DETERMINATION OF THE EFFECT OF EGG SHELL POWDER AND LIME STABILIZATION ON ENGINEERING PROPERTIES OF CLAY SOIL.

BY

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SUBMITTED TO THE

DEPARTMENT OF CIVIL ENGINEERING
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CERTIFICATION

This is to certify that I, Nwali Jeremiah ugonna with registration number 20167224083 personally carried out this project till completion for the award of Bachelor's degree in Engineering (B.Eng.), department of Civil Engineering, Nnamdi Azikiwe University Awka. This work to the best of my knowledge has not been to any form submitted for same purpose in the field of Civil Engineering.

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APPROVAL

This research work "determination of the effect of egg shell powder and lime stabilization on engineering properties of clay soil." is an authentic academic work undertaken by Nwali Jeremiah ugonna has been assessed and approved by the department of civil engineering, Nnamdi Azikiwe University, Awka Anambra state.

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DEDICATION

This project is dedicated to God Almighty whom his infinite grace and mercy has accompanied me to the successful completion of this project. I equally dedicate this work to my beloved family, friends, and everyone who served as a real source of inspiration towards my academic pursuit.

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This work was made possible first by the divine assistance of God Almighty, the giver of life. His protection and guidance are immeasurably appreciated, and his guidance and protection throughout my stay in Nnamdi Azikiwe University.

I am extremely thankful to my parents whose constant prayers and love kept me going. My thanks, also goes to my Engr. Dr Onoduagu for his time and guidance in the accomplishment of this project. Special thanks to the Head of Department of Civil Engineering, Eng. Dr C.A. Ezeagu His guidance and assistance remain outstanding. I will also like to extend my heartfelt appreciation to all my lecturers who taught me (academic and non-academic staff) of the department of civil engineering, I say thank you. . To my academic coach Uzobuike Kenechukwu (Gplus) I extend my gratitude and say a thank you very much for all you did. To my friends and well-wishers. I appreciate the love received throughout my years here. God bless you all.

ABSTRACT

Improvement of soil properties is necessary in the modern scenario as soils with the required properties are not readily available for construction activities. There have been a lot of issues reported when structures were constructed on weak and soft soils like problems of shear failure, excessive settlement, differential settlement etc. The alternatives left with us are making the soil at site suitable for the expected load by improving its properties. Eggshell Powder (ESP) has not been used as a stabilizing material; however, it could be a supplement for industrial lime. This study was meant to study the effect of eggshell powder on the stabilizing potential of lime on an expansive clay soil. Tests were carried out to determine the optimal quantity of lime and the optimal percentage of lime-ESP combination; the optimal quantity of lime was gradually replaced with suitable amount of eggshell powder. The lime stabilized and lime-ESP stabilized mixtures were subjected to engineering tests. The optimal percentage of lime-ESP combination was attained at a 4% ESP + 3% lime, which served as a control. Results of the Maximum Dry Density (MDD), California Bearing Ratio (CBR), unconfined compression test and Undrained triaxial shear strength test all indicated that lime stabilization at 7% is better than the combination of 4% ESP + 3% lime.

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CHAPTER ONE

Introduction

1.1 Background to the Study

Improvement of soil properties is necessary in the modern scenario as soils with the required properties are not readily available for construction activities. There have been a lot of issues reported when structures were constructed on weak and soft soils like problems of shear failure, excessive settlement, differential settlement etc. The alternatives left with us are making the soil at site suitable for the expected load by improving its properties or adopting a deep foundation. Adoption of deep foundation is not at all an economically viable solution. Hence improvement of soil properties and in-situ treatment of soil are gaining importance these days. Soil stabilization is an economically feasible solution to one of the major construction problems. One of the most common methods of fine soil improvement is to stabilize it using additives that improve soil properties through physical and chemical changes. It is, however, worth noting that fine soils behavior should be well studied before deciding on the method of improvement. Soil modification or stabilization is usually carried out to achieve the following goals:

- Increasing soil strength, geotechnical properties and bearing capacity
- Preventing structure subsidence
- Reducing adhesion in highly adhesive soils
- Increasing adhesion in soils with low adhesion (sands)
- Increasing safety factor against slope, levees and earth dam sliding
- Reducing soil plasticity index.

Over the last years, environmental issues have prompted human to use industrial wastes as alternatives to some construction materials. Both earthwork researchers and engineers have paid considerable attention to using wastes in soil stabilization and improving physical and mechanical properties of soils. This may help both remove environmental problems and contribute to the economy. Industrial wastes such as fly ash, iron slag, wood ash, plastic wastes and iron filings show considerable potential to stabilize soils, which are occasionally used to improve geotechnical properties of poor soils. Researchers all over the world have studied various materials which can be used in conjunction with soil to improve soil properties. Basha (2005) studied the stabilization of soil with Rice husk ash and cement. Brooks (2009) conducted stabilization studies on stabilization using fly ash and rice husk ash. Kamon (1991) conducted researches on stabilization of soil using lime and industrial wastes. Paul (2014) conducted studies on soil stabilization using egg shell powder and quarry dust. Anoop (2017) conducted studies on improving soil using lime and elephant dung strips. Of all the various studies conducted around the world, the most commonly used and effective stabilizer was found to be lime. Lime is a not an industrial waste or a by-product, the manufacture of lime requires heating in the order of 750° C, which contribute to the fact that stabilization of large areas of soil with lime alone will increase the cost required for stabilization. In this study an attempt is made to replace lime with egg shell powder and to find out the extent up to which lime can be replaced by egg shell powder without compromising on the strength. Egg shell powder is an ideal material to replace lime in the stabilization process due to its similar chemical composition. The chief ingredient in egg shell powder is calcium carbonate as in the case of lime. Egg shells are disposed from hotels, restaurants etc. in huge quantities and they are currently facing disposal problems. Use of egg shell powder in soil stabilization reduces the disposal problems associated with egg shell generations. Moreover, powdering of egg shell can be done easily. Egg shell powder generation does not involve generation of CO_2 as in the case of lime where heating is done up to 750°C. Hence use of egg shell powder in soil stabilization will make the overall stabilization process economical, sustainable and eco-friendly.

Therefore in the present study, Egg Shell Powder (ESP) will be used to study its effect on the engineering properties of clayey soil. An improvement in the strength properties of soil by addition of ESP will help to find an application for waste materials to improve the properties of clayey soil and can be used as a better stabilizing agent.

1.2 Statement of Problem

Due to the destruction of buildings, roads and pavements by the swelling/shrinking nature of clayey soil (expansive soil) and the increase in cost of lime for its stabilization.

this project will help in finding out if certain percentage of lime stabilizers can be substituted with egg shell powder (ESP), which will help reduce the amount spent on the stabilization of the soil as well as aid in reduction of waste material (egg shells) in the country. The appalling state of Nigerian economy has brought about inflation, resulting in a continuous increase in prices of materials. The use of lime for stabilization is therefore becoming very expensive therefore requiring an economical replacement. Egg shells are primarily known to contain lime, calcium and protein. It has been used in agriculture to replace lime, which confirms that lime is present in a considerable amount in egg shells.

There have been findings revealing that ESP has been used for stabilizing cohesion less soils in different countries. Therefore this study is thus directed towards identifying eggshell powder as an effective stabilizing agent by replacing a certain percentage of the lime in the stabilized soil with ESP (egg shell powder). Since the quantity of eggshells that may be required for stabilization of a large area may not be met, it is suggested that the ESP (egg shell powder) be used as a supplement in the lime stabilization.

In a well-organized environment, disposal of waste poses a great difficulty, as regards where and how to effectively dispose of the waste product without any adverse effect on the society.

Researches have shown, in the year 2018 global egg production was 78 million metric tons, contributing approximately 8.58 million metric tons of egg shells which was discarded mostly as waste Waheed et al (2020). In the United Kingdom, the estimated quantity eggshell waste to be disposed each year is between 10,000 - 11,000 tons. Thus, an estimate of about 5,000 - 6,000 tons could also be expected in Nigeria annually, therefore, in the absence of an effective waste disposal policy, the utilization of eggshells for soil improvement, either for agriculture or engineering purposes will be a welcome development.

1.3 Aim and Objectives of the Study

The aim of the study is to determine the effect of egg shell powder and lime stabilization on engineering properties of clay soil.

The objectives include:

(i) To investigate the effect of different percent of ESP-lime on the engineering properties of Clay soil.

(ii) To make relevant recommendations with regards to the use of (egg shell powder) ESP-Lime in clay soil.

(iii) To determine how stabilized clay soil gets when (egg shell powder) ESP-lime is added.

1.4 Significance of Study

This research is very important because Nigeria is one the largest consumers for concrete production and this method will help to contributes to reduce the construction cost. Hence, it resolves arising issues of waste disposal problem including support to our environmental problem that mainly caused pollution to public health. Besides that, it will improve to the properties strength and the durability of concrete characteristic with the certain concrete testing thus helps our economy industries of construction. The project aims at serving as a guide for people in the Geological and Construction companies/industries. Geologists and Engineers can use the success of this project/research to determine the right quantity of ESP-lime to use for the stabilization of clay soil as it will reduce the cost of using only lime for the stabilization of the soil. It will also serve as a guide if whether or not ESP can be used in the stabilization of clay soil.

1.5 Scope of study

This research work will be limited to the effect of egg shell powder when it is used to replace certain percentage of lime stabilizers, in the stabilization of expansive soil. The optimal percentage of lime was gradually reduced and complemented with suitable percentages of ESP in the following ratios: 10% lime + 0% ESP, 8.5% lime + 1.5% ESP, 7% lime + 3% ESP, 5% lime + 5% ESP. For each variation, Atterberg's limit tests (liquid and plastic limit) were carried out to determine the optimal lime-ESP combination. The effects of ESP on lime-stabilized soil were assessed further by subjecting the natural soil, the natural soil plus lime and the natural soil plus optimal lime-ESP mixture to general classification and laboratory strength tests which includes: Compaction Test (standard proctor), California Bearing Ratio Test, and Unconfined Compression Test.

CHAPTER TWO

Literature Review

2.1 General

A developing country with a big geographical area and population, such as Nigeria, necessitates extensive infrastructure, such as a network of highways and buildings. Land is being used for a variety of structures ranging from simple houses to sky scrapers, bridges to airports, and rural roads to expressways. Almost every civil engineering structure is built on a different soil layer. A material made up of rock particles, sand, silt, and clay is known as soil. It is generated by the progressive disintegration or decomposition of rocks as a result of natural processes, such as disintegration of rock as a result of pressures caused by temperature fluctuations. In most geotechnical projects, it is not possible to obtain a construction site that will meet the design requirements without ground modification. The current practice is to modify the engineering properties of the native problematic soils to meet the design specifications. Nowadays, soils such as, soft clays and organic soils can be improved to the civil engineering requirements. This state of the art review focuses on soil stabilization method which is one of the several methods of soil improvement. Soil stabilization aims at improving soil strength and increasing resistance to softening by water through bonding the soil particles together, water proofing the particles or combination of the two Sherwood (1993). Usually, the technology provides an alternative provision structural solution to a practical problem. The simplest stabilization processes are compaction and drainage (if water drains out of wet soil it becomes stronger). The other process is by improving gradation of particle size and further improvement can be achieved by adding binders to the weak soils Rogers et al (1996). Soil stabilization can be accomplished by several methods. All these methods fall into two broad categories (FM 5-410) namely; - mechanical stabilization Under this category, soil stabilization can be achieved through physical process by altering the physical nature of native soil particles by either induced vibration or compaction or by incorporating other physical properties such as barriers and nailing. Mechanical stabilization is not the main subject of this review and will not be further discussed. –chemical stabilization under this category, soil stabilization depends mainly on chemical reactions between stabilizer (cementitious material) and soil minerals (pozzolanic materials) to achieve the desired effect. A chemical stabilization method is the fundamental of this review and, therefore, throughout the rest of this report, the term soil stabilization will mean chemical stabilization. Through soil stabilization, unbound materials can be stabilized with cementitious materials (cement, lime, fly ash, bitumen or combination of these). The stabilized soil materials have a higher strength, lower permeability and lower compressibility than the native soil (Keller bronchure 32-01E). The method can be achieved in two ways, namely; (1) in situ stabilization and (2) ex-situ stabilization. Note that, stabilization not necessary a magic wand by which every soil properties can be improved for better (Ingles and Metcalf, 1972). The decision to technological usage depends on which soil properties have to be modified. The chief properties of soil which are of interest to engineers are volume stability, strength, compressibility, permeability and durability (Ingles and Metcalf, 1972; Sherwood, 1993; EuroSoilStab, 2002). For a successful stabilization, a laboratory tests followed by field tests may be required in order to determine the engineering and environmental properties. Laboratory tests although may produce higher strength than corresponding material from the field, but will help to assess the effectiveness of stabilized materials in the field. Results from the laboratory tests, will enhance the knowledge on the choice of binders and amounts (EuroSoilStab, 2002).

When poor soil is encountered on a construction site, the following options for the problems will be considered:

(i) Leaving the poor soil site for a new one.

(ii) Excavation to deep foundation level.

(iii) Removal of the poor soil and subsequent replacement with a more suitable soil.

(iv) Redesigning the structure to match the poor conditions of the soil.

(v) Treating the poor soil to improve its properties.

Option (v) is also known as STABILIZATION. Stabilizing the soil in order to acquire the desired property would be the most suitable choice in situations where alternative sites are not available, cost of burrow materials are very high and land disputes are likely to occur over relocation.

Clay soil (expansive soil), often requires treatment by stabilization. They are characterized by the phenomenon of swelling when moisture is absorbed and as well as shrinkage when moisture is lost; this makes them highly problematic as foundation soils.

Stabilization is aimed at improving the engineering properties of the soil, which may involve the following:

(i) Increasing the soil density

(ii) Increase in cohesion

(iii) Frictional resistance and

(iv) Reduction of plasticity index.

Some of the material that have been used in soil stabilization include Lime, cement, fly ash etc. They have been used solely and in combination with other stabilizing materials to effect a chemical change in the soil.

Eggshell powder (ESP) has not been in use as a stabilizing agent and it could be a good replacement for industrial lime, since it shares a similar chemical composition to that of lime. Chicken shell is a waste material from domestic sources such as poultries, hatcheries, homes and food joints/restaurants. This amounts to environmental pollution. Egg shell waste if subjected to adequate scrutiny, they could be suitable for soil stabilization.

The appalling state of Nigerian economy has brought about inflation, resulting in a continuous increase in prices of materials. The use of lime for stabilization is therefore becoming very expensive therefore requiring an economical replacement. Egg shells are primarily known to contain lime, calcium and protein. It has been used in agriculture to replace lime, which confirms that lime is present in a considerable amount in egg shells. Now, we need to ask ourselves: should we continue to buy lime at a high cost when we can get a cheap replacement in eggshells?

There have been findings revealing that ESP has been used for stabilizing cohesion less soils in different countries. Therefore this study is thus directed towards identifying eggshell powder as an effective stabilizing agent by replacing a certain percentage of the lime in the stabilized soil with ESP. Since the quantity of eggshells that may be required for stabilization of a large area may not be met, it is suggested that the ESP be used as a supplement in the lime stabilization.

In a well-organized environment, disposal of waste poses a great difficulty, as regards where and how to effectively dispose of the waste product without any adverse effect on the society. Researches have shown, in the year 2018 global egg production was 78 million metric tons, contributing approximately 8.58 million metric tons of egg shells which was discarded mostly as waste Waheed et al (2020). In the United Kingdom, the estimated quantity eggshell waste to be disposed each year is between 10,000 - 11,000 tons. Thus, an estimate of about 5,000 - 6,000 tons could also be expected in Nigeria annually, therefore, in the absence of an effective waste disposal policy, the utilization of eggshells for soil improvement, either for agriculture or engineering purposes will be a welcome development.

2.2 Clay Soil

Clay minerals are called secondary silicates, because they are formed from the weathering of primary rock-forming minerals. Clay minerals occur in small particle sizes (<0.002 mm) and are very fine grained and flake shaped; they are separated from sand, gravel and silt due to the negative electrical load on the crystal edges and positive electrical load on the face. Clay minerals consist of two basic structures. First, silica tetrahedral sheet is formed through the bonding of silicon ions to the oxygen atoms on all four sides (tetrahedron). Second, an octagon forms with aluminum and magnesium ions coordinated on eight-sides with oxygen and hydroxyl ions (octahedron). All clay minerals are formed from octahedral and tetrahedral sheets with certain types of cations, which are in various forms and connected to each other in a certain system. Changes in the structures of the octahedral and tetrahedral sheets result in the formation of different clay minerals. More common clay mineral groups include kaolinite, illite and smectite (montmorillonite). Kaolinite consists of silica and alumina plates, and these plates are connected very strongly, because kaolin clay is very stable (Figure 2.1a). Illite has layers made from two silica plates and one alumina plate (Figure 2.1b). However, Illite contains potassium ions between each layer; this characteristic makes the structure of the clay stronger than smectite. Smectite has layers made from two silica plates and one alumina plate. Because there is a very weak bond between the layers, large quantities of water can easily enter the structure (Figure 2.1c). This event causes the swelling of such clay.

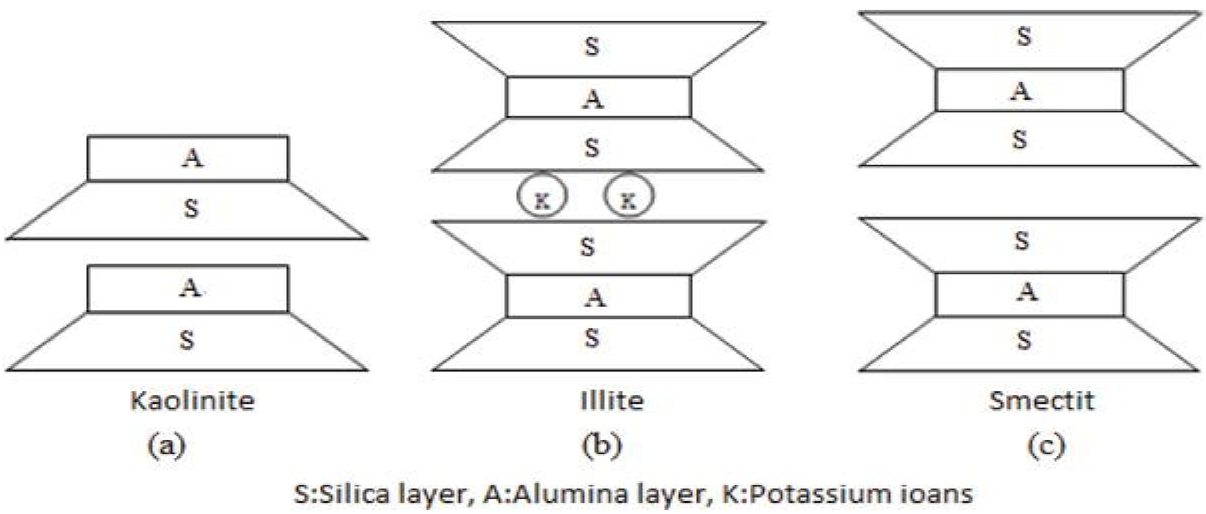
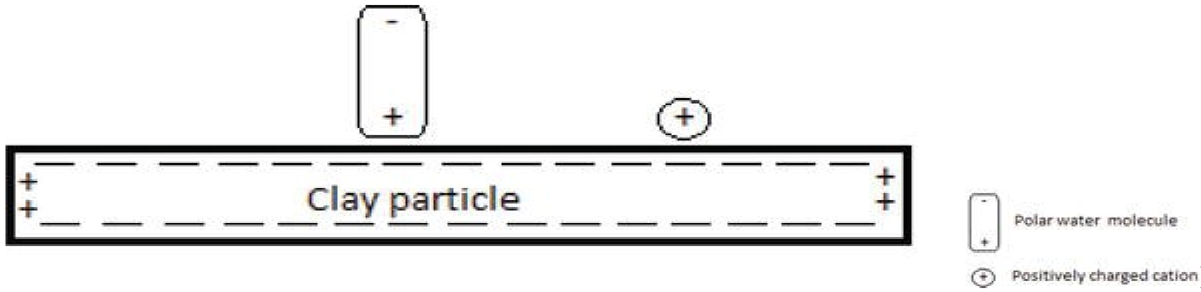


Plate 2.1: Display of structure of common clay minerals.

Source: (Braja, 2002).

2.2.1 Clay Properties

Certain features of the clay affect the structure of the soil, which determines its properties such as strength, hydraulic conduction, settlement and swelling. These features include isomorph substitution and surface anion and cation exchange capacity, Hillier, S. (2003).This event is called isomorphic substitution if the octahedral or tetrahedral sites are replaced by a different atom normally found elsewhere. The specific surface area is the property of solids, which is defined as the total surface area of a material per unit of mass. With the separation of hydroxyl ions from the clay surface, which results in crystal deficiency at the crystal head, anions subsequently attach to the surface and organic molecule content causes an electrical load imbalance. This imbalance results in clay’s extreme affinity to water and cations in the environment (Figure 2.2). Water is a dipolar molecule, namely, it has one positive and one negative charge. The surface of the clay crystal is electrostatically held to the water molecule. In addition, water is held to the clay crystal by hydrogen bonding. Also, negatively charged clay surfaces attract cations in the water. The cation/anion changes in the clay minerals are different between clay minerals. Therefore, it is expected that the clay that attracts more water molecules to the surface will have more plasticity, more swelling/shrinkage and more volume change, depending on the load on it. Thus, water influences clay minerals. For example, the water content changes consistency limits and this affects the ground plasticity. Ultimately the change in clay plasticity directly affects the mechanical behavior of the soil. Studies generally accept clays as fully saturated in geotechnical engineering. Therefore, the behavior of clays is affected by the individual clay particle arrangements and pore water content. The surfaces of clays are negatively charged, and so they tend to adsorb the positively charged cations in pore water. In this way, the cations on the surface of a clay particle that are entering the water spread into the liquid. This spreading is called the double layer. Briefly, the cations are distributed around the negatively charged surface of the clay particles, with the greatest density near the surface and decreased density with increasing distance from the surface. The cations form a positively charged layer and the double layer is created with a negatively charged surface of the clay particles. The double layer affects the arrangements of the clay particles, and hence, the physical and mechanical properties of the soil are also affected. The interaction of these forces controls the engineering behavior of soils to a great extent. At the same time, this interaction leads to the formation of different compositions and settlements in the soil planes, which are defined as structures in clay soils. Environmental temperature, precipitation, groundwater level and pH and salinity all play roles in clay properties, as well as in the conversion of rock into clay. Clay derived from the same rock can be different under different environmental conditions.



Plat

e 2.2: Clay particle and surface charge display.

Source: (Braja, M.Das, 2002).

2.2.2 Structure of Clay and Physio-Chemical Properties

- Around the clay that is faced with a liquid, there are distance-varying push-pull curves. If there is a force lifting the two clay minerals, the particles clump together. This is called flocculation. If the net force is thrust, the particles are separated from each other; this is called dispersion. Particle orientations of soils vary between flocculated and dispersed (Figure 2.3). Forces between the particles are important for clay, because the behavior of clay depends on the geological history and structure. This difference in the orientation of fine-grained soils affects the engineering behavior of the soil. The geological process in the formation of soils in nature constitutes the arrangement of soils, Weaver and Pollard (1973).For this reason, geotechnical engineering studies are interested in the physical and mechanical behavior of soils-bearing structures, as well as the strength between the structure, texture and performance of soils. There are many studies on the soil-orientation effects on the soil properties such as strength, hydraulic conductivity and the swelling-shrinkage relative to each particle. O.G. Ingles et al. (1966) examined soil fabric during consolidation. Due to an increase in the degree of particle orientation, the total volume of the voids was reduced.

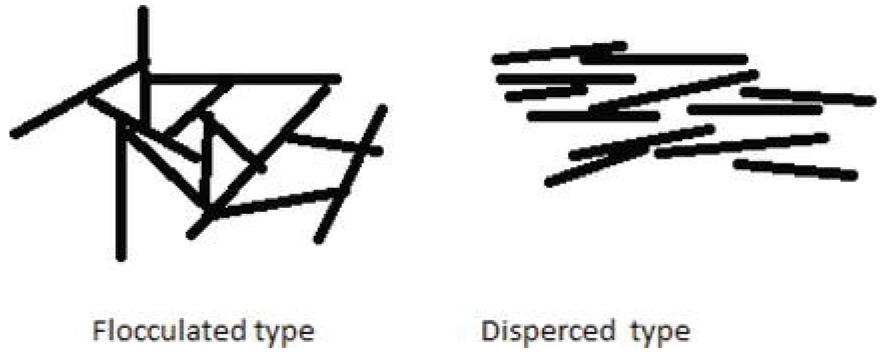


Plate 2.3: Clay particle orientations.

Source: (Braja, 2002).

Flocculation increases depending on the electrolyte concentration, ion valence, temperature, decreasing dielectric constant, hydrated ion diameter, pH value and surface absorbed ions. Soil engineering properties depend on the size, shape, a high amount of surface area and a negative surface charge of clay particles. In 1925, Terzaghi proposed the concept of clay arrangement. He said that clay minerals stick to each other at the points of contact, with forces sufficiently strong to construct a honeycomb structure. In 1932, it was shown by Casagrande that this honeycomb shape is a special structure in clay-containing soils and this structure can vary depending on many characteristics of the environment. Figure 2.4 shows further compression with progress of soil sedimentation. Later, other researchers also proposed fabric models.

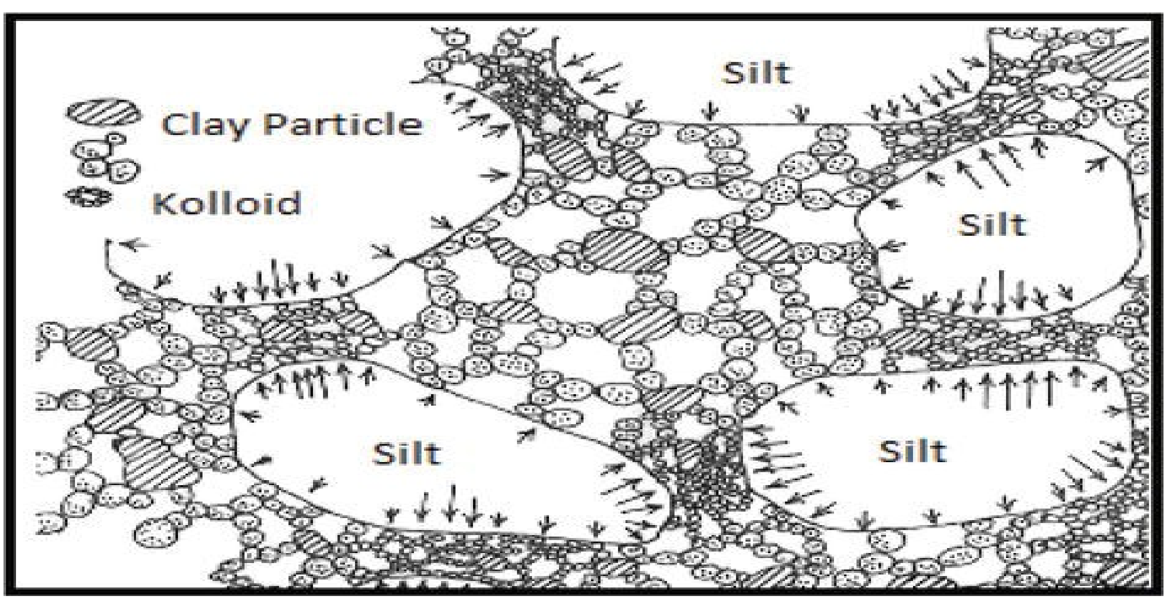


Plate 2.4: Casagrande’s fabric model (1932).

Source: (Braja, 2002).

Collins and McGown (1973) defined the elementary particle arrangement, particle assemblages and pore spaces in the fabric model. Researchers provided an elementary particle arrangement, a single clay, silt or sand, which is shown in Figure 2.5a and 2.5b; the group effect of clay plates is shown in Figure 2.5c, and the interaction between silt and sand is shown in Figure 2.5d. Particle assemblages contain one or more elementary particle arrangements or small particle clusters. Pore spaces are defined with spacing between elementary particle arrangements and particle assemblages. Bennet and Hulbert suggested that the fabric of soils is mostly determined by the physical arrangement of particles, which is gained at the time of sediment deposition by the physical–chemical conditions of the depositional environment. The fabrics of soils describe clusters, clusters are formed by other clusters and space between the clusters and structure of the soils describes the fabric, mineral content and decontamination forces. Also, the fabrics of soils can sometimes be visualized under a microscope. The structure of soils can be examined more extensively by an X-ray diffractometer (XRD) and a scanning electron microscope (SEM).

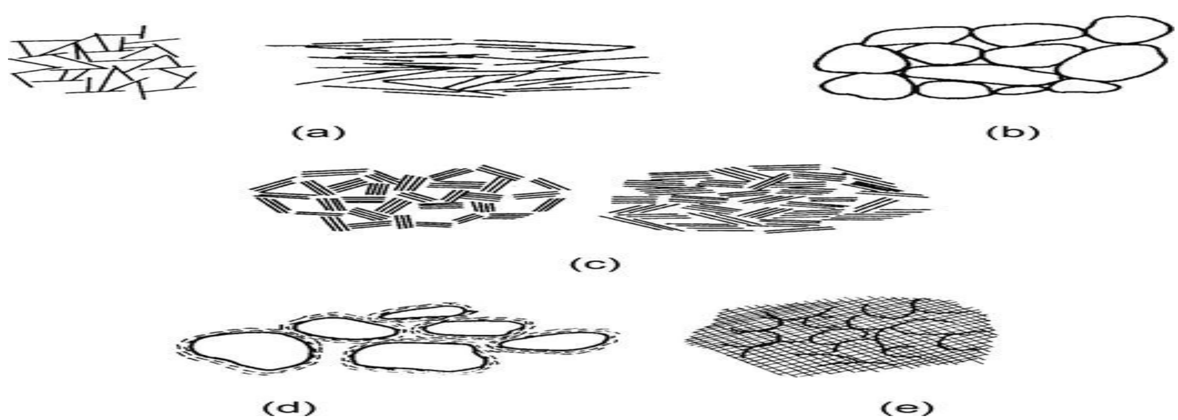


Plate 2.5: Arrangement of clay particles.

Source: (Braja 2002).

(a) Elementary particle arrangements of clay

(b) Elementary particle arrangements of sand and silt

(c) Clay assemblages

(d) Clay coated silt and sand arrangement

(e) Not fully defined arrangement.

2.2.3 Role of Clay in Geotechnical Engineering

Studies on soil behavior that do not consider the physico-chemical and microstructural properties of clay soils may be missing important information regarding the soil's physical and mechanical properties. This is because most physical and mechanical behaviors can be explained by the soil's physico-chemical and microstructural properties. In general, clay is an unwanted material because it creates significant engineering problems. Unlike other minerals of the same size, clay forms mud when mixed with water. Clay has plasticity and can be shaped into dough, and when cooked it turns into a solid with great strength increments. Clay generally shows a volume increase when wet, and when it is dried, its volume decreases, which creates many cracks.

2.2.4 Physical and Mechanical Behavior of Clay.

In geotechnical engineering, it is important to identify a clay type, as the type directly affects the important properties of clay, such as Atterberg's limits, hydraulic conductivity, swelling-shrinkage, settlement (compression) and shear resistance. Atterberg's limits, known as consistency limits, define the relationship between ground particles and water and the state of the soil relative to varying water contents. With increasing moisture content, clay changes from solid state, to semisolid state, to plastic state and to liquid state, which is given in Figure 2.6. In Figure 2.6, the clay-water mixture shows a total volume reduction, which is equivalent to the volume of water lost around the liquid and plastic limits, as the clay transitions from liquid to dry, and if the decrease in water content continues, no reduction in volume is observed. Braja (2002). This limit value is called the shrinkage limit. Therefore, the shrinkage limit is the moisture content at which the soil volume will not reduce further if the moisture content is reduced. The plastic limit is the moisture content at which the soil changes from a semisolid to a plastic (flexible) state. The liquid limit is the moisture content at which the soil changes from a plastic to a viscous fluid state. In geotechnical engineering, the liquid and plastic limits are commonly used. These limits are used to classify a fine-grained soil, according to the Unified Soil Classification system, AASHTO system or TS1500 (Turkey).

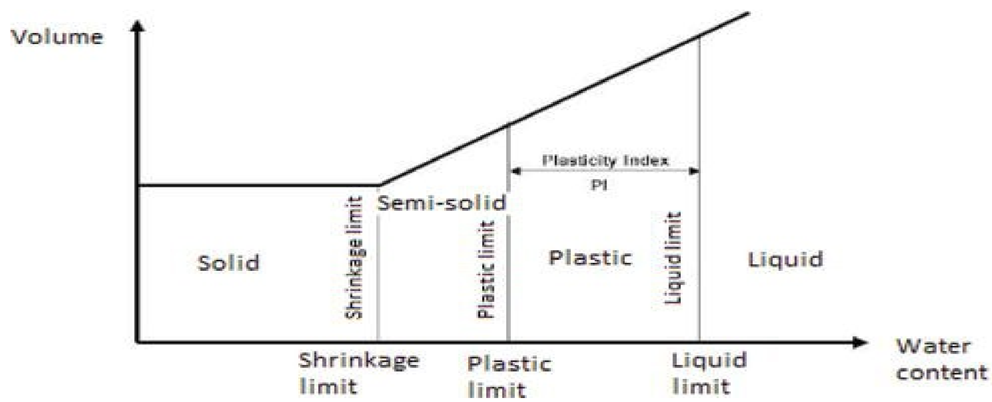


Plate 2.6: Water content-volume relationship of soils.

Source: (Braja 2002).

2.2.5 Hydraulic Conductivity Properties of Clay

Water is a problem in geotechnical engineering, such as water in voids in the ground mass, flowing in pores, or in the pressure or stress that water creates in the pores. Clay plays an important role in the emergence of water problems, especially on fine soils, and these problems include permeability, shear resistance, setting and swelling problems, Braja (2002). In addition, capillarity, freezing and infiltration can be additional issues. Structures built on clay and slope stability are particularly problematic when affected by water. Dams and dikes also cause the destruction of structures without leakage and piping. Therefore, it is necessary to estimate the quantity of underground seepage under various hydraulic conditions to investigate problems that involve pumping water for underground construction and for stability analyses of earthen dams and earth-retaining structures that are subject to seepage forces.

The hydraulic conductivity coefficient commonly used in geotechnical engineering is also used for permeability. Hydraulic conductivity is a property that expresses how water flows in the soil. Soils are permeable due to the existence of interconnected voids, through which water can flow from the points of high energy to the points of low energy. Fluid viscosity, pore-size distribution, grain-size distribution, void ratio, roughness of particles and the degree of soil saturation affect the hydraulic conductivity of soils, Braja, M.Das, (2002). Clay soil has electrical ions, so the hydraulic conductivity of clays affects the ionic concentration and thickness of water layers held to the clay particles. Table 2.1 provides the typical values for soils. The hydraulic conductivity value of soils determines the constant head test (for coarse soils) and the falling head test (for fine-grained soils).

Table 2.1: Hydraulic conductivity of soils.

SOIL TYPE	K (CM/S)
Clean gravel	100 – 1.0
Coarse sand	1.0 – 0.01
Fine sand	0.01 – 0.001
Silty clay	0.001 – 0.00001
Clay	< 0.000001

Source: (Braja, 2002).

2.2.6 Swelling-Shrinkage Behavior of Clay

The effect of swelling-shrinkage on fine-grained soils is often seen as a problem in geotechnical engineering applications. Shrinkage behavior in clay soils is effective in reducing the strength in a slope and a foundation's bearing capacity. Shrinkage is usually visible from evaporation in dry climates, reduction of groundwater and sudden arid periods. Swelling can be seen due to rising water. These volume changes are harmful to heavy construction and road coverings. Swelling occurs when the inflation pressure is greater than the pressure from the covering or structure. The material damage from the swelling-shrinkage of soils is more likely to occur in the United States due to greater water pressure, floods, typhoons and earthquakes.

Jones and Holtz estimated that shrinking and swelling soils cause approximately \$2.3 billion in damage annually to small buildings and road surfaces in the United States. This amount of damage is twice the amount of damage incurred from floods, earthquakes and hurricanes. Krohn and Slosson (2012) estimated that swelling soils cause approximately 200 milliion naira in damage each year. According to Holts and Hart 60% of 250,000 newly constructed homes incur minor expansive soil damages and 10% incur significant expansive soil damage each year in the United States. Coduto (2001) noted that expansive soils caused 16 million naira in damage to a building over a 6-year period. The estimated annual cost due to significant structural damages, such as cracked driveways, sidewalks and basement floors, heaving of roads and highway structures, condemnation of buildings; and disruption to pipelines and other utilities in Colorado, is 16 billion naira, according to AMEC.

Swelling pressure depends on the type of clay mineral, soil structure and fabric, cation exchange capacity, pH, cementation and organic matter. Any cohesive soil can involve clay minerals, but montmorillonite or bentonite clay minerals are more active regarding swelling-shrinkage. Swelling is calculated by swelling experiments with chemical and mineralogical analysis, soil indices and some empirical formulas from soil classifications. The shrinkage limit is determined from a laboratory test or approximate calculation recommended by Casagrande. Properties of clay improve with chemical additives such as cement, lime, lime-fly ash, cement-fly ash, calcium chloride and so on.

Structures transfer loads to the subsoil through their foundations. The imposed stress from the structure compresses the subsoil. This compression of soil mass leads to a decrease in the volume of the mass, which results in the settlement of the structure, and this should be kept within tolerable limits. Therefore, settlement (compression) should be estimated before construction. The settlement is defined as the compression of a soil layer due to the construction of foundations or other loads. The compression is seen in deformation, relocation of soil particles and expulsion of water or air from void spaces. Braja (2002).

In general, the soil settlement under load falls into three categories: immediate or elastic settlement, which is caused by the elastic deformation of dry soil or moist and saturated soils without change in the moisture content; primary consolidation settlement, which is the result of a volume change in saturated cohesive soils because of the expulsion of water occupying void spaces; and secondary consolidation settlement is the volume change under a constant effective stress due to the plastic adjustment of soil fabrics. The consolidation settlement is seen when a structure is built on saturated clay or the water level is permanently lowered. Simultaneously, consolidation settlement is seen under its own weight or the weight of soils that exists above the clay, Braja (2002). The consolidation settlement of clay takes a long time, and the reason for this is the low hydraulic conductivity and slow drainage of clay. Settlement of the soil is determined by one-dimensional consolidation (odometer) and hydraulic consolidation (Rowe). In experiments, the vertical loads and void ratios are recorded. Afterwards, the relationship between the pressure and void ratio is determined from the measured data. These data are also useful in determining the consolidation coefficient. The consolidation coefficient is determined by the root of time method and the log-t method. Figure 2.7 shows the relationship between the void ratio and stress for a typical odometer test for consolidation.

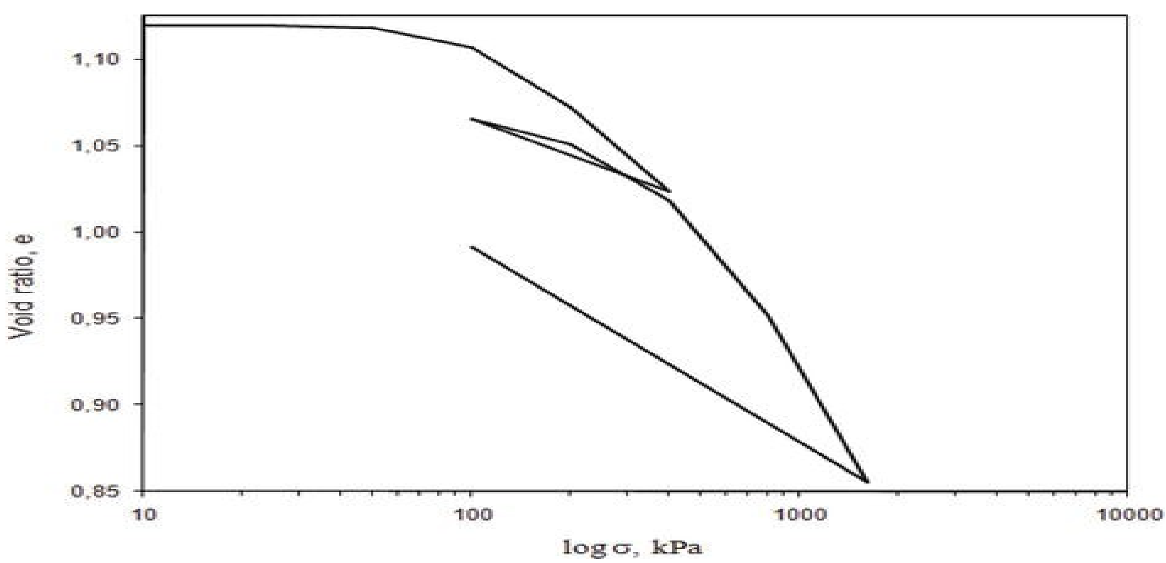


Plate 2.7: The graph of a typical test for consolidation test by oedometer.

Source: (Braja 2002).

2.2.7 Shear Strength Behavior of Clay

The shear strength of soils is one of the most important aspects of geotechnical engineering. The strength of the soil provides safety for geotechnical structures. The bearing strength, slope stability and bearing wall of the bases are influenced by the shear strength of the soils. Failure in the soils occurs in the form of shear. If the stresses in the soil exceed the shear strength, failure occurs. The shear failure of the soil depends on the interactions between the soil particles. These interactions are divided into friction strength and cohesion strength. When the clay soils are subjected to shear, the volume change in the drainage shear depends on the environmental pressure, as well as the stress history of the soil. In addition, loading on clay soils does not allow water to escape from the pores, and thus, this creates excess water pressure. Braja, M.Das, (2002). If the loading does not cause failure, the excess water pressure is dampened, consolidation occurs and volume change is observed. The long process of this volume change in the clays is due to very low hydraulic conductivity. Determination of the shear strength of the clay is performed by a direct shear test, triaxial compression test, vane test and standard penetration tests. Figure 2.8 presents the relationship between the shear stress and normal stress for a typical shear strength test and triaxial compression test. After the failure envelope is drawn, the cohesion (c) and internal friction angle (f) are obtained.

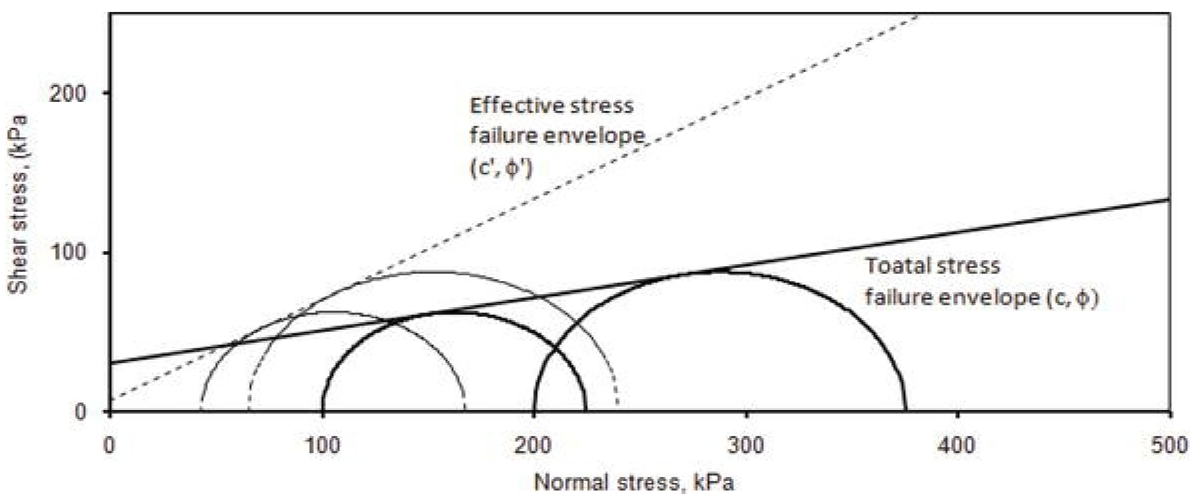


Plate 2.8: The graph of a typical test for shear strength test by triaxial compression test. Source: (Braja 2002).

2.2.8 physio-Chemical and Microstructure Behavior of Clay

For the determination of the physio-chemical and microstructural properties of clay soils, X-ray diffractometer (XRD) and scanning electron microscope (SEM) are commonly conducted. In addition, to determine the physico-chemical properties and structure of the soils, a pH test, electrical conductivity, cation exchange capacity, helium pycnometer, mercury intrusion porosimetry (MIP), surface area analysis (SSA), Brunauer-Emmett-Teller (BET) method or likewise, zeta potential and wavelength dispersive X-ray fluorescence test and Differential Thermal Analysis (DTA) are conducted, O.G. Ingles et al. (1966).The pH value indicates the degree of H+ or OH− ions present. The change in pH affects the soil-water relations. Low pH indicates flocculation, and high pH indicates dispersion. The electrical conductivity of clay is defined by its ion number and type. Cation exchange capacity is a measure of isomorph displacement capacity. Isomorph displacement is when other ions of equal or different valence to those of the ions are left. This change emerges from the unbalanced electrical charge for every change. To prevent this imbalance, the cations in the environment enter the edges of the clays and between the blocks.

X-ray diffractometer (XRD) analyses: The mineralogical composition of soils is critical due to its significant influence on soil behavior; the soils are affected at first degree, especially by physical, chemical and mechanical properties of clay and by the mineral content. In geotechnics, it is important to find the type of minerals present in clay, as well as their proportions to understand the mechanical behavior. The XRD curve for typical clay is displayed in Figure 2.9. The X-ray diffraction patterns of clay show a mineralogical composition of montmorillonite, anorthite, quartz, calcite and silica.

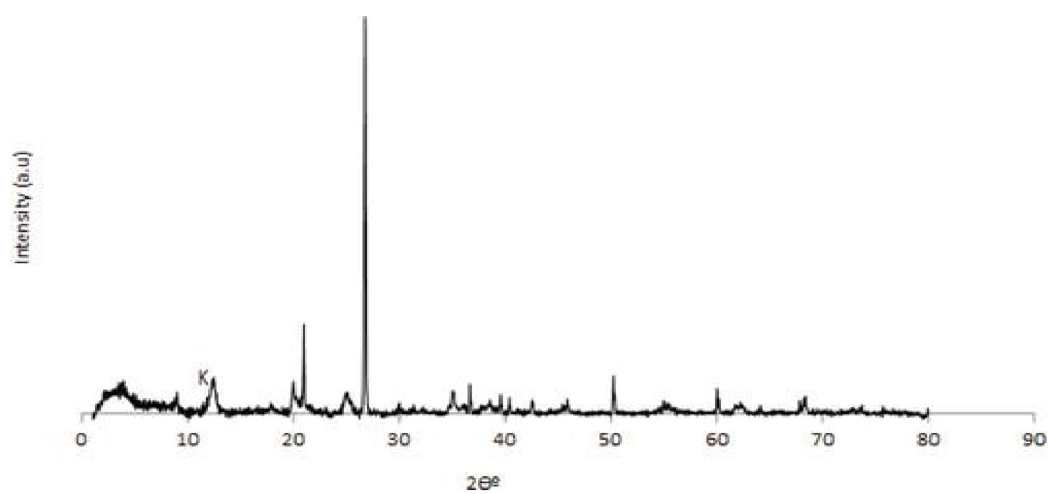


Plate 2.9: The XRD curve for typical clay.   Source: (Braja 2002).

Mercury intrusion porosimetry (MIP) analyses: In geotechnical engineering, the pore-size distributions for clay significantly influence the geotechnical behavior of soil. The pore-size distributions for typical clay from the MIP tests are displayed in Figure 3.0. This figure shows the relationship between incremental intrusion and pore-size diameter.

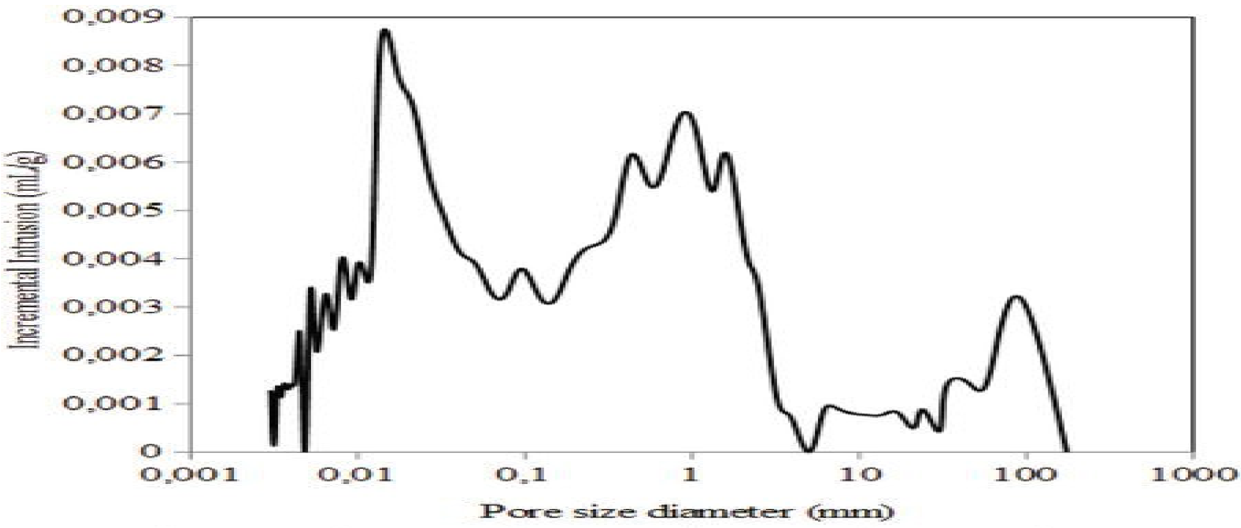


Figure 3.0: The pore size distributions for typical clay from the MIP tests.

Source: (Braja 2002).

Scanning electron microscope (SEM): The microstructure of soils, especially clays, is observed using a versatile, analytical and ultrahigh-resolution field-emission SEM. An SEM provides a high level of magnification. Soil specimens that are magnified up to 1,000,000 times enable the evaluation of differences in the surface by imaging the surface structures. The changes in the microstructural development of soils play a significant role in the behavior of soils. In particular, these parameters could lead to a better understanding of the engineering properties of compacted soils. The SEM images of typical clays are present in Figure 3.1. Thus, flocculated and dispersed structures are observed in the soil samples.

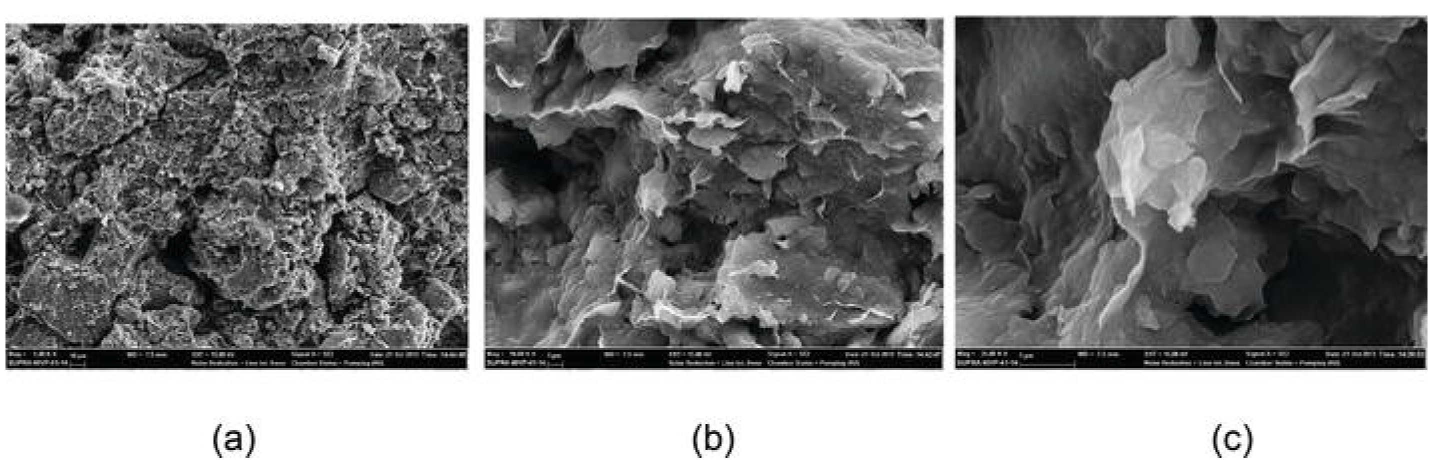


Plate 3.1: The SEM images of typical clay for different magnification (a. 1000×, b. 10,000×, c. 35,000×).

Source: (Braja 2002).

Surface area analysis (SSA): The specific surface area is affected by grain-size distribution and the types and amounts of different clay minerals. Specific surface area is affected by the physical and chemical properties of soils.

2.2.9 Expansive Soil

Expansive soils are soils which have the tendency to increase in volume when water is absorbed and to decrease in volume when water is removed. These volume changes are the causes of damages/problems in constructions built on expansive soils as its subgrade material without adequate stabilization and treatment.

The swelling potential of the expansive soil is directly related to the plasticity index of the soil thus the higher plasticity index of the soil the greater the quantum of water that can be imbibed within the expansive soil structure and hence the greater the swelling potential, Croney, D., Coleman, J.D. (1954). Furthermore, a low shrinkage limit indicates that the expansive soil would begin to swell at a low water content. It is also necessary to note that a high colloidal content means a greater possibility of expansive high Liquid Limit also means high Moisture Content.

Many criteria are available to identify and characterize expansive soils, such as Liquid Limit (Table 2.2), Plasticity Index (Table 2.3), Shrinkage Limit, Shrinkage Index, Free swell index, Activity and percent of Free Swell (Table 2.4).

Table 2.2: Soil Expansivity Predictions by Liquid Limit

Degree of Expansion	Liquid Limit (%)	
	Chen	IS 1498
Low	<30	20-35

Medium	30-40	35-50
High	40-60	50-70
Very High	>60	70-90

Table 2.3: Soil Expansivity Predictions by Plastic Limit

Degree of Expansion	Plastic Limit (%)	
	Chen	IS 1498
Low	0-15	<12
Medium	10-35	12-23
High	20-55	23-32
Very High	>35	>32

Table 2.4: Soil Expansivity predictions by other Measures

Degree of Expansion	Colloid Content	Shrinkage Limit	Shrinkage Index	Free Swell	Expansion in Oedometer

	(% minus 0.001)mm	(%)	(%)	Index (%)	As per Holts and Gibbs (%)	As per Seed et al (%)
Low	<17	>13	<15	<50	<10	0-1.5
Medium	12-27	8-18	15-30	50-100	10-20	1.5-5.0
High	18-37	6-12	30-60	100-200	20-30	5-25
Very High	>27	<10	>60	>200	>30	>25

Source: (Braja 2002).

Expansive soils are mainly clay soils of fine grained particles, plastic, cohesive, and almost impervious with high compressibility and poor shear strength. These characteristics make it unsuitable for use as a subgrade in flexible parameters. Clay minerals are basically formed from the chemical weathering of rock forming minerals with the main mineral composition of hydrous aluminosilicates $(OH)_4\ O_{10}$ and other metallic ions. There are 3 major types:

a. Kaolinite = $(OH)_4\ AL_4\ Si_4\ O_{10}$
b. Montmorillonite = $(OH)_4\ Al_4\ Si_4\ O_{10}\ 4H_{20}$
c. Illite $(OH)_4$ KySigng Aly) $(Al_4\ Mg_6\ Fe_4\ Fe_6)\ O_{20}$

Montmorillonite swells considerable upon imbibing water than other types of clay.

2.3 Egg-Shell Powder

Eggshell consists of several mutually growing layers of $CaCO_3$, the innermost layer-maxillary layer (=100 μm) grows on the outermost egg membrane and creates the base on which palisade layer constitutes the thickest part (=200 μ) of the eggshell. The top layer is a vertical layer (=5.8 μ) covered by the organic cuticle.

Tocan(2003) stated that eggshell primarily contains calcium, magnesium carbonate (lime) and protein. He however evaluated the quantity of the lime in eggshell waste to be as almost the same as ground chalk or limestone tonne for tonne. Eggshell waste does have a theoretical value either as an animal feed or as a fertilizer or lime substitute. In many other countries, it is the accepted practice for eggshell to be dried and use as a source of calcium in animal feeds. The quality of lime in eggshell waste is influenced greatly by the extent of exposure to sunlight, raw water and harsh weather conditions. Froning and Bergquist (1990), specified that eggshell waste should be ground not more than 2 days after recovery from source to prevent depletion of lime content.

The physical nature of the shell waste and the foul rotten egg odors produced when the material degrades, reduce the lime value and renders the waste difficult to recycle to land. Ideally, the waste should be dried at source, transported to a site where it would be finely ground immediately and used as source of lime to agriculture and for other applications. In order to maximize the recycling opportunities for eggshells, Froning and Bergquist (1990), recommended that eggshell waste should be incinerated independently of other wastes so that the calcium/magnesium content of the shell will be converted into calcium/magnesium oxide (quicklime) and the resultant burnt lime could be used as a liming agent.

2.4 Lime

The word “lime” refers to products derived from heating limestone. It originates with its earliest use as building mortar and has the sense of “sticking or adhering“. The rocks and minerals from which these materials are derived, typically limestone or chalk, are composed primarily of calcium carbonate (CaCO3).

2.4.1 Types of Lime Used in Construction

The four main types of limes used in construction are:

i. Quick Lime
ii. Slaked Lime
iii. Fat Lime
iv. Hydraulic Lime

i. Quick Lime

Quick lime is also called caustic lime (Calcium Oxide). This is produced by burning pure limestone (Calcination process). It is the cheapest form of lime available which is highly amorphous and caustic. Quick lime has a great affinity towards moisture. Quick lime is one of the key ingredients in the manufacture of cement. This is also an important material used for the treatment of drinking water.

ii. Slaked Lime

Slaked lime is obtained by the process of slaking, where quick lime is combined with water. Slaked lime is also known as Hydrate of lime. This is available as pure lime in the form of a white powder. Slaked lime when exposed to the atmosphere, absorbs carbonic acid in the presence of water content. It is also called as calcium hydroxide or calcium hydrate or lime hydrate. Slaked lime formed by adding quick lime and water gives a slurry material. This is highly suitable for mortar applications. It can also be used in plastering works and in cement as a binder.

iii. Fat Lime

The lime that contains calcium oxide in high content is called as fat lime. This is also called as white lime or rich lime or high calcium lime or pure lime. It slakes vigorously when water is added to it. This slaking increases its volume to two and a half times greater than that of quick lime. Fat lime is used for pointing in masonry works, foundation, with surkhi to thicken the masonry walls etc.

iv. Hydraulic Lime

Hydraulic lime is also called as water lime. It contains 30 percent of silica and 5 percent of alumina along with iron oxide. This lime sets when water is added to it. Hydraulic lime has the following classification:

a. Eminently Hydraulic: Eminently hydraulic lime has a chemical composition similar to Ordinary Portland cement (OPC). This consists of 25 to 39 percentage of silica and alumina. This type can be used for good quality mortar works.

b. Semi-Hydraulic: Semi-hydraulic lime consists of 15 to 25 percentage of silica and alumina. It is used for good quality works of masonry, brickwork, and mortar. This is less strong than eminently hydraulic lime.
c. Non-Hydraulic: Non-hydraulic lime has silica and alumina in quantities of less than 15%. On the process of slaking, it undergoes less expansion. For less important works, mortar made from non-hydraulic lime is accepted.

2.4.2 Uses of Lime in Construction

Major uses of lime are enlisted below:

i. Lime is used in the manufacturing of steel in order to remove the impurities.
ii. Soil stabilization for construction of roads, airfields and building foundation demands lime in large quantity.
iii. Lime slurry is used as mortar for masonry work and for plastering.
iv. Lime is used as an additive in asphalt which helps to improve the property of cohesion. Lime helps to increase the resistance of asphalt towards stripping and aging.

2.4.3 Lime Production

When limestone (calcium carbonate) is heated, at about 1000 °C it undergoes thermal decomposition. It loses carbon dioxide and turns into quicklime (calcium oxide).

$$\text{calcium carbonate} \xrightarrow{\text{Heat}} \text{calcium oxide} + \text{carbon dioxide.}$$
$$CaCO_{3(s)} \rightarrow CaO_{(s)} + CO_{2(g)}$$

Plate 3.2a: thermal decomposition of calcium carbonate

Source: (Braja 2002).

The reaction is carried out in specially constructed lime kilns (a kiln is a high temperature oven). Limestone is added at the top and quicklime is removed from the bottom in a continuous process.

Plate 3.3: Lime Cycle

Source: (Braja 2002).

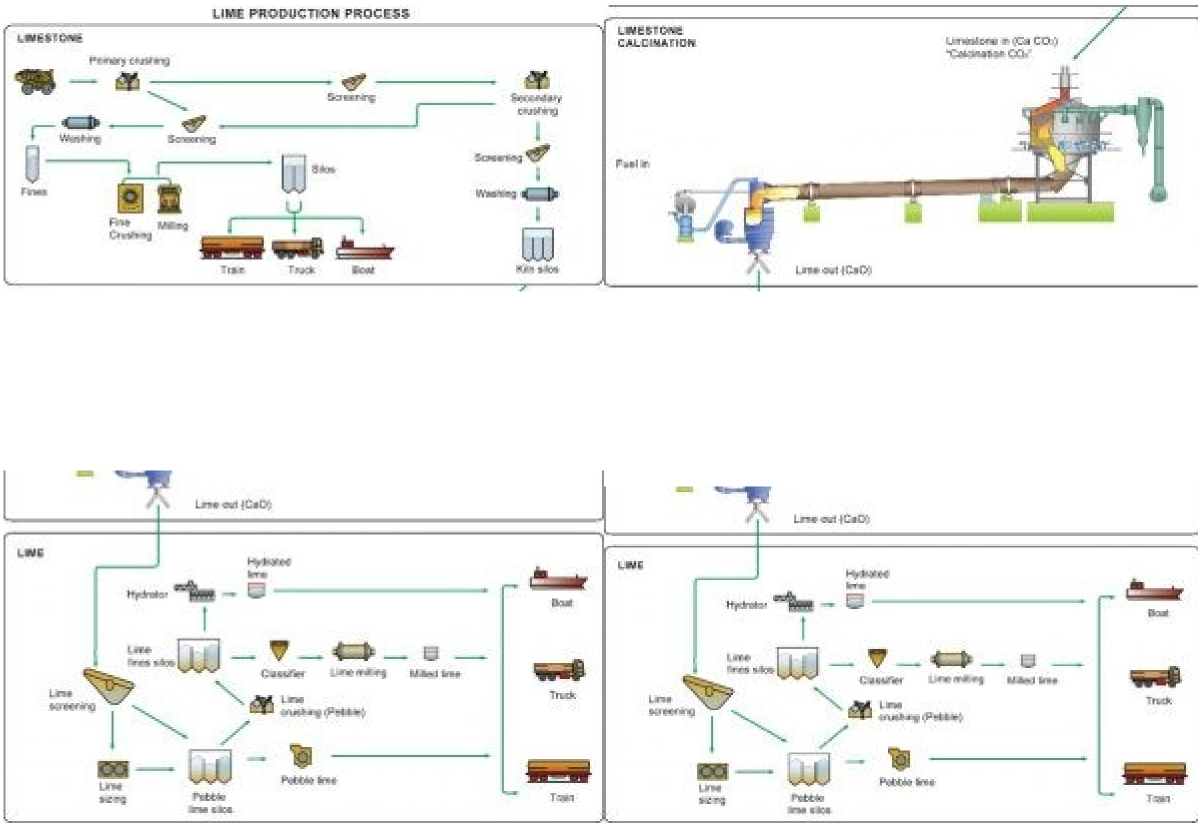


Plate 3.4: Lime Production

Source: (Braja 2002).

CHAPTER THREE

MATERIALS AND METHOD

3.1 collection of Materials

a) Clay Soil: The soil sample used were collect from burrow pits in Efab Properties Estate Amansea Awka, which is a new estate project being built along Enugu-Onitsha Express-way. The soil sample collected was gotten with the aid of a digger from the site. The samples were collected in cement bags and transported to the schools laboratory.

b) Lime: Slaked lime also called hydrated lime, calcium hydroxide ($Ca\,(OH)^{2}$) is a white powder produced commercially when quicklime is mix with water. The slaked lime used was purchased at Eke Awka market from there it was conveyed to the school laboratory and was stored properly.

c) Egg Shell Powder (ESP): Eggshells consist of calcium carbonate, it is a major chemical component required to generate binder gel (Calcium Silicate Hydrate) in cementitious materials. Therefore, in powder form, it can be used as cement replacement in construction materials. The egg shells where gotten from fast food restaurants in Ifite and were sun-dried before being pounded and then blended into a fine powder passing through a 75μ sieve.

3.2 Methodology of Study

The following methods were applied during the course of carrying out this research;

3.2.1 Specific Gravity Test

Specific gravity is the ratio of mass of unit volume of soil at a stated temperature to mass of equal volume of gas-free distilled water at the same temperature Krishna (2002). Also as defined by Braja (2006), Specific gravity can be defined as the ratio of unit weight of a material to unit weight of water. The specific gravity of soil solids is often needed for various calculations in soil mechanics. It can be determined accurately in the soil laboratory. Table 3.1 shows the specific gravity ranges for some common minerals found in the soil.

Table 3.1 Ranges of Specific Gravity Values for different Mineral.

MINERAL	SPECIFIC GRAVITY
Quartz	2.65
Kaolinite	2.6
Illite	2.8
Montmorillonite	2.65 – 2.80
Halloysite	2.0 – 2.55
Potassium Feldspar	2.57
Sodium and Calcium	2.62 – 2.76
Chlorite	2.6 -2.9
Biotite	2.8 – 3.2
Muscovite	2.76 – 3.1
Homblende	3.0 – 3.47
Limonite	3.6 – 4.0
Olivine	3.27 – 3.7

Source: (Braja 2002).

The apparatus employed for this experiment includes:

i. Density bottle of 50ml capacity and a stopper.
ii. Desiccator containing anhydrous silica gel.
iii. Thermostatically controlled oven with temperature of about 80-110OC.
iv. Weighing balance of 0.01g sensitivity.
v. Mantle heater.
vi. Plastic wash bottle.
vii. Distilled water.
viii. Funnel
ix. Thin glass rod for stirring.
x. 425um Sieve.
xi. Dry piece of cloth for cleaning.
xii. Masking tape for identification of sample.
xiii. Exercise book and pen for recording of result.

Plate 3.4: Density bottle and Funnel

The Procedure for Computation of result obtained are as follows:

Specific gravity (GS) = $\frac{(M2-M1)}{(M2-M1)-(M3-M4)}$ (3.1)

Where M1= weight of density bottle + stopper

M2= Weight of density bottle + air-dried soil + stopper.

M3= Weight of density bottle filled with water + wet soil + stopper.

M4= Weight of density bottle filled with water + stopper

Test Procedure

The density bottle used was properly cleaned and was rinsed with distilled water before been placed in a desiccator to remove any moisture present. The empty bottle was weighed together with the stopper and the weight recorded as (M1). About 10-15g of soil passing through sieve 425micrometer was poured into the density bottle and the weight of density bottle +dry soil + stopper recorded as (M2). Distilled water was added to fill about half to three-fourth of the density bottle and it was allowed to saturate for 24hours. The density bottle was gently stirred using thin glass rod and thereafter connected to a mantle heater to de-air the sample. The density bottle was allowed to cool at room temperature and then filled with distilled water up to the specific mark (at lower meniscus level). The exterior surface of the density bottle was dried with a clean dry cloth and the weight of the density bottle + stopper +soil filled with water was determined and weight recorded as (M3) and thereafter the density bottle was emptied, cleaned and rinsed with distilled water. The density bottle was filled with distilled water up to the same mark after which the exterior surface was dried. The weight of the density bottle filled with distilled water + stopper was determined and the weight recorded as (M4). The procedure was repeated for two more trials after which the average specific gravity was determined.

3.2.2 Atterberg Limit Test

Atterberg Limits testing is widely used in the design stage of construction to ensure that the soils being used exhibit the proper consistency to support structures even as their moisture levels change. Soils for engineering use are often classified based on properties relative to foundation support or how they might perform under pavements and in earthworks. In the early 1900s, the Swedish chemist Albert Atterberg developed a classification system and method with which these states of consistency could be determined. His methods were later defined by Arthur Casagrande. The method is based on the determination of the water content at distinct transitions between different states of soil consistency. These transitions are defined as shrinkage limit, plastic limit, and liquid limit, and collectively are referred to as Atterberg limits. The liquid limit and plastic limit tests are the most commonly used of the Atterberg limits tests. The values for these limits are dependent on various soil parameters (e.g., particle size, specific surface area of the particles that are able to attract water molecules). Geotechnical classification systems are designed to make it easy to equate field observations to estimates of engineering properties. As moisture content of a fine-grained, clay-like soil increases, it goes through four distinct states of consistency: solid, semi-solid, plastic, and liquid. Each stage is defined by significant changes in strength, consistency and behavior. As a hard, rigid solid in the dry state, fine grained soil becomes a crumbly (friable) semisolid when certain moisture content, or shrinkage limit, is reached. This soil will also begin to swell as shrinkage limit is exceeded. Increasing the water content beyond the soil's plastic limit will transform it into a plastic mass, which causes additional swelling. The soil will remain in this plastic state until its liquid limit is exceeded, which causes it to transform into a viscous liquid that f lows when jarred. These limits can be determined with the three tests that make up the Atterberg limits tests. They are:

(i) Liquid Limit (LL)

(ii) Plastic Limit (PL)

(iii) Shrinkage Limit (SL)

I. Liquid Limit Test

It is the water content at which the soil has a small shear strength that it flows to close a groove of standard width when jarred in a specified manner. It is the minimum water content at which the soil tends to flow like a liquid. When a soil is mixed with an excessive amount of water, it will be in a liquid state and flow like a viscous liquid. When the viscous liquid dries gradually due to loss of moisture it will pass into a plastic state. With further loss of moisture, the soil will pass into a semi-solid state. With even further reduction of moisture, the soil will pass into a solid state. The moisture content (%) at which a cohesive soil will pass from liquid state to plastic state is referred to as the liquid limit of the soil.

In order to study the liquid limit of the soil Casagrande test was conducted. Liquid limit is generally determined by the mechanical method using Casagrande apparatus or the standard liquid limit test apparatus. With respect to this method, the liquid limit is defined as the moisture content at which 25 blows or drop in standard liquid limit apparatus will just close a groove of standardized dimension cut into sample by a grooving tool at a specified amount (Aroja, et al 2017).

The apparatus used for liquid limit determination is outlined below:

(i) Liquid limit device (Casagrande's type)

(ii) Grooving tool

(iii) Moisture content tins

(iv) Porcelain evaporating dish

(v) Spatula or pellet knife

(vi) Thermostatically controlled oven

(vii) Weighing balance sensitive to 0.01g

(viii) Plastic wash bottle containing distilled water

(ix) Paper towels

(x) 425µm Sieve

(xi) Airtight container

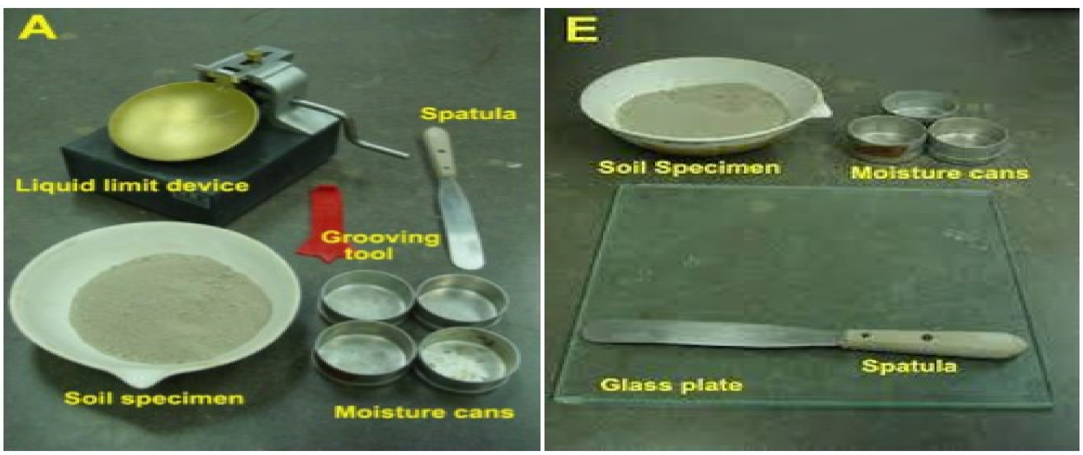


Plate 3.5: Apparatus for Atterberg Limit Test.

The Procedure employed for the Computation of the Result obtained is as Follows:

Moisture content = $\frac{Weight\ of\ water}{weight\ of\ dry\ soil} \times 100 = \frac{W2 - W3}{W3 - W1} \times 100$

Where W1 = Weight of empty tin.

W2 = Weight of tin + wet soil.

W3 = Weight of tin + oven-dried soil.

Test procedure

150g of soil sample passing through 425micometer sieve was weighed out and was placed on a glass plate, thereafter, distilled water were added and pallet knife was used to mix it thoroughly to form a thick homogenous paste after which the sample was place in an airtight container and was left to mature for 24ours. Five moisture content tins were weighed and the mass recorded as (M1). The matured sample was placed on an evaporating dish and little water were added using plastic squeeze bottle after which the soil was properly mixed to ensure uniform distribution of moisture. A portion of the paste was placed on the liquid limit device and it was levelled to obtain a maximum depth of 1cm. The grooving tool was used to cut a groove along the symmetrical axis of the cup. The handle of the liquid limit device was rotated at the rate of 2 revolution per second and the number of blows that closed the groove at a distance of 13cm were counted. About 10g of the sample was taken out from the closed groove for moisture content determination, weigh the sample + moisture content tin and record the mass as (M2). The rest of the soil in the liquid limit device cup were removed and the cup was cleaned properly using towel. The water content of the sample were altered and the process were repeated to get the number of blows for a specific range of blows. A graph of moisture content against the log of number of blows was plotted and the moisture content corresponding to 25 blows on the abscissa gives the value of the liquid limit.

II. Plastic Limit Test

The plastic limit of a soil is the moisture content expressed as a percentage of the weight of oven-dried soil at the boundary between the plastic and the semi-solid state of consistency. It is the moisture content at which a soil will just begin to crumble when rolled into a uniform 3mm diameter thread using a glass plate or other recommended surface for rolling. Soil used for Atterberg limit test can be classified based on the plasticity index of the soil. The plasticity index is the amount of water required to change a soil from its plastic limit to liquid limit, in other word it is the numerical difference between the liquid limit and the plastic limit of soil. Table 3.2 is used to classify soil based on the ranges of it plasticity index.

Table: 3.2 Plasticity Ratings for Fine grained Soil

PLASTICITY INDEX	PLASTICITY

0	Non-Plasticity
<7	Low Plasticity
7-17	Medium Plasticity
17-35	High Plasticity
>35	Very High Plasticity

Source: (Braja 2002).

The apparatus used for this experiment includes:

i. A smooth glass plate about 300mm square and 10mm thick.
ii. A palette knife or spatula
iii. A short length of 3mm metal rod
iv. Moisture content tins
v. Plastic squeeze bottle
vi. Weighing balance with 0.01g sensitivity
vii. Veneer caliper

The Computation for Plastic Limit is as follows:

$$\text{Plastic limit} = \frac{Weight\ of\ water}{Weight\ of\ oven-dried\ soil} \times 100 = \frac{W2-W3}{W3-W1} \times 100 \quad (3.2)$$

Where W1 = Weight of empty tins.

W2 = Weight of tin plus wet soil

W3 = Weight of tin plus oven-dried soil

Test procedure

The sample was prepared by the method described in the liquid limit. The moisture content tin to be used was weighed and the mass recorded as (M1). About 20g of prepared soil paste on a porcelain dish was taken out, thereafter, it was mixed thoroughly with water until the paste became plastic enough to be rolled into ball. A portion of the ball was rolled on a glass plate with the palm of the hand into a thread of uniform diameter throughout its length by rolling forward and backward. The rolling and remolding were continued until the thread just start to crack at a distance of 3mm. the crumbled pieces were collected and placed in a moister content tin and weighed and the mass was recorded as (M2). The moisture tin was placed in the oven at a constant temperature of 80-110˚C for a period of 16-24hours, after 24hours the tin was removed from the oven and the mass of the dry soil plus the tin was determined and the mass recorded as (M3). The test were repeated for two trials and the average plastic limit value was determined.

3.2.3 Compaction Test

Compaction is the process of increasing the bulk density of the soil by driving out air. It involves the densification of soils by mechanical means thereby increasing the DRY UNIT WEIGHT of the soil. According to (Shruthi, 2017) Compaction of soil is the process by which the soil solid are packed more closely together by mechanical means, thus increasing its DRY UNIT WEIGHT. It could also be stated as the process of packing the soil particle more closely together usually by tamping, rolling or other mechanical means, thus increasing the DRY UNIT WEIGHT of the soil. It is achieved through the reduction of the volume of air void in the soil with little or no reduction in water content. The process must not be confused with consolidation in which water is squeezed out under the action of steady static load. Consolidation is a natural process and result in dense packing of the soil.

In civil engineering practice soil compaction is essential for the following reasons:

a) Increasing the bearing strength of foundation
b) Provide stability to slope and foundation.
c) Prevention of undesirable settlement of structures
d) Reduction of water seepage from structure

The compaction methods to be adopted for this research are:

1. British Standard Light (BSL) for the natural samples of laterite collected from borrow pit at Ugwu-oba.

Details of British Standard Compaction Process

Table 3.3 Details of Compaction Mold.

TYPE	DIAMETER (mm)	HEIGHT (mm)	VOLUME (cm^3)
BRITISH STANDARD	105	115.5	1000

Table 3.4 Details of Compaction Procedure.

TYPE OF TEST	MOULD (cm^3)	RAMMER (kg)	DROP (mm)	NO OF LAYERS	BLOW PER LAYER
BS Light	1000	2.5	300	3	25

The apparatus used for the test are as follows:

i. Compaction mold with a detachable base plate and removable extension collar.
ii. Metal rammer (either 2.5kg or 4.5kg)
iii. Measuring Cylinder 200ml or 500ml
iv. Large Metal tray (600mm×600mm ×600mm)
v. Balance up to 10kg readable to 1g
vi. Small tools such as palette knife, steel straight edge about 300mm long.
vii. Drying oven temperature of 105-110OC
viii. Apparatus for moisture content determination

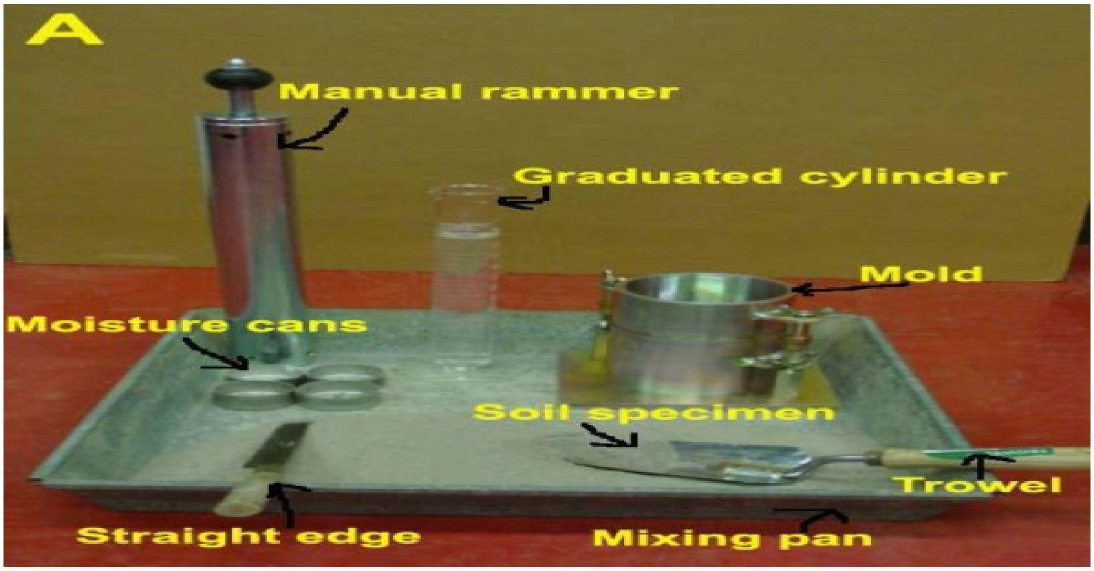


Plate 3.5: Apparatus employed for Compaction Test. Source: (Braja, M.Das, 2002).

The Computation of the result obtained is as follows:

Determination of DRY UNIT WEIGHT (P_d).

Wt of mold (kg) = W1

Wt of mold + wet soil (kg) = W2

Wt of wet soil (kg) = W2-W1

Volume of mold (M3) = W4

Bulk Density (kg/m3) = $\frac{Wt\,of\,wet\,soil\,(kg)}{Vol\,of\,mould\,(m3)} = \frac{W2 - W1}{W4}$

Moisture Content (%) = $\frac{mosture\,content\,(top) + moisture\,content\,(bottom)}{2}$

DRY UNIT WEIGHT (kN/m3) = $\frac{Bulk\,density}{1 + moisture\,content\,(\%)} = \frac{Pb}{1 + w/100}$

Determination of Moisture Content (w) for top and bottom respectively.

Wt of tin (kg) = W1

Wt of tin + wet soil = W2

Wt of wet soil (kg) = W3 = W2-W1

Wt of tin + dry soil (kg) = W4

Wt of dry soil (kg) = W5= W4-W1

Wt of water (kg) =W6 = W3-W5

Moisture Content (%) = $\frac{wt\ of\ water}{wt\ of\ dry\ soil} X\ 100 = \frac{w6}{w5} X\ 100$

Test procedure.

Firstly, the mold, extension collar and base plate were cleaned and greased, the mold was weighed and the mass was recorded as (M1), the rammer was checked for free fall. The extension collar was attached to the mold. 3kg of soil sample was weighed out using a weighing balance, after which the weighed sample was placed in a mixing tray. 4% of water was added, thereafter, it was mixed thoroughly before been shared into three layers for British Standard Light (BSL) compaction. The first layer was poured inside the mold and was compacted by applying 25 blows using 2.5kg rammer falling freely from a height of 300mm. the blows were distributed uniformly over the surface of the mold. After completion of the compaction, the extension collar was removed and the mold was carefully levelled off by means of straight edge. The mold plus the compacted soil was weighed and the mass recorded as (M2). Small portions of the sample were collected from the top and bottom of the mold for moisture content determination. The procedures were repeated for 8%, 12%, 16% and 20% of water and the values obtained were recorded. A graph of dry unit weight (DUM) against moisture content were plotted and optimum moisture content (OMC) corresponding to maximum dry unit weight (MDUW) was determined.

CHAPTER FOUR

RESULT AND DISCUSSION

The results of the different laboratory test are presented and discussed in the following sections:

4.1 Classification of Clay Sample, Lime and ESP Mix.

The following table can be used to discuss extensively on the index properties of the soil sample and the additive (lime and egg shell powder (ESP)) and this are enumerated as follows:

Table 4.1 Index properties of the soil

PROPERTIES	CLAY SOIL
Specific Gravity (Gs)	2.48
Liquid Limit (%)	32.45
Plastic Limit (%).	14.01
Plasticity Index (%).	18.44
Plasticity	HIGH
Maximum DRY UNIT WEIGHT (kN/m^3)BSL	21.8
Optimum Moisture Content (%).BSL	13.6

Table 4.2 Index properties used for the research

PROPERTIES	CLAY SOIL (NO ADDITIVE)	10% LIME +0% ESP	8.5% LIME + 1.5% ESP	7% LIME +3% ESP	5% LIME +5% ESP
SPECIFIC GRAVITY (Gs)	2.48	-	-	-	-
LIQID LIMIT (%)	32.45	25.53	26.11	26.70	28.85
PLASTIC LIMIT (%)	14.01	15.32	15.23	14.91	14.41
PLASTICITY INDEX (%)	18.44	10.21	10.88	11.79	14.44
PLASTICITY	HIGH	MEDIUM	MEDIUM	MEDIUM	MEDIUM

4.1.2 Specific Gravity

The specific gravity of the soil is defined as the ratio of weight of the soil to the weight of equal volume of water. It is used to obtain the unit weight of the soil in the presence of water. For the soil samples designated, the average specific gravity computed is 2.48 as shown in table 4.0. The values obtained, fall within the range of specific gravity.

4.1.3 Atterberg Limit

From table 4.1 shown above, the atterberg limit test concluded indicates that the clay soil sample has an average plasticity index of 18.44 and liquid limit of 32.45. Therefore the clay sample can conveniently be classified as a clay soil with high plasticity. From table 4.2, the liquid limit value obtained for LIME-ESP mix indicates that the liquid limit of 10%LIME-0% ESP has the lowest

Liquid limit of 25.53 the results started increasing when percentages of lime was removed and egg shells powder added. This observation indicates that the reduction of percentages of lime will lead to an increase in the liquid limit. The plastic limit on the other hand decreases as the percentage of lime is reduced. The reduced plastic limit generally results in an increase of the plastic properties of the clay sample. The graph depicting the various liquid values obtained from the test is shown below:

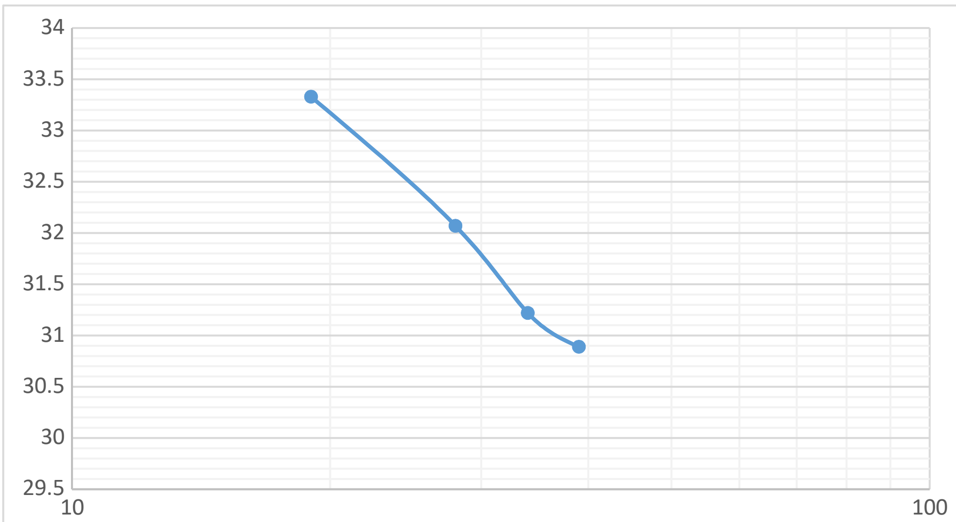


Figure 4.1: Moisture Content (%) against No. of blows (log scale)

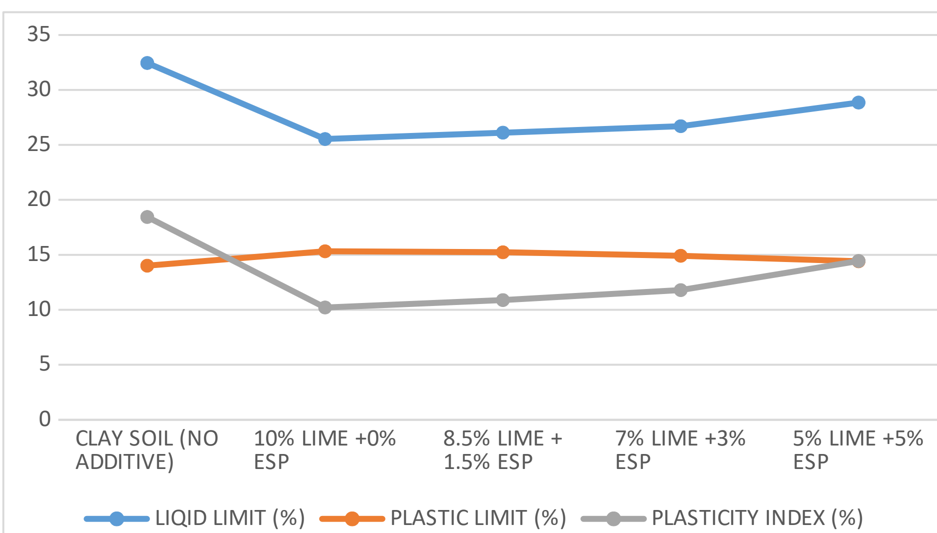


Figure 4.2: The effect of adding different percentages of lime-ESP on the liquid limit, plastic limit and plasticity index properties of the soil.

4.1.3 Compaction Test Result

Table 4.3 Moisture content and MDUW of soil at different percentage of lime and ESP

	CLAY SOIL	10% LIME + 0% ESP	8.5% LIME + 1.5% ESP	7% LIME + 3% ESP	5% LIME + 5% ESP
OMC (%)	13.6	10.3	12.65	12.7	12.35
MDUW (kN/m^3)	21.8	22.59	21.57	21.56	21.64

The Maximum dry unit weight (MDUW) of the soil sample and the specimen (clay soil mixed with lime and egg shell powder) was determined using British Standard Light (BSL). Table 4.3 shows the OMC and MDUW of the soil with all percentages of mixtures (Lime and ESP), the highest MDUW occurred at 10% Lime and 0% ESP with the value of 22.59kN/m^3 and the reading drops and varies at different percentages.

The OMC on the other hand decreases from the clay sample of 13.6% to 10.3% for clay soil mixed with 10%lime and 0% ESP and started increasing again after percentages of Lime were removed and replaced with ESP.

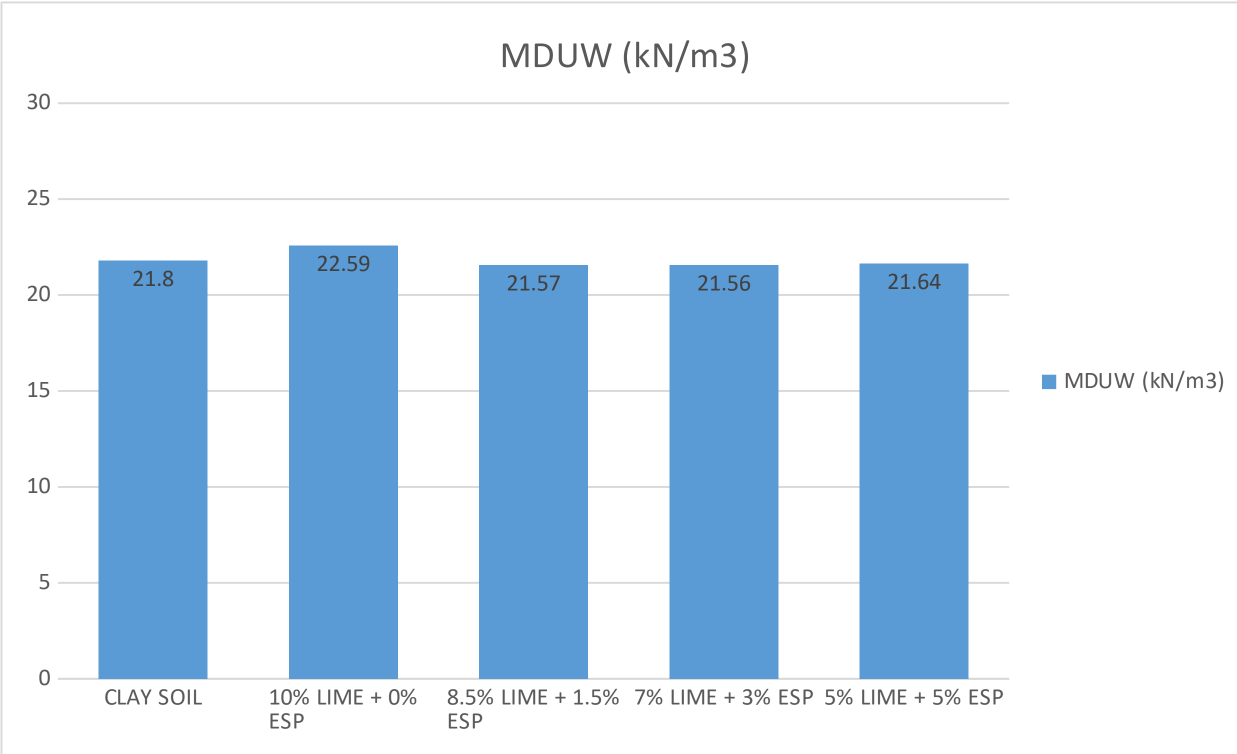


Fig 4.2 Maximum dry unit weight at different percentages of lime-ESP mix

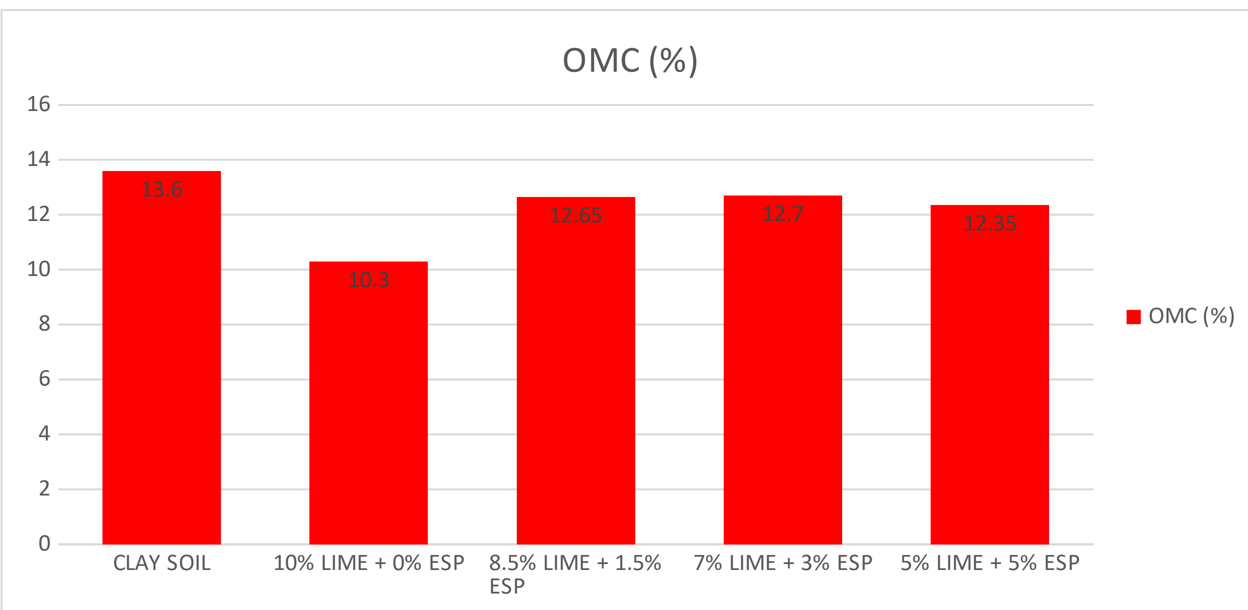


Fig 4.3 Optimum moisture content at different percentages of lime-ESP mix

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

This study intended to find out the effect of Egg shells and lime stabilizer on the engineering properties of clayey soil. The data presented in this study shows that egg shells might not have much effect on the engineering properties of clayey soil, only when there is a higher percentage of lime in the mix ratio can there be a possibility of stabilization on the engineering properties of clayey soil.

5.2 Recommendation

- If this method should be used a higher percentage of lime must be used or only lime powder should be used as an additive in the study.
- Egg shell powder cannot be used as a substitute for lime in an expansive soil e.g. clay soil.
- Further research should still be carried out to determine any further effect.

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APPENDIX 1

LABORATORY TEST RESULTS OF SPECIFIC GRAVITY OF CLAY SOIL

SPECIFIC GRAVITY OF CLAY SOIL

	EMPTY BOTTLE (W1)	BOTTLE + DRY SAMPLE (W2)	BOTTLE + SAT. SOIL (W3)	BOTTLE + WATER (W4)	SPECIFIC GRAVITY
S1	24.65	36.41	86.11	79.04	2.51
S2	25.03	37.43	86.25	78.85	2.48
S3	24.53	36.81	87.01	79.73	2.46
AVERAGE SP. GRAVITY					2.48

APPENDIX 2

LABORATORY TEST RESULTS OF ATTERBERG LIMIT OF CLAY-LIME-ESP MIXTURE

ATTERBERG LIMIT TEST (CLAY)

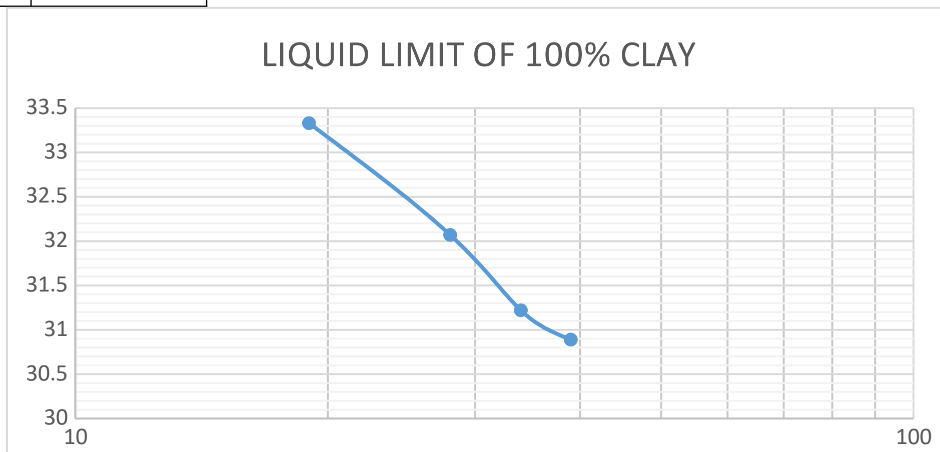
LIQUID LIMIT

NUMBER OF BLOWS	WEIGH T OF CAN (g)	CAN + WET SAMPLE (g)	WET SAMPLE (g)	CAN + DRY SAMPLE (g)	DRY SAMPLE (g)	WEIGH T OF WATER	MOISTURE CONTENT (%)
19	14.89	21.53	6.64	19.87	4.98	1.66	33.33
28	15.01	23.37	8.36	21.34	6.33	2.03	32.07
34	15.74	23.04	7.3	21.44	5.70	1.78	31.22
39	14.71	21.66	6.95	20.02	5.31	1.64	30.89

PLASTIC LIMIT

WEIGHT OF CAN (g)	CAN + WET SAMPLE (g)	WET SAMPLE (g)	CAN + DRY SAMPLE (g)	DRY SAMPLE (g)	WEIGHT OF WATER	MOISTURE CONTENT (%)

13.78	15.00	1.22	14.85	1.07	0.15	14.01
LL	32.45					
PL	14.01					
PI	18.44					



ATTERBERG LIMIT TEST (CLAY+10% LIME+0%ESP)

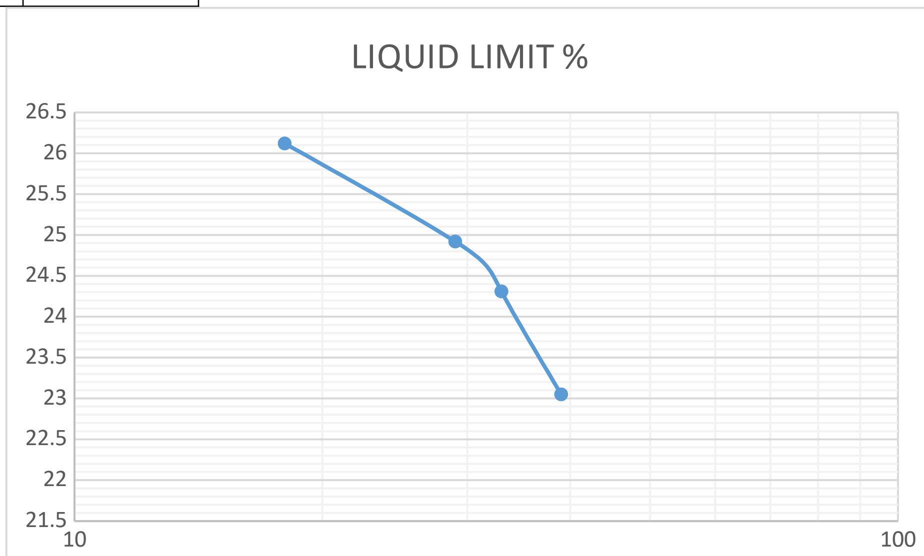
LIQUID LIMIT

NUMBER OF BLOWS	WEIGHT OF CAN (g)	CAN + WET SAMPLE (g)	WET SAMPLE (g)	CAN + DRY SAMPLE (g)	DRY SAMPLE (g)	WEIGHT OF WATER	MOISTURE CONTENT (%)
18	14.89	20.78	5.89	19.56	4.67	1.22	26.12
29	15.01	22.43	7.42	20.95	5.94	1.48	24.92
33	15.74	22.9	7.16	21.5	5.76	1.4	24.31
39	14.71	21.65	6.94	20.35	5.64	1.3	23.05

PLASTIC LIMIT

WEIGHT OF CAN (g)	CAN + WET SAMPLE (g)	WET SAMPLE (g)	CAN + DRY SAMPLE (g)	DRY SAMPLE (g)	WEIGHT OF WATER	MOISTURE CONTENT (%)

13.78	15.06	1.28	14.89	1.11	0.17	15.32
LL	25.53					
PL	15.32					
PI	10.21					



ATTERBERG TEST (CLAY+8.5% LIME+1.5%ESP)

LIQUID LIMIT

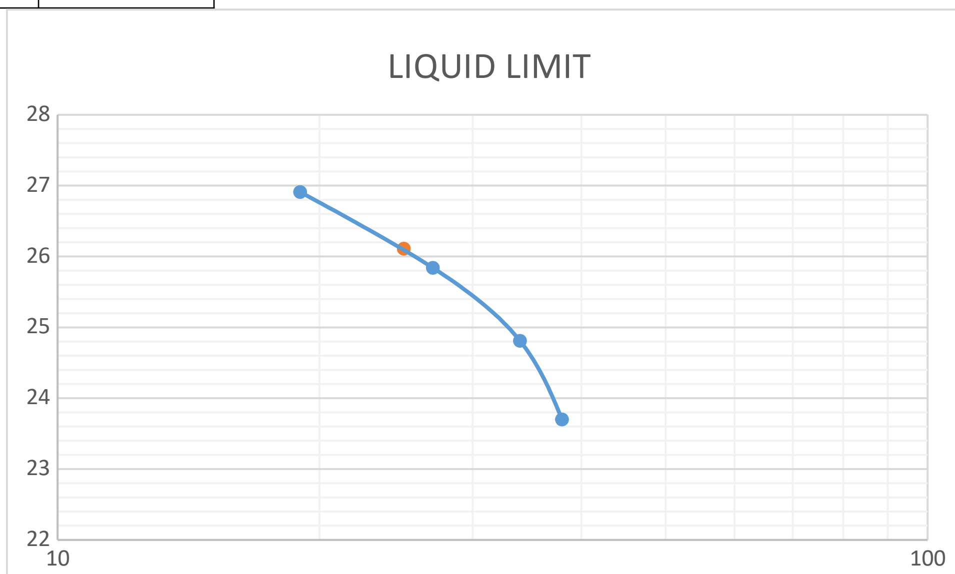
NUMBER OF BLOWS	WEIGHT OF CAN (g)	CAN + WET SAMPLE (g)	WET SAMPLE (g)	CAN + DRY SAMPLE (g)	DRY SAMPLE (g)	WEIGHT OF WATER	MOISTURE CONTENT (%)
19	14.89	21.21	6.32	19.87	4.98	1.34	26.91
27	15.01	21.78	6.77	20.39	5.38	1.39	25.84
34	15.74	22.43	6.69	21.1	5.36	1.33	24.81
38	14.71	21.13	6.42	19.9	5.19	1.23	23.7

PLASTIC LIMIT

WEIGHT OF CAN	CAN + WET	WET SAMPLE	CAN + DRY	DRY SAMPLE	WEIGHT OF	MOISTURE CONTENT

(g)	SAMPLE (g)	(g)	SAMPLE (g)	(g)	WATER	(%)
13.78	15.52	1.74	15.29	1.51	0.23	15.23

LL	26.11
PL	15.23
PI	10.88



ATTERBERG TEST (CLAY+7% LIME+3%ESP)

LIQUID LIMIT

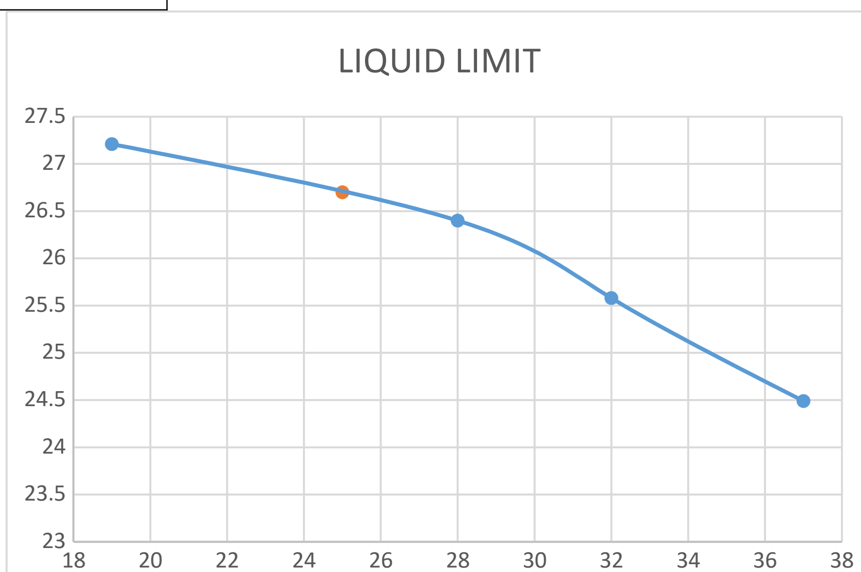
NUMBER OF BLOWS	WEIGHT OF CAN (g)	CAN + WET SAMPLE (g)	WET SAMPLE (g)	CAN + DRY SAMPLE (g)	DRY SAMPLE (g)	WEIGHT OF WATER	MOISTURE CONTENT (%)
19	14.89	20.78	5.89	19.52	4.63	1.26	27.21
28	15.01	20.9	5.89	19.67	4.66	1.23	26.4

32	15.74	21.14	5.4	20.04	4.3	1.1	25.58
37	14.71	20.86	6.15	19.65	4.94	1.21	24.49

PLASTIC LIMIT

WEIGHT OF CAN (g)	CAN + WET SAMPLE (g)	WET SAMPLE (g)	CAN + DRY SAMPLE (g)	DRY SAMPLE (g)	WEIGHT OF WATER	MOISTURE CONTENT (%)
13.78	15.09	1.31	14.92	1.14	0.17	14.91

LL	26.7
PL	14.91
PI	11.79



ATTERBERG TEST (CLAY+5% LIME+5%ESP)

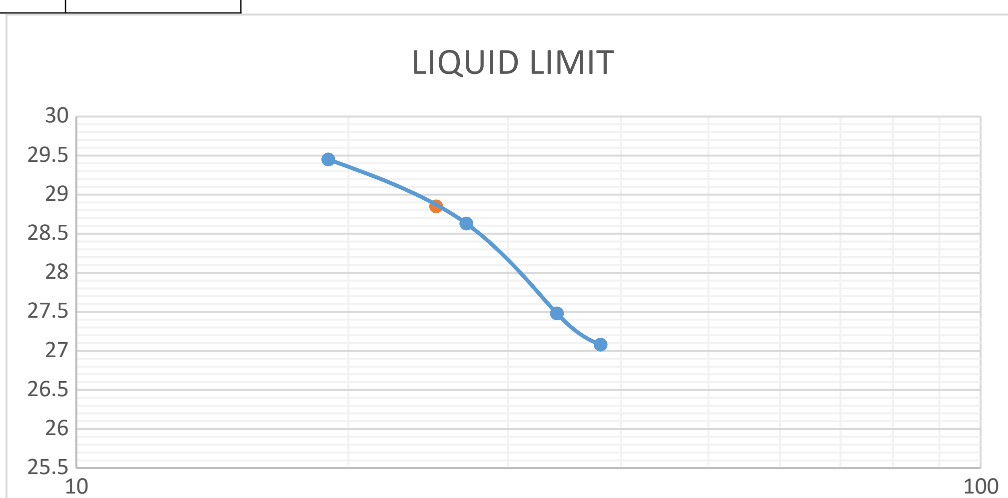
LIQUID LIMIT

NUMBER OF BLOWS	WEIGHT OF CAN (g)	CAN + WET SAMPLE	WET SAMPLE	CAN + DRY SAMPLE	DRY SAMPLE	WEIGHT OF	MOISTURE CONTENT

		(g)	(g)	(g)	(g)	WATER	(%)
19	14.89	21.22	6.33	19.78	4.89	1.44	29.45
27	15.01	21.03	6.02	19.69	4.68	1.34	28.63
34	15.74	20.89	5.15	19.78	4.04	1.11	27.48
38	14.71	21.14	6.43	19.77	5.06	1.37	27.08

PLASTIC LIMIT

WEIGHT OF CAN (g)	CAN + WET SAMPLE (g)	WET SAMPLE (g)	CAN + DRY SAMPLE (g)	DRY SAMPLE (g)	WEIGHT OF WATER	MOISTURE CONTENT (%)
13.78	15.05	1.27	14.89	1.11	0.16	14.41
LL	28.85					
PL	14.41					
PI	14.44					

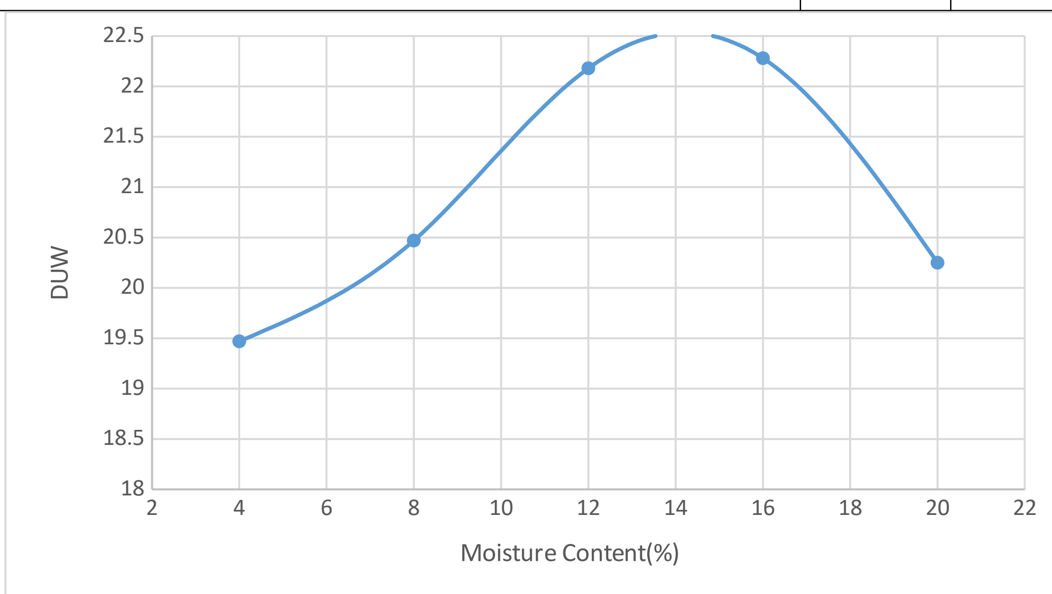


APPENDIX III

COMPACTION RESULT

COMPACTION 1 BS LIGHT (CLAY)

% OF WATER	VOL OF MOLD (cm3)	WT OF MOLD (g)	MOLD + WET SOIL(g)	WT OF WET SOIL(g)	BULK DENSITY (g/cm3)	MOISTURE CONTENT (%)	DRY DENSITY (kN/m3)	DUW (kN/m3)
4	826.9	4200	5900	1700	2.0559	3.58	20.17	19.47
8	826.9	4200	6050	1850	2.2373	7.2	21.95	20.47
12	826.9	4200	6250	2050	2.4791	9.63	24.32	22.18
16	826.9	4200	6300	2100	2.5396	11.8	24.91	22.28
20	826.9	4200	6150	1950	2.3582	14.2	23.13	20.25
							MDUW (Kn/m3)	22.5
							OMC %	14.01



COMPACTION WITH ADDITIVE (LIME AND ESP)

OPTIMUM MOISTURE CONTENT = 11.1% (will be used as the % of water for all compactions with different additives)

% OF WATER	VOL OF MOLD (cm3)	WT OF MOLD (g)	MOLD + WET SOIL(g)	WT OF WET SOIL(g)	BULK DENSITY (g/cm3)	MOISTURE CONTENT (%)	DRY DENSITY (kN/m3)	DUW (kN/m3)
COMPACTION FOR 10% LIME + 0% ESP								
13.6	826.9	4200	6300	2100	2.540	10.3	24.92	22.59
COMPACTION FOR 8.5% LIME + 1.5% ESP								
13.6	826.9	4200	6250	2050	2.479	12.67	24.31	21.57
COMPACTION FOR 7% LIME + 3% ESP								
13.6	826.9	4200	6250	2050	2.479	12.715	24.31	21.56
COMPACTION FOR 5% LIME + 5% ESP								
13.6	826.9	4200	6250	2050	2.479	12.35	24.31	21.64

MOISTURE CONTENT OF ALL ADDITIVE COMPACTION

	WT OF CAN (g)	WT OF CAN + SOIL (g)	WT OF WET SOIL (g)	WT OF CAN + DRY SOIL (g)	WT OF DRY SOIL (g)	WT OF WATER (g)	MOISTURE CONTENT
MOISTURE CONTENT FOR ADDITIVE (10% LIME + 0% ESP)							
TOP	15.04	50.68	35.64	47.44	32.4	3.24	10
BOTTOM	16.84	55.12	38.28	51.45	34.61	3.67	10.60
MOISTURE CONTENT FOR ADDITIVE (8.5% LIME + 1.5% ESP)							
TOP	16.17	51.29	35.12	47.43	31.26	3.86	12.35
BOTTOM	18.28	65.76	47.48	60.3	42.02	5.46	12.99

MOISTURE CONTENT FOR ADDITIVE (7% LIME + 3% ESP)						
TOP	18.69	58.6	39.91	54.13	35.44	4.47	12.61
BOTTOM	16.96	55.95	38.99	51.52	34.56	4.43	12.82
MOISTURE CONTENT FOR ADDITIVE (5% LIME + 5% ESP)							
TOP	15.01	48.24	33.23	44.58	29.57	3.66	12.38
BOTTOM	13.77	50.51	36.74	46.48	32.71	4.03	12.32

	10% LIME + 0% ESP	8.5% LIME + 1.5% ESP	7% LIME + 3% ESP	5% LIME + 5% ESP
Average moisture content	10.3	12.67	12.715	12.35