

EXPERIMENTAL STUDY ON STRENGTH IMPROVEMENT OF WEAK SUBGRADE
CLAY SOIL USING COPPER SLAG AND HYDRATED LIME AS AN ACTIVATOR

By

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A dissertation Submitted to the Faculty of Engineering and Technology in fulfilment of the
requirements for the award of the Degree of Master of Engineering in Geotechnical
Engineering, BIUST.

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I _____ confirm that the dissertation submitted is my own work and appropriate credit has been given where reference has been made to the work of other authors. This dissertation has not been presented to any other university for similar purpose or for any other degree.

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CERTIFICATION

The undersigned certifies that he has read and hereby recommends for acceptance by the Faculty of Engineering and Technology a dissertation titled: “**Experimental Study on Strength Improvement of Weak Subgrade Clay Soil using copper slag and hydrated lime as an activator**”, in fulfilment of the requirement for the degree of Master of Engineering in Geotechnical Engineering in the Department of Civil and Environmental Engineering, Faculty of Engineering and Technology, BIUST.

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ABSTRACT

Roads constructed on weak sub-grade clay soils with high plasticity may be affected by the behaviour of the clay soils. Weak sub-grade clay soils undergo volume change due to the changes in the moisture content which triggers cracking, settlement, heaving and damage of the road pavement by breaking up the pavement. Stabilizing these types of weak sub-grade clay soil is required to overcome the swelling of the clay soil and increase the soil strength and improve the performance. The main aims of this research were to study the effect of copper slag (CS) without and with hydrated lime on the engineering behaviour (compaction, swelling potential, plasticity characteristics and unconfined compressive strength) of the composite and to identify the reaction products of the stabilized soil through X-ray diffraction and scanning electron microscopy. To accomplish these aims, extensive laboratory tests were conducted on various mixes (from 2 % up to 10 % of CS by dry weight of the weak subgrade clay soil and from the ratios of 9:1 up to 7:3 replacements by hydrated lime) which were prepared and cured under two conditions (35 ° C with relative humidity of 90 – 100 % referred to as (CC1) and 45 ° C with a relative humidity of 50 – 60 % referred to as (CC2) for up to 74 days. The plasticity characteristics and compaction properties were determined soon after mixing while the unconfined compressive strength (UCS) and the swelling potential were determined after longer curing periods.

In general, the study showed that the engineering properties (plasticity, unconfined compressive strength and swelling potential) improved with CS addition and with increasing temperature and curing period. The addition of copper slag and hydrated lime to the soil resulted in an enhancement of about 76 % increase in strength of the composite soil reported in the study. For the compaction test, the maximum dry density (MDD) reduced while the optimum moisture content (OMC) increased with increment in CS and hydrated lime content.

The main alterations in the swelling and the unconfined compressive strength are due to the development of new cementitious materials. The analytical work verified two main reactions when hydrated lime and CS were added to the weak subgrade clay soil, the hydration of CS activated by hydrated lime yield calcium aluminium silicate hydrate paste and hydrotalcite type of phase and the clay soil-hydrated lime reaction yield calcium silicate hydrate, calcium alumina hydrate and calcium alumino silicate hydrate. The scanning electron microscopy showed that aluminium silicate (Al/Si) ratio and amount of Si in the formed calcium silicate hydrate increased significantly with an increase in the curing temperature and curing period which suggests a stable and perfect crystalline form of calcium silicate hydrate. The results indicate that the use CS alone or preferably with hydrated lime significantly decrease the swelling potential and increase the strength of the soil.

The recommendations for further investigations include a study on the effect of cyclic loading of the composite soil and field trials to evaluate or measure the suitability of the composite.

LIST OF ABBREVIATIONS AND SYMBOLS

Å	Angstroms (10^{-10})
BC	Binder Content
°C	Degree Celsius
CC1/CC2	Curing conditions
CS	Copper slag
D	Lattice spacing
N	Order of diffraction
E ₄₀	Secant modulus
K	Kilo (10^3)
Kg	Kilogram
M	Milli (10^{-3})
Mm	Millimeter
M	Mega (10^6)
M	Micro (10^{-6})
Mm	Micrometer (10^{-6})
N	Newton
AASHO	American Association of state highway official
ASTM	American society of Testing and Materials
CBR	California Bearing Ratio
CC1	Curing Condition 1 (35 °C with relative humidity 90 % to 100 %)
CC2	Curing Condition 2 (45 °C with relative humidity 50 % to 60 %)
C-A-H	Calcium aluminium hydrate
C-S-H	Calcium silicate hydrate
C-A-S-H	Calcium aluminium silicate hydrate
LL	Liquid Limit
L. O. I	Loss of Ignition
MDD	Maximum dry density
NS	Natural soil
OMC	Optimum Moisture Content
PL	Plastic Limit
PI	Plasticity Index
Ph	Log ₁₀ (H ⁺ Concentration)
SEM	Scanning Electron Microscopy
T	Temperature
UCS	Unconfined Compressive strength
XRD	X-ray diffraction
XRF	X-ray fluorescence
θ	Angle of diffraction
λ	Lambda wavelength

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CHAPTER 1

INTRODUCTION

1.1 General Introduction

The growth of any country depends on a good and well-connected road infrastructure network and from which all other infrastructural developments gain successively. Highway construction and maintenance globally today requires the use of millions of tons of naturally occurring aggregates. This has become obvious where vast lengths of roads are required to be constructed to help the growth of the economies including commerce, agriculture and industries. The road infrastructures are built over sub-grade soils with many uncertainties such as settlement, heave, ground water table layer etc. It is vitally necessary to study the soil condition before carrying out the design of the pavement, since pavement and traffic loads are transferred to the sub-grade soils (Bari, 1995; Kok and Khairul, 2001).

Globally, many countries have experienced rapid increase in civil engineering construction projects and the construction industry involvement in the utilization of natural aggregates for infrastructural developments. The unavailability of the suitable naturally occurring aggregates for development of infrastructures is threatened by such increase in civil engineering construction projects. The naturally occurring aggregates are depleting at a fast rate while wastes produced from mining industries are on the increase considerably (Al-Jabri et al., 2009).

Good quality sub-grade soils are more vastly valuable for the construction of durable roads. The pavement designing entity may be confronted with unsuitable and weak sub-grade soil. To overcome such a problem in highway construction, there are many methods to utilize but, in this study, only three are considered.

First method is to import suitable material from the closest convenient source and substitute the material on the site with the imported material. Second method is by improving the in-situ geotechnical properties of the soil by standard proctor compaction method and design for the modification. The third method is to improve upon the existing sub-grade soil material

properties by adding or integrating other materials; termed “soil stabilization” (Ingles and Metcalf, 1972; Little, 1995, 1996 and Puppala et al., 2003).

Biswas and Davenport (2002) refers to slag as a glass-like by-product after a desired metal has been separated from its raw ore during the smelting process. It is extracted from ore either hydro-metallurgically or pyrometallurgically. Metallurgical industries have antecedently considered slag as a waste product (Gorai et al., 2002). According to the European Commission (2012) extractive industries waste is to be rightly supervised to check semi-permanent stability of disposal facilities, to prevent or reduce any water and soil pollution developing from acid or alkalic drainage and heavy metals leaching. Production of value-added products, recycling and recovering metal, and the disposal in slag dumps, stockpiles or tailing dams are the present management options.

Copper tailings dam failures have been related with stringent environmental impacts over the past decade. Grimalt et al. (1999) reported on the 1998 Aznalcollar tailings pond failure in Spain, where approximately 2 million cubic meters of mud containing heavy metals spread over 4,286 ha of land and surface water. Lungu (2008) highlighted that tailings spillage of Nchanga Copper Processing Plant in 2000 in Zambia released high concentration of heavy metals into the nearby surface water, thereby contaminating the local source of water supply. Similarly, Mutombo et al. (2011) noticed contamination of the Katamanda River in Lubumbashi in Democratic Republic of Congo. They observed that an adjacent heap of slag tailings generated by Electronic Foundry Panda Copper Plant (FEP) in Lubumbashi released metallic trace elements such as Co, Cd, Pb, Zn and Cu into the Katamanda River. Therefore, the copper tailing and copper slag should be well managed and be used as one of the best alternative, environmentally-friendly construction material.

Lime is a by-product formed from heating process of limestone at a high temperature in an industry (<https://www.lime.org> accessed June 2015). Limestone purely composed of calcium carbonate [CaCO_3], can be transformed into quicklime (CaO) through a calcination process at high temperatures in kilns, while quicklime can be transformed into hydrated lime [$\text{Ca}(\text{OH})_2$] by slaking it with water as shown in Eqns 1.1 and 1.2 (Boynton, 1980).

Quick Lime reaction: $\text{CaCO}_3 + \text{heat} \rightarrow \text{CaO} + \text{CO}_2$ [Eqn. 1.1]

Hydrated Lime Reaction: $\text{CaO} + \text{H}_2\text{O} \rightarrow \text{Ca}(\text{OH})_2$

[Eqn. 1.2]

One of the oldest and most vital chemicals ever to exist is lime. The ancient Romans utilized lime in building and road construction, and its usage continues up to today. Lime is among one of the earliest industrial commodities or trade goods cognizant to man (<https://www.lime.org> accessed June 2015). Lime for today's commercial purpose must satisfy the demanding chemical and physical specifications.

Blast furnace slag, fly ash, pulverized fuel ash, steel slag, silica fumes and lime are being utilized successfully by construction industries as admixtures, binders, partial replacement of coarse or fine aggregate in concrete, fine or coarse aggregate in pavement construction, coarse and fine aggregate in improving weak subgrade construction materials. Mining industrial waste management schemes are developed by researchers worldwide for particular demands to meet an environmental-friendly, economic and sustainably management utilization (Siddique and Khan, 2011; Chen, 2006; Al-Jabri et al., 2009; Wu et al., 2010).

1.2 Justification

The Serowe sports complex in Botswana was constructed on top of an approximately 6 km band of black cotton clay soil which runs up to 12 meters deep in some places resulting in some structural defects. The sports complex holds a stadium which has a seating capacity of 6,000. It houses a warehouse, netball, softball, basketball, volleyball and tennis courts as well as an administrative building. The Serowe soil under study covers approximately 25 km of black cotton clay soil with high plasticity.

Figures 1.1 and 1.2 illustrate structural defects to some of the now abandoned facilities of the Serowe sports complex due to damage as a consequence of the inadequate geotechnical properties of the underlying soil.

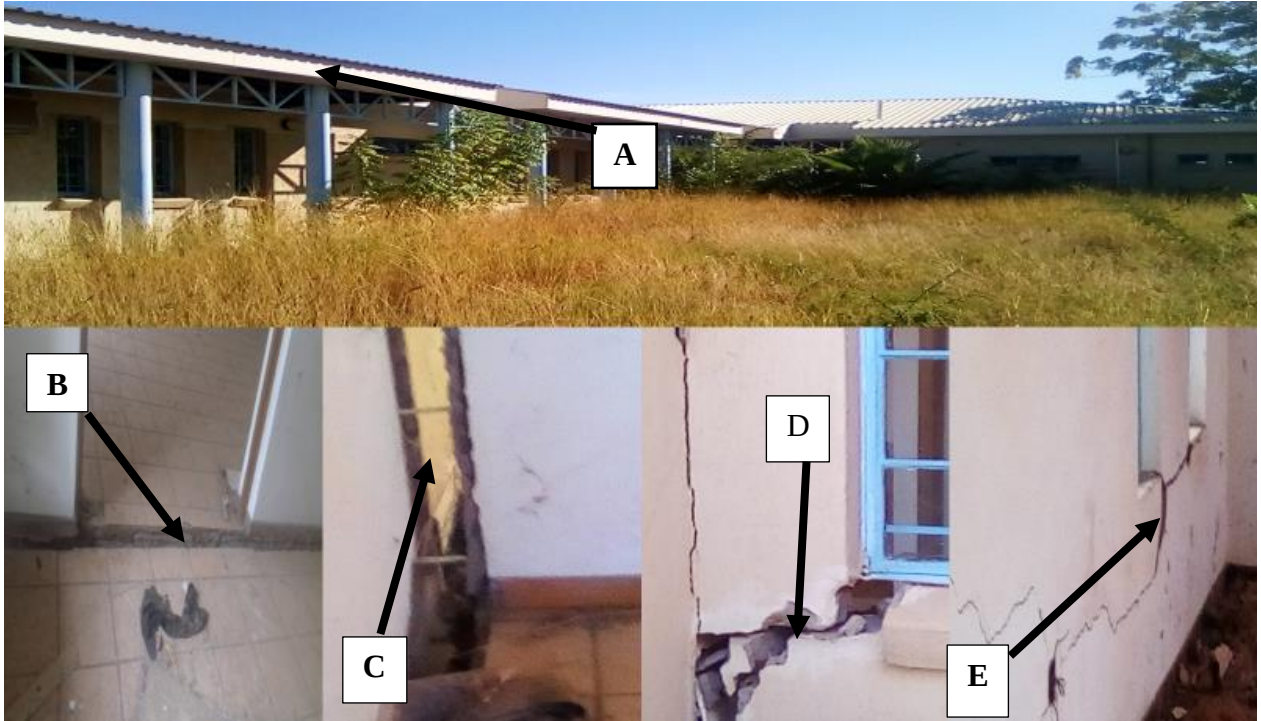


Figure 1.1: A) abandoned administrative building, B) left section of the floor and foundation separated from the center section, C) building wall separated, D and E) Cracks in the building wall (Serowe Sports complex in Botswana, May 16, 2015)

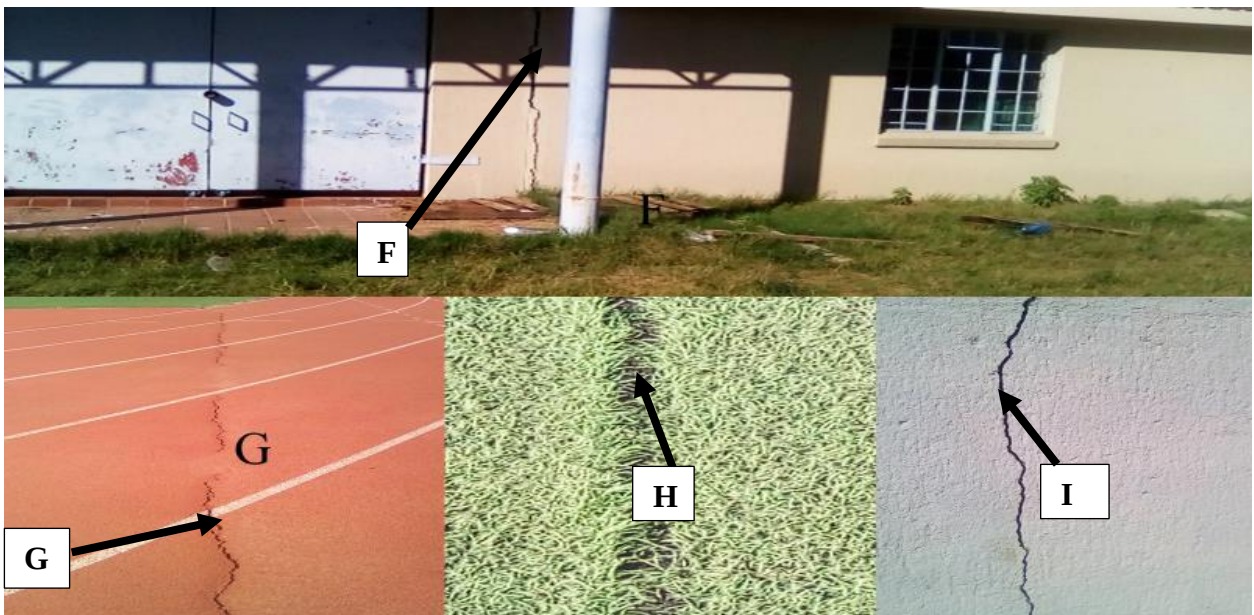


Figure 1.2: F) cracks in abandoned warehouse, G) cracks on tracking perimeter, H) cracks in artificial turf, I) crack in wall around the sports complex (Serowe sport complex in Botswana, May 18, 2015)

Previous studies conducted to evaluate the suitability of copper slag use in the construction industry has considered copper slag as follows (Collins and Ciesielski, 1994; Zong, 2003 and Caliskan and Behnood, 2004 and Al-Jabri, 2006):

- Pavement material or aggregate for highway construction
- Portland cement replacement (as a binder)
- As an abrasive tool
- Fine and coarse aggregate replacement in concrete etc.

Numerous studies conducted on copper slag utilization has mostly concentrated on improving the strength of concrete while limited study has been conducted on the utilization of copper slag in the improvement of weak subgrade clay soils for road construction (Collins and Ciesielski, 1994; Zong, 2003 and Caliskan and Behnood, 2004; Behnood, 2005; Al-Jabri, 2006 and Wu et al., 2010). Notwithstanding, there is limited research conducted regarding the utilization of copper slag as admixture/stabilizer or a binder to improve weak clay subgrade material. This current study was executed in order to provide information on the possible application of copper slag as admixture/binder for strength improvement of clay subgrade material; and to determine its suitability as subgrade material for road construction. Copper slag is used in conjunction with hydrated lime to improve weak clay subgrade.

The BCL mine produces copper slag as a waste/by product locally in very large quantities during the production of copper and Nickel. The open dumping storage of the BCL mine copper slag is approximately above 500 million tonnes which has environmental effects on the surrounding region to some extent. This therefore necessitates the need for an alternative use of the by-product that is stored up on vast land in the mine locality.

The motivation of this study is based on the environmental concerns and economic cost of disposal of waste copper slag over the past decades and the lack of naturally occurring quality subgrade materials for road construction that leads to pavement failure in Botswana. The usage of this material is thought to have environmental and economic benefits.

1.3 Aim

The main aim of this research was to study the stabilization of weak subgrade clay soils utilizing copper slag without and with hydrated lime as an activator.

The specific objectives of the research are as follows:

1. To study the effect of copper slag on the engineering behaviour of the weak subgrade clay soil with and without hydrated lime as an activator.
2. To identify the reaction products of the stabilized soil through X-ray diffraction and scanning electron microscopy.

1.4 Scope

This research was to study the stabilization of weak subgrade clay soils utilizing copper slag (CS) with and without hydrated lime as an activator in improving the engineering properties of weak subgrade clay soil. In order to achieve the objectives of this research, the scope is as follows:

- The materials comprising Serowe clay soil, hydrated lime and copper slag were characterized. Characteristics investigated include particle size distribution, specific gravity, liquid and plastic limits, optimum moisture content, maximum dry density and the main oxides of the stabilizers and the soil.
- To study the effect of copper slag on the engineering behaviour of the weak subgrade clay soil without hydrated lime as an activator (mainly the unconfined compressive strength, (UCS), compaction, plasticity and swelling characteristics) of the soil under varying curing conditions and periods.
- To study the effect of copper slag with hydrated lime as an activator on the engineering behaviour of the soil with respect to unconfined compressive strength, compaction; plasticity and swelling characteristics.
- To study the effects of the quantity of copper slag replacement by hydrated lime on the UCS, compaction, swelling and plasticity and with varying effect on curing conditions were also studied.
- To identify the reaction products of the stabilized soil through X – ray diffraction (XRD) and scanning electron microscopy (SEM).

1.5 Thesis Structure

This thesis comprises of seven Chapters; Chapter 1 is an introduction of the thesis. It comprises a general introduction, justification, aim, and scope and thesis structure. Chapter 2 which is the literature review of the thesis, describes clay soil mineralogy and structure in general with more emphasis on kaolinite, montmorillonite clay minerals, copper slag stabilization and effect of CS on the engineering properties of the soil and lime-clay reactions. In Chapter 3, the materials used in the research are discussed and the description of the testing equipment and testing methods are presented. Chapter 4 presents results of the engineering tests conducted while Chapter 5 presents the results of the analytical tests conducted. Chapter 6 discusses the tests results. The conclusions derived from the investigation, with recommendations are presented in Chapter 7.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

In this chapter, a literature review is carried out on clay soil mineralogy, the structure of the clay mineral in particular montmorillonite and kaolinite, problems associated with expansive clay, production and composition of copper slag (CS), properties of copper slag (CS), and copper slag (CS) stabilization. The effect of copper slag (CS) on the engineering properties of the soil and the clay-lime reactions. Particular consideration is given to lime stabilization, including the general soil-lime reaction, volume stability, plasticity, strength and the compaction characteristics. Currently, very limited literature exists in the application of CS in stabilizing weak sub-grade soils.

Gibbs and Holtz (1956) refer to expansive soil as soil which displays high swelling and shrinkage characteristics, moderate to high plasticity and low to moderate strength. Expansive soils exhibit large volume change when subjected to seasonal variations in moisture content (Nelson and Miller, 1992). The potential swelling characteristic is related to the percent swelling and swelling pressure.

According to Bell (1996) when lime is added to clay soil, the changes that occur are divided into two classes called stabilization and modification. In stabilization, calcium ions attack clay minerals due to chemical reaction between clay minerals and calcium hydroxide, which leads to the formation of new materials namely calcium silicate hydrate, calcium aluminate hydrate and calcium aluminate silicate hydrate. In the modification process, calcium ions are adsorbed by clay mineral particles in cation exchange reaction. The amount of lime content that would effect modification has been established to range between 1 % - 3 % by dry weight of the soil, while in stabilization the range is 3 % - 8 % by dry weight depending on the proportion and type of clay minerals present in the soil (Diamond and Kinter, 1965; Bell, 1988a; Ingles and Metcalf, 1972).

The clay-lime reaction, clay minerals and soil structure are vital in the understanding of soil stabilization process in general.

2.2 The Composition of Soil and Clay Minerals

2.2.1 Soil Composition

According to Peck and Terzaghi (1948) soil “is a natural aggregate of mineral grains that can be classified by gentle means of agitation in water”. Soil is regarded in general as any loose material at the earth’s crust.

2.2.2 Clay Minerals

Any fine-grained, natural earth material which when mixed with water develops plasticity is called clay. The fundamental characteristic of clay soil is plasticity. Clay comprises of alumina, water and silica with little amount of alkali and iron. Clay that does not have plasticity when mixed with water is called flinty clay (Grim, 1953). Civil engineers define the maximum clay particle size as 2 μm .

Clay minerals play an essential role in industry product and building construction (Van Olphen, 1977; Mateos, 1964). In addition, clays are of particular concern to geotechnical engineers, highway engineers and civil engineers globally in construction projects. The presence of clay minerals in the soil is essential as it controls the soil properties such as volume stability, plasticity and reactivity. Non-clay minerals may also be present in the soil and these can be identified by X-ray diffraction analysis, where different non-clay minerals are separated from the clay minerals on the X-ray diffraction diagram.

2.2.3.1 General Clay Minerals Structure

According to Grim (1968), the atomic lattice of the majority of clay minerals consists of a tetrahedral silica unit and an octahedral aluminium unit or an octahedral magnesium unit. The tetrahedral silica unit comprises of silicon atom at equal distance of four oxygen atoms or hydroxyl ions that are needed to balance the electrical charge of the structure. The tetrahedral silicate groups are usually placed to form a hexagonal network that is repeated to create a sheet with distinctive composition of $\text{Si}_4\text{O}_6(\text{OH})_4$ (Figure 2.1). The tetrahedral units are positioned in such a manner that all the points above are in alignment with the plane of their bases (Brown, 1984).

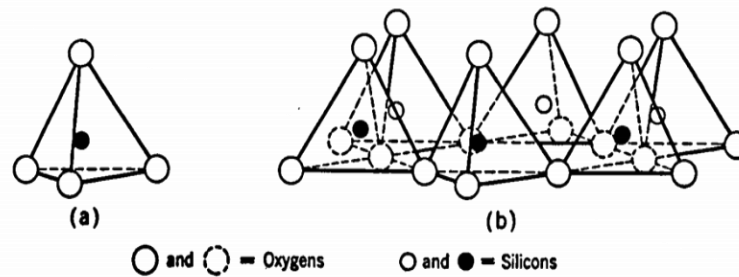


Figure 2.1 Sketch showing (a) single silica tetrahedron unit and (b) the sheet structure of the tetrahedra piled in hexagonal network after Grim (1962)

The octahedral crystal unit structure center is occupied with aluminium, magnesium or iron atom which is surrounded by six hydroxyls (Figure 2.2). When aluminium is present, two thirds of the potential cations positions are filled up to balance the structure electro-charge forming gibbsite with a chemical formula $\text{Al}_2(\text{OH})_6$. The presence of magnesium only indicates that all potential positions are occupied resulting in the clay mineral brucite with a formula $\text{Mg}_3(\text{OH})_6$ (Grim, 1968).

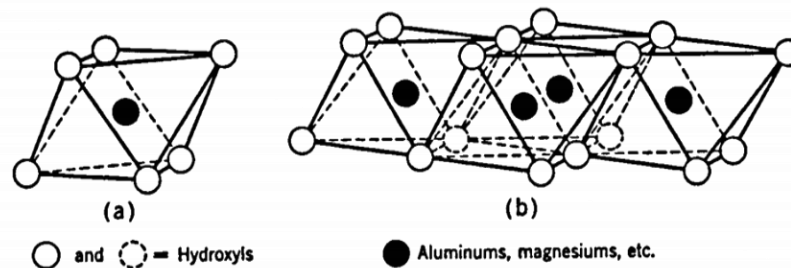


Figure 2.2 Sketch showing (a) single octahedron unit and (b) sheet structure of octahedral unit (Grim, 1962)

Majority of the clay minerals consist of two main structural layers piled in various sorts. The order of arrangement of the layers influences the clay mineral chemical and physical characteristics greatly. There are clay minerals that are fibrous with different structural units from the above mentioned one (Grim, 1953; Van Olphen, 1977). The structural unit consists of silica tetrahedron placed in double chain order (Figure 2.3). The structure looks similar to that of silica tetrahedrons sheet, with one plane in a continuous direction while the other is subjected to the width which is about 11.5 Å.

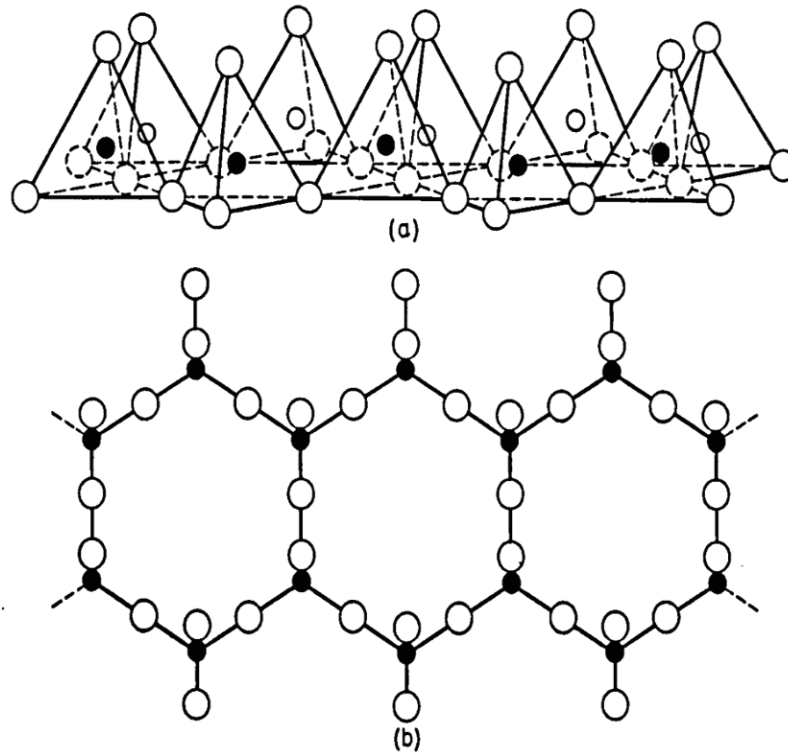


Figure 2.3 Sketches of double chains silica tetrahedra as in the amphibole structure types of clay minerals (a) in perspective (b) projected on the plane of the bases of tetrahedrons (Grim, 1962)

According to Grim (1962) montmorillonite, kaolinite and illite are the most common clay minerals in engineering practices during construction.

2.2.3.1 MONTMORILLONITE

The weathering of volcanic ash under deplorable drainage condition results in the formation of montmorillonite. Montmorillonite holds a higher liquid limit and higher activity than kaolinite and illite. The shrinkage and swelling potential of montmorillonite are several times higher than its volumetric dryness. The basic structural unit of montmorillonite is a gibbsite sheet $\text{Al}_2(\text{OH})_6$ between two sheets of silicate. This mineral structure is three-layers sheet units arranged one above another (Yong and Warkentin, 1966).

Relatively weak hydrogen bonds exist between successive crystal units. The bond strength lies upon exchangeable cations involved. Other cations and water molecules can go in between the sheets, making them to be separated and cause expansion of the particles and split the mineral

up into unit layers. Montmorillonite has a small particle size with a specific surface area of 800 g/m² considering area of the surface charge and it displays high plasticity, swelling, cohesion and shrinkage characteristics based upon exchangeable cation involved.

Grim (1968) showed the chemical formula for montmorillonite as $(OH)_2 Si_8Al_4O_{20}.nH_2O$ with approximate composition of 5 % of H₂O, 28.3 % of Al₂O₃ and 66.7 % of SiO₂. Aluminium can partially substitute silicon in the tetrahedral sheet and aluminium can be substituted by magnesium. This process of substitution is called isomorphous substitution which is the prime factor of influences in cation exchange capacity. Soils with large magnitude of montmorillonite are unsuitable sub-grade material due to the propensity of large quantity of water adsorption and large volume changes during wet and dry conditions (Mitchell, 1976).

2.2.3.2 KAOLINITE

The kaolinite structure consists of a single silicate tetrahedral sheet and single alumina octahedral sheet. Kaolinite minerals consist of piled layers with hydrogen bonding between hydroxyls of alumina sheet and oxygen of the silicate sheet (Grim, 1968). Due to the strong hydrogen bonding, the typical kaolinite crystal comprises of approximately 100 layers of kaolinite mineral piled unitedly and it is very hard to disunite. Kaolinite clay mineral has the largest crystals and smallest specific area of 10-20 g/m² which is due to the strong hydrogen bonding, water molecules and ions between layers which make it hard to break through the layers. The lattice of kaolinite is regarded as non-expansive based on the surface area at which water molecules is attractable to; is limited to the external surface. Kaolinite has a very low plasticity than other clay mineral types.

Kaolinite clay mineral has a lattice ratio of 1 silica :1 alumina sheet. The structural formula of this mineral is $Si_4Al_4O_{10}(OH)_8$ with the approximation of the composition of 13.96 % of H₂O, 39.50 % of Al₂O₃ and 46.54 % of Si₂. The interlayer basal unit spacing is 7.2 Å and the number of isomorphous substitution is less and the flawlessness of the crystal is greater than other clay minerals (Ross and Kerr, 1931; Grim 1962).

2.2.3.3 *ILLITE*

Illite has a structure similar to that of montmorillonite. What make them different are their individual layers which are joined together by potassium ions. The system is especially stable because the potassium ions precisely fit within the hexagonal space formed by oxygen ions on the silicate sheets surface. The surface activity and specific surface are lesser than those of montmorillonite. It has an intermediate activity between montmorillonite and kaolinite (Grim, 1962).

2.3 Adsorption of Water at Clay Surface

According to Gillot (1987); Yong and Warkentin (1975) clay particles in soils are surrounded by layers of water molecules adsorbed on to the clay particle always during wet conditions which keeps the soil hydrated. These water molecules are regarded as a part of the surface of clay due to the consideration of the behaviour of clay soils. Clay soil properties including strength, plasticity, compaction and water movement in soil are all affected by the water layer.

Bell (1983) indicates that due to the changes in moisture content resulting in swelling and shrinkage, clay soils undergo volume changes. Clay soil ability to absorb water contributes to swelling while when it is dry it shrinks. According to Gillot (1987) clay soil properties including cation exchange capacity, organic matter content, degree of consolidation and specific surface area are influenced by cementitious agents' presence. The minerals are bonded together by cementitious agents that result in the suppression of the swelling of the clay soil by surface area open to moisture reduction, and by enhancing the materials strength which places an internal expansion constraint.

2.4 Plasticity and Activity of Clay

Atterberg (1911) proposed that soil plasticity properties should be indicated in terms of liquid limit (LL), plastic limit (PL) and plasticity index (PI). Allen (1942) defined plastic limit as the lowest moisture content expressed as a percentage by weight of oven dried soil at which the soil can be rolled into threads 3mm in diameter without breaking into pieces. Liquid limit is the moisture content expressed as a percentage by weight of the oven dried soil at which the soil will just begin to flow when jarred somewhat. Plasticity index is defined as the difference

between the liquid limit and plastic limit. It is the range in which the moisture content of the soil is plastic. According to Yong and Warkentin (1966); Terzaghi (1925) water content below which the physical properties of the soil no longer correspond to those of free water is considered as the lowest water content at which the cohesion between particles or groups of particles is sufficient to allow movement but higher to allow particles to maintain the molded position.

Grim (1962) refers to the activity of the soil as the ratio of the plasticity index to the clay size fraction (percentage by weight of the particles finer than $2\mu\text{m}$). Activity has been considered a very useful value in indicating plasticity index of the clay fraction of the soil. Activity indicates the ability of clay soils to react with chemical agents' present in the soil. The activity of kaolinite varies from 0.01 to 0.41 and that of montmorillonite varies from 0.5 to 7 all depending on the proportion of clay.

2.5 Ion Exchange

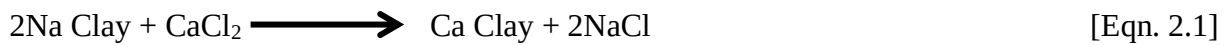
The substitution of one ion on the clay lattice surface by another ion is called ion exchange. Clay soils physical properties depend on exchangeable ion and the exchange of ions are of great importance where clay is being used. The types of exchangeable cation available in the clay make the plastic properties of clay enormously different.

According to Grim (1962, 1968) clay soil surface is generally negatively charged. This leads to the attraction of cation to the surface of the particle. He indicated that the most important sources of negative charge on the clay are as follows:

- a. Replacement with the lattice structure of trivalent aluminium Al^{3+} for quadrivalent silicon Si^{4+} in the tetrahedral sheet and lower valence ions Mg^{2+} for trivalent aluminium Al^{3+} in the Octahedral sheet leads to unbalanced charge within the clay structure of some minerals of the clay. The imbalanced charges might be balanced either by other lattices (OH^-) for O^{2-} adsorption of positive cations.

- b. Due to broken bonds around the edges of the silicate-alumina unit leaving unbalanced charges which are balanced by adsorbed cations. The exchange capacity increases as the particle size decreases based on the number of broken bonds per unit mass.

Yong and Warkentin (1975) referred to exchangeable cation as the positively charged ion from salts in the pore water that is attached to the clay particles surface to balance the surplus negative charge. The occurrence of cation exchange is because one cation is substituted by similar valence. For instance, when clay with sodium as the exchangeable cation is washed with calcium chloride solution, each one of the calcium ions will substitute two sodium ions and this leads to the expulsion of sodium from the solution (see Eqn 2.1).



The cations are arranged in series on the basis of their substituting power. In general, the substitution order of adsorbing cation is: $\text{Li}^+ < \text{Na}^+ < \text{H}^+ < \text{K}^+ < \text{NH}_4^+ < \text{Mg}^{2+} < \text{Ca}^{2+} < \text{Al}^{3+}$. Any cation at equal concentration will tend to substitute those to its left in the series (Grim, 1968).

Exchangeable cation is defined as the cation exchange capacity which is expressed in milliequivalent of cations per 100 gram of dry clay soil ($\text{meq}^{100}/100\text{g}$). The definition of milliequivalent is possibly one milligram of hydrogen ions (H^+) or number of other cations which will substitute it on the surface of the clay mineral. The measurement of cation exchange capacity is at pH 7 and when the pH is less than 5, the cation exchange capacity is at a constant (Grim, 1962). Table 1.1 shows the cation exchange capacity for the three most common clay minerals.

Table 2.1 Values of Cation exchange capacities by (Wu, 1976)

Clay Mineral	Exchange Capacity (meq ³ /100g)
Montmorillonite	80 – 150
Illite	10 – 40
Kaolinite	3 – 15

It is observed from Table 2.1 that the clay mineral with largest net negative charge is montmorillonite due to its large specific surface area. The cation exchange capacity of montmorillonite is tremendously high in comparison to illite and kaolinite.

2.6 Expansive Soil

According to Fadl (1971) the clay fraction of the mineralogical composition of expansive soils is preponderantly montmorillonite. There might be fewer quantities of kaolinite and illite present. Magnesium and calcium are the main or major exchangeable cations with lesser quantities of potassium and sodium. Montmorillonite clay minerals always swell when there is wet condition while it shrinks when there is a dry condition. Figure 2.4 illustrates soil with higher montmorillonite clay mineral when dry (Gradusov, 1974).



Figure 2.4 Illustration of cracks in montmorillonite clay mineral rich dry soil (after Brady, 1990).

The engineering properties of expansive soils (Lyon Association, 1971; Chen, 1975) are summarized in Table 2.2. Jones and Holtz (1973) showed that expansive soils cause damage to structures specifically pavements and buildings than any natural disaster. There are many reported failure problems relating to structures founded on expansive soils worldwide such as cracking and break up of pavement, heaving and building foundation failure. Highways are the most affected by expansive soil than other structures (Xidakis, 1979). Many authors have examined the initial water content effect on the amount of swelling and swelling pressure. It has been observed that each expansive soil specifically possesses definite initial water content that does not allow swelling to occur.

Table 2.2 Expansive clay soils engineering properties, (Chen, 1975; Anon, 1971)

Engineering Properties	Percentage and Unit
Liquid Limits	22 % - 124 %
Plasticity Indices	11 % - 74 %
Shrinkage Limits	10 % - 30 %
Classification using AASHTO	A-7-5 to A-7-6
Organic Matter Content	0.5 - 2.5 %
Specific Gravity	2.65 - 2.75
CBR reported after 4 days of soaking period	< 2
Effective Cohesion	2.5 - 3.00 kN/m ²
Effective Angle of Internal Friction	15 ° - 25 °
Volume Expansion	40 % - 50 %
Free Swelling	100 % - 400 %
Swelling Pressure	1000 - 3000 kN/m ²

2.6.1 Expansive Soil In-Situ Heave

Heave in soil is due to swelling of the soil while settlement is due to shrinkage that occurs in the soil when dry. Notwithstanding, heave is most severe to roads structures and other structures than compression since brittle structures are amenable to damage due to doming related to heave. Doming induces tensile stress in the topmost portions of the structures while shrinkage might cause ditches in the foundation of the structures leading to tensile stresses development in the foundation (Xidakis, 1979).

Various methodologies and laboratory tests have been designed to establish in-situ heave, however it is hard to find the laboratory duplication of the field conditions. Heave is primarily referable to moisture content increase beneath surface area covered. This process is very slow and there is no environmental condition dependency. There is unpredictable heaving that can break sewer and water pipes that can lead to dangerous damage in some special cases. Seasonal variations of soil water content around highway pavements and building surrounding area is associated with cyclic expansive compression development (Kassiff et al., 1969; Chen, 1975).

Parcher and Liu (1965) indicate that soil swelling in general is presumed or assumed to take place normal to the surface since the adjacent soil restrains lateral swelling. Lateral swelling

might be relatively large and might be greater than the normal swelling to the surface area due to the seriousness of the cracking and fissuring.

The principal element which may affect the in-situ movement of expansive soil is the time factor, which is one of the significant factors. Because extremely expansive clay soils tend to display less permeability, ample time is essential for the swelling to finish. The total swelling potential might not be attainable for the design life of the building (about 30-40 years) if there isn't enough time (Gromko, 1974).

2.6.2 Treatment of Expansive Soils

Treatment of expansive soils is not always easy and economical to conduct. Below is the summary of some of the methods:

- a. Reduce the water content change in expansive clay soil after construction. The simplest way is to build suitable drainage control systems and put vegetation coverage under control.
- b. Realignment of the project, (for example, highway or airfield) to avoid expansive soil deposits or excavating and backfilling of highways, buildings and airfield, where ever possible.
- c. Utilizing an appropriate structural design on the expansive soils established on the estimated mean vertical heave for environmental conditions.
- d. Utilizing any of the several methods of soil stabilization. Heave potential of soil can be controlled in several ways which include compaction, where compaction is controlled at higher moisture content and at a lower density in a chemical stabilization with cement and lime. In this instance of expansive clay soils, lime stabilization is encouraged and recommended than cement stabilization due to the retardation montmorillonite may pose on cement hydration and workability of the clay cannot be improved by cement. If there is higher percentage of organic matter content in the soil, chemical stabilization might not be of use (Kassiff et al., 1969; Anon., 1971; Gromko, 1974 and Chen, 1975).

One of the most valuable pieces of research carried out on Egyptian swelling soils is the work of Mowafy et al., (1990). They developed three successful techniques for the treatment of Egyptian expansive soils. They found a decrease in the magnitude of swelling and swelling pressure with an increase in the initial water content. They suggested compacting these soils in the field at high moisture content to suppress the swelling potential. They also found that mixing coarse fractions of granular material caused a substantial decrease in the swelling potential of the swelling soils, due to the reduction in the clay fraction in soil mixtures.

2.6.3 Organic Matter Content Determination

Head (1992) suggested that the organic matter content of soils should be determined as a pre-treatment to eliminate colloidal organic matter from the soil. He used the hydrogen peroxide digestion method to pre-treat the soil to eliminate the organic matter, where the loss due to hydrogen peroxide treatment is the organic matter content (OMC) expressed in Eqn. 2.2 as:

$$\text{Loss} = \frac{m_3 - m_2}{m_1} \times 100 \% \quad [\text{Eqn. 2.2}]$$

Where Loss = Organic Matter content, m_1 = is the weight of the oven dry soil sample placed in the conical flask, m_2 = weight of the soil sample after the application of hydrogen peroxide, filtered and washed with distilled water, m_3 = the weight of the dish and its content.

2.6.4 Free Swelling Percent

Head (1992) defined free swell or differential free swell as the increase in volume of the soil from loose dry powder to after it is poured into water which is expressed as a percentage of the original volume. Figure 2.5 demonstrates the procedure in determining free swelling. The free swell is expressed in Eqn. 2.3 as:

$$\text{Free swell} = \frac{V - 10}{10} \times 100 \% \quad [\text{Eqn. 2.3}]$$

Where 10 = the volume of dry soil specimen read from the graduated cylinder as 10 ml, v = the volume of soil specimen read from the graduated cylinder containing distilled water.

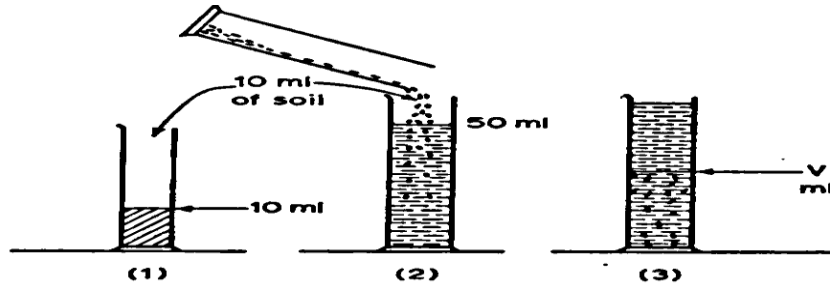


Figure 2.5: Diagram of free swelling test (Head 1992)

2.6.5 pH Value Determination

According to BS: 1924 (1990) pH recording at room temperature should be corrected to a 25 °C standard temperature. The standard suggests Eqn 2.4 to be used for correcting room temperature to standard temperature which is expressed in Eqn. 2.4 as:

$$pH_{25} = pH_T + 0.03 (T-25) \quad [Eqn. 2.4]$$

Where pH_{25} is the pH at 25 °C

pH_T = pH at prevailing temperature (°C)

T = prevailing temperature

2.6.6 Specific Gravity

IS: 2720 – Part 3; Section 2 (1980) defines specific gravity as the ratio of mass in air of a given volume of dry soil solid to the mass of equal volume of distilled water at 4 °C or the ratio of unit weight of the soil to that of water. In this method, Specific gravity (G_s) is determined by means of the pycnometer and it is expressed in Eqn. 2.5 as:

$$G_s = \frac{(m_2 - m_1)}{(m_2 - m_2) - (m_3 - m_4)} \quad [Eqn. 2.5]$$

Where m_1 = mass of dry pycnometer with its cap.

m_2 = weigh of oven dried soil of the under size of 4.75 mm sieve and the pycnometer.

m_3 = mass of the pycnometer, soil and water. Where water is added to cover the soil and

the cap screwed on. The pycnometer is then shaken well and thereafter connected to a vacuum pump to remove entrapped air for about 10 to 20 minutes. After the air has been removed, the pycnometer is filled with water and weighed. The pycnometer is then clean by washing thoroughly and filled completely with water upto its top with cap screw on then weighed after drying the outside thoroughly (m_4). G_s = specific gravity of the sample

2.7 Soil Stabilization

Several authors have described the properties of soil altered by the addition of other materials (Grim, 1968; Ingles and Metcalf, 1972; Arabi and Wild, 1989 and Higgins et al., 1998). These properties include strength, volume change and plasticity. Some chemical agents utilized include cement, fly ash, sodium chloride, alkali-activated blast furnace, lime etc. with their modification abilities greatly dependent on the magnitude of the clay minerals, type of stabilizer and percentage, moisture content, percentage of clay fraction, temperature, organic matter content in the soil, and curing time and conditions (Mitchell and Hooper, 1961).

According to Mitchell and Hooper (1961) soil stabilization is used to amend deficient soil mechanical properties in civil engineering infrastructure construction projects such as highways pavement, sub-grades, sub-bases, shallow foundation and runways. Hence, the stabilization types selected in any construction project depends on the nature of the project and the properties of the soil (Abdi, 1992).

2.7.1 Copper Slag Production

Copper slag is produced by hydro-metallurgical or pyrometallurgical production of copper from copper ores and it contains material like iron, alumina, calcium oxide, silica etc. (Gorai et al, 2002). The hydro-metallurgical method is considered as flotation method of recovering copper from secondary copper slag or by-product that still has about 0.8 % Cu and other valuable minerals in the slag content produced by pyrometallurgical method. The pyrometallurgical method of production is the only method applicable to ores containing copper-iron-sulphide minerals namely chalcopyrite and chalcobornite which are the most plethoric. The properties of copper slag comprise of the mineralogical and chemical compositions, physical and mechanical properties.

Copper slags are generally suitable for recycling, reuse and use as alternative materials in roads construction and other geotechnical applications. Copper slag can serve as an effective stabilizing agent for the improvement of weak subgrade clay soil, highway construction, embankment fill, sub-base and sub-grade. Copper slag has high angularity and friction angle up to 52 ° of aggregate which contribute to stability and load bearing capacity. It can be used as an alternative aggregate in bituminous mix and other construction aggregates (Collins and Ciesielski, 1994; Zong, 2003 and Caliskan and Behnood, 2004 and Al-Jabri, 2006).

BS 6699 (1986) suggests that whenever chemical stabilization is to be done, the chemical modulus should be calculated in order to ensure an acceptability level of reactivity of the chemical to be used (Eqn 2.6).

$$\frac{\text{CaO} + \text{MgO} + \text{Al}_2\text{O}_3}{\text{SiO}_2} > 1 \quad [\text{Eqn. 2.6}]$$

DIN 1164 also suggests that the chemical modulus should be calculated as shown in Eqn. 2.7 a and b. The chemical modulus should be more than one and the lime/silica ratio (CaO/SiO₂) should be less than 1.4.

$$\frac{\text{CaO} + \text{MgO} + \frac{1}{3}\text{Al}_2\text{O}_3}{\text{SiO}_2 + \frac{2}{3}\text{Al}_2\text{O}_3} > 1.0 \quad [\text{Eqn. 2.7a}]$$

$$1.4 < \frac{\text{CaO} + \frac{1}{4}\text{MgO} + 0.56\text{Al}_2\text{O}_3}{\text{SiO}_2} < 2.2 \quad [\text{Eqn. 2.7b}]$$

2.7.2 Mineralogical Composition of Copper Slag

Kiyak et al. (1999) through microscopic studies indicated that most copper slags are well crystallized. Moreover, iron oxides and other oxides such as silica, alumina, lime and magnesia constitute 95 % or more of the oxides. They indicated that X-ray diffraction pattern or shape shows 2FeOSiO₂, Fe₃O₄ and Ca (Mg, Fe) (SiO₂)₃ as the principal phases present in copper slag.

According to Najimi et al. (2011) the mineralogical compounds found in copper slag are magnetite (Fe₃O₄), anorthite (CaAl₂Si₂O₈), quartz (SiO₂) pyroxene (CaZnSi₂O₆) and fayalite

(Fe_2SiO_4) similar to the principal compound of copper slag used in previous study such as magnetite, fayalite and quartz (Tixier et al., 1997; Arino and Mobasher, 1999; Moura et al., 1999 and Sanchez de Rojas et al., 2008 and Alp et al., 2008).

2.7.3 Chemical Property of Copper Slag

The chemical properties and composition of copper slag from different regions around the world are presented in Table 2.7. The properties vary based on the type of processing methods used in the extraction of copper and other minerals.

Table 2.3: Typical chemical property and the composition of copper slag

No	Origin of copper slag	Al_2O_3 (%)	SiO_2 (%)	CaO (%)	Fe_2O_3 (%)	Cu (%)	S (%)	MgO (%)	K_2O (%)	Na_2O (%)
1	National Iranian Copper, Iran	3.78	40.7	5.24	44.78	0	1.06	1.16	0	0
2	Kure Copper Slag, Turkey	6.8	26.1	0.7	47.8	0.82	1.5	1	0	0
3	Copper Queen, Prince, USA	14.7	27.16	17.42	34.62	1.64	0.33	3.51	0	0
4	Etibank Ergani Copper Plant, Turkey	2.4	31.95	3.95	39.65	1.01	0	2.82	0	0
5	Tsumeb, Namibia	2.82 - 18.5	9.83 - 35.5	2.29 - 17.44	8.42 - 64.22	0.49 - 12.13	0	0.69 - 5.10	0.10 - 0.93	0.25 - 1.87
6	Copperbelt Province, Zambia	3.9 - 12.60	15.53 - 60.18	3.18 - 21.87	0.68 - 18.79	< 2	0	0.64 - 6.45	1.13 - 4.83	0.03 - 0.30
7	Okiep copper district, South Africa	3.50 - 12.16	43.76 - 66.52	1.57 - 4.22	12.76 - 29.87	0.015 - 0.53	0	1.89 - 10.28	0.32 - 0.82	0.19 - 1.54

1. Marghussan et al., (1999)
2. Yocel et al., (1999)
3. Mobasher et al., (1996)
4. Kiyak et al., (1999)
5. Etter et al., (2009)
6. Vítková et al., (2010)
7. Rozendaal and Horn, (2013)

2.7.4 Physical and Mechanical Properties of Copper Slag

Copper slag (air cooled and granulated) has many favorable physical and mechanical properties that make it suitable as construction material. Due to the high internal friction angle of copper slag, it has excellent soundness characteristics, good abrasion resistance, good skid resistance and good stability. The air-cooled copper slag aggregates are black in color and typically have a glassy appearance while granulated copper aggregates are similar to the air-cooled type but more vesicular. Copper slag can be processed into fine or coarse aggregate and screened to produce aggregates which satisfy the gradation requirements. The unit weight of air-cooled copper slag is 2800 kg/m^3 to 3800 kg/m^3 while granulated copper slag has a low unit weight than air-cooled slag ranging from 1000 kg/m^3 to 1500 kg/m^3 . The unit weight of copper slag is comparably higher than that of conventional aggregate. Water absorption capacity of air-cooled slag is typically very low about (0.13 %), while granulated copper slag has higher water absorption (Scullion et al., 2008). Granulated copper slag is more porous, has low specific gravity and is made up of regular shape, angular particles, mostly between 0.075 mm and 4.75 mm in size which is similar to conventional fine aggregate for concrete particle size range (Hughes and Haliburton, 1973; Emery, 1986). Table 2.8 shows the typical physical and mechanical properties of copper slag.

Table 2.4 Typical physical and mechanical properties of copper slag

Properties	Appearance and unit
Physical state at 20 °C and 101.3 kPa	Solid
Color	Black glassy
Melting Point	1027 ° C - 1341 ° C
Unit Weight	2800 kg/m^3 - 3800 kg/m^3
Bulk Density	2300 kg/m^3 - 2600 kg/m^3
Angle of internal friction	40 ° C -50 ° C
Specific gravity	2.8 - 3.8
Hardness	6 - 7 mol
Moisture Content	< 5 %
Abrasion loss	24.1 %
Sodium sulphate soundness loss	0.90 %
Water absorption	0.13 %

After Hughes and Haliburton, (1973), Feasby, (1975), Emery, (1986), EU No. 453 Commission Regulation, (2010) and Das et al., (1993)

2.5.2 Slag Activation

Slag can be activated in many different ways but the most common is by chemical activation. In chemical activation, an activator or an alkali medium is needed. Many activators have been proposed to activate slags. The most commonly used activators are ordinary Portland cement, calcium sulphate, sodium hydroxide, sodium sulphate, calcium hydroxide and sodium carbonate (Gjorv, 1989). Wu et al. (1990) proposed that sodium sulphate, potassium aluminum sulphate and sodium hydroxide can be used as activator and can assist in breaking the Al-O and Si-O bonds. According to Wild and Tasong (1999) when lime was employed as an activator in a study to influence slag in the sulphate resistance of lime-stabilized kaolinite, they noticed an optimum slag/lime ratio of 1:5 to activate the slag and to prevent it from attack by the excess sulphate solution. They also observed that the lime activated slag hydration reaction is faster than that of pozzolanic reaction of lime-clay. The principal reaction products of slag activated by lime are calcium aluminate silica hydrate (C-A-S-H) gel and hydrotalcite type phases made up of magnesium due to the high alumina and silica content.

The most commonly used activator is Portland cement. The reaction of slag with Portland cement and water is a complex process. Water hydration of Portland cement produces mainly calcium hydroxide $\text{Ca}(\text{OH})_2$ and calcium silica hydrate (C-S-H) gel. In the hydration of blended Portland cement minor quantities of alkalis are released while slag is mainly activated by the hydration product $\text{Ca}(\text{OH})_2$ (Hakkinen, 1993). Therefore, lime in the form of $\text{Ca}(\text{OH})_2$ may be added as an additive or released from Portland cement hydration due to the high aluminum, iron and silica content in slag which produces slightly different hydrates from Portland cement (Higgins et al., 1998). The principal reaction products of slag hydration are calcium aluminate hydrate, calcium silicate hydrate and small quantities of calcium hydroxide.

According to Maphee et al. (1989) slag hydration reactions are slower than the hydration of Portland cement and have a pore-blocking effect which leads to a greater ultimate strength and lower permeability. When slag is mixed with lime, it decreased $\text{Ca}(\text{OH})_2$ and improved bonding and absorptive effects of the mixture and increases the resistance of slag concrete to sulphate attack.

Daimon (1980) and Taylor (1990) indicated that a reactant participates meaningfully in the reaction process while an activator creates a suitable environment for the reaction process without necessarily playing a substantial role in the reaction. In order to understand how the addition of slag alters the soil properties, slag hydration should be studied in some detail.

2.7.6 Slag Hydration

According to Wild et al. (1989) the clay-lime reaction system is the main reaction in clay soil stabilization. Wild et al. (1989) and Diamond et al. (1964) noticed that principal reaction products in clay-lime reaction are the calcium silicate hydrate (C-S-H) with low Ca/Si ratio, crystalline calcium aluminate phases namely C_3AH_6 , C_4AH_{13} and calcium aluminate silicate hydrate C-A-S-H phases (C_2ASH_8). In slag cement blends, C-S-H also contains aluminum. The Al/Ca ratio varies with Si/Ca ratio according to Eqn. 2.8 (Richardson and Groves, 1993 and Richardson and Groves, 1997);

$$Si/Ca = 0.4277 + (2.366 \times Al/Ca) \quad [Eqn. 2.8]$$

Regourd (1980) indicates that the addition of slag to a clay-lime system modifies the reaction products of clay-lime system. Slag provides extra calcium, magnesia, silica, iron and alumina to the mixture based on the type and quantity of slag replacement. Meanwhile the main reactants present in the clay-lime system and the reaction products of clay-lime-slag system are relatively similar to those of clay-lime system.

Slag hydration effectiveness depends mainly on many factors. These factors are the chemical composition of the slag, alkali concentration of the reacting system, temperature, glass content of the slag and fineness of the slag (Kinuthia, 1997). Caijun and Day (1993) indicated that the hydration of Canadian slag forms a layer of rich Si-Al-O on the slag particle surface when water comes into contact with the slag. This layer absorbed the H^+ which resulted in an increase in the OH^- and pH of the solution but indicates an insufficient H^+ to break the bonds of the Al-O and Si-O to allow the formation of calcium silicate hydrate, calcium aluminate hydrate and calcium aluminate silicate hydrate components. According to Daimon (1980) during the preliminary reaction of slag hydration, coatings of aluminosilicate are produced on the surface

of slag grains within a few minutes of exposure to water and these layers are impermeable to water, impeding further hydration reactions. Thus, the use of slag alone shows little hydration.

2.7.6.1 Hydration Products and Mechanism of Slag Cement Mixture

Wild et al. (1999) and Tasong et al. (1999) indicate that slag hydration products utilizing alkali are mainly calcium silicate hydrate and hydrotalcite type phase containing magnesium. Song et al. (2000) indicated the formation of the hydrotalcite type phase in higher pH paste along with calcium silicate hydrate and also indicated that the pH of the mixing solution may affect the nature of the calcium silicate hydrate and its Ca/Si ratio. Similarly, the solubility of Si increases with higher pH solution while that of Ca decreases with a higher pH solution which is a result of C-S-H with a lower Ca/Si ratio. The hydration process when water is added to slag-cement mixture can be summarized as follows:

- Water begins to combine with Portland cement and calcium silicate hydrate begins to form.
- The other reactions products of Portland cement are calcium hydroxide and later sodium and potassium hydroxides.
- These alkalis activate the slag which reacts with the water to produce hydrate similar to those produced by Portland cement hydration.
- The excess silicates and aluminates from the slag hydration combine with the calcium hydroxide in a pozzolanic reaction.

Hydrotalcite is a mineral that can be easily crushed into a white powder similar to talc. It is also a hydroxyl carbonate of magnesium and aluminium and occur in nature in foliated and contorted plates or fibrous masses. The hydrotalcite type of hydroxyl compounds observed during slag cement hydration reaction have the compositions of the type in Eqn. 2.9;



Many of these minerals and synthetic compounds have the composition of this type with $R^{2+} = Mg^{2+}$, the divalent cation at the octahedral site within hydroxyl layers, $R^{3+} = Al^{3+}$ or Fe^{3+} , the trivalent cation at the octahedral site within hydroxyl layers, $R^r = OH^-$, SO_4^{4-} , is an exchangeable interlayer anion, $J = 1$; k lies mainly in the range of 0.20 – 0.33 and is equal to the ratio of $R^{3+}/$

($R^{2+} + R^{3+}$). Among natural minerals, hydrotalcite is typical with the approximate formula $Mg_6Al_2(OH)_{16}CO_3 \cdot 4H_2O$ which corresponds with $k = 0.25$ in the general formula. The brucite-like hydroxide layers of this material structure consist of positively charged k per formula unit balance by equal negatively charged interlayer anions. The layer spacing depends on the nature of the interlayer anions and the state of hydration and to a lesser extent on the magnitude of the electrostatic attraction between layers and interlayers (Brindley and Kikkawa, 1979; Mascolo and Marino, 1980; Miyata, 1983; Mills et al., 2012).

2.7.7 Effect of Copper Slag on the Engineering Behaviour of Clay Soil

The activation of slag by lime affects many engineering properties of soil. The effects of slag activated by lime on the plastic limit (PL), liquid limit (LL), plasticity index (PI), maximum dry density (MDD), volume stability and swelling potential, optimum moisture content (OMC) and the UCS are discussed below.

2.7.7.1 Effect of Slag on the Consistency Characteristics

The addition of slag to clay soil changes the consistency characteristics of the clay soil. According to Akinmusuru (1991) the consistency, strength and compaction characteristics of lateritic soil effects are depending on the slag content that varied from 0 % to 15 % by dry weight of soil. He observed a reduction in both the plastic and liquid limits and an increase in the plasticity index with increasing proportion of added slag.

Wild et al. (1996) noticed that small additions of lime to kaolinite clay produce an easily noticeable rise in the plastic limit while the liquid limit might decrease or increase in a way that there is a rise in plasticity index with an increasing percentage of lime.

2.7.7.2 Effect of Slag on the Volume Stability and Swelling Potential of Soil

Higgins et al. (1998) site trial investigation on the effect of slag and lime on the swelling characteristics of kaolinite and kimmeridge clay showed that the slag successfully decreased the swelling caused by sulphate.

Higgins et al. (1998) indicated that the effect of slag on strength and swelling properties of stabilized kaolinite-lime in the presence of sulphate revealed that kaolinite clay comprising of

slag and stabilized with lime produced a large expansion when in contact with water. They indicated that the laboratory investigation with an all-out trial demonstrating lime/slag combined can be used for soil stabilization in the presence of sulphate to prevent swelling attack.

Wild et al. (1996) suggested that the decrease in swelling is not a result of the rise in interparticle bonding, since slag addition tends to produce a slight reduction in strength. Tasong et al. (1999) also indicated that a number of factors that govern swelling are caused by ettringite formation. Ettringite is a mineral consisting of hydrous base sulphate of calcium and aluminum. The expansion rises with an increase in sulphate content and rises linearly with the absorption of water.

2.7.7.3 Effect of Slag on the Compaction Characteristics of Soil

Akinmusuru (1991) indicated that the addition of slag up to 10 % to soil raised the maximum dry density and above which the maximum dry density will decrease. This could have been due to the rise in fine powder in the mixture which contributes to the reduction in the proportion of the coarse material making it not easy to achieve better compaction.

The addition of lime to kaolinite clay reduces the maximum dry density and raises the optimum moisture content. Nevertheless, a reduction in the ratio of lime/slag produced a slight rise in the maximum dry density and a non-systematic optimum moisture content variation. They concluded that the presence of gypsum slightly raised the moisture content and the maximum dry density (Wild et al., 1996).

2.7.7.4 Effect of Slag on the strength of Soil

The primary advantages of lime/slag usage are relative to the use of lime alone at which the rate of strength development is slow and prolongs the construction time to completion and increase the long-term strength that improves the performance of the structure.

Wild et al. (1999) indicated that slag decreased the strength of kaolinite clay in the presence of sulphate as the ratio of slag/lime was raised. The outcome from slag activated by lime reaction is more rapid than kaolinite-lime reaction.

Akinmusuru (1991) noticed an increase in cohesion with an increasing slag content up to 10 % after which there is decrease in the internal friction angle with increase in percentage of slag content. He noted an increase in the CBR with an increase in slag percentage up to 10 % slag content and a reduction in value thereafter.

2.8 Lime Stabilization

Lime stabilization of soil dates back to the early part of the century. Lime in modern time utilization in road construction probably dates back to 1925, when the American state of Missouri carried out a brief observational on earth road treated with hydrated lime to decrease grooving during snow and rain (McDowell, 1966). According to Johnson, (1948) hydrated lime was used to decrease the plasticity of the soil used in the construction of Texas airfield. The lime treated segment did show good durability despite the heavy traffic volume while the segment treated without hydrated lime failed and needed extensive restoration. From that time onward, a lot of research has been conducted to decide on an adequate and efficient method of utilizing lime as a stabilizer for soils (Johnson, 1948).

After World War two, lime was largely used in many states in United States of America road construction projects (Dumbleton, 1962). Johnson (1948) indicates that approximately 5 percent of hydrated lime significantly decreased the plasticity of cohesive soils and increased the strength of fine and coarse-grained soils. Galloway and Buchanan (1951) proposed that lime effect on the soils is because of the exchange of calcium ions adsorbed by cation on clay particles surface. It was observed that soil reactivity towards hydrated lime increases the plasticity index as well as the cation exchange capacity of the soil.

Lime used in soil stabilization comes in three major forms namely hydrated lime $\text{Ca}(\text{OH})_2$, quick lime CaO and dolomitic lime $\text{CaMg}(\text{CO}_3)_2$. Quick lime is a direct product of limestone calcination. It is most effective in stabilizing wet soil to absorb water. Lime stabilization is defined as the “reaction between silica and alumina within clay soil structure with lime and water to form calcium silicate hydrate and calcium aluminate hydrate colloidal gel which bond the crystallized structures together” (Rogers et al., 1997). According to Ormsby and Kinter (1973); Diamond et al. (1964) there are two terms to consider for the basic chemical reactions, short-term and long-term. The short-term reaction comprises of cation exchange and

flocculation. In this instance, lime is a strong alkaline base that reacts with the clay chemically and leads to base exchange. Divalent calcium ions force out potassium, sodium and the monovalent hydrogen cations and change the electrical charge density around the clay particles. This results in the interparticle attraction increase which leads to flocculation and aggregation with a reduction in the soil plasticity. The long-term reaction is pozzolanic reaction, where calcium from the lime reacts with the soluble alumina and silica from the clay in the presence of water to produce stable calcium silicate hydrates (CSH), calcium aluminate hydrate (CAH) and calcium aluminate silicate hydrates (CASH) that give long-term strength gain and amend the geotechnical soil properties (Sloane, 1965; Choquette et al., 1987).

In general, between 1-3 % by dry weight of the soil of hydrated lime is needed for soil modification; while 2-8 percent by dry weight of the soil is needed for cementation to occur. Bell (1988a) proposed that 1 % of lime by weight of dry soil is needed for the stabilization of each 10 percent of clay. The exact quantity of lime required to be used should always be established at a great extent of preliminary testing.

2.8.1 Lime Stabilization Process (Clay-Lime Reactions)

Mixing lime with clay soils enhances the compressive strength and also alters other physical and chemical properties of the soil (Fessberg, 1959; Brand and Schoenberg, 1959; Wang et al., 1963; Jan and Walker, 1963; Andrew, 1966).

The practical rationale of addition of lime to clay soils are to enhance better compaction, workability and decrease the swelling and shrinkage characteristics with calcium ions by saturating the clay. Chemical interaction between lime and clay is regarded to be a significant component of the long-term improvement due to the establishment of cementitious materials that lead to the increase in soil-lime mixtures strength. This reaction can be stronger in the instance where the soil has higher silicate content (Little, 1996).

According to Bell and Coulthard (1990) there are several types of chemical reactions that can occur at the same time when lime is added to a clay soil in the presence of water. The most significant reactions include I) Carbonation, II) pozzolanic reactions, III) cation exchange, and flocculation and agglomeration (Thompson, 1966a; Bari, 1995).

2.8.1.1 Carbonation

When lime reacts with carbon dioxide in the atmosphere, it leads to the formation of calcium carbonate or magnesium carbonate; a process called ‘Carbonation’. Goldberg and Klien (1952) noticed calcium carbonate formation when soil-lime mixtures were tested in open-air curing and confirmed by Eades et al. (1962).

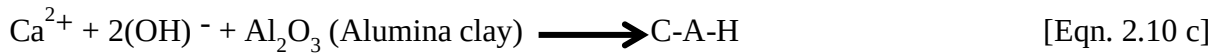
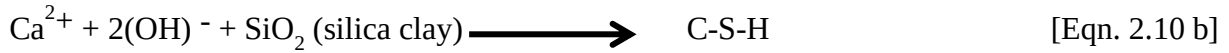
2.8.1.2 Pozzolan Reactions

Eades et al. (1962) referred pozzolanic reaction as the process where water, lime and the alumina and silica in clay react to form cementitious material. The primary origins of an increase in strength in the mixtures of soil-lime are mostly considered to be due to cementing agents. The mixtures of clay-lime reaction products are identically alike to those in cement hydration process. The reaction between lime and clay particles enhances bonding between the particles of clay and lime, and therefore the mixture’s strength increases as the duration of curing period increases where there is continuous dissolution of Al_2O_3 and SiO_2 while the reaction still proceeds (Croft, 1964).

The crystalline calcium silicate hydrate (CSH) or amorphous, crystalline calcium aluminate hydrate (CAH) and the calcium aluminosilicate hydrate (CASH) stages are the principal reaction products that are constituted in the clay lime reactions (Arabi and Wild, 1989; Sloane, 1965 and Diamond and Kinter, 1966).

Cabrera and Nwakanma (1979) through studies on red tropical soil and lime reaction mechanisms and pozzolanic reaction observed that lime got consumed at a much faster rate in the reaction at the beginning for about 5 to 7 days. The rate decreased after 7 days. Between 7 and 28 days the rate constituted about 8 % of lime consumed during the early 7 days. Suggestion was made that the soil-lime mixtures strength increase outside the 7 days is not predictable in terms of pozzolanic products reaction due to a small quantity of lime consumption between 7 days and 28 days. It was proposed that increase in strength with addition in curing period demonstrated by the soils was primarily because of the changes in the structure of the cementitious products throughout the pozzolanic reaction; that is the hydration and crystallinity increase of the products reaction did not create a new product. Pozzolanic reaction is improbable to have finished after 7 days. The pozzolanic reaction and cementitious fabrics

formation is dependent upon the type and content of clay, moisture content and curing conditions. Eqn. 2.10 a, b and c illustrate the typical soil-lime reactions.



Many authors including Eades and Grim (1960) established the identity of products that might be formed in soil lime reaction procedure through X-ray diffraction (XRD) and differential thermal analysis for products identification. The reaction conditions and the clay mineralogy are the two conditions that the long-term cementitious materials considerably depend upon.

X-ray diffraction and differential thermal analysis were used by Eades and Grim (1960) to study the reaction product of pure kaolinite clay in conjunction with hydrated lime for a mixture cured at a temperature of 60 °C (140 °F). They indicated that as the content of lime and the curing time increased, the kaolinite was consumed, and crystalline calcium silicate hydrate was formed. They proposed that large number of unlikely substance constituents may be formed before the actual silica and lime arrive at equilibrium. Eades and Grim (1960) indicated that illite and lime reaction occurs in a similar manner as with kaolinite. They indicated that X-ray data for reaction on montmorillonite shows that the mineral structure was consumed with small new materials formed. Hence, the treated montmorillonite appears to show that the compressive strength value increases based on the reaction product formed. The XRD did not show the presence of amorphous in the reaction product of montmorillonite and lime mixture. Amorphous are structureless minerals that are formed and have two characteristic properties. When cleaved or broken, they produce fragments with irregular, often curved surfaces; and they have poorly defined patterns when exposed to X-rays because their components are not arranged in a regular array.

Hilt and Davidson (1960) examined the long-term reaction products of montmorillonite-lime mixture. It was established that the calcium aluminate hydrate and calcium silicate hydrate crystals were formed at normal temperature of 20 ° C. Glenn and Handy (1963) also indicated

that the formation of calcium silicate hydrate and calcium aluminate hydrate (C_4AH_{13}) together in the mixture of lime-montmorillonite is similar to Hilt and Davidson (1960).

Eades and Grim (1963) through petrographic microscope studies on soil containing quartz and mica indicated that a relatively large amount of reaction took place in the mixture. The particles of the mica and quartz developed a cementing visible gel using X-ray diffraction to examine the same soil. They observed that calcium silicate hydrate and calcium carbonate were present in the mica and quartz soils mixed with lime.

Calcium hydroxide, kaolinite and montmorillonite mixtures were cured at 60 °C and the reaction products that were formed were examined by Diamond et al. (1964). They indicated that calcium silicate hydrate was present in the kaolinite while calcium silicate hydrate and calcium aluminate hydrate (C_3AH_{10}) existed in the montmorillonite clay. They proposed that in montmorillonite clay, the release of alumina by lime in the calcium silicate hydrate lattice stage was partly added or integrated. They also examined and noticed in another study utilizing mica and illite in conjunction with lime, that the products formed were calcium aluminate hydrate and tobermorite (Diamond et al., 1964).

Diamond and Kinter (1966) indicated that in the semi-permanent clay-lime mixtures, two major reaction products of calcium silicate hydrate and calcium aluminate hydrate were formed. It is understood from their study that calcium silicate hydrate may come or be formed in three dissimilar forms based on their reaction conditions namely C-S-H gel, C-S-H (I) and C-S-H (II).

Six sets of soils from the south-eastern part of the United States of America were treated with lime by using thermogravimetric analysis, X-ray diffraction (XRD) and the scanning electron microscopy (SEM) in order to study the reaction products (Charles et al., 1982). The soils were stabilized with 6 % by dry weight of high calcium hydrated lime (Ford, 1978). The properties determined are presented in Table 2.3.

Table 2.5: Properties of soils from the south-eastern United State of America used by (Charles et al., 1982)

Soil Name	Family	Natural pH	Liquid Limit (%)	Plastic Limit (%)	Percent passing No. 200
Cecil	Clayey, kaolinite	4.9	52	33	85
Chewacla	Fine-loamy	8.2	24	17	43
Eutaw	Very fine	5.4	70	37	99
Sumter	Fine-silty carbonic	8.4	51	24	88
Tatum	Clayey mixed	4.3	33	24	83
Wilcox	Fine-montmorillonite	4.3	72	42	90

2.8.1.3 Cation Exchange

Cation exchange in lime-clay stabilization is due to the physico-chemical reaction products where Ca^{2+} ions in lime cause magnesium or sodium ions naturally present in the clay soil to move from their original places and then displace them in the mixture. When lime is added to a soil, it produces a concentration free Ca^{2+} that substitutes different adsorptive cations on the clay colloidal surface.

2.8.1.4 Flocculation and Aggregation (Agglomeration)

Hilt and Davidson (1960) refer to flocculation as the process whereby particles of clay cluster together to create larger size aggregates in a very quick manner. Aggregation is the mutual attraction between particles via force or chemical bonding while agglomeration is the process where particles dispersed in liquid phase stick to each other and spontaneously form irregular particle flocs or aggregates. This process of clustering together to form an aggregate is believed to be induced by cation exchange. Eades and Grim (1960) and Hilt and Davidson (1960) proposed that the exchange of cation and an ensuing alteration in the double electrical layer change the electrical charge density around the area of the clay particles which lead them to be attracted electrically to one another. Hertzog and Mitchell (1961) indicated that the occurrence of flocculation was imputable to an electrolyte concentration increase in the ion exchange and

the pore water. Flocculation can develop an apparent transformation in the texture as a result of the larger aggregates size while the soil turns to be friable.

Clay soil alteration in conjunction with lime will solely and considerably be contingent upon the original dominant cation adsorbed in the double layer on the clay type. For instance, attaining calcium saturation and full flocculation in montmorillonite sodium base clay which has relatively high cation exchange capacity, will need a comparatively high lime percentage to be added. Hilt and Davidson (1960) proposed that the presence of the “lime fixation point” present the best possible percentage of lime addition and at this point there is no advancement of calcium cations that may cluster on the particles of clay. The addition of excess lime will make no modifying of the flocculation or on the plasticity of the soil.

Flocculation and aggregation are known to be predominantly held accountable for the semi-permanent change in the soil (Ingles and Metcalf, 1972; Wild et al., 1989). Flocculation and aggregation were also observed by Thompson (1966a) to be held accountable for change in shrinkage, plasticity and workability of the mixture. On the contrary, Thompson (1968) established that flocculation and aggregation, and cation exchange are not considered to be basic soil-lime reactions for the strength increase that has been widely known for lots of soil–lime mixture.

2.8.2. Effects of Lime on the Engineering Behaviour of Clay Soils

Lime additions to clay soils affect many engineering properties of the soils. Engineering properties which are affected by lime include plastic limit (PL), liquid limit (LL), plasticity index (PI), maximum dry density (MDD), unconfined compressive strength (UCS) and optimum moisture content (OMC).

2.8.2.1 Effect of Lime on the Plasticity Characteristics

Following Sabry and Parcher (1979); Diamond and Kinter (1965) indicated that the addition of lime to clay soil in general normally increases the plastic limit of the soil as cited in Akoto and Singh (1981) and Brandle (1981). The quantity of lime required to affect the alteration of the plastic limit of the soil varies from one percent to four percent by dry weight of the soil, contingent upon the mineral type and quantity existing in the soil (Bell and Coulthard, 1990).

Hilt and Davidson (1960) demonstrated that the addition of lime causes an increase in the plastic limit for all soil types. The biggest plastic limit increase was found when the primary clay mineral present is montmorillonite. Montmorillonite has the largest plastic limit than illite which is followed by kaolinite in that order. The least or minimum quantity of lime addition needed for montmorillonite clays to achieve the maximum increase in plastic limit is known as $PL_{min} = (\% \text{ of 2-micron clay}/35) + 1.25$ (Mateo, 1964).

Arabi and Wild (1989); Rogers (1988) indicated that montmorillonite liquid limit quickly reduces while its plastic limit increased when lime is added. Kaolinite liquid limit might stay fixed after the treatment with lime and also might increase. The reduction in liquid limit of montmorillonite and the plastic limit gains lead to the rapid reduction in plasticity index. Majority of the studies done on kaolinite have established that the liquid limit might likely increase or stay fixed when lime is added depending on the quantity of lime addition which makes kaolinite likely capable of change (Daniels, 1971; Abdi, 1992).

According to Dumbleton (1962) the plastic limit and liquid limit at a low lime content increase and decrease gradually respectively with an increase in lime content. He concluded that clay type, time and the addition of lime affect the plasticity. The effect of lime was also examined by Abd-el-Kader and Hamdani (1985) on stabilization of soil samples from two locations Damanhour and Elfayoum in Egypt. They observed that both plastic limit and liquid limit increase at low lime content while there was a decrease in plasticity index.

Sherwood et al. (1993) showed that London clay turned out to be very easy to compact and with improved workability after the addition of lime. They observed a change in liquid limit at low lime content while the plastic limit needed greater addition of lime to achieve maximum alteration. They also observed that different clays require different curing time to attain maximum modification.

Rogers et al. (1997) showed a general increase in the liquid limit at low content of lime. The plastic limit however needed greater addition of lime to achieved substantial alteration. Clare and Crutchley (1957) detected that one percent of lime addition increased the clay liquid limit from 72 % to 88 % and they concluded that any additional lime decreased the value. They

established that increase in curing period decreases the liquid limit at a mix of one percent lime, whereas liquid limit values increased after curing at a higher content of lime.

Majority of studies on the effect of curing period on plasticity properties of soil-lime mixtures have shown a reduction in plasticity index with increase in the plastic limit and curing period (Dumbleton, 1962; Akoto and Singh, 1981). Significant increase in plasticity index for soil-lime mixtures cured for 48 hours were compared to that of a sample examined quickly after lime addition. Curing for a longer time of about 7 days to 28 days reversed the effect with significant reduction in the plasticity index in most instances. There is no possibility of conducting Atterberg limits test after long curing time as newly cementitious fabric is formed because of the pozzolanic reaction of lime with the clays which might give misleading outcomes.

Arabi (1987) and Wolfe and Allen, (1964) studies on the effect of the curing time and curing temperature on lime-illite stabilized soil plasticity properties demonstrate a decrease in the plasticity index with increase in curing time. It was also determined that the temperature for curing had a greater extent of substantial effect on the plasticity characteristics as compared to curing period.

Howeedy and Zedan (1991) examined fly ash and lime for stabilization of alluvial soil. They observed a reduction in plasticity index of the soil by 55 % and 73 % respectively upon the addition of 10 % of lime and 10 % fly ash. The physical, mineralogical and chemical properties of the soil, lime and fly ash are presented in Table 2.4 and 2.5 respectively while the effect of the lime and fly ash on the soil Atterberg limits is presented in Table 2.6.

Table 2.6 Physical properties of clay soil and fly ash (after Howeedy and Zedan, 1991)

Properties	Clay Soil	Fly Ash
AASHO Classification	A-7-5 (16)	-
Natural water Content %	7.25	0
Liquid limit %	74	0
Plastic Limit %	34	0
Shrinkage Limit %	14.5	0
Plasticity Index %	40	0
Specific Gravity	2.6	8.52
Optimum Water Content %	28	47

Maximum Dry Density lb/cu. ft	96.4	60.65
Organic Content %	1	0

Table 2.7 Mineralogical and chemical analyses of clay, lime and fly ash (Howeedy and Zedan, 1991).

Mineral	Oxide Content %		
	Clay soil	Lime	Fly Ash
SiO ₂	51.5	15.02	50.77
Al ₂ O ₃	16.68	9.9	9.01
Fe ₂ O ₃	9.92	0.65	6.82
TiO ₂	2.52	0.25	1.77
CaO	3.52	38.94	9.24
MgO	4.75	5.07	5.7
Na ₂ O	1.96	1.65	2.7
K ₂ O	1.23	0.21	7.95
SO ₃	0.68	1.16	1.42
Loss of Ignition	6.96	27.12	4.83
Total	99.73	99.97	100.21

Table 2.8 Atterberg limits of clay soil, fly ash-soil and lime-fly ash-soil mixtures (Howeedy and Zedan, 1991).

Atterberg Limit	Clay Soil	Percentage of Fly Ash- Soil (%)					Percentage of Lime: Fly Ash (%)				
		2 %	4 %	6 %	8 %	10 %	2.2 %	4.4 %	6.6 %	8.8 %	10:10 %
LL %	74	72	71	61	57	56	71	68	65	58	22
PL %	34	35	36	36.5	38	38	35	36	38	40	41
PI %	40	37	36	34.5	18	18	36	32	27	18	11

Generally, it has been shown that there is an increase in the LL and PL of the soil at low lime content. Any further increase of lime content results in gradual reduction in liquid and plastic limits. The reduction in these properties is affected by curing period and the clay types.

2.8.2.2 Effect of Lime on Volume Stability and Swelling Potential of the Clay Soil(s)

The addition of lime to clay soils changes the moisture content and reduces the volume changes that take place in the soil (Al-Rawi and Awad, 1981). Wang and Handy (1966) noticed that lime addition to clay soil increased shrinkage limit. It was also demonstrated by Mateos (1964) that the addition of a small amount of up to 4 % lime raised the shrinkage limit of clay soil. He

concluded that effectiveness of lime in decreasing shrinkage characteristics of the soil is the quick lime than that of the hydrated lime. It was observed by Dumbleton (1962) that lime was more effective in decreasing shrinkage of London clay and silty clay than cement is.

Bell and Coulthard (1990) indicated that hydrated lime is more effective when calcium oxide content is higher, that is, when lime is added to the clay soil, it increases the calcium oxide content. They indicated that about 8 % lime addition causes similar increase in the shrinkage limit. They concluded that the addition of 4 % lime led to an increase in the shrinkage limit of 7 % to 26 %.

Contrary to the above on shrinkage limit increase, Stocker (1972) indicated that the addition of lime reduces shrinkage limit. Abd-el-Kader and Hamdani (1985) also noticed insignificant change in the shrinkage potential of two Egyptian soils. They concluded that the effect on shrinkage limit is mainly based on the types of clay.

Lime stabilization of clay soil is a means to attain a decrease in swelling pressure and swelling potential. These alteration characteristics are ascribed to the replacement of other cations by calcium in the stabilization process. There are two primary factors that are involved in swelling reduction namely, reduction of water affinity of saturated calcium clay and the cementitious materials formation which makes it possible for the prevention of expansion (Ingles and Metcalf, 1972; Mitchell and Hooper, 1961). The influence of lime on the swelling characteristics of expansive California clay was examined by Mitchell and Hooper (1961). They indicated that the swelling decreases significantly with lime alone after curing. They noticed that 4 % addition of hydrated lime to expansive clay soil decrease the swelling from 17 % to 3 %. It was also observed that lime addition to such expansive clay soil increased shrinkage limit.

Arabi and Wild (1989) indicated that when clay soil is subjected to water, swelling occurs. The negatively charged particle surfaces strongly adsorbed the water. Therefore, due to concentration gradient between the bulk solution and the electrical double layer comprising of molecules of water and exchangeable cation, an extensively surface assimilation is formed. Lime addition alters electrical double layer while the thickness of adsorbed water layer is decreased and the swelling capacity reduced.

2.8.2.3 Effect of Hydrated Lime on Compaction Characteristics

It has been observed that the addition of lime to clay soil induces two main alterations in compaction characteristics namely (a) an increase in the optimum moisture content and (b) reduction in maximum dry unit weight (Johnson, 1948; Lund and Ramsey, 1959; Andrew, 1966). The decrease in the maximum dry density and an increase in the optimum moisture content are primarily ascribable to aggregation/agglomeration and flocculation (Rogers, 1988). Also, Abdi (1992) indicated that clay substitution with lime led to maximum dry density decrease on account of higher density of clay than the lime. According to Lu et al. (1957) the type of lime affects the level to which the soil compaction characteristics are altered by addition of lime. They noticed dolomitic lime reduces the maximum dry unit weight to a smaller degree than hydrated lime. Lees et al. (1982a) conducted three soil compaction tests on 10 %, 30 % and 50 % clay, and the remaining comprising sand. They indicated that lime addition ensues in 5 % reduction in the maximum dry density at 30 % and 50 % clay content.

Also following Andrews and O'Flaherty (1968), they indicated that the decrease in density was dependent not only upon the lime percentage, but also on the clay content and the clay minerals. Thus, the optimum moisture content increases with increasing clay fraction as the specific surface area increases and thus needs more water for lubrication. Also, the maximum dry density decreases as a result of the difference in density between soil and lime.

2.8.2.4 Effect of Hydrated Lime on Strength Characteristics of Clay Soil

It has been established that the strength gain is influenced by various factors such as lime type and content, clay type and content, curing period and curing conditions, the elapsed time between mixing the chemical additives and compaction (Ingles and Metcalf, 1972; Bell and Coulthard, 1990).

2.8.2.4.1 Lime Type

There are two main types of lime used in soil stabilization namely, quick lime and hydrated lime. According to Bell (1988a) quick lime appears to be much more an effective stabilizer than the hydrated lime. Ingles and Metcalf (1972) observed that montmorillonite clay produces lower strength when mixed with dolomite lime instead of calcium lime. Bell and Coulthard

(1990) indicated that kaolinite clay attained the greatest strength when mixed with hydrated lime and the lower strength when mixed with calcium lime.

2.8.2.4.2 Content of Lime

Generally, the addition of lime to clay soil results in gain of strength of the soil to a certain limit and if the addition of the lime exceeds a certain limit, it tends to reduce the strength (Bell, 1996). According to Hilt and Davidson (1960) the optimum lime content is between 4 % and 8 % with higher replacement values needed in soils with higher fraction of clay. They suggested lime fixation between 2 % and 4 % lime. However, the addition of lime to the soils beyond the lime fixation point required to produce a substantial strength improvement due to the pozzolanic reaction activities will decrease the strength of the soil. For economic purposes, it is important to utilize minimum quantity of lime which is needed for strength development or improvement. To determine the optimum amount of lime needed for soil stabilization, pH test is quickly conducted to know the pH. In this method the pH of the soil-lime solution at different quantities of lime is measured after one hour of reaction at room temperature. When lime addition to the soil is made, the soil-lime mixtures pH increases to a maximum of 12.4 (the pH of lime-saturated water). Allen (1942) indicated that the optimum lime content pH does not increase more than the above stated value.

2.8.2.4.3 Clay Type

The type of clay mineral present in the soil has been established to have significant effect on the strength properties and each different clay mineral type affects the reaction products. Lime can attack all clay mineral types both with three layers (montmorillonite) which are more effective than the two layers of clay minerals (kaolinite). The reaction with montmorillonite is faster than kaolinite and the final strength attained is greater in kaolinite (Bell, 1988a).

Mateos (1964) indicated that montmorillonite and kaolinite clays attain higher strength development when they are mixed with lime than chlorites and illite. It was reported by Bell and Tyrer (1987) that the initial strength of expansive clay rapidly raised the final strength attained and is greater for kaolinite. Many authors have confirmed that kaolinite and montmorillonite react with lime better than chlorite and illite (Thompson and Harty, 1973; Bell, 1996).

On the contrary, Lees et al. (1982a) study on the effect of lime on the strength of artificial soil comprised of kaolinite, calcium base montmorillonite and sand indicated that lime treatment on the soil raised the UCS of the artificial soil. The calcium-based montmorillonite soil attained greater strength than the kaolinite achieved. Generally, bulk of the strength gain was due to the addition of lime at 2 % content for the kaolinite clay soil and 8 % for the montmorillonite clay soil. Also, montmorillonite treated with 4 % lime content after 28 days of curing increase the elastic modulus. The elastic modulus values of the specimens with 10 %, 30 % and 50 % of montmorillonite clay reached 32.0 MN/m², 49.0 MN/m² and 41.5 MN/m² respectively (an increase of 237 %, 145 % and 232 %) over the corresponding values before treatment. No further increase in the elastic modulus occurs for lime content greater than 2 %. Modulus values at this level achieved 15.5 MN/m² and 14.1 MN/m² for specimens with 30 % and 50 % clay respectively (an increase of 101 % and 182 %) over the corresponding values before treatment.

2.8.2.4.4 Time Passed Between Mixing and Compaction

The effect of the time passed between mixing and compaction on the final strength of the soil was studied by Mitchell and Hooper (1961). They observed that a delay of 24 hours between mixing and compaction led to as much as 30 % reduction in the cured strength compared to samples compacted right after mixing. All of the samples were cured for 21 days in a moist atmosphere prior to testing.

Holt and Freer-Herwish (1998) study on the effect of mellowing period on the compaction characteristic of London and Oxford clay mixed with lime, shows a rise in the optimum moisture content and a reduction in the maximum dry density with increase in the mellowing.

2.9 Clay - Copper Slag - Lime Reactions

The reaction of clay-lime-slag is different from the reaction of clay-lime. Surely there are two contending reactions instead of one. The first reaction is the clay-lime reaction which produces C-A-S-H and calcium aluminates and alumino- silicates. The second reaction is hydration of slag activated by lime to produce C-A-S-H gel and hydrotalcite type phases comprising of magnesium (Meng et al., 1998). The pozzolanic reaction of clay activated by lime is slower than that of the slag hydration activated by lime (Wild and Tasong, 1999). The strength of clay-

lime-slag mixture is ruled by the same factors noticed in slag-ordinary Portland cement blend hydration. The factors observed include the properties of C-S-H gel, permeability, structure and porosity. Lime presence in the clay-lime mix provides the needed alkaline for slag hydration and activation.

The total binder content (slag + lime) as proposed by Wild et al. (1999) would be decided by the required engineering properties. They recommended that the lime content should be replaced partially with 60 % to 80 % slag to reduced sulphate expansion. They reported that lime replacement by slag increases the strength and the slag-lime system should be used rather than that of lime alone to reduce total binder content. Thus, a preliminary swelling and strength tests must be carried out in order to establish effective and appropriate lime-slag content.

2.10 Curing Condition

The curing conditions comprising time, temperature and relative humidity are of great significance in influencing the strength increase and the final strength of the soil mixtures. According to Laguros et al. (1956) the pozzolanic reaction rate increases with increase in the temperature. Bell (1988 a, b) indicate that higher curing temperatures speed up the reaction and result in an early higher strength to be achieved. Many authors have confirmed this including Marks and Halliburton (1972) and Al-Rawi and Awad (1981); Wild et al. (1989). It was established by Mateos (1964) that clay soil specimens cured at 35 °C produced twice the strength of those that were cured at 25 ° C. Thompson (1970) indicated that no pozzolanic reaction can occur below 4 ° C because it is low temperature that retard the strength of development.

2.11 X-Ray Fluorescence (XRF)

X-ray fluorescence (XRF) is an analytical tool that was first used to analyze lead (Pb) in paint in the 1970s during abatement and exposure studies (Kalnicky and Singhvi, 2001) and has since been used in geological/geotechnical materials testing of soil or composite, environmental analysis of alloy, sediments, glasses, with very minimal sample preparation and treatment (Sitko, 2009; Lawryk et al., 2009). Over the years, the XRF has gained acceptance from science and engineering research community as a viable analytical tool because of the efficiency of the radioisotope source excitation coupled with extremely sensitive detectors and other electronics,

hence offering multielement analysis capability, economy, high speed and simplistic operation, where its advantages and limitations are well understood.

The basics of the XRF lie in the atoms of the receiving sample emitting different energies when they are excited by X-rays. The excited photons enable the qualitative and quantitative analysis of most elements in a given sample (Kalnicky and Singhvi, 2001). First the X-rays dislodge an atom from the inner shell. The atom from an outer shell fills the inner shell (K or L). The excited atom releases energy in the X-ray region of the wavelength as it returns to the ground state. The released photons with energy are equivalent to the difference between the two different shells. For instance, the transition from the L-shell to the K-shell results in a spectral line, which is designated $K\alpha$, while the transition from the M-shell to the K-shell provides a spectral line, which is designated $K\beta$. Thus, each element possesses different characteristic lines in the spectrum because each type of orbital transition produces a distinct X-ray. Clark and Brown (1999) showed that when certain atoms are excited, they release energy in the form of fluorescence as they return to the unexcited state. The photons emitted are then detected by the instrument.

2.12 X-Ray Diffraction (XRD)

X-ray diffraction is one of the several methods most extensively used in the identification of clay mineralogical composition. The method has been utilized to identify the reaction products of lime-clay mixtures, specifically after finding plain evidence of the similarity of these reaction products to the hydrated cementitious minerals (Abdelkader, 1981). X-ray diffraction (XRD) method was also utilized to identify the reaction products of clay with slag activated hydrated lime. There is no possibility to determine accurate proportion of each mineral present or the reaction products present in the mixtures. To understand and interpret the diffraction patterns of any materials, it is established on the fact that each of the crystalline material possesses its individual atomic characteristic structure which XRD diffract in their characteristics pattern (Abdi, 1992; Barr et al., 1995).

Crystals are made up of many closely similar units of cells, which are the smallest repeating units with the same structure (Yong and Warkentin, 1975). As the wavelength (λ) of the radiation and the distance between the atomic planes in the crystals are related, diffraction at

certain angles occurs when the X-ray strikes the atomic planes. The diffraction patterns are used to find the spacing between the planes of atoms in the crystals. The spacing among the planes (d spacing) characterize the shape and the size of the repeated unit cell of the crystals. Therefore, by characterizing and estimating the positions of the reflections crystals and their structure can be established (Gillot, 1987; Abdi, 1992).

The X-ray diffraction is being described by Bragg's law (Eqn. 2.11 a and b). Figure 2.6 presents X-ray beam reflected from parallel crystal plane spaces of d unit apart. In assuming two rays, 1 and 2, ray 2 then moves an extra length called ABC. From ABC, $AB = d \sin \theta$, where d is the spacing for the crystal (d-spacing) and θ is the peeking angle. "For many peeking angles the path length difference is not an integer number of wavelengths, and the wave interfere mostly in a destructive manner. When the path length difference is an integer number of wave length, then $(AB + BC = n\lambda)$ and the reflected waves are in phase and interfere constructively. It complies that a bright reflection should be noticed when the peeking angle agrees with Bragg's law" (Atkins, 1998).

$$n\lambda = 2 d \sin \theta \text{ (Bragg's law)} \quad [\text{Eqn 2.11a}]$$

Where n is the integer (1, 2, 3, 4 etc.), and λ is the x-ray wave length and θ is the peeking angle. It is normal in this modern time, to absorb the n into the d and rewrite the equation as

$$\lambda = 2 d \sin \theta \text{ (Bragg's law modified)} \quad [\text{Eqn 2.11b}]$$

The utilization of x-ray diffraction method principal advantage over other methods and procedures is that it is a non- destructive and only little quantity of the materials is needed. The disadvantage of this x-ray method is that, it is not sufficient to identify all the clay minerals present in the reaction products as it is specifically limited to crystallinity phases identification alone. Also, it is not easily possible to identify roughly some clay minerals that are present in the structure similarities (Kinuthia, 1997).

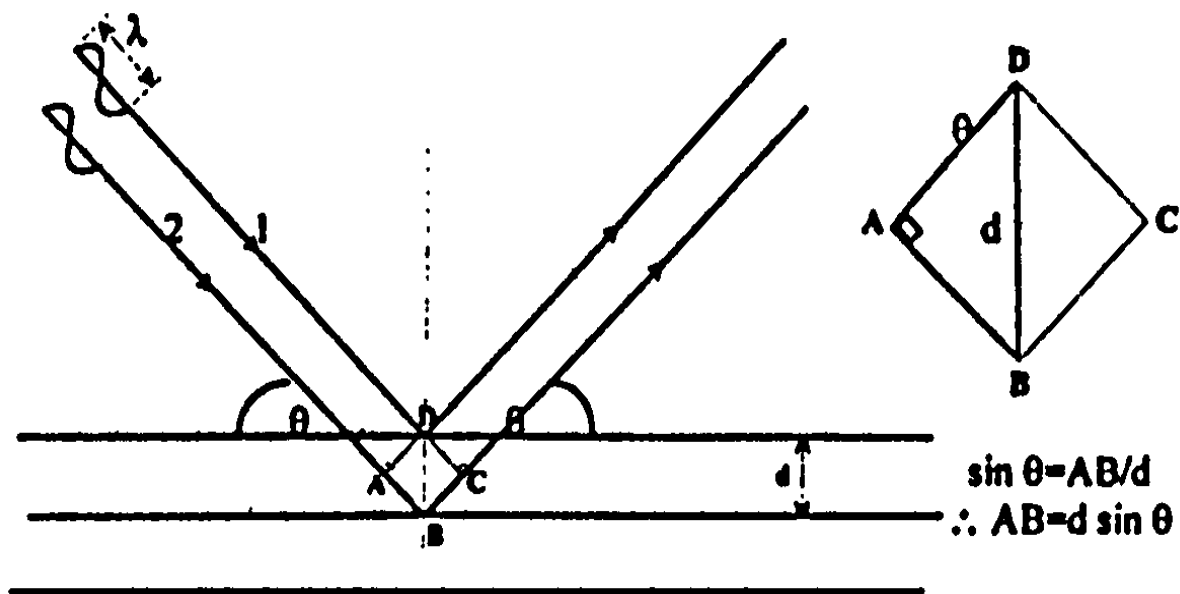


Figure 2.6 X-ray diffraction according to Bragg's law, after Kinuthia (1997)

2.13 Scanning Electron Microscopy (SEM)

The scanning electron microscopy is one of the powerful instruments and most common tool in material analysis. It is more desirable than the optical microscopy when higher magnification or increased depth of field is required; also, when some form of composition and elemental analysis is required. The scanning electron microscopy is used to generate images of the surface and subsurface of the specimen at the magnification range of 20x to 20000x. It can be utilized to study the micro-structure of the specimen and to find the particle crystallinity. Scanning electron microscopy might be utilized to characterize and identify specific detail phases and their forms and shape. Also, it has the advantage of providing three dimensional images of superior depth resolution and focus capabilities as compared to the optical micrographs (Kinuthia, 1997; Ananta, 1999).

2.13.1 Nature of Backscattered Scanning Electron Images

The scanning electron microscopy can provide various images of points, composition, and individual crystallographic structure. The backscattered electrons are emitted or uttered from a cathode and then speed up by the voltage between the cathode an anode. A fine investigation is formed on the surface of the specimen by using a number of lenses. When the primary beam

electrons (PE) collide with the surface of the specimen, they scatter after multiple collisions with the specimen atom and then re-emerge from the surface of the specimen. There are two groups of electrons which are ejected; electrons with PE energy greater than (50 eV) called backscattered electrons (BSE) and the other electrons called the secondary electrons having energy less than (50 eV). Both of these electrons leave the specimen from a thin surface layer. The secondary electrons can only provide surface topographic information of the specimens. The backscattered electron can transmit through the thin specimens and provide information on the internal structure of the specimens. SEM can provide average atomic number information and their crystallographic surface information. Also, SEM can be used for the chemical and elemental analysis (Richardson, 2001).

CHAPTER 3

MATERIALS AND METHODS

3.1 Introduction

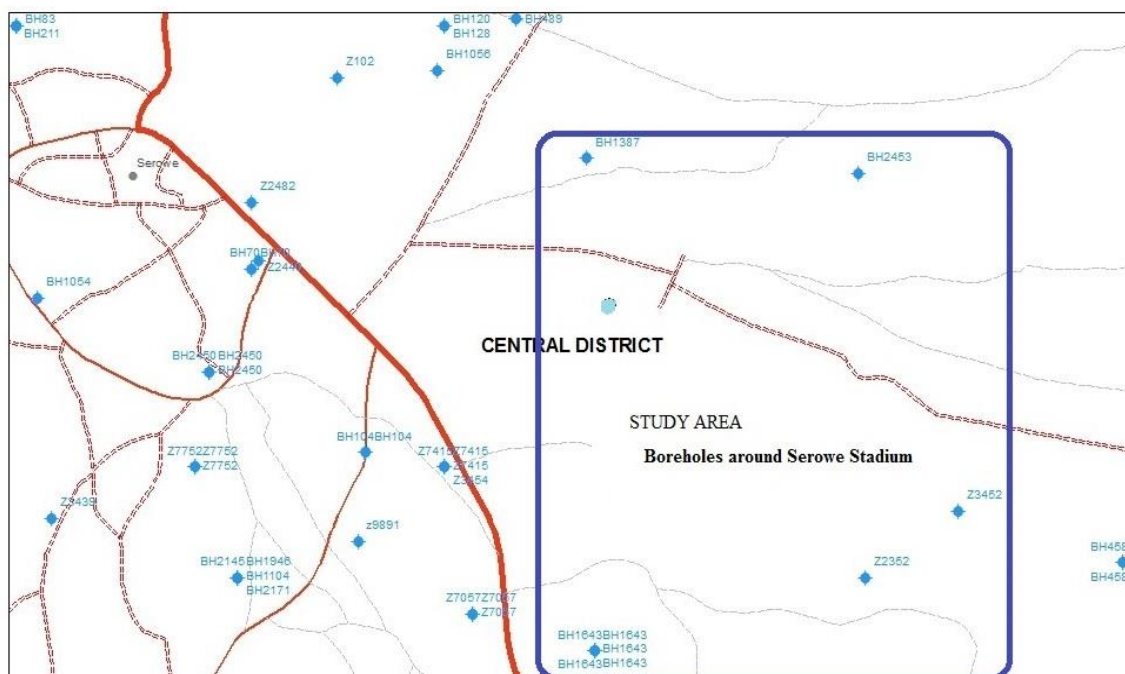
In this chapter, a description of the materials used and the primary testing methods applied in the study are presented. Two types of materials namely natural soil from boreholes in Serowe and stabilizers comprising of copper slag and hydrated lime were used.

The testing procedure is subdivided in three parts namely; characterization tests, engineering tests and analytical methods. The characterization tests include specific gravity, organic matter content, Atterberg limits and grain size distribution tests. The engineering tests comprise of compaction and unconfined compressive strength tests while the analytical methods used include X-ray diffraction and Scanning electron microscope probe. The scanning electron microscopy (SEM) and X-ray diffraction (XRD) analyses were used to provide some indications of the characteristics of the primary materials and the reaction products.

3.2 Materials

3.2.1 Serowe Clay Soil

Soils used for this research were obtained from 6 boreholes drilled in the claystones deposits around the Serowe stadium. The samples were selected from random boreholes over the claystone deposit (Figure 3.1). Studies conducted on the soil samples taken at different depths included X-ray fluorescence, X-ray diffraction and scanning electron microscopy studies.



3.4 Testing Methods

3.4.1 Organic Matter Content Test

The organic matter content test was conducted on the natural soil from the 6 boreholes using hydrogen peroxide digestion method as part of the pre-treatment of the soil to terminate colloidal organic matter present in the soil by adding 40 % of hydrogen peroxide concentration to 120 g of soil passing 2 mm sieve. The organic matter content was calculated using Eqn 2.2 in Chapter 2.

3.4.2 Specific Gravity Measurement

The specific gravity of the soil, CS and hydrated lime were determined by means of the pycnometer method. 130 g of each sample type namely the soil, copper slag and hydrated lime passing through 2 mm sieve was placed in separate pycnometers for the specific gravity determination. The specific gravity (G) was calculated using the relationship in Eqn 2.5 in Chapter 2.

3.4.3 Grain Size Distribution

The Laser light scattering diffraction method was used in the determination of the grain size distribution of the material using the Malvern Mastersizer 3000 equipment. In this method, the particles pass through a laser beam and the light scattered by them is collected over a range of angles in the forward direction. The angles of diffraction are in the simplest case inversely related to the particle sizes (Figure 3.2).

The particles pass through an expanded and collimated laser beam in front of a lens in whose focus plane is positioned a photosensitive detector consisting of a series of concentric rings. Distribution of scattered intensity is analyzed by a computer to yield the particle size distribution. It is a non-intrusive method and utilizes a low power laser beam. (www.fritsch-sizing.com).

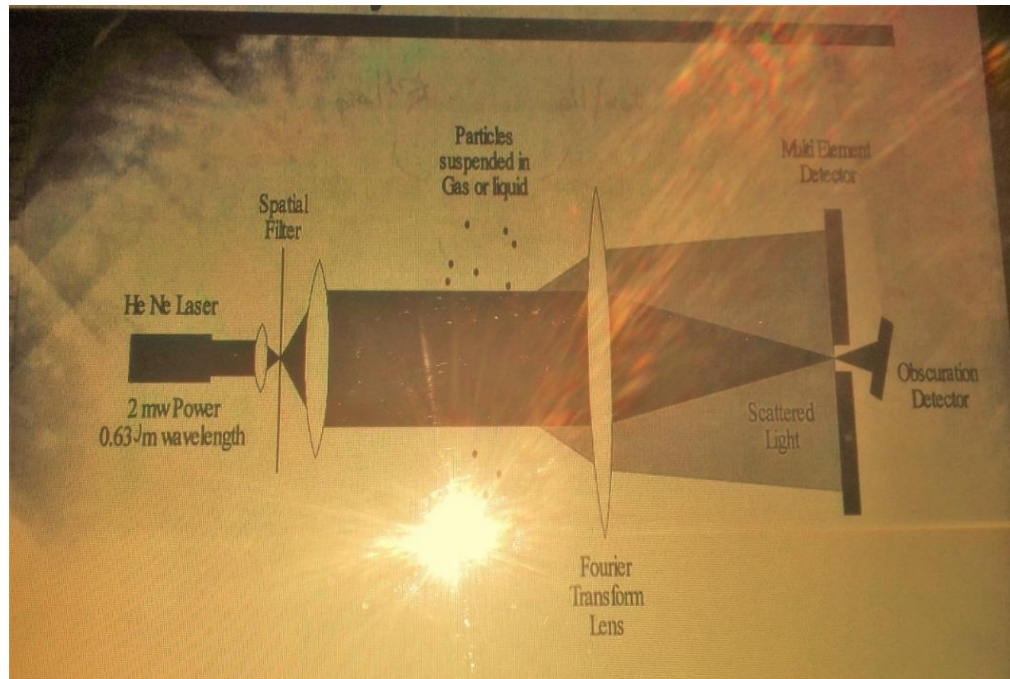


Figure 3.2: Typical diffraction pattern

3.4.4 X-Ray Fluorescence (XRF)

The chemical analysis was done using X-ray fluorescence (XRF) to determine the percentage of the oxide contents in the clay soil, hydrated lime and copper slag. The X-ray fluorescence tests were conducted at the Department of Geological Survey, Botswana Geoscience Institute.

3.4.5 Modified Proctor Compaction Test

BS: 1377 (1990) Test 3 was adopted for the determination of the optimum moisture content and maximum dry density. This test encompasses the determination of the mass of dry soil per cubic meter when the soil is compacted over a range of selected moisture contents. 45 kg mass of soil passing 0.075 mm test sieve was divided into nine representative samples of about 5 kg each. Each sample was carefully mixed with different proportion of additives in the Hobart mixer followed by the addition of distilled water and mixed for 4 minutes. The soil was compacted in five layers in CBR mould at a selected water content with each layer compacted by 27 blows. Uniform compaction could not be achieved using this method due to the high plasticity of the soil. Therefore, dynamic automatic proctor/CBR compaction machine was used to perform the compaction test in order to achieve uniform compaction. The machine was operating at a speed of 0.9 mm/min. The compacted specimens were extruded from the mould using a manual

hydraulic jack. The releasing mould oil was clean off the specimens using tissue paper. The extruded specimens were weighed and wrapped with cling film and placed in polyethylene bags to assure minimum moisture loss during the curing period, labelled and stored in separate curing rooms.

The mixtures under study comprised of the soil containing 2 %, 4 %, 6 %, 8 % and 10 % of copper slag expressed by dry unit weight of the soil. In addition, another test was conducted with CS been replaced by hydrated lime at the ratios of 9:1, 4:1 and 7:3 of the mixture comprising of soil, copper slag and hydrated lime. The dry unit weight of the composite was determined and two samples were extracted for the determination of moisture content and the mean of the two samples was taken as representative moisture content. The dry density was plotted against the moisture content in order to obtain the maximum dry density (MDD) and the optimum moisture content (OMC). Figure 3.3 presents the Hobart mixer and Figure 3.4 presents modified proctor compaction mould and dynamic automatic proctor-CBR compaction machine.



Figure 3.3: Hobart mixer used for mixing of samples



Figure 3.4: a) modified proctor compaction mould and b) dynamic automatic proctor-CBR compaction machine

3.4.6 Curing Conditions

The specimens were divided into two batches with separate curing conditions. Batch one was cured in a room with temperature maintained at 35 °C and relative humidity ranging from 90 % to 100 % which in this report is referred to as curing condition 1 (CC1). The second batch was cured in a room at 45 °C and the relative humidity of 50 % to 60 % referred to as curing condition 2 (CC2). The curing time varied from 7 days to 74 days. The selection of 35 °C temperature with a relative humidity of 90 – 100 % to cure the specimens, is to increase the strength of the specimens due to the lack of uniform distribution of moisture inside the specimens. While for the specimens under 45 °C temperature with relative humidity of 50 – 60 % is to also increase the strength of the specimens at a lower relative humidity and higher temperature. The maximum curing period of 74 days was selected in order to know the types of reaction products formed during the reaction of soil-copper slag-hydrated lime system.

3.4.7 Atterberg Limits

The Atterberg limits tests comprising of liquid limit and plastic limit and the plasticity index were conducted in accordance with BS: 1377 (1990), Part 2 Tests 4 and 5. The tests were carried out on the natural clay soil and composites of soil and copper slag in the proportion of 2 %, 4 %, 6 %, 8 % and 10 % by dry unit weight of copper slag (CS) and also mixtures of CS +

hydrated lime as binder in the proportions of 2 %, 4 % and 6 % with replacement of copper slag by hydrated lime at ratios of 9:1, 4:1 and 7:3 by dry unit weight of copper slag.

3.4.8 Free Swelling Tests

Free swelling test was conducted on all specimens that were subjected to the unconfined compressive strength tests. Approximately 60 g of soil was oven dried at 55 °C, crushed and passed through 212 µm sieve. 10 ml of the under size of 212 µm sieve of soil was placed in a dry 10 ml cylinder without shaking or compacting. 50 ml of distilled water was then added to the 10 ml of soil in a 70 ml cylinder for about 35 minutes to allow the principal solid to settle (Figure 2.5). The volume of the solid that settled was measured (V_{ml}). The free swell was then calculated using Eqn. 2.3 as shown in Chapter 2. The test was repeated thrice for each of the specimens and the average of the three values reported as the free swell.

3.4.9 pH Value Determination

The pH test was determined in accordance with BS: 1924 (1990) on the natural soil and the mixtures of soil and lime with a range of 2 %, 4 %, 6 % and 9 % lime content and also on the soil with copper slag mixes in the proportion of 2 %, 4 %, 6 % and 9 % CS content. In addition, the test was conducted on the soil with lime and copper slag at 4 %, 6 %, 11 %, 16 % and 27 % binder content for both replacement ratios of 4:1 and 7:3 by hydrated lime. The pH of the lime solution was measured using Corning 240 digital pH meter. The change in proportion for copper slag (CS) activated by hydrated lime was confirmed during the preliminary tests, to establish the lime solution at which the optimum total binder content is achieved. BS: 1924 (1990) recommends a 25 °C temperature for pH correct measurement at room temperature to standard temperature which was calculated using Eqn 2.4 in Chapter 2.

3.4.10 Specimen Preparation for Unconfined Compressive Strength Test

The mixing was executed using the Hobart variable speed mixer (Figure 3.3) at the lower speed of 3 millimeters per minute before adding the calculated quantity of distilled water in order to produce six compacted cylindrical test samples with dimension of 38 mm diameter and 76 mm length. The distilled water used was that required to give the optimum moisture content as found

from the compaction test. The optimum moisture contents were used in this study to obtain the maximum dry density for each individual mixture.

3.4.11 Unconfined Compressive Strength of Specimens

Variation occurs in the moisture content of the test specimens due to the curing period. As a precaution, samples with loss in moisture content at the time of curing greater than $\pm 3\%$ and greater than $\pm 6\%$ of the initial moisture content for the first 28 days and 74 days respectively were insignificant and considered as minimum loss obtained for the unconfined compressive strength test.

After curing, the specimens were removed from the cling film polyethylene bags and any water traces on them was removed using tissue paper. The specimens were weighed to the nearest 0.1 gram. Specimens selected were tested in Hounsfield type H 20 K-W compression machine at a loading rate of 1.2 mm/min to permit the specimen to fail in approximately 4 - 5 minutes (Figure 3.5). The minimum of three specimens of each mixture were tested for the UCS and the mean result recorded. Samples were taken from the failed specimens for free swell test.



Figure 3.5: Typical unconfined compressive strength test machine

3.5 Analytical Tests

Two analytical methods namely the X-ray diffraction (XRD) and the scanning electron microscopy (SEM) were used to determine the nature of the raw materials and the alteration achieved after stabilization with CS and after activation of CS with hydrated lime.

3.5.1 X-Ray Diffraction (XRD)

X-ray diffraction was used to identify the crystallinity phases of the reaction products. The minerals of the raw materials and the reaction products were identified from the diffraction line. It is however not possible to accurately determine the proportion of each mineral and the reaction products present in the mixture using this method. In order to understand and interpret X-ray test result, diffraction patterns were established on the fact that each of the crystalline material holds its own atomic characteristic structure which the XRD diffract in its own characteristic pattern.

X-ray diffraction analysis was limited to only two carefully selected specimens namely the soil as a control sample and the composite sample at the ratio of 7:3 and 4:1 CS/hydrated lime cured under CC2 after 74 days. After curing, about 60 grams of each specimen was taken and oven dried at 55 °C for 24 hours. The dried specimens were then ground into powder to pass through a 63 µm mesh before mounting in the aluminium holding of the X-ray diffractometer.

The samples were analyzed using the Bruker D8 Advance Powder diffractometer with a 3.0 kW generator and a copper (Cu) tube x-ray source and a Lynx-Eye XE energy dispersive strip detector (Figure 3.6). The x-ray diffraction data was assessed using Bruker software EVA for phase identification, structure refinement, and quantitative phase analysis. Radiation used was a copper (Cu-K α) with wavelength of 1.54056 Å with the value of 2 θ between 5 to 90-degrees scanning speed of 0.02 degree/second.



Figure 3.6 Bruker D8 advance powder diffractometer X-ray

3.5.2 Scanning Electron Microscope (SEM)

The scanning electron microscope method was used to analyse the composite specimen and individual samples used for the study.

3.5.2.1 Specimen Preparation for Microscopic Study

The composite specimens and copper slag were pulverized. Individual samples including copper slag, soil, hydrated lime, and the composite specimens were used in the SEM investigation. Figure 3.7 presents the pulverizer equipment and Figure 3.8 presents the samples used in the investigation. Figure 3.9 presents specimens and samples during carbon coating.

The procedure consists of coating of the specimen surface with carbon for about 10 minutes for full coating. The specimens were each placed in the scanning electron microscope for the energy dispersive X-ray analysis which takes about an hour for a single sample. Carbon coating was selected for the coating of the specimens and samples because it gives better results compared to gold coating under the backscattered electron.



Figure 3.7: Pulverized equipment



Figure 3.8 Composite specimens and samples including CS, hydrated lime and soil



Figure 3.9 Coating of samples for SEM analysis

CHAPTER 4

ENGINEERING PROPERTIES TESTS RESULTS

4.1 Introduction

The effects of CS with and without hydrated lime on some engineering properties that include swelling characteristics, plasticity characteristics, compaction and unconfined compressive strength (UCS) of the soil are presented. The probe of swelling characteristics and the unconfined compressive strength were carried out on specimens that were cured under two conditions namely: curing under 35 °C with a relative humidity ranging from 90 % - 100 % and curing under 45 °C with a relative humidity ranging from 50 % - 60 %. The curing times were varied between 7, 16, 28, 42, 56 and 74 days. The plasticity test was conducted on composite soils aged 1 and 3 days after preparation of composite samples. Compaction test was conducted soon after the addition of the binder materials.

4.2 Chemical and Engineering Properties of the Soil, Hydrated Lime and Copper Slag

Figure 4.1 presents samples of the natural soil, copper slag and hydrated lime. Results of the particle size distribution analysis of the natural soil, copper slag and hydrated lime are presented in the grading curves shown in Figure 4.2. Table 4.1 presents the physical, mineralogical and engineering properties of the natural soil, copper slag and hydrated lime while Table 4.2 presents the chemical oxides and their proportions in the natural soil, copper slag and hydrated lime.

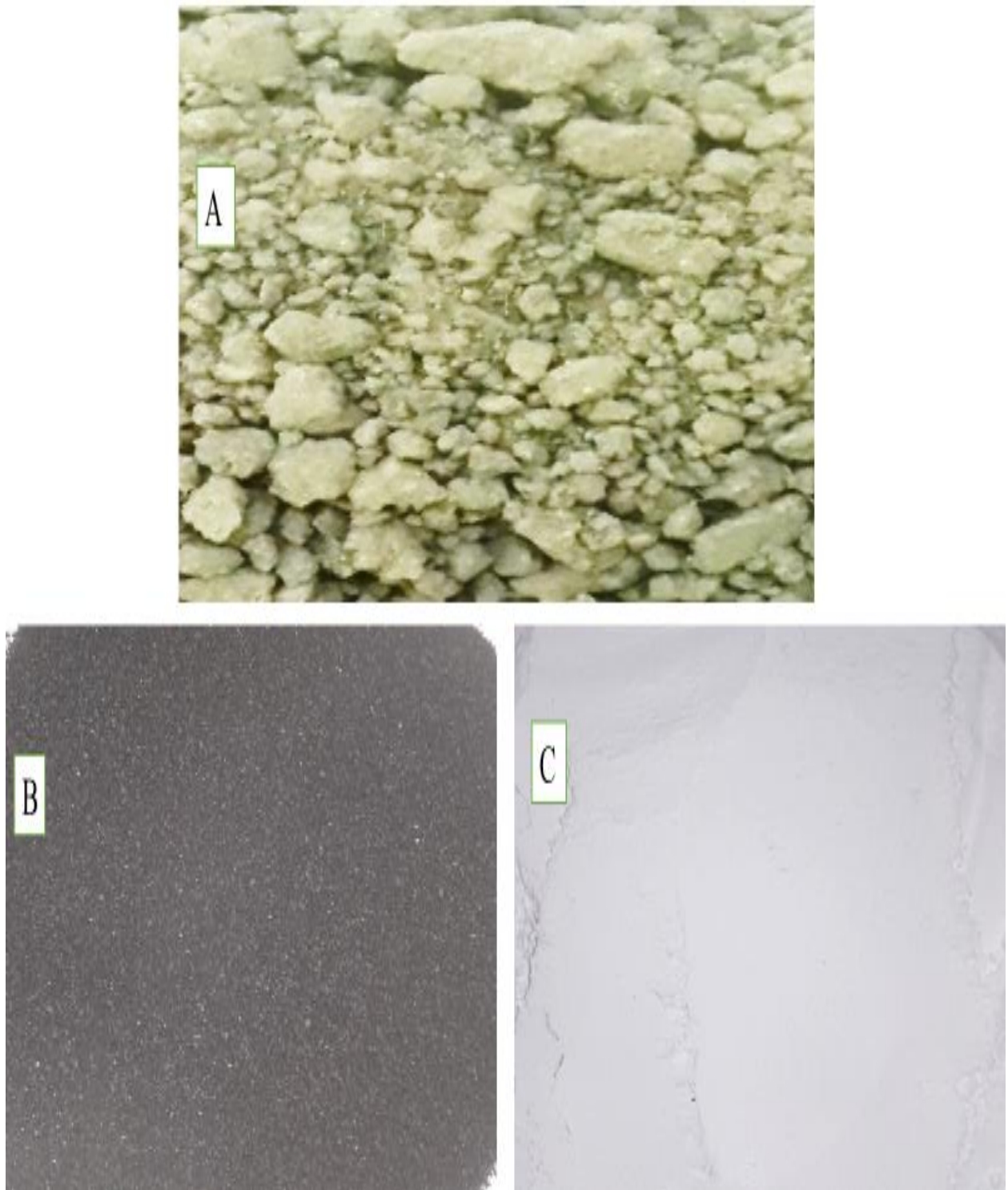


Figure 4.1: A) Natural soil, B) Copper slag and, C) Hydrated lime

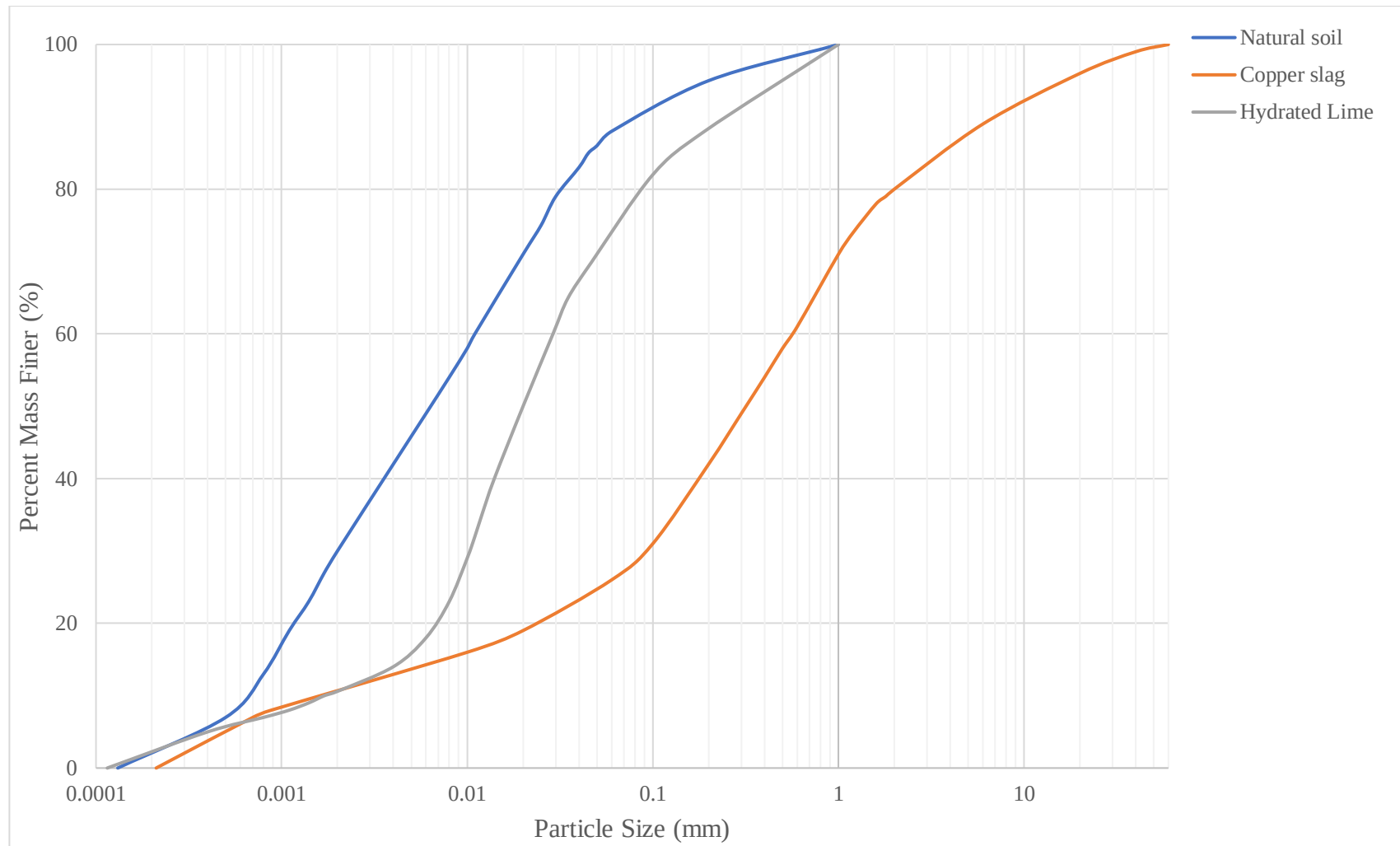


Figure 4.2 Particle size distribution curves for the natural soil, copper slag and hydrated lime

Table 4.1: Physical, mineralogical and engineering properties of the natural soil, copper slag and hydrated lime

Physical Property	Natural soil	Copper slag	Hydrated lime
Gravel (20 mm – 2 mm)	0	20 %	0
Sand (2 mm - 0.063 mm)	14 %	55 %	27 %
Silt (0.063 mm - 0.002 mm)	56 %	15 %	62 %
Clay (< 0.002 mm)	30 %	10 %	11 %
Specific gravity	2.5	3.2	2.63
Liquid Limit	82 %		
Plastic Limit	37 %		
Plasticity Index	45 %		
Organic matter content	1.3 %		
Activity = PI/clay	0.78 %		
Maximum Dry Density (Mg/m ³)	1.92		
Optimum Moisture Content (%)	18.81	< 5	
Casagrande	MH		
Textural	Clay		
AASHTO Engineering	A-7-5		
Clay minerals			
Montmorillonite (%)	19.75		
Mica (%)	5		
Chlorite (%)	6.25		
Illite (%)	9		
Kaolinite (%)	30		
Non-Clay minerals			
Quartz (%)	14		
Calcite (%)	6		
Feldspars (%)	10		
Physical state at 20 °C and 104.8 kPa:		Solid	
Appearance:		Black glassy, vesicular when granulated	Dry white powder
Unit Weight		2800 Kg/m ³	
Water Absorption		0.01 %	
Bulk Density		2300 Kg/m ³	480
Conductivity		500 µs/cm	
Hardness		5 .65 mol	
Abrasion loss		23.60 %	
Sodium sulphate Soundness loss		0.85 %	
Angle of internal friction		46°	
Specific surface area			1100 Kg/m ²
Melting point/decomposition temperature		1259	500 °C

Table 4.2 Chemical oxides and the percentage present in natural soil, copper slag (BCL Mine) and hydrated lime (Neelkanth Lime Limited, Lusaka) from XRF studies

Chemical Oxides	Natural soil (%)	Copper Slag (%)	Hydrated lime (%)
SiO ₂	34.27	24.84	4.21
Al ₂ O ₃	26.38	7.86	1.83
Fe ₂ O ₃	16.67	48.28	1.15
CaO	8.17	16.89	20.91
MgO	4.58	11.86	
K ₂ O	2.42	0.18	
Na ₂ O	3.34	0.43	
SO ₃	2.2	0.03	
Mn ₃ O ₄	0.78	0.15	
TiO ₂	1.2	0.31	
CuO		0.56	
Chloride		0.25	
CaCO ₃			2.13
Ca(OH) ₂			65.09
H ₂ O			0.58
CaSO ₄			0.1
Insoluble residue		31.63	

4.3 Compaction Characteristics of the Soil, Copper Slag and Lime as Activator

4.3.1 Moisture Content and Dry Density Relationship

Moisture content - dry density relationship was established through the modified proctor compaction test. The test was conducted on the soil with varying quantities of added copper slag (CS) and also mixtures of soil with 2 %, 4 % and 6 % total binder content (CS + lime) by weight of dry soil. Furthermore, the test was performed on soil in which CS was replaced by hydrated lime at a ratio of 9:1, 4:1 and 7:3 with total binder content of 2 %, 4 % and 6 %. Compaction curves for these samples are presented in Figures 4.3, 4.4 and 4.5. From the compaction test, the natural soil dry density increases to a maximum value of 1.92 Mg/m³ called the maximum dry density (MDD). The moisture content corresponding to the maximum dry density termed as the optimum moisture content (OMC) was determined to be 18.81 %. In Figure 4.3, 2 % addition of binder content to the natural soil resulted in a decrease in the MDD to 1.86 Mg/m³ and an increase in the OMC to 19.96 %. The replacement of CS by hydrated lime at the ratios 9:1, 4:1 and 7:3 result in decrease in the MDD to 1.83 Mg/m³, 1.80 Mg/m³ and 1.77 Mg/m³ respectively and the corresponding OMCs increased to 20.30 %, 20.92 % and 21.73 %. A further increase in the binder content to 4 % resulted in a decrease in the MDD to 1.84 Mg/m³ and an increase of the OMC to 20.57 %. Further replacement of CS by hydrated lime at the ratios of 9:1, 4:1 and 7:3, resulted in a decrease in the MDD to 1.79 Mg/m³, 1.75 Mg/m³ and 1.72 Mg/m³.

The corresponding OMCs are 21.11 %, 22.01 % and 22.46 % (Figure 4.4). A further decrease in the MDD was observed with 6 % binder content to 1.75 Mg/m³ and an increase in the OMC to 21.29 %. The replacement of CS by hydrated lime at the ratios of 9:1, 4:1 and 7:3 also decreased the MDD to 1.72 Mg/m³, 1.70 Mg/m³ and 1.67 Mg/m³ with their corresponding OMCs of 22.62 %, 22.73 % and 23.35 % (Figure 4.5).

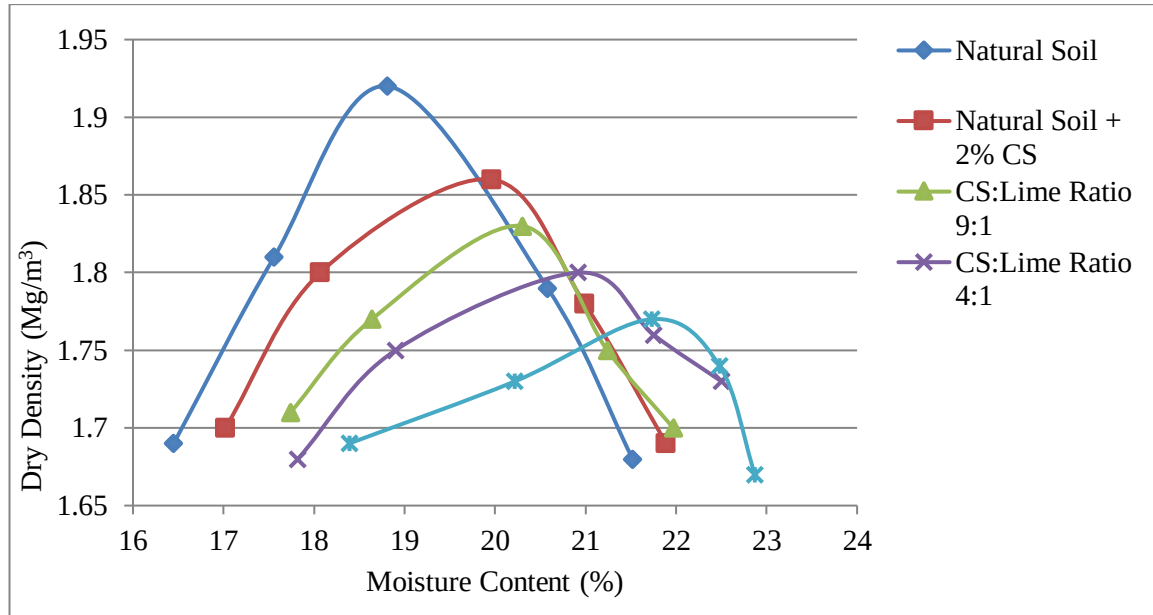


Figure 4.3: Compaction curves for the natural soil, natural soil + 2 % CS and composite soil + 2 % binder content (CS and hydrated lime)

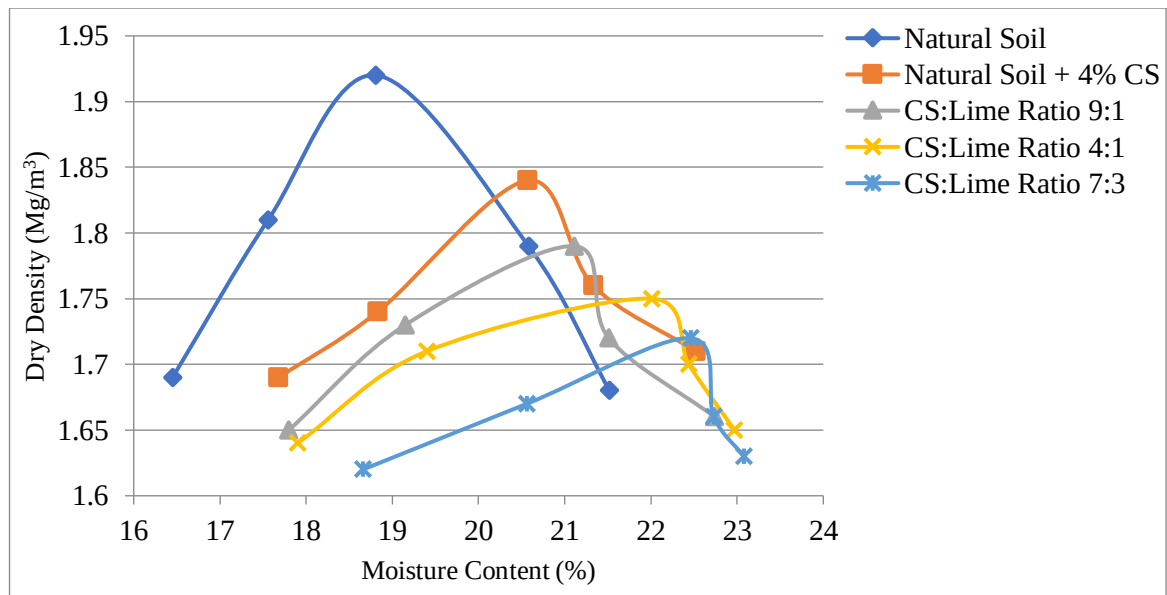


Figure 4.4: Compaction curves for the natural soil, natural soil + 4 % CS and composite soil + 4 % binder content (CS and hydrated lime)

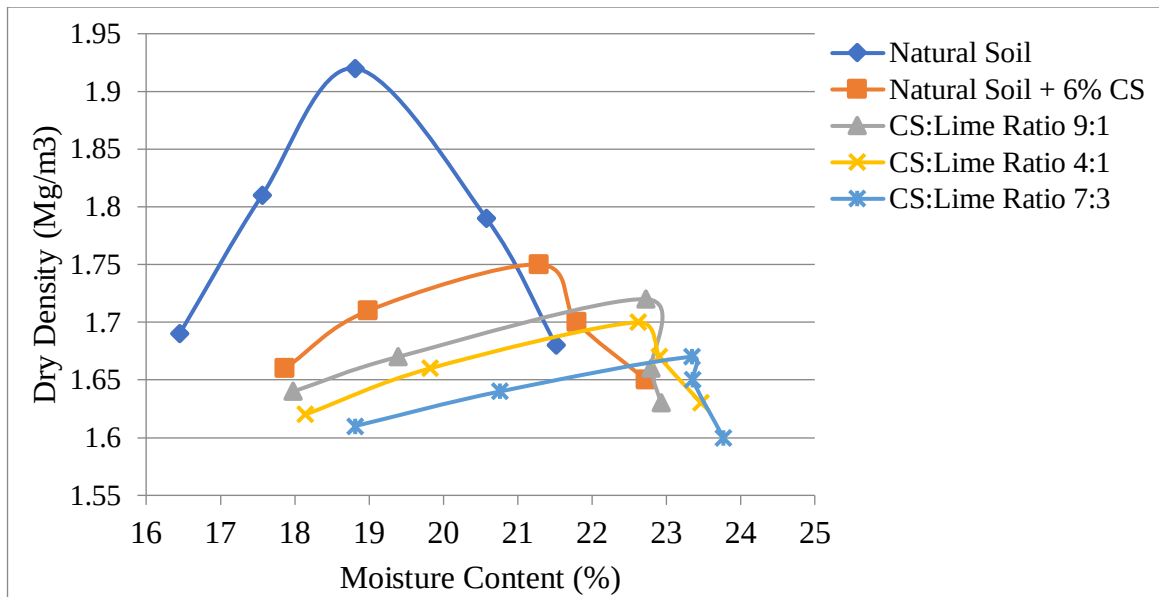


Figure 4.5: Compaction curves for the natural soil, natural soil + 6 % CS and composite soil + 6 % binder content (CS and hydrated lime)

4.3.2 Effect of Copper Slag on Compaction Characteristics of Natural Soil

Results of the effect of copper slag percentage on the optimum moisture content and the maximum dry density of the soil and composite soil are presented in Figure 4.6. As the CS percentages increase from 2 % to 10 % by 2 % incremental value, the optimum moisture content increased from 18.81 % to 22.57 % while the maximum dry density decreased from 1.92 Mg/m³ to 1.68 Mg/m³.

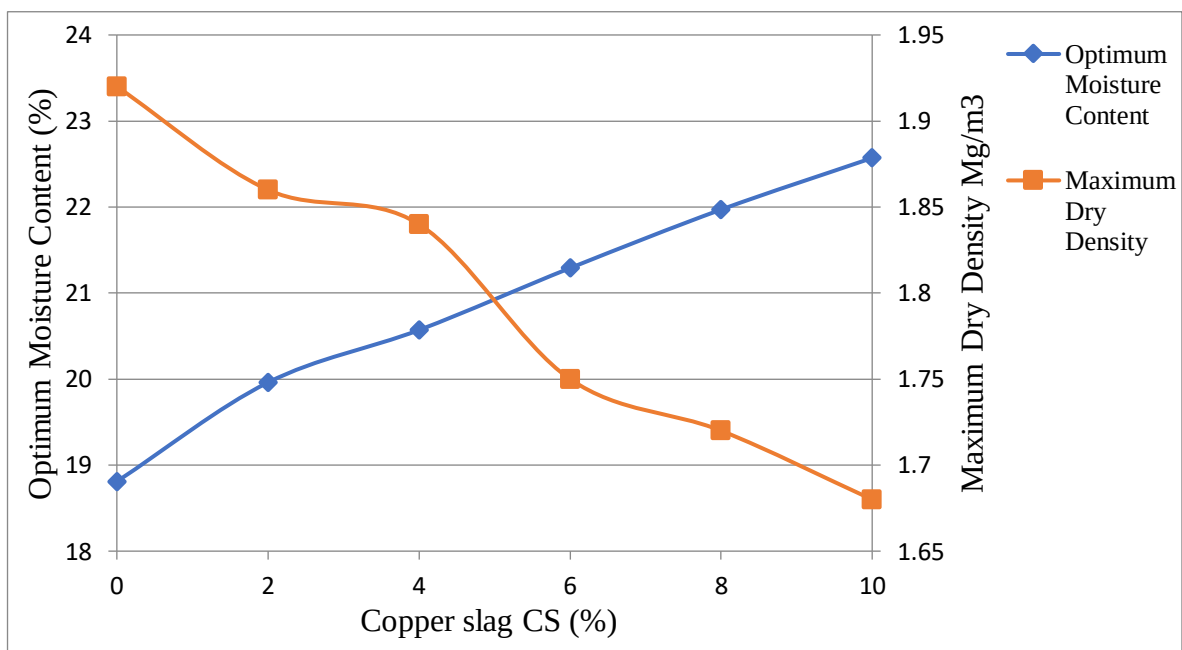


Figure 4.6: Effect of CS on the MDD and OMC of the soil against binder content

4.3.3 Effect of Copper Slag and Hydrated Lime on Compaction Characteristics of Natural Soil

Results of the effect of the partial replacement of CS by hydrated lime on the maximum dry density of the soil and composite soil have been shown in Figures 4.3, 4.4 and 4.5. Figure 4.7 presents the effect of the proportion of total binder content on the MDD of the soil. It is observed from the figure that the maximum dry density reduces with an increase in CS/lime ratio with constant total binder content. For the same binder content of CS/lime ratio, the maximum dry density decreases as the proportion of lime replacement increased from the ratio of 9:1 to 7:3. For example, at 6 % binder content the MDD reduced from 1.75 Mg/m^3 to 1.67 Mg/m^3 with an increase in the replacement of CS by hydrated lime content from a ratio of 9:1 to 7:3.

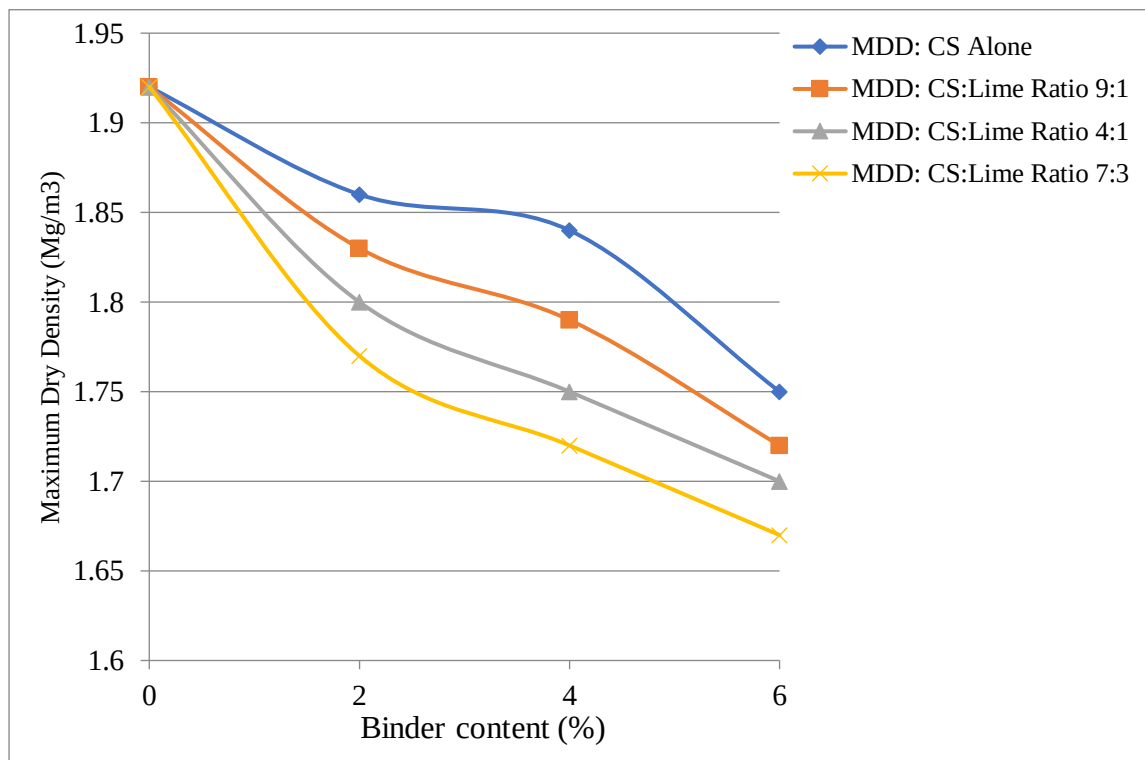


Figure 4.7: Effect of the proportion of total binder content on the MDD of the soil

Figure 4.8 also presents the effect of the proportion of total binder content on the OMC of the soil. The OMC is observed to increase with an increase in CS/lime ratio with constant total binder content. For the same binder content of CS/lime ratio, the optimum moisture content increases as the proportion of lime replacement increased from a ratio of 9:1 to 7:3. For example, at 6 % binder content the OMC increased from 21.29 % to 23.35 % with an increase in the replacement of CS by hydrated lime content from a ratio of 9:1 to 7:3.

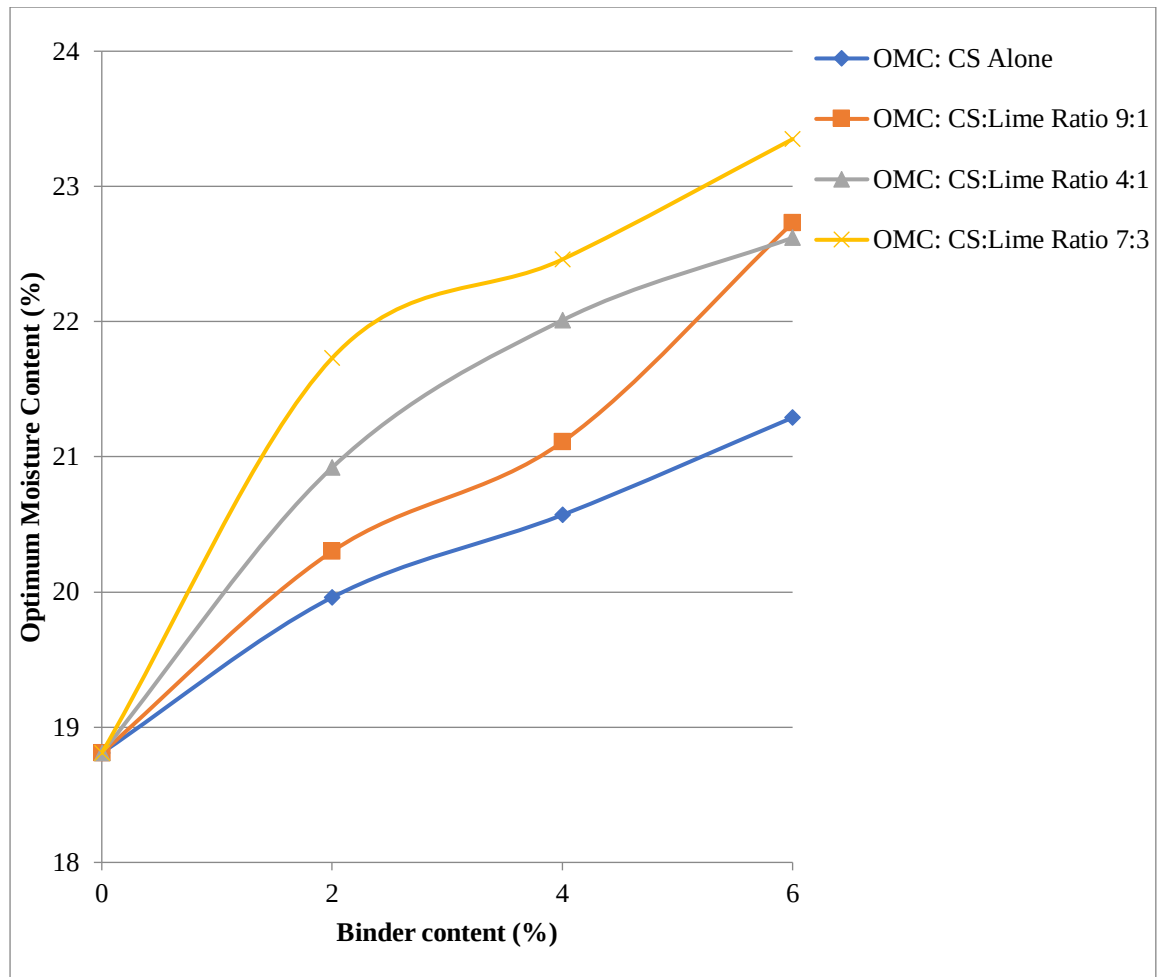


Figure 4.8: Effect of the proportion of total binder content on the OMC of the soil

4.4 Plasticity Characteristics of Copper Slag-Soil, Copper Slag-Hydrated Lime-Soil and Hydrated Lime-Soil Mixtures

Atterberg limits tests were conducted on the soil in accordance with BS: 1377 (1975) Test 2 (A) and Test 3 after 1 day and 3 days of mixing. The first phase of the plasticity characteristics was to study the effect of CS on the natural soil by varying the percentage of CS from 2 %, 4 %, 6 %, 8 % to 10 % by the weight of dry soil. The outcomes are presented in Figure 4.9. The second phase was to study the effect of CS (2 %, 4 %, 6 %, 8 % and 10 % by the weight of dry soil) activated by hydrated lime at a ratio of 9:1 by weight of CS on the natural soil. The results are presented in Figure 4.10. The third phase was to study the effect of the replacement of CS by hydrated lime at the ratio of 4:1 and 7:3 on the natural soil as presented in Figures 4.11 and 4.12 respectively. The classification of the natural soil and composite is presented in Figure 4.13. The effects of hydrated lime only on plasticity characteristic results are presented in Figure 4.14.

4.4.1 Effect of Copper Slag on the Plasticity Characteristics of the Natural Soil

Figure 4.9 presents the result of the effect of copper slag on the Atterberg limits of the natural soil. It shows the changes in the liquid and plastic limits and the plasticity index of the natural soil for 1-day mix and 3 days after mixing with various quantities of added CS. The liquid and plastic limits and the plasticity index of the soil at zero percent binder content are 82 %, 37 % and 45 % respectively.

The liquid limit for 1-day mix represented as D1-NS-CS-LL marginally reduced from 82 % to 81 % at 2 % CS content. Further decrease was observed in the D1-NS-CS-LL from 81 % to 79 % with an increase in the quantity of CS from 2 % to 10 % by 2 % increment. The plastic limit for the 1-day mix represented as D1-NS-CS-PL was noticed to slightly increase to 38 % with the addition of 2 % CS. Further increase was observed in the D1-NS-CS-PL from 38 % to 39 % with an increase in CS content ranging from 2 % to 10 % by 2 % increment. The plasticity index for 1-day mix represented as D1-NS-CS-PI decreased slightly from 45 % to 40 % with an increase in CS content ranging from 2 % to 10 %. The liquid limit after 3-days mixing represented as D3-NS-CS-LL marginally reduced from 82 % to 78 % with the addition of CS content ranging from 2 % to 10 %. The plastic limit after 3-days mixing represented as D3-NS-CS-PL increased from 37 % to 41 % as the CS content was increased ranging from 2 % to 10 % while the plasticity index after 3-days mixing represented as D3-NS-CS-PI significantly decreased from 45 % to 37 %. Other values of Atterberg limits obtained from the varying CS content are presented in Appendix 1.

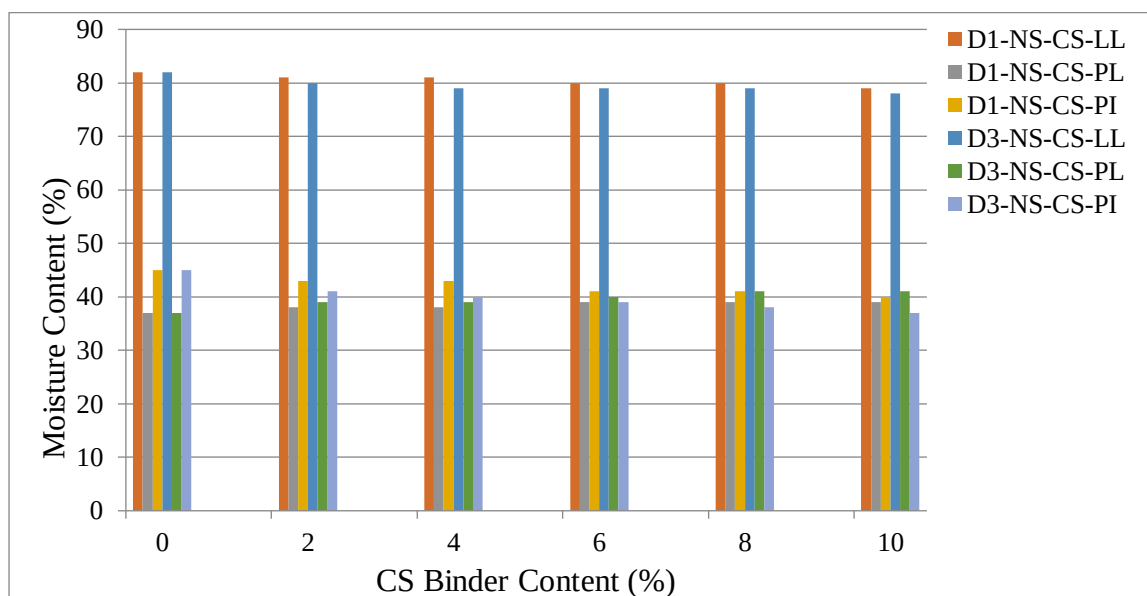


Figure 4.9: Atterberg limits for the natural soil and copper slag (CS) for 1 day and 3 days

4.4.2 Effect of Copper Slag and Hydrated Lime on the Plasticity Characteristics of the Natural Soil

Figures 4.10 to 4.12 present the results of the effect of CS activated by hydrated lime on the Atterberg limit of the soil with CS content replaced by hydrated lime at the ratios of 9:1, 4:1 and 7:3 for 1-day mix and 3 days after mixing. The liquid limit for the 1-day mix represented as D1-NS-BC-LL increased from 82 % to the peak values of 88 %, 86 % and 85 % at 2 % binder content with an increase in CS/lime ratio from 9:1, 4:1 and 7:3 respectively. Further addition of binder content to the soil ranging from 4 % to 10 %, reduced the D1-NS-BC-LL to 69 %, 68 % and 60 % at 10 % binder content respectively. The plastic limit for 1-day mix represented as D1-NS-BC-PL increased from 37 % to 40 %, 41 % and 42 % at 2 % binder content with an increase in CS/lime ratio from 9:1, 4:1 and 7:3 respectively. Further addition of binder content to the soil ranging from 4 % to 10 %, increase the D1-NS-BC-PL to 44 %, 46 % and 47 % at 10 % binder content with an increase in CS/lime ratio from 9:1, 4:1 and 7:3 respectively. The plasticity index for the 1-day mix represented as D1-NS-BC-PI decreased from 45 % to 43 % at 2 % binder content at the CS/lime ratio of 9:1. It increases to 48 % at 2 % binder content at the CS/lime ratio of 4:1 while it remains constant at 45 % at 2 % binder content at the CS/lime ratio of 7:3. Further addition of binder content to the soil from 4 % to 10 % by 2 % increment, decreased the D1-NS-BS-PI to 25 %, 22 % and 13 % at 10 % binder content with an increase in CS/lime ratio from 9:1, 4:1 and 7:3 respectively (see Appendix 1).

The liquid limit after 3 days mixing represented as D3-NS-BC-LL increased from 82 % to a peak of 87 %, 86 % and 85 % at 2 % binder content with an increase in CS/lime ratios from 9:1, 4:1 and 7:3 respectively. Further addition of binder content to the soil from 4 % to 10 % by 2 % increment continued to decrease the D3-NS-BC-LL up to 10 % to 68 %, 66 % and 60 % respectively. The Plastic limit after 3 days mixing represented as D3-NS-BC-PL increased from 37 % to 43, 44 % and 44 % at 2 % binder content with an increase in CS/lime ratios from 9:1, 4:1 and 7:3 respectively. Further addition of binder content to the soil ranging from 4 % to 10 %, increase the D3-NS-BC-PL to 46 %, 46 % and 48 % at 10 % binder content respectively. The plasticity index after 3 days mixing represented as D3-NS-BC-PI decreased from 45 % to 43 %, 44 % and 42 % at 2 % binder content with increase in CS/lime ratio from 9:1, 4:1 and 7:3 respectively. Further addition of binder content to the soil from 4 % to 10 % by 2 % increment also decreased the D3-NS-BC-PI to 22 %, 20 % and 12 % respectively (see Appendix 1).

It was observed that the addition of 10 % binder content to the natural soil with increasing CS/lime replacement ratios from 9:1 to 7:3 (Figures 4.10 to 4.12) moved the soil from being classified as very high plasticity silty-clay to being classified as high plasticity silt in the Casagrande chart (Figure 4.13). The modified soil has become friable and the particle sizes turn bigger which shows significant change in plasticity characteristics of the modified soil.

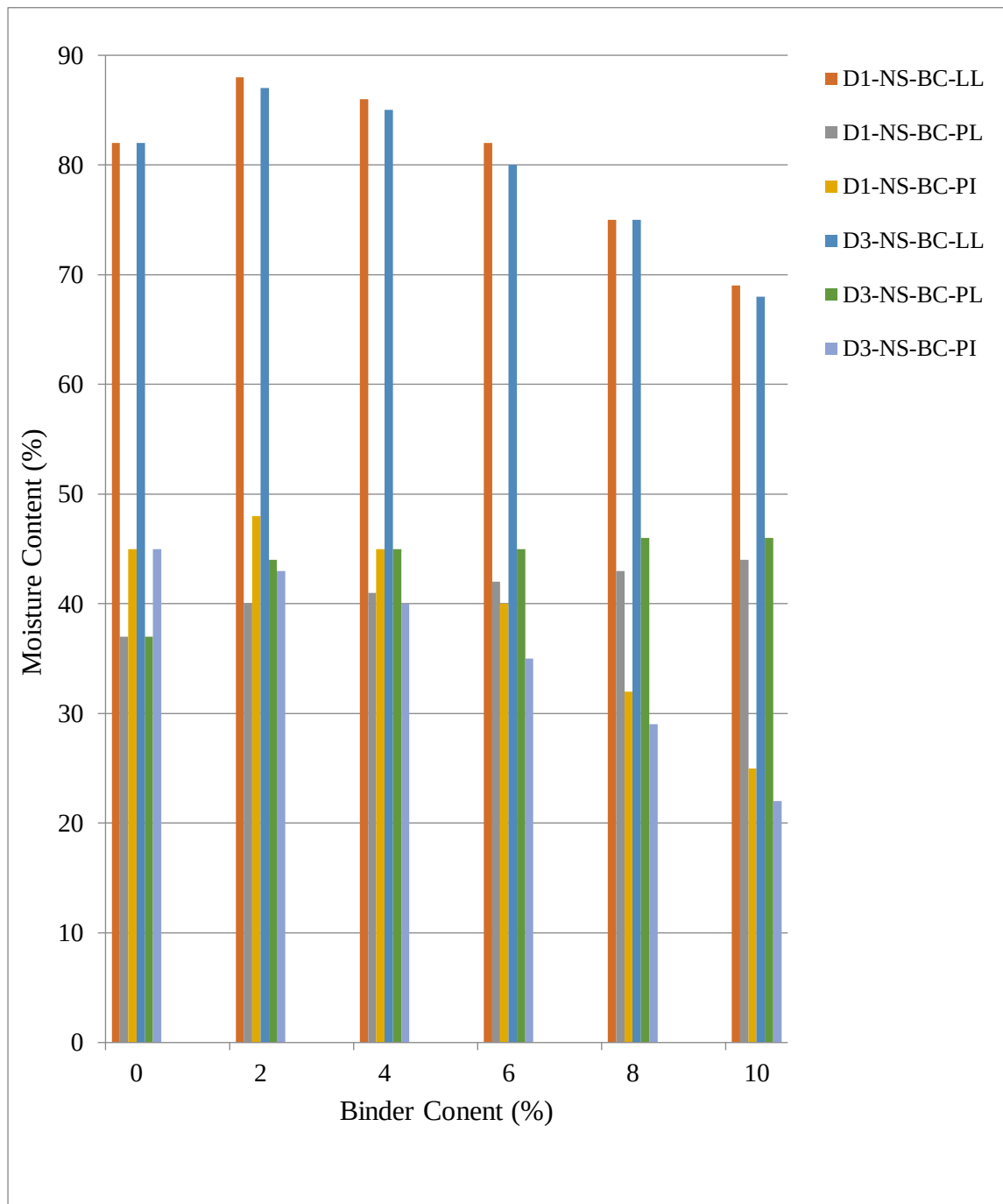


Figure 4.10: Atterberg limits for the composite of natural soil, CS and hydrated lime at the ratio of 9:1 CS replacement

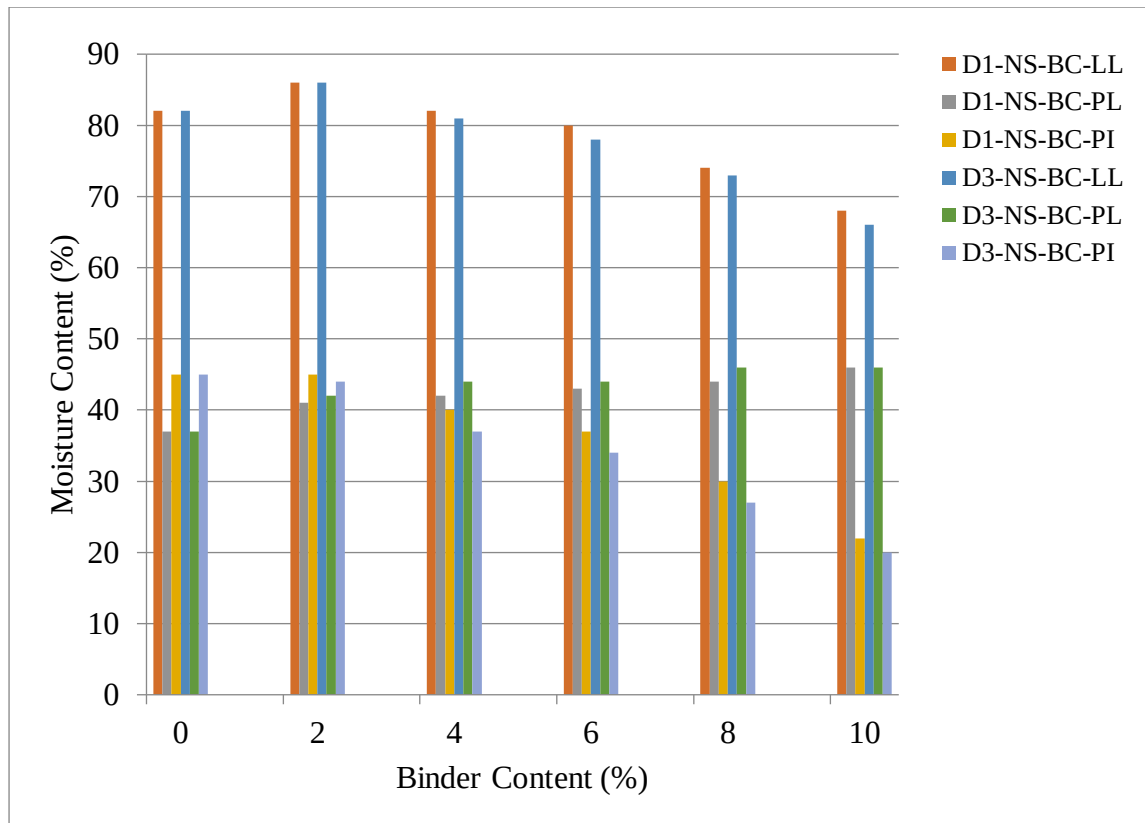


Figure 4.11: Atterberg limits for the composite of natural soil, CS and hydrated lime at the ratio of 4:1 CS replacement

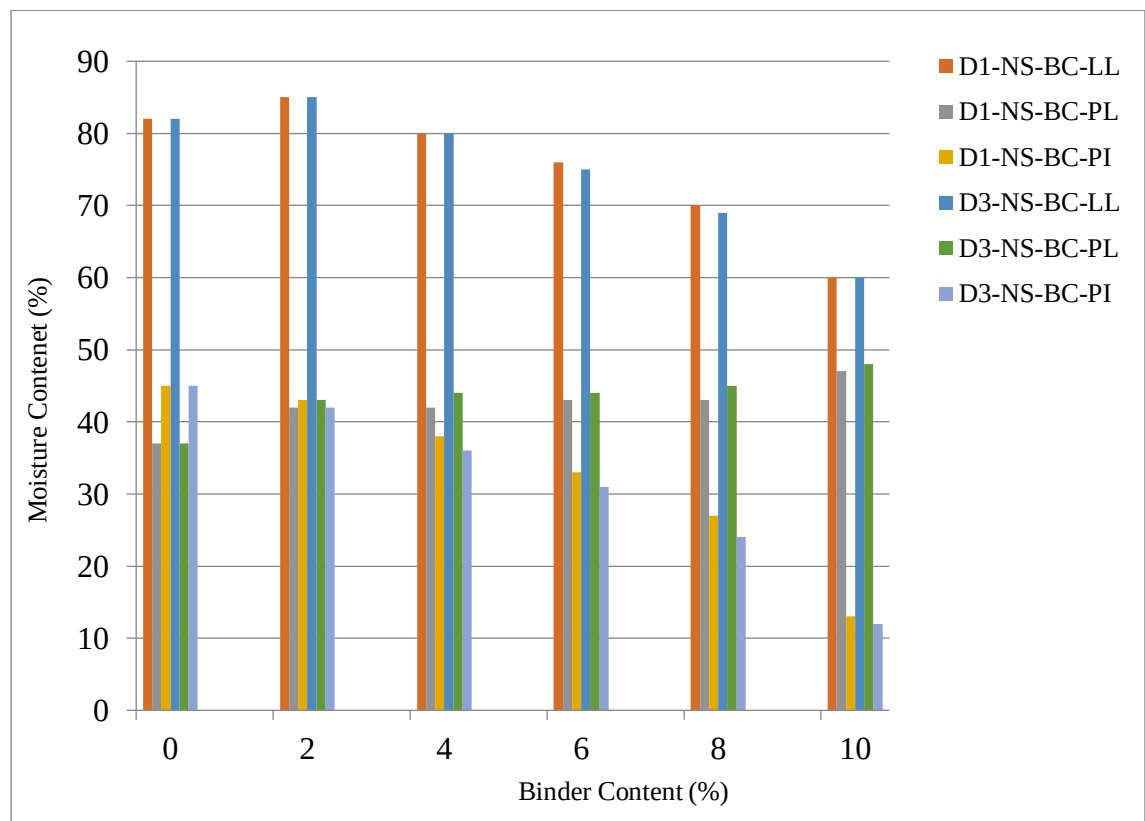


Figure 4.12: Atterberg limits for the composite of natural soil, CS and hydrated lime at the ratio of 7:3 CS replacement

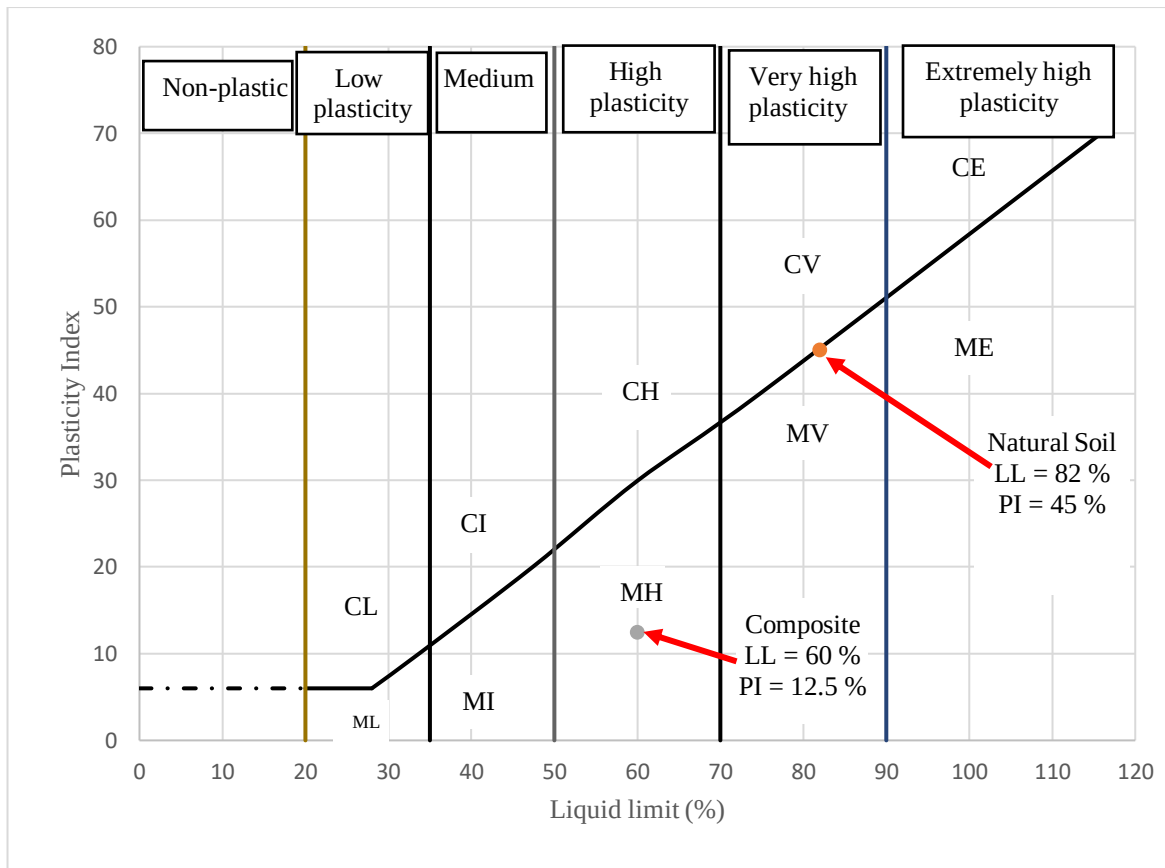


Figure 4.13: Effect of CS and hydrated lime on plasticity change in the natural soil in a typical Casagrande chart for the classification of the composite

4.4.3 Effect of Hydrated Lime on the Plasticity Characteristics of the Natural Soil

Figure 4.14 presents the results of the effect of hydrated lime on the Atterberg limits of the soil for 1-day mix and 3 days after mixing. The liquid limit for the 1-day mix represented as D1-NS-LC-LL increased from 82 % to a peak value of 86 % at 4 % lime content. Further addition of lime content to the soil from 6 % to 10 % by 2 % increment reduced the D1-NS-LC-LL to 80 % at 10 % lime content. The plastic limit for the 1-day mix represented as D1-NS-LC-PL increased from 37 % to 47 % at 4 % lime content. Further addition of lime content to the soil from 6 % to 10 % by 2 % increment, increased the D1-NS-LC-PL to 50 % at 10 % lime content. The plasticity index for 1-day mix represented as D1-NS-LC-PI decreased from 45 % to 39 % at 2 % lime content. Further addition of lime content to the soil from 4 % to 10 % by 2 % increment, decreased the D1-NS-LC-PI to 30 % at 10 % lime content (See Appendix 1).

The liquid limit after 3 days mixing represented as D3-NS-LC-LL decreased from 82 % to 79 % at 2 % lime content. Further addition of lime content from 4 % to 10 % by 2 %

increment to the soil decreased the D3-NS-LC-LL to 76 % at 10 % lime content. The plastic limit after 3 days mixing represented as D3-NS-LC-PL also increased from 37 % to 49 % at 4 % lime content. Further addition of lime content to the soil from 6 % to 10 % by 2 % increment, also increased the D3-NS-LC-PL to 50 % at 8 % and 10 % lime content. The plasticity index after 3-days mixing represented as D3-NS-LC-PI decreased from 45 % at 2 % lime content. Further addition of lime content to the soil from 4 % to 10 % by 2 % increment, decreased the D3-NS-LC-PI to 26 % at 10 % lime content (See Appendix 1).

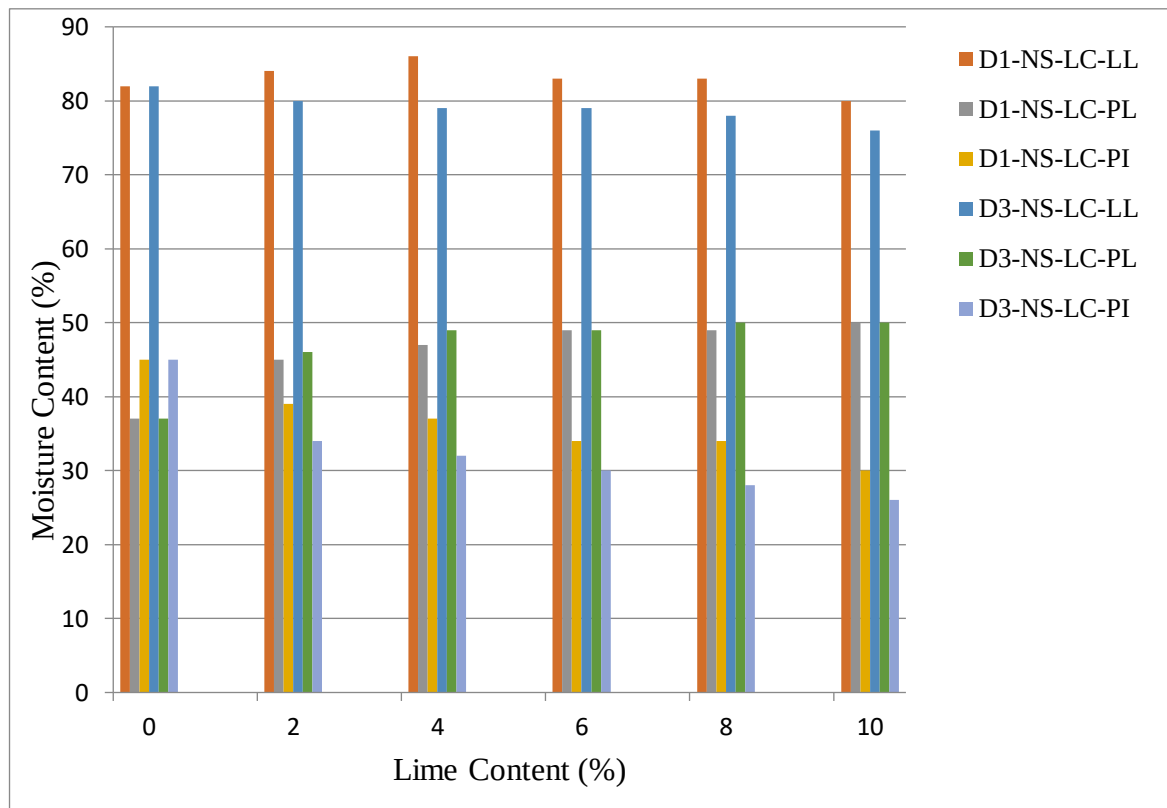


Figure 4.14: Atterberg limits for the natural and hydrated lime

4.5 Effect of Copper Slag and hydrated Lime on the pH of the Natural Soil

Figure 4.15 presents the results of the effect of copper slag and hydrated lime on the pH of the natural soil with different percentages. The minimum binder content sufficient to cause the silica and aluminium in the clay to dissolve in the natural soil-copper slag-hydrated lime system was determined at 4 % CS and 2 % lime for the stabilization process.

The addition of 2 % CS to the natural soil raised the pH significantly from 8.21 to 9.71 (see Figure 4.15). An increase in the CS resulted in a sharp increase in the pH up to 10.42 at 4 % CS. The 4 % CS with 4:1 ratio of lime addition at total binder content to natural soil

increased the pH to 11.55 and thereafter, an increase in the binder content to 6 % resulted in further increase in the pH to 12.55. A Further increase in the binder content up to 11 % and 15 %, increase the pH of the natural soil to 12.65. The effect of CS with 24 % lime content gives the same result as 11 % lime content. The addition of lime content from 11 %, 15 % and 24 % lime to the natural soil increased the pH to 12.65 which remained constant without change in the pH value.

It is observed that the pH of the soil remained constant at 12.65 with the addition of lime or CS at 11 % binder content. Thus, 6 % binder content is sufficient to cause full modification of the natural soil-CS-lime mixture. The initial binder content was confirmed by the plastic limit test at 4 % binder content called binder fixation point depending on the CS/lime ratio.

The changes in each of the pH ($\approx 91\%$) took place at 6 % binder content of the composite of natural soil + CS, natural soil + lime and natural soil + CS + lime. Therefore, 6 % binder content was chosen as the minimum amount of binder needed for both modification and stabilization of the natural soil. Thus, all probes of natural soil-CS-lime testing including UCS and swelling characteristics were conducted on test specimens with maximum binder content of 6 %.

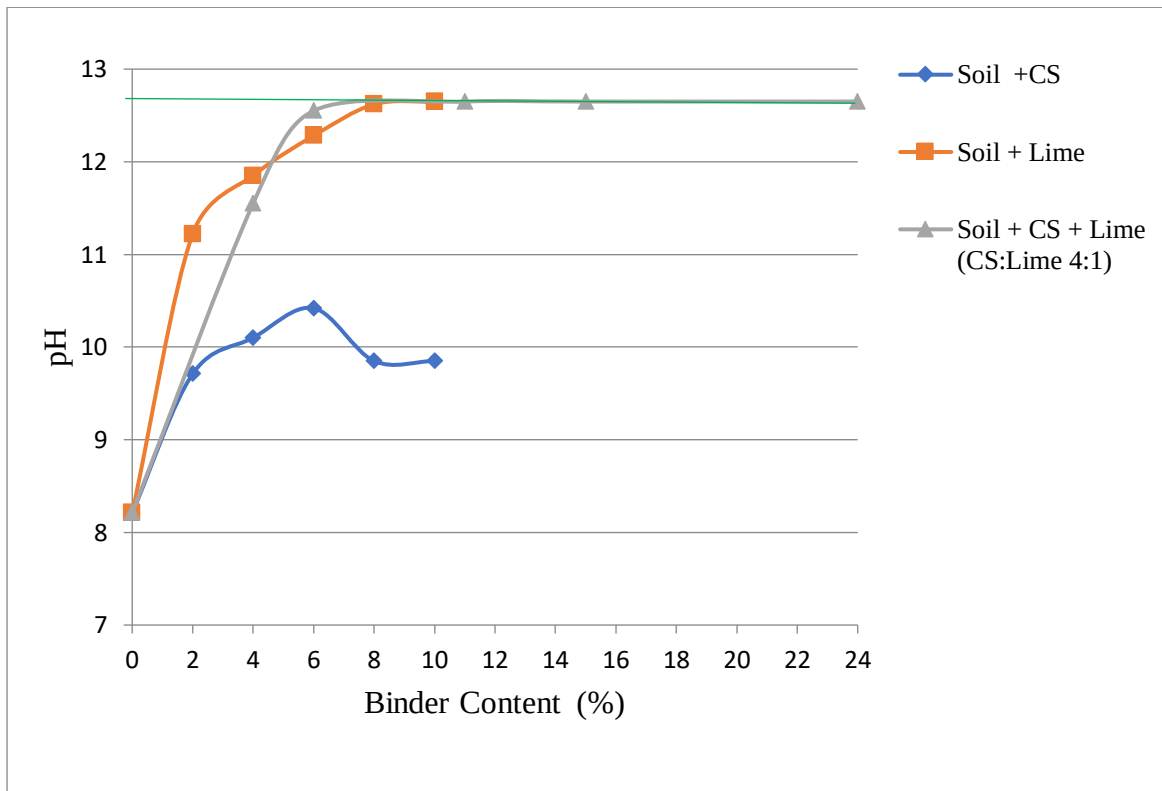


Figure 4.15: Effect of binder content on the pH value of the natural soil at 22 °C

4.6 Unconfined Compressive Strength of Soil-Copper Slag and Soil-Copper Slag-Hydrated Lime

The effect of CS without and with hydrated lime on the UCS characteristics of the soil was determined in accordance with BS: 1377 (1990) and ASTM 1988. Six curing periods namely: 7 days, 16 days, 28 days, 42 days, 56 days and 74 days were applied to the specimens under the two curing conditions referred to as CC1 and CC2.

4.6.1 Effect of the Addition of Copper Slag on the Unconfined Compressive Strength of the Natural Soil

Figures 4.16 and 4.17 present the results of the effect of CS percent on the UCS of the soil under CC1 and CC2 after 7, 16, 28, 42, 56 and 74 days. Generally, as the CS content and curing period increased, the UCS also increased under both conditions. Under CC1 and CC2, two groups of UCS increase are observed. The first group is for the 7 days and 16 days curing period, which show that the UCS slightly increased to the maximum peak values of 799 kN/m² and 875 kN/m² at 4 % CS content under CC1. Under CC2 the maximum peak values of 1000 kN/m² and 1111 kN/m² at 4 % and 6 % CS content respectively were achieved. Both CC1 and CC2 UCS values slightly decreased as the CS content was progressively increased to 10 % (See Appendix 2).

The second group of results are identified with the 28 days, 42 days, 56 days and 74 days curing period. In this group, sharp increase in the UCS values were observed. The UCS in this group increase up to maximum peak values of 990 kN/m², 999 kN/m², 999 kN/m² and 1000 kN/m² at 4.6 %, 5 %, 5 %, 5.7 % CS content under CC1 respectively while under CC2 the UCS increased up to the maximum peak values of 1285 kN/m², 1329 kN/m², 1371 kN/m² and 1399 kN/m² at 6.5 % CS content respectively. Both UCS values under condition 1 and 2 slightly decrease as the CS content was progressively increased to 10 %.

The lowest increase in UCS values under CC1 and CC2 were at 4 % CS content after 7 days curing with UCS values of 799 kN/m² and 1000 kN/m² respectively with their corresponding proportions of CS content at 5.7 % and 6.5 % CS after 74 days curing.

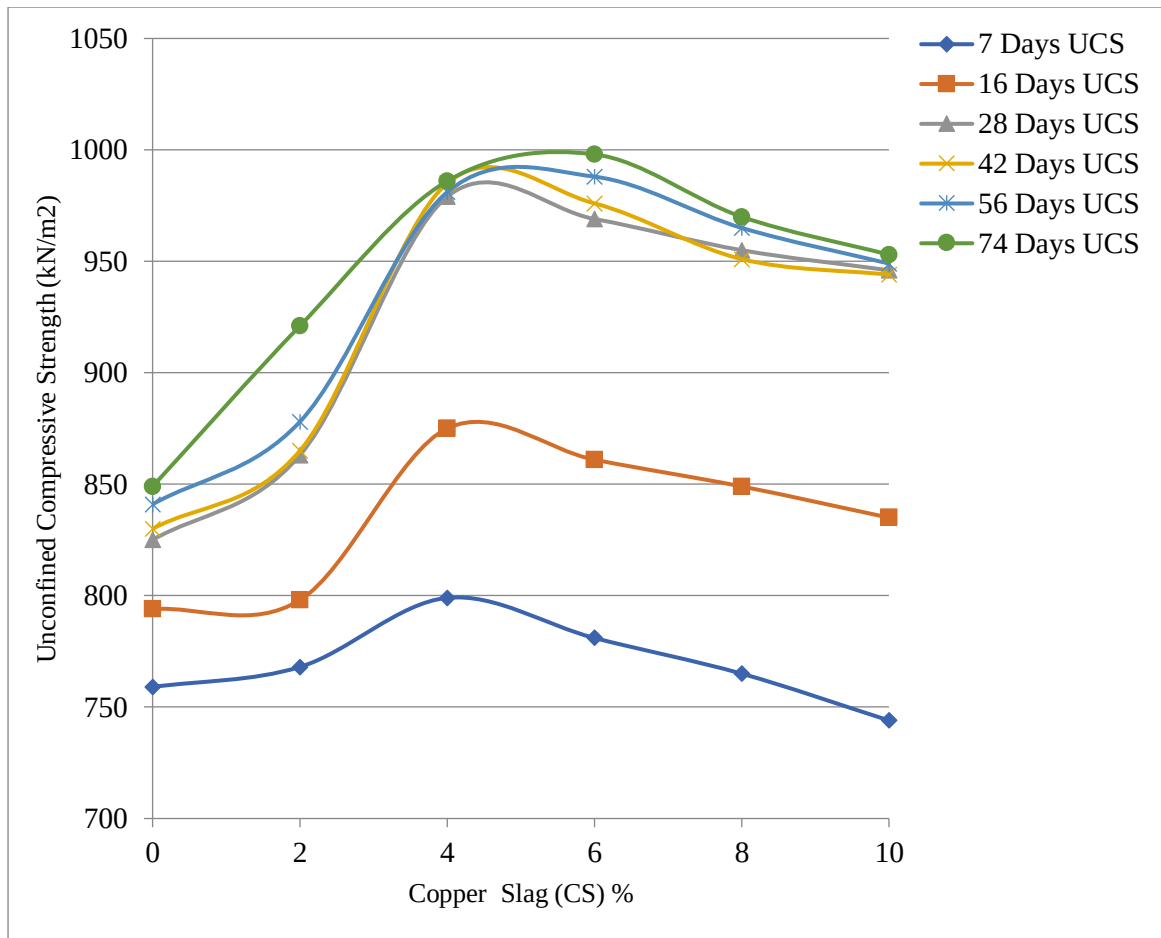


Figure 4.16: Effect of CS percent on the UCS of the soil under CC1 after 7 days, 16 days, 28 days, 42 days, 56 days and 74 days

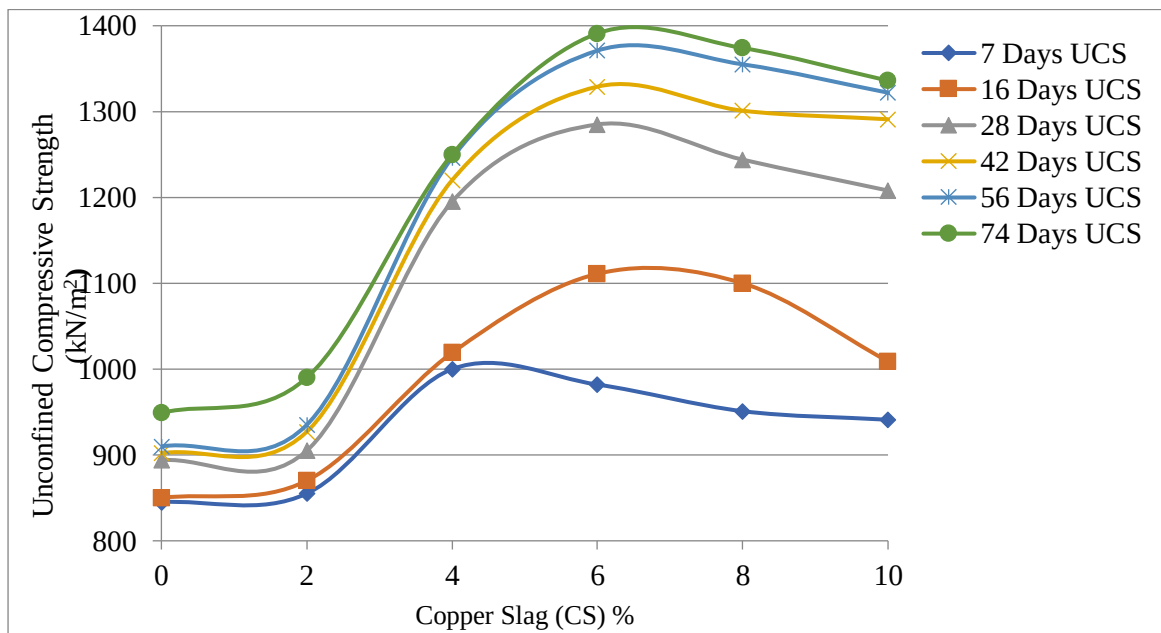


Figure 4.17: Effect of CS percent on the UCS of the soil under CC2 after 7days, 16 days, 28 days, 42 days, 56 days and 74 days

4.6.1.1 Effect of Copper Slag on Unconfined Compressive Strength of the Natural Soil under Varying Curing Conditions

The results of the effect of CS on unconfined compressive strength of the natural soil under CC1 and CC2 are presented in Figures 4.16 and 4.17. All specimens were cured for 7 days to 74 days. After 56 days and 74 days curing period, under CC1 the maximum strength increased to 990 kN/m² and 1000 kN/m² at 5 % and 5.7 % CS content respectively. However, under CC1, upon the addition of CS to the soil; an increase in the strength was observed for 7, 16, 28, and 42 days curing of the specimens at 4.6 %, 5 %, 5 %, 5.7 % CS binder content of their respective UCS values presented in Section 4.6.1. Under CC2 the strength achieved after 16, 28, 42 56 days curing was at 6 % CS content while 74 days curing was at 6.3 % CS content as presented in Section 4.6.1.

The strength for 7 days curing period at 4 % CS content was determined to be 1000 kN/m² under CC2. Thereafter, the maximum strength under both CC1 and CC2 gradually decreased as shown in Figures 4.16 and 4.17 respectively (see Appendix 2). With longer curing time, significant increase in strength is observed with higher degree of CS at a higher temperature (see Figures 4.16 and 4.17 and Appendix 2).

Generally, as the binder content and curing period of the specimens increased, a peak strength value is attained between 4 % and 5 % binder content except in the case of 74 days curing period when the peak strength was achieved between 4 % and 6.3 % binder content under CC1. Similarly, for the case of CC2, the strength increased as the curing period and binder content increased. However, the peak strength values were obtained at about 6.5 % binder content except for the 7 days curing period when the peak strength is attained at 4 % binder content.

4.6.1.2 Effect of Curing Periods on the Unconfined Compressive Strength of the Natural Soil and Composite Soil under Different Curing Conditions

The results of curing period effect on the unconfined compressive strength of the natural soil under both CC1 and CC2 for a curing period of 7 days to 74 days are presented in Figures 4.18 and 4.19 respectively. Generally, as the curing period increased from 7 days to 74 days and CS content increased from 2 % to 10 % and curing temperature from 35 °C to 45 °C, the UCS also increased under both conditions. The UCS under both CC1 and CC2 sharply increased at 28 days and then continued to increase up to 74 days curing. The strength of the natural soil increased from 759 kN/m² to 849 kN/m² under CC1 and under

CC2 from 845 kN/m² to 949 kN/m². This may be attributed to higher curing period and temperature.

Under CC1, there are two groups of UCS increase that are observed. The first group is for the 0 % and 2 % CS binder, which the UCS values sharply increase after 28 days curing to 825 kN/m² and 863 kN/m² under CC1 while under CC2, UCS values slightly increased after 28 days curing period to 894 kN/m² and 927 kN/m² respectively (see Figures 4.19 and 4.20). The increase in the UCS values under CC1 is higher than for CC2 which slightly increased as the curing period increases up to 74 days.

The second group is the 4 %, 6 %, 8 % and 10 % binder content. In this group, sharp increase was observed in the UCS at 28 days curing with UCS values of 985 kN/m², 969 kN/m², 955 kN/m² and 946 kN/m² under CC1 respectively while under CC2 at 28 days curing, the UCS values are 1195 kN/m², 1285 kN/m², 1244 kN/m² and 1208 kN/m² respectively. Both UCS values under conditions 1 and 2 slightly increased as the curing period and binder content increased up to 74 days and 10 % respectively. In this second group, the UCS sharp increase was observed at 4 % CS binder at 28 days up to 42 days curing and slightly decreased below the 6 % CS binder content up to 74 days curing under CC1. The UCS of the 10 % CS binder content decreased below the 4 %, 6 % and 8 % binders under CC1. Under CC2, the UCS sharp increase was observed at 6 % CS binder at 28 days curing and slightly continued to increase as the curing period increased up to 74 days. For the 10 % CS binder content, the UCS decreased below the 6 % and 8 % binders under CC2 (see Figures 4.18 and 4.19 and Appendix 2).

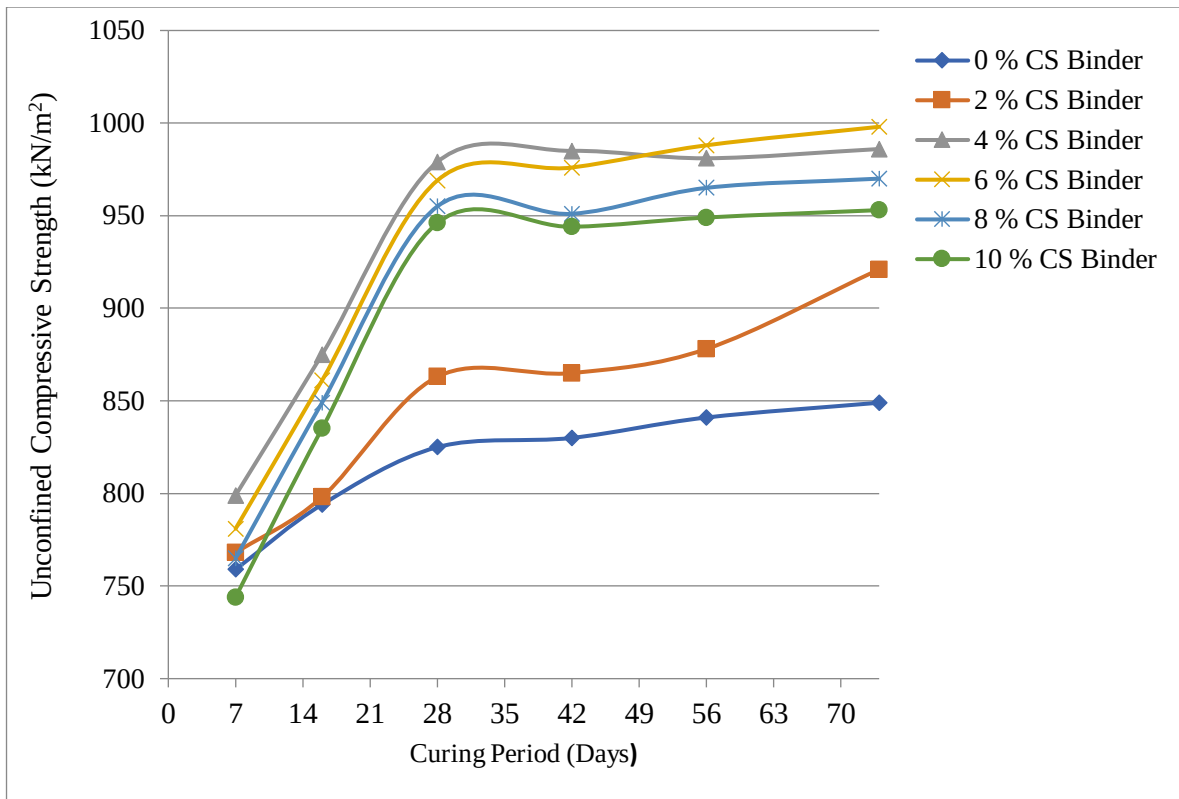


Figure 4.18: Effect of curing period on the UCS of the soil mixed with 0 %, 2 %, 4 %, 6 %, 8 and 10 % CS under CC1

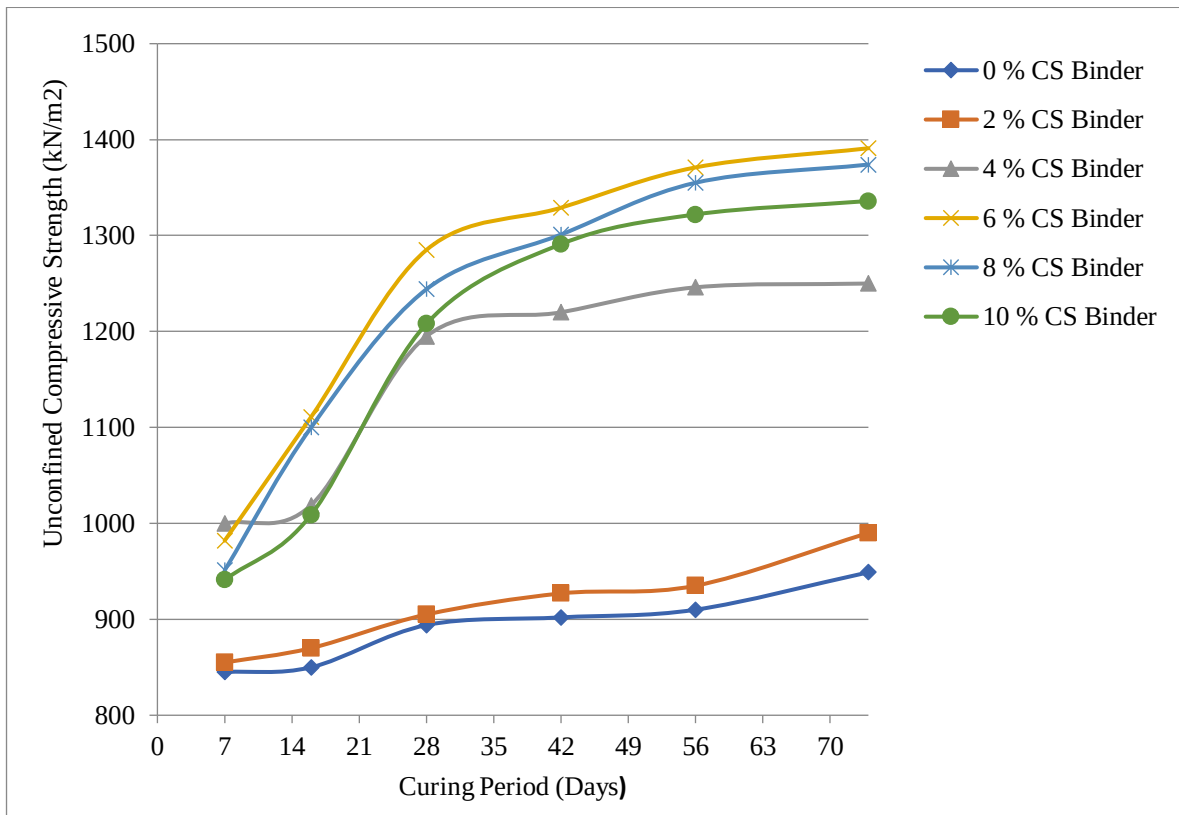


Figure 4.19: Effect of curing period on the UCS of the soil mixed with 0 %, 2 %, 4 %, 6 %, 8 % and 10 % under CC2

4.6.1.3 Effect of Curing Period on the Moisture Content of the UCS of the Composite Soil

Figures 4.20 and 4.21 present the effect of curing period on the moisture content of the UCS of the composite comprising of natural soil and CS. Generally, as the curing period varies from 7 days to 74 days and the CS binder content increased from 2 % to 10 % by 2 % increment under CC1 and CC2, the moisture content of the UCS decreased slightly. Under CC1 and CC2, two groups of moisture content are observed from Figures 4.20 and 4.21. The two groups of moisture content under both conditions are 0 %, 2 % and 4 % CS-UCS-MC and the 6 %, 8 % and 10 % CS UCS-MC. The decrease in the moisture content of the 0 %, 2 % and 4 % CS-UCS-MC are less than the decrease in the moisture content of the 6 %, 8 % and 10 % CS-UCS-MC.

The decrease in the UCS moisture content is due to an increase in the soil pore water suction and also due to some chemical reactions particularly; during long curing period at a higher temperature. The pore water suction was not measured in this study directly, however, the effect of the change in moisture content on the unconfined compressive strength of the composite soil was measured as the pore water suction; calculated as per the lost in moisture content on the UCS in terms of pore suction and pozzolanic reaction products.

The curing period effect on the moisture content of the UCS test results and percentage of increase in strength of the composite is calculated as shown in Appendix 3. For any mix of the natural soil and CS, the UCS of the natural soil is subtracted from a specific mix at the needed curing period and conditions. For example, the unconfined compressive strength of the natural soil under CC1 after 7 days is 759 kN/m². An increase in the curing period to 28 days caused an increase in the strength to 825 kN/m² (8 % increase). When 4 % CS content was added, the strength increased to 979 kN/m² (16 % increase) at 28 days curing period. From this example, it is observed that the increase in the UCS due to the change in pore water suction is 56 kN/m² while the increase due to the formation of pozzolanic reaction product is 154 kN/m² under CC1. As a further example, the UCS for the natural soil under CC2 after 74 days is 949 kN/m². Increasing the binder content to 6 %, the unconfined compressive strength increased to 1391 kN/m² (40 % increase). It is observed that the increase in the unconfined compressive strength due to change in pore water suction is 104 kN/m² and the increase due to chemical reaction is 442 kN/m² under CC2. It is observed that the effect of change in pore water suction on the unconfined compressive strength

slightly increased with an increase in curing period and temperature. Therefore, the change in the UCS due to change in the pore water suction is comparative to the increase due to pozzolanic reaction of the natural soil and CS alone. The increase due to the pozzolanic reaction is comparatively less than that due to the chemical reaction. Thus, it is speculated that CS was activated by one or more oxides present in the mixture with high calcium quality.

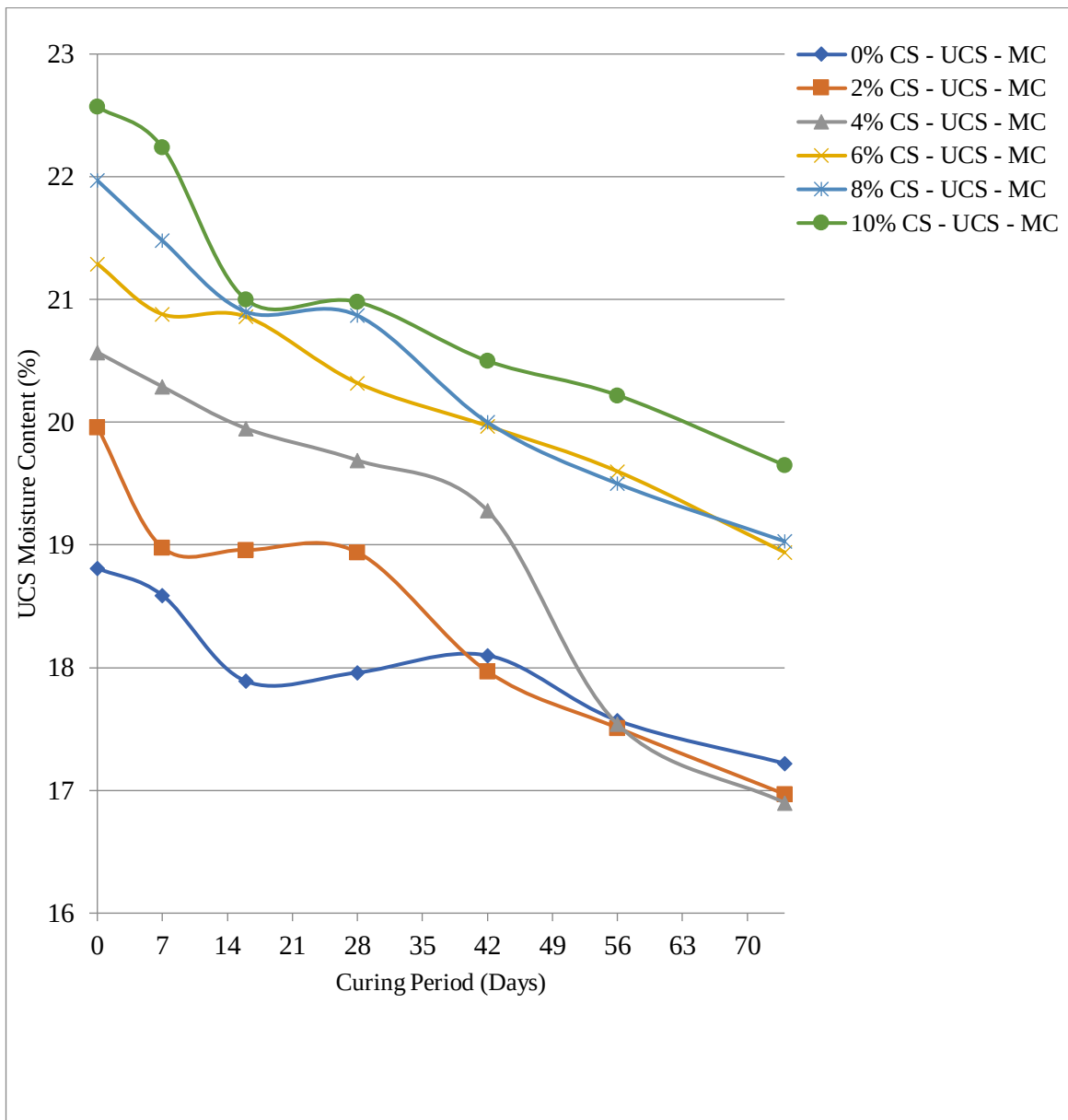


Figure 4.20: Moisture content against curing period of the soil and CS alone under CC1

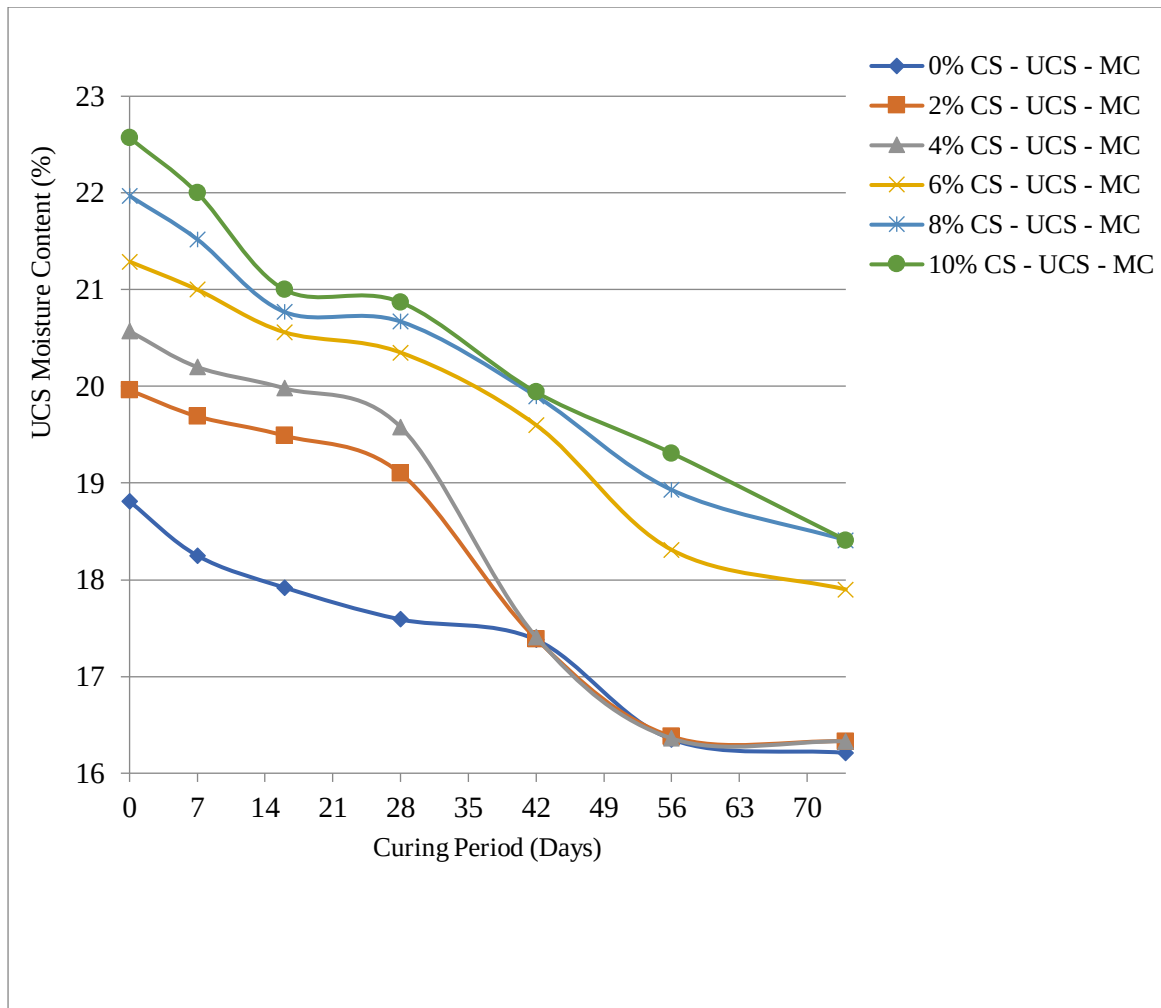


Figure 4.21: Moisture content against curing period of the soil and CS alone under CC2

4.6.2 Effect of CS and Hydrated Lime on the Unconfined Compressive Strength

The results of the effect of CS activated by the hydrated lime on the unconfined compressive strength of the composite soil are presented in Table 4.3 and Figures 4.22 to 4.24. Three total binder (CS and Hydrated lime) percentages; 2 %, 4 % and 6 % together with varied ratios of CS replacement by hydrated lime in the ratios 9:1, 4:1 and 7:3 were used for the second stage of the UCS test. The curing periods and curing conditions were maintained as in the previous sections. Generally, the UCS of the natural soil increased with increase in the total binder content and with an increase in CS/lime ratio under both conditions. There is also increase in the UCS with an increase in curing time and curing temperature for the same CS/lime ratio and total binder content under both conditions. There are three groups of binder contents under CC1 and CC2 namely, 2 % total binder content, 4 % total binder content and the 6 % total binder content.

Table 4.3: Effect of CS and hydrated lime on the UCS of the natural soil

CS (%)	Lime ratio (%)	UCS (kN/m ²) CC1						UCS (kN/m ²) CC2					
		Curing period											
		7 Days	16 Days	28 Days	42 Days	56 Days	74 Days	7 Days	16 Days	28 Days	42 Days	56 Days	74 Days
2	0	768	798	863	865	878	900	855	870	905	927	935	990
	10	791	820	924	994	996	1012	870	889	1097	1109	1149	1169
	20	824	850	959	1000	1110	1223	875	895	1169	1190	1251	1290
	30	846	866	990	1121	1220	1297	890	919	1299	1324	1352	1386
4	0	799	875	979	985	981	986	1000	1019	1195	1220	1246	1250
	10	1009	1020	1096	1116	1139	1159	1263	1276	1494	1568	1629	1658
	20	1040	1100	1210	1292	1321	1374	1781	1812	2295	2330	2390	2444
	30	1240	1320	1394	1425	1469	1490	2200	2224	2769	2791	2894	2931
6	0	781	861	969	976	988	998	982	1111	1285	1329	1371	1391
	10	1276	1295	1815	1900	2019	2191	1234	1254	1496	1596	1653	1691
	20	1799	1820	2597	2620	2691	2768	1865	1992	2556	2610	2695	2785
	30	2290	2310	2864	2988	3010	3138	2290	3090	3168	3225	3294	3321

The first group of total binder content of 2 % is presented in Figure 4.22. This figure presents the effect of CS/lime ratio with 2 % total binder content on the UCS of the composite soil under CC1 and CC2. In this group, the UCS for 7 days and 16 days curing periods increased from 768 kN/m² to 846 kN/m² and from 798 kN/m² to 875 kN/m² under CC1 as the CS/lime ratio increased ranging from 10 % to 30 % (9:1 to 7:3) respectively. The UCS under CC2 for 7 days and 16 days curing period increased from 855 kN/m² to 890 kN/m² and from 870 kN/m² to 919 kN/m² under CC2 as the CS/lime ratio increased ranging from 10 % to 30 % (9:1 to 7:3) respectively. The UCS under both CC1 and CC2 for 7 days and 16 days curing periods have less increase in strength as compared with the values for 28 days curing period to 74 days curing period. The increase in UCS for 7 days and 16 days curing under CC2 is much higher than the UCS under CC1. For the 28, 42, 56 and 74 days curing under CC1, the UCS increased to 990 kN/m², 1121 kN/m², 1220 kN/m² and 1297 kN/m² as the CS/lime ratio increased from 10 % to 30 % (9:1 to 7:3) while under CC2, the 28, 42, 56 and 74 days curing UCS increased to 1299 kN/m², 1324 kN/m², 1352 kN/m² and 1386 kN/m² as the

CS/lime ratio increased from 10 % to 30 %. The highest increase in UCS under CC1 and CC2 was at 74 days curing period (see Figure 4.9).

The second group of total binder content of 4 % is presented in Figure 4.23. In this figure, the effect of CS/lime ratio with 4 % total binder content on the UCS of the composite soil under CC1 and CC2 are presented. This group includes the 7 days, 16 days, 28 days, 42 days, 56 days and 74 days curing periods. The UCS of these curing periods increased to 1240 kN/m², 1320 kN/m², 1394 kN/m², 1425 kN/m², 1469 kN/m² and 1490 kN/m² under CC1 while under CC2 to 2200 kN/m², 2224 kN/m², 2769 kN/m², 2791 kN/m², 2894 kN/m², 2931 kN/m² as the CS/lime ratio increased from 10 % to 30 % (9:1 to 7:3) respectively. The 7 days and 16 days curing periods under CC2 have low increase in UCS than the 28 days, 42 days, 56 days and 74 days curing. The increase in UCS under CC2 is much higher than the UCS under CC1 (see Table 4.9).

Figure 4.24 presents the third group of total binder content of 6 %. This figure presents the effect of CS/lime ratio with 6 % total binder content on the UCS of the composite soil under CC1 and CC2. In this group, 7 days and 16 days curing period have the lowest increase in UCS from 781 kN/m² to 2290 kN/m², 861 kN/m² to 2310 kN/m² under CC1 and from 982 kN/m² to 2290 kN/m², 1111 kN/m² to 3090 kN/m² under CC2 as the CS/lime ratio increased from 10 % to 30 % (9:1 to 7:3) respectively. The increase in UCS for 7 days and 16 days curing periods under CC2 is much higher than the UCS under CC1. The increase in UCS for the 28, 42, 56 and 74 days curing under CC1, increased to 2864 kN/m², 2988 kN/m², 3010 kN/m² and 3138 kN/m² as the CS/lime ratio increased from 10 % to 30 % (9:1 to 7:3) while under CC2, the 28, 42, 56 and 74 days curing UCS increased to 3168 kN/m², 3225 kN/m², 3294 kN/m² and 3321 kN/m² as the CS/lime ratio increased from 10 % to 30 % (9:1 to 7:3). The highest increase in UCS under CC1 and CC2 was at 74 days curing period. The UCS for the 28, 42, 56 and 74 curing periods under CC1 has sharp increase up to 20 % (4:1) CS/lime ratio and slightly fall while under CC2 the UCS slightly increased as the CS/Lime ratio increased 30 % (7:3) (see Table 4.9).

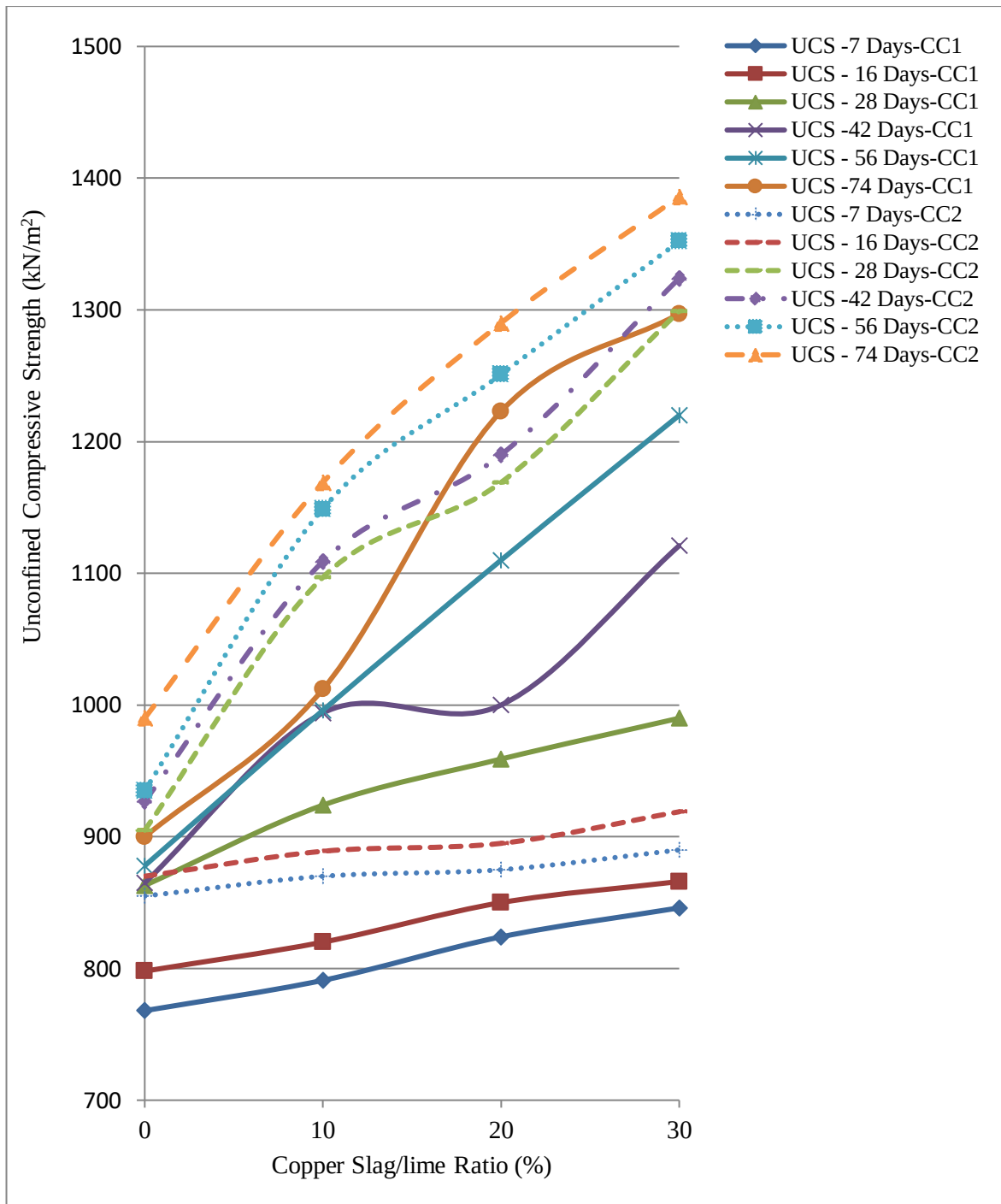


Figure 4.22: Effect of CS/lime ratio on the UCS of the composite soil at 2 % total binder content under CC1 and CC2

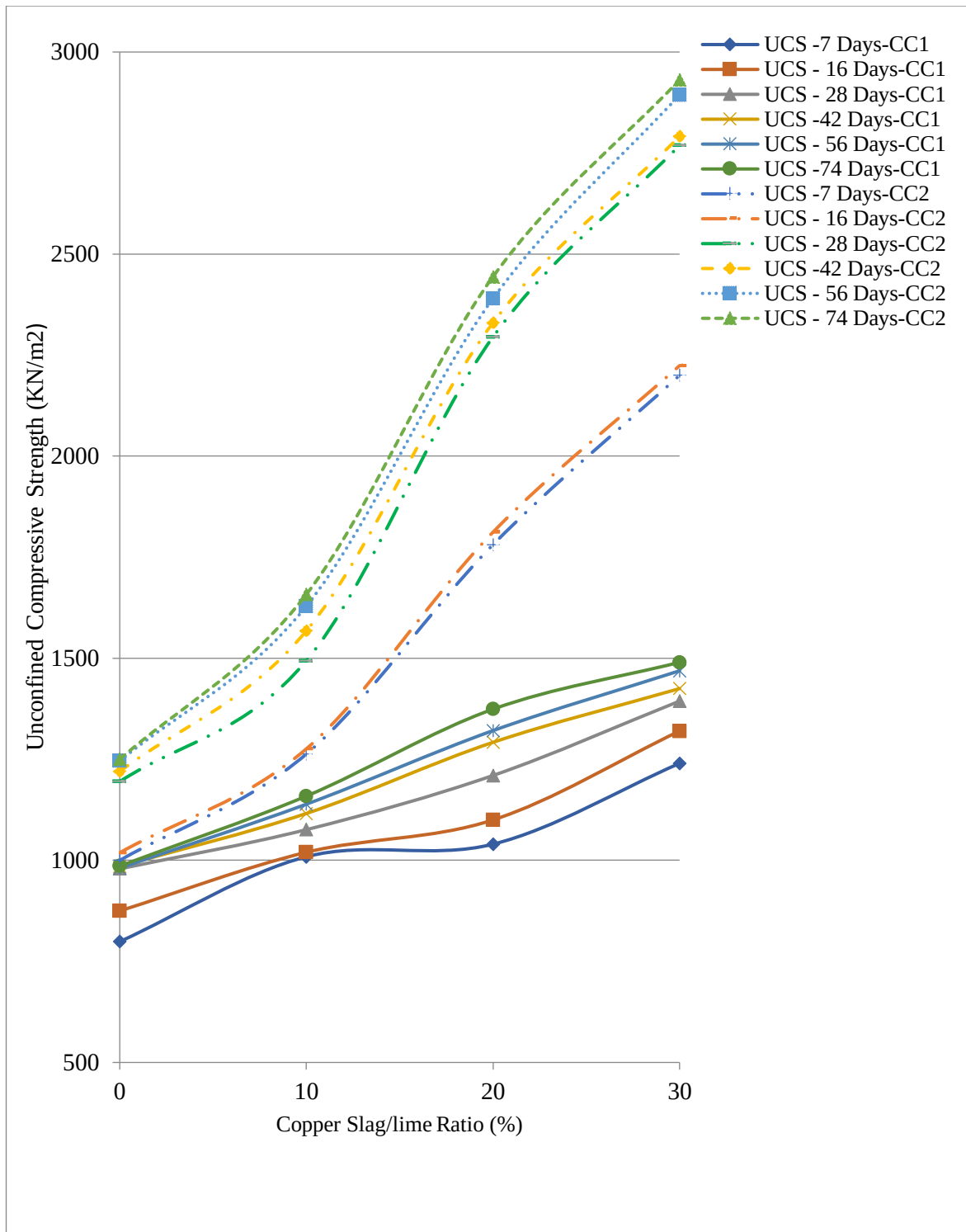


Figure 4.23: Effect of CS/lime ratio on UCS of the composite soil at 4 % total binder content under CC1 and CC2

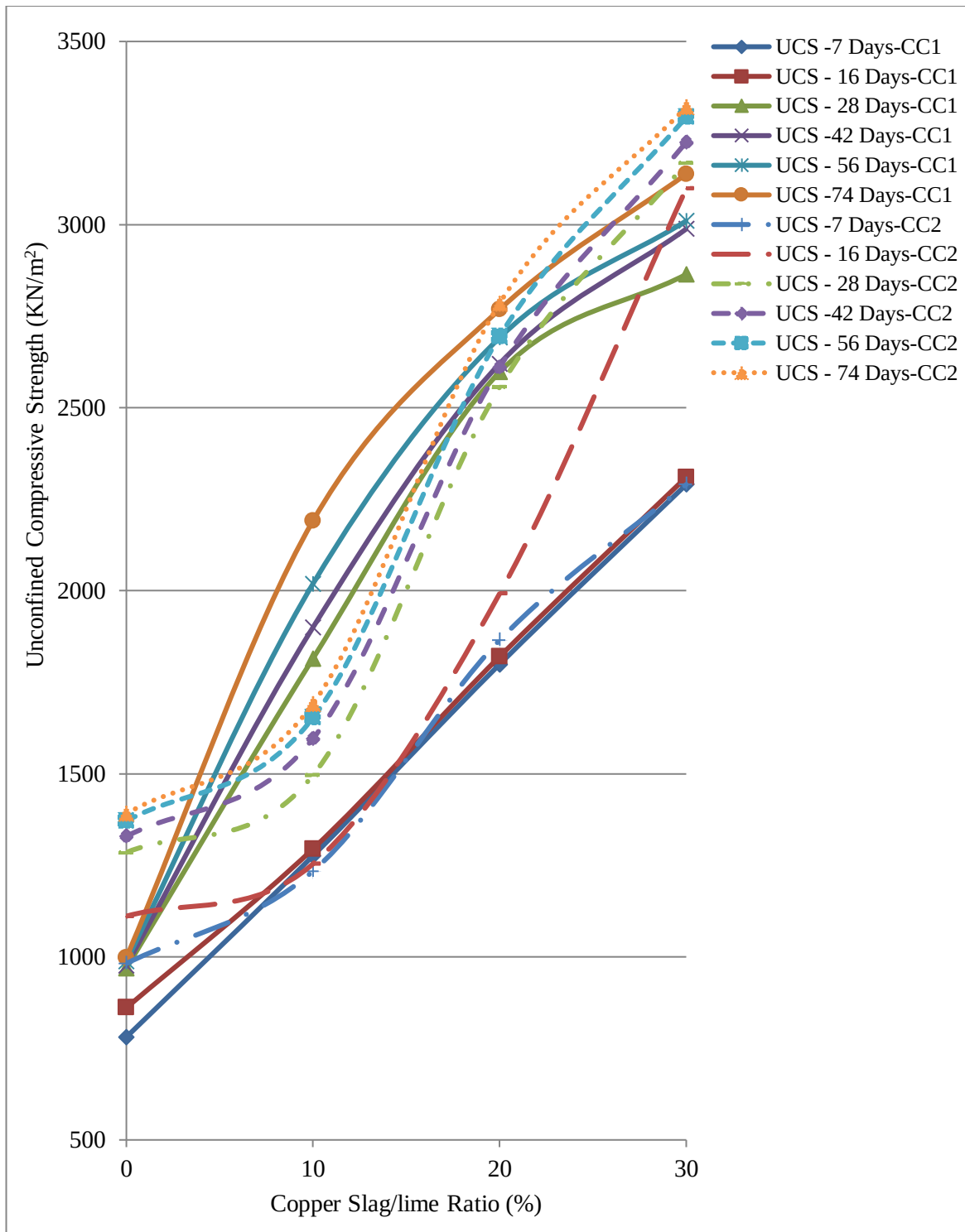


Figure 4.24: Effect of the CS/lime ratio on UCS of the composite soil at 6 % total binder content under CC1 and CC2

4.6.2.1 Effect of Curing Period on the Initial Moisture Content of the Soil Treated with CS and Hydrated Lime

Figures 4.25 and 4.26 present the results of the effect of curing period on the initial moisture content of the composite soil under both CC1 and CC2. The initial moisture content decreased as the curing period increase from 7 days to 28 days. The effect of curing period on the initial moisture content and percentage of increase in UCS of the composite, it is calculated as mentioned in Section 4.6.1.3 and shown in Appendix 3. For example, the unconfined compressive strength under CC1 at 4 % total binder content after 7 days is 799 kN/m². An increase in the curing period at the same total binder content of 4 % under CC1 after 74 days at the ratio of 7:3 CS replacement by hydrated lime increased the UCS to 1490 kN/m² (47 % increase). At 6 % total binder content after 16 days at the ratio of 7:3 CS replacement by hydrated lime under CC2 caused an increase in the UCS to 3090 kN/m² (74 % increase). An increase in the curing period to 74 days at 6 % total BC content at the ratio of 7:3 CS replacement by hydrated lime, significantly increased the unconfined compressive strength to 3321 kN/m² (76 % increase) under CC2. From this example, it is observed that the increase in the unconfined compressive strength due to the change in pore water suction is 681 kN/m² while the increase due to the chemical reaction of CS activated by hydrated lime is 1831 kN/m². From the previous example, it is observed that the effect of change in pore water suction on the UCS is little compared to the effect of the chemical reaction on the UCS of the composite soil.

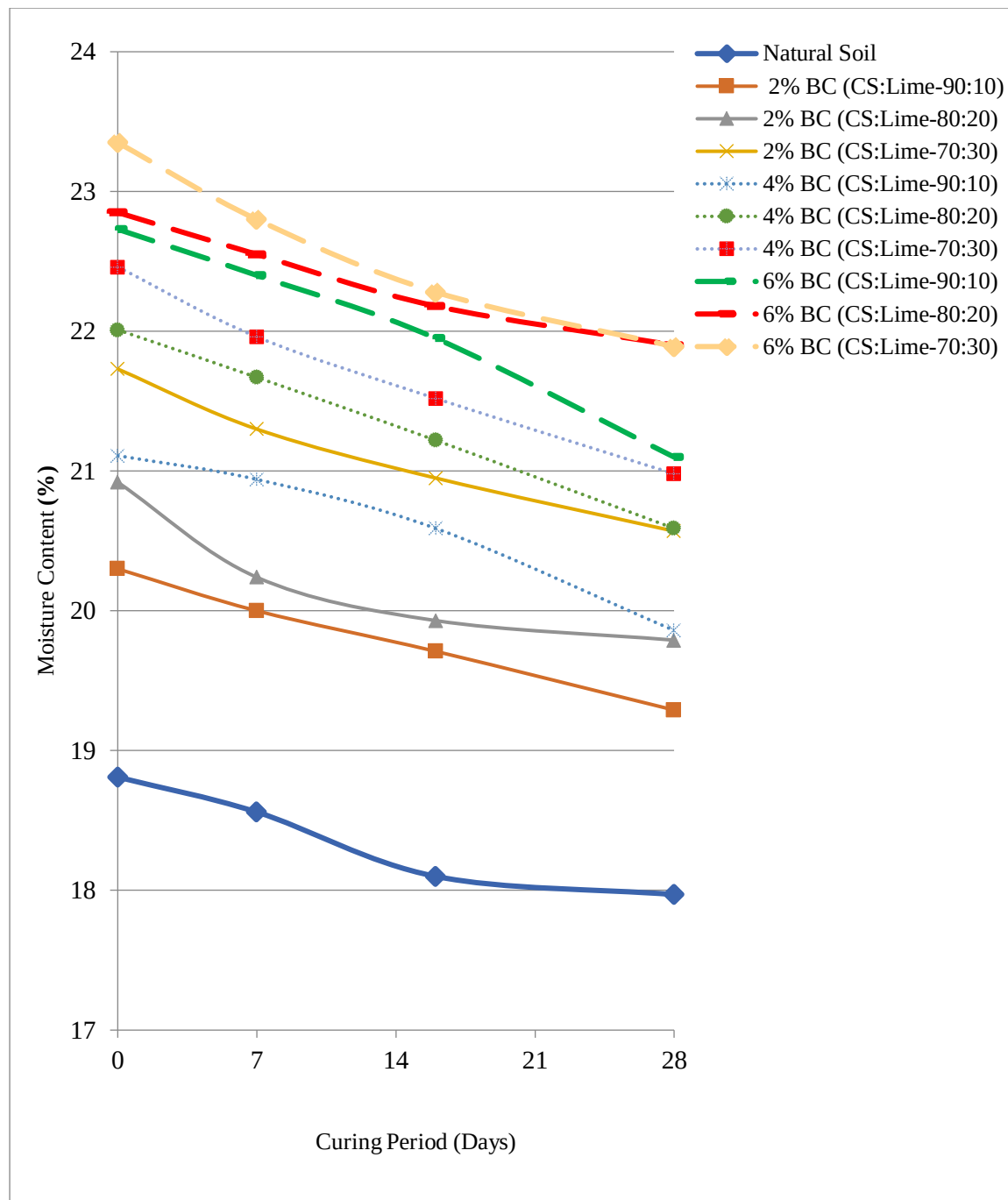


Figure 4.25: Effect of curing period on initial moisture content for the composite soil under CC1

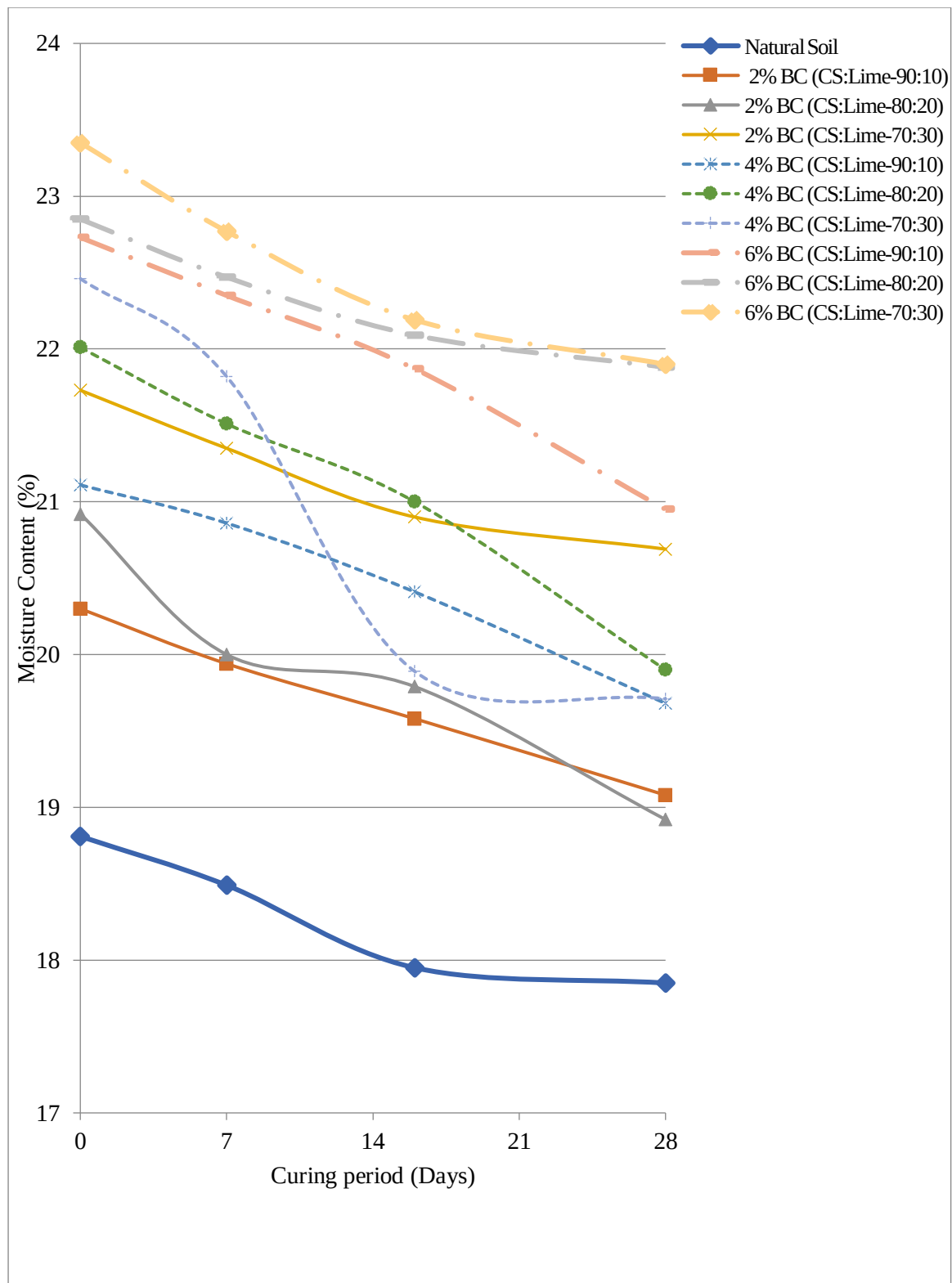


Figure 4.26: Effect of curing period on the initial moisture content for the composite soil under CC2

4.6.3 Effect of CS and Hydrated Lime on the Modulus of Elasticity of the Composite Soil

Figures 4.27a, 4.28a and Table 4.4 present the results of the effect of CS and hydrated lime total binder content on the elastic behaviour of the composite soil. The elastic modulus is described as the E_{40} of the composite. The E_{40} is the ratio of 40 % of the maximum unconfined compressive strength to the corresponding strain. The E_{40} was used to study the effect of CS and hydrated lime on the elastic behaviour of the composite soil. In general, the modulus of elasticity (E_{40}) increases with an increase in the total binder content and curing period. For example, there was a slightly significant change in the E_{40} of the composite soil at 10 % binder content after 28 days curing under CC2 with an increase in the total binder content of 6 %. The E_{40} also increases with the increase in CS/lime ratio from 10 % to 30 % (9:1 to 7:3) and curing times from 7 days to 74 days and with increase in curing temperature under CC1 from 35 °C to 45 °C under CC2. Therefore, the increase in the E_{40} with an increase in the binder content, curing temperature, curing time and CS/lime ratio may be due to the alteration in the composition and formation of cementitious materials that may have direct effects on the deformation properties of the composite.

In addition, there are three CS/lime ratios in this test under CC1 and CC2. The first is the 10 % or 9:1 ratio. In this ratio, the elastic modulus sharply increased from 32.11 GPa at zero percent binder to 46.14 GPa, 51.11 GPa, 59.12 GPa, 71.11 GPa, 89.13 GPa and 104.24 GPa at 2 % total binder content as the curing period increased from 7 days to 74 days respectively and thereafter, continued to increase up to 6 % total binder content under CC1. Under CC2, the elastic modulus sharply increased to 50 GPa, 52.13 GPa, 57.93 GPa, 62.12 GPa, 76.34 GPa and 85.25 GPa at 2 % total binder content as the curing period increased from 7 days to 74 days respectively and continued to increase up to 6 % total binder content. Therefore, as the CS/lime ratios increased from 10 % to 30 % or 9:1 to 7:3 and the curing period increased from 7 days, 16 days, 28 days, 42 days, 56 days to 74 days with increase in total binder content, the elastic modulus increased. The remaining Figures are presented in Appendix 4.

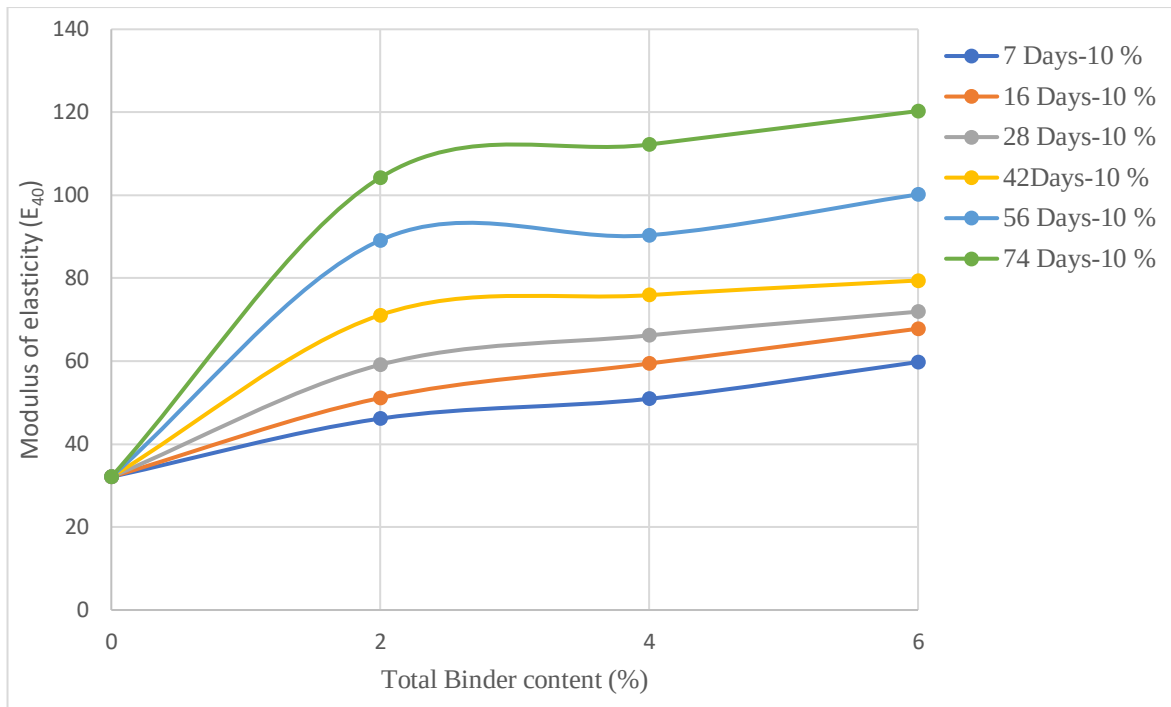


Figure 4.27a: Effect of total binder percent on the modulus of elasticity E_{40} under CC1 at CS/lime ratio of 10 % (9:1)

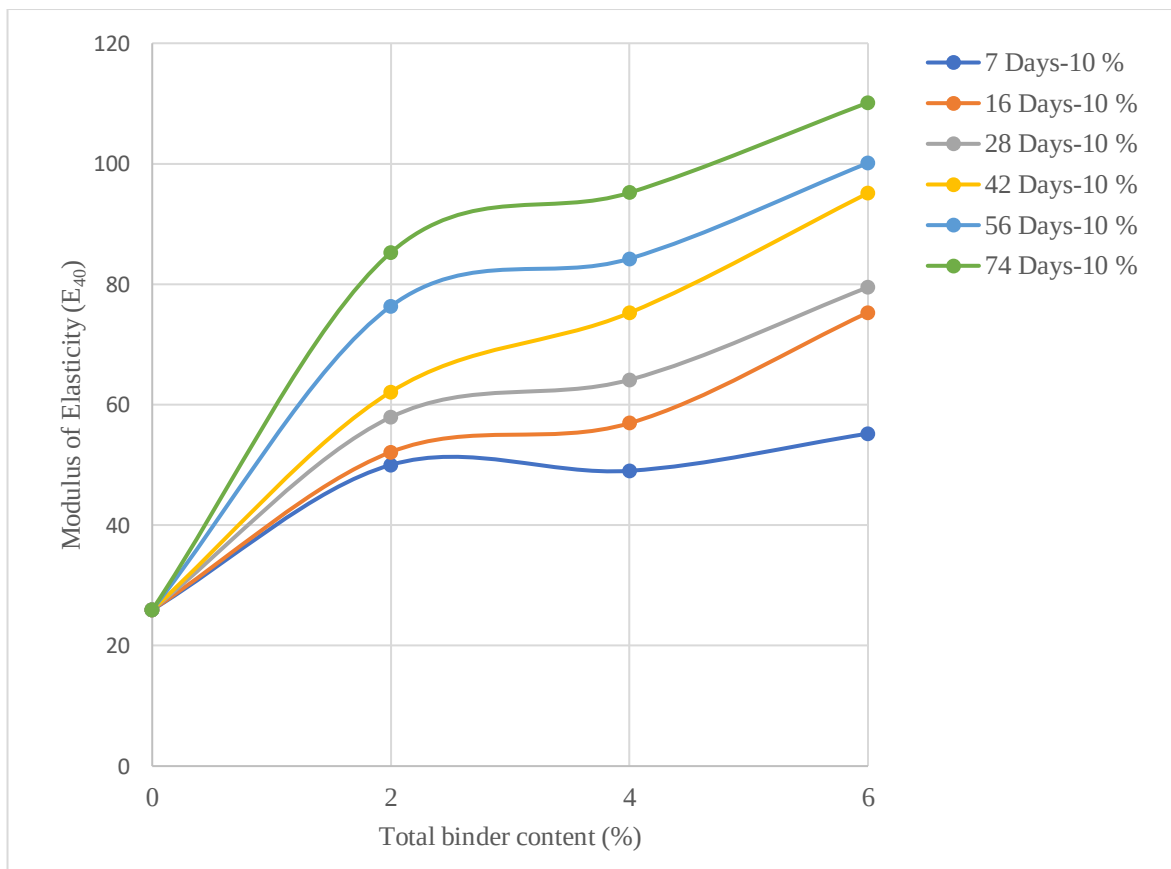


Figure 4.28a: Effect of total binder percent on the modulus of elasticity E_{40} under CC2 at CS/lime ratio of 10 % (9:1)

Table 4.4: Elastic Modulus

BC %	45°C	35°C	45°C	35°C	45°C	35°C
	7 Days (GPa)		16 Days (GPa)		28 Days (GPa)	
0	32	25	32	25	32	25
2(10 %)	46	50	51	52	59	57
4(10 %)	50	49	59	56	66	64
6(10 %)	59	55	67	75	71	79
0	32	25	32	25	32	32
2(20 %)	48	48	52	53	63	59
4(20 %)	51	47	61	58	68	68
6(20 %)	62	59	69	80	83	84
0	32	25	32	25	32	32
2(30 %)	50	54	57	56	67	61
4(30 %)	55	59	64	60	71	71
6(30 %)	65	70	74	81	85	90
BC %	42 Days (GPa)		56 Days (GPa)		74 Days (GPa)	
0	32	25	32	25	32	25
2(10 %)	71	62	89	76	104	85
4(10 %)	75	75	90	84	112	95
6(10 %)	79	95	100	100	120	110
0	32	25	32	25	321	25
2(20 %)	76	66	95	79	111	89
4(20 %)	80	78	99	86	118	98
6(20 %)	87	98	108	101	132	112
0	32	25	32	25	32	25
2(30 %)	80	70	100	81	125	904
4(30 %)	84	80	105	90	123	104
6(30 %)	90	100	110	104	144	124

1. 2(10 %) = E_{40} at 2 % total binder content with 10 % replacement of CS by hydrated lime
2. 4(10 %) = E_{40} at 4 % total binder content with 10 % replacement of CS by hydrated lime
3. 6(10 %) = E_{40} at 6 % total binder content with 10 % replacement of CS by hydrated lime
4. 2(20 %) = E_{40} at 2 % total binder content with 20 % replacement of CS by hydrated lime
5. 4(20 %) = E_{40} at 4 % total binder content with 20 % replacement of CS by hydrated lime
6. 6(20 %) = E_{40} at 6 % total binder content with 20 % replacement of CS by hydrated lime
7. 2(30 %) = E_{40} at 2 % total binder content with 30 % replacement of CS by hydrated lime
8. 4(30 %) = E_{40} at 24% total binder content with 30 % replacement of CS by hydrated lime
9. 6(30 %) = E_{40} at 6 % total binder content with 30 % replacement of CS by hydrated lime

4.7 Swelling Characteristics of Copper Slag-Soil and Copper Slag-Lime-Soil Mixtures

The free swelling percentage was used as an index for the classification of the expansive soil for engineering reasons and to measure the effect of CS without and with hydrated lime on the swelling properties in two phases. The first phase was to study the effect of CS alone on the swelling characteristics of the natural soil cured under different conditions. The second phase was to study the effect of CS activated by hydrated lime on the swelling characteristics of the natural soil.

4.7.1 Effect of Copper Slag Only on the Free Swelling Characteristics of the Soil

The results of the CS effect on the swelling characteristics of the natural soil under CC1 and CC2 are presented in Table 4.5. Generally, from Table 4.5, the percentage of free swelling reduces with an increase in CS content and curing period with increased temperature. For example, under CC1 free swelling decreased from 33 % to 16 % at 10 % CS content while under CC2, the free swelling also decreased from 33 % to 13 % at 10 % CS content. Under CC1 and CC2, the maximum decrease in free swelling percentages are observed in CC2 while under CC1 the decrease in free swelling is less than CC2. As the curing period and binder content increase the free swelling potential decreases with increase in temperature.

Table 4.5: Effect of copper slag on the free swelling percentage of the natural soil

Curing Conditions (CC)	CS (%)	Swelling Percentage (%)					
		Curing Period					
		7 Days	16 Days	28 Days	42 Days	56 Days	74 Days
CC1 35 °	0	33	33	33	33	33	33
	2	32	32	31	31	31	31
	4	30	30	30	29	29	29
	6	21	20	19	17	16	15
	8	20	16	14	13	12	10
	10	16	15	13	12	10	10
CC2 45 ° C	0	33	33	33	33	33	33
	2	28	28	28	28	27	27
	4	24	21	20	18	16	15
	6	19	17	17	14	13	10
	8	16	13	12	12	8	6
	10	13	11	10	10	5	4

4.7.2 Effect of Curing Period on the Swelling Characteristics of the Natural Soil Treated with Copper Slag

Figures 4.29 and 4.30 present the results of the effect of curing periods on the free swelling characteristics of the soil under CC1 and CC2. Generally, as the curing period and CS content increase the swelling potential decreases under CC1 and CC2. Figure 4.30 presents two categories of swelling potential decrease under CC1. The swelling potential of 2 % CS and 4 % CS content slightly decreased with increase in curing period while the swelling potential of 6 % CS, 8 % CS and 10 % CS content significantly decreased more than the swelling potential of 2 % CS and 4 % CS content as the curing period increased from 7 days to 74 days. Figure 4.31 also presents two categories of swelling potential decrease under CC2. The swelling potential of 2 % CS content slightly decreased as the curing period increased from 7 days to 74 days while the swelling potential of 4 %, 6 %, 8 % and 10 % CS content significantly decreased as the curing period increased from 7 days to 74 days.

The maximum decrease in swelling potential was observed in Figure 4.31 under CC2 while in Figure 4.29 under CC1 the decrease in swelling potential was less as compared to CC2.

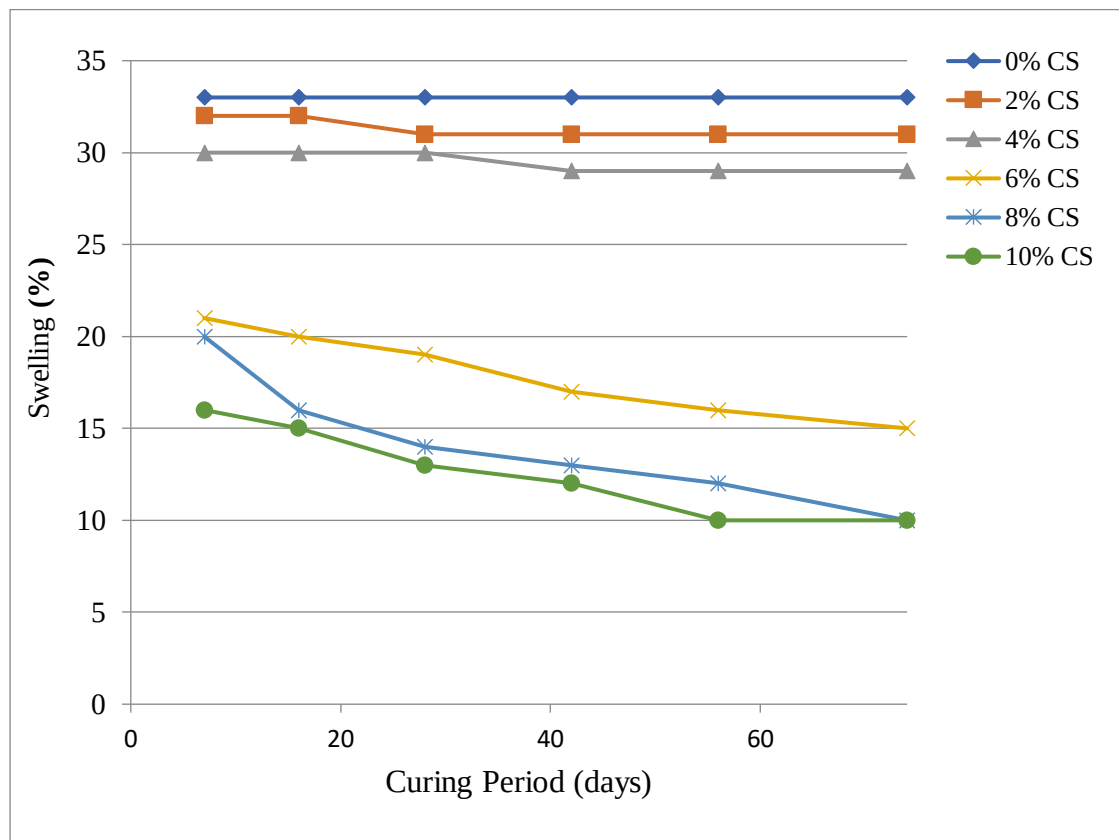


Figure 4.29: Effect of CS (%) on the free swelling of the soil under CC1 at different period

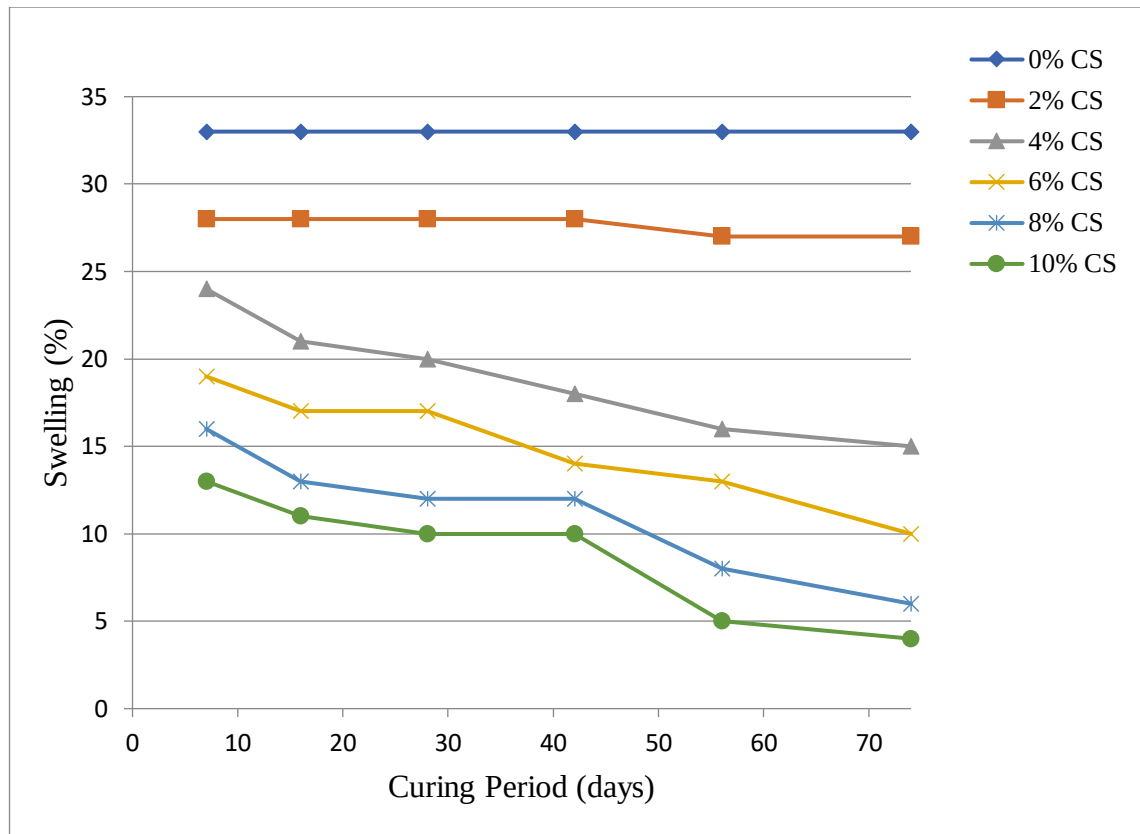


Figure 4.30: Effect of CS (%) on the free swelling of the soil under CC2 at different period

4.7.3 Effect of Copper Slag Activated by Hydrated Lime on the Swelling Characteristics of the Natural Soil

Figures 4.31 to 4.36 and Tables 4.6 A and B present the results of the effect of total binder content with different CS/lime ratio on the free swelling potential of the specimens cured under CC1 and CC2. Six curing periods namely 7 days, 16 days, 28 days, 42 days, 56 days and 74 days were considered to examine the effect of CS activated by hydrated lime on the free swelling behaviour of the composite soil. Generally, it is observed from Figures 4.32 to 4.37 that the free swelling potential decreases with an increase in total binder content and CS/lime ratio, with increase in the curing temperature and curing period.

For example, an increase in the binder content up to 4 % at 24 % CS/lime ratios decreased the percentage of free swelling to 25 % after 7 days of curing under CC1 while under CC2 the percentage of free swelling decreased to 20 % after 7 days. At 4 % binder content with 24 % CS/lime ratio the free swelling percentage decreased from 33 % to 16 % after 56 days of curing under CC1 and under CC2 to 12 %. The free swelling percentage decreased from 33 % to 14 % under CC1 and under CC2 to 12 % at 6 % binder content after 28 days of curing. At 6 % binder content at 24 % CS/lime ratio the free swelling percentage decreased

from 33 % to 8 % after 74 days under CC1 and under CC2 to 6 %. Further increase in CS/lime ratio to 44 % at 6 % binder content decreased the free swelling percent from 33 % to 13 % under CC1 and under CC2 to 12 % after 16 days of curing. At 6 % binder content with 44 % CS: lime ratio the free swelling percent decreased from 33 % to 6 % under CC1 and under CC2 to 5 % after 74 days of curing period.

For a total binder content of 6 % the percentage of free swelling decreased from 33 % to 17 % under CC1 and under CC2 to 15 % after 16 days curing period at 12 % CS/lime ratios. Increasing the CS/lime ratio to 44 % the percentage of free swelling decreased to 9 % at 6 % binder content after 56 days curing period under CC1 and under CC2 to 8 %.

Table 4.6 A: Effect of CS and hydrated lime on the swelling characteristic of the natural soil and composite soil

Total Binder content		Free Swelling (%) CC1					
Lime	CS	7 Days	16 Days	28 Days	42 Days	56 Days	74 Days
0%	0	33	33	33	33	33	33
	2	32	32	31	31	31	31
	4	30	30	30	29	29	29
	6	21	20	19	17	16	15
12%	0	33	33	33	33	33	33
	2	30	28	22	19	15	12
	4	24	20	17	14	13	11
	6	19	17	15	13	11	10
24%	0	33	33	33	33	33	33
	2	31	30	29	28	27	26
	4	25	20	19	17	16	13
	6	18	16	14	12	10	8
44%	0	33	33	33	33	33	33
	2	30	30	29	28	26	25
	4	25	20	19	17	15	12
	6	15	13	12	10	9	6

Table 4.6 B: Effect of CS and hydrated lime on the swelling characteristics of the natural soil and composite

Total Binder content		Free Swelling (%) CC2					
Lime	CS	7 Days	16 Days	28 Days	42 Days	56 Days	74 Days
0%	0	33	33	33	33	33	33
	2	31	31	30	30	30	30
	4	28	28	28	27	27	26
	6	20	20	19	16	14	13
12%	0	33	33	33	33	33	33
	2	27	26	20	18	14	11
	4	22	19	16	12	11	10
	6	17	15	13	11	10	8
24%	0	33	33	33	33	33	33
	2	27	27	26	26	25	24
	4	20	18	16	14	12	7
	6	16	15	12	10	8	6
44%	0	33	33	33	33	33	33
	2	26	26	26	25	25	25
	4	21	19	15	14	12	9
	6	14	12	11	9	8	5

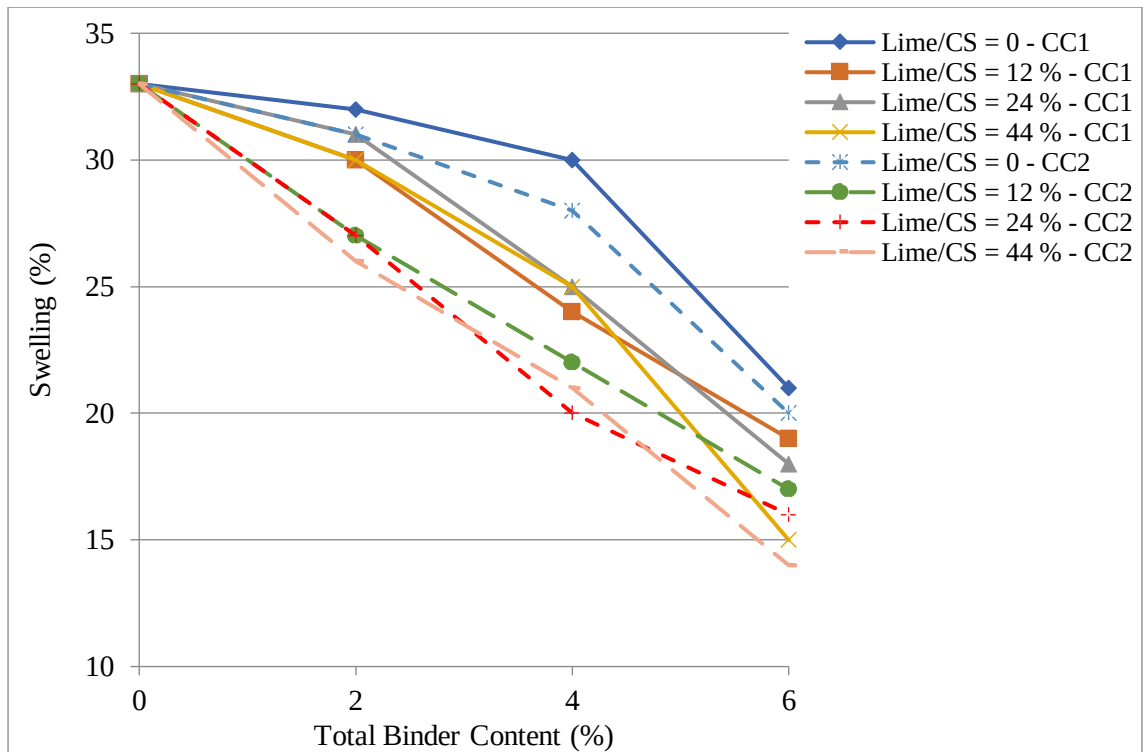


Figure 4.31: Effect of CS and hydrated lime on free swelling of the composite soil under CC1 and CC2 after 7 days.

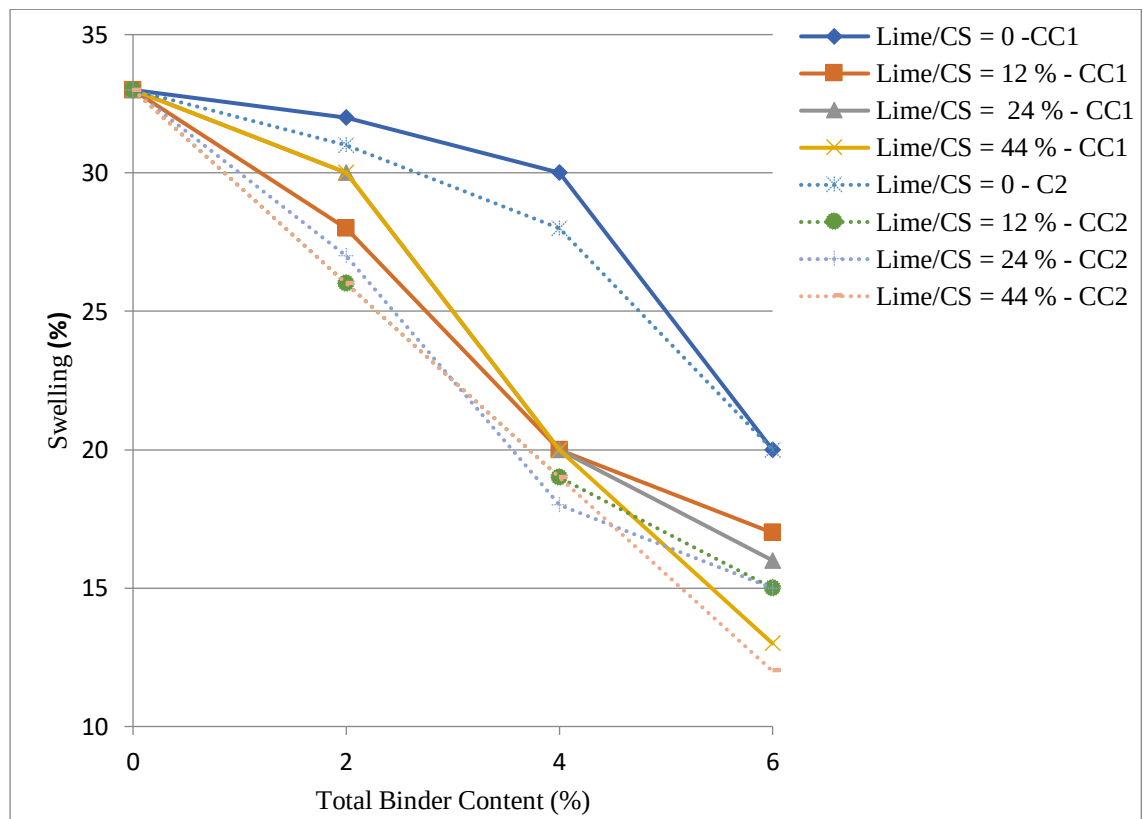


Figure 4.32: Effect of CS and hydrated lime on the free swelling of the composite soil under CC1 and CC2 after 16 days.

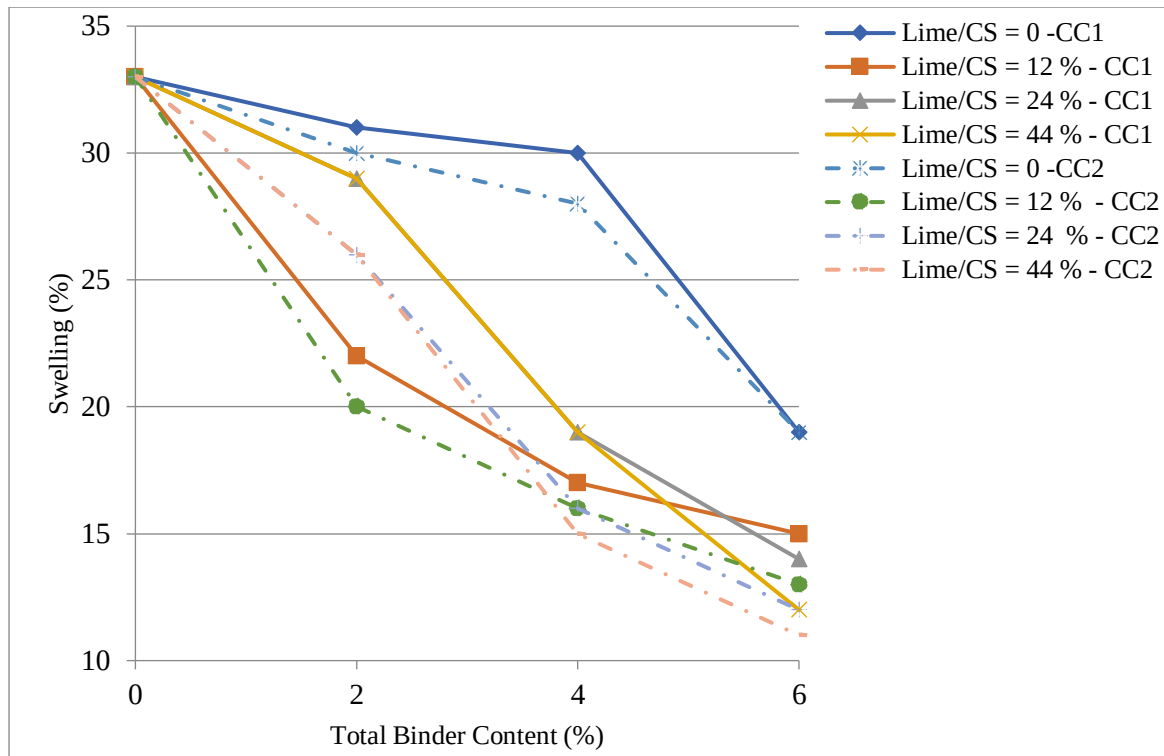


Figure 4.33: Effect of CS and hydrated lime on the free swelling of the composite soil under CC1 and CC2 after 28 days.

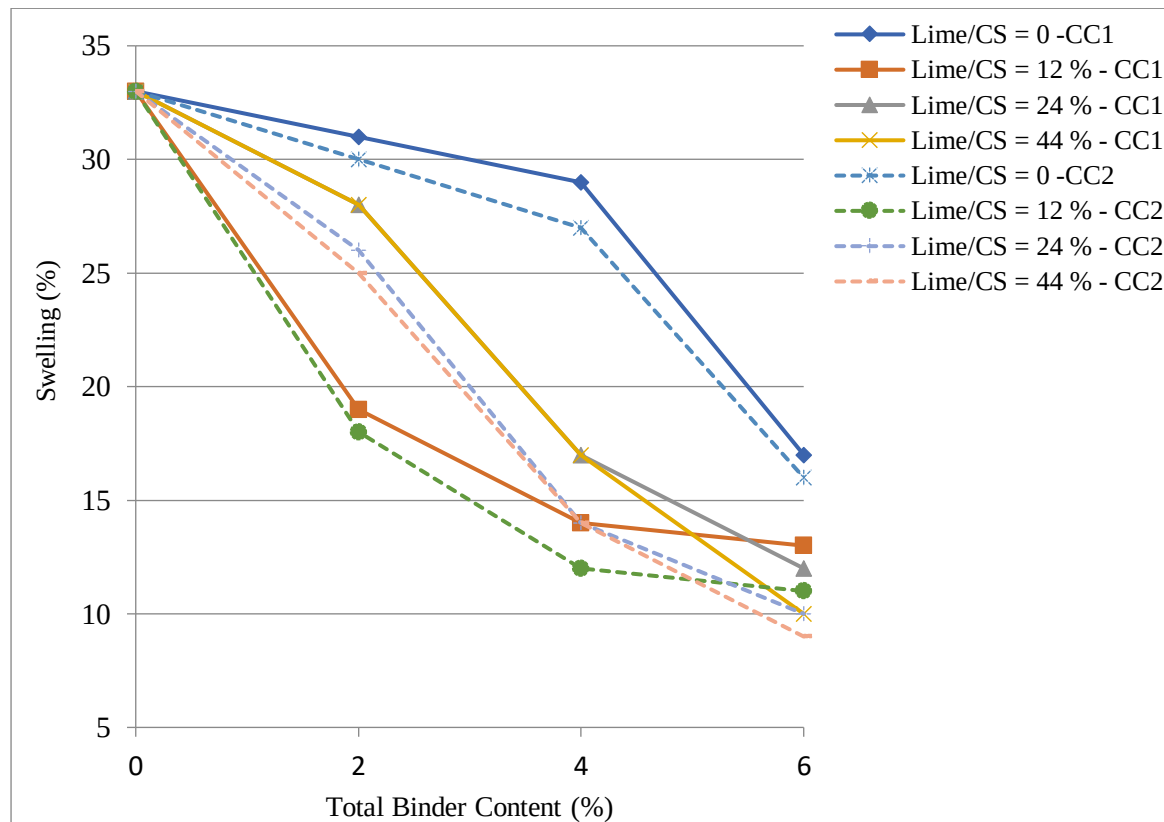


Figure 4.34: Effect of CS and hydrated lime on the free swelling of the composite soil under CC1 and CC2 after 42 days

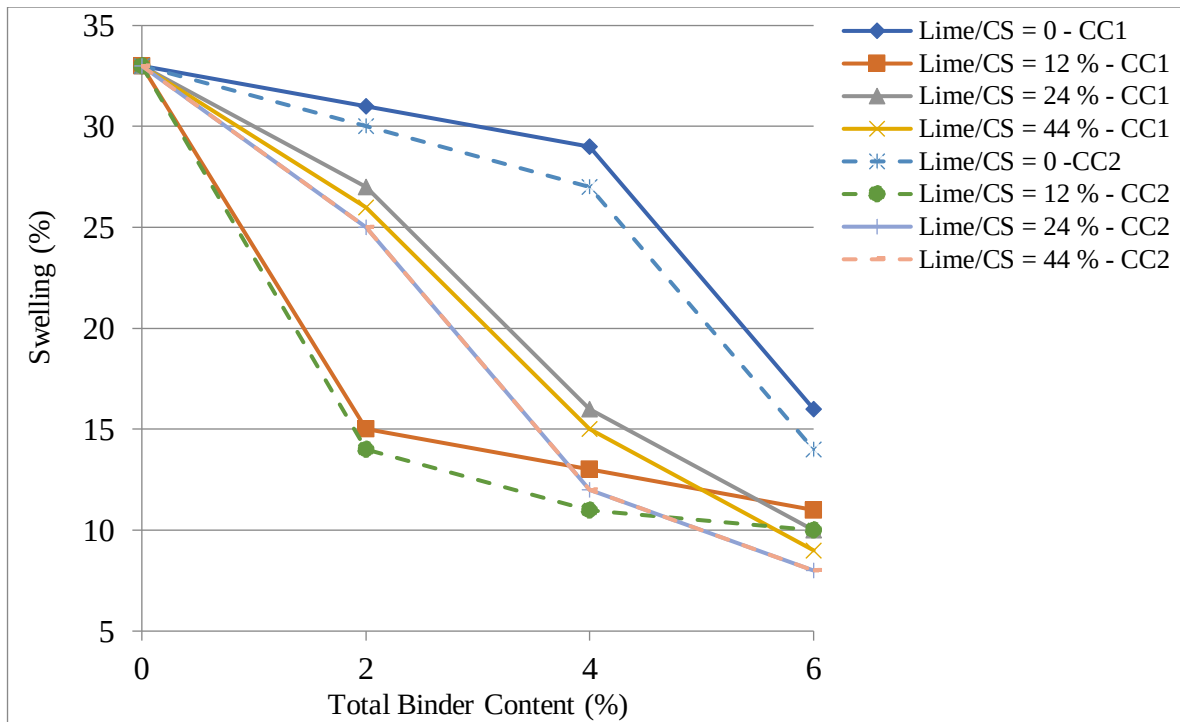


Figure 4.35: Effect of CS and hydrated lime on the free swelling of the composite soil under CC1 and CC2 after 56 days

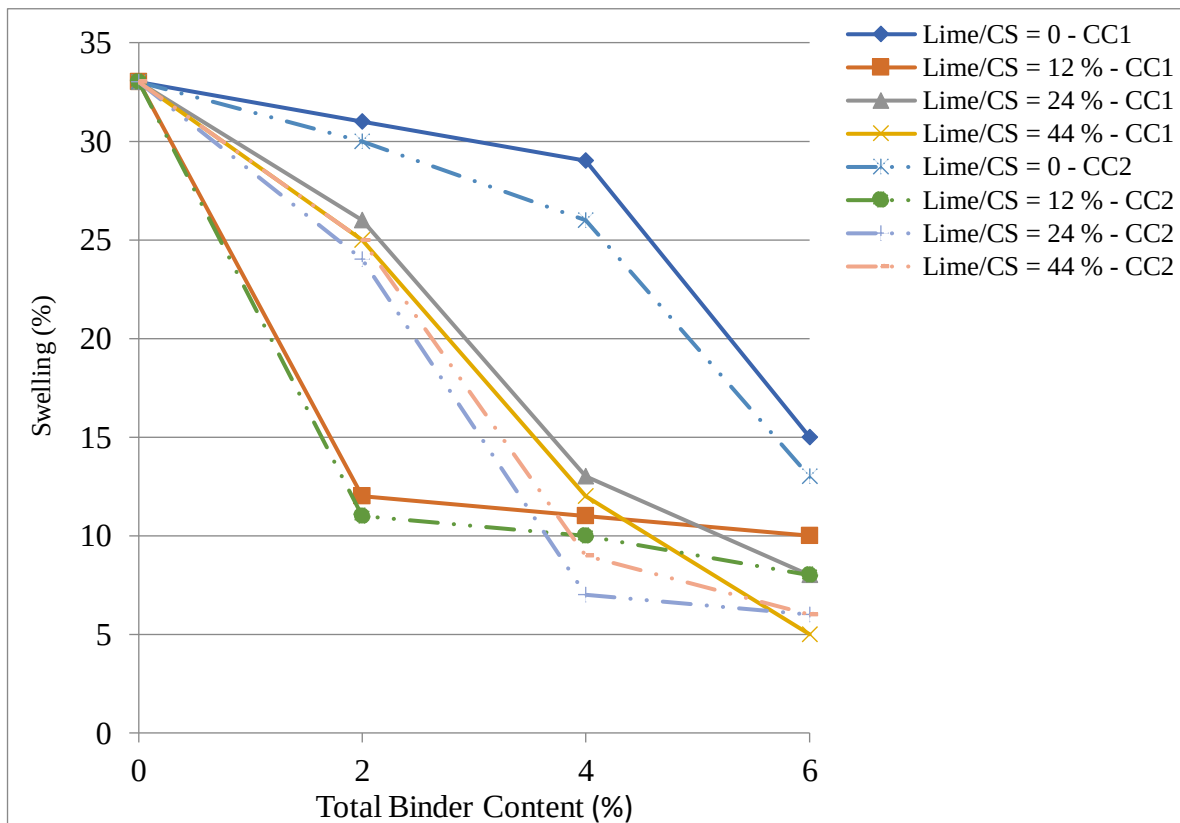


Figure 4.36: Effect of CS and hydrated lime on the free swelling of the composite soil under CC1 and CC2 after 74 days.

CHAPTER 5

ANALYTICAL TESTS RESULTS

5.1 Introduction

Five X-ray diffraction (XRD) and four scanning electron microscopy (SEM) studies were conducted on small quantities of pulverized samples for the tests. The specimens were cured for 74 days under CC1 and CC2.

5.2. X-Ray Diffraction Results

The X-ray diffraction patterns for individual raw samples are presented in Figures 5.1 to 5.4. The diffraction patterns were found between Bragg angles of 5° and 90°. The X-ray diffraction results are presented in two parts. The first part is the tracing of minerals peaks in the natural soil and tracing of strong peaks, medium peaks and weak peaks in hydrated lime, cooper slag and the hydrated sample which are presented in Appendix 9. The second part of the results present the reaction products formed in the composite samples which are also classified into groups. In the X-ray diffraction patterns of the natural soil, each mineral present in the natural soil has its own individual diffraction pattern within main X-ray diffraction of the natural soil (Figure 5.1) which can only be seen by the mean of XRD analyzer software, and the peaks of these minerals were extracted from their individual patterns as presented in Appendix 9. The strong peaks of the samples show the longer peaks, the medium peaks show the next peaks less than the strong peaks in height and longer than the weak peaks in height. The XRD patterns, from strong to weak peaks showed the dominant Angstroms (Å) and the diffraction angle of the samples or minerals. The trace of clay minerals begins with Kaolinite which showed 8 main peaks two of which are strong, five mediums and one weak peak. The trace of montmorillonite showed 10 main peaks with five strong peaks, four medium peaks and one weak peak. The trace of illite showed 13 main peaks with five strong peaks and eight medium peaks. Traces of non-clay minerals also occur. The trace of the quartz showed 6 main peaks with one strong peak one medium peak and four weak peaks. The trace of calcite showed 5 main peaks with one strong peak, two medium peaks and two weak peaks (Figure 5.1). The trace of hydrated lime showed 6 main peaks with two strong peaks, two medium peaks and two weak peaks (Figure 5.3). Copper slag (CS) trace showed no peak values due to its magnetic amorphous composition (Figure

5.2). The X-ray diffraction trace of the hydrated sample showed new peaks with seven strong peaks; nine medium peaks and two weak peaks were observed (Figure 5.4).

After 74 days of curing, the illite strong peaks of 10.02 Å (8.816°), 5.01 Å (17.685°), 4.51 Å (19.672°), 4.50 Å (19.700°), 4.47 Å (19.819°) and the medium of 4.39 Å (20.169°), 4.31 Å (20.566°), 4.24 Å (20.908°), 4.11 Å (21.593°), 3.98 Å (22.308°), 3.89 Å (22.834°), 3.73 Å (23.787°) and 3.59 Å (24.417°) totally disappeared. The intensities of the montmorillonite strong peaks at 15.00 Å (5.887°), 7.50 Å (11.790°), 5.00 Å (17.725°), 4.49 Å (19.757°) and 4.48 Å (19.770°) considerably disappeared in the hydrated sample or composite. The medium peaks of 4.30 Å (20.632°), 4.29 Å (20.645°), 3.85 Å (23.068°), 3.75 Å (23.707°) and the weak peak of 3.33 Å (26.672°) totally disappeared. The calcium hydroxide strong peaks of 4.90 Å (18.059°), 3.11 Å (28.670°) and the medium peaks of 2.62 Å (34.093°) and 2.45 Å (36.587°) including the weak peaks of 1.92 Å (47.129°) and 1.79 Å (50.789°) totally disappeared, indicating that the hydrated lime content was consumed at this stage of the reaction.

The reaction products formed in the composite sample were classified into eight groups. Group A shows the formation of hydrotalcite phase and has the strong peak at 3.34 Å (26.603°). Group B shows the formation of CaCO₃ with the strong peak at 3.85 Å (23.048°). Group C shows the formation of semi-crystallized calcium silicate hydrate (C-S-H) gel with the strong peak at 4.25 Å (20.849°), while group (D) shows the formation of calcium aluminate hydrate C₄AH₁₃ and has strong peak at 7.16 Å (12.349°). Group E shows the formation of α₁ C₄AH₁₉ which has the strong peaks at 3.04 Å (29.261°) and 4.79 Å (18.490°). Group F showed the formation of calcium silicate hydrate II with the strong peak at 7.15 Å (12.365°). Group (G) which has the strong peaks at 10.03 Å (8.817°), 10.06 Å (8.780°) and 10.00 Å (8.832°) showed the formation of a member of the hydrogarnet solid series solution (C₃AH₆-C₃AS₃H₁₃). Group (H) which has the strong peaks at 11.14 Å (7.930°), 14.36 Å (6.147°), 14.40 Å (6.132°) and 15.01 Å (5.888°) shows the formation of calcium aluminate calcium carbonate hydrate C₃AIC $\hat{C}$ H₁₂.

XRD results showed the presence of the crystalline calcium aluminate hydrates (C₄AH₁₁ and α₁ C₄AH₁₇ together with semi-crystalline C-S-H gel. Hydrotalcite type phase was also present in the hydrated sample. The XRD results also showed that the lime and CS were consumed while the minerals present in the natural soil were transformed to other phases during the reaction.

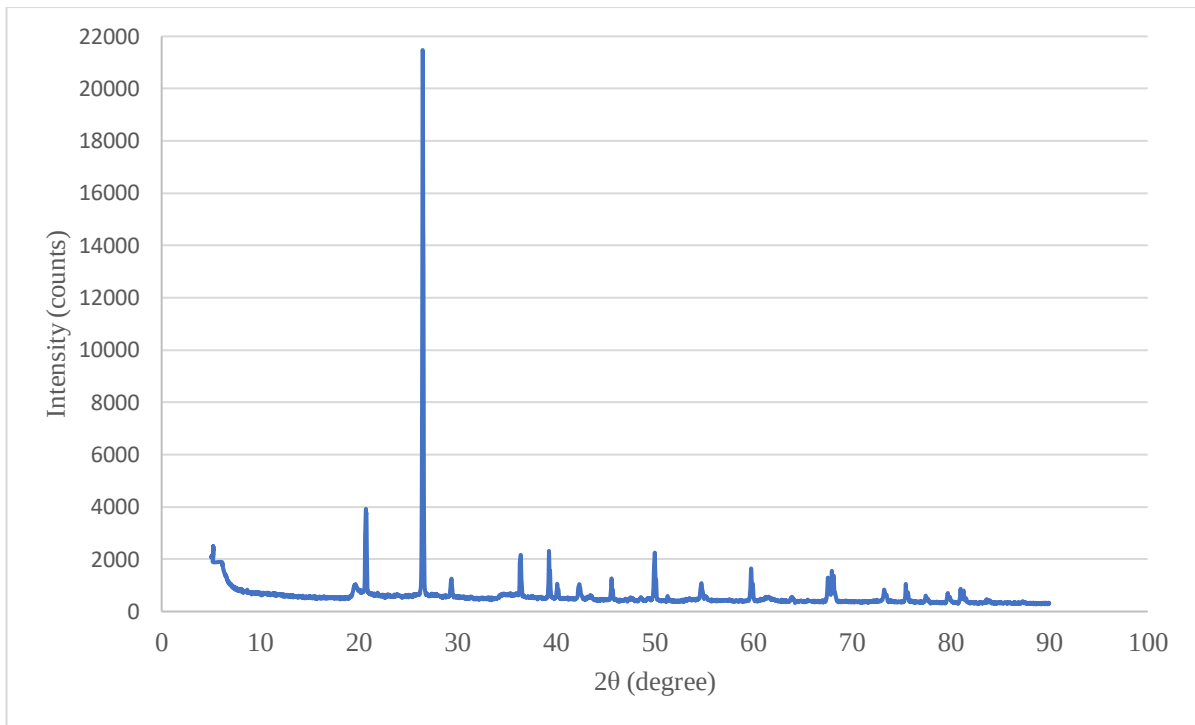


Figure 5.1: X-ray diffraction pattern for the soil sample

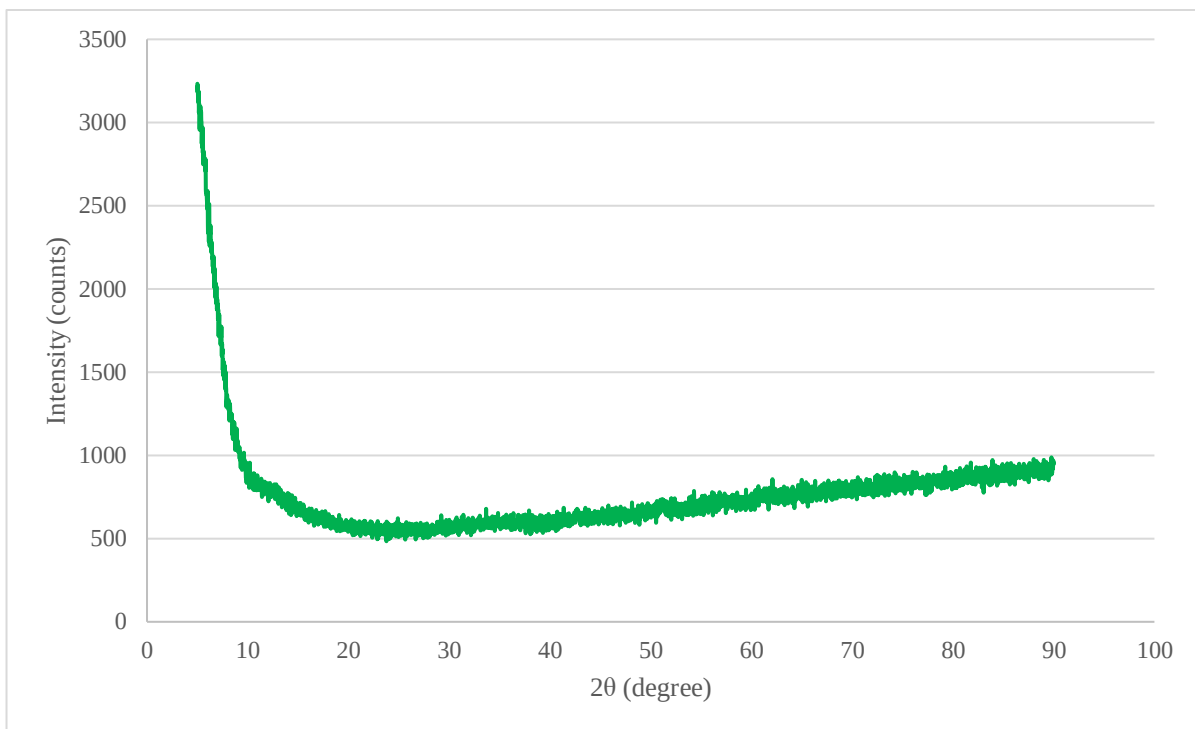


Figure 5.2: X- ray diffraction pattern for copper slag (CS) sample

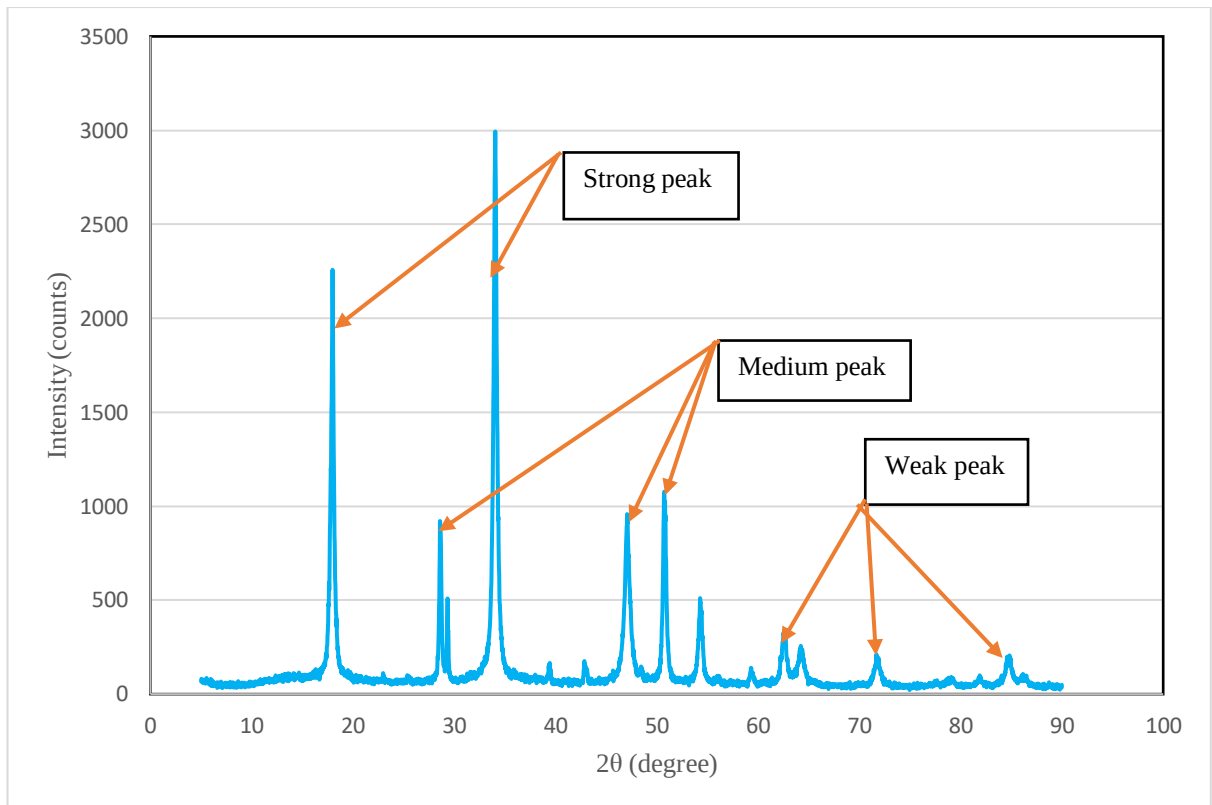


Figure 5.3: X – ray diffraction pattern for hydrated lime sample

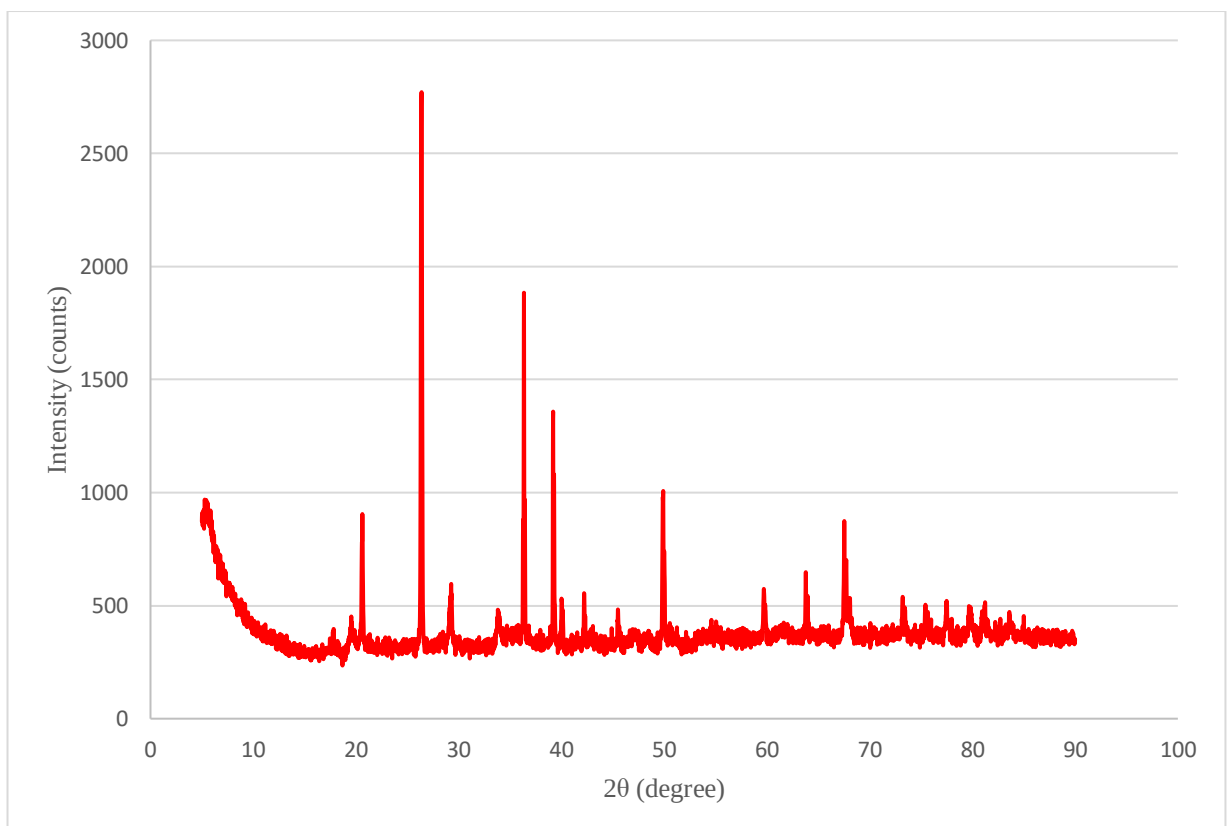


Figure 5.4: X – ray diffraction pattern for hydrated composite sample

5.3. Scanning Electron Microscope (SEM) Results

The scanning electron microscope was used to analyze four samples namely natural soil, copper slag, hydrated lime and the composite specimen. Two composite samples were cured for 74 days under CC2. Figure 5.5 (A, B and C) present the plain SEM micrograph of the natural soil, hydrated lime and copper slag samples surfaces at different magnifications respectively. In order to achieve clear resolution of the image of the natural soil, hydrated lime and copper slag samples, variety of magnification was used. The loop was used to determine the magnification at which a clear resolution of the image of the samples may appear on the screen of the analyzing computer. Also, to get a clear resolution of the image of the samples, it depends on the handling of the equipment, samples and other factors which may be responsible for the variation in magnification. The natural soil, hydrated lime and copper slag samples have a mutual difference in their dimensions, shapes, origins, mineralogical compositions and chemical and physical properties. The SEM micrograph in Figure 5.5 (A, B and C) shows that the microstructure of the natural soil and hydrate lime clear resolution could only be achieved in higher magnification of x1500 and x5000 in order to conduct individual point analysis. The copper slag clear resolution was achieved at lower magnification of x70 in order to conduct point analysis. This confirms that the particles of the natural soil, hydrated lime and copper slag have a porous and cracking structure which is responsible for many of their properties. The natural soil sample is mainly composed of kaolinite, montmorillonite and illite with little quantity of mica and chlorite using the X-ray diffraction data.

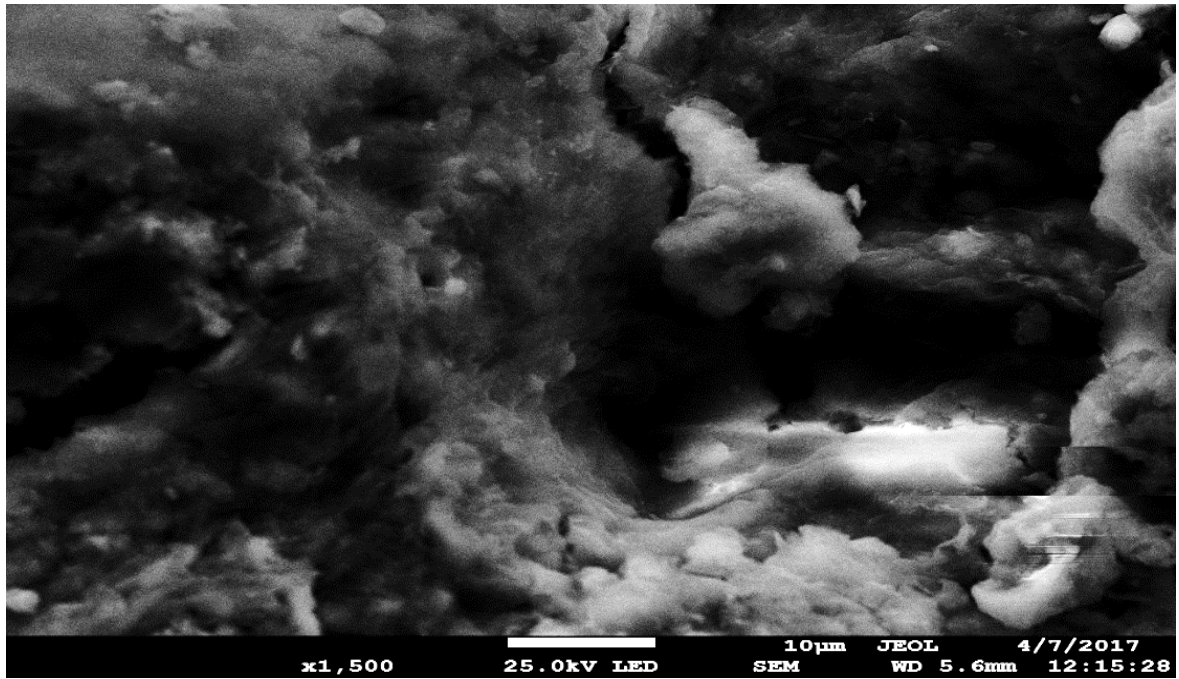


Figure 5.5 A: Plain SEM micrograph of the natural soil surface

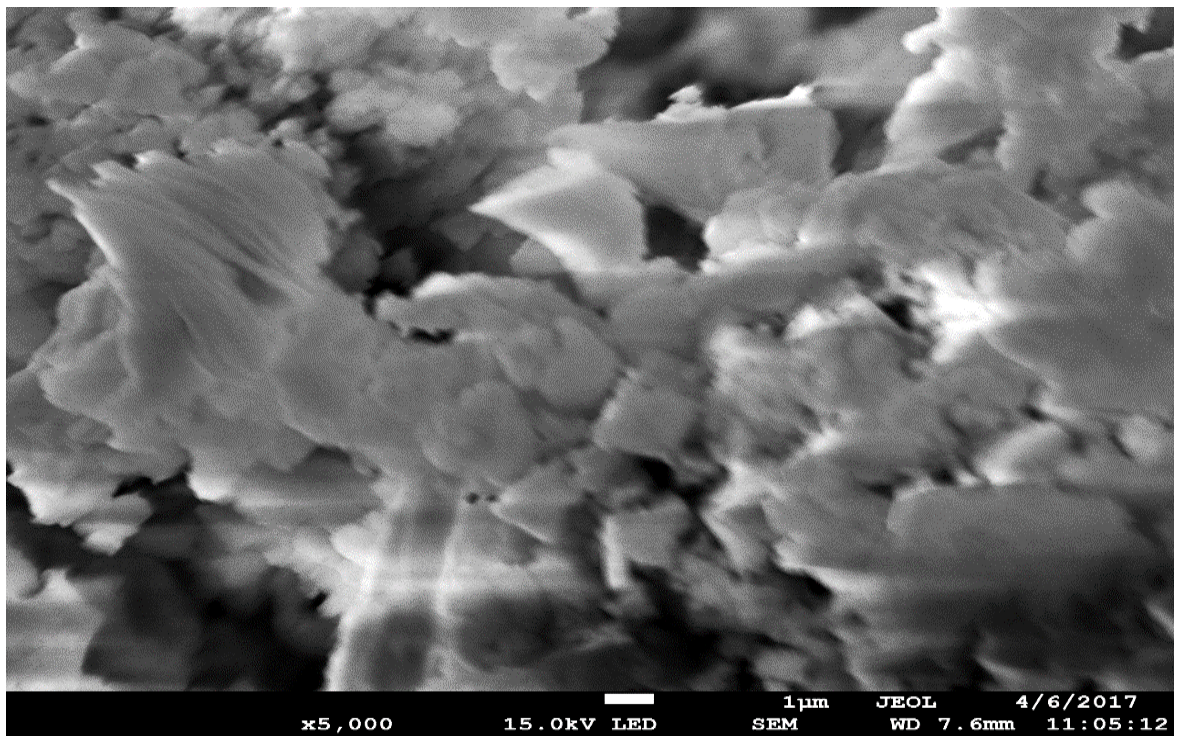


Figure 5.5 B: Plain SEM micrograph of hydrated lime surface

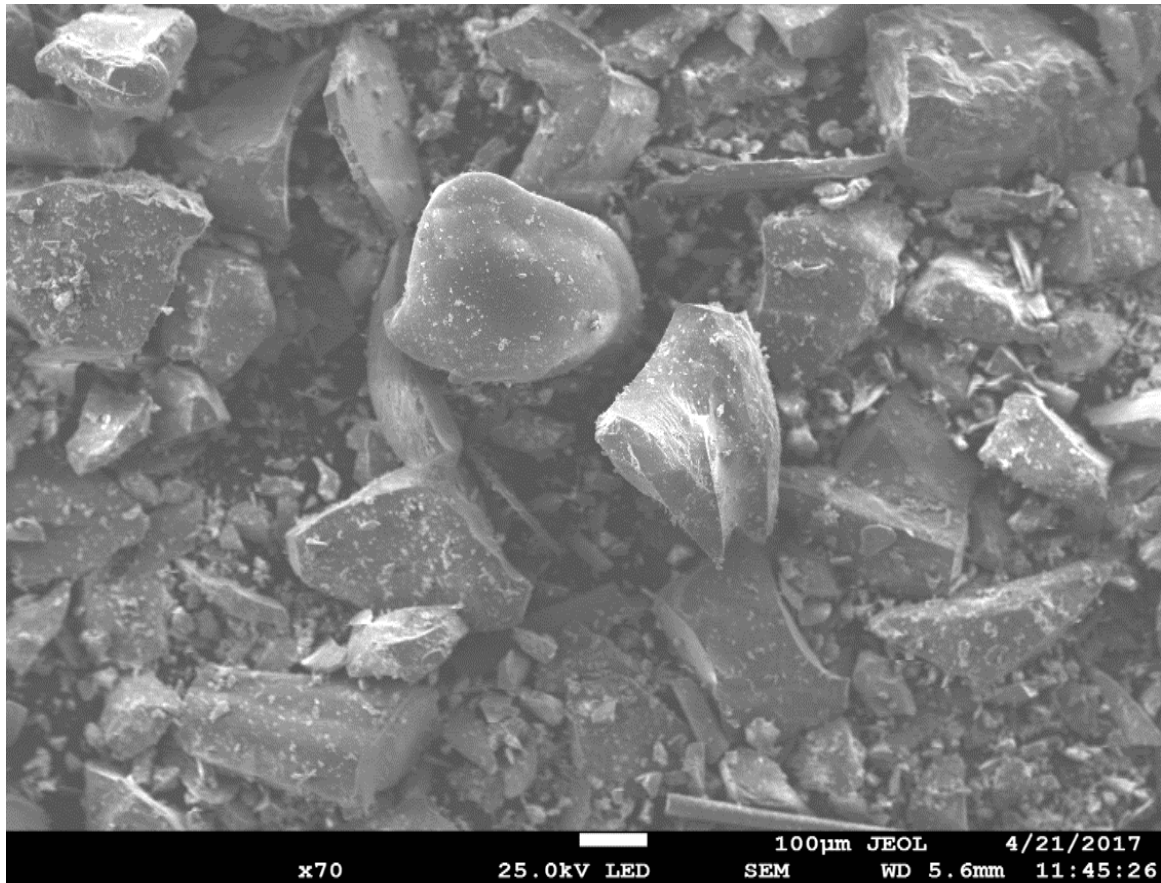


Figure 5.5 C: Plain SEM micrograph of copper slag surface

5.3.1 Control Specimen

Figure 5.6 A, B and C present the general appearance of SEM micrograph of the stabilized dry composite at different magnifications. In order to achieve clear resolution of the image of the stabilized dry composite, higher magnifications of x22,000 and x23,000 were used for two primary points in Figure 5.6 A and B while one primary point image was achieved at slightly low magnification of x4,500 (Figure 5.6 C). The stabilized dry composite has slightly mutual difference in its dimension, shape, origin, mineralogical composition and chemical and physical properties. The SEM micrograph in Figure 5.6 (A, B and C) confirms that the particles of the stabilized dry composite have an improvement in its structure due to the new cementitious gel formation which is responsible for the change in properties.

Figure 5.6 A, B and C, presents three reaction products formed in the stabilized dry composite namely: the calcium silicate hydrate (C-S-H), calcium aluminate hydrate (C-A-H) and the calcium aluminate silicate hydrate (C-A-S-H). The atomic percentages for individual points are provided in Appendix 5 while the atomic ratios for individual points

are presented in Appendix 7. Discussion of these reaction products are presented in Chapter 6.

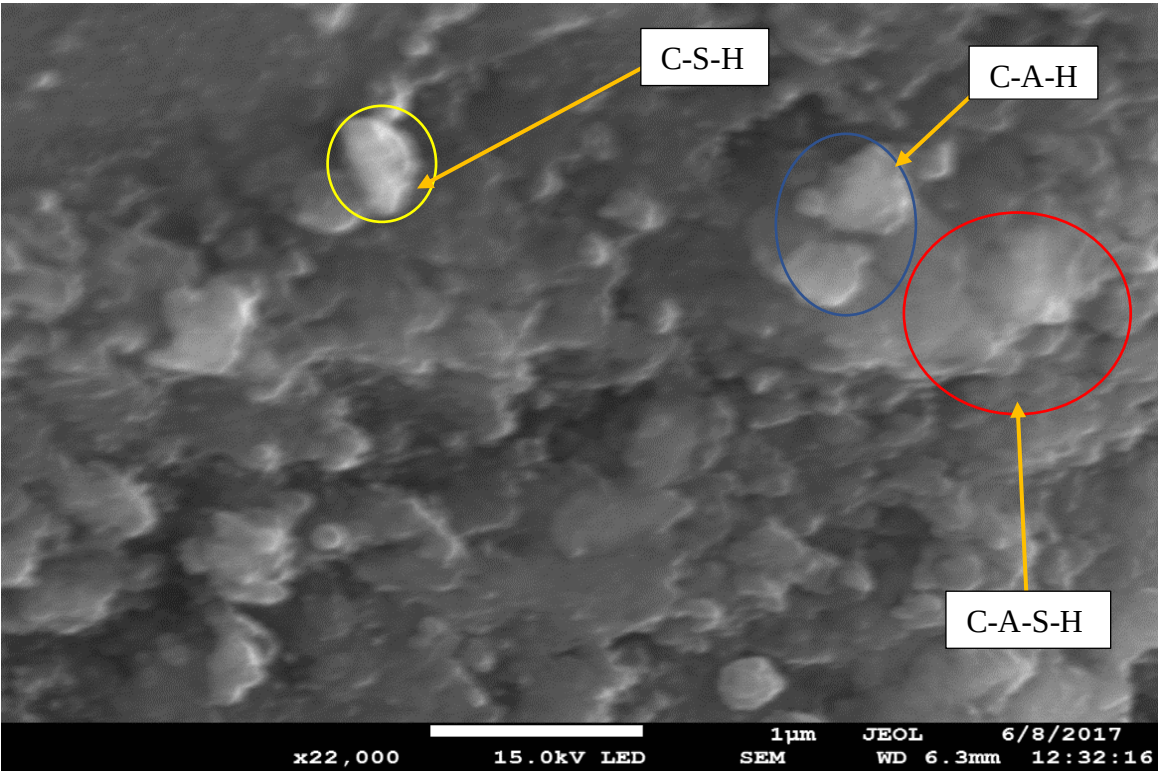


Figure 5.6A: SEM micrograph of the stabilized dry composite

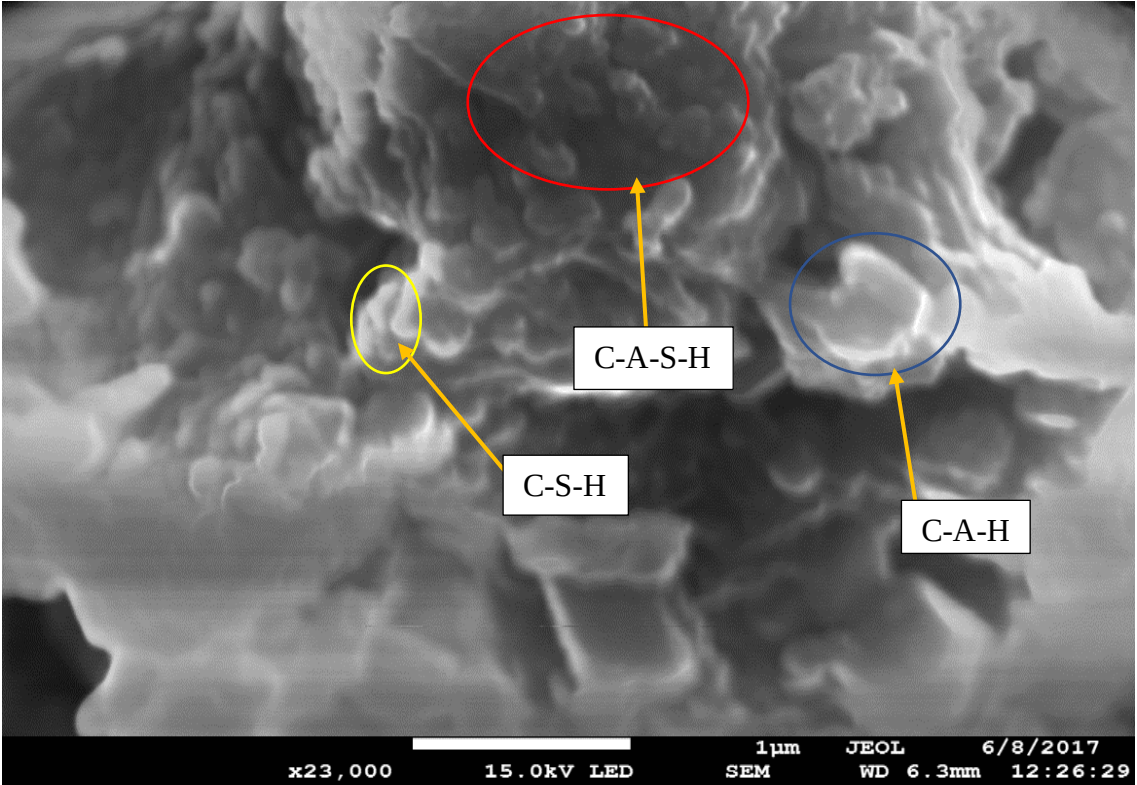


Figure 5.6 B: SEM micrograph of the stabilized dry composite

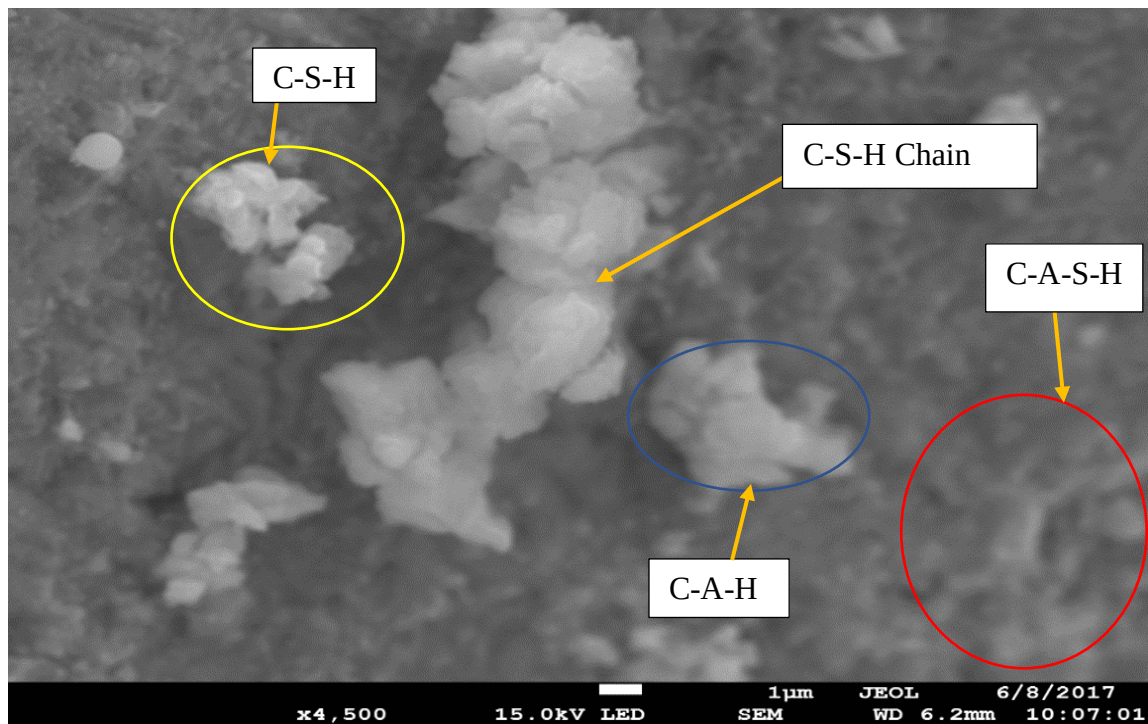


Figure 5.6 C: SEM micrograph of the stabilized dry composite

Figure 5.7 presents the relationship between atomic ratio of Mg/Si against (Mg +Al + Fe + Ti +Mn)/Si (Octahedral/Tetrahedral) atomic ratio for the energy dispersive X-ray analysis (EDX) in the control specimen (stabilized dry composite) presented in Appendix 7. Figure 5.7 also shows how the reactant elements atomic ratios of Mg/Si against (Mg +Al + Fe + Ti +Mn)/Si atomic ratio are distributed within the stabilized dry composite. Figure 5.8 presents the relationship between Si/Ca atomic ratio against Si/Al atomic ratio of the stabilized dry composite whose atomic ratios are presented in Appendix 7. In Figure 5.8, it also shows how the reactant elements atomic ratios of Si/Ca against Si/Al are distributed within the stabilized dry composite. The atomic ratios for individual points are presented in Appendix 7. Table 4.2 in Chapter 4 gives oxides percentage by weight of the soil, copper slag and hydrated lime while scanning electron microscope also confirms the results of the oxides. Scanning electron microscope also give atomic ratios of the elements investigated (see Appendix 7).

The principal oxides found in the control samples given by the scanning electron microscopy are those of silicon, calcium, aluminum, iron, magnesium and little titanium which agrees with the X-ray diffraction and X-ray fluorescence results.

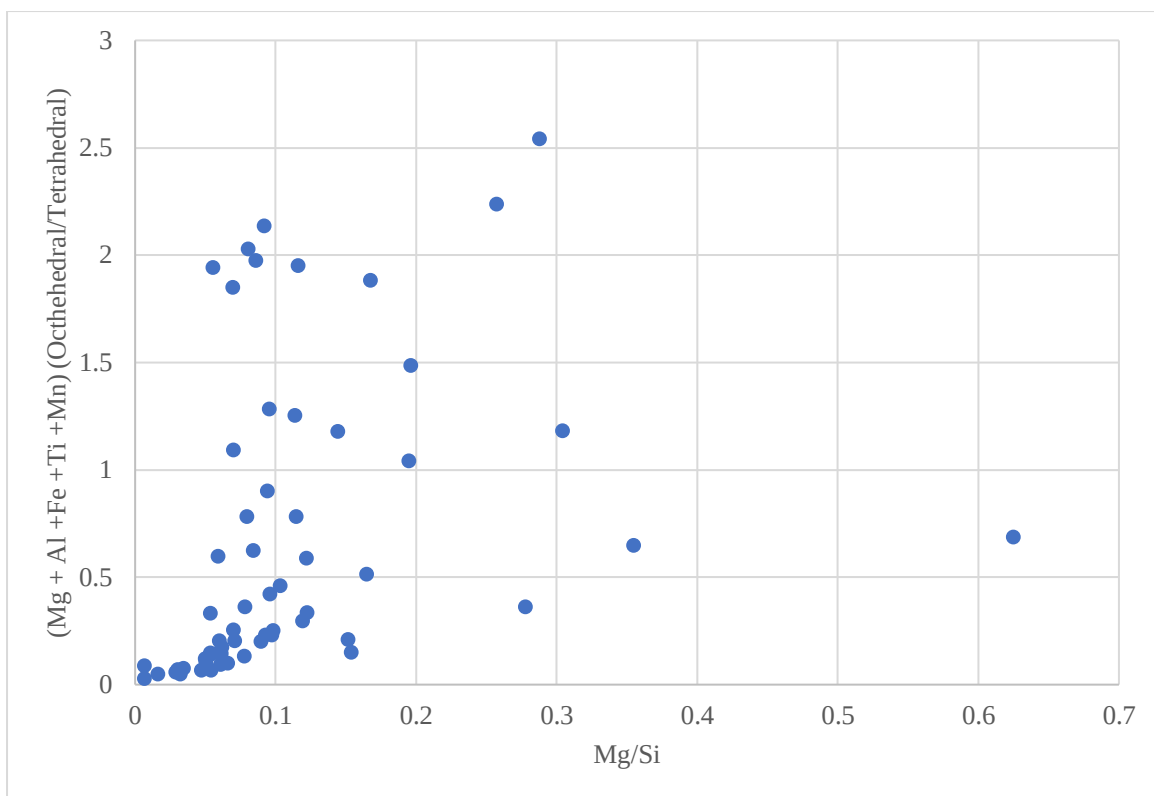


Figure 5.7: Mg/Si against (Mg + Al + Fe + Ti + Mn)/Si (Octahedral/Tetrahedral) atomic ratio for the EDX data in Figures 5.6 A, B and C

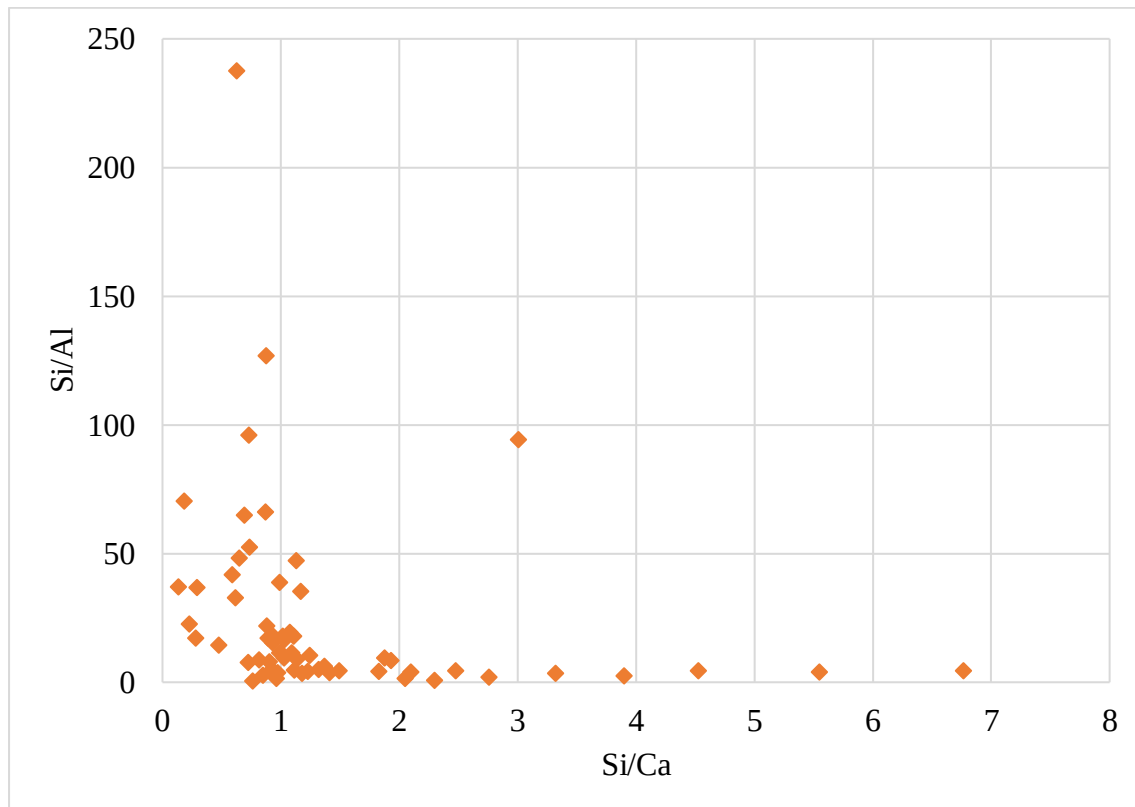


Figure 5.8: Si/Ca against Si/Al atomic ratio for the EDX data in Figures 5.6 A, B and C

Figure 5.9 (A, B, C, D, E, F and G) present the general appearance of SEM micrograph of the stabilized hydrated composite at different magnifications. In order to achieve clear resolution of the image of the stabilized hydrated composite, a variety of high magnifications were used for primary points analysis. The hydrated composite also has mutual difference in its dimension, shape, origin, mineralogical composition and chemical and physical properties. The SEM micrograph in Figure 5.9 (A, B, C, D, E, F and G) confirms that the particles of the stabilized hydrated composite have an improvement in its structure due to the new cementitious gel (C-S-H, C-A-H and C-A-S-H) formation which is responsible for the change in properties. The atomic parentages for individual points of the images are provided in Appendix 6 while the atomic ratios for individual points are presented in Appendix 8.

Figure 5.9 (A, B, C, D, E, F and G), presents three reaction products formed in the stabilized hydrated composite micrograph namely: the calcium silicate hydrate (C-S-H) in the yellow circle, calcium aluminate hydrate (C-A-H) in the blue circle and the calcium aluminate silicate hydrate (C-A-S-H) in the red circle. Discussion of these reaction products are presented in Chapter 6.

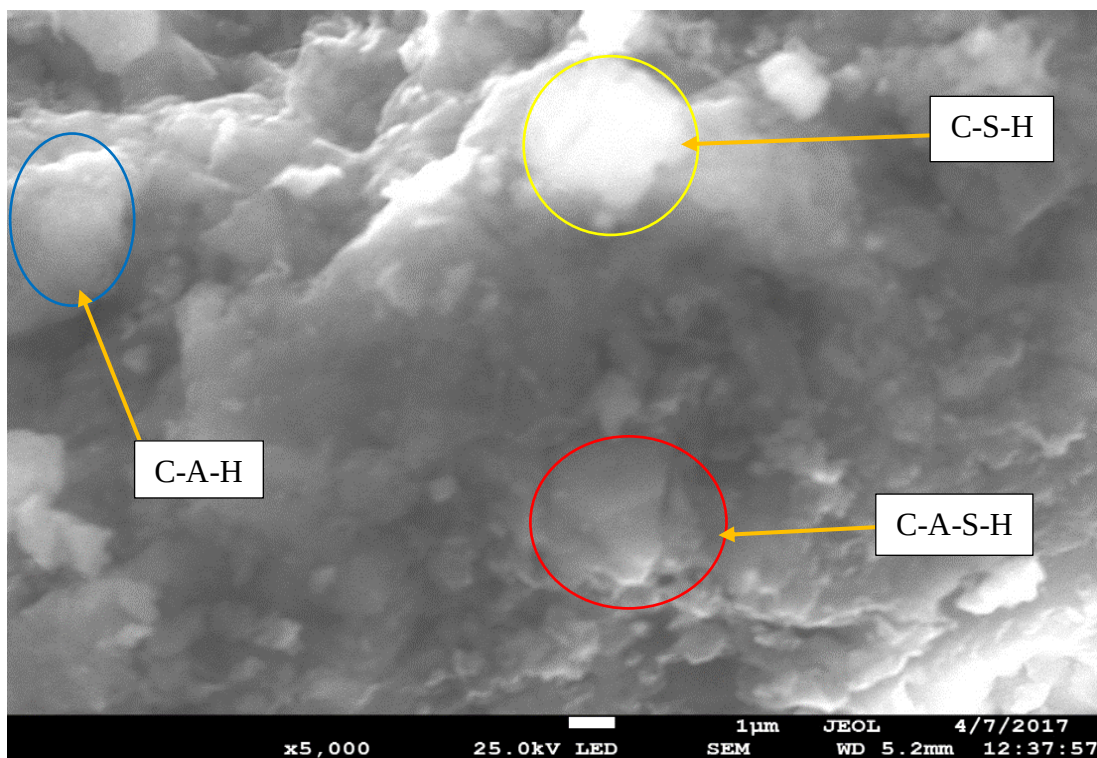


Figure 5.9 A: SEM micrograph of hydrated specimen (Composite)

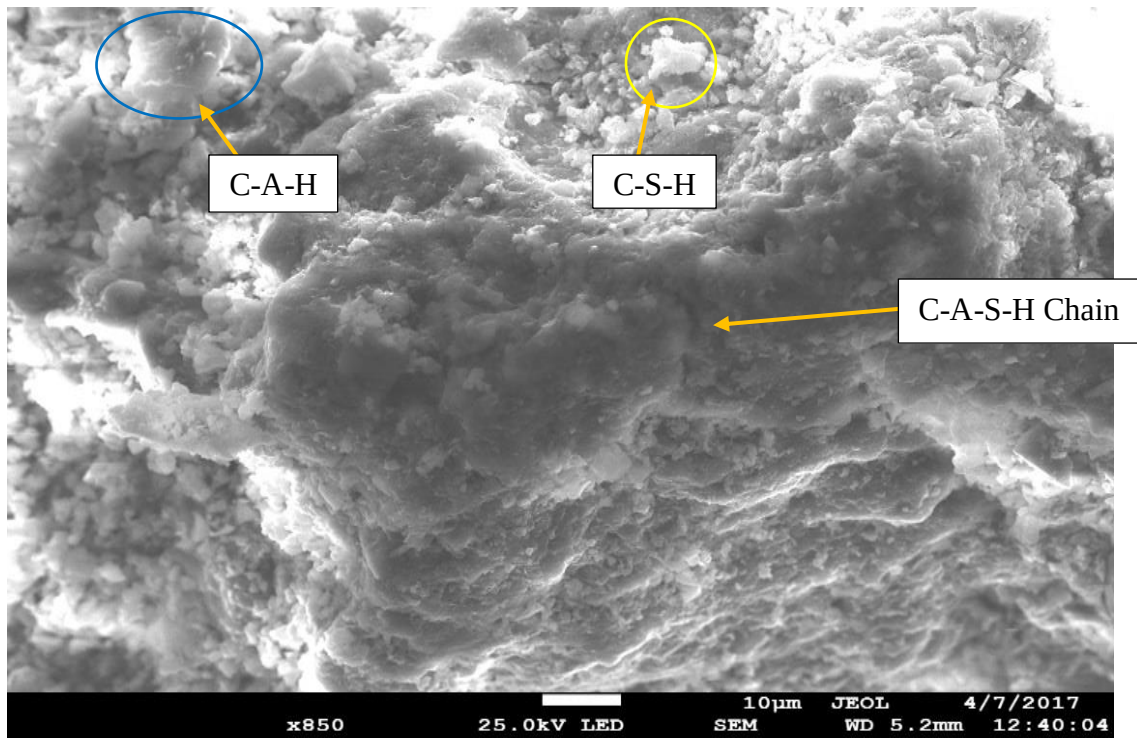


Figure 5.9 B: SEM micrograph of the hydrated specimen (composite)

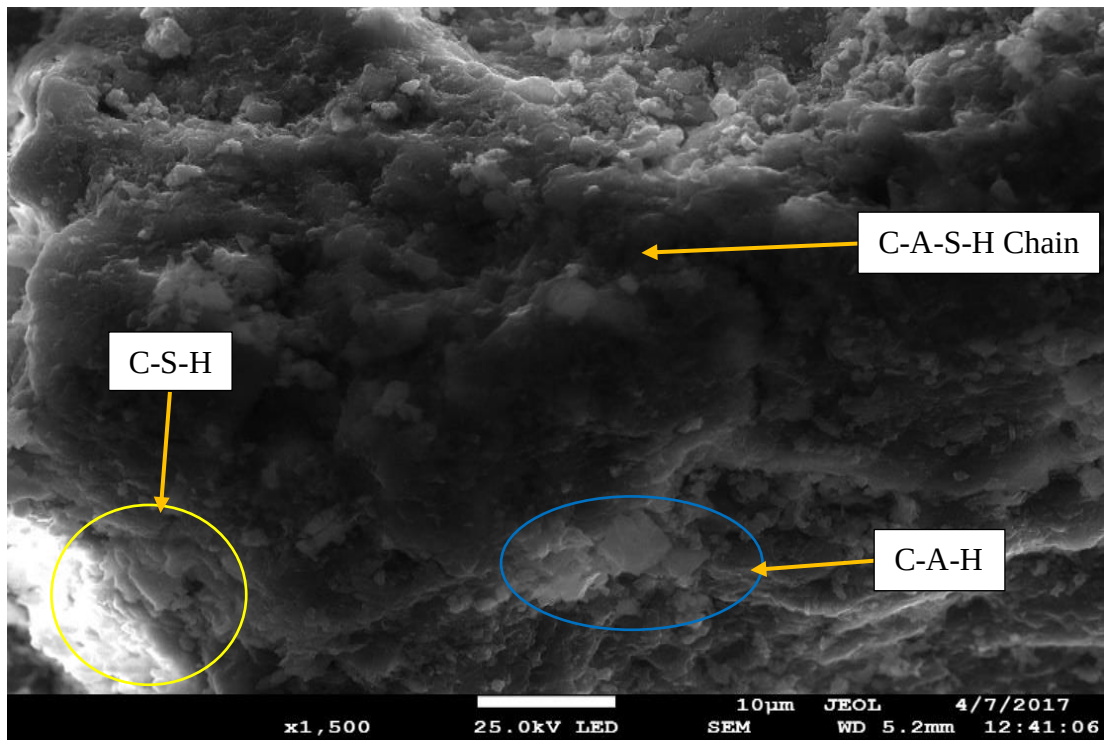


Figure 5.9 C: SEM micrograph of the hydrated specimen (Composite)

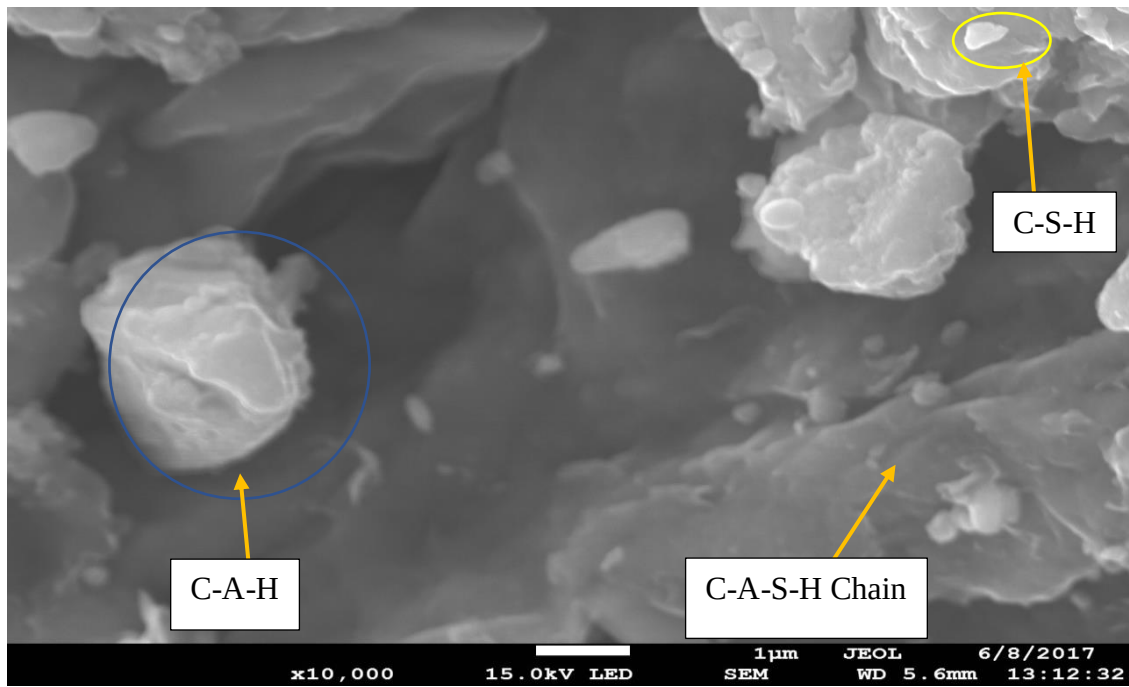


Figure 5.9 D: SEM micrograph of the hydrated specimen (Composite)

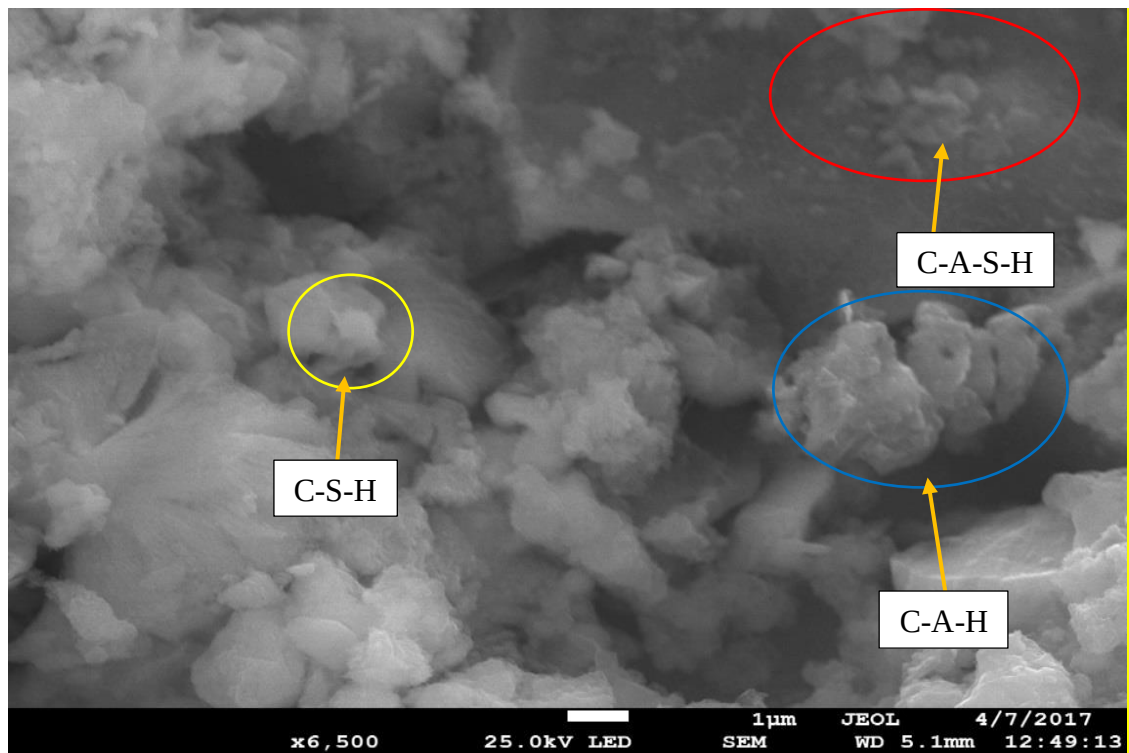


Figure 5.9 E: SEM micrograph of the hydrated specimen (Composite)

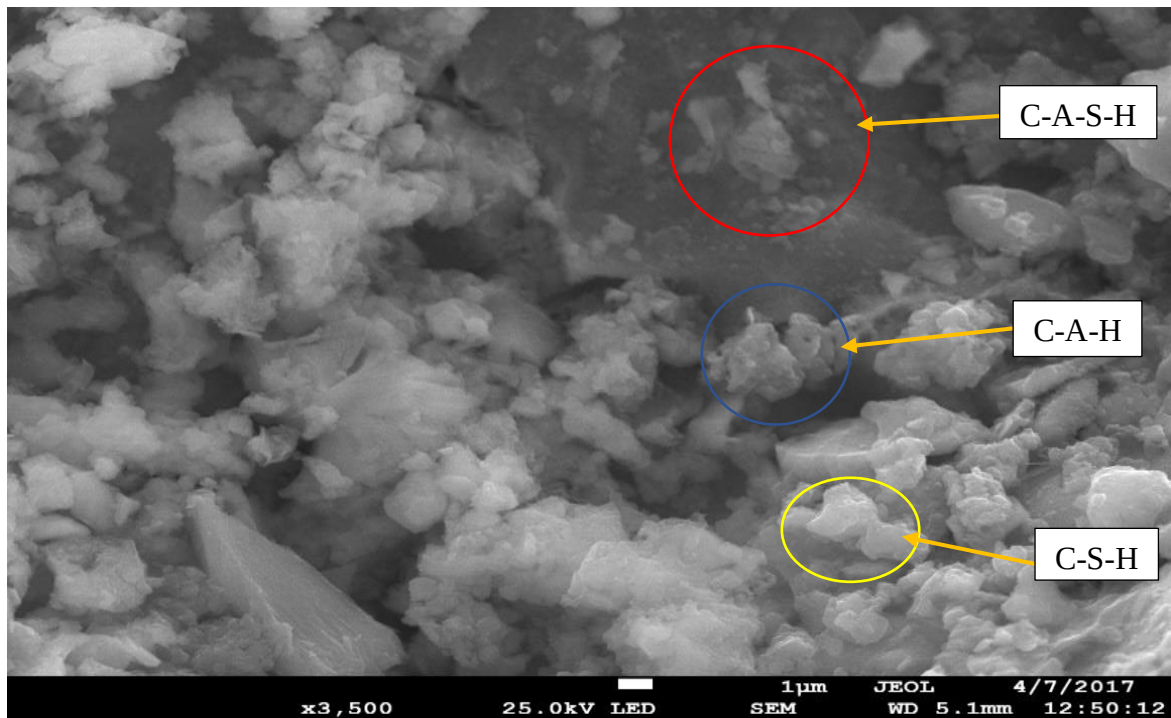


Figure 5.9 F: SEM micrograph of the hydrated specimen (Composite)

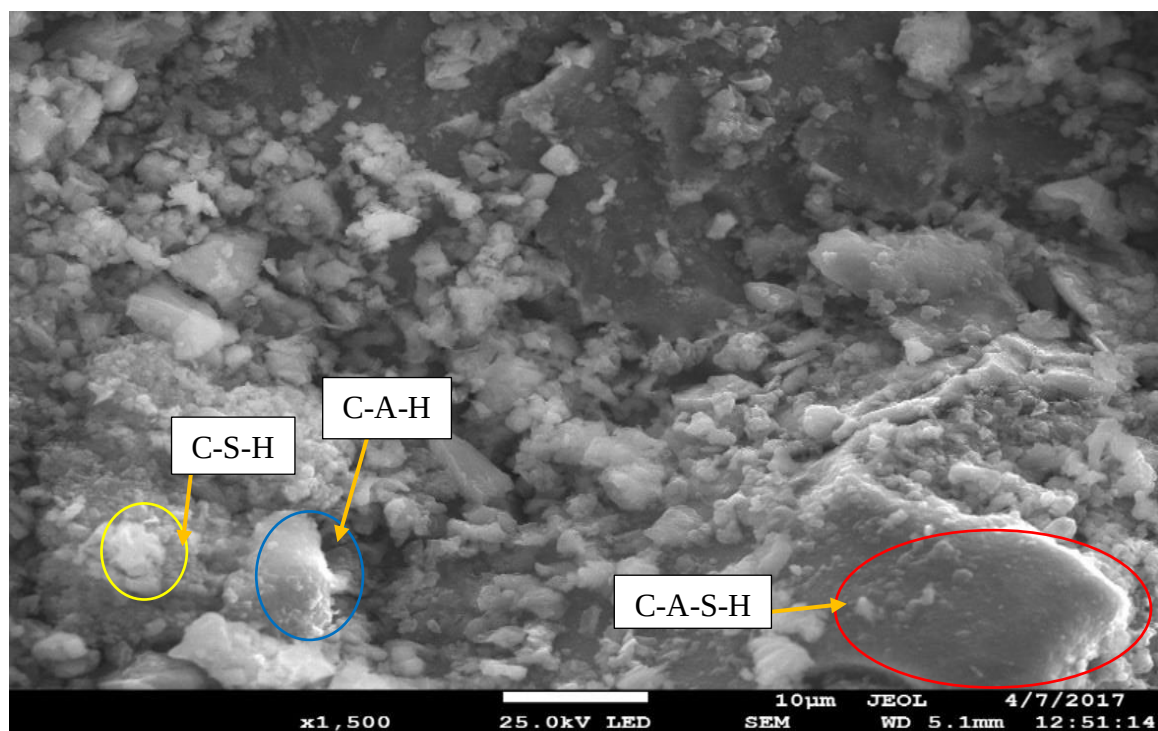


Figure 5.9 G: SEM micrograph of the hydrated soil specimen (composite)

Figure 5.10 A and B show the relationships between Mg/Si atomic ratio against (Mg +Al +Fe Ti +Mn +Ca)/Si (Octahedral/Tetrahedral) atomic ratio for the energy dispersive X-ray analysis (EDX) in the stabilized hydrated composite and stabilized dry composite. Figure 5.10 A shows how the reactant elements of low Mg/Si atomic ratio against (Mg +Al +Fe Ti

+Mn +Ca)/Si atomic ratios are distributed in the hydrated specimen and dry specimen, while Figure 5.10 B shows how the reactant elements of intermediate Mg/Si atomic ratio against (Mg +Al +Fe Ti +Mn +Ca)/Si atomic ratios are distributed in the hydrated specimen and dry specimen (composites). Figure 5.11 shows the relationship between Ca/Si atom ratio against Al/Si atom ratio of the hydrated composite and stabilized dry composite. Figure 5.11, also shows how the reactant elements atomic ratios of Ca/Si against Al/Si atomic ratios are distributed in the dry specimen and the hydrated specimen (composites). Figure 5.12 presents the relationship between Si/Ca atomic ratios against Al/Ca atomic ratios of the stabilized dry composite and the hydrated composite. In Figure 5.12, the distribution of reactant elements atomic ratios in the stabilized dry composite and the hydrated composite are shown. The atomic ratios for individual points are presented in Appendix 8. It is observed that the Al/Ca ratio linearly rose with an increase in Si/Ca ratio which satisfies Eqn. 2.8 presented in Chapter 2.

$$\text{Si/Ca} = 0.4277 + (2.366 \times \text{Al/Ca}) \quad [\text{Eqn. 2.8}]$$

In Figure 5.12, Si/Ca is plotted against Al/Ca together with the straight-line representing Equation 5.1. Equation 5.1 is based on soil-lime structure of C-A-S-H and the equation is used due to the similarity between calcium aluminate silicate hydrate (C-A-S-H) system and the calcium aluminate silicate hydrate formed in natural soil-CS-lime system. The hydrotalcite type containing magnesium was identified as the principal reaction product in CS hydration. The most interesting finding is the calcium aluminate silicate hydrate (C-A-S-H), C-S-H and C-A-H, in most of the images in both the stabilized dry composite and stabilized hydrated composite. In the first phase aluminum substituted C-A-S-H was formed due to CS hydration by hydrated lime. In the second phase of C-A-S-H substitution, the clay-lime reaction indicates that in this phase, the lime is being mixed with clay particles and it has low calcium content. See Section 6.7 for how reaction products were identified.

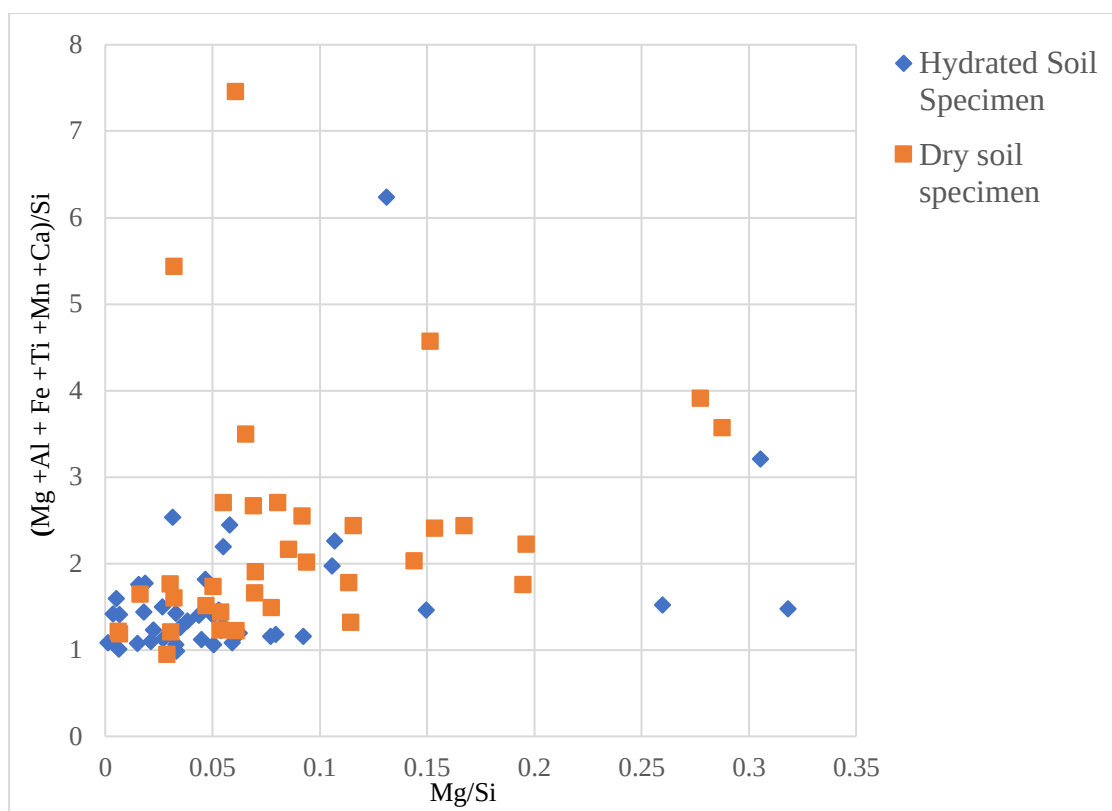


Figure 5.10 A: Mg/Si against (Mg +Al +Fe +Ti +Mn +Ca)/Si atomic ratio for the EDX data in Figure 5.9 (A, B, C, D, E, F and G) of hydrated specimen with low Mg/Si from the EDX analysis

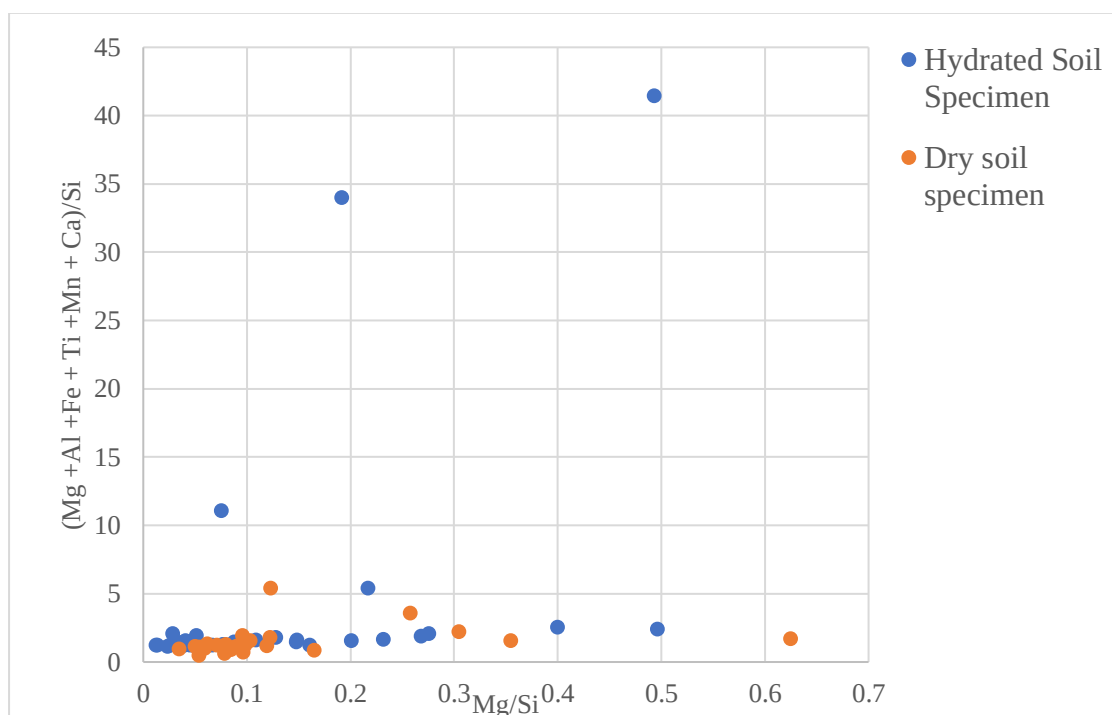


Figure 5.10 B: Mg/Si against (Mg +Al +Fe +Ti +Mn +Ca)/Si atomic ratio for the EDX data in Figure 5.9 (A, B, C, D, E, F and G) of hydrated specimen with intermediate Mg/Si from the EDX analysis

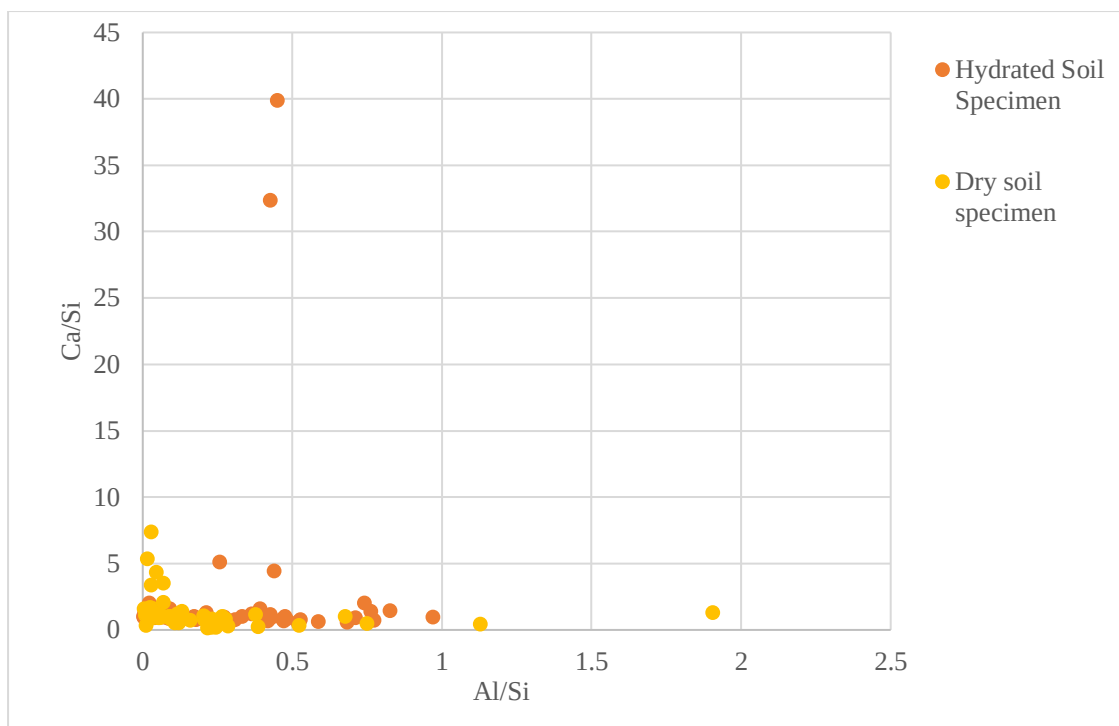


Figure 5.11: Al/Si against Ca/Si of the ratio for the EDX data in Figure 5.10 (A, B, C, D, E, F and G) of the hydrated specimen

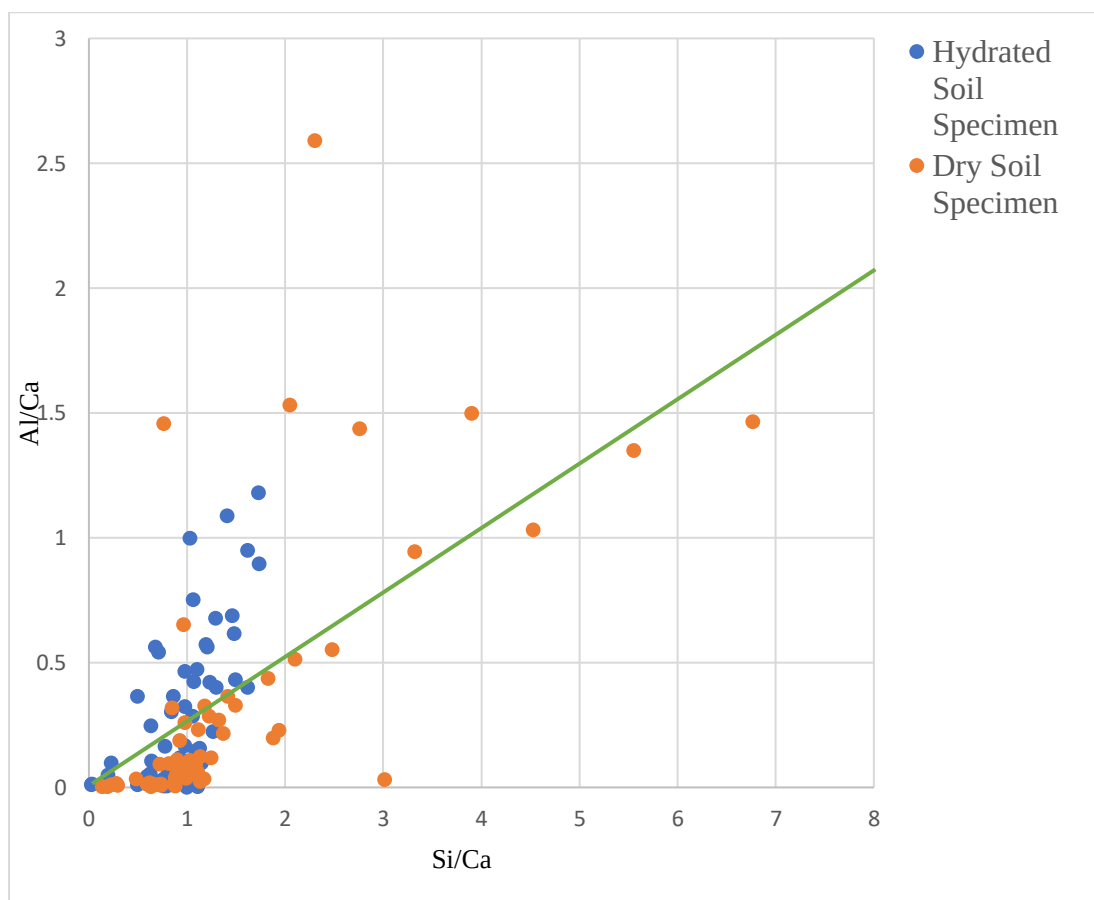


Figure 5.12: Si/Ca against Al/Ca atomic ratio for the EDX data in Figure 5.10 (A, B, C, D, E, F and G)

CHAPTER 6

DISCUSSION OF TESTS RESULTS

6.1 Introduction

This chapter discusses the results of the engineering and analytical tests in relation to the composition and changes in properties. The changes in the soil-CS-hydrated lime system is explicated in terms of short-term changes in chemical and physical properties that occur after the addition of CS and hydrated lime (between few hours and three days) during the initial stabilization process and the long-term changes in material properties occurring over a long period of time.

The changes in properties of the soil are mainly physical and chemical changes. The physical changes include change in moisture content, plasticity characteristics, compaction characteristics and particle size distribution. The chemical effects include flocculation, pozzolanic reaction and change in mineralogical composition that affect the strength and volume stability of the soil.

6.2 Chemical and Engineering Properties of the Soil, Hydrated Lime and Copper Slag

The natural soil, copper slag and hydrated lime chemical oxides have been presented in Table 4.2 in Chapter 4. The physical, mineralogical and engineering properties of the natural soil, copper slag and hydrated lime are presented Table 4.1. The natural soil liquid limit and plasticity index fall on the A-line shown in Figure 4.13 and was classified as silty clay with very high plasticity. The addition of CS and hydrated lime to the natural soil, changed the soil from very high plasticity silty clay to high plasticity silt (Figure 4.13).

It is important to understand the chemical composition and the physical properties of copper slag and hydrated lime before their usage in road construction due to the existence of heavy metal content which may cause both soil and ground water pollutions in the surrounding region. A knowledge of the percentage of the chemical composition of the stabilizers help to know the amount of heavy metal content in the stabilizers and help to not exceed the acceptance level of heavy metal content in the materials.

The investigation on the physical, geochemical and geotechnical characteristics of copper slag and lime shows that the materials have favorable engineering properties which make them suitable for use as stabilizers or admixtures to improve the strength of weak subgrade soil. Copper slag contains a high amount of Fe_2O_3 (48.28 %), SiO_2 (24.84 %), CaO (16.89 %), and other chemicals which make it amenable for chemical stabilization (Das et al., 1993). Hydrated lime also contains a high amount of $\text{Ca}(\text{OH})_2$ of about 65.09 % and CaO of about 20.91 % and other chemicals which make it suitable for use in chemical stabilization. Therefore, it is the properties of these materials which qualify them to be suitable alternative materials for use as stabilizers or binders in improving subgrade clay materials strength.

6.3 Effect of CS and Hydrated Lime Addition on Compaction Characteristics of the Soil

6.3.1 Effect of CS Addition on the Compaction Characteristics of the Natural Soil

The effect of CS addition on the compaction characteristics of the composite soil are presented in Section 4.3. The results show an increase in OMC from 18.81 % to 20.57 % and a decrease in MDD from 1.92 Mg/m^3 to 1.68 Mg/m^3 with increasing CS content ranging from 2 % to 10 %. The increase in the OMC with increasing CS content is thought to be due to the CS particles being finer than the soil particles. Therefore, because of the greater specific area that has to be lubricated, the soil-CS mixture is anticipated to require more moisture to attain maximum dry density than the untreated soil. The decrease in the maximum dry density corresponding to CS addition is probably associated with the replacement of the soil particles by glassy angular shaped CS particles in a given volume. The voids between the soil particles are partially filled by CS particles to prevent them from coming into closer state of packing and leaving large volume of voids. In this stage of mixing there is unlikely to be any pozzolanic reaction between CS and the soil particles.

6.3.2 The Effect of CS and Hydrated Lime on the Compaction Characteristic of the Soil

Cation exchange occurs between the cations and the negatively charged clay particles in the pore solution in the clay soil–lime system. The increase in cation concentration in the pore solution leads to the neutralization of the particles negative charge. The neutralization of clay particle negative charge causes them to come closer together thus leading to flocculation and agglomeration of the flocculated particles. Any further lime addition has

no principal effect on flocculation. The pore volume is increased due to flocculation that leads to the clay particles becoming bigger and leads to reduction in the maximum dry density.

Stabilization of the natural soil with CS activated by hydrated lime resulted in a decrease in the maximum dry density from 1.92 Mg/m^3 to 1.67 Mg/m^3 which may be due to the formation of cementitious products (see Section 5.3.1) which reduce the compatibility of the treated soil. The optimum moisture content also increases as a result of an increase in the pore volume due to flocculation.

Three reactions products were observed to be present in the soil-CS- lime system, the hydration of CS activated by hydrated lime to produce calcium aluminate silicate hydrate (C-A-S-H) gel and the hydrotalcite type of phases containing magnesium. The remaining two reaction products are the calcium aluminate hydrate (C-A-H) and calcium silicate hydrate (C-S-H) present in the soil-CS-lime system (see Section 5.3.1). The principal reaction in the short-term is the hydration of CS activated by hydrated lime which normally begins immediately after mixing the dry material with the required distilled water. The CS hydration usually consumes a relatively large quantity of water and relatively little quantity of lime. However, some free lime is likely to be present after CS hydration in the mixtures depending on the percentage of lime addition required to activate the CS. As a result of CS hydration, the available water in the mixture decreases the air voids in order for the mixture to be lubricated. Therefore, the optimum moisture content increases with an increase in the total binder content as presented in Figure 4.8.

Increase in CS/lime ratio in the soil mixture causes changes in the system. The exchange of cation and flocculation of the composite soil occur immediately after the addition of water and only little quantity of lime is consumed in the reaction. The formation of cementitious materials as a result of the pozzolanic reaction was due to the excess quantity of lime in the soil mixtures. Some researchers including Abdelkader and Hamdani (1985); Wild et al (1987) observed the formation of cementitious materials immediately after adding water to the mixture. The mixture comprising of the natural soil activated by hydrated lime indicated the presence of cementitious material such as the C-A-S-H, C-A-H and C-S-H formation after mixing of the dry samples with hydrated lime (see Section 5.3.1). The hydration of the slag activated by lime is faster than the clay-lime reaction. The formation of pozzolanic

materials in such a system is possible. The material fills a relatively large part of the voids between the soil particles thereby lowering the voids space.

On the other hand, the optimum moisture content increases with an increase in CS/lime ratio, keeping the total binder content constant. Increasing the lime content in such systems unquestionably consumes large part of the mixing water in hydration. Mixtures with high lime content require more water to attain the same level of lubrication and workability, thus increasing the OMC. There is however, an increase in strength of the soil (see Section 4.6) which may be due to pozzolanic reactions which compensated more for the changes in compaction characteristics (maximum dry density decreases and the optimum moisture content increase). It is an added advantage for CS and hydrated lime addition to the soil which help improve the soil strength (Kinuthia, 1997).

6.4 Effect of CS Addition and CS and Hydrated Lime on the Plasticity Characteristics of the Natural Soil

6.4.1 Effect of CS on the Plasticity Characteristics of the Natural Soil

Copper slag addition to the natural soil from 2 % to 10 % by 2 % increment for 1-day mix and 3 days after mixing, causes small decrease in liquid limit for 1-day mix represented as D1-NS-CS-LL from 82 % to 79 % and liquid limit after 3 days mixing represented as D3-NS-CS-LL from 82 % to 78 % respectively. A small increase was observed in both plastic limit for 1-day mix represented as D1-NS-CS-PL from 37 % to 39 % and 3 days after mixing represented as D3-NS-CS-PL from 37 % to 41 % respectively. The plasticity index for 1-day mix represented as D1-NS-CS-PI and 3 days after mixing represented as D3-NS-CS-PI decreased from 45 % to 40 % and 45 % to 37 % respectively (Figure 4.9 and Appendix 1). The changes in the Atterberg limits in comparison to the effect of CS and hydrated lime on the plasticity behaviour are relatively little. For example, the maximum change in the liquid limit for 1 day and 3 days are 3 % and 4 % at 10 % binder content respectively. The maximum change in the plastic limit for 1 day is 2 % at 6 %, 8 % and 10 % CS binder content while for 3 days is 4 % at 8 % and 10 % binder content. The maximum change in the plasticity index for 1 day and 3 days are 5 % and 7 % at 10 % binder content respectively. These changes are considered to be due to the replacement of natural soil particles by CS particles. There is unlikely any chemical change taking place at the early stage of the reaction of CS-natural soil as the test was after 3 days (see Appendix 1).

6.4.2 Effect of Hydrated Lime on the Plasticity Characteristics of the Natural Soil

Two main properties that control the clay soil plasticity are the plate-like structure and the associated adsorbed water. The interaction between the adsorbed water of each clay platelet and the effect it has on the particle's interaction can bond the platelets together, and the strength of this bond depends on the thickness of oriented water layers to some great extent. As stated in Chapter 2 soils which have great specific surface area and electrical charge and higher degree of plasticity than kaolinite soil is the montmorillonite soil. The large electrical charge associated with montmorillonite results in much larger capacity for water than kaolinite (Grim, 1953; Daniels, 1971).

The addition of 2 % hydrated lime to the natural soil created linkages between the clay particles and the hydrated lime particles. An increase in the hydrated lime content from 2 % to 10 % allows cation exchange to begin with direct effect on the plasticity of the natural soil (see Figure 4.14 and Appendix 1). The exchange of cation begins after a mellowing period of 3 days with an increase in hydrated lime content from 2 % to 10 % to the natural soil. The clay particles will then be surrounded by some diffuse hydrous double layers modified by the ion exchange of calcium. This leads to the alteration of the density of the electrical charge around the clay particles which causes the clay particles to change and to form silt size particles (flocules). This is principally responsible for plasticity modification of the clay soil when treated with lime (Sherwood, 1993).

The addition of 2 % lime to the natural soil leads to an increase in the liquid limit from 82 % to 84 %. An increase in lime content up to 4 % by dry weight of natural soil increases the liquid limit from 82 % to 86 % (Figure 4.14 and Appendix 1). The early increase in the liquid limit is due to the linkage created between the clay particles and hydrated lime particles. The rise in the plastic limit from 37 % to 50 % with the addition of hydrated lime content ranging from 2 % to 10 % after 3 days of maturing period is sufficient for all calcium cations to get involved in the exchange of cations and the beginning of flocules formation which leads to the formation of voids in the composite material comprising of the soil and hydrated lime. Increasing the lime content in the mixture to 6 % and 8% may raise the cation exchange and the flocules formation, therefore leading to the reduction of the liquid limit to a minimum value of 83 %. The liquid limit also reduced from 82 % to 76 % at 10 % lime content (Figure 4.14 and Appendix 1) over 3 days mellowing period which was sufficient to get the clay particles saturated with lime cations and form more flocules. Soil-lime

mixture flocculation also causes a decrease in the liquid limit by creating weak hydrogen bonds between floccules. Little proportion of water is necessary to lubricate them to the stage where the required movement would take place with shearing action. Addition of hydrated lime to the natural soil raised the plastic limit from 37 % to 50 % at 10 % binder content for both 1 day and 3 days after mixing. As explained by Daniels, (1971), the rise in the plastic limit is caused by the water being held by capillary forces inside the newly flocculated clay structure and the electrostatic forces on the clay mineral surface.

Due to the reduction in the liquid limit from 82 % to 76 % and the rise in plastic limit from 37 to 50 %, the plasticity index decreased with an increase in lime content up to 4 % at lime fixation point, and then continued to decrease down to 10 % lime content (Figure 4.14). Further reduction in the plasticity index was observed over 3-day mellowing period due to the decrease in liquid limit described above.

6.4.3 Effect of CS and Hydrated Lime on the Plasticity Characteristic of the Natural Soil

The natural soil-CS-lime system is alike to that of natural soil-lime system to some extent, particularly in the short-term. When CS and lime are added to the natural soil, CS hydration begins to occur as it is a relatively rapid reaction as indicated by Meng et al., (1998). The rapid reaction consumes a part of the lime and dries out to some extent part of the mixing water. A little percentage of lime is sufficient to activate the CS (between 10 % and 30 % of the total binder content depending on the soil type), and the remaining of the lime is used for cation exchange and flocculation of the clay particles (see Figures 4.10, 4.11 and 4.12).

The liquid limit for 1-day mix and 3 days after mixing increased to the maximum value of 88 % and 87 % respectively upon the addition of 2 % minimum binder content to the soil at the ratio of 9:1 CS replacement by lime and thereafter drops to 69 % and 68 % respectively at 10 % maximum binder content addition. The liquid limit for both 1-day mix and 3 days after mixing increased to the maximum value of 86 % upon the addition of 2 % minimum binder content to the soil at the ratio of 4:1 CS replacement by hydrated lime and thereafter decreased to 68 % and 66 % respectively at 10 % maximum binder content addition. The liquid limit for both 1-day mix and 3 days after mixing increased to the maximum value of 85 % upon the addition of 2 % minimum binder content to the soil at the ratio of 7:3 CS replacement by lime and thereafter dropped to 60 % at 10 % maximum binder content addition for both 1 day and 3 days after mixing. Liquid limit also decreased for 9:1, 4:1 and

7:3 replacements after 3 days mellowing period which may be sufficient for calcium cations to get involved in the cation exchange and begin forming floccules. Increasing the binder content from 0 % to 10 % lead to further reduction in the liquid limit as the binder content rises and more floccules are formed. The flocculated nature of the soil-CS-lime system, causes a decrease in the liquid limit of the soil by increasing relatively weak hydrogen bonds between floccules (see Figures 4.10 to 4.13).

The addition of CS activated by hydrated lime to the clay soil raises the plastic limit to a maximum value of 47 % at 10 % binder content. The rise in plastic limit (see Figures 4.10 to 4.12) may be due to the formation of floccules and reasons presented above. The plastic limit values in the case of CS activated by hydrated lime are lower than those of lime alone at the same binder content. This is a result of a little quantity of lime being consumed in the CS hydration. The plasticity index also reduced due to the reduction in liquid limit and the rise in the plastic limit.

6.5 Effect of CS Addition and CS-Hydrated Lime on Strength Properties of the Natural Soil

6.5.1 Effect of CS Addition Alone on Strength Properties of the Natural Soil

The UCS of the composite soil increased significantly from 759 kN/m² to 998 kN/m² under CC1 and 845 kN/m² to 1391 kN/m² maximum under CC2 with an increase in CS content and increase in curing period (see Appendix 2). This behaviour is attributed to both short-term and long-term changes. The short-term changes occur during the initial curing period up to 28 days after which the long-term changes occur up to 74 days of curing period. There is little change in the UCS for the 7 days curing period. Thereafter, the UCS increased as the CS percentage increased up to 4 % after 16, 28 and 42 days. The UCS also increase for 56 and 74 days up to 6 % CS content curing under CC1. Further increase in the CS content beyond 4 % for 7, 16, 28 and 42 days resulted in a reduction in the UCS while further increase beyond 6 % CS for 56 days and 74 days curing period decrease the UCS under CC1. The UCS increased as the percentage of CS content increased up to 6 % for 16, 28, 42 56 and 74-days curing whereas the UCS increased for 7 days up to 4 % CS content under CC2. Further increase beyond 6 % for 16, 28, 42, 56 and 74 decreased the UCS while for 7 days curing beyond 4 % the UCS decreased (see Appendix 2). The reduction in moisture content during the curing periods of the UCS after 7 and 74 days varied based on the curing temperature and CS content (see Figures 4.20 and 4.21). The rise in the UCS was due to the

reduction in moisture content, hydration reaction of CS (where the cementitious materials mix with water resulting in the production of C-S-H, C-A-H and C-A-S-H), and also changes due to the pozzolanic reaction (where the addition of water to slag cement leads to the formation C-S-H and C-H) and resulting in strength improvement (see Section 5.3.1). There was little amount of calcium, aluminium, silica and water in the composite which led to the formation of C-A-S-H specifically during long curing period under CC2. The UCS increase was, due to the presence of high calcium hydrates in the composite soil which indicates that the natural soil was activated by the hydrated lime and some of the reactant elements found in the stabilizers. The addition of CS only to the soil resulted in the formation of C-A-S-H after 42 days of curing period under CC1 and CC2. The little reduction in the unconfined compressive strength with increasing CS content beyond 6 % is thought to be due to the reduction in the MDD.

The long-term rise in the unconfined compressive strength is thought to be a result of the increase in the amounts of C-A-S-H present due to the longer curing period, and also a result of the rise in the pore water suction is due to the decrease in the moisture content. The effect of the pore water suction on the unconfined compressive strength was higher at higher curing temperature than at lower curing temperature due to the reduction in water content (see Figures 4.20 and 4.21).

6.5.2 Effect of CS and Hydrated Lime on the Strength Properties of the Natural Soil

The nature of the long-term cementation in clay soil stabilization depends on the effect of curing period and curing conditions which determine and control the strength of the mixtures among other factors (for example, compaction). The principal cementitious agent in the clay soil- lime stabilization system is the calcium alumino silicate hydrate (C-A-S-H) gel which comprises of calcium, aluminium, silica and water. The principal cementing agent dominant in the clay-CS-lime system is still C-A-S-H gel. Kinuthia (1997) indicated that the pore solution of these systems contains silicate and aluminate ions which are formed by the dissolution of the clay particles in the highly alkaline environment provided by the dissolved lime. The high content of alumina and iron in CS was responsible for some of the alumina to replace silica in the mixture of CS, lime and clay soil system to allow C-A-S-H gel to be formed. The X ray diffraction results displayed the formation of crystalline calcium aluminate hydrate (C-A-H). In this study, the results of the test reveal rise in the unconfined compressive strength with an increase in total binder content, by replacing CS with an

increased lime content up to the ratio of 7:3 with an increase in curing period and temperature. The addition of 2 % minimum binder content to the natural soil resulted in a small increase in strength of the composite up to 28 days curing period for all the CS percentage replacements by hydrated lime at a lower curing temperature. The minimum quantity of lime that is required for the complete modification to begin the pozzolanic reaction is principally dependent on the clay type and content and lime content. A minimum quantity of 4 % binder content was used for the full natural soil modification before beginning the pozzolanic reaction.

From the pH test results (Figure 4.15), the minimum quantity of binder that caused the full modification of the soil was at 4 % CS activated by hydrated lime at the CS replacement ratio of 4:1. These values were adequate to allow an increase in the pH to 12.55 which is close to the saturated lime pH solution of 12.65. Therefore, 2 % binder content activated by 4:1 and 7:3 ratios of hydrated lime had a small chemical effect on the soil and no cementitious materials were anticipated. In this situation, the principal reason for the rise in strength is thought to be the rise in pore water suction due to the little reduction in the moisture content (see Figures 4.25 and 4.26).

At 6 % binder content, the pH rises to 12.28 and above. Thus, the pore water suction effect on the composite soil was considered to be minor compared to the pozzolanic reactions effect. Therefore, the principal reason for the rise in the strength is due to the pozzolanic reaction and hydration reaction, where CS is activated by hydrated lime and the reactions of natural soil -lime. An increase in the binder content up to 6 % CS led to an increase in the UCS of the mixtures with ratios of 4:1 and 7:3 CS replacement by hydrated lime. The rise in the strength is mostly due to the increase in the cementitious product content. The rise in the CS/lime ratio is up to the ratio of 7:3 which caused an increase in the strength of the composite soil. There was no further replacement of CS by hydrated lime beyond the ratio of 7:3 that was investigated. In the author's opinion the optimum CS/lime ratio depends principally on the clay type and content, curing period and curing conditions as also indicated by Higgins et al, (1998) and Wild et al., (1999).

When CS gets exposed to water the Si, Al, Fe and O rich layer is formed on the CS particles surfaces. The rich layer absorbs the H^+ from the water which results in an increase in OH concentration and also increases the pH of the solution to the values closer to the pH of the saturated lime solution of 12.65 (Caijun and Day, 1993). The Si-O, Al-O and Fe-O are

broken down at high pH values to allow the formation of a semi-crystalline C-A-S-H, crystalline C-A-S-H and calcium aluminate hydrate (C-A-H). The rise in the C-A-S-H content indicates an increase in the strength while the rise in the chain length did not appear on the surface of the composite soil specimen but produces a low Ca/Si ratio. Therefore, it increases the degree of crystallinity and homogeneity of the C-A-S-H. The rise in the crystallinity and the homogeneity of C-A-S-H indicates that there is a stable reaction product that led to the increase in UCS. Many factors determine the characteristics of the clay soil-CS- lime mixtures which include the clay soil content and type, mixing, moisture content, and cementitious product content, curing conditions and periods, and the ratio of CS/lime. These factors determine the amount and properties of the reaction products formed and thus affect the UCS of the soil under test.

The SEM test results present crystalline C-A-S-H together with semi-crystalline C-A-H and C-S-H as the principal reaction products in Figures 5.6 A, B, C, and 5.9 A, B, C, D, E, F and G. The increase in the C-S-H content implies an increase in the UCS of the composite soil while the decrease in the calcium hydroxide indicates the consumption of the hydrated lime by the activation of CS in the formation of the C-S-H. Also, the increase in C-A-S-H content implies an increase in strength while the chain length of C-A-S-H produces lower Ca/Si ratio (Figure 5.9 B, C and D), this was understood from the SEM results. The scanning electron microscopy image shows the homogeneity and crystallinity of the specimen cured for 74 days under both CC1 and CC2.

Bell (1996) observed that the strength developed in clay-lime system is influenced by the quantity of cementitious gel produced and consequently on the amount of lime consumed. He indicated that extended curing periods and elevated temperatures promote pozzolanic reactions. For instance, significantly improved strength can be developed with relatively small increase in temperature. Conversely, if the temperature is very low, pozzolanic reaction may cease. Similar behaviour was found in the soil-CS-lime system. Extended curing periods and elevated curing temperature promoted the pozzolanic reactions and the cementitious gel produced was influenced by the factors presented above, based also on XRD and SEM data.

6.6 Effect of the Addition of CS Alone and CS and Hydrated Lime on the Swelling Characteristics of the Soil

Many problems have been observed in structures built over expansive clay soils due to the change in their moisture content and the unpredictable movements of the structures. The change in volume and the swelling pressure in clay soils mostly depend on pore volume, clay soil type, forces of particle to particle interaction and the pore solution (Xidakis, 1979; Mowafy et al., 1990). The change in volume and swelling pressure are most closely associated and the estimation of the change in volume of soils gives suggestion of the swelling pressure. In the chemical soil stabilization processes, using lime or slag activated by lime, it has been established that new cementitious materials are formed and alter the particle-to-particle forces through the cementitious effect that these reaction products impart. This demonstrates that when these cementitious materials are formed, the pore pressure and pore fluid will be decreased and therefore reduce the swelling pressure and the swelling potential and lead to the stability of the volume (Kinuthia, 1997).

6.6.1 Effect of CS and Hydrated Lime on the Swelling Potential of the Natural Soil

The swelling test outcomes have demonstrated that CS addition to the soil activated by hydrated lime significantly decreased the swelling potential of the soil (see Table 4.6 A and B). The swelling percent reduces with an increase in total binder content, CS/lime ratio, curing temperature and period. To explain the effect of all factors on free swelling percent, two analytical tests (XRD and SEM) were conducted on the composite soil, CS and hydrated lime to identify the reaction products formed during curing (see Chapter 5).

The enhancement of cementitious properties leads to further gains in the binding forces between clay soil particles with further attack by the C-A-S-H gel of the clay soil particles. The hydration reaction of slag has demonstrated to produce a pore blocking effect which results in an increase in long-term strength and lower permeability that also contribute to the decrease in the swelling potential. The X-ray diffraction test results affirmed the semi-crystallinity properties of the formed gel (see section 5.2). Increasing the crystallinity and the homogeneity of the formed gel certainly has desirable effect on the long-term changes in the properties including a rise in the strength and an enhancement in the permeability and a decrease in the swelling potential of the bulk materials. Therefore, the decrease in the permeability of the bulk materials means a decrease in water absorption and decrease in the

probability of the stabilized soil being affected by the changes in the environmental conditions which lead to the change in volume.

In comparison of free swelling with the UCS test, it is observed that using CS alone has a slow and little effect on the swelling potential of the soil which can be seen in Figures 4.27 and 4.28. The binder content and the UCS on the other hand have an impact on the composite soil over their expected curing periods (see Figures 4.14 and 4.15). It is observed that using CS activated by hydrated lime has a significant effect on both the swelling potential in Figures 4.29 to 4.34 and the UCS in Figures 4.20 to 4.22. It is observed from the above results that the swelling potential of the composite soil was suppressed in two possible ways. The first is by using higher percentage of CS only with longer curing period particularly at higher temperature. The second is using lower percentage (2 % or 4 %) of CS activated by hydrated lime with short curing time up to 28 days. For instance, if the allowable swelling is about 6 %, this could be found by either using 4 % CS with 74 days curing period under CC1 or utilizing 2 % CS with 28 days curing period under CC 2. The design engineer can better choose the best suitable system of curing period, curing condition, binder content and CS/lime ratio based on the design specification of the project and also according to the nearest available material on site and the project area environmental conditions.

6.7 Reaction Products

The reaction products are described in two fundamental reactions that are occurring, the first is the CS hydration by hydrated lime and the second is the natural soil-hydrated lime reaction. The CS hydration reaction consumes a comparatively little amount of the hydrated lime which begins right after mixing with water. This reaction under normal conditions produces C-A-S-H gel with low Ca/Si ratio (≈ 1.11), low Al/Si ratio (≈ 0.15) and a Ca/(Si+Al) ratio (≈ 1.06) (see Appendix 6 and 7). The other part noticed in the CS hydration is the hydrotalcite type phase which contains magnesium in a general form in Eqn. 2.9;



Where $R^{2+} = Mg^{2+}$; $R^{3+} = Al^{3+}$ or Fe^{3+} , $R^{r-} = OH^-$ SO_4^{2-} , $K = 1$ and $J = CO_3^{2-}$. For the hydrotalcite type phase in general $\sim 0.2 \leq K \leq 0.33$ (Brindley and Kikkawa, 1979; Mascolo and Marino, 1980).

The clay-lime reaction is the second reaction to produce colloidal C-A-S-H gel. In this phase of the reaction, it has a low calcium content and is mixed with clay particles with a lower average of Ca/Si ratio ≈ 0.28 , an average of Al/Si ratio ≈ 0.33 and Ca/(Si + Al) ratio ≈ 0.22 . This part in the clay-CS-lime reaction is the calcium aluminate hydrate of C_4AH_{11} and $\alpha 1C_4AH_{17}$ noticed in the X-ray diffraction traces, almost similar to Diamond and Kinter (1966) (also see section 5.2). Any high percentage of hydrated lime replacement by CS with enough lime to activate the CS hydration, may not allow the reaction of clay-lime to begin. Thus, raising the CS/lime ratio assures the presence of clay-lime reaction and formation of colloidal C-A-S-H gel and calcium aluminate hydrate. In this study, lime/CS ratio from 12 % to 44 % was sufficient to allow reactions to occur where the swelling potential decreases as the curing period increases from 7 days to 74 days. The composition and nature of the reaction products primarily/mainly depend upon the curing conditions and curing period.

In order to identify C-S-H, C-A-H and C-A-S-H in the microstructure of the dry and hydrated composites in Figures 5.6 (A, B and C) and 5.9 (A, B, C, D, E, F and G) individual specific points shown in the mentioned figures were analyzed for major reactant elements in the microstructure of the composites. C-S-H had two major reactant elements identified in the microstructure of the dry and hydrated composites which include calcium and silica. C-A-H also had two major reactant elements identified in the microstructure of the dry and hydrated composites which include calcium and aluminium. Due to the existence of these three reactant elements of calcium, aluminium and silica in the mixture of CS, hydrate lime and the soil, whenever water is added to the mixture, it leads to the formation of C-S-H and C-A-H over a period of time. when these three reactant elements react with the addition of water, it's also lead to the formation C-A-S-H.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The main aim of this work was to study the effect of copper slag without and with hydrated lime on the geotechnical properties of weak subgrade clay soil from Serowe. From the tests results and discussions, the following conclusions have been drawn:

1. The natural soil was classified as silty clay with very high plasticity. The addition of copper slag and hydrated lime to the natural soil, changed the soil from very high plasticity silty clay to high plasticity silt.
2. The addition of copper slag to the natural soil greatly decreased the maximum dry density from 1.92 Mg/m^3 to 1.68 Mg/m^3 and increased the optimum moisture content from 18.81 % to 22.57 %. The optimum moisture content increased with an increase in total binder content at a constant copper slag/lime ratio while the maximum dry density decreased.
3. The addition of copper slag from 0 % to 10 % to the soil does not result in any substantial changes in the Atterberg limits.
4. The addition of lime to the clay soil gradually increased the liquid limit from 82 % to a peak of 86 % at 4 % lime content and thereafter, reduced it gradually to 80 % at 10 % lime content. The plastic limit increased from 37 % up to 49 % at 4 % lime content and thereafter, increased up to 10 % lime content for 1-day maturity while the plasticity index for 3 days maturity ceased to increase. The plasticity index reduced from 39 % to 27 % with increase in lime content from 0 % to 10 %.
5. The copper slag content at which maximum increase was observed in the unconfined compressive strength of the composite from 7 days to 42 days was at 4 % while for 56 days and 74 days curing, the maximum increase in the unconfined compressive strength was observed at 6 % copper slag content under curing condition 1. Under curing condition 2, the copper slag content at which maximum increase was observed in unconfined compressive strength for 7 days was noticed at 4 % while from 16 days to 74 days curing was noticed at 6 % copper slag content.
6. The addition of 6 % copper slag to the natural soil after 74 days under curing condition 2, increase the unconfined compressive strength to 1391 kN/m^2 (40 % increase, see

Appendix 2). While the addition of copper slag and hydrated lime to the natural soil after 74 days at 6 % total binder content at the ratio of 7:3 under curing condition 2, increase the unconfined compressive strength to 3321 kN/m² (76 % increase, see Table 4.3).

7. The unconfined compressive strength of the composite soil increased with an increase in the copper slag content activated by hydrated lime. The unconfined compressive strength also increased with increase in the ratio of copper slag/lime and also increase with an increase in the curing period and temperature. After long curing periods of 42 days to 74 days, the unconfined compressive strength of the soil significantly increased with an increase in copper slag content and with an increase in the curing period and the temperature.
8. The swelling potential of the composite soil reduced with an increase in copper slag/lime ratio and with increase in curing period and temperature.
9. Two principal reactions were observed in the cementitious formation of the mixture when copper slag and hydrated lime were added to the soil namely:
 - a) the hydration of copper slag activated by hydrated lime produced calcium alumino silicate hydrate gel and the hydrotalcite type phase containing magnesium and iron and
 - b) the clay–lime reaction produced calcium silicate aluminate hydrate. The X-ray diffraction exemplified the formation of two major phases which are the semi-crystalline calcium silicate hydrate and crystalline calcium aluminate hydrate.
10. The Si percentage in calcium aluminate silicate hydrate increases with an increase in curing period under curing condition 1 while keeping all other parameters constant. However, no important change was noticed with a rise in the curing period under curing condition 2. The Si percentage in calcium alumino silicate hydrate increased with rise in the curing temperature while it significantly increased with an increase in binder content under curing condition 2.
11. Scanning electron microscope images showed that calcium aluminate silicate hydrate formed is homogeneously spread in the specimens and the clay particles forming a very dense homogeneous and non-permeable material. The aluminium silicate (Al/Si) ratio from the calcium aluminate silicate hydrate stay the same with an increase in the binder content and lime content while Al/Si ratio significantly increased with an increase in the curing period and temperature.

7.2 Recommendations

The following recommendations are made:

1. Further investigation is required on the effective and maximum binder content usage. Also, the critical copper slag/lime ratio to attain the best results in strength, swelling potential should be established on the clay-copper slag-lime reaction system. In the present study, the maximum copper slag/lime ratio used was 7:3, and a little quantity of lime was observed to be sufficient to activate the copper slag (copper slag/lime ratio = 1/9). Further investigations should include using high copper slag/lime ratio up to total replacement of copper slag by hydrated lime. Thus, the economic content of copper slag and hydrated lime which are sufficient to satisfy the requirements of the project should be determined.
2. The present work was limited to studying the effect of static loading on the strength behaviour of the Serowe clay soil. Further investigations should be conducted on the effect of cyclic loading on the strength and swelling behaviour as roads are subjected to dynamic loads together with static loads.
3. Following the laboratory investigation results, it is recommended to carry out field trials to assess the suitability of the stabilized materials for field application.
4. The effect of using copper slag activated by hydrated lime on the unconfined compressive strength and the swelling behaviour was limited to short curing periods of 7 days to 74 days. A further study should be conducted to investigate the effect of long curing periods on the unconfined compressive strength and the swelling behaviour of copper slag activated by hydrated lime.

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APPENDICES

APPENDIX 1: Plasticity Characteristics of Copper Slag-Soil, Copper Slag-Hydrated Lime-Soil and Hydrated Lime-Soil Mixtures

Effect of copper slag on plasticity characteristics of the natural soil				
Actual Values (0 % binder content)		LL (%)	PL (%)	PI (%)
		82	37	45
2	1 Day	-1	1	-2
	3 Days	-2	2	-4
4	1 Day	-1	1	-2
	3 Days	-3	2	-5
6	1 Day	-2	2	-4
	3 Days	-3	3	-6
8	1 Day	-2	2	-4
	3 Days	-3	4	-7
10	1 Day	-3	2	-5
	3 Days	-4	4	-8
Effect of copper slag and hydrated lime on plasticity at the CS/lime ratio of 9:1				
2	1 Day	6	+3	3
	3 Days	5	6	-2
4	1 Day	4	4	0
	3 Days	3	8	-5
6	1 Day	0	5	-5
	3 Days	-2	8	-10
8	1 Day	-7	6	-13
	3 Days	-7	9	-16
10	1 Day	-13	7	-20
	3 Days	-14	9	-23
Effect of copper slag and hydrated lime on plasticity at the CS/lime ratio of 4:1				
2	1 Day	4	4	0
	3 Days	4	7	-1
4	1 Day	0	5	-5
	3 Days	-1	7	-8
6	1 Day	-2	6	-8
	3 Days	-4	7	-11
8	1 Day	-8	7	-15
	3 Days	-9	9	-18
10	1 Day	-14	9	-23
	3 Days	-16	9	-25
Effect of copper slag and hydrated lime on Plasticity at the CS/lime ratio of 7:3				
2	1 Day	3	5	-2
	3 Days	3	7	-3
4	1 Day	-2	5	-7
	3 Days	-2	7	-9
6	1 Day	-6	6	-12
	3 Days	-7	7	-14
8	1 Day	-12	6	-18
	3 Days	-13	8	-21
10	1 Day	-22	10	-32
	3 Days	-22	11	-33
Effect of hydrated lime on plasticity characteristics of the natural soil				
2	1 Day	2	8	-6
	3 Days	-2	9	-11
4	1 Day	4	10	-8
	3 Days	-3	12	-13
6	1 Day	1	12	-11
	3 Days	-3	12	-15
8	1 Day	1	12	-11
	3 Days	-4	13	-17
10	1 Day	-2	13	-15
	3 Days	-6	13	-19

APPENDIX 2: Unconfined Compressive Strength of the Soil and CS

Curing period	Copper Slag	UCS (kN/m ²)	
		Curing Temperature	
		35 °C	45 °C
7 Days	0	759	845
	2	768	855
	4	799	1000
	6	781	982
	8	765	951
	10	744	941
16 Days	0	794	850
	2	798	870
	4	875	1019
	6	861	1111
	8	849	1100
	10	835	1009
28 Days	0	825	894
	2	863	905
	4	979	1195
	6	969	1285
	8	955	1244
	10	946	1208
42 Days	0	830	902
	2	865	927
	4	985	1220
	6	976	1329
	8	951	1301
	10	944	1291
56 Days	0	841	910
	2	878	935
	4	981	1246
	6	988	1371
	8	965	1355
	10	949	1322
74 Days	0	849	949
	2	921	990
	4	986	1250
	6	998	1391
	8	970	1374
	10	953	1336

APPENDIX 3: Calculating the effect of Curing Period on the Moisture Content of the UCS of the Composite Soil

$$S = U_a - U_w$$

S = metric suction

U_a = pore air pressure

U_w = pore water pressure = 10 kN/m²

$$U_a = (825 - 759) \text{ kN/m}^2$$

$$U_a = 66 \text{ kN/m}^2$$

$$S = (66 - 10) \text{ kN/m}^2$$

$$S = 56 \text{ kN/m}^2$$

Effect of Curing Period on the Initial Moisture Content of the Soil Treated with CS and Hydrated Lime

$$U_a = (1490 - 799) \text{ kN/m}^2$$

$$U_a = 691 \text{ kN/m}^2$$

$$S = (691 - 10) \text{ kN/m}^2$$

$$S = 681 \text{ kN/m}^2$$

Increase in UCS due to pozzolanic reaction

$$\text{Pozz}_{\text{reaction product}} = (949 - 845) \text{ kN/m}^2$$

$$\text{Pozz}_{\text{reaction product}} = 104 \text{ kN/m}^2$$

Increase in UCS due to chemical reaction

$$\text{CH}_{\text{reaction}} = (1391 - 949) \text{ kN/m}^2$$

$$\text{CH}_{\text{reaction}} = 442 \text{ kN/m}^2$$

➤ Increase in UCS due to chemical reaction

$$\text{CH}_{\text{reaction}} = (3321 - 1490) \text{ kN/m}^2$$

$$\text{CH}_{\text{reaction}} = 1831 \text{ kN/m}^2$$

APPENDIX 4: Elastic Modulus

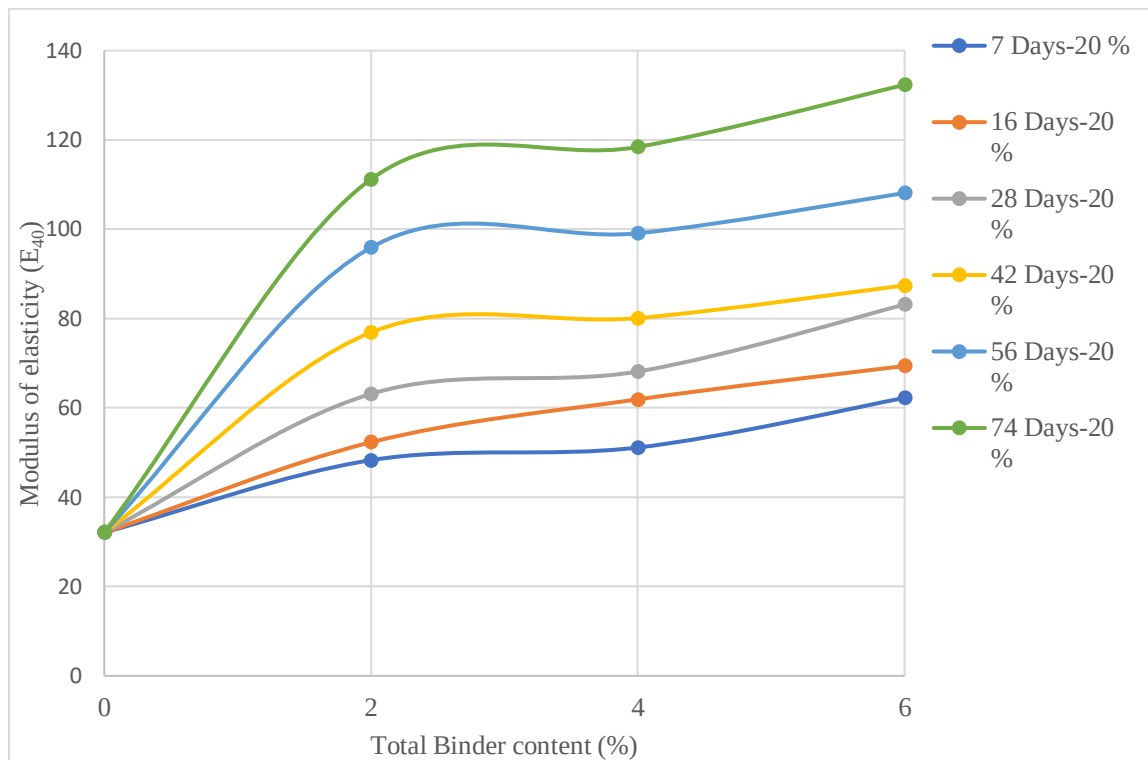


Figure 4.27 b: Effect of total binder percent on the modulus of elasticity E_{40} under CC1 at CS/lime ratio of 20 % (4:1)

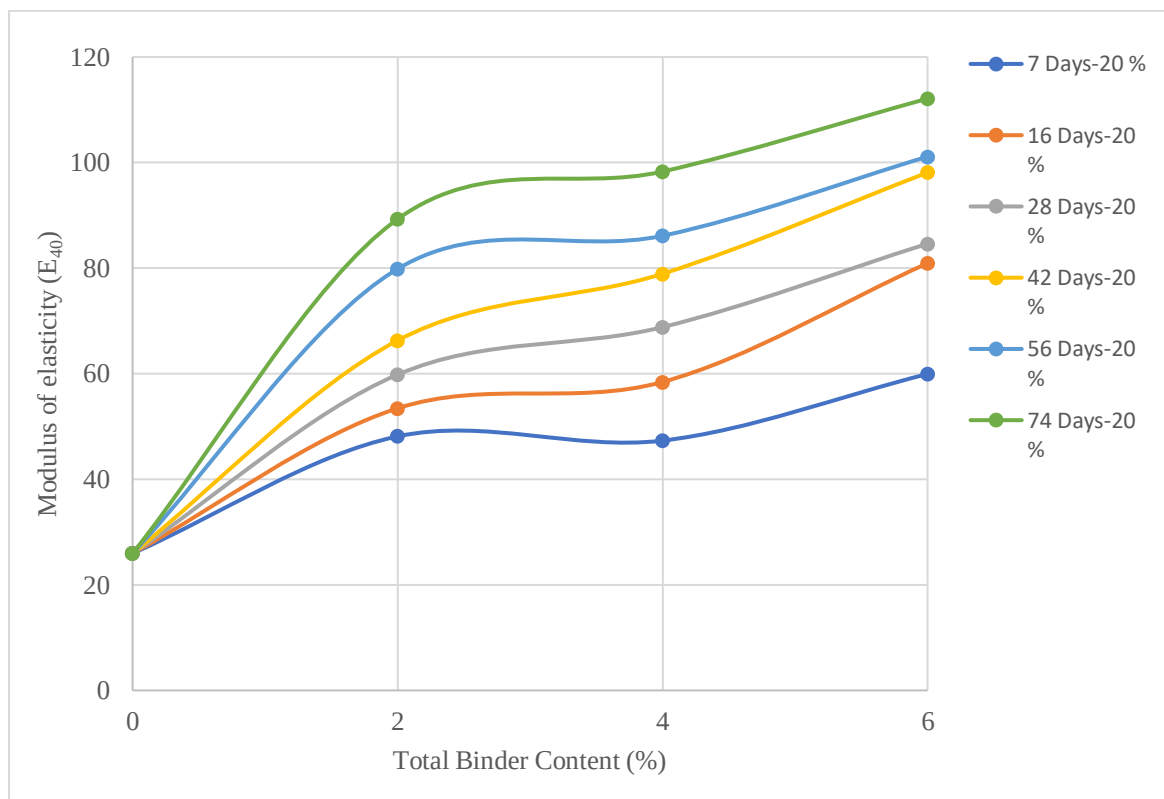


Figure 4.28 b: Effect of total binder percent on the modulus of elasticity E_{40} under CC2 at CS/lime ratio of 20 % (4:1)

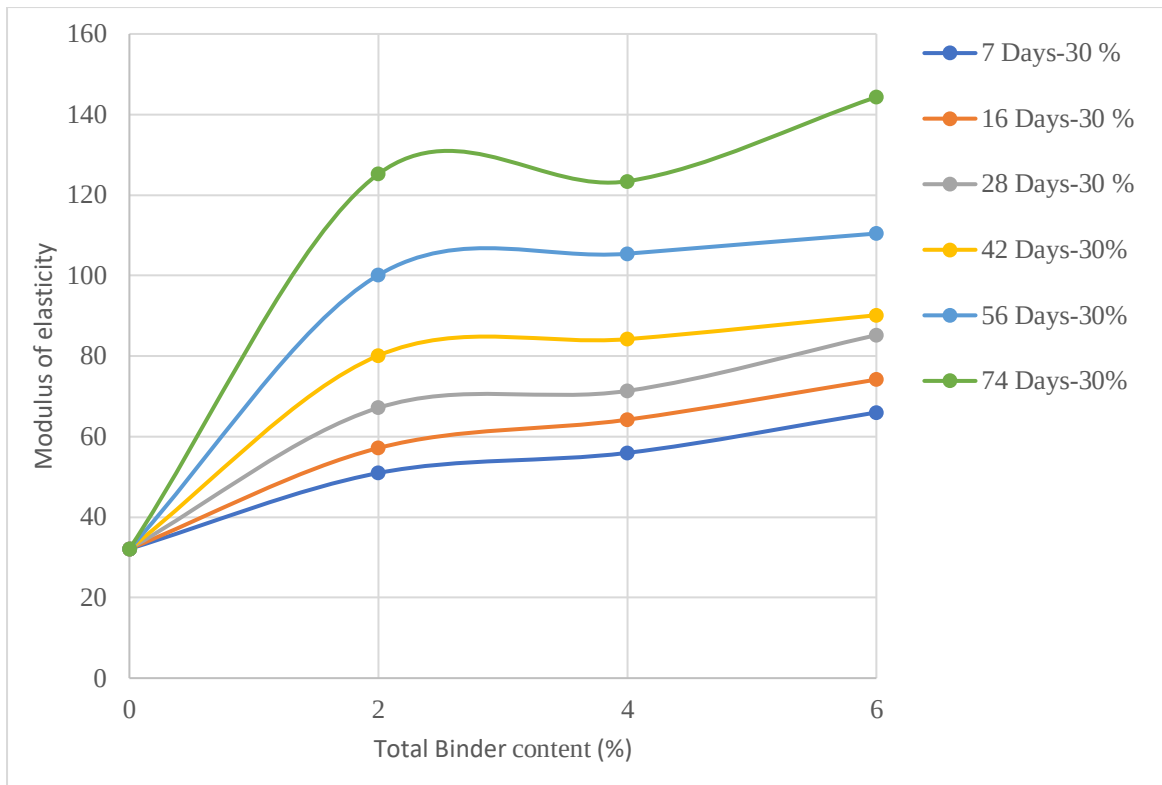


Figure 4.27 c: Effect of total binder percent on the modulus of elasticity E_{40} under CC1 at CS/lime ratio of 30 % (7:3)

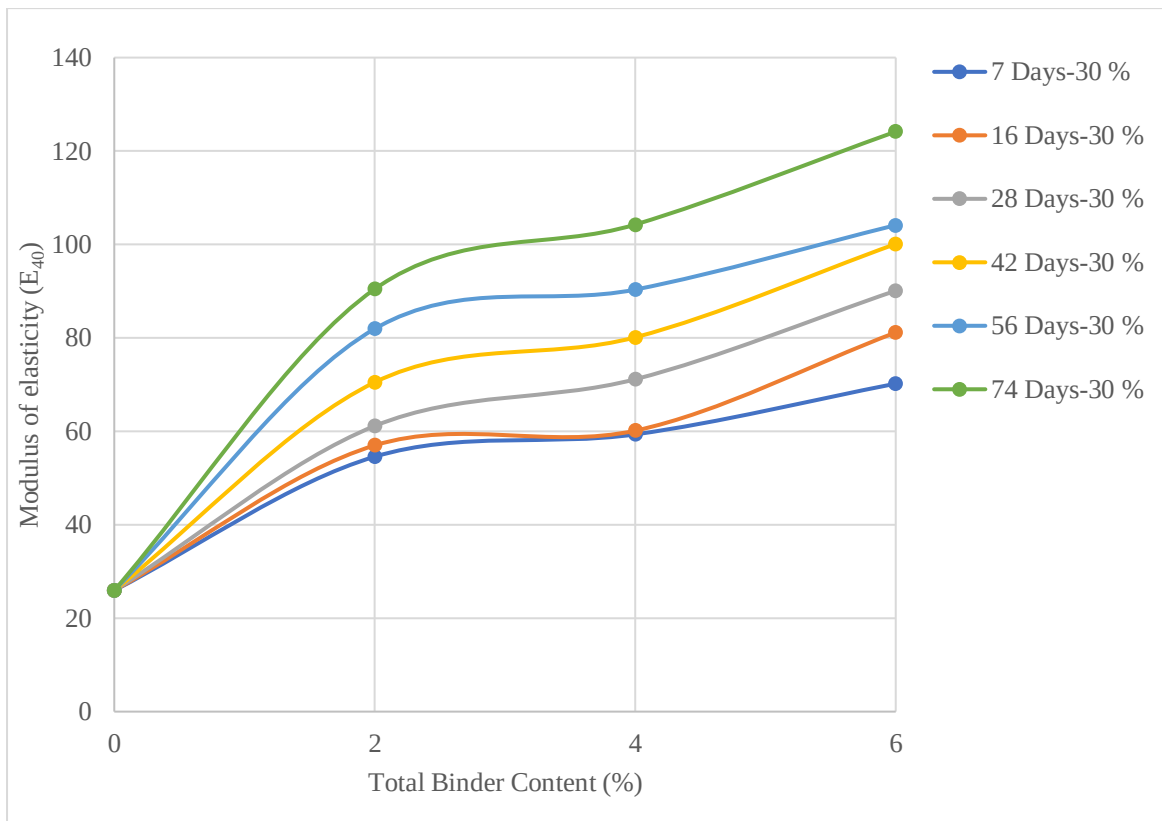


Figure 4.28 c: Effect of total binder percent on the modulus of elasticity E_{40} under CC2 at CS/lime ratio of 30 % (7:3)

APPENDIX 5: SEM DATA FOR THE DRY SPECIMEN (Y1, Y2 and Y3)

Y1	Atomic O K	Atomic Na K	Atomic Mg K	Atomic Al K	Atomic Fe K	Atomic Si K	Atomic S K	Atomic K K	Atomic Ca K	Atomic % Ti Ti K
	O (%)	Na (%)	Mg (%)	Al (%)	Fe (%)	Si (%)	S (%)	K (%)	Ca (%)	Ti (%)
POINT 1	24.14	-0.42	3.41	15.55	6.27	40.49	0.08	0.09	10.38	0.03
POINT 2	35.42	0.23	2.33	9.41	2.36	43.46	0.05	0.06	6.42	0.02
POINT 3	64.52	1.04	2.18	0.14	4.42	13.22	0.02	10.01	4.39	0.04
POINT 4	63.96	-0.35	4.69	10.42	3.11	15.41	0.06	0.42	15.95	0
POINT 5	57.24	-0.26	2.1	6.13	1.54	26.9	0.03	0.11	5.94	0.01
POINT 6	51.43	0	2.71	8.01	1.11	28.21	0.01	0.2	8.49	0.05
POINT 7	50.11	-0.25	1.49	13.14	0.41	25.21	0.07	0.41	9.14	0.02
POINT 8	55.32	-0.36	1.56	18.43	0.95	16.35	0.06	0.23	7.11	0.03
POINT 9	61.08	-0.4	2.19	16.21	0.59	8.51	0.09	0.15	11.12	0.06
POINT10	46.95	0.36	0.82	20.59	0.12	27.53	0.04	0.18	13.43	0.04
POINT11	69.61	0	1.34	4.14	0.97	10.98	0	0	12.94	0.02
POINT12	57.42	0	1.79	2.03	0.66	19.32	0	0	18.78	0
POINT13	72.14	0	0.66	0.74	0.21	13.24	0	0	13.01	0
POINT14	68.19	0	1.52	1.44	1.21	12.41	0	0	15.19	0.04
POINT15	49.96	0	6.97	4.11	1.66	19.65	0	0	17.65	0
POINT16	64.54	0	1.94	1.79	1.09	16.29	0	0	14.35	0
POINT17	52.77	0	10.94	0.44	0.63	17.51	0	0	17.71	0
POINT18	58.11	0	1.09	1.04	0.71	20.25	0	0	18.8	0
POINT19	70.61	0	0.52	0.43	0.22	15.22	0	0	13	0
POINT20	61.9	0	1.62	1.01	1.04	18.11	0	0	16.32	0
POINT21	66.7	0	1.47	0.92	1.1	15.11	0	0	14.7	0
POINT22	59.92	0	1.75	1.6	1.13	17.85	0	0	17.75	0
POINT23	47.99	0	2.11	4.16	3.14	20.47	0	0	22.13	0
POINT24	65.25	0	0.93	0.69	1.01	15.06	0	0	17.06	0
POINT25	60.71	0	1.11	1.63	1.07	18.52	0	0	16.96	0
POINT26	50.49	0	1.59	1.94	1.03	22.4	0	0	22.55	0

Y2	Atomic O K	Atomic Na K	Atomic Mg K	Atomic Al K	Atomic Fe K	Atomic Si K	Atomic S K	Atomic K K	Atomic Ca K	Atomic % Ti Ti K
	O (%)	Na (%)	Mg (%)	Al (%)	Fe (%)	Si (%)	S (%)	K (%)	Ca (%)	Ti (%)
POINT1	20.44	0.09	4.5	0.94	0.43	16.21	0	0	57.39	0
POINT2	21.5	0	2.14	0.62	0.21	14.11	0	0	61.42	0
POINT3	30.45	0.02	1.02	0.42	0.1	15.49	0	0	52.52	0
POINT4	49.5	0.01	0.95	0.31	0.09	20.15	0	0	28.99	0
POINT5	15.43	0	0.61	0.27	0.06	10.01	0	0	73.62	0
POINT6	13.95	0	0.43	0.19	0.04	13.39	0	0	72	0
POINT7	39.66	0	1.66	1.76	0.41	27.18	0	0	29.19	0.14
POINT8	46.74	0	1.4	2.61	1.08	20.05	0	0	27.64	0.48
POINT9	64.14	0	1.62	0.73	0.94	10.55	0	0	22.02	0
POINT10	69.92	0	0.94	0.23	0.43	12.1	0	0	16.38	0
POINT11	44.56	0	0.01	0.48	0.33	200.11	0	0	33.91	0
POINT12	47.98	0	0.96	0.58	0.46	19.08	0	0	30.94	0
POINT13	70.69	0	0.44	0.32	0.11	15.11	0	0	13.33	0
POINT14	62.19	0	0.11	0.99	0.43	17.14	0	0	19.14	0
POINT15	49.9	0	0.31	0.08	0.54	19	0	0	30.17	0
POINT16	58.76	0	0.51	0.33	0.11	15.9	0	0	24.39	0
POINT17	50.99	0	1.09	0.21	0.08	20.15	0	0	27.48	0
POINT18	66.44	0	0.47	0.12	0.41	15.24	0	0	17.32	0
POINT19	71.11	0	0.09	0.2	0.1	13.36	0	0	15.24	0
POINT20	39.48	0	1.46	1.49	1.06	27.25	0	0	29.26	0

Y3	Atomic OK	Atomic Na K	Atomic Mg K	Atomic Al K	Atomic Fe K	Atomic Si K	Atomic SK	Atomic KK	Atomic Ca K	Atomic Ti K
	O (%)	Na (%)	Mg (%)	Al (%)	Fe (%)	Si (%)	S (%)	K (%)	Ca (%)	Ti (%)
POINT1	60.55	0	1.09	2.64	21.6	11.87	0.04	0.39	4.79	0.03
POINT2	63.5	0	1.01	2.86	19.37	11.77	0.02	0.36	2.12	0.01
POINT3	63.56	0	1.4	2.96	19.2	12.07	0.01	0.36	5.75	0
POINT4	62.96	0	0.95	2.59	20.36	11.78	0.3	0.39	7.88	0
POINT5	63.92	0	0.85	2.84	18.99	12.26	0.02	0	9.98	0
POINT6	64.17	0	0.64	2.37	19.52	11.6	0.5	0.51	8.78	0
POINT7	49.91	0	2.43	3.46	21.43	14.52	0.4	0.31	7.94	0
POINT8	54.53	0	2.91	2.68	20.11	10.11	0	0	10.34	0
POINT9	59.12	0	0.99	1.34	13.11	14.12	0	0	11.33	0
POINT10	65.41	0	1.08	1.43	7.84	11.49	0.02	0	12.73	0
POINT11	56.11	0	3.11	4.12	9.41	15.96	0	0	11.29	0
POINT12	45.63	0	2.33	2.43	20.96	20.49	0	0	10.59	0
POINT13	48.56	0	3.14	2.53	18.11	16	0	0	11.66	0
POINT14	49.95	0	2.39	4.58	12.52	16.56	0	0	14	0
POINT15	60.44	0	1.96	1.81	9.59	17.1	0	0	9.1	0
POINT16	61.47	0	0.84	1.07	1.11	19.58	0	0	16.66	0.1
POINT17	58.55	0	0.66	1.1	0.94	21.56	0	0	17.99	0
POINT18	57.43	0	1.09	2.78	3.86	16.03	0.01	0.05	18.55	0.2
POINT19	52.42	0	0.43	2.02	1.99	22.68	0	0.02	20.44	0
POINT20	51.33	0	0.23	1.11	2.54	19.98	0.31	0	17.12	0.03

APPENDIX 6: SEM DATA FOR HYDRATED SPECIMEN (B1, B2, B3 and B4)

B1	Atomic O K	Atomic Na K	Atomic Mg K	Atomic Al K	Atomic Si K	Atomic S K	Atomic K K	Atomic Ca K	Atomic Ti K	Atomic Mn K	Atomic Fe K
	O (%)	Na (%)	Mg (%)	Al (%)	Si (%)	S (%)	K (%)	Ca (%)	Ti (%)	Mn (%)	Fe (%)
POINT1	49.77	0	4.46	6.91	16.21	0.02	0.44	18.87	0.1	0.64	2.1
POINT2	54.02	0	1.56	3.16	7.19	0.01	0	31.95	0.16	0.34	1.6
POINT3	61.42	0	3.45	5.11	14.91	0.03	0	12.11	0.41	0.99	2.41
POINT4	41.96	0	8.49	2.92	17.11	0.05	0	26.92	0.05	0.51	1.71
POINT5	68.61	0	2.93	4.69	10.94	0.1	0.09	9.94	0.03	0.31	2.41
POINT6	56.84	0	1.99	3.29	15.56	0	0.04	20.04	0.19	0.29	1.66
POINT7	62.31	0	2.99	1.9	14.93	0	0	16.12	0	0.05	1.7
POINT8	71.04	0	2.1	1.11	13.11	0	0	11.4	0.04	0.1	1.1
POINT9	45.19	0	1.96	2.12	22.45	0	0	25.72	0.03	0.42	2.11
POINT10	61.86	0	1.11	0.99	13	0	0	21.87	0.1	0.11	0.96
POINT11	46.49	0	0.91	0.85	24.11	0	0	26.02	0.4	0.09	1.12
POINT12	63.11	0	0.59	0.46	14.52	0	0	20	0.31	0.07	0.94
POINT13	57.58	0	0.49	0.51	19.02	0	0	21.3	0.23	0.06	0.81
POINT14	50.43	0	0.71	0.9	20.49	0	0	26.82	0.13	0.01	0.51
POINT15	44.51	0	2.13	0.14	23.6	0	0	25.72	0.19	0.12	1.59

B2	Atomic O K	Atomic Na K	Atomic Mg K	Atomic Al K	Atomic Si K	Atomic S K	Atomic K K	Atomic Ca K	Atomic Ti K	Atomic Mn K	Atomic Fe K
	O (%)	Na (%)	Mg (%)	Al (%)	Si (%)	S (%)	K (%)	Ca (%)	Ti (%)	Mn (%)	Fe (%)
POINT1	47.9	0	1.86	6.98	24.14	0.02	0.05	16.19	0.21	0.14	2.56
POINT2	44.27	0	1.59	7.84	25.43	0	0.02	19.56	0.11	0.24	0.96
POINT3	63.19	0	1.03	4.19	16.92	0.01	0	10.46	0.59	0.54	2.97
POINT4	62.54	0	1.06	3.19	17.94	0.04	0.04	14.16	0.43	0.2	0.4
POINT5	56.14	0	0.94	1.61	17.81	0.03	0	21.95	0.29	0.53	0.7
POINT6	53.91	0	0.96	5.19	19.14	0	0.06	18.11	0.51	0.46	1.66
POINT7	70.11	0	0.54	1.06	4.12	0.05	0.04	20.98	0.34	0.66	2.11
POINT8	62.21	0	0.41	6.12	15.41	0.06	0.2	14.42	0.49	0.65	1.03
POINT9	43.49	0	0.76	12.19	23.14	0	0.4	17.95	0.47	0.53	2.07
POINT10	58.18	0	0.59	9.92	19.2	0	0	11.08	0.21	0.32	0.5
POINT11	49.67	0	0.91	14.29	20.93	0	0	12.1	0.23	0.2	1.67
POINT12	57.61	0	0.31	8.11	17.4	0	0	14.41	9.14	0.91	1.11
POINT13	60.12	0	0.66	5.16	14.2	0	0	16.94	0.51	0.62	1.79
POINT14	54.21	0	0.77	10.12	13.29	0	0	18.65	0.68	0.41	1.87
POINT15	45.91	0	0.93	16.41	16.94	0	0	16.42	0.49	0.8	2.1
POINT16	50.19	0	0.44	11.66	14.1	0	0	20.74	0.65	0.9	1.32
POINT17	49.61	0	0.1	15.02	19.42	0	0	13.79	0.41	0.34	1.31
POINT18	48.92	0	0.29	13.19	18.58	0	0	17.52	0.22	0.14	1.14
POINT19	68.11	0	0.52	7.98	13.62	0	0	8.41	0.01	0.09	1.26
POINT20	64.19	0	0.24	6.12	12.91	0	0	13.16	0.1	0.53	2.75

B3	Atomic O K	Atomic Na K	Atomic % Mg K	Atomic Al K	Atomic Si K	Atomic S K	Atomic K K	Atomic Ca K	Atomic Ti K	Atomic Mn K	Atomic Fe K
	O (%)	Na (%)	Mg (%)	Al (%)	Si (%)	S (%)	K(%)	Ca (%)	Ti (%)	Mn (%)	Fe (%)
POINT1	80.59	0	0.69	0.58	6.52	0	0.01	10.41	0.04	0.15	1.01
POINT2	69.12	0	1.01	0.21	9.46	0	0	19.16	0.02	0.1	0.92
POINT3	49.63	0	5.19	3.43	19.99	0	0	20.44	0.21	0.08	1.03
POINT4	70.62	0	2.13	5.16	6.98	0	0	14.12	0.11	0.13	0.75
POINT5	52.19	0	6.14	0.51	19.3	0	0	19.4	0.2	0.12	2.14
POINT6	56.91	0	0.61	0.94	20.12	0	0	20	0.18	0.11	1.13
POINT7	62.64	0	1.03	1.08	17.1	0	0	17.02	0.14	0.09	0.9
POINT8	66.11	0	0.34	0.8	15.21	0	0	16.21	0.17	0.06	1.1
POINT9	64.61	0	0.45	0.64	16.64	0	0	17.42	0.09	0	0.15
POINT10	43.41	0	0.57	0.48	26.97	0	0	27.37	0.1	0.08	1.12
POINT11	20.63	0	0.21	0.53	14.15	0	0	13.72	0.05	0.1	0.61
POINT12	54.91	0	0.72	0.69	21.87	0	0	21.4	0	0	0.41
POINT13	58.66	0	1.01	1.09	20.06	0	0	18.97	0	0	0.21
POINT14	65.49	0	2.1	0.69	14.04	0	0	16.9	0	0	0.51
POINT15	71.42	0	1.04	0.43	13.12	0	0	13.32	0	0	0.67
POINT16	57.48	0	0.9	1.08	20.06	0	0	20.05	0	0	0.43
POINT17	50.99	0	1.19	2.11	22.02	0	0	23.38	0	0	0.31
POINT18	60.52	0	0.66	0.78	19.89	0	0	17.89	0	0	0.26
POINT19	66.92	0	0.79	1.03	14.5	0	0	16.56	0	0	0.2
POINT20	49.98	0	2.14	3.12	23.21	0	0	21.37	0	0	0.18
POINT21	72.82	0	0.42	0.41	12.01	0	0	14.13	0	0	0.22
POINT22	62.77	0	0.1	0.14	15.43	0	0	29.42	0	0	2.14
POINT23	59.17	0	0.06	0.11	16.91	0	0	22.19	0	0	1.56
POINT24	57.95	0	0.13	0.09	20.93	0	0	18.94	0	0	1.96
POINT25	51.81	0	0.03	0.05	23.12	0	0	23.13	0	0	1.86

B4	Atomic O K	Atomic Na K	Atomic Mg K	Atomic Al K	Atomic Si K	Atomic S K	Atomic K K	Atomic Ca K	Atomic Ti K	Atomic Mn K	Atomic Fe K
	O (%)	Na (%)	Mg (%)	Al (%)	Si (%)	S (%)	K(%)	Ca (%)	Ti (%)	Mn (%)	Fe (%)
POINT1	57.69	0	0.26	7.99	19.16	0	0	12.96	0.21	0.18	1.42
POINT2	67.1	0.01	0.18	0.4	0.94	0	0	30.41	0.11	0.1	0.75
POINT3	70.71	0	0.34	0.31	0.69	0	0	27.51	0.09	0.21	0.14
POINT4	68.62	0	0.29	0.27	10.31	0	0	17.88	0.1	0.11	2.42
POINT5	72.91	0	0.5	1.04	10.99	0	0	12.04	0.4	0.16	1.46
POINT6	62.65	0	1.56	6.9	14.37	0	0	12.06	0.21	0.14	2.11
POINT7	59.81	0	2.42	7.72	16.42	0	0	11.22	0.12	0.17	2.12
POINT8	34.92	0	0.72	4.32	30.92	0	0	27.43	0.13	0.12	1.44
POINT9	46.19	0	0.95	0.99	25.06	0	0	25.24	0.11	0.1	1.36
POINT10	55.96	0	2.5	5.61	16.9	0	0	17.25	0.34	0.22	1.22
POINT11	64.14	0	4.05	3.98	10.14	0	0	16.11	0.2	0.19	1.09
POINT12	66.78	0	1.09	1.43	14.51	0	0	14.71	0.29	0.17	1.02
POINT13	69.46	0	1.02	1.21	13.41	0	0	13.4	0.26	0.13	1.11
POINT14	54.94	0	1.52	1.94	19.98	0	0	19.96	0.23	0.08	1.31
POINT15	70.19	0	0.61	0.78	13.53	0	0	13.69	0.2	0.04	0.96
POINT16	52.79	0	1.11	1.52	21.57	0	0	21.29	0.19	0.12	1.41
POINT17	80.11	0	0.11	0.41	8.99	0	0	8.61	0.18	0.16	1.43
POINT18	68.74	0	0.93	0.51	14.02	0	0	13.9	0.14	0.15	1.61
POINT19	50.99	0	1.08	2.14	22.06	0	0	21.86	0.1	0.41	1.36
POINT20	63.74	0	1.01	1.22	16.21	0	0	16.14	0.11	0.4	1.17

APPENDIX 7: ATOMIC RATIOS SEM DATA FOR STABILIZED DRY COMPOSITE
(Y1, Y2 and Y3)

Y1	Si/Ca	Al/Ca	Si/Al	Al/Si	Mg/Si	(Mg+Al+Fe+Ti+Mn)/Si	Ca/Si	Ca/Si + Al
POINT1	3.9008	1.4981	2.6039	0.384	0.0842	0.6239	0.2564	0.1852
POINT2	6.7695	1.4657	4.6185	0.2165	0.0536	0.3311	0.1477	0.1214
POINT3	3.0114	0.0319	94.429	0.0106	0.1649	0.5129	0.3321	0.3286
POINT4	0.9661	0.6533	1.4789	0.6762	0.3043	1.1823	1.035	0.6175
POINT5	4.5286	1.0319	4.3883	0.2279	0.0781	0.3636	0.2208	0.1798
POINT6	3.3227	0.9435	3.5218	0.2839	0.0961	0.4211	0.3009	0.2344
POINT7	2.7582	1.4376	1.9186	0.5212	0.0591	0.5974	0.3626	0.2383
POINT8	2.2996	2.5921	0.8871	1.1272	0.0954	1.2826	0.4349	0.2044
POINT9	0.7653	1.4577	0.5249	1.9048	0.2573	2.2385	1.3067	0.4498
POINT10	2.0499	1.5331	1.3371	0.7479	0.0795	0.7835	0.4878	0.2791
POINT11	0.8485	0.3199	2.6522	0.377	0.122	0.5893	1.1785	0.8558
POINT12	1.0288	0.1081	9.5172	0.1051	0.0927	0.2319	0.972	0.8796
POINT13	1.0177	0.0569	17.892	0.0559	0.0498	0.1216	0.9826	0.9306
POINT14	0.8169	0.0949	8.6181	0.116	0.1225	0.336	1.224	1.0968
POINT15	1.1133	0.2329	4.781	0.2092	0.3547	0.6483	0.8982	0.7428
POINT16	1.1352	0.1247	9.1006	0.1099	0.1191	0.2959	0.8809	0.7937
POINT17	0.9887	0.0356	39.796	0.0251	0.6248	0.6859	0.9806	0.9866
POINT18	1.0771	0.0553	19.471	0.0514	0.0538	0.1402	0.9284	0.883
POINT19	1.1708	0.0331	35.395	0.0283	0.0342	0.0769	0.8541	0.8307
POINT20	1.1097	0.0619	17.931	0.0558	0.0895	0.2027	0.9012	0.8536
POINT21	1.0278	0.0626	16.424	0.0609	0.0973	0.2309	0.9729	0.917
POINT22	1.0056	0.0901	11.156	0.0896	0.098	0.2509	0.9944	0.9126
POINT23	0.9249	0.1879	4.9207	0.2032	0.1031	0.4597	1.0811	0.8985
POINT24	0.8828	0.0404	21.826	0.0458	0.0618	0.1746	1.1328	1.0832
POINT25	1.0919	0.0961	11.362	0.088	0.0599	0.2057	0.9158	0.8417
POINT26	0.9933	0.086	11.546	0.0866	0.0709	0.2036	1.0067	0.9265

Y2	Si/Ca	Al/Ca	Si/Al	Al/Si	Mg/Si	(Mg+Al+Fe+Ti+Mn)/Si	Ca/Si	Ca/Si + Al
POINT1	0.2825	0.0164	17.245	0.0579	0.2776	0.3621	3.5404	3.3464
POINT2	0.2297	0.0101	22.758	0.0439	0.1517	0.2105	4.3529	4.1697
POINT3	0.2949	0.0079	36.881	0.0271	0.0658	0.0994	3.3906	3.3011
POINT4	0.6951	0.0107	65	0.0154	0.0471	0.0669	1.4387	1.4169
POINT5	0.1359	0.0037	37.074	0.0269	0.0609	0.0939	7.3546	7.1615
POINT6	0.1859	0.0026	70.474	0.0142	0.0321	0.0493	5.3771	5.3019
POINT7	0.9311	0.0003	15.443	0.0648	0.0611	0.1461	1.0739	1.0086
POINT8	0.7254	0.0094	7.6819	0.1302	0.0698	0.2539	1.3786	1.2198
POINT9	0.4791	0.0332	14.452	0.0692	0.1536	0.1494	2.0872	1.9521
POINT10	0.7387	0.014	52.609	0.019	0.0777	0.1322	1.3537	1.3285
POINT11	0.593	0.0142	41.896	0.0239	0.0303	0.0706	1.6862	0.1691
POINT12	0.6167	0.0187	32.897	0.0304	0.0503	0.1048	1.6216	1.5738
POINT13	1.1335	0.024	47.219	0.0212	0.0291	0.576	0.8822	0.8639
POINT14	0.8955	0.0517	17.313	0.0576	0.0064	0.0893	1.1167	1.0557
POINT15	0.6298	0.0027	237.5	0.0042	0.0163	0.0489	1.5879	1.5812
POINT16	0.6519	0.0135	48.182	0.0208	0.0321	0.0597	1.5339	1.5028
POINT17	0.7333	0.0076	95.952	0.0104	0.0541	0.0685	1.3678	1.3497
POINT18	0.8799	0.0069	127	0.0079	0.0308	0.0656	1.1365	1.1276
POINT19	0.8701	0.0131	66.3	0.0151	0.0068	0.0294	1.1493	1.1239
POINT20	0.9313	0.0509	18.289	0.0547	0.0536	1.472	1.0738	1.0181

Y3	Si/Ca	Al/Ca	Si/Al	Al/Si	Mg/Si	(Mg+Al+Fe+Ti+Mn)/Si	Ca/Si	Ca/ Si + Al
POINT1	2.4781	0.5511	4.4962	0.2224	0.0918	2.1365	0.4035	0.3301
POINT2	5.5519	1.3491	4.1154	0.2429	0.0858	1.9754	0.1801	0.1449
POINT3	2.0991	0.5148	4.0777	0.2452	0.1159	1.9519	0.4764	0.3826
POINT4	1.949	0.3287	4.5443	0.2198	0.0806	2.0289	0.6689	0.5484
POINT5	1.2285	0.2846	4.3169	0.2316	0.0693	1.8499	0.814	0.6609
POINT6	1.3212	0.2699	4.8945	0.2043	0.0552	1.9422	0.7569	0.6285
POINT7	1.8287	0.4358	4.1965	0.2383	0.1674	1.8815	0.5468	0.4416
POINT8	0.9778	0.2591	3.7724	0.2651	0.2878	2.542	1.0227	0.8084
POINT9	1.2462	0.1183	10.534	0.0949	0.0701	1.0935	0.8024	0.7329
POINT10	0.9026	0.1123	8.0349	0.1245	0.0939	0.9008	1.1079	0.9853
POINT11	1.4136	0.3649	3.8738	0.2581	0.1949	1.0426	0.7074	0.5623
POINT12	1.9348	0.2295	8.4321	0.1186	0.1137	1.2552	0.5168	0.462
POINT13	1.3722	0.2169	6.3241	0.1581	0.1963	1.4863	0.7288	0.6292
POINT14	1.1829	0.3271	3.6157	0.2766	0.1443	1.1793	0.8454	0.6623
POINT15	1.8791	0.1989	9.4475	0.1058	0.1146	0.7813	0.5322	0.4812

APPENDIX 8: ATOMIC RATIOS SEM DATA FOR HYDRATED COMPOSITE (B1, B2, B3 and B4)

B1					
Si/Ca	Al/Si	Mg/si	(Mg+Al+Fe+Ti+Mn+Ca)/Si	Ca/Si	Ca/Si + Al
0.859	0.4263	0.2751	2.0407	1.1641	0.8162
0.225	0.4395	0.2169	5.3922	4.4437	3.087
1.2312	0.4219	0.2314	1.6419	0.8122	0.6049
0.6356	0.1707	0.4962	2.3729	0.9889	1.344
1.1006	0.4287	0.2678	1.8565	0.9086	0.636
0.7764	0.2114	0.1279	1.7648	1.2879	1.0631
0.9262	0.1273	0.2003	1.5244	1.0797	0.9578
1.15	0.0847	0.1602	1.209	0.8696	0.8017
0.8729	0.0944	0.0873	1.4414	1.1457	1.0468
0.5944	0.0762	0.0508	1.9338	1.6823	1.5633
0.9266	0.0353	0.0377	1.2189	1.0792	1.0425
0.726	0.0317	0.0406	1.5406	1.3774	1.3351
0.8929	0.0268	0.0258	1.2303	1.1199	1.0906
0.7639	0.0439	0.0347	1.4192	1.3089	1.2539
0.9176	0.0908	0.0903	1.3513	1.0898	1.0834

B2					
Si/Ca	Al/Si	Mg/Si	(Mg+Al+Fe+Ti+Mn+Ca)Si	Ca/Si	Ca/Si + Al
1.491	0.2891	0.0771	1.1574	0.6707	0.5202
1.3001	0.3083	0.0625	1.1915	0.7692	0.5879
1.6176	0.2476	0.0609	1.169	0.6182	0.4955
1.2669	0.1778	0.0591	1.0836	0.7893	0.6701
0.8114	0.0904	0.0528	1.4609	1.2325	1.1303
1.0569	0.2712	0.0502	1.4049	0.9462	0.7443
0.1964	0.2573	0.1311	6.2354	5.0922	4.0502
1.0687	0.3971	0.0266	1.5003	0.9358	0.6698
1.2891	0.5268	0.0328	1.4248	0.7757	0.5081
1.7329	0.5167	0.0307	1.1781	0.5825	0.3805
1.7298	0.6828	0.0435	1.4047	0.5781	0.3436
1.2075	0.4661	0.0178	1.4362	0.8282	0.5649
0.8383	0.3634	0.0465	1.8141	1.1929	0.875
0.7126	0.7615	0.0579	2.4454	1.4033	0.7967
1.0317	0.9687	0.0549	2.193	0.9693	0.4924
0.6798	0.8269	0.0312	2.5326	1.4709	0.8051
1.4083	0.7734	0.0051	1.5947	0.7101	0.4004
1.0605	0.7099	0.0156	1.7549	0.9429	0.5515
1.6195	0.5859	0.0382	1.3414	0.6175	0.3894
0.981	0.4741	0.0186	1.7738	1.0194	0.6915

B3					
Si/Ca	Al/Si	Mg/Si	(Mg+Al+Fe+Ti+Mn+Ca)/Si	Ca/Si	Ca/Si + Al
0.6263	0.0889	0.1058	1.9755	1.5966	1.4662
0.4937	0.0222	0.1068	2.2643	2.0254	1.9814
0.9779	0.1716	0.2596	1.5198	1.0225	0.8728
0.4943	0.7393	0.3052	3.2092	2.0229	1.1631
0.9948	0.0264	0.3181	1.4772	1.0052	0.9793
1.006	0.0467	0.0303	1.1933	0.994	0.9497
1.0047	0.0632	0.0602	1.1848	0.9953	0.9362
0.9383	0.0526	0.0224	1.2281	1.0657	1.0125
0.9552	0.0385	0.027	1.1268	1.0469	1.0081
0.9854	0.0178	0.0211	1.0983	1.0148	0.9971
1.0313	0.0375	0.0148	1.0756	0.9696	0.9346
1.0219	0.0316	0.0329	1.0617	0.9785	0.9486
1.0575	0.0543	0.0503	1.0608	0.9457	0.8969
0.8308	0.0684	0.1496	1.4579	1.2037	1.1473
0.9849	0.0328	0.0793	1.1784	1.1052	0.983
1.0005	0.0538	0.0449	1.1196	0.9995	0.9484
0.9418	0.0958	0.054	1.2257	1.0618	0.9689
1.1118	0.0392	0.0332	0.9849	0.8994	0.8655
0.8756	0.071	0.0545	1.2814	1.1421	1.0663
1.0861	0.1344	0.0922	1.1551	0.9207	0.8116
0.8506	0.0341	0.0349	1.2639	1.1757	1.1377
0.7445	0.0091	0.0065	1.4128	1.2586	1.8895
0.7621	0.0065	0.0035	1.4145	1.3122	1.3038
1.1051	0.0043	0.0062	1.0091	0.9049	0.901
0.9996	0.0022	0.0013	1.0843	1.0004	0.9983

B4					
Si/Ca	Al/Si	Mg/Si	(Mg+Al+Fe+Ti+Mn+Ca)/Si	Ca/Si	Ca/Si +Al
1.4784	0.417	0.0136	1.2015	0.6764	0.4773
0.0309	0.4255	0.1915	33.9894	32.3511	22.694
0.0251	0.4493	0.4928	41.4493	39.8696	27.51
0.5766	0.0262	0.0281	2.0436	1.7342	1.69
0.9128	0.0946	0.0455	1.4649	1.0955	1.0008
1.1915	0.4802	0.1086	1.5992	0.8392	0.567
1.4635	0.4702	0.1474	1.4476	0.6833	0.4648
1.1272	0.1397	0.0233	1.1048	0.8871	0.7784
0.9929	0.0395	0.0379	1.1472	1.0072	0.9689
0.9797	0.3319	0.1479	1.6059	1.0207	0.7663
0.6294	0.3925	0.3994	2.5365	1.5888	1.1409
0.9864	0.0986	0.0751	11.0462	1.0138	0.9228
1.0007	0.0902	0.0761	1.2774	0.9993	0.9166
1.001	0.0971	0.0781	1.2553	0.9989	0.9106
0.9883	0.0576	0.0451	1.2033	1.0118	0.9567
1.0132	0.0705	0.0515	1.1887	0.987	0.922
1.0441	0.0456	0.0122	1.2125	0.9577	0.916
1.0086	0.0364	0.0663	1.2297	0.9914	0.9566
1.0091	0.097	0.0489	1.2217	0.9909	0.9033
1.0043	0.0753	0.0623	1.2369	0.9957	0.926

APPENDIX 9: NATURAL SOIL, HYDRATED LIME AND HYDRATED SAMPLE PEAKS

Mineral	Strong Peaks	Medium Peaks	Weak Peaks
Kaolinite	7.15 Å (12.365°)	4.41 Å (20.140°)	2.79 Å (32.053°)
	4.46 Å (19.891°)	4.26 Å (20.846°)	
		4.04 Å (21.989°)	
		3.78 Å (23.488°)	
		3.58 Å (24.877°)	
Montmorillonite	15.00 Å (5.887°)	4.30 Å (20.632°)	3.33 Å (26.672°)
	7.50 Å (11.790°)	4.29 Å (20.645°)	
	5.00 Å (17.725°)	3.85 Å (23.068°)	
	4.49 Å (19.757°)	3.75 Å (23.707°)	
	4.48 Å (19.770°)		
Illite	10.02 Å (8.816°)	4.39 Å (20.169°)	
	5.01 Å (17.685°)	4.31 Å (20.566°)	
	4.51 Å (19.672°)	4.24 Å (20.908°)	
	4.50 Å (19.700°)	4.11 Å (21.593°)	
	4.47 Å (19.819°)	3.98 Å (22.308°)	
		3.89 Å (22.834°)	
Quartz	4.25 Å (20.849°)	3.34 Å (26.624°)	2.45 Å (36.528°)
			2.28 Å (39.442°)
			2.23 Å (40.271°)
			2.12 Å (42.432°)
Calcite	3.02 Å (29.466°)	2.83 Å (31.516°)	2.09 Å (43.243°)
		2.49 Å (36.038°)	1.92 Å (47.212°)
Hydrated Lime	4.90 Å (18.059°)	2.62 Å (34.093°)	1.92 Å (47.129°)
	3.11 Å (28.670°)	2.45 Å (36.587°)	1.79 Å (50.789°)
Hydrated Sample	3.34 Å (26.603°)	10.03 Å (8.817°)	4.91 Å (18.048°)
	3.85 Å (23.048°)	10.06 Å (8.780°)	2.63 Å (34.003°)
	3.04 Å (29.261°)	10.00 Å (8.832°)	
	4.25 Å (20.849°)	11.14 Å (7.930°)	
	4.79 Å (18.490°)	14.36 Å (6.147°)	
	7.16 Å (12.349°)	14.40 Å (6.132°)	
	7.15 Å (12.365°)	15.01 Å (5.888°)	
		21.49 Å (4.107°)	
		9.76 Å (9.048°)	

APPENDIX 10: ABBREVIATION

1 Day- natural soil- copper slag liquid limit	D1-NS-CS-LL
1 Day- natural soil-binder content-liquid limit	D1-NS-BC-LL
1 Day- natural soil-binder content-plastic limit	D1-NS-BC-PL
1 Day- natural soil-copper slag plastic limit	D1-NS-CS-PL
1 Day- natural soil-lime content-liquid limit	D1-NS-LC-LL
1 Day- natural soil-lime content-plastic limit	D1-NS-LC-PL
1 Day-natural soil-binder content-plasticity index	D1-NS-BC-PI
1 Day-natural soil-copper slag-plasticity index	D1-NS-CS-PI
1 Day-natural soil-lime content-plasticity index	D1-NS-LC-PI
3 Days- natural soil- copper slag- liquid limit	D3-NS-CS-LL
3 Days- natural soil-binder content-liquid limit	D3-NS-BC-LL
3 Days- natural soil-binder content-plastic limit	D3-NS-BC-PL
3 Days- natural soil-copper slag plastic limit	D3-NS-CS-PL
3 Days- natural soil-lime content-liquid limit	D3-NS-LC-LL
3 Days- natural soil-lime content-plastic limit	D3-NS-LC-PL
3 Days-natural soil-binder content-plasticity index	D3-NS-BC-PI
3 Days-natural soil-copper slag-plasticity index	D3-NS-CS-PI
3 Days-natural soil-lime content-plasticity index	D3-NS-LC-PI
American Association of State Highway Official	AASHTO
American society of Testing and Materials	ASTM
Binder Content	BC
Calcium aluminium hydrate	C-A-H
Calcium aluminium silicate hydrate	C-A-S-H
Calcium silicate hydrate	C-S-H
Copper slag	CS
Curing Condition 1 (35 °C with relative humidity 90 % to 100 %)	CC1
Curing Condition 2 (45 °C with relative humidity 50 % to 60 %)	CC2
Giga Pascal	GPa
Liquid Limit	LL
Maximum dry density	MDD
Natural soil	NS
Optimum Moisture Content	OMC
Plastic Limit	PL
Plasticity Index	PI
Scanning Electron Microscope	SEM
Unconfined Compressive strength	UCS
X-ray diffraction	XRD
X-ray fluorescence	XRF