Geotechnical Engineering and Coastal Engineering

Faculty of Agricultural and Environmental Sciences

# The Effect of Coffee Husk Ash on Geotechnical Properties of Expansive Soil

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Meskerem Kebede Atahu (M.Tech)

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**Gutachter:**

Prof. Dr. Fokke Saathoff, Agrar- und Umweltwissenschaftliche Fakultät, Universität Rostock

Prof. Dr. Alemayehu Gebissa, , Agrar- und Umweltwissenschaftliche Fakultät, Universität Rostock

Prof. Dr. Emer Tucay Quezon, P.Eng, Ambo Institute of Technology, Ambo University

Dr. Abrham Woldemichael, Institute of Technology, Hawassa University

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## Abstract

Expansive soil is one of the widespread typical problematic soils that poses several challenges for civil engineers. Expansive soils are hard and strong when they dry out; swell and soften when the moisture content increases. Particularly roads built on expansive soil are susceptible to early damage due to the swelling and shrinkage characteristics of this soil under changing moisture conditions. The most common technique used to improve the properties of problematic soil is stabilization using admixtures (lime and cement), which has been used to prevent and/or remediate infrastructural damage caused by expansive soils throughout the world. However, despite the global acceptance of traditional additives for treating expansive soil, there is a growing tendency to replace traditional stabilizers with industrial and agricultural by-products due to limited resources and large-scale demand for traditional admixtures. One such alternative is from coffee production that generates large amount of coffee waste. Abundant storage and inapplicability of this solid residue is resulting in disposal and environmental problems.

This study consists of experimental studies on the treatment of expansive soil using Coffee husk ash (CHA). Coffee husk is a by-product of coffee production, and CHA is the resulting ash after burning it. This study aims to evaluate the effectiveness of CHA treatment using Atterberg limits, swell potential, compaction characteristics, compressibility, unconfined compressive strength, durability (wetting-drying, W/D), California bearing ratio (CBR) and shear strength behaviors of the studied soil. In addition, further investigation was carried out to evaluate the effectiveness of CHA in combination with lime. Furthermore, changes in the mineralogical composition and microstructures of untreated and treated samples were studied using X-ray diffraction (XRD) and Scanning electron microscope (SEM), respectively. The results showed that the soil treated with CHA generally shows improvements. The addition of 20% CHA increases the bearing capacity and reduced the swelling capacity of the soil by about three-fold compared to the untreated soil. In addition, the morphological studies of the soil samples treated with 10% CHA and 15% CHA indicated the formation of hydrated particles and cementitious compounds as a result of the reaction between the soil and CHA. On the other hand the laboratory test results demonstrated that expansive soil treated with the mixture of lime and CHA is more effective compared to lime-treated soil by reducing the plasticity, swelling and improving bearing capacity. From the elemental analysis, it was observed that the concentration of calcium increased for increased CHA content. In addition, X-ray diffraction (XRD) reveals that the appearance of cementitious product on treated samples, which majorly contributed to the improvement in the geotechnical properties.

This investigation reveals the potential use of CHA for road sub-grade construction. It is not limited to the socio-economic advantages in infrastructure developments, but could also play a significant role in reducing the environmental impact arising from the storage of the waste.

## Zusammenfassung

Expansiver Boden ist einer der weit verbreiteten typischen problematischen Böden, der für Bauingenieure mehrere Herausforderungen darstellt. Expansive Böden sind hart und stark, wenn sie austrocknen; und erweichen, wenn der Feuchtigkeitsgehalt zunimmt. Vor allem Straßen, die auf expansivem Boden gebaut werden, sind auf Grund der Quellung und Schrumpfeigenschaften dieses Bodens unter wechselnden Feuchtigkeitsbedingungen anfällig für frühzeitige Schäden. Die häufigste Technik zur Verbesserung der Eigenschaften problematischer Böden ist die Stabilisierung durch Beimischungen (Kalk und Zement), die verwendet wurde, um Infrastrukturschäden zu verhindern und/oder zu beheben, die durch expansive Böden auf der ganzen Welt verursacht werden. Trotz der weltweiten Akzeptanz traditioneller Zusatzstoffe zur Behandlung von expansiven Böden wächst jedoch die Tendenz, traditionelle Stabilisatoren durch industrielle und landwirtschaftliche Nebenprodukte zu ersetzen, da die Ressourcen begrenzt sind und die Nachfrage nach traditionellen Beimischungen sehr hoch ist. Eine solche Alternative ist die Kaffeeproduktion, die eine große Menge an Kaffeeabfällen erzeugt. Die reichliche Lagerung und Unanwendbarkeit dieses festen Abfalls führt zu Entsorgungs- und Umweltproblemen.

Diese Untersuchung besteht aus experimentellen Studien zur Behandlung von expansiven Böden mit Kaffeeschalenasche (CHA). Kaffeeschale ist ein Nebenprodukt der Kaffeeproduktion, und CHA ist die resultierende Asche nach dem Verbrennen. Diese Studie zielt darauf ab, die Wirksamkeit der CHA-Behandlung an den Plastizitätsgrenzen, dem Quellungspotenzial, den Verdichtungseigenschaften, der Kompressibilität, der uneingeschränkten Druckfestigkeit, der Dauerhaftigkeit (Benetzungstrocknung), dem California Bearing Ratio (CBR) und der Scherfestigkeit die Eigenschaften des untersuchten Bodens zu bewerten. Darüber hinaus wurde eine ähnliche Untersuchung durchgeführt, um die Wirksamkeit von CHA und Kalk in Kombination zu bewerten. Darüber hinaus wurden Veränderungen in der mineralogischen Zusammensetzung und Mikrostrukturen unbehandelter und behandelter Proben mit Hilfe von Röntgenbeugung (XRD) bzw. Rasterelektronenmikroskop (SEM) untersucht. Die Ergebnisse zeigten, dass der mit CHA behandelte Boden im Allgemeinen Verbesserungen zeigt. Die Zugabe von 20% CHA erhöht die Tragfähigkeit und reduziert die Quellkapazität des Bodens um etwa das Dreifache im Vergleich zum unbehandelten Boden. Darüber hinaus zeigten die morphologischen Untersuchungen der mit 10% CHA und 15% CHA behandelten Bodenproben die Bildung von hydratisierten Partikeln und zementierten Verbindungen als Ergebnis der Reaktion zwischen dem Boden und der CHA. Andererseits zeigten die Labortestergebnisse, dass expansive Böden, die mit der Mischung aus Kalk und CHA behandelt werden, im Vergleich zu kalkbehandeltem Boden effektiver sind, indem die Plastizität und Quellung reduziert und die Tragfähigkeit verbessert wird. Aus der Elementaranalyse wurde festgestellt, dass die Kalziumkonzentration für einen erhöhten CHA-Gehalt erhöht wurde. Darüber hinaus zeigt die Röntgenbeugung (XRD), dass das Auftreten von zementhaltigem Produkt

auf behandelten Proben, was wesentlich zur Verbesserung der geotechnischen Eigenschaften beigetragen hat.

Diese Untersuchung zeigt die mögliche Verwendung von CHA für die Untergrundbehandlung im Straßenbau. Sie beschränkt sich nicht auf die sozioökonomischen Vorteile bei der Entwicklung von Infrastrukturen, sondern könnte auch eine wichtige Rolle bei der Verringerung der Umweltauswirkungen spielen, die sich aus der Lagerung der Abfälle ergeben.

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## Declaration of Authorship

I, Meskerem Kebede Atahu, declare that I have written this dissertation titled `The Effect of Coffee Husk Ash on Geotechnical Properties of Expansive Soil ` independently.

I confirm that:

- This thesis is entirely my own work unless otherwise referenced.
- I have acknowledged all main sources of help.
- This dissertation has not been submitted for qualification to any other academic institution.

Rostock, ____________________ ________________________

Meskerem Kebede Atahu

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## Abbreviations and Acronyms

| | |
|---|---|
| AASHTO | American association of state highway and transportation officials |
| ASTM | American society of testing and materials |
| BC | Black cotton |
| CBR | California bearing ratio |
| CH | Clay of high plasticity |
| CHA | Coffee husk ash |
| CL | Clay of low plasticity |
| EDX | Energy dispersive x-ray |
| FSI | Free swell index |
| LL | Liquid limit |
| MDD | Maximum dry density |
| MH | Silt of high plasticity |
| ML | Silt of low plasticity |
| OMC | Optimum moisture contents |
| PI | Plasticity index |
| PL | Plastic limit |
| SEM | Scanning electron microscope |
| SL | Shrinkage limit |
| UCS | Unconfined compressive strength |
| USCS | Unified soil classification system |
| Vd | Dry volume |
| Vf | Final volume |
| Vi | Initial volume |
| Vw | Wet volume |
| W/D | Wetting/Drying |
| XRD | X-ray diffraction |

# 1 Introduction

## 1.1 General Introduction

Soil is loose surface of the earth, a mixture of minerals, organic matter, gases, liquid and air. The soil quality affects the development of civil engineering infrastructures. Fundamentally, different soil types exist; one of them is expansive soil. Any earth material that has a potential for shrinking or swelling under changing moisture condition is generally termed as expansive soil. The expansive characteristic of soil is due to the presence of swelling clay minerals [1]. As the soils get wet, the clay minerals absorb water molecules and expand; conversely, as getting dry the soils shrink and leave voids in the underground. The shrink-swell properties of soils detrimentally influence construction project design, performance and life time, especially of lightweight civil engineering infrastructures. Major engineering problems are volume changes due to cyclic swelling and shrinkage upon wetting and drying of the soils. The cyclic swelling and shrinkage of the soils can lead to differential heave, settlement and creep (decrease in bearing capacity and high- erosion susceptibility), instability in natural slopes, road cuts or open excavations; and difficult in workability conditions.

Damages to civil engineering infrastructures due to the swell-shrink characteristics of soils are estimated in billions of dollars [2, 3]. Regions with significant climatic variation between dry and wet seasons are particularly prone to these problems [1]. Engineers are often faced with the problem of constructing roadbeds on or with expansive soils. In construction of roadways a soft sub-grade is one of the most frequent problems for highway construction in many parts of the world [4]. These problematic soils do not possess enough strength to support the wheel loads upon them either in construction or during the service life of the pavement [4].

The usual approaches to solve the problems arise from expansive soils nature are to remove the soft soil, and replaces it with stronger materials and stabilization (mechanical and chemical). Chemical additives, lime and cement stabilization was a popular trend for decades in enhancing property of poor soils. Even though researching for new stabilization agent (effective, economical and eco-efficient) is in high importance because of increasing global demand for raw materials.

Using waste materials in civil engineering infrastructure has a big role, for sustainable development. Therefore, exploring low cost materials for the improvement of soil properties to avoid damages to engineering structures is a crucial issue. Considering this, in this study, CHA was investigated for its applicability in the treatment of shrink-swell soils to use as sub-grade material.

## 1.2 Statement of the Problem

Geotechnical properties of soils affect the design and cost of construction projects. Expansive soils, which commonly known as BC soil in Ethiopia, are the main soil type in the country upon which road pavements are constructed [5]. Consequently, it has been observed that several road constructed on expansive soils suffer numerous problems in this country. Most of the roads constructed in Ethiopia on the shrink-swell soil damaged before expected design life. Severe crack on the surface of roads constructed on expansive soils is a common failure in the country. This happens few years or sometimes even months after being open for use. However, the extent and specificity of the damages have not been adequately addressed.

Construction of roadways over a soft sub-grade is one of the most common problems for highway construction in many parts of the world as well as in Ethiopia. BC soils are known by their high expansion and shrinking properties due to a change in atmospheric conditions (wet, dry). The shrinkage behaviour of these soils leads to development of cracks. These cracks create weakness zones in a soil mass, which increase the compressibility and reduce strength [6]. This could result in an early serious damage or even a complete collapse of civil engineering structures. There are serious problems in Ambo town (sampling area) and its surrounding infrastructural development due to the properties of this soil. Figure 1.1 a) shows typical BC soil cracks during dry season; these cracks can measure up to 25cm wide and 90cm deep, both pictures are taken from the sample area. Figure 1.1 b) shows typical road damage in and around Ambo town. In this city and its surrounding, seeing cracks on floors and walls, even a complete damage of the house are a typical scene (see Figure 1.1 c).


Figure 1.1: Typical BC soil damages in Ambo, Ethiopia: a) BC soil cracks during dry season, b) typical crack scene on roads and c) a serious damage on a house constructed on BC soil

Stabilization of soils using calcium based admixtures (lime and cement) to improve the geotechnical property of expansive soils is the most commonly used technique. However, the manufacturing processes of these admixtures contribute significantly to greenhouse gas emissions [7]. On the other hand Ethiopia produces huge amount of coffee and utilization of the waste, coffee husk, generated during coffee production is poor. Disposal of this waste poses major environmental concerns, as its abundant storage can lead to serious environmental problems due to the presence of toxic materials, such as caffeine and tannins [8, 9]. In addition, investigation of locally available low cost materials (such as coffee husk) for stabilization of geotechnically injurious soil types has not been well explored. Therefore, exploring the potential use of this waste material as admixture for the stabilization of shrink-swell soils for road construction is crucial in planning and development of road infrastructures in developing countries.

The potential use of this waste material as admixture for soil stabilization has dual advantages in reducing disposal problem, environmental concern and improving soil properties.

## 1.3 Research Objectives

### 1.3.1 General Objective

The main aim of this research is to study the potential use of CHA to improve the geotechnical properties of expansive soil.

### 1.3.2 Specific Objectives

The specific objectives of the study reported in this thesis are as follows:-

- To investigate the effect of CHA on various geotechnical properties (Atterberg limits, swelling, unconfined compressive strength, bearing capacity, compressibility, durability) of expansive soil.
- To investigate the combined effect of lime and CHA on geotechnical properties of expansive soil.
- To characterize expansive soil, additives (lime and CHA) and treated samples using XRD, EDX and SEM.

## 1.4 Research Methodology

To achieve the aim of this study; the following methodology were adopted (see Figure 1.2).

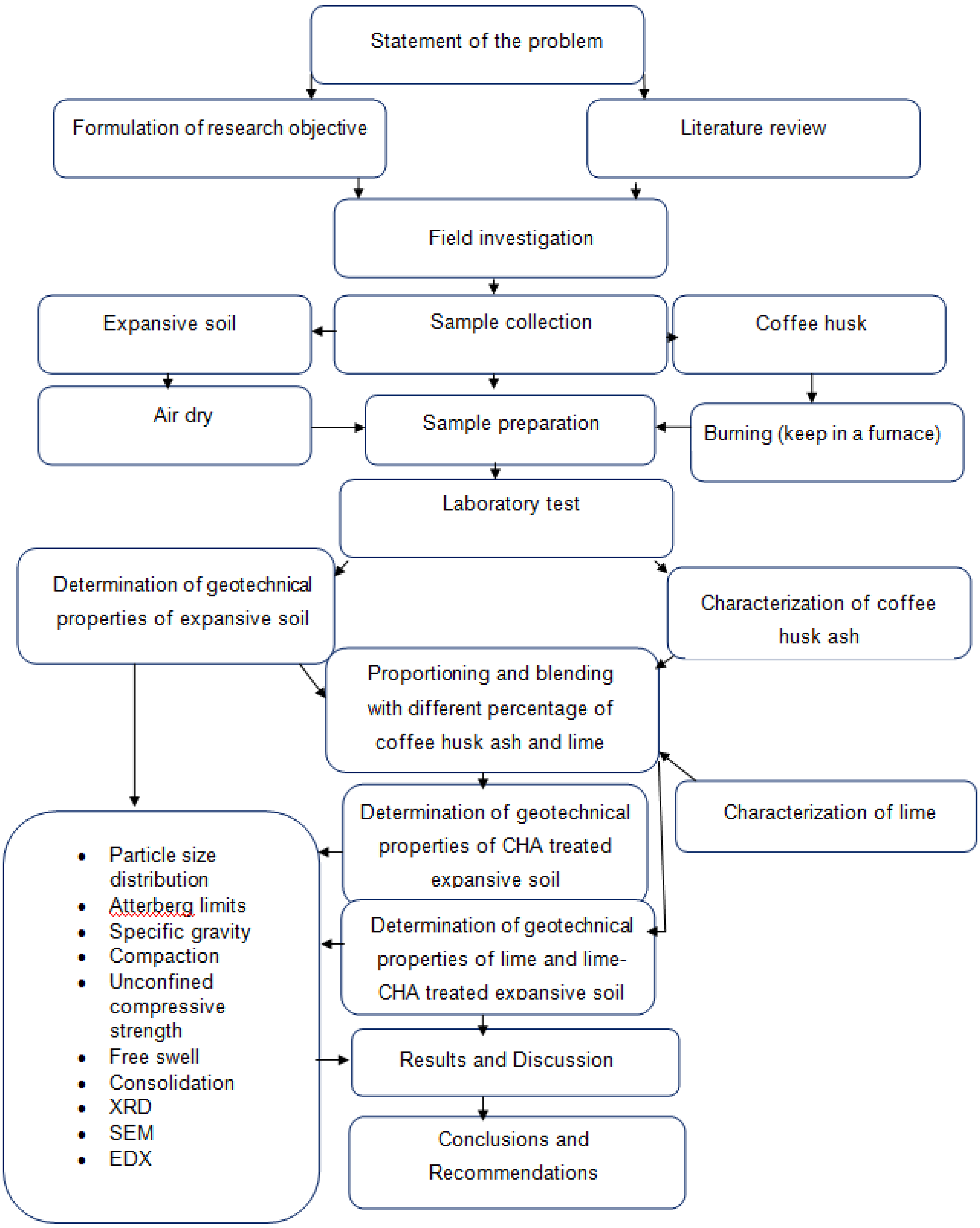



Figure 1.2: Diagrammatic illustration of research program

Review of literature on expansive soils properties, coverage, the extent of damage caused and improvement methods was under taken. This is followed by selection of site covered by expansive soil. Different samples were taken, from different depth and different location of Ambo, Ethiopia and then the representative soil were chosen for detailed study.

The research methodology further includes characterization of expansive soil using laboratory tests such as Atterberg limit, compaction, CBR, unconfined compressive strength, consolidation and shear strength. In addition, laboratory tests were also conducted in order to investigate the effects of additives (CHA and lime) on geotechnical properties of expansive soil. Three different treatment methods, CHA treated, lime treated and lime-CHA treated soil were conducted in accordance with ASTM standards. Further, Chemical and mineralogical composition of the soil, CHA, lime and treated samples were studied.

The test methods used in expansive soil characterisation were also used for treated soils, the results of untreated and treated samples were analysed to study the influence of CHA introduction on swell-shrink properties, plasticity behaviour, strength and stability characteristics of expansive soil.

## 1.5 Thesis Organization

This thesis consists of seven chapters, containing from preliminary to detailed information about expansive soil properties, related causes and improvement methods. The research work was laboratory based and the thesis organization is as follows:

The first chapter (this chapter) provides preliminary information about expansive soil and summarises the main aims of the research to address the problems associated with expansive soil.

Chapter two presents reviews of existing literature to explore the magnitude of the problems and to justify aims of the research. Expansive soil distribution, characteristics, classification, and the damage associated with such soil as well as improvement methods are reviewed in this chapter. In this chapter the production of coffee husk, related disposal problems and application of coffee husk for different purpose are discussed.

In the third chapter, geographical location and climate condition of study area, geotechnical properties of expansive soil used for this research and the two additives, coffee husk ash and lime are discussed. This chapter also describes the methods used in this research for characterization of the samples.

Chapter four covers analysis on the laboratory test results of untreated and CHA treated samples and effects of CHA on plasticity, compaction, compressibility and unconfined compressive strength characteristics of studied expansive soil. In addition durability of CHA treated samples and critical discussions on the possible reasons behind the observed characteristics of treated samples are presented in this chapter.

The fifth chapter is an extension of chapter four, in this chapter SEM and XRD analysis to evaluate the influence of CHA as an additive on micro-structural and mineralogical properties of expansive soil are presented.

This chapter also deals with bearing capacity and shear strength behaviour of expansive soil treated with CHA. Further the effectiveness of CHA as a stabilizer was evaluated.

Chapter six presents the geotechnical properties (plasticity, compaction, bearing capacity and unconfined compressive strength) of expansive soil treated with lime and lime-CHA mixture. To implement the field condition, effect of wet-dry cycle on compressive strength of lime-CHA treated soil are analysed and discussed.

The final chapter of the thesis, chapter seven, summarises the thesis work and presents the major conclusion drawn from the research. This chapter also recommends areas for further research.

# 2 Literature Review

## 2.1 Expansive Soils

Soil can be classified as a soil with high and low compressibility, sensitive and insensitive, high plastic and low plastic, very soft to stiff clay, loose and dense sand, expansive and non expansive etc [10]. Expansive soils are soils those contains high content of expansive clay minerals which have a tendency to expand as moisture content increases and shrink as they dry out. Expansive soils generally found in nature are in an unsaturated state and the swell-shrink behavior of these soils is mainly governed by the in-situ condition [11].

Expansive soils can be identified by performing expansion test, soil that cracks or fractures when it dries is often a sign that it is expansive; a lack of cracks does not necessarily indicate that the soil is not expansive [12].

Expansive soils are one of the widespread typically problematic soils in the world [13]. The expansion behaviour of these soils is more of a problem in arid and semi-arid areas. Expansive soils are hard and strong when they dry out and swell and soften when the moisture content increases leading to ground movement [14]. The unpredicted movements due to high swell-shrink behavior of these soils cause severe damage to overlying structures or cracks on the pavements resting on them [15]. The problems of a ground movement are predominantly related to the presence of montmorillnite clay mineral which has a capacity to absorb water in monsoon season and shrink by leaving cracks in drier seasons [1, 16, 17]. The continues variation of expansive soil volume, makes the material unsuitable for construction. However, the rapid population growth and increase in urban development has made it difficult to avoid areas covered by these soils [18]. Due to the global distribution of expansive soils many different ways have been developed to tackle the problem and these can vary considerably [19]. Removing the problematic soil and replacing by a good quality material, or treating using chemical or mechanical stabilization is common solutions. In practice, very widely used chemical stabilizers are lime and cement. In recent years, due to a large-scale demand and limited resources, there is a growing tendency to replace traditional stabilizers with industrial and agricultural by-products. The partial or full replacement of cementitious material, lime and cement by industrial and agricultural by-products has a big role in enhancing the properties of the soil, reducing cost and environmental conservation [20-22].

### 2.1.1 Distribution of Expansive Soils

Expansive soils can be found almost all over the world [23]. By the extension of constructional activities, more expansive soil regions are being discovered every year, especially in underdeveloped countries [1]. Expansive soils are found particularly in arid and semiarid regions, as well as where wet conditions occur after prolonged periods of drought. Distribution of an expansive soil is generally a result of geological history, sedimentation and local climatic conditions [24].

Countries where expansive soils occur and give rise to major construction costs include Ethiopia, Ghana, Kenya, Morocco, South Africa, and Zimbabwe in Africa; China, India, Iran and Japan in Asia; Canada, Mexico and USA in north America; Argentina and Venezuela in the south America; Romania, Spain and Turkey in Europe; and Australia (see Figure 2.1) [24]. In these areas expansive soils are known by different names such as "black cotton soils" of central and eastern Africa and India; "black earth" of Australia and northern Africa: "swell-shrinking soils or cracking soils" of China; "problem soils" of Japan; "London clay" of UK and "hidden hazard and grumusols" of USA [25].

In regions of expansive soil deposits, the evaporation rate is higher than the annual precipitation, so there is usually a moisture deficiency in the soil. The deficiency in rainfall leading to the lack of leaching processes is believed to have aided the formation of smectite minerals in semi-arid zones of the world [1]. The presence of montmorillonite clay (a smectite group) in these soils imparts them high swell–shrink potentials [1]. The distribution of expansive soil is dependent on the formation, accumulation, and preservation of montmorillonite, its formation is mainly facilitated by weathering and diagenetic alteration of pre-existing minerals [26].

Arid climatic conditions and severe weathering environment prevailing in north eastern part of Africa promote the widespread occurrence of expansive soils in this region [27]. In Ethiopia, expansive soils cover significant area of the country (see Figure 2.2). The textural classification of soils in Ethiopia is shown in Figure 2.3. Expansive soils are observed in area such as central Ethiopia, following the major trunk road like Addis Ababa - Ambo, Addis Ababa - Weliso, Addis Ababa - Debere Berehan, Addis Ababa – Gohatsion and Addis Ababa - Mojo. They also cover the areas like Mekelle, Bahirdar, Gambela, Arba Minch and the most Southern, South-west and south-east part of the capital Addis Ababa area in which the most major recent construction are being carried out [28].

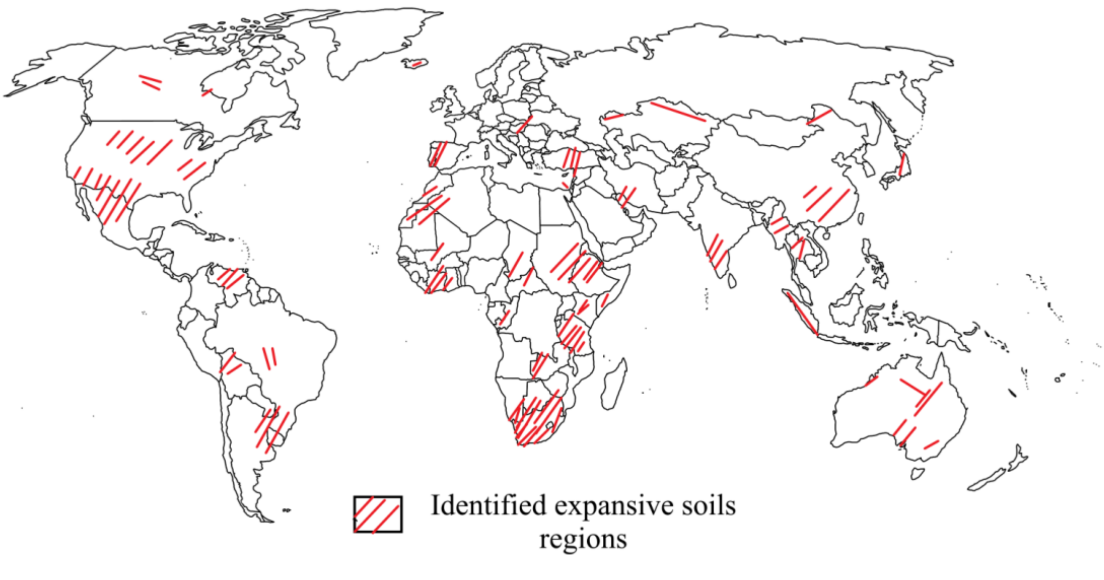



Figure 2.1: Global distribution of expansive soils (After [1, 29])

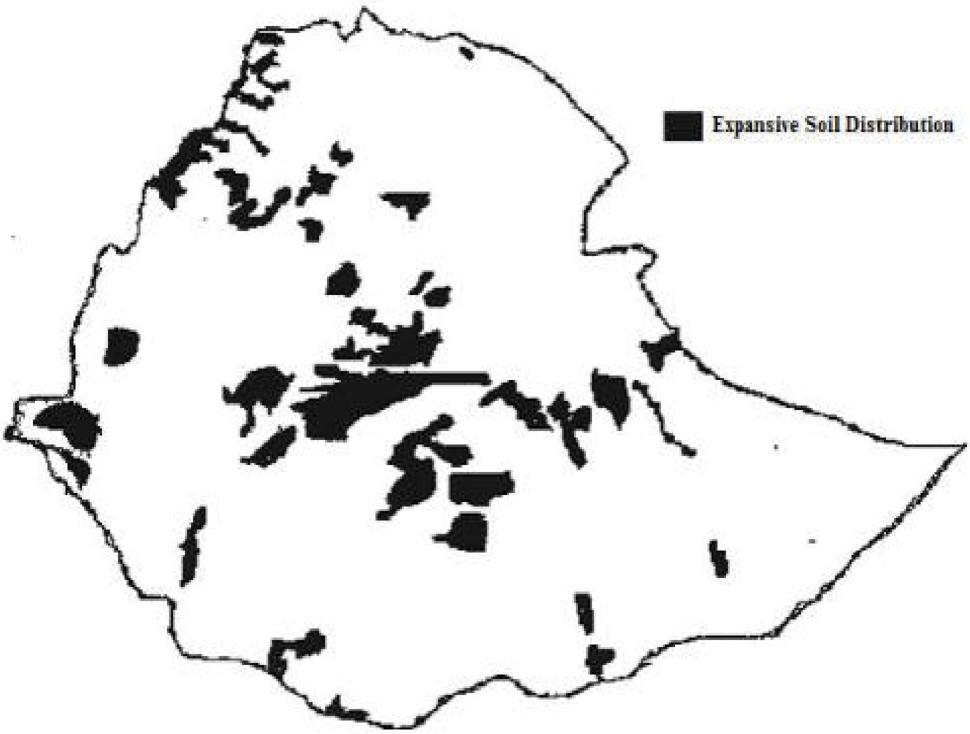



Figure 2.2: Distributions of expansive soils in Ethiopia (After [28])

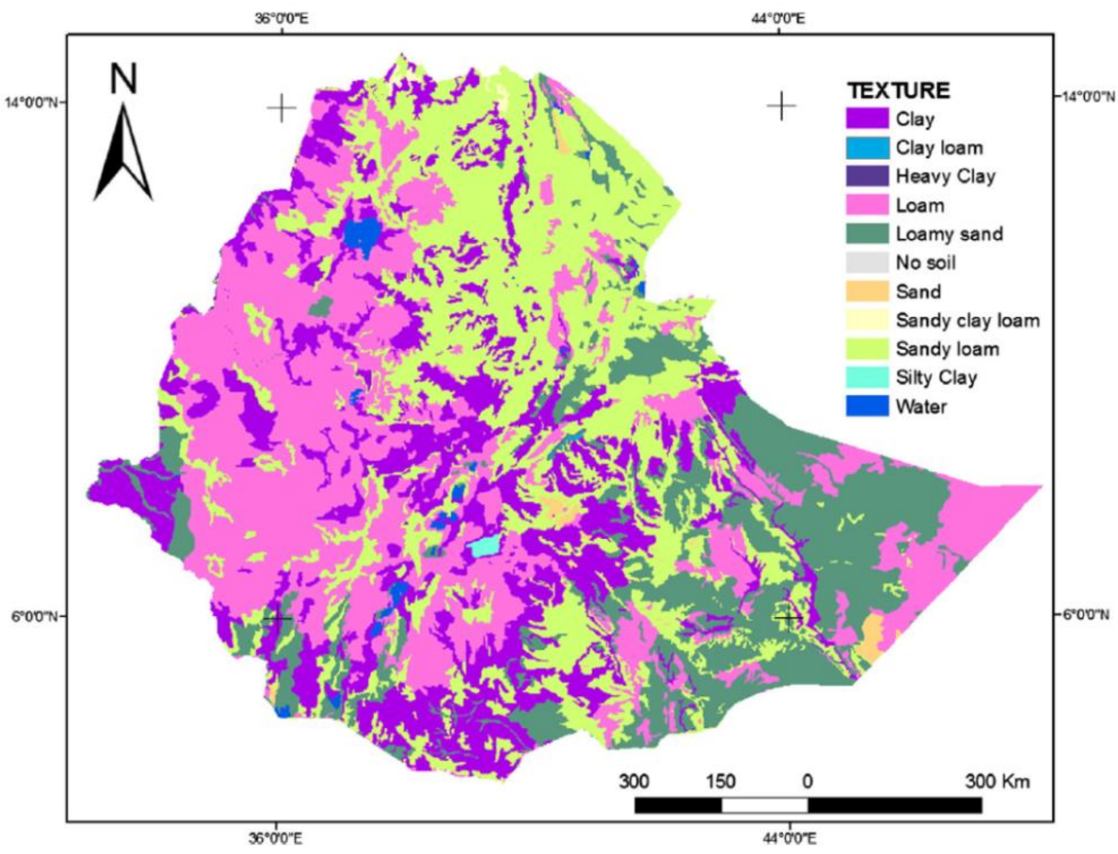



Figure 2.3: Textural classification of Ethiopian soils (After [30])

### 2.1.2 Identification and Classification of Expansive Soils

Identification of soils that has an expansive nature and associated problems is very important to select an appropriate design and construction methods. There are various methods to identify an expansive nature of soils either visually or manually. Visual identification, surface examination, geological and geomorphological description can indicate the nature of soil. A visual identification of an expansive soil can be carried out based on its colour, texture and observing extent of desiccation cracks at the field. The extents of these cracks rely on the degree of swell [31]. In addition, soils containing expansive clays become very sticky and plastic when wet, and hard when dry.

Physical and mechanical properties of soils can also determine their swelling potential; these properties can be obtained by performing geotechnical index property tests such as grain size distribution, Atterberg limits, volume change, and mineralogical compositions tests. The swelling soils are commonly known by the name of expansive soils. For swelling to occur, these soils must be initially unsaturated at some water content. If the unsaturated soil gains water content, it swells. On the other hand, if a decrease in water content occurs the soil shrinks. A major concern in geotechnical engineering is identification of expansive soils and estimation of their swelling magnitudes when subjected to changes in environment [32].

There are three different methods of expansive soils identification namely mineralogical Identification, direct measurement and indirect methods. Mineralogical identification can be useful in the evaluation of the material. The various methods of mineralogical identification are important in a research laboratory in exploring the basic properties of clays, but are impractical and uneconomical for practicing engineers [1]. The mineralogical composition and pore size distribution of soils mainly detect the swell-shrink behaviour of expansive soils [33]. The direct methods consists essentially performing swelling tests, while the indirect methods based on empirical correlation of the measured soil´s geotechnical properties and swelling capacity [34].

Direct methods measure the swell potential of a soil directly, the most useful and reliable assessment of swell potential of a soil could be obtained from the conventional odometer swell tests [35].

The criterion developed by Holtz and Gibbs [36] for the classification of degree of soil expansivity (see Figure 2.4) is the total volume change of a soil from air dry to a saturated condition under a surcharge of 7 kPa in an odometer.

According to the Bureau of Indian Standards [37], an expansive soil can be *classified* based on FSI value.

$$FSI\ (\%) = \frac{(Vd - Vk)}{Vk} * 100 \qquad \text{Eq. 2.1}$$

where, $V_d$ and $V_k$ are the equilibrium sediment volumes of 10 g oven dried soil samples passing 425 µm sieve placed in 100 ml graduated measuring jars containing distilled water and kerosene respectively, after an equilibration period of a minimum of 24 h. The swell potential of the soil based on FSI is classified as per the Indian Standards [37] (see Figure 2.5).

The indirect methods make use of soil index properties such as clay content and Atterberg limits, these properties are the most widely used in practice to estimate the swell potential of soils and classifying expansive soils [15].

Chen [1] and Holtz and Gibbs [36] predicted the swell potential of fine-grained soils using percentage of clay size fraction (content of particles of size less than 0.002 mm) and colloid content (content of particles of size less than 0.001 mm) respectively (see Table 2.1).

The Atterberg limits are a basic method to measure the critical water contents of a fine-grained soil: its LL, PL and SL.

*LL* is determined in the laboratory by the conventional Casagrande method or by the fall cone method. Liquid limit of a soil is the moisture content at which soil changes from a plastic to a liquid state, which in turn has been taken as a measure of soil swell potential [39].

*PI*: It is the difference between the liquid limit and plastic limit of fine-grained soils. The higher the plasticity index, more plastic the soil is and higher will be the soil swell potential.

*SL*: It represents the lower bound water content for any volume reduction of a soil mass and hence, regarded as a measure of volume stability of the in situ soil [39]. Expansive soil classification based on Atterberg limits are presented in Table 2.2.

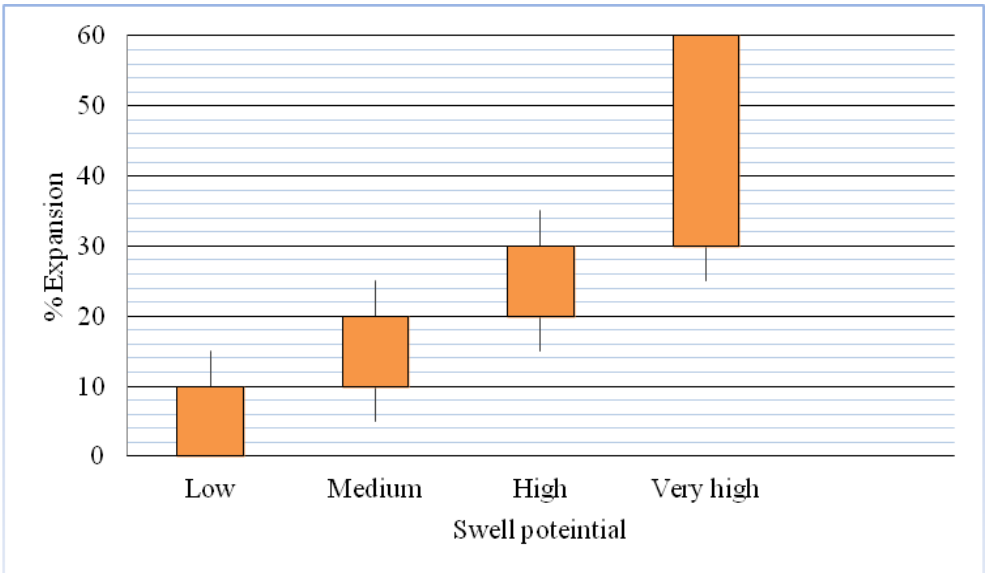



Figure 2.4: Expansion versus swell potential (After [36])

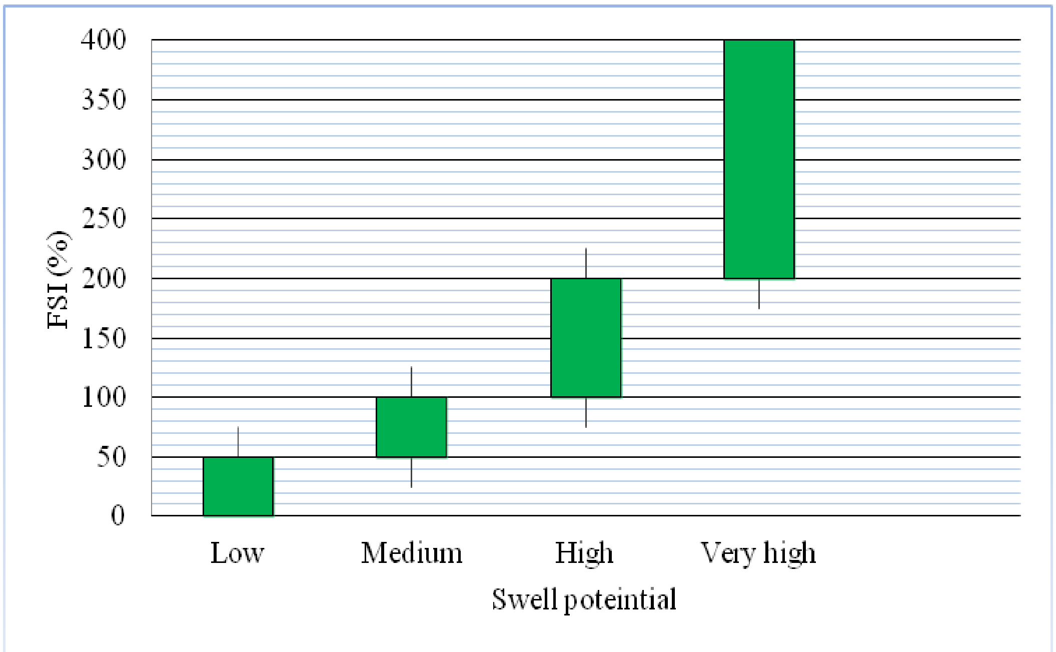



Figure 2.5: FSI versus swell potential (After [37])

Table 2.1: Degree of expansivity based on particle size composition (After [1] and [36]).

| Degree of expansivity/swell potential | Percent clay size fraction | Colloid content (<0.0001 mm) |
|---|---|---|
| Low | <30 | <15 |
| Medium | 30–60 | 13–23 |
| High | 60–95 | 20–31 |
| Very high | >95 | >28 |

Table 2.2: Swell potential based on Atterberg limits (After [1, 36, 37]).

| Swell potential | Liquid limit (%) | | Plasticity index (%) | | Shrinkage limit (%) |
|---|---|---|---|---|---|
| After | [1] | [37] | [36] | [37] | [36] |
| Low | <30 | 20-35 | <18 | 0–15 | >15 |
| Medium/ marginal | 30–40 | 35–50 | 15–28 | 10–35 | 10–16 |
| High | 40–60 | 50–70 | 25–41 | 20–55 | 7–12 |
| Very high | >60 | 70–90 | >35 | >35 | <11 |

### 2.1.3 Clay Mineralogy

Minerals occur in soils as a result of inheritance from parent materials, by crystallization from solution and by altering of existing minerals into new species [40]. Minerals are natural inorganic compounds with definite physical, chemical, and crystalline properties. Minerals particles (sand, silt, clay) and organic matter make up about 50% of the volume of soils [41]. The other 50% of the volume is made up of pore space filled with varying proportions of air and water. Minerals are indicators of the amount of weathering that has taken place, and the presence or absence of particular minerals gives clues how soils formed. They are classified as primary or secondary, silicates or non-silicates, and crystalline or non-crystalline minerals. Primary minerals have not been substantially altered chemically since deposition or crystallization from molten lava and are usually found in the sand and coarse silt fractions [42]. Secondary minerals form as a result of the weathering of primary minerals and are found in the clay and fine silt fractions [42]. However, many exceptions to this size partitioning do occur. Iron and aluminium oxide as well as carbonates, sulphates, sulphides and secondary silica minerals, often are components of the sand and silt fractions. In addition, some primary minerals, especially quartz and feldspar, are present in the coarse clay fraction of many soils [40].

Quartz is found in almost every geological environment. It is so abundant in soils and sediments and common constituent in most of the rock types and soil groups [43]. Granite, sandstone, limestone, and most of the igneous, sedimentary and metamorphic rocks contain quartz. Quartz contains mainly oxygen and silicon, these two constituents make up to 75 % of the earth's crust, the other 25% are aluminium, iron, calcium, sodium, potassium, magnesium etc [43]. An alternate name for the quartz group is the silica group. The structure of quartz is built from $SiO_4$ tetrahedra which are linked by sharing each corner with another tetrahedron [43], see Figure 2.6.

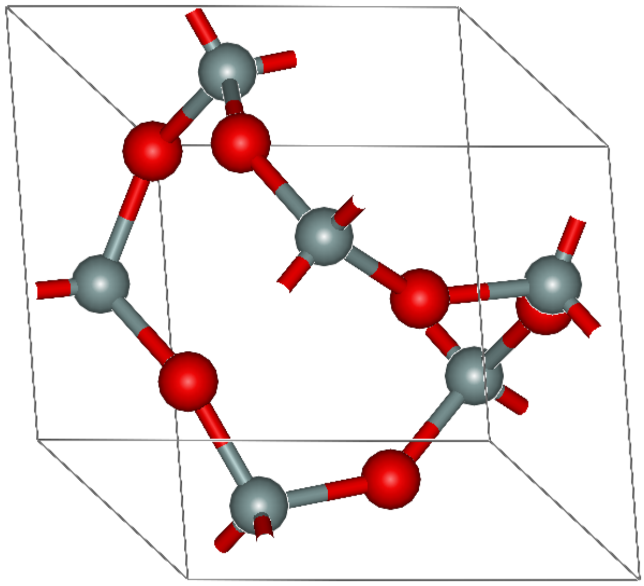


Figure 2.6: Structure of quartz mineral (red balls are oxygen and grey balls are silicon) (After [44])

Clay minerals are secondary minerals formed from weathering of rock. Clay is composed of an accumulation of clay-sized mineral particles of which clay minerals generally make up a significant proportion [45]. These minerals are similar in chemical and structural composition to the primary minerals that originate from the Earth's crust; however, transformations in the geometric arrangement of atoms and ions within their structures occur due to weathering [46]. The clay minerals are crystalline hydrous aluminosilicates having a lattice structure in which the atoms are placed in layers, similar to the pages of a book. And each of the clay minerals has a different arrangement and chemical composition which determine the type of clay mineral [47]. Most common clay minerals are kaolinite (non-expanding type with two layers), montmorillonite (expanding type with three layers) and illite (see Figure 2.7).

In silica rich rocks and minerals in which calcium and magnesium cations are present, under climatic conditions in which evaporation exceeds precipitation, the pH is high and leaching conditions are favourable for the formation of montmorillonite (smectite group). On the other hand, in rocks and minerals in which alumina is abundant, magnesium and calcium cations are absent, and in areas where rainfall is relatively high with good drainage, conditions favour the formation of kaolinite [48].

The expansive nature and negative charge of smectites cause them to be extremely reactive in soil environments. In seasonally wet and dry climates, they are responsible for most of the swelling and shrinking that occurs in soils. Smitites in the surface of soil may be responsible for the adhesive property that helps prevent sheet erosion while those in the subsurface may absorb large quantities of water, diminish in shear strength, and move down slope taking buildings with them [49].

Montmorillonite is the most common member of smectite clay minerals; it has high expansion capacity due to the intra-crystalline swelling or expansion of crystal lattice under wet conditions [50]. Smectite minerals have high water absorption capacity that reduces the soil strength. Montmorillonite is the main mineral in some clays and shales and in some residual soils formed from volcanic ash [47].

The general structural characteristics of the clay minerals have two structural units, octahedral and tetrahedral. The octahedral sheet (gibbsite sheet) is usually composed of aluminium, which consists of two sheets of close-packed oxygens or hydroxyls between which aluminum atoms are embedded in such position that they are equidistant from 6 oxygens or hydroxyls. The tetrahedral sheet is composed of silica tetrahedral ($SiO_4$) groups linked to neighboring tetrahedra by sharing three corners, resulting in a hexagonal network [51, 52], see Figure 2.8.

Montmorillonite consists of structural units of one aluminium octahedral sheet (gibbsite sheet) between two sheets of silica tetrahedral groups (see Figure 2.7). The structural units are stacked one above another and they are loosely held together with water present between the units and the mineral is said to have an expanding lattice [51].

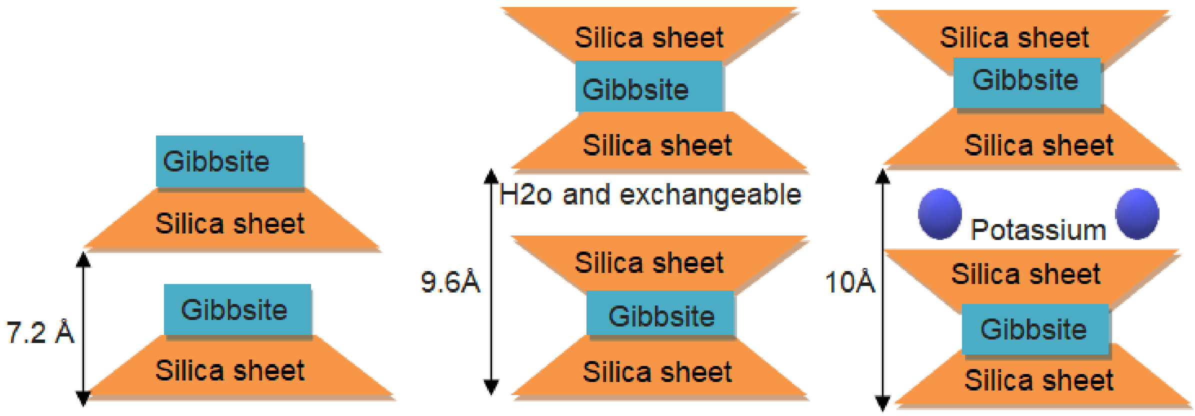



Figure 2.7: Structure of clay minerals Kaolinite, Montmorillite and illite (After [53])

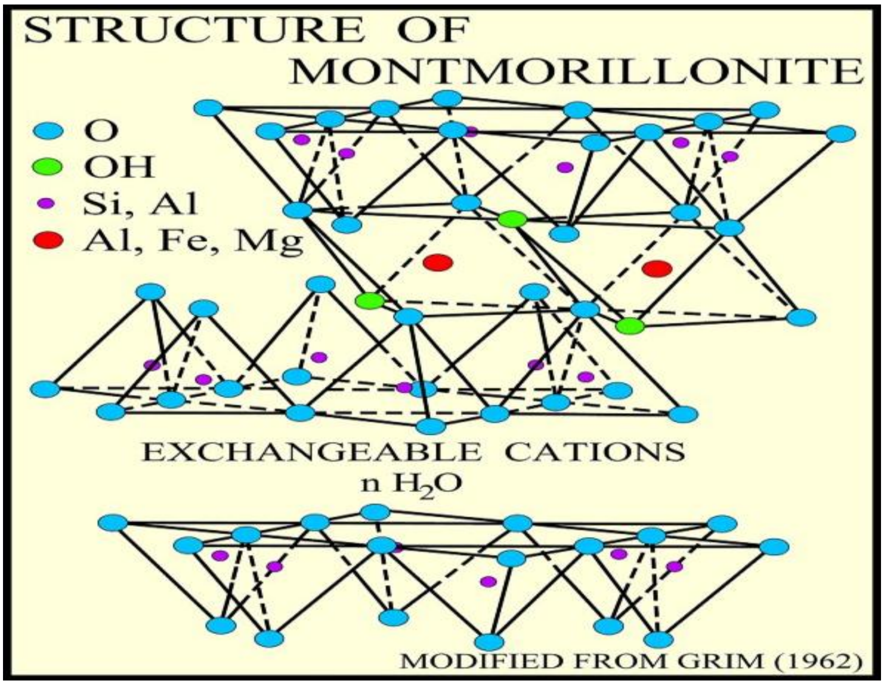



Figure 2.8: Structure of montmorillonite (After [54])

The swelling of the clays is mainly attributed to the water absorption capacity of highly reactive mineral like montmorillonite [16]. Figure 2.9 shows the Scanning SEM of Montmorillonite. The large swelling capacity of montmorillonites, marks these minerals as the most troublesome ones with respect to engineering design and construction [48].

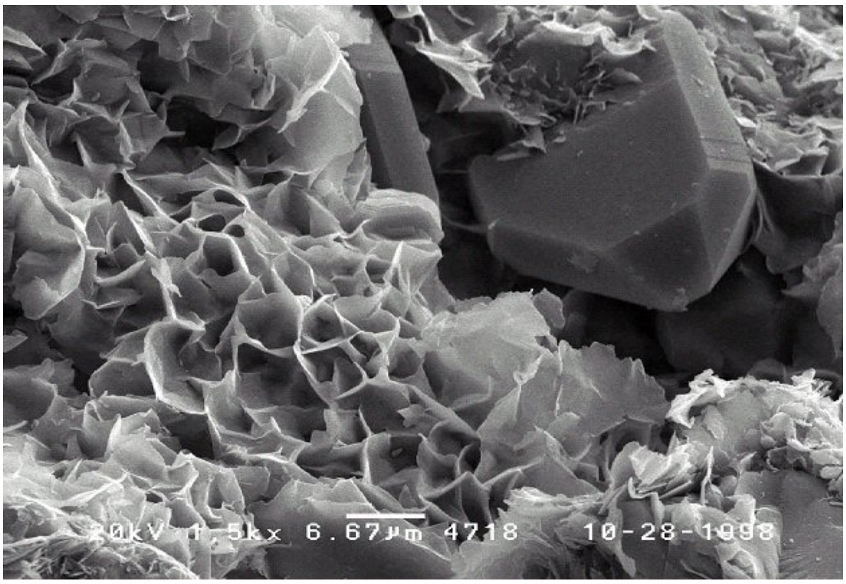



Figure 2.9: SEM of montmorillonite (After [55])

Kaolinite is one of the most commonly present clay minerals in sedimentary and residual soils [51]. Kaolinite consists of repeating layers of elemental silica-gibbsite sheets in a 1:1 lattice [53], as shown in Figure 2.10.

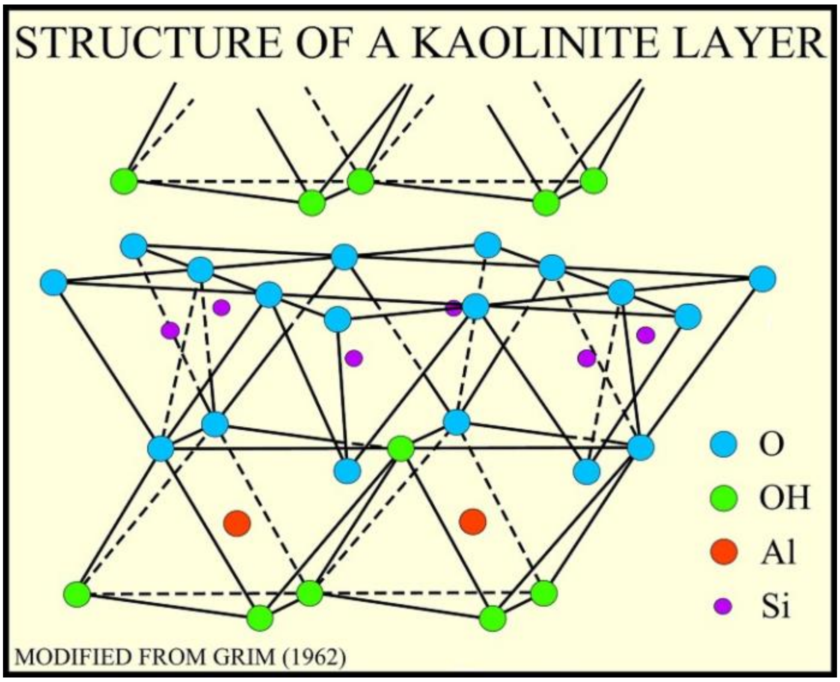



Figure 2.10: Structure of kaolinite (After [56])

Clay composed of kaolinite may be viewed as aggregate of uncharged small flakes that are not as small as those of montmorillonite or some illite clay. There is, therefore, slight tendency to form water films on the flakes and for the individual flakes to attract each other. Consequently bond strength, drying shrinkage, and general plastic properties should be low. The plasticity of kaolinite should be well below that of montmorillonite clays and subject to variation depending on the size of the kaolinite particles [51]. Figure 2.11 shows the SEM of Kaolinite.

Illite (sometimes called clay mica) consists of a gibbsite sheet bonded to two silica sheets; the layers are bonded by potassium ions [53], see Figure 2.12. Clays composed of illite are aggregates of flakes that have attractive forces on their surfaces. Like clays composed of montmorillonite, conditions are proper for the development of water films surrounding the flakes and for the existence of the plastic state when the clay is worked with water. However, as flakes of illite are larger than those of montmorillonite, fewer are present in a given volume of clay [51]. The total surface area in illite clay would therefore be smaller and fewer water films between flakes along which slippage would be present. The plasticity of illite clays would therefore be relatively less than that of montmorillonite clays. It follows that the plasticity, drying shrinkage, and the influence of exchangeable bases on surface properties would be lower in illite than in montmorillonite clays [51].

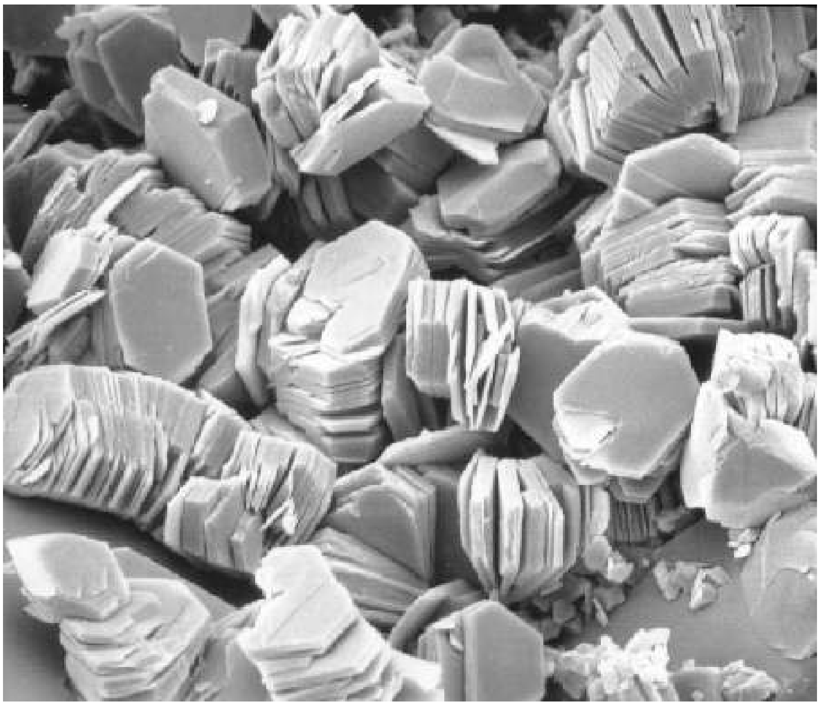


Figure 2.11: SEM of Kaolinite (After [57])

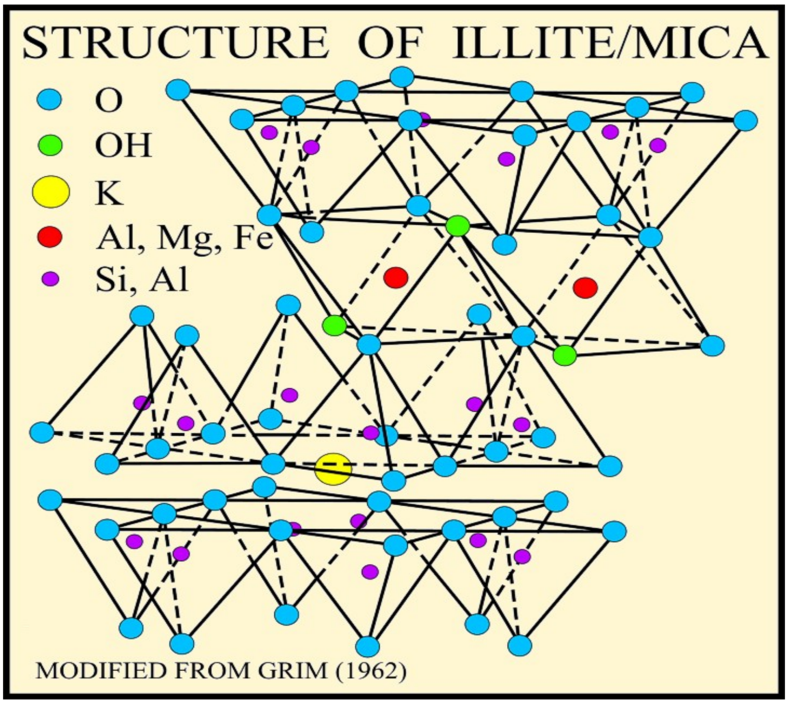



Figure 2.12: Structure of Illite (After [58])

Expansive soils are fine grained soils; mineralogical composition and pore size distribution of these soils pay a fundamental role in their moisture absorption capacity [31, 33, 47].

Clay minerals in a soil usually influence properties in a manner far greater than their abundance [59]. The physical and chemical characteristics of soil minerals are important considerations in planning, constructing and maintaining buildings, roads and airports [41]. The swell potential of any clay could be estimated by identifying the mineralogical composition of the soil [1]. Clay minerals can be identified using XRD, Differential thermal analysis, Electron microscopy.

## 2.2 Problems and Damages Associated to Expansive Soils

The behaviour of the soil at the location of any structure and the interactions of the earth materials during and after construction has major influence on the economy and safety of the structure. All engineering structures may experience damage in their life time, the level and cause of damages may vary. The damage causes might be due to design errors, poor workmanship, climate conditions, and properties of construction materials (eg. expansive soils). Primary characteristics of expansive soils are their potential to swelling and shrinkage in response to moisture content increase and decrease. An increase in volume (swelling) from expansive soil can exceed the downward pressure exerted by light weight structure and hence cause deformation and development of cracks [5]. Expansive soils awe their characteristics to their mineralogical assemblage that is a presence of active clay mineral and their amount [5]. Expansive soils consist of plastic clays and clay shales that often contain colloidal clay minerals such as the montmorillonites. Some of these soils, especially dry residual clayey soil, may heave under low applied pressure but collapse under higher pressure. Heaving by swelling soils can also cause slope failures, roads to buckle, driveways to shift, and structural walls to collapse [60].

Structures built on expansive soils tend to undergo moderate to severe cracking problems due to swelling and shrinkage behavior [15]. The extent of cracking and damages of the pavement leads to high rehabilitation cost. Major problem of expansive soils is large ground deformations in and around the structures due to swelling and shrinking of soil on wetting and drying. These excessive movements can damage structures [61]. The problems may be large ones or small ones and these have tremendous negative impact on the structure performance in terms of time and cost. A knowledge and understanding of geology of earth materials is necessary while dealing with expansive soils [61].

Damage on infrastructure founded on expansive soil has been extensively reported by several researchers around the world. For example, USA (each year in the United States, expansive soils cause $2.3 billion in damage to houses, other buildings, roads, pipelines, and other structures); South Africa; Nigeria; Middle East; Ethiopia and India [14, 32, 62, 63].

The cost of the damage to civil engineering structures caused by expansive soils around the world has been estimated in billions of dollars [14, 64], which exceeds damage costs of natural disasters (earthquakes, floods, tornados, and hurricanes) [1]. The problems arise from the nature of this soil are also severe in Ethiopia especially on durability of pavement structures, but no statistics are available on the cost of damages due to expansive soil alone [65].

In recent years, many researchers have been devoted to study the phenomena and developing constitutive and computational models applied to the engineering behaviour of expansive soils [66].

### 2.2.1 Pavement Structures

Pavements are built up in several layers, consisting of sub-grade, sub-base, base and surface layer. These layers together constitute the pavement (see Figure 2.13). The sub-grade is the compacted soil layer that forms the foundation of the pavement system, consists of naturally occurring material on which the road is built, or the imported fill material to supporting the load transmitted from the overlying layers. Sub-bases acts as a load bearing layer, and strengthens the pavement structure directly below the pavement surface, providing drainage for the pavement structure and reducing the stress applied to the sub-grade. However, it is critical to note that the sub-base layer will not compensate for a weak sub-grade. Sub-grades with a CBR of at least 10 should provide adequate support for the sub-base [67]. The base course is the portion of the pavement structure immediately beneath the surface course and plays a prominent role in the support and dispersion of the traffic loads [68]. Surface course may consist wearing course, or binder course and wearing course depending on traffic. Binder course layer works as a supporting, dispersing traffic load and resists shear, while the topmost layer (wearing course) resists abrasion and prevent skidding [69].

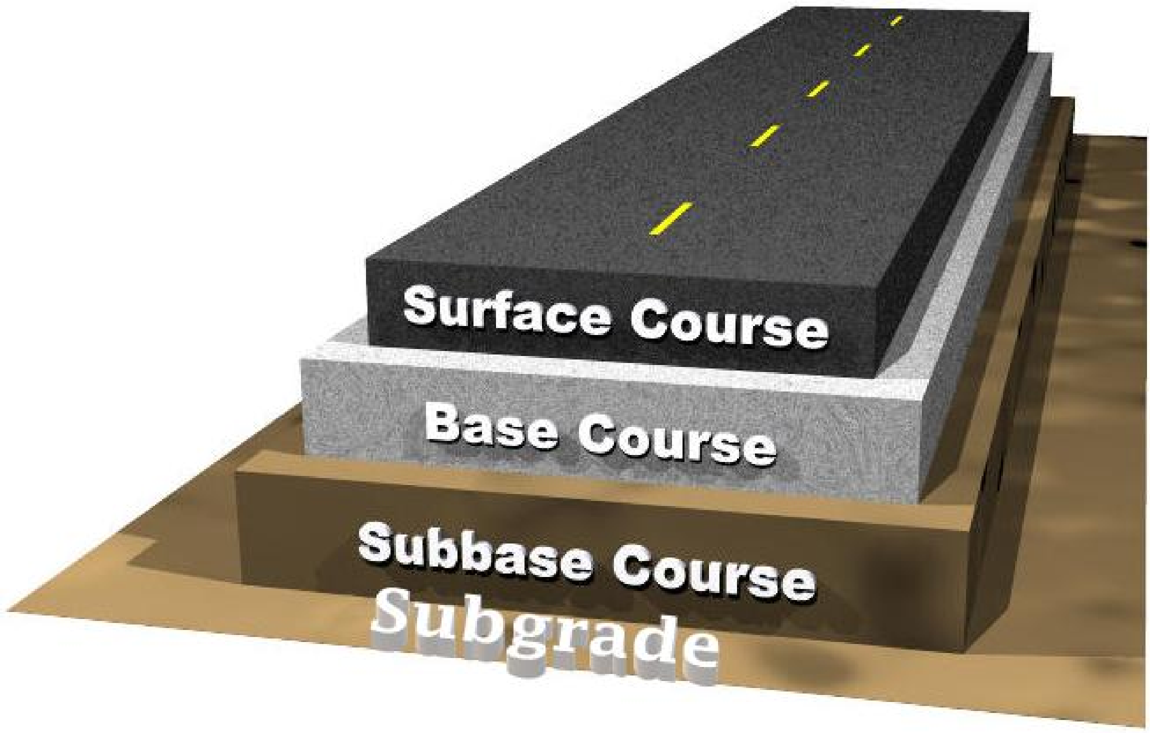



Figure 2.13: Flexible pavement layers (After [70])

The performance of a pavement depends on the quality of its sub-grade and sub-base layers. The sub-grade and sub-base layers play a key role in mitigating the detrimental effects of climate and the static and dynamic stresses generated by traffic. Therefore, building a stable sub-grade and a properly drained sub-base is vital for constructing an effective and long lasting pavement system [67].

Soil is an integral part of the road pavement structure as it provides the support to the pavement from beneath. Poor soils can seriously impede construction of adequate sub-grades, as well as affect the long-term performance of a pavement during its service life [67].

Expansive soils are susceptible to volume change with seasonal fluctuations in moisture content. This volume change of clay-type soils can result in longitudinal cracks (see Figure 2.14) near the pavement's edge and significant surface roughness (varying swells and depressions) along the pavement's length [67].

Pavements with clay soils sub-grades are resulted to significant pavement distress because of low sub-grade support values and moisture-induced volume changes [71].

Building highways on water-sensitive expansive soils often is a major problem due to the occurrence of differential movement because of variations in profile thickness of any expansive strata [32].

Distortion and cracking of the pavement is a common problem caused by expansive soils. Desiccation cracking of the underlying sub-grade soil during dry period imparts poor support to the pavement layers, and induces the failure of the pavement. In addition, swelling of the expansive sub-grade soil at the locations of poor drainage conditions result in heaving of the pavement layers [72]. Differential swelling of the pavement layers induce shear forces and moments on pavement, if the pavement is not designed to address such kind of loadings, failure of the pavement is inevitable. The extent of cracking and damages of the pavement layers can be extensive and sometimes the repair cost can be higher than the original construction costs [72].

A soft sub grade in construction of roadways is one of the most frequent problems for highway construction in many parts of the world [69]. Das [73] related the quality of the sub-grade soil used in pavement applications using unconfined compressive strength values, see Table 2.3. Bowles [74] related the quality of the sub-grade soil used in pavement applications using CBR values, see Table 2.3. Depending on these values, if the soil is considered unstable for supporting the wheel loads, the properties of soil should be improved.

Table 2.3: Quality of sub-grade materials based on UCS and CBR values (After [73 and 74]).

| UCS (kN/m²) | Quality of sub-grade (consistency) | CBR (%) | Quality of sub-grade |
|---|---|---|---|
| 0-25 | Very soft | 0-3 | Very poor |
| 25-50 | Soft | 3-7 | Poor to fair |
| 50-100 | Medium | 7-20 | Fair |
| 100-200 | Stiff | 20-50 | Good |
| 200-400 | Very stiff | >50 | Excellent |
| >400 | Hard | - | - |

Figure 2.14: Typical longitudinal crack developed on pavements over expansive clays (After [75])

### 2.2.2 Building Structures

The main problem of expansive soils can be attributed to poor understanding of the volume changes caused by moisture fluctuations, this continuous change in soil volume can cause homes built on this soil to move unevenly and crack [29].

The most obvious way in which expansive soils can damage building foundations is by uplift as they swell with moisture increases. Swelling soils lift up and crack lightly-loaded, continuous strip footings, and frequently cause distress in floor slabs [76].

Reaction of an expansive soil to changed environmental conditions is to swell or exert large pressures against non-yielding structures; but it may also exhibit a high degree of shrink–swell reversibility with changes in moisture content, leading to deformation and damage to buildings [16].

The predominant movement observed in buildings on expansive soils is one of upward heave or a dome-shaped heave with the greatest upward movement occurring at the center of the building [32], see Figure 2.15. When the soil is subsequently wetted either by rainfall or domestic watering, the middle of the footing would be experience more expansion than the corners due to the smaller surcharge over the former. During the summer the soil shrinks by desiccation. Since the soil around corners is exposed to more drying than at the sides of the house, more shrinkage would be expected at the corners than at the middle of the footings. This would result in larger differential movement, differential movement of these buildings may cause undesirable floor tilting (moments), cracks, and other hazards [32].

Expansive soils can also damage multi-story buildings; walls may not be heavy enough to resist soil swelling (uplifting). The same walls are most vulnerable if supporting soil dries out and shrinks, removing wall support [14].

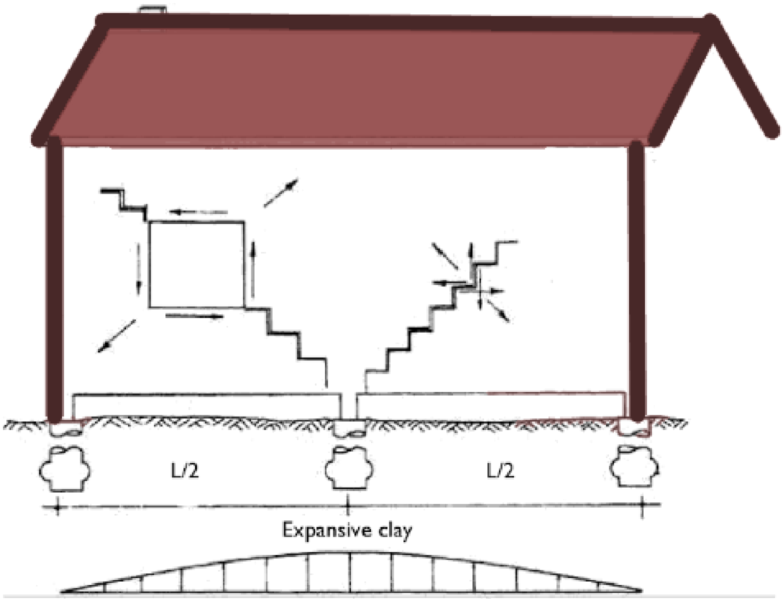



Figure 2.15: Structure subjected to center column heave (differential heave) (After [32])

## 2.3 Improvement Methods

Different treatment methods have been adapted to improve the geotechnical properties of expansive soils and to avoid excessive swelling. The most popular treatment methods are removing the problematic soil and replacing by a good quality material or treating using mechanical and/or chemical stabilization. Among these treatment techniques, chemical stabilization (treatment with binders) has a very long history in view of its effectiveness and adoptability [77].

In practice, chemical stabilizers, i.e. lime and cement, are widely used [71]. In recent years, there is a tendency to replace traditional stabilizers with industrial and agricultural by-products due to the limited resources. Several researchers investigated various waste materials (e.g. rice husk ash, bagasse ash, locust bean pod ash, marble dust and coir waste) for the purpose of soil stabilization, to enhance the properties of these soils and to reduce the construction cost. The partial or full replacement of cementitious material, lime and cement by pozzolanic materials is effective in enhancing the properties of the soil, reducing cost and environmental conservation [20-22]

### 2.3.1 Removal and Replacement

Replacement of in situ expansive clays by a suitable material is a common method of dealing with problems of expansive soil sub-grades, but the expense of removing in situ material, transporting and replacing with a stable material, is very high [15].

The depth to which non-expansive backfill should be placed will be governed by the weight necessary to restrain the expected uplift pressures and the ability of the backfill to mitigate differential displacements [15]. One mechanism by which the removal and replacement method mitigates expansive potential is by control of the moisture content in the underlying clay layer. Most of the seasonal moisture content fluctuation takes place in the non expansive backfill. If the clay exhibits a moderate to low expansion potential the reduced volume of expansive clay in the upper, high moisture variation layer may be sufficient to prevent large movements at the surface. However, if there is a high potential for volume change in the underlying soil the reduced volume of expansive material may not adequately prevent surface heave or shrinkage. If water can infiltrate into the clay layer due to surface runoff, water line breakage, or other factors, excessive movements will most likely result [15].

Expansive soils cover large geographical area and in areas where it occurs, cover a significantly large area that avoiding it is not possible [78]. As a result, the soil replacement option is not cost effective. In addition, it is essential to minimize earth moving as much as possible for environmental consideration.

### 2.3.2 Stabilization

Soil stabilization is a method by which the geotechnical characteristics of the virgin soil get improved. Improving the quality of soil can be achieved through mechanical or chemical process.

Stabilization of the existing soil will normally be a much more sustainable solution than soil replacement option [79]. The stabilization mechanism may vary widely from the formation of new compounds binding the finer soil particles to coating particle surfaces by the additive to limit the moisture sensitivity [79]. Therefore, a basic understanding of the stabilization mechanisms involved with each additive is required before selecting an effective stabilizer suited for a specific application [79].

The most important factor in the initial timely stabilization of clayey soils is the ability of the stabilizer to supply an adequate amount of calcium. Both Portland cement and lime can supply this necessary ingredient, and both, when used properly, can effectively stabilize clay soils [80].

The main aims of soil stabilization are altering strength, bearing capacity, stiffness, compressibility, permeability, workability, swelling potential and volume change behavior of natural soil so that the treated soil responds to any particular set of engineering requirement [81].

Long-term performance of pavement structures is significantly impacted by the stability of the underlying soils [79]. In situ sub-grades often do not provide the support required to achieve acceptable performance under traffic loading and environmental demands. Although stabilization is an effective alternative for improving soil properties, the engineering properties derived from stabilization vary widely due to heterogeneity in soil composition, difference in micro and macro structure of soils, heterogeneity of geologic deposits, and due to differences in physical and chemical interactions between the soil and candidate stabilizers [79].

Stabilization of expansive soils using different industrial and agricultural waste for enhancement of engineering properties and to create scope for best utilization of abundantly available waste material at a substantially low cost is a recent trend.

#### 2.3.2.1 Mechanical Stabilization

Mechanical Stabilization is the process of improving the properties of the soil by changing its gradation. This method includes soil compaction and densification by application of mechanical energy using various sorts of rollers, rammers, vibration. Compaction is defined as the rearrangement of primary particles into a state of higher bulk density-and lower porosity when a load is applied to a soil [82]. Compaction leads to the loss of pore space between aggregates as the soil volume is decreased. The loss of inter-aggregate pore space has a major effect on water infiltration and drainage [82]. Densification occurs as air is expelled from soil voids without much change in water content. The stability of the soil in this method relies on the inherent properties of the soil. In particular, this method is effective for cohesion-less soils where mechanical compaction cause interlocking of soil particles by rearrangement. But, the technique may not be effective if these soils are subjected to significant moisture fluctuations. Also with increase in fine content, fraction smaller than about 75 µm, the efficiency of compaction may also diminish. This is because cohesion and inter particle bonding interferes with particle rearrangement during compaction [79].

Mechanical stabilization can also be achieved by inducing nailing or sand columns or by inclusion of reinforcement in the soil. Mechanical stabilization results in increase of the strength as well as reduction in settlement of soil. Altering the physio-chemical properties of fine-grained soils by means of chemical stabilizers/modifiers is a more effective form of durable stabilization than densification [79].

#### 2.3.2.2 Chemical Stabilization

Chemical stabilization involves the inter-mixing of a chemical binder within a body of soil with the intent to improve the engineering performance of the treated material [83]. Chemical stabilization improves soil strength and stiffness and also enhances durability. More importantly, it also limits the volumetric swelling-shrinkage of the soils [32].

Chemical stabilization may only be effective for shallow sub-grades, for deep expansive profiles, it cannot completely eliminate the damage that can be caused by seasonal movements, however reduces the problem to a limited extent [50].

Lime and cement are the most widely used and effective additive for expansive soil stabilization [84, 47, 71]. These additives are produced from industrial processes and are associated with the emission of greenhouse gases such as carbon dioxide ($CO_2$), methane ($CH_4$), and nitrous oxide ($N_2O$) [7].

Another drawback of stabilization using lime or cement is their small particle size. Dust can be a problem, and its management is generally inadequate in populated areas. In addition to the high volumetric weight of such additives, which makes them more expensive to transfer, the dosage is altered in places where it is very windy. Moreover, the hydration process is more expensive when done in a plant rather than doing it at the site of applications [85].

According to Gueddouda [86], the study of the treatment of clays using several methods of stabilization (i.e addition of lime, cement, and association lime+ cement, association lime + salt and NaCl salt,) show that for certain combinations the reduction rate in swelling potential is more than 90%.

**Lime**: Traditional stabilizers generally rely on pozzolanic reactions and cation exchange to modify and/or stabilize. Among all traditional stabilizers lime is probably the most routinely used material and it is prepared by decomposing limestone at elevated temperature [79]. Lime stabilization is a form of soil stabilization or a ground improvement technique that has been used in civil engineering projects. Expansive soils treatment using lime to improve its use in construction is an advanced method and already proved technique that has been used throughout the world for decades [84].

Clay soil can be treated by the addition of small percentages, by weight, of lime, thereby enhancing many of the engineering properties of the soil and producing an improved construction material [84]. When the soil is stabilized with lime, the lime reacts with clay minerals to instigate a series of physico-chemical processes which improves the engineering properties of clay soils [83]. For most clay soils, chemical reactions between lime and clay minerals will continue for months and years, leading to ongoing, gradual improvements in strength. The processes responsible for the rapid change in properties are different to those that cause the long term strength increase; however, both processes have the potential to substantially influence the durability of a treated soil [87].

Lime stabilization is most often technique used to improve sub-base and sub-grades in a road construction compare to other techniques [84]. This technique is also found to be an effective approach to reduce the distress on pavements [64].

There are different types of lime; however, hydrated lime and quick lime are the most commonly used types to stabilize fine-grained soils. The quantity of lime used to stabilize most soils usually is in the range from 5 to 10% [88].

Lime-soil reactions that lead to the improvement of clay soil's property involve two primarily steps: primary reaction (short term treatment) and secondary reaction (long term treatment) [88]. The earlier one occurs within a few hours or days after lime is added, and at this stage three main chemical reactions occurs, namely, cation exchange, flocculation and agglomeration, and carbonation that bring about rapid textural and plasticity changes [79]. The later one takes months to years to occur and the main reaction at this stage is pozzolanic reaction (see Figure 2.16). Improvement in soil properties such as workability, decrease in plasticity index, decrease in volume change and decrease in clay size particles are attributed to rapid effects. While the increase in soil strength and the durability is attributed to effects of long term treatment. Although pozzolanic reaction processes are slow, some amount of pozzolanic strength gain may occur during the primary reactions [79].

The clay minerals have negatively charged sites on their surfaces which adsorb and hold cations by electrostatic force. Cations are positively charged ions such as calcium (Ca2+), magnesium (Mg2+), and potassium (K+). The capacity of the soil to hold on to these cations called the cation exchange capacity (CEC). In the cation exchange and reactions, the monovalent cations generally associated with clays are replaced by the divalent calcium ions [88].

Flocculation is the process of clay particles altering their structure from a flat, parallel structure to a more random orientation [88]. Agglomeration is thought to occur as the flocculated clay particles begin to form weak bonds at the edge-surface interfaces of the clay particles, because of the deposition of cementitious material at the clay-particle interfaces [80]. Flocculation and agglomeration produces a change in the texture of clay soils from that of a plastic, fine grained material to that of a granular soil, clay particles tend to clump together to form larger particles, thereby improving Atterberg limits and workability properties of a soil [88].

Carbonation is a partial or complete transformation of calcium hydroxide to carbonate phases due to reaction with carbon dioxide.

Pozzolanic reaction between soil and lime involves a reaction between lime and the silica and alumina of the soil to form cementing material [88].

Lime is considered effective for stabilization of soils with fine contents more than 25% because it makes the soil more variable with less plastic and hence easier to work [89]. Lime, either alone or in combination with other materials, can be used to treat a range of soil types. The reactivity of lime is dependent on the mineralogical properties of the soils, and the strength gains of soil is mainly by the chemical reactions between the lime, clay minerals and amorphous constituents in the soil [89].

Lime react with most of the clay minerals, it has significant effect on the soils containing montmorillonite and less effect on kaolinite containing soils, this is attributed to high cation exchange capacity of montmorillonite compared to kaolinite [84].

**Cement**: Cement is being increasingly used as a stabilizing material for soil, particularly in the construction of highways and earth dams [88]. Granular soils and clayey soils with low plasticity are obviously most suitable for cement stabilization [88]. For these reasons, proper care should be taken in the selection of the stabilizing material. Cement stabilization is an effective technique for clayey soils when the liquid limit is less than about 50 and it is not suitable for the soils with plasticity index higher than 20%, because of excessive drying shrinkage, lower compressive strength and inadequate stability of these soils [91].

Soft clay, when mixed with cement, will be stabilized because cement and water react to form cementitious calcium silicate and aluminate hydrates, which bind the soil particles together. Like lime, cement helps in increasing the strength of soils, and the strength increases with a curing time [88].

The principal mechanism of ground improvement is done by forming chemical bonds between the soil particles. When the soil particles are bonded they will be strengthened and become more stable physically and mechanically [86].

Ordinary Portland Cement (OPC) is one of the most successfully used soil stabilizer in reducing soil plasticity and swelling. OPC and soil mixed at the proper moisture content has been used increasingly in recent years to stabilize soils in special situations. The hardening process of cement stabilized soils happens immediately upon mixing soil with cement slurry. The hardening agent produces the hydrated calcium silicates, hydrated calcium aluminates, and calcium hydroxide and forms hardened cement bodies [86].

Cement contents may range from as low as 3% to a high of 16% by dry weight of the soil, depending on the type of soil and properties required [84]. Generally, as the clayey portion of the soil increases, the quantity of the cement required also increases.

The degree of the crystallinity of the minerals and particle size distribution are some of the factors influencing solubility. The high pH environment of a soil cement system increases the solubility and reactivity of the silica and alumina present in the clay particles, a prerequisite for the formation of additional cementing materials is the solution of silica and alumina from clay components [80]. .

Cementitious materials stabilize soils and modify their properties through cation exchange, flocculation and agglomeration, and pozzolanic reactions. Additionally, cement provides hydration products, which increase the strength and support values of the base materials as well as enhance the performance of the treatment [80].

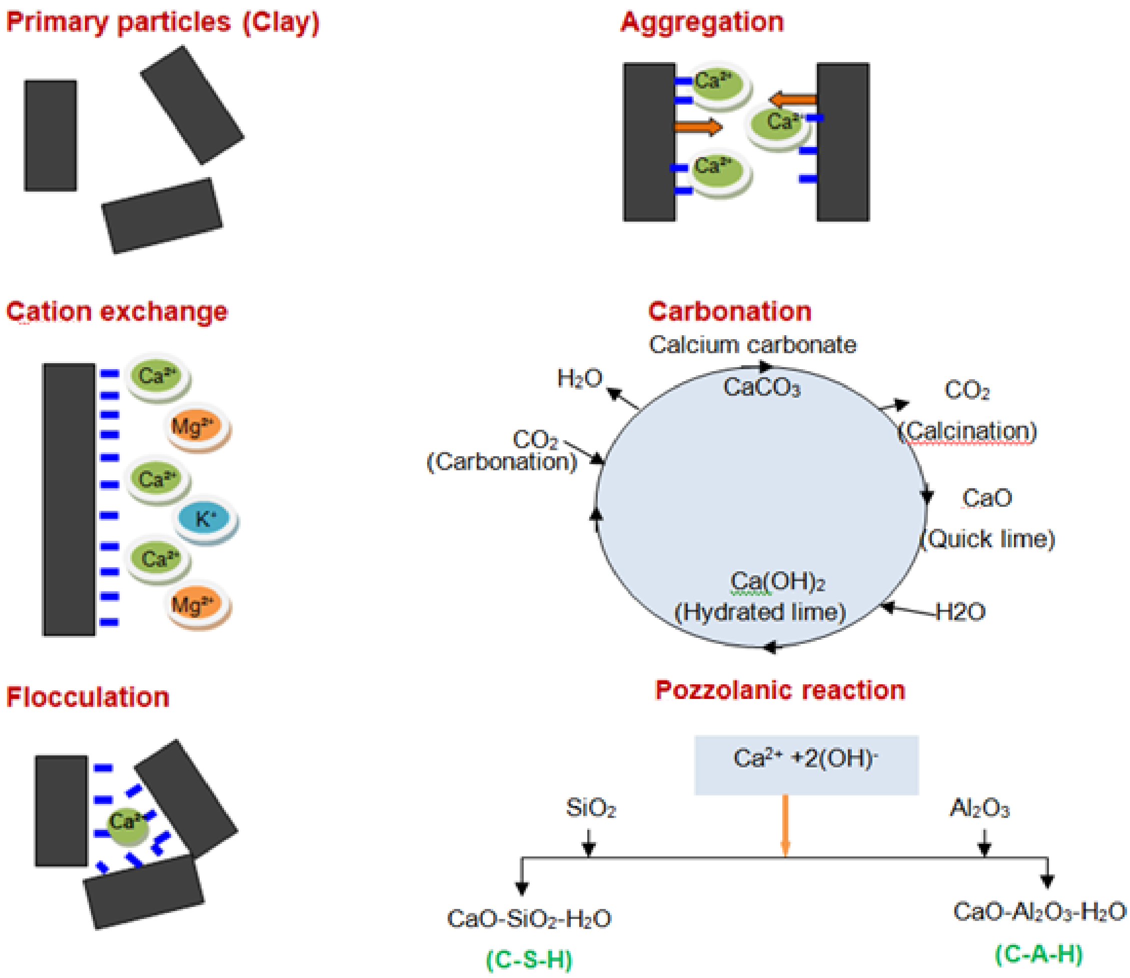



Figure 2.16: Reactions that lead to the improvement of clay soil's property (After [92])

**Industrial and agricultural by-product:** It has always been a great challenge for geotechnical engineers to improve the characteristics of expansive soil through various innovative and cost effective techniques. Stabilization is one of the techniques used to improve the geotechnical properties of the expansive soils. Stabilization can be achieved by using different additives. The traditional construction materials (additives) are being produced from the existing natural resources. This is causing an environmental damage due to continuous exploration and depletion of natural resources. Besides, the industrial and urban management systems are generating solid wastes, and most often dumping them in open fields. Moreover, various toxic substances such as high concentration of carbon monoxide, oxides of sulfur, oxides of nitrogen, and suspended particulate matters are invariably emitted to the atmosphere during the manufacturing process of construction materials [93]. These activities pose serious detrimental effects on the environment, the emission of toxic matters contaminates air, water, soil, flora, fauna and aquatic life, and thus influences human health as well as their living standard [93].

Recent research works in the field of geotechnical engineering and construction materials focuses more on the search for cheaper and locally available materials, industrial and agricultural by-product, to use in a construction industry.

The industrial by-product such as lime kiln dust, cement kiln dust, fly ash, dolochar, ground granulated blast furnace slag (GGBS), pond ash etc are generated from various industries. The industries generally dump the industrial by-product in their vicinity causing environmental hazards. Using these industrial by-product as additives are relatively low cost, additionally, $CO_2$ emissions can be reduced significantly by the increased use of such supplementary materials [7]. Hence, there is an urgent need to explore and exploit the use of the above industrial waste.

Most of the industrial by-product usually coming from the combustion process, such as fly ash and bottom ash, have pozzolanic properties [94]. Pozzolanic reactions take place when significant quantities of reactive CaO, $Al_2O_3$ and $SiO_2$ are mixed in presence of water, to form compounds with having cementitious properties [94]. From different researches, it has been proved that the geotechnical properties of expansive soil can be enhanced by using the mixture of industrial by-product and efficiently lime.

The utilization of different industrial by-product that meant to improve soil engineering properties are reviewed and listed in Table 2.4.

Table 2.4: Utilization of industrial by-product.

| No | Name of industrial waste | Mixture | Utilization potentials | Effect | Reference |
|---|---|---|---|---|---|
| 1 | Fly ash | Lime and Polyester Fibers | Soil stabilization | Improved compaction and strength properties of expansive soil | [95] |
| 2 | Cement kiln dust | - | Soil stabilization | Improved plasticity and UCS of soil | [20] |
| 3 | Granulated blast-furnace slag | Fly ash | Soil stabilization | Improved plasticity and UCS of soil | [7] |
| 4 | Silica fume | - | Soil stabilization | Improved plasticity and swelling potential | [96] |
| 5 | Iron ore tailings | Lime | Soil stabilization /sub-base for low volume roads | Improved strength | [78] |
| 6 | Lime kiln dust | Fly ash | Soil stabilization | Improved bearing capacity | [97] |
| 7 | Copper slag | - | Flexible pavement subgrade | Improved bearing capacity | [98] |
| 8 | Ceramic dust | - | Flexible pavement subgrade | Improved swelling pressure and strength | [99] |
| 9 | Pond ash | Cement | Soil stabilization | Improved strength | [100] |

Annually, large quantities (millions of tonnes) of various agricultural by-product are produced around the world. Agricultural by-product increased annually at an average rate of 5%-10% and the random abandon would result in air pollution, soil contamination, and so on [101].

Several researchers investigated the application of various agricultural waste materials (rice husk ash, bagasse ash, locust bean pod ash, coir waste, burned olive waste) as a stabilizing agent to enhance the properties of expansive soils and to reduce the construction cost [100, 102-107].

The partial or full replacement of cementitious material, lime and cement by pozzolanic materials has a big role in enhancing the properties of the soil, reducing cost and environmental conservation [20-22]. Over the last years, the use of agricultural by-product as a meant to improve soil geotechnical properties are reviewed and listed in Table 2.5.

Table 2.5: Utilization of agricultural by-product.

| No | Name of agricultural waste | Mixture | Utilization potentials | Effect | Reference |
|---|---|---|---|---|---|
| 1 | Rice husk ash | Cement | Highway construction material | Improved bearing capacity | [100] |
| 2 | Bagasse ash | lime | Road sub-grade stabilization | Improved strength | [102] |
| 3 | Coir waste | - | Road sub-grade stabilization | Improved bearing capacity | [103] |
| 4 | Wheat straw | - | Soil property improvement | Improved shear strength | [104] |
| 5 | Ground nut shell ash | Cement | Road construction | Improved strength and durability | [105] |
| 6 | Burned olive waste | - | Soil stabilization | Improved plasticity , swelling pressure and strength | [106] |
| 7 | Palm oil fuel ash | Cement | Soil stabilization | Improved plasticity and strength | [107] |

The engineering properties of the additives are a determining factor for their effectiveness and application in construction. The characteristics of the material contributing on the quality and stability of a structure through the addition into the soil must satisfy the engineering requirement. Since stabilization of soils by using only lime or cement is costly, the use of by-product along with very little lime or cement is a preferred option economically. The potential use of waste materials as a stabilization agents offer two major advantages over traditional construction materials. First, they have a positive effect in reducing the cost related to stabilization due to the fact that they are a waste product that already needs to be disposed of. Second, using these waste materials reduces the disposal problems and environmental concerns related to it.

## 2.4 Coffee Husk

Coffee husk is a waste material obtained from coffee processing. Inapplicability of this solid residue is resulting in disposal and environmental problems.

Properties of coffee by-products are less known and less research has been conducted on how to use these waste materials in an effective manner. Combustion of this type of waste material is a common practice in farms. The coffee husk is becoming a worrying factor for environmentalists [108].

The bulk density of the coffee husks is low, raising the costs of transportation and storage. Furthermore, it is not suitable for a long distance transportation [109]. Coffee husks are therefore more suitable for firing within the production region. Due to the low contents of sulphur, SO2 emission problems would not be expected during coffee husks combustion [109].

### 2.4.1 Description and Production

Coffee is one of the most important agricultural commodities produced and commercialized worldwide [110]. Coffee arabica and coffee robusta are the two principal varieties of the genus cultivated all over the world for commercial production [111].

Coffee beans are processed in two ways, wet and dry processing (see Figure 2.17). Dry processing is the simplest technique for processing coffee cherries; after harvesting, the coffee cherries are dried, thereafter; the coffee beans are separated by removing the material covering the beans in a de-hulling machine [109].

Wet processing, on the other hand does not require drying of the cherries. Here, first the outer skin and pulp are mechanically removed and then the beans obtained are fermented to remove a layer of remaining pulp material [109]. The dry method is used for about 95% of Arabica coffee produced in Brazil, Haiti, Indonesia and Paraguay, and some Arabica produced in India, Ecuador and most coffee produced in Ethiopia [112]. In Ethiopia dry processing accounts for about 71% of all processed coffee [113]. Ethiopia is the origin of coffee Arabica and listed among top coffee producing countries in the world [114]. Ethiopia produces coffee abundantly, 393,000 metric tons coffee from an area of 532,000 ha, in the year 2017/2018 [115]. Coffee production is important to the Ethiopian economy with about 15 million people directly or indirectly deriving their livelihoods from coffee [116]. Coffee is also a major Ethiopian export commodity and it has economical, environmental as well as social significance to the country.

Most of the coffee production areas and processing plants in Ethiopia are found in two regions, Oromia and the Southern Nations, Nationalities, and People Regions (SNNPR), in the south and west of the country.

Coffee processing generates significant amounts of agricultural waste (coffee husks, peel and pulp), ranging from 30% to 50% the weight of the total coffee produced [117]. There may be difference in percent composition of the constituents, depending upon the processing mode and efficiency, crop variety, cultivation conditions such as soil type, etc [111].

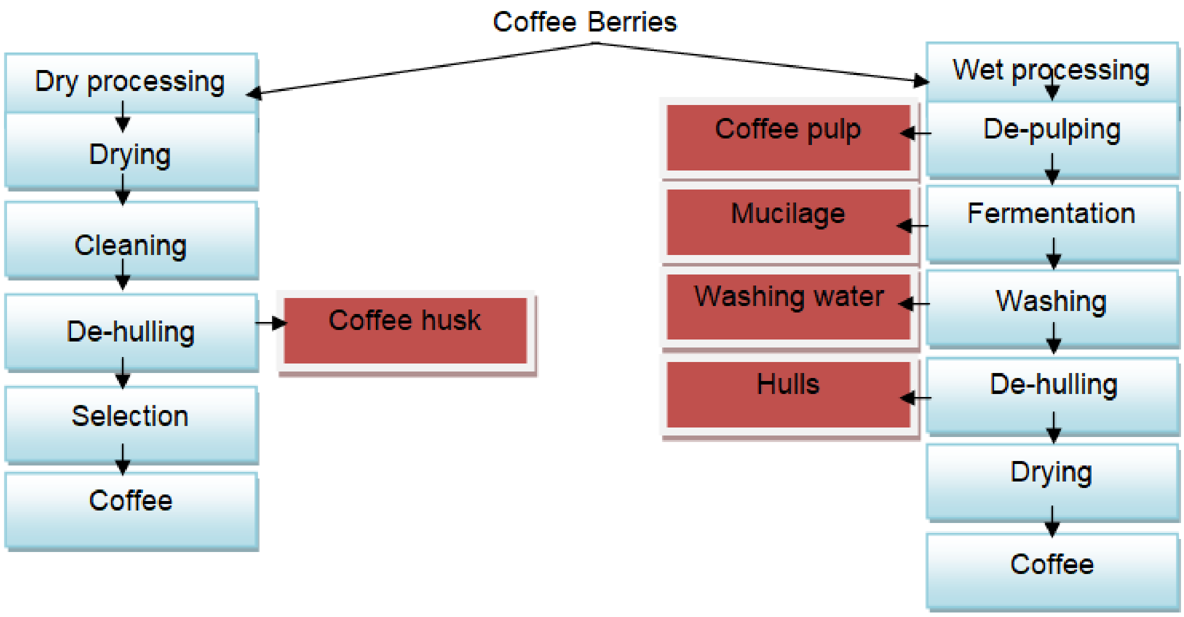



Figure 2.17: Dry and wet coffee processing (After [110])

### 2.4.2 Disposal of Coffee Husk

In Ethiopia the utilization of coffee husk generated during coffee production is still poor. Currently, coffee pulp generated during wet processing is discharged into local streams and rivers where it tends to produce a highly acidic effluent which pollutes the water, destroying aquatic life and generating an offensive odour [118].

Coffee husk contains some amount of caffeine and tannins, which makes it toxic in nature, resulting in disposal problems [110]. Caffeine is an active compound, one of the nature's most powerful and addictive stimulants. It is the principal substance causing the mild stimulation effect of coffee. It is also present in coffee pulp and husk at about 1.3% concentration on dry weight basis [111]. Tannins are generally thought to be an anti-nutritional factor and prevent coffee pulp from being used at greater than 10% of animal feed [111]. Coffee husk contains more than 9% phenolic compounds, and as such, their direct release into the environment could inhibit plant root growth and lead to an increase in greenhouse gas emissions through anaerobic decomposition [116].

### 2.4.3 Application of Coffee Husk

Since more than 30% of the coffee fruit is not used for production of the commercialized green coffee and, therefore, is discarded during processing, it should be interesting to find applications for these byproducts [117].

Coffee husk is rich in organic matter, which makes it an ideal substrate for microbial processes for the production of value-added products, such as fertilizers, livestock feed, compost, etc. However, these applications utilize only a fraction of available quantity and are not technically very efficient [110].

Bekalo and Reinhardt [119] studied utilization of coffee husk and their study showed that a partial replacement of wood by coffee husk and hulls (which contain a great amount of cellulose and hemicellulose which makes them similar to wood) up to 50% is possible at the production of particleboards

The water drained from coffee cherry extract is another potential source of biogas production. According to the estimates, from one ton of coffee pulp, about 131$m^3$ of biogas could be produced by anaerobic digestion [110,117]. In addition, coffees husk present excellent potential for residue-based ethanol production [120].

The potential use of coffee husks in the production of mushrooms is an option for recovering this by-product because of the fungi's ability to degrade these compounds and convert the lignocellulosic residues to food with high commercial and nutritional value. Moreover, with the use of these fungi, this by-product can also be used to enrich mushrooms with selenium or other minerals important for human health and to produce lignocellulolytic enzymes [112].

Coffee husk ash constituted mainly of calcium and potassium [108,121]. This makes it interesting to investigate the possibility of using this material for soil stabilization.

The possible utilization of CHA for soil stabilization can lead to reduction in disposal problem, in addition to reducing economical and environmental concern.

The motive of this study is to investigate the effect of CHA on geotechnical properties of expansive soil specifically BC soil.

# 3 Materials and Methods

This chapter presents methodologies, procedures and experiments conducted in order to investigate the viability of coffee husk ash as an expansive soil treatment agent alone and with lime admixture.

The main experimental investigations conducted in this study comprise a grain size test, Atterberg limit test, free swell test, standard Proctor compaction test, UCS test, CBR test, shear test and consolidation test. They are conducted in conformity with approved standards on untreated and treated soil to evaluate their plasticity, swelling, compaction, strength, durability and compressibility characteristics. In order to determine mineral composition a mineralogical analysis was conducted. A chemical identification and microanalysis were also performed.

The following successive sections will present the properties of the materials, test methods and experimental procedures conducted in this research work.

## 3.1 Expansive Soil

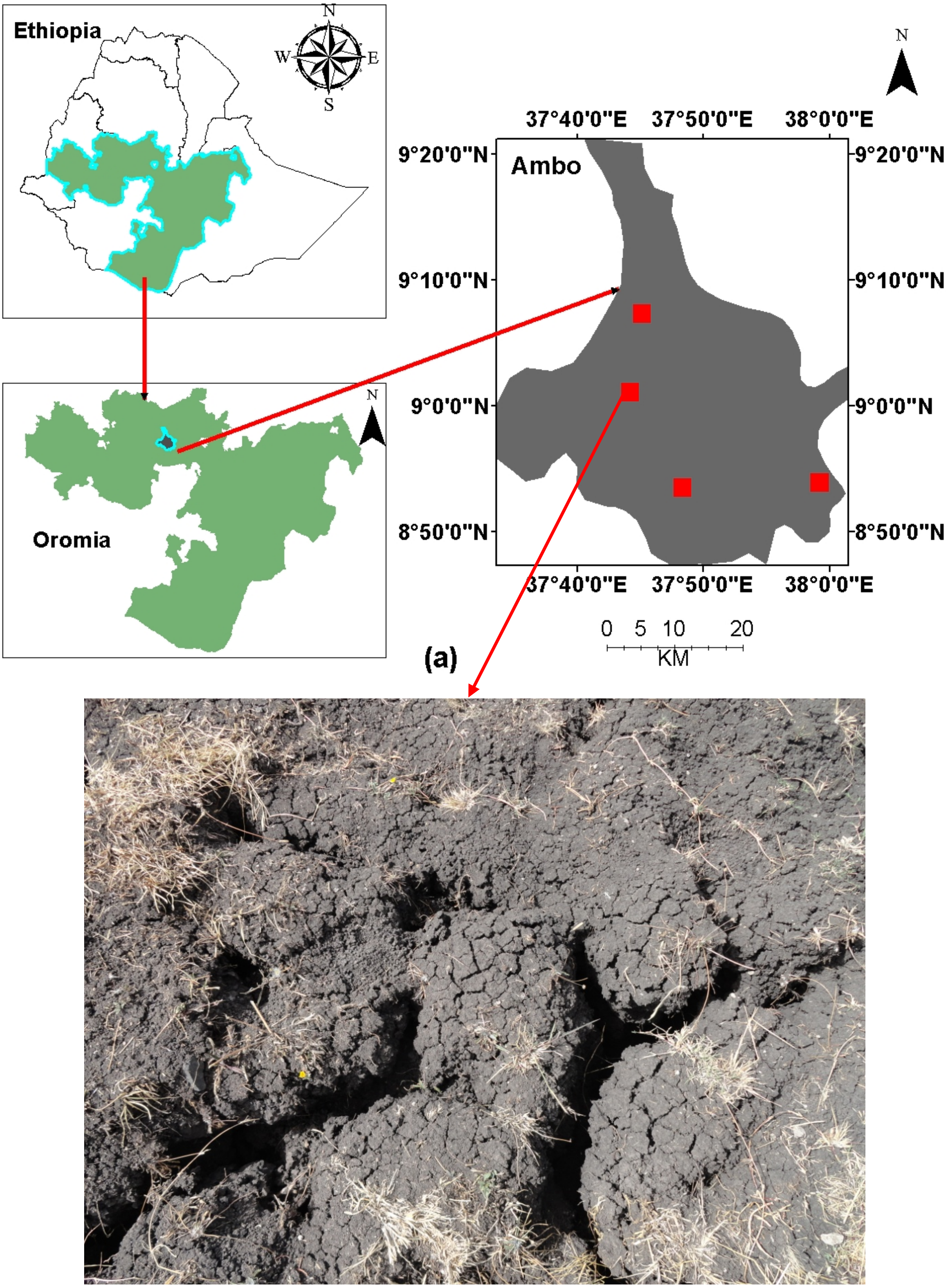
### 3.1.1 Study Area

The site selection and sampling were carried out in west Shoa zone of Oromia state, around Ambo town, Ethiopia. Ambo is located in the central part of Ethiopia around 120km west of Addis Ababa. The geographical location of Ambo town is approximately between $8^{o}$ 56'30'' N - $8^{o}$ 59'30'' N latitude and between $37^{o}$47'30" E - $37^{o}$ 55'15" E longitude with an elevation of 2101 meters. The location where the soil was collected is shown in Figure 3.1.

For this study, so many samples from so many sampling areas were collected from different sites in and around Ambo town. Then representative soil sample was selected and collected in such a way that the sample represents a wide spectrum of factors.

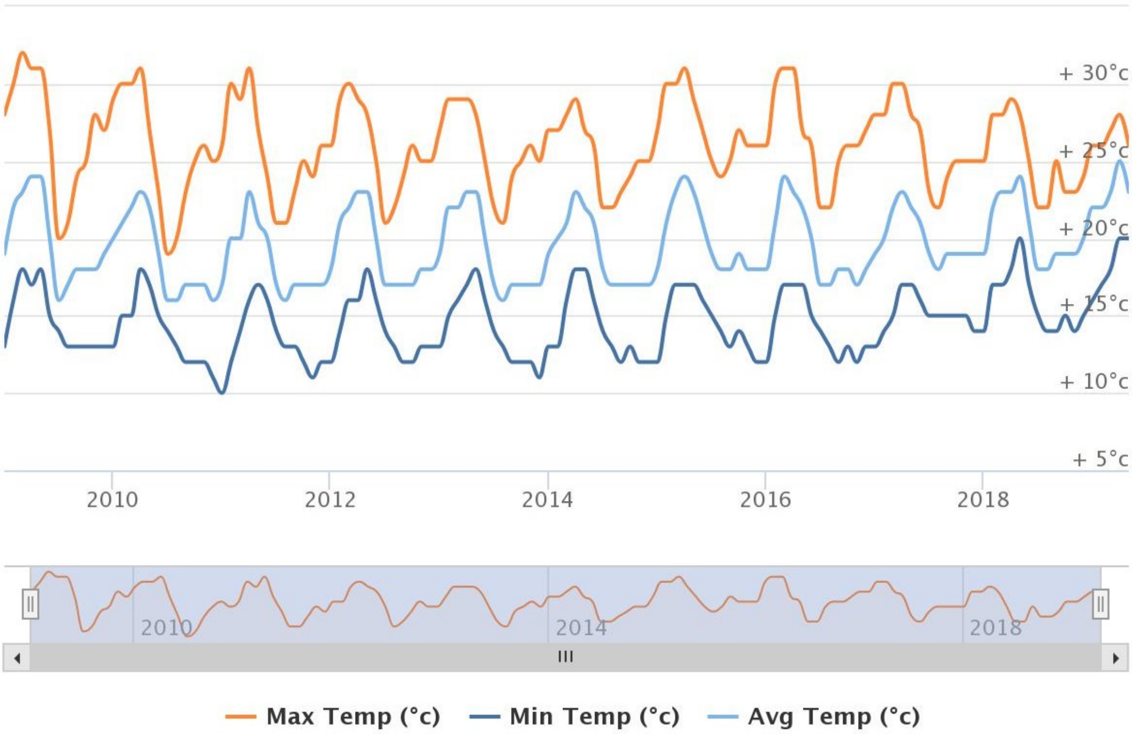
### 3.1.2 Climate Conditions of the Study Area

The volumetric change behaviour of BC soil is dependent on the climate conditions. As a result the knowhow of an interest area climate condition is a crucial factor to be observed before any engineering construction will be constructed on this soil. There are two main seasons in the sampling area (Ambo), dry and wet. In the wet season (June to September), there is a heavy rainfall. BC soil swells, in the periods of precipitation by absorbing water, and shrinks, in the dry season when water content reduces due to evaporation. Temperature and rainfall of Ambo area is shown in Figure 3.2a and Figure 3.2b respectively.

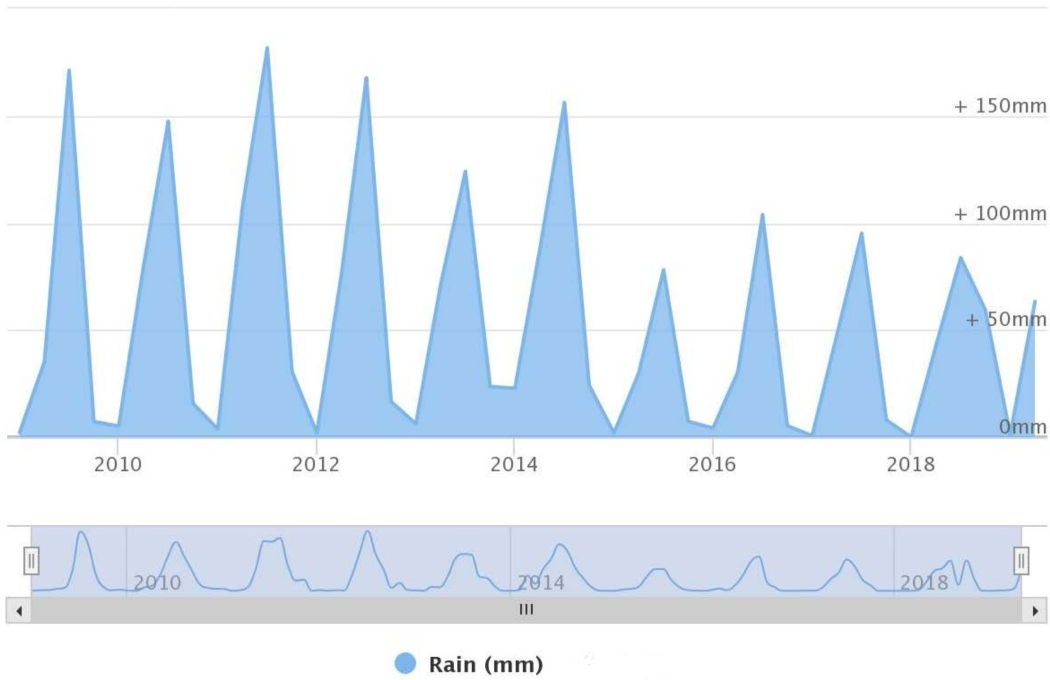



(b)

Figure 3.1: Location map of sampling area (a) (source: [122]) and BC soil in dry season (b)




(a)




(b)

Figure 3.2: Temperature and rainfall of Ambo area in the years 2010-2018 (source: [123])

### 3.1.3 Characterization of BC Soil

To characterize the sample, different laboratory tests such as particle size distribution, Atterberg limits, swelling index, specific gravity and permeability tests were performed according to the guidelines of the ASTM and the index properties of the tested soil are given in Table 3.1. Grain size distribution and grain shape influence the geotechnical properties of the soil. The studied soil has a clay content of 54.8%. The clay fraction has a significant influence on many physical and chemical processes that occur in the soil, because the small particles have such a large and reactive surface area. In addition, as the clay fraction (finer than 0.002 mm in soil diameter) increases, the swelling potential increases [124]. Further classification was done using Atterberg limit results, using a plasticity chart, and the results show that the soil is classified as A-7-5 and CH according to AASHTO and USCS, respectively. Figure 3.3 shows the grain size distribution of BC soil, CHA and lime.

Table 3.1: Index property and classification of the BC soil.

| | | |
|---|---|---|
| Grain size | Gravel% | 4.7 |
| | Sand% | 10.1 |
| | Silt% | 30.33 |
| | Clay% | 54.87 |
| Atterberg limits | Liquid limit | 93.4 |
| | Plastic limit | 40.46 |
| | Plasticity index | 52.94 |
| Soil classification | AASHTO | A-7-5 |
| | USCS | CH |
| | Colour | Black |
| Specific gravity | Gs ($g/cm^3$) | 2.685 |
| Coefficient of permeability | K ($ms^{-1}$) | $2.885x10^{-11}$ |
| Activity of soil | A | 0.96 |
| Swell characteristics | Free swell ratio | 2.5 |

The MDD and OMC were determined from compaction curves corresponding to standard Proctor results and the average was taken.

Unconfined compression tests and CBR tests were conducted using the data obtained from standard Proctor test (MDD and OMC) according to the guidelines of ASTM and the geotechnical properties of the tested soil are given in Table 3.2. To characterize the compressibility of the BC soil one dimensional consolidation tests was performed and the corresponding results are presented in Table 3.2.

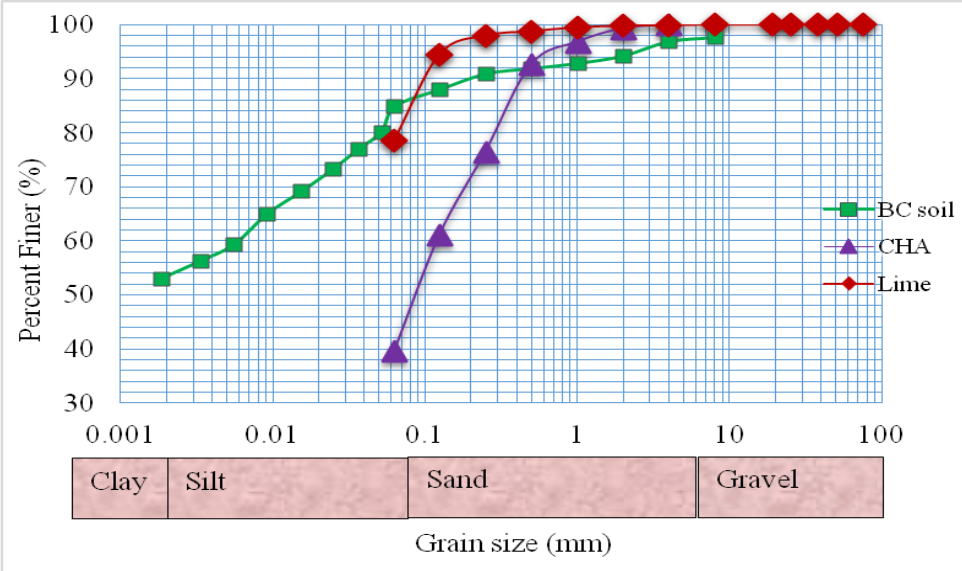



Figure 3.3: Grain size distributions of the BC soil, CHA and lime

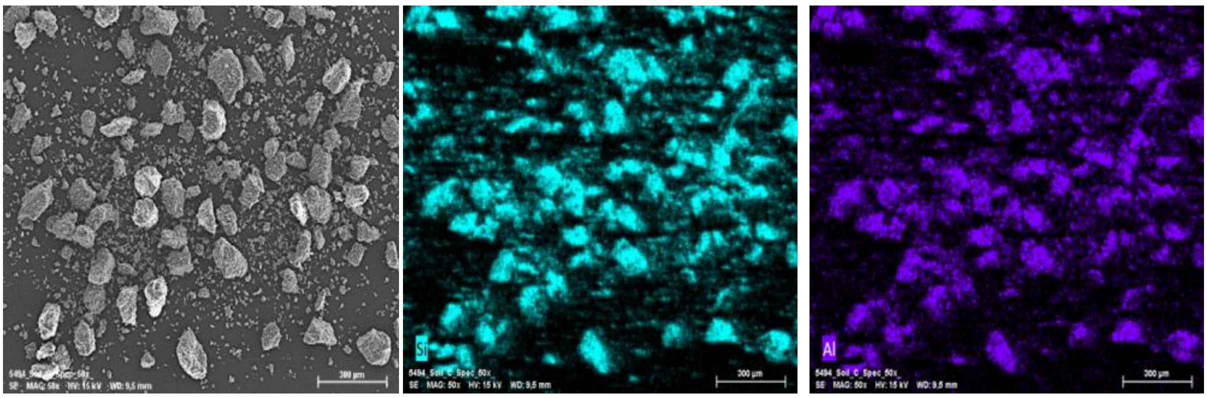
Table 3.2: Geotechnical properties of the BC soil.

| | | |
|---|---|---|
| Compaction characteristics | Optimum moisture content (%) | 37.2 |
| | Maximum dry density ( $gcm^{-3}$) | 1.242 |
| UCS | UCS (KPa) | 81.16 |
| CBR | CBR Un-soaked (%) | 8,30 |
| | CBR Soaked (%) | 1,02 |
| | CBR Swell (%) | 10,08 |
| One dimensional consolidation | Compression index | 0,41 |
| | Recompression index | 0,08 |
| | Swell index | 0,06 |
| | Coefficient of compressibility, av ($m^2/MN$) | 1,21 |
| | Coefficient of volume compressibility, mv ($m^2/MN$) | 0,55 |

Chemical analysis results of the BC soil are presented in Table 3.3. To study the microstructure of the BC soil SEM was used. The BC soil mainly composed of Silica and Alumina as shown in Figure 3.4.

Table 3.3: Chemical composition of BC soil.

| Oxide | Value (%) |
|---|---|
| Silica ($SiO_2$) | 49.18 |
| Alumina ($Al_2O_3$) | 13.30 |
| Iron oxide ($Fe_2O_3$) | 7.80 |
| Calcium oxide (CaO) | 6.32 |
| Magnesium oxide (MgO) | 2.28 |
| Sodium oxide ($Na_2O$) | 0.12 |
| Potassium oxide ($K_2O$) | 1.28 |
| Manganese oxide (MnO) | 0.24 |
| Titanium dioxide ($TiO_2$) | 0.44 |
| Phosphorpentoxid ($P_2O_5$) | 0.08 |
| Loss on ignition (LOI) | 10.90 |




BC soil Si Al

Figure 3.4: SEM images showing Si and Al distribution maps of BC soil

## 3.2 Coffee Husk Ash (CHA)

The coffee husk used for this study was collected from farmlands and factories and kept at 550°C in a furnace for five hours to get the resulting ash. The sample and the properties of the CHA that are used in this study are shown in Figure 3.5 and Table 3.4 respectively.

Table 3.4: Properties of CHA.

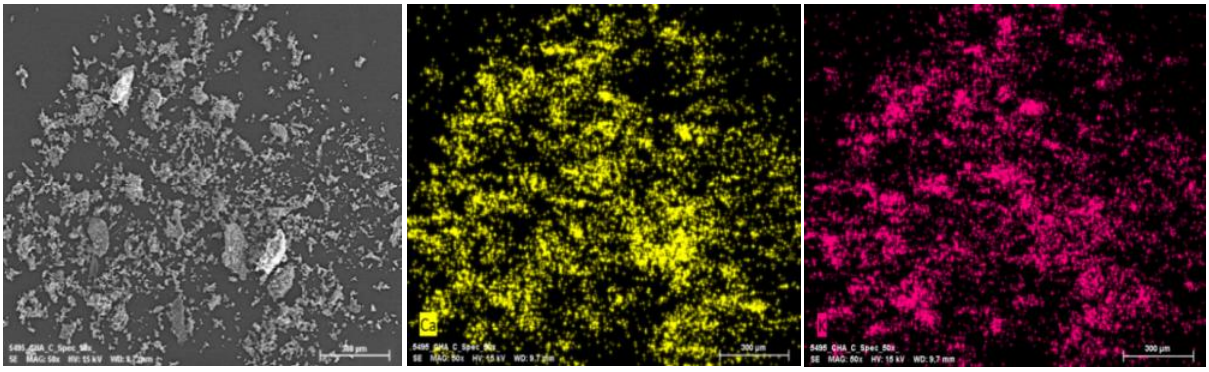
| | | |
|---|---|---|
| Colour | | Light grey |
| Size | <75 μm | 42.9% |
| | 75-425 μm | 46.26% |
| | 425 μm-2 mm | 10.17% |
| | 2-4.75 mm | 0.67% |
| Plasticity | | Non plastic |
| Specific gravity | | 2.03 |

Figure 3.5: CHA used for the study

According to Acchar et al. [108], the main minerals compositions of a CHA are calcium carbonates, calcium silicates, calcium phosphate and potassium sulphate. They also determined the chemical composition of CHA using X-ray fluorescence, as presented in Table 3.5. Microstructure of CHA as observed using SEM and the CHA mainly constitutes potassium and calcium (see Figure 3.6).

Table 3.5: Chemical compositions of CHA (After [108]).

| Oxide | Value (%) |
|---|---|
| Silica (SiO2) | 1.24 |
| Alumina (Al2O3) | 0.58 |
| Iron oxide (Fe2O3) | 0.56 |
| Calcium oxide (CaO) | 17.70 |
| Magnesium oxide (MgO) | 4.51 |
| Sodium oxide (Na2O) | 0.14 |
| Potassium oxide (K2O) | 46.46 |
| Manganese oxide (MnO) | 0.06 |
| Titanium dioxide (TiO2) | 0.08 |
| Phosphorpentoxid (P2O5) | 3.85 |
| Sulphur oxide (SO3) | 3.75 |
| Loss on ignition (LOI) | 21.07 |



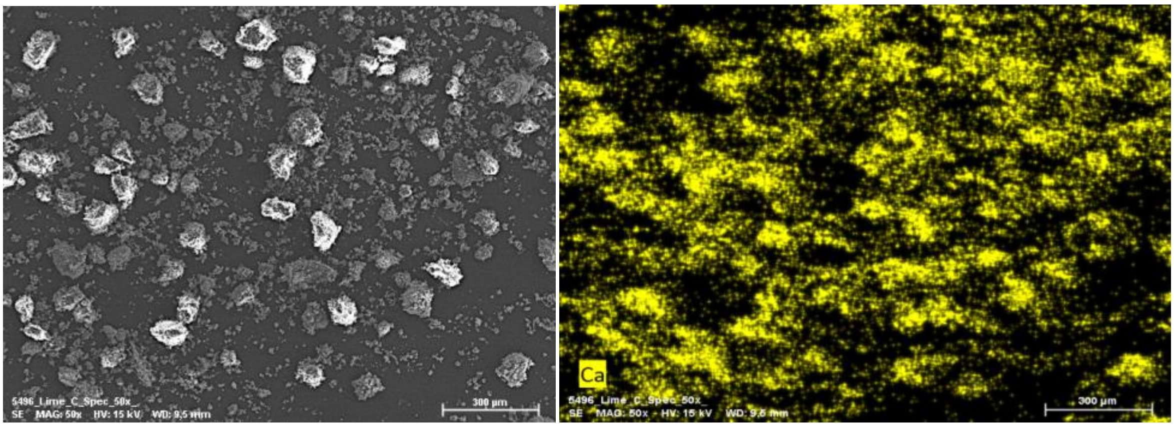
CHA Ca K

Figure 3.6: SEM images showing K and Ca distribution maps of CHA

## 3.3 Lime

There are different forms of lime, including quicklime (made by heating limestone ($CaCO_3$+ heat → $CaO$+ $CO_2$) and hydrated lime (produced by treating quicklime with water ($CaO+H_2O$ → $Ca(OH)_2$) [89]. Soil stabilization using lime in the quick or hydrated form in order to improve the properties of poor soil is an obvious technique. Hydrated lime reacts with clay particles and permanently transforms them into a strong cementitious matrix [90]. According to ASTM [125], hydrated lime used for soil stabilization shall not have more than 3% retained on a 590 µm sieve and not more than 25% retained on a 75 µm sieve. As shown in Figure 3.3, the hydrated lime used for this study met the grain size requirement for soil stabilization.

The lime used in this investigation is hydrated lime obtained from the Senkele lime factory in Ambo, Ethiopia. Figure 3.7 shows the microstructure of lime. The chemical compositions of the hydrated lime used in this study are presented in Table 3.6 below. Energy-dispersive X-ray (EDX) spectrums of the materials (BC soil, lime, and CHA) used in this study is shown in Figure 3.8. From this result, the main elements present in the BC soil are silicon (Si) and aluminium (Al). Calcium (Ca) and potassium (K) are the main elements in the CHA, and the lime mainly contained Ca which is in agreement with SEM results.




Lime Ca

Figure 3.7: SEM images showing Ca distribution maps of lime

Table 3.6: Oxide composition of the hydrated lime (After [126]).

| Oxide | Composition by weight (%) |
|---|---|
| Silica (SiO2) | 6.21 |
| Alumina (Al2O3) | 2.18 |
| Iron oxide (Fe2O3) | 3.57 |
| Calcium oxide (CaO) | 59.47 |
| Magnesium oxide (MgO) | 3.91 |
| Sodium oxide (Na2O) | 0.61 |
| Potassium oxide (K2O) | 0.79 |
| Titanium dioxide (TiO2) | 0.32 |
| Phosphorpentoxid (P2O5) | 0.20 |
| Manganese oxide (MnO) | 0.27 |
| Sulfur trioxide (SO3) | 0.58 |
| Loss on ignition | 17.04% |

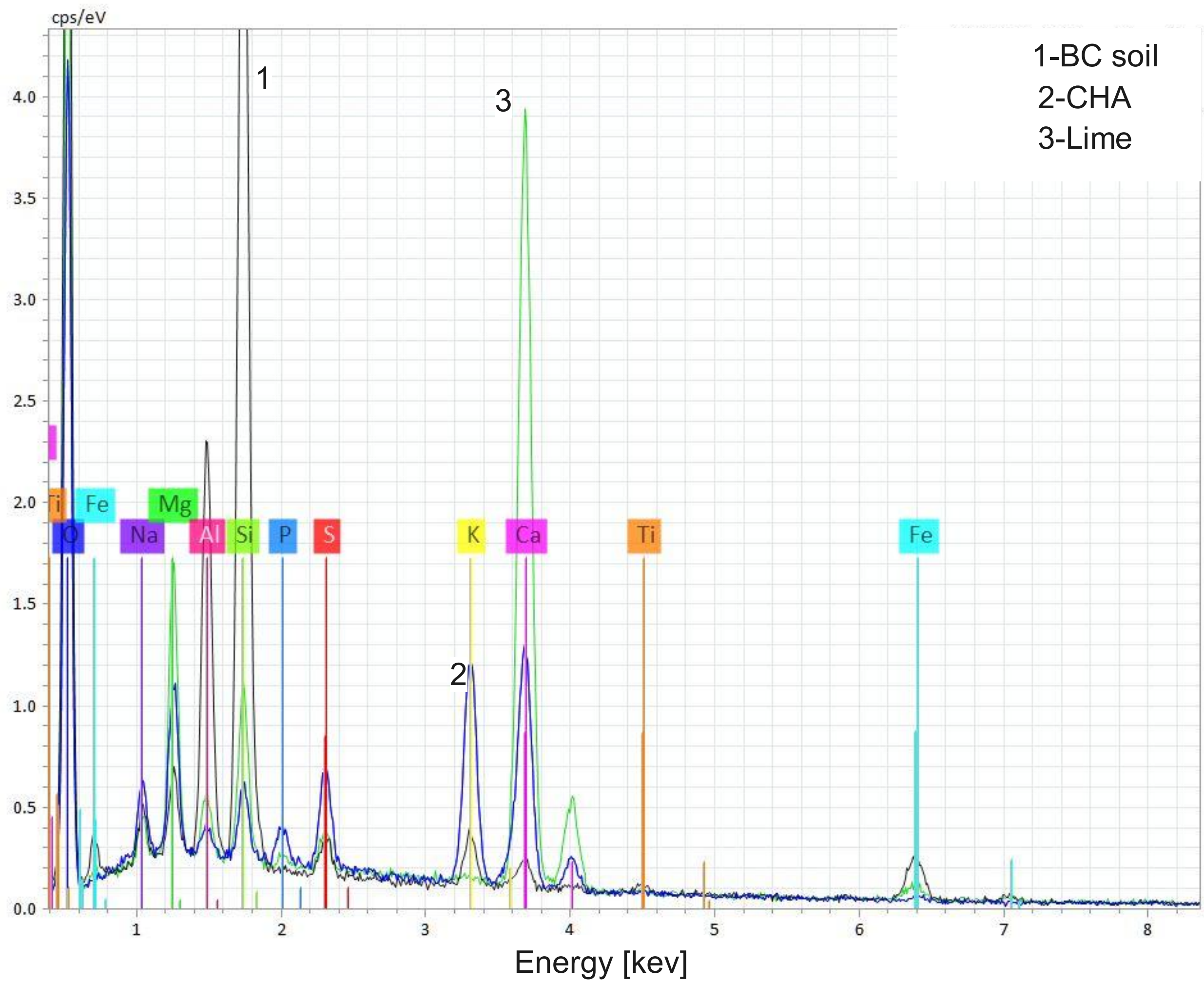



Figure 3.8: EDX spectra of BC soil, lime and CHA

## 3.4 Methodology

All samples are prepared and conditioned before conducting the desired laboratory tests.

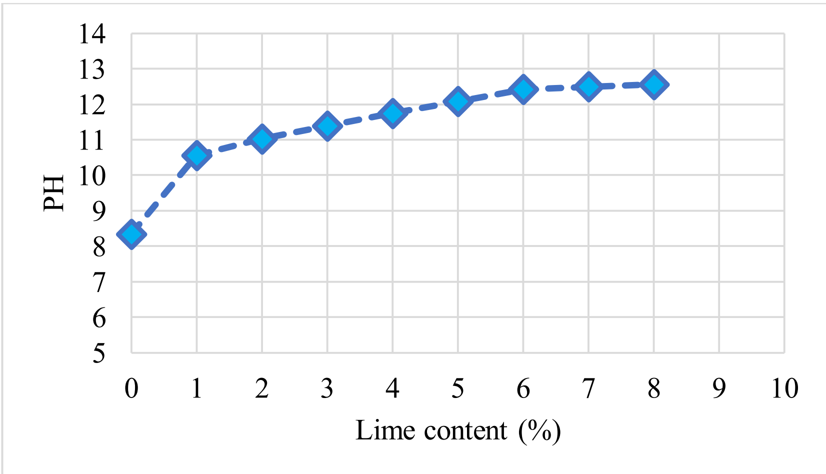
The soil sample passing through the 4.75 mm sieve was taken for the testing. The conditioned CHA, lime and CHA-lime mixture are added to the soil based on the targeted tests. The concentration of lime and CHA is defined by a percentage of the weight of each additive in the dry weight of the soil (presented in Table 3.7). The CHA was added to the soil samples starting with 5% and up to 20% by dry weight of soil mixture with an increment of 5%.

Initial lime consumption (ICL) was estimated using PH test according to ASTM D6276−99a (2006). The lowest percentage of lime in the soil that produces a laboratory pH of 12.4 is the minimum lime percentage required to stabilize the soil [90]. An air-dried sample each equivalent to 25.0 g of oven-dried soil, passed through 425µm was put into a measuring jar (see Figure 3.9a). Then 100mL distilled water was added to the bottles and each of the soil-lime-water mixtures were shake for 30 seconds every 10 minutes until the samples are thoroughly mixed. Then the pH value for each soil-lime-water mixture was recorded. A PH meter shown in Figure 3.9b was used to measure the soil fluid pH in this study.

As shown in Figure 3.9c the amount of the lime to get a PH value of 12.4 is 6% by dry weight of the soil. In this study the amount of lime selected for the BC soil treatment are 4%, 6% and 8% by dry weight of the soil.

(a)


(b)




(c)

Figure 3.9: Soil- lime mixture recorded for PH test (a), PH apparatus (b) and PH values for different lime contents (c)

Different concentrations of lime (4%, 6% and 8%) were added to the soil, and with the mixture of the CHA. Thus, a total amount of 20 mixtures were prepared. The samples are then tested in the laboratory to determine their Atterberg limits, Procter density, specific gravity, unconfined compressive strength, shear strength, CBR, swelling potential and consolidation characteristics.

For each sample, three replicate tests were performed to ensure repeatability of test results. The test results presented in this study are the average of three tested samples.

Table 3.7: Mixture proportion (BC soil (S), CHA (C), lime (L)).

| CHA % | 0%Lime | 4%Lime | 6%Lime | 8%Lime |
|---|---|---|---|---|
| 0 | BC soil | SL4 | SL6 | SL8 |
| 5 | SC5 | SL4C5 | SL6C5 | SL8C5 |
| 10 | SC10 | SL4C10 | SL6C10 | SL8C10 |
| 15 | SC15 | SL4C15 | SL6C15 | SL8C15 |
| 20 | SC20 | SL4C20 | SL6C20 | SL8C20 |

### 3.4.1 Particle Size Distribution

The grain size analysis is carried out for the BC soil, CHA and lime. Materials passing through a 4.75 mm and retained on 75 micron sieves are subjected to sieve analysis method, whereas, the hydrometer analysis method is adopted for the soil particles passing through a 75 micron sieve. Wet sieving tests were performed to obtain the grain size distribution of fine particles. The tests were performed according to ASTM D 422–63. The representative test samples were dried weighed to the nearest 0.01 g. The weighed samples were soaked in water for 24 hours (at a room temperature). To prevent the sample's particles coagulation, sodium hexametaphosphate was added into the immersed samples. After the complete soaking of the sample, the samples were stirred and washed through the sieves using running water. Samples are continuously washed till clear water through the 75 micron sieve is obtained. Then the washed samples are carefully kept in the oven for drying. Samples are dried at 105$^{o}$C temperature till the state of constant mass is obtained. After drying, the fractions retained on the sieves are weighed and results noted down to determine the size of the particles.

Hydrometer analysis is based on the principle of sedimentation of soil grains in water. When a soil samples is dispersed in water, the particles settle at different velocities, depending on their shape, size, weight, and the viscosity of the water. It is assumed that all the soil particles are spheres and that the velocity of soil particles can be expressed by Stokes'law, larger particles fall more quickly in a suspending fluid, while finer particles remain in suspension for a longer time.

To determine the silt and clay fractions, the portion with a grain size less 75µm were taken for the hydrometer analysis. Then mixed with distilled water to make the soil slurry, sodium hexametaphosphate is added and stirred. The soil slurry is transferred to 1000 ml measuring cylinder then distilled water is added until the jar is 1000ml (see Figure 3.10). Then thoroughly stirred and allowed to settle prior to the decantation of the suspension. After the hydrometer is allowed to float freely in the soil suspensions in the cylinder, the consecutive hydrometer readings were taken. The reading on the hydrometer determines the amount of that size, while the time at which the hydrometer readings are taken determines the size of the particle remaining in the suspension.

After taking the reading, hydrometer is removed slowly, rinsed in distilled water and kept in the nearby 1000 ml measuring cylinder containing water with sodium hexametaphosphate solution at the same temperature as that of the soil suspension.

Then the hydrometer is re-inserted in the soil suspension, the process was repeated several times, and the final reading was taken after 24 hours.

The grain size curve has been plotted to determine the silt and clay contents in the test samples (see Figure 3.3).

Figure 3.10: Hydrometer test in progress

### 3.4.2 Atterberg Limits Testing

One of the main steps for evaluating the geotechnical properties of the fine materials is to determine their Atterberg limits. The Atterberg limits tests were carried out in accordance with ASTM D4318-05. These limits can give the approximate indication of the expansion and shrinkage potential of the soils.

The water content at the boundary between liquid state and plastic state of the soil is termed as liquid limit. On the other hand the water content at the boundary between plastic and semi-solid state is called plastic limit.

The dry soil materials passing through the 425 μm sieve was mixed with distilled water until a thick homogenous paste was formed. The soil paste then was sealed in plastic bags and left to stand for a 24 hours so that the moisture inside the samples will be uniformly distributed.

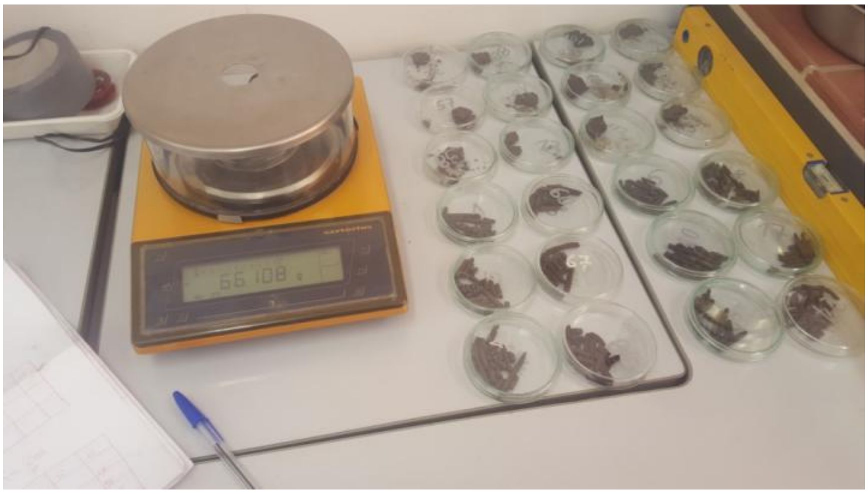
After the completion of 24 hrs, the sample then re-mixed thoroughly before conducting the test. A portion of the prepared paste is placed in the cup of the liquid limit device (see Figure 3.11a), smoothed with spatula to a uniform thickness (10mm). Then cut from back to front using a grooving tool so that the paste is partitioned into two equal halves with a clear gap of 12 mm in the middle. The number of blows causing the groove made by the spatula in the soil to close was counted. Blows are recorded which fall within the range of 15 to 35 blows. A portion of the samples paste is taken from the cup and moisture content is determined. Then a little more distilled water is added to the samples paste remaining in the bowel and is mixed thoroughly. The operations are repeated for different water content.

In each case the number of blows in the manner elaborated above is recorded. A flow curve was plotted for each soil to determine their liquid limit. The water content corresponding to the 25 number of blows is the corresponding liquid limit.

The plastic limit is defined as the moisture content in percent, at which the soil crumbles, when rolled into threads of 3mm in diameter. To determine the plastic limit of the soil, homogeneous paste was allowed to dry in air until it become plastic enough to be shaped into a ball. The soil was then rolled by hand on a ground glass plate to crumble at 3mm diameter (see Figure 3.11b). Then pieces of crumbled samples thread are kept in an air tight container and moisture content is determined which represents the plastic limit of the samples.

Finally, the plasticity index, which characterize the plasticity characteristics of the soil were obtained by subtracting plastic limits from the liquid limits of the samples and the results are given in sections 4.1 and 6.1.


(a)



(b)

Figure 3.11: Plasticity test: a) liquid limit and b) crumbled samples for plastic limit

### 3.4.3 Specific Gravity

Specific gravity is defined as the ratio between the masses of equal volume of soil solids and water. Specific gravity (Gs) is an important parameter in determining weight volume relationships of soils and needed for various calculations in soil mechanics. The specific gravities of the soils were determined in accordance with standard test methods for specific gravity of soil solids by water pycnometer method (ASTM D854-14). A dry soil sample passed through 2 mm sieve was used for the test. The sample is transferred to the density bottle and weighed. Water is added so that the sample in the bottle is just covered. Then entrapped air is removed by heating the density bottle on the water bath. The bottle then taken out of the bath and wiped dry. Then the density bottle is cleaned and filled with air free distilled water (see Figure 3.12) and weighed, and then the specific gravity of the sample is determined, the results are given in section 4.4.

Figure 3.12: Specific gravity tests in progress

### 3.4.4 Compaction

Standard Proctor tests were conducted in accordance with standard test methods for laboratory compaction characteristics of the soil using standard effort (ASTM D698). Compaction test indicates two main factors of the soil, the OMC and the MDD. The soil type and the applied energy on the soil layer are two main factors which primarily affect the compaction of the soil. Air dried samples passed through 4.75mm are put on the tray and water is added starting from 3% by dry weight of the sample. The samples and water are mixed thoroughly to get a homogeneous mixture. The mixture is then compacted in the Proctor mould in equal three layers, each layer being uniformly compacted by applying 25 blows. The process is repeated until a decreased in bulk density is observed. The corresponding water content in the sample is also determined by oven drying method. The moisture content versus dry density curves were plotted from the tests results (see section 4.3 and 6.2). From the curves, the MDD and the corresponding OMC were found.

### 3.4.5 Unconfined Compressive Strength (UCS)

Unconfined compressive strength tests were conducted according to the ASTM D 2166 standard test method for UCS of the cohesive soil. The samples were compacted in the Proctor compaction mould at their respective MDD and OMC. After compaction, the samples were trimmed into another mould and stored at a room temperature and for 1, 7 and 14 days. UCS is taken as the maximum load attained per unit area or the load per unit area at 15 % axial strain, whichever is secured first during the performance of a test. UCS for untreated and treated samples is determined and the results are given in sections 4.5 and 6.3.

### 3.4.6 Swell

The swelling characteristics of the soil samples were determined according to Holtz and Gibbs [36]. The samples passing through 425 µm were used. The free swell test consists of pouring slowly 10 $cm^3$ of oven dried soil into a 100 $cm^3$ measuring jar filled with distilled water (see Figure 3.13). Then by gentle shaking and string with glass rod any entrapped air is removed and the samples are allowed to stand for 24 hours. After 24 hours, readings of volumes are noted. The free swell value is then calculated as the increase in the volume of the soil expressed as a percentage of the initial volume and the results are given in section 4.2.

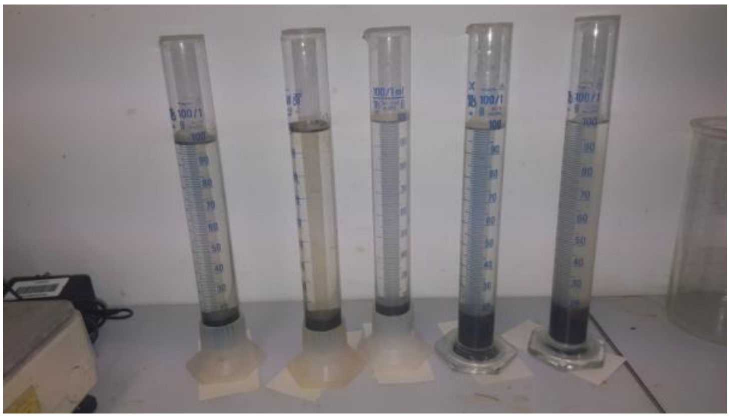


Figure 3.13: Free swell test in progress

### 3.4.7 CBR and CBR-swell

The CBR test was performed in accordance with ASTM D1883−16, the standard CBR test method for laboratory-compacted soils. Samples were blended with an optimum amount of water and then were compacted in the CBR mould. A surcharge mass of 4.5 kg was applied for both soaked and un-soaked states. For soaked CBR tests, samples were completely immersed into freshwater for four days (see Figure 3.14). For un-soaked CBR tests, samples were cured for four days at a room temperature. The treated and untreated samples were prepared for tests, and then a penetration piston with a diameter of 4.9cm was pushed into the samples at a loading rate of 1.27mm/min. The CBR value is obtained by dividing a test load by a standard load at the same depth of penetration (see section 5.1 and 6.4).

Swelling data were also determined from CBR tests. A swell dial gauge was mounted on the CBR mould. Then, the initial height of samples was recorded before immersing the samples in to the water and the final height was taken after samples had been soaked for four days.

Figure 3.14: Soaked samples for CBR tests

### 3.4.8 Direct Shear

The soil and soil-CHA mixture were prepared with their respective optimum moisture content and maximum dry density then compacted in proctor mold. After extruding the samples from the mold, trimmed into the ring with diameter of 71.36mm and thickness of 25mm. The shear strength parameters of soil and soil-CHA mixture were determined by direct shear test, for varying percentage of CHA (0, 5, 10 & 20%).

The hydraulic conductivity of clay is very small compared with that of sand when a normal load is applied to a clay soil samples; a sufficient length of time must elapse for full consolidation that is for dissipation of excess pore water pressure. For this reason the shearing load must be applied very slowly [53]. Samples were consolidated for 24hours and then shearing started at a controlled rate until reaching horizontal strain of 15mm.

The *Mohr-Coulumb* Strength Theory was used for the determination of shear strength parameters, the intercept of a line gives cohesion and the slope of the line is the angle of internal friction (see section 5.2).

### 3.4.9 Consolidation

The consolidation test was performed in accordance with ASTM D2435-11, the standard test method for one-dimensional consolidation properties of soils using an incremental loading. This test is performed to determine the magnitude and the rate of volume decrement that a laterally confined soil sample undergoes when subjected to different vertical pressures.

Samples were compacted in a compaction mould with a dry density equal to a MDD and a water content equal to OMC, obtained from the standard Proctor test result. Compacted samples were trimmed into the consolidation ring in order to prepare for the one dimensional odometer test. A consolidation ring that has a diameter of 7.14 cm and 2 cm thickness was used. The bottom porous stone is placed centrally on the base of the consolidation cell followed by the bottom filter paper, sample with ring, top filter paper and top porous stone one by one. Then the loading pad is placed at the top. A seating pressure of 5 kPa is applied to the sample. The consolidation cell is filled with distilled water and the sample is allowed to reach equilibrium for 24 h. The samples were subjected to a compression and recompression; loading and unloading were conducted by doubling and halving, respectively. The time duration for all load increments and decrements was twenty-four hours, the results are given in section 4.4.

### 3.4.10 Wetting-Drying (W/D)

Durability describes the performance of a civil engineering material used in construction subjected to freeze-thaw and W/D cycles. Depending on the geographical area and climate condition one can perform W/D or freeze-thaw tests; in general, Ethiopia is one of the countries subjected to both wet and dry conditions but not to freezing.

Durability tests were conducted loosely based on ASTM D559 standard test, this procedure helps to simulate the sample’s resistance to the W/D condition of the field in short time frame.

Samples were prepared for durability test at their respective OMC and MDD. The samples were wrapped in plastic bags and cured for seven days at room temperature, prior to subjecting them to wetting and drying cycles (Figure 3.15). After seven days of curing the samples were subjected to W/D cycles, each cycle consists of one wetting and drying condition. The soil samples were submerged in potable water for a period of 5 hours at room temperature, and then removed. To complete one W/D cycle the samples was transferred to an oven heated for 42 hours at 70 °C, then removed, which means that each test requires 2 days to complete. The durability test results are given in sections 4.6 and 6.3.

Figure 3.15: Samples prepared for W/D cycles

### 3.4.11 X-Ray Diffraction (XRD)

The XRD test was performed on untreated BC soil, additives and treated samples. The samples used for XRD test were cured at a room temperature for consecutive 7, 14 and 28 days. At the end of the curing process, samples were oven dried at 105°C for 24 hours, then powdered with a mortar and pestle until the material can pass through 63 μm sieve. Then put in a glass and ready for the test (see Figure 3.16).

The analysis was carried out to identify changes in mineralogical phases present in the samples using Bruker CCD-diffractometer. The samples were scanned at angle of 2θ; range from 5° to 70° was chosen to provide enough XRD peaks to identify the most common minerals. `MATCH! ´(CRSTAL IMPACT) software was used to analyze the data obtained from the diffractometer, and the results are given in sections 5.3 and 6.5.

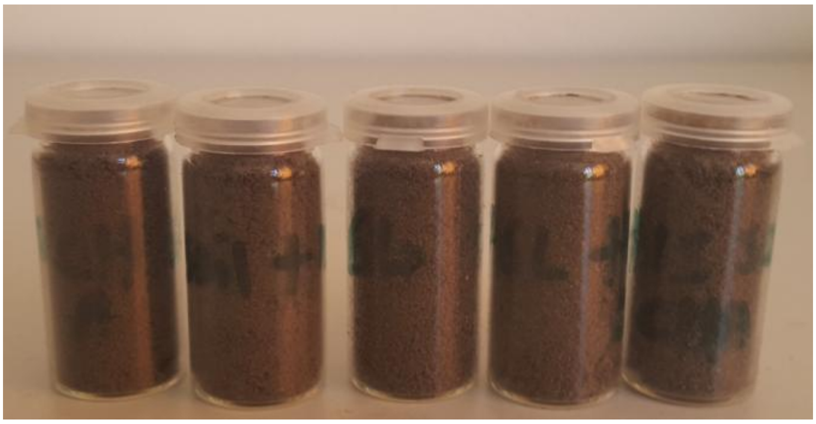


Figure 3.16: Samples ready for XRD test

### 3.4.12 Scanning Electron Microscopy (SEM)

SEM provides a useful tool for visual characterisation of particle structure and bonding agents developed between soil particles due to cementation. SEM analysis was performed on untreated BC soil, additives and treated samples. Samples were cured at a room temperature for consecutive 7, 14, 28 days for micro-structural analysis. At the end of the curing process, samples were oven dried at 105°C for 24 hours, then powdered with a mortar and pestle until the material can pass through 63 µm sieve. Then the samples were placed on the sample holder as shown in Figure 3.17. Samples were analyzed by a field emission SEM (SEM, MERLIN® VP Compact, Co. Zeiss, Oberkochen) equipped with EDX detector (XFlash 6130, Co. Bruker, Berlin).

Representative areas of the samples were analyzed and mapped for elemental distribution on a basis of the EDX-spectra data by QUANTAX ESPRIT Microanalysis software (version 2.0). Samples were mounted on SEM-carrier with adhesive conductive carbon tape and coated with carbon under the vacuum. Conditions of SEM-images like magnification, accelerating voltage, working distance are given in the corresponding images (see sections 5.4 and 6.6).

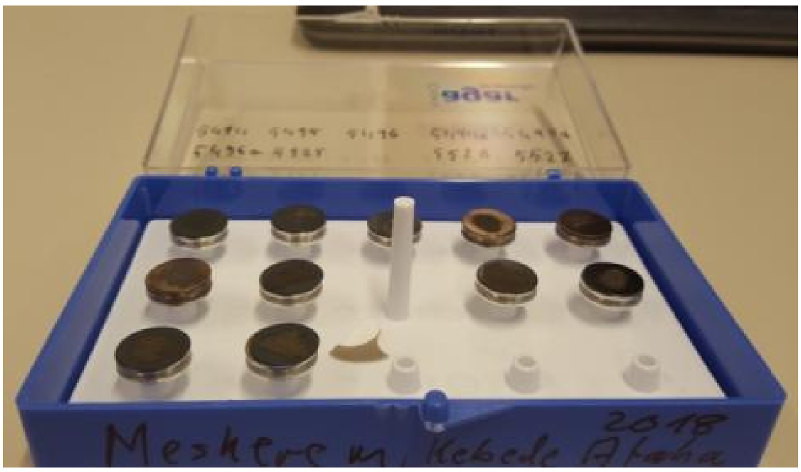


Figure 3.17: Samples placed on sample holder

# 4 Effect of Coffee Husk Ash on Plasticity, Compaction and Unconfined Compressive Strength of Black Cotton Soil

## 4.1 Effect on Plasticity

BC soils are troublesome materials to use in road construction due to their high plasticity behaviour. The studied soil has a LL of 93% and a PI of 52%. One of the important aims of this study was to evaluate the effects of CHA on plasticity behaviour of the BC soil. The effect of CHA on consistency behaviour of the soil sample is determined by conducting Atterberg limit tests. Atterberg limits (LL, PL and PI) indicate some of the geotechnical problems such as swell potential and workability. Thus, Atterberg limits play an important role in soil identification and classification (see Table 2.2).

The plasticity of the BC soil was significantly affected by the addition of CHA, reductions in LL from 93% to 71%, and PI from 52% to 22%, respectively, were observed for addition of 20% CHA (see Figure 4.1-4.3). The addition of 15% CHA alone reduced the PI by 51%. According to USCS (see Figure 4.4), the BC soil is classified as CH. As the content of additives increased, both LL and PI decreased shifting downward on the PI-axis and left on the LL-axis from an area of CH to MH. The improvement of CHA treated BC soil could be attributed to the chemical reaction between CHA and the BC soil. This includes the replacement of naturally carried cations on clay surfaces with Ca2+ cations. The increased $Ca^{2+}$ concentration in pore solution also causes flocculation and agglomeration of clay particles. This interplay reduces the water absorption of the soil particles, making them less plastic [127]. The reduction in the plasticity of the BC soil indicates that it became less sensitive to water, leading to less expansive and better compactable. Consequently, the treated soil gains stability with respect to the deformations due to the seasonal variations of water content. In addition, the decrease in the PI of treated samples indicates an improvement in the workability of the soil, and the lower the PI, the smaller the swelling potential.

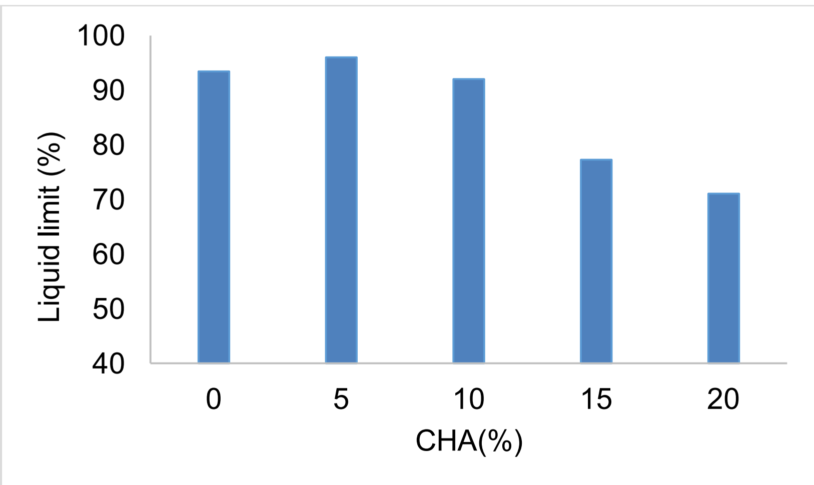



Figure 4.1: Liquid limits of samples treated with varying amounts of CHA

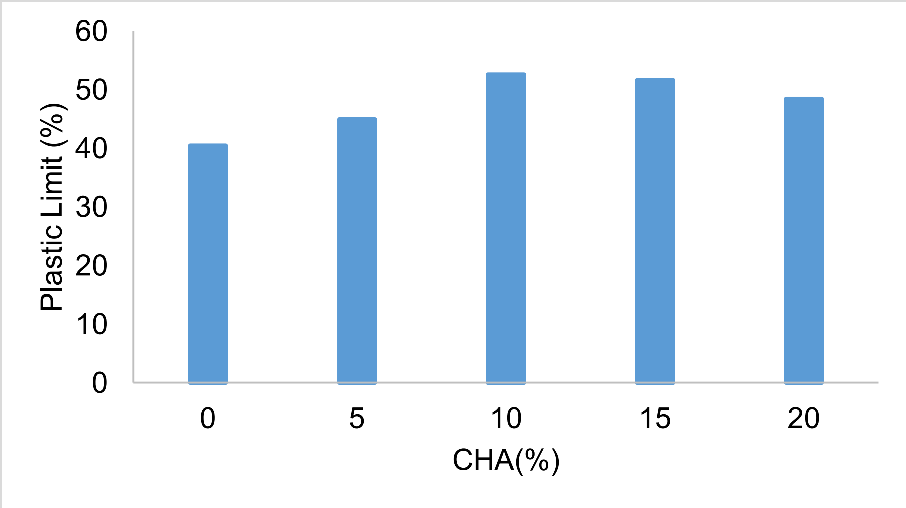



Figure 4.2: Plastic limits of samples treated with varying amounts of CHA

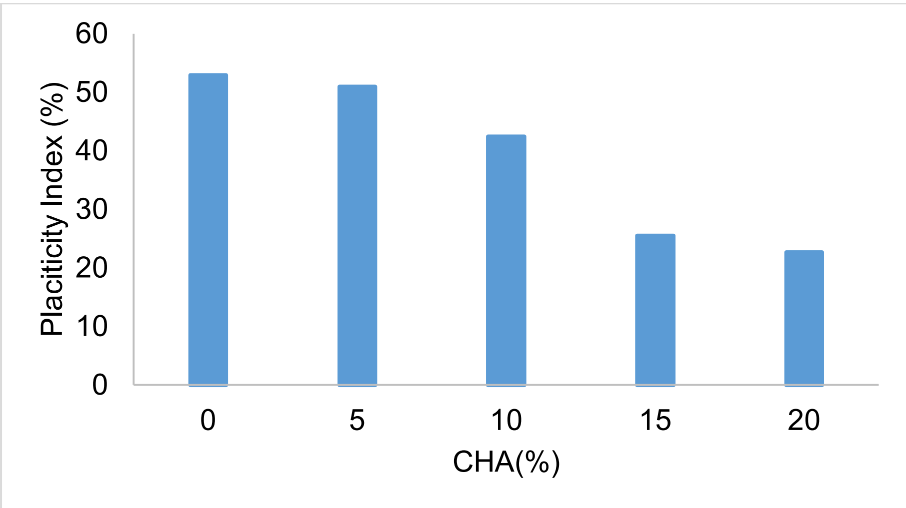



Figure 4.3: Plasticity index of samples treated with varying amounts of CHA

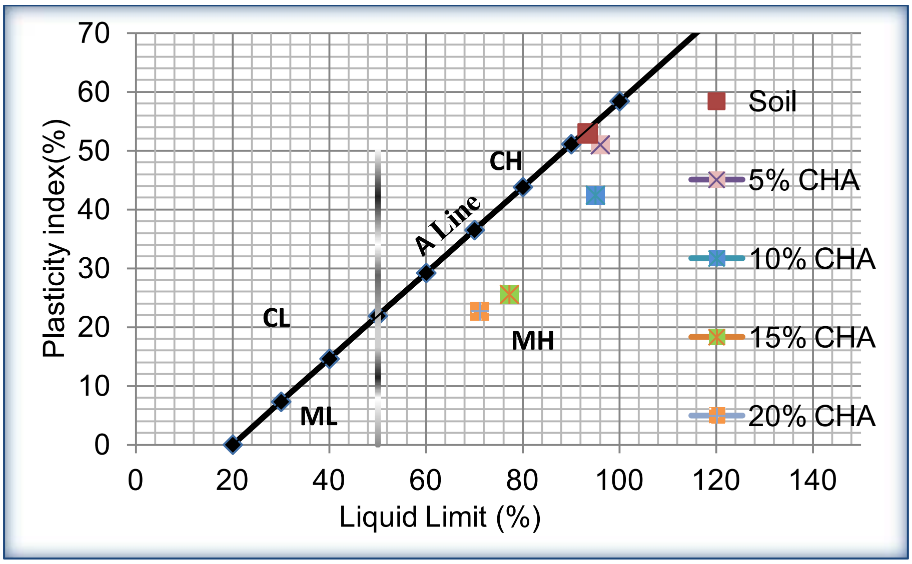



Figure 4.4: Plasticity index versus liquid limit of untreated and treated samples (USCS chart)

## 4.2 Swell-Shrink

Soil swelling is an expansion in volume that causes significant problems leading to serious damage and economic consequences in construction sectors, mainly in road construction.

The free swell index (FSI) test was performed on the BC soil and CHA treated samples according to the procedure stated in section 3.4.6. The FSI was determined using Equation 4.1 and the results are shown in Figure 4.5.

$$FSI\ (\%) = \frac{(Vf - Vi}{Vi} * 100 \qquad \text{Equation 4.1.}$$

Volumetric shrinkage (VS) is the decrease in volume of a soil mass when the water content is reduced from a given percentage to the shrinkage limit and it is expressed as percentage of dry volume of the soil mass. The VS of soil and soil-CHA mixtures is determined using Equation 4.2.

$$Vs(\%) = \frac{(Vw - Vd)}{Vd} * 100 \qquad \text{Equation 4.2.}$$

Volumetric changes of the soils may cause unfavourable effects such as damage to buildings and cracks in roads. The effect of the addition of CHA on the volumetric shrinkage of soils is shown in Figures 4.6 and 4.7. The BC soil used for this study has a high swelling and shrinking capacity, and the addition of CHA on soil samples shows significant change on both swelling capacity and volumetric shrinkage. The FSI value decreases almost by half by adding 10% CHA (see Figure 4.5). The Vs value decreases from 143% to 84% by adding 20% CHA, which indicates about a two-fold reduction compared to the BC soil.

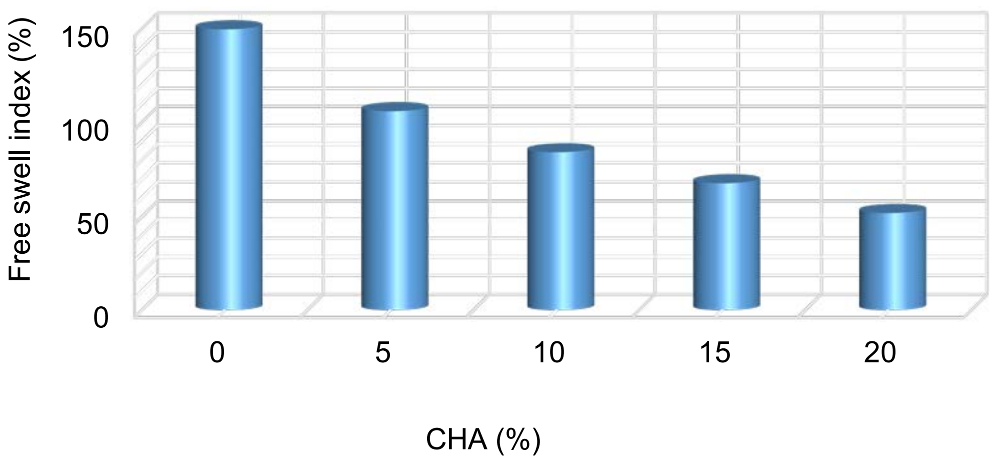



Figure 4.5: Free swell index values versus CHA amount

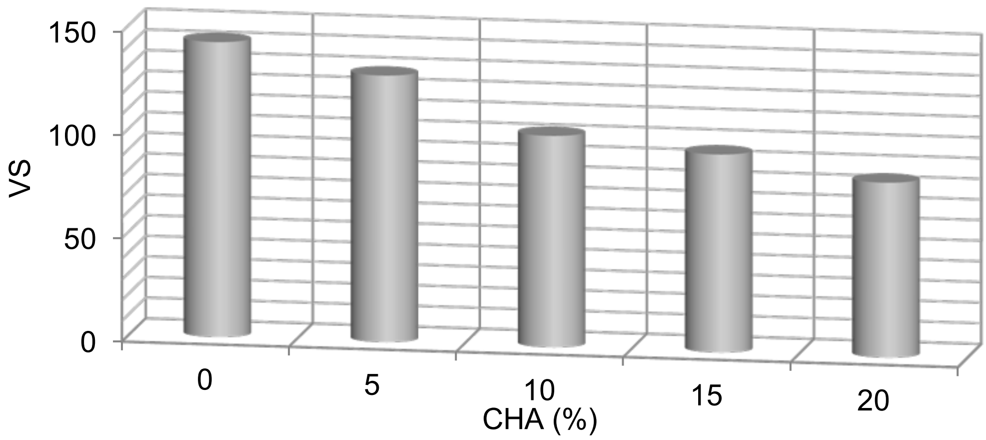



Figure 4.6: Volumetric shrinkage values versus CHA amount


Figure 4.7: Shrinkage behaviour (1) BC soil, (2) 10% CHA and (3) 20% CHA

## 4.3 Effect on Compaction

Compaction is the artificial improvement of the mechanical properties of the soil. The properties of soil, such as compressive strength, CBR and compressibility are dependent on the compaction parameters.

Compaction tests were conducted to find out the effect of CHA on moisture content and dry density of the BC soil by following the procedure discussed in section 3.4.4. The moisture content- dry density relationship of the untreated BC soil and CHA treated samples are represented by compaction curves, see Figure 4.8. These curves show the influence of CHA on the compaction characteristics of the BC soil. The OMC and MDD of the BC soil and with varying amount of CHA are summarized in Figure 4.9. It can be seen that the MDD increased and the OMC decreased as the amount of CHA increases. The reduction in the OMC of CHA- treated samples could be attributed to the amount of water needed to reach an optimum state is lower for CHA-treated samples compared to untreated samples; this could be due to the lower affinity of CHA particles for water.

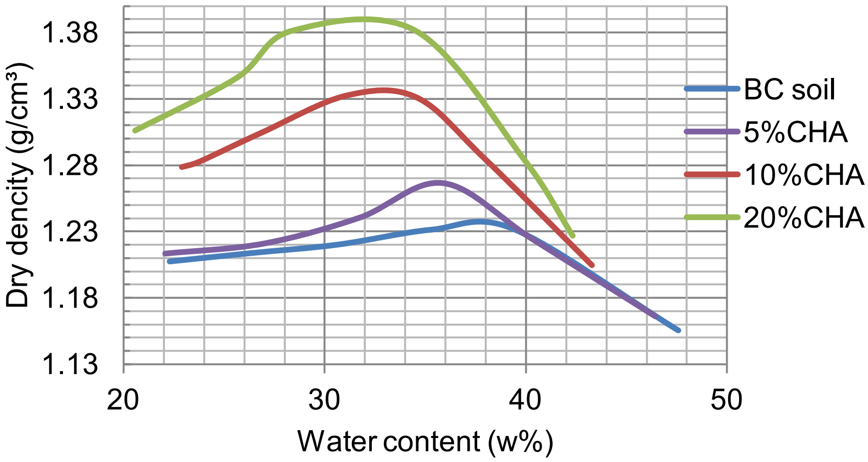



Figure 4.8: Moisture–density relationship of the BC soil and CHA treated samples

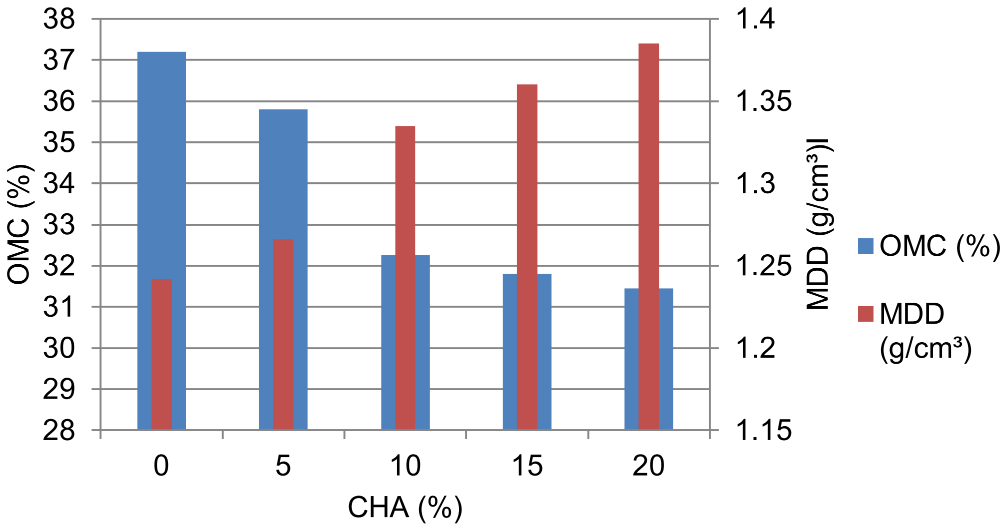



Figure 4.9: MDD and OMC values of the BC soil and CHA treated samples

## 4.4 Compressibility Behavior

The specific gravity of a soil is used to calculate the phase relationships of soils, such as void ratio. The specific gravity of the soil and the soil-CHA mixture was determined and the results are shown in Figure 4.10. The specific gravity of the investigated expansive soil is 2.68 g/cm³, after stabilization, the value decreases as the percentage of CHA content increases, which indicates that CHA particles are lighter than expansive soil particles.

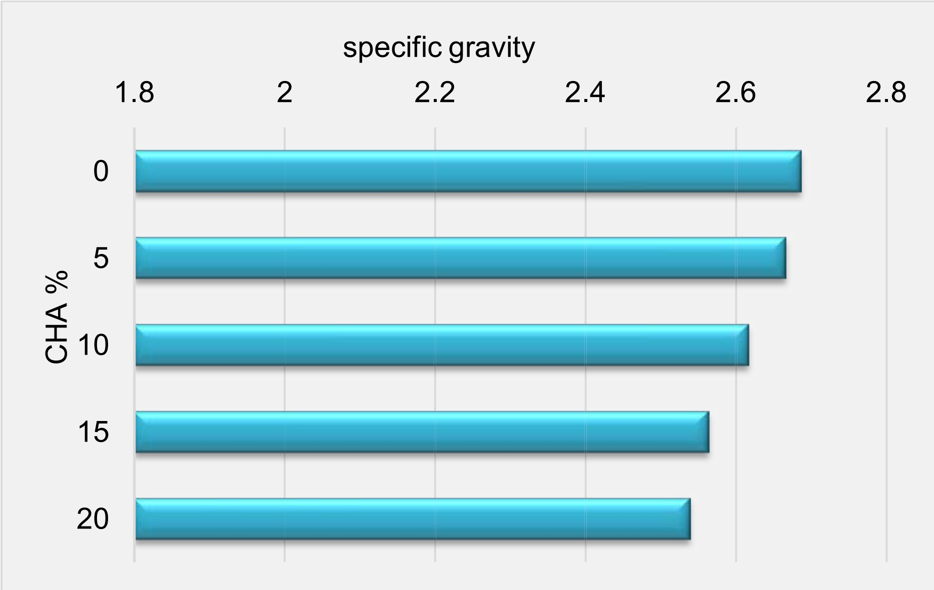



Figure 4.10: Specific gravity of the soil and soil-CHA mixture

The application or removal of loads on a soil layer will result in a deformation or swelling, deformation due to an increase in applied stress with time is known as the consolidation process.

The consolidation test is used to describe the compressibility characteristics of fine grained soils. The properties of expansive soils can be affected by different factors including the moisture variation and loading conditions. Under the application of loads in excess of the bearing capacity of the soil, downward movement of supporting soil occurs. Heave and settlement behaviours of an expansive soil cause a vertical movement on pavements, which usually resulted in a serious road damage and high maintenance cost.

In this study, one-dimensional consolidation tests were performed to determine consolidation parameters for the untreated soil and soil treated with different percentages of CHA. The settlement results are shown in Figure 4.11. Decrement in settlement value as the CHA percentage increases is observed. On the other hand, the settlement value increases as the pressure increases.

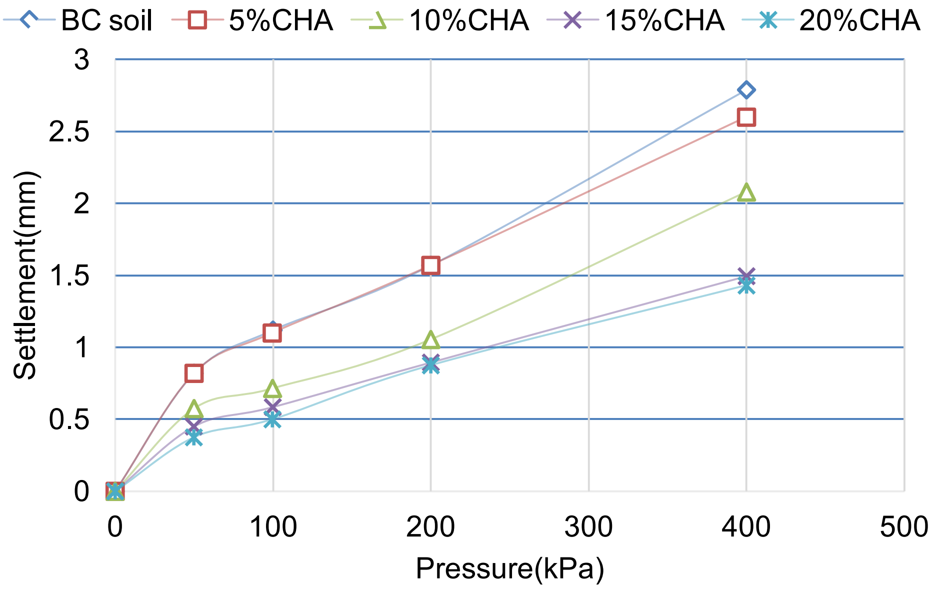
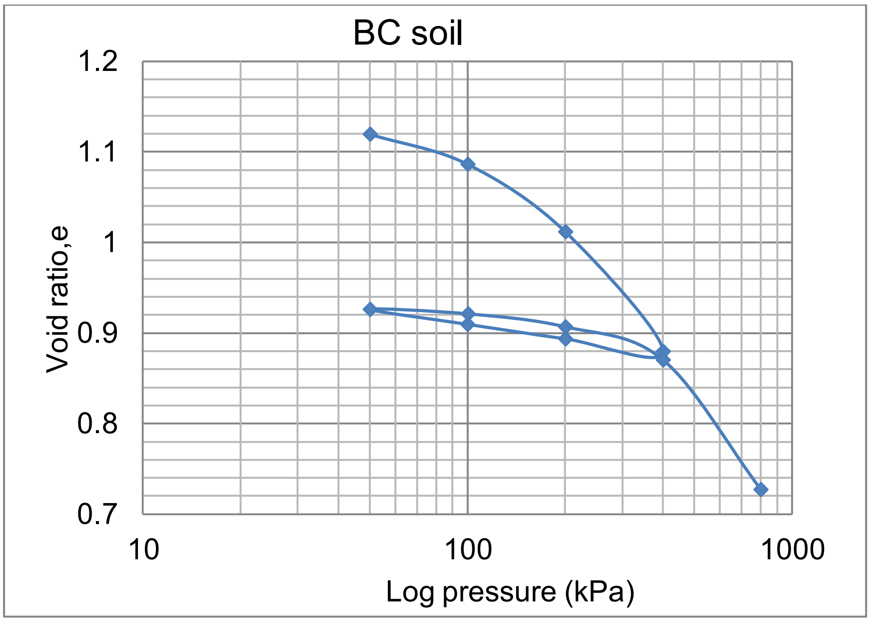
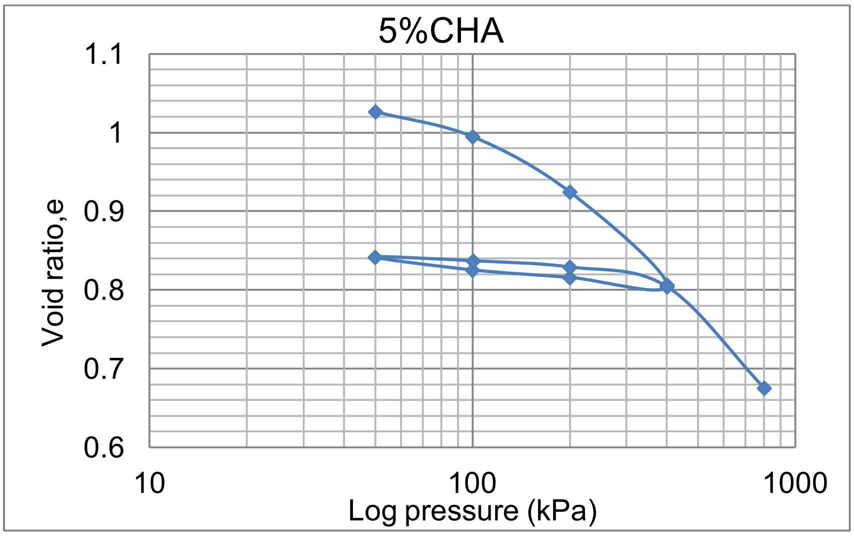



Figure 4.11: Settlement versus pressure for varying percentage of CHA

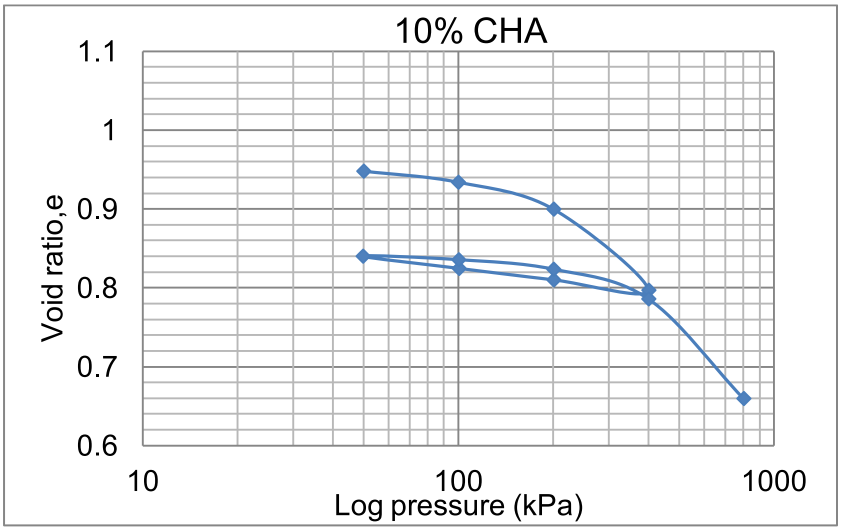
Consolidation curves (void ratio versus log pressure ) for the untreated BC soil (see Figure 4.12 a) and for different percentage of the soil-CHA mixture are presented in Figures 4.12 b-e. It can be seen that the initial void ratio of samples decreased as the CHA concentration increased for the same loading and saturation conditions. Further, as the load increases, continues reduction in void ratio is observed for all samples. The reduced void ratio of CHA treated samples could be attributed to the closer of voids by CHA constituents and due to joined particles by the formation of cementitious compounds.



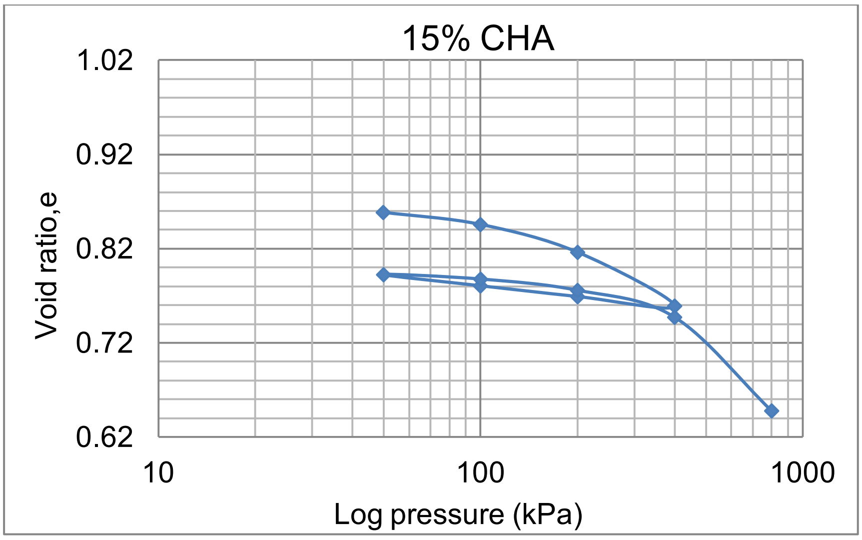

(a)


5%CHA
1.1
1
0.9
0.8
0.7
0.6
Void ratio,e
10
100
1000
Log pressure (kPa)

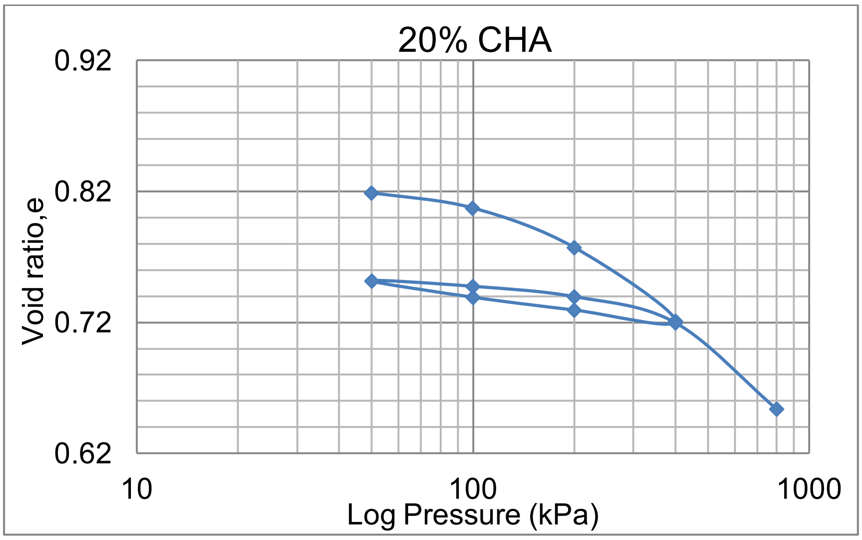
(b)


10% CHA
1.1
1
0.9
0.8
0.7
0.6
Void ratio,e
10
100
1000
Log pressure (kPa)

(c)


15% CHA
1.02
0.92
0.82
0.72
0.62
Void ratio,e
10
100
1000
Log pressure (kPa)

(d)




(e)

Figure 4.12: Void ratio versus log pressure graphs for different amounts of CHA

The main reason behind performing odometer test is to examine the effect CHA on the compression, recompression and swelling characteristics of expansive soil as applied pressure changes. The compression index (Cc), recompression index (Cr) and swell index (Cs) values of the BC soil and BC soil-CHA mixture were determined from consolidation curves. The index magnitudes (Cc, Cs and Cr) are the slope of compression, swell and recompression curves respectively. The addition of CHA leads to a significant change in these curves for similar loading conditions. The corresponding results are shown in Figure 4.13. The values of Cc, Cr and Cs decreased as the added percentage of CHA increases. The reason behind the decrement in consolidation parameters (Cc, Cs and Cr) may be due to the addition of CHA, filling the inter-aggregate pores and resulting in reduced compressibility characteristics. The decrement in Cc values of treated samples shows a better tendency of treated samples to resist the external load in comparison to the untreated samples.

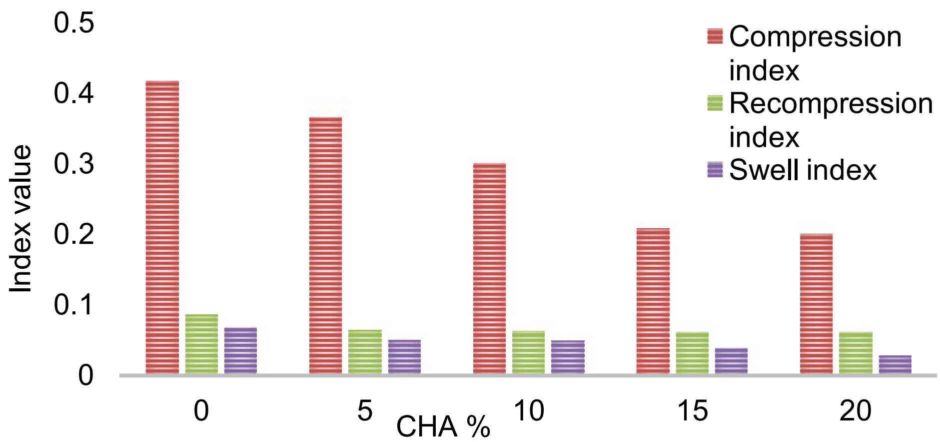



Figure 4.13: Index values versus percentage of CHA

The coefficient of compressibility and volume change are determined, and are illustrated in Figure 4.14 and Figure 4.15 respectively. The coefficient of compressibility (av) is represented by the slope of void ratio to applied pressure, and is an important indicator of the magnitude of soil compressibility. The coefficient of volume compressibility (mv) is defined as the volume change per unit increase in effective stress for a unit volume of soil. For untreated soil, the value of mv decreases to 0.37 m²/MN from 0.82 m²/MN as applied pressure increases from 50 kPa to 400 kPa, which shows mv decreases as the pressure increases. Both coefficients of compressibility and volume change of CHA treated samples are decreased as the effective stress and concentration of CHA increases. This might be attributed to a reduction in volume of voids due to rearrangement of the soil particles and decrease in plasticity behaviour as CHA increases, thus, making the soil particles more compact and less compressible.

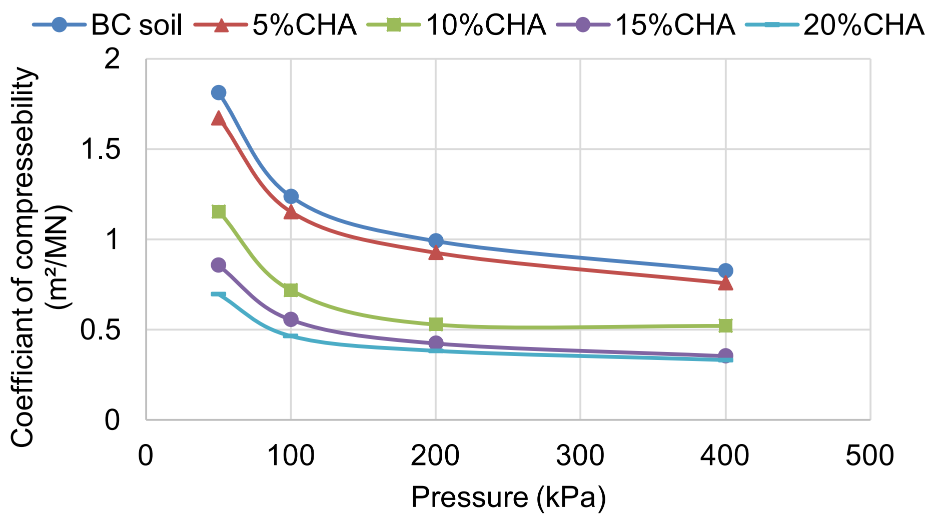



Figure 4.14: Coefficient of compressibility versus pressure

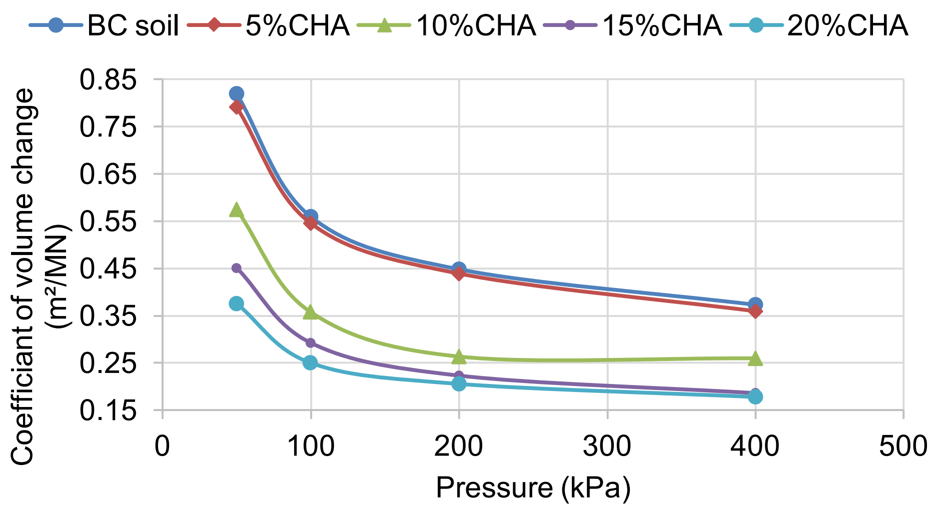



Figure 4.15: Coefficient of volume change versus pressure

## 4.5 Unconfined Compressive Strength

The UCS test is a standard and the most common strength test used in evaluating the effectiveness of stabilizer. Compressive strength is an important factor to estimate the design criteria for the use of soil as a pavement material. As indicated in section 3.4.5 samples for the UCS test were prepared and cured at room temperature for 1, 7 and 14 days.

The UCS results of the BC soil and CHA treated samples with different percentages of CHA and curing time are presented in Figures 4.16 - 4.18. The addition of CHA to soil samples shows considerable improvement. It can be seen that the BC soil attained UCS of 81 kPa. As the percentage of CHA increases from 0 to 10% the UCS has increased to 219 kPa, and increased to 231 kPa for 15%CHA. The decrease in the UCS after 15% CHA content may be due to excess CHA that was not mobilized in the reaction, which consequently occupies spaces within the sample and therefore reducing bond in the soil–CHA mixtures.

The UCS of BC soil treated with 15% CHA increase by more than double compared to that of the untreated BC soil. The strength gain of CHA treated samples is primarily caused by the formation of cementation material. In addition, this improvement could be due to better interlocking of BC soil-CHA particles consequently yielding a higher load. For further addition the UCS values decreases. The UCS values of both BC soil and BC soil-CHA mixtures increased for 7 days and 14 days curing (see Figures 4.17 and 4.18)

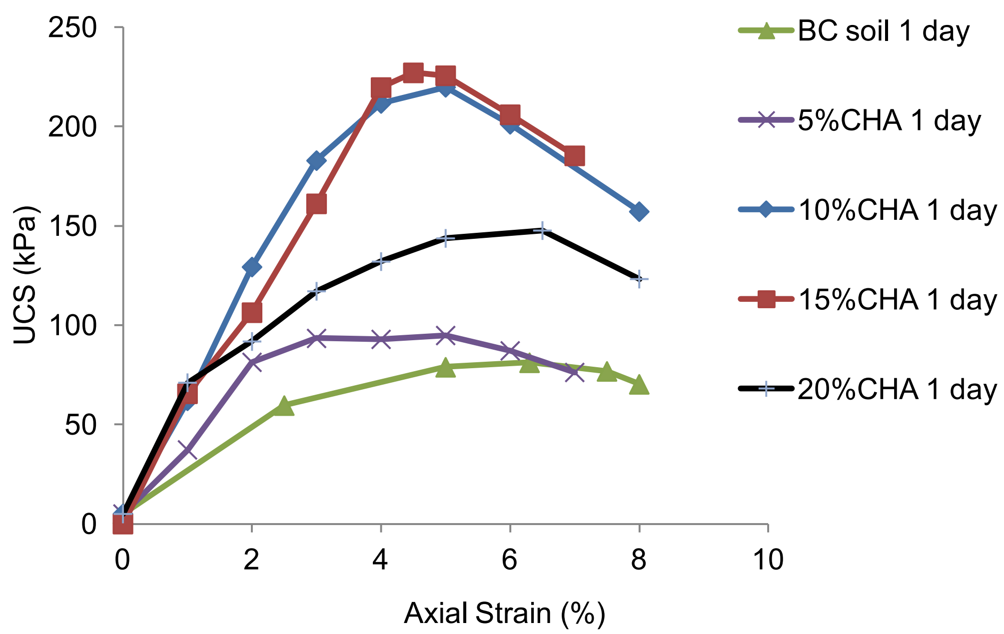



Figure 4.16: UCS of the BC soil and CHA treated samples cured for 1 day

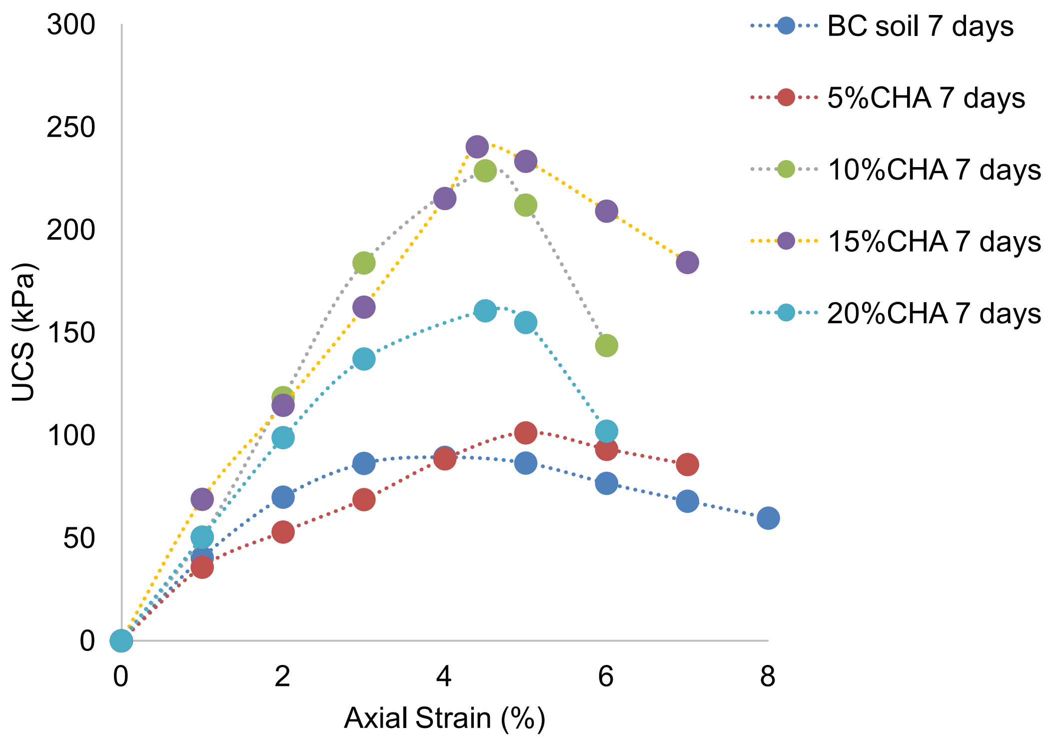



Figure 4.17: UCS of the BC soil and CHA treated samples cured for 7 days

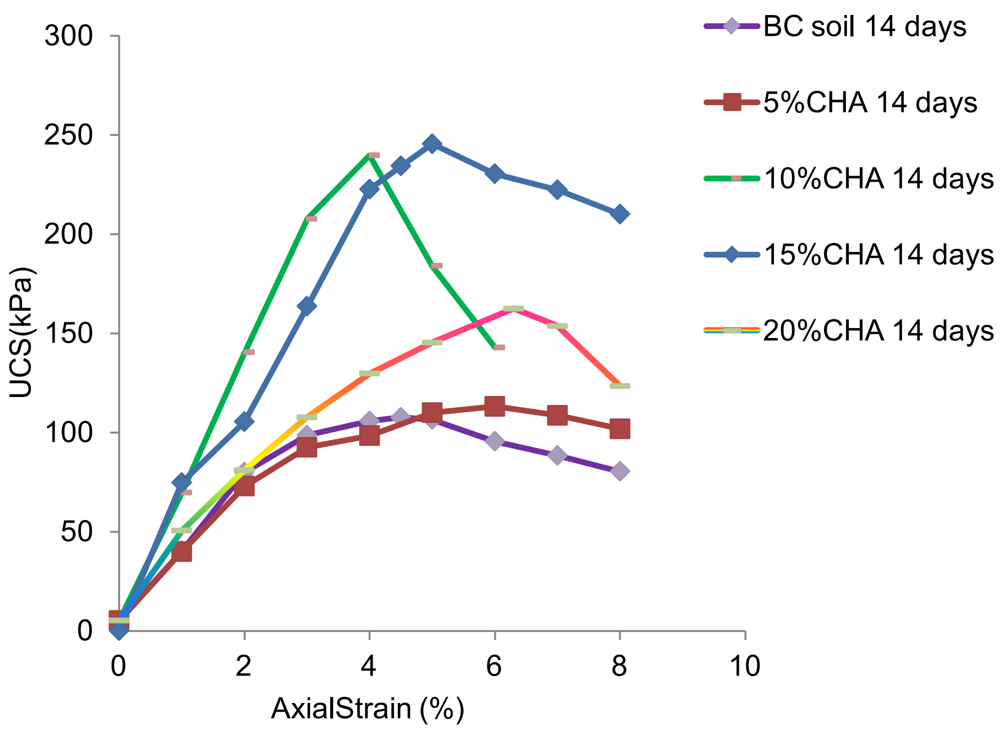



Figure 4.18: UCS of the BC soil and CHA treated samples cured for 14 days

### 4.6 Resistance to Wetting-Drying (W/D) Cycles

Durability of the BC soil and CHA treated samples were evaluated by simulating moisture variation in the field that occur during different seasons using W/D method. To estimate the strength loss in the worst possible field conditions (eg. water saturation) the treated soils were subjected to durability investigation. To prove the stabilizers used are effective, it is important to investigate their ability to withstand environmental condition. In this study, durability of the BC soil before and after treatment was performed by subjecting the samples to W/D cycles (see Figure 4.19).

Effects of variation in moisture and temperature on behaviour of samples were investigated on four different amounts of CHA (5%, 10%, 15% and 20%). All the samples were compacted at their respective OMC and MDD.

Figure 4.19: Samples subjected to W/D cycles

This study attempted to understand the long-term stability of CHA treated samples by focusing on the UCS of samples. The UCS tests were used to evaluate the weathering effect on strength and durability of studied soil before and after treatment. Figure 4.20 show stress-strain curve of the BC soil cured for 7 days at room temperature prior to W/D cycle.

As water moves in and out of the pore network during W/D, capillary pressure develops and acts on the walls of the pores such that the pore structure, tensile strength, inter-particle friction and cohesion of the material will then dictate how the soil will respond to this capillary pressure [77].

The BC soil samples were collapsed in the first wetting cycle as shown in Figure 4.21. This could be due to that the attractive forces between the BC soil particles were so weak (van der waal forces) that capillary pressure that developed on the walls of the pores cause the samples to collapse [77].

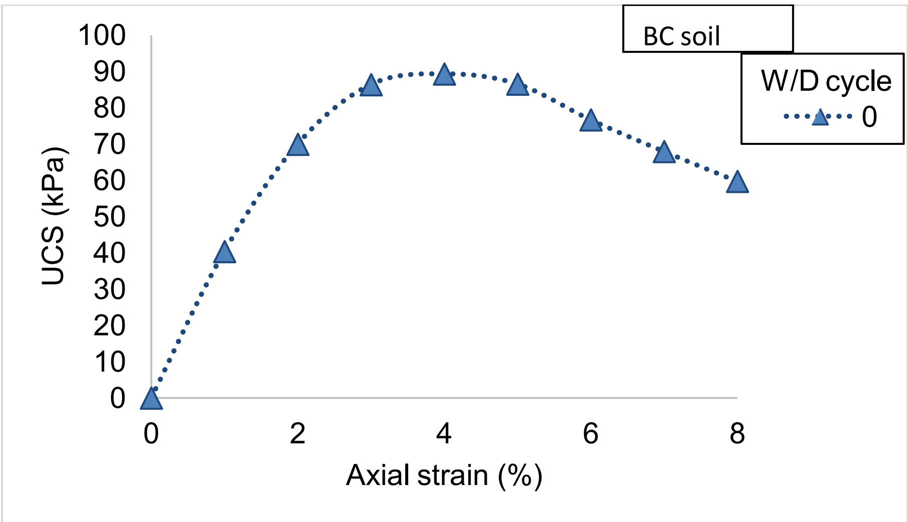



Figure 4.20: Stress-strain curve of the BC soil before subjected to W/D cycles

Figure 4.21: The BC soil samples failed in the first cycle

The stress-strain curves of samples treated with 5% CHA are shown in Figure 4.22. These samples only withstand one W/D cycle, they failed in the second cycle. At the 5% concentration, the stabilizer failed to strengthen the soil mixture sufficiently possibly due to inadequate reaction to generate enough cementitious compounds that stabilizes the soil to resists the wetting.

For samples containing 10%, 15% and 20% CHA, the likely formation of cementitious gel resulted in greater strength development of the mixtures, hence improved resistance to W/D condition (Figures.4.23-25). These samples (containing 10%, 15% and 20% CHA) resists two W/D cycles (see Figure 4.26). The ASTM D559 test standard recommended 12 cycles of W/D but all the samples failed at the end of the 3rd cycle (see Figure 4.27). This could be related to the fact that the standard was designed for soil-cement mixes and thus the time duration of wetting and drying may not apply to the type of samples used in this investigation.

Although the treated samples failed in the third cycle, it can be concluded that CHA treated samples are more stable (resist W/D conditions) compared to the untreated BC soil samples.

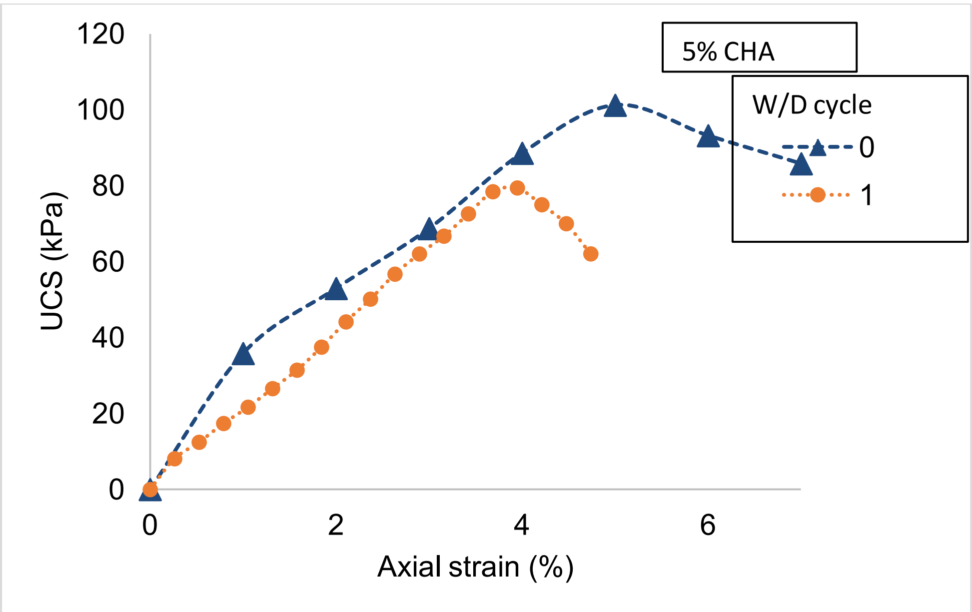



Figure 4.22: Stress-strain curves of samples treated with 5%CHA before and after subjected to W/D cycles

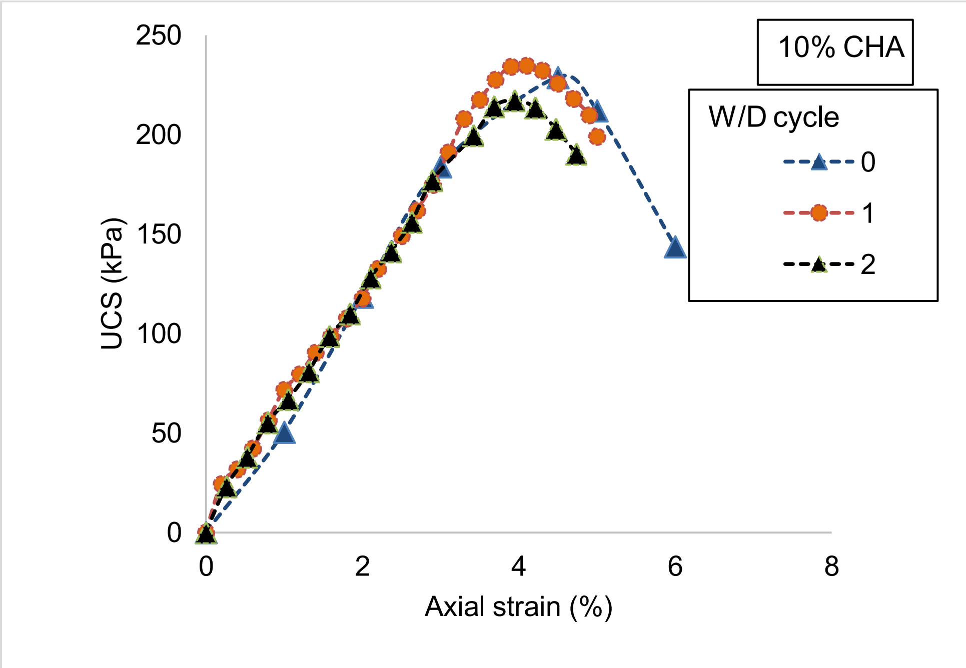



Figure 4.23: Stress-strain curves of samples treated with 10%CHA before and after subjected to W/D cycles

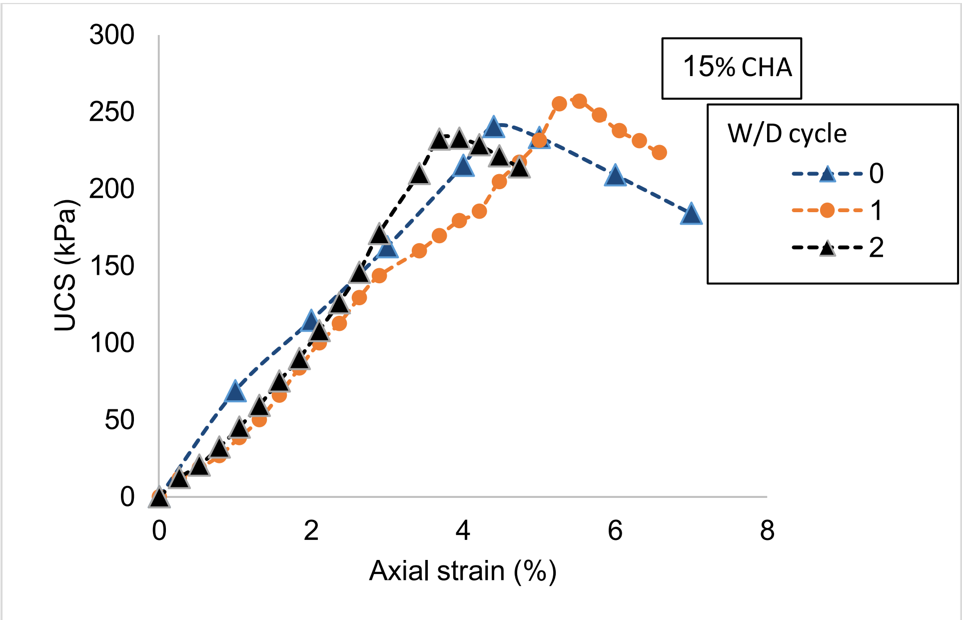



Figure 4.24: Stress-strain curves of samples treated with 15%CHA before and after subjected to W/D cycles

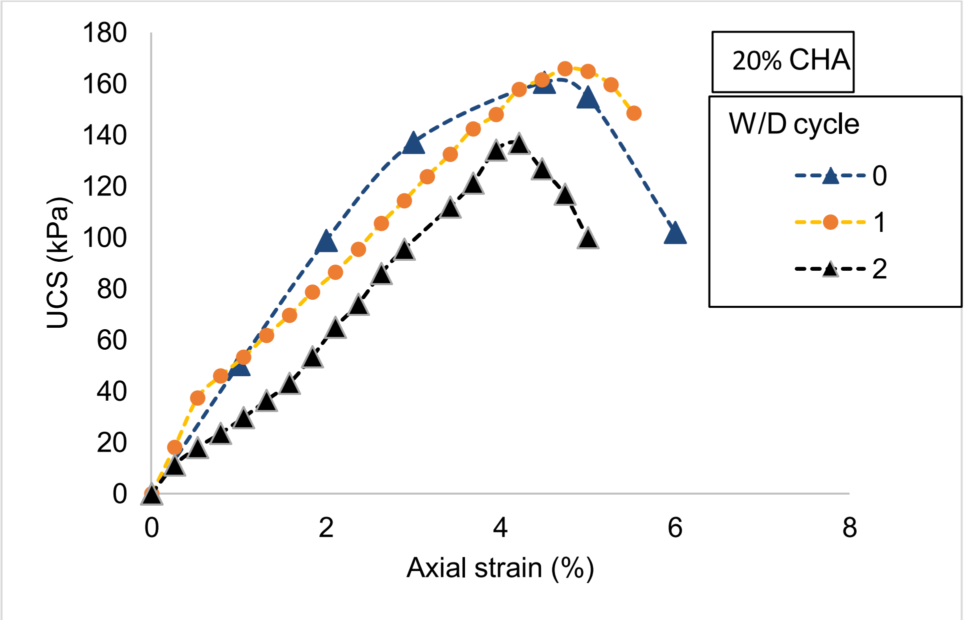



Figure 4.25: Stress-strain curves of samples treated with 20%CHA before and after subjected to W/D cycles

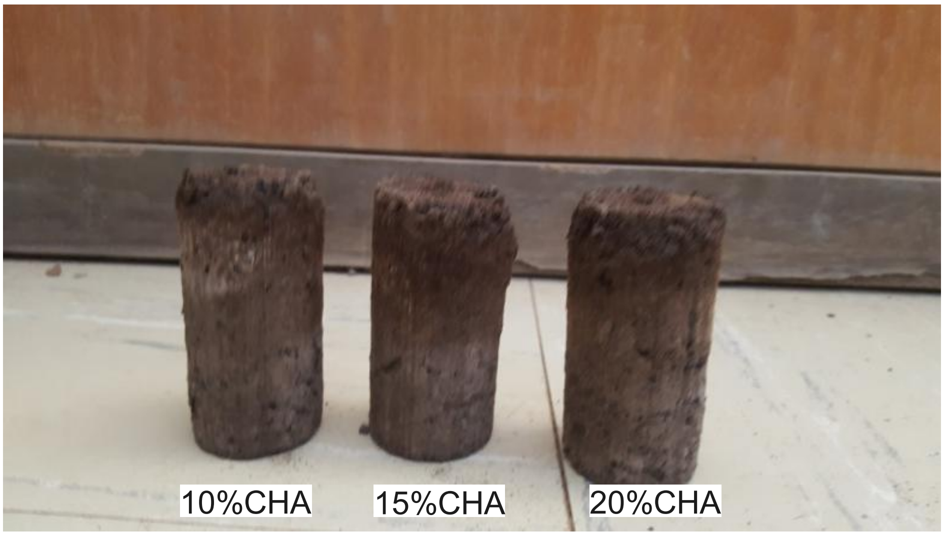



Figure 4.26: Samples treated with 10% CHA, 15% CHA and 20% CHA after second W/D cycles

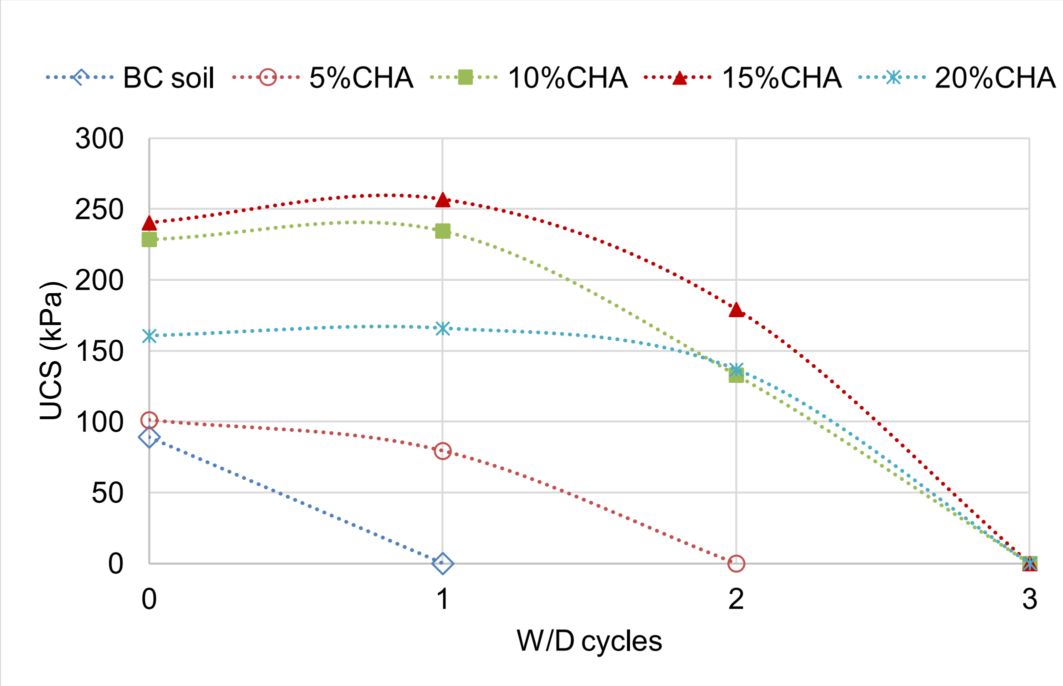



Figure 4.27: UCS values of samples for different W/D cycles

### 4.7 Conclusions

In this study the potential use of CHA to improve the geotechnical property of the BC soil was investigated. Commonly used laboratory tests such as Atterberg limits, free swell index, standard Proctor, consolidation and UCS tests were performed to evaluate the behaviour of the BC soil treated with different percentages (5%, 10%, 15% & 20%) of CHA.

The tested soil (BC soil) is classified as CH with high swell–shrink capacity. The obtained result reveals that the addition of CHA reduces the plasticity of the soil. The LL and PI decreased as the amount of CHA increased. This could be attributed to the flocculation of clay particles and increases in the number of coarser particles as a result reduce the plasticity.

The swell and shrinkage tests indicated that the swelling and shrinking capacity of the CHA treated samples reduced significantly. The BC soil treated by 20% CHA shows reduction of swelling capacity by about three-fold and shrinking capacity by about a two-fold compared to the untreated BC soil.

The OMC was found to decrease while the MDD increased with increasing CHA content.

The results obtained from one-dimensional consolidation tests show that the void ratio declined as the magnitude of stress and CHA concentration increased, this indicates that the volume of voids decreased. As the percentage of CHA increases, the compression and recompression index values decrease, which shows the decrement in soil compressibility. Furthermore, the compressibility and volume change coefficients are declined as CHA percentage increased.

The UCS increased with increasing CHA content (from 5 to 15%) and decreased with continuous increase in CHA content. Further, the strength increased with curing time.

In durability test (W/D) the BC soil failed in the first cycle. The samples treated with 5%CHA resists the first cycle, however, failed in the second cycle. The samples treated with 10% CHA, 15% CHA and 20% CHA resists two W/D cycles. From these results it can be concluded that CHA treated samples are more stable compared to the BC soil samples.

# 5 Strength, Microstructural and Mineralogical Characterization of Coffee Husk Ash Stabilized Black Cotton Soil

## 5.1 California Bearing Ratio (CBR)

During the construction of roads, the strength of soils to be used is usually evaluated by their CBR values. CBR-test was conducted to characterize the bearing capacity of the BC soil and CHA treated BC soil. The test procedures and the preparation of the samples were achieved according to section 3.4.7. The results of the CBR test, for varying percentage of CHA are represented by load versus penetration graphs and are shown in Figure 5.1 (a) and (b) for un-soaked and soaked conditions respectively.

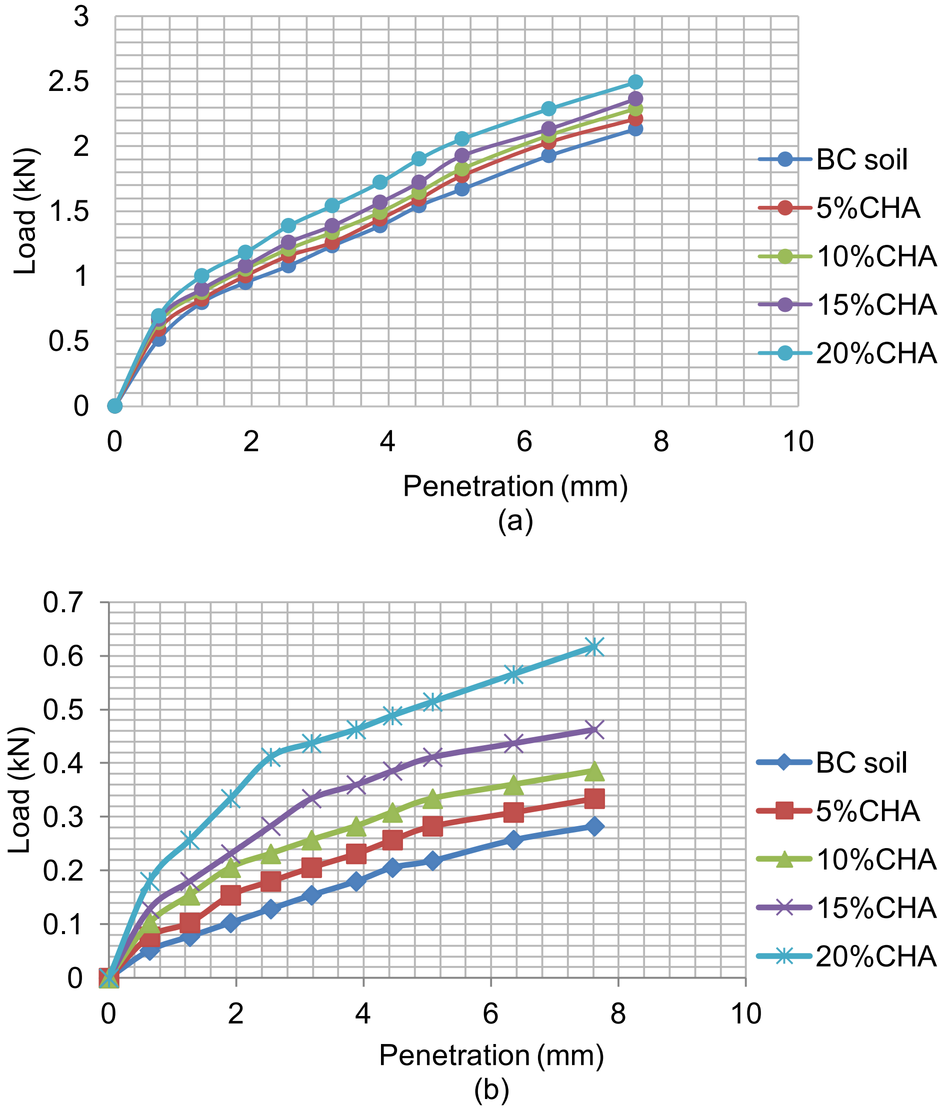



Figure 5.1: Load versus penetration curves for (a) un-soaked and (b) soaked conditions

The soaked CBR of the BC soil used in this study is 1%. Sub-grade materials, which have a CBR value of less than two needs a special treatment [65]. According to AASHTO M 145-91 (2008), the BC soil used in this study is classified as A-7-5, clayey soil in silt-clay groups; these groups will range fair to poor in quality to be used as a sub-grade material. After stabilization, it can be concluded that the CBR value increases as the percentage of CHA increases, the relations are presented in Figure 5.2. The CBR value of untreated soil is 1% and 8.3% for soaked and un-soaked conditions, respectively. The CBR values of soils treated with 20% CHA are found to be 3.1% and 10.6%, in case of soaked and un-soaked conditions, respectively. From the test results, it is observed that the stabilization of BC soil with CHA improves the CBR value, which is an indicator for the load carrying capacity improvement. The addition of CHA showed a significant improvement in both soaked and un-soaked conditions. With a concentration of 20% CHA, the soaked CBR value increased by around 205% (three times greater than the natural BC soil) and the un-soaked CBR value increased by around 28%. From these results, it can be observed that CHA is more effective for soaked condition. The improvement can be attributed to the reaction between the soil and CHA, forming cementitious material. The formation of these cementitious material bounds the particles together, by covering the soil grain and filling the inter-aggregate pores [78]. The result indicates that the BC soil treated with CHA performs better as a sub-grade material; which requires lower thickness of the pavement structure compared to the untreated BC soil.

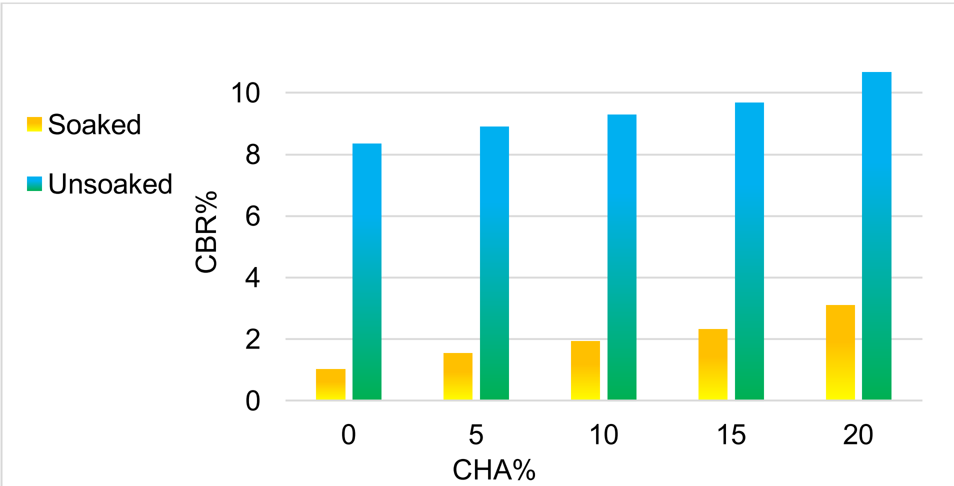



Figure 5.2: CBR value versus CHA percentage for soaked and un-soaked conditions

The CBR swell test was also performed for varying combination of soil-CHA mixtures. The CBR swell of untreated soil is found to be 10%, as mentioned in Figure 5.3, which shows high swelling capacity. When the bottom layer of pavements is found to be a BC soil, it has a high risk of swelling and deformation, which will result in cracks and rutting to the surface of roads. Low quality soils like BC soil with high swelling potential can be improved through stabilization to use as sub-grade for highway construction. The addition of CHA decreases the CBR swell with an increase in CHA dosage. The swell potential decreases approximately by two fold with the addition of

20% CHA. The usage of CHA as a stabilizing agent has also showed a significant change in CBR swell in addition to CBR value. The formation of aggregations could be the reason for the reduction of swell. Furthermore, the reduction in the swelling capacity of CHA treated soil may also relate to the non-swelling property of CHA.

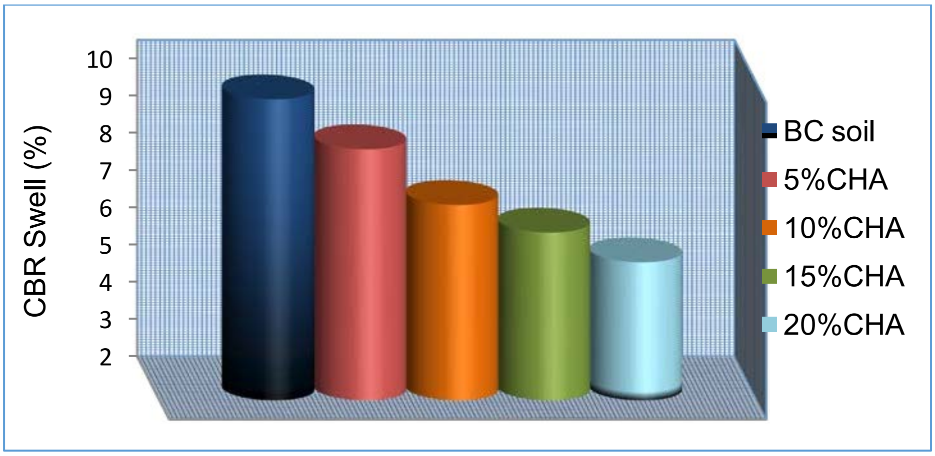



Figure 5.3: CBR swell versus CHA percentage

## 5.2 Shear Strength

The evaluation of the shear strength of soils is very important in geotechnical engineering. The stability of dams and the bearing capacity of building foundations and pavements are affected by the shear strength of the soil they are constructed on. Soils shearing resistance is the result of resistance to movements at inter-particle contacts [47].

The angle of internal friction (φ) and Cohesion (C) for varying percentages of CHA are shown in Table 5.1. From the results, decrease in cohesion as the content of CHA increased was observed. The angle of internal friction of CHA treated samples increased. The results are in agreement with unconfined compressive strength values which increased as the CHA content increased (see section 4.5). Internal friction arises from inter-particle proximity and a decrease in the PI is a sign of aggregation, which leads to a larger angle of internal friction.

Table 5.1: Cohesion (C) and angle of internal friction (φ) at various CHA content.

| CHA | *Φ(º)* | C(kPa) |
|---|---|---|
| 0 | 18.14 | 94.38 |
| 5 | 21.96 | 94.18 |
| 10 | 24.47 | 82.72 |
| 15 | 25.88 | 77.14 |
| 20 | 26.05 | 76.33 |

### 5.3 X-ray Diffraction Analysis

XRD analysis has been carried out to identify the minerals present in BC soil, and to investigate the effect of CHA on mineralogy of the soil. The XRD patterns of BC soil and BC soil treated with 10% CHA and 15% CHA are presented in Figure 5.4. The result obtained indicates the presence of clay minerals namely, quartz and kaolinite, the BC soil also contains some amount of montmorillonite mineral, which is responsible for the expansion character. The result of XRD is in agreement with EDX analysis, which shows the presence of elements such as Si, Al, Fe, Na, Mg, in the BC soil (see Figure 3.8).

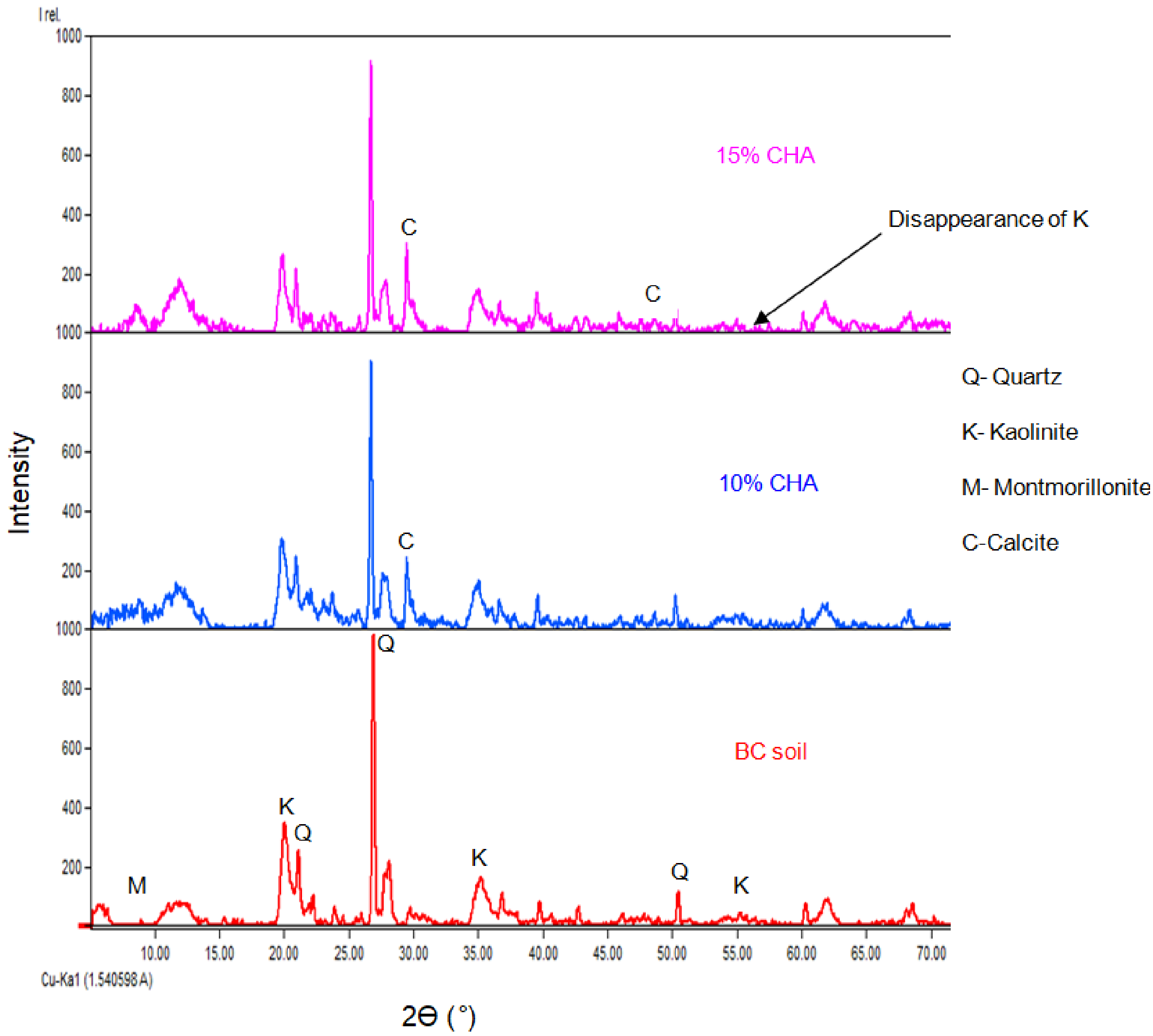



Figure 5.4: Mineralogical analysis of BC soil and samples treated with 10% CHA and 15% CHA

The XRD pattern of CHA treated soil shows the formation of Calcite. The formation of calcite is an evidence of the fact that pozzolanic or cementing material has taken part in chemical reactions with the clay minerals [100]. In addition, reductions in the intensity of some minerals for CHA treated samples are observed. For instance, a decrement in the intensity of quartz is observed for both samples treated with 10% CHA and 15% CHA. Comparatively, XRD patterns indicated higher peaks of calcite for samples treated with 15% CHA than treated with 10% CHA. This is due to the availability of additional calcium in 15% CHA. Apart from reduction in the peak intensity of quartz and the appearance of calcite, the disappearance of kaolinite is also observed in samples treated with 15% CHA. The peak disappearance has been accompanied by a simultaneous appearance of new peaks in the XRD patterns. These changes in XRD pattern and peak height could be attributed to reactions between clay mineral and the CHA.

Figure 5.5 shows X-ray diffraction patterns of BC soil treated with 10% CHA and cured for 7d, 14d and 28d. The intensities of peaks related with cementitious compound (calcite) increased with increasing curing time. In addition, additional calcite peaks appeared for samples cured for 14d and 28d. These results demonstrate that the pozzolanic reactions start in the short term over 7 days and continue over a long period; the quantity of the reaction product increases with curing time.

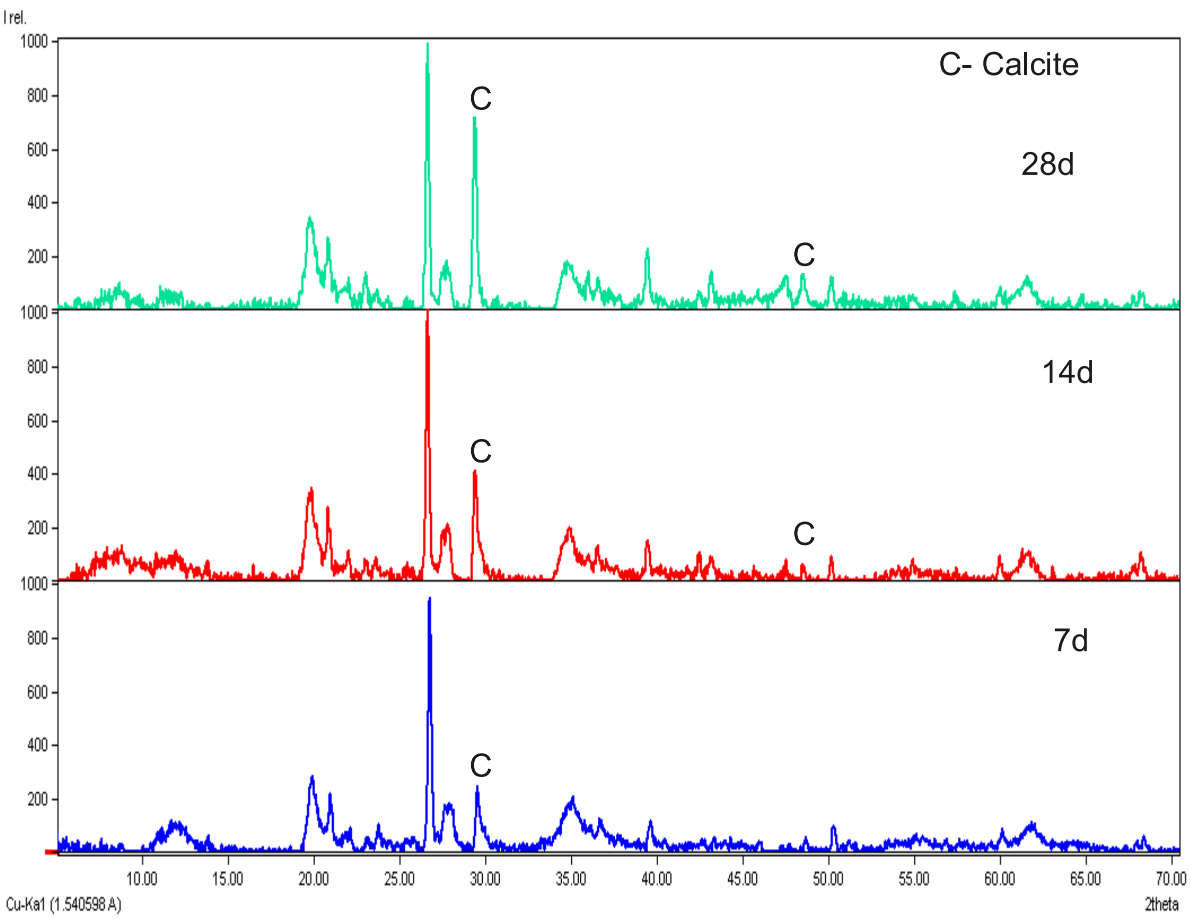



Figure 5.5: X-ray diffraction of samples treated with 10% CHA and cured for 7d, 14d and 28d

## 5.4 Micro-Structural Analysis

In this study, SEM images have been captured to observe the morphology features of untreated and CHA treated samples. Furthermore, EDX was used to examine the chemical composition of samples. According to EDX analysis incorporated with SEM, major elements present in BC soil are silicon (Si) and aluminium (Al) (Figures.3.4 and 5.6c), whereas CHA mainly constitutes potassium (K) and calcium (Ca), as shown in Figures. 3.6 and 5.7c).

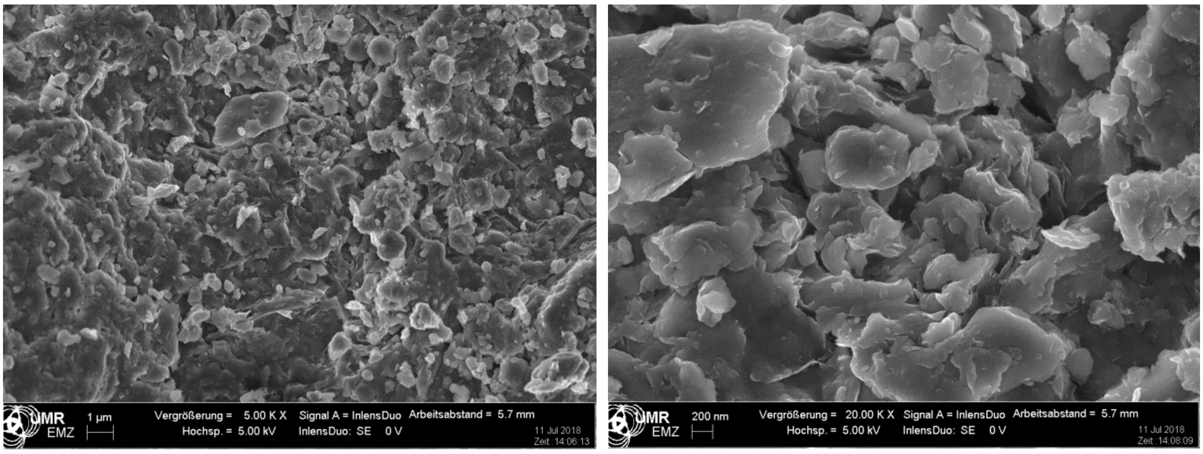
The chemical composition of stabilizing agents provides a good indication about their effectiveness in soil stabilization. The silica and alumina present in the soil could react with calcium hydroxide present in the additive to produce cementitious products [7].

To get an overall observation about the micrograph of the samples, first, the SEM images were magnified at low ranges (less than 200×). Afterwards, representative region was selected for higher magnifications. Micrographs of the BC soil, CHA and CHA treated samples were taken from the selected region at magnifications of 5000x and 20,000x.

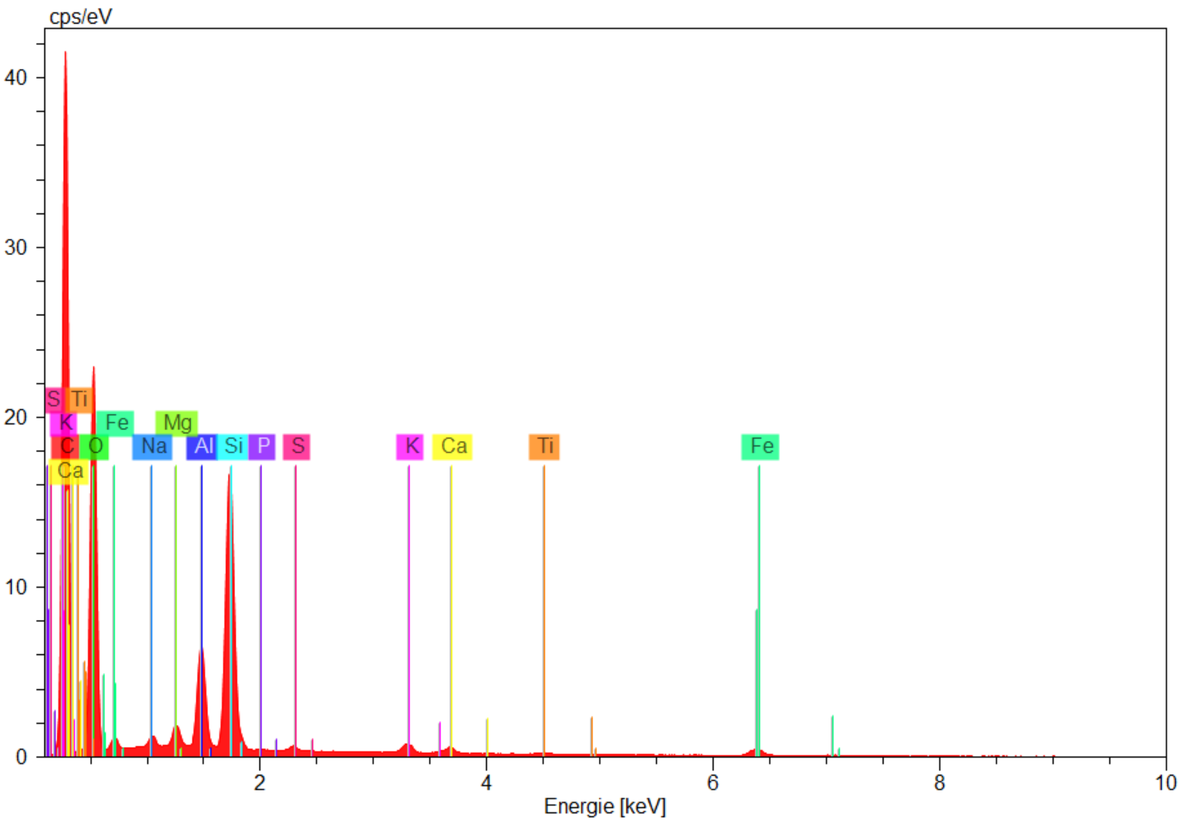
Figure 5.6 shows the micrograph of BC soil, at magnification of 5000x, and many small particles with different shapes are observed. For higher magnification (20,000x), a number of large and small pores with various shapes can be seen with no appearance of aggregations. In terms of particle shape and surface features, CHA is different from BC soil (Figure. 5.7).

By comparison, the surface morphology of the BC soil had changed considerably after CHA treatment. Samples treated with 10% and 15% CHA after curing (for 7d, 14d and 28d) shows changes in micro-structural particle orientation. The micrographs of BC soil treated with 10% and 15% CHA are presented in Figures. 5.8 and 5.9, respectively. From these figures the presence of flocculated particles is observed, showing few pores and continuous clay matrices.

The observed large pores among the fine-grained BC soil particles provide a large portion of the total void ratio. These pores connected and diminish for both samples treated with 10% and 15% CHA, respectively. These morphological changes could be due to the coating mechanisms and cation exchange processes between the soil and the CHA. Further, it might be attributed to the replacement of clay particles by CHA particles. Furthermore, the micrograph of CHA treated samples shows compact clay matrices and the micro-structure becomes less open and more continuous. This might be due to the settling of CHA particles in the pore spaces among the soil particles and aggregation of particles. This textural event could contribute significantly to the reduction of the swelling capacity of the soil. The reason behind observing a more stable micro-structure in CHA treated samples compared to untreated samples could be attributed to the formation of cementitious products resulting from the reaction between BC soil and CHA particles.

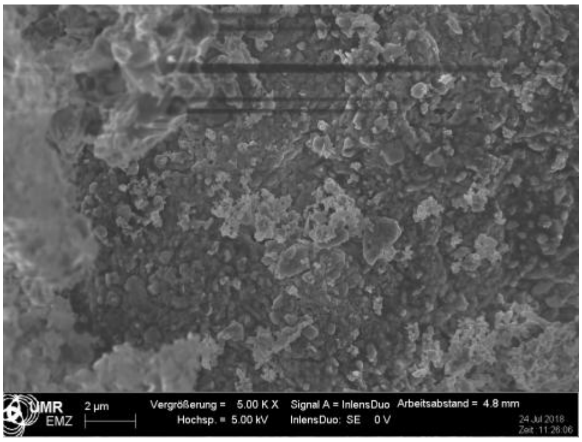



(a) (b)

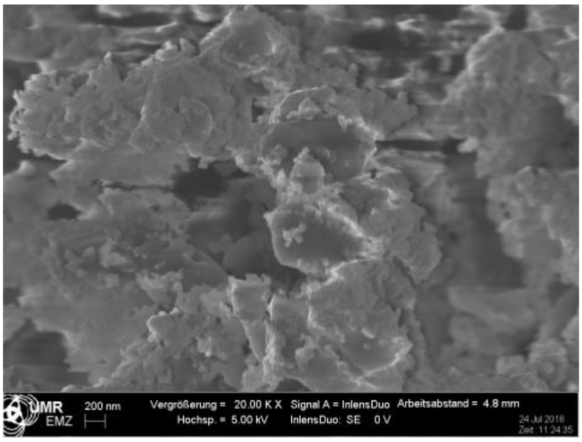



(c)

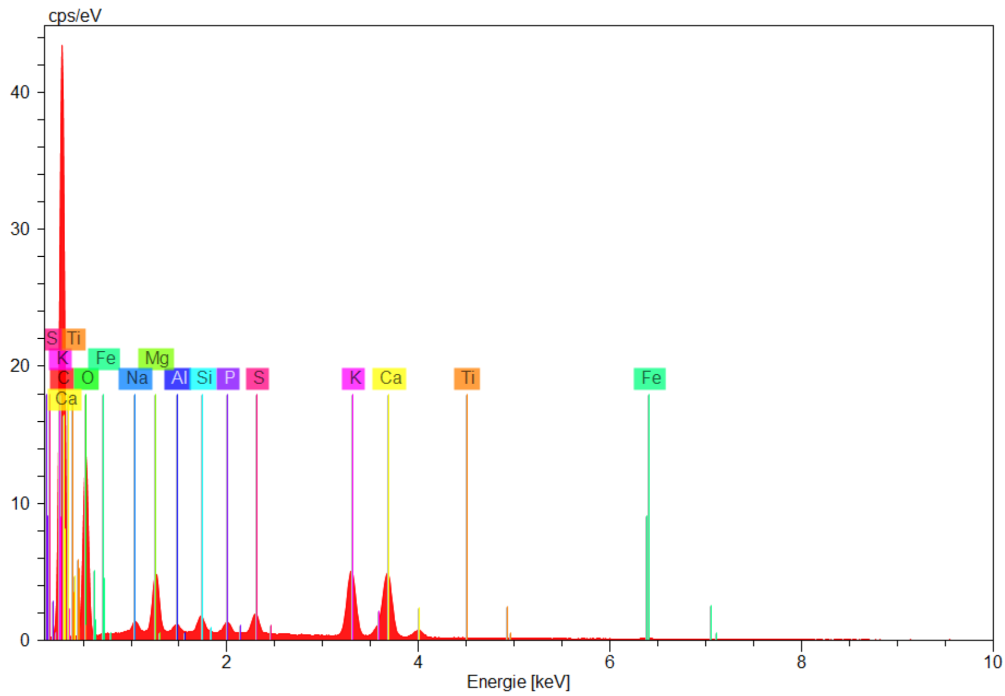
Figure 5.6: SEM images of the BC soil with (a) 5000 and (b) 20,000 times magnification, and (c) EDX spectrum of BC soil




(a)




(b)

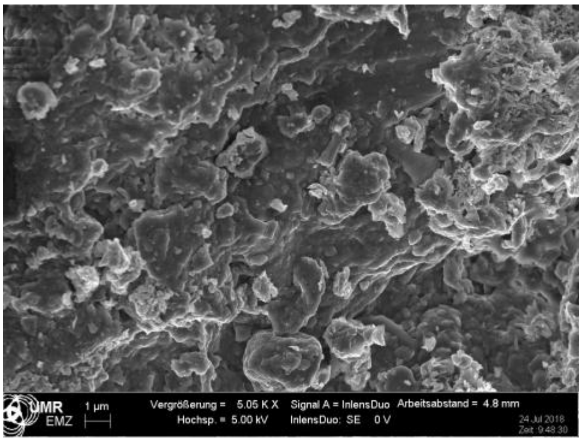



(c)

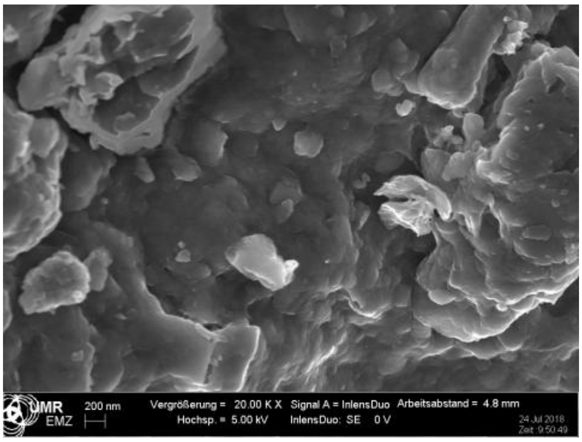
Figure 5.7: SEM images of the CHA with (a) 5000 and (b) 20,000 times magnification, and (c) EDX spectrum of CHA

Figures 5.8 and 5.9 illustrates the micrograph of CHA treated samples cured for 7d 14d and 28d. In these figures the formation of cementitious compounds after long-term curing as a result of the pozzolanic reaction filling the voids and coating the aggregates was also observed. Figure 5.10 shows the distribution of Ca and development of network of reinforcement, which may lead to an increase in the strength. In the long term curing the formation of new cementitious compounds grow within the pore spaces resulting in a reduction of the pore spaces,

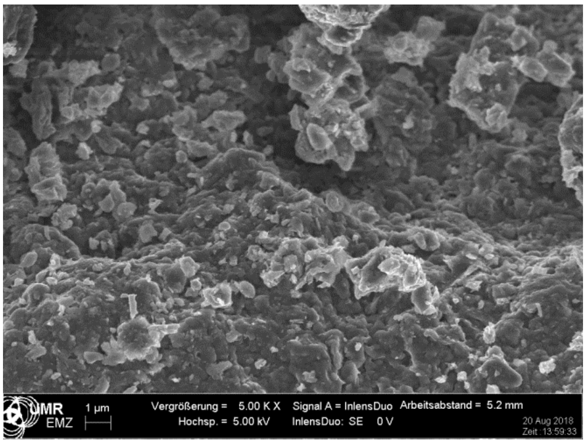
EDX analysis of CHA treated samples presented in Figures 5.8 (g) and 5.9 (g) shows an increment in Ca and K contents and a decrement in Si and Al contents. The decrement in Si and Al contents could be due to the pozzolanic reaction, in which the Ca from the CHA reacts with Al and Si from the soil in the presence of water to form cementitious compounds, which enhances the long-term strength and improves the properties of the soil. In general, SEM images of CHA treated samples reveal a better interface condition compared to BC soil.



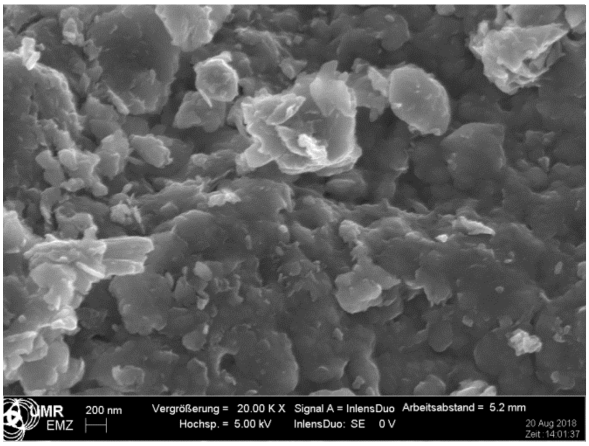

(a)



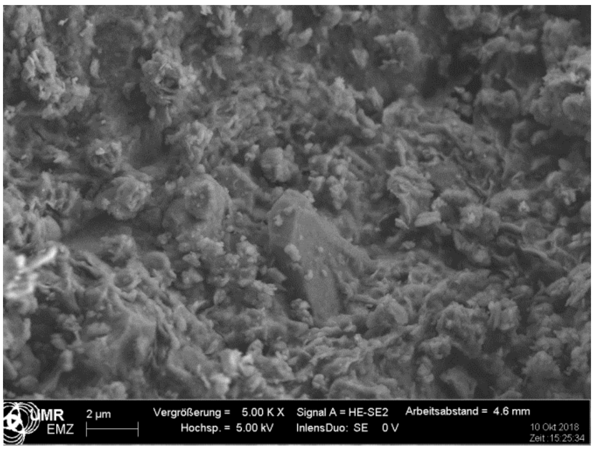

(b)



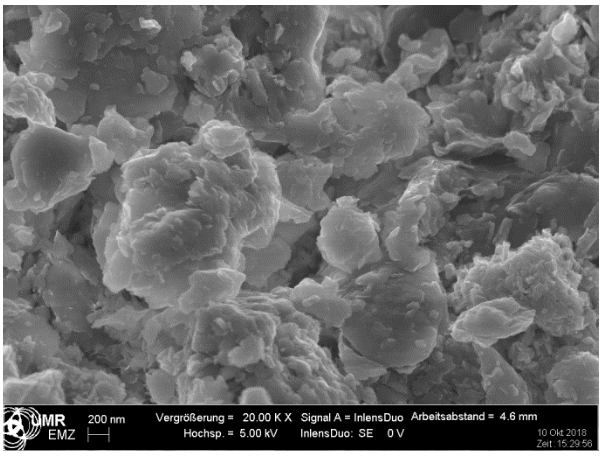

(c)



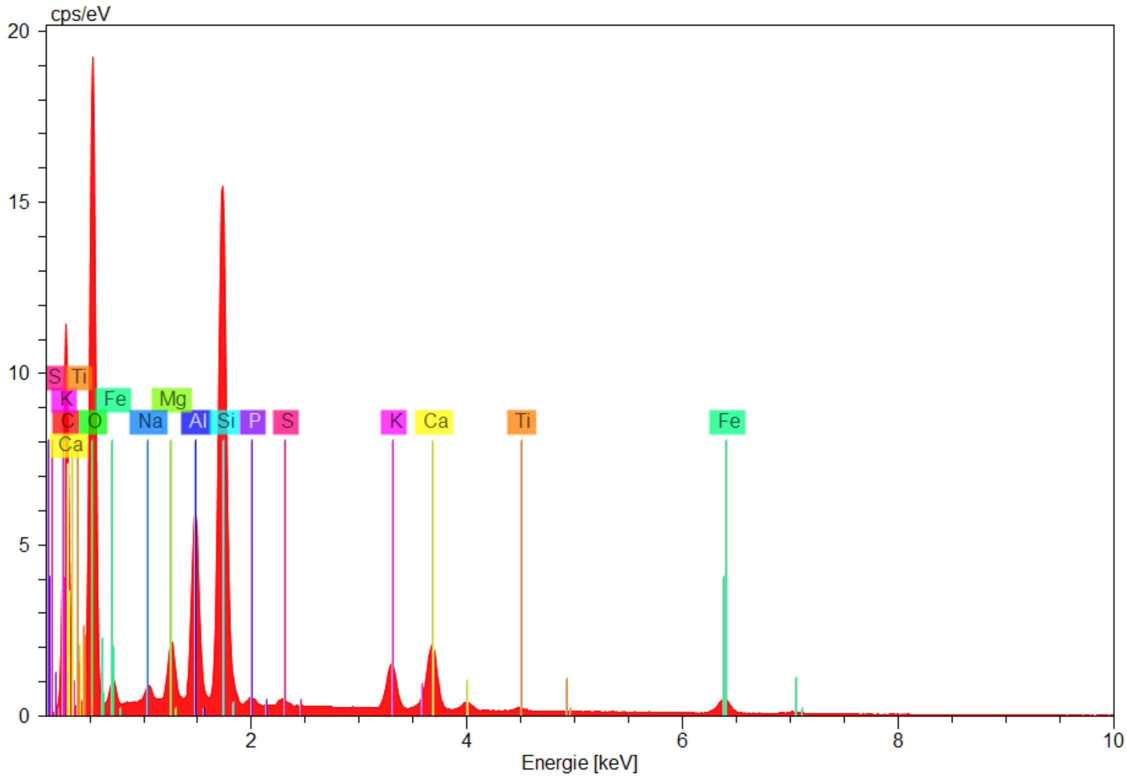

(d)




(e)

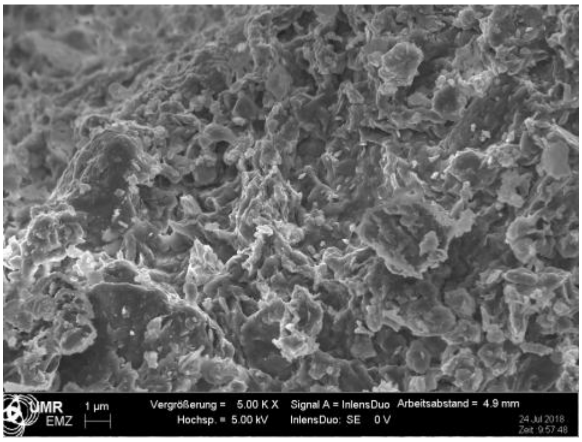



(f)

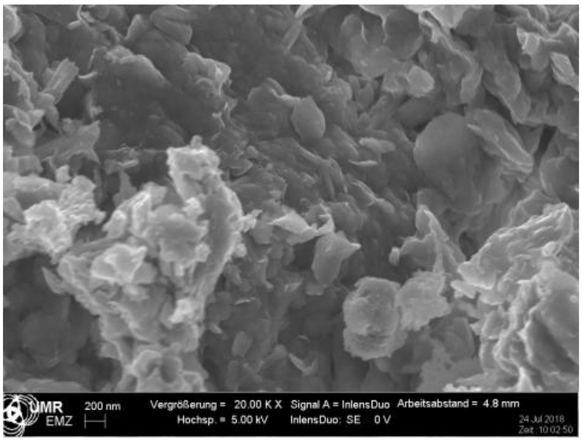



(g)

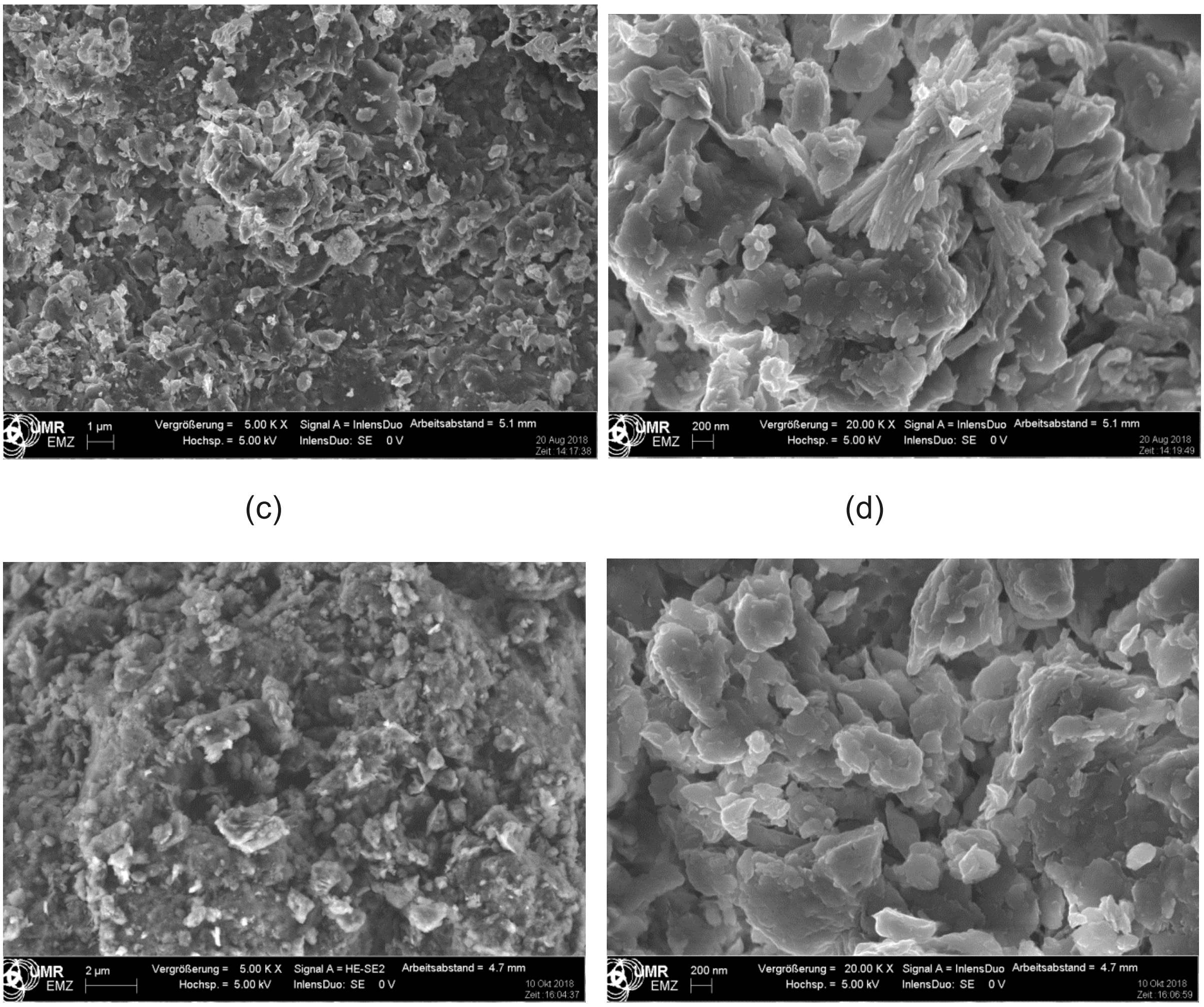
Figure 5.8: SEM images of the BC soil treated with 10% CHA (a) 5000x, 7d; (b) 20,000x, 7d; (c) 5000x, 14d; (d) 20,000x, 14d; (e) 5000x, 28d; (f) 20,000x, 28d and (g) EDX spectrum of BC soil treated with 10% CHA after 7 d of curing




(a)

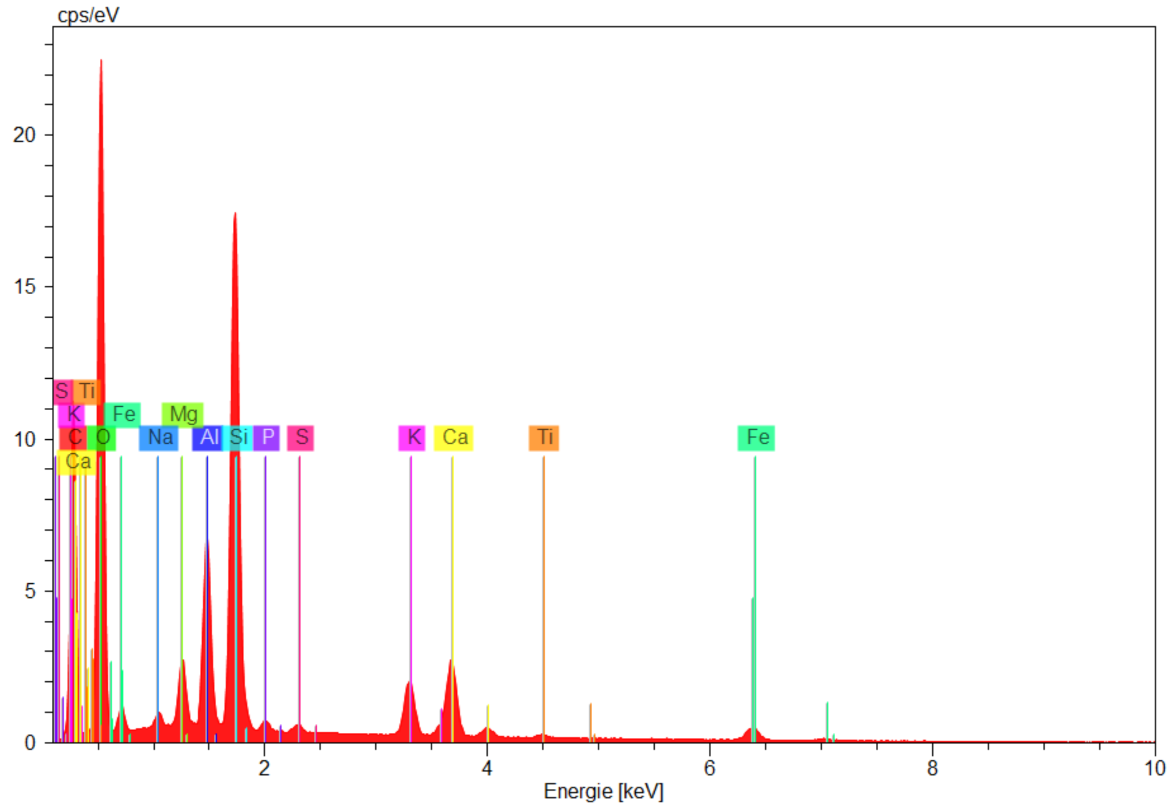



(b)


UMR EMZ
1 µm
Vergrößerung = 5.00 K X Signal A = InlensDuo Arbeitsabstand = 5.1 mm
Hochsp. = 5.00 kV InlensDuo: SE 0 V
UMR EMZ
200 nm
Vergrößerung = 20.00 K X Signal A = InlensDuo Arbeitsabstand = 5.1 mm
Hochsp. = 5.00 kV InlensDuo: SE 0 V
UMR EMZ
2 µm
Vergrößerung = 5.00 K X Signal A = HE-SE2 Arbeitsabstand = 4.7 mm
Hochsp. = 5.00 kV InlensDuo: SE 0 V
UMR EMZ
200 nm
Vergrößerung = 20.00 K X Signal A = InlensDuo Arbeitsabstand = 4.7 mm
Hochsp. = 5.00 kV InlensDuo: SE 0 V

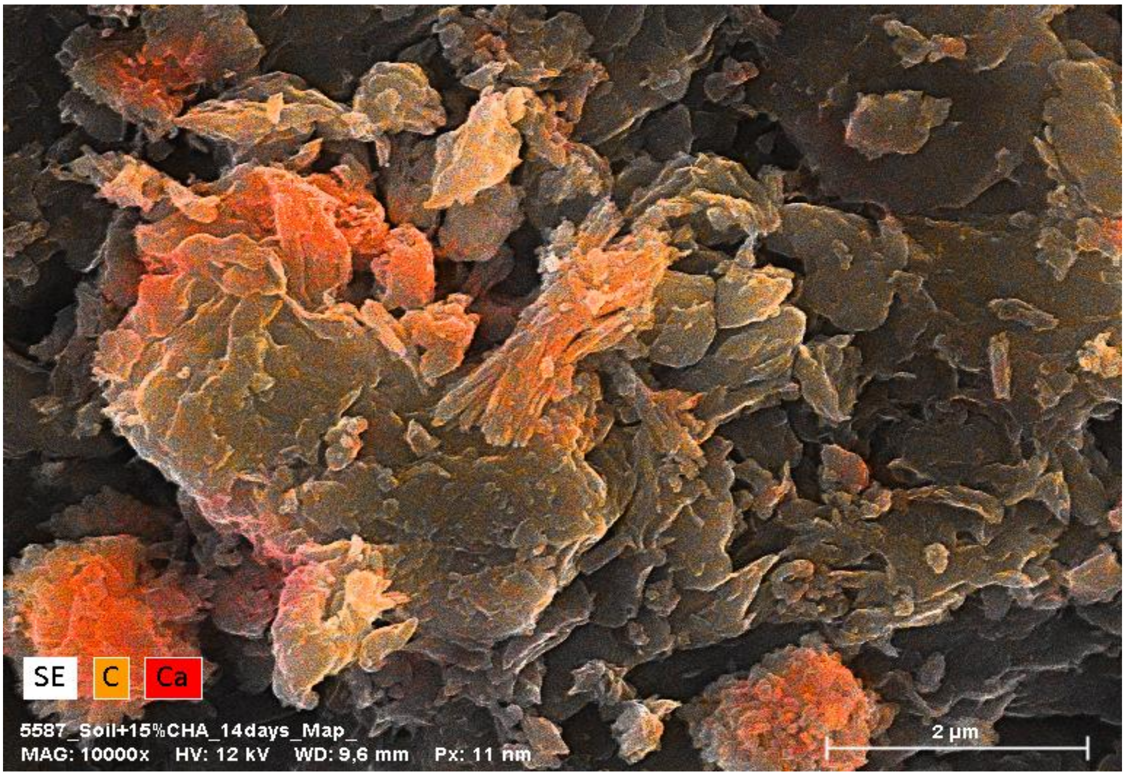
(c) (d)

(e) (f)

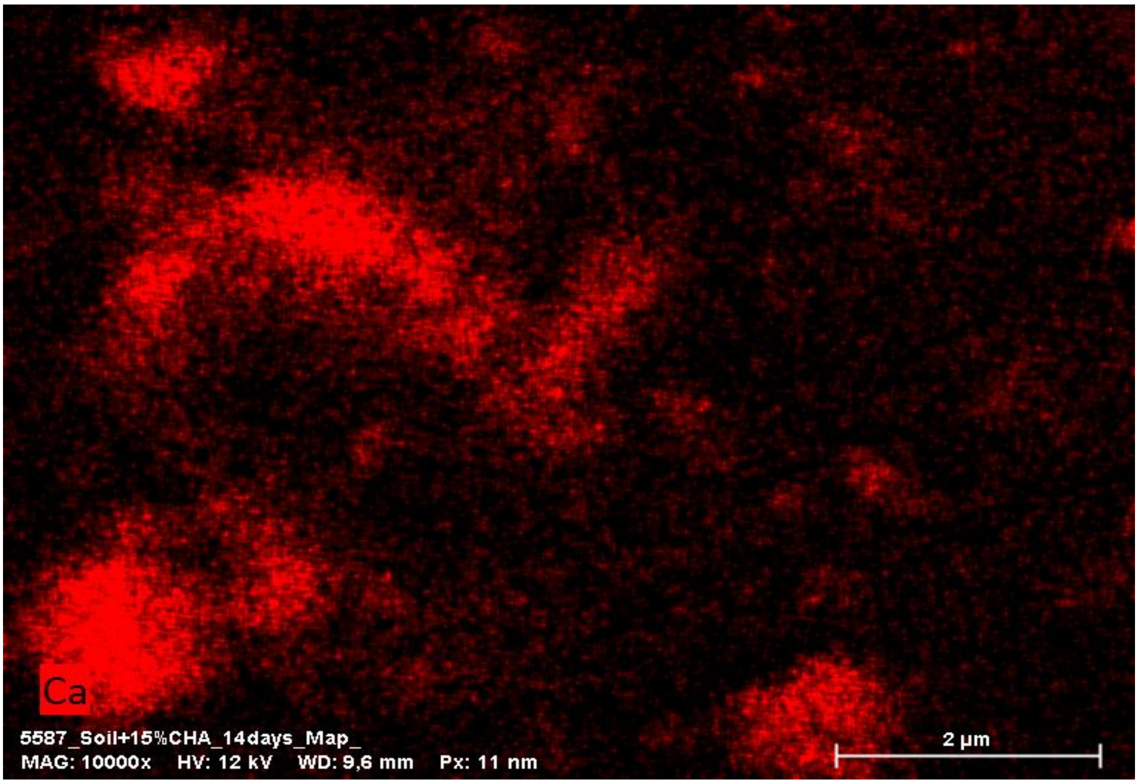



(g)

Figure 5.9: SEM images of the BC soil treated with 15% CHA (a) 5000x, 7d; (b) 20,000x, 7d; (c) 5000x, 14d; (d) 20,000x, 14d; (e) 5000x, 28d; (f) 20,000x, 28d and (g) EDX spectrum of BC soil treated with 15% CHA after 7 d of curing




(a)




(b)

Figure 5.10: SEM images of the BC soil treated with 15% CHA (a) showing Ca distribution (b)

## 5.5 Effectiveness of the Proposed Method

The effectiveness of the CHA as a stabilizer was evaluated using the results from Atterberg limits, UCS and CBR tests. Holtz and Gibbs [36] related the plasticity index value with swelling potential (see Table 5.2). According to this relation the swelling potential of the BC soil is high. The addition of CHA content 15% and above reduces the swelling potential to medium swelling.

The bearing capacity of CHA treated BC soil to use as sub-grade material was evaluated using the relation given by Bowles [74] (see Table 5.3). The CBR value of the BC soil was classified as very poor. The quality of the BC soil improved when treated with 20% CHA, which classified as poor to fair.

Das [73] related the UCS values and the quality of the soils used in pavement applications. The consistency of the BC soil is medium, after stabilization it became stiff to very stiff (see Table 5.4). Strength gain factor is given as the ratio of the UCS of the cured (7 and 14 days) and CHA treated soil to that of the untreated BC soil cured for 1 day. The highest strength gain factor was found by addition of 15%CHA and after 14d curing.

Table 5.2: Swelling potential depending on PI values (CHA treated).

| Mixture | PI | Swell potential after PI value |
|---|---|---|
| BC soil | 52.94 | Very high |
| 5CHA | 51 | Very high |
| 10CHA | 42.43 | High |
| 15CHA | 25.55 | Medium |
| 20CHA | 22.69 | Medium |

Table 5.3: Quality of sub-grade depending on CBR values (CHA treated).

| soil mixture | Condition | CBR value (%) | CBR gain | Quality after CBR value |
|---|---|---|---|---|
| BC soil | soaked | 1.02 | 1 | Very poor |
| | un-soaked | 8.35 | 1 | Fair |
| 5%CHA | soaked | 1.55 | 1.51 | Very poor |
| | un-soaked | 8.89 | 1.06 | Fair |
| 10%CHA | soaked | 1.94 | 1.89 | Very poor |
| | un-soaked | 9.29 | 1.11 | Fair |
| 15%CHA | soaked | 2.33 | 2.27 | Very poor |
| | un-soaked | 9.69 | 1.15 | Fair |
| 20%CHA | soaked | 3.11 | 3.03 | Poor to fair |
| | un-soaked | 10.67 | 1.27 | Fair |

Table 5.4: Consistency depending on UCS values (CHA treated).

| Mixture | Curing time (days) | UCS value (kPa) | Strength gain | Consistency after UCS value |
|---|---|---|---|---|
| BC soil | 1 | 81.16 | 1 | medium |
| | 7 | 89.27 | 1.09 | medium |
| | 14 | 107.05 | 1.31 | stiff |
| 5%CHA | 1 | 94.87 | 1.16 | medium |
| | 7 | 101.17 | 1.24 | stiff |
| | 14 | 113.03 | 1.39 | stiff |
| 10%CHA | 1 | 219.36 | 2.70 | very stiff |
| | 7 | 228.95 | 2.82 | very stiff |
| | 14 | 239.69 | 2.95 | very stiff |
| 15%CHA | 1 | 231 | 2.84 | very stiff |
| | 7 | 240.3 | 2.96 | very stiff |
| | 14 | 245.4 | 3.02 | very stiff |
| 20%CHA | 1 | 147.69 | 1.81 | stiff |
| | 7 | 160.58 | 1.97 | stiff |
| | 14 | 162.33 | 2.00 | stiff |

## 5.6 Conclusions

The addition of CHA to the BC soil led to increase the CBR value. The soaked CBR value of untreated BC soil is found to be 1.02%, after treated with 20% CHA, the value increased by three-fold and found to be 3.1%. For the un-soaked CBR test, the addition of 20% CHA increases the bearing capacity from 8.3% to 10.67%. The percentage increment of CBR value for un-soaked condition is 28%, while it increases by 203% for soaked condition. From these results, it can be concluded that CHA is more effective for wet condition than dry condition. The results unveil that CHA improves the bearing capacity of the investigated soil.

The study also reveals that the CBR swell percentage of the BC soil decreases as the concentration of the CHA increases. For maximum experimented concentration of CHA (20%), the swell percentage decreases from 10% to 5.7%, this indicates a significant decrement of about 43%. The shear strength of the BC soil also showed improvement after CHA treatment which may attributed to the agglomeration of particles and decreased void ratio of CHA treated samples.

From SEM analysis, it is observed that the structure of a material has a big influence on engineering properties. SEM images of CHA treated samples indicates a reduction in pores, which may attributed to the formation of cementitious products that are perhaps contributed to the improvement of BC soil properties. In addition, the

XRD results confirmed the formation of cementitious compounds in CHA treated samples.

In general, it can be concluded that the stabilization of BC soil using CHA improves the bearing capacity, shear strength and the microstructure of the BC soil. In addition, the potential use of this waste material as a stabilization agent has a positive effect in reducing the cost related to stabilization and furthermore it reduces the disposal problems and environmental concerns related to it.

# 6 Stabilization of Black Cotton Soil with Lime and Coffee Husk Ash Admixture

## 6.1 Effect on Plasticity

The LL, PL and PI of samples treated with lime and lime-CHA mixture are presented in Figures 6.1-6.3. It can be observed that with addition of lime there is reduction in the LL. The PL of the samples increased on the addition of lime. The amount of this increase varies directly with the amount of lime added. With the addition of 4% lime alone, the PI was reduced from 52% to 33%. It is obvious that the addition of lime improves the plasticity behaviour of the soil; a similar trend was found by several researchers [83, 124 and 127].

The treatment with CHA could be an effective treatment option in reducing the plasticity of the BC soil. However, the mixture of lime and CHA shows a better reduction in LL and PI. PI of less than 15% was found for samples with the following mixtures SL6C15, SL6C20, SL8C10 SL8C15 and SL8C20. The classification of untreated and treated samples according to USCS and AASHTO, respectively, are shown in Figures 6.4 and 6.5. According to USCS (see Figure 6.4), the BC soil was classified as CH, after stabilization, both LL and PI decreased shifting the samples downward on the PI-axis and left on the LL-axis from an area of high plasticity clay to high plasticity silt and low plasticity silt. From Figure 6.5 (AASHTO chart), it can be observed that the BC soil is classified as A-7-5, which shifted to A-5 with the addition of 6% lime and 15% CHA, and goes downward left as the percentage of additives increases.

According to the ratings of sub-grade materials by AASHTO, the treated samples perform better as a sub-grade material compared to the untreated BC soil.

The improvement observed in the plasticity behavior of stabilized samples is attributed to the cation exchange reaction (replacement of the exchangeable sodium, magnesium, or other cations previously held by the clay, by calcium cations derived from the additives). In most cases the effect of lime on the plasticity of clay soil is due to that the clay particles move towards each and other and tend to clump together to promote flocculation and particle aggregation [88]. The aggregates behave like particles of silt, thereby decreasing the liquid limit and increasing the plastic limit. The observed effects, decreasing in the liquid limit and increasing in the plastic limit generally leads to a reduction in the PI. The subsequent decrease in the water affinity will increase the workability, and improve the strength and deformation properties of a soil.

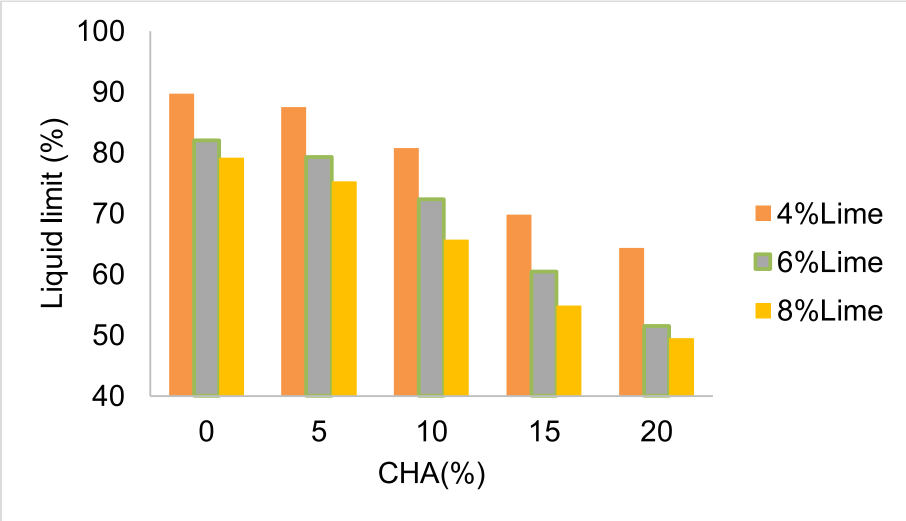



Figure 6.1: Liquid limits of samples treated with lime and with the mixture of CHA

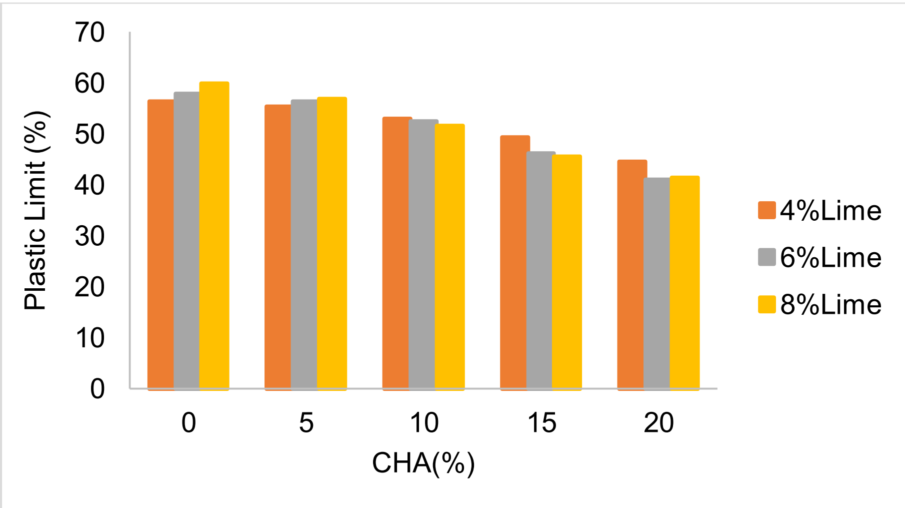



Figure 6.2: Plastic limits of samples treated with lime and with the mixture of CHA

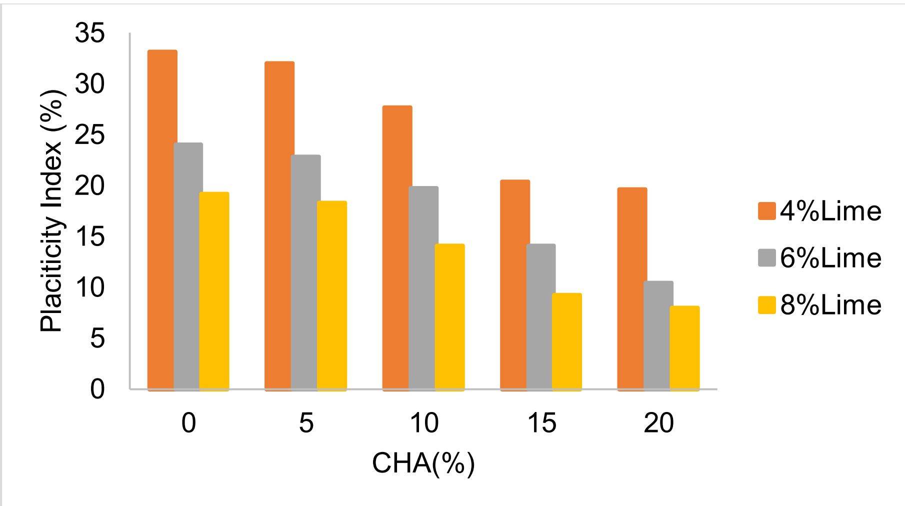



Figure 6.3: Plasticity index of samples treated with lime and with the mixture of CHA

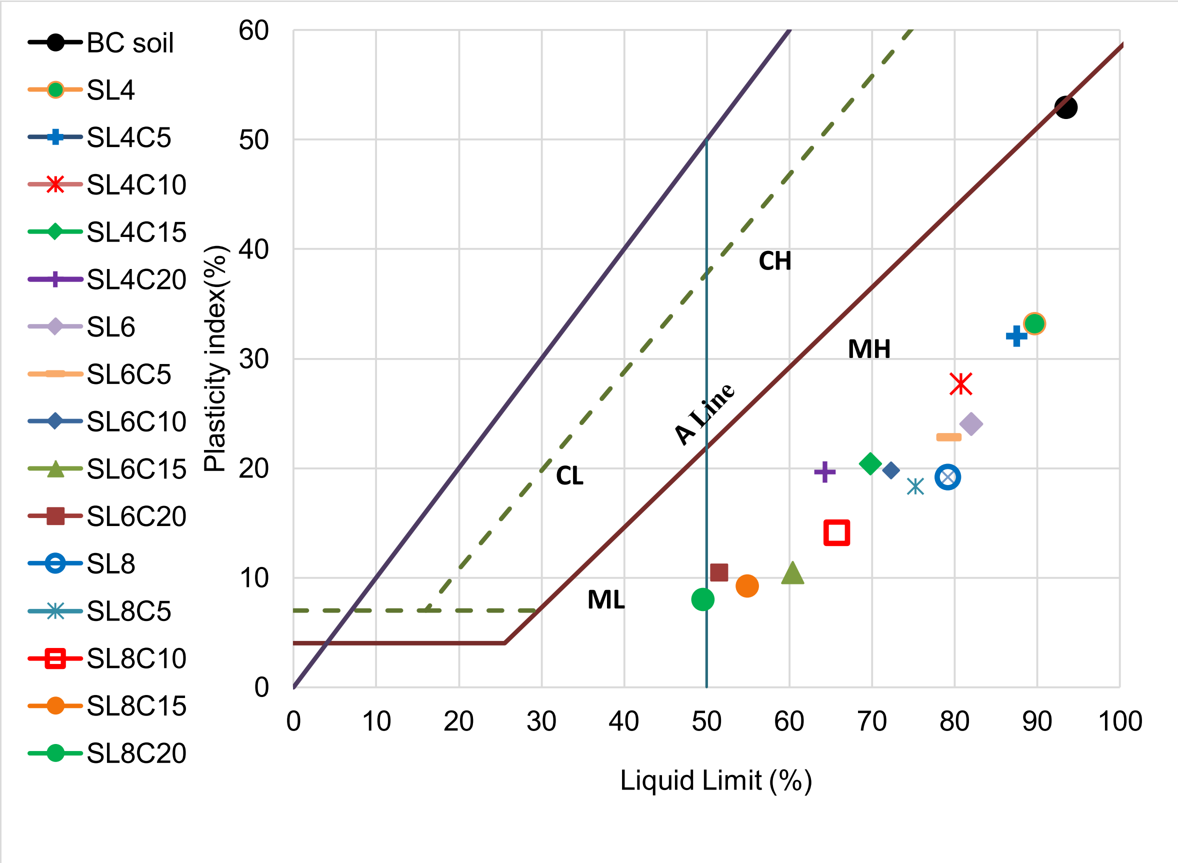



Figure 6.4: Plasticity index versus liquid limit for untreated and treated samples (USCS chart)

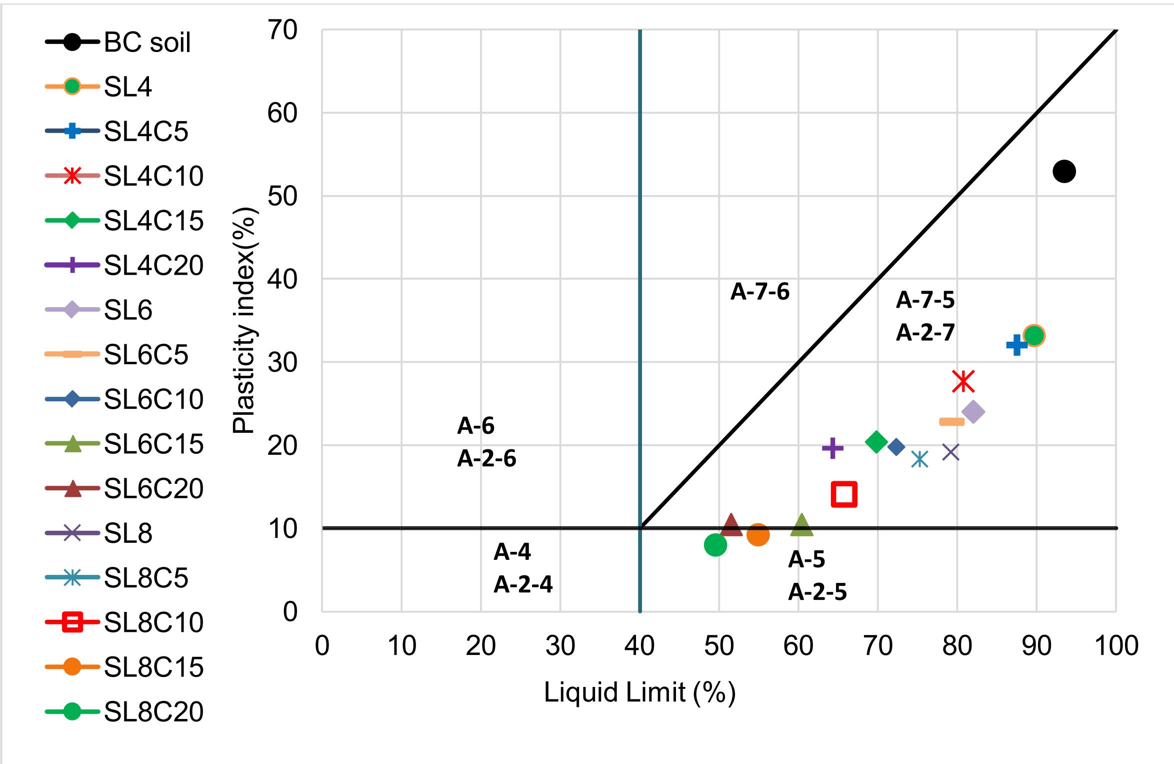



Figure 6.5: Plasticity index versus liquid limit for untreated and treated samples (AASHTO chart)

## 6.2 Effect on Compaction

The effect of CHA on the compaction characteristics of the lime-treated BC soil was evaluated by conducting a standard proctor test. It can be seen that the MDD of untreated and CHA-treated soils decreases as the lime content increases (see Figure 6.6). Conversely, the OMC increases as the lime content increases (see Figure 6.7). Similar behaviours in MDD and OMC were observed by other researchers ([78, 94]) when lime was added to expansive soils. The OMC increased from 37% to 44%, when the amount of lime added increased from 0% to 8%. These changes in compaction behaviour due to lime treatment could be the result of a reaction between the soil and the lime, which is responsible for the increase in OMC. The aggregation of the particles to occupy larger spaces due to the addition of lime and the lower specific gravity of the lime could be responsible for the decrease in MDD.

On the contrary, the addition of CHA to the lime-stabilized soil slightly increases the MDD and decreases the OMC, as shown in Figures 6.6 and 6.7. The OMC decreases as the percentage of CHA increases for all lime content. As discussed in section 4.3 the reduction in the OMC of CHA- treated samples could be attributed to the amount of water needed to reach an optimum state is lower for CHA-treated samples compared to untreated samples; this could be due to the lower affinity of CHA particles for water.

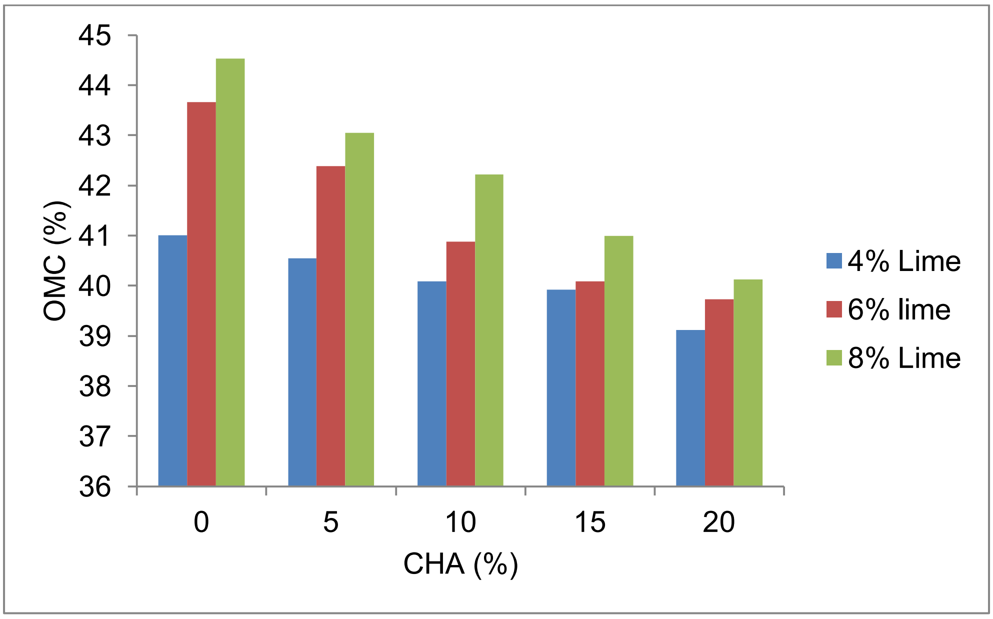



Figure 6.6: Variation of OMC with additive content

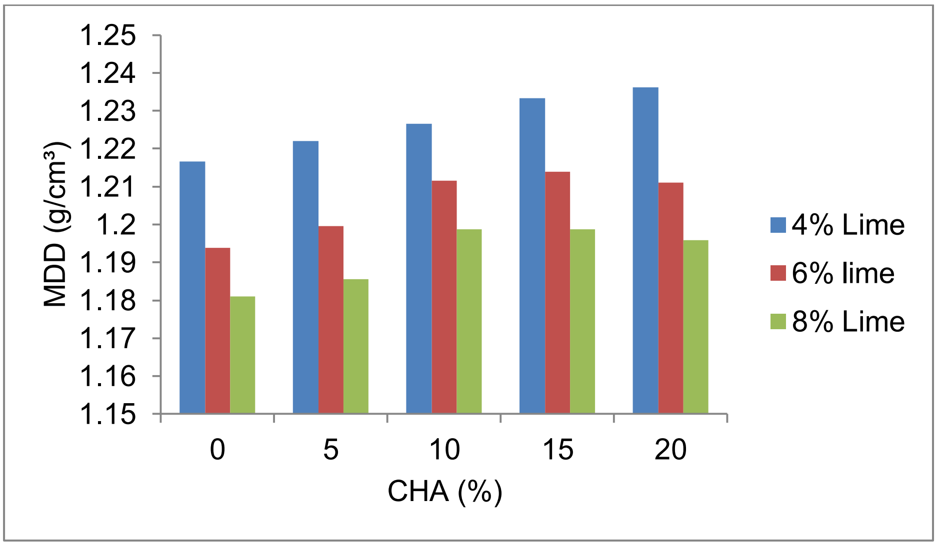



Figure 6.7: Variation of MDD with additive content

## 6.3 Unconfined Compressive Strength and Resistance to Wetting-Drying Cycles

UCS of samples, BC soil-lime and BC soil-lime-CHA mixtures were prepared at their respective OMC and MDD as discussed in section 3.4.5. The UCS was measured after 7 days of curing.

The general effect of lime and lime-CHA treatment on the BC soils is illustrated in Figure 6.8. The addition of 6% lime to the BC soil led to a radical increment in the UCS. All samples, treated with lime and lime-CHA mixture showed improvement in UCS values compared to the BC soil. However, decrement of these values was observed for CHA content of more than 15%. The highest UCS value was attained for samples stabilized with 6% lime and 15% CHA. The improvement observed are due to the additives forming cementitious compounds that gives rise to stiffened response.

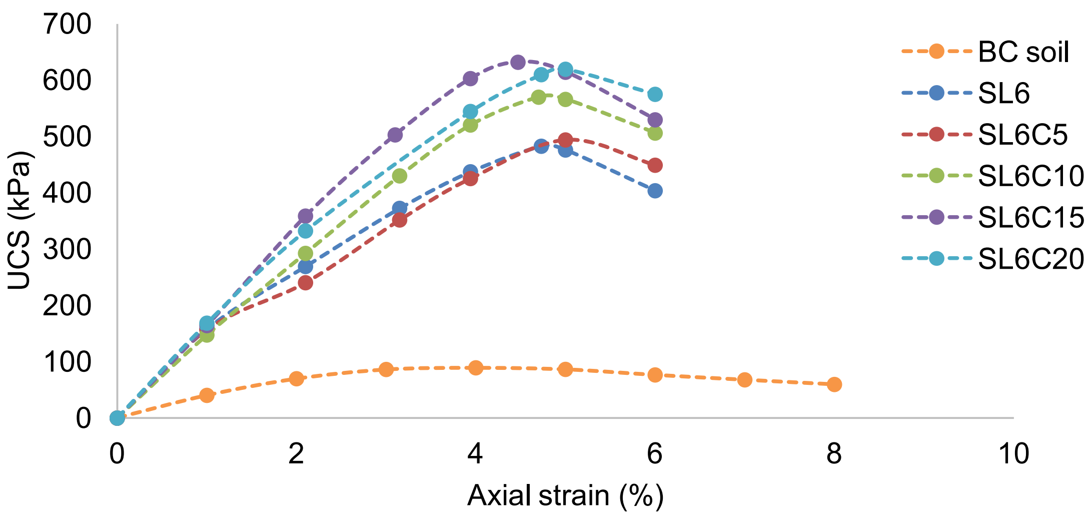



Figure 6.8: Unconfined compressive strength of lime and lime-CHA stabilized samples

UCS test was additionally carried out to access the durability of lime and lime-CHA stabilized samples after completion of each W/D cycle. As previously discussed in section 4.6, the untreated BC soil samples did not survive the first W/D cycle. This fact is attributed to the high water absorbition capacity of the soil. It can be stated that samples treated with a mixture of lime and CHA resist effect of W/D cycles better compared to samples treated with CHA alone.

The strength of lime treated soil increased slightly in the first W/D cycle. After that, the strength declined as the W/D cycles increased as indicated in Figure 6.9. The slightly increased strength with first W/D cycle is attributed to the additional gain of curing for treated soil during W/D cycle.

The treated soil starts to lose strength with time due to increasing the rate of water absorption when the number of W/D cycles is increased. This expansion helps to cause a reduction for particles interlocking within the treated soil and creates a change in the structure of treated soil. Subsequently, the compressive strength declines with time. This effect may be able to disturb the existing cementation between soil particles and additives in the treated soil, and the change of soil structure may reduce the performance of strength.

Generally, adding lime to CHA treated samples improved the durability as presented in Figures 6.10- 6.13. In addition resists more W/D cycles compared to samples treated with CHA only. This is attributed to the fact that lime develops sufficient rigidity for soil-CHA particles; thus it can resist the effect of W/D actions on the compressive strength.

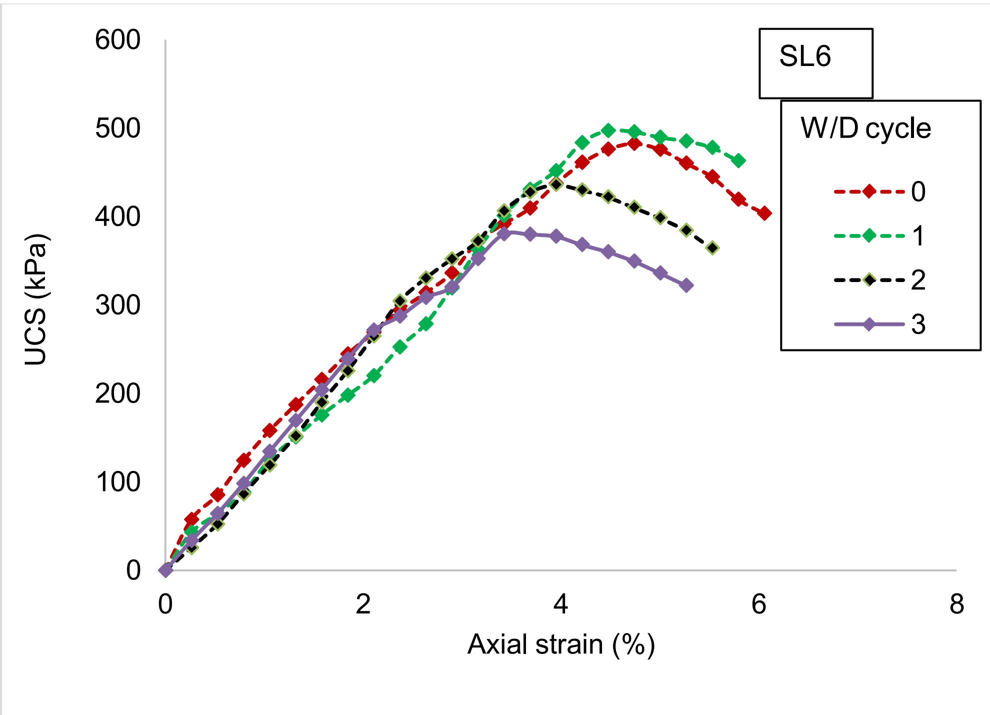



Figure 6.9: Stress-strain curves of samples treated with 6% lime before and after subjected to W/D cycle

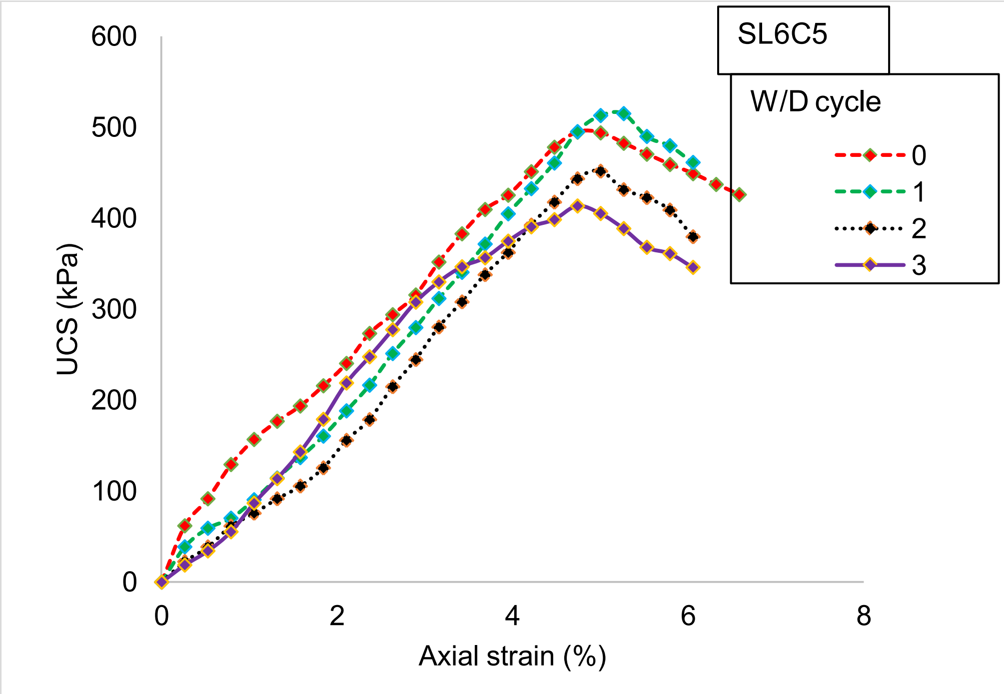



Figure 6.10: Stress-strain curves of samples treated with a mixture of 6% lime and 5%CHA before and after subjected to W/D cycle

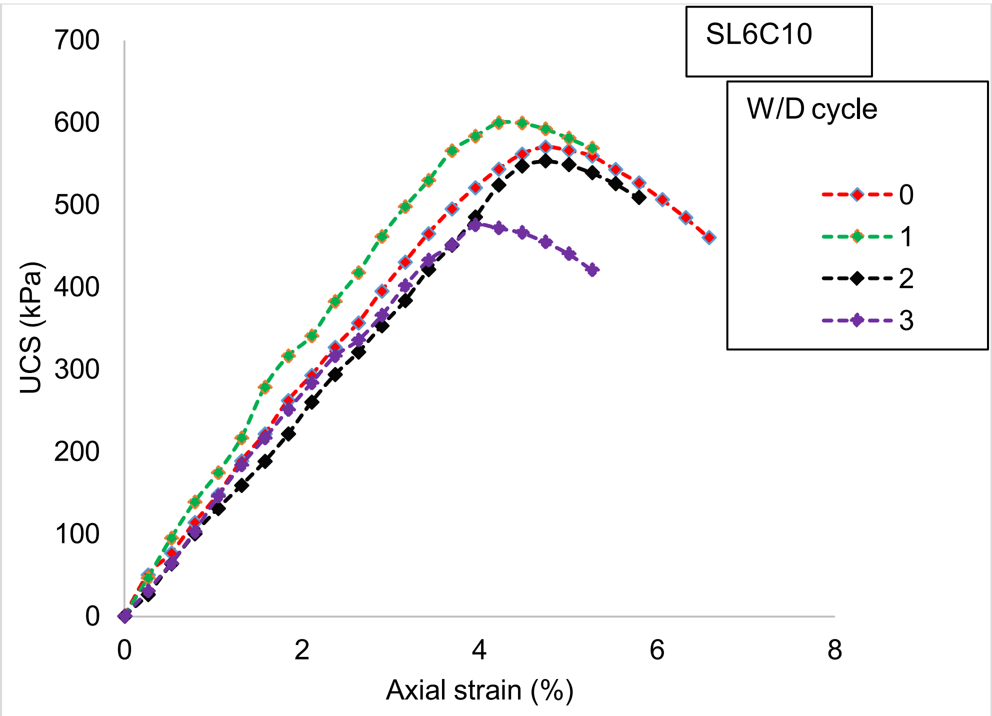



Figure 6.11: Stress-strain curves of samples treated with a mixture of 6% lime and 10%CHA before and after subjected to W/D cycle

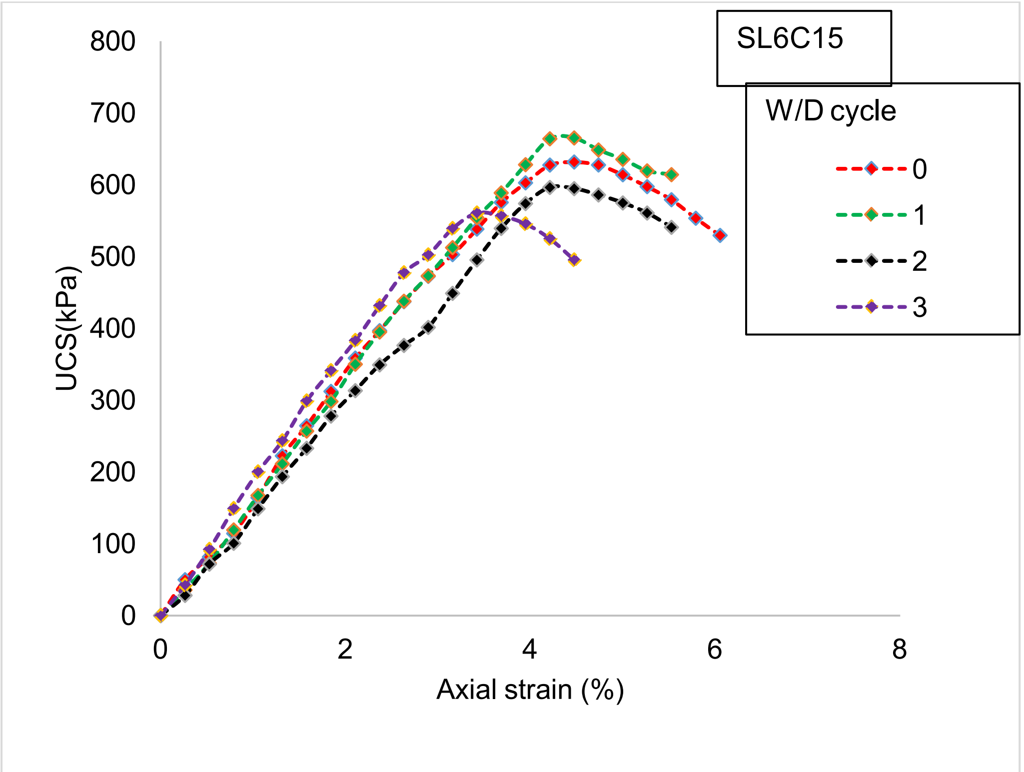



Figure 6.12: Stress-strain curves of samples treated with a mixture of 6% lime and 15%CHA before and after subjected to W/D cycle

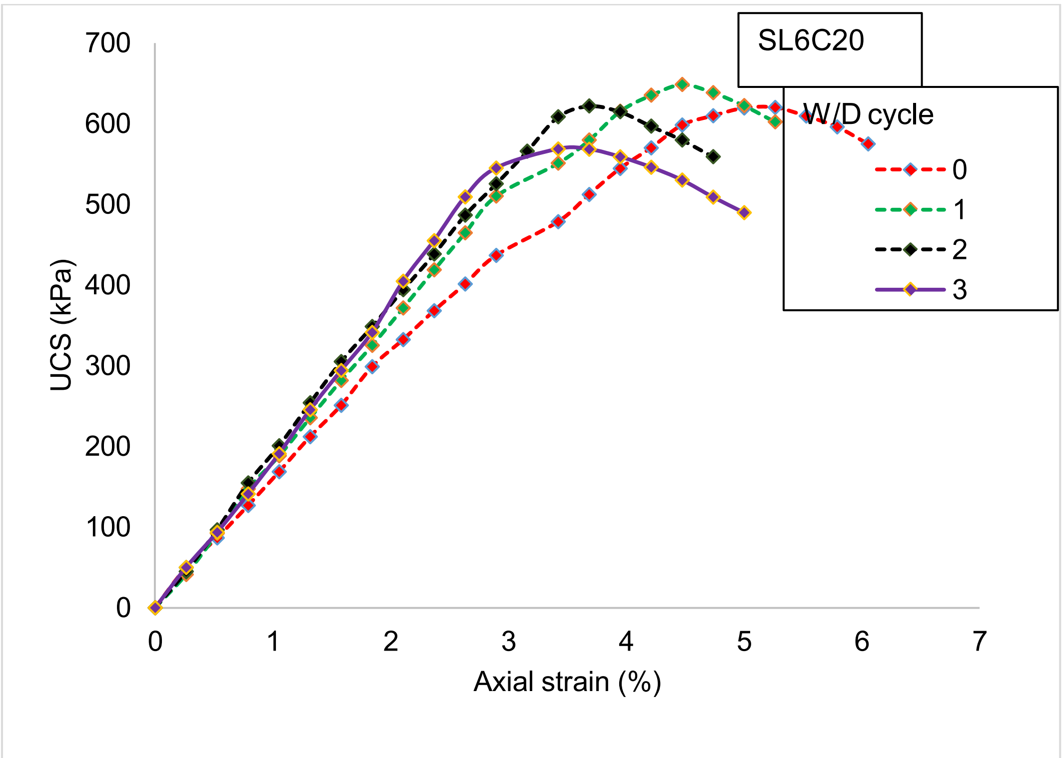


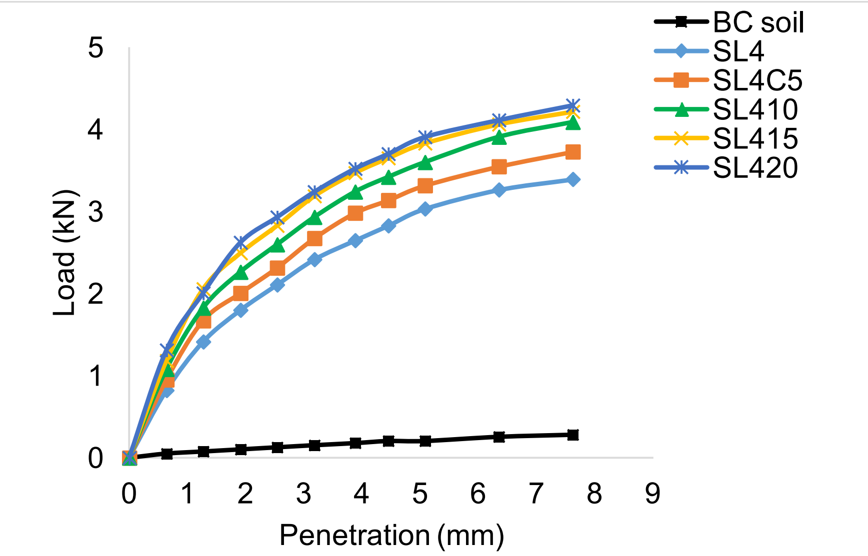

Figure 6.13: Stress-strain curves of samples treated with a mixture of 6% lime and 20%CHA before and after subjected to W/D cycle

## 6.4 California Bearing Ratio

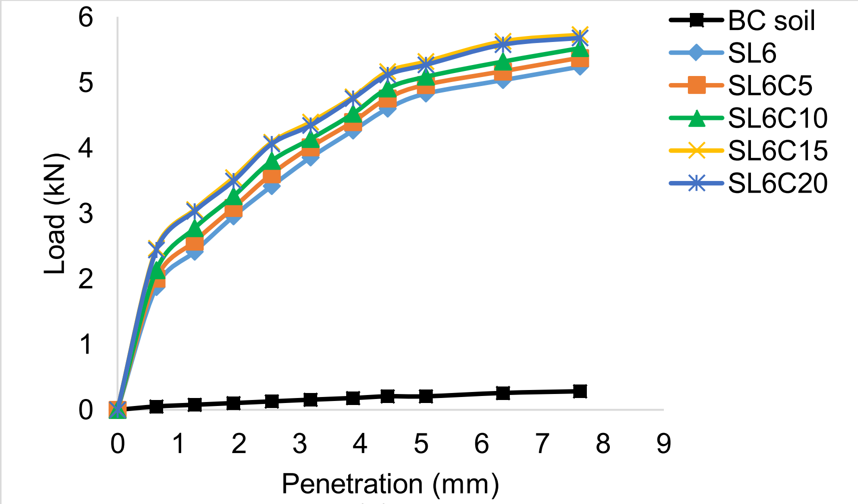
The CBR test is performed to evaluate the bearing capacity of CHA treated BC soil with the addition of lime. BC soil-lime and BC soil–lime-CHA mixtures were prepared with the lime content of 4%, 6%, and 8%. The samples from all mixture proportions were compacted at their respective OMC and MDD, and then CBR tests were conducted after four days of soaking.

The results obtained from conducted CBR tests on the BC soil treated with lime and with the mixture of lime and CHA are shown in Figure 6.14 as load-penetration curves. The addition of 4% lime alone increased the CBR value to 16% from 1% for the untreated BC soil. The mixture of lime and CHA treatment showed more enhancements on the bearing capacity of the BC soil compared to both lime and CHA treatment. As shown in Figure 6.15, the CBR values of the samples are almost inversely proportional to their plasticity index. The highest CBR values were found at 20% for CHA treatment (as indicated in section 5.1), at 8% for lime treatment and at 6% lime and 15% CHA for the mixture treatment, respectively.

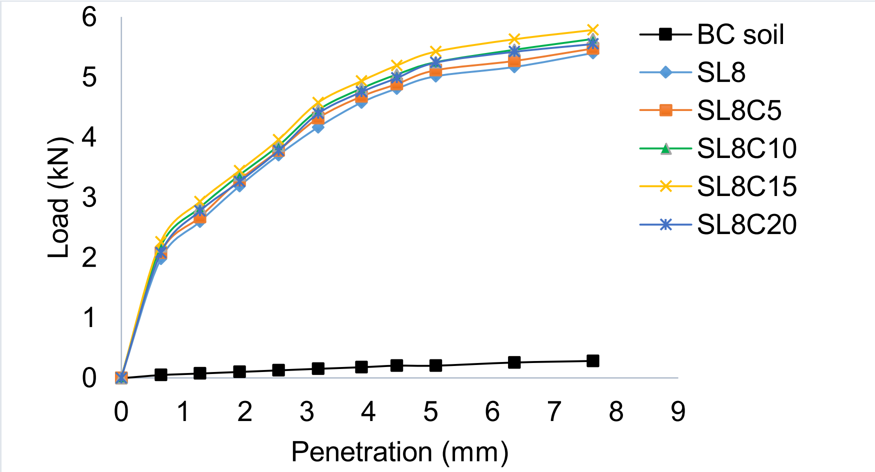
All the samples treated with lime attained CBR value more than 15%, however better bearing capacity was observed after the addition of CHA. This result indicates that CHA treated samples perform better as a sub-grade material; which requires lower thickness of the pavement structure compared to both the untreated and only lime treated BC soil.



(a)




(b)




(c)

Figure 6.14: Load versus penetration curves for 4% lime (a) 6% lime (b) and 8% lime (c)

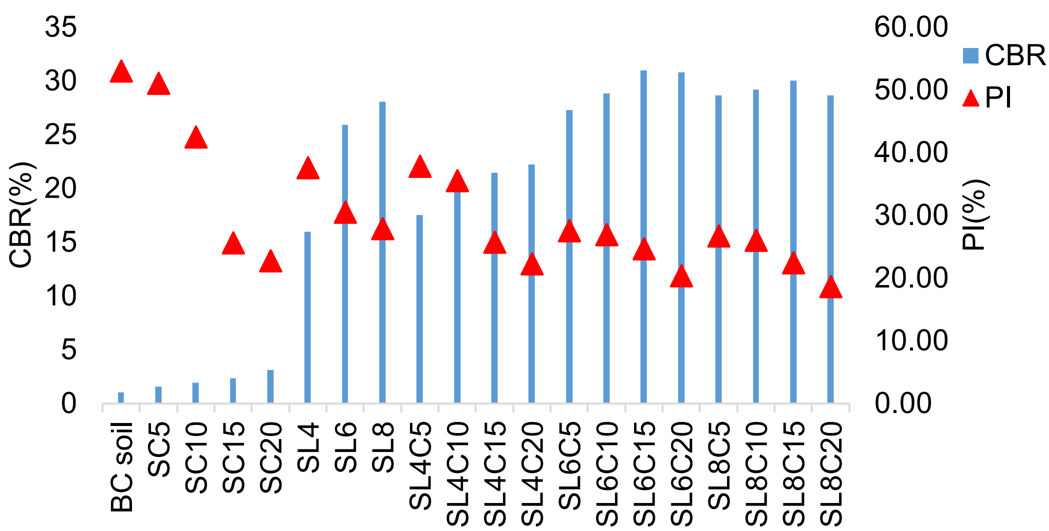



Figure 6.15: Variation of CBR and PI values for different concentrations of additives

CBR swell tests results of untreated and treated soil with different lime and lime-CHA contents are conducted. As shown in Figure 6.16, the CBR swell values decreased as the percentage of CHA, lime and lime-CHA increased. Both CHA and lime treatment reduces the CBR swell values. However, the change was significant for lime treated samples compared to CHA treated samples. The mixture of CHA and lime was more effective in reducing the CBR swell compared to both lime and CHA treatment. The increased CBR values and reduced swell measurements for higher additives content are probably as a result of the development of sufficient cementitious matrix. In addition, the replacement of swelling clay with CHA contributes to a notable reduction in CBR swell.

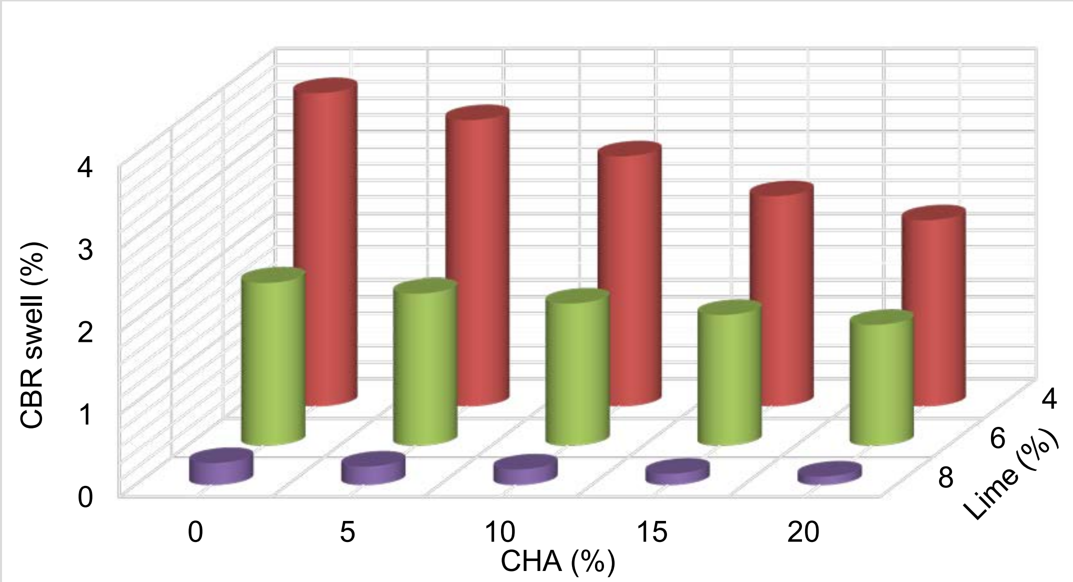



Figure 6.16: CBR swell values for different concentrations of additives

## 6.5 X-ray Diffraction Analysis

XRD analysis was carried out to observe the effect of lime, and lime-CHA mixture on the mineralogical properties of the BC soil. The minerals present in the BC soil are quartz, kaolinite and montmorillonite (see section 5.3). Figure 6.17 shows the XRD patterns of the untreated BC soil, SL6, SL6C10 and SL6C15. It was observed that the peak intensities of these minerals changed after the treatment. The mineral, quartz, appears in all samples. However, there was a reduction in the peak intensities of quartz for all mixtures. The disappearance of kaolinite peaks was also noticed. The disappearance of peaks could be related to the appearance of new peaks in the pattern due to the reaction of additives with the soil, forming crystalline phases such as calcite [100].

Both lime and CHA treatment leads to the formation of new minerals, mainly calcite. Calcite is known as a hardening material, which helps in improving the strength of the treated samples. It could be formed from the reaction between the calcium in the additives, water in the samples and carbon dioxide in the atmosphere. For SC10, the calcite peak appeared and for SC15, the intensities of this peak became higher (see

Figure 5.4). Further, additional calcite peaks also appeared for SC15. CHA-treated samples showed lower peak intensities of clay minerals and the appearance of crystalline phases. However, lime-treated samples showed more and higher calcite peak intensities compared to samples treated with CHA only. The increase in the intensities of calcite peaks may be attributed to the availability of more calcium from the additives, as the concentration of the additives increased. The formation of more cementitious compounds, mainly calcite, could also be due to admixture content.

Figure 6.18 shows X-ray diffraction patterns of BC soil treated with a mixture of 6% lime and 15% CHA and cured for 7d, 14d and 28d. The intensities of peaks related with cementitious compound (calcite) increased with increasing curing time. These results demonstrate that the quantity of the reaction product increases with curing time.

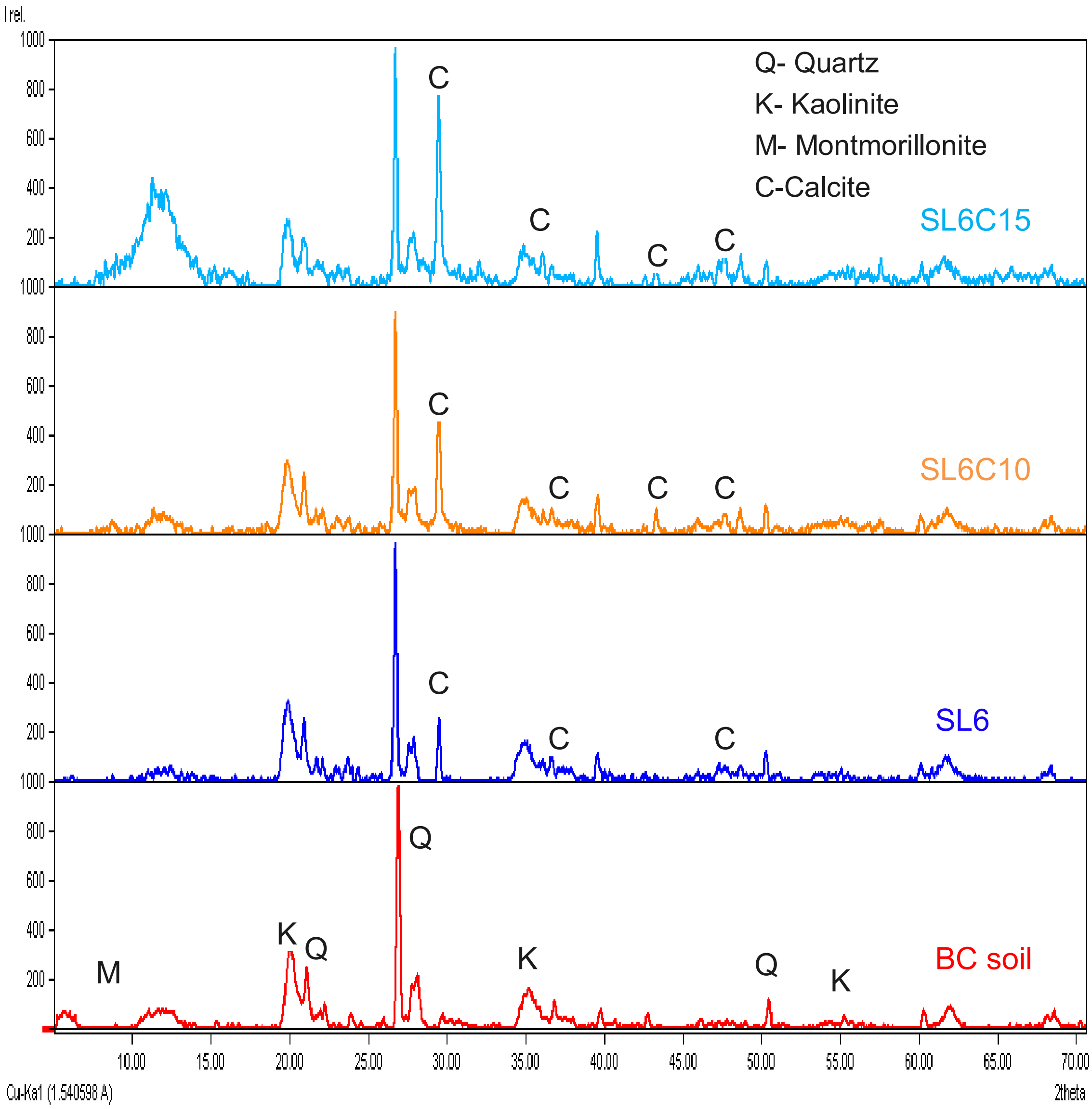



Figure 6.17: X-ray diffraction of samples of untreated BC soil and samples treated with lime and lime-CHA mixture, SL6, SL6C10 and SL6C15

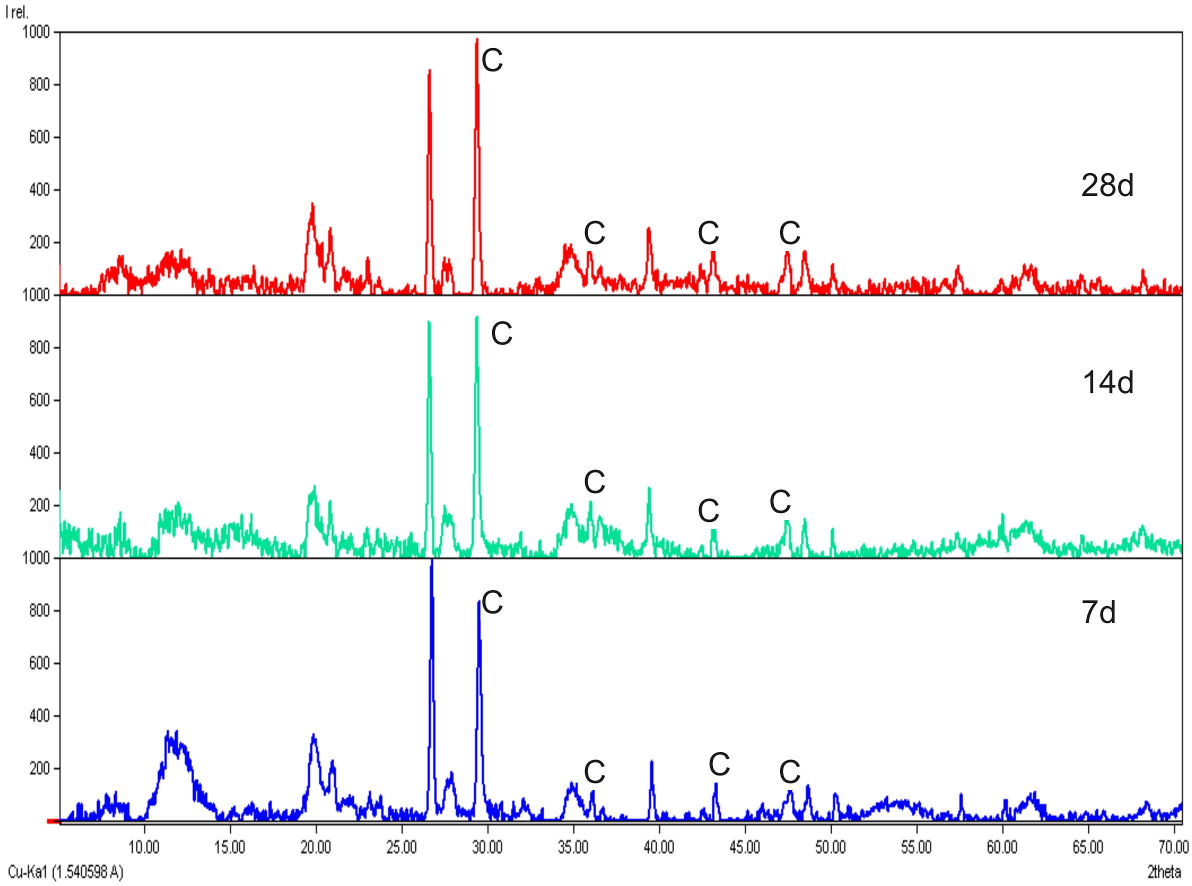
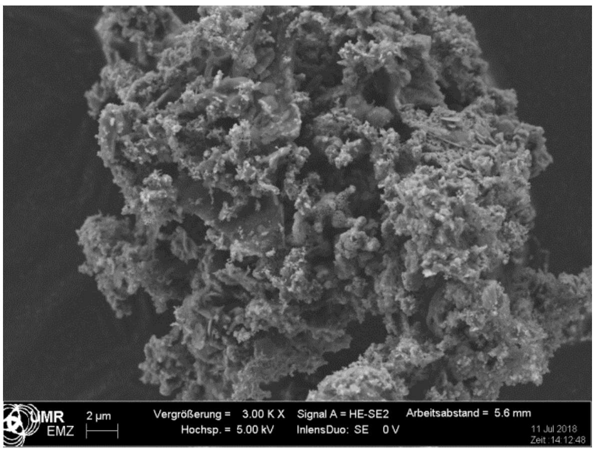


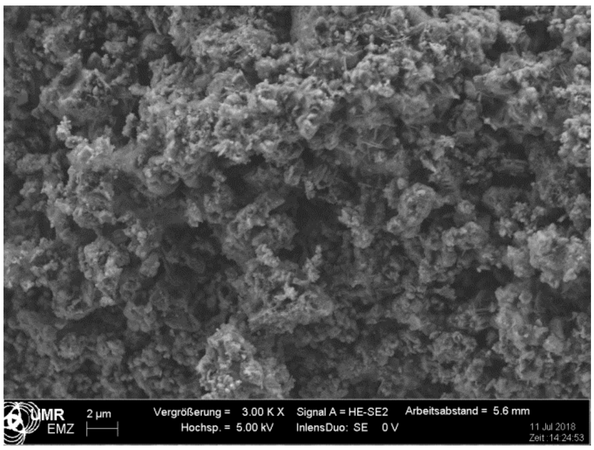

Figure 6.18: X-ray diffraction of samples treated with a mixture of 6% lime and 15% CHA (SL6C15) and cured for 7d, 14d and 28d

## 6.6 Micro-Structural Analysis

Scanning electron microscopy was used to observe the effects of lime and lime-CHA mixture on the morphological structure of the BC soil. The SEM images of additives, CHA and lime shows similarity in their microstructure (see Figure 6.19).




CHA




Lime

Figure 6.19: Microstructure of CHA and lime at magnification of 3000x

Figures 6.20-6.22 illustrates micrographs of the BC soil treated with 6% lime, and with mixture of 10%CHA and 15%CHA (SL6, SL6C10, and SL6C15), and cured for 7d, 14d and 28d. The micrographs of the samples were taken at magnifications of 5000x and 20000x.

As shown in Figure 5.6, the presence of pores is evidenced in the microstructure of the BC soil, and these pores are more visible compared to treated samples (see Figures 6.20-6.22), which could be attributed to the nonexistence of hydration products to form a continuous surface in the untreated sample.

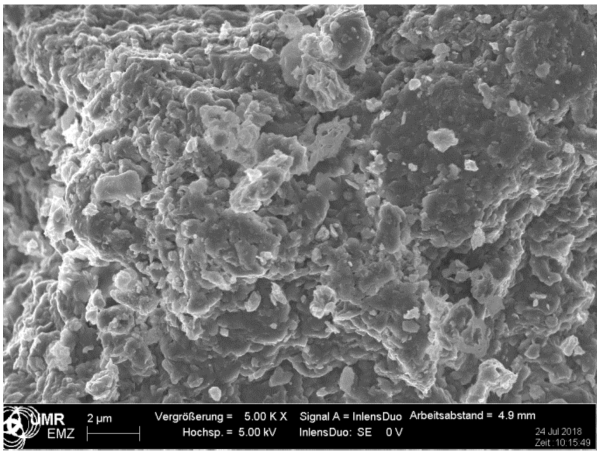
The microstructure of soils play significant role in their geotechnical properties. It is well known that stabilization of expansive soils using lime improves the geotechnical properties of these soils due to the ionic exchange and flocculation, and by the pozzolanic reactions thereafter with curing periods. Similar trend were observed in this study for both lime and lime-CHA treated samples.

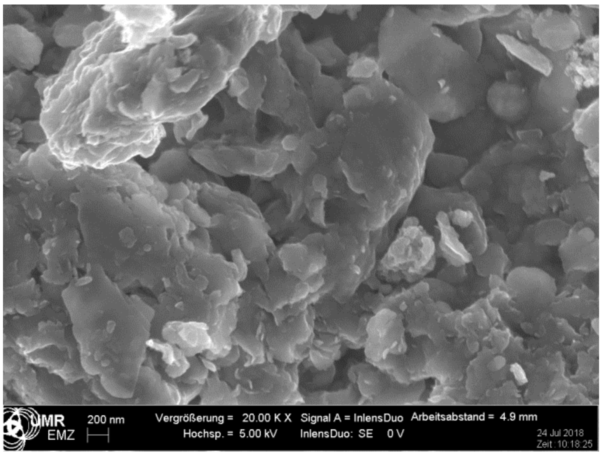
After treating the BC soil with lime and lime-CHA mixtures, changes in its microstructure were observed. For both lime and lime-CHA treated samples a granular structure with cementitious compounds is observed (see Figures 6.20-6.22). The granular structure of the treated samples proves that there was flocculation due to the addition of additives.

The formation of new minerals proved that there was pozzolanic reaction. Pozzolanic reaction between clay minerals and the additives result in formation of the cementitious compounds. The increase in the amount of additives could be a reason for the formation of additional cementitious compounds and more with curing time. In addition, the treated samples showed the formation of white lumps of calcium. The micrograph of treated samples cured for 14 and 28 d shows the formation of additional cementitious compounds after long-term curing as a result of the pozzolanic reaction.

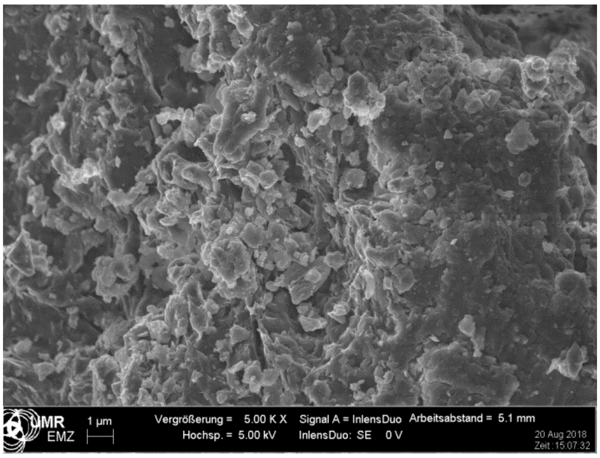
The EDX spectrum was used to analyse the changes occurring in the chemical composition of the BC soil after treatment. EDX analyses were performed at representative areas of the examined samples. From EDX results (see Figure 5.6), it was observed that the BC soil contains mainly Si and Al. After treatment, changes in the elemental composition of the BC soil were observed. The amount of Si and Al decreases as the additive content increases (see Figure 6.23). Besides, this result is supported by the EDX spectrum of Ca and K as shown in Figure 6.24. It is clear that the increase in the additives content has a significant effect on the amount of Ca, and the formation of cementitious products. The increase in the calcium content gave an indication of the formation of additional cementitious product; this result is in agreement with the XRD analysis (see Figures 5.4 and 6.17), which indicated the formation of additional peaks of the calcite as the content of the additives increased.

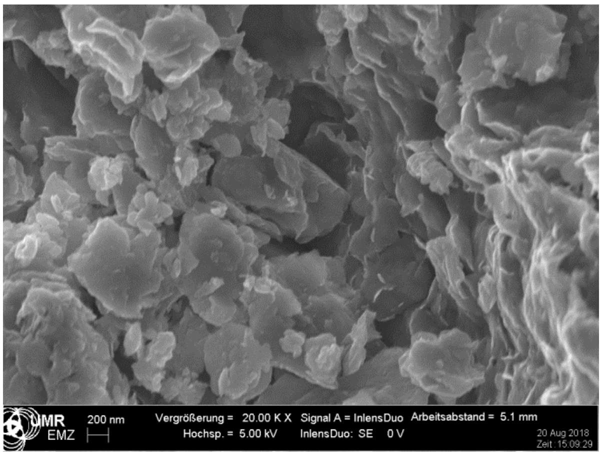
The formation of cementing materials indicated that the reaction had taken place between Si and Al from the soil, and Ca from the additives. These cementitious products formed from this reaction are characterized by their high strength and low volume change [127]. The observed changes in the microstructures of the treated samples could also be a reason for the improvement in the geotechnical characteristics (plasticity, swelling, compressibility and strength) of the BC soil.



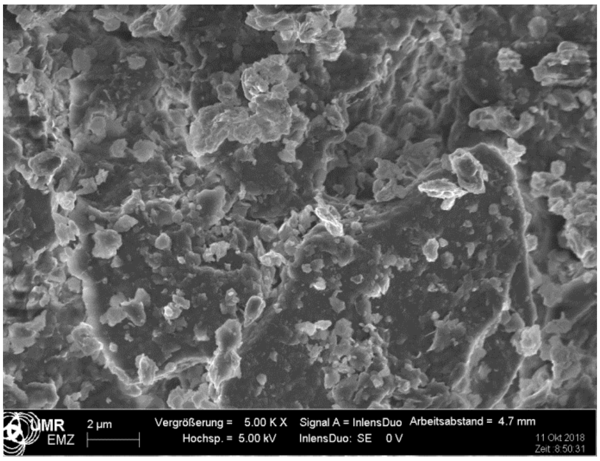



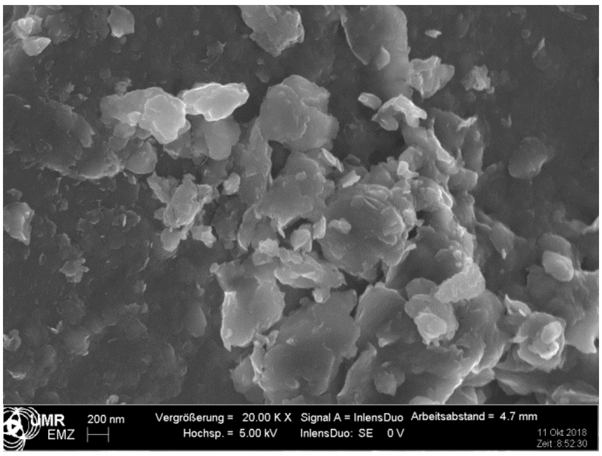

(a) (b)



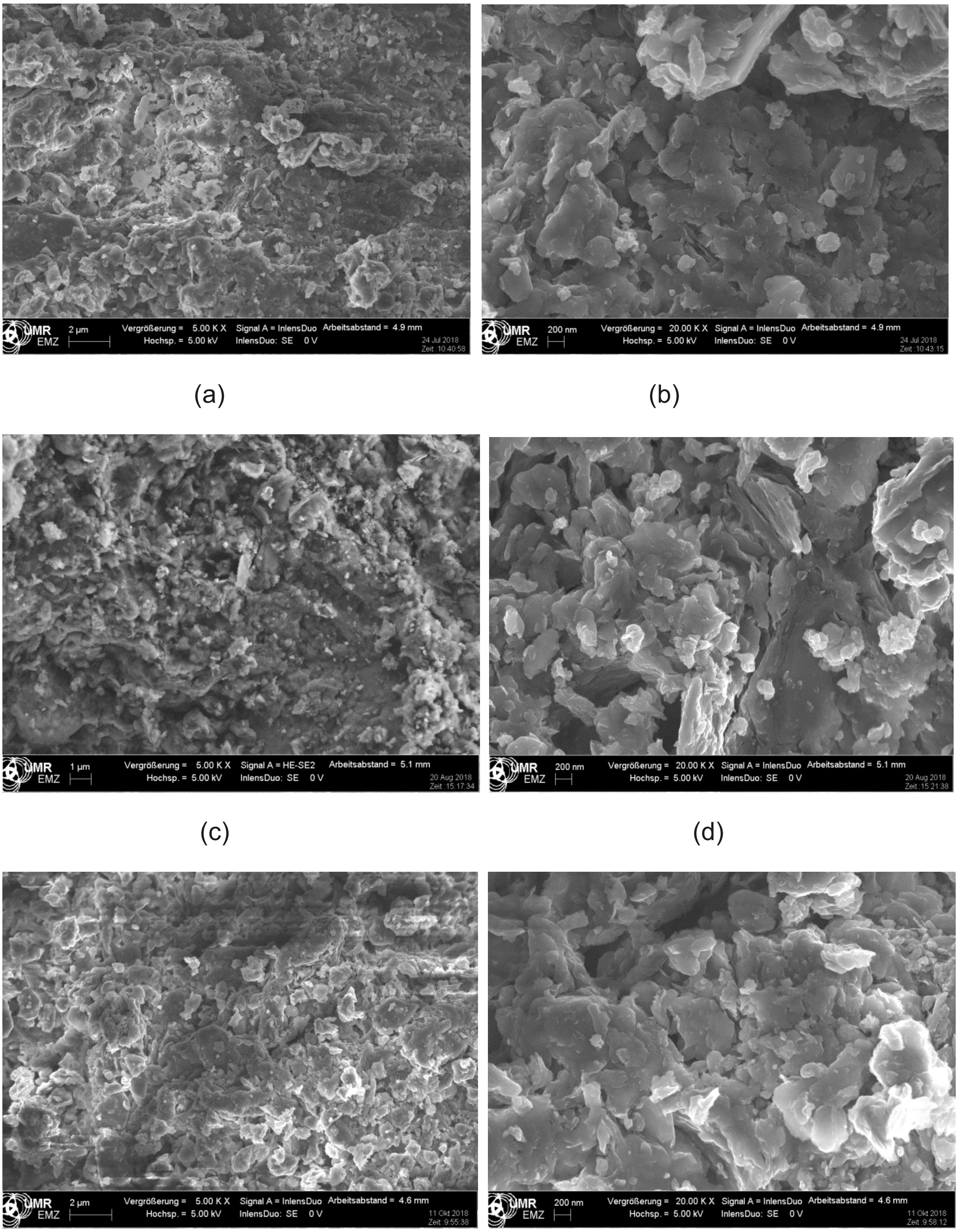

(c) (d)




(e) (f)

Figure 6.20: SEM images of the BC soil treated with 6% lime (a) 5000x, 7d; (b) 20,000x, 7d; (c) 5000x, 14d; (d) 20,000x, 14d; (e) 5000x, 28d and (f) 20,000x, 28d

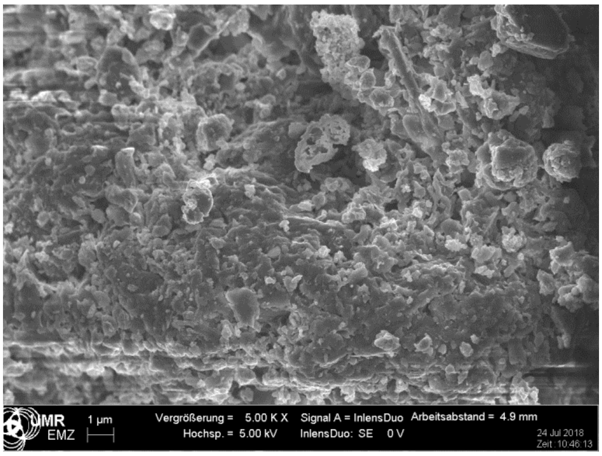


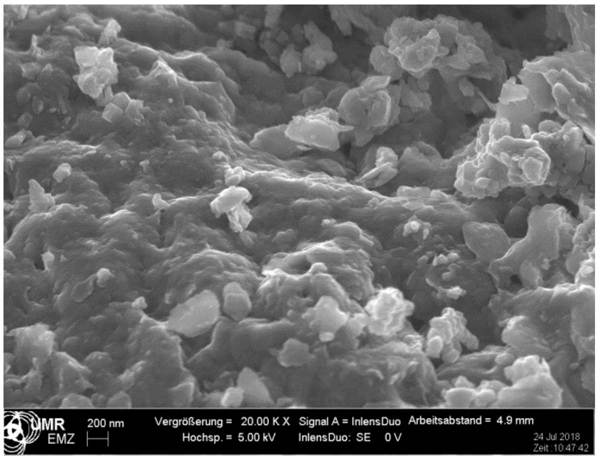
(a) (b)

(c) (d)

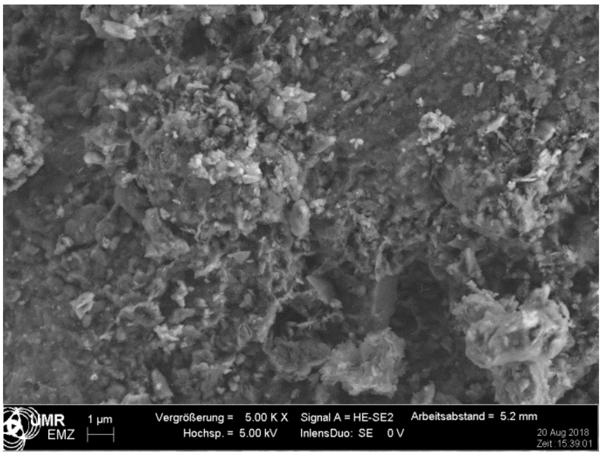
(e) (f)

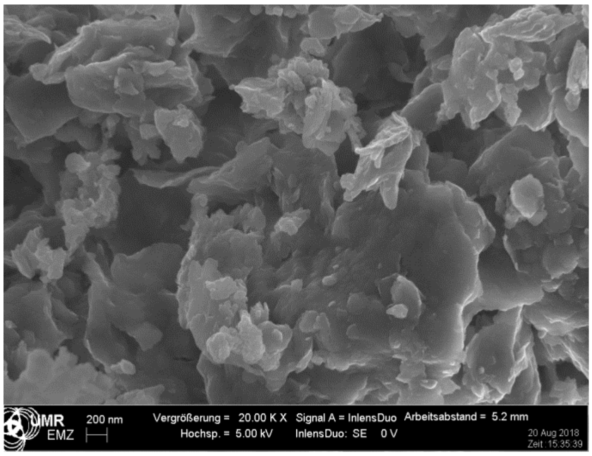
Figure 6.21: SEM images of the BC soil treated with 6% lime and 10% CHA mixture (SL6C10) (a) 5000x, 7d; (b) 20,000x, 7d; (c) 5000x, 14d; (d) 20,000x, 14d; (e) 5000x, 28d and (f) 20,000x, 28d



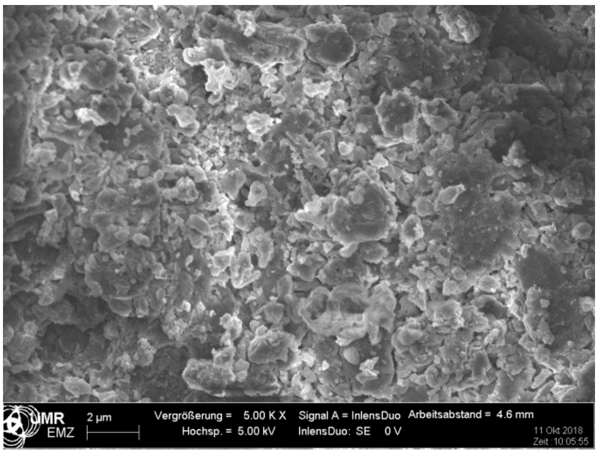



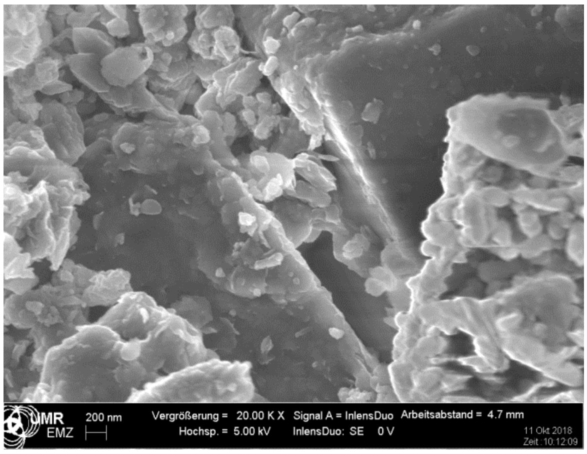

(a) (b)



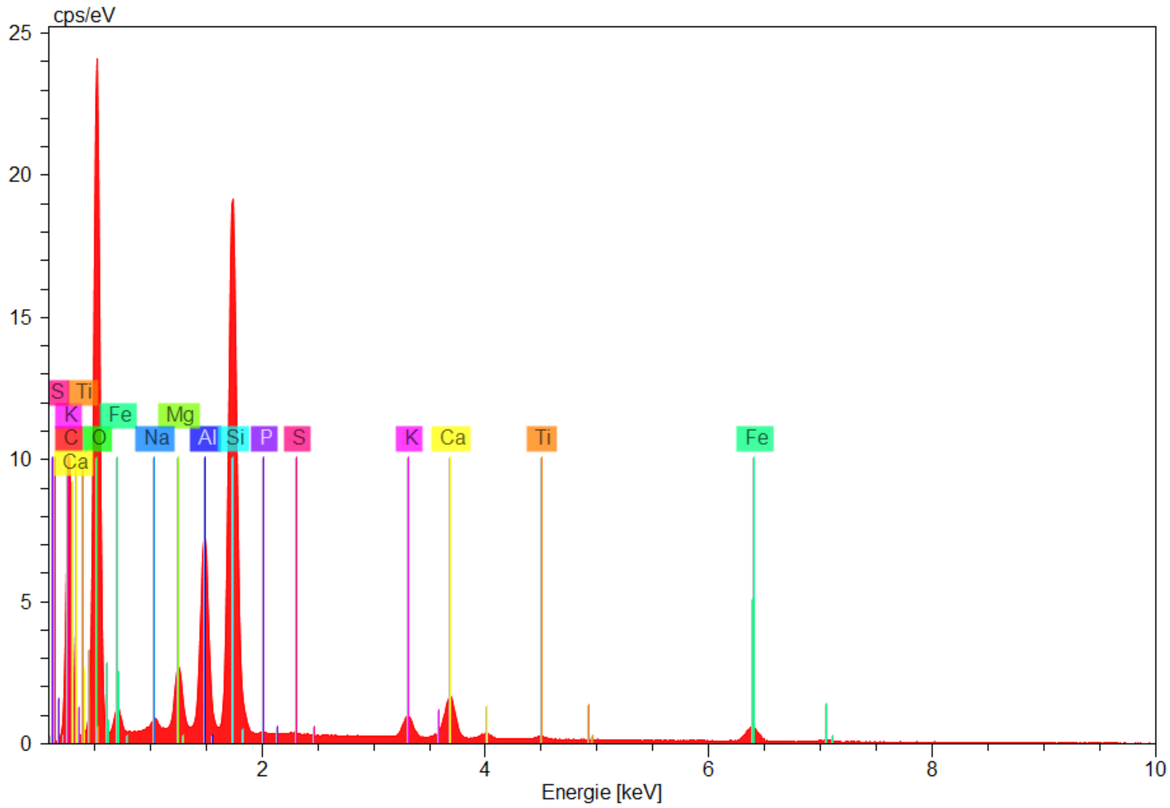

(c) (d)



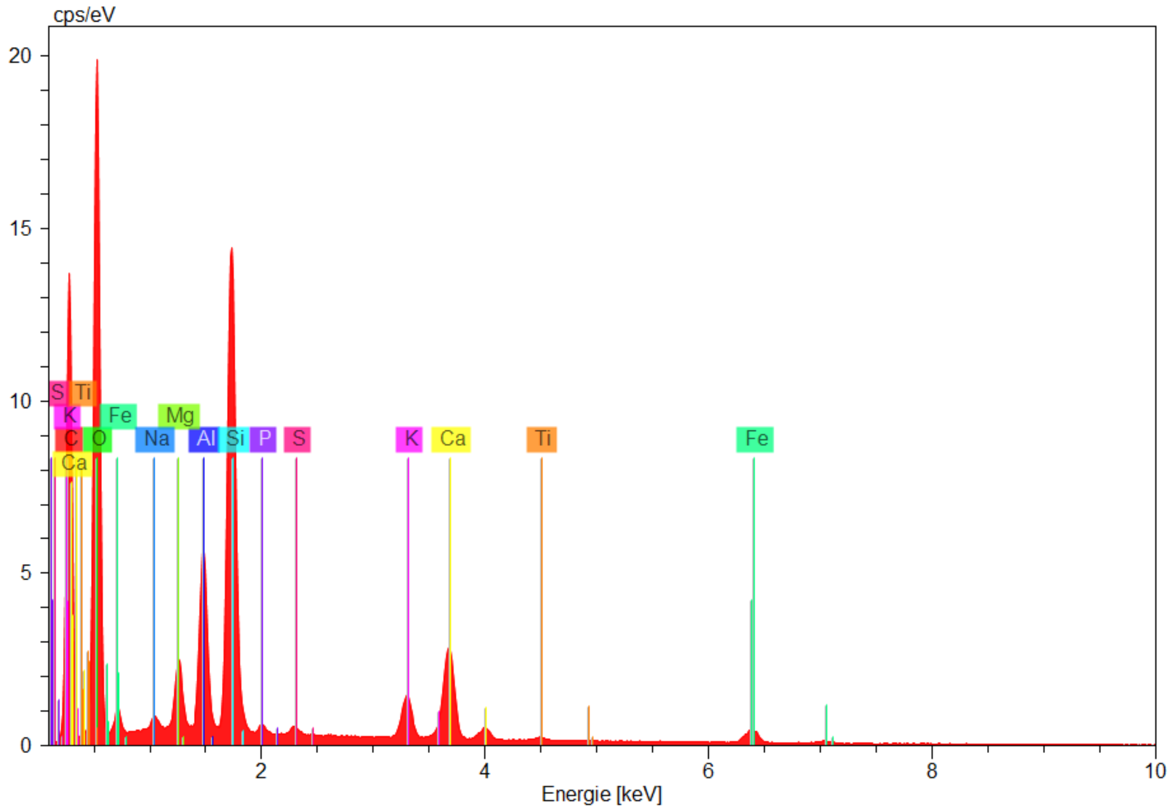




(e) (f)

Figure 6.22: SEM images of the BC soil treated with 6% lime and 15% CHA mixture (SL6C15) (a) 5000x, 7d; (b) 20,000x, 7d; (c) 5000x, 14d; (d) 20,000x, 14d; (e) 5000x, 28d and (f) 20,000x, 28d


cps/eV
25
20
15
10
5
0
S Ti
K Fe Mg
C O Na Al Si P S K Ca Ti Fe
Ca
2
4
6
8
10
Energie [keV]

(a)


cps/eV
20
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C O Na Al Si P S K Ca Ti Fe
Ca
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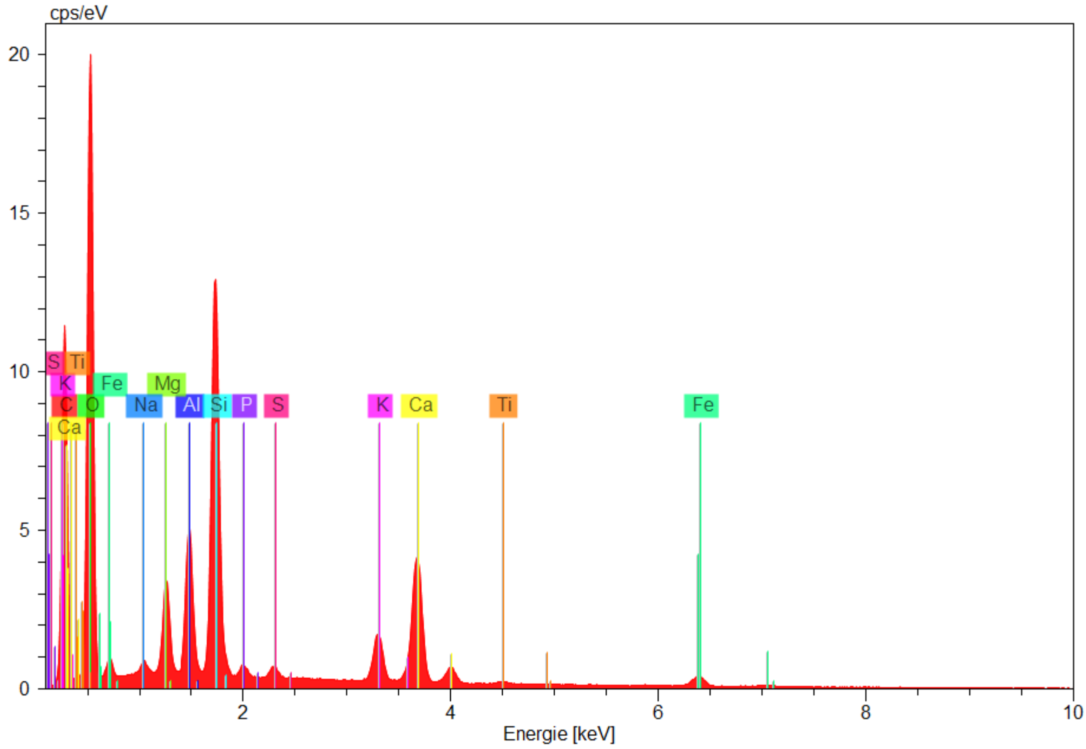
Energie [keV]

(b)




(c)

Figure 6.23: EDX spectrum of (a) 6% lime, (b) 6% lime and 10% CHA (SL6C10) and (c) 6% lime and 15% CHA (SL6C15)

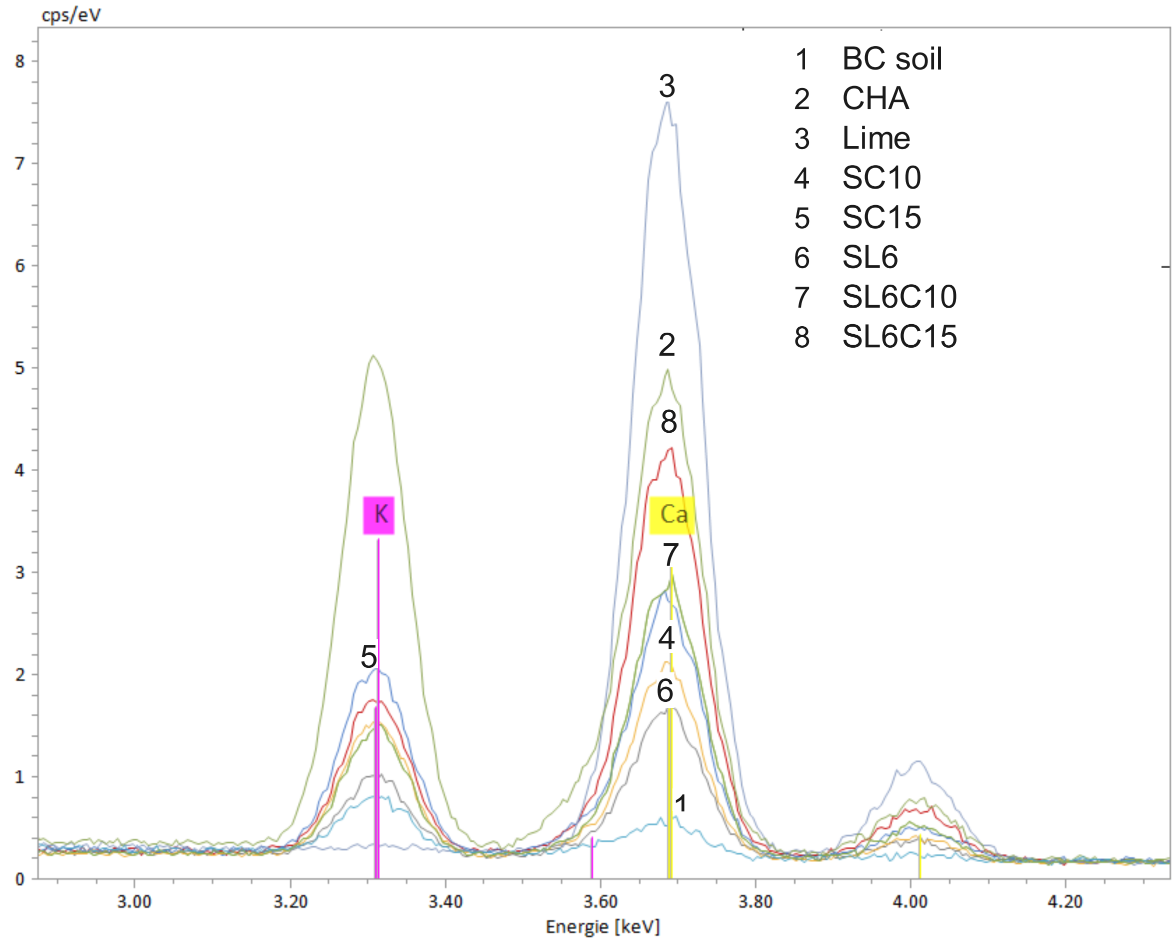



Figure 6.24: EDX spectrum of K and Ca for untreated and treated samples

## 6.7 Effectiveness of Lime-CHA Treatment

The effectiveness of samples treated with lime and lime-CHA mixtures were evaluated using PI, UCS and CBR values. The swelling potential of lime treated samples reduced significantly (see Table 6.1). The swelling potential of the BC soil was very high and when treated with 4% lime, reduced to high swelling. Further, as the lime content increases and with the mixture of lime and CHA the swelling potential reduced to medium to low swelling.

The UCS gain of stabilized soil comparable to untreated soil samples were determined by dividing the UCS value of lime and lime-CHA treated soil to that of the untreated soil samples. The maximum strength gain factors found were 5.4 and 7.0 for lime alone, and for lime-CHA mixtures respectively (see Table 6.2).

According to Bowles [74] the quality of the BC soil to use as sub-grade material was very poor, after treatment with 4% lime the quality become fair. For 6% and 8% lime content and with the mixture of CHA the quality became good (see Table 6.3). The best results were found at additives content of 8% for lime alone and 6% lime and 15% CHA (SL6C15) for the mixture treatment.

Generally the treatment of BC soil with CHA and lime mixture leads to decrease in the swelling capacity and increase in the compressive strength and bearing capacity of the soil. Therefore, the effective utilization of CHA with lime is cost-effective and efficient material for use in road construction.

Table 6.1: Swelling potential depending on PI values (lime and lime-CHA treated).

| Soil mixture | PI | Swell potential |
|---|---|---|
| SL4 | 33.20 | High |
| SL4C5 | 32.07 | High |
| SL4C10 | 27.71 | High |
| SL4C15 | 20.41 | Medium |
| SL4C20 | 19.66 | Medium |
| SL6 | 24.04 | Medium |
| SL6C5 | 22.84 | Medium |
| SL6C10 | 19.81 | Medium |
| SL6C15 | 14.13 | Low |
| SL6C20 | 10.48 | Low |
| SL8 | 19.20 | Medium |
| SL8C5 | 18.35 | Medium |
| SL8C10 | 14.11 | Low |
| SL8C15 | 9.25 | Low |
| SL8C20 | 8.02 | Low |

Table 6.2: Consistency depending on UCS values (lime and lime-CHA treated).

| Soil mixture | UCS (kPa) cured for 7 days | Strength gain | Consistency after UCS value |
|---|---|---|---|
| SL6 | 482,71 | 5,40 | hard |
| SL6C5 | 493,88 | 5,53 | hard |
| SL6C10 | 569,83 | 6,38 | hard |
| SL6C15 | 631,84 | 7,07 | hard |
| SL6C20 | 620,09 | 6,94 | hard |

Table 6.3: Quality of sub-grade depending on CBR values (lime and lime-CHA treated).

| Soil mixture | CBR value (%) (soaked) | CBR gain | Quality after CBR value |
|---|---|---|---|
| SL4 | 15.97 | 15.53 | Fair |
| SL4C5 | 17.52 | 25.18 | Fair |
| SL4C10 | 19.67 | 27.27 | Fair |
| SL4C15 | 21.42 | 17.04 | Good |
| SL4C20 | 22.20 | 19.12 | Good |
| SL6 | 25.90 | 20.83 | Good |
| SL6C5 | 27.26 | 21.59 | Good |
| SL6C10 | 28.82 | 26.51 | Good |
| SL6C15 | 30.96 | 28.03 | Good |
| SL6C20 | 30.77 | 30.11 | Good |
| SL8 | 28.04 | 29.92 | Good |
| SL8C5 | 28.63 | 27.84 | Good |
| SL8C10 | 29.21 | 28.40 | Good |
| SL8C15 | 29.99 | 29.16 | Good |
| SL8C20 | 28.63 | 27.84 | Good |

## 6.8 Conclusions

Based on the test results obtained from the Atterberg limits, compaction, CBR, micro-structural and mineralogical tests, the following conclusion are drawn.

The BC soil used in this study was classified as high plasticity clay according to USCS. As the content of the additives (lime, lime-CHA) increased, both LL and PI decreased, shifting the classification of the BC soil to high plasticity silt and low plasticity silt. These changes in the plasticity behaviour indicate that the treated samples became more workable and stable.

From the compaction test results, it was observed that the MDD of the lime-treated samples slightly decreased and the OMC increased as the lime content increased. In contrast, the addition of CHA on lime treated soil, slightly increases the MDD and decreases the OMC.

The lime and lime-CHA-treated samples resulted in a considerably higher CBR than the untreated BC soil. The tests results are summarised in Table 6.4, from the results the optimum amount of additives to treat the BC soil is found to be the mixture of 6% lime and 15% CHA (SL6C15).

Table 6.4: Summery of test results with different amount of CHA and lime.

| Parameters / Mixture (%) | MDD($g/cm^3$) | OMC (%) | CBR (%) | PI (%) | Swell potential | Quality after CBR value |
|---|---|---|---|---|---|---|
| BC soil | 1.24 | 37.2 | 1.02 | 52.95 | Very high | Very poor |
| SC5 | 1.26 | 35.8 | 1.55 | 51 | Very high | Very poor |
| SC10 | 1.33 | 32.25 | 1.94 | 42.43 | High | Very poor |
| SC15 | 1.36 | 31.81 | 2.33 | 25.55 | Medium | Very poor |
| SC20 | 1.38 | 31.45 | 3.11 | 22.69 | Medium | Poor to fair |
| SL6 | 1.19 | 43.65 | 25.90 | 24.04 | Medium | Good |
| SL6C5 | 1.19 | 42.39 | 27.26 | 22.84 | Medium | Good |
| SL6C10 | 1.21 | 40.88 | 28.82 | 19.81 | Medium | Good |
| SL6C15 | 1.21 | 40.09 | 30.96 | 14.13 | Low | Good |
| SL6C20 | 1.21 | 39.72 | 30.774 | 10.48 | Low | Good |

The micrograph of the BC soil indicated discontinuity in its structure and more visible voids. The SEM images of the treated samples showed that the addition of lime, and lime-CHA has a marked change on the microstructure of the BC soil. EDX results showed a decrement in Si and Al and an increment in Ca content, as the amount of the additives increased. In addition, the XRD results confirmed the formation of cementitious compounds, which is responsible for the improvement in geotechnical properties of the investigated soil.

# 7 Conclusions and Recommendations

## 7.1 Conclusions

In this research the potential use of CHA for the treatment of geotechnical properties (e.g. Atterberg limit, compaction characteristics, consolidation parameters, durability, strength) of expansive soil was studied in detail. In addition, the effect of the lime and CHA mixture in altering geotechnical engineering properties of the studied soil were investigated and reported. Furthermore, change on the microstructure and mineralogy of the studied soil after treatment was investigated using XRD and SEM techniques.

Based on the laboratory investigation results the following conclusions have been made.

The obtained result reveals that the addition of CHA reduces the plasticity of the soil. The LL and PI decreased as the amount of CHA increased (reduction in LL from 93% to 71%, and PI from 52% to 22%, respectively, were observed for addition of 20% CHA). Similar trends occurred in the sample treated with 8% lime where the LL decreased from 93% to 79% with a corresponding reduction in PI from 52% to 19%. For lime-CHA mixture treatment it was noted that the LL decreased from 93% to 49%, with a corresponding decrease in the PI from 52% to 8% for additives content of 20% CHA and 8% lime. Since PI is a good indicator of the swelling behavior of soils, the reduced PI helped to decrease the swell potential of treated soil.

The results of swell-shrink tests indicated that CHA is effective in controlling the swell potential of an expansive soil.

From the compaction test results, the MDD of CHA treated samples increased and OMC decreased as the amount of CHA increased. The MDD of lime-treated samples slightly decreased and the OMC increased as the lime content increased. In contrast, the addition of CHA on lime treated soil, slightly increases the MDD and decreases the OMC.

The void ratio of CHA treated samples decreased as the amount of CHA increased, this change is due to the formed cementitious compounds (as a result of the chemical reactions between the silica and the alumina and the CHA) reduced the volume of the void spaces and joined the soil particles.

The addition of CHA improved the bearing capacity of the BC soil. CBR-values of the BC soil increased with the increase in CHA content from 5% to 20%. Both lime and lime-CHA treated samples also shows increment in CBR value as the amount of additives increases. The highest CBR-values were found for the BC soil treated with the mixture of lime and CHA (SL6C15). The quality of the BC soil to use as sub-grade material was very poor, after treatment with CHA, the quality become poor to fair. For lime and lime-CHA mixture treatment (6% and 8% lime content for lime treatment, and 6% and 8% lime mixed with 5%, 10%, 15% and 20% CHA for mixture treatment) the quality became good.

The unconfined compressive strength of samples treated with CHA increased with the increase in CHA content from 5% to 15%. Above 15% CHA (eg. 20%), decrease in the UCS value was observed.

The addition of lime and lime-CHA together increased the UCS and the strength gain values of the BC soils. The strength gain of samples treated with the mixture of lime and CHA was higher compared to the addition of lime and CHA separately.

The durability of untreated and treated samples was evaluated using W/D tests. In durability test the BC soil failed in the first cycle. The samples treated with 5% CHA resists the first cycle, however, failed in the second cycle. The samples treated with 10% CHA, 15% CHA and 20% CHA resists two W/D cycles. All samples treated with optimum lime content (according to PH test) and samples treated with the mixture of lime and CHA resists three W/D cycles. From these results it can be concluded that CHA stabilized samples are more stable compared to the BC soil samples. The lime and lime-CHA mixture treated samples resists more W/D cycles compared to samples treated with only CHA.

A micro-structural analysis showed that the microstructures of the tested soils was changed due to the addition of CHA, lime and lime-CHA mixture, and developed through the long-term curing. SEM-micrographs of treated samples indicated that the formation of cementitious compounds as a product of the reaction between soil and the additives and through the long-term curing contributing to increasing the strength gain. In addition, the XRD results confirmed the formation of cementitious compounds, which is responsible for the improvement in geotechnical properties of the investigated soil. The formation of these cementitious compounds leads the soil particles to join together and highly contribute to the increase in the strength and decrease in the swelling capacity.

This investigation reveals the potential use of CHA for road sub-grade construction. CHA treated samples perform stronger as sub-grade material compared to the untreated BC soil. Because of the stronger sub-grade, a reduced thickness of pavement structure results from a reduced thickness of pavement layers. This yields several benefits: construction cost savings and construction material saving. It is not limited to the socio-economic advantages in infrastructure developments, but could also play a significant role in reducing the environmental impact arising from the storage of the waste.

## 7.2 Recommendations

Below are some recommendations for the beneficial use of this novel treatment option, including the additional tests required for the effective use of CHA.

Analysis of laboratory data revealed that CHA could be a very good alternative in controlling the swell potential of expansive soil because it has low affinity for water. In addition as noted in the previous chapters, CHA has improved the geotechnical properties of tested soil. However, for this treatment option to be applicable the following points should be considered.

The chemical composition of CHA specifically the amount of Ca present in the CHA may vary from place to place (sources) and manufacturing method. In addition the effectiveness of CHA treatment may depend on the type of soil. Therefore it is recommended that standard laboratory tests should be conducted prior to field applications, to prove how effective the treatment would be on a particular type of soil.

This study was only conducted in the laboratory. The use of CHA for expansive soil treatment seems to be a viable solution in terms of waste management and construction cost, but the significant reduction in swell behaviour achieved in the laboratory may not be achieved in field applications where conditions are much more complex. Therefore, field testing and monitoring will be needed to build confidence in this novel treatment option.

Field measurements should be conducted on the leaching characteristics of the CHA and its potential environmental impact on groundwater.

The CHA treatment has improved the properties of the studied soil significantly, so a study should be carried out to assess the longevity of this improvement. Because the lifespan of a road is long and general approval for new and non-traditional treatment option would require long term field trials to be conducted and to build confidence in this novel treatment option on its limitations and benefits.

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## Theses

1. Expansive soils are soils which have a tendency to expand as moisture content increases and shrink as they dry out.
2. Expansive soils are one of the widespread typically problematic soils in the world. The cost of the damage to civil engineering structures caused by expansive soils around the world has been estimated in billions of dollars.
3. Major problem of expansive soils is large ground deformations in and around the structures due to swelling and shrinking of these soils on wetting and drying. These excessive movements can lead to structural damages.
4. Different treatment methods have been adapted to improve the geotechnical properties of expansive soils and to avoid excessive swelling. Stabilization is one of the possible methods to improve its poor properties by adding a stabilizing agent.
5. Ethiopia is listed among top coffee producing countries in the world. Coffee husk is a waste material obtained from coffee processing and disposal of this solid residue can lead to environmental problems.
6. This research aimed to study the potential use of coffee husk ash to improve the geotechnical properties of expansive soil.
7. The site of the study is West Shoa zone of Oromia state around Ambo town, Ethiopia. This area is mainly covered by highly expansive black cotton soil.
8. The study includes the investigation of coffee husk ash as a potential agent alone and with lime combination.
9. Experimental investigations were conducted in conformity with approved standards on untreated and treated soil to evaluate their characteristics.
10. The experimental results reveal that coffee husk ash improves the plasticity, compressibility and strength of the black cotton soil.
11. The addition of coffee husk ash shows improvement on the bearing capacity of the black cotton soil.
12. The X-ray diffraction result shows the formation of cementitious compounds, which is responsible for the improvement in geotechnical properties of the black cotton soil.
13. Coffee husk ash treated samples are more resistance to wetting and drying conditions compared to the untreated BC soil samples.
14. The mixture of lime and coffee husk ash treatment shows more enhancements on the properties of the black cotton soil compared to coffee husk ash alone.
15. The potential use of coffee husk ash as a stabilization agent will not be limited to the socio-economic advantages in infrastructure developments, but could also play a significant role in reducing the environmental impact arising from the storage of the waste.
16. This study was entirely conducted in the laboratory. In order to build confidence in this novel treatment option a field testing and monitoring will be needed.

## Publications

- Atahu, Meskerem Kebede: Saathoff, Fokke: Gebissa, Alemayehu: Effect of Coffee Husk Ash on Geotechnical Properties of Expansive Soil. International Journal of Current Research, Vol. 9(2017), Issue, 02, pp.46401-46406.
- Atahu, Meskerem Kebede: Saathoff, Fokke: Gebissa, Alemayehu: Strength and Compressibility Behaviors of Expansive Soil Treated with Coffee Husk Ash. Journal of Rock Mechanics and Geotechnical Engineering, Vol. 11(2019), pp.337-348.
- Atahu, Meskerem Kebede: Saathoff, Fokke: Gebissa, Alemayehu: Engineering Properties of Black Cotton Soil Stabilized with Coffee Husk Ash and Lime. Global Journal of Engineering Science and Research Management, Vol. 7(2020), pp.20-38.

**Seminars**

- Presenter and attendant- Impacts of Climate Change on Soil Quality in Ethiopia. ``Climate Change and Renewable Energies in Africa´´, 03-06 December 2017, Hechingen, Germany.
- Attendant - ``Education for Life, Education for Development-Changing Approaches in a Changing World´´ 1–4 December 2016, Dire Dawa, Ethiopia.
- Attendant - “Insurance" systems: social and cultural background. 27-30 March 2017 Bonn Germany.
- Attendant - "Industry 4.0 - Who Wins? Who Loses?“ 14-17 June 2018, Münster, Germany.