

**EVALUATION OF METAKAOLIN-BASED GEOPOLYMER  
AS A STABILIZING AGENT FOR EXPANSIVE SOIL**

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**MASTER OF SCIENCE IN  
MATERIALS SCIENCE AND ENGINEERING**

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## **CERTIFICATION**

This is to certify that the thesis titled “Evaluation of Metakaolin-Based Geopolymer As A Stabilizing Agent for Expansive Soil” submitted to the school of postgraduate studies, African University of Science and Technology (AUST), Abuja, Nigeria for the award of the Master’s degree is a record of original research carried out by Ahmed A. Dukuly in the Department of Materials Science and Engineering.

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AS A STABILIZING AGENT FOR EXPANSIVE SOIL**

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## DEDICATION

To my late Mother "Mrs. Mariam Dukuly"

You so much desired to see me educated, and you went through  
a lot to see me successful. I owe you so much than I can say.

## ACKNOWLEDGEMENT

The success of this worthy undertaking was by Allah's grace and guidance. All praise and thanks, therefore, be to Allah, the Custodian of the universe for His enormous blessing and endowment bestowed upon me to commence and accomplish this all important task.

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## ABSTRACT

The prevailing method currently used in treating and stabilizing expansive soil is chemical stabilization. The two materials that are widely used in this regard are lime and cement. They have proven over the years to be very effective in improving the geotechnical properties of soils. Nevertheless, they have high sustainability index, long term durability problem and high susceptibility to forming an expansive mineral -ettringite- in a sulfate rich soil, which necessitate their replacement with an alternative material that is more sustainable and less susceptible to ettringite formation.

A potential material that stands the chance of replacing these conventional stabilizers is geopolymer, a solid and stable aluminosilicate material formed by alkali hydroxide or alkali silicate activation of metakaolin or fly-ash. Metakaolin-based geopolymer is, therefore, being evaluated by this research as an alternative stabilizing agent for expansive soils.

In so doing, a metakaolin-based geopolymer was produced and utilized to treat an expansive soil obtained from Abuja, Nigeria. Subsequently, the geopolymer treated soil was tested for basic geotechnical and engineering properties, and the results were compared with the results obtained from testing of the control soil. The geopolymer was found very effective in enhancing the geotechnical properties of the control soil. Dramatic increase was observed in the unconfined compressive strength, CBR and maximum dry density, while drastic reduction was observed in the optimum moisture content, plasticity index, swell and shrinkage potentials of the control soil upon the addition of 15% geopolymer content.

In order to evaluate geopolymer as an alternative to lime in soil treatment and stabilization, the soil was treated with lime and tested for geotechnical properties, and the results obtained were compared to those of geopolymer. Generally, the results revealed that geopolymer is an adequate substitute to lime for soil treatment and stabilization.

It is therefore hoped that future research will appreciate the findings of this work in the execution of pavement design as a more sustainable alternative to conventional soil stabilizers.

**KEYWORDS:** Metakaolin, geopolymer, expansive soil, geotechnical properties, soil stabilization, California Bearing Ratio, compressive strength.

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# CHAPTER 1

## INTRODUCTION

### 1.1. Background to the Problem

Expansive soils are highly problematic by virtue of their innate capacity to undergo volumetric changes corresponding to changes in the moisture regime. Expansive soils are soils that swell when they absorb water and shrink when they dry out. These periodic changes in volume lead to cracks into structures that are built on them. Expansive soils exist everywhere in the world, and every year their damages sum up to billions of dollars. The consequence is even more severe in countries where the rainy season is immediately followed by the dry season. These alternating rainy and dry seasons result in repeated cycles of swelling and shrinking movements, which induce additional stresses in the infrastructure for which it was not designed, damage heighten with time. In the world, more than a billion dollars has been spent for the restoration of damages for infrastructure built on expansive soils. For several decays, concerted efforts have been made to address this issue by stabilizing expansive soils to reduce their moisture content and thereby reducing the effect of their swelling and shrinking.

The stabilization techniques that have been utilized over the years to treat expansive soils include physical, mechanical and chemical stabilizations. The most efficient and reliable technique among the techniques that have been employed over the years to treat expansive soils is the chemical stabilization method, more especially the calcium-based stabilizers (Chakraborty & Nair, 2018, 2020; Petry & Little, 2003; Puppala, Congress, & Banerjee, 2019). Conventionally, chemical stabilization methods use cementitious materials such as lime and cement to enhance unconfined compressive strength and stiffness properties of expansive soils. Although these calcium-based stabilizers are widely used, but their major problem is that they tend to form mineral Ettringite in sulfate-rich soils, which consequently promote excessive soil swelling that ultimately result into infrastructure failure. Moreover, conventional soil stabilizers

do not offer sustainable and eco-friendly treatments, as their production has been flagged as major contributors of carbon dioxide emissions worldwide (Zhang et al., 2013). As such, there is a need for soil stabilizers that are sustainable and durable as well as capable to significantly improve engineering properties of expansive soils. Non-traditional stabilizers are therefore being investigated as improved alternatives for soil stabilization. The use of non-traditional stabilizers is usually prompted when traditional calcium-based stabilizers are unavailable or lacking in quality. While most non-traditional stabilizers are usually classified as either polymeric, ionic, or enzymatic in nature, many can be classified as recycled waste materials that are industry by-products. As a result, many non-traditional stabilizers are inherently more eco-friendly in nature, although they are not widely accepted for soil stabilization procedures. The reluctance to accept non-traditional stabilizers for mainstream work can be attributed to the stabilizers being too novel in most places and the lack of comprehensive, peer-reviewed research on their ability to enhance multiple engineering properties of soils. A relatively new class of materials known as 'geopolymers' have been hailed as sustainable alternatives to Ordinary Portland Cement (OPC) and Lime additives (Joseph Davidovits, 2020; John L. Provis & van Deventer, 2009). Geopolymers can be synthesized from by-products of existing industrial processes and are known to have high compressive strength, low shrinkage, as well as heat and fire-resistant properties. Geopolymers have a much lower carbon footprint than lime and OPC (Gartner, 2004), and they are therefore being considered as an alternative to conventional soil stabilizers. While much information is available on the synthesis of geopolymers for use in the ceramics and resin industries, very few studies have investigated their viability as soil stabilizers. Additionally, the few studies that do explore the ability of geopolymers to stabilize soils mostly tend to focus only on the enhancement of the unconfined compressive strength of soil on geopolymer treatment, or do not consider their ability to stabilize expansive soils (Cristelo et al., 2012; Du et al., 2017; Phummiphan et al., 2017). This study makes an attempt to address some of the limitations in this regard. In this study, geopolymers are synthesized in-house and applied

in different dosages as the sole stabilizer to expansive soils found in Nigeria. In addition to considering enhancement of unconfined compressive strength of soils, the effect of geopolymers on volume-change, stiffness, CBR and long-term performance characteristics of expansive soils are also investigated. Additionally, geopolymer-treated soils will be compared to lime-treated soils, to evaluate their comparative effects on expansive soils.

## 1.2. Statement of the Problem

The underlying foundation soil of most of our infrastructures is the expansive soil. Expansive soil expands when absorbs water and shrinks when dries up. This periodic volumetric changes in the soil induces additional stresses in the infrastructures that are built on it, which consequently with time results into cracks. This has imposed huge financial burden on the world to restore damages to infrastructures as the result of volumetric changes in the soil. Every year, over a billion of dollars is spent to maintain and rehabilitate damaged structures. To address this issue, efforts have been made over the years to enhance the properties of the soil against this effect by reducing the moisture content of the soil. Among those efforts is the stabilization of the soil, and the most efficient and effective stabilization mechanism currently in place is the stabilization of the soil with cement. Cement is largely made of gypsum, clinker and clay. Hence, their production requires a temperature of approximately 1500<sup>o</sup>C, which is energy intensive. Moreover, the process also to a larger extend releases lot of greenhouse gases into the atmosphere as the result of the decarbonation of calcium during the coking process. According to McCaffrey (2002), the production of one ton of Portland cement generates and releases approximately one ton of carbon dioxide gas into the atmosphere. another drawback associated with OPC is that, when it is mixed with sulfate-rich soil, it forms a highly expansive mineral called Ettringite which causes excessive swelling and shrinkage of the soil. Due to this high energy consumption and carbon footprint, Portland cement has therefore raised serious environmental concerns that necessitate it replacement with an environmental-friendly material.

For these reasons, the need to transit from OPC to viable and sustainable alternative binders which has less CO<sub>2</sub> emission into the atmosphere is imperative.

Researchers, through several studies, have found supplementary cementitious materials such as fly ash and other pozzolans satisfactory in the making of green binders. Preferably of these is the inorganic alumino-silicate polymer-based material synthesized predominantly from silicon (Si) and aluminum (Al) materials of geological origin call geopolymers (Joseph Davidovits, 2020). The geopolymer is activated with alkaline liquids from alkali metals (sodium or potassium based) to produce the Cementitious properties like Portland cement. In this regard, the two major constituents of the geopolymer binders are the source materials (Fly ash, Rice-husk ash, Silica fume, Slag, Red mud etc) and the alkali liquids (Sodium hydroxide with sodium silicate, potassium hydroxide with potassium silicate), Rangan, 2008. It has been discovered by many researchers that geopolymer technology requires less consumption of energy, and it reduces the emission of carbon dioxide by 90% compared to ordinary Portland cement; Hence, geopolymer is the most viable alternative to remedy the ecological hazard posed by OPC.

### 1.3. Aim and Objectives

The main aim of the study was to synthesize a metakaolin-based geopolymer and experimentally evaluate its viability as an effective stabilizing agent for expansive soils. To achieve this aim, below are listed the specific objectives the study will follow:

1. synthesize metakaolin-based geopolymer
2. Use the metakaolin-based geopolymer to treat the expansive soils at different mixes and determine the geotechnical properties and microstructural characterization test on geopolymer-treated soils.
3. Use a conventional stabilizer like lime to treat the expansive soils and compare the improvement in the soil properties due to geopolymer treatment and lime treatment.

4. Investigate the results to assess the viability of geopolymers as an effective soil stabilizing agent and determine appropriate mix of the geopolymer to be employed for the enhancement of the engineering properties of the soils.
5. The evaluation of the potential use of metakaolin-based geopolymer as a stabilizer of expansive clays, in contrast to lime.

## CHAPTER 2

### LITERATURE REVIEW

#### 2.1. Introduction

Most of our infrastructure, if not all, are placed on the ground that is expected to give them adequate support. However, many of the sites where these constructions or installations are carried out consist of soils that are expansive, weak, or very compressible -deficient of the desirable strength required to support the loads that are placed on them (Zhang et al., 2013). Expansive soils are soils that undergo volumetric changes as the moisture content changes. (Adem & Vanapalli, 2015; Julina & Thyagaraj, 2019). When they absorbed water, they swell, and when they lose the water, they shrink. These periodic swelling and shrinking of the soils that underlie our infrastructures cause severe damage to the structures that are built on them, making them (expansive soils) potential hazard to existing and prospective infrastructure developments due to their susceptibility to volumetric changes with changes in their moisture content. This imposes recurrent expenditure on nation government or infrastructure funding agencies. About billions of dollars are set aside every year for the reconstruction and rehabilitation of these infrastructures (Jones and Holtz 1973, (Atahu et al., 2019). Hence, soils of such nature are improved by stabilization of the soil, where the engineering properties of the soil are boosted by utilizing materials that have cementitious property and/or have binder materials (Makusa et al. 2012). Effort to devise methods for the stabilization of these hazardous soils to cut down construction and maintenance cost has long been in the making. The stabilization of expansive soil has been carried out for so many years by different procedures including the utilization of chemical additive (Tayabji et al. 1982). Aside from enhancing the mechanical properties of problematic soils, high efficiency has been demonstrated by conventional stabilizers, for example, lime and cement, in enervating the expanding and shrinkage capability of extensive soils (Prusinski & Bhattacharja, 1999). Nevertheless, lime and

cement production require high energy consumption which consequently impact our atmosphere adversely due to the carbon dioxide footprint it leaves behind (Garcia-Lodeiro et al., 2015). Furthermore, calcium-based stabilizers in sulfate-rich soils lead to the development of the mineral known as Ettringite which causes excessive heaving and volumetric changes (Katz et al., 2001; Puppala et al., 2005; Zhang et al., 2015). Moreover, conventional stabilizers are additionally restricted in terms of longevity because they are consistently wash off by water as they percolate into the soil, which influence, in the long run, the way they work. Available documents are, therefore, reviewed in this chapter to understand the properties and nature of expansive soils, their stabilization mechanisms and the intricacy involved in their stabilization. Section 1 discusses expansive soils and its relationship with clay mineralogy. Section 2 explains on conventional soil stabilization mechanisms. Section 3 and 4 present geopolymers and past studies carried out on treating soils utilizing geopolymers.

## 2.2. Overview of Expansive soils

Regular variations in moisture content makes expansive soils entirely prone to regular expansion and contraction. Expansive soils are pervasive everywhere in the world, especially in zones with semi-arid to arid atmospheres where the yearly evapo-transpiration rates are more than the yearly precipitation (Banerjee & Puppala, 2015; Puppala et al., 2013; Puppala, Ruttanaporamakul, et al., 2019; Talluri et al., 2013). This volumetric change characteristic makes expansive soils very much unreliable and inappropriate for buildings. Regardless of their inaptitude for construction, expansive soils can not be avoided because of forceful urbanization and dramatic population increase (Williams, 2003) or perhaps their abundance occurrence. Damages caused by expansive soil to infrastructures also adversely affect sophisticated nations. Extensive soils related destructions have been accounted for by numerous nations, for example, Australia, Canada, China, India, Israel, South Africa, and the US (Ramana, 1993; Saride et al., 2010).

There are three key factors that influence the swelling and shrinking capability of expansive clays; they include soil characteristics, environmental impacts and state of stresses (Ramana, 1993). Clay mineralogy, soil-water chemistry, soil suction, plasticity, soil structure and fabric, permeability, as well as dry density are soil characteristics that affect the swell/shrinkage potential of soil; while moisture variations, climate, groundwater, drainage, vegetation, temperature, loading and stress history are environmental impacts that affect the swell/shrinkage potential of soil (Ramana, 1993).

Among the soil characteristics essential for the establishment of the soil swell/shrinkage potential is the clay mineralogy. The following minerals have been isolated as classes of clay mineral: Vermiculite, Chlorite, Kaolinite, Illite, Halloysite and Smectite, where kaolinite, Illite and smectite are the most frequently occurring ones (BAILEY et al., 1979).

The predominant mineral found in smectite is the montmorillonite, the mineral that is largely responsible for clay soil expansive nature (Mitchell & Soga, 2005) .

Consequently, smectite and montmorillonite are often used interchangeably. Among the three main clay minerals, montmorillonite has the smallest basal spacing (9.6 Å), and its molecules are bonded together by van der Waals forces.

The shrinkage/swelling potential of clay minerals largely depends on the disparities in their specific electrical fields. The shrinkage/swelling potential of clay minerals is directly proportional to electrical fields. Clay minerals most often contained negative charge (-) because of their inherent physical-chemical properties, the characteristic that make them to have high affinity for dipolar molecules of water and cations of salt such as sodium (Na), calcium (Ca), potassium (K) and magnesium (Mg) to keep the charges neutral.

Clay minerals are usually hydrophilic in nature. Their particles usually store large amount of water through cations hydration and clay crystal adsorptive forces. Consequently, double-layer water is always formed around the clay minerals that leads to alteration in volume (Ramana, 1993). The negatively charged clay particle with the solution at its interface that has the

exchangeable cations is known as the diffuse double layer (DDL) (Bohn et al. 1985). Special repulsive force, called micro-scale swelling pressures, are engendered by the overlying DDLs between clay particles. Hence, the swelling potential of soils upsurge as the thickness of their DDLs increases (Ramana, 1993).

The amount of exchangeable cations essential to neutralize the negative charge on the clay particle surface in the soil is referred to as the cation exchange capacity (CEC), and it is expressed in terms of milliequivalents (meq) per gram of dry clay. The CEC is usually determined in the laboratory by substituting the adsorbed cations which are substituted by saturating the exchange sites with a known chemical species; the known cations that is needed to saturate the exchange sites can therefore be computed (Nelson and Miller, 1992). The clay mineral and its CEC are very connected. Hence, CEC is an excellent determinant of the soil swell potential. CEC is directly proportional to the soil swell potential. I.e. higher CEC implies higher swell potential.

Table 2.1 presents some common clay minerals and their characteristic CEC values, where Montmorillonite of huge swelling potential is shown to have the largest value of CEC among other clay minerals.

Table 2. 1: Common clay minerals and their characteristic CEC values (Nelson and Miller, 1992).

<b>Clay mineral</b>	<b>CEC (meq/100g)</b>
Kaolinite	3-15
Illite	30-40
Montmorillonite	80-150

Atterberg limits and density are also other properties that meaningfully impact the expansive soils swelling potential. Atterberg limits furnish information on the plasticity of soils. Some researchers have categorized soil swelling potential based on their plasticity index (PI). Table 2.2 shows the different categories of swelling potential that have been made by different researchers. Many researchers hold the view that a soil can have high potential for swell when

the PI exceed 20. It is important to note that this not a generally accepted consensus among the researchers (Seed et al. 1962, Terzaghi and Peck 1967, Raman 1967).

Table 2. 2: Categories of swell potential according to PI

<b>Swelling Potential</b>	<b>Seed et al., 1962</b>	<b>Terzaghi &amp; Peck, 1967</b>	<b>Raman, 1967</b>	<b>Nelson &amp; Miller, 1992</b>
Low	0-10	0-15	0-12	0-18
Medium	10-20	10-35	12-23	15-28
High	20-35	20-55	23-32	25-41
Very High	> 35	> 55	>32	>35

### 2.3. Conventional Soil Stabilization Techniques

Expansive soils are among the major contributing factors responsible for the damages of our infrastructures. Its impacts ranges from foundation failure, door jamming, slope failure, pipeline cracking, pavement distress and failure, etc. (Congress et al., 2019; Jafari et al., 2019; Puppala, Congress, & Banerjee, 2019; Puppala, Ruttanaporamakul, et al., 2019). These effects are more severe around the surface of the ground where confining stresses are easily surmountable by swell pressure that is applied by expansive soils. (Punthutaecha et al., 2006). Soil treatment and stabilization are the surest way of alleviating the adverse impacts of expansive soils.

Stabilization of expansive soils are frequently carried out by means of moisture-regulation methods, ponding or prewetting, and other mechanical or chemical stabilization methods. In a moisture-control method, the already existing water content of the soil is maintained, and changes in water content is managed to prevent swelling of the soil. Though it is tremendously hard to avoid variations in water content of soil, it is likely to regulate its rate of variation to curtail fluctuation during the seasons in a long run (Nelson and Miller, 1992). Moisture barriers or drains are frequently utilized for this purpose. The efficiency of this method is not impressive in expansive soils, because of their low permeability and high soil suction, which require more

times for moisture change regulation. In ponding or prewetting method, possible heaves are induced into the soil before the commencement of construction. Like moisture-control method, this method is not applicable in expansive soils. They only work perfectly in soil with high permeability.

### 2.3.1. Mechanical Stabilization

Mechanical stabilization is the process of improving the soil mechanically by utilizing materials that do not change the intrinsic properties of the soil (Ingold, 1994). Some common methods of mechanical stabilization of soils include deep densification, precompression, and reinforcement. Deep densification is a mechanical stabilization technique used to enhance the soil mass density by speedily getting rid of the air within the voids of the soil. Reducing the pore size of the soil, greatly restrict permeability, improved the shear strength of the soil and mitigate settlement. Densification of soil can be achieved by either dynamic compaction or vibro-compaction (Kirsch and Bell 2012).

Densification by dynamic compaction employs high energy impact in densifying loose fine grain soils. The adequacy of the density attained depends on the tamper weight, drop height, and the number of drops. Densification through this technique drastically decrease the pore size and immensely enhance the engineering properties, such as bearing capacity and strength, of the soil. The efficiency of dynamic compaction is mostly restricted to soils that are cohesionless, and it is impractical for clayey soils (Nicholson, 2014). Densification by vibro-compaction method utilizes techniques that involve vibration to reduce void ratio, which help in improving consolidation and thus a rise in shear strength. This technique also enhances the bearing capacity of the soil, mitigate settlement and moderates liquefaction potential (Nicholson, 2014; Banerjee et al., 2018). Because it enables appropriate drainage of the soil, vibro-compaction could be utilized for the stabilization of expansive soils, but it has shown to be more efficient in cohesionless soils (Nicholson, 2014). Moreover, heaving of expansive soils can be mitigated by low-density compaction; however, it must be ensured that the soil has enough strength at the

low density. It is worth noting that the efficiency of dynamic compaction and vibro-compaction are particularly restricted to only cohesionless soils; however, particle rearrangement and interlocking by compaction are very hard to achieve in cohesive soils. Reinforcement method of soil stabilization is achieved utilizing geosynthetic materials, such as geotextiles, geogrids, and Geocells, for varieties of structures (el Sawwaf, 2007; N. Biswas & Ghosh, 2018; Nripojyoti Biswas & Ghosh, 2019). Reinforcements have been employed to reinforce pavements, slopes, earth-retaining walls, etc.

### 2.3.2. Chemical Stabilization

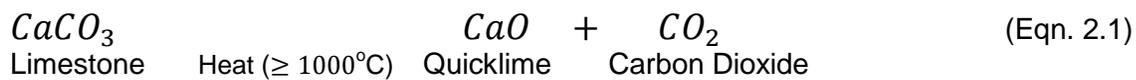
Chemical stabilization is the process employed to improve the engineering properties of a soil by the addition of chemical additive to the soil, that alters the physico-chemical properties of the soil. For the past many decades, chemical stabilization has been the most common and efficient method of stabilization of expansive soils (Firoozi et al., 2017; Madhyannapu & Puppala, 2014; Puppala, 2016). The kind of stabilizer to be employed depends greatly on the type of soil to be treated, its plasticity, clay content, as well as its mineralogy. Strength and stiffness, such as unconfined compressive strength, and resilient modulus, of soils are greatly enhanced by chemical stabilization; it is also instrumental in the reduction of swelling potential of the soil (Khoury et al., 2013; Puppala et al., 2011, 2003). The two main classifications of chemical stabilizers are traditional and non-traditional stabilizers,

#### 2.3.2.1. Traditional Stabilizers

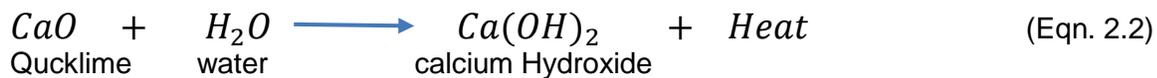
The most utilized chemical stabilizers of soils conventionally are lime, cement, and fly-ash. The means of stabilization for these additives depends on the exchange of cation, and pozzolanic reactions between the additive and the soil minerals. Traditional stabilizers are the most widely used additive, because they are efficient and provide relatively low cost solution for the treatment of expansive soils.

##### 2.3.2.1.1. Lime

Since prehistoric days, lime has been utilized by the Romans, the Chinese and the Indians for the purpose of construction. Lime can be efficiently applied in expansive soils stabilization. The two categories of lime that are utilized for stabilizing expansive soils are calcium oxide (CaO) commonly referred to as quicklime and calcium hydroxide (Ca(OH)<sub>2</sub>) commonly referred to as hydrated lime. Quicklime is made by the calcination of high-quality limestone at very high temperatures at about 1000°C which releases the carbon dioxide in it by the process called vaporization as shown in equation 2.1:



Hydrated lime is therefore produced by slaking quicklime with water. The process of hydration is presented in equation 2.2:



The reactions are reversible process. I.e., Both calcium oxide (quicklime) and calcium hydroxide (hydrated lime) undergo reaction with carbon dioxide to produce calcium carbonate (limestone). (Boynton 1980). Quicklime is extremely hydrophilic, so it has the tendency of absorbing moisture from the atmosphere enabling a process that leads to hydrated lime formation, whose solubility in water is slightly higher than quicklime.

Expansive soil treatment with lime is extensively employed worldwide as a major soil stabilization technique, because it is highly efficient and relatively inexpensive.

Lime treatment of soils involves a process that is carried out in two-fold. The first step involves the exchange of cations between the lime and the clay mineral. This exchange of cations results into the formation of flocs and agglomerates that alter the plasticity and texture of the soils (Little, 1995). The process through which flocs or agglomerates are formed through the reorganization of clay particles is known as Flocculation or agglomeration. Soil mineralogy and

soil-water chemistry severely impact the Flocculation or agglomeration process. These flocs and agglomerates make the soils crumbly, workable and compactable. The second step involves a pozzolanic process that binds the flocs and agglomerates of the clay particles. This step results into the improvement of the strength of the soil. Pozzolans are basically fine detached siliceous or aluminous materials that undergo reaction with hydrated lime (calcium hydroxide) and water that result into robust cementitious compounds formation soils (Little, 1995). The rich silicate and aluminate minerals in the clay and the cementitious compounds formed represent the pozzolan phases and the calcium-silicate-hydrate (CSH) or calcium-aluminate-hydrate (CAH) respectively.

The movement of hydrated lime through water in the soil to react with the silicate -aluminate rich mineral in the clay soil is the basic means by which pozzolanic process occur (P. Duxson et al., 2007). The key factors that determine the rate of pozzolanic process are the reactivity and solubility of the pozzolans and the PH of the soil. Hence, reaction completion time is not definitive. The duration of pozzolanic processes could last for months or even years, base on the PH of the soil and how reactive and soluble the pozzolan is.

The properties of the soil are enhanced in so many ways when expansive soil is treated with lime. The treatment drastically reduces swell potential and plasticity index and at the same time increases shear strength, unconfined compressive strength (UCS), California Bearing Ratio (CBR), durability, workability, etc. (Little, 1995; Puppala et al., 1996). However, lime is very impractical in granular and sulfate-rich soil treatments. When lime is used to treat sulfate-rich soils, it results into extreme heaving and distress in the soil (Puppala, Congress, Talluri, et al., 2019). Lime also has the tendency of being washout by water as they percolate into the soil, which lead to long-term performance problem. Furthermore, lime production adversely impacts our environment. It is an energy-consuming process which emits tens of million metric tons of greenhouse gases annually.and therefore has severe negative impact on the environment.

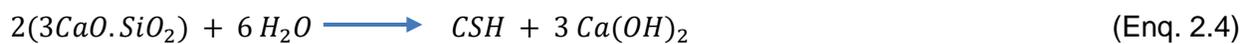
#### 2.3.2.1.2 Cement

Another common binder comprising of cementitious materials that is utilized in the stabilization of expansive soils is the Ordinary Portland Cement (OPC) frequently called ‘cement’. The three main raw materials that are involved in the production of Cement are limestone, clay, and gypsum. The lime (CaO) is furnished by the limestone, while clay is generally the source of silica (SiO<sub>2</sub>), alumina (Al<sub>2</sub>O<sub>3</sub>) and iron (Fe<sub>2</sub>O<sub>3</sub>). There are three main stages involved in the production of cement: 1) the proportioning and mixing of limestone and clay, 2) calcination of limestone and clay to form cementitious materials called ‘clinker’, and 3) the crushing of clinker follows by the adding of retarding agent that is known as gypsum. The complex chemical reactions that take place to form the variety of cementitious compounds which form cement can be simplified primarily as shown in equation 2.3:



For many decades now, cement has been utilized effectively for the treatment and stabilization of soils. Cement treatment consists of the mixing of the proper proportion of cement with water and soil. Like lime, the process of stabilizing soil with cement consists of the exchange of cations between the lime and the clay mineral, that result into the formation of flocs and agglomerates that alter the plasticity and texture of the soil; and the cementitious hydration and pozzolanic reactions that cement the flocs and agglomerates of the clay particles.

The main phases of oxides cement consist of are dicalcium silicate (2CaO.SiO<sub>2</sub>), tricalcium silicate (3CaO.SiO<sub>2</sub>), tricalcium aluminate (3CaO. Al<sub>2</sub>O<sub>3</sub>) and tetracalcium aluminoferrite (4CaO.Al<sub>2</sub>O<sub>3</sub>. Fe<sub>2</sub>O<sub>3</sub>). During the process of hydration, the two former phases release calcium hydroxide and calcium-silicate-hydrate introducing the stabilization process (Prusinski and Bhattacharja 1999). Calcium hydroxide and calcium-silicate-hydrate are formed through the hydration reactions shown in equations 2.4 and 2.5 below:



Cement treatment of soils increases the unconfined compressive strength (UCS), shear strength, and California Bearing Ratio (CBR) as well as reduces the plasticity index and the swell potential of soils (Sharma & Sivapullaiah, 2016). Cement treatment has shown to be more efficient and an inexpensive solution for granular soil stabilization. Like lime, the production of cement is an energy-intensive process that adversely affect the climate. Moreover, it is inclined to ettringite formation, cracking and brittle failures; and tend to wash way as water tries to percolate through the soil.

#### 2.3.2.1.3. Fly ash

Fly ash is a grey particulate-like material that are pozzolanic in nature due to their silica and alumina glass content; it is produced as a result of burning powdered coal (Mateos & Davidson, 1962). Unlike cement and lime that are synthetic products, fly ash is the by-product of coal-production industry. Consequently, the chemical make-up of fly ash can differ significantly based on the source and the synthesizing process. The two broad classifications of fly ash according to AASHTO and ASTM are Class C (self-cementing) fly ash and Class F (non-self-cementing) fly ash. The latter needs an activating agent like lime or cement, because it contains small lime content, while the former is capable of performing as a sole binder, because it contains sufficient lime quantity.

Fly ash enhances the UCS and CBR characteristics as well as lessens the plasticity index of the treated soils (Zulkifley et al. 2014)

Fly ash stabilization and treatment of soils is a two-step process. The initial step involves the process that flocculates particles of soil to ensure their workability. And the second step is the pozzolanic processes that produce cementitious products enabling improvement in the engineering properties of the soil. The chemical make-up, the type of soil, the quantity of fly ash applied to the soil, the moisture content and temperature of the mixture are factors that immensely influence the efficiency and reaction rate of the fly ash (Usmen & Bowders Jr, 1990).

Problems associated with fly ash range from variation in reaction and setting times, air-quality contamination, contraction of respiratory diseases by laborers and the unlikelihood of being an efficient binder on its own in reasonable quantity.

#### 2.3.2.2. Non-traditional Stabilizers

Expansive soil stabilization is carried out worldwide using traditional stabilizers because of its efficiency. However, non-traditional stabilizers have been utilized in other instances at what time traditional stabilizers are unavailable, or their quality are unacceptable. Moreover, the issues of high energy consumption, high cost, sulfate attach on them in sulfate-rich environment and greenhouse gases emission associated with them are root causes of the desire to finding alternative materials for soil stabilization. Majority of the non-traditional stabilizers consist of by-products of production industries or wastes of other materials for which there are need for proper and efficient disposal mechanisms. Some examples of non-traditional stabilizers are granulated blast furnace slag, enzymes, polymers, mine tailings, sulfonated oils, resins, kiln dust, salts, rubber tires, rice husk and fibers (Caballero et al., 2016; Diniz et al., 2017; Kumar & Singh, 2008). Addition of non-traditional stabilizers to balancing additives or activators have proven to be effective in soil stabilization, although not as satisfactorily as traditional stabilizers (Sharma & Sivapullaiah, 2016; Tingle et al., 2007). Though research in this area is gaining serious attention and momentum, there are not too many available literatures, and most of the available literatures concentrate mainly on performance instead of the mechanism of stabilization. This evaluation of performance is found to be insufficient as many of them concentrate entirely on UCS and swell potential, whereas many of the other engineering characteristics are left unexamined.

### 2.4. Introduction to Geopolymers

Geopolymers are a comparatively novel type of cementitious materials that depend on alkali-aluminosilicate reactions, and they have characteristics that can be compared to OPC but with a relatively lesser energy consumption and lesser carbon footprint. Geopolymers are prepared by the activation of aluminosilicate-rich materials with alkali metals, and they comprise of huge three-dimensional (3-D) networks of alumino-silicates that are bonded covalently. Geopolymers are noted for low shrinkage, high compressive strength and durability properties (P. Duxson et al., 2007). These alkalis activated products can be prepared from a comparatively economical aluminosilicate precursors (clay, metakaolin, fly ash, and others) (J. Davidovits, 1991; van Jaarsveld et al., 2002; Gordon et al., 2005), solidify at ambient temperatures in a relatively short period of time (Lizcano, Gonzalez, et al., 2012; Lizcano, Kim, et al., 2012), and are thus well-thought-out to be a more sustainable and environmentally-friendly alternative to conventional stabilizer.

#### 2.4.1. Geopolymer History

A German researcher by the name of Kuhl was the first in 1908 to patent the preparation of alkali-activated materials (AAMs) relative to Portland cement when he mixed an alkali source (alkali sulfate or carbonate) with an alumino-silicate precursor (vitreous slag) (John L Provis & van Deventer, 2014). This basic idea was further improved upon by Purdon. Varieties of blast furnace slags activated with calcium hydroxide ( $\text{Ca(OH)}_2$ ) and sodium hydroxide (NaOH) solutions were tried by him to prepare materials with tensile, flexural and final strengths relative to that of Portland cements (Purdon, 1940). It was later discovered by Glukhovsky in 1950 that binder products activated with alkali could be formed utilizing low-calcium or calcium-free aluminosilicate (clay) (Krivenko, 2017). His discovery is regarded as the first synthesis registered about geopolymers.

In the 1980s, Joseph Davidovits, a French material scientist prepared geopolymer binders by activating naturally occurring materials such as kaolinite, dolomite and limestone with alkali (J.

Davidovits, 1991). Upon patenting many aluminosilicate materials and marketing them as fire-resistant resins, Davidovits has drawn significant attention to geopolymer. From that time till now, geopolymers and their practices have been employed in many fields such as material science, chemistry, mineralogy, and engineering applications, for variety of usages like thermal insulation, fire-resistant materials, corrosion-resistant coatings, containment of radioactive materials and adhesives, cements, composites as well as concretes for infrastructure applications (J. Davidovits, 1991; P. Duxson et al., 2007; J. Temuujin et al., 2015; Jadambaa Temuujin et al., 2011; van Jaarsveld et al., 1999).

#### 2.4.2. Terminology of Geopolymer

Geopolymers can be commonly defined as a solid and firm aluminosilicate material synthesized by utilizing alkali hydroxide or alkali silicate to activate a precursor that is most often provided in solid powder form (Joseph Davidovits, 2020). They are gels that are very firm, and they can be prepared at room temperature and pressure conditions to generate near-net dimension bodies, then transformed to materials that are crystalline or amorphous (Bell et al., 2009).

The products discussed here as 'geopolymers' have also been labeled by researchers as 'mineral polymers', 'inorganic polymer glasses', 'inorganic polymers', 'alkali-bonded ceramics', 'soil cements', 'alkali ash material', 'hydroceramics', and a different of other designations (Joseph Davidovits, 2020). This variety of names for the same products have left researchers who are unfamiliar with this domain unaware of essential research on the topic. An instance of this is the earlier work done by Rahier and colleagues (Rahier et al. 1996a, 1996b, 1997), who described the material as 'inorganic polymer glass' instead of 'geopolymer,'. Hence, their papers have been less referenced than their worth and relevance deserve.

It was not until in the 1980s when Davidovits, a French material scientist coined the term 'geopolymer'. Since then, the term has become ubiquitous in the academic research field.

Geopolymers are a subclass of a group of materials referred to as AAMs. AAMs are prepared by the activation of an aluminosilicate precursor with an alkaline activator, and they have properties relative to those of a traditional stabilizer like cement; hence, geopolymers are principally AAM binders produced utilizing less or no calcium, and they are frequently prepared using metakaolin or fly ash as the aluminosilicate precursor (John L Provis & van Deventer, 2014).

Figure 2.1 portrays a generally recognized classification of AAMs by van Deventer, where the region that is darker is indicative of larger alkaline (Na and/or K) concentrations. Figure 2.1 also relates calcium sulfo-aluminate cements and Portland-based cements with AAMs on the basis of their aluminum and calcium contents.

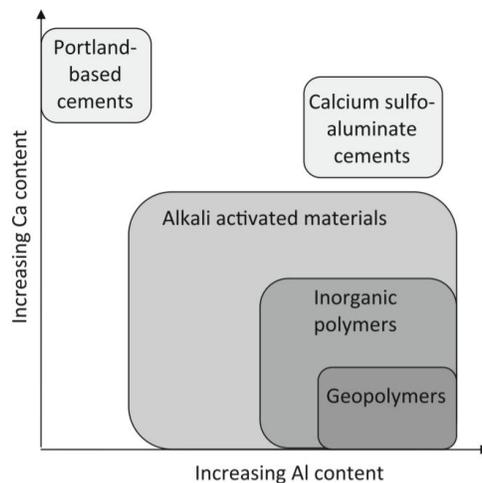


Figure 2. 1: General classification of AAMs (van Deventer et al., 2010).

### 2.4.3. Geopolymer Structure

The term sialate is normally used to describe the structure of aluminosilicate (Joseph Davidovits, 1982). The bond type Si-O-Al ('silicon-oxo-aluminate') is called the sialate bond, while the bond type Si-O-Si is called the siloxo bond. This made it very easy to describe the configuration of geopolymers based on their Si/Al ratio, with a ratio of 1 being a poly(sialate), 2.0 being a poly(sialate-siloxo), and 3.0 a poly(sialate-disiloxo). These oligomeric units are presented in figure 2.3.

Polysialates are described chain or ring polymers where  $\text{Si}^{4+}$  and  $\text{Al}^{3+}$  are present in four-fold coordination confined by one more of the linked oxygens in every aluminate tetrahedron and balanced by the cations furnished by the alkali metal of the activating solution (see Figure 2.3), and they range from amorphous to semi-crystalline in nature (J. Davidovits, 1991). Geopolymers can be expressed by the empirical formula of polysialates as presented in Equation 2.6.

$$M_n [-(\text{SiO}_2)_z - \text{AlO}_2]_n \cdot w\text{H}_2\text{O} \quad (\text{Eqn. 2.6})$$

Where,  $M$  is the alkali metal cation (such as Na, K, or Ca),  $z$  is the silicon to aluminum (Si:Al) ratio (normally 1, 2, or 3),  $n$  is the degree of polycondensation and  $w$  is the the amount of molar water.

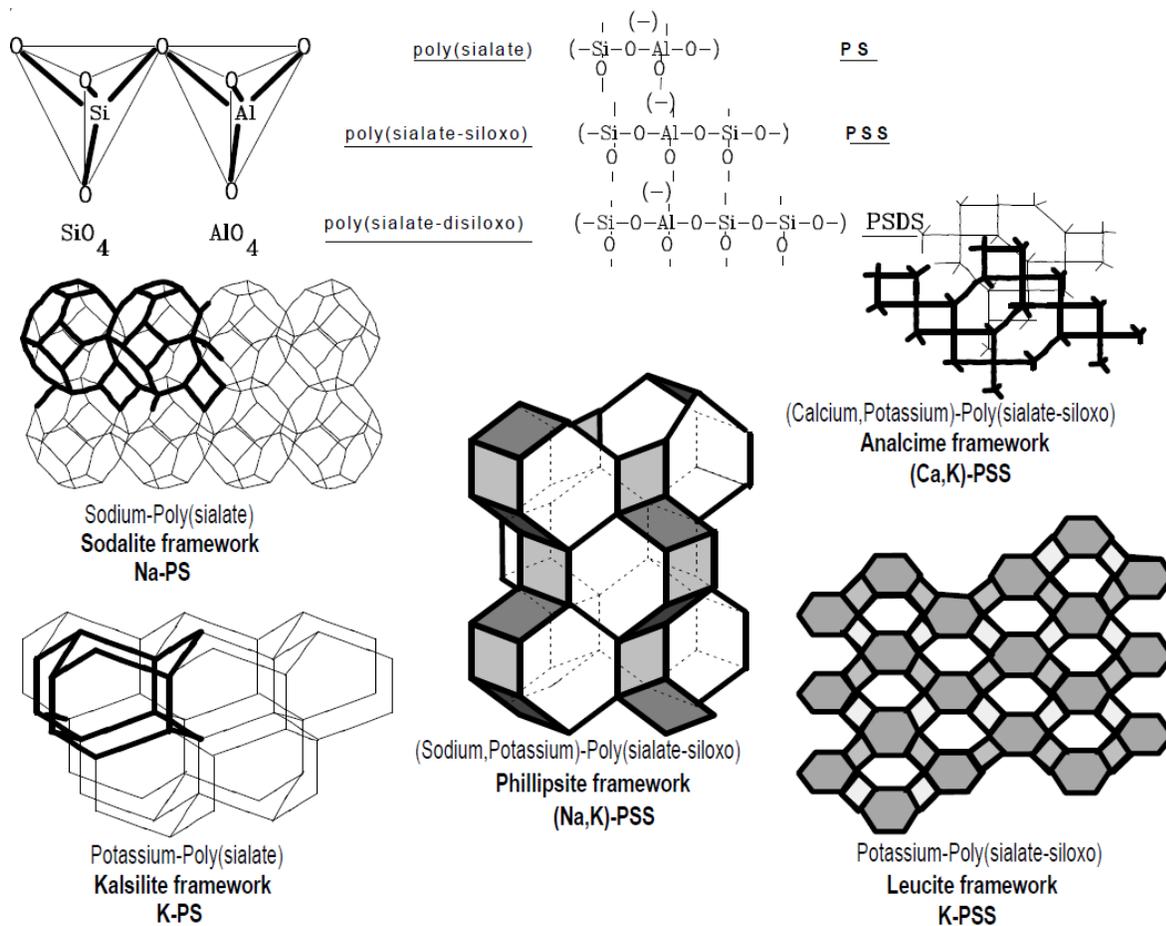


Figure 2. 2: Molecular framework of geopolymers and their associated crystalline structures (J. Davidovits, 1991).

The particles of many geopolymers have a non-crystalline arrangement and an X-ray amorphous structure. Because of this phenomenon, the X-ray diffraction (XRD) of geopolymers has a 'broad diffuse halo' shape as opposed to the normal anticipated sharp peaks of diffraction (J. Davidovits, 1991). The X-ray diffraction of geopolymers with their typical broad diffuse halo is portrayed in figure 2.3. The structure of geopolymers heavily relies on the curing temperature. Geopolymers cured at the temperature above 80°C form crystalline structure (Barbosa & MacKenzie, 2003), while geopolymers cured at or below 80°C normally become amorphous (J. Davidovits, 1991; Kriven et al., 2004), but form crystalline phases at elevated temperatures (Barbosa & MacKenzie, 2003). According to (John L. Provis & van Deventer, 2009) geopolymers cured at elevated temperature have structure comparable to the minerals called zeolitic at atomic scales.

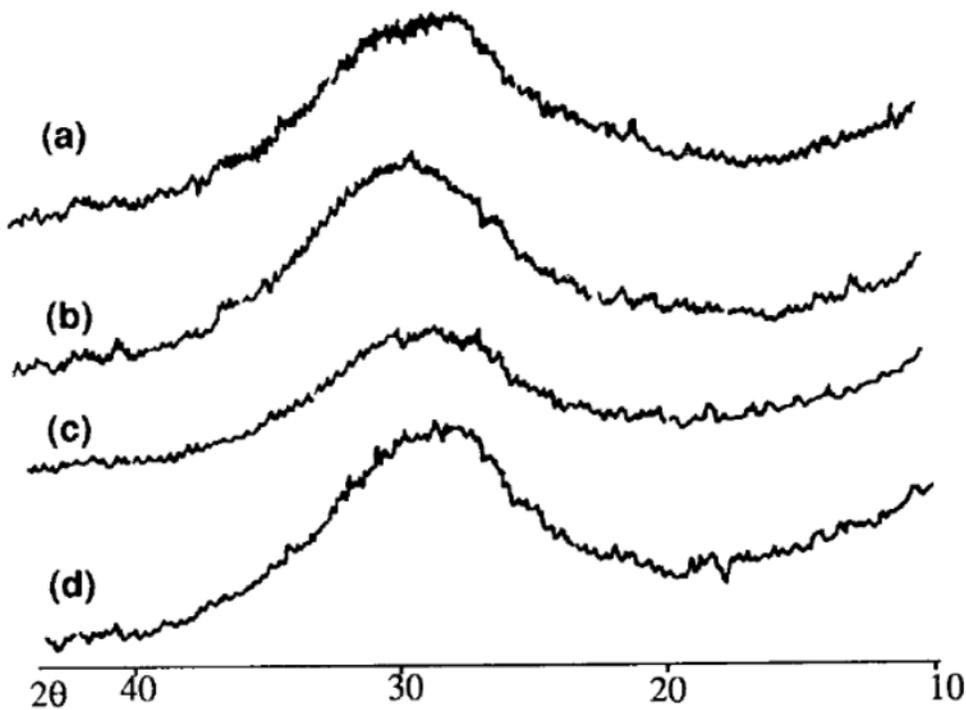


Figure 2. 3: X-ray diffractogram of two types of [Na,K]-PSS (a, b), and [K]-PSS (c, d) (Davidovits 1988).

With the exception of the typical crystalline formations that are illustrated in Figure 2.2, which are obviously the ultimate structures of finished crystallization processes at higher

temperatures, the curing of most geopolymers is done at very low temperatures, and they are thus non-crystalline in structure and are essentially a lengthy framework of aluminosilicate chain or ring polymers as portrayed below in Figure 2.4 (J. Davidovits, 1991). Within an incompletely formed ring or chain structures, free aluminate and silicate groups of a geopolymer that has not undergone complete reaction with its components will finally polycondense to complete the links.

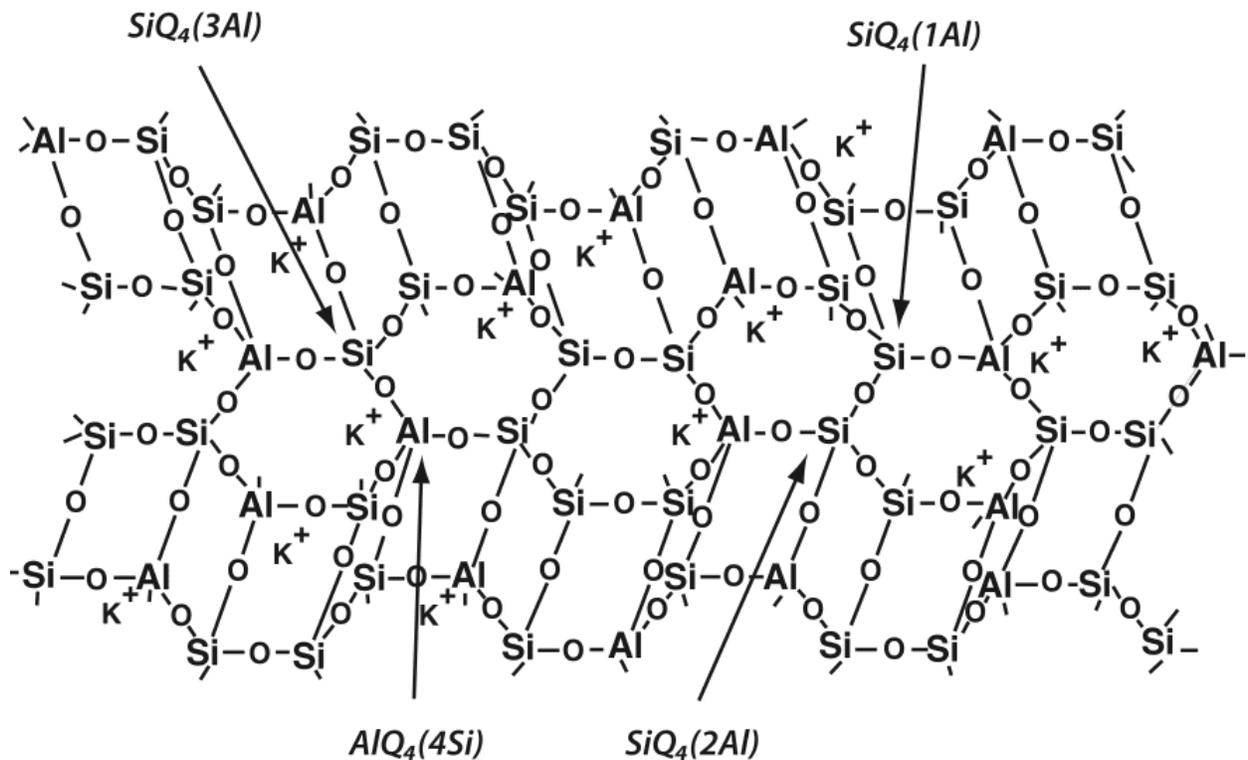


Figure 2. 4: Theoretical 3-D structural model of a fully reacted Potassium-based geopolymer (J. Davidovits, 1991).

#### 2.4.4. Conceptual Process of Geopolymerization

The geopolymerization process is basically a polycondensation process of alkaline activated materials. Polycondensation is the term use to describe the formation of polymers where molecules join losing small molecule as byproducts such as water and methanol (Bhat and Kandagor 2014). The term was mentioned by Glukhovsky in the 1950s when he set forth a

general process of geopolymerization which he summarized in a three-step processes such as destruction–coagulation, coagulation–condensation and condensation–crystallization. Researchers have over the years endeavored to describe the process of geopolymerization more expansively in view of various advancements in technology that have allowed a better understanding into the process (J. Davidovits, 1991; John L. Provis & van Deventer, 2009). Figure 2.5 portrays a model that summaries conceptual process of geopolymerization, and describes key procedures involved. The steps that the process of Geopolymerization involves are dissolution, speciation equilibrium, gelation, reorganization and polymerization and hardening (P. Duxson et al., 2007; Medri et al., 2010). All these steps usually happen in chorus, not consecutively as they may seem to appear.

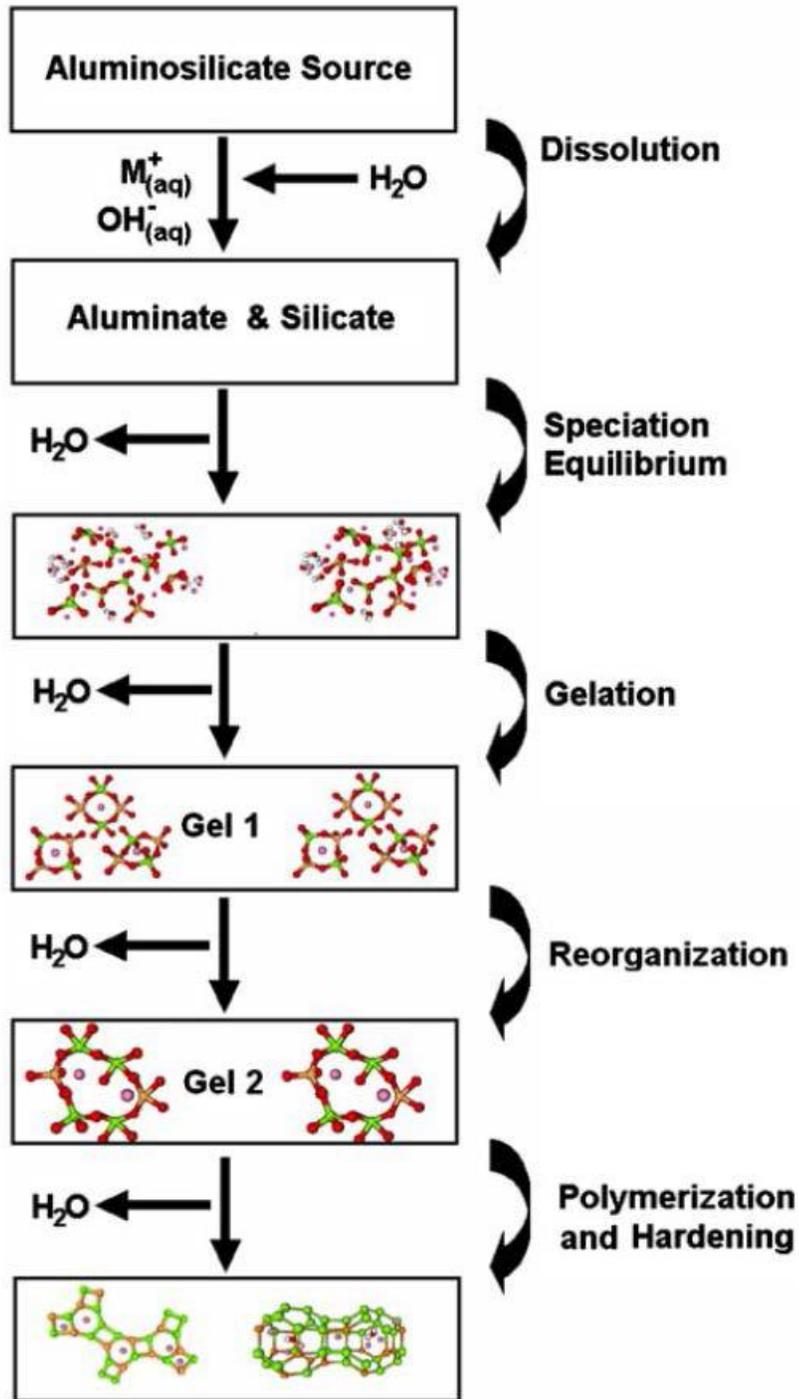


Figure 2. 5: Diagram of geopolymerization process (P. Duxson et al., 2007).

The combination of the aluminosilicate precursor and the alkaline activating solution initiate the process of dissolution. As the precursor and the alkaline solution interacts, the vitreous/amorphous component of the aluminosilicate precursor rapidly dissolve in the solution

and release aluminate ( $\text{Al}(\text{OH})_4^-$ ) and silicate ( $\text{Si}(\text{OH})_4$ ) monomers through the process of hydrolysis. The interaction of the monomers with each other results into the formation of new species such as dimers, trimers, tetramers, etc. Dimers react with monomers to form trimers, and the trimers react with monomers to form tetramers and on, on and on. As the solution becomes saturated, which indicates the end of the speciation process, aluminosilicate gel starts to form. The gel formed initially (designated in the diagram as Gel 1) will usually consist of more aluminate concentration than the silicate concentration since reactive aluminum dissolve more quickly than reactive silicon because the bonds of the Al-O bond are feebler than the Si-O bond. However, as the reaction continues, more silicates begin to dissolve into the solution, and the solution starts to contain more silicon concentration, thereby leading to the formation of gel 2. These structural reorganisation processes determine the final composition of the polymer as well as pore microstructure and distribution in the material, which are critical factors in the development of many physical properties of the resulting cement (Fernández-Jiménez et al., 2006).

#### 2.4.5. Geopolymer Synthesis and Influencing Factors

There are three key materials needed for the synthesis of geopolymers: 1) a rich source of aluminosilicate, 2) an alkali-metal cation source and 3) water. The synthesis mechanism involves two main steps: 1) the preparation of the alkaline activator solution and 2) the preparation of the geopolymer slurry. The first step involves the combination of the alkali metal cation source and water in their appropriate proportions to prepare the alkaline activator solution, while the second step is the mixing of the alkaline activator solution and the aluminosilicate precursor to form the geopolymer slurry. These are followed by curing the slurry at the required temperature, which is one of the parameters that really affects the quality of the geopolymer produced. Davidovits first used the word geopolymer to describe the alkali hydroxide/silicate activated metakaolin (calcined clay) (John L. Provis & van Deventer, 2009).

Other raw materials, in addition to metakaolin, that are mostly used for geopolymers preparation are fly ashes and slags (P. Duxson, 2009). Factors that influence the rate of gelation and solidification processes of geopolymer are the mix design, curing environment, and the amount of contaminants in the mix. The quantity of alkaline activator, the amount of alumina and water content as well as the purity of the aluminosilicate precursor are important factors that affect the properties of the geopolymer (Buchwald et al., 2007; Fletcher et al., 2005; Lizcano, Gonzalez, et al., 2012; Rowles & O'Connor, 2003).

#### 2.4.5.1 Aluminosilicate Precursor

Among the variety of sources of aluminosilicate mentioned earlier, metakaolin seems to be the most ideal one, because they are undoubtedly the most reproducible, pure and high reactive aluminosilicate precursor for geopolymer. It is a pozzolanic material that is made through the calcination of kaolinite at temperatures ranging from 500°C to 800°C, making their production less energy-intensive, and thus contribute less carbon footprint. This makes them more environmentally friendly relative to cement and lime (Liew et al., 2016). Factors that must be considered during the synthesis of geopolymer are the purity, particle size and crystallinity of the kaolinite for they determine the variation in the quality of the metakaolin. Metakaolin based geopolymer attains higher polymerization relative to aluminosilicate precursors from other sources, especially when cured in a closed environment at higher temperatures (John L. Provis & van Deventer, 2009). However, a rather negative feature is that the water demand is higher for metakaolin-based geopolymers due to its flat particle shape and very large surface areas, which is a major drawback for their large-scale production. In addition, due to its high alkali content, metakaolin-based geopolymers are quick to effloresce (John L. Provis & van Deventer, 2009). Figure 2.6 presents the image of the Scanning Electron Microscopy (SEM) of metakaolin-based geopolymer, providing a perception into its microstructure.

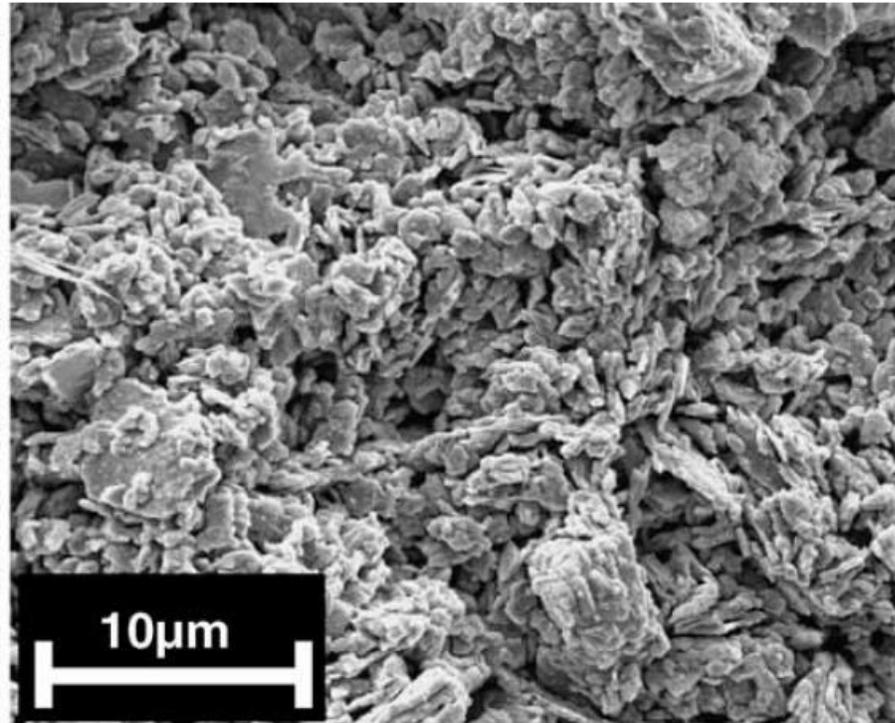


Figure 2. 6: SEM image of metakaolin-based geopolymer (Peter Duxson et al., 2005).

Another widely used aluminosilicate precursor for the synthesis of geopolymer is fly ash. Fly ashes are relatively abundant and ubiquitous and inexpensive precursor, but its composition varies highly from one source to another and even on a everyday basis at a given source. Fly ashes properties are, therefore, based on heavily on the class of coal it is attained from, as well as the process of production. Hence, the reactivity and the concentration of aluminosilicate of fly ashes differ immensely from each other depending on the sources they were obtained.

This variation in composition from one source to another makes the proportioning of mix design employing fly ashes a complex process. Overall, the rate at which alumina is released from fly ashes is much slower compared to metakaolin, and it is highly dependent on the type and amount of alkali present (Fernández-Jiménez & Palomo, 2003; van Jaarsveld et al., 1999). Studies are still being carried out to link the concepts of geopolymer chemistry and the dissolution of ash regardless of these drawbacks of fly ash. The SEM image of a fly ash-based geopolymer is shown in figure 2.7.

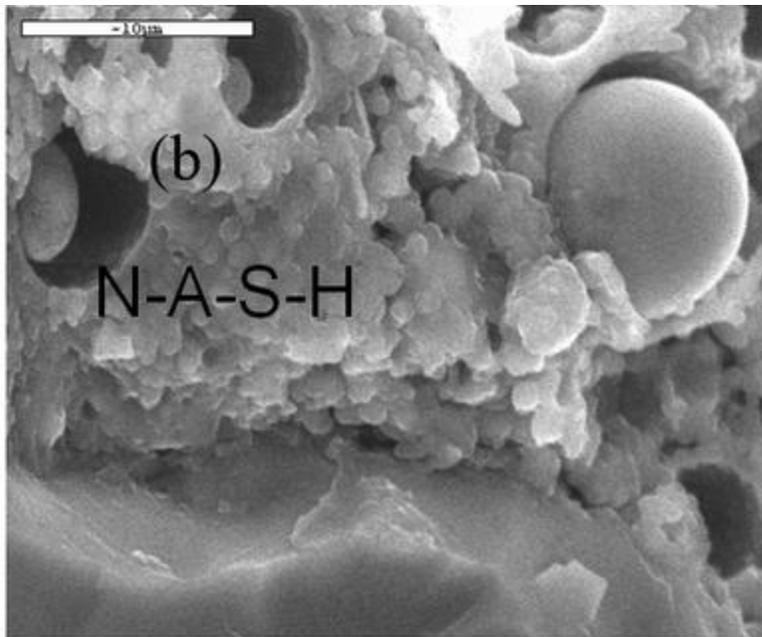


Figure 2. 7: SEM image of fly ash-based geopolymer (Garcia-Lodeiro et al., 2015)

#### 2.4.5.2. Alkali Metal Cation

Three-dimensional (3D) polymerization is fully achieved when the geopolymer is properly cured, and there are little or no non-bridging oxygen in the tetrahedral framework. To ensure that the bridging oxygens in the tetrahedral framework are fully bonded, the negative charges must be balanced. Alkali metal cations, denoted as M, are therefore very crucial in charge-balancing each tetrahedral site of aluminum in the geopolymer structure. Na<sup>+</sup> and K<sup>+</sup> ions are the most frequently used cations. Other cations like rubidium (Rb<sup>+</sup>) and cesium (Cs<sup>+</sup>) are also used, but their frequency is not as compare to Na<sup>+</sup> and K<sup>+</sup> ions. The geopolymerization rate, reactivity and porosity of the geopolymer systems are largely depended on the kind and the amount of cations present. The more excess alkali in the solution, non-bridging oxygen sites connected to some of the framework sites will be present. This subsequently leads to reduction in the rate of geopolymerization, due to aluminosilicate precursor slow rate of dissolution (Steins et al., 2012). The alkali cation source is combined with water and additional silica source (if required) to prepare the solution that is applied to activate the aluminosilicate precursor.

#### 2.4.5.3. Si:Al ratio

Si/Al ratio and the nature of the alkali cations present govern the precise details of the local structure of geopolymer to a substantial degree. It is arguably one of the most important factors that determine geopolymer features. The development of polysialate, the smallest unit of geopolymer framework, depends on this ratio (Joseph Davidovits, 2020; Lizcano, Gonzalez, et al., 2012; Phair & van Deventer, 2002; Xu & van Deventer, 2000). Si:Al ratio is directly proportional to the compressive strength as a result of the more complex systems of polysialates produced during polycondensation (Peter Duxson et al., 2005). But this does not mean that the ratio must be increased indefinitely. Previous studies have shown that significant increase in geopolymer strength properties is observed when Si:Al ratio ranges from 1 to 3 (J. Davidovits, 1991).

#### 2.4.5.4. Water

The water content of geopolymer framework highly affects on the overall properties such as workability, strength, density, durability, and microstructure characteristics of the final hardened geopolymer (Lizcano, Gonzalez, et al., 2012; John L. Provis & van Deventer, 2009). Excess water leads to the formation of large pore volume in the geopolymer chains. As the excess water evaporates during the process of gelation and curing, it leaves behind those pore voids in the geopolymer framework, that causes decrease in density and increase in porosity. Consequently, the compressive strength and the thermal conductivity of geopolymer reduce correspondingly (Lizcano, Gonzalez, et al., 2012). Moreover, excess water in the structure also causes reduction in pH of the aqueous solution, which result into deficiency of alkaline necessary to maximize the rate of dissolution.

#### 2.4.5.5. Alumina Content

Another significant factor that determines geopolymer properties such as strength, microstructure, acid resistance, strength development profile and rate of curing is the alumina

content of the system (Fernández-Jiménez et al., 2006; Weng et al., 2005). The rate of geopolymerization and its final curing time can be decided by the rate at which alumina is given up from the different aluminosilicate precursors (Rees et al., 2008). Moreover, the alkali metal cation and aluminum ratio (M/Al) is very crucial in the geopolymerization process. For all the oxygen in the tetrahedral framework to be fully bonded, the M/Al molar ratio should be 1. (Barbosa et al., 2000) suggested the M:Al of 1:1 as the stoichiometric balance for optimum geopolymer formation. The positive  $Al^{3+}$  charge ion is stabilized by the  $K^+$  or  $Na^+$  ions slackly associated with the aluminosilicate framework (J. L. Provis et al., 2005). An increase in M:Al ratio is assumed to thwart the complete formation of geopolymer chains (Lizcano, Gonzalez, et al., 2012; John L. Provis & van Deventer, 2009).

#### 2.4.5.6. Curing Environment

The curing environment has serious impacts on the structural and mechanical characteristics of geopolymers. Early researchers in this domain reported initially that the curing temperatures for geopolymers were around 150°C to 180°C (Joseph Davidovits, 1982), but researchers are currently curing geopolymers at temperature as low 20°C to 90°C (Chindaprasirt et al., 2011; Deb & Sarker, 2017; Peter Duxson et al., 2005; Shadnia & Zhang, 2017). Geopolymers have been successfully cured by researchers at ambient temperatures (20-23°C) making the synthesis process less energy-intensive and simpler; geopolymers cured at lower temperatures have been reported to have low strength and slower setting times at early stages (John L. Provis & Bernal, 2014). Increasing the curing temperature, according to literature, shows corresponding increase in the compressive strength and reduction in the rate of setting up to a temperature of about (85-95°C), beyond which no addition in strength is observed (Shadnia & Zhang, 2017). The impact of elevated temperatures on diverse geopolymers have also been investigated, and it has been observed that crystalline formation starts to take place at around a temperature of 120°C (P. Duxson et al., 2007). Specimens have also been cured in both sealed

and unsealed environments where relative humidity was varied from 40-90% to assess the impact of humidity on the curing of geopolymer.

## 2.5. Geopolymer stabilized soil

Several types of geopolymers have been synthesized over the year for a different usage in engineering and built industries, such as high-tech resins, low-tech building materials, composites for infrastructure repair and reinforcement, fire and acid-resistant materials, and 'green' cements and concrete. Global concern and the wish for more sustainable construction materials have given geopolymer the edge to serve as possible substitute for Portland cement since many studies over the years have shown that the both have comparative properties. Geopolymer has been suggested by most of these studies in the recent time as a perfect substitute material for the treatment of expansive soil. It started gaining attention in 2010, when studies on the effectiveness of geopolymers for soil treatment and stabilization started. Hence, not many literatures are available on this subject. Table 2.3 below furnish a collection of the recent existing literatures on the treatment and stabilization of soils using geopolymer, which clearly indicates that most studies in this area were conducted within the recent years. Since then, different kinds of geopolymers have been produced to treat different kinds of soils. For example, fly ash-based geopolymers have been applied to silty and clayed sand to improve its strength (Cristelo et al., 2012; Dungca & Codilla, 2018; Rios et al., 2016), in addition to some low-plasticity soil (Liu et al., 2016), and high-plasticity clays (Phetchuay et al., 2014). Very few researchers have also employed metakaolin-based geopolymer to enhance the swell and strength of a artificial lean clay (Zhang et al. 2013, 2015) and a high plasticity clay (Khadka et al., 2018). Geopolymers have also been produced by some researchers by blending fly ash and calcium carbide residue (CCR) (Phetchuay et al., 2016; Phummiphan et al., 2017) or ground-

granulated blast-furnace slag (GGBFS) (Abdullah et al., 2017; Mohammadinia et al., 2016) to improve the properties of soils.

Palm-oil fuel ash (POFA) (Pourakbar et al., 2016), GGBFS (Du et al., 2017) and volcanic ash (VA) (Miao et al., 2017) are other precursor materials that have been employed solely for the formulation of geopolymer for soil treatment and stabilization. Some researchers have successfully proved the capability of geopolymers in improving soil properties such as the UCS, and only in few cases the shrinkage, swell and California Bearing Ratio (CBR). While geopolymer has been applied so well in enhancing the UCS of treated soils, other engineering properties such as durability, CBR and volume-change (shrinkage and swell) properties still need to be further investigated. To better understand the process of geopolymer stabilization, some researchers also, in addition to the engineering property tests, perform tests such as microstructural investigations through mineralogical and elemental characterizations using either SEM, XRD, XRF or EDS.

<b>Author(s) &amp; Publication Year</b>	<b>Soil Type (USCS)</b>	<b>Geopolymer Precursor</b>	<b>Engineering Properties Tested</b>
Cristelo et al. (2012)	SM	Class-F FA	UCS
Zhang et al. (2013)	CL	MK	UCS, Shrinkage
Phetchuay et al. (2014)	CH	FA	UCS, LL, PL
Zhang et al. (2015)	CL	MK	UCS, 1-D Swell
Du et al. (2016)	CL	GGBFS	UCS, Hydraulic Conductivity
Liu et al. (2016)	CL	Class-F FA	UCS
Mohammadinia et al. (2016)	GW	Class-F FA + GGBFS	UCS, MR
Phetchuay et al. (2016)	CH	Class-F FA + CCR	UCS
Phummiphan et al. (2016)	SC-SM	Class-C FA + CCR	UCS
Pourakbar et al. (2016)	CH	POFA	UCS, Bearing Capacity (lab load test)
Rios et al. (2016)	SM	Class-F FA	UCS, Durability, S and P-wave velocity
Abdullah et al. (2017)	CH	Class-F FA + GGBFS	UCS, LL, PL
Miao et al. (2017)	CH	VA	UCS, Swell, LL, PL
Dungca & Codilla (2018)	SM	Class-F FA	UCS, CBR
Khadka et al. (2018)	CH	MK	UCS, Swell

LL: Liquid Limit  
PL: Plastic Limit  
USCS: Unified Soil Classification System  
UCS: Unconfined Compressive Strength  
CBR: California Bearing Ratio  
MR: Resilient Modulus  
S and P-wave: Shear and Compression Waves

Table 2. 3: Available literature on geopolymer stabilization of soil

Based on characterization tests that have been conducted by some researchers, it has been observed that soil-geopolymer mix does not produce new mineral, and hence, it has been concluded that no direct chemical reaction takes place between soil and geopolymer (Zhang et al., 2013). The process of geopolymer treatment is physical. The geopolymer gel combines with the soil and causes them to bind together, resulting into enhancement of properties, such as strength and swell and shrinkage, of the soil (Liu et al., 2016).

Potassium hydroxide has not only proven to be the best cations source among all the cation sources, but it has also proven to produce geopolymer products with very high Unconfined Compressive Strength (UCS) (Liu et al., 2016; Pourakbar et al., 2016). K<sup>+</sup> ions agitate water in their closest surrounding so that it is incapable of attracting water molecule. Their hydration sphere is therefore very small which make intervolving polycondensation reactions very easy, and subsequently resulting into geopolymer-treated soil of higher strength (Pourakbar et al., 2016).

Metakaolin-based geopolymer has shown significant improvement in strength, and reduction in swell relative to other precursors used for geopolymer production (Khadka et al., 2018).

For curing temperature, there is no general consensus on that yet. Some specimens of geopolymer-treated soil have been cured at ambient temperature (< 23°C), while others have been treated at elevated temperatures (23°C-60°C) and low relative humidity. But most of these specimens cured at elevated temperatures and under low relative humidity conditions furnished ambiguous results for the acquired strength due to dehydration of the specimens (Cristelo et al., 2012; Phetchuay et al., 2014; Zhang et al., 2013, 2015).

Not much research on geopolymer treated soil has been performed on high plasticity soil (CH); most of them have been, however, focused on silty clay and low plasticity clays (CL). Results from geopolymer treatment and stabilization of soil are encouraging, but lack of consistency among the type of soils, content of raw materials, curing condition and environment, activator, water content, etc. have significantly made it difficult to relate, calculate and replicate identical results.

Hence, it is imperative to carry out additional studies to obtain a deeper insight on the characteristics of geopolymer regarding expansive soils treatment and the optimization of curing methods on geopolymer-treated soils.

## 2.6. Summary

Existing literature on expansive soils and their properties were highlighted and discussed in this chapter. It was shown that the clay mineralogy determines the swelling potential of expansive clays, and that smectite, largely consisting of montmorillonite, is noted for having the most swelling potential. Systematic examination of the different conventional stabilization methods utilized to treat expansive soils and their procedures were carried out in profundity. The efficiency of calcium-based stabilizers in lowering shrinkage and swelling potential of expansive soils was shown to be relatively impressive; but the adverse effects of calcium-based stabilizers on the environment, in addition to their inadequacies in sulfate-rich soils, necessitate their replacement with substitute stabilizers that are more sustainable and environmentally friendly. Afterward, the materials referred to as Geopolymers were presented, where their history, structure and production process were introduced. Lastly, the shortcoming of geopolymers as soil stabilizers were introduced and discussed. It was shown that not many studies have been carried out on expansive soils, and the available ones focused mainly on strength improvement. Several precursor materials that are utilized to produce geopolymers and their combinations in

different proportions were shown to be different in properties depending on the area they were sourced from. This research emphasizes the production of a metakaolin-based geopolymer that is employed to treat expansive soils in different dosages. The metakaolin-based geopolymer is therefore assessed for its viability as a lone stabilizer of expansive soils.

## CHAPTER 3 RESEARCH METHODS

The sole aim of this chapter is to illustrate the synthesis of geopolymers used for this study, and to assess the effectiveness of geopolymer as a stabilizing agent of expansive soil. The geopolymer synthesis section will discuss the raw materials required for the preparation of geopolymer mix, the procedure involved in the preparation of the geopolymer mix, the curing conditions, mix proportioning, comparative analysis of different geopolymer mixes, and the selection process of the final mix for this study. The assessment section will cover the procedure used in the classification of the soil as expansive soils, soil characterization tests, engineering characterization tests, Geopolymer treatment of expansive soils, and Lime treatment of expansive soils. Soil characterization tests include basic, chemical, and microstructural tests, while engineering characterization tests include strength, stiffness, volume change, and long-term performance tests.

It is anticipated that the microstructure test will give an insight on plausible means of stabilization as well as offer a lucid explanation for the macro-behavior of geopolymer-treated soils. Moreover, durability tests conducted on geopolymer-treated soils will aid in establishing the best rates of dosage of geopolymer. The research methodology is detailed in the process flowchart below.

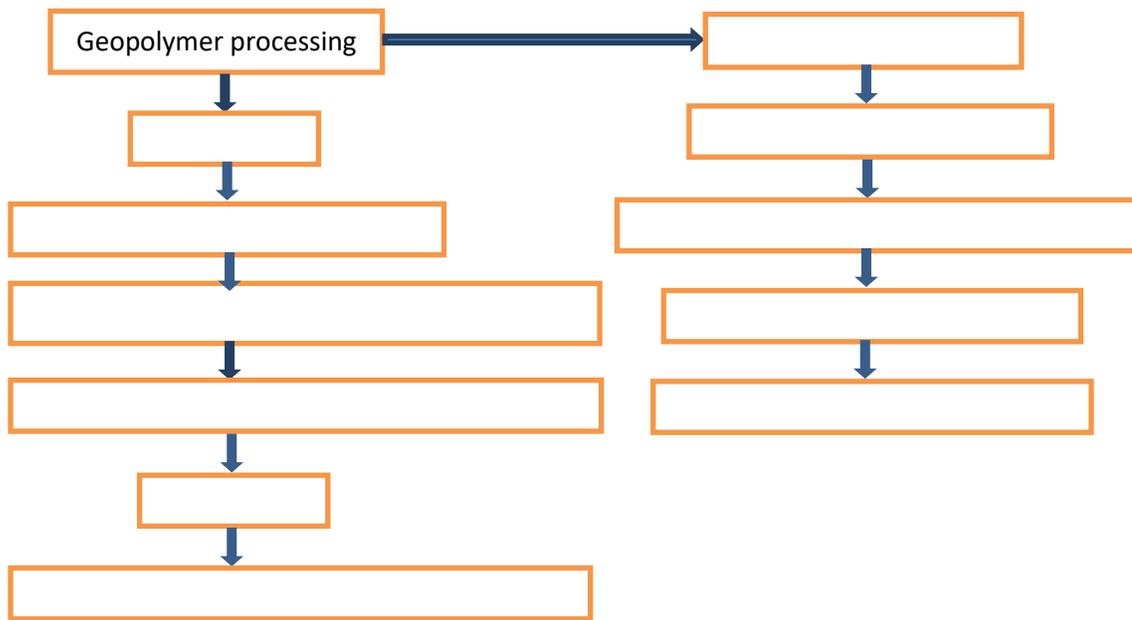


Figure 3. 1: Process Flow Chart

### 3.1. Geopolymer Synthesis

#### 3.1.1. Selection of Raw Materials

Generally, the four main raw materials essential for geopolymer synthesis are: (1) an aluminosilicate precursor, (2) an alkali-metal cation source, (3) additional silica source, and (4) water. While it is true that there are many choices of raw materials available, the decision governing the selection of the right raw materials for the synthesis of geopolymer is squarely based on the intended usage of the geopolymer, availability and cost. The selection of our raw materials was based on these factors.

### 3.1.1.1. Aluminosilicate precursor

Sources of aluminosilicate precursors for geopolymer synthesis are numerous. For this research, the material chosen for the aluminosilicate precursor was white kaolin, because it is relatively abundant and inexpensive. The kaolin, originally sourced from Jose State, Nigeria, was obtained from a Lab technician at NBRRI, a research institute in Abuja, Nigeria.



Figure 3. 2: Kaolin Sample

Kaolin is deemed a perfect precursor as it normally comprises very small quantities of impurities relative to other probable precursors. Kaolin is a highly reactive pozzolanic material which has a good source of reactive alumina and silica needed for geopolymerization. Relative to other frequently utilized precursors such as fly ash, kaolin is recognized to have unimportantly minor amounts of calcium oxide than fly ash, which can usually contain calcium oxide in the range of 10 to 40% (Fan et al., 2015; McManis & Arman, 1989). In order to eschew possibility of further expansion of the soil in the event where the soil might contain some sulfate content, the aluminosilicate precursor used for this study is kaolin.

A SEM characterization was performed on this material to determine its Elemental composition. The result given in Table 3.1 shows the aluminum and silicon content to be relatively higher at 33.12% and 62.47% respectively providing a Si/Al ratio of about 1.89.

Table 3. 1: Elemental Composition of Raw Kaolin Sample

Element	Weight %	Atomic %	Error %	Net Int.	K Ratio	Z	R	A	F
O K	4.41	7.39	86.74	0.56	0.0106	1.0995	0.954	0.2179	1
Al K	33.12	32.94	5.58	37.12	0.3107	0.9808	0.9972	0.9375	1.0202
Si K	62.47	59.67	8.21	46.47	0.4013	1.0027	1.0044	0.6395	1.0018

#### 3.1.1.2. Alkaline metal cation source

The aqueous alkaline activator solution (AAS) is a blend of a soluble base metal cation source, an extra silica source and deionized water. The impact of the alkaline metal cation, Na<sup>+</sup>, was considered for the synthesis of geopolymer. Lab grade NaOH of 98% purity, as appeared in Figure 3.3, was gotten from AUST Lab to make Na-based aqueous AAS.



Figure 3. 3: Sodium hydroxide

#### 3.1.1.3. Additional silica source

The additional silica source used to formulate the AAS used to adjust the Al/Si ratio was a liquid sodium silicate ( $\text{Na}_2\text{Si}_2\text{O}_5 \cdot 5\text{H}_2\text{O}$ ) which was purchased from Home Guard Chemical Industries LTD. (A local chemical vendor) in Abuja, Nigeria. According to the manufacturer declaration, the  $\text{SiO}_2/\text{Na}_2\text{O}$  were mixed in the ratio of 1:2.

#### 3.1.1.4. Water

A dirt free distilled or deionized water was utilized for the making of alkaline solution and geopolymer. The PH of the water was tested and found to be 6.75 using a handheld PH meter shown in figure 3.13.

### 3.1.2. Preparation of Metakaolin and characterization

For the synthesis of geopolymer, the primary precursor material, the metakaolin utilized in this study was prepared and characterized in the laboratory of AUST. In a cole-parmer electric furnace, the metakaolin was processed through the thermal treatment of the raw kaolin material obtained from Jos, plateau state under controlled variables. Samples of the kaolin were first calcined at two different temperatures of 650°C and 700°C for the duration of 2 hours to determine the best calcination temperature that could transform the kaolin from crystalline clay to amorphous metakaolin. The metakaolin sample, as shown in figure 3.2, became pink in color after calcination. The heat-treated samples were analyzed utilizing the EDX. The elemental composition revealed by the EDX showed that the best calcination temperature for the transformation of the kaolin clay from crystalline to amorphous can be achieved at the temperature of 700°C for the duration of 2 hours.

Table 3. 2: The elemental composition of the metakaolin calcined at 700°C for 2 hours

Element	Weight %	Atomic %	Error %	Net Int.	K Ratio	Z	R	A	F
KrL	32.61	36.73	3.35	276.74	0.2964	1.0141	0.9779	0.8816	1.0168
Al K	32.61	36.73	3.35	276.74	0.2964	1.0141	0.9779	0.8816	1.0168
Si K	52.63	56.95	5.84	303.68	0.3353	1.0370	0.9856	0.6120	1.0037
Nb L	9.71	3.17	18.46	18.52	0.0479	0.8028	1.1765	0.6093	1.0084
Tc L	0.92	0.28	59.88	1.94	0.0049	0.7944	1.1861	0.6657	1.0110
K K	3.30	2.57	19.86	13.84	0.0243	0.9641	1.0185	0.7551	1.0109

### 3.1.3. Preparation of Geopolymer

The preparation of geopolymer entailed the mixing of Alkaline activator reagent and metakaolin in their proper proportions. This is followed by curing of the geopolymer slurry at a desire temperature for proper hardening. The processes involved in the preparation of geopolymer is depicted in a diagrammatic form in figure 3.4 below:

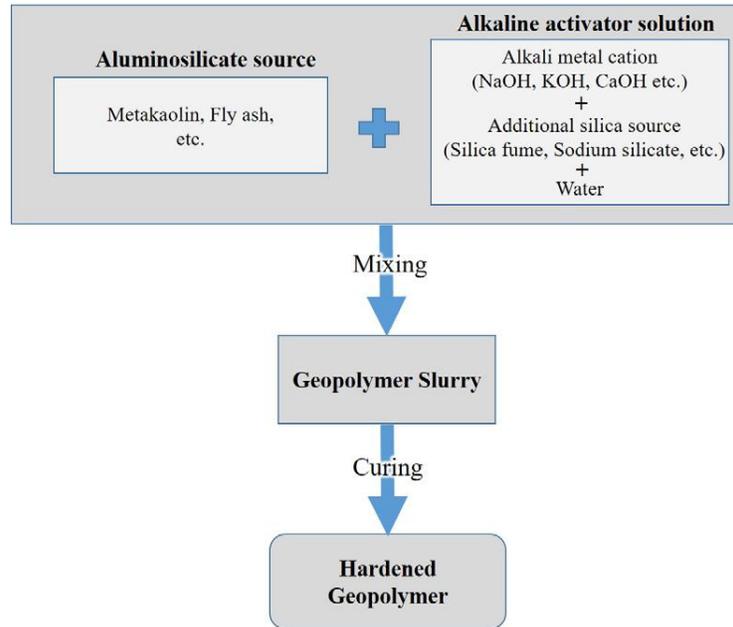


Figure 3. 4: geopolymer synthesis schematic

### 3.1.3.1. Preparation of the Reagent (Alkaline Activator)

A given mass of sodium hydroxide was first dissolved in a given mass of distilled water to form an alkaline solution of 1000ml. The concentration of NaOH varies from 8M to 16M (Hardjito & Rangan, n.d.; M. D.J. Sumajouw, 2006). The NaOH concentration, therefore, used in this study was 10M. The process proved to be exothermic by given up heat, so it was allowed to cool for few hours while being stirred by mechanical stirrer. After cooling, desirable amount of liquid  $\text{Na}_2\text{SiO}_3 \cdot 5\text{H}_2\text{O}$  was added to the alkaline solution in the ratio of NaOH/  $\text{Na}_2\text{SiO}_3 \cdot 5\text{H}_2\text{O}$  of 10 to adjust the Al/Si ratio of the metakaolin. The mixture was stirred for 24 hours by a mechanical stirrer and was stored in a HDPE container.

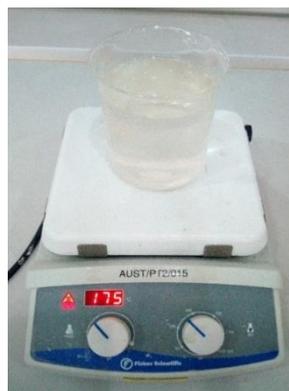


Figure 3. 5: magnetic stirred AAS

### 3.1.3.2. Preparation of the Geopolymer Slurry

The metakaolin was added to the prepared Alkaline Activator reagent and subsequently followed by the addition of water to enhance the workability of the geopolymer mix. The slurry was then thoroughly mixed manually due to the unavailability of high sheared mixer. As stated in the preceding section, all alkaline activator reagents were prepared in keeping with their  $\text{Na}_2\text{O}/\text{SiO}_2$  molar ratio (between 0 and 2), with  $\text{H}_2\text{O}/\text{Na}_2\text{O} = 11$ , and stoichiometrically combined with metakaolin to yield an overall Na/Al ratio of 1. The slurry was poured in a paper mold and allowed to harden for a period of 3 day. After the three days, the specimen was removed from the mold and kept for additional curing days.



Figure 3. 6: Homogeneous GP slurry

### 3.1.3.3. Curing temperature

One of the key parameters that really influence the mechanical strength of a geopolymer is the curing temperature. Higher temperature of curing enhances the mechanical strength of geopolymer (Fernández-Jiménez et al., 2005; Mo et al., 2014; Škvára et al., 2005). However, the specimen, for the purpose of this research, was cured at room temperature (20 to 25°C) and at a relative humidity of 50% or less.



Figure 3. 7: Hardened GP sample

#### 3.1.3.4. Mix design.

Mix design is another key parameter that influences the mechanical strength and long-term performance of geopolymer (Fernández-Jiménez et al., 2005; Mo et al., 2014; Škvára et al., 2005). Geopolymer, therefore, needs to be carefully designed to ensure satisfactory performance in the long run. The geopolymer mix was obtained by varying the alkaline activator reagent and the metakaolin. The alkaline activator reagent was formulated using NaOH/Na<sub>2</sub>SiO<sub>3</sub>.5H<sub>2</sub>O molar ratio of 10. Since the Si/Al ratio is usually fixed in the metakaolin, the alkaline activator reagent was varied to yield an overall ratio of Na/Al ratio of 1. Table 3.3 summarizes the mix as followed.

Table 3. 3: Summary of the Geopolymer mix

Si/Al	H <sub>2</sub> O/NaOH	NaOH/Na <sub>2</sub> SiO <sub>3</sub> .5H <sub>2</sub> O	Na <sub>2</sub> O/Al
1.6	5	10	1

### 3.2. Stabilization of laterite soil

This study seeks to evaluate the efficacy of a metakaolin-based geopolymer as a stabilizing agent of expansive clays. Both lime and metakaolin-based geopolymer were used to treat the expansive soil, and results from the different treatments were compared to determine how they affect the engineering properties of the soils. This determination was done through a series of characterization and geotechnical tests. Different dosages were used for the metakaolin-based geopolymer since there is no established theory for the determination of a fixed dosage like

lime. The experimental methodology for this study is discussed in detail in the following sub-sections:

### 3.2.1. selection process of the expansive soil

The world is extensively covered with expansive soil. As the result of this, it is necessary to focus on finding a solution (stabilization of the expansive soil) to address the negative impact posed by expansive soils. The primary engineering property used to determine the swelling and expansive natures of the soil was the plasticity index (PI), as higher PI specifies higher swelling and expansive nature of soils (see Table 3.2). A typical subgrade soil generally found in most part of Africa was obtained from Abuja, Nigeria to be treated with geopolymer. The soil was obtained through a NBRRI staff. The particle size of the soil was analyzed, and the fraction of the soil retained on 425- $\mu\text{m}$  (No. 40) sieve were tested for Atterberg limits and classified according to the Unified Soil Classification System (USCS). The test result revealed that the soil had a PI of 30%, which can be considered to have a medium swell potential and can be classified as expansive soil in nature. The various tests performed on the soil are detailed below in the proceeding sections:



Figure 3. 8: The control soil

### 3.2.2. Soil & Engineering Characterization Tests

The chemical composition and the geotechnical properties of the soil were investigated through varieties of tests ranging from basic, chemical to microstructural test. These tests are very helpful in comprehending the engineering properties of both the controlled and treated soils.

Chemical and microstructural tests are mostly performed on control (untreated) soil. However, in order to be able to determine any variation in the properties of the soil after treatment with geopolymer, few characterization tests are performed on the geopolymer-treated soil. Nevertheless, the significance of these test, sufficient chemical characterization could not be performed due to the limitation of time and research funding.

The geotechnical tests performed during this study on both the controlled and treated soils included specific gravity, Atterberg limits, gradation analyses, and moisture content – dry density relationship tests, swell test, CBR and Undefined compression strength test (UCS). The methodology of each test is detailed in the upcoming sections.

#### 3.2.2.1. Specific Gravity (Gs) Test

Specific gravity (Gs) is the ratio of the weight of a given volume of a material to the weight of the same volume of water. It is essential in obtaining the weight-volume relationships of soils. The specific gravity of the control soil was determined according to ASTM D854-14. It was conducted on fraction of the soil passing the 4.75 mm (No. 4) sieve using water pycnometer. The value of the specific gravity found for the control soil can be seen in Table 3.2. The determined value was found to be the same as the anticipated value.

#### 3.2.2.2. Atterberg Limit

The Atterberg limits is the collective name for the liquid, plastic and shrinkage limits. They give clear understanding and information about the ability of cohesive soils to shrink and swell. They are also used to determine the plasticity of soils. The level of water content at which the cohesive soil begins to behave like a liquid is known as the liquid limit of the soil. Likewise, the

water level at which the soil makes a transition from a plastic to a semisolid state and from a semisolid state to a solid state are called the plastic limit and the shrinkage limit, respectively.

The soil fraction passing the 425- $\mu\text{m}$  (No. 40) sieve was used to determine the liquid limit (LL) of the soil. The experiment was conducted using the motorized Casagrande apparatus according to the ASTM D4318-17. The plastic limit was measured by rolling a soil sample between the two palms to a 3 mm diameter cylindrical shape. Figure 3.9 shows the apparatus that was used to determine the Atterberg limit of the soil. The result obtained for the Atterberg limits is

summarized in Table 3.1 (properties of soils) were used for the treatment.

and lime-treated properties upon

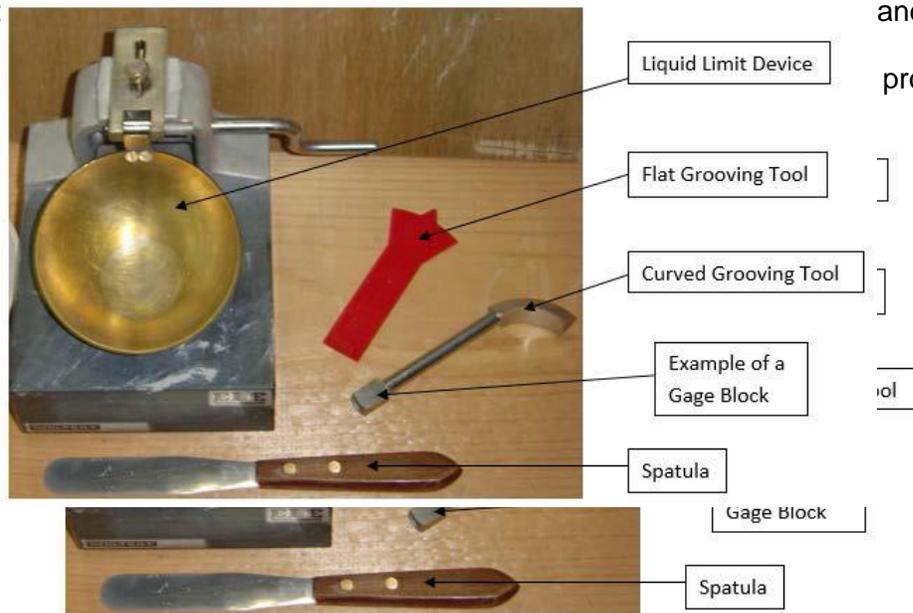


Figure 3. 9: Casagrande apparatus used to determine Atterberg limits.

### 3.2.2.3. Gradation Analysis

Gradation analysis is conducted to classify soils. Sieves are utilized to isolate soil particles and group them according to their different sizes for engineering application. This test is utilized solely for the classification of soil either according to AASHTO or USCS. The classification method used in this study is the latter. The soil was tested according to the ASTM D6913-17 standard procedure laid down. Based on the results obtained from the gradation analysis and



Atterberg limit tests, the soil was found to be a low-plasticity clay (CL), as shown in Table 3.4. The gradation analysis curves of control soils is depicted in Figure 3.11

Figure 3. 10: Sieves utilized for the analysis.

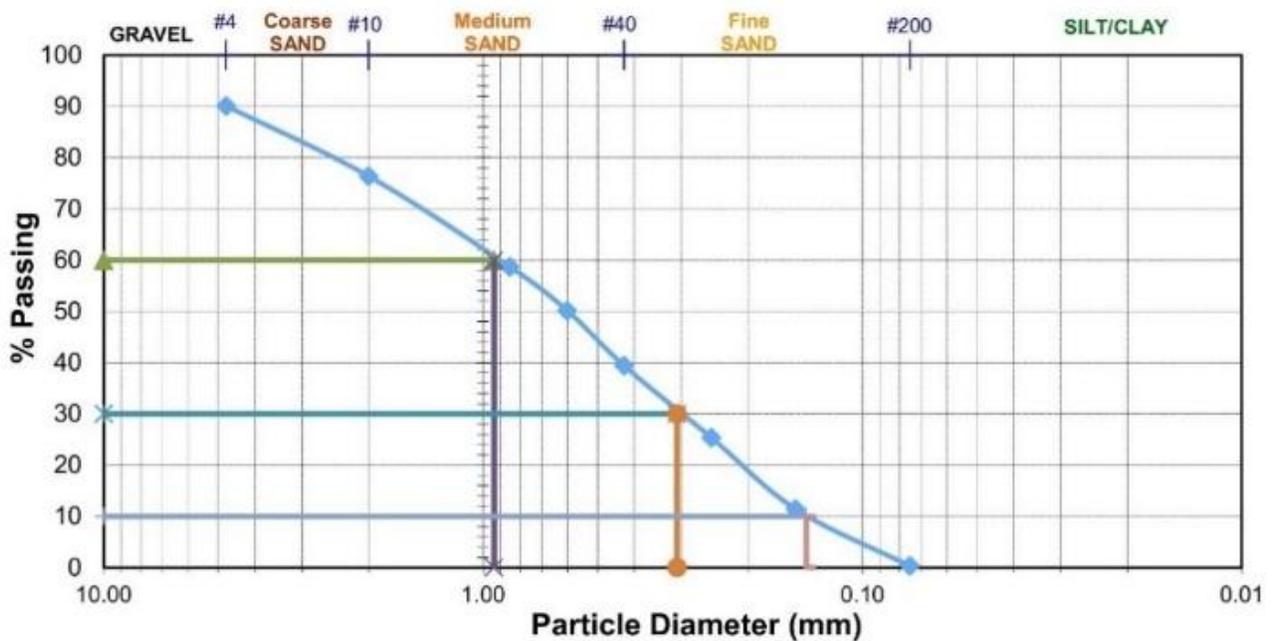


Figure 3. 11: Gradation Analysis of control soils.

#### 3.2.2.4. Moisture Content-Dry Density Relationship Test

Soil placed as a fill (embankment, foundation pads, road bases) is compacted to a dense state to obtain satisfactory engineering properties such as shear strength, compressibility and permeability. In order to achieve proper compaction field, it is imperative to determine the Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) in the laboratory. The

Moisture Content-Dry Density Relationship helps to achieve proper compaction in the field. The Optimum Moisture Content (OMC) and the Maximum Dry Density (MDD) of control soil were determined using a standard compaction test consistent with ASTM D698-12. The apparatus used was a cylindrical mold of diameter 152.4mm and height of 116.4mm. the specimens were placed in three layers, and the number of blows applied to each layer was 56 blows. The same compactive effort was applied to all soil specimens to ensure consistency in the results. The effort of compaction applied to all specimens of soil was 600KN-m/m<sup>3</sup> which is consistent with the standard compaction equipment usually used. Effort of compaction is the energy required to compact a specimen. The effort of compaction (CE) can be determined using Equation 4.1 as

follows: 
$$CE = \frac{\text{Drop height} \times \text{Hammer weight} \times \text{No. of drops} \times \text{No. of layers}}{\text{Volume of compaction mold}} \quad \text{Eqn 3.1.}$$

Results of the compaction tests of the control soils are shown in Table 3.4, and their moisture content – dry density relationship curves are shown in Figure 3.12.

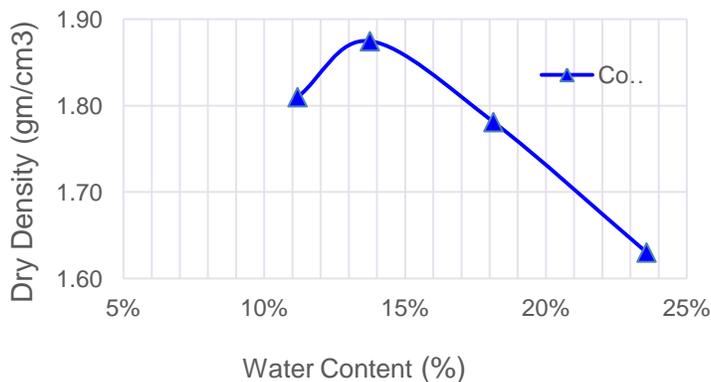


Figure 3. 12: Moisture content – dry density relationship curves of control soils

Table 3. 4: Basic geotechnical properties of control soil determined in this study.

Property	Amount
Passing No. 10 sieve (%)	76.4
Passing No. 40 sieve (%)	39.4
Passing No. 200 sieve (%)	0.3
Silt content (<75micron) %	0.29
Liquid limit (%)	51.5

Plastic limit (%)	21.3
Plasticity index (%)	30
Specific Gravity ( $G_s$ )	2.56
Max. Dry Density ( $Kg/m^3$ )	2.65
Optimum Moisture Content (%)	13.75
USCS-ID	CH

### 3.2.2.5. PH Test

The degree of acidity or basicity of a soil is determined by a Soil PH Test. It is convenient in finding out how mobile and soluble the ions and minerals in the soil are, respectively. The PH's of both the controlled and the soils were measured using a handheld PH meter (figure 3.13). The tests were conducted in accordance with ASTM D4972-19. Prior to every test, it was ensured that the pH meter was calibrated as instructed by the manufacturer. The pH of the soils was determined using distilled water. The tests were carried out on both control and treated soils after the amelioration periods to determine the availability of enough basicity for ongoing reaction processes, and at the same time to observe the changes in pH and to comprehend the long-term effects of the treatments on the initial advancing chemical reactions.



Figure 3. 13: Handheld PH-meter

### 3.2.2.11. Unconfined Compressive Strength Test

Unconfined compressive strength (UCS) is the maximum axial compressive strength a right cylindrical soil mass can endure under confined condition prior to failure, when confining stress is zero. It is an important geotechnical test designed to measure the shear strength of a clay soil

mass. The relationship between the shear strength ( $s_u$ ) and the stress at failure ( $q_u$ ) is given equation 3.4 as follows (Das and Sobhan 2013):

$$S_u = \frac{q_u}{2} \quad \text{Eqn 3.4}$$

Both control and treated soils were tested to compare the strengths of the soil before and after treatment and to evaluate the efficiency of the geopolymer in enhancing the strength of the soil. Three different dosages (5%, 10% and 15% by weight) of geopolymer were applied to determine which one is more effective. The UCS was performed in keeping with ASTM D2166-13, using the Geocomp Load Trac-II machine. The soil specimens were mixed with water at their appropriate OMC and compacted in a cylindrical mold of dimension: 100mm diameter and 116mm height. The right cylindrical soil specimens were extruded from the mold. The control soil specimen was immediately crushed to failure by subjecting them to a strain-controlled uniaxial loads at a rate of 2%/min, treated soil specimens were kept in a humidity-controlled room for 7 days. The maximum value at which the specimen failed on the stress-strain curve is



Figure 3. 14: UCS test of the specimen

#### 3.2.2.13. One-dimensional Swell Test

The soil swell/collapse strains can be used to approximate heave or settlement of the confined soil profile subjected to one dimensional heave or settlement, or stress-induced settlement following wetting-induced heave/settlement. They can also be used to approximate the pressure that would be essential to avert swelling.

The free swell is the percent swell undergone by a soil following absorption of water at a seating pressure of 0.145 psi (1 kPa). The swell was determined using a swell setup (Figure 3.15) as recommended by ASTM D4546-08. The specimens were statically compacted at its OMC in a cylindrical mould to form specimens with diameter and height of 152.4mm and 116.4mm, respectively. A seating pressure of 1kpa was applied to the top of the soil specimen. A dial gauge (deformation-measuring device) was then placed on the capping for the reading of the vertical strain due to the absorption water. This was followed by fully submerging the consolidation cell by inundating it with water and the taking of the swell reading. This process was carried out for the period of 72 hours on both the control and treated soil specimens. The swell curve was then attained by plotting the percent swell against time on a semi-logarithmic scale.



Figure 3. 15: Modified 1-D swell test set-up.

#### 3.2.2.14. California Bearing Ratio (CBR)

The California Bearing Ratio (CBR) is the load test applied to the surface and used in soil investigations as an aid to pavement design. This test was conducted on both the control and the treated soils. It is used to evaluate the potential strength of the subgrade, subbase and base course materials for use in road and airfield pavements. It is expressed as the ratio of the unit load on the piston required to penetrate 0.1 in (2.5mm) and 0.2 in (5mm) of the test soil to the unit load required to penetrate a standard material of well graded crushed stone. The tests

were performed in keeping with ASTM D1883-07. The samples were compacted in 150.24mm diameter molds in accordance with ASTM 698-12. A surcharge of 10lb was placed on the compacted soil prior to testing. Before seating the penetration load, a surcharge of 5lb annular weight was first placed on the surface of the compacted soil to prevent upheaval of the soil into the hole of the surcharge weight, after which the remainder of the surcharge was added. The laboratory test uses a circular piston to penetrate material compacted in mold at constant rate of penetration of approximately 0.05 in (1.27mm/min). Both the load and the penetration gauges were set at zero prior to resumption of the test. The load reading was recorded at the penetrations of 0.25, 0.5, 0.75, 1.0, 1.25, 1.5, 1.75, 2.0, 2.25, 2.5, 2.75, 3.0, 3.25, 3.5, 3.75, 4.0, 4.25, 4.5, 4.75, 5.0, 5.25, 5.5, 5.75, 6.0, 6.25, 6.5, 6.75, 7.0, 7.25, 7.5cm. The load reading at penetrations of 2.5 and 5 were used to compute the CBR using the standard load of 6.9MPa and 10.3MPa, respectively.



Figure 3. 16: CBR Testing Equipment

#### 3.2.2.15. Linear Shrinkage Test

The linear shrinkage of soil is defined as the specific volume change of soil with respect to its water content. This test is used to evaluate the ability of soil to shrink. At the shrinkage limit, further water loss is not expected to result in additional volume reduction of the soil. The test is performed according to ASTM D4943. This test was conducted on a soil fraction passing the 0.425mm sieve. The soil sample was mixed with a distilled water to make a creamy paste that could be readily worked without entrapping air bubbles. This was followed by coating the

shrinkage dish (a metal bar) with a fuel waste to avoid the soil sticking to the surface of the metal bar. The soil paste was placed in the center of the shrinkage dish of about one-third of the volume of the dish and was tapped and allowed to flow to the edges. This process was repeated until the dish was filled and excess soil paste projects out about its edges. The excess was striped with a straight edge to even the top. The sample was allowed to dry in the open air until its color change from dark to light. It was then placed in the oven for drying for about 24 hours at the temperature 105°C. Afterward, it was removed and allowed to cool in a desiccator. After cooling, the length of the bar was measured to compute the shrinkage limit using equation 3.6 as follows:

$$L_S = \frac{L_W - L_D}{L_W} \times 100 \quad (\text{Eqn. 3.6})$$

Where  $L_S$  is the linear shrinkage usually represented in percent,  $L_W$  is the length of the wet soil bar, 5 inches (127 mm), and  $L_D$  is the length of the dry soil bar, inches (mm). Linear shrinkage tests were performed on both control and treated soils.



Figure 3. 17: Linear shrinkage bar test setup

### 3.2.3. GP treatment of Expansive soils

Consistent with the objective of this research, the control soil was treated with the synthesized geopolymer, and the treated soil was subjected to all the basic geotechnical tests that were performed on the controlled soil. The purpose of this was to assess the impact of the geopolymer on the engineering properties of the soil. Three different dosages in the percentages of 5, 10 and 15 by weight were applied to the control soil to determine the appropriate dosage that would be effective for the treatment of the soil. In each case, basic geotechnical tests such as the compaction, Atterberg limits, shrinkage, swell, CBR and UCS tests were performed.

#### 3.2.3.1 Sample preparation and testing

For the compaction test, the soil was mixed with the appropriate proportion of the geopolymer mix and allowed to mellow for 24hr in an air-tight polyethylene container. The treated soil was then compacted in a mold of diameter and height of 152.4mm and 116.4mm in keeping with ASTM D698-12 to obtain the OMC and MDD. The determined OMC and MDD were used to prepare the samples for 1-D swell, CBR and UCS tests which were conducted according to ASTM D4546-08, ASTM D1883-07 and ASTM D2166-13 respectively. The samples of the 1-D swell and the CBR were molded in a cylindrical mold of diameter and height 152.4mm and 116.4mm respectively while the sample of the UCS molded in a cylindrical mold of diameter and height of 100mm and 116.4mm. prior to molding the samples, the treated soil was placed in an air-tight polyethylene container and allowed to mellow for 24hr. 1-D swell was performed immediately after molding, and this was done for three days. The CBR samples were submerged under water for 96 hours prior to conducting the test. The UCS samples were cured for 7 days before testing. The curing period of each test can be summarized in table 3.4 below:

Table 3. 5: Engineering test variable for Geopolymer treated soil

Test	Curing periods (days)
Compaction	0
Atterberg	0

UCS	0, 7
1-D Swell	0, 3
CBR	0, 3

### 3.2.4. Lime treatment of Expansive soils

Among the many methods available for the stabilization of expansive soil, lime is the most frequently use. In so doing, the geopolymer treated soil was compared with lime treatment to evaluate the viability of geopolymer as an effective alternative to lime as a stabilizing agent.

In order to treat a soil sample with lime, it is imperative to determine the soluble sulfate content of the soil and the appropriate dosage of lime required for the treatment. The following subsections detailed the procedure of determining the sulfate content and the appropriate dosage of lime:

#### 3.2.4.2. Lime Dosage Determination

The minimum dosage of lime required to stabilize expansive clay as described by Eades and Grim was performed in keeping with ASTM D6276-19. According Eades and Grim, the minimum percentage of lime required to stabilize expansive soil is the percentage of lime that gives a PH of 12.4, the PH required to yield a pozzolanic reactions.

The test was performed on the fraction of the soil passing the No. 40 sieve. Six different samples of 30g of soil were placed in beakers, and six different dosages of lime in the proportion of 0, 2%, 4%, 6%, 8% and 10% by weight were added to the samples, respectively. This was followed by the addition of 150 ml of distilled water to each of the soil-lime mixture. They were mixed thoroughly for about an hour before taking the reading of the PH's.



Figure 3. 18: Determination of the lime dosage

The result of the PH versus lime dosage can be seen in figure 3.19.

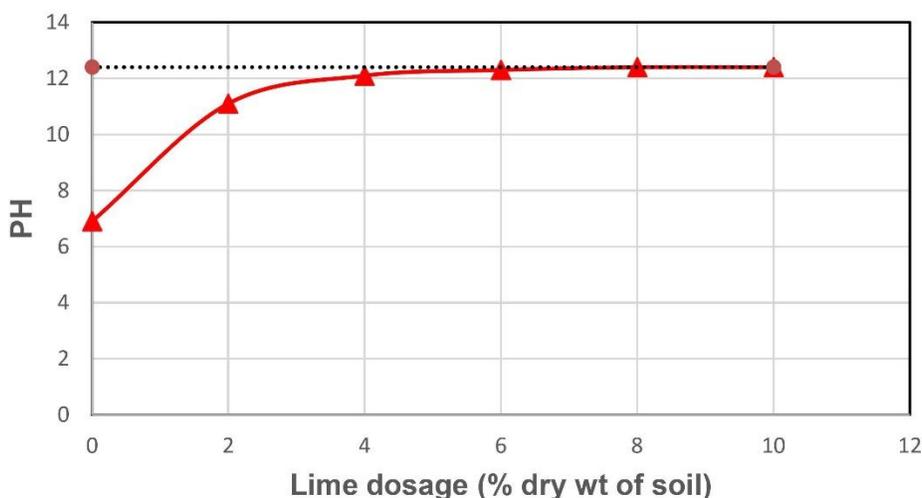


Figure 3. 19: Variation of pH with lime dosage rate

#### 3.2.4.3. Specimen Preparation and Testing

In order to understand the variations that occur in geotechnical properties of soil when treated with lime, the lime treated soil was also subjected to those basic geotechnical tests that were performed on the control soil. The Atterberg limits of the lime treated soil were determined by mixing the soil with the predetermined lime dosage obtained by the procedures described by Eades and Grim, and the lime-soil mixture was kept for an amelioration period of 24 hour for proper hydration to occur prior to testing. The test was performed in line with ASTM D4318-17.

Following this was the compaction test conducted as per ASTM D698-12 to obtain the OMC and the MDD of the lime-treated soil. The soil was mixed with the appropriate lime demand, kept in a polyethylene bucket to avoid absorption of moisture from the air and allowed to ameliorate for 24 hours prior to the performance of the test. The soil-lime mixture was then compacted in a cylindrical mold of diameter and height of 152.4mm and 116.4mm respectively at the OMC of the control soil previously determined. The soil was placed in three layers, and 56 blows was applied to each layer. After the determination of the OMC and the MDD of the lime-treated soil,

the CBR was subsequently determined. The lime-treated soil was now remolded using the standard compaction procedure described above at the new OMC and MDD. The remolded sample was submerged in water for proper soaking for 96 hours prior to performing the CBR. The test was performed with a digital CBR Testing machine from ELE International as per ASTM D1883-07.

UCS soil specimens were molded to cylinders of approximate diameter and height of 100mm and 116.4 mm, respectively, while 1-D swell specimens were molded to form specimens with diameter and height of 6 152.4 mm and 116.4 mm, respectively. Molded test specimens were placed for curing in a moisture room at 100% RH for the prescribed curing periods. The parameters tested for comparing engineering characteristics of both lime-treated soils in this study are given in Table 3.7

Table 3. 6: Engineering test variables for lime-treated soils

Test	Curing periods (days)
Compaction	0
Atterberg	0
UCS	0, 7
1-D Swell	0, 1
CBR	0, 4

Results obtained from these tests were correlated with those of geopolymer-treated soils in order to evaluate the viability of geopolymer as an effective substitute stabilizer of expansive clay soils.

### 3.3. Summary

This chapter discussed the synthesis of metakaolin-based geopolymer and its evaluation as an effective stabilizing agent for expansive soil. The procedures involved in the synthesis of geopolymer, treatment mechanisms of expansive soil using geopolymer and lime and how geopolymer affect expansive soil were fully highlighted in this chapter. Upon the synthesis of geopolymer, control soil was subjected to variety of geotechnical, chemical and microstructural testing essential for its classification. These were followed by its treatment with both geopolymer

and lime to compare the variation that will occur in their engineering properties as the result of the both treatments. In so doing, the effectiveness of the geopolymer as a stabilizer could easily be appraised. The succeeding chapter fully discussed how the results attained from the experimental methodology impacted the engineering properties of the soil after treatment.

## CHAPTER 4

### EFFECT OF VARIOUS TREATMENTS ON GEOTECHNICAL PROPERTIES OF EXPANSIVE SOILS

#### 4.1 Introduction

This chapter seeks to analyze the results of the various engineering and microstructural tests that were performed on the different treated soils in the preceding chapters. The effectiveness of both geopolymer and lime in stabilizing the control soil are first evaluated, and both the geopolymer and lime treated soils are compared and analyzed aim at evaluating the viability of metakaolin-based geopolymer as a perfect alternative to lime in soil treatment.

#### 4.2 Engineering Test

The efficiency of both lime and geopolymer in treating expansive soil is assessed in this section through the discussion of the results of some of the engineering tests that were performed on the control and treated soils. Hence, engineering tests such as Atterberg limit, moisture content-

dry density relationship, UCS, CBR, 1-D swell and linear shrinkage are highlighted in the upcoming subsections.

#### 4.2.1. Atterberg limit

The Atterberg limits or consistency limits furnish information on the ability of the soil to retain water, as well as its ability to swell. Results of the Atterberg limits test of the control and treated soils after 24 hrs. are shown in table 4.1. Changes in the PI of treated soils at different geopolymer demand and control soil are depicted in figure 4.1. Addition of metakaolin-based geopolymer to the soil led to reduction in the plasticity of the soil indicating enhancement in property as depicted in figure 4.1. Similar reduction in the PI was observed when a predetermined dosage of lime was added to the native soil. The immediate reduction of PI is possibly due to a rapid cation exchange reaction occurring between the chemical additive and clay particles. A soil has low plasticity if the liquid limit is less than 35%, and it is considered to have intermediate plasticity if the liquid limit is between 35% and 50%. Furthermore, the soil is said to have high plasticity if the liquid limit is between 50% to 90%, and it is considered to be highly plastic with the liquid limit above 90% (Areola O. et al).

The addition of metakaolin geopolymer in the proportion of 5, 10 and 15% and lime in its predetermined dosage to the sample altered the engineering properties of the sample, which are indicated in Table 4.1. These reductions in plasticity indices are indicators of soil improvement.

Table 4. 1: Summary of variations in soil properties due to addition of GP and lime

<b>Properties</b>	<b>Additives</b>				
	<b>Geopolymer</b>				<b>Lime</b>
	<b>0%</b>	<b>5%</b>	<b>10%</b>	<b>15%</b>	<b>8%</b>
<b>Liquid limit %</b>	51	35	30	31	31

<b>plastic limit %</b>	21	22	23	26	23
<b>Plasticity index %</b>	30	13	7	5	8

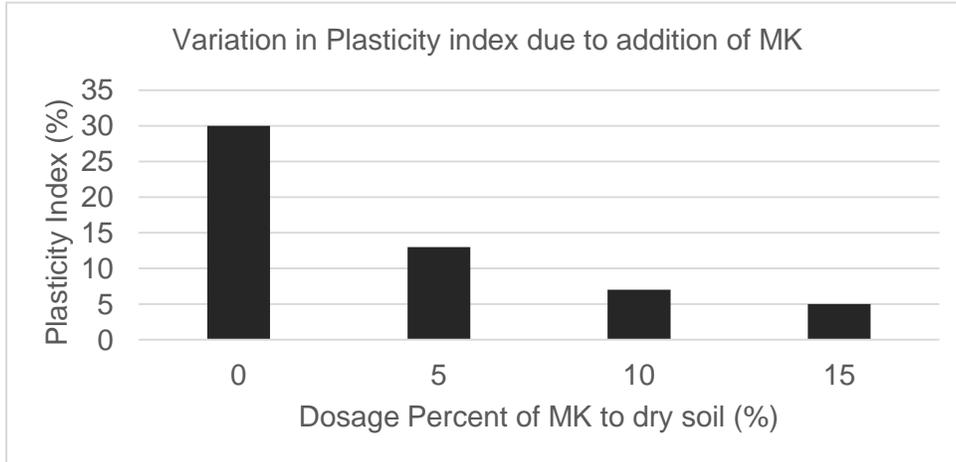


Figure 4. 1: variation in PI due to MK addition

#### 4.2.2. Moisture content-dry density relationship

The addition of geopolymers and lime to the soil specimen effect changes in the compaction properties of the specimen. Fig. 4.2 depicts the effect of geopolymers and lime on the compaction properties of expansive clay. The addition of geopolymers and lime to the soil specimen led to a reduction in the MDD and an increase in the OMC. This phenomenon is typical in lime treated soil. According to (Little 1998) lime causes agglomeration and flocculation of soil particles when added to soil, and the flocs reorient themselves and formed a denser state that enhance its water retention potential. However, each increment in the geopolymers composition resulted into an increase in MDD and a reduction in the OMC. For instance, the MDDs and OMCs obtained for 5, 10 and 15% addition of geopolymers to the soil are 1.64gm/cm<sup>3</sup> and 15.04%, 1.77gm/cm<sup>3</sup> and 14.10% and 1.84gm/cm<sup>3</sup> and 13.85% respectively. This is a clear indication that the maximum geopolymers demand was not reached. It is assumed that continuous increase in the geopolymers content could give higher MDD than the MDD of the control soil. The MDD and OMC of the lime treated was determined to be 1.81gm/cm<sup>3</sup> and 14.39% respectively.

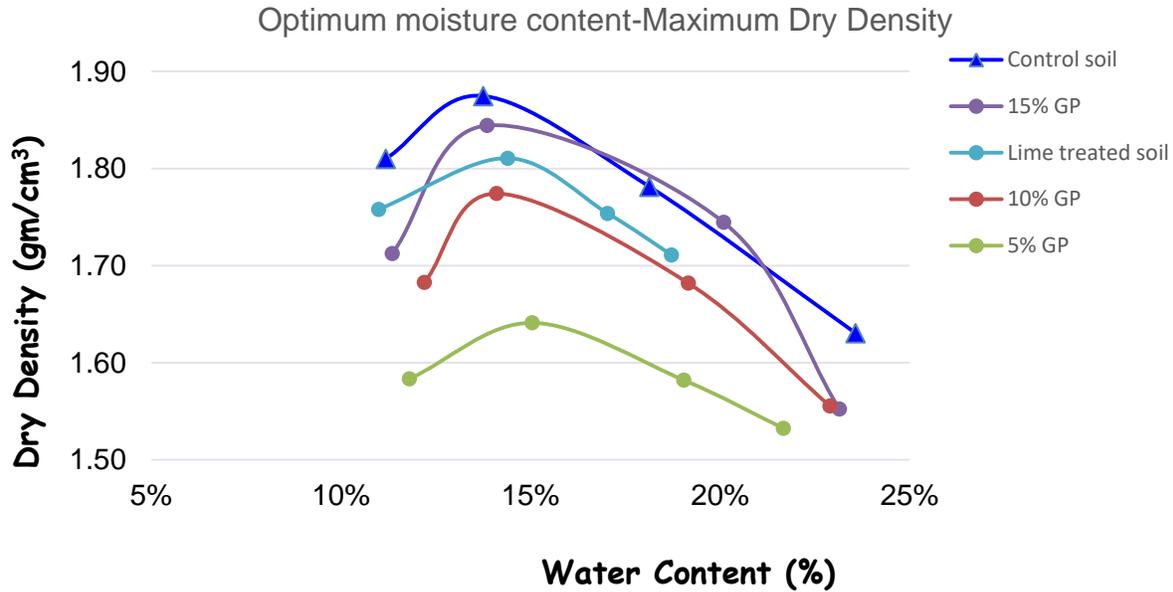


Figure 4. 2: Optimum Moisture Content-Maximum Dry Density relationship of the different dosages

#### 4.2.3. UCS

The unconfined compressive strength (UCS) of the control soil and the treated soils were studied through the stress-strain characteristics of cylindrical specimens. The samples were mixed with the appropriate dosages and kept in an air-tight HDPE bucket to mellow for 24hr before molding them into test specimens. The specimens were molded according to ASTM D2166-16 and allowed to cure for additional 7 days before testing. Fig. 4.3 depicts the stress-strain behaviors of the control soil and the treated soils with the predetermined lime dosage and different geopolymer dosages. The peak stress or failure stress, which is the unconfined compressive strength (UCS), increased with the addition of lime and geopolymer. This may be due to the flocculant structure the mixture attains when the soil was blended with the geopolymer and the lime. Curing the samples for 7 days leads to cementation which enhances the strengths of the specimens. The peak stresses were 71.54, 95.15, 106.95 and 118.04kPa, respectively, for the control soil and geopolymer contents of 5, 10 and 15%. The peak stress for the lime soil mixture was 101.05. The result also reveals that failure strain reduced with increasing geopolymer content. This reduction evinces higher stiffness.

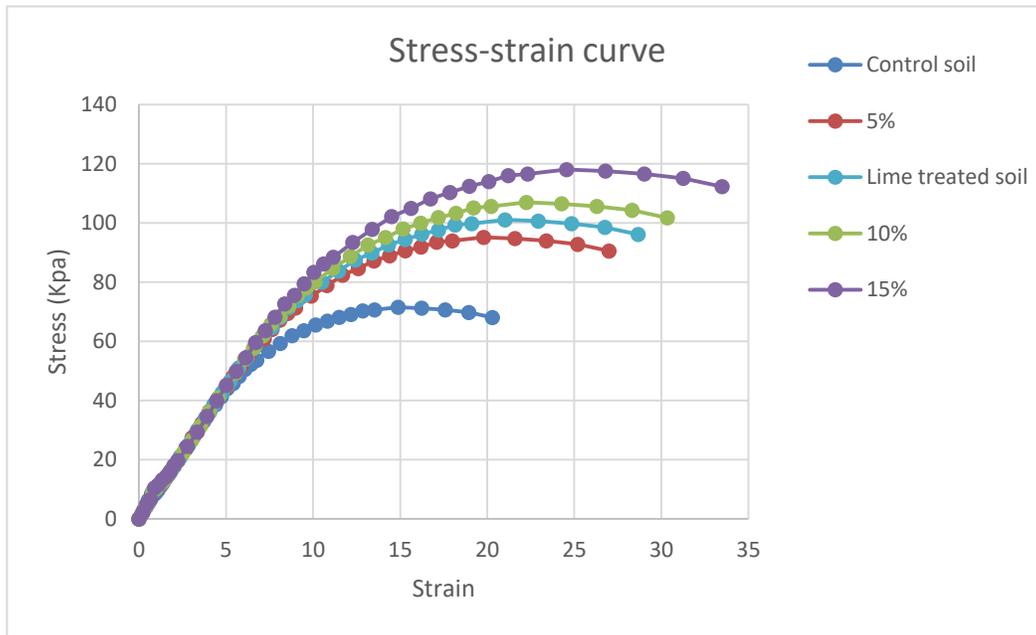


Figure 4. 3: UCS

#### 4.2.4. 1-D Swell

Both the control and the treated soils were tested for 1-D swell tests in keeping with ASTM D4546-08. A single predetermined dosage of lime treated and three different dosages of geopolymer (5%, 10%, and 15% MK) treated soil were tested after curing periods of 24 hours. The test samples were prepared by compacting the soil specimens at 95% of their corresponding MDDs and OMCs. Results obtained from the swell test are depicted in Figures 4.4.

The swell potential of the soil dramatically reduced after its treatment with geopolymer and lime. The swell potential of the lime treated soil was observed to be negligible after 24hrs. For geopolymer, the reduction was incremental as the concentration of the geopolymer increases. The swell potential of the control soil was observed to have reduced by about 50, 75 and 87.5% with corresponding dosage of 5, 10 and 15%. It is assumed, therefore, that the swell would have become insignificant with longer curing time or/and geopolymer content. It is likely that the fast reduction in swell is due to a swift exchange of cations between the geopolymer and lime on

one hand and the clay minerals on another hand. This process weakens the capacity of clay minerals to absorb water.

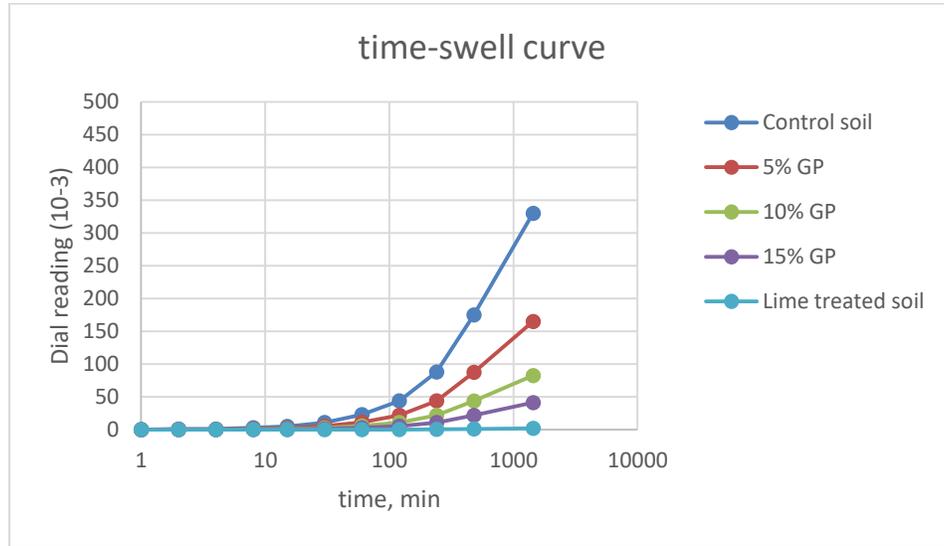


Figure 4. 4: time-swell curve of varying geopolymer dosage

#### 4.2.5. CBR

Table 4.2 shows the results of the CBR test that were performed at a predetermined lime dosage and varying geopolymer dosages. The tests were performed in keeping with ASTM D1883-07. The addition of lime and geopolymer to the soil enhances the CBR index. The CBR index of the lime treated soil was determined to be 33.65%, while the index gets more improved with increasing geopolymer concentration. This enhancement in the CBR indices with increasing geopolymer concentration is as the result of the strong structure created between the geopolymer and the soil during the geopolymerization process. The CBR indices obtained were 18.24, 43.31 and 54.25%, respectively, for 5, 10 and 15% by weight of geopolymer concentration.

Table 4. 2: CBR results of the stabilized soil

	Control	Lime	Geopolymer		
			5%	10%	15%

CBR Index (%)	12.41	33.65	18.24	43.31	54.25
General Rating	fair	good	fair	good	good
Uses	subbase	Base, subbase	subbase	Base, subbase	Base, subbase

4.2.6. Linear Shrinkage

Both control and treated soils were tested for linear shrinkage in keeping with TEX-107-E, for three geopolymer dosages (5%, 10%, and 15% GP) and a predetermined dosage of lime treated soils. All specimens were cured for a period of 24 hours prior to testing. Both treatments were observed to have dramatically lowered the shrinkage ability of the expansive soil virtually to a negligible level. The addition of lime to the soil specimens reduced the soil shrinkage ability of about 17.85% by 90%. Similar drastic reductions were observed for the geopolymer treated specimens. Addition of geopolymer in the proportion by weight of 5, 10 and 15% reduced the shrinkage potential by 77.55%, 83.07% and 98.51% respectively. The shrinkage became virtually negligible following the addition of the 15% geopolymer. The results of the shrinkage test for geopolymer treated soils are presented in figure 4.5.

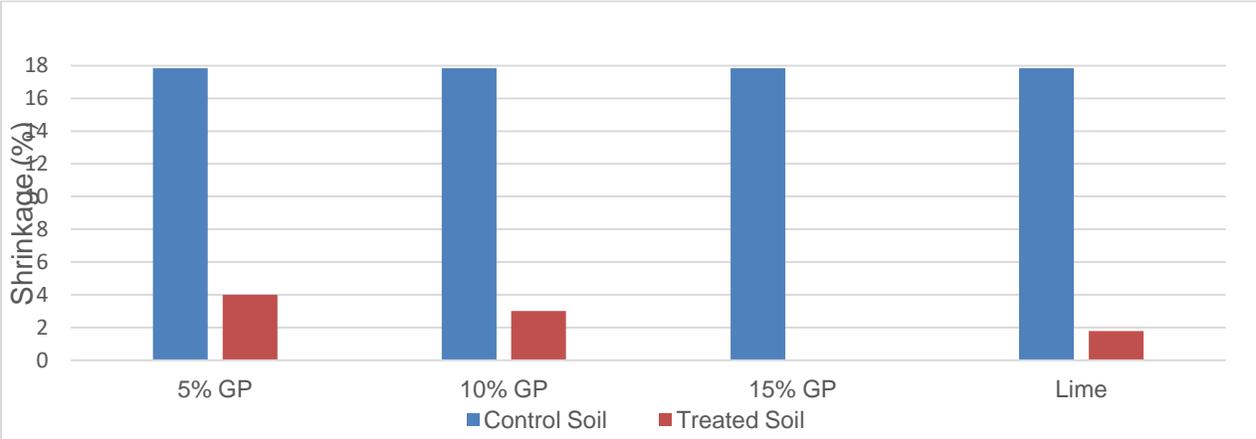


Figure 4. 5: Shrinkage plot for control and treated soils

#### 4.2.7. Comparison of lime and geopolymer treatment

In this section we endeavored to correlate the effects of lime on the expansive soil to that of the geopolymer treated soil to enable us measure the viability of geopolymer as a perfect stabilizer of expansive soil. Only test results of soils treated with the highest dosage of geopolymer (15% GP) were employed in this comparison since their results seem to be more effective in most of the cases, if not all.

##### 4.2.7.1. Atterberg limit

Figure 4.6 compares the Atterberg limits of geopolymer treated soil to that of lime treated soil, where geopolymer was observed to be more effective in lowering the water absorption, the shrinking and the swell potentials of the expansive soil than lime. The geopolymer and the lime reduced the PI of the native soil by 83.3% and 73.3% respectively, which indicate that the geopolymer was more effective than the lime by 12% in the treatment of the soil.

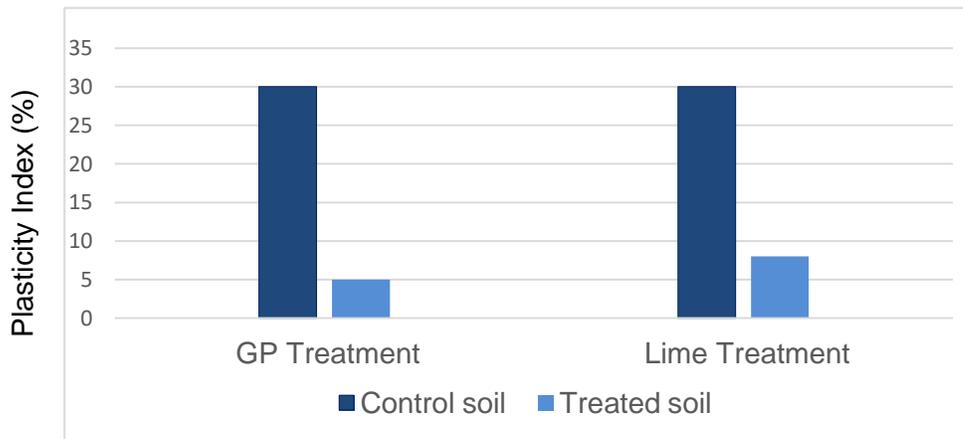


Figure 4. 6: PI comparison of GP and lime treated soil

##### 4.2.7.2. Compaction

Figure 4.7 compares the results of the compaction of GP and lime treated soils, where the MDD of the geopolymer treated soil was observed to be higher than the lime treated soil, and the OMC of the geopolymer treated soil was observed to be lowered than the lime treated soil. The MDDs and OMCs of the geopolymer and lime treated soils were 1.84gm/cm<sup>3</sup> and 13.5% and 1.81gm/cm<sup>3</sup> and 14.39% respectively.

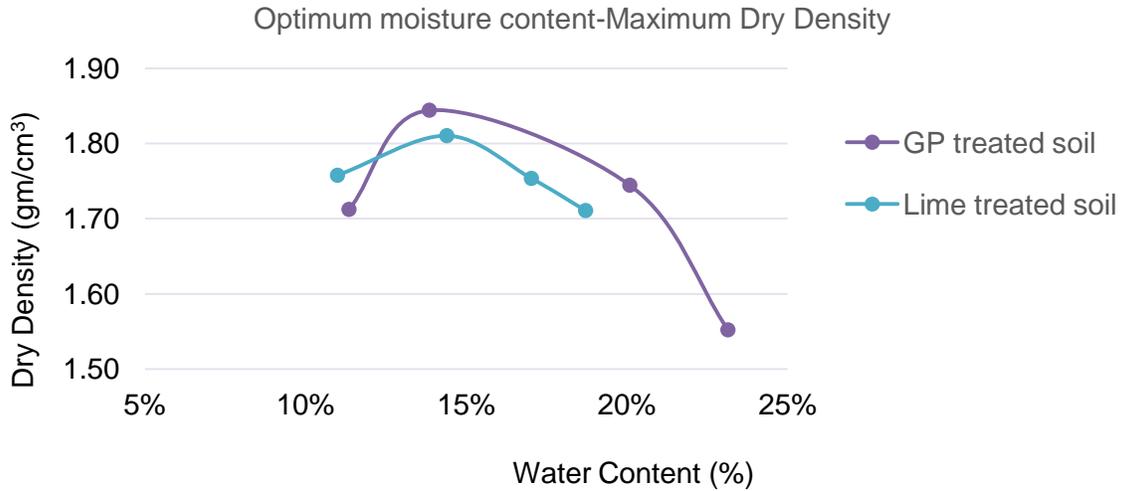


Figure 4. 7: compaction comparison of geopolymer and lime treated soils

#### 4.2.7.3. UCS

Figure 4.8 shows the correlation between the UCS of the lime-treated and the geopolymer treated soils, where the strength achieved eventually was noticed to be larger in geopolymer-treated soils than lime-treated soils. Geopolymer treatment led to an increase in strength of about 65% of the strength of the native soil, while lime treatment resulted into an increase in strength of about 40.66% of the strength of the native soil. It is necessary to indicate that the molding of the lime-treated samples was done at 5% more than the OMC value to mitigate ettringite formation.

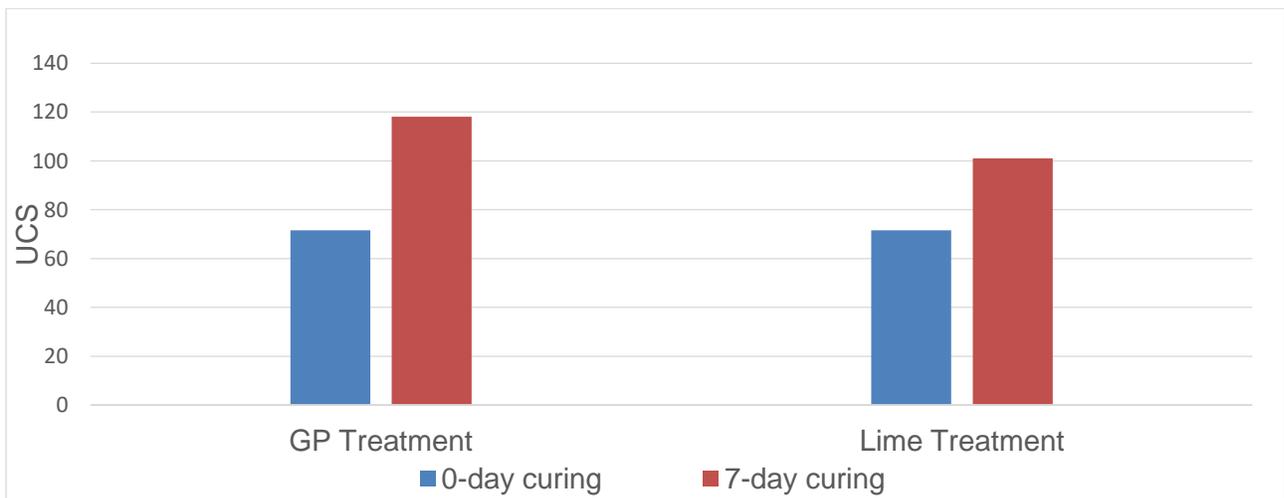


Figure 4. 8: UCS comparison of geopolymer treated and lime treated soils.

#### 4.2.7.4. 1-D Swell

Figure 4.9 shows a correlation of swell potential of GP-treated and lime-treated soils. When lime was added to the soil, the swell immediately became almost negligible within a 24-hour period. Nevertheless, geopolymer would have proven more effective if the curing period was extended. Both geopolymer and lime treatments, however, immediately lowered swell and produced almost similar outcomes with respect to decreasing of the swell potential.

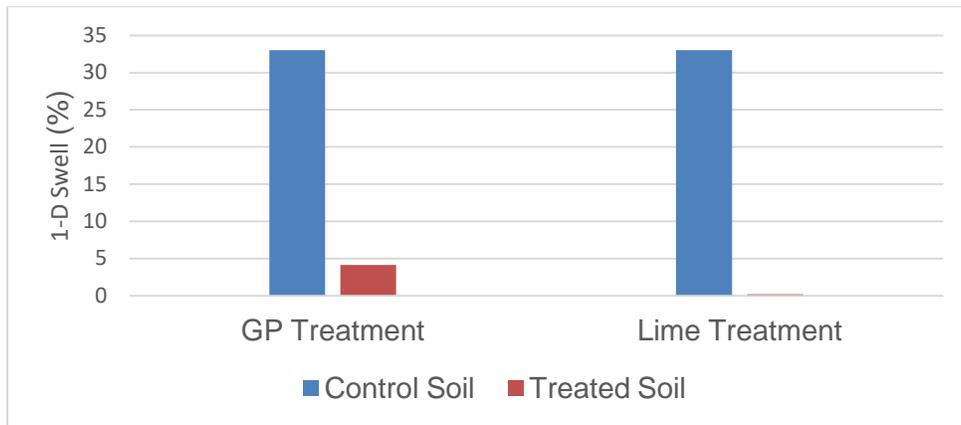


Figure 4. 9: 1-D Swell comparison of geopolymer and lime treated soils

#### 4.2.7.5. CBR

Figure 4.10 presents CBR the comparison between the geopolymer treated soil and the lime treated soil, where the CBR of the geopolymer treated soil was observed to be higher than the lime treated soil. The CBRs of the geopolymer and lime treated soils were 54.25 and 33.65% respectively. The geopolymer treatment proved to be 22.88% more effective than the lime treatment.

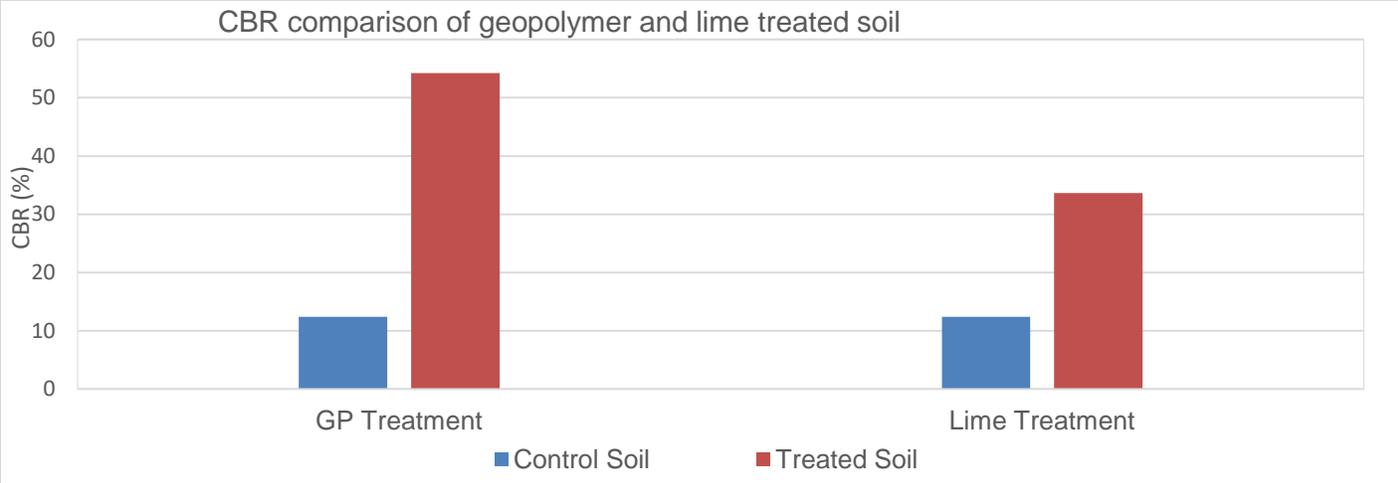


Figure 4. 10: CBR comparison of geopolymer and lime treated soils.

4.2.7.6. Shrinkage

Figure 4.11 presents the shrinkage comparison between the geopolymer treated soil and the lime treated soil, where the shrinkage of the geopolymer treated soil was observed to be lower than the lime treated soil. The shrinkages of the geopolymer and lime treated soils were 0.2 and 1.79% respectively. The geopolymer treatment proved to be 11.17% more effective than the lime treatment.

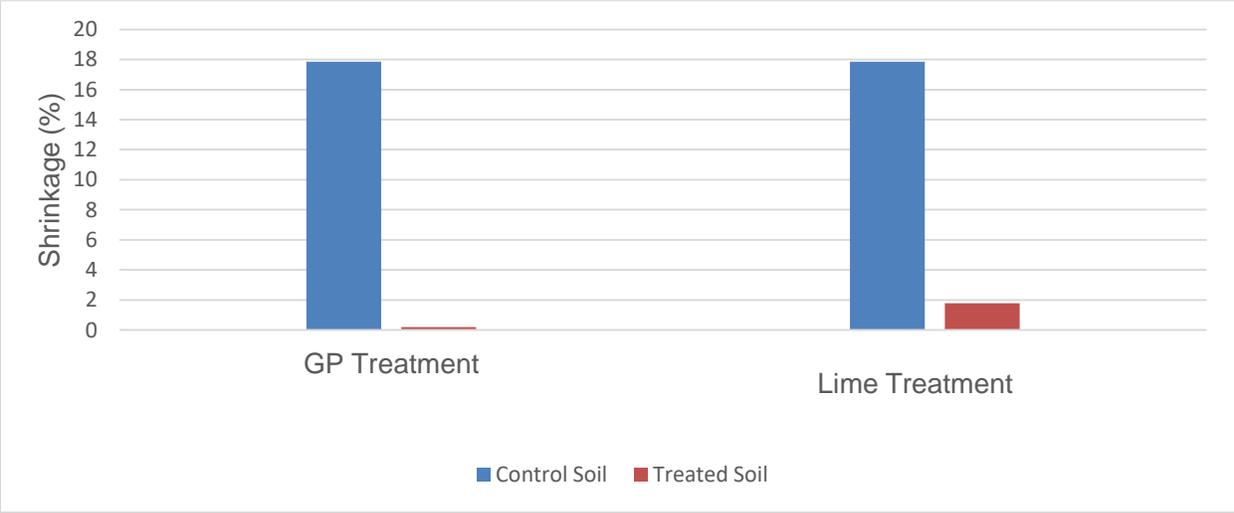


Figure 4. 11: Shrinkage comparison of Geopolymer and lime treated soil

#### 4.2.8. Summary

Variety of engineering tests were performed in this study to determine the characteristics of both control and treated soils. These characteristics included plasticity index, moisture content-dry density relationship, unconfined compressive strength, 1-D swell, CBR and linear shrinkage. The treatment was done with two different materials (lime and geopolymer) to compare the effect of each on the native soil and to determine the viability of geopolymer as an effective alternative to lime in soil stabilization. Results of these tests were discussed and compared in this chapter.

The addition of 8% lime and 15% geopolymer to the soil reduced the plasticity index of the native soil by 73.3 and 83.3% respectively, where the geopolymer proved to be more effective than the lime by 12%. The geopolymer treatment, at the same proportion, also resulted into slightly higher MDD and lower OMC than the lime treatment. The MDDs and OMCs attained as the result of geopolymer and lime treatments were 1.84gm/cm<sup>3</sup> and 13.5% and 1.81gm/cm<sup>3</sup> and 14.39% respectively. Impressive improvement was also observed in the unconfined compressive strength (UCS) of the soil when lime and geopolymer were added to it. The geopolymer increased the UCS of the soil by 65%, while the lime increased it by 40.66%.

Lime proved to be more effective within short period of application than geopolymer in reducing the swell potential of the soil. However, with little extension in curing period, they all seem to be comparative in the long run.

Immense enhancement was also observed in the CBR upon the addition of geopolymer and lime to the control soils. The CBR of the soil was increased by 337.15% and 171.15% by geopolymer and lime respectively.

The shrinkage was also found to have undergone drastic reduction from 17.85 to 0.2% following the addition of geopolymer.

Generally, the results reveal that both treatments are somewhat comparable, and that geopolymer is an outstanding stabilizing agent for expansive soil.

## CHAPTER 5

### SUMMARY, CONCLUSION AND RECOMMENDATION

#### 5.1 Introduction

Presently, lime and ordinary Portland cement are the materials widely utilized for the stabilization of expansive soils, but the production of these materials requires the consumption of high energy which emits lot of carbon dioxide gases into the atmosphere, that contributes highly to global warming and climate change. Moreover, another drawback associated with calcium-based treatments is that they tend to make the soil more expansive in the event where the calcium-based stabilizer is used to stabilize a soil that contains high sulfate content.

To find a workable alternative for these conventional stabilizers, this study sought to assess the adequacy of another material that is referred to as geopolymers as an effective stabilizing material.

The geopolymer was synthesized from a metakaolin and an alkaline activator (NaOH and Na<sub>2</sub>SiO<sub>3</sub>) in the mix of Si/Al molar ratio of 1.6 and NaOH/Na<sub>2</sub>SiO<sub>3</sub>.5H<sub>2</sub>O molar ratio of 10. These materials were thoroughly mixed stoichiometrically to obtain a alkali metal cation and aluminum oxide ratio of 1:1 to provide for better geopolymer formation, and the geopolymer slurry was cured at room temperature (20 to 25°C) and at a relative humidity of 50% or less to form solid geopolymer.

A typical expansive subgrade soil generally found in most part of Africa was obtained from Abuja, Nigeria and treated with three different dosages of the geopolymer.

Series of engineering tests were conducted on the native soils and the treated soils to determine the adequacy of the geopolymer in improving engineering properties such as plasticity, swell, compressive strength, CBR, etc. of the soil.

Also, to determine the viability of the geopolymer as a perfect alternative to conventional stabilizers currently in use, lime was also utilized to treat the expansive soil and tested for engineering properties. The results obtained was compared to those obtained for geopolymer treated soil.

## 5.2 Summary of results

The key discoveries of this study are briefly outlined in this section below:

1. The soil was classified as a low plasticity clay (CH) with a PI of 30%.
2. The geopolymer was applied to both expansive soils at dosages of 5%, 10%, and 15% MK.

The moisture content – dry density relationship tests performed on control and geopolymer-treated expansive soils revealed that the lowest GP dosage resulted in higher OMC and lower MDD than control soil, while the intermediate and high GP dosage treated soils showed an increasing MDD and decreasing OMC trend than the lowest GP dosage. This is similar to the compaction curves of lime-stabilized soils.

3. Geopolymer treatment was observed to reduce the PI of the expansive soil almost immediately. Significant reduction in PI by about 50 to 90% was observed for the lowest and highest GP dosages, respectively, after a 7 day curing period.
4. Geopolymer-treated soils were observed to have significantly higher unconfined compressive strength than control soil, especially for the higher GP dosages after 7 days of curing. Strength gain was observed to be immediate for lower GP dosages for the soil.
5. Immediate and significant reduction of swell potential was observed in the geopolymer-treated expansive soil. The swell potential of the control soil was determined to be about 15%, and it was found to be negligible after being treated with the geopolymer dosage after only 3 days of curing.
6. Shrinkage of the control soil was significantly reduced upon geopolymer treatment.
7. Geopolymer treatment was also observed to significantly reduce the water absorption capacity of soil specimens.
9. CBR was greatly improved upon geopolymer treatment.
10. Both soils were treated with lime and tested for UCS and 1-D swell characteristics. Both lime-treated and geopolymer-treated soils were found to have comparable UCS and swell properties.

### 5.3 Conclusion

Engineering properties of the expansive soil were improved immensely when the soil was treated with Geopolymer. The geopolymer was also found to be comparable with lime as relate to their efficiencies in soil treatment. The viability of metakaolin-based geopolymer as an effective agent for stabilization of expansive soil has therefore been evidently validated by this research. Moreover, manufacture of metakaolin-based geopolymers is found to be more ecofriendly and sustainable than lime. This further justified the need for the use of this more environmentally sustainable alternative material, such as geopolymer, in place of lime for soil stabilization.

## 5.4 Recommendations for future works

After thorough investigation of the effectiveness of metakaolin based geopolymer as a stabilizer for expansive soil and as a potential alternative to lime, the following suggestions are recommended for study in order to fully evolve the concept of this material known as geopolymer:

1. Setting time of geopolymer be determined. This is an important parameter for the purpose of soil stabilization.
2. Evaluation of the sustainability and resiliency benefits be carried out in order to determine and assess the resource consumption, environmental impact, and socio-economic impact of lime and geopolymer treatments, and to evaluate the toughness of a system to bear regular operational loads as well as extreme events.
3. Use other conventional stabilizers in combination with metakaolin based geopolymer to determine whether the quantity of geopolymer could be used at the minimal.
4. Conduct triaxial tests on geopolymer-treated soils to examine variations in its shear strength parameters, and to determine its resilience modulus.
5. Relationship between metric suction and water content be established to determine water storage capacity of geopolymer-treated soils, and to carry out a study on changes in permeability with geopolymer treatment.
6. Durability and leachability testing must be performed to establish long-term performance of geopolymer-treated soils.
7. Intensive chemical and microstructural analyses, such as XRD, XRF, CEC, SSA, TGV, and DSC, FESEM, etc. be carried out, which are utilized to investigate geopolymer-treated soils in order to understand the geopolymerization process in soils.

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