SOIL STABILIZATION IN SKARDU REGION USING DOMESTIC WOOD ASH

By

Muhammad Mazahir Haider

(NUST-2016-MS GEOTECH-00000171210)

A thesis submitted in partial fulfillment of the requirements for the degree of

Master of Science
In
Geotechnical Engineering
NUST Institute of Civil Engineering (NICE)
School of Civil and Environmental Engineering (SCEE)
National University of Sciences and Technology (NUST)
H-12 Sector, Islamabad, Pakistan

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THESIS ACCEPTANCE CERTIFICATE

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SOIL STABILIZATION IN SKARDU REGION USING DOMESTIC WOOD ASH

Submitted by

Muhammad Mazahir Haider

has been accepted towards the partial fulfillment
of
the requirements
for
Master of Science in Geotechnical Engineering

Dr. Liaqat Ali
Associate Dean
NUST Institute of Civil Engineering (NICE)

DEDICATED

TO

MY BELOVED FAMILY

WHO GAVE ME A LOT OF SUPPORT
AND
ENCOURAGEMENT

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List of Abbreviations

AASHTO:	American Association of State Highway and Transportation Officials
ASTM:	American Society for Testing and Materials
CBR:	California Bearing Ratio
MDD:	Maximum Dry Density
NLA:	National Lime Association
OMC:	Optimum Moisture Content
UCS:	Unconfined Compressive Strength
XRD:	X-Ray Diffraction

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Abstract

In Gilgit Baltistan region there is an extensive use of wood as a source of energy for cooking and heating purpose which results in a large amount of combustion residues known as wood ash. After combustion these ashes are discarded and dumped in dig holes and tranches, resulting in wastage of stabilizing agent and useful land. Wood ash contain CaO due to which it behave like a pozzolanic material therefore wood ash can be used as a stabilizer of soil. It is found that wood ash enhances the engineering properties of natural soil including unconfined compressive strength and California Bearing Ratio.

The present research is intended to examine the effect of wood ash on the engineering properties of soil found in Skardu region and evaluate their potential use for the stabilization and improvement of engineering properties of these soils. Two types of soil were selected from Skardu region, soil type-I (CL-ML) and soil type-II (CL). Both types of soil were mixed with 0, 5, 10, 15 and 20 percent wood ash. Both types of soil are reactive with wood ash. In unconfined compressive strength test, optimum percentage of wood ash required for stabilizing the soil is in range of 5 to 10 percent for both soil type-I and soil type-II. Similarly, CBR increases by mixing of 5 to 10 percent wood ash mixed with both types of soil and swell potential decreases.

Chapter 1

INTRODUCTION

1.1 GENERAL

Soil stabilization is the procedure in which a problematic natural soil is mixed with a binding admixture, or any other chemical material to improve its engineering properties. Gilgit-Baltistan region covers 72,496 square kilometer territory consisting ten districts, i.e., Skardu, Gilgit, Ghizer, Diamer, Hunza, Nagar, Astore, Shigar, Kharmang and Ghanche. The climate of the Gilgit-Baltistan is very cold in winter and pleasant in summer. Temperature varies between a maximum of 27ºC and a minimum of 8º C during summer and drop to below -10º C in winter. In winter season most of the precipitation received in this area is in the form of snow. The average annual rainfall is about 208 mm.

Skardu region is situated on the bank of Indus River and many other seasonal rivers and streams pass through this area due to which the soil of this region is uneven, it consists of clay, silty clay, silt, sand and gravel. Those areas nearby river bank are mostly clays and those near the mountains are mostly sand and gravel. Due to presence of clay, high water table (due to river) and the extreme climate conditions, soil of this area is found to be problematic, results in development of crack in buildings, damage of roads and other infrastructure.

With the rapid growing population, energy crises is also increasing day by day in the region, due to lack of gas and shortage of electricity the only main source of energy for heating and cooking is wood, because wood is cheap and easily available. From the month of October to March the residents of this region use “Bukhari” (a wooden combustion chimney) for heating purpose in their homes, shops and offices. As per the estimate of local market approximately a single house hold uses 1600 to 3200 kg wood in winter for heating purpose only.

Along with the advantage of wood as a source of heating energy, there is one important drawback which is the remaining by-product as wood ash. During wood combustion process the majority of organic material is oxidized and release into air leaving the wood ash which includes all the essential elements like Ca (Calcium), K (Potassium) and Mg (Magnesium). As wood ash is highly alkaline therefore, it has highly alkali carbonated which act as pozzolanic agent. Wood ash is disposed off without its proper utilization in Gilgit Baltistan. Wood ash can be used stabilization agents which is low-cost and it has also added advantage of solving the environmental problem.

1.2 NEED FOR RESEARCH

In construction industry soil stabilization offers an economical and technically feasible solution to many problems regarding engineering throughout world, detailed field and laboratory investigations are carried out to recommend a specific stabilization technique. Therefore, this study is conducted to find out feasible economical solution for soil issues in Skardu region and to improve their engineering properties using locally available free of cost material.

1.3 OBJECTIVES OF RESEARCH

The core objective of this research work is to study soil stabilization in Skardu region, to improve engineering properties using locally produced domestic wood ash by laboratory investigation and proper utilization of wood ash in infrastructure development. This particular study will focus on:

- Increasing unconfined compressive strength.
- Reducing the swell potential of the soil.
- Settlement control.
- Evaluating the use of wood ash as a stabilizer.
- Improvement of geotechnical properties.

1.4 SCOPE AND METHODOLOGY

The sample collection from Skardu region, soil classification, characterization of Atterberg's limit and identification of minerals present in soil will be included in this study. Further investigation carried out for change in engineering properties of soil after using wood ash as a soil stabilizing agent. This study is carried out in five phases as follow:

- Phase 1: Sample Collection
- Phase 2: Characterization of Untreated Soil
- Phase 3: Optimization of Admixture Content
- Phase 4: Strength and Durability of Soil Admixture
- Phase 5: California Bearing Ratio Test

1.5 THESIS OUTLINE

This thesis comprises five chapters. Chapter no. 1 provides the brief outline of thesis which includes introduction, brief geography of the area, background of the problem, introduction to the stabilizing material, need for stabilization of soil and objectives of this research .Chapter no.2 presents a concise literature review regarding soil stabilization types and methods, mineralogy of clayey soil, introduction of admixtures i.e., wood ash and chemical aspects of the wood ash soil mixture. Chapter no. 3 presents the working methodology adopted for this research work including the lab testing techniques, testing equipment and procedure. Chapter no. 4 presents the detail lab test results and discussion on the results. Chapter no.5 presents the conclusion from laboratory testing results and recommendations for future research work.

Chapter 2

REVIEW OF LITERATURE

2.1 GENERAL

Due to the construction of infrastructure projects in a large scale the importance of ground improvement techniques is increasing day by days. Now Scientists and Researchers are working on use of cost effective and easily available local materials from industrial, agricultural and domestic wastes to enhance the engineering properties of weak soils and also to minimize the construction cost.

Waste materials offer an economic method for soil stabilization using fly ash, bottom ash (Koteshwara *et al.*, 2012). Along with the soil stabilization another important benefit is to save depleting landfills as these waste materials are mostly used for filling of landfills.

2.2 SOIL STABILIZATION

2.2.1 General

The geotechnical properties of unsuitable soils can be improved by using different methods and techniques. These techniques include pore water pressure reduction, densifying by compaction or preloading, use of reinforcing elements and bonding of soil particles. For many works including geotechnical applications such as roadways, pavement structures, channel and reservoir linings, building foundations, sewer lines and water lines, chemical stabilization of problematic soil is important to avoid the damages due to the swell action of expansive soil or due to the settlement of the soft soil. Generally, the stabilization concept is as old as 5000 years. History reveals that in Egypt earthen roads were stabilized and soil-lime mixtures were used for stabilization in ancient by Mesopotamia, Romans and the Greeks. The initial experiments on subgrade stabilization were done in USA with clay sand mixtures around 1906. Soil stabilization pertinent to road construction was used in Europe in the 20th century (Pokalwar *et al.,* 2014).

2.2.2 Stabilization Components

Stabilizing agents (binder materials) are used for soil stabilization of weak soils to improve their geotechnical engineering properties. Improved results can be such as improvement in strength, durability, compressibility and permeability. Stabilization components include soil or soil minerals and stabilizing agents or binders (cementitious materials).

2.2.3 Mechanism of Stabilization

The mechanism of stabilization may include coating the particles surfaces by stabilizing agents to control the moisture sensitivity and new compounds are formed of finer particles as bonded. That's why a proper understanding of each additive for stabilization mechanisms is required before selection of an additive. In chemical stabilization the soil is mixed with chemically active compounds such as lime, Portland cement, fly ash, sodium or calcium chloride or with bitumen (visco-elastic materials). There are broadly three groups of chemical stabilizers: Traditional stabilizers, non-traditional stabilizers and by-product stabilizers.

2.2.4 Traditional Stabilization

The important traditional stabilizers are Portland cement, hydrated lime and fly ash. The most commonly used chemical additives are Portland cement, lime, and fly ash (Petry and Little, 2002). Traditional stabilizers broadly rely on pozzolanic reactions and exchange of cat-ions to stabilize soil. Commonly lime is used for that objective. Decomposing limestone at elevated temperature produces lime. Lime is the oldest and most common stabilizing agent being used.

Soil-Lime reactions are two step complex process; the primary reaction involves flocculation/agglomeration and exchange of cat-ion and cause the textural and plasticity changes (Little, 1995). Larger particle agglomerates are formed due to short term pozzolanic reaction and cat-ion exchange which makes the soil more friable and more workable. In the second step a cementing process among agglomerates and flocculates occur that leads to pozzolanic reaction. Considerable strength increase depends on quantity of pozzolanic matter and on reactivity of soil minerals with the additives.

2.2.5 Non-Traditional Stabilization

Stabilization mechanism for non-traditional stabilizers may include enzymes, ionic stabilizers sulfonated oils etc., (Little and Petry, 2002). Stabilizers in presence of higher sulfate contents in the soil play a great role in stabilization and limits the applicability of traditional stabilizers.

2.2.6 By - Product Stabilization

For most of the by-product stabilizers, cat-ion exchange and pozzolanic reactions are the primary processes, like traditional stabilizers. The by-product from Portland cement, Lime and Wood are Wood ash, Lime Kiln Dust and Cement Kiln Dust respectively. Lime Kiln Dust generally is composed of 30 to 40 percent lime. The lime is either combined with pozzolans or free lime in the kiln. The fuel used to provide energy is the source of these pozzolans. Cement Kiln Dust is obtained as by product from the production of Portland cement. The level of pozzolanic reactivity is supported by the reactive pozzolans generated by Portland cement production.

Cement Kiln Dust generally contains 20 to 25 percent pozzolanic material and about 30 and 40 percent calcium oxide CaO same like Wood ash generally contains 15 to 33 percent of pozzolanic material and about 33 percent calcium oxide.

2.3 CLAYEY SOILS

For a civil engineer clayey soil is a soil or a fraction of soil which contain some clay minerals as well as other minerals, has plasticity and is cohesive (Holtz, D.R. and Kovacs, D.W 1981). The term “clay” is used to refer both to size as well as mineralogy of the soil (Mitchell, 1993). In term of size, it indicates soil particles smaller than 0.002 mm. As a term mineral, it refers to the specific clay mineral distinguished by a low hydraulic conductivity, plasticity when mixed with water, net negative electrical charge, more swelling potential and highly weathering resistance.

2.3.1 Clay Structure

All the clay minerals are arranged in several layers, crystalline hydrous aluminosilicates having a lattice structure in which the atoms are arranged similar to the pages of a book. The structure of clay minerals is made up of combination of two simple structural units or blocks, the silicon tetrahedron or silica sheet and the aluminum octahedron or the alumina sheet (Terzaghi, 1996; Mitchell, 1993). These blocks form tetrahedral and octahedral layers specific combinations of which produce a unit sheet of the various varieties of clays. The association and the chemical composition of these layers decide the kind of clay mineral. The different clay mineral groups are characterized by the stacking arrangement of these sheets and the manner in which successive layers are held together.

2.3.2 Clay Mineralogy

Three principal groups of clay minerals which are distinguished by their structure which controls their behavior. These groups are the Montmorillonite, the Kaolinite and the Illite. All three groups are composed principally of tiny crystals with platy shapes. These crystals are made out of layers of atoms stacked like tablet paper. These tabular crystals, of which all three above mentioned groups are composed and are combinations of either two or three sheets of atoms of two main kinds; one of silicon and oxygen atoms (silica layer) and the other a combination of aluminum with oxygen or hydroxyl atoms (the alumina or aluminum hydroxide layer) (Holeman, 1965).

2.3.2.1 Kaolinite group

The name of this mineral group was derived from the Chinese word “Kao Ling” meaning “High Hill”. This group of minerals is generally having low iron content and almost white in color (Holeman, 1965). The minerals of this group are made up of alternating alumina sheet and silica sheet. The tops of the silica tetrahedron and one of the planes of atoms in the octahedral sheet are common. The tops of the tetrahedron point in the same direction towards the center of the layer. The structural formula of this group is $Si_4Al_4O_{10}(OH)_8$. Both Van der Waals Forces and hydrogen bonds provide bonding between successive layers. Kaolinite particles are charged negatively in a high pH environment (basic), while positively on their edges in a low pH environment (acidic). When alumina is in abundance and silica is rare, the formation of kaolinite is favored (Mitchell, 1993). The minerals of this group are nacrite, kaolinite, dickite and halloysite.

2.3.2.2 Illite group

Prof. R.E Grim of the University of Illinois has discovered the illite group. It is 2:1 mineral which refers that alumina sheet is sandwiched between two silica sheets.

However, the interlayers are combined together with a potassium atom. Illite group has a crystal structure that resembles to the mica minerals but as compared to mica minerals it has less potassium and less isomorphous substitution. Hence, these minerals are chemically more active than other micas (Holtz *et al.,* 1981). Illite is the most common mineral found in engineering practice. Sometimes iron and magnesium can also be found in the octahedral sheet of illite (Mitchell, 1993). Illites have a non-expanding lattice structure (Holeman, 1965). The structural formula of illite is "$K_{0.6}(H_3O)_{0.4}Al_{1.3}Mg_{0.3}Fe^{2+}{}_{0.1}Si_{3.5}O_{10}(OH)_2(H_2O)$".

2.3.2.3 Montmorillonite group

The montmorillonite is composed of expending lattice structure. This group is consisting of one alumina octahedral sheet sandwiched among two silica tetrahedral sheets, owing to which this group is called as 2:1 mineral (Mitchell, 1965). The van der Waals forces provide bonding between the layers. Water and exchangeable ions can enter and separate the layers. Montmorillonite crystals may be very tiny but at the same time they have very strong affinity for water. Due to which montmorillonite makes the soil very vulnerable of swelling. Montmorillonite is also known as smectite (Holtz *et al.,* 1981). Therefore, these sheets can separate and allow water and chemicals in solution to enter between the sheets.

2.3.3 Cation Exchange

Clay consist of generally very small particles. According to the definition, all the soil elements lesser than a particular size of 0.002 millimeter are regarded as clay particles. Mostly the clay particles have platy shape and in some cases they have needle or tubular like shape. When these clay particles are in contact with water and other dissolved ions, then several force fields are produced on the interface of clay and water. Later on, the properties of soil like strength and volume change etc., are influenced by these forces (Mitchell, 1993). The edges of the clay minerals have negative electrical charge because they are chemically incomplete. The discontinuities and irregularities of the crystal structure are also responsible for the negative charge on the clay minerals. As water is consist of polar molecules, the absorption of water by the clay minerals is enhanced by the negative charge. In addition to this, negatively charged ions of clay minerals have tendency to attract other positive ions present in the water like ions of magnesium, calcium, potassium, hydrogen and sodium. All these ions can be exchanged and have a substantial influence on the properties of soil (Holeman, 1965).

2.3.3.1 Sources of cation exchange

Sources of cation exchange are as follows:

2.3.3.2 Isomorphous substitution

Substitution of one type of ion with another type of ion with the same or different valence but with the preservation of same crystal structure is called as Isomorphous substitution. Isomorphous replacement has a vital role to determine the configuration and characteristics of clay (Mitchell, 1993). The replacement of one type of ions almost of same physical size with other type of ions is termed as isomorphous substitution (Holtz *et al.,* 1981). Common examples of isomorphous substitution are aluminum replaced by silicon, magnesium in place of aluminum and ferrous iron (fe^{2+}) replaced by magnesium (Mitchell, 1993).

2.3.3.3 Broken bonds

Broken bonds produce negative charges along the edges of both silica and alumina layer which are adjusted by the cations. The broken bonds appear on the vertical planes of the two layered, three layered and mixed layered clay minerals and on the horizontal plane of the chain structure type minerals (Scholen, 2007). In the two-layer minerals, broken bonds are the main reason of cation exchange. As the particle size of clay mineral decreases the number of broken bonds and the cation exchange caused by them would increase.

2.3.3.4 Replacements

As the clay minerals have hydroxyl groups around the edges and there is a tendency that the hydrogen ions of these hydroxyl groups can be substituted with the exchangeable cations. This effect of replacement on the exchange capacity would be significant for halloysite and kaolinite as these groups have sheet of exposed hydroxyl on the basal side of alumina layer (Holeman, 1965).

2.3.3.5 Cation exchange capacity (CEC)

The amount of the exchangeable cations present in the clay minerals is termed as the cation exchange capacity and represented by CEC. CEC is generally expressed in milli equivalents per hundred grams of dry clay (mEq/100 gm) (Mitchell, 1993). Each group of clay minerals has different values of CEC subjected to the conditions and environment of that time (Holeman, 1965). The CEC of different clay mineral groups at neutrality i.e., at pH value of 7 are presented in Table 2.1.

Table 2.1: Typical values of CEC for various clay minerals (Mitchell, 1993)

Colloid Type	**CEC (meq/100gm)**
Kaolinite	2-15
Montmorillonite	80-150
Chlorite	10-40
Hydrous Mica (Illite)	10- 40

2.3.4 Identification of Clay Minerals

Clay minerals, being very small are difficult to be identified by conventional techniques and thus special methods are employed for their identification. Some of the methods used for identification of clay minerals are summarized below:

2.3.5 X-ray Diffraction

This method is based on the principle that materials with regular or repeating crystalline structure will diffract X-rays in similar fashion and each different mineral will have a specific diffraction pattern of its own (Holtz *et al.,* 1981). The minerals of a soil are identified by comparing the patterns obtained from the diffracted X-rays with the standard published for the common minerals. The method however gives only a qualitative analysis of the present minerals and not the detailed quantities of minerals.

2.3.5.1 Differential thermal analysis

In this method, the soil specimen is heated to several hundred degrees in an electric furnace. The changes occurring at different stages are recorded. These changes associated with a specific clay mineral occur at specific temperatures. Minerals are then identified by comparing the record with standards associated with known clay minerals (Holtz *et al.,* 1981).

2.3.5.2 Electron microscopy

This technique is also used to identify clay minerals; however, the procedure is not very simple. Moreover, it does not give the quantitative analysis.

2.4 WOOD ASH

Wide use of wood as a fuel and energy sources has led to a considerable increase in the amount of combustion residue. There is an increasing amount of wood ash generation in the world because wood is a renewable resource for energy and an environmentally friendly material therefore, there is an increased interest in using wood for energy production. Currently, most of the wood ash generated in the USA is either landfilled (about 70 percent) or applied on land as a soil supplement (about 25 percent) (Tarun *et al.,* 2001).

Wood ash consists of two different types of materials, i.e., fly ash and bottom ash. The chemical and physical properties depend upon several factors including source and type of wood design and operating parameters of the combustor equipment (especially combustion temperature) and the method of collection of ash. Currently, *ASTM* specifications do not exist for wood ash for determining their suitability for use as construction materials. Therefore, *ASTM C 618*, developed for volcanic ash and coal fly ash used.

Wood ash is a gray material produced by total combustion of wood, which may be considered as a category of fly ash. It lacks plastic properties with particles that are mostly smaller than 0.075 millimeters in size (sieve No. 200).

2.4.1 Wood Ash as a Pozzolan

Wood ash is used as a potential soil stabilizer through the chemical reaction. Chemical composition of wood ash implies that it can be used as a substitute of CaO (that contains about 30 percent CaO) for soil stabilization. The calcium-oxide content in the presence of water turns into calcium-hydroxide. In the air calcium-hydroxide reacts with the carbon-dioxide, which results in the formation of calcium-carbonate. As a result, wood ash enhances the lime content of the soil. Lime is also known to be used to improve the geotechnical properties of clay soils.

2.4.2 Lime

Lime has been used for centuries to improve the behavior of clayey soils. Lime can be used as hydrated lime $Ca(OH)_2$, quicklime (CaO) or lime slurry to treat soils. Quicklime is obtained by transforming calcium carbonate ($CaCO_3$) into CaO chemically.

Lime improve strength and reduce compressibility of clayey soil. Lime first react with soil particles and decrease the elasticity and increase the workability. This will lead to swell reduction and strength increment. Some typical characteristics of quicklime and hydrated lime is shown in Tables 2.2 & 2.3 simultaneously.

Lime has calcium that reacts with silica and alumina present in the soil. This generates a cementation reaction making calcium aluminate silicates. This cementitious matter has a great strength and less volume change capacity. The studies showed that small lime additions (2 - 8) percent considerably decrease the maximum dry density (MDD), plasticity index (PI) and swell potential. This also enhance the strength of clayey soils (Basma *et al.*, 1998).

Table 2.2: Characteristics of quicklime (National Lime Association)

Primary Components	CaO
Specific Gravity	3.20-3.40
Bulk Density (lbs./cu.ft.)	55-60
Angle of Repose	55°

Table 2.3: Characteristics of hydrated lime (National Lime Association)

Primary Components	$Ca(OH)_2$
Specific gravity	2.30-2.40

Barasa studied lime, fly ash and combination of lime and fly ash to treat expansive soil. First, particles size distribution was determined from grading test, secondly different dosages (4, 5, & 6) percent of lime were introduced to treat clayey soil and then plasticity index and CBR (California bearing ratio) value was evaluated. There was a reduction in plasticity index (PI) of the treated clayey soil with increasing the quantity of lime and fly ash in all the samples. It is well-known that the quick lime seems to be more effective than hydrated lime (Bell, 1993). Quick lime is more concentrated and economical than hydrated lime. CaO is utilized broadly for soil treatment. Selection of the type of lime depends upon the availability of particular type in that region. In Europe, quick lime is more common (Rolling, 1996). Stabilization take place when the suitable amount of lime is introduced to the soil (Petry *et al.*, 2002).

2.4.2.1 Curing conditions

Temperature, moisture and time are important factors in strength gain of lime stabilized soils and must be strictly controlled. Test specimens can be cured at ambient temperature for 28 days. Various agencies have also recommended accelerated curing at different temperatures for different curing periods. NLA recommends curing of samples at 40^{o} C. According to *ASTM D 5102,* accelerated curing for desired period is to be done at 40.6^{o} C and curing temperature must not exceed 48.9^{o}C. Similarly study on lime fly ash mixtures reported that 28 days curing at ambient temperature (22^{o} C) or 7 days curing at 40^{o} C or 3 days curing at 49 - 50^{o} C yields approximately the same strength (Little 1999). Specimens must be sealed to prevent loss of moisture and carbonation of lime. Plastic bags or steel metal cans, etc., can be used for this purpose (Little *et al.*, 1987).

2.4.3 Pozzolan

ASTM C 618 defines Pozzolan as "A siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value, but will, when in finely divided form and in the presence of moisture, chemically reacts with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties." *ASTM C 618* classifies pozzolans into Class N, Class F and Class C. Class F and Class C fly ashes are produced from bituminous and sub-bituminous coals respectively. Whereas, a Class N pozzolan is a raw or natural pozzolan containing at least 70 percent of mixture of Silicon dioxide (SiO_2) plus Aluminum oxide (Al_2O_3) plus iron oxide (Fe_2O_3), a maximum of 3 percent moisture and/or 4 percent of Sulfur-trioxide(SO_3) and a maximum loss of 10 percent on ignition.

2.4.4 Pozzolanic Reactions

Reactions occurring in soil and fly ash (lime) mixtures are not fully understood. The complex nature of these reactions is compounded by the heterogeneous nature of both the ash and the soil. The most important reaction with respect to paving application is one responsible for producing cementitious gel. Temperature and moisture conditions are also extremely important for pozzolanic reactions to take place (Little, 1995). The reactive properties of wood ash with soil are concluded that wood ash presents a high pozzolanic reactivity and could provide an increase in strength when used as a binder (Okagbue; 2007). As the chemical composition of ash vary with its source and method of production, it is imperative that ash be tested for its composition prior to use.

2.4.5 Fineness of Fly Ash

Reactivity of fly ash depends largely on its fineness. A sizeable fraction of ash must be passing Sieve No. 200 for fly ash to be reactive. However, it is practically difficult to control the fineness in the field. Experience has shown that if the fly ash has at least 75 percent passing Sieve No. 10, it will normally contain sufficient finer fraction to sustain suitable reaction (Little *et al.,* 1987).

2.4.6 Production of Wood Ash in Skardu Region

Skardu region is an agro based region, most of the people in this region are farmers. Tree plantation is very important in this region because it is the source of wood which is used for construction, cooking and heating purpose. Approximately 1600 to 3200 kg of wood is used by a single house hold for heating purpose only in a year. Therefore, a lot of wood ash is producing in Gilgit Baltistan region. But there is not accurate data regarding the production of wood ash in this region.

2.4.7 Potential Uses of Wood Ash

The pozzolanic nature of wood ash and its cheap availability makes it an attractive material for utilization in engineering applications. Major engineering applications of wood ash are:

- Soil stabilization in conjunction with or without lime or cement.
- Partial replacement of cement in concrete mixes.
- Manufacturing of low cost mud blocks for building construction.

Chapter 3

RESEARCH METHODOLOGY

3.1 INTRODUCTION

To achieve research objectives described in previous chapter, a detailed methodology is presented in this chapter. The details of materials being used, preparation of test specimen and different tests are discussed in this chapter.

3.2 INVESTIGATION PROGRAM

This research work covers characterization and stabilization of soil in Skardu region of Gilgit Baltistan. Soil samples were collected from different areas of Skardu region. All laboratory investigations were performed as per ASTM Standards. The research program was divided into five phases:

Phase 1: Sample Collection

Soil sample were collected from different areas of Skardu region.

Phase 2: Characterization of Untreated Soil

- Soil Classification
    - Sieve analysis
    - Hydrometer analysis
- Atterberg's Limits
    - Liquid Limit (LL)
    - Plastic Limit (PL)
    - Plasticity Index (PI)
- Specific Gravity
- X-ray Diffraction Test

- Compaction Characteristics (Modified Compaction Test)
- Unconfined compressive strength test (Soaked)
- Unconfined compressive strength Test (Un-soaked)
- California Bearing Ratio Test (CBR)
    - Un-soaked
    - Soaked

Phase 3: Optimization of Admixture Content

Modified compaction test carried out at different proportion of Wood ash with soil to optimize the moisture content with the maximum dry density.

Phase 4: Strength and Durability of Soil Admixture

- UCS test at varying percentage of wood ash with soil at 2, 7, 14 and 28 days. (Un-soaked)
- UCS test at varying percentage of wood ash with soil at 2, 7, 14 and 28 days. (Soaked)

Phase 5: California Bearing Ratio Test

- CBR at varying percentage of wood ash. (Un-soaked)
- CBR at varying percentage of wood ash. (Soaked)
- Swell Potential at optimum percentage of wood ash mixed soil.

Soil characterization was carried out to determine properties and characteristics of untreated soil and to establish the potential of soil for stabilization. On the basis of strength parameters and moisture-density relationship, a specific admixture percentage was decided for stabilization of soil. On the basis of optimized amount of admixture, further strength test of treated soil were carried out.

3.3 PHASE 01: SAMPLE COLLECTION

3.3.1 Soil Sample

The soil samples used in the research were collected from Skardu region. Samples were collected from five different areas, Kachura, Chunda, Gamba Skardu, Hussainabad and Skardu. Selected soil classified in Kachura valley as CL (Lean Clay), in Chunda valley ML (Silt) Soil type, whereas in Gamba Skardu, Skardu and Hussainabad soil type is varying depending upon the location, gravels are found near the mountains, while towards Indus river bank, CL-ML (Silty Clay) and CL (Lean Clay) are found. Oven dried soil was used throughout the study.

Figure 3.1: Location map of collected samples from Skardu region

Figure 3.2: Excavation of test pits for sample collection

Figure 3.3: Clay deposits near Indus river bank at Skardu region

3.3.2 Wood Ash

Wood ash was obtained from Skardu, produced by burning of local wood in houses. The grain size analysis of wood ash is shown in Figure 3.4 and Table 3.1.

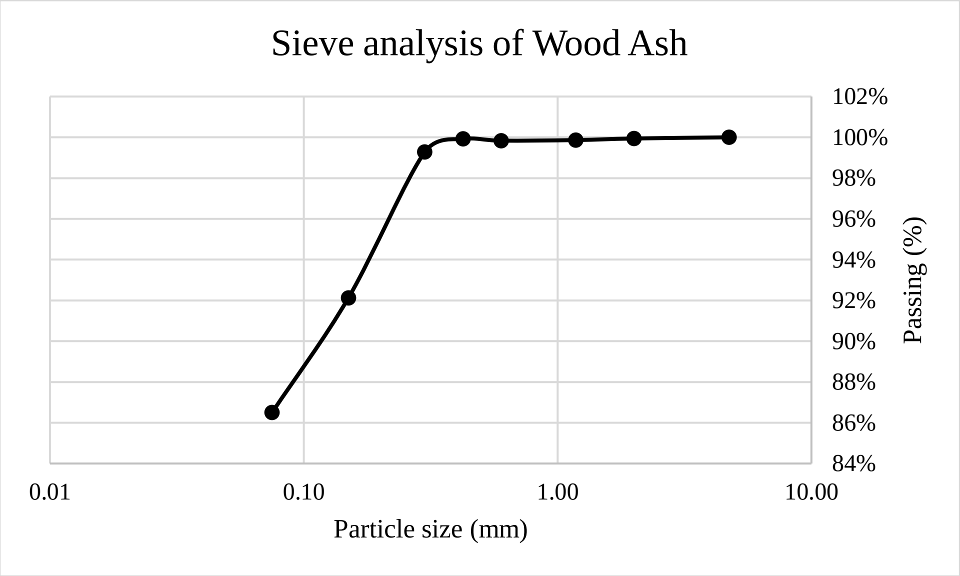

Figure 3.4: Grain size distribution curve of wood ash

Table 3.1: Grain size analysis of wood ash

Sieve Opening (mm)	Sieve No.	Retain Mass (gm)	Passing (gm)	% Pass	% Retain
4.75	4	0	207.82	100%	0.0%
2.00	10	0.12	207.70	100%	0.1%
1.18	16	0.28	207.54	100%	0.1%
0.60	30	0.34	207.48	100%	0.2%
0.43	40	0.17	207.65	100%	0.1%
0.30	50	1.49	206.33	99%	0.7%
0.15	100	16.36	191.46	92%	7.9%
0.08	200	28.06	179.76	86%	13.5%
Pan		161			77.5%
Total Mass		207.82			

3.4 PHASE 02: CHARACTERIZATION OF UNTREATED SOIL

Following tests were performed on untreated soil:

- Grain size distribution (Sieve analysis)
- Hydrometer analysis
- Atterberg's limit test (LL, PL)
- Modified proctor test (Moisture-density relationship)
- XRD (X-Ray diffraction)
- UCS test
- CBR test

3.4.1 Grain Size Distribution

To find the gradation of soil, *ASTM D 422* was followed. Total 400 gm sample was weighed, pulverized and then tested. A sieve set of sizes 4.75 mm, 2 mm, 0.425 mm, 0.25 mm, 0.16 mm and 0.075 mm was used and arranged in decreasing order. The percentages of passing from each sieve was calculated in a tabular form. A semi-log graph for gradation curve was developed by placing sieve sizes along the abscissa and percentage passing along ordinate axis.

3.4.2 Hydrometer Test

As the percentage passing through sieve No. 200 was more than 70 percent, so Hydrometer test was conducted to find a complete gradation curve. This test was performed as per *ASTM D 422*.

3.4.3 Atterberg’s Limits Test

Atterberg’s limits test was performed according to *ASTM D 4318-10* standard. To find out liquid and plastic limit of soil samples, a sample passing sieve no. 40 was collected. For soil classification, liquid limit and plastic limit were used with sieve analysis data. Soil was classified using Unified Soil Classification System (USCS) and AASHTO classification system. Plasticity index (PI) of the soil along with percent passing Sieve No. 200 serves as an indicator for feasibility of soil stabilization.

3.4.4 Modified Proctor Test

Modified Proctor test was performed according to *ASTM D 1557* standard. This test is performed for calculation of the maximum dry density and optimum moisture content of soil.

3.4.5 X-Ray Diffraction Test

The test was carried out at SCME labs, NUST Islamabad. For XRD analysis, the soil is crushed to powder form and 5 to 10 gm of sample is pressed into a sample holder and put into the XRD system. Fully automated XRD system (RIGAKU GEIGER FLEX–ME 20/PAS) with computer control of the diffractometer, digital data collection and computerized search-match facilities which can scan the whole XRD database was employed in this research. The system automatically detects clay minerals by matching the results with database.

Figure 3.5: XRD apparatus (CASEN, NUST Islamabad)

3.4.6 UCS Test

UCS test was carried out as per *ASTM D 5102-09* standard, procedure A. Proving ring reading (stress) was noted at constant interval of deformation dial reading. After put on corrections, stress at each deformation was find out by calculations. After peak stress value, it was tried to note two readings at least; however, due to brittle nature of material it was not easy in most cases. In such case, a sudden drop was noticed. Average strength was calculated for two specimens of each curing period.

3.4.7 CBR Test

ASTM D 1883-99 standard was followed to perform the CBR tests. The test comprises load application to a small penetration piston (3 square inch) at a rate of 1.3 mm per sixty seconds and recording the total load at penetrations ranging from 0.025 inch up to 0.500 inch. Samples were prepared at optimum moisture content and at maximum dry density of soil. Soil was compacted in 5 layers, each layer bearing 65 number of blows. Soaked and un-soaked tests were conducted, and swell potential of soil was also determined using CBR mould.

3.5 PHASE 03: OPTIMIZATION OF ADMIXTURE CONTENT FOR STABILIZATION OF SOIL

3.5.1 Optimization of Wood Ash content

3.5.1.1 Moisture–density relationship (OMC - MDD)

ASTM D 1557-02 was used to develop the moisture density relationship at 5, 10, 15 and 20 percent wood ash content by weight.

3.5.1.2 Unconfined compressive strength testing (UCS)

Samples were fabricated at the above mentioned wood ash percentages in accordance with *ASTM D 5102-09* at a moisture level of respective OMC. The samples were fabricated and cured for 2 days, 7 days, 14 days and 28 days at 40^{o} C. The strengths were compared and optimum wood ash content was confirmed. For the comparison of average strength, two samples were fabricated and tested.

3.6 FABRICATION AND CURING OF SAMPLES AT OPTIMUM CONTENT AND TESTING

3.6.1 Sample Fabrication

For UCS testing procedure of *ASTM D 5102-09* was used. The procedure requires that test specimen should have a height to diameter ratio of 2:1. A special mould having 6-inch height and 3-inch diameter was fabricated. During modified compaction procedure, the compaction effort was found out using calculations with respect to energy delivered per unit volume of soil. Weight of hammer, number of layers, and height of fall was kept same and only number of blows were satisfied. Adjusted blows came out to be 22. Several trial specimens were prepared as per adjusted blows and their density was checked to ascertain the validity of adjusted number of blows.

3.6.2 Curing of Samples

Each sample was sealed in a plastic sheet to make it air tight. Samples were then placed in an oven at 40^{o} C for the desired curing period.

Figure 3.6 Un-soaked curing of samples for UCS test

3.7 PHASE 04: STRENGTH AND DURABILITY OF SOIL ADMIXTURE

UCS test was carried out as per *ASTM D 5102-09*, procedure A. Proving ring reading (stress) was noticed against constant interval of deformation dial reading. After using corrections, stress at each deformation was calculated. After peak stress value, it was tried to note two readings at least; however, due to the brittle nature of material it was not possible in most cases.

3.7.1 Un-soaked Testing

After 2, 7, 14 and 28 days curing periods, un-soaked tests were carried out. Average strength was calculated for two specimens of each curing period.

Figure 3.7: Sample testing arrangement in UCS apparatus

3.7.2 Soaked Testing

After curing samples were covered in a permeable cloth and positioned over a porous stone in a vacuum desiccator. Up to the top of porous stone, desiccator was filled with water. There was no direct contact of sample with water. Samples were tested after soaking for 24 hours. In this case also, average strength of two samples were reported.

Figure 3.8 Soaked curing of samples for UCS test

3.8 PHASE 05: CALIFORNIA BEARING RATIO TEST

Soil was compacted in 5 layers, giving 65 blows to each layer. Soaked and un-soaked tests were conducted, swell potential of soil wood ash mixture was also determined using the same mould.

Figure 3.9: Soaking arrangement of samples for CBR test

3.9 TOTAL TESTS MATRIX

On the basis of methodology, total of 100 tests are prepared.

Table 3.2: Total tests matrix

Sr. No.	Sample Condition	Test Name	Quantity	ASTM Standards
1	Untreated Soil	Sieve analysis	3	D-2487
		Atterberg’s limit test (LL & PL)	2	D-4318
		Specific gravity	3	D-854
		Modified proctor Test	2	D-1557
		UCS test	4	D-5102 & D-2166
		CBR test	4	D-1883
		XRD test	2	
2	Wood Ash Treated Soil	Modified proctor test	8	D-1557
		UCS test	64	D-5102 & D-2166
		CBR test	8	D-1883
3	Total		100	

Chapter 4

RESULTS AND DISCUSSION

4.1 GENERAL

This chapter includes discussion on the tests performed on the two type of soils selected from Skardu region, i.e., soil Type-I (CL-ML) and soil Type-II (CL). These two type of soils were characterized and then stabilized using locally produced wood ash.

4.2 PHASE 02: CHARACTERIZATION OF UNTREATED SOIL

To determine the engineering properties of soil, various tests were conducted. Table 4.1 summarizes the different tests results for untreated soil and subsequent paragraphs explained the detail of results.

4.2.1 Soil Classification and Characterization

Soil Type-I has been classified as A-4 according to AASHTO classification system and low plastic Silty clay (CL-ML) as per USCS (Unified soil classification system) and Soil Type-II has been classified as A-6 according to AASHTO classification system and low plastic lean clay (CL) as per USCS. For soil Type-I, UCS tests shows that when soil subjected to capillary soak condition for 2 days, it looses strength (i.e., reduction of strength was from 240.8 kPa to 80.2 kPa). Similarly, for soil Type-II, also looses strength when soaked for 2 days’ time (i.e., reduction of strength from 255.4 kPa to 54.2 kPa). CBR test results for soil Type-I is 6.6 percent and soil Type-II is 4.4 percent.

Table 4.1: Untreated soil characterization

Soil Properties		**Soil Type-I (CL-ML)**	**Soil Type-II (CL)**
Percent passing sieve no. 200		71%	90%
Liquid limit		26.2	29
Plastic limit		20	17.1
Plasticity index		6.2	11.9
Specific gravity		2.51	2.67
Maximum dry density, (g/cm^3)		1.95	1.84
Optimum moisture content, (%)		12.2	14.3
Clay minerals		Quartz,Albite, Muscovite, Illite	Quartz, Albite, Muscovite, Illite, Kaolinite
Unconfined compressive strength (UCS), (kPa)	Un-soaked	240.8	255.4
	Soaked	80.2	54.2
CBR, (%)	Un-soaked	6.6	4.4
	Soaked	5.4	2.2
One dimensional swell potential, (%)		2.65	4.66

X-ray Diffraction test results are presented in figure 4.1 and figure 4.2. This test identifies Quartz, Muscovite, Illite and Kaolinite minerals present in both type of soils. Presence of clay minerals in soil shows that it is reactive with admixture.

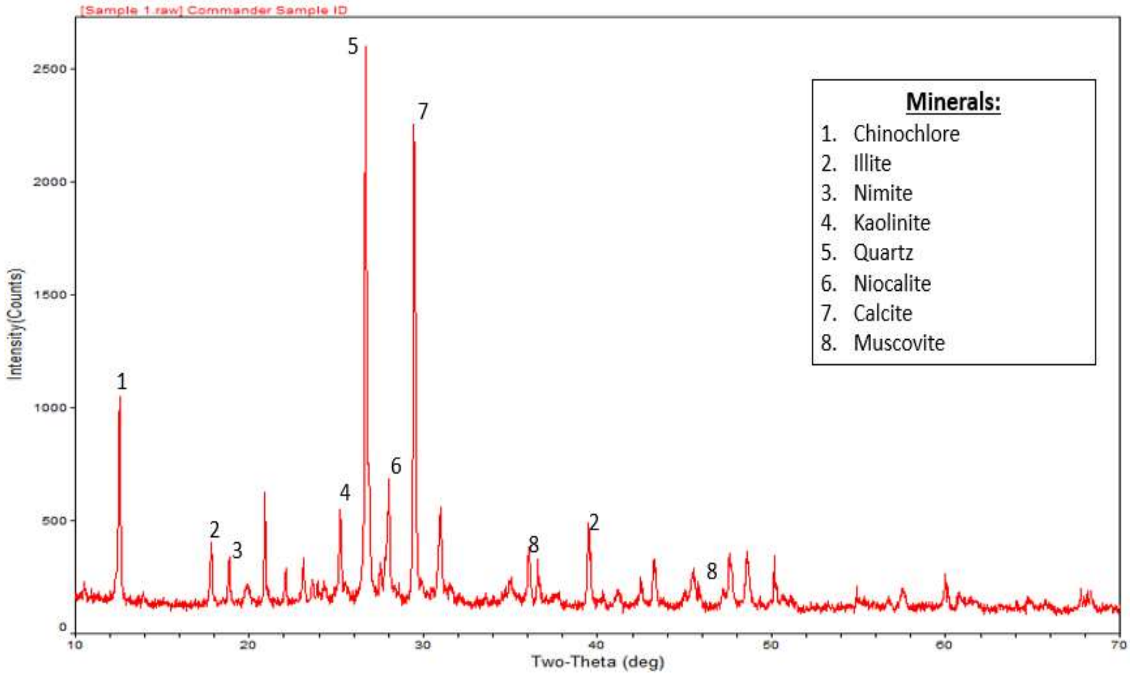

Figure 4.1: X-ray diffraction test results of soil Type-I

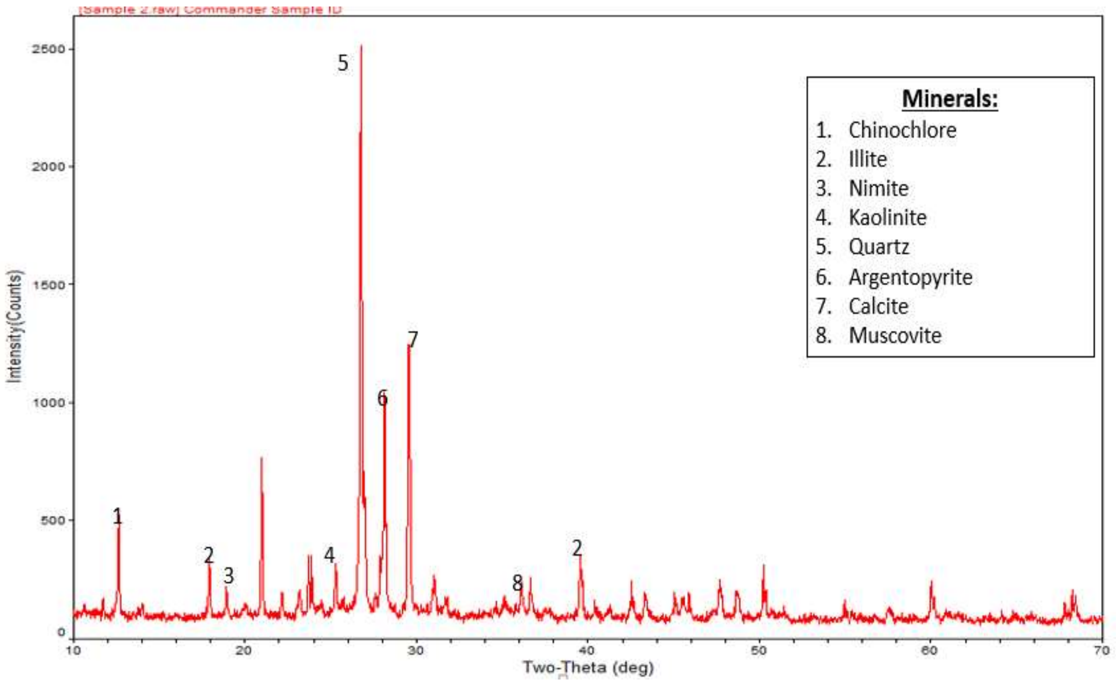

Figure 4.2: X-ray diffraction test results of soil Type-II

4.3 PHASE 03: OPTIMIZATION OF ADMIXTURE CONTENT

4.3.1 Wood Ash Content

Modified Proctor tests were performed to determine the maximum dry density (MDD) and optimum moisture content (OMC) for different percentages of wood ash mix with both type of soils. Effect of wood ash on density and optimum moisture content against different percentages is shown in figure 4.3 to figure 4.6. From figure 4.3 and figure 4.4 for soil Type-I shows that addition of wood ash decrease maximum dry density and increase optimum moisture content of wood treated soil Type-I.

From figure 4.5 and figure 4.6 minimum value of MDD is achieved at 10 percent mix of wood ash with soil Type-II, with further addition of wood ash MDD increases. OMC increases up to 10 percent wood ash and with further addition of wood ash OMC value decreased. This represent that addition of wood ash effect the MDD and OMC values.

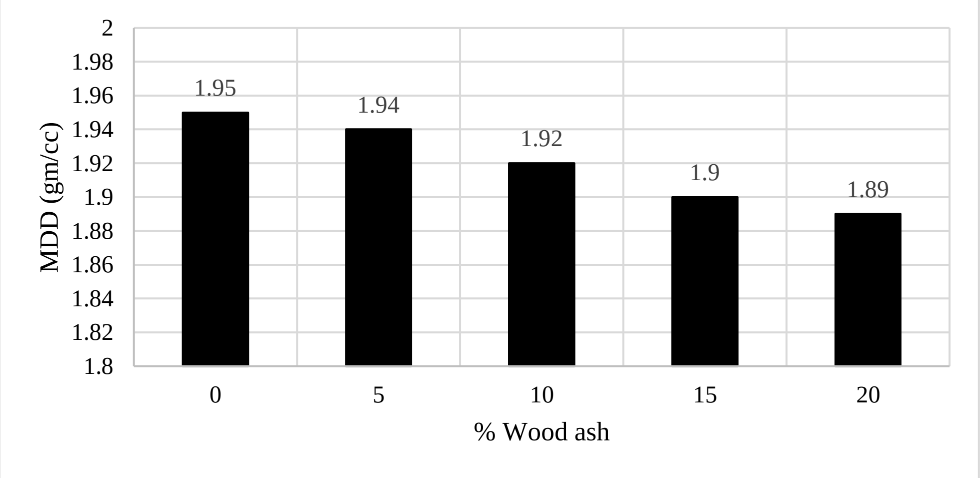

Figure 4.3: Effect of wood ash on MDD of soil Type-I

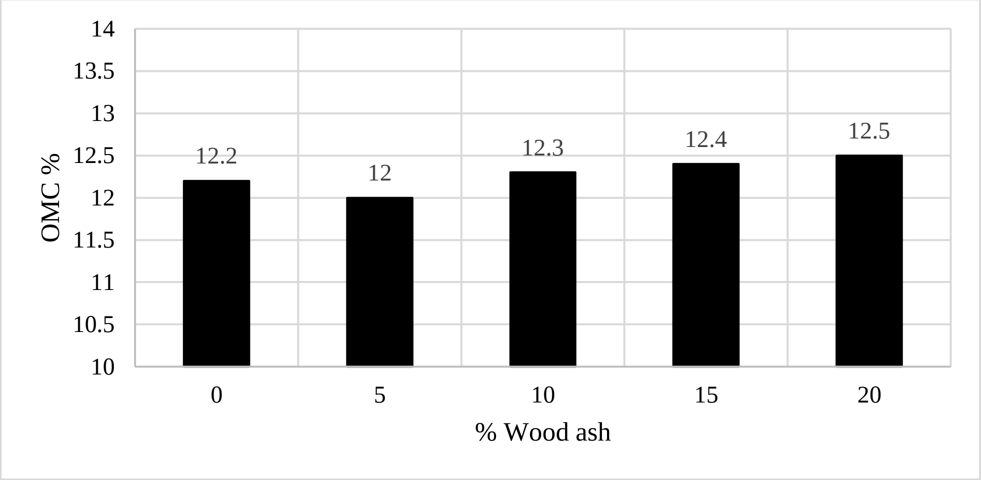

Figure 4.4: Effect of wood ash on OMC of soil Type-I

Many researchers suggest that, the reason behind the change in MDD values is because of flocculation and agglomeration of the clay particles which is caused by the cation exchange reaction resulting in the change in weight to volume ratio (Okagbue, 2007). Increase in OMC values is due to the exothermic reactions, disassociation of the calcium hydroxide into Ca^+ and OH^- ions which required more water for the reaction (NLA, 2004; Okagbue and Yakubu, 2009).

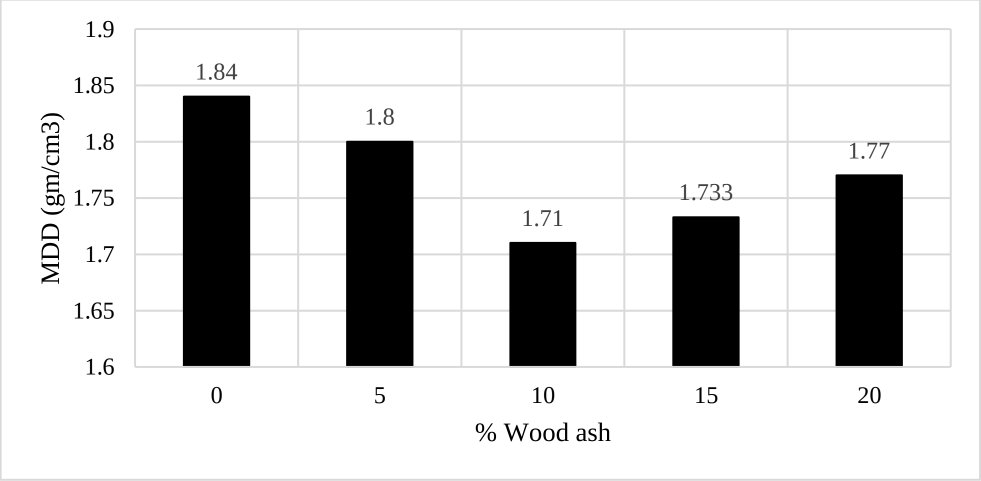

Figure 4.5: Effect of wood ash on MDD of soil Type-II

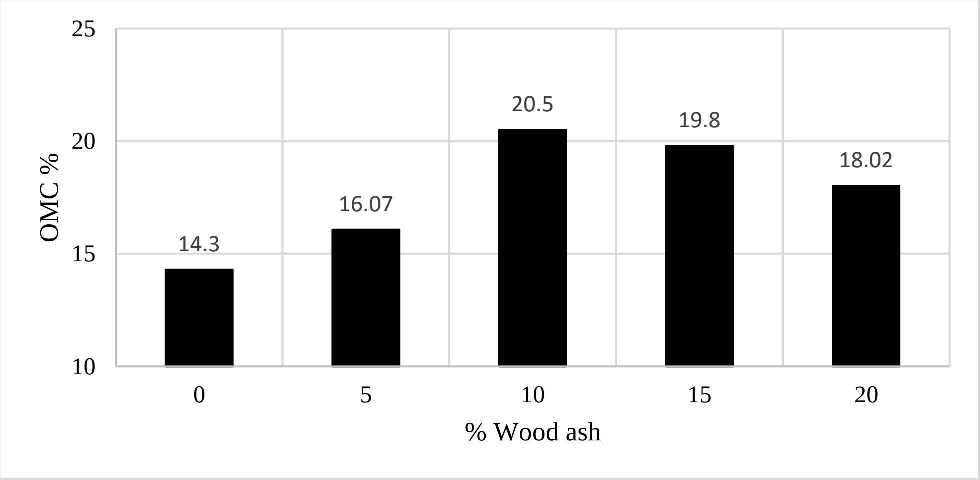

Figure 4.6: Effect of wood ash on OMC of soil Type-II

4.3.2 Pozzolanic Properties of Wood Ash

Table 4.2 presents the chemical compositions of wood ash and its comparison with fly ash and bottom ash. This shows that wood ash reacts with water forms calcium hydroxide, ettringite, gypsum and others which is assumed to be a noteworthy part in deciding wood ash adequacy as soil stabilizers.

Table 4.3 present different researcher's work on UCS, CBR and Swell potential of soil with the variation of different types of ashes. All studies shows that improvement in engineering properties of soils were observed with the addition of ashes due to the presence of pozzolanic material. (Manmey, 2015; Güllü, 2014; Cadersa *et al.*, 2014).

Table 4.2: Chemical composition of ashes

Sr. No.	Chemical Composition	Wood ash (%)	Fly ash (%)	Bottom ash (%)
1	CaO	32.34	0.2 – 7.87	3.85
2	SiO_2	53.4	38-60	23-73
3	Al_2O_3	1.33	31-44	13-27
4	Fe_2O_3	5.05	3-4	1-14
5	P_2O_5	0.02	1.11	0.19
6	MgO	1.4	2.32	0.58
7	Na_2O	2.32	0.39	0.80
8	K_2O	0.29	3.39	2.07

Table 4.3: Comparison of UCS, CBR and Swell Potential values of soil with variation of ashes

Ash (%)	Wood ash	Fly ash	Bottom ash	Wood ash	Fly ash	Bottom ash	Wood ash	Fly ash	Bottom ash	Wood ash	Fly ash	Bottom ash
	UCS (kPa)			CBR (soaked) (%)			CBR (un-soaked) (%)			Swell Potential (%)		
0	255	197	291	2.2	4.44	40	4.4	6	55	4.66	2.02	0.17
5	581	-	304	2.7	-	-	4.5	-	-	1.02	-	-
10	574	195	374	4.8	4.4	-	8.6	10.5	-	0.8	1.92	-
15	282	-	297	3.8	-	40	5.4	-	62	-	-	0.14
20	270	206	375	3.1	5.28	-	4.7	16.6	-	-	1.84	-
25	-	-	316	-	-	-	-	-	-	-	-	-
30	-	174	-	-	5.52	85	-	9.95	140	-	1.77	0.08
40	-	170	-	-	4.08	65	-	8.61	115	-	1.63	0.04
50	-	166	-	-	3.48	40	-	8.51	55	-	1.53	0.17

4.4 PHASE 04: STRENGTH AND DURABILITY OF SOIL - ADMIXTURE

4.4.1 Wood Ash with Soil

4.4.1.1 UCS (un-soaked curing) of wood ash mixed with soil type-I

Figure 4.7 and figure 4.8 demonstrate effect of wood ash and the curing time on UCS (un-soaked curing) on wood ash and soil Type-I mixture.

As shown in table 4.4, UCS un-soaked, untreated soil have UCS value of 240.8 kPa and it remains unchanged with curing period. With the addition of 5 percent wood ash, UCS value increases to 352.0 kPa after two days, 423.6 kPa after 7 days and 447.8 kPa after 28 days of curing. With addition of 10 percent wood ash, UCS value increases to 349.2 kPa after two days, 420.3 kPa after 7 days and 430.8 kPa after 28 days of curing. With addition of 15 percent wood ash, UCS value increases to 333.8 kPa after two days, 377.1 kPa after 7 days and 388.8 kPa after 28 days of curing. For addition of 20 percent wood ash UCS value increases to 300.2 kPa after two days, 375.7 kPa after 7 days and 383.7 kPa after 28 days of un-soaked curing.

Table 4.4: UCS test results of un-soaked soil Type-I

Curing Time (Days)	**Wood ash (%)**				
	0	**5**	**10**	**15**	**20**
2	240.8	352.0	349.2	333.8	300.2
7	240.8	423.6	420.3	377.1	375.7
14	240.8	429.7	419.2	382.7	378.1
28	240.8	447.8	430.8	388.8	383.7

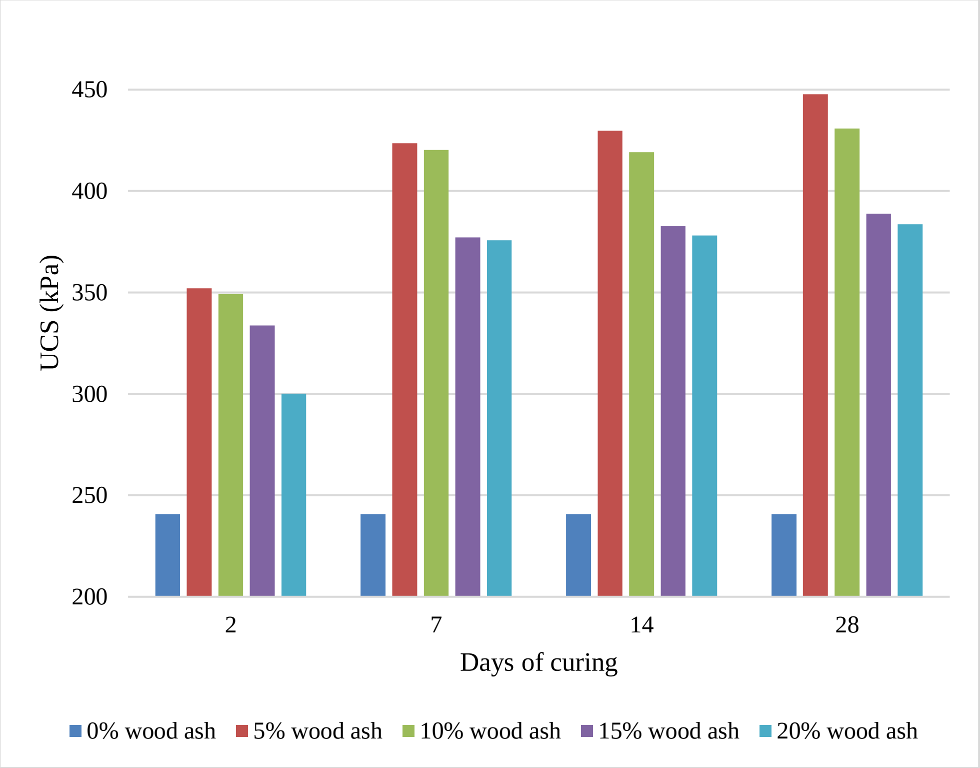

Figure 4.7: UCS (un-soaked) results for soil Type-I

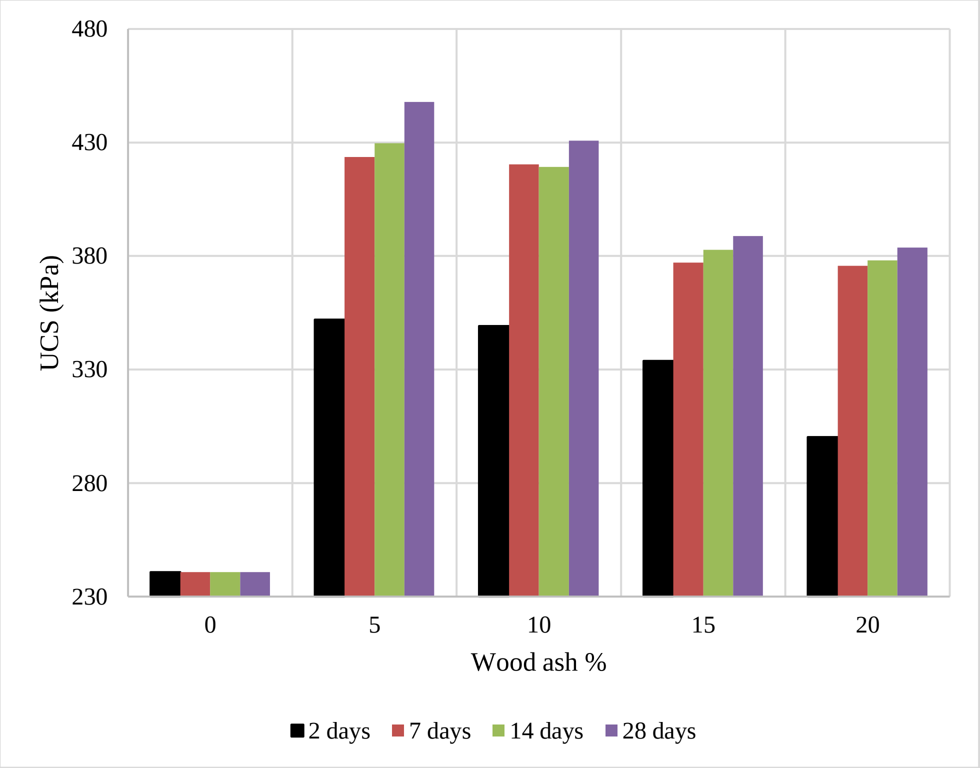

Figure 4.8: UCS (un-soaked) results with variation of wood ash soil Type-I

For un-soaked curing of soil Type-I with the addition of wood ash from 5 to 10 percent, UCS value increases and with further addition of wood ash, UCS values decreases.

4.4.1.2 UCS (soaked curing) of wood ash mixed with soil type-I

Figure 4.9 and figure 4.10 demonstrate effect of wood ash and the curing time on UCS (soaked curing) on wood ash and soil Type-I mixture.

As shown in table 4.5, UCS soaked test results, untreated soil have UCS value of 80.2 kPa, with the addition of 5 percent wood ash UCS value increases to 175.4 kPa after two days, 184.6 kPa after 7 days and 189.4 kPa after 28 days of curing. With addition of 10 percent wood ash, UCS value increases to 118.4 kPa after two days, 138.5 kPa after 7 days and 154.2 kPa after 28 days of curing. With addition of 15 percent wood ash, UCS value increases to 109.4 kPa after two days, 110.2 kPa after 7 days and 114.4 kPa after 28 days of curing. With addition of 20 percent wood ash, UCS value increases to 108.6 kPa after two days, 109.4 kPa after 7 days and 112.3 kPa after 28 days of soaked curing.

Table 4.5: UCS test results of soaked curing soil Type-I

Curing Time (Days)	Wood ash (%)				
	0	5	10	15	20
2	80.2	175.4	118.4	109.4	108.6
7	80.2	184.6	138.5	110.2	109.4
14	80.2	186.2	144.6	112.6	111.5
28	80.2	189.4	154.2	114.4	112.3

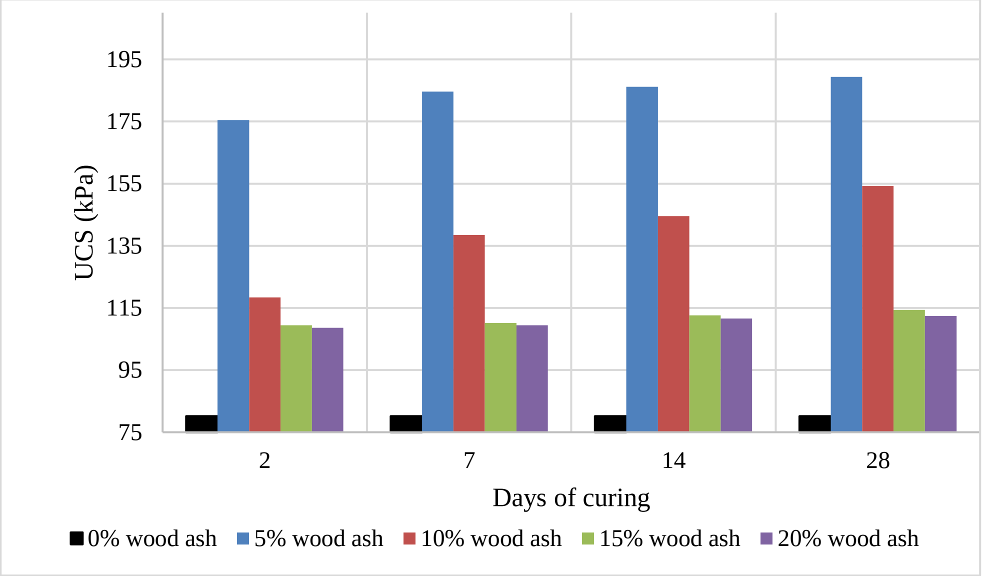

Figure 4.9: UCS (soaked curing) results soil Type-I

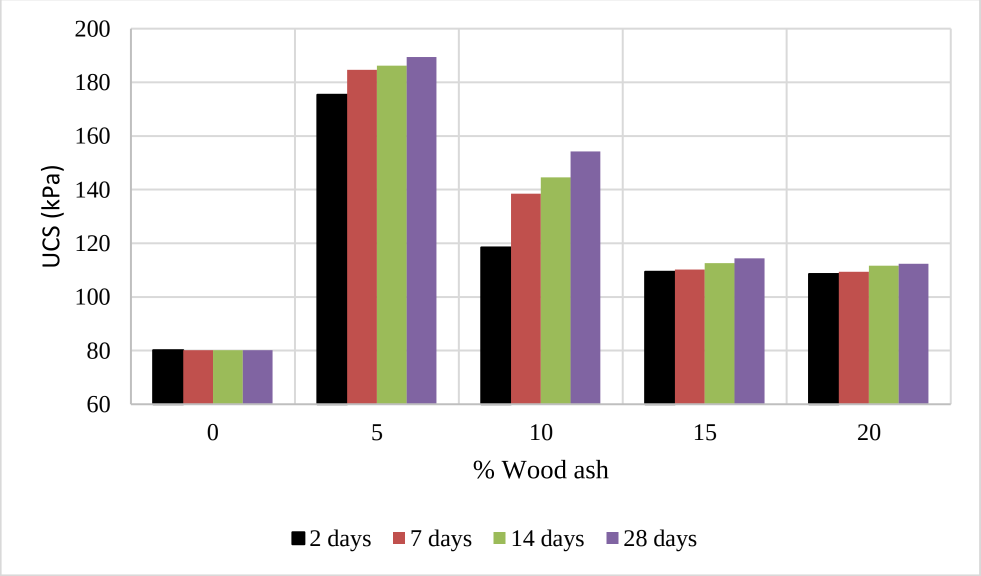

Figure 4.10: UCS (soaked curing) results with variation of wood ash soil Type-I

For soaked curing of soil Type-I, with the addition of 5 percent wood ash UCS value increases and after further addition of wood ash, UCS values decreases.

4.4.1.3 UCS (un-soaked curing) of wood ash mixed with soil type-II

Figure 4.11 and figure 4.12 demonstrate effect of wood ash and the curing time on UCS (un-soaked curing) on wood ash and soil Type-II mixture.

As shown in table 4.6, UCS un-soaked untreated soil have UCS value of 255.4

kPa. With the addition of 5 percent wood ash, UCS value increases to 581.0 kPa after two days, 767.6 kPa after 7 days and 795.6 kPa after 28 days of curing. With addition of 10 percent wood ash, UCS value increases to 574.2 kPa after two days, 737.8 kPa after 7 days and 782.6 kPa after 28 days of curing. With addition of 15 percent wood ash, UCS value increases to 282.8 kPa after two days, 388.3 kPa after 7 days and 401.2 kPa after 28 days of curing. With addition of 20 percent wood ash, UCS value increases to 270.4 kPa after two days, 303.8 kPa after 7 days and 311.7 kPa after 28 days of curing.

Table 4.6: UCS test results of un-soaked curing of soil Type-II

Curing Time (Days)	Wood ash (%)				
	0	5	10	15	20
2	255.4	581.0	574.2	282.8	270.4
7	255.4	767.6	737.8	388.3	303.8
14	255.4	779.6	772.7	396.0	300.7
28	255.4	795.6	782.6	401.2	311.7

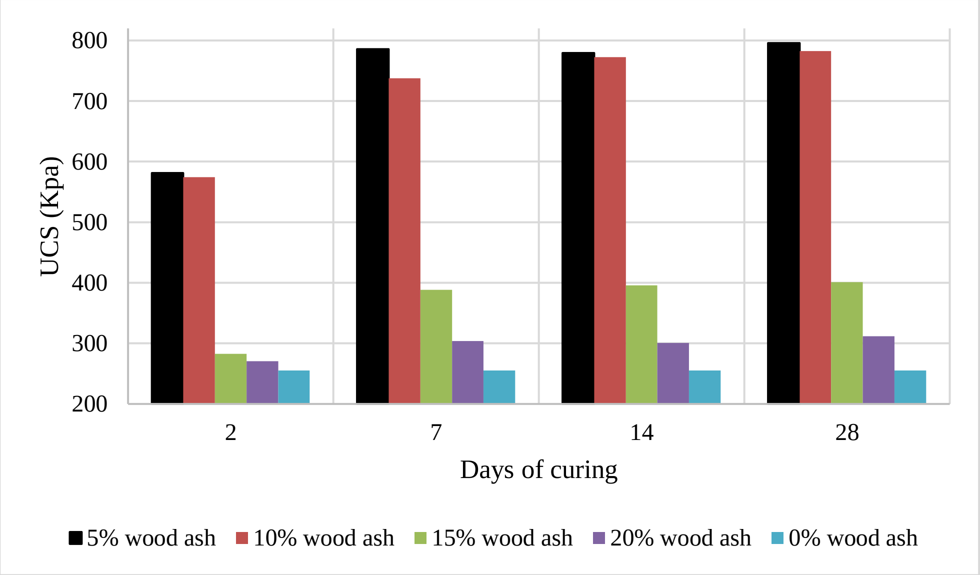

Figure 4.11: UCS (un-soaked curing) results of soil Type-II

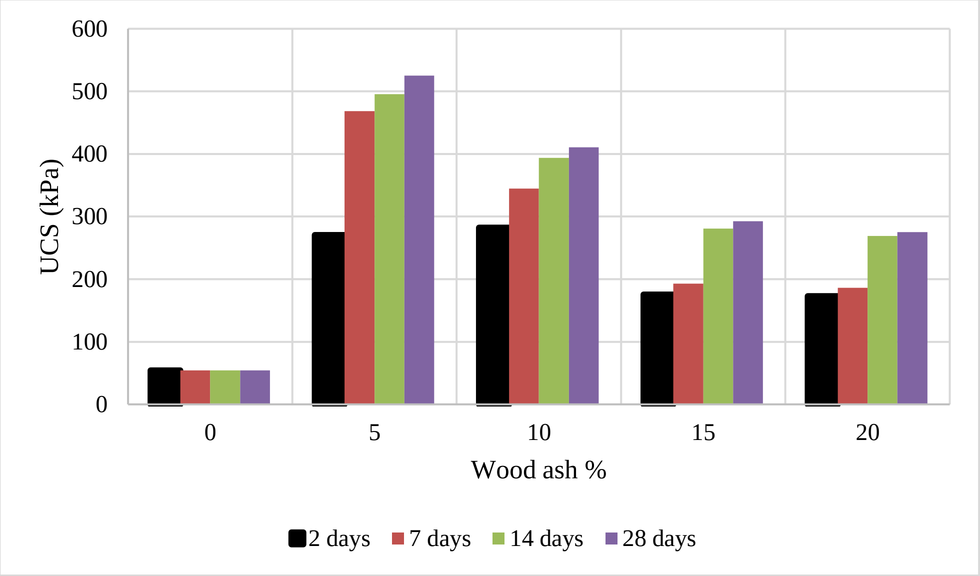

Figure 4.12: UCS (un-soaked curing) results with variation of wood ash soil Type-II

For un-soaked curing of soil Type-II with the addition of wood ash from 5 to 10 percent, UCS value increases and further addition of wood ash, UCS values decreases.

4.4.1.4 UCS (soaked curing) of wood ash mixed with soil type-II

Figure 4.13 and figure 4.14 demonstrate effect of wood ash and the curing time on UCS (soaked curing) results on wood ash and soil Type-II mixture.

Table 4.7: UCS test results of soaked curing soil Type-II

Curing Time (Days)	Wood ash (%)				
	0	5	10	15	20
2	54.2	270.0	265.0	175.4	173.1
7	54.2	468.5	344.6	193.1	186.2
14	54.2	495.4	393.8	280.8	269.2
28	54.2	525.4	410.9	292.3	275.4

As shown in table 4.7, UCS soaked test results, untreated soil have UCS value of 54.2 kPa. With the addition of 5 percent wood ash, UCS value increases to 270.0

kPa after two days, 468.5 kPa after 7 days and 525.4 kPa after 28 days of curing. With addition of 10 percent wood ash, UCS value increases to 265.0 kPa after two days, 344.6 kPa after 7 days and 410.9 kPa after 28 days of curing. With addition of 15 percent wood ash, UCS value increases to 175.4 kPa after two days, 193.1 kPa after 7 days and 292.3 kPa after 28 days of curing. With addition of 20 percent wood ash, UCS value increases to 173.1 kPa after two days, 186.2 kPa after 7 days and 275.4 kPa after 28 days of curing.

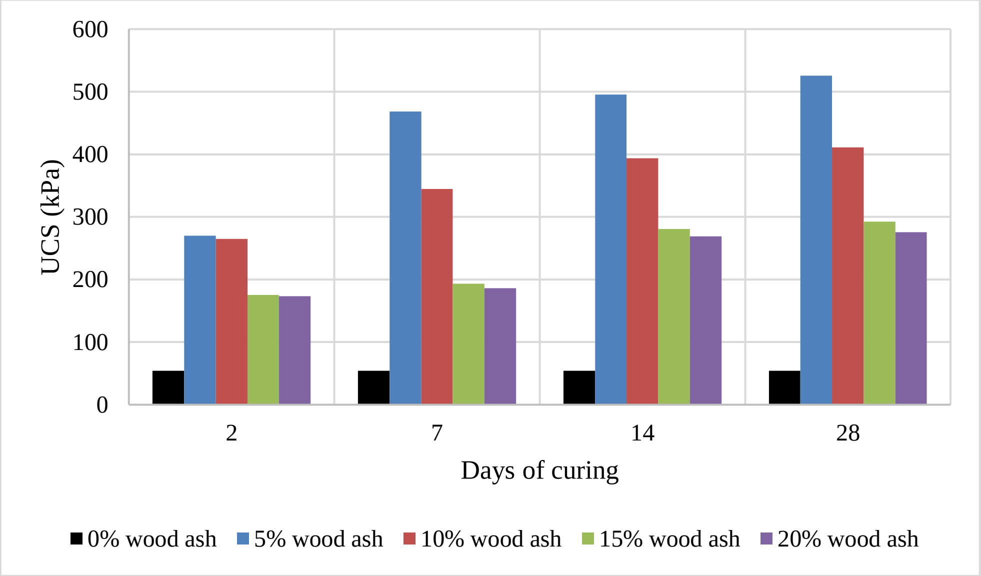

Figure 4.13 UCS (soaked curing) results of soil Type-II

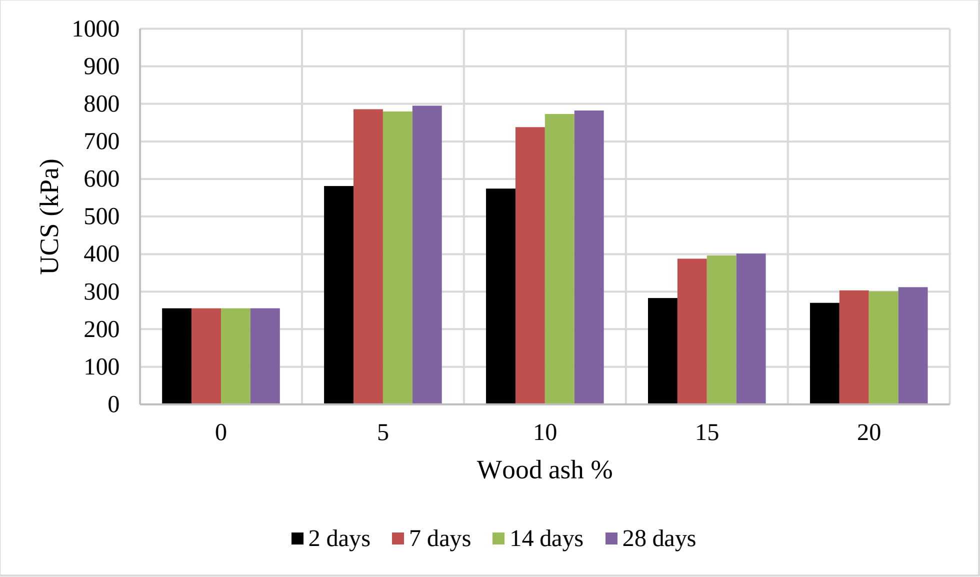

Figure 4.14: UCS (soaked curing) results with variation of wood ash soil Type-II

For soaked curing of soil Type-II with the addition of wood ash from 5 to 10 percent, UCS value increases and further addition of wood ash, UCS values decreases.

4.4.2 Discussion on UCS Test Results of Wood Ash Treated Soil

The increase in unconfined compression strength during curing period is due to the formation of cementing gel materials produced due to the pozzolanic reactions which take place over a period of time. In pozzolanic reaction strength increases by forming silicate gel which initially binds and coats together the lumps of clay, with the passage of time it forms crystals which binds the soil particles which result in the gain of strength (Terrel *et al.*, 1979; Hadi *et al.*, 2008).

$$\mathrm{Ca\,(OH)_2} \text{ ------------------------ } \mathrm{Ca^{2+} + 2(OH)}$$

$$\mathrm{Ca^{2+} + OH^- + SiO_2}\text{--------------}\mathrm{CSH}\text{ (Calcium silicate hydrate)}$$

$$\mathrm{Ca^{2+} + OH^- + Al_2O_3}\text{------------}\mathrm{CAH}\text{ (Calcium alumina hydrate)}$$

The main source of $Ca(OH)_2$ is wood ash which reacts with clay minerals found in soil. After 5 to 10 percent of wood ash content, the UCS value decreases which may be due to the quick use of lime in the pozzolanic reaction while the additional quantity of wood ash acts as unbounded particles or filler which have no any cohesion or any friction which results in decrease of unconfined compression strength (Bell, 1996; Abdullahi, 2006).

4.5 PHASE 05: CALIFORNIA BEARING RATIO TEST

4.5.1 CBR of Wood Ash Treated Soil Type-I

4.5.1.1 CBR (un-soaked) of wood ash treated soil type-I

From figure 4.15, un-soaked untreated soil have CBR value of 6.6 percent. With the addition of 5 percent wood ash CBR values increased to 10.0 percent, with the addition of 10 percent wood ash CBR value increases to 11.6 percent, with the addition of 15 percent wood ash CBR value increases to 10.3 percent and with the addition of 20 percent wood ash CBR value increases to 10.1 percent.

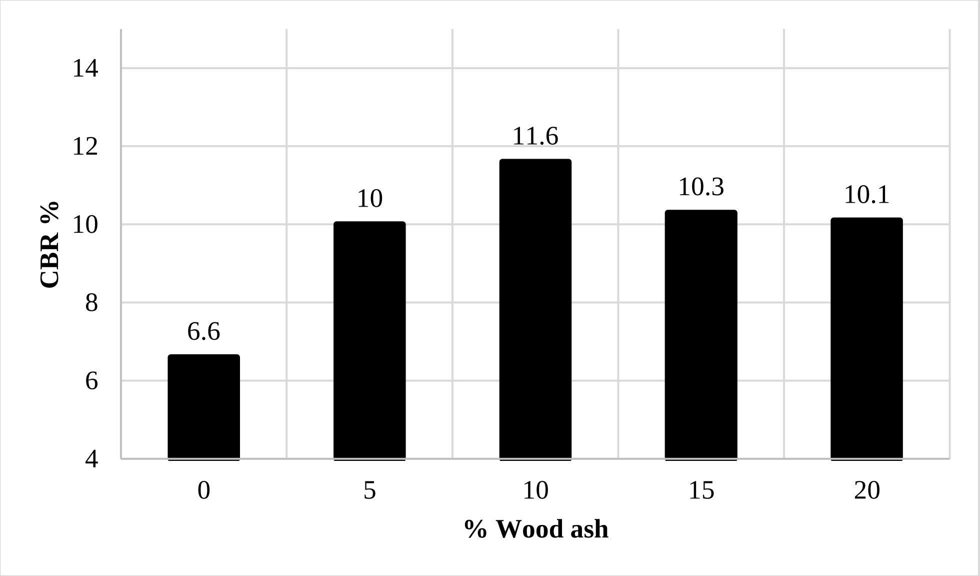

Figure 4.15: CBR (un-soaked) test result on wood ash treated soil Type-I

In un-soaked CBR test of soil Type-I, 10 percent wood ash gives higher CBR value and with the further addition of wood ash CBR value decreases.

4.5.1.2 CBR (soaked) of wood ash treated soil type-I

As shown in figure 4.16, soaked untreated soil have CBR value of 5.4 percent, with the addition of 5 percent wood ash CBR values increased to 6.8 percent, with the addition of 10 percent wood ash CBR value increases to 8.5 percent, with the addition of 15 percent wood ash CBR value increases to 6.6 percent and with the addition of 20 percent wood ash CBR value increases to 5.8 percent.

For soaked CBR test of soil Type-I, 10 percent wood ash gives higher CBR value, with the further addition of wood ash CBR value decreases.

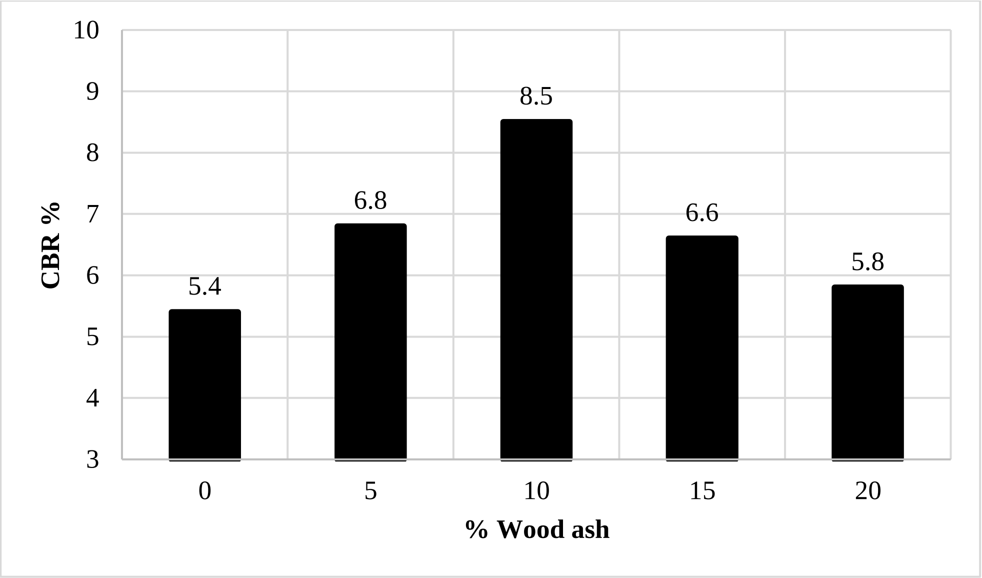

Figure 4.16: CBR (soaked) test result on wood ash treated soil Type-I

4.5.2 CBR of Wood Ash Treated Soil Type-II

4.5.2.1 CBR (un-soaked) of wood ash treated soil type-II

As shown in figure 4.17, un-soaked untreated soil have CBR value of 4.4 percent. With the addition of 5 percent wood ash CBR values increased to 4.5 percent, with the addition of 10 percent wood ash CBR value increases to 8.6 percent, with the addition of 15 percent wood ash CBR value increases to 5.4 percent and with the addition of 20 percent wood ash CBR value increases to 4.7 percent.

Therefore, 10 percent wood ash gives higher CBR value, with the further addition of wood ash to soil Type-II, CBR value decreases.

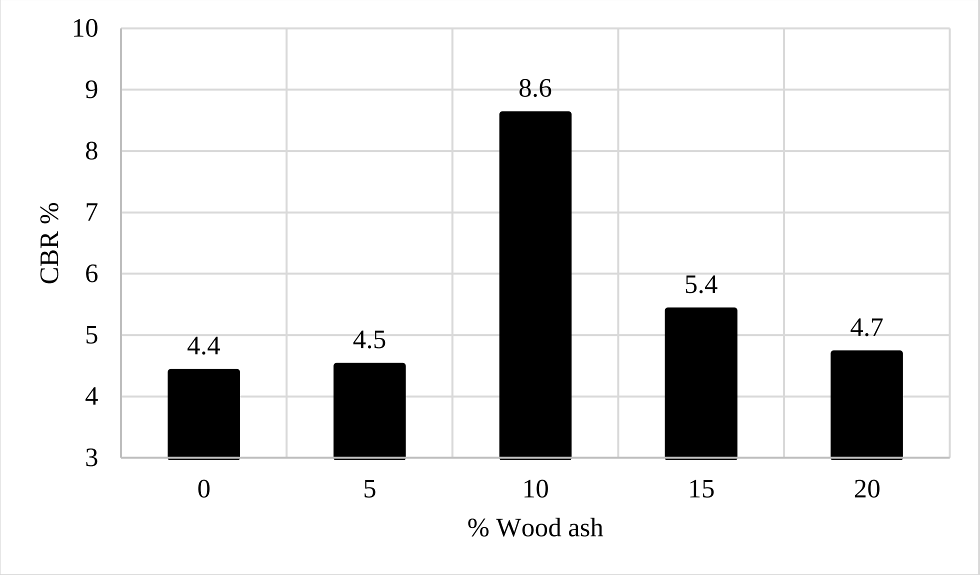

Figure 4.17: CBR (un-soaked) test result on wood ash treated soil Type-II

4.5.2.2 CBR (soaked) of wood ash treated soil Type-II

As shown in figure 4.18, soaked untreated soil have CBR value of 2.2 percent. With the addition of 5 percent wood ash CBR values increased to 2.7 percent, with the addition of 10 percent wood ash CBR value increases to 4.8 percent, with the addition of 15 percent wood ash CBR value increases to 3.8 percent and with the addition of 20 percent wood ash CBR value increases to 3.1 percent.

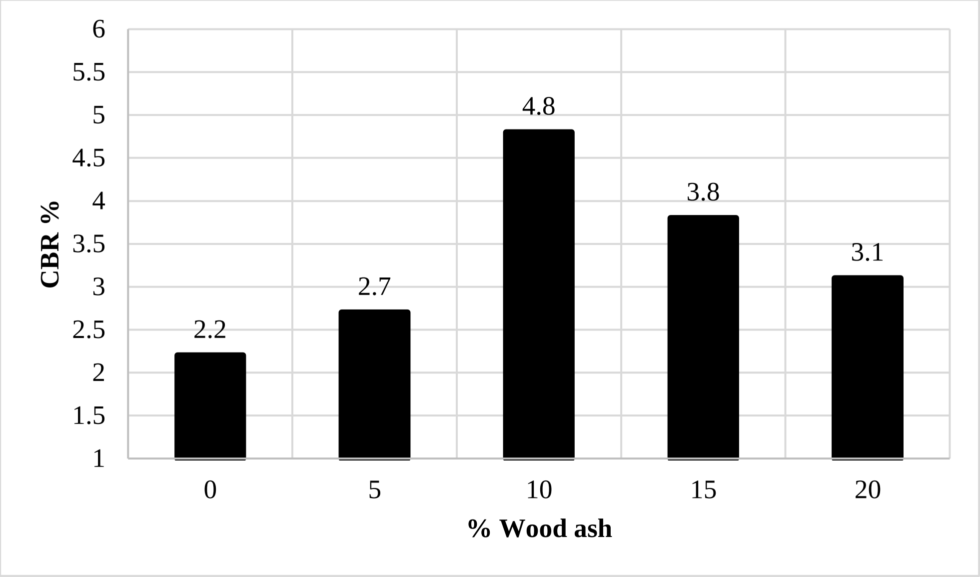

Figure 4.18: CBR (soaked) test result on wood ash treated soil Type-II

4.5.3 Discussion on CBR of Wood ash Treated Soil

Improvement in CBR value with the addition of wood ash is due to the soil lime reaction which forms flocks of soils, they retain their individuality when kneaded and compacted (Van Ganse, 1974; Ola, 1977). Another reason for gain in CBR value is due to the formation of tetracalcium alumina hydrates and silicate hydrates which bond together clay particles (Diamond and Kinter, 1965; Bell, 1993; Montohar and Hantoro, 2000; Okagbue and Yakubu, 2000; Wartman and Hantoro, 2000). Another reason for improvement of CBR values are due to the immediate cation exchange flocculation and agglomeration reaction (Neubauer and Thompson, 1972; Arora, 1997).

4.5.4 One Dimensional Swelling Potential of Wood Ash Treated Soil

Table 4.8 shows one dimensional swell potential test results. After soaking for 96 hours, untreated soil type-I have one dimensional swell potential value of 2.65 percent, after adding 5 percent wood ash its value decreased to 1.17 percent and after adding 10 percent wood ash its value decreased to 0.89 percent. After soaked for 96 hours, untreated soil Type-II have one dimensional swelling potential value of 4.66 percent. With the addition of 5 percent wood ash its value decrease to 1.02 percent and after addition of 10 percent wood ash its value decreased to 0.8 percent.

From these test results, it is conformed that with the addition of wood ash one dimension swell potential decreases. The reason for decrease of swell potential is due to the cation exchange reaction in which sodium ions are replaced with calcium ions (Yilmaz and Civelekoghu, 2009).

Table 4.8: One dimensional swell potential of wood ash treated soil

Wood Ash (%)	Soil Type-I (%)	Soil Type-II (%)
0	2.65	4.66
5	1.17	1.02
10	0.89	0.8

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

Following conclusion can be made from the experimental results of this research study:

- Presence of clay minerals in both type of soils detected by XRD analysis suggest that soil is reactive with wood ash.
- For both types of soil, maximum dry density is decreased by the addition of wood ash while increase in optimum moisture content is observed. Decrease in dry density is due to flocculation of soil particles. Soil become more friable and difficult to compact. While the increase in optimum moisture content is due to the increased surface of soil particles due to the addition of wood ash which is finer particles.
- Change in UCS of soil with varying content of wood ash is observed. For soaked and un-soaked curing, with 5 to 10 percent of wood ash mixed with both type of soils have higher strength. The improvement in UCS was associated with the improvement in chemical bonding between soil particles due to the pozzolanic reaction between soil minerals with pozzolanic agent present in wood ash.
- Improvement in CBR of treated soil under soaked and un-soaked condition is observed for both types of soil. At 10 percent wood ash mixed with both types of soil gave higher CBR values. Increase in CBR may be due to the cementation between soil particles by wood ash.

- From one dimensional swell tests on soil wood ash mixture, one dimensional swell potential decreased with the addition of wood ash. Reduction in swell potential of soil was associated to the cation exchange reaction in which sodium ions are replaced with calcium ions.
- Based on results of laboratory testing, it can be concluded that wood ash improved the engineering properties of soil in an effective manner.

5.2 RECOMMENDATIONS

Following recommendations are made for further studies:

- Effect of wood ash on engineering characteristics of highly plastic and collapsible soils should be studied.
- Shear strength parameters and modulus of resilience of ash stabilized soil can be studied.
- One dimensional swell was taken into consideration for this research work. The overall free swell should also be determined.
- Cost comparison may also be studied to determine the economic effects of stabilizing the problematic soil using wood ash.

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