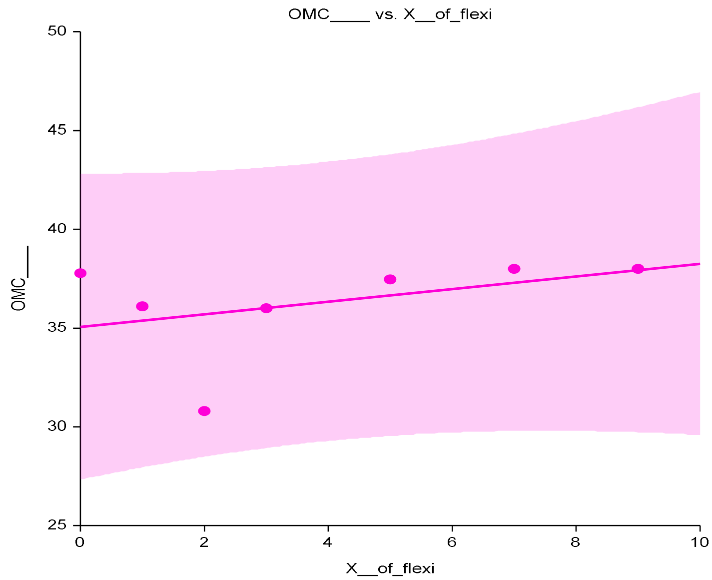


Figure4. 17 Linear regression plot of OMC (%) v_s % of flexi

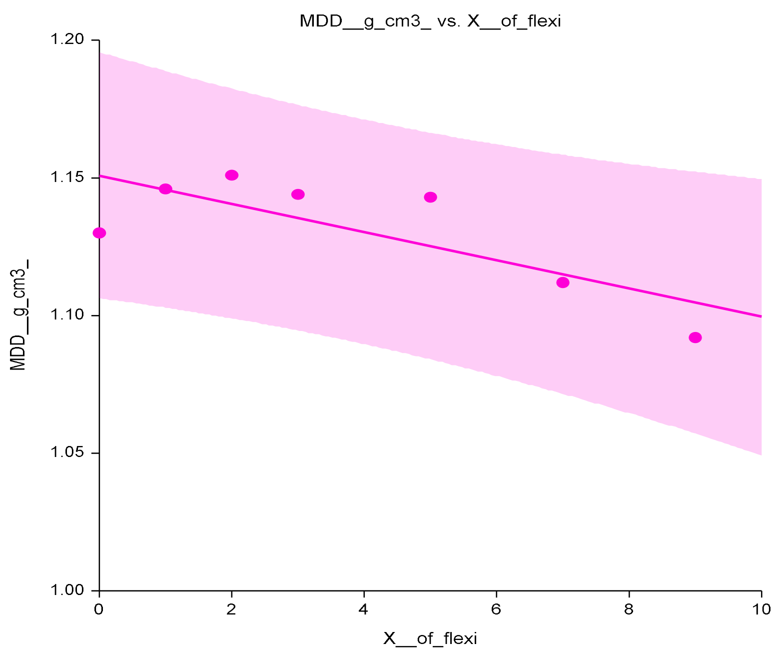


Figure4. 18 Linear regression plot of MDD (g/cm^3) v_s % of flexi

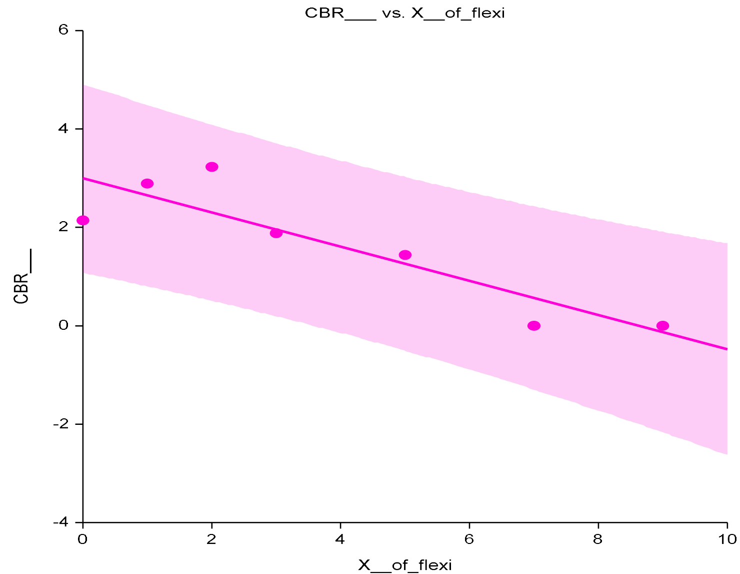


Figure4. 19 Linear regression plot of CBR (%) v_s % of flexi

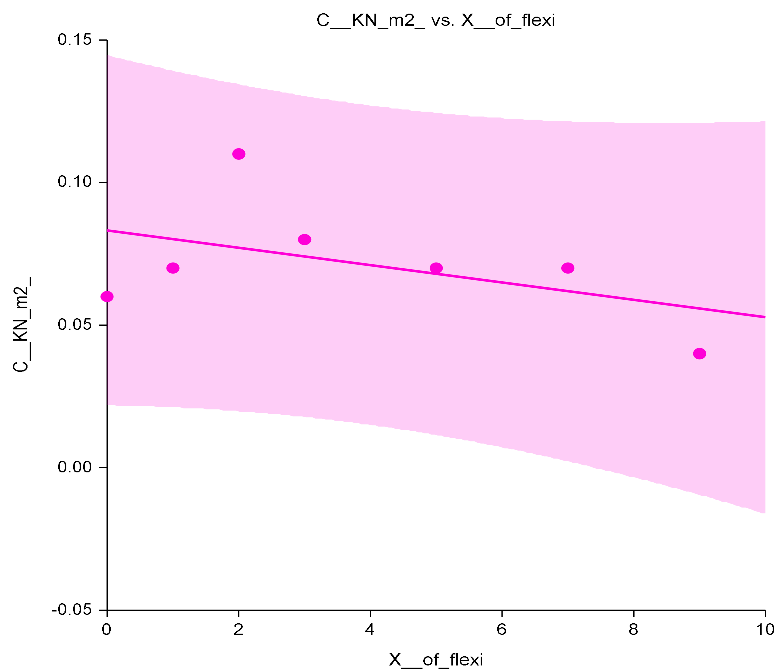


Figure4. 20 Linear regression plot of C (KN/m^2) v_s % of flexi

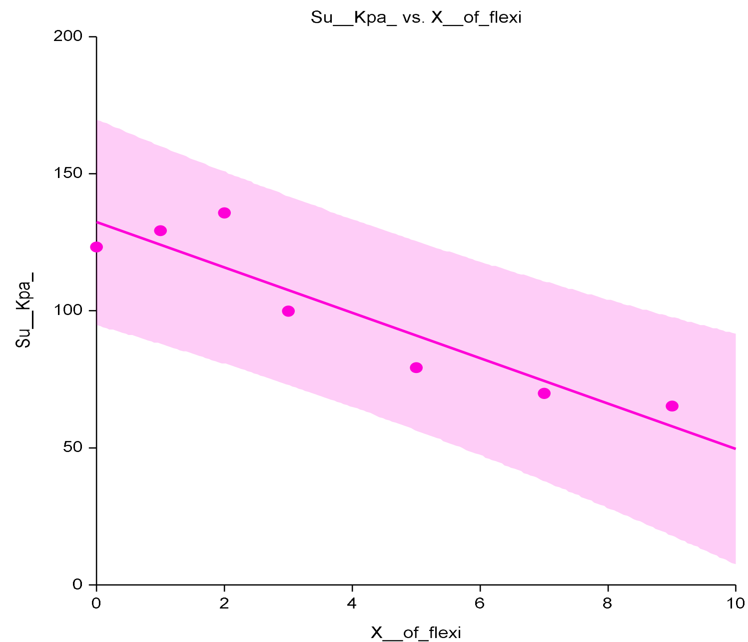


Figure4. 21 Linear regression plot of S_u (Kpa) v_s % of flexi

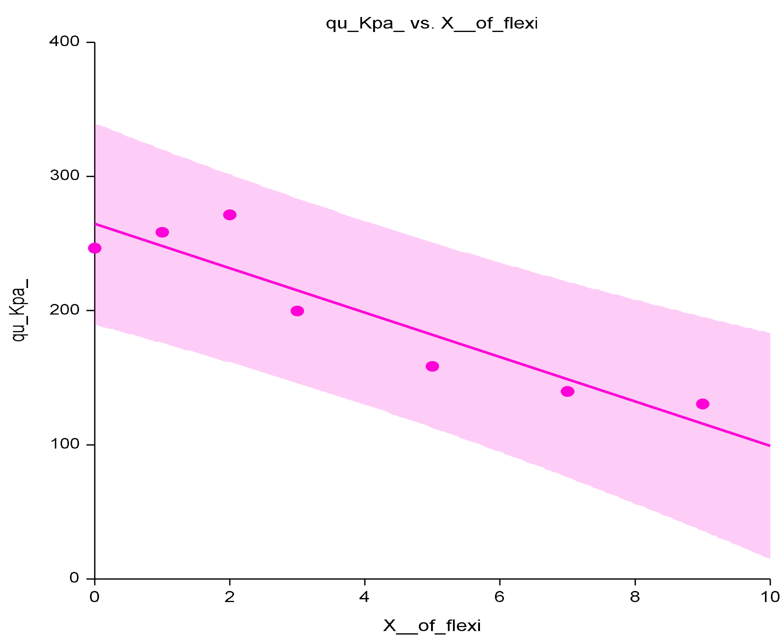


Figure4. 22 Linear regression plot of q_u (Kpa) v_s % of flexi

The other linear regression plots were done by using index property of the stabilized black cotton soil means the PI (%) values, which are gained from Cassa Grande cup method, as an independent variable and the compaction & strength laboratory test results which are listed in chapter four as dependent variables. The linear regression plots are presented in the figures between 4.23 and 4.28.

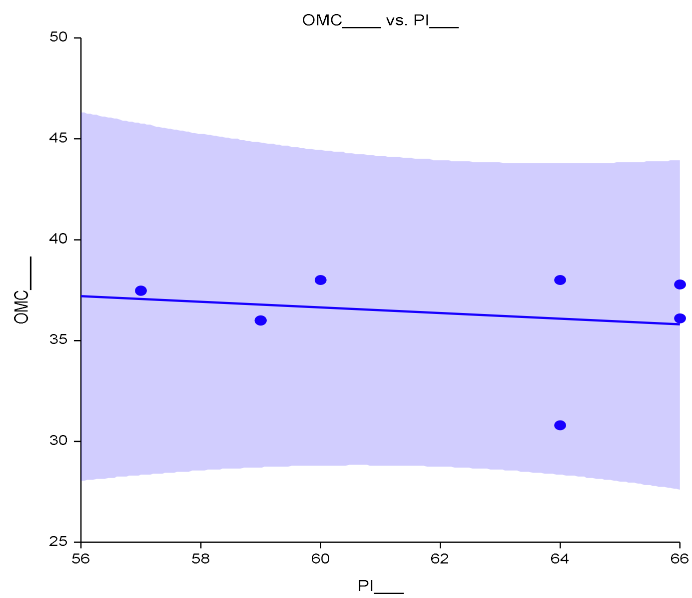


Figure4. 23 Linear regression plots of OMC (%) v_s PI (%)

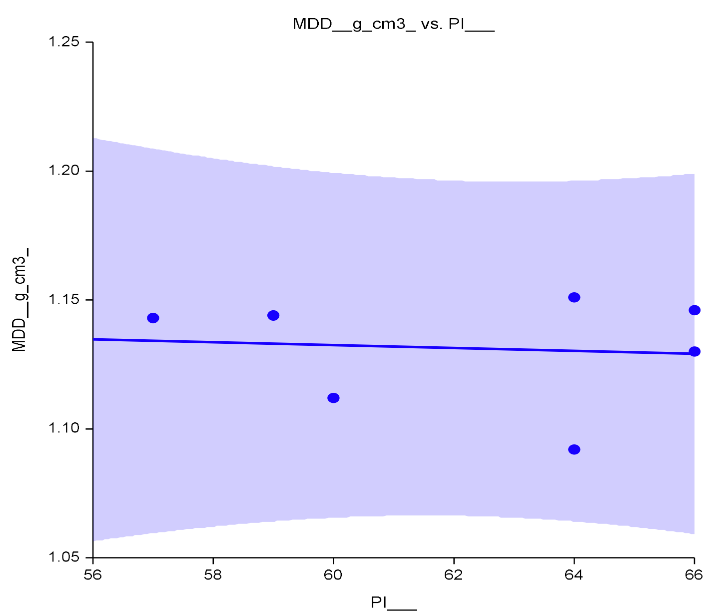


Figure4. 24 Linear regression plots of MDD (g/cm^3) v_s PI (%)

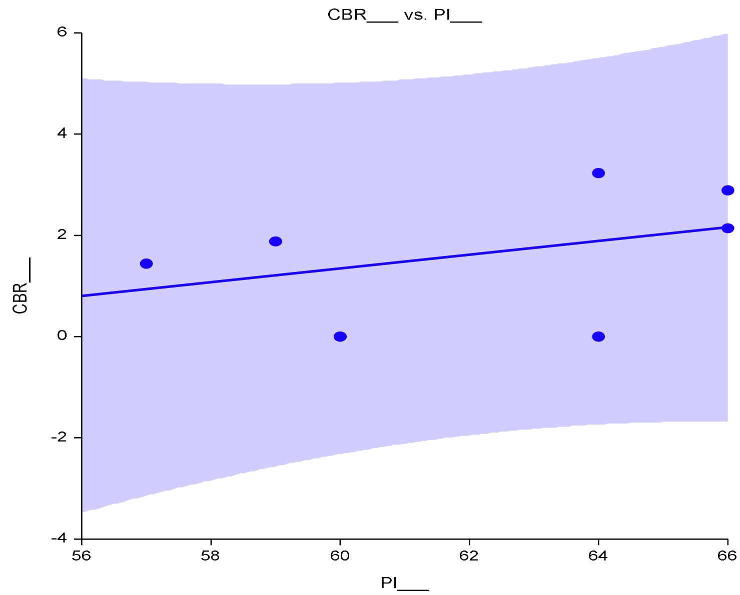


Figure4. 25 Linear regression plot of CBR (%) v_s PI (%)

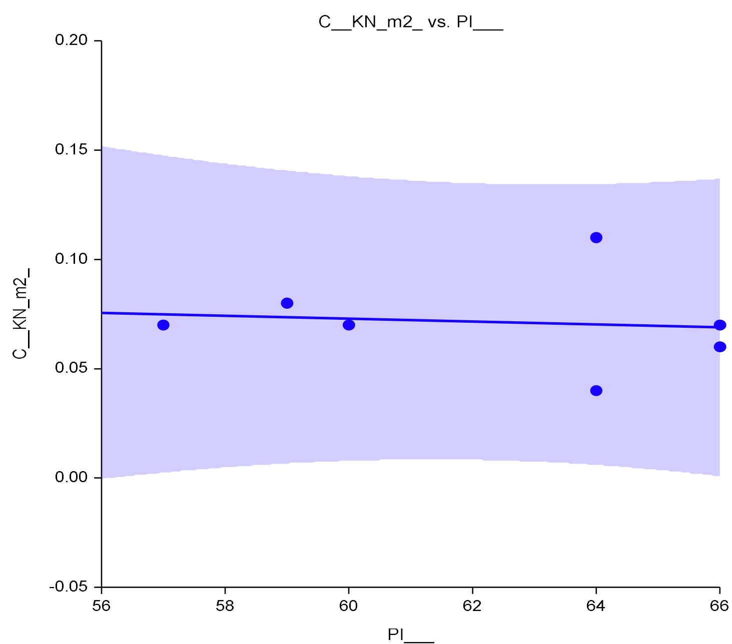


Figure4. 26 Linear regression plot of C (KN/m^2) v_s PI (%)

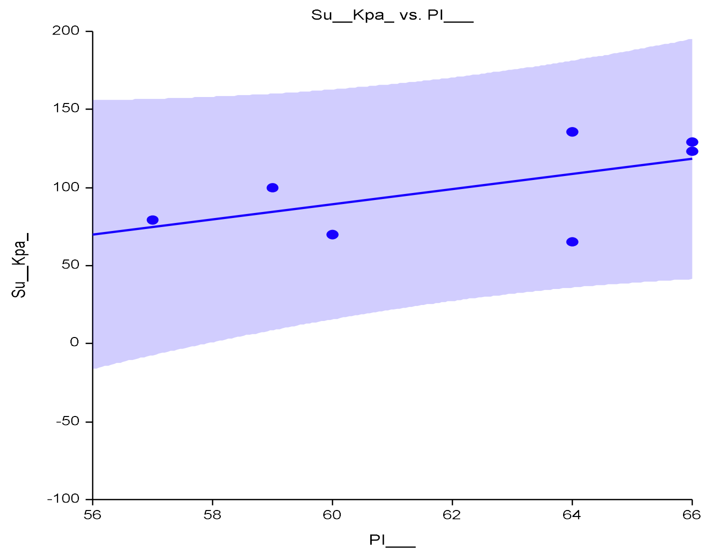


Figure4. 27 Linear regression plot of S_u (Kpa) v_s PI (%)

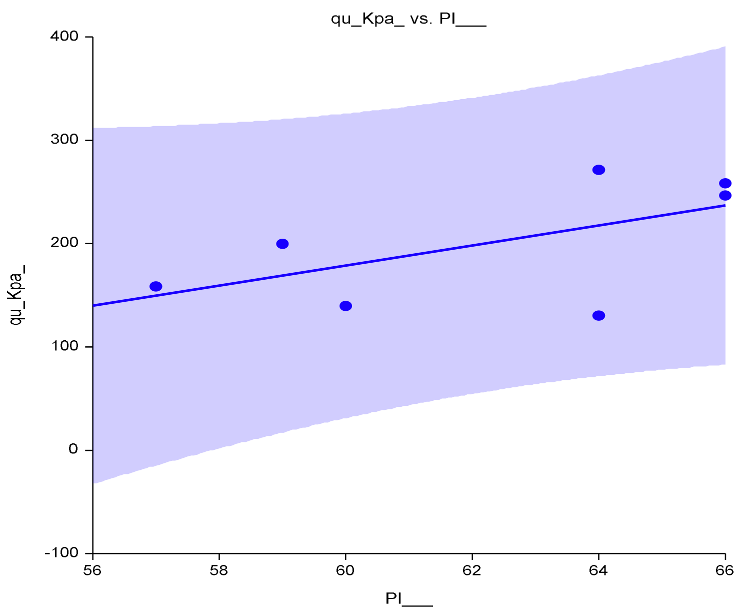


Figure4. 28 Linear regression plot of q_u (Kpa) v_s PI (%)

4.10.2. Determination Coefficient and Regression Equations

The squared correlation coefficient, R^2, is a measure of the size of the effect described by the linear relationship. R^2 is the proportion of variance in y that can be accounted for knowing x. Conversely, since r is symmetric with respect to x and y, R^2 is the proportion of variance in x that can be accounted for by knowing y. But this isn't quite true. If you keep the spread about the regression line the same but increase the slope of the line, R^2 increases. The correlation coefficient for zero slopes will be zero regardless of the amount of scatter about the line. Generally, R^2 is % of variance in Y accounted for by the regression of Y on X. If the fit is perfect, all residuals are zero and thus R^2= 1(100%) and vice versa. Additionally, R^2 become a good predictor if its value is between 0.6 and 0.9 values (Douglas and George, 2003).The regression equation and R^2 that get from the NCSS software for independent variables of % of flexi & PI (%) values and the different dependent variables of the laboratory test results are presented in tables 4.14 and 4.15 respectively.

Table4. 14 Regression equation and R^2 values for independent variable of % of flexi

Parameters	Regression equation	R^2
PI from % of flexi	PI (%) = (64.0529) + (-0.4581) % of flexi	0.1758
OMC from % of flexi	OMC (%)= (35.0727) + (0.3200) % of flexi	0.1673
MDD from % of flexi	MDD (g/cm^3) = (1.1510) + (-0.0051) % of flexi	0.6092
CBR from % of flexi	CBR (%) = (2.9909) + (-0.3465) % of flexi	0.7935
C from % of flexi	C (KN/m^2) = (0.0832) + (-0.0031) % of flexi	0.2264
S_u from % of flexi	S_u (Kpa) = (132.2612) + (-8.2723) % of flexi	0.8511
q_u from % of flexi	q_u(Kpa) = (264.5220) + (-16.5446) % of flexi	0.8511

Table4. 15 Regression equation and R^2values for independent variable of PI value

Parameters	Regression equations	R^2
OMC from PI	OMC (%) = (45.0306) + (-0.1401) PI (%)	0.0382
MDD from PI	MDD (g/cm^3) = (1.1660) + (-0.0006) PI (%)	0.0086
CBR from PI	CBR (%) = (-6.7370) + (0.1347) PI (%)	0.1432
C from PI	C (KN/m^2) = (0.1139) + (-0.0007) PI (%)	0.0134
S_u from PI	S_u (Kpa) = (-201.7119) + (4.8497) PI (%)	0.3492
q_u from PI	q_u (Kpa) =(-403.4257) + (9.6994) PI(%)	0.3492

4.11. Discussion on Regression Analysis

By carefully studying the data on the linear regression plot which illustrate the difference of the actual & predicted values of dependent variables, the regression analysis revealed the following discussions for % of flexi and PI values as independent variables. When the % of flexi has been an independent variable, the dependent variables of PI, OMC and C values are influenced by % of flexi by achieving coefficient of determination of 17.58%, 16.73% and 22.64% respectively. From the actual and predicted values of PI, OMC and C, the regression lines were fitted poorly. In addition, the MDD, CBR, S_u and q_u values with % of flexi linear relationships account for 60.92%, 79.35%, 85.11% and 85.11% of the variation in the data respectively. Therefore; depending on the above values of R^2 coefficient, the regression lines were moderately fitted for MDD and approximately perfect fit for CBR, S_u and q_u laboratory test results of dependent variables. In other case; when PI value is an independent variable, the regression analysis revealed that OMC, MDD, CBR, C, S_u and q_u values are influenced by PI by achieving coefficient of determination of 3.82%, 0.86%, 14.32%, 1.34%, 34.92% and 34.92% respectively. This means that all the above dependent variables with the independent variable of PI were poorly fitted with the original test results.

4.12. Validation of the Developed Equations

One of the specific objectives of this study is to develop a simple model for the prediction of the strength and stability laboratory test results of flexi stabilized black cotton soil without getting into tedious work by taking a number of days and by using costly laboratory tests. Depending on the above equations; the actual and predicted values of OMC, MDD, CBR, C, S_u and q_u with a limited seven sample sizes were not equal or approximate by using PI as an independent variable. This indicates that there was weak prediction of all laboratory test values even if some of the tests have a moderate relation. In the case of % of flexi as an independent variable, the actual and predicted values of PI, OMC, C and MDD are not equal or approximate which have weak and moderate prediction as a dependent variables respectively. But for actual and predicted values of CBR, S_u and q_u dependent variables based on the independent variable of % of flexi were approximate to each other.

Therefore; among the developed equations, the following equations are selected for checking their validation depending on their R^2 values. That is equations with high value of R^2 which have been selected for % of flexi as a dependent variable.

- CBR (%) = (2.9909) + (-0.3465) % of flexi……………………………R^2=0.79
- S_u (Kpa) = (132.2612) + (-8.2723) % of flexi…………………………..R^2=0.85
- q_u(Kpa) = (264.5220) + (-16.5446) % of flexi………………………….R^2=0.85

By substituting the values of % of flexi in the above equations, $CBR_{predicted}$, $S_{u\ predicted}$ and $q_{u\ predicted}$ were determined and presented in tables 4.16, 4.17 and 4.18 respectively.

Table4. 16 Summary of actual values, predicted values and their variations of CBR results

% of flexi (1)	**CBR_{actual} (%)** (2)	**$CBR_{predicted}$ (%)** (3)	**Variation** $/(2)-(3)/$
0	2.14	2.99	0.85
1	2.89	2.64	0.25
2	3.23	2.3	0.93
3	1.88	1.95	0.07
5	1.44	1.26	0.18
7	0.07	0.57	0.5
9	0.04	-0.13	0.17

Table4. 17 Summary of actual values, predicted values and their variations of S_u results

% of flexi (1)	**$S_{u\ actual}$ (Kpa)** (2)	**$S_{u\ predicted}$ (Kpa)** (3)	**Variation** $/(2)-(3)/$
0	123.268	132.261	8.99
1	129.221	124	5.22
2	135.715	115.7	20.02
3	99.881	107.4	7.52
5	79.25	90.9	11.65
7	69.891	74.4	4.51
9	65.25	57.8	7.45

Table4. 18 Summary of actual values, predicted values and their variations of q_u results

% of flexi (1)	**$q_{u\ actual}$ (Kpa)** (2)	**$q_{u\ predicted}$ (Kpa)** (3)	**Variation** $/(2)-(3)/$
0	246.536	264.52	17.98
1	258.441	264	5.56
2	271.43	247.98	23.45
3	199.761	214.89	15.13
5	158.5	181.8	23.3
7	139.781	148.7	8.92
9	130.5	115.6	14.9

4.13. Summary

Construction of pavement on weak or soft soil (expansive soil) is highly risky because such soil is susceptible to swell, have poor shear strength and high compressibility. One of the causes of pavement distress is climatic environmental conditions. This may cause surface irregularities and structural weaknesses on the pavement. The increment of moisture content cause wetting of soil and consequently swelling (heave) appears. This is often localized movement that can distort the shoulders, causing settlement & failures. The surface of the road becomes so slippery when wet at that time the wheels of the vehicles lose traction and the vehicle starts slipping to the extent. In addition, Longitudinal cracking and rutting occurred from the construction of expansive soil as a sub-grade material.

The strength and plasticity of a black cotton soil are improved with the addition of both flexi and sand in a mechanical stabilization method. In the case of flexi stabilizer, it doesn't share the natural soil's moisture content but reduce the void space by replacing some amount of natural soil. The results of the data show that both the LL and PL also decreased linearly with an increase of sand content. This is because of the sand material shares the moisture content of black cotton soil and it has good workability with low plasticity behavior. In general, the laboratory test results of black cotton soil that stabilized with different percents of flexi and sand stabilizers depending on different aspects are presented as follow;

- ❖ Flexi stabilizer is the better one at the optimum stabilizer content by modifying the poor engineering properties of black cotton soil with a minimum amount of stabilizer
- ❖ The availability of flexi stabilizer is high
- ❖ The cost of flexi stabilizer is minimum and
- ❖ Flexi stabilizer is the one that reduce environmental pollution by recycling plastic wastes in to resources.

In order to assure the flexi and sand stabilizer improvement, the laboratory tests were done repeatedly to drop out uncertainty of different results.

In addition, the better compaction method from standard and modified for the preparation of remolded sample in strength laboratory tests is examined. Consequently; the standard method is well again for strength tests of flexi stabilized black cotton soils as a sub-grade material.

This is because of very high swelling characteristics of stabilized black cotton soil in heavy compaction load with low optimum moisture content than the light compaction load. The optimum flexi stabilized black cotton soil results of the above laboratory test are listed in table 4.19.

Table4. 19 Summary of laboratory results of optimum flexi stabilized black cotton soil

Optimum % of flexi	Laboratory tests	List of description	Results
5%	Atterberg limit (Cassa Grande cup method)	LL (%)	91
9%		PL (%)	32
5%		PI (%)	57
9%	Atterberg limit (Cone penetration method)	LL (%)	74
		PL (%)	32
		PI (%)	42
2%	Standard compaction	OMC (%)	30.8
		MDD(gm/cm^3)	1.151
1%	Modified compaction	OMC (%)	21.49
		MDD(gm/cm^3)	1.421
2%	standard soaked CBR	CBR (%)	3.23
	Standard un-soaked CBR	CBR (%)	12
2%	Modified Shear strength	S_u(Kpa)	1158.95
		C(KN/m2)	0.15
All		Φ	0
2%	standard Shear strength	S_u(Kpa)	135.715
		C(KN/m2)	0.11
All		Φ	0
2%	Standard UCS	q_u (Kpa)	271.43

2%	Modified UCS	q_u (Kpa)	2317.89

The laboratory results of sand stabilized black cotton soil of the optimum stabilizer content depending of this study are illustrated in table 4.20.

Table4. 20 The laboratory results of black cotton soil with the optimum sand percentages

Optimum % of sand	Laboratory tests	List of descriptions	Results
3%	Atterberg limit (Cassa Grande cup method)	LL (%)	92
		PL (%)	31
		PI (%)	61
7%	Modified compaction	OMC (%)	21.23
		MDD(gm/cm3)	1.47
0%	Modified Shear strength	S_u(Kpa)	686.55
7%		C(KN/m2)	0.3
All		Φ	0
0%	Modified UCS	q_u (Kpa)	1373.1

From the analysis of correlation and regression equation of the laboratory test results, the results of PI as an independent variable had not the chance to predict all the dependent variables. Their correlation except S_u and q_u were poor. In other case; most of the test results except CBR, S_u and q_u that has been predicted from the independent variable of % of flexi had not equal or approximate actual and predicted values. The correlation of CBR, S_u, q_u and C were also good and moderate respectively. By considering the above actual and predicted values of CBR, S_u and q_u, the exact values of CBR, S_u and q_u from the developed equation were not found but a good approximation were produced.

5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

The black cotton soil that stabilized by flexi material was improved their poor engineering properties. This is proved by using different laboratory test results of index property, compaction and strength tests. The PI of 5% flexi stabilized soil was decreased by 13.6% than the natural black cotton soil in Cassa Grande cup method. From the result of strength tests; the effective amount of stabilizer was 2% for laboratory tests such as CBR, shear strength and UCS by using the standard OMC to attain the appropriate MDD in the preparation of remolded soil samples. The values of OMC and MDD at optimum flexi stabilizer were smaller by 18.5% and larger by 1.9% than the natural black cotton soil respectively. The improved amount of CBR value at the optimum flexi with respect to the natural black cotton soil was 50.9% with less percent of swell. The values of C at effective flexi stabilizer were 83.3% greater than the C value of natural black cotton soil with non improved values of Φ. Both the values of q_u and S_u at optimum stabilizer were 10.1% greater than natural black cotton soil.

The modified compaction is not an appropriate method for black cotton soil, because of high swelling characteristics of soil mineral after applying heavy load with low moisture content & soaking in water respectively. In addition, the flexi stabilizer is advisable to use than sand stabilizer with a perspective of improving the poor engineering properties of black cotton soil slightly by using less amount of stabilizer, having less economic cost and environmental pollution of the country through plastic wastes will be minimized by recycling wastes into resource.

The correlation and regression model helped to analyze the above laboratory test results each other. In this case, the relationship of each compaction and strength laboratory results such as OMC, MDD, CBR, C, S_u and q_u tests with the index property test of PI were poor to medium level while the Regression analysis of these tests by getting the equations of the independent variable of PI and dependent variables of OMC, MDD, CBR, C, S_u and q_u had not good fit with the original data's. Whereas the estimated values of CBR, q_u and S_u by using percent of flexi as an independent variable were approximate similar with the original values of CBR, q_u and S_u by R^2 values of 0.79, 0.85 & 0.85 respectively.

5.2. Recommendation

Depending on various limitation of this study, the recommendation is given in terms of different perspective as follow.

- ❖ Try to use more fine than 2mm size of plastic waste for superior mix with the problematic black cotton soil.
- ❖ It is better to use the mixture of optimum percentage of flexi material in terms of dry soil weight and other chemical stabilizer material together to get better CBR value and less expansive potential of soil.
- ❖ In the correlation and regression model of the laboratory test results, it is appropriate to use large sample sizes for better fit of regression equation line with the original data's.

Recommendation for different government bodies, road construction authorities and plastic waste recycle companies as follow. They should aware that plastic waste is one type of stabilizer, which should be practiced for better sub-grade layer with less total economic cost of pavement construction as well as getting healthy environment condition by recycling wastes into resource.

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APPENDIX

Appendix A: Detail Laboratory Test Results of Flexi Stabilized Black Cotton Soil

Table A. 1 Sieve analysis for portions coarser than No. 10 (2 mm) sieve

sieve size (mm)	sieve size (ASTM)	Mass Retained (g)	Cumulative mass retained (g)	Mass passing (g)	Percentage passing (%)
75	3-in	0.0	0	10.90	100.00
50	2-in	0.0	0	10.90	100.00
37.5	1 1/2-in	0.0	0	10.90	100.00
25	1-in	0.0	0	10.90	100.00
19	3/4-in	0.0	0	10.90	100.00
9.5	3/8-in	0.0	0	10.90	100.00
4.75	No. 4	0.0	0	10.90	100.00
2	No. 10	0.6	0.6	10.30	94.50

Table A. 2 Sieve analysis for portions finer than No. 10 (2 mm) sieve

sieve size (mm)	sieve size (ASTM)	Mass Retained (g)	Cumulative mass retained (g)	Mass passing (g)	Percentage passing (%)
2.000	No. 10	2.9126	2.91262136	50.00	94.50
0.850	No. 20	2.2	5.11262136	47.80	90.34
0.425	No. 40	2.6	7.71262136	45.20	85.42
0.250	No. 60	0.8	8.51262136	44.40	83.91
0.150	No. 100	2.3	10.8126214	42.10	79.57
0.075	No. 200	0.1	10.9126214	42.00	79.38

Table A.3 Hydrometer analysis for portions finer than NO. 200 (75 ɥm) sieve

Time interval (min)	Hydro meter reading	Tempr ature (^0C)	Calibrati on cylinder reading	Compo site correcti on	Corrected Hydromet er reading	Percentage passing (%)	K factor	Particle size, D (mm)
2	1.03	19.6	1.004	-0.004	1.026	81.896	0.0144	0.02951
4	1.0295	19.6	1.004	-0.004	1.0255	80.3211	0.0144	0.02111
8	1.028	19.5	1.004	-0.004	1.024	75.5963	0.0144	0.01519
15	1.027	19.5	1.004	-0.004	1.023	72.4465	0.0144	0.01128
30	1.026	19.4	1.004	-0.004	1.022	69.2966	0.01449	0.00811
60	1.0245	19.5	1.004	-0.004	1.0205	64.5719	0.0144	0.00588
120	1.023	19.6	1.004	-0.004	1.019	59.8471	0.0144	0.0042
240	1.022	20	1.004	-0.004	1.018	56.6972	0.01431	0.00299
480	1.0208	20	1.004	-0.004	1.0168	52.9174	0.01431	0.00217
1440	1.019	20	1.004	-0.004	1.015	47.2477	0.01431	0.00127

Table A. 4 Summary of particle size distribution of a natural black cotton soil

Sieve size (Particle size) (mm)	Sieve size (Particle size) (ASTM)	Percentage Passing (%)	Sieve size (Particle size) (mm	Sieve size (Particle size) (ASTM)	Percentage Passing (%)
75	3-in	100.00	0.015	-	75.60
50	2-in	100.00	0.011	-	72.45
37.5	1 1/2-in	100.00	0.008	-	69.30
25	1-in	100.00	0.006	-	64.57
19	3/4-in	100.00	0.004	-	59.85
9.5	3/8-in	100.00	0.003	-	56.70
4.75	No. 4	100.00	0.002	-	52.92
2	No. 10	94.50	0.001	-	47.25
0.850	No. 20	90.34			
0.425	No. 40	85.42			
0.250	No. 60	83.91			
0.150	No. 100	79.57			
0.075	No. 200	79.38			
0.030	-	81.90			
0.021	-	80.32			

Table A. 5 LL, PL and PI Results of natural black cotton soil in Cassa Grande Cup method

Type of Test	Liquid Limit				Plastic Limit	
Number of Blows	35	26	22	15		
Container Number	120	156	171	87	89	68
Mass of Empty Container, M_c	33.3	33.0	33.4	33.2	33.2	33.9
Mass of Container + Wet Soil, M_{csw} (g)	44.7	46.5	45.0	46.6	40.4	41.0
Mass of Container + Dry Soil, M_{cs} (g)	39.1	39.7	39.1	39.6	38.5	39.1
Mass of Dry Soil, M_s (g)	5.80	6.70	5.70	6.40	5.30	5.20
Mass of Water, M_w (g)	5.60	6.80	5.90	7.00	1.90	1.90
Water Content, *w* (%)	96.55	101.49	103.51	109.38	35.85	36.54

Liquid Limit, LL or w_L (%)	102
Plastic Limit, PL or w_p (%)	36
Plasticity Index, PI (%)	66

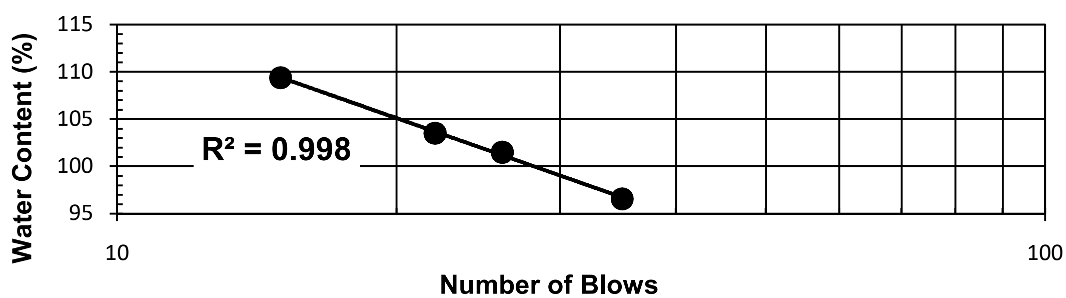


Figure A. 1 LL value at 25 number of blow for natural black cotton soil

Table A. 6 LL, PL and PI Results at 5% flexi stabilized soil in Cassa Grande Cup method

Type of Test	Liquid Limit				Plastic Limit	
Number of Blows	32	26	21	15		
Container Number	172	81	91	184	177	131
Mass of Empty Container, M_c	33.4	32.8	32.8	33.2	33.1	33.6
Mass of Container + Wet Soil, M_{csw} (g)	46.2	46.9	46.5	46.4	38.9	38.4
Mass of Container + Dry Soil, M_{cs} (g)	40.2	40.2	39.9	39.9	37.4	37.2
Mass of Dry Soil, M_s (g)	6.80	7.40	7.10	6.70	4.30	3.60
Mass of Water, M_w (g)	6.00	6.70	6.60	6.50	1.50	1.20
Water Content, *w* (%)	88.24	90.54	92.96	97.01	34.88	33.33

Liquid Limit, LL or w_L (%)	91
Plastic Limit, PL or w_p (%)	34
Plasticity Index, PI (%)	57

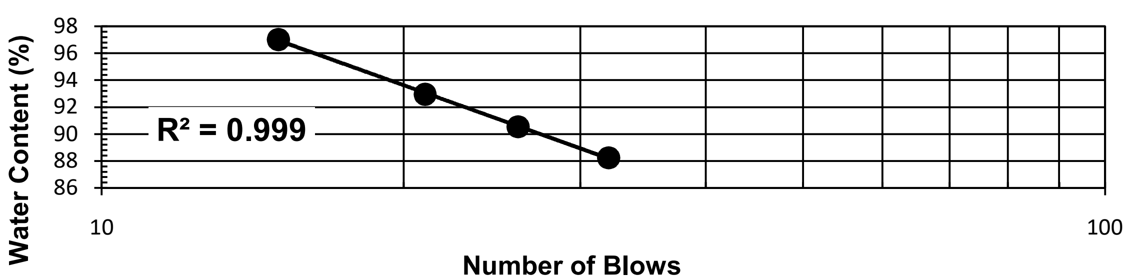


Figure A. 2 LL value at 25 number of blow for 5% flexi stabilized black cotton soil

Table A. 7 LL, PL and PI Results of natural black cotton soil in Cone penetration method

Type of Test	**Liquid Limit**				**Plastic Limit**	
Penetration(mm)	16.7	20.7	26.6	29.5		
Container Number	193	322	330	306	89	68
Mass of Empty Container, M_c	14.64	13.68	14.99	14.89	33.2	33.9
Mass of Container + Wet Soil, M_{csw} (g)	46.09	38.38	49.22	40.71	40.4	41.0
Mass of Container + Dry Soil, M_{cs} (g)	32.07	27.15	33.44	28.77	38.5	39.1
Mass of Dry Soil, M_s (g)	17.43	13.47	18.45	13.88	5.30	5.20
Mass of Water, M_w (g)	14.02	11.23	15.78	11.94	1.90	1.90
Water Content, *w* (%)	80.44	83.37	85.53	86.02	35.85	36.54

Liquid Limit, LL or w_L (%)	83
Plastic Limit, PL or w_p (%)	36
Plasticity Index, PI (%)	47

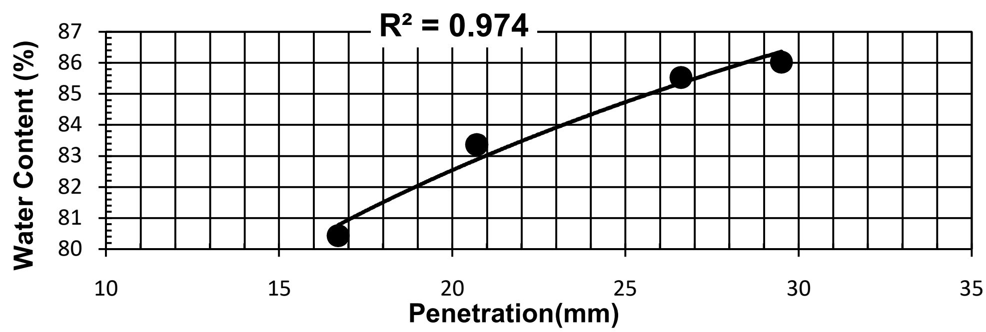


Figure A. 3 LL value at 20 mm penetration depth for natural black cotton soil

Table A. 8 LL, PL and PI Results at 9% flexi stabilized soil in Cone penetration method

Type of Test	Liquid Limit				Plastic Limit	
Penetration(mm)	17.8	19.2	21.0	24.8		
Container Number	338	393	63	65	88	120
Mass of Empty Container, M_c	14.88	14.96	14.76	14.85	33.5	33.0
Mass of Container + Wet Soil, M_{csw} (g)	29.75	32.65	34.58	35.95	41.0	40.6
Mass of Container + Dry Soil, M_{cs} (g)	23.58	25.25	26.03	26.55	39.2	38.7
Mass of Dry Soil, M_s (g)	8.70	10.29	11.27	11.70	5.70	5.70
Mass of Water, M_w (g)	6.17	7.40	8.55	9.40	1.80	1.85
Water Content, *w* (%)	70.92	71.91	75.87	80.34	31.58	32.46
Liquid Limit, LL or w_L (%)	74					
Plastic Limit, PL or w_p (%)	32					
Plasticity Index, PI (%)	42					

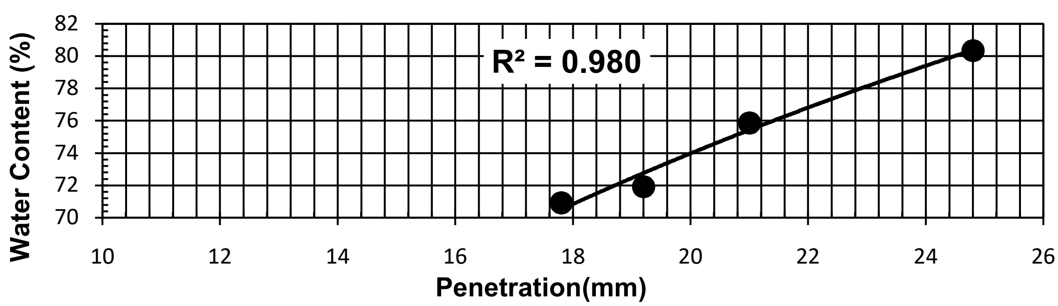


Figure A. 4 LL value at 20 mm penetration depth for 9% flexi stabilized black cotton soil

Table A. 9 LL, PL and PI Results at 3% sand stabilized soil in Cassa Grande cup method

Type of Test	Liquid Limit				Plastic Limit	
Number of Blows	34	27	22	17		
Container Number	98	128	172	164	135	79
Mass of Empty Container, M_c	33.4	33.2	33.5	33.8	33.3	33.1
Mass of Container + Wet Soil, M_{csw} (g)	45.9	49.1	47.4	49.0	42.2	42.3
Mass of Container + Dry Soil, M_{cs} (g)	40.0	41.5	40.7	41.6	40.1	40.1
Mass of Dry Soil, M_s (g)	6.60	8.30	7.20	7.80	6.80	7.00
Mass of Water, M_w (g)	5.90	7.60	6.70	7.40	2.10	2.20
Water Content, *w* (%)	89.39	91.57	93.06	94.87	30.88	31.43
Liquid Limit, LL or w_L (%)	92					
Plastic Limit, PL or w_p (%)	31					
Plasticity Index, PI (%)	61					

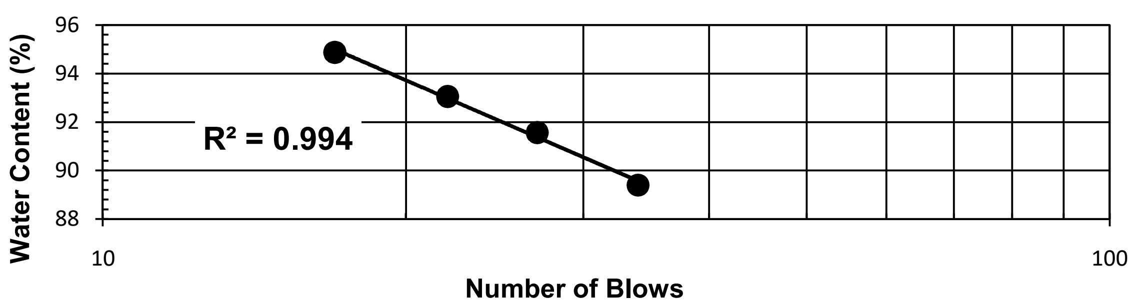


Figure A. 5 LL value at 25 number of blow for 3% sand stabilized black cotton soil

Table A. 10 OMC and MDD Results of natural black cotton soil in modified compaction

Molding Moisture Content Determination				
Container Number	172	171	89	129
Mass of Empty Container, M_c	33.0	33.5	33.3	33.6
Mass of Container + Wet Soil, M_{csw} (g)	199.0	145.4	129.8	128.0
Mass of Container + Dry Soil, M_{cs} (g)	181.5	126.5	108.7	102.2
Mass of Dry Soil, M_s (g)	148.5	93.0	75.4	68.6
Mass of Water, M_w (g)	17.5	18.9	21.1	25.8
Water Content, *w* (%)	11.8	20.3	28.0	37.6
Density Determination				
Weight of soil + Mould, M_{MS} (g)	6029.0	6194.2	6318.6	6261.4
Weight of soil, M_S (g)	1464.7	1629.9	1754.3	1697.1
Bulk Density, ρ (g/cm^3)	1.458	1.623	1.747	1.690
Dry Density, ρ_d (g/cm^3)	1.304	1.349	1.365	1.228

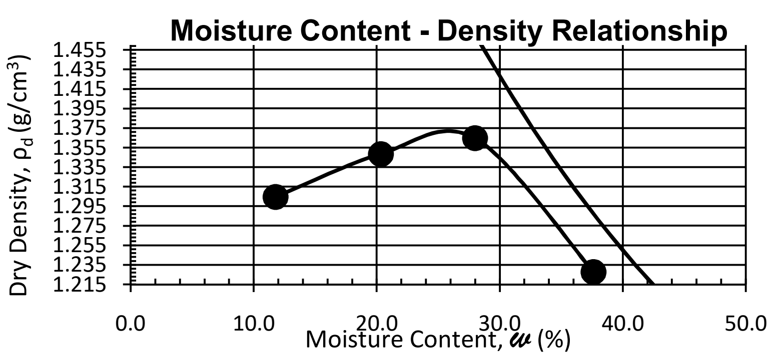


Figure A. 6 Modified compaction curve of OMC and MDD results of natural black cotton soil

Table A. 11 OMC and MDD Results of 1% flexi stabilized soil in modified compaction

Molding Moisture Content Determination				
Container Number	97	127	106	82
Mass of Empty Container, M_c	32.8	33.4	33.3	33.3
Mass of Container + Wet Soil, M_{csw} (g)	142.8	116.5	120.7	105.1
Mass of Container + Dry Soil, M_{cs} (g)	128.7	100.4	100.4	85.1
Mass of Dry Soil, M_s (g)	95.9	67.0	67.1	51.8
Mass of Water, M_w (g)	14.1	16.1	20.3	20.0
Water Content, *w* (%)	14.7	24.0	30.3	38.6
Density Determination				
Weight of soil + Mould, M_{MS} (g)	6106.5	6325.6	6305.3	6198.3
Weight of soil, M_S (g)	1540.6	1759.7	1739.4	1632.4
Bulk Density, ρ (g/cm^3)	1.534	1.752	1.732	1.625
Dry Density, ρ_d (g/cm^3)	1.337	1.412	1.329	1.172

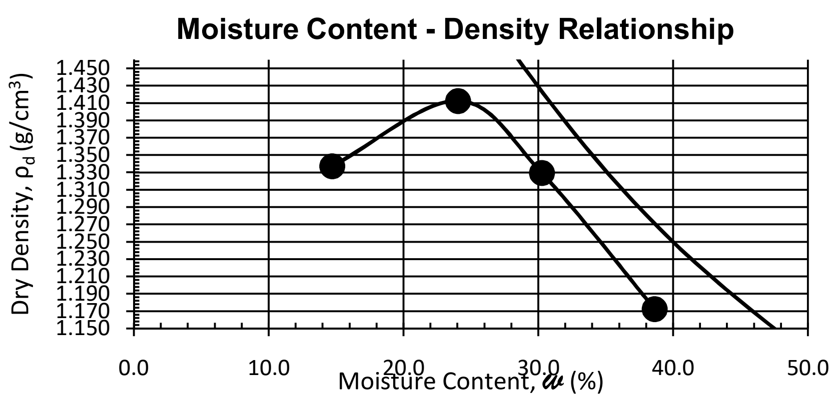


Figure A. 7 Modified compaction curve of OMC and MDD results of 1% flexi stabilized soil

Table A. 12 OMC and MDD Results of 7% sand stabilized soil in modified compaction

Molding Moisture Content Determination				
Container Number	171	120	97	150
Mass of Empty Container, M_c	33.5	33.4	32.9	33.0
Mass of Container + Wet Soil, M_{csw} (g)	141.0	116.0	115.6	121.0
Mass of Container + Dry Soil, M_{cs} (g)	126.7	100.6	96.2	96.8
Mass of Dry Soil, M_s (g)	93.2	67.2	63.3	63.8
Mass of Water, M_w (g)	14.3	15.4	19.4	24.2
Water Content, *w* (%)	15.3	22.9	30.6	37.9
Density Determination				
Weight of soil + Mould, M_{MS} (g)	6159.7	6376.7	6320.0	6236.6
Weight of soil, M_S (g)	1593.1	1810.1	1753.4	1670.0
Bulk Density, ρ (g/cm^3)	1.586	1.802	1.746	1.663
Dry Density, ρ_d (g/cm^3)	1.375	1.466	1.336	1.205

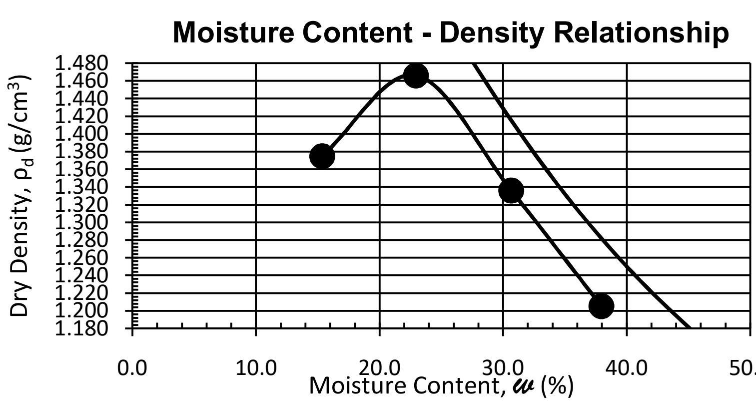


Figure A. 8 Modified compaction curve of OMC and MDD results of 7% sand stabilized soil

Table A. 13 CBR and % swell Results of natural black cotton soil in standard compaction

Blow/ Layer		56/3			
Swell, %		4.30	**Optimum Most. Content**		53.62
CBR Value, %		**2.14**	**Max. Dry Density**		1.06
Penet.	**Ring Reading**	**Load**	**Stress**	**Standard stress**	**CBR (%)**
(mm)	(Div.)	(N)	(N/mm^2)	(N/mm^2)	
0.00	0.0	0	0.00		
0.64	2.4	116	0.06		
1.27	4.3	208	0.11		
1.96	5.5	266	0.14		
2.54	5.9	286	0.15	6.9	2.14
3.18	6.0	290	0.15		
3.81	6.0	290	0.15		
4.45	6.1	295	0.15		
5.08	6.2	300	0.16	10.3	1.51
7.62	6.2	300	0.16		
10.16	6.3	305	0.16		
12.70	6.4	310	0.16		

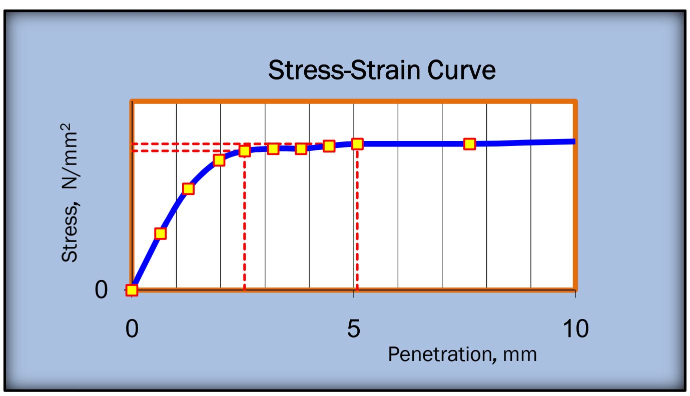


Figure A. 9 Stress v_s penetration curve of CBR result of natural black cotton soil

Table A. 14 CBR and % swell Results of 2% flexi stabilized soil in standard compaction

Blow/ Layer		56/3			
Swell, %		1.22	**Optimum Most. Content**		45.70
CBR Value, %		**3.23**	**Max. Dry Density**		1.14
Penet.	**Ring Reading**	**Load**	**Stress**	**Standard stress**	**CBR (%)**
(mm)	(Div.)	(N)	(N/mm^2)	(N/mm^2)	
0.00	0.0	0	0.00		
0.64	5.2	252	0.13		
1.27	7.1	344	0.18		
1.96	8.7	421	0.22		
2.54	8.9	431	0.22	6.9	3.23
3.18	9.0	436	0.23		
3.81	9.0	436	0.23		
4.45	9.0	436	0.23		
5.08	9.1	440	0.23	10.3	2.21
7.62	9.2	445	0.23		
10.16	9.4	455	0.24		
12.70	9.5	460	0.24		

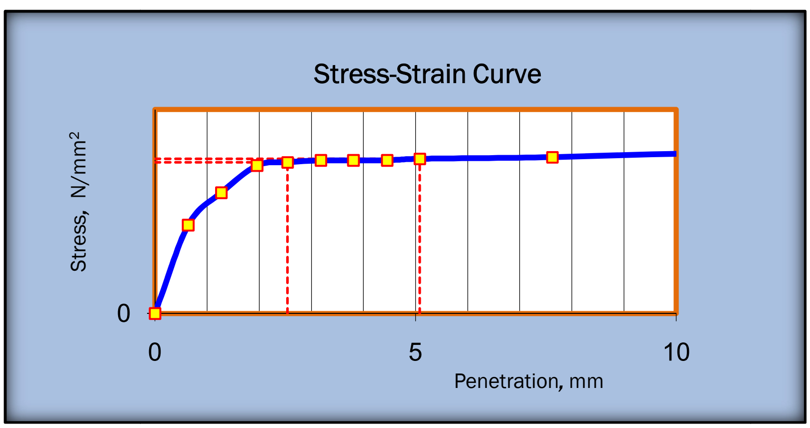
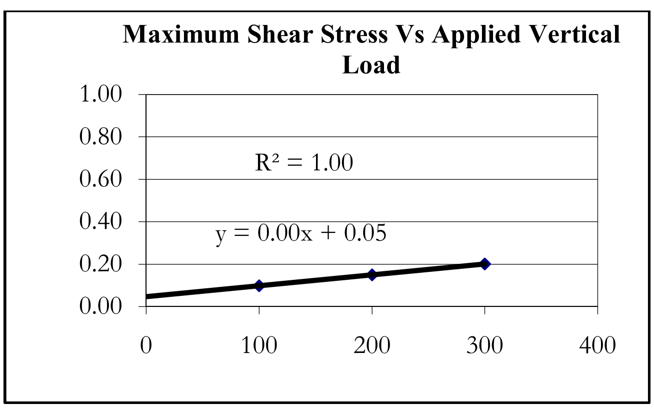


Figure A. 10 Stress v_s penetration curve of CBR result of 2% flexi stabilized soil

Table A. 15 Shear strength Results of natural black cotton soil in modified compaction

Ring Calib. Factor		0.002866 N/div	Applied Vertical Stress								
Rate of strain		0.5 mm/min	**100 kPa**			**200 kPa**			**500 kPa**		
Horizontal Displacement [div]	**Horizontal Displacement [mm]**	**Corrected Area [mm2]**	**Proving Ring Reading**	**Shear Load [N]**	**Shear Stress [kPa]**	**Proving Ring Reading**	**Shear Load [N]**	**Shear Stress [kPa]**	**Proving Ring Reading**	**Shear Load [N]**	**Shear Stress [kPa]**
0	0.0	2827.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
30	0.3	2809.4	1.00	0.00	0.00	2.00	0.01	0.00	3.00	0.01	0.00
60	0.6	2791.4	16.00	0.05	0.02	25.00	0.07	0.03	115.00	0.33	0.12
90	0.9	2773.4	22.00	0.06	0.02	44.00	0.13	0.05	150.00	0.43	0.16
120	1.2	2755.4	30.00	0.09	0.03	63.00	0.18	0.07	178.00	0.51	0.19
150	1.5	2737.4	37.00	0.11	0.04	87.00	0.25	0.09	188.00	0.54	0.20
180	1.8	2719.4	47.00	0.13	0.05	115.00	0.33	0.12	190.00	0.54	0.20
210	2.1	2701.4	58.00	0.17	0.06	128.00	0.37	0.14	163.00	0.47	0.17
240	2.4	2683.4	67.00	0.19	0.07	140.00	0.40	0.15	159.00	0.46	0.17
270	2.7	2665.4	85.00	0.24	0.09	98.00	0.28	0.11	139.00	0.40	0.15
300	3.0	2647.4	90.00	0.26	0.10	92.00	0.26	0.10	126.00	0.36	0.14
330	3.3	2629.4	60.00	0.17	0.07	80.00	0.23	0.09	114.00	0.33	0.12
			Maximum shear stress, kPa **0.10**			*Maximum shear stress, kPa* **0.15**			*Maximum shear stress, kPa* **0.20**		

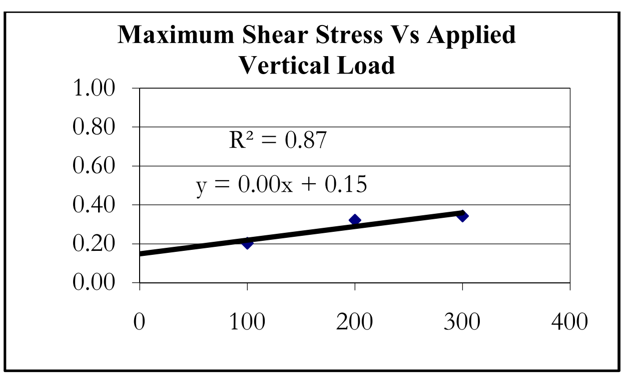


Cohesion , C (kN/m^2) =0.05
Angle of internal friction, ϕ = 0.0

Figure A. 11 C and Φ values of natural black cotton soil in shear stress v_s vertical load curve

Table A. 16 Shear strength Results of 2% flexi stabilized soil in modified compaction

Ring Calib. Factor =		0.002866 N/div	Applied Vertical Stress								
Rate of strain =		0.5 mm/min	100 kPa			200 kPa			500 kPa		
Horizontal Displacement [div]	**Horizontal Displacement [mm]**	**Corrected Area [mm2]**	**Proving Ring Reading**	**Shear Load [N]**	**Shear Stress [kPa]**	**Proving Ring Reading**	**Shear Load [N]**	**Shear Stress [kPa]**	**Proving Ring Reading**	**Shear Load [N]**	**Shear Stress [kPa]**
0	0.0	2827.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
30	0.3	2809.4	1.00	0.00	0.00	1.00	0.00	0.00	8.00	0.02	0.01
60	0.6	2791.4	2.00	0.01	0.00	2.00	0.01	0.00	32.00	0.09	0.03
90	0.9	2773.4	13.00	0.04	0.01	20.00	0.06	0.02	55.00	0.16	0.06
120	1.2	2755.4	22.00	0.06	0.02	34.00	0.10	0.04	75.00	0.21	0.08
150	1.5	2737.4	34.00	0.10	0.04	58.00	0.17	0.06	95.00	0.27	0.10
180	1.8	2719.4	48.00	0.14	0.05	77.00	0.22	0.08	116.00	0.33	0.12
210	2.1	2701.4	57.00	0.16	0.06	100.00	0.29	0.11	135.00	0.39	0.14
240	2.4	2683.4	65.00	0.19	0.07	119.00	0.34	0.13	154.00	0.44	0.16
270	2.7	2665.4	74.00	0.21	0.08	140.00	0.40	0.15	172.00	0.49	0.18
300	3.0	2647.4	89.00	0.26	0.10	160.00	0.46	0.17	191.00	0.55	0.21
330	3.3	2629.4	98.00	0.28	0.11	180.00	0.52	0.20	208.00	0.60	0.23
360	3.6	2611.4	103.00	0.30	0.11	197.00	0.56	0.22	226.00	0.65	0.25
390	3.9	2593.4	115.00	0.33	0.13	219.00	0.63	0.24	266.00	0.76	0.29
420	4.2	2575.4	123.00	0.35	0.14	234.00	0.67	0.26	279.00	0.80	0.31
450	4.5	2557.4	134.00	0.38	0.15	249.00	0.71	0.28	299.00	0.86	0.34
480	4.8	2539.4	146.00	0.42	0.16	262.00	0.75	0.30	304.00	0.87	0.34
510	5.1	2521.4	152.00	0.44	0.17	273.00	0.78	0.31	292.00	0.84	0.33
540	5.4	2503.4	160.00	0.46	0.18	280.00	0.80	0.32	285.00	0.82	0.33
570	5.7	2485.4	176.00	0.50	0.20	265.00	0.76	0.31	273.00	0.78	0.31
600	6.0	2467.4	151.00	0.43	0.18						
630	6.3	2449.4	139.00	0.40	0.16						
			Maximum shear stress, kPa **0.20**			***Maximum shear stress, kPa*** **0.32**			***Maximum shear stress, kPa*** **0.34**		

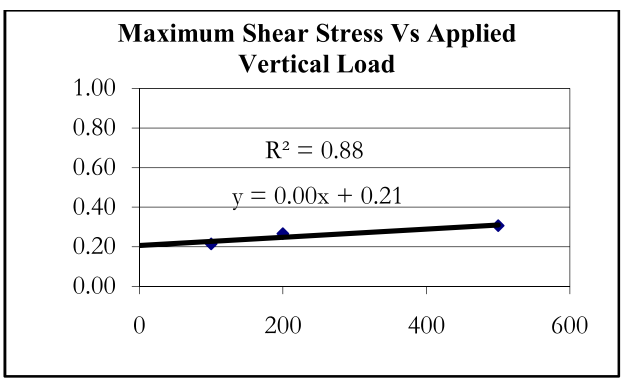


Cohesion , C (kN/m^2) =0.15	
Angle of internal friction, ϕ = 0.0	

Figure A. 12 C and Φ values of 2% flexi stabilized soil in shear stress v_s vertical load curve

Table A. 17 Shear strength Results of 3% sand stabilized soil in modified compaction

Ring Calib. Factor =		0.002866 N/div	Applied Vertical Stress								
Rate of strain =		0.5 mm/min	100 kPa			200 kPa			500 kPa		
Horizontal Displacement [div]	Horizontal Displacement [mm]	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0	0.0	2827.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
30	0.3	2809.4	16.00	0.05	0.02	20.00	0.06	0.02	51.00	0.15	0.05
60	0.6	2791.4	33.00	0.09	0.03	62.00	0.18	0.06	110.00	0.32	0.11
90	0.9	2773.4	51.00	0.15	0.05	99.00	0.28	0.10	188.00	0.54	0.19
120	1.2	2755.4	99.00	0.28	0.10	112.00	0.32	0.12	254.00	0.73	0.26
150	1.5	2737.4	133.00	0.38	0.14	193.00	0.55	0.20	291.00	0.83	0.30
180	1.8	2719.4	158.00	0.45	0.17	218.00	0.62	0.23	291.00	0.83	0.31
210	2.1	2701.4	181.00	0.52	0.19	251.00	0.72	0.27	259.00	0.74	0.27
240	2.4	2683.4	201.00	0.58	0.21	211.00	0.60	0.23	220.00	0.63	0.23
270	2.7	2665.4	192.00	0.55	0.21	200.00	0.57	0.22	210.00	0.60	0.23
300	3.0	2647.4	178.00	0.51	0.19	189.00	0.54	0.20	201.00	0.58	0.22
330	3.3	2629.4	149.00	0.43	0.16	160.00	0.46	0.17	180.00	0.52	0.20
360	3.6	2611.4				120.00	0.34	0.13	160.00	0.46	0.18
			Maximum shear stress, kPa =		**0.21**	*Maximum shear stress, kPa =*		**0.27**	*Maximum shear stress, kPa =*		**0.31**



Cohesion , C (kN/m^2) =0.21
Angle of internal friction, ϕ = 0.0

Figure A. 13 C and Φ values of 3% sand stabilized soil in shear stress v_s vertical load curve

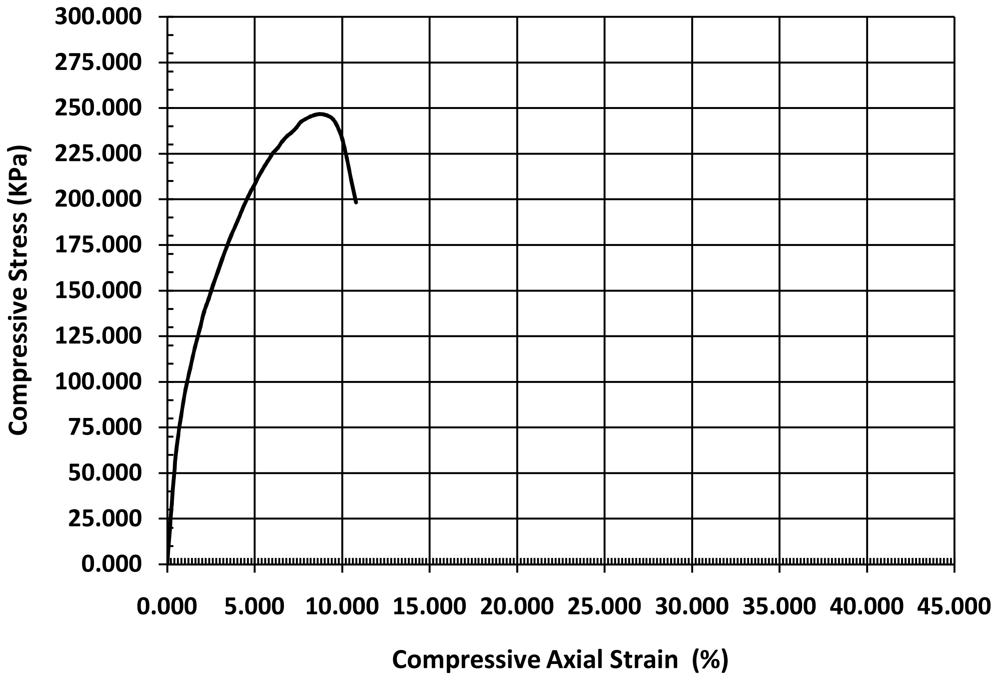


Figure A. 14 Stress-strain diagram of natural black cotton soil in standard UCS test

Table A. 18 UCS test Results of natural black cotton soil in standard compaction

Unconfined Compressive Strength, q_u (KPa)	246.536
Undrained Shear Strength, S_u (KPa)	123.268
Faliure Strain, E_f (%)	8.684
Wet Density, ρ (g/cm^3)	1.801
Dry Density, ρ_d (g/cm^3)	1.263
Average Moisture Content, *w (%)*	38.23

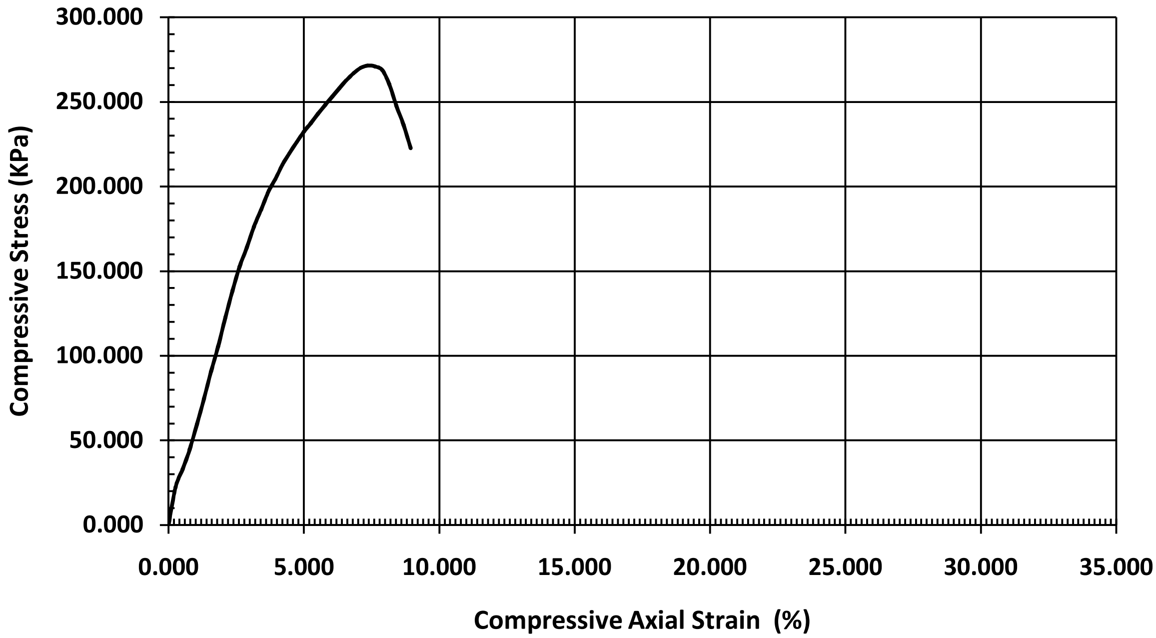


Figure A. 15 Stress-strain diagram of 2% flexi stabilized soil in standard UCS test

Table A. 19 UCS test Results of 2% flexi stabilized soil in standard compaction

Unconfined Compressive Strength, q_u (KPa)	271.430
Undrained Shear Strength, S_u (KPa)	135.715
Faliure Strain, E_f (%)	7.368
Wet Density, ρ (g/cm^3)	1.731
Dry Density, ρ_d (g/cm^3)	1.273
Average Moisture Content, *w (%)*	35.99

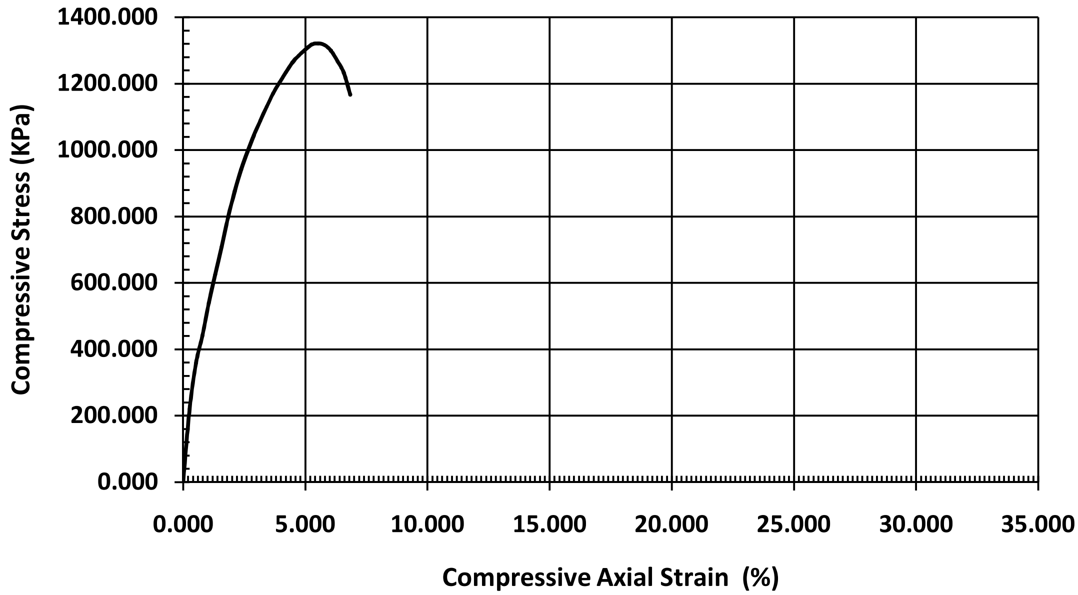


Figure A. 16 Stress-strain diagram of 7% sand stabilized soil in modified UCS test

Table A. 20 UCS test Results of 7% sand stabilized soil in modified compaction

Unconfined Compressive Strength, q_u (KPa)	1320.515
Undrained Shear Strength, S_u (KPa)	660.257
Faliure Strain, E_f (%)	5.526
Wet Density, ρ (g/cm^3)	1.975
Dry Density, ρ_d (g/cm^3)	1.614
Average Moisture Content, *w (%)*	22.36

Appendix B: Detail Results of Correlation and Regression Analysis

Table B. 1 Regression Estimation Section between PI and % of flexi

Parameter	Intercept B(0)	Slope B(1)
Regression Coefficients	64.0529	-0.4581
Lower 95% Confidence Limit	58.4498	-1.5985
Upper 95% Confidence Limit	69.6559	0.6822
Standard Error	2.1797	0.4436
Standardized Coefficient	0.0000	-0.4193
T Value	29.3864	-1.0328
Prob Level (T Test)	0.0000	0.3490
Reject H0 (Alpha = 0.0500)	Yes	No
Power (Alpha = 0.0500)	1.0000	0.1354
Regression of Y on X	64.0529	-0.4581
Inverse Regression from X on Y	72.3365	-2.6058
Orthogonal Regression of Y and X	67.0441	-1.2337

Table B. 2 Regression Estimation Section between OMC and % of flexi

Parameter	Intercept B(0)	Slope B(1)
Regression Coefficients	35.0727	0.3200
Lower 95% Confidence Limit	31.0389	-0.5009
Upper 95% Confidence Limit	39.1065	1.1410
Standard Error	1.5692	0.3194
Standardized Coefficient	0.0000	0.4090
T Value	22.3503	1.0021
Prob Level (T Test)	0.0000	0.3623
Reject H0 (Alpha = 0.0500)	Yes	No
Power (Alpha = 0.0500)	1.0000	0.1303
Regression of Y on X	35.0727	0.3200
Inverse Regression from X on Y	28.9265	1.9135
Orthogonal Regression of Y and X	34.1336	0.5635

Table B. 3 Regression Estimation Section between MDD and % of flexi

Parameter	**Intercept B(0)**	**Slope B(1)**
Regression Coefficients	1.1510	-0.0051
Lower 95% Confidence Limit	1.1277	-0.0099
Upper 95% Confidence Limit	1.1743	-0.0004
Standard Error	0.0091	0.0018
Standardized Coefficient	0.0000	-0.7805
T Value	127.0556	-2.7920
Prob Level (T Test)	0.0000	0.0384
Reject H0 (Alpha = 0.0500)	Yes	Yes
Power (Alpha = 0.0500)	1.0000	0.6120
Regression of Y on X	1.1510	-0.0051
Inverse Regression from X on Y	1.1637	-0.0084
Orthogonal Regression of Y and X	1.1510	-0.0051

Table B. 4 Regression Estimation Section between CBR and % of flexi

Parameter	**Intercept B(0)**	**Slope B(1)**
Regression Coefficients	2.9909	-0.3465
Lower 95% Confidence Limit	1.9923	-0.5498
Upper 95% Confidence Limit	3.9896	-0.1433
Standard Error	0.3885	0.0791
Standardized Coefficient	0.0000	-0.8908
T Value	7.6989	-4.3830
Prob Level (T Test)	0.0006	0.0071
Reject H0 (Alpha = 0.0500)	Yes	Yes
Power (Alpha = 0.0500)	1.0000	0.9334
Regression of Y on X	2.9909	-0.3465
Inverse Regression from X on Y	3.3388	-0.4367
Orthogonal Regression of Y and X	3.0292	-0.3565

Table B. 5 Regression Estimation Section between C and % of flexi

Parameter	Intercept B(0)	Slope B(1)
Regression Coefficients	0.0832	-0.0031
Lower 95% Confidence Limit	0.0513	-0.0096
Upper 95% Confidence Limit	0.1152	0.0034
Standard Error	0.0124	0.0025
Standardized Coefficient	0.0000	-0.4758
T Value	6.6926	-1.2096
Prob Level (T Test)	0.0011	0.2805
Reject H0 (Alpha = 0.0500)	Yes	No
Power (Alpha = 0.0500)	0.9993	0.1677
Regression of Y on X	0.0832	-0.0031
Inverse Regression from X on Y	0.1236	-0.0135
Orthogonal Regression of Y and X	0.0832	-0.3565

Table B. 6 Regression Estimation Section between S_u and % of flexi

Parameter	Intercept B(0)	Slope B(1)
Regression Coefficients	132.2612	-8.2723
Lower 95% Confidence Limit	112.7179	-12.2498
Upper 95% Confidence Limit	151.8046	-4.2949
Standard Error	7.6027	1.5473
Standardized Coefficient	0.0000	-0.9226
T Value	17.3966	-5.3463
Prob Level (T Test)	0.0000	0.0031
Reject H0 (Alpha = 0.0500)	Yes	Yes
Power (Alpha = 0.0500)	1.0000	0.9868
Regression of Y on X	132.2612	-8.2723
Inverse Regression from X on Y	137.8428	-9.7194
Orthogonal Regression of Y and X	137.7741	-9.7016

Table B. 7 Regression Estimation Section between q_u and % of flexi

Parameter	Intercept B(0)	Slope B(1)
Regression Coefficients	264.5220	-16.5446
Lower 95% Confidence Limit	225.4351	-24.4996
Upper 95% Confidence Limit	303.6090	-8.5897
Standard Error	15.2055	3.0946
Standardized Coefficient	0.0000	-0.9226
T Value	17.3965	-5.3463
Prob Level (T Test)	0.0000	0.0031
Reject H0 (Alpha = 0.0500)	Yes	Yes
Power (Alpha = 0.0500)	1.0000	0.9868
Regression of Y on X	264.5220	-16.5446
Inverse Regression from X on Y	275.6853	-19.4388
Orthogonal Regression of Y and X	275.6506	-19.4298