ADAMA SCIENCE AND TECHNOLOGY UNIVERSITY

SCHOOL OF CIVIL ENGINEERING AND ARCHITECTURE

DEPARTMENT OF CIVIL ENGINEERING

STABILIZATION OF EXPANSIVE SOILS BY STEEL SLAG, CASE OF EXPANSIVE CLAY SOIL IN ADDIS ABABA

A THESIS SUBMITED TO THE DEPARTMENT OF

CIVIL ENGINEERING (MAJOR GEOTECHNICAL ENGINEERING)

BY

TESFAY KIROS

SEPTEMBER 2016

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ETHIOPIA

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A THESIS SUBMITED TO THE DEPARTMENT OF CIVIL ENGINEERING IN PARTIAL FULFILLMENT OF THE REQUIREMENT FOR THE AWARD OF MASTERS OF SCIENCE IN CIVIL ENGINEERING [MAJOR GEOTECHNICAL ENGINEERING].

BY

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SEPTEMBER 2016

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DECLARATION

I Tesfay Kiross, declare this thesis is my own original work for the requirements of MSC and has not been presented and submitted to any other University for similar or any other degree award.

Signature: ____________

Date: ____________

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ABSTRACT

Investigation of industrial waste by products as geo material over the ordinary and market oriented stabilizers has both economic and environmental significance. Steel slag is one of the waste products which is investigated in this as geo material for stabilization in this study.

This research is aimed at evaluating the suitability of steel slag for stabilization of expansive soils. The laboratory investigations involved Atterberg limit tests and grain size distributions for classifying the fine grained soils. Accordingly, the investigation of the untreated soil belongs to A -7-6 class of soil in the AASHTO classification system and CH in the USCS with very poor strength. Atterberg limits, compaction, free swell, specific gravity, CBR and UCS tests were used to evaluate properties of the stabilized soil. The soil was stabilized with varied proportions of steel slag like 3%, 5%, 7% and 9% by dry weight of the soil. The treated soil samples for UCS, Atterberg limits and CBR were also cured for 3 and 7 days both in desiccator and none permeable hard plastic at room temperature respectively.

The optimum steel slag required for stabilizing the expansive soil was determined based on strength values of moisture density relation, UCS and soaked CBR which is obtained as 7%.

Analysis of the test result indicates an improvement in engineering properties of steel slag stabilized soils and liquid limit, plastic limit, plasticity index, OMC and free swell slightly reduced. As the amount of steel slag increases, all the engineering properties of the treated soil (UCS, MDD and CBR) also increase until the optimum amount of the stabilizer.

Key words: Expansive soils, Steel Slag, Index properties, engineering properties (CBR, UCS, moisture density relation), soil stabilization.

Contents

LIST OF SYMBOLS AND ABRIVATIONS

ASTM – American Society for testing Materials.

ABMI - Akaki Basic Metallic Industry.

ERA - Ethiopian Road Authority.

MC – Moisture content

CBR – California bearing ratio.

SGT – specific gravity test

UCS – Unconfined compressive strength.

BOF – Basic Oxygen Furnace slage.

EAF - Electric Arc Furnace slag.

XRD X-ray Diffraction

XRF X-ray Fluorescence.

MDD – Maximum dry density.

OMC –Optimum moisture content

AASHTO - American Association of State

Highway and Transport officials.

ANSS - Anyway Natural Soil Stabilizer.

SS – Steel Slag

USCS- Unified Soil Classification System.

GW – Well graded Gravel.

GP –Poor graded Gravel.

SW- Well graded Sand.

SP – Poor graded Sand.

SM –Silt sand

CH – High plasticity in organic clay.

cc – Centimeter cube.

mm – millimeter.

KPa – kilo Pascal

CEC- Cationic exchange capacity.

qu – Unconfined compressive Strength.

C- Cohesive.

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1. INTRODUCTION

1.1 Preamble

Expansive soils are major problematic soils due to their seasonal cyclic variations ,cyclic swell during wet seasons and cyclic shrink during dry seasons .The cyclic seasonal variation results in a volume change which leads to un expected damages and costs . Expansive soils are major geo technical hazards that can pose several limitations on performance and life time of light weight engineering structures. Major problems are volume changes due to swelling and shrinking, which can lead to differential settlement and creep, decrease in bearing capacities and shear strength when saturated, Failure of pavements, settlements, heave depression, cracking and unevenness are among the most problems of expansive soils up on seasonal cyclic variations. Such Problems of expansive soils in civil structures were not well studied since 1930's.The origin of expansive soils is related to a complex combination of conditions and processes that result in the formation of clay minerals having a particular chemical makeup which, when in contact with water, will expand. All clay soils are not expansive and the degree of expansion varies with the type of clay mineral predominantly present in the soil mass. The presence of montmorillonite in these soils imparts them high swell-shrink potentials. These soils are very hard when dry, but lose strength completely when wet .Such problematic soils are distributed worldwide in arid and semi-arid climates. Argentina ,Iran, Australia Mexico, Burma, Morocco, Canada, Rhodesia, Cuba, South Africa, Ethiopia ,Spain, Ghana ,Turkey, India ,U.S.A.,Israel ,Venezuela are among the countries in the world in which the existence of expansive soils are reported (Chen1988) .

According to Hbtamu Solomon,(2011) , as cited by Alemayehu Mengistu, (2006), expansive soils form a major soil group in Ethiopia and occur in the highlands dominantly in the Western, Central and south eastern parts of the country, on the colluvial slopes and alluvial plains, on the vast lime stone plateaus of Central Harerge and in sites like Granitic colluvium in basins with seasonal drainage deficiencies in southern Sidamo which occupy 116,785 sq.km which accounts to about 10.5% of the total area of the country. The largest extents of vertisols are found on the volcanic plateaus. In most cases the major clay mineral components are montmorillonite.

Identifying expansive soils and quantifying their potential expansiveness is crucial to ensure propose site selection, environmentally compatible, economically feasible designing and construction especially of lightly loaded structures.

1.2 Justification for the study

- Cost saving than the other ordinary stabilizers and methods of stabilization like replacement, as steel slag is waste by product.
- Traditional stabilizers are not environmentally friendly stabilizer, rather they are market oriented.
- Both economical and waste management can be facilitated for the nearby environment by distributing the slag in different engineering fields of application.
- The fast growth of the country in different industrial activities will enhance a direct contact with the construction Industry by intention to wards such waste by products with various engineering applications.

1.3 Objectives of the study

The general objective of this study is to evaluate the suitability of steel slag as a stabilizing agent for expansive soils. The main objective of this study is achieved through the following specific objectives.

The specific objectives of the study are:

1. To increase the strength or stability of the weak soil by using steel slag (locally available waste material).
2. To evaluate the effect of steel slag on the properties of the expansive soil for Index and Engineering properties (CBR and UCS).
3. To compare the change in properties of the natural soil with steel slag treated or stabilized soil.
4. To evaluate the optimum stabilizer required for stabilization in the study area.

1.4 scope of the study

The study is mainly focused on the determination of some engineering properties of the stabilized soil by conducting laboratory tests and by detail interpretation of the test results both for untreated soil and treated soil with steel slag mixtures. In addition, the study will provide transforming steel slag which is an industrial waste into valuable materials or resources for further utilization in different applications of the construction industry. This study has been supported by secondary resources and a series of laboratory experiments. However, the findings of the research are limited to one soil samples considered in the research. The results are also specified to the type of stabilizer used and test procedures that have been adopted in the experimental work. Therefore, findings should be considered indicative as performed from simple test samples, rather than definitive as it is not conducted for several samples collected from different areas and special laboratory tests for the stabilizer and the proposed expansive soils due to time and un availability of some sensitive tests .

1.5 Organization of the report

The paper contains five chapters. The first chapter describes the introduction part, justification of the study, objective of the study and scope of the study. Chapter two explains literature review on some conceptual points. The third chapter describes about materials and methodology. Chapter four explains test results and discussions as well as detail result interpretations. The last chapter describes about conclusions and recommendations from findings of the study about the stabilized material for the proposed soil by steel slag.

2. LITERATURE REVIEW

2.1 Permeable

Expansive soils are a general indication of soils which has a potential for shrink and swell under moisture variation seasons. Their moisture variation causes unexpected problems up on different civil infrastructures. Expansive soils are alternatively called black cotton (BC) for areas in which cotton grow are widely spread like in India and problematic soils for the field of engineering. These problematic soils are major geotechnical hazards that can pose severe limitations on performance and life time of light weight engineering infrastructures worldwide. Major problems encountered due to these problematic soils are volume changes due to swelling and shrinking, which can lead to differential settlement and creep, decrease in bearing capacities and shearing strength when saturated, high erosion susceptibility when exposed in cuts or open excavations and difficult workability conditions. Mostly effects posed due to expansive soils are light buildings and pavements than any other natural hazards like , Earth quake and flooding (Jones and Holtz ,1973) .

Identifying expansive soils and quantifying their potential expansiveness is crucial to ensure proper site selection, environmentally compatible and economically feasible designing and construction especially of lightly loaded structures. However, common geotechnical practices of characterizing expansive soils need dense sampling, thus are costly, labour intensive, time consuming and difficult to get a continuous representation of soil masses in space. Though expansive soils occur anywhere of the world; arid and semiarid parts are the most areas dominated for these soils due to seasonal moisture fluctuations. These soils are characterize in areas of poor drainage, low rain fall, exceeding heat and in areas of precipitation exceeds rainfall (Chen 1988).
Ethiopia is among the countries located in the tropic of the world map that reports the existence of expansive soils in different parts of the country. Existence of these problematic soils retards the construction industry which has a significance effect on the continuous transformation growth and leads an economic crisis for the nation.

2.2 Origin of Expansive Soils

The origin of expansive soil is the result of the combination and processes that leads to the formation of clay minerals having a particular chemical and mineralogical make up, which, when in contact with water expands. Variations in the conditions and processes may also form other clay minerals, most of which are non-expansive. The conditions or processes that determine the clay mineralogy include composition of the parent material and degree of physical and chemical weathering to which the materials are subjected. The combinations and processes may include parent material, drainage and climate [Chen, F.H., 1988].

I. Parent materials

The constituents of the parent material during the early and intermediate stages of the weathering Process determine the type of clay formed. The nature of the parent material is much more important during these stages than after intense weathering for long periods of time. The parent materials that can be associated with expansive soils are classified into two groups. The first group comprises the basic igneous rocks, such as the basalts, the dolerite sills and dykes, gabbros and norities. In these soils, the feldspar and pyroxene minerals of the parent rocks have decomposed to form montmorillonite and other secondary minerals. The second group comprises the sedimentary rocks that contain montmorillonite as a constituent which breaks down physically to form expansive soils. Bedrock shale, shale of the Ecca Series, marls and lime stones and the more recent Laramie and Denver Formations are examples of this type of rock. The basic igneous rocks are comparatively low in silica, generally about 45% to 52%. Rocks that are rich in metallic base such as the pyroxenes, amphiboles, biotite and olivine fall within this category. Such rocks include the gabbros, basalts and volcanic glass, Shale and clay stones are one of sedimentary rocks that contain montmorillonite as a constituent. Limestone and marls rich in magnesium can also weather to clay. These constituents of the shales and clay stones contain varying amount of volcanic ash and glass, which are subsequently weathered to montmorillonite [Chen, H., 1988].

II. Weather condition and Climate

Climate and topography are other very significant conditions in the formation of tropical black cotton soils soils. Expansive soils are common in areas of moderate rainfall and poor drainage.
Under these conditions and in an alkaline environment where there is adequate supply of magnesium, Silica and alumina, montmorillonite, which is the principal clay mineral, is formed. These Soils are rich in alkaline earths. Horizons of calcium sometimes develop or lime concretions may be found scattered through the profile [Nelson D. and Holtz 1997].

2.2.1 Minerals related to Clay soils

Clays are fundamentally very different from gravels, sands, and silts. All the later consist of relatively inert bulky particles and their engineering properties depend primarily on the size, shape, and texture of these particles. In contrast, clays are made up of very small particles that are usually plate-shaped.

The engineering properties of clays are strongly influenced by the very small size and large surface area of these particles and their inherent electrical charges.

Clay minerals are complex aluminum silicates composed of two basic units:

(1) *Silica tetrahedron* and

(2) *Alumina octahedron.* Each tetrahedron unit consists of four oxygen atoms surrounding a silicon atom. As described by Braja M.das (Cited in Grim, 1995) several different clay minerals occur in nature, the differences being defined by their chemical makeup and structural configuration.

Three of the most common clay minerals are:

1. Kaolinite
2. Illite
3. Montmorillonite

The different chemical compositions and crystalline structures of these minerals give each a different susceptibility to swelling. Swelling occurs when water infiltrates between and within the clay particles, causing them to separate.

2.2.1.1 Kaolinite Mineral

Its basic structural unit consists of aluminum sheet (gibbsite) (G) combined with a silica sheet (S) .Tips of the silica sheet and one base of the aluminum sheet form a common interface. The total thickness of the structural unit is about 7 Angstrom (A_o), where one Angstrom is equal to 10^{-10}m or 10^{-7}mm. The kaolinite mineral is formed by stacking one over the other several such basic structural units. The structural units join together by hydrogen bond, which develops between the oxygen of silica sheet and hydroxyls of alumina sheet. As the bond is fairly strong, the mineral is stable. Moreover, water cannot easily enter between the structural units and cause expansion.

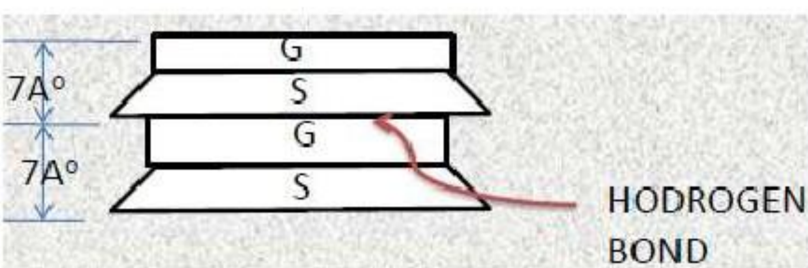


Figure2.1 Arrangements of structural units of kaolinite mineral

2.2.1.2 Montmorillonite Mineral

The basic structural unit consists of an aluminum sheet sandwiched between two silica sheets. Successive structural units are stacked one over another, like leaves of a book. The thickness of each structural unit is about 10A₀.The two successive structural units are joined together by a link between oxygen ions of the two silica sheets. The negatively charged surfaces of the silica sheet attract water in the space between two structural units.

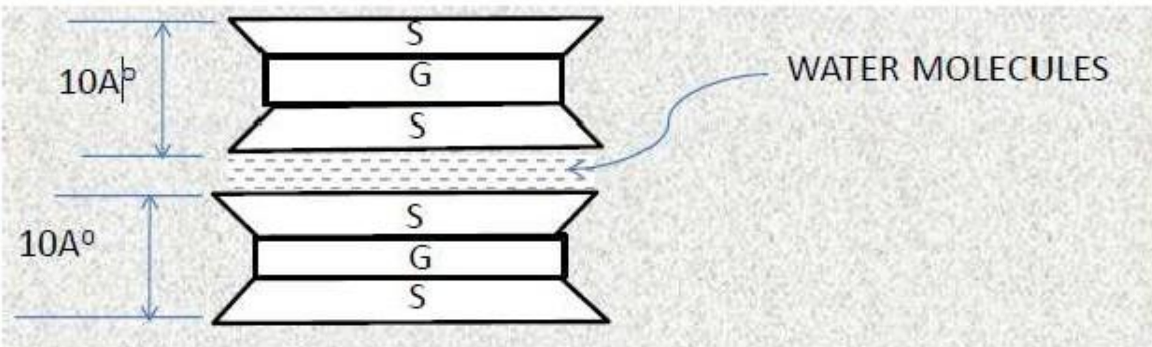
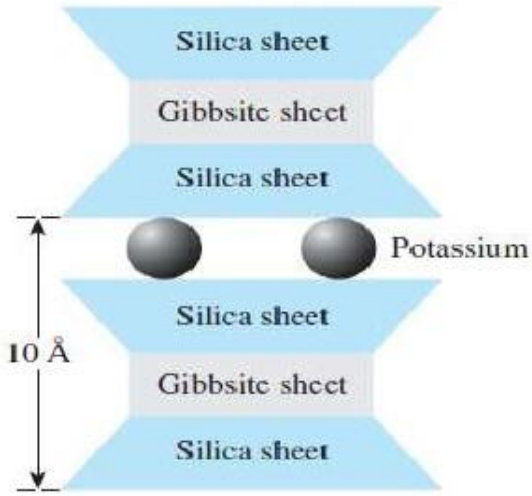


Figure2.2 Arrangement of structural unit of montmorillonite mineral.

2.2.1.3 Illite Mineral

The basic structural unit is similar to that of the mineral montmorillonite. However, the link between different structural units is through non-exchangeable potassium (K^+) and not through water. This bonds the units more firmly than in montmorillonite. Illite swells less than montmorillonite. However, swelling is more than in kaolinite.



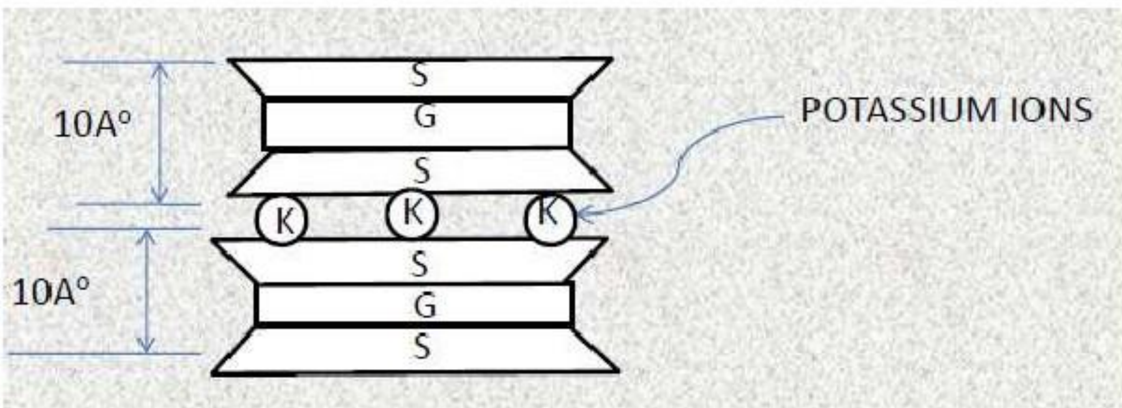


Figure2. 3Arrangement of structural unit for Illite mineral

2.3 Distribution of Expansive Soils

Expansive soils are wide spread in the African continent, occurring in South Africa, Ethiopia, Kenya, Mozambique, Morocco, Ghana, Nigeria etc. In other parts of the world case of expansive soils have been widely reported in U.S.A , Australia , Canada , Spain ,Israel , Turkey ,Argentina , Venezuela etc. (Tefera , A. , and Leikun . , M.1999).

Expansive soils form a major soil group in Ethiopia and occur in the highlands dominantly in the Western, Central and south eastern parts of the country, on the colluvial slopes and alluvial plains , on the vast lime stone plateaus of Central Harerge and in sites like Granitic colluvium in basins with seasonal drainage deficiencies in southern Sidamo . In most case the major clay mineral components are montmorillonite. In Ethiopia these expansive soils or vertisols occupy 116,785sq.km which accounts to about 10.5% of the total area of the country (Geo science Data center Geological Survey of

Ethiopia, December 2009). Expansive soil found in different parts of Ethiopian extent and range of distribution of this Problematic soil has been studied thoroughly by different researchers. Southern, South- East and South - West part of the city of Addis Ababa areas, where most of the recent construction are being carried out and central part of Ethiopia following the major trunk roads like Addis-Modjo, Addis- Ambo, Addis Welliso, Addis- Debre Birhan, Addis- Gohatsion area are covered by expansive soils and also areas like Mekelle and Gambela are covered by expansive soils. The distributions of expansive black cotton soils of Addis Ababa are shown in figure 2.4.

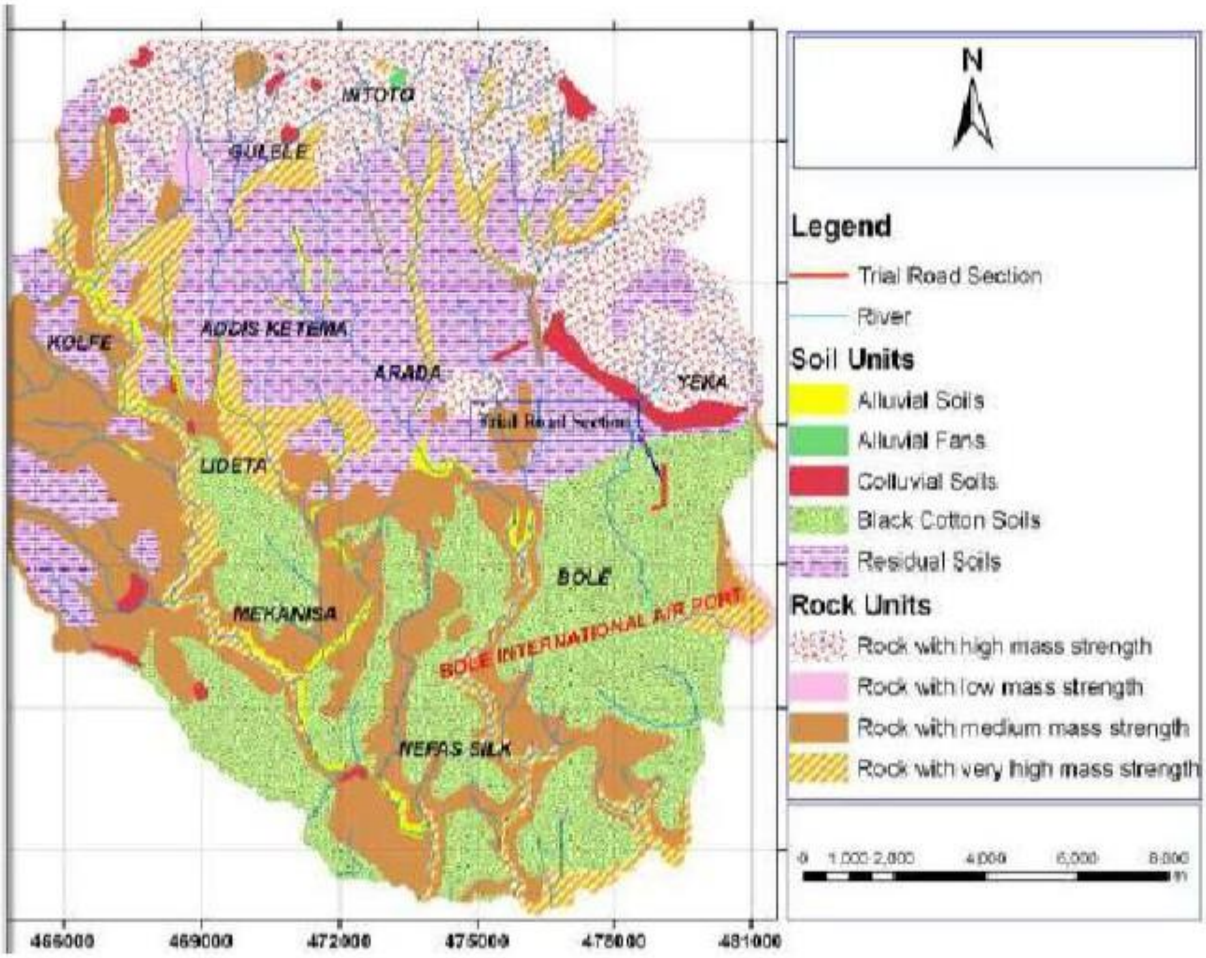


Figure2. 4 Distribution of expansive soils of Addis Ababa (Source Engineering Geological Map of Addis Ababa).

2.4 Identification of Expansive Soils.

Expansive soils can be investigated through two mechanisms both in field and in laboratory. These mechanisms are visual identification and sampling of materials and examining their physical properties.

2.4.1 Field Identification of Expansive Soils

Soils that are characterized by high swelling behaviors can be identified in the field during reconnaissance and preliminarily investigations (Chen F.H., 1988 and JohnD.Nelson 1992).

The following are among the characteristics that are used for field identifications of expansive soils.

1. Usually have either grey or black color.
2. Plasticity characteristics
3. Dilatency properties
4. High dry strength and low Wet strength
5. Appearance of deep or wide surface cracks
6. Visibility of identifiable cracks on different structures

2.4.2 Laboratory Identifications

Expansive soils can be identified in the laboratory through mineralogical tests, direct and indirect test methods.

2.4.2.1 Mineralogical Identifications

The mineralogical composition of expansive soils has an important bearing on the swelling potential as explained under clay Structure. The negative electric charges on the surface of the clay minerals, the strength of the interlayer bonding, and the cation exchange capacity all contribute to the swelling potential of the clay. Hence, it is claimed by the clay mineralogist that the swelling potential of any clay can be evaluated by identification of the constituent mineral of this clay. Techniques which may be used as mineral identification methods are as follows (Chen, F.H., 1988 and John, D., Nelson 1992).

1. X-ray diffraction.
2. Differential thermal analysis.
3. Dye adsorption.
4. Chemical analysis.
5. Electron microscope resolution.

These tests cannot be considered as routine tests due to the following limitations.

1. Test results are time consuming.
2. Extremely expensive and
3. Interpretation of test results requires a special and skilled technician.

2.4.2 .2 Indirect Test Method

Simple soil property tests can be used for the evaluation of the swelling potential of expansive soils. Such tests are easy to perform and should be included as routine tests in the investigation of building sites in those areas having expansive soil. Such tests may include

I. Atterberg limit's tests.

Plasticity index and liquid limit are useful indices for determining the swelling characteristics of most clay. Plasticity index alone can be used as a preliminary indication of swelling characteristics of most clay (Chen, F.H., 1988).

Table 2. 1 Relation between swelling potential of clays and plasticity index.

Swelling Potential	Plasticity Index
Low	0-15
Medium	10-35
High	20-35
Very High	35 and above.

II. Free swell Test

This test is a simple tests conducting in laboratory for determination of degree of expansiveness. Free swell tests consist of placing a known volume of dry soil in water and noting the swelled volume after the material settles, without any surcharge, to the bottom of a graduated cylinder. The difference between the final and initial volume, expressed as a percentage of initial volume, is the free swell value. The swell test is very crude and was used in the early days when refined testing methods were not available. Experiments indicated that a good grade of high swelling commercial betonies will have a free swell value from 1200 to 2000 percent. Soils having free swell value as low as 100 percent can cause considerable damage to lightly loaded structures, and soils having free swell value below 50 percent seldom exhibit appreciable volume change even under very light loadings (Alemayo T.and Leikun,. M., 1999 and Chen F.H., 1998, Nelson 1992).

$$freesweel\,(\%) = \frac{Vf - Vi}{Vi}$$

Where:

V_f = Final Volume after twenty four hours (ml)

V_i = Initial Volume placed on the cylindrical plate (10ml)

Table2. 2 Relation between free and condition of expansiveness

Fee swell (%)	Description for expansiveness
<50	Not expansive
50-100	Marginal
>100	Expansive

III. Cationic Exchange Capacity (CEC)

Cationic exchange capacity is the charge or electrical attraction for cation per unit mass as measured in milliequivalent per 100 grams of soil. The cation exchange capacity of different types of clay minerals may be measured by washing a sample of each with a solution of a salt such as ammonium chloride NH_4CL and the amount of adsorbed NH_4^+ by measuring the difference between the original and the final concentration of the washing solution. The exchangeable ions are held around the outside of the silica-alumina clay-mineral structural unit, and the exchange reaction does not affect the structure of the silica-alumina pocket. In clay minerals, the most common exchangeable cations are Ca^{2+}, Mg^{2+}, F, T,K^+ ,$NH4^+$ and Na^+ frequently in about that order of general relative abundance. The existence of such charges is indicated by the ability of clay to absorb ions from the Solution. Cations (positive ions) are more readily absorbed than anions (negative ions). Hence, negative charges must be predominant on the clay surface. A cation, such as Na^+ is readily attracted from a salt solution and attached to a clay surface. However, the absorbed Na^+ ion is not permanently attached; it can be replaced by K^+ ions if the clay is placed in a solution of potassium chloride KCL.

2.4.2.3 Direct method

This method is the most reliable and easiest method of determining the swelling potential and swelling pressure of expansive clay is by direct measurement. Direct measurement of expansive soils is determined by the use of the conventional one-dimensional consolidometer. The consolidometer can be either platform type, scale type, or other arrangement. In this case load is applied with air as in the case of Conbel consolidometer or by direct weight as in the case of cantilever consolidometer. The soil sample is enclosed between two porous plates and confined in a metal ring. Such a device enables an easy and accurate measurement of the swelling potential of clay under various conditions (Chen., F.H., 1988).

2.5 Classification of Expansive Soils

A soil classification system is the arrangement of different soils having similar properties into groups and sub-groups based on their application which provides a common language to express briefly the general characteristics of soils. There are several methods of classifying soils. The most widely used classification systems by engineers are described here.

2.5.1 Classification by General Methods

The following two classification systems are the most widely used classification schemes in the field of engineering.

I. Unified Soil Classification System

This classification system is the most widely used in all types of engineering problems involving soil. It is based on recognition of the type and predominant the constituents considering grain size, plasticity and gradation. This method classifies soils in to three major groups.

1. Coarse grained soils (GW, GP, S W, SP)
2. Fine grained soils (M, H, C, O and Pt.)
3. Highly organic peat (Pt.)

Based on the correlation of swelling potential and the Unified Soil Classification system the above three groups can be described interms of their degree of expansion as follows.

Table 2.3 Relatioshionship between USCS and Degree of Expansion.

Soil Classification Category	Degree of Expansion
GW ,GP, Gm, SW,SP and SM	Little or no Expansion
GW,SC,MC, and MH	Moderate Expansion
CL ,OL, CH and OH	High volume change of Expansion
Pt.	No visible Rating of Expansion

II. AASHTO Soil Classification System

The AASHTO classification system, also called Public Roads Administration (PRA) classification, is based on the following three soil properties.

1. Grain size distribution.
2. Liquid limit and
3. Plasticity Index

This system is generally used by highway engineers, for classification of sub-grade soils for highway pavements. According to this system soils are classified into seven major groups, A-1 through A-7.

2.6 Stabilization of Expansive soils

Soil stabilization techniques for road construction are used in most parts of the world. Although the circumstances and reasons for restoring to stabilization vary considerably; stabilization is generally the alteration of one or more soil properties, by mechanical or chemical means to get a desired improved soil material which full fill an engineering properties of the soil for various applications in construction industry. Stabilization Process can be either blending or mixing of the soil and the stabilizer which may be locally and commercially available materials to get the desired improved soil for construction. As per Ethiopian Roads Authority (ERA) and ACRA Standard Technical Specifications, stabilization of materials is used in the construction of the in-situ subgrade by the addition of a chemical stabilizing agent or by the mechanical modification of the material by mixing various materials or by treating the material with a bituminous stabilizing agent. It includes the furnishing, spreading and mixing of the stabilizing agent or soil binder. In the case of chemically stabilized material, the layer is given a curing treatment.

Material to be stabilized shall be any naturally occurring cohesive or non-cohesive soil or processed material of weak strength, the properties of which can be permanently improved by the process of Stabilization to satisfy with the requirements of the Specification for the relevant of the material for a particular geo application. Stabilizing the existing expansive soils includes; other process of mixing materials with the soil to improve certain properties of the soil, other blending of soils with other materials to achieve a desired gradation or other mixing of commercially available additives that change the gradation, texture or plasticity of the soil.

2.6.1 Applications of Stabilization

Pavement design and construction relies on the permissible structural quality of each component layer which should not show any deformation, excessive settlement, seepage and other related structural failurities for proper serviceable use of the road. The same is true for any structure rested on different soil strata. As the quality of the different layer increases, the load distributed over the corresponding cross sectional area also increases results on suitable design matter for the required road section.

Most commonly advantages of treated soil to be attained are:

1. Reduction of Thickness: As the strength of sub grade and sub base attains in good stable manner the corresponding design thickness reduces such as avoid of capping layers.
2. Quality improvement: stabilized soils reduces the index properties [liquid limit , plastic limit , plasticity index and shrinkage limits] swelling potential, free swell and free swell index and increases in engineering properties such as CBR and UCS for the required purpose.

2.6.2. Techniques of Soil Stabilization

Techniques which are currently available for stabilizing of expansive soils include;

- ❖ Remolding and Compaction
- ❖ Surcharge Loading
- ❖ Prewetting
- ❖ Moisture Control
- ❖ Mechanical stabilization
- ❖ Chemical Admixtures.

2.6.2.1 Stabilization by Compaction and Remoulding

Soil compaction is the process whereby soil particles are constrained to peak more closely together through a reduction in the amount of air contained in the soil mass. It is common experience that the stability of a soil can be increased by increasing its state of compaction.

Advantages of remoulding and compacting include;

- ❖ Avoiding of cost of importing selected material.
- ❖ Proper compaction of the soil minimizes infiltration of water into underlying soils.
- ❖ It is economically feasible to scarify, pulverize, and re - compact such expansive soils.

Some disadvantages of remoulding and compaction include;

- ❖ The lower bearing capacity of the low density compaction may not be adequate.
- ❖ Some soils have such high potential for volume change that compaction control doesn’t reduce swell potential significantly and replacement or stabilizing may be necessary.
- ❖ Compacting at specified densities and water contents may necessitate frequent testing to maintain quality control, which may increase the cost of the project [Nelson, D., 1990].

2.6.2.2 Surcharge loading

The swell property of an expansive soil can be prevented using surcharge load which is large enough to counteract the expected swell pressures. This method is most effective when swell pressures are low. However, many soils exhibit swell pressure too high to be controlled by normal surcharge loads alone.

2.6.2.3 Prewetting

Prewetting is based on the theory that increasing the moisture content in the expansive foundation soils will cause heave to occur prior to construction and there by eliminate problems afterward. If the high moisture content is maintained, there will be no appreciable increase in soil volume to damage the pavement structure. The procedure has serious drawbacks that limit its application. Expansive soils typically exhibit low hydraulic conductivity and the time required for adequate wetting can be up to several years. Furthermore, after the water has been applied for long periods of time serious loss of soil strength can result causing reductions in bearing capacity.

2.6.2.4 Moisture control

Soil expansion problems are primarily the result of fluctuations in water content. Non uniform heave will result from either non uniform water content changes, non-uniform soil conditions, or both. If fluctuations in water content over time can be minimized and if the water content in the subsoil can be made uniform, a major part of the expansion problem can be mitigated. The placement of a structure or pavement on the ground surface will change the evapotranspiration from the surface. Changes in land use, such as irrigation, will change the potential for infiltration. These changes will, in turn, change the water content and its distribution in the sub soil. If the change in water content can be made to occur slowly and if the water content distribution can be made uniform, differential heave can be minimized (Nelson, 1990).

2.6.2.5 Mechanical Stabilization

This technique is a process whereby the grading of a soil is improved by the incorporation of other material which affects only the physical properties of the soil.

2.6.2.6 Chemical Stabilization

Stabilization through different admixtures is the most ground improvement techniques applied in most geo technical engineering areas. Expansive soils undergo modification when a catalyst agent is introduced into the soil. In turn, the chemical reaction converts the engineering properties of the material which was unsuitable to suitable for road sub base or sub grade materials.

This stabilization is mixing of soil with one or more combination of admixtures, slurry or liquid for the achievement of stability in both the index and engineering properties. Cationic exchange, flocculation and pozzolanic reactions are the most types of chemical stabilization of soils.

I. Cationic Exchange

The excess of ions of opposite charge of the surface clay over those of like charges present in the diffuse double layer are considered to be exchangeable ions. These ions can be replaced by a group of different ions having the same total charge by altering the chemical composition of the equilibrium electrolyte solution. Natively charged clay particles adsorb cations of specific type and amount. The ease of replacement or exchange of cations depends on several factors, primarily the valance of the cation. Higher valance cations easily replace cations of lower valance. For ions of the same valance, the size of the hydrated ion becomes important, the large the ion the greatest the replacement power. If other conditions are equal, trivalent cations are held more tightly than the divalent and divalent cations are held more tightly than the monovalent cations.

Atypical replacement series is as shown below.

$Na^{+} < Li^{+} < K^{+} < Rb^{+} < Cs^{+} < Mg^{2+} < Ca^{2+} < Ba^{2+} < Cu^{2+} < Al^{3+} < Fe^{3+} < Th^{4+}$

The exchangeable cations may be present in the surrounding water or gain from the stabilizer.

Typical example of cation exchange

Ca^{2+} [from stabilizer] + Na^{+} [from expansive clay] → Ca^{2+} - Clay + Na^{+}

The thickness of the diffused double layer decreases as replacing the divalent ions [Ca^{2+}] from stabilizer with monovalent ions [Na^{+}] of clay. Thus swelling potential decreases.

Flocculation and Agglomeration

This process is facilitating by reducing the high `surface of expansive soils , Cationic exchange reactions result in the flocculation and agglomeration of the soil particles with consequent reduction in the amount of clay size materials and hence the soil surface area which inevitably accounts for the reduction in plasticity. Due to change in texture, a significant reduction in the swelling of the soil occurs.

II. Pozzolanic Reaction

Time dependent pozzolanic reactions play a major role in soil stabilization, as they are responsible for the improvement in the various soil properties. Pozzolanic constitutes produces calcium silicate hydrated (CSH) and calcium Aluminate hydrated (CAH).

$Ca^{2+} + 2(OH)^{-} + SiO_2$ (clay silica) $\rightarrow$ CSH

$Ca^{2+} + 2(OH)^{-} + Al_2O_3$ (Clay Aluminum) $\rightarrow$ CAH

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

2.7 Industrial wastes as a Soil Stabilizer

Areas of Geotechnical engineering mainly focuses on the search of most economical and locally available materials which can be either agricultural or industrial wastes for the critical improving of the engineering properties of weak soils for serving as a good construction material . The applications of such waste by products become widely applicable in the construction industry which has both economic and environmental sustainable uses.

Fly ash, Steel Slag , Copure slag , marble dust, brick dust , Coconut husk ash , rice husk ash and Bagasse ash are some typical waste by products widely used in the construction industry as a soil stabilizer and other multi purposes in road construction ,concrete technology , rail way engineering and other fields by minimizing resource scarcity , environmental polusion , un wanted energy consumption .The research focuses mainly on the potential use of steel slag as soil stabilizer for expansive soils.

2.7.1 Steel Slag

Steel slag is an industrial by product obtained from the steel and Iron manufacturing industries. Based on the mode of production it can be metal production processes and slag derived from waste heat-treatment and melting. Slag derived from metal Production processes can be further categorized in to ferrous slag and non-ferrous slag. Steel- and iron making industries generate different types of slags. Blast furnace slag which is a by-product of iron making process has a high SiO_2 content and hence, rapidly-cooled blast-furnace slag has an amorphous structure and pozzolanic properties. Due to its beneficial cementitious properties, blast-furnace slag generated each year is fully utilized by the cement and concrete industry. Traditionally unutilized steel slag was stockpiled in the steel plants, and eventually land filled at slag disposal sites. Since the current methods of stockpiling and landfilling are not sustainable, disposal of steel slag has become a significant concern both to slag processer companies and to environmental agencies. Though steel slag is a waste for steel and iron industry; it can be used for different purposes in construction industry particularly in the field of geo technical engineering and Highway engineering as a soil Stabilizer and as a source of aggregate respectively. Use of steel slag for geotechnical engineering projects, such as in the construction of highway Companies and to environmental agencies becomes familiar since the last decades. Due to the presence of free lime and also the similar chemical composition and mineralogy to Portland cement in steel slage; there is a great potential for furthering the usage of steel slag in the construction industry for soil stabilization. . Steel slage is produced in large quantities during the steel-making operations which utilize Electric Arc Furnaces (EAF) and smelting iron ore in the Basic Oxygen Furnace (National Slag association 2011).

2.7.1.1 Properties of Steel Slag

Calcium and aluminum silicate are the main component of steel slag. It also usually Contains elements like sulfur(S), selenium (Se), carbon(C), cadmium (Cd), lead (Z), copper (CU) and mercury (Hg). Several of the residual elements are encased within a glassy matrix. The matrix of the steel slag is soluble and releases calcium and manganese oxides, which can raise the pH value of the dissolving liquid to certain values .The permeability of the steel slag is reduced only when it is compacted or grounded into smaller particles [Akinwumi, I., 2012].

2.7.1.2 Physical and Mechanical Properties of Steel slage

2.7.1.2.1 Physical properties of Steel slage

According to National Slag Association (2011) steel slag aggregates are fairly angular, roughly cubical pieces having flat or elongated shapes. They have rough vesicular nature with many non-interconnected cells which gives a greater surface area than smoother aggregates of equal volume. It also has a high degree of internal friction and high shear strength. The cellular shape is formed by the gases entrapped in the hot slag at the time of cooling and solidification. Steel Slag has high bulk specific gravity and usually less than 6 % water absorption, its aggregates have high density and they have high resistance to abrasion and impact.

Table2. 4Typical Physical Properties of Steel Slag [National Steel Slag, 2011]

Property	Value
Specific gravity	3.2 -3.6
Approximate dry unit weight (Kg/m3)	1600- 1920
Water absorption (%)	2-6

2.7 .1.2 .2 Mechanical Properties of Steel slage

Steel slag has favorable mechanical properties for use as aggregates in construction; these include good abrasion resistance, good soundness characteristics, and high bearing strength. The following table shows some of the mechanical properties of steel slage. Table 3 shows some of the mechanical properties of steel slage [National slag Ass. 2011].

Table2. 5 Some typical Mechanical Properties of Steel Slag [National Steel Slag Association, 2011]

Property	Value
Loss Angles abrasion	20-25
Sodium sulfate soundness loss	<12
Angle of internal friction	$40_0 - 50_0$
Hardness (measured by Moh's scale mineral 6-7 hardness)	

2.7.3.3 Chemical and Mineralogical Properties of steel slage

The primary constituents of slag are aluminum, calcium, iron, magnesium, manganese, phosphate, silicon and sulphur. The main constituents of iron and steel slags are Silica, Alumina, Calcium and Magnesia, which together makes about 95 % of the total composition. Steel slag usually, contains four major oxides, namely lime, magnesia, silica and alumina. Minor elements include sulfur, iron, manganese, alkalis and trace amount of several others (National Slag association, 2011).

The chemical composition of slag is generally expressed in terms of simple oxides calculated from elementary analysis determined from x-ray fluorescence or x-ray diffraction. The mineralogical form of slag is highly dependent on the rate of cooling of slag in the steel making process. The rate of cooling of steel slag is generally kept lower so that the crystalline compounds are formed. The predominant compounds in steel slag are dicalcium silicate, tricalcium silicate, dicalcium ferrite, merwinite, calcium aluminate, calcium magnesium iron oxides, some free lime and magnesia. Steel slag is mildly alkaline, with a solution pH generally in range of 8 -10. However, the pH of leachate from steel slag can exceed 11, a level that can be corrosive to aluminum or galvanized steel pipes placed in direct contact with the slag. The amount of free calcium and magnesium oxides is not completely consumed in steel slag. The hydration of quick lime and magnesia in contact with moisture is largely responsible for the expansive nature of most steel slag (Akinwumi, I., 2012).

The free lime hydrates very rapidly and can cause large changes in volume over a relatively short period of time and weeks, while magnesia hydrates much more slowly and contributes to long term expansion which may continue for many years.

Free lime (CaO) + water (H_2O) → Calcium Carbonates ($CaCO_3$) → Rapid process.

Magnesium oxide (MgO) + water (H_2O) → Magnesium carbonates ($MgCO_3$) → slow process

According to Isaac (as Cited in Thomas Research Services 2005), basic steel slags commonly contain significant quantities of free CaO and free MgO. These free oxides are well known for the massive expansion associated with their hydration. In practical terms, it is impossible to forecast when hydration will take place, but it can be up to decades after the material was cooled or placed. The reasons are complex, but include the varying density of the oxides, due to the variation in temperatures at which the products have been held in the furnace.

Table2. 6Typical Chemical Composition of Steel Slag (Akinwumi I., 2012 and National steel slag Asso 2011.)

Constituent	**% by Weight**	**Composition (%), by TFHRC**	**Composition (%), by NSA**
Calcium Oxide (CaO)	30.70 - 52.00	40 – 52	42
Silicon Oxide (SiO_2)	10.00 -19.00	10 -19	15
Iron Oxide (FeO)	10.00 - 40.00	10.00 – 40	24
Manganese Oxide (MnO)	3.99 - 5.00	5–8	5

Magnesium Oxide (MgO)	5.00 - 9.95	5 – 10	8
Aluminum Oxide (Al_2O_3)	1.00 - 5	1–3	1–5
Phosphorus Oxide ($P2O5$)	0.50 - 1.00	00.5 – 1	0.8
Sulfur, S		< 0.1	0.08
Chromium Oxide (Cr_2O_3)	0.00 - 0.28	--	--
Potassium Oxide (K_2O)	0.00 - 0.05	--	--
Sodium Oxide (Na_2O)	0.00 - 0.09	--	--
Titanium Oxide) (TiO_2	0.00 - 0.65	--	--
Vanadium Oxide (V_2O_5)	0.00 - 0.21	--	--

2.8 Advantages of steel slag over the other stabilizers

Steel slag has a versatile application in the field of engineering .The following are among the wide range applications of steel slage in various construction industry as a basic source of material. Applications of steel slag as cited by Irem Zeynep (2009) from different literature include

I. **Civil engineering applications:**

- Cement production
- Concrete aggregate
- Asphalt aggregate
- Road bases and sub-bases
- Soil stabilization
- It highly applicable for base and sub base of high way construction.
- Backfill of all types and slope protection
- Graded aggregate for pavements and bituminous pavements.

- Skid resistance surface
- Agricultural liming and soil conditioning
- lightweight concrete block
- Concrete floor fill.

II. Miscellaneous applications:

- Steelmaking
- Fertilizer production
- Linings for waterways
- Landfill daily covers
- Railroad ballast
- Miscellaneous environmental applications

3.5 Review on Previous Studies

Negusse, E., (2011) studied Evaluation of Sodium Silicate and its combination with cement and lime for soil stabilization on expansive soils taken from Addis Ababa Bole Senior Secondary Preparatory School and Bole-Gergi road section.

Grain size analysis, Atterberg limit test, Proctor test and California Bearing Ratio tests were used to evaluate properties of treated soils. Montmorillonite clay was treated using 2%, 4% and 6% lime, 1%, 2.5% and 6% liquid sodium silicate and the respective combinations of the additives by dry weight of the soil. Clayey sand with gravel was treated with 3%, 5% and 7% cement, 1%, 2.5% and 6% liquid sodium silicate. Samples were cured for 3 and 7 days before testing and additional 28 days of curing were considered for Atterberg limit test.

The following Conclusions were drawn from the study.

- Lime reduced the plasticity index of treated soils.
- Atterberg limit samples treated with cement resulted in non-plastic soil.
- Treating expansive soil with sodium silicate or combination of lime and sodium silicate gave odd curves from which a maximum dry density or optimum moisture content could not be determined.
- Blending clayey sand with sodium silicate or its respective combination with cement proportionally decreased the dry density of the soil.
- Sodium silicate or its respective combination with lime gave decreased strength values and increased swelling properties compared to the respective lime treatments.
- The addition of sodium silicate changes clayey sand sample in to non-plastic which led to the belief that sodium silicate decreases plasticity of soils.

- Carbonation has detrimental reversing effects on plasticity index of soils attained by stabilization.
- Mixing sodium silicate with lime is not a viable option for montmorillonite clay (expansive soil) stabilization.
- Applying 1% sodium silicate by dry weight of the soil in concert with ordinary Portland cement has positive implications on strength development of soil-cement.
- Sodium silicate has inhibiting effect on strength development of soil-cement when its quantity is increased to and beyond 2.5% by dry weight of the soil.
- Proper curing is mandatory for strength development of cementitious stabilizers. Curing also enhances strength of soils treated with sodium silicate.
- Varying the mode of mixing does not bring a change in dry density of expansive clay treated with sodium silicate; however, sodium silicate should not be directly applied to cementitious stabilizers as it forms hydration products before the soil can be engaged in the reaction.

Wubshet, M. (2014) investigated Bagasse Ash as Subgrade Soil Stabilizing material on black cotton soils from Addis Ababa Bole Medhane Alem. Index properties ,moisture density relationship , particle size distribution , specific gravity ,Free swell , Free swell ratio , Free swell ratio ,CBR and CBR swell of the proposed soil were determined .

The findings and conclusions drawn from the study are;

- As the amount of bagasse increases plasticity index slightly reduces and curing has no effect on plasticity.
- As the bagasse ash content increases maximum dry density decreases and the optimum moisture content increases.
- Free swell, free swell index, free swell ratio decreases with increase in Bagasse ash content.
- CBR value slightly increases with addition of bagasse ash, curing has no effect on CBR values.
- Sodium silicate is not a suitable additive for montmorillonite clay (expansive soil) stabilization.

- The plasticity index reduces with the addition of both bagasse ash and lime and increases with curing durations.
- The addition of lime and bagasse ash together reduces more MDD and increases OMC than separately blending; the combination of the two admixtures improves CBR and has a great effect on strength as curing duration increases.
- Bagasse ash alone does not contribute as a soil improvement.

Argu, Y. (2008) studied Stabilization of light grey and red clay Subgrade Soil using Sa-44/ls-40 chemical and lime on expansive soils collected from Addis Ababa by conducting different laboratory tests like soaked CBR and CBR swell tests, swelling pressure test, Atterberg limit tests, percent swell test and free swell tests, in which the modifications in engineering properties were evaluated after curing the treated sample for fourteen days.

Conclusions and Findings drawn from the study are:

- The applications of SA-44/LS-40 chemical alone results in promising reduction in the swelling Pressure of the light grey clay soil
- However, it is ineffective in improving the soaked CBR value of this soil.
- In addition the applications of lime alone results in an increase in the soaked CBR value and reduction in the swelling pressure of the light grey clay soil.
- Furthermore the applications of the chemical with lime results in even more increase in the soaked CBR value and reduction in the swelling pressure of the light grey clay soil.

Solomon, H. (2011) investigated Chemical Stabilization of Expansive Subgrade Soils Performed evaluation on Road section from North East Parts of Addis Ababa.

The expansive soil was characterized based on Atterberg limits, linear shrinkage, CBR and percent swell of CBR.

Test results and applicability of the stabilization for the study were summarized as follows.

- The addition of 2% of either of the chemical additives; hydrated lime or ANSS, does not assure the improvement of the sub-grade soil to the desired engineering properties.
- The addition of 4% of either of the chemical additives; hydrated lime or ANSS, has sufficiently stabilized the sub-grade soil. With further increment of the curing period the sub-grade soil has further improved
- While addition of either of the two additives dark gray soil is improved than grey clay soil.
- The performance of hydrated lime is higher than ANSS for stabilization.
- As the curing duration increases cracks were visible due to moisture lose as curing duration increases.

Isaac Ibukun Akinwumi **(2012)** studied utilization of steel slage for stabilization of Lateritic soils found in Nigeria. X-ray fluorescence spectroscopy, index properties, compaction characteristics (maximum dry density, MDD and optimum moisture content, OMC), strength characteristics (California bearing ratio, CBR and unconfined compressive strength, UCS) and permeability of the natural and treated soil were evaluated for lateritic soil of the given area.

Table 2.7 XRF Result of Steel Slag Sample

Element	Concentration value unit	
K_2O	2.3682	Wt.%
CaO	7.7254	Wt.%
TiO	1.8562	Wt.%
MnO	15.517	Wt.%
Fe	91.968	Wt.%
V	997	ppm
Cr	2.0811	Wt.%
Ni	341	Ppm
Cu	1086	Ppm
Zn	4986	Ppm

Zr	757	Ppm
Zb	388	ppm

As steel slag serves as soil stabilizer due to its self-cementing properties and similar chemical composition and mineralogy to Portland cement, comparison between steel slag and Portland cement about their chemical composition was also investigated (Table 2.8)

Table2. 8 Comparison of chemical Composition of Steel Slag and Portland cement

Constituent	Steel slag (%)	Portland cement (%)
SiO	13.8	22
CaO	44.3	64.2
Al_2O_3	1.5	5.5
T-Fe	17.5	3
MgO	6.4	1.5
S	0.07	2
MnO	5.3	-------
TiO	1.5	------

Generally, a finding of the study indicates improved of the engineering properties of the treated soil and a decrease in index properties as the addition of steel slag additive increases.

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3. MATERIALS AND METHODS

In this section description and preparation of materials used for the research and testing procedures are presented. Soil tests were done in the Geotechnical Engineering and Highway Engineering laboratories of Addis Ababa University except CEC which were conducted in Water works and Design Bureau of FDRE.

3.1 Materials Used for the study

3.1.1 Expansive Soil

Samples of soil which represent expansive soils for the study were collected from Addis Ababa Bole Arabsa, project five. From the specified area disturbed soil samples were collected at a depth not less than 1.5 meter from the ground after the removal 50cm of the top soil. The soil sample was stored and kept dry in sack in the Geotechnical and highway engineering laboratories of Addis Ababa University. The collected sample was sieved through different sieve size openings which are standardized for fine soils to performing the desired laboratory test of the mixed material to get an appropriate batching.

3.1.2 Steel Slag

In addition, steel the steel slag stabilizer for the study were taken from Hibret Machine Building Factory and Akaki Basic Metallic Industry which had been exposed to weather for about four Months and more as shown in figure 3.1, which was previously disposed and stock pilled without any use.

Figure3. 1 Steel slag samples Collected from Hibret Machine Factory and ABMI.

3.1.2.1 Availability of Steel Slag

The availability of steel slag depends on the gross production amount of Steel and Iron productions. The iron and steel industry involves a myriad of operations which generate vast volumes of air emissions, liquid effluents and solid wastes. Significant quantities of sludge and slag are generated as waste material or byproduct every day from steel industries. They usually contain considerable quantities of valuable metals and materials. It is generally possible to recover some values by physical or chemical mineral processing techniques. Transforming these solid wastes from one form to another to be reused either by the same production unit or by different industrial installation is very much essential not only for conserving metals and mineral resources but also for protecting the environment (Akinwumi, I., 2012).

In the steelmaking process, slag is formed during the addition of limestone, dolomite or lime to the molten iron ore. These calcium compounds mixed with aluminum, silica, phosphorus and other impurities in the iron to form slag. What results is a much stronger steel product and a pile of amorphous, glass-like, calcium alumino-silicate oxides, more commonly known as slag. The slag floats to the top of the melt and is poured off into piles for disposal. Due to its extremely high temperature, approximately 2700 degrees Fahrenheit, it cools almost immediately. Molten pig iron and slag are produced as a result of the processing of iron ore combined with a flux material (limestone or dolomite). Steel slag is the by-product of the steel-making process, which makes use of either a basic-oxygen furnace or an electric-arc furnace (National Slag Association, 2011).

As cited by Akinwumi, I., (2012), in van Oss (2003), approximately 10 to 12 million metric tons of blast furnace slag are produced each year in the US. Globally, this number jumps to 150 to 180 million metric tons. Similarly, it is estimated that 9 to 14 million metric tons of steel slag are produced each year in the US, with the worldwide number ranging from 90 to 135 million metric tons per year. Besides, around 15 million tons of steel slag is produced annually in Europe, resulting from different types of steel technology. In order to assess the potential of steel slage production in Ethiopia, it is important to evaluate the steel and iron yield factories in the country. In Ethiopia about 1 to 2.16 million tons of steel is produced annually, from which 0.1 to 0 .216 million tons of steel slag is generated annually.

3.2 sample preparation

Air dried pulverized and disturbed expansive soil samples were prepared for the experimental work. For the safety of conducting the required laboratory test, the prepared disturbed expansive soil samples were sieved through the required particle size according to ASTM [as shown in table 3.1] test manual. Classification of soil was determined by running index property and grain size analysis tests. The following particularities were considered with respect to the additives and the mixing ratios used.

I. Expansive clay was treated by varying steel slag additives until the optimum stabilizer is achieved. Accordingly, the expansive soil was stabilized through the proportions of steel slag as 0%, 3%, 5%, 7% and 9% of Steel slag by dry weight of the soil.

II. Curing durations were varied at 0, 3 and 7 days for Atterberg limit tests, UCS and CBR samples.

Improvements in the index properties of the soil were assessed using Atterberg limit test, free swell and specific gravity tests. Strength of the stabilized soil was studied using Unconfined Compressive strength, California Bearing Ratio (CBR) and moisture density relation tests.

3.3 Standard Testing Methods

For the different tests conducted in this study, both ASTM and AASHTO standards were used for testing soils in the laboratory. Specific test manual standards considered in the study are summarized in table 3.1.

Table3. 1 Standard Test Methods

Test No.	Test Type	Test Description	ASTM Standard Designation	AASHTO Standard Designation
1	Natural Moisture content	Standard test method for the determination of natural moisture content of a given soil.	D2216-98	T 265-93
2	CEC and PH	Mineralogical type and pH of the natural soil respectively.	D6276-99a and D4972-01	

3	Atterberg Limit Tests	Standard Test Methods for Liquid Limit, Plastic Limit and Plasticity Index of Soils carried out for classification purpose.	D4318-00	T 89-96, T90-96
4	Standard Compaction test	Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort for the determination of MDD and OMC.	D698-00a	T 99-95
5	Modified Compaction Test	Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort for the determination of MDD and OMC	D 1557-0	T 180-95
6	Specific gravity Test	Standard test method used for the determination of specific gravity of a given material	D854-00	T 100-95
7	Particle size Analysis test	Standard Test Method for Particle-Size Analysis of Soils both for coarse and fine particles.	D422-63	T-88-93 (1996)
8	Hydrometer Test	Standard Test Method for Particle-Size Analysis of Soils only for fine particles.	D422-63	
9	CBR	Standard Test Method for the California Bearing Ratio for both	D-1883-99	T 193-93

		soaked and un soaked cases.		
10	UCS	Standard Test methods for the determination of unconfined strength (qu) and cohesion (c) of cohesive soils.	D -2166-00	T 208-92
11	One dimensional Consolidation	Standard test method used for the determination of void ratio, CC, Swelling pressure and Settlement of cohesive soils.	D-2435-02	T 216-94

3.3.1 Cationic Exchange Capacity (CEC) and PH meter Tests

CEC is the quantity of exchangeable cations required to balance the negative charges on the surface of the clay particles expressed in milliequivalent per 100 grams of dry clay. This test was performed for soil samples passing 0.6 mm sieve size for the determination and identification of mineralogical type of the expansive soil. Accordingly, the types of clay mineral were deduced as Montmorillonite. The detailed data and test results of CEC are presented in the annex part.

3.3.2 Specific Gravity test

The specific gravity determination of a sample of soil both treated and untreated was made by displacement in water using pycnometer (volumetric bottle). Specific gravity for both the slag and the expansive soil were determined through ASTM standard test method. Representative of disturbed sample was conducted for both the natural soil and the stabilized soil. The test was conducted based on ASTM D 854-00 by placing 25 gram weight of oven dried soil sample carefully put in a pycnometer which is then half filled with distilled water. The air entrapped in the soil sample was removed by means of vacuum pumps for about ten minutes. The bottle was then topped up with distilled water up to the calibration mark and brought up to a constant temperature. After carefully wiped and dried, it was weighed in a balance.

3.3.3 Grain size Analysis Test

Grain size analysis of the natural soil was carried out through two stages:

3.3.3.1 Sieve analysis for coarse grained particles [wet sieve analysis]

This test was conducted in accordance with ASTM D 422-63 test method. This test was carried out by sieving a 1000 gram weight of air dried soil sample through the set of sieves placed one below the other after washing by water. The washed air dried soil sample and retained on sieve size 0.075 mm was dried in oven dried for 24 hours at 110 ^{o}C. After dried the sample was sieved so that the openings decrease in size from the top sieve down, with a pan at the bottom of the stock after oven dried of the sample. The whole set of sieves were given a horizontal shaking for about ten minutes until the weight of soil remaining on each sieve reaches a constant value.

3.3.3.2 Hydrometer analysis for fine grained particles

Soil particle sizes smaller than 0.075 mm (passing 200 mesh sieves) was determined by hydrometer test method. It is based on the process of sedimentation of soil particles in water by gravity. Coarse soil suspension settles out more rapidly than the finer ones of the same specific gravity. Hydrometer analysis was conducted to know the silt and clay fractions based on ASTM test standard. 50 gm of soil passing sieve No. 200 were agitated with water and dispersing agent (Sodium hexametaphosphate) in 1000 ml jar.

Readings were taken at intervals of 0.75, 1, 2, 4, 8, 15, 60, 120,240, and 1440 minutes with the hydrometer remaining in the suspension all the time and the corresponding temperature of the suspension were also recorded with thermometer. After the twenty four hour reading all the reading data were compiled and determined different sieve sizes for the different hydrometer reading values. Then after, the combined of wet sieve and hydrometer tests were merged by determine the combined percent finer and plotted these percent finer versus Particle sizes. The shape of the plotted curve indicates the gradation of the given tested soil.

3.3.4 Atterberg Limit Tests

The following Atterberg limit tests were conducted both for the natural and the treated soils. In order to understand the effect of Steel Slag on the natural soil property, five samples were tested (starting from 0% SS to 9% SS) by weight of the dry soil. These tests (LL, PL and PI) were conducted for both Curing and uncuring cases for the stabilized soil. All the samples were tested According to ASTM standard which pass the number 40 mm Sive through a Casagrande apparatus.

3.3.4.1 Liquid Limit test

Liquid limit values were determined for a portion of soil finer than the No.40 sieve (ASTM). 200 gm of soil were prepared and mixed thoroughly with distilled water into a uniform paste. A portion of the paste were placed in the cup and leveled. A groove was cut at the center of the soil paste using the standard grooving tool. Four test trials had been carried out for each proportion of both the natural and the stabilized soil by adjusting the water contents in such a way that the number of blows required to close the groove fall within the range of 15 to 35 blows.

The water content of the soil taken near the closed groove was found out the water content of the soil and the cup was altered. The test trials were repeated. A plot of water content against the log of blows was made and the water content at 25 numbers of blows was determined as the liquid limit.

3.3.4.2 Plastic Limit test

15 gm of soil passing through sieve No. 40 was mixed thoroughly with water. The soil was rolled on a flat glass plate with hand, until it becomes 3 mm in diameter. This procedure of mixing and rolling was repeated until the soil shows signs of crumbling and the diameter becomes 3 mm. The water content of the crumbled portion of the thread was determined as plastic limit.

3.3.5 Moisture density relation

This test was carried out by standard compaction method for improving the engineering properties of soils by reducing the air void and by packing the particles together. Six pulverized samples for each of the treated soil mixed (3%SS, 5%SS, 7%SS and 9%SS) as well as seven samples for the un treated soils passing number 4 mm sieve with 1500gram for each sample were mixed with different amount of water starting with 8% according to ASTM test standard method until the final weight falls down for the determination of moisture density relation. From this test both MDD and OMC for both treated and untreated soil was determined by plotting a graph that shows a relation between these two.

3.3.6 Free Swell Test

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. Free swell test indicates the fair approximation of degree of expansiveness a given soil sample. The test was performed by pouring 10ml of dry soil passing a sieve size 0.425mm (No. 40) into a 100ml graduated jar filled with water. An oven dried soil sample was prepared and filled in to a small tube with initial volume of 10 ml and filled with water the bottle up to the upper mark and take the reading after 24 Hours. Then the swelled volume of the soil after the material settles with in twenty four hours was measured.

3.3.7 Unconfined Compressive Strength (UCS) Test

In this test, a cylindrical cohesive specimen without any lateral support was subjected to axial loading till the sample fails either due to shear along a diagonal plane or by the lateral bulging through an undrained (quick) test.This value of σ_1 (major principal stress) at failure commonly called unconfined compressive strength designated as **qu** was determined from each test result which is the peak value [the point at which the soil fails in shear or diagonal].

The Unconfined compressive strength test samples were prepared with optimum moisture content and maximum dry density from standard proctor test. The prepared sample was cured for 0days, 3days and 7days through impermeable hard plastic in desiccator which maintains moisture at equilibrium.

The test was conducted on sample having a dimension of 38 mm diameter and 76 mm height. Unconfined compressive strength testing was performed on all extracted specimens using a strain rate of approximately one percent per minute. A data acquisition system was used to record the applied load and deformation. Corrections to the cross-sectional area were applied prior to calculating the compressive stress on the specimen. Each specimen was loaded until peak stress was obtained. The following calculations were made prior to determination of qu. Throughout the computation 0.00142 N/div load factor and 0.01mm deformation factor was used, and from each test results the ultimate or maximum value was taken as the unconfined compressive strength for that specific test result.

3.3.8 CBR Test

This test was conducted according to ASTM D 1883-99 for prediction of the bearing resistance of a soil to an external load or stress applied on the soil. While conducting the test 6 kg pulverized and air dried for both treated and untreated soil samples was prepared and mixed with OMC obtained from standard compaction test result. Soaked and unsoaked as well as curing and uncuring CBR tests were conducted. CBR curing test was conducted by covering an impervious hard plastic placing at room temperature for three and seven days. Soaked CBR test were conducted by placing on water bath for 96 hours and by placing 4.5 surcharge loads on the top of the compacted soils. After 96 hours soak by removing the CBR mould from the water bath, penetration of the soaked soil on a load of 28 KN plungers with a rate of 1.27 mm/min speed were performed and determined the CBR values at 2.54 mm and 5.08 mm penetration by dividing the calculated stress to the standard stress on these two penetration points. Standard stresses for 5.08 mm penetration and for 2.54mm penetration are 6.9 and 10.3 N/mm^2 respectively.

CBR swell was measured by placing a tripod on the CBR mould during the initial soaking date and finally when removing from the water bath after 4 days for both natural and treated soil samples.

4. TEST RESULTS AND DISCUSIONS

In this chapter results of the different laboratory tests for treated and untreated soils, engineering application of the different tests, classification of the natural soil based on test results and their analysis are briefly discussed.

4.1 Test Results for Natural Soil

All the Index and engineering properties for the untreated soil samples are summarized in table 4.1.

Table4. 1 Results of index and Engineering properties of natural expansive soil.

Summary of un treated soil sample Test Results	
Location : Bole Arabsa	
Tested by: Tesfay Kiross	
Material Description: Expansive Clay	
1. Index properties	
1.1 Natural moisture content (%)	42.6
1.2 Specific gravity	2.7
1.3 Liquid limit (%)	119
1.4 Plastic limit (%)	67
1.5 Plasticity index (%)	52
1.6 AASHTO Soil classification system	A -7-6 (20)
1.7 USC soil Classification system	CH
2. Soil fraction sizes (%)	
2.1 Gravel (%)	0
2.2 Sand (%)	14.86
2.3 Silt (%)	31.14
2.4 Clay (%)	54
Optimum moisture content (%)	42
3. Engineering properties	
3.1 Maximum dry density ,MDD (g/cc)	1.2
3.2 UCS (kPa)	60

3.3 Un soaked CBR (%)	12.3
3.4 Soaked CBR (%)	1.13
CEC	74.38
Soil PH	8.38

4.2 Gradation Test Results

The grain size distribution of the expansive soil sample is shown in figure 4.1. The combined wet Sive and hydrometer test curve indicates more than 50% expansive clay soil fractions with no gravel fractions. The curve and the amount of clay contents satisfy the requirement for fine grained soils according to ASHTO for fine particle specifications.

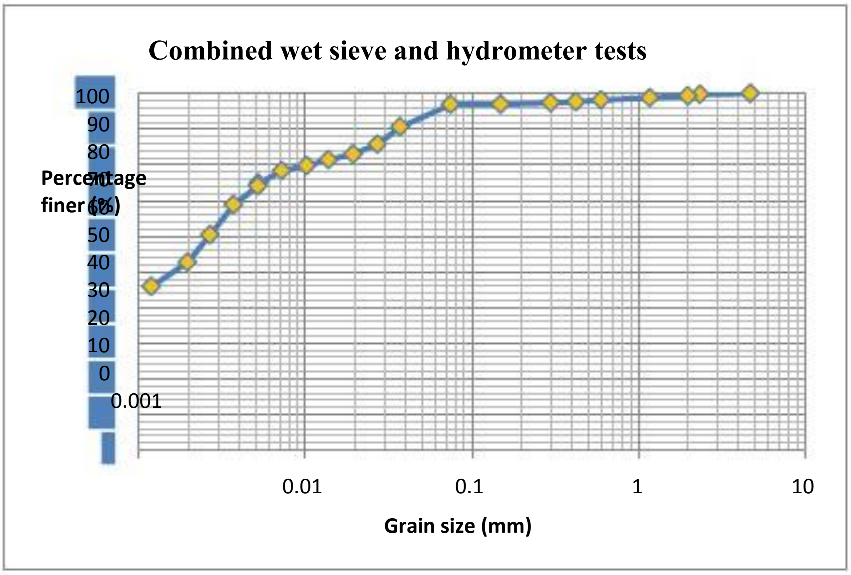

Figure4. 1 Combined wet Sive and Hydrometer Test Results

4.2.1 Soil Classification

The soil was classified based on Atterberg limit test results and grain size distribution curves through AASHTO and USC soil classification systems. Accordingly, the classification of the soil through these methods is summarized in the table.

Table4. 2 Summery of soil classification for the natural soil.

LL	PL	PI	USCS Classification	AASHTO Classification
119	67	52	CH	A-7-6 [20]

4.3 Effect of steel slag on Specific gravity

Variation of Specific gravity test result for both treated and untreated soil samples with steel slag is shown in figure 4.2.

The figure shows the average specific gravity of the expansive soil increase from 2.7 to 3.09 with increase of steel slag content. This increment in specific gravity for the treated soil is due to the existence of high amount of iron content in the slag and high specific gravity than the finer expansive soil.

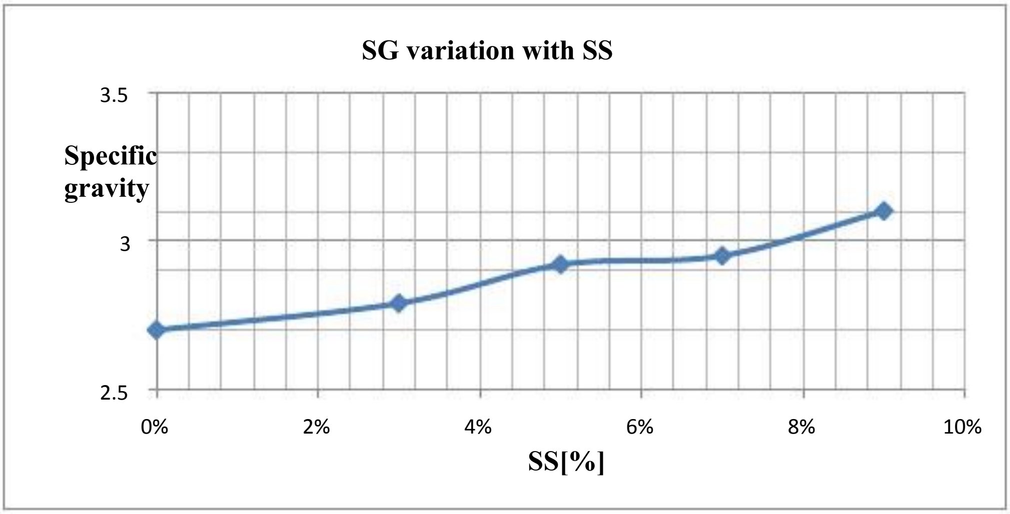

Figure4. 2 Variation of Specific gravity with steel slag.

4.4 Effect of Steel Slag on Atterberg limit Tests

This test was conducted for the determination of liquid limit, plastic limit and plasticity index for both the untreated and treated soil samples as well as curing variations for three and seven days .The test results show that as the percentage of SS (Steel –Slag) stabilizer increases the Atterberg limit values slightly reduce to a certain degree. However, the curing duration has an insignificant effect on the Atterberg limit test results.

4.4.1 Effects of Steel Slag on Liquid Limit

The variation of liquid limit with steel slag without curing and curing for three and seven days is presented in figures 4.3 and 4.4 respectively. In all the different trial tests conducted for liquid limit determination, as the amount of steel salg increases the corresponding liquid limit slightly decreases. The decrement in a liquid limit is due to the self-cementing property of the stabilizer which reduces the high surface area of the expansive clay through compression of diffused double layer by flocculation. However, the variation of liquid limit has an insignificant effect up on the curing durations as shown in figure 4.4.

Laboratory test results of liquid limit 3%, 5%, 7% and 9% steel slag treated and untreated soil samples are presented in the annex part.

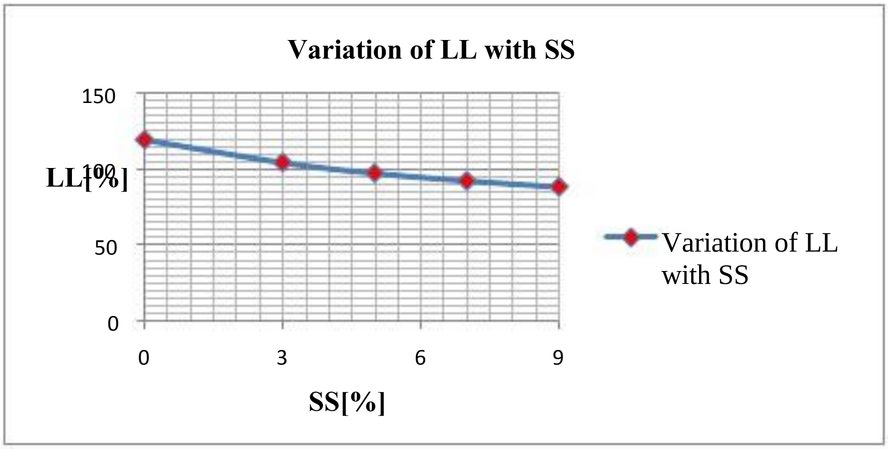

Figure4. 3 Variation of Liquid limit with steel slag without curing.

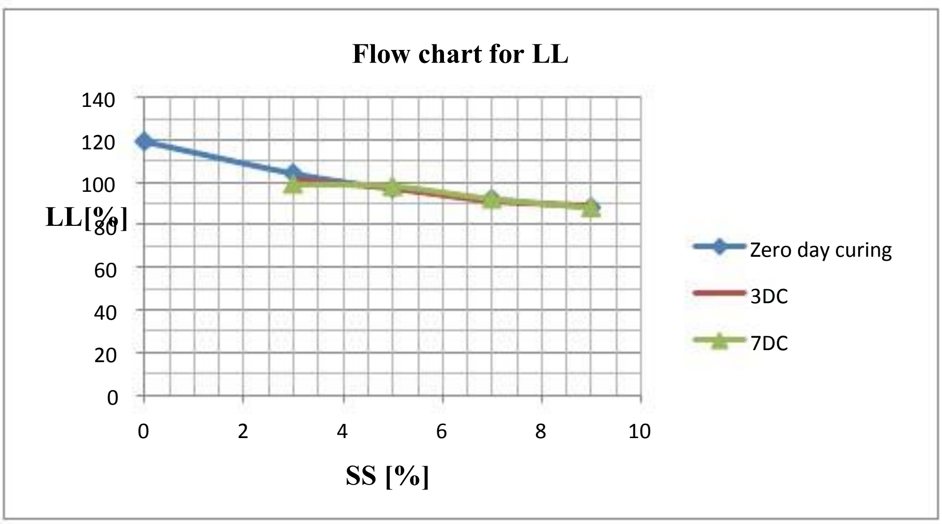

Figure4. 4 Variation of Liquid limit with steel slag and curing

4.4.2 Effects of Steel Slag on Plastic Limit

Test results show that as the amount of steel slage stabilizer increases the plastic limit slightly decreases. The reduction in plastic limit for the treated soil as in the case of the increase in steel slag is due to the self-cementing property of the stabilizer which reduces the high surface area of the expansive clay through compression of diffused double layer by flocculation. Details of plastic limit of both the treated and natural soil is presented in the figure 4.5.

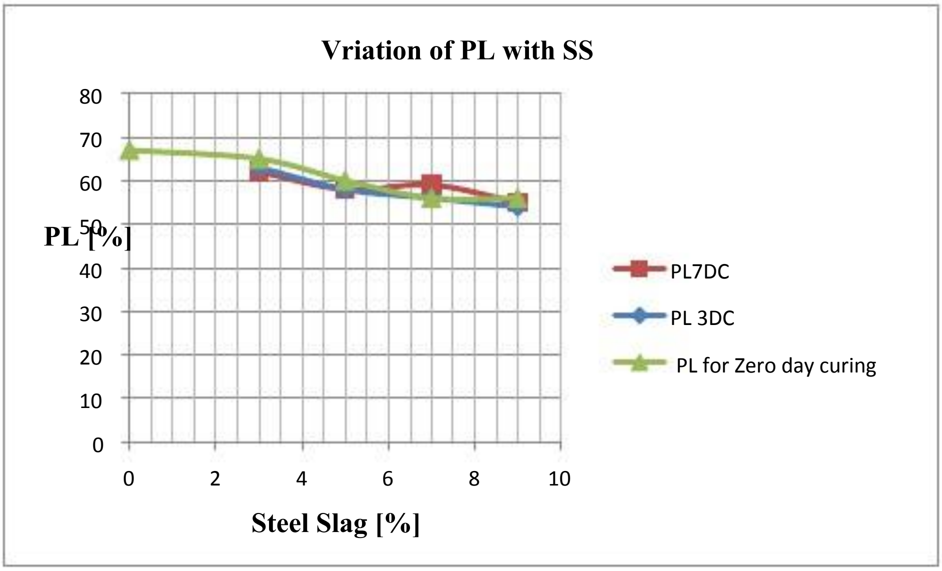

Figure4. 5 Variation of Plastic limit with steel slag and curing.

4.4.3. Effects of Steel Slag on Plasticity Index

Simply this test is the difference of liquid limit and plastic limit test result values.

Plasticity Index results for a given soil shows that;

- The range of water content over which the soil exhibits plasticity.
- The presence of clay in a given soil
- In general, a high numerical value of plasticity index means a high percentage of clay fractions.

Figure 4.6 shows the results obtained from the Atterberg limit tests of the PI for both the natural and treated soil. The PI of the treated soil slightly decreases as the amount of steel slage increases.

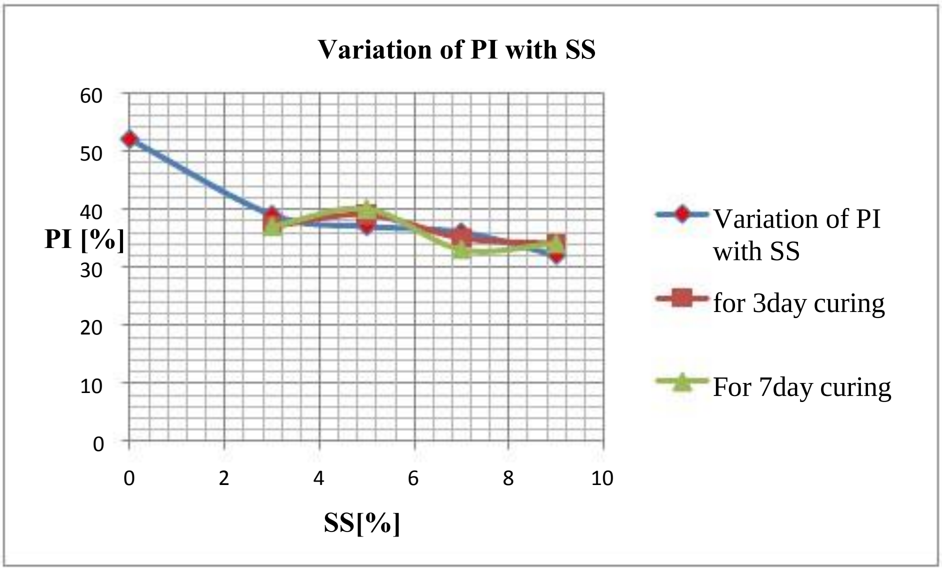

Figure4. 6 Variation of PI with Steel slag and curing

4.5 Effect of Steel Slag on Free swell

Free swell test conducted with cylindrical tube without any surcharge load measures the degree of swell for a particular soil. Free swell for the treated soil slightly decreases as the amount of steel slage required to stabilize the soil increases. The reduction of free swell index falls in the marginal range for degree of expansiveness.

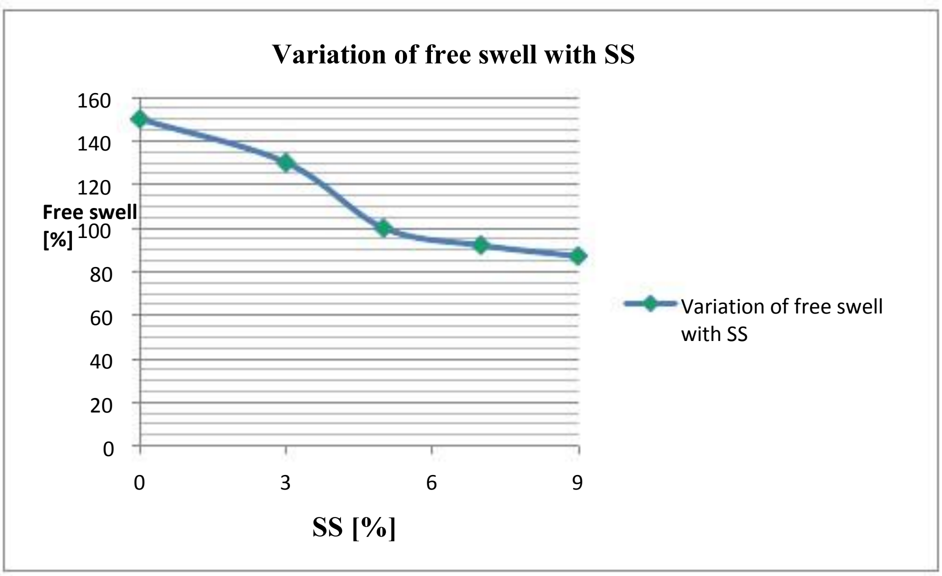

Figure4. 7 Variation of Free swell with steel slag

4.6 Effect of Steel Slag on engineering properties

Engineering properties are properties that determine the strength value of a given material. Test results show that all the engineering properties of the stabilized soil increases as the percentage of steel slag increases until the optimum and slightly decreases beyond the optimum of the stabilizer. This increase in strength of the treated soil is due to the self-cementing properties of the high strength stabilizer which facilitates the chemical reaction through agglomeration and flocculation. Beyond the optimum the decrease value indicates the agglomeration of the steel slag occupies the soil particles and become difficult to resist the stabilizer.

4.6.1 Effect of steel slag on Moisture Density Relation

As the amount of steel slag stabilizer increases the corresponding dry density of the stabilized soil also increases until the optimum amount of the stabilizer, beyond the optimum it slightly decreases. This increase in dry density indicates that the soil with loss or low density mixed with slag of high density due to self – cementing of the slag resulting in density improvement. In contrast the optimum moisture content falls as the amount of steel slag increase which shows that as the particles of the slag are slightly larger than that of the fine soil, these larger particles have less water absorption and requires less water for complete the whole reaction. Results of the different test trials for MDD and OMC obtained from moisture density relation are shown in figures 4.8 and 4.9 respectively.

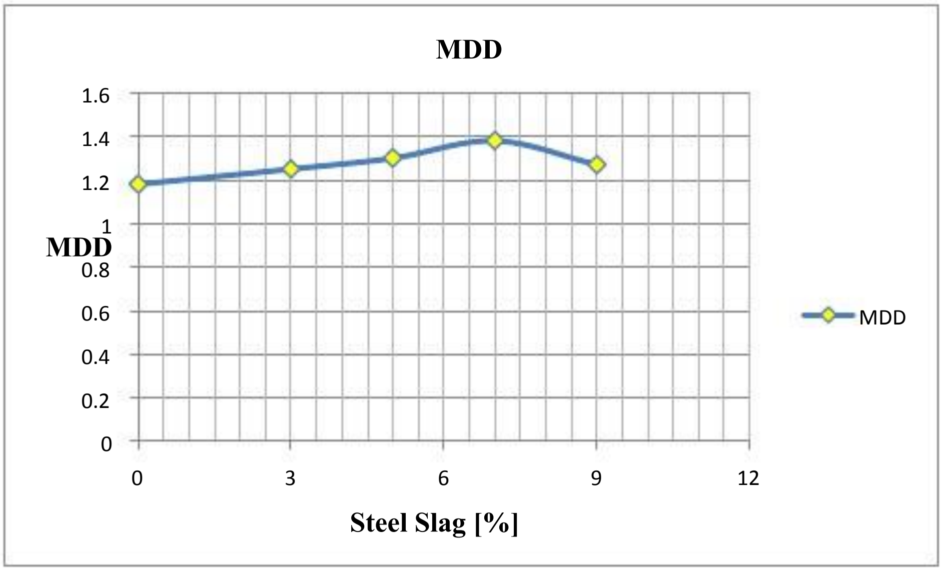

Figure4. 8 Variation of MDD with Steel Slag

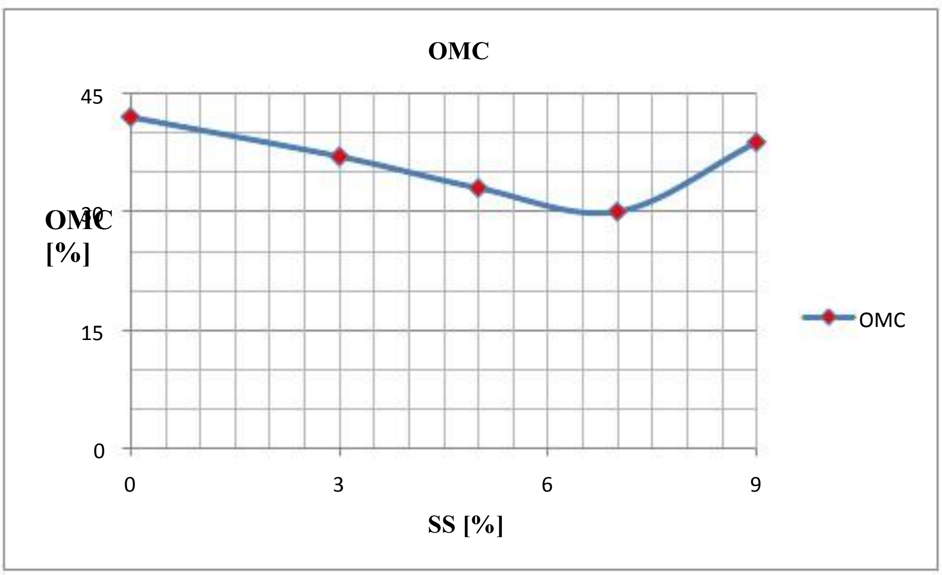

Figure4. 9 Variation of OMC with Steel Slag

4.6.2 Effects of Steel Slag on Unconfined Compressive Strength (UCS)

The variation in unconfined compressive strength with steel salg for both curing and uncuring cases can be easily seen in figures 4.10, 4.11, 4.12 and 4.13. The unconfined compressive strength of the treated soil increases with increase of steel slag. The increment of the unconfined compressive strength as the steel slag required increases is due to the long term effect as well as due to the effect of secondary phases resulted by CSH (Calcium silicate hydrated) and CAH (Calcium aluminate hydrated) which resulted in agglomeration and flocculation upon clay particles. The subsequent reduction in the unconfined compressive strength beyond the optimum of steel slag stabilizer content also suggests that the limit of influence of the solidification effect is reached. All the laboratory test results of the unconfined compressive strength test for treated and untreated as well as curing and uncuring are presented in the annex part.

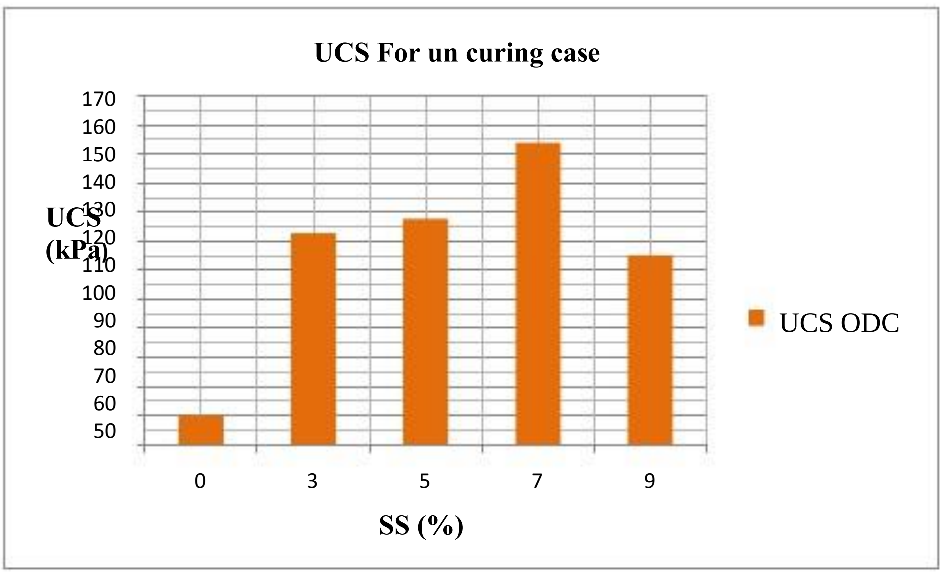

Figure4. 10 Variation of UCS with Steel for zero day curing.

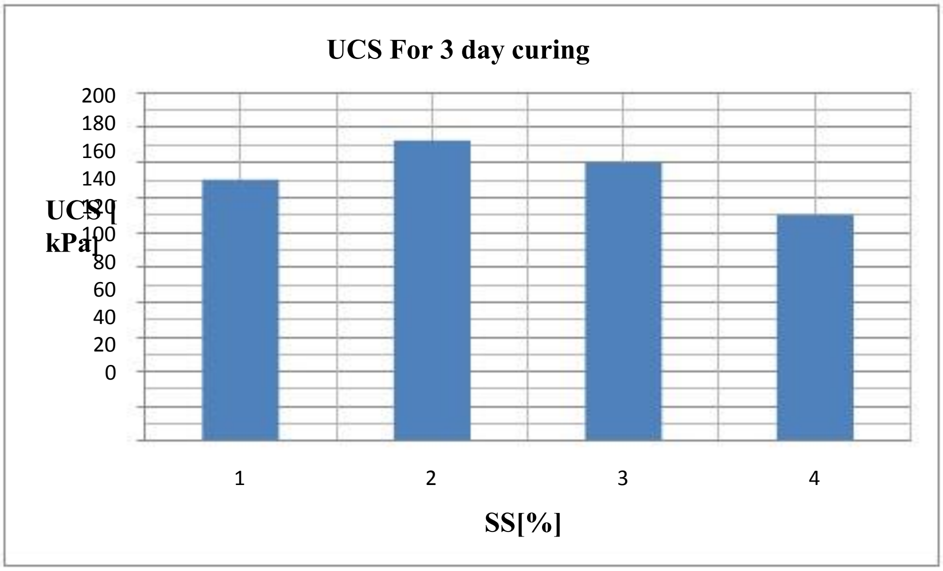

Figure4. 11 Variation of UCS with Steel Slag for 3DC

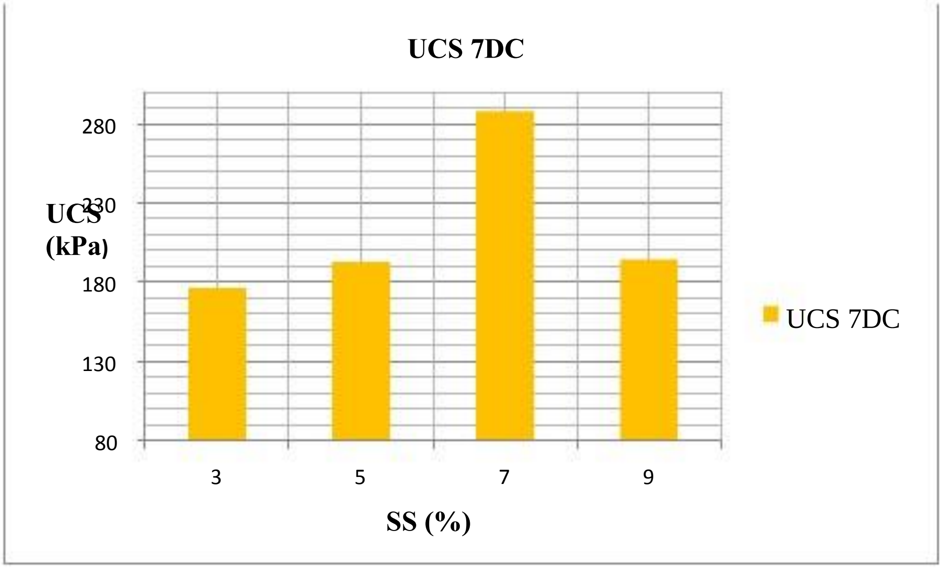

Figure4. 12 Variation of UCS variation with Steel Slag for 7DC

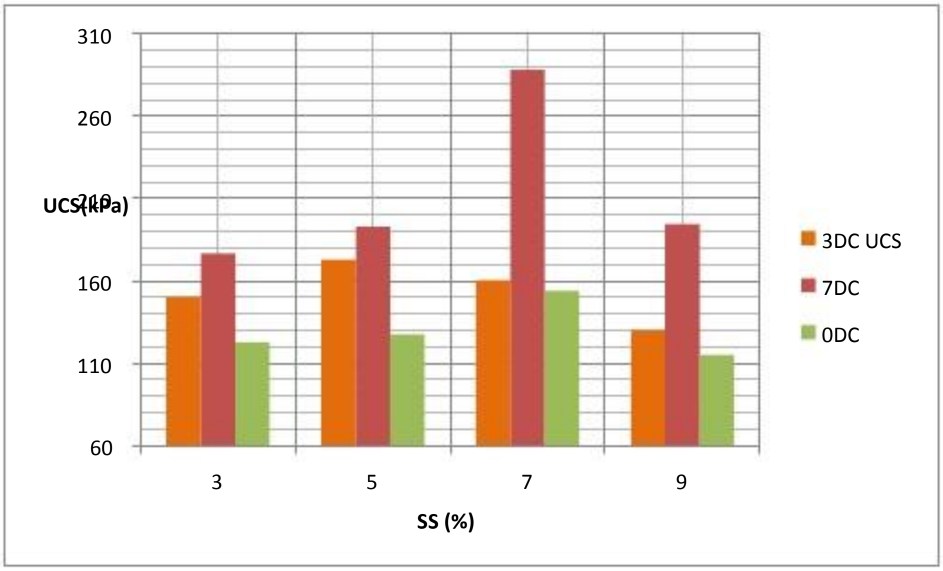

Figure4. 13 Variation of UCS with Steel Slag both curing and uncuring

4.6.3 Effects of Steel Slag on CBR

CBR Value for both the treated and untreated soil increases for unsoaked and soaked cases. CBR variation value with slag content and variation in curing days are presented in Figures 4.14 and 4.15. Figure 4.14 indicates that the unsoaked CBR value for the soil-slag mixture progressively increased from 12.3 % for the 0 % steel slag content and from 40 % to 50% for uncuring and curing durations for 7 % slag addition and decrease to 38.35 % and 44.73% for 9 % steel slag content for uncuring and curing cases respectively. Similarly, the change in soaked CBR value and swell potential with slag content are also presented annex. The significant increment of the unsoaked curing CBR value from 12.3 % for 0 % slag content to 40% for 7 % slag content is due to the effect of secondary phases resulted by CSH (Calcium silicate hydrated) and CAH (Calcium aluminate hydrated) which resulted in agglomeration and flocculation upon clay particles.

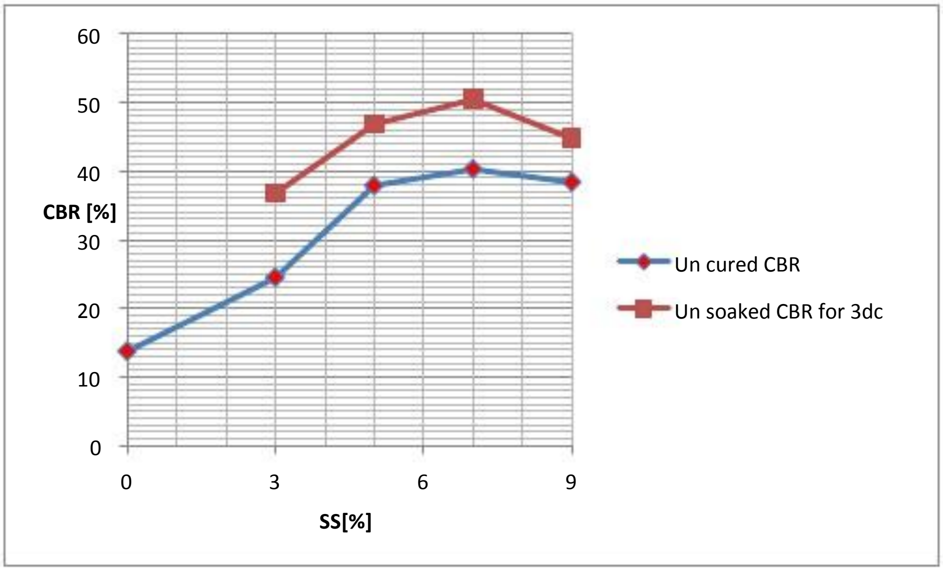

Figure 4. 14Variation of Unsoaked CBR with SS

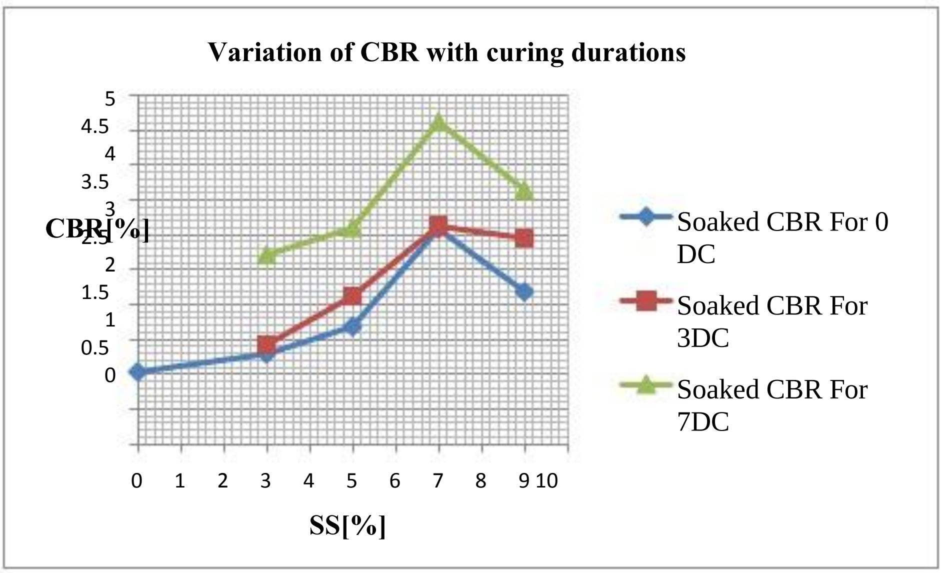

Figure4. 15Variation of Soaked CBR with SS

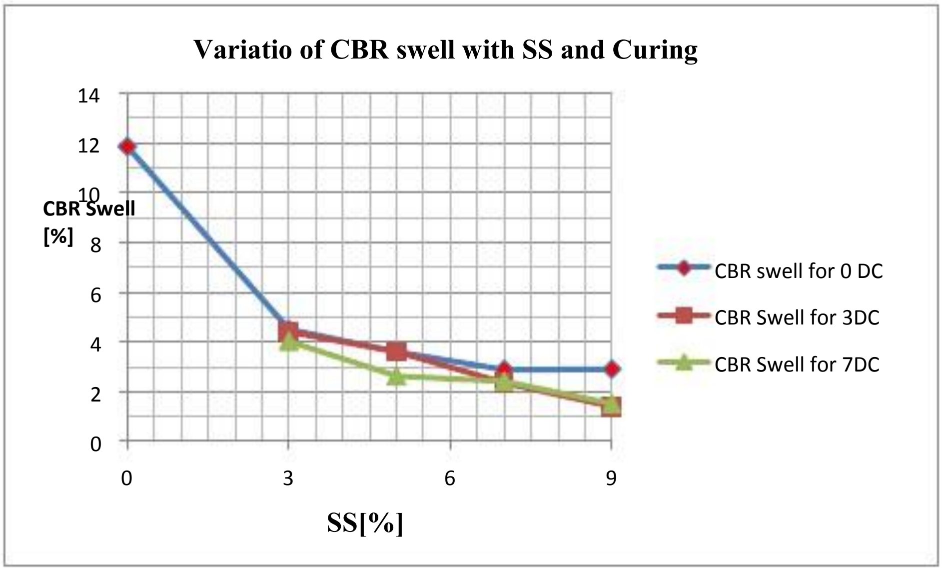

Figure4. 16Variation of CBR swell with SS

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The following conclusions can be drawn from the results and investigations from the proposed study.

1. The optimum steel slag required for stabilizing the expansive soil was found 7% by dry weight of the dry soil from engineering property test results of the treated soil [UCS and CBR].
2. As the amount of steel slag increases the Maximum dry density increases up to the optimum and the optimum moisture content decreases.
3. Unsoaked and soaked CBR values increase as the amount of steel slag increase up to the optimum both for curing and uncuring cases.
4. The unconfined comprehensive strength value (UCS) both for curing and un curing cases increases as the amount of steel slag increases up to the optimum.
5. UCS value increases further with the curing duration as the amount of steel slag increases up to the optimum.
6. Liquid limit, plastic limit and plasticity index all slightly decrease as the percentage of steel slag increases.
7. Specific gravity of treated soil increase with increase in steel slag.
8. Curing has insignificant effect on Atterberg limits, but has a significant effect on engineering properties indicating the strength of steel slag stabilized soil attains its strength with curing.
9. Therefore 7% optimum stabilization of the A-7-6 improves the engineering properties of the natural soil and slightly reduces the Atterberg limits as well as the free swell index.

5.2 RECOMMENDATIONS

- In view of the property of steel slag, 7 % is optimal stabilization for the A-7-6 [20] soil in this study.
- According to ERA specification to use steel slag as sub grade material, it is best if primary activators are added for further reduction of plasticity index to meet the specification requirement (should be less than 30%) and free swell index (should be less than 50%).
- Further investigation should be conducted to determine the effect of adding steel slag on some other expansive soils.
- The effect of the environmental impacts of steel slag application and weather condition on the treated soils was not taken in to account while undertaking the investigation. It may be helpful to study the effect of different weather conditions and environmental effects on the response of the soils.
- The results of the investigations were obtained based on immediate compaction after treating the soil. The effect of delay between mixing and compaction of the soils should be investigated.
- The study was performed only for one sample collected from one area, it is recommended to conduct other study on different soil samples varied with depth.
- The detail engineering properties of the stabilizer should be well investigated as one main research detailed for further applications geo material.
- Properties of blending with other materials should also be investigated for detailed information about the degree of effects of the chemicals up on stabilization.
- The other methods of blending methods of stabilizer and the soil to be stabilized like matrix method should be investigated.
- Finally, the results and findings of the present study should be considered as indicative only as these findings were based on limited parameters and on small number of samples. It is also recommended if more elaborate sampling and testing would be performed.

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ANNEX: LABORATORY TEST RESULTS

Table 1 Wet sieve Analysis

Sieve opening Weight of Weight of sieve + retained number [mm] sieve [g]soil [g][g]%RET		Weight of soil			CUM%R	%PASS
4.751262.81282.800					0	100
2.36800.9803.62.70.27					0.27	99.73
2950.39554.70.47					0.74	99.26
1.18895.59004.50.45					1.19	98.81
0.6824.9833.76.70.67					1.86	98.14
0.425786.47884.60.46					2.32	97.68
0.3748.4751.53.10.31					2.63	97.37
0.15776.8780.94.10.41					3.04	96.96
0.075767767.40.40.04					3.08	96.92
pan386.71355.796996.9					99.98	-----

Table 2 Hydrometer Test Result

Hydrometer Test Analysis Result Sheet

Soil Specific gravity =2.7

Elapsed time [min]	Actual Hydrometer reading	Composite correction	Corrected hydrometer reading	Effective depth [cm]	Coefficient [K]	Grain size [mm]	Percent finer [%]	Percent. Finer combined [%]
3/4	1.0320	-0.00256	1.0294	7.84	0.013328	0.0373	93.52	90.64
1	1.0300	-0.00254	1.0275	8.36	0.013312	0.0272	87.23	85.76
2	1.0295	-0.00254	1.0270	8.50	0.013312	0.0194	85.64	83.00
4	1.0290	-0.00256	1.0264	8.63	0.013328	0.0138	83.99	81.40
8	1.0285	-0.0026	1.0259	8.76	0.01336	0.0102	82.27	79.74
15	1.0280	-0.00256	1.0254	8.89	0.013328	0.0073	80.81	78.32
30	1.0270	-0.0027	1.0243	9.16	0.013424	0.0052	77.19	74.81
60	1.0265	-0.00244	1.0241	9.29	0.013232	0.0052	76.43	74.07
120	1.0245	-0.00216	1.0223	9.82	0.013014	0.0037	70.96	68.78
240	1.0215	-0.0019	1.0196	10.61	0.012745	0.0027	62.26	60.34
480	1.0190	-0.0019	1.0171	11.27	0.01279	0.0020	54.32	52.64
1440	1.0172	-0.0023	1.0149	11.75	0.0132	0.0012	47.33	45.87

Table3 Test Result of CEC

Cationic Exchange capacity and PH	Result of Concentration [meq /100 gram of soil	Class of Clay Mineral [based on Chen 19 88]
PH-H_2O (1:2.5)	8.38	**Montmorillonite**
PH-KCL (1:2.5)	7.53	
EC(ms/cm) (1:2.5)	0.26	
Exchangeable Na.(meq/100 gram of soil	5.6	
Exchangeable K(meq/100 gram of soil	2.48	
Exchangeable Ca(meq/100 gram of soil	47.01	
Exchangeable Mg (meq/100 gram of soil	17.18	
CEC(meq/100gram of soil)	74.06	
Sum of cations (meq/100gram of soil).	66.26	
Exchangeable Sodium % (ESP)	2.40	

Table 4 Summary of Atterberg Limit Test Results

Summary of Atterberg Limit Test Results				
Location : Bole Arabsa				
Tested by: Tesfay Kiross				
Material description , Expansive clay				
SS (weight by % of soil)[Days] LL[%]PL [%]PI [%] 0011967	Curing time			
	010465 310163			52 39
379962	09760 39758			38 37 37
579858	09256 39155			39 40 36
779259	08856 38954			35 32 32
978854				35 34

Table 5 Summary of Moisture density relation for both treated and un treated soil

Summary of Moisture density relation test results		
Location : Bole Arabsa		
Tested by : Tesfay Kiross		
Material Description : Expansive Clay		
SS%	MDD[g/cc]	OMC [%]
0	1.18	42
3	1.25	37
5	1.3	33.0
7	1.38	30
9	1.27	38.8

Table 6 Summery of UCS values varied with SS amount and Curing.

Summary of UCS Test Results			
Location : Bole Arabsa			
Tested by : Tesfay Kiross			
Material Description : Expansive Clay			
SS%	Curing [Days]	UCS[N/mm2]	ΔUCS [N/mm2]
0	0	60.00	--------
	0	122.69	62.69
	3	150.23	90.23
3	7	176.50	116.50
	0	127.68	67.68
	3	172.75	112.75
	7	192.77	132.77
5	0	153.96	93.96
	3	160.24	100.24
	7	287.90	172.72
	0	115.18	55.18
7	3	130.02	70.02
	7	194.44	134.44
9			

Table 7 Summary of CBR test results for both stabilized and unstablized soil

Summary of CBR Test Results					
Location : Bole Arabsa					
Tested by : Tesfay Kiross					
Material Description : Expansive Clay					
SS%	Curing time [Days]	CBR (%)		CBR Swell % (%)	Reduction in swell
		Unsoaked	Soaked		
0	0	12.33	1.03	11.83	
3	0	24.45	1.29	4.49	58
	3	26.57	1.42	4.99	62
	7	--------	2.71	4.01	66
5	0	37.8	1.68	3.59	69.6
	3	46.3	2.12	2.62	77.8
	7	---------	3.08064	2.62	78.7
7	0	40.24	3.08	2.87	75.73
	3	50.42	3.12	2.34	80.2
	7	---------	4.62	2.41	80.
9	0	38.35	3.28	2.98	74.78
	3	44.73	2.95	1.39	88.27
	7	------	2.84	1.55	86.88

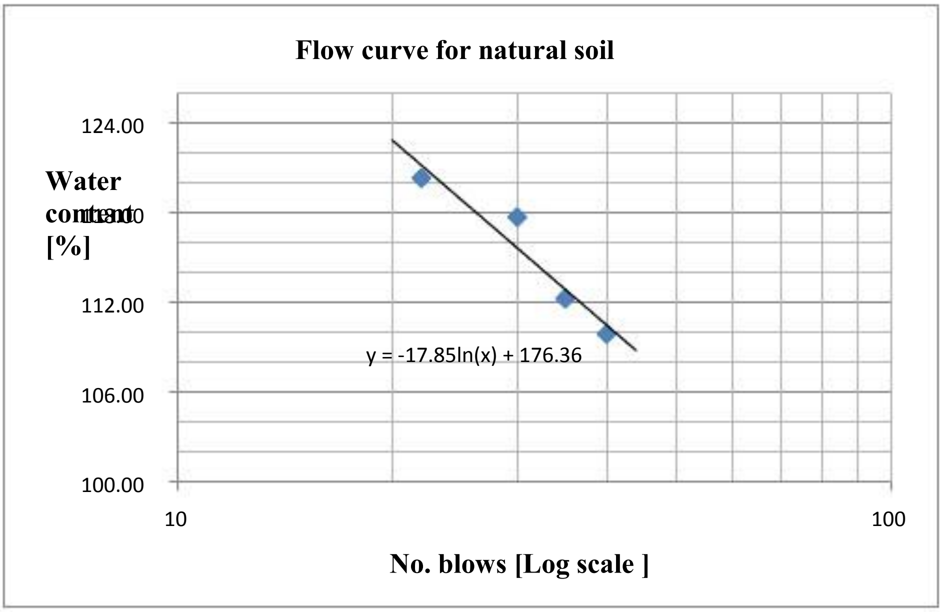

Figure 1 Liquid Limit for Natural soil

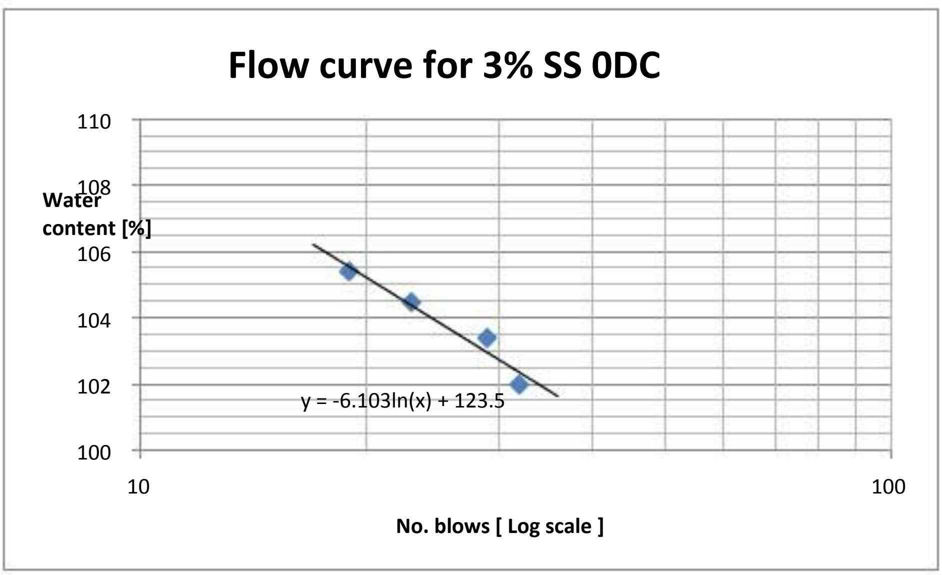

Figure 2 Liquid Limit For 3% Steel Slag.

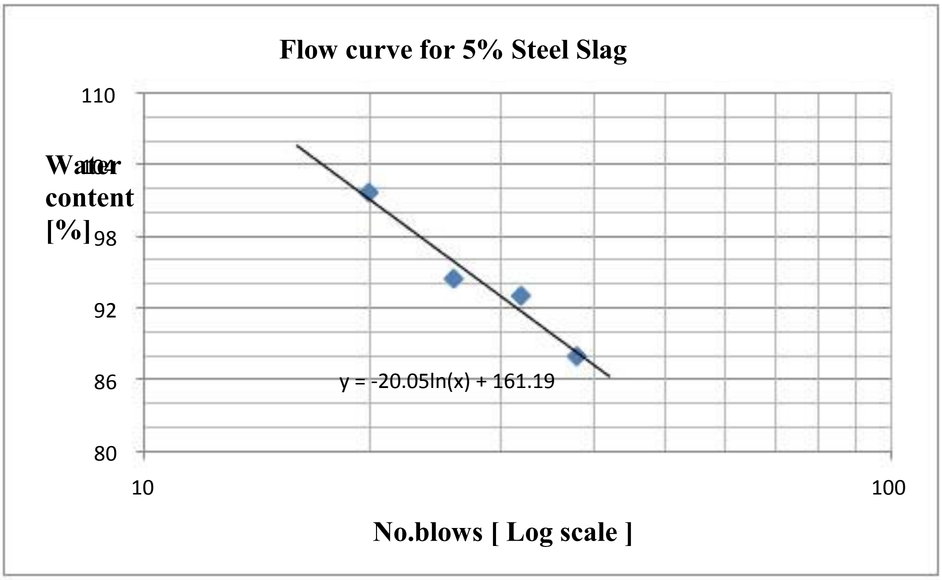

Figure 3 Liquid Limit For 5% Steel Slag.

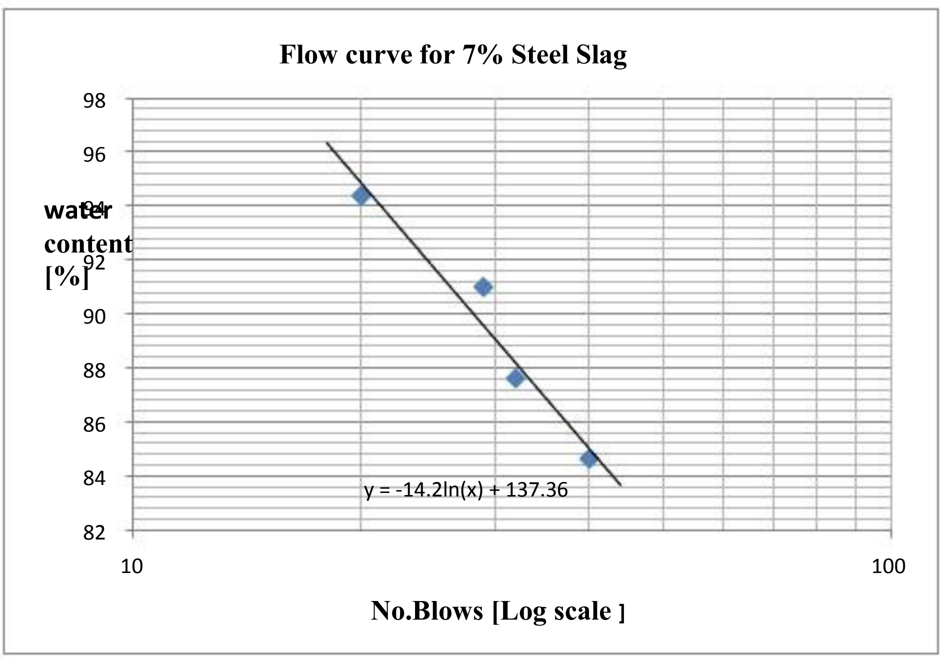

Figure 4 Liquid Limit For 7% Steel Slag.

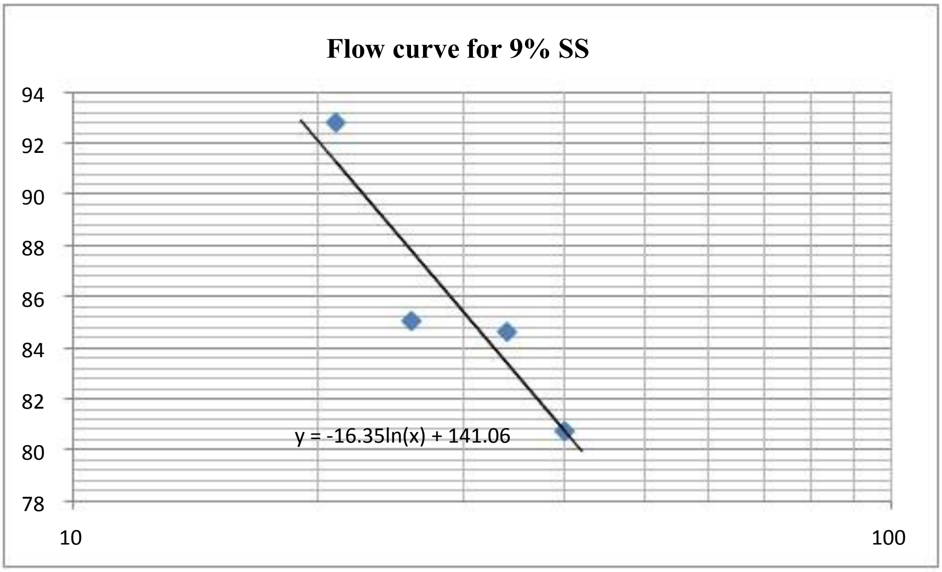

Figure 5 Liquid Limit For 9% Steel Slag.

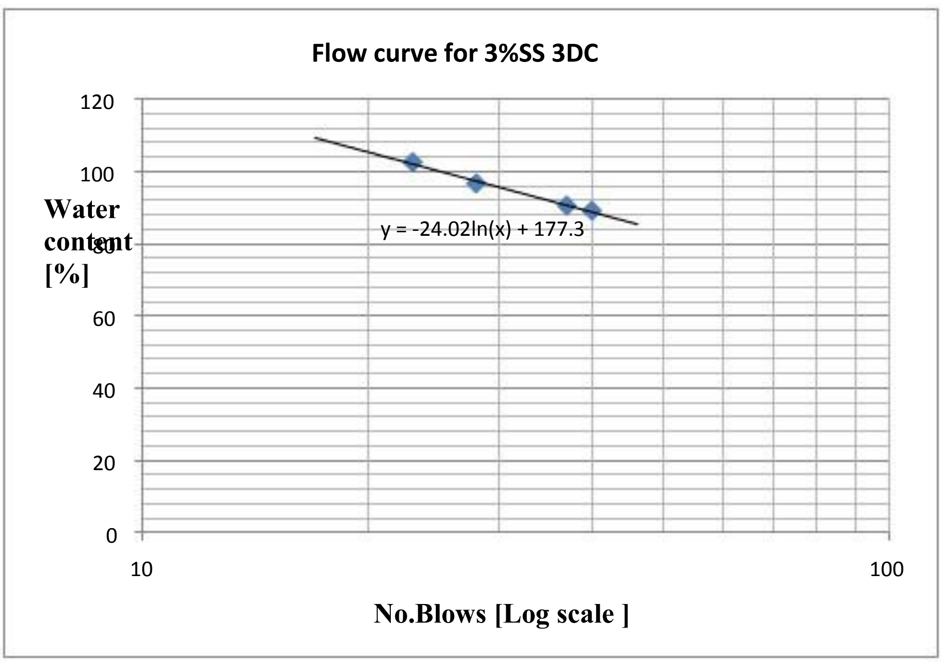

Figure 6 Liquid Limit For 3% Steel Slag for 3 day curing.

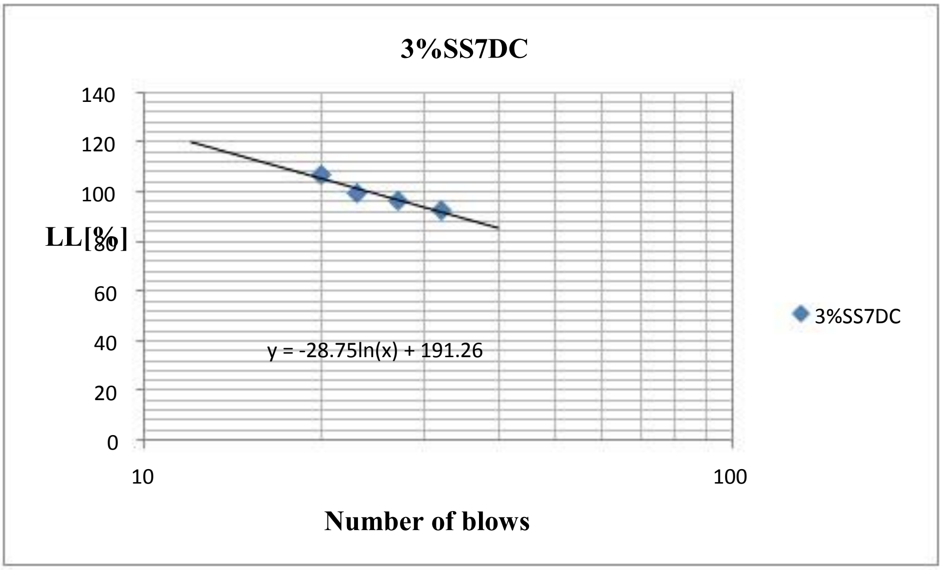

Figure 7 Liquid Limit For 3% Steel Slag for 7 day curing.

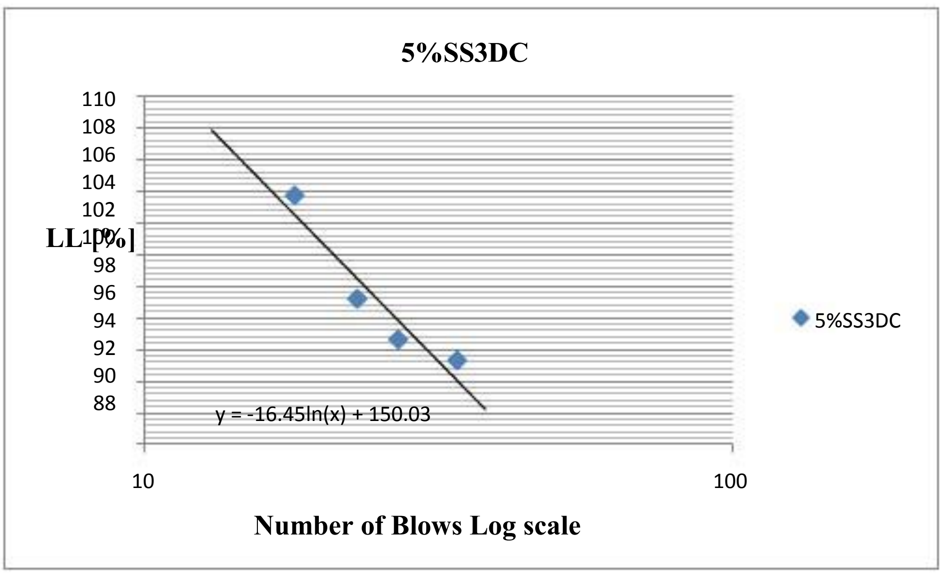

Figure 8 Liquid Limit For 5% Steel Slag for 3 day curing.

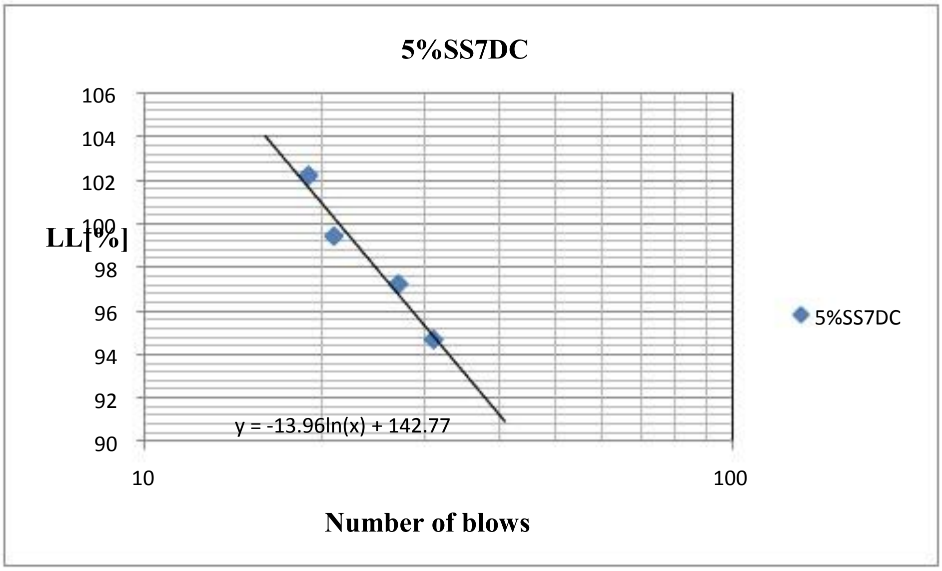

Figure 9 Liquid Limit For 5% Steel Slag for 7 day curing.

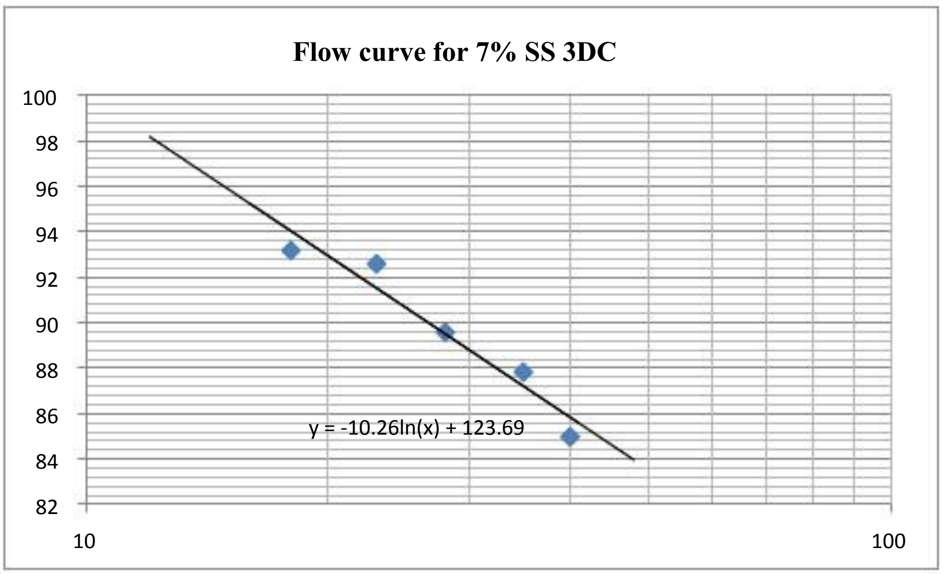

Figure 10 Liquid Limit For 7% Steel Slag for 3 day curing.

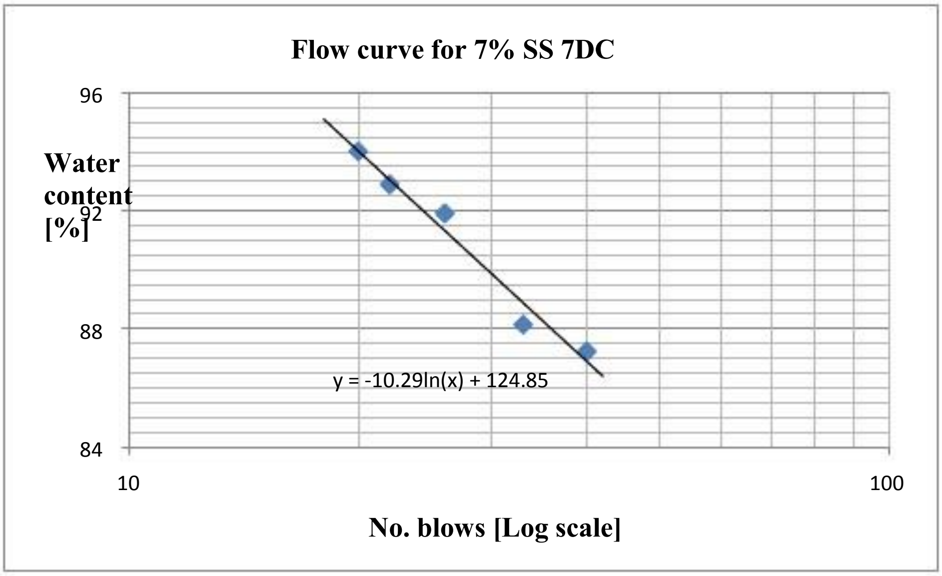

Figure 11 Liquid Limit For 7% Steel Slag for 7 day curing.

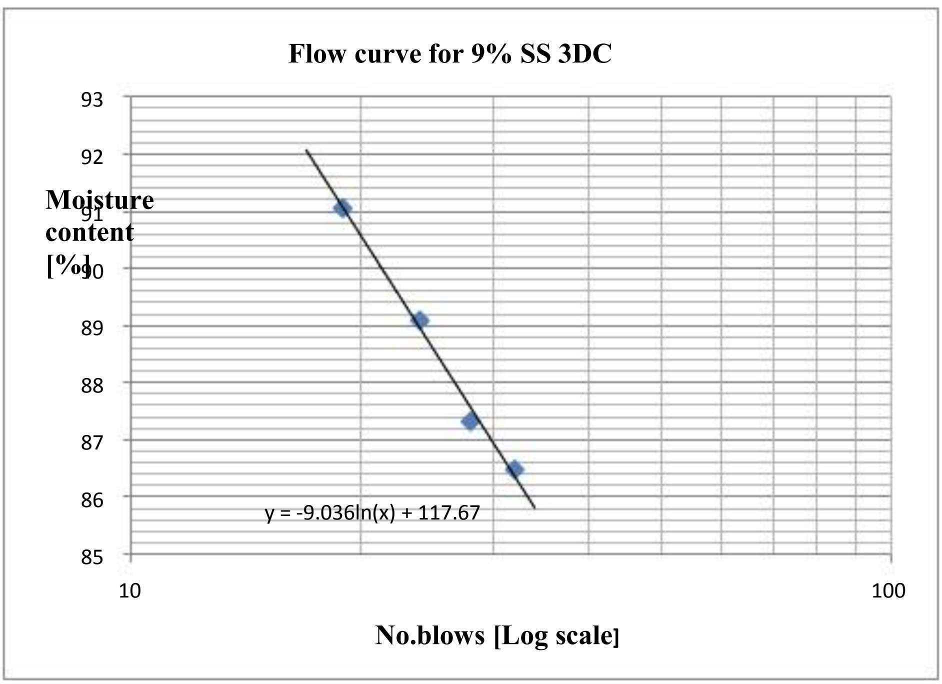

Figure 12 Liquid Limit For 9% Steel Slag for 3 day curing.

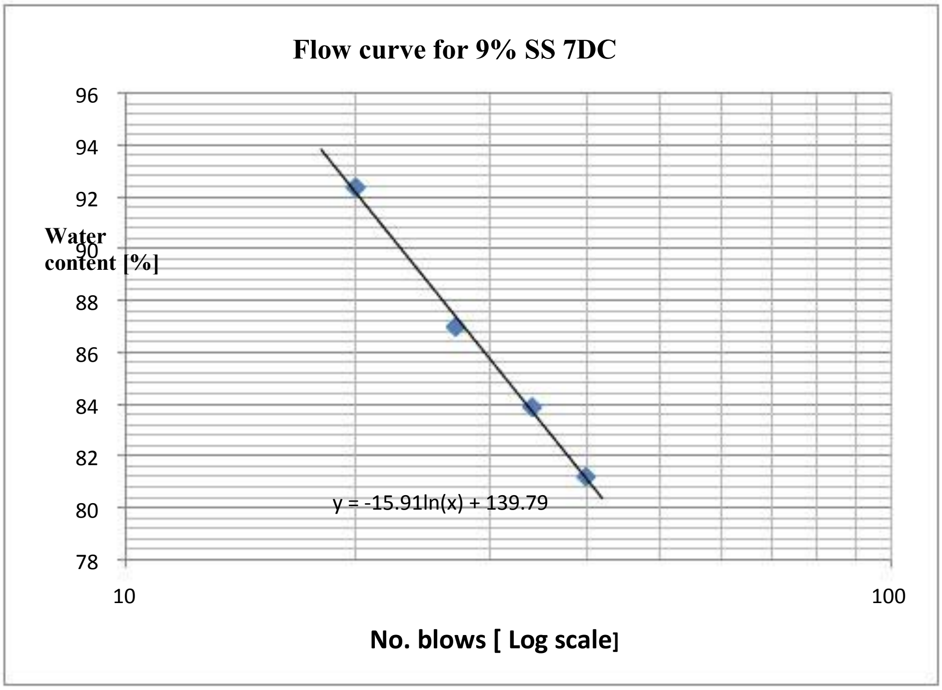

Figure 13 Liquid Limit For 9% Steel Slag for 7 day curing.

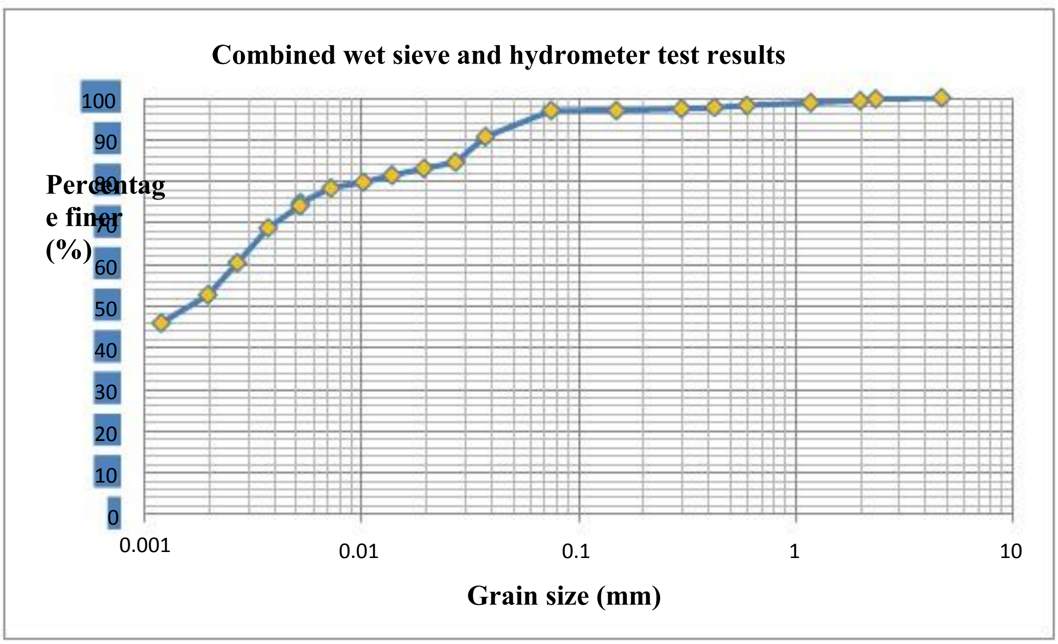

Figure 14 Combined Wet Sieve and H hydrometer Test analysis

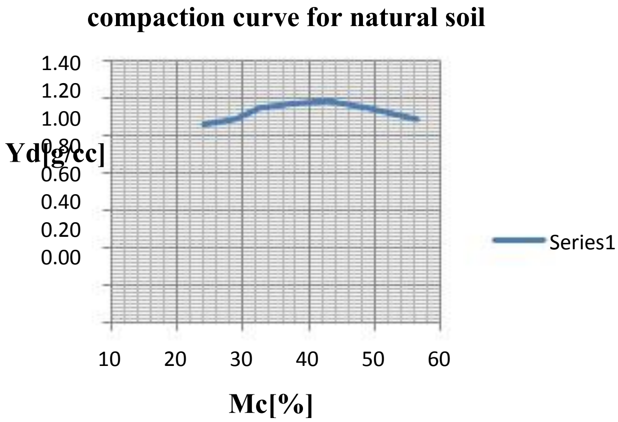

Figure 15 Compaction Test result for Natural soil

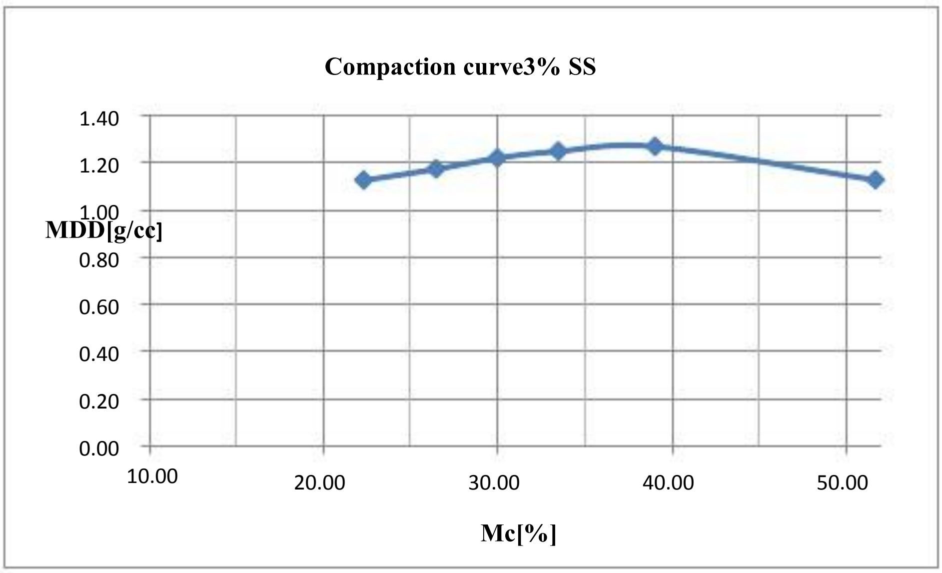

Figure 16 Compaction Test Result for 3% Steel Slag

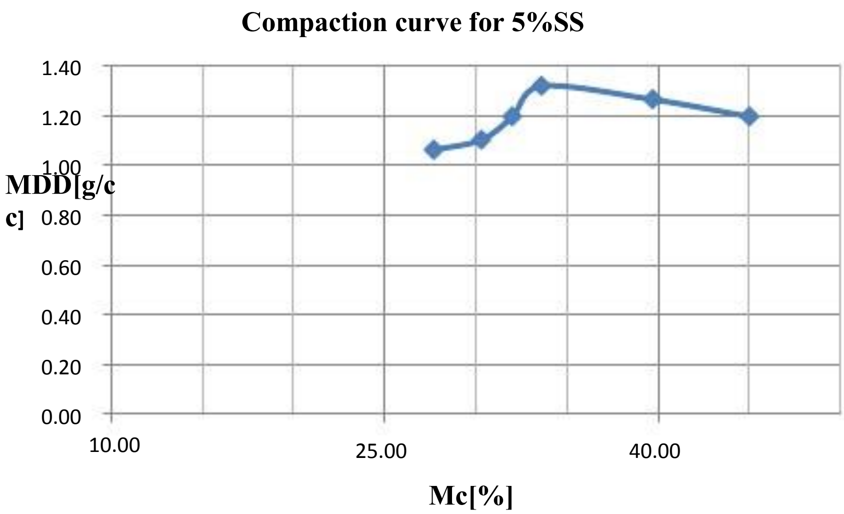

Figure 17 Compaction Test Result for 5% Steel Slag

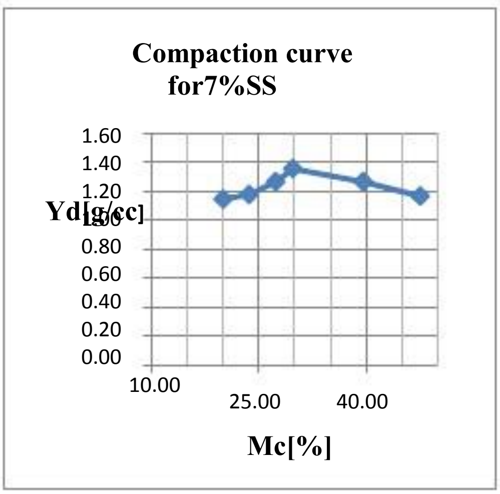

Figure 18 Compaction test Result for 7% Steel Slag

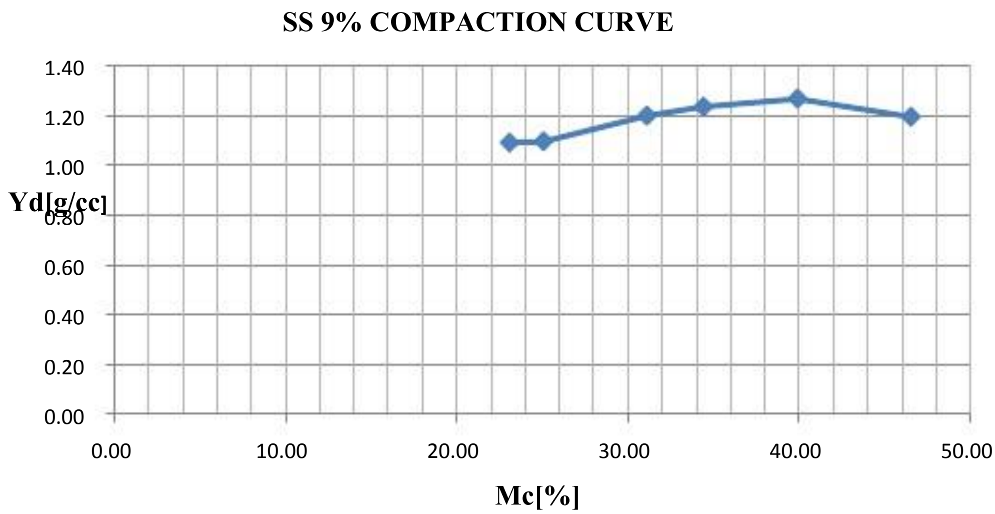

Figure 19 Compaction Test Result for 9% Steel Slag

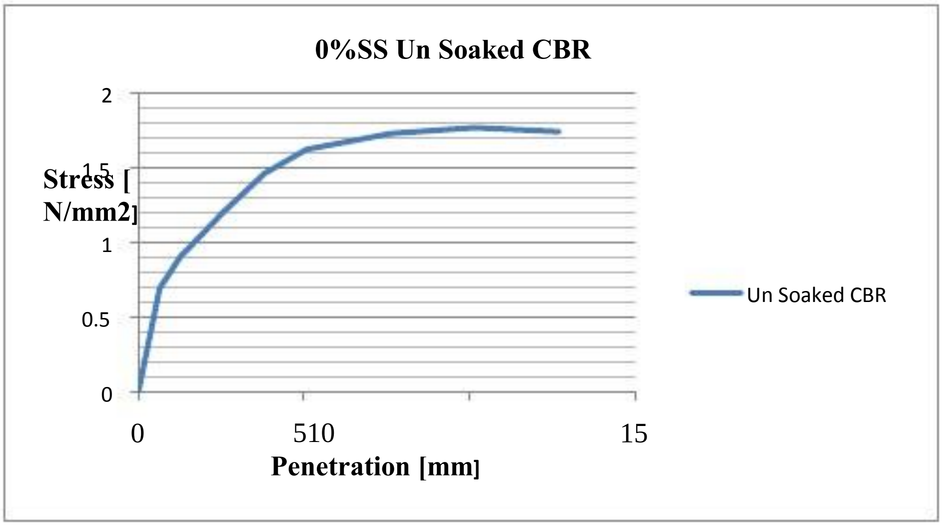

Figure 20 Stress - Penetration for 0 % Steel Slag Unsoaked CBR

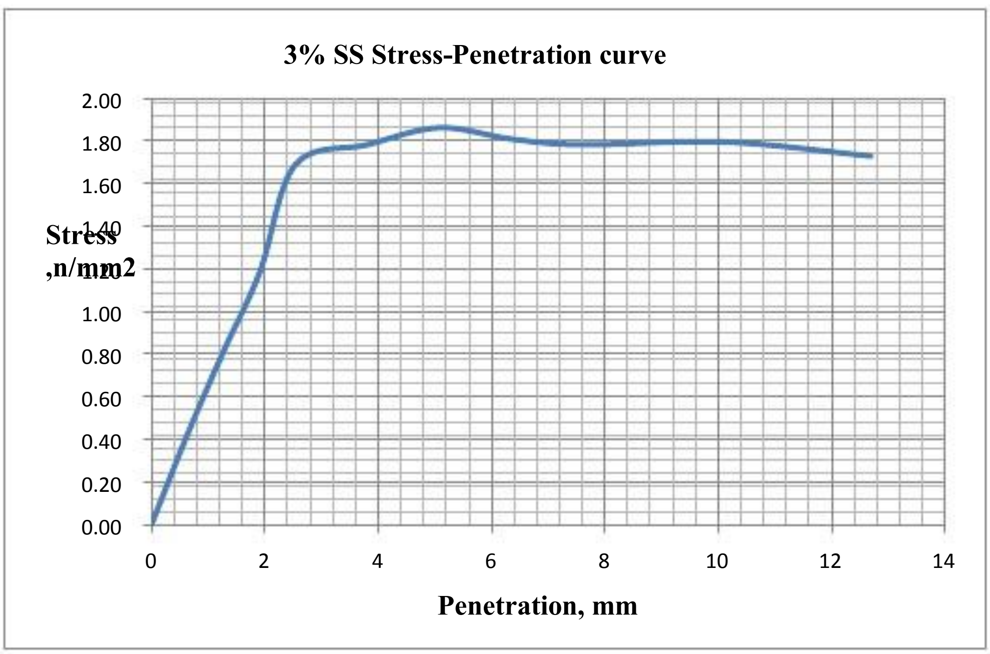

Figure 21 Stress- Penetration 3% Steel Slag Unsoaked CBR

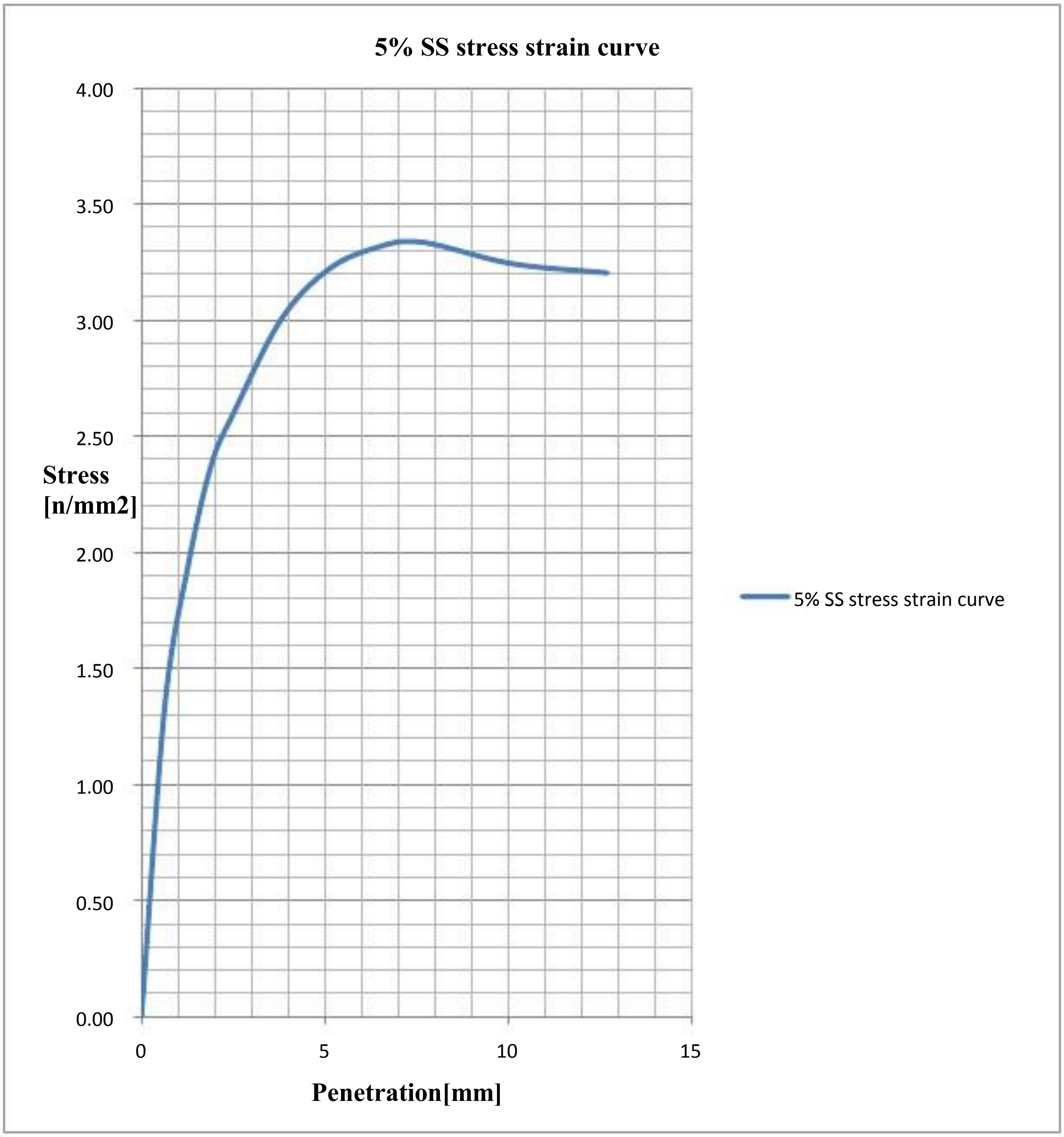

Figure 22 Stress- Penetration for 5% Steel Slag unsoaked case

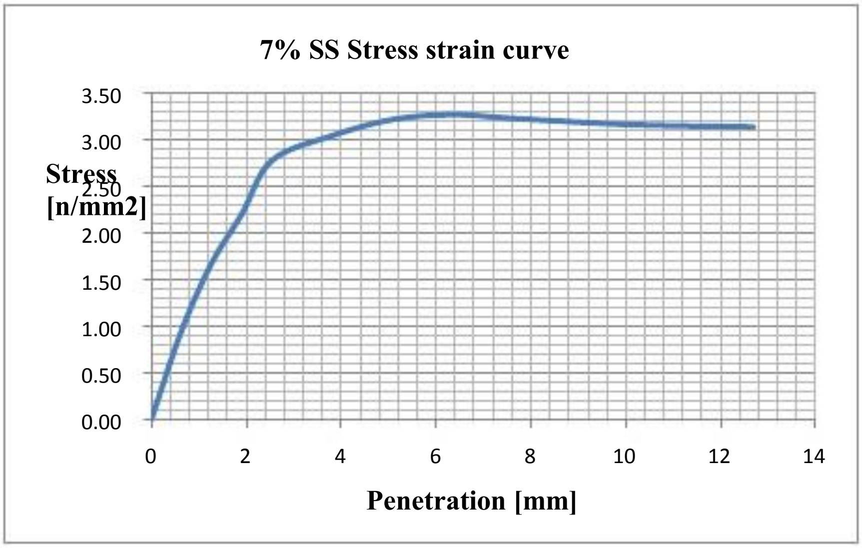

Figure 23 Stress- Penetration 7% Steel Slag

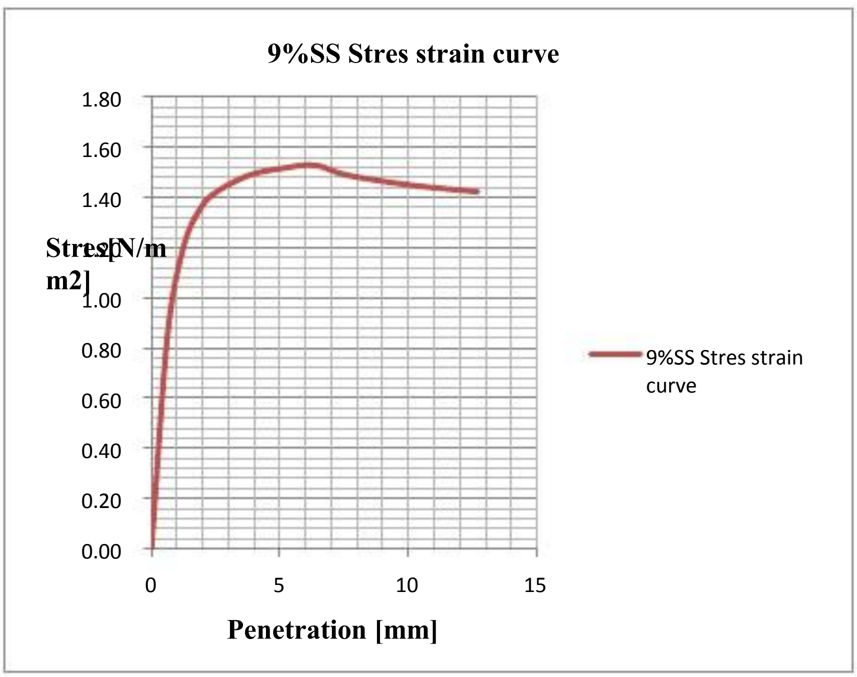

Figure 24 Stress- Penetration for 9% Steel Slag

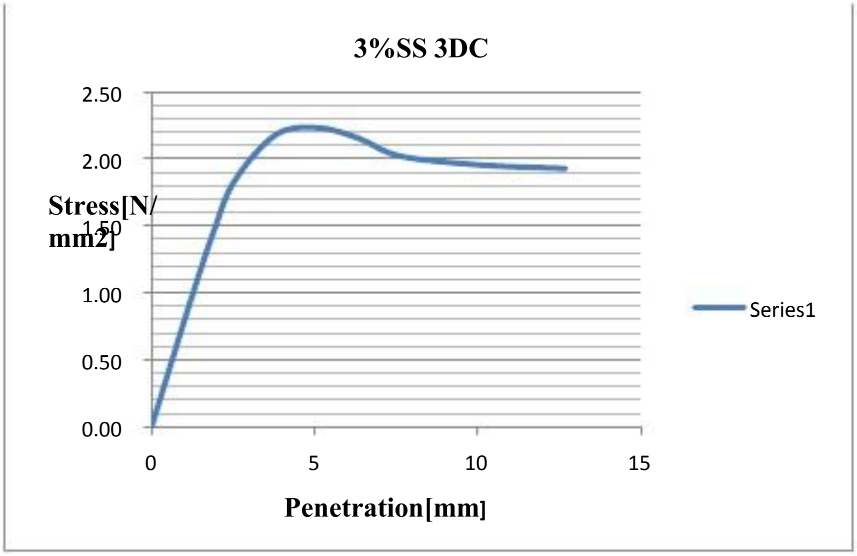

Figure 25 Stress- Penetration For3% Steel Slag unsoaked case

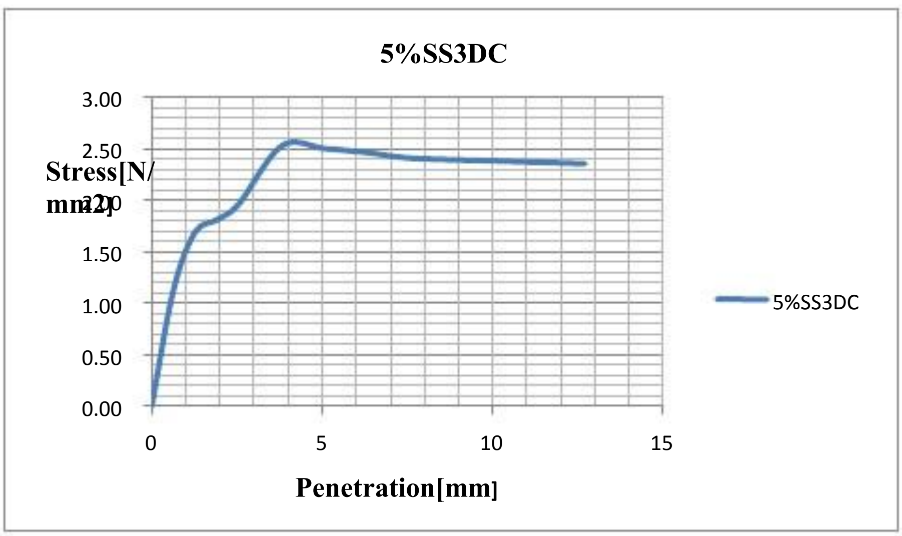

Figure 26 Stress- Penetration for 5% Steel Slag 3DC

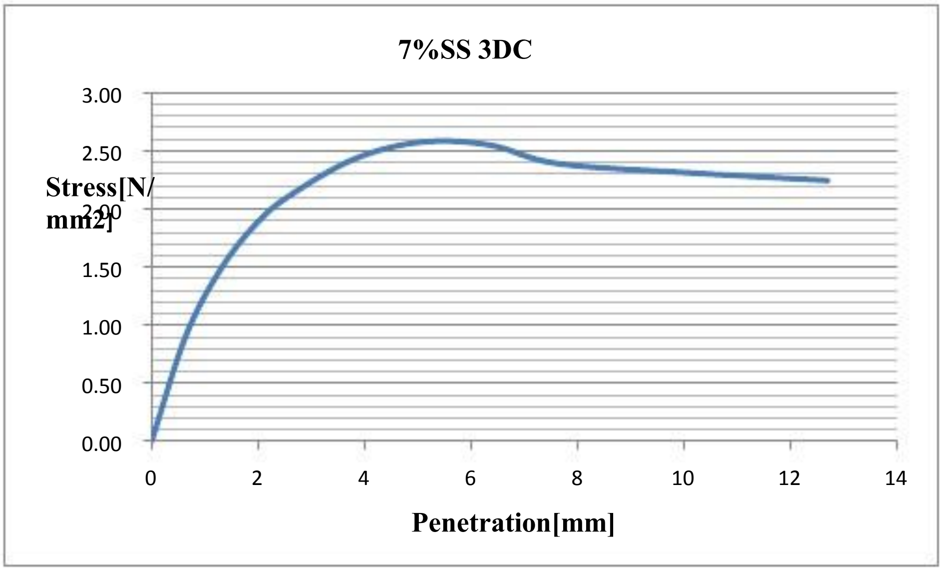

Figure 27 Stress- Penetration for 7% Steel Slag 3DC

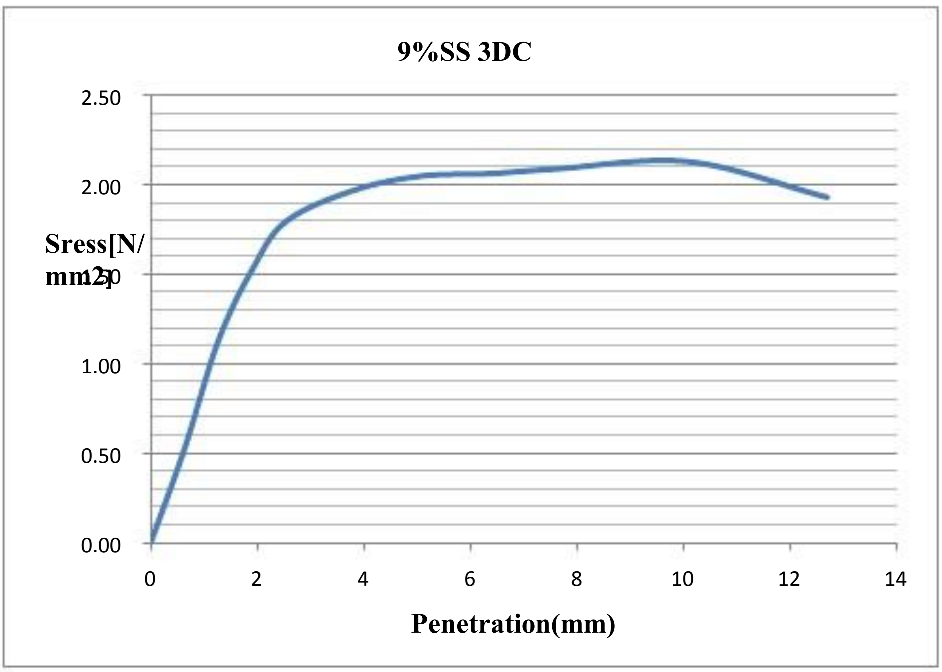

Figure 28 Stress- Penetration 9% Steel Slag 3DC

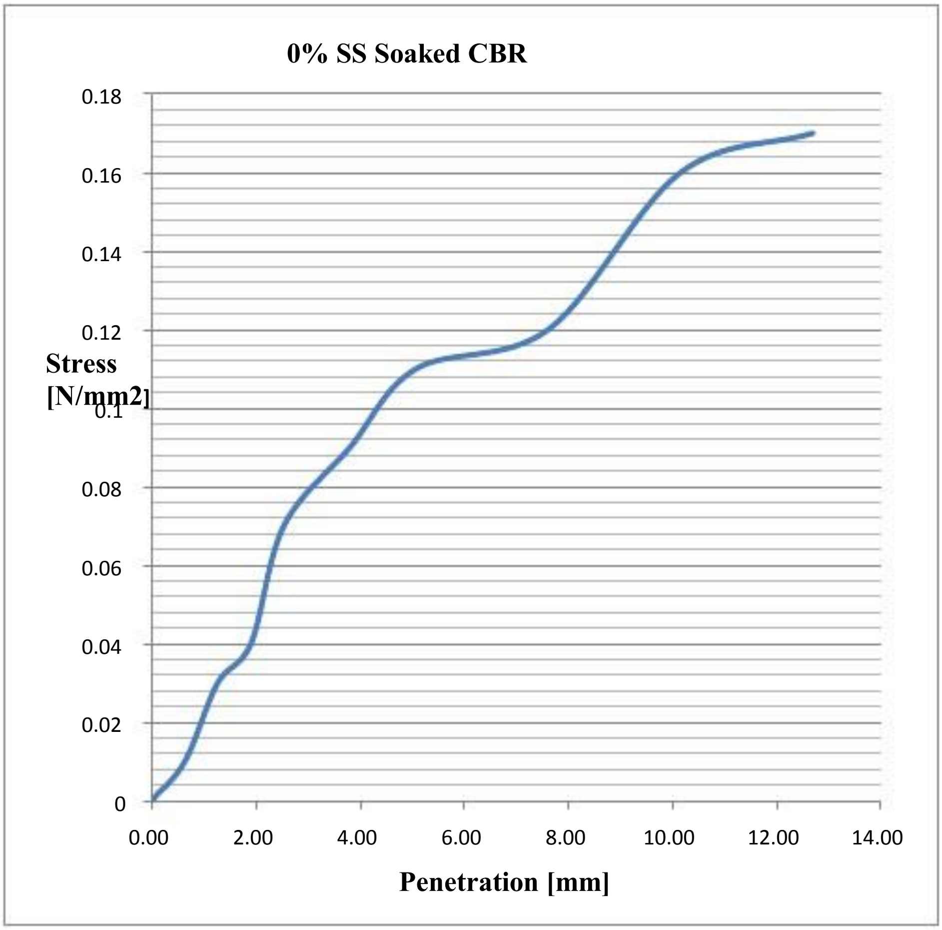

Figure 29 Stress- Penetration for 0% Steel Slag

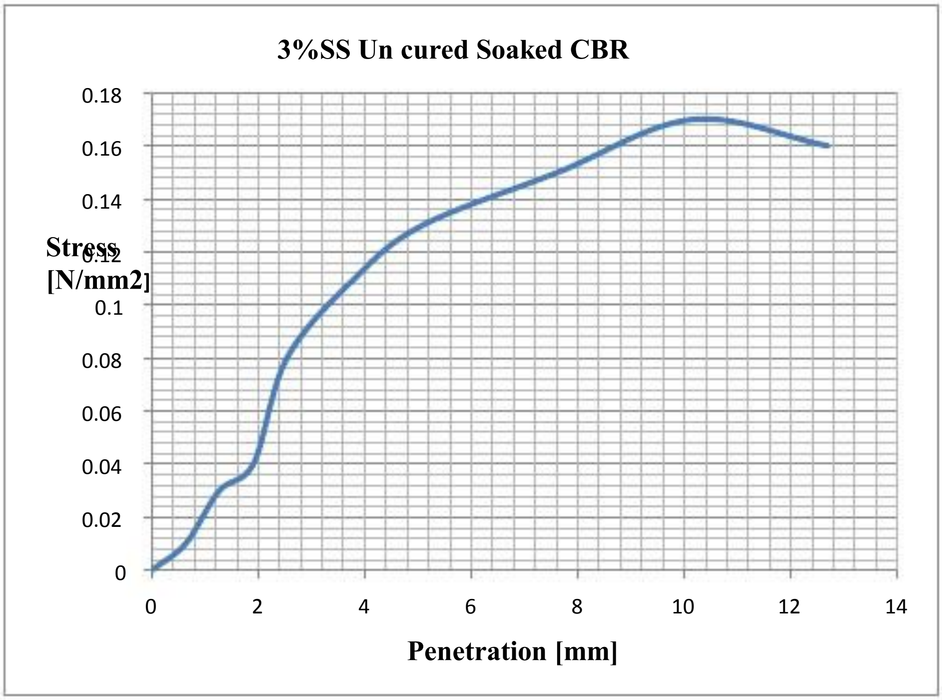

Figure 30 Stress Strain curve for 3% Steel Slag for UN curing Soaked CBR

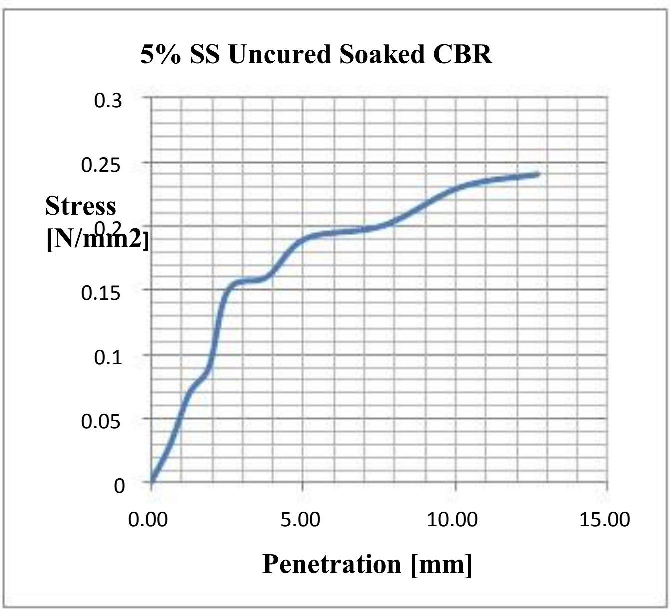

Figure 31 Stress penetration for 5% Steel Slag Soaked Case

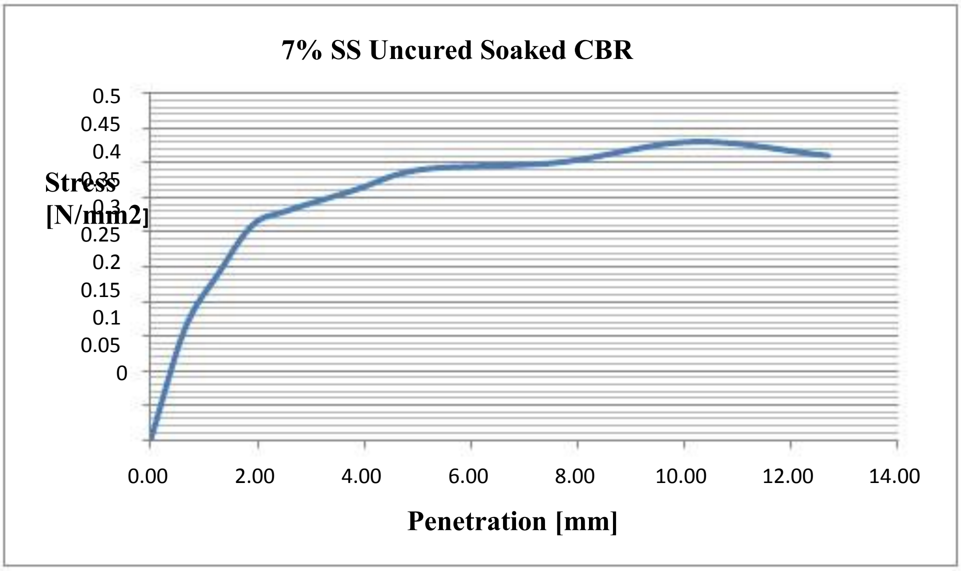

Figure 32 Stress – penetration curve for 7% Steel Slag Soaked case

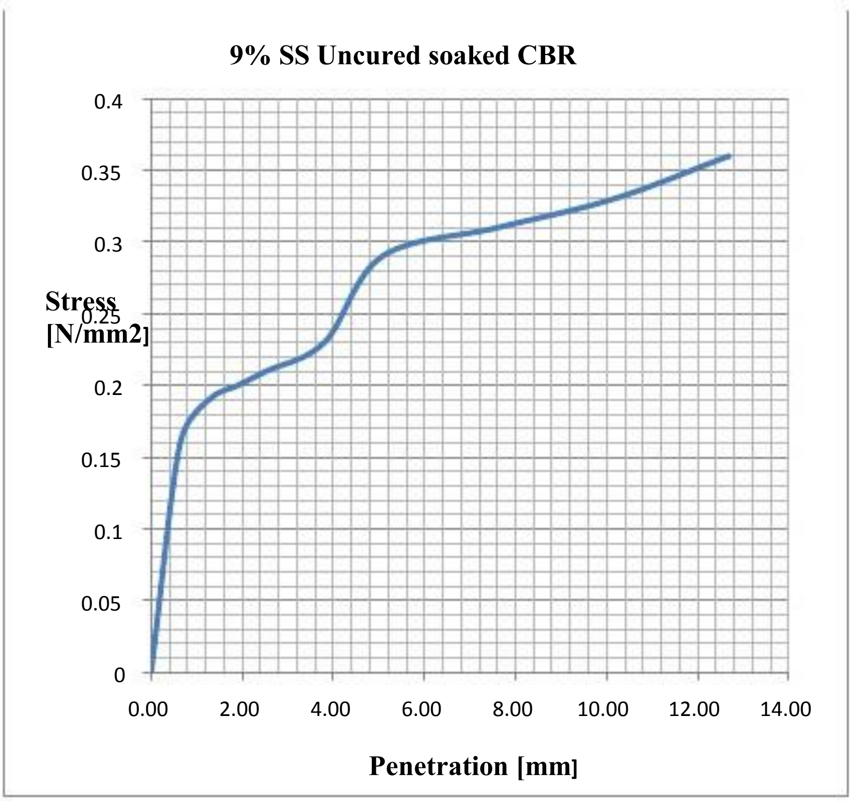

Figure 33 Stress – Penetration Curve for 9% Steel Slag soaked Case

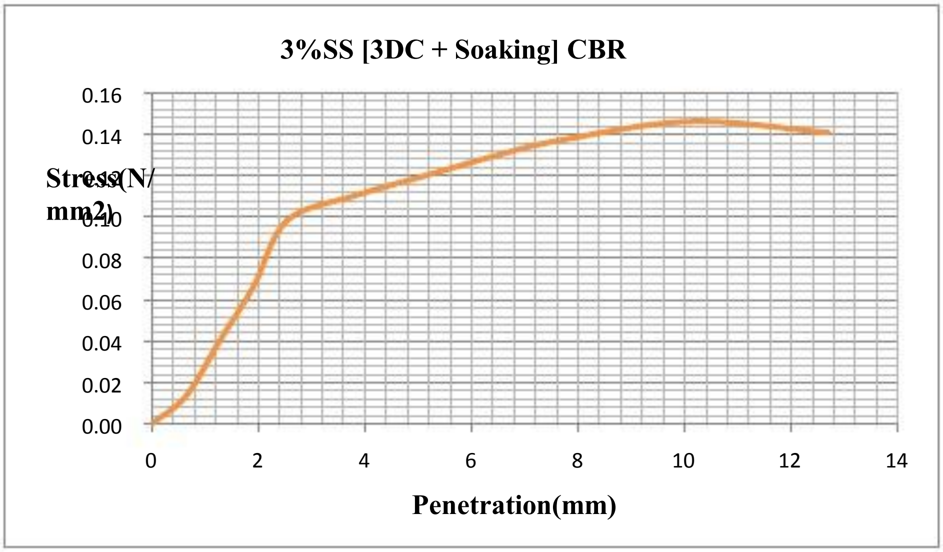

Figure 34 Stress –penetration for 3% SS [3DC+SO]

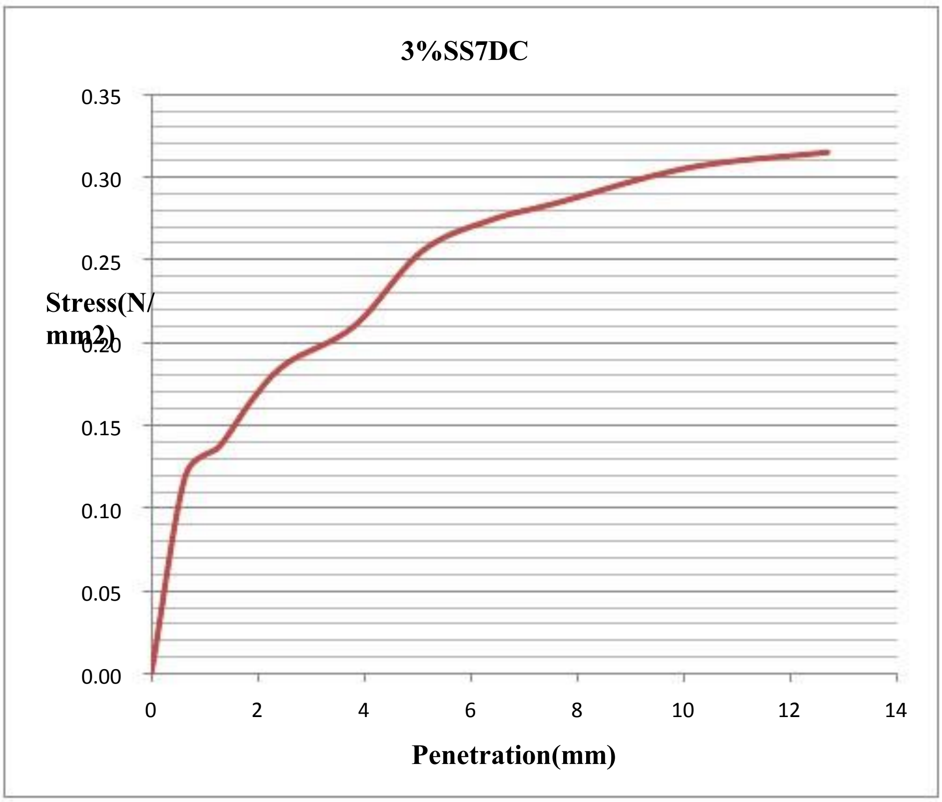

Figure 35 Stress versus Penetration for 3% SS 7DC+ So

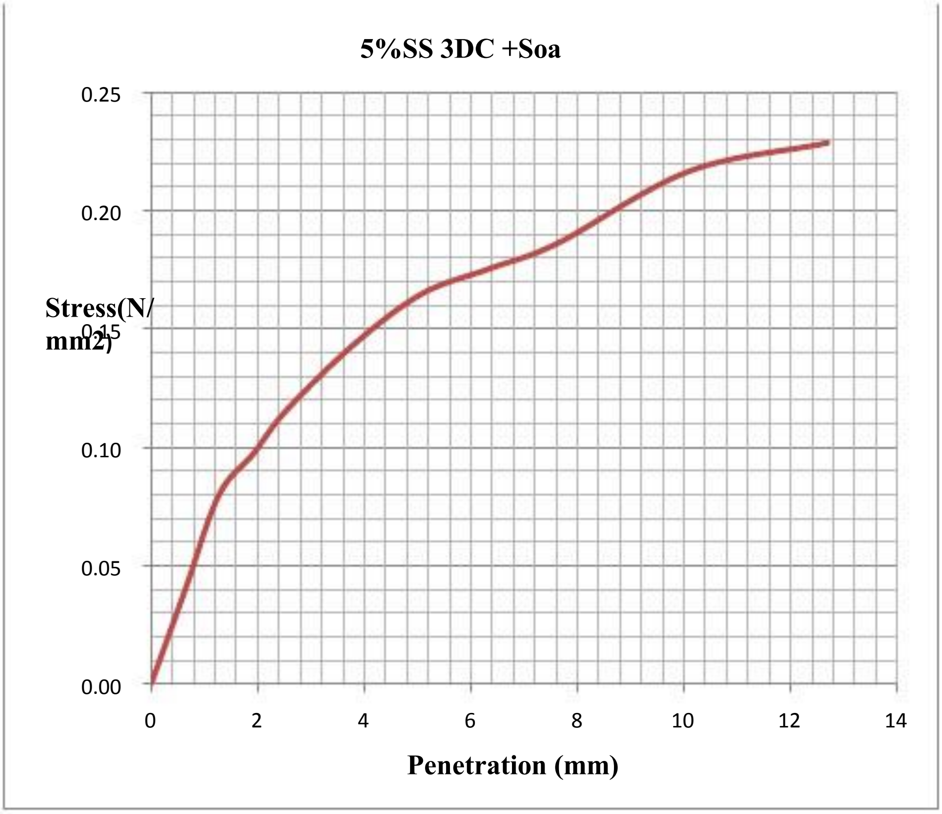

Figure 36 Stress – Penetration for 5% SS 3DC+So

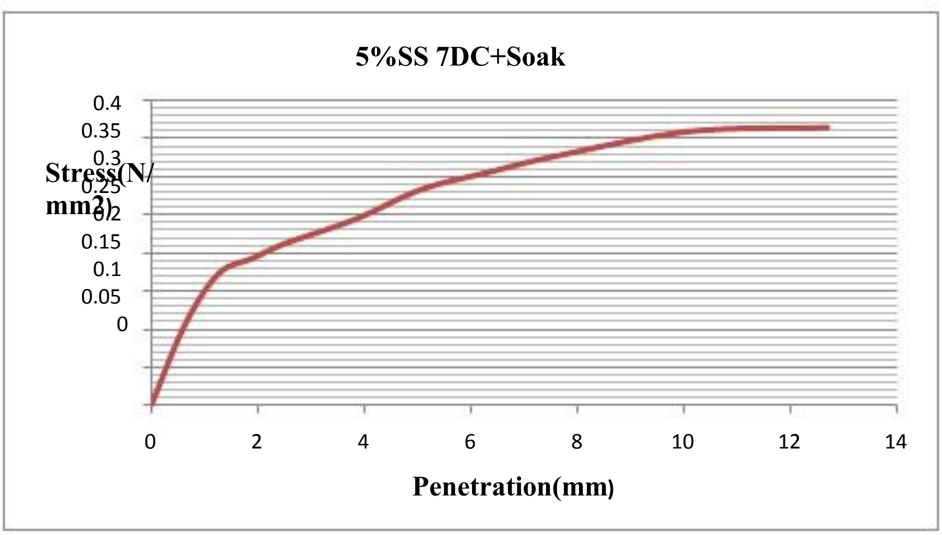

Figure 37 Stress- Penetration for 5% SS 7DC+SO

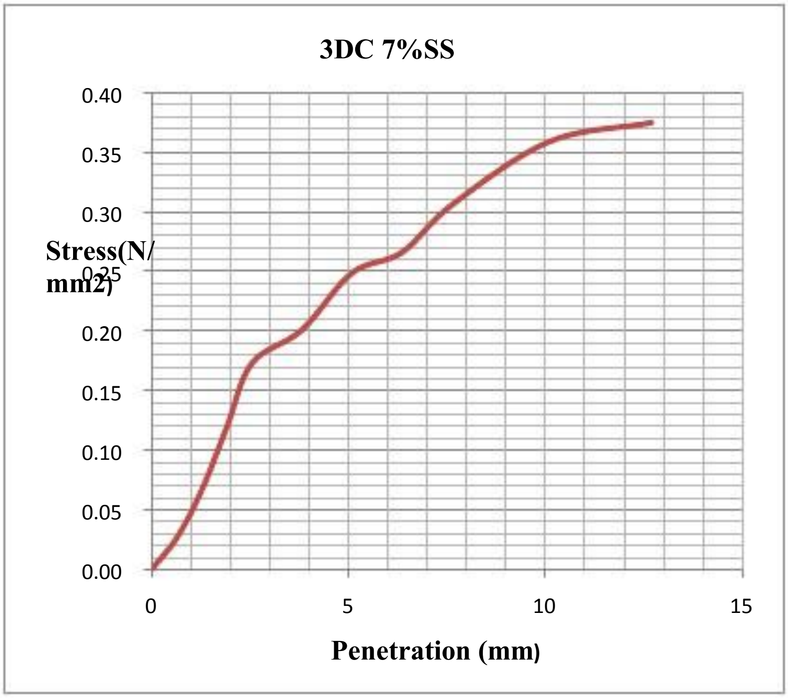

Figure 38 Stress - Penetration for 7% SS 3DC

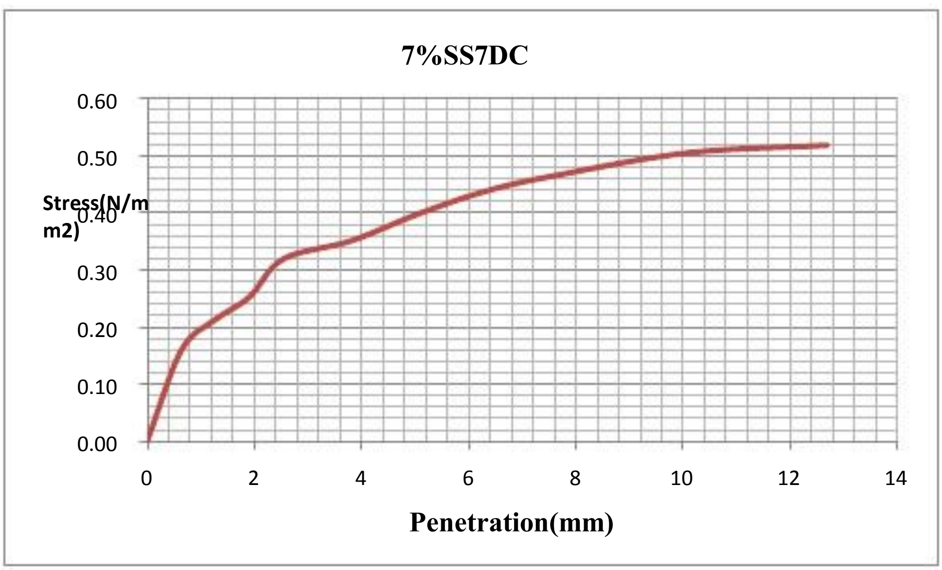

Figure 39 – Stress- Penetration for 7% SS 7DC+S

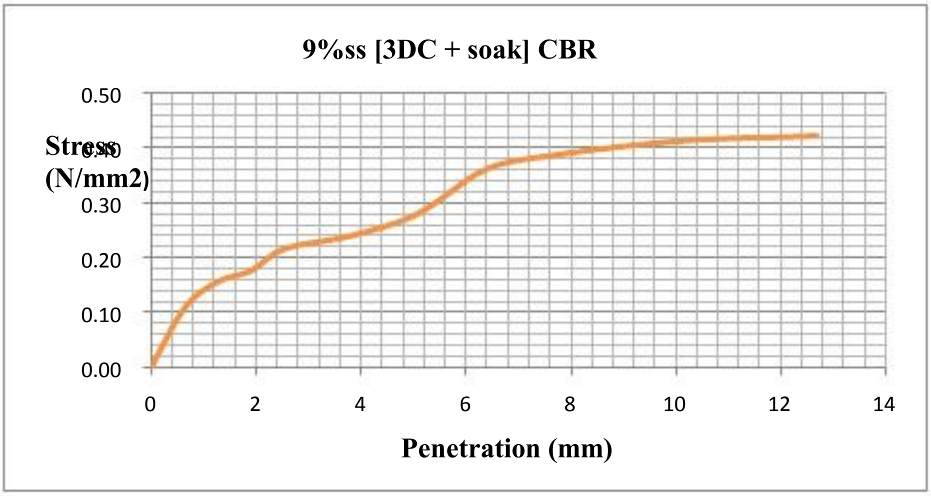

Figure 40 Stress - Penetration for 9% SS 3DC+ S

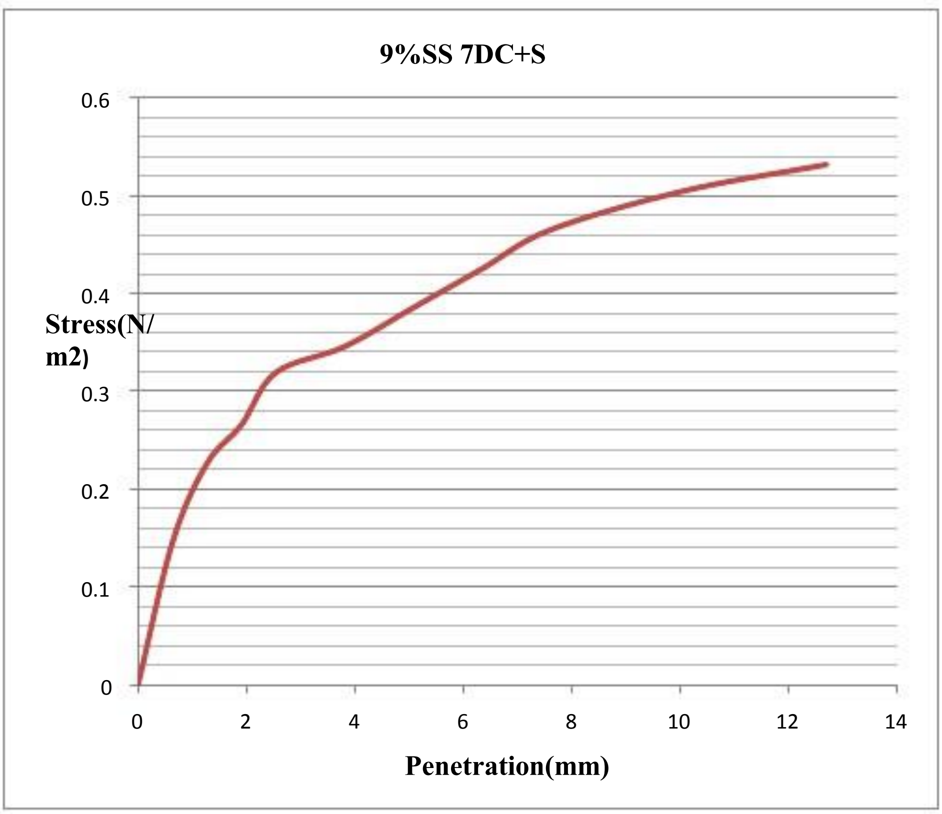

Figure 41 Stress - Penetration for 9% SS 3DC+ SO

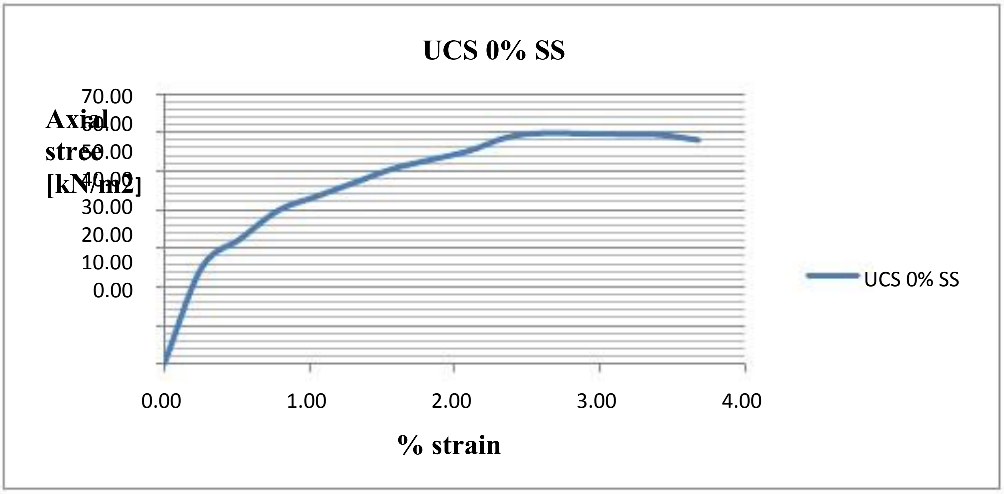

Figure 42 Stress- Strain for 0% SS

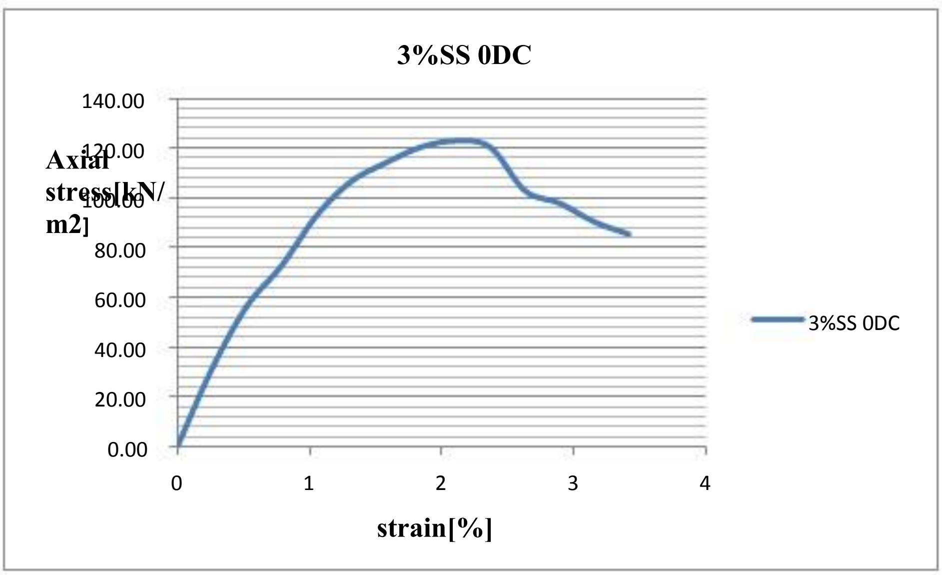

Figure 43 .Stress- Strain curve for 3% SS 0 DC

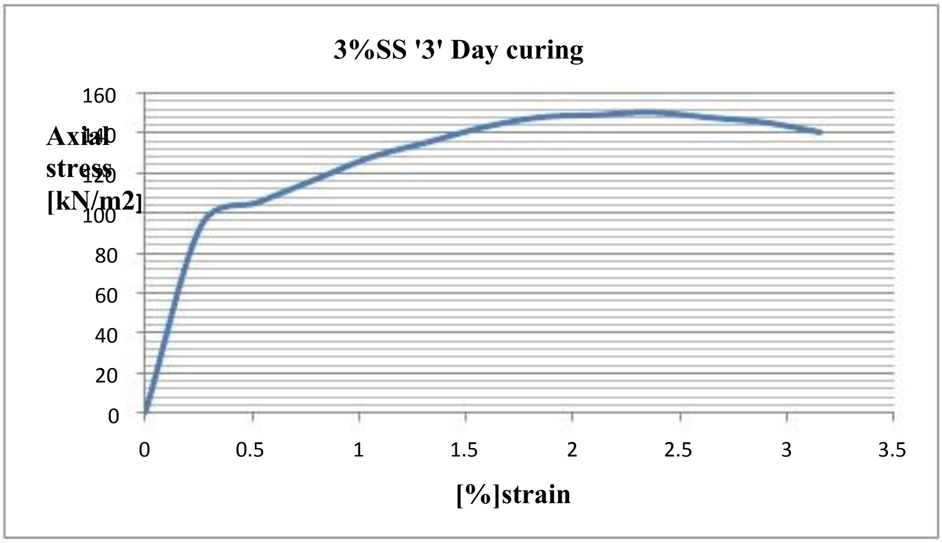

Figure 44 Stress- Strain curve for 3% SS 3DC

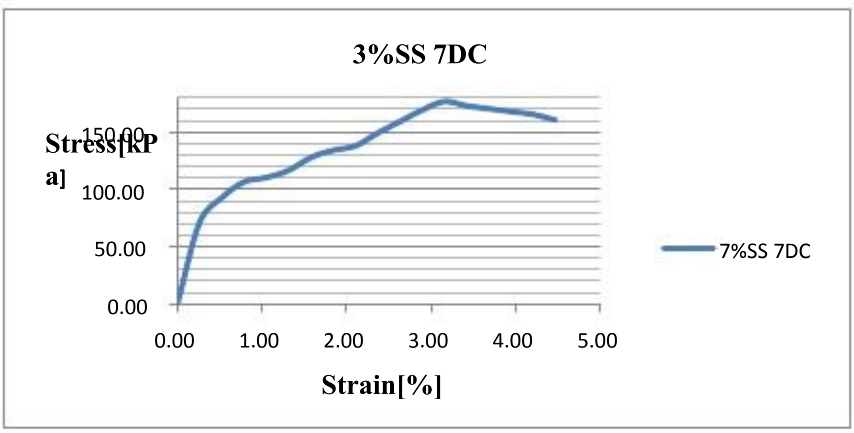

Figure 45 Stress – Strain curve for 3% SS 7DC

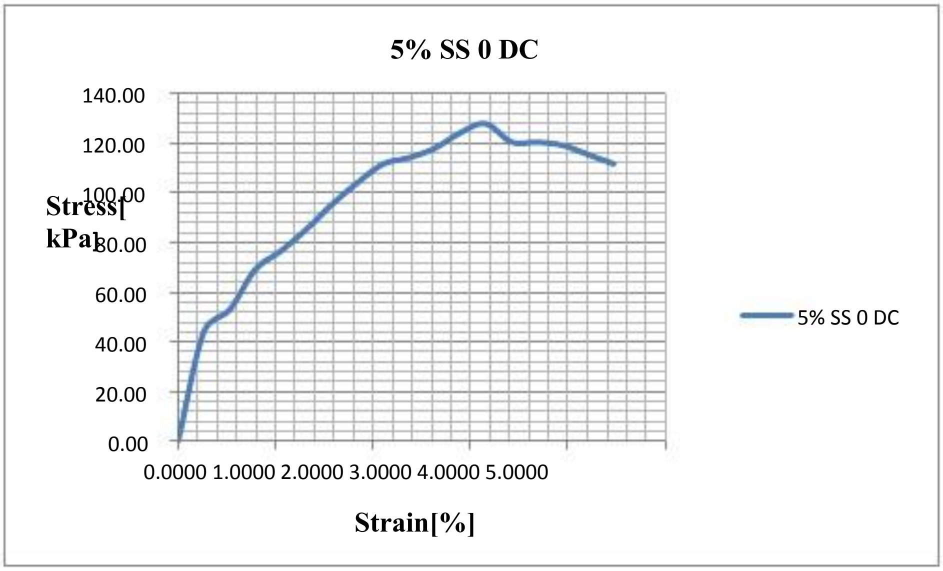

Figure 46 Stress – Strain curve for 5% SS 0 DC

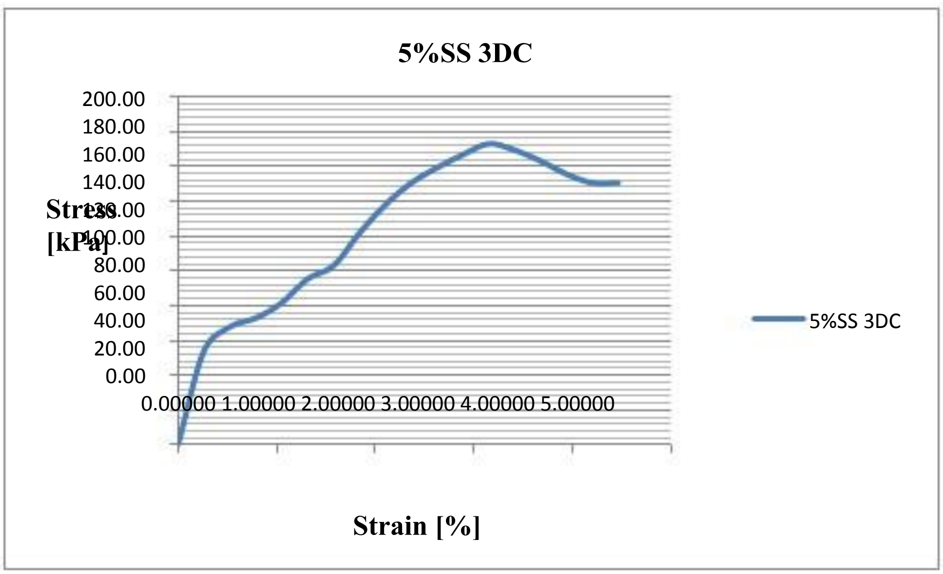

Figure 47 Stress- Strain curve for 5% SS 3DC

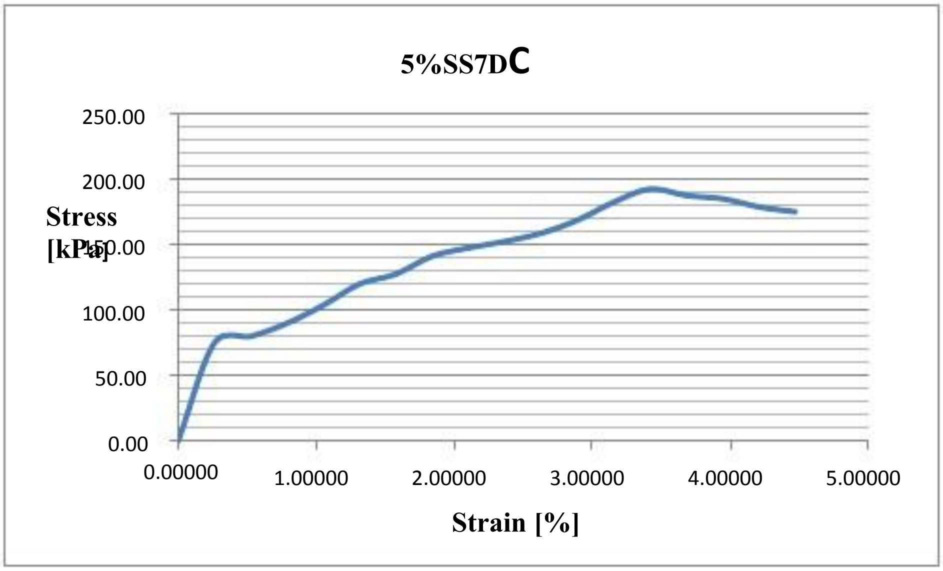

Figure 48 Stress –Strain Curve for 5% SS 7DC

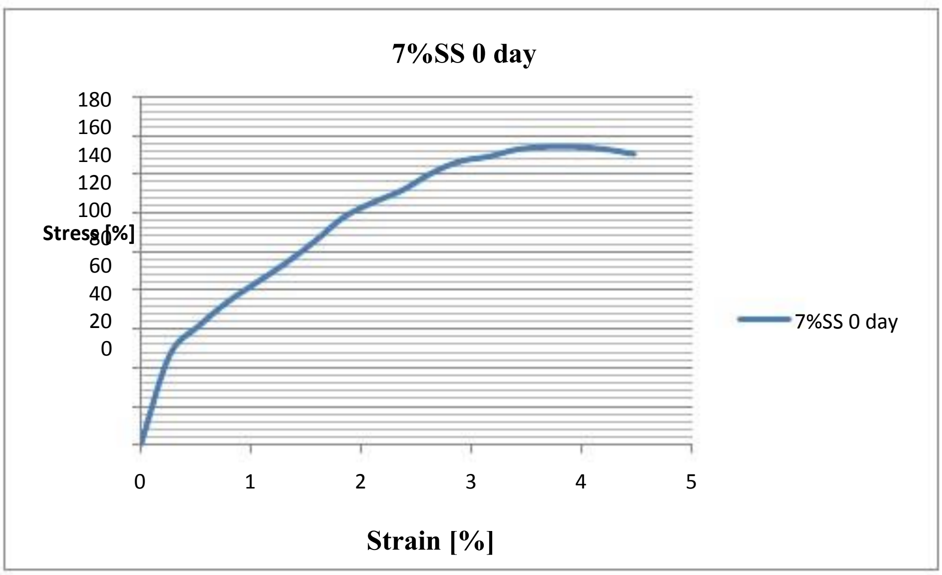

Figure 49 Stress- Strain curve for 7% SS Zero Day curing

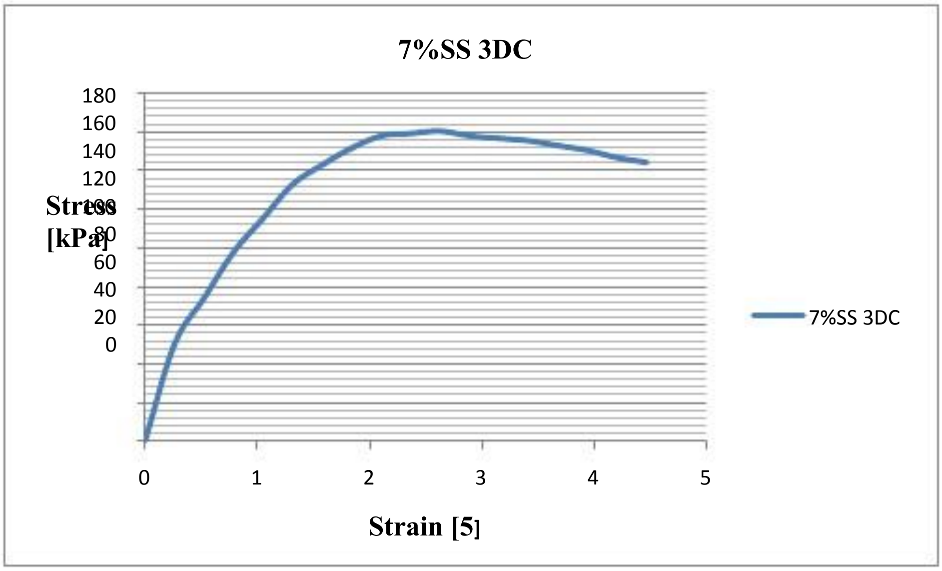

Figure 50 Stress- Strain Curve for 7% SS 3DC

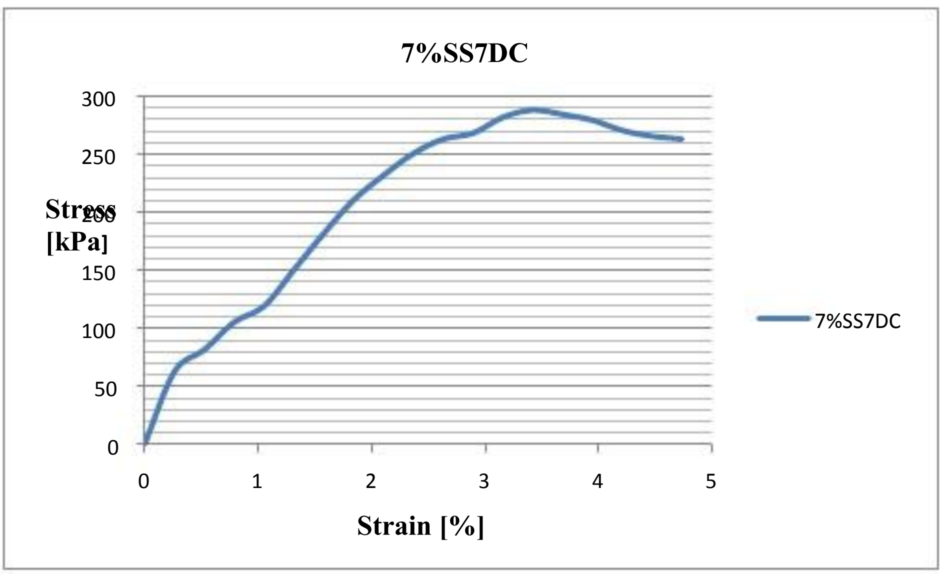

Figure 51 Stress- Strain Curve for 7% SS 7DC

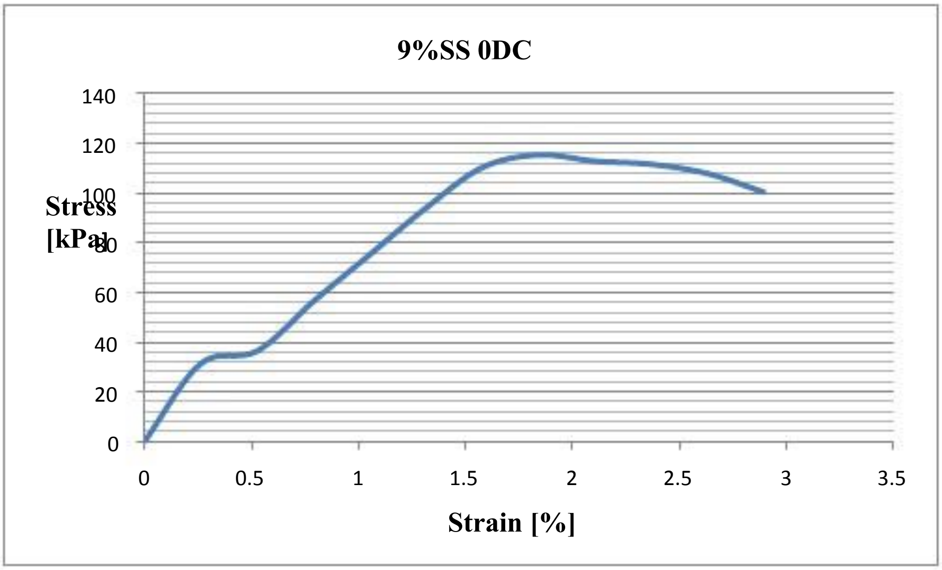

Figure 52 Stress- Strain Curve for 9% SS Zero Day Curing

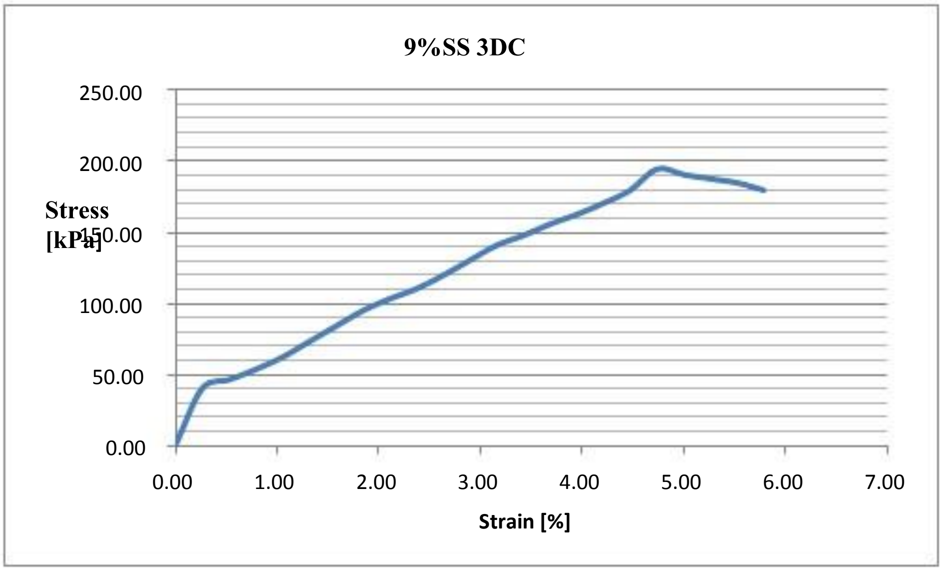

Figure 53 Stress- Strain Curve for 9% SS 3DC

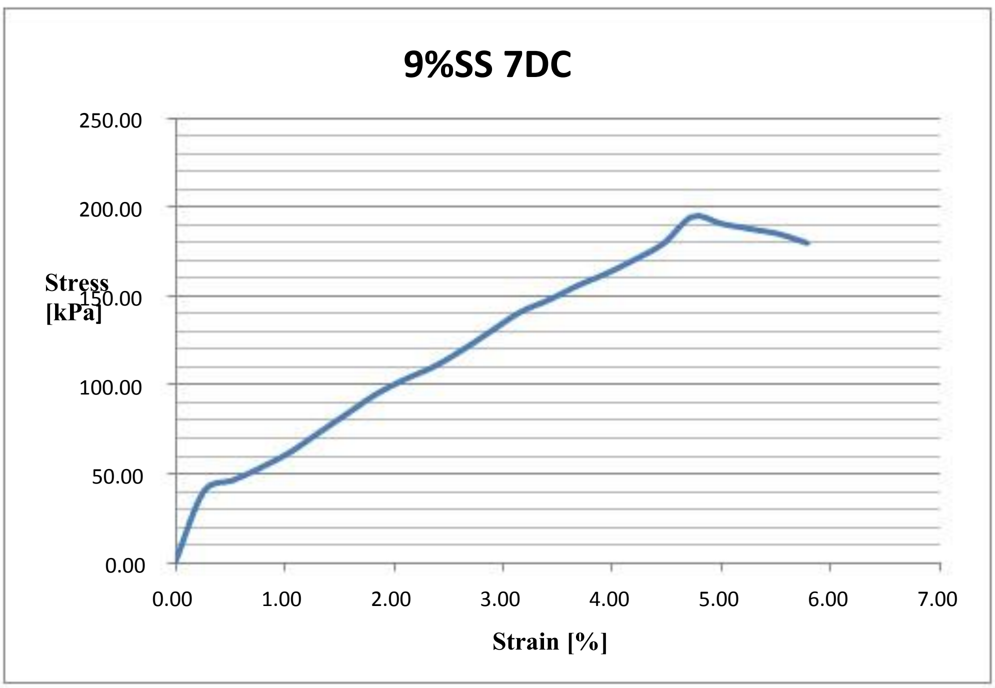

Figure 54 Stress- Strain Curve for 9% SS 7DC