

**EFFECT OF LIME-STABILIZATION ON STRENGTH CHARACTERISTICS OF
BRICKS MADE WITH SOKOTO RED CLAY SOIL**

By

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DECLARATION

I declare that the work in this thesis entitled “Effect of Lime-Stabilization on Strength Characteristics of Bricks made with Sokoto Red Clay Soil” has been carried out by me in the Department of Building. The information derived from the literature has been duly acknowledged in the text and a list of references is provided. No part of this thesis was previously presented for another degree or diploma at this or any other Institution.

Name of student

Signature

Date

CERTIFICATION

This thesis entitled “EFFECT OF LIME-STABILIZATION ON STRENGTH CHARACTERISTICS OF BRICKS MADE WITH SOKOTO RED CLAY SOIL” by Billihaminu Ayodeji OLAWUMI meets the regulations governing the award of the degree of Master of Science (Construction Technology) of Ahmadu Bello University, Zaria and is approved for its contribution to knowledge and literary presentation.

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DEDICATION

This thesis is dedicated to my late parents. May their gentle souls continue to rest in perfect peace. Amin.

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ABSTRACT

Clay soils have been used locally in Sokoto to make bricks without stabilizing the soil. The bricks usually experience structural failure such as cracking and seasonal swelling when used for the construction of wall. Collapsing of wall also occurs in the area where non-stabilized bricks are used for wall construction. This research aims to assess the effect of lime-stabilization on strength of bricks made with Sokoto Red Clay Soils with a view to determining the physical properties of the soil and the optimum level of lime-stabilization required in the case of Sokoto Red Clay Soil (RCS). RCS used in this research were stabilized by lime and the method adopted for the stabilization is additive method. The method involves addition of certain percentages of stabilizer(s) to soil to improve the strength properties and durability of the soil. This study carries out laboratories work on the physical tests of the soil sample and the compressive strength test on the bricks produced. The Optimum Moisture Content of the soil and stabilized soil were determined to be 14.6% and 16.8% respectively. Percentages of lime-stabilization used in this research are 0%, 3%, 6%, 9%, 12%, 15%, 18% and 21%. A total of 120 bricks were produced and tested for compressive strength after stipulated curing period. The curing method adopted is moist curing method for 3, 7, 14, 21 and 28 days. The results of compressive strength at 0%, 3%, 6%, 9%, 12%, 15%, 18% and 21% in 28 days are; 0.5N/mm², 1.35N/mm², 1.24N/mm², 0.85N/mm², 0.82N/mm², 0.63N/mm², 0.47N/mm² and 0.37N/mm². A continuous increase in compressive strength of the bricks from 3 days to 28 days in both 3% and 6% stabilizations were further observed. At 3% and 6% stabilization, the highest average compressive strengths recorded in 28 days are 1.35N/mm² and 1.24N/mm² respectively.. However,

the highest value is not up to 2.8N/mm^2 as stipulated in BS 5628 part 1 (1978). This might be as a result of the low value of plasticity index (10.4) and the stabilizer used. It was also found that the highest compressive strength of the bricks at 0% stabilization was 1.11N/mm^2 in 21days and the compressive strength reduced to 0.50N/mm^2 at 28days. The reduction in compressive strength derived at 0% stabilization in 28days might be as a result of cracks showed on the surface of the bricks which may be attributed to lack of stabilization. Compared to 3% stabilization, compressive strength of 1.11N/mm^2 was recorded in 3days due to the effect of lime on the bricks. The highest value recorded at 3% stabilization indicates that lime alone cannot be used to stabilize red clay soil for brick production. This shows that lime should be used together with one or two stabilizers to stabilize red clay soil so as to improve the strength properties of the bricks. It is also recommended that red clay soil with higher plasticity index should be adopted for lime-stabilization.

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ABBREVIATIONS AND SYMBOLS

AASHTO	America Association of State Highway and Transport Officials.
ASTM	American Society for Testing and Material
BS	British Standard
BSCS	British Soil Classification System
BS EN	European Standard with British standard status
CCNN	Cement Company of Northern Nigeria
LL	Liquid Limit
NLA	National Lime Association
OMC	Optimum Moisture Content
OWC	Optimum Water Content
PL	Plastic Limit
PI	Plasticity Index
RCS	Red Clay Soil
IS	Indian Standard
USCS	United Soil Classification System
WT	Weight

CHAPTER ONE

INTRODUCTION

1.1 Background of the Study

Different types of stabilizers have been used in different parts of the world to enhance the engineering properties of laterite (Moh, 1962). The most common ones are cement, lime and bitumen and the choice of a particular stabilizer depends on the soil type and degree of effectiveness of the stabilizers (Rahaman, 1968). Mustapha (2006) defined soil stabilization as the treatment of soils to enable their strength and durability to be improved such that they become totally suitable for construction beyond their classification if left untreated. Stabilization as viewed by Afjman (1994) is the process of blending and mixing materials with a soil to improve certain properties of the soil. Afjman (1994) further explained that stabilization process may include the blending of soils to achieve a desired gradation or the mixing of commercially available additives that may alter the gradation, texture or plasticity or act as a binder for cementation of the soil. Stabilization can be used to function as mechanical stabilization, additive stabilization or modification.

Mechanical stabilization is accomplished by mixing or blending soils of two or more gradations to obtain a material meeting the required specification. The soil blending may take place at the construction site, a central plant or a borrow area. Additive stabilization is achieved by the addition of proper percentages of cement, lime, fly-ash, bitumen or combination of these materials to the soil. The selection of the type and determination of the percentage of additive to be used is dependent upon the soil classification and the degree of improvement in the soil quality desired. Modification stabilization refers to the

stabilization process that results in improvement in some properties of the soil, but does not by design result in a significant increase in soil strength and durability (Afjman,1994). Lime, basically is derived from the process of burning limestone at temperature above 900°C and the lime produced as a result of the above decomposition, is generally referred to as quicklime. The quicklime is subsequently slaked with water to produce hydrated lime (Mukerji and Strulz, 1988). Mukerji and Strulz (1988) also pointed out the fact that the production of lime kilns is a more than 2000 years old technology, believed to have been developed by the Romans around 300 B.C. Lime is also obtained as a by-product in the form of lime sludge (which contains calcium carbonate and various impurities) from sugar manufacture, and from acetylene and paper industries. The building lime is used for preparing building mortars, concrete, binders and for manufacturing artificial stones, slabs and miscellaneous building parts (Kumar, 1987). Lime plays an important role in soil stabilization, mortar for brick bonding, production of block, renders and in lime paints (Vazquez, Marquez and Macias, 2005). Lime is used as stabilizer in construction with clayey soils, because the lime reacts with clay to form a binder and when it is mixed with a pazzolana (rice husk ash, fly ash, blast furnace slag e.t.c.) to produce a hydraulic binder, which can partially or completely substitute cement, depending on the required performance (Mukerji and Strulz, 1988). The chemical process of lime burning shows that the main constituent in the raw material (lime stone) is necessarily calcium carbonate (CaCO_3) and lime stone can have CaCO_3 contents exceeding 98% (as in chalk and various types of shells and coral) or as low as 54% (in pure mineral dolomite) (Mukerji and Strulz, 1988).

It has been reported by Craig (2005) that to the Civil Engineer, soil is any un-cemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks, the void space between the particles containing water and/or air. Craig (2005) further explained that the weak cementation can be due to carbonates or oxides precipitated between the particles or due to organic matter. Soil is naturally occurring materials that are used for the construction of all except for the construction of surface layers of pavements (i.e. concrete and asphalt) and these are subject to classification tests (ASTM D 2487) to provide a general concept of their engineering characteristics (Afjman, 1994).

According to Williams (2006) clay is sticky and plastic-like to handle when wet, the individual particles are extremely small and can only be seen with the aid of an electron microscope. The three main soil texture classifications are: sandy, silt and clay. Sand soils are coarse-textured, clay soils are fine-textured and silt soils are intermediate in texture (Williams, 2006). Clay minerals all have a great affinity for water, some swell easily and many double in thickness when wet, while most have the ability to soak up ions (electrically charged atoms and molecules) from a solution and release the ions later when conditions change (Nora,1999). Compare to other soil types, clay soil particles are small in size and have very large surface area relative to their volume, this large surface is highly reactive and has the ability to attract and hold positively charged nutrients ions (Pidwirny, 2006). Before clay soil can be stabilized with lime the plasticity index (PI) must be measured, when the plasticity index is at least 10, according to the Atterberg limits (Consistency limits), the soil can be stabilized with lime (Vazquez, et al, 2005). Vazquez et al (2005) shows that the Atterberg limit expresses the difference between the liquid limit and plastic limit in soils and that if plasticity index is less than 12, means a low plastic soil

and if plasticity index is greater than 40 means a very high-plastic soil (almost liquid). This research work intends to proffer solution to problem associated with wall constructed with unstabilized red clay bricks.

1.1 Statement of the Research Problem

Weathering of rocks and soil is the primary way that clays and clay minerals form at the earth's surface today. The weathering process involves physical disaggregation and chemical decomposition that change original minerals to clay minerals; weathering is uneven, and many stages of breakdown may be found in the same clay sample (Nora, 1999). Clay soil can be easily sticky when mixed with water. The individual particles of clay are much smaller than those of sand. Locally, clay soils have been used in many ways in Sokoto state. For instance, in making bricks, used as mortar in block laying, plastering and rendering of wall surfaces. The clays in use as explained above had been applied directly after mixing with water only. In most villages in Sokoto State, the red clay bricks, used in construction of walls are not usually stabilized. Even those that were stabilized were not done according to the required standard. Cement is commonly used as stabilizer but is being applied in small quantities. Thus, due to the fact that the red clay soil is not usually stabilized or not adequately stabilized, most of the walls constructed with the red clay bricks experienced cracking, seasoning swelling and even collapse of the entire wall. This is because the bricks were not produced in accordance with the standard specification. And since the cost of purchasing lime is cheaper than cement, therefore, lime was adopted as stabilizer for this research work. If the strength of the clay is enhanced by stabilizing it with lime, these problems of cracking and collapsing of wall can be checked. Moreover,

this study established the effect of lime stabilization on brick made with “Sokoto red clay soil”

1.2 Justification of the Study

Clay soils are deposited in large quantities in almost all the local government areas in Sokoto State, Nigeria (Field survey, 2009). Sokoto state has very good quality limestone deposited in more than three local governments in the state. Presently, cement is being manufactured in the state since early 1980s at “*Kalambaina*” in Wamakko local government due to large quantities of limestone deposited in the area compare to other local governments (CCNN, 2010). However, it was observed that due to the quality of the limestone in Sokoto State, the red clay soil in the area perhaps possess some qualities which can make it suitable for building construction when well treated. Therefore, this brought about the reason for this research work.

Blocks and bricks are two major material used for construction of wall. Lot of researches have been carried out on the sandcrete block (either hollow or solid) such as partial replacement of cement to make block, compressive strength sandcrete block, an assessment of the quality of sandcrete hollow block production of different block industries in a state, preliminary study of properties of compressed building blocks stabilized with rice husk ash, soil blocks admixed with bitumen and ordinary Portland cement for building construction etc. Blocks are more common in used for wall construction than bricks. Probably, that is the reason why researchers focused more on it than brick. More research works have not been documented on how to improve strength characteristics of clay bricks. Also, in Sokoto state, blocks are common in use for construction of wall compare to brick. However, red clay soil is deposited in large

quantities in Sokoto State, may be that is the reason why most of the walls of buildings in the villages are constructed by red clay bricks. Hence, it is imperative to look inwards on how to improve the strength properties of the red clay bricks.

1.3 Aim and Objectives

1.4.1 Aim

The aim of this research is to determine the effect of lime stabilization on strength characteristic of bricks made with Sokoto Red Clay Soil with a view to enhancing the strength and durability of bricks produced with “Sokoto” Red Clay Soil.

1.4.2 Objectives

1. To determine the physical properties of Sokoto red clay soil.
2. To produce brick samples made with stabilized Sokoto red clay soil.
3. To test the brick samples for mechanical properties.
4. To determine the optimum level of lime stabilization required in the case of “Sokoto” red clay soil.

1.5 Scope and Limitation

1.5.1 Scope

This study was conducted to produce brick samples from stabilized sokoto red clay soil. Zero percent stabilized bricks were produced to serve as control samples. Physical properties of the soil were examined before used. After the bricks were produced and cured accordingly, the samples were tested for mechanical properties. This was done so as to confirm the effect of lime-stabilization on the compressive strength of the brick produced. All the tests on the soil and brick samples were carried out in accordance with the relevant standard code of practice.

1.5.2 Limitations

Red Clay Soils used for this research were sourced by method of disturbed sampling. And that the soil was removed at about 300mm depth from the top most part of the soil. The soil below 300mm depth perhaps possesses physical properties different to the surface part of the soil. Thus, this study only covered the physical properties of the top most part of the soil.

The brick samples were cured by spraying water on the samples every 24 hours before covering with sacks. There is possibility that some portion of the water might not get to the whole part of the samples compared to the specimens immersed in curing tank. Hence, the curing method adopted may have influence on the strength properties of the bricks.

CHAPTER TWO LITERATURE REVIEW

2.1 Introduction

This chapter consists of reviews of the literature on soil and its classification, soil stabilization and brick as a material used in the construction of wall of buildings. Emphasis will be on physical and chemical properties of clay and clay minerals. Since the research focused on stabilization of soil, different method of soil stabilization were also discussed.

2.2 Soil

The term “soil” according to Ranjan and Rao (2005) has different connotations for scientists belonging to different disciplines. The definition given to soil by an agriculturalists or geologist is different from the one used by civil engineer (Ranjan and Rao, 2005). To an agriculturalist, soil merely means the top layer of the earth which is responsible for supporting plant life and to geologist, soil is the thin outer layer of loose sediments within which plant roots are present (Ranjan and Rao, 2005). Ranjan and Rao, (2005) further argued that, a geologist refers to the rest of earth’s crust as rock irrespective of how strong or weak the bonding forces of the sediments are. To a civil engineer and builder, soils mean all natural occurring, relatively unconsolidated earth material-organic or inorganic in character that lies above the bedrock (Ranjan and Rao, 2005)

Smith and Smith (2003) explained soil as the actions of frost temperature, gravity, wind, rain and chemical weathering are continually forming rock particles that eventually become soils. According to Atkinson (2000), the term “soil” means different things to different people, to geologist it represents the products of past surface processes, to a pedologist it represents currently occurring physical and chemical processes and to a civil engineer and builder, it is a natural material that can be built on (e.g. foundation to

building, bridge) built in (like, tunnels, culverts, basements), built with (like roads, runways, embankments, dams), supported (like retaining walls, quays). As opined by Smith and Smith (2003) there are three types of soil when considering modes of formation; these are, transported soil, residual soil and organic soil.

2.2.1 Transported Soil.

Most soils have been transported by water as a stream or river loses its velocity, it tends to deposit some of the particles it is carrying, dropping the larger, heavier particles first. Hence on the higher reaches of a river, gravel and sand are found whilst on the lower or older part, silts and clays predominate, especially where the river enters the sea or lake and loses its velocity, also ice has been another important transportation agents, and large deposits of boulder clay and moraine are often encountered (Smith and Smith, 2003).

2.2.2 Residual soil.

These soils are formed in situ by chemical weathering and may be found on level rock surfaces where the action of the elements have produced a soil with little tendency to move. The residual also occur whenever that rate of breakup of the rock exceeds the rate of removal laterites are formed by chemical weathering under warm, humid tropical conditions when the rain water leaches out the soluble rock material leaving behind the insoluble hydroxides of iron and aluminum, giving them their characteristic red-brown colour (Smith and Smith, 2003).

2.2.3 Organic Soil

These soils contain large amounts of decomposed animal and vegetable matter that are usually dark in colour and give off a distinctive odour. Deposits of organic silts and clays have usually been created from river or lake sediments (Smith and Smith, 2003).

2.3

Soil Description and Classification

According to Craig (2005), it is essential that a standard language should exist for the description of soil and a comprehensive description should include the characteristics of both the soil material and the in-situ soil mass. He further discussed that material characteristics can be determined from disturbed samples of the soil, i.e. samples having the same particle size distribution as the in-situ soil but in which the in-situ structure has not been preserved. Smith and Smith (2003) posited that soil classification system have been in use for a very long time the first being created some 4000 years ago by Chinese engineer 1896, a soil classification system was proposed by the Bureau of soils, United States Department Agriculture in which the various soil types were classified purely on particle size and it is interesting to note that, the limiting sizes used are more or less the same as those in use today. Further improved systems allowed for the plasticity characteristics of soil and a modified form of the system proposed by casagrande in 1974 is the basis of the soil classification system used in Britain (Smith and Smith, 2003).

Atkinson (2000) argued that it is necessary to adopt a formal system of soil description and classification in order to describe the various materials found in ground investigation such a system must be comprehensive (covering all but rarest of deposit), meaningful in engineering context (so that civil engineers and builders will be able to understand and interpret) and yet relatively concise. He further explained that it is important to distinguish between soil descriptions and soil classification. Craig (2005) shares the same view. Description of soil is a statement describing the physical and nature and the state of the soil. It can be a description of a sample, or a soil in situ. It is arrived at using visual examination, simple tests, observations of site conditions, geological history etc.

(Atkinson, 2000). Soil classification on the other hand is the separation of soil into classes or groups each having similar behavior. A classification for engineering purposes should be based mainly on mechanical properties such as permeability, stiffness, strength. The class to which a soil belongs be used in its description (Atkinson, 2000).

Craig (2005) pointed out the fact that the principal material characteristics are particle size distribution (or grading) and plasticity, from which the soil name can be deduced. He claims that particle size distribution and plasticity properties can be determined either by standard laboratory tests or by simple visual and manual procedures, the secondary material characteristics are the colour of the soil and the shape, texture and composition of the particles. Mass characteristics should ideally be determined in the field but in many cases they can be detected in undisturbed samples such as samples in which the in-situ soil structure has been essentially preserved. A description of mass characteristics should include an assessment of in-situ compactive state (coarse soils) or stiffness (fine soils) and details of any bedding, discontinuities and weathering (Craig, 2005). Soil description includes details of both material and characteristics, and therefore it is unlikely that any two soils will have identical descriptions, while in soil classification, a soil is allocated to one of a limited number of groups on the basis of material characteristics only and this independent of the in-situ condition of the soil mass.

Soil classification and soil description as explained by Smith and Smith (2003) are two operations which are not necessarily the same, an operator who has not even visited the site from which the information obtained from grading and plasticity tests carried out on disturbed samples. Such tests are necessary if the soil is being considered as a possible

construction material and the information obtained from them must be included in any description of the soil.

2.3.1 Soil Classification System by Different Code of Practice

Ranjan and Rao (2005) said, soil in general, may be classified as cohesion-less and cohesive or as coarse-grained and fine-grained. These terms however, are two general and include a wide range of engineering properties. Hence, additional means of categorization are necessary to make the terms more meaningful in engineering practice. These terms are compiled to form Soil Classification Systems. Several classification systems were evolved by different organizations having a specific purpose as the object.

Ranjan and Rao (2005) submitted that Casagrande 1948 describes the systems developed and used in highway engineering, airfield construction, etc. The two classification systems, which are adopted by the US engineering agencies and the State Departments, are the Unified Soil Classification System (USCS) and the American Association of State Highway and Transport Officials (AASHTO) system. Other countries, including India, have mostly the USCS with minor modification (Ranjan and Rao, 2005).

Smith and Smith (2003) explained that the British Standard BS 5930 (1981), code of practice for site investigations, gives a full description of the BSCS. The system divides soil into two main categories. If at least 35percent of soil can pass through a 63UM sieve then it is a fine soil. Conversely, if the amount of soil that can pass through the 63UM sieve is less than 35percent then it is a coarse soil (Smith and Smith, 2003). Each category is divided into groups, depending upon the grading of the soil particles not passing the 63UM sieve and upon the plasticity characteristics of the soil particles passing the 424UM

sieve. Atkinson (2000) is also of the same opinion that the recommended standard for soil classification is the British soil classification system, BSCS.

a) The United Soil Classification System

The United Soil Classification System (USCS) originally developed by Casgrande (1948), was intended for use in airfield construction during World War II. Later in 1952, it was slightly modified to make it applicable to foundations, dams and other constructions. According to the USCS, the coarse-grained soils are soils (whose behaviour is controlled by plasticity) on the basis of their plasticity characteristics. Coarse-grained soils are those having 50 percent or more retained on the ϕ 200 (0.075mm) sieve and they are further subdivided into gravel (C_2). If 50 percent or more of the coarse fraction (+75 ϕ) is retained on the ϕ 4 (4.74mm) sieve; otherwise they are designated as sand (S). Fine-grained soils having more than 50 percent material passing the ϕ 200 (0.075mm sieve). These are subdivided into silt and clay, based on their liquid limit and plasticity index (Ranjan and Rao, 2005).

b) AASHTO Soil Classification System

The US Bureau of Public Roads (now the Federal Highway Administration) developed in the late twenties, a classification system called the Public Roads Administration Classification System which was especially meant for use in road construction. The original system has been revised several times and the present AASHTO (1978) Soil Classification System is essentially the system revised in 1945 (Ranjan and Rao, 2005).

The system is based on both the particle size and plasticity characteristics. According to the AASHTO system, the soils are classified into eight groups; A-1 through A-7 with an

additional group A-8 for peat or muck and system includes several sub-groups (Ranjan and Rao, 2005).

c) Indian Standard Soil Classification System.

The Indian standard soil classification system (ISSCS) for classification of soils for general engineering purpose, first published in 1959, was revised in 1970. The revision is based on the unified soil classification system (USCS) with the modification that the fine-grained soils have been subdivided into three groups as low, medium and high compressibility against only two in the USCS (Ranjan and Rao, 2005). The classification of coarse-grained soils is done on the basis of their grain and gradation characteristics, while the fine-grained soils are classified on the basis of their plasticity characteristics using I.S plasticity chart (Ranjan and Rao, 2005).

Coarse-grained soils which contain more than 12% fines (<75U) are classified as Gravel-Silt or Sand-Silt, if the fines are silty in character, they are classified as Gravel-clay or Sand-clay if the fines are clayey in character. Coarse-grained soils having 5% to 12% fines are borderline cases and given a dual symbol. The first part of the dual symbol is indicative of the gradation, while the second part indicates the nature of fines (Ranjan and Rao, 2005).

Table 2.1: Division of Soil Fractions on the Basis of Grain Sizes according to ISSCS

	Coarse-grained Soils				Fine-grained soils			
Boulder	Gravel		Sand		Silt and Clay size			
Cobble	Coarse	Fine	Coarse	Medium				
7300mm 300	80	20	4.75	2.0	0.425	0.075	<0.075mm	

Source: Ranjan and Rao, (2005)

2.3.2 Particle Size Tests and Analysis

The particle size analysis of a soil sample involves determining the percentage by mass of particles within the different size ranges (Craig, 2005) and according to Atkinson (2000) the aim of particle size tests is to measure the distribution of particle sizes in the sample. Atkinson (2000) says that when a wide range of sizes is present, the sample will be subdivided, and separate tests carried out on each sub-sample.

The particle size distribution of a coarse soil can be determined by the method of sieving. The soil sample is passed through a series a series of standard test sieves having a successively smaller mesh sizes. The mass of soil retained in each sieve if determined and the cumulative percentage by mass passing each sieve is calculated. If fine particles are present in the soil, the sample should be treated with deflocculating agent and washed through the sieves (Craig, 2005). Atkinson (2000) shares similar opinion that wet sieving can be used to separate fine grains from coarse grains by washing the soil specimen on a 60 μ M sieve mesh and dry sieving analysis can only be carried out on particles >60 μ M. Samples (with fines removed) are dried and shaken through a nest of sieves of descending size. Atkinson (2000) stated that sedimentation is used only for fine soils and that soil particles are allowed to settle from a suspension. He went further to say that the decreasing density of the suspension is measured at time intervals, sizes are determined from settling velocity and times recorded while percentage between sizes are determined from density differences.

The particle size distribution of a fine soil or fine fraction of coarse soil can be determined by the method of sedimentation. This method is based on Stokes' law which governs the velocity at which spherical particles settle in a suspension. The larger the particles, the

greater is the settling velocity and vice versa. The law does not apply to particles smaller than 0.0002mm, the settlement of which is influenced by Brownian movement. The size of particle is given as the diameter of sphere which would settle at the same velocity as the particle (Craig, 2005). Craig (2005) further observed that the particle size distribution of soil can be presented as a curve on a semi-logarithm plot, the ordinate being the percentage by mass of particles smaller than the size given by the abscissa. The flatter the distribution curve, the larger the range of particle sizes in the soil; the steeper the curve the smaller the size range. A coarse is described as well graded if there is no excess of particles in any size range and if no intermediate size are lacking. In general, a well graded soil is represented by a smoothly concave distribution curve. A coarse soil is described as poorly graded, if a high proportion of the particles have sizes within narrow limits (a uniform soil) or if particles of both large and small sizes are present but with a relatively low proportion of particles of intermediate size (a gap-graded or step-graded soil). Particle size is represented on a logarithmic scale so that two soils having the same degree of uniformity are represented by curves of the same shape regardless of their positions on the particle size distribution plot (Craig, 2005).

According to Atkinson (2000), the cumulative percentage quantities finer than certain sizes (e.g. passing a given size sieve mesh) are determined by weighing, points are then plotted of % finer (passing) against log size. He further discussed that a smooth S-shaped curve drawn through these points is called Grading Curve, and the position and shape of the grading curve determines the soil class.

A grading curve is a useful aid for soil description and is often included in investigation reports. The results of grading tests can be tabulated using geometric properties of the grading curve. These properties are called, Grading Characteristic (Atkinson, 2000).

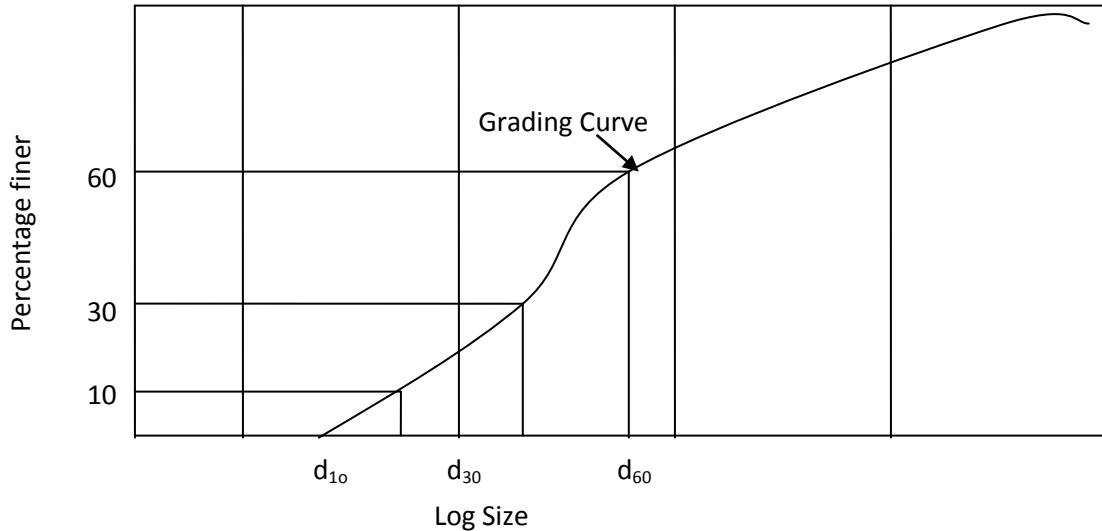


Fig 2.1: Grading Characteristic (Source: Atkinson 2000)

First of all, three points are located on the grading curve:

d_{10} = the maximum size of the smallest 10% of the sample.

d_{30} = the maximum size of the smallest 30% of the sample.

d_{60} = the maximum size of the smallest 60% of the sample (Atkinson, 2000).

From these grading characteristics are calculated: Effective size = d_{10} ,

Uniformity coefficient,

$$C_u = \frac{d_{60}}{d_{10}}$$

Coefficient of gradation,

$$C_K = \frac{d_{30}^2}{d_{60} d_{10}}$$

Both C_u and C_K will be 1 for a single-sized soil.

$C_u > 5$ indicates a well-graded soil.

$C_u < 3$ indicates a uniform soil

CK between 0.5 and 2.0 indicates a well-graded soil.

CK < 0.1 indicates a possible gap-graded soil (Atkinson, 2000).

In the British classification system, soils are classified into named basic soil type groups according to sizes and the groups further subdivided into coarse, medium and fine sub-groups. (Atkinson, 2000)

Table 2.2: Size Range of Grains

Very Coarse Soils	BOULDERS		>200mm
	COBBLES		60-200mm
Coarse Soils	G	Coarse	20-60mm
	GRAVEL	Medium	6-20mm
		Fine	2-6mm
		S	Coarse
	SAND	Medium	0.2-0.6mm
		Fine	0.06-0.2mm
M		Coarse	0.02-0.06mm
Fine Soil	SILT	Medium	0.006-0.02mm
		Fine	0.002-0.006mm
C	CLAY		< 0.002mm

Source: Atkinson(2000).

2.3.3 Size Identification and Shape of Grains

According to Atkinson (2000), soils possess a number of physical characteristics which can be used as aids to size identification in the field. He further stated that a handful of soil rubbed through the fingers can yield the following:

SAND (and coarse) particles are visible to the naked eye.

SILT-particles become dusty when dry and are easily brushed off hands and boots.

CLAY-particles are greasy and sticky when wet and hard when dry, and have to be scrapped or washed off hands and boots.

The majority of soils may be regarded as either SANDS or CLAYS: sands include gravelly-sands and gravel-sands. Sand grains are generally broken rock particles that have been formed by physical weathering, or they are the resistant components of rocks broken down by chemical weathering. Sand grains generally have a rotund shape. Clays include silty clays and clay-silts; there are few pure silts (e.g. area formed by windblown loess). Clay grains are usually the product of chemical weathering of rocks and soils. Clay particles have a flaky shape (Atkinson, 2000).

The thickness of clay is very small relative to their length and breadth, in some cases as thin as $\frac{1}{100\text{th}}$ of the length; therefore have high to very high specific surface values. These surfaces carry a small negative electrical charge, which will attract the positive end of water molecules. This charge depends on the soil mineral and may be affected by an electrolyte in the pore water; this causes some additional forces between the soil grains which are proportional forces between the soil grains which are proportional to the specific surface (Atkinson, 2000).

2.3.4 Plasticity of Fine Soils

Plasticity is an important characteristic in the case of fine soils. Craig (2005) corroborated that the term plasticity describing the ability of a soil to undergo unrecoverable deformation without cracking or crumbling. He further said depending on its water content (defined as the ration of the mass of water in the soil to the mass of solid particles), a soil may exist in one of the liquid, plastic, semi-solid and solid states. Clay soils have flaky

particles to which water adheres, thus imparting the property of plasticity (Atkinson, 2000).

If water content of a soil initially in the liquid state is gradually reduced, the state will change from liquid through plastic and semi-solid, accompanied by gradually reducing volume, until the solid state is reached. Plasticity is due to the presence of a significant content of clay mineral particles (or organic material) in the soil, the void space between such particles is generally very small in size with the result that water is held at negative pressure by capillary tension. This produces a degree of cohesion between the particles, allowing the soil to be deformed or molded. Adsorption of water due to the surface forces on clay mineral particles may contribute to plastic behavior (Craig, 2005).

According to Atkinson (2000) the consistency of a soil can range from (dry) solid to semi-solid to plastic to liquid (wet). He went further to say that consistency varies with the water content of the soil and that the water content at which the consistency changes from one state to the next are called CONSISTENCY LIMIT or (ATTERBERG LIMIT). Two factors are utilized in the classification of fine soil, these are; liquid limit (WL) (change of consistency from plastic to liquid) and plastic limit (WP) (change of consistency from brittle/crumby to plastic) (Atkinson, 2000).

Atkinson, (2000) submitted that the consistency of most soils in the ground will be plastic or semi-plastic or semi-solid and soil strength and stiffness behavior are related to range of plastic consistency. He then asserted that the range of water content over which a soil has plastic consistency is termed the Plasticity Index (IP or PI).

$IP = \text{Liquid Limit } (W_L) - \text{Plastic Limit } (W_P),$

$IP = W_L - W_P$

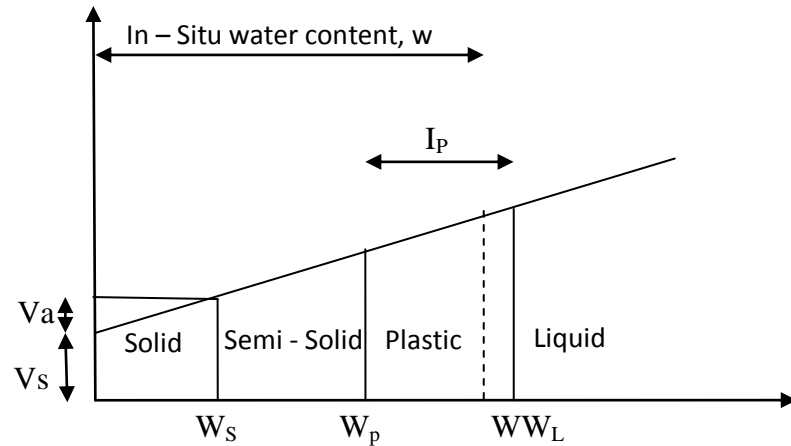


Fig 2.2: Plasticity Index of Soil
Source: Atkinson (2000)

Craig (2005) is of the same opinion that the upper and lower limit of the range of water content over which the soil exhibits plastic behavior are defined as the liquid limit (WL) and Plastic limit (WP), respectively. He then explained that the water content range itself is defined as the plasticity Index (IP), i.e. $IP = WL - WP$. However, the transitions between the different states are gradual and the liquid and plastic limits must be defined arbitrarily. The natural water content (W) of a soil (Adjusted to an equivalent water content of fraction passing the 425 - μm sieve) relative to the liquid and plastic limits can be represented by means of the Liquid Index (IL), where

$$IL = \frac{W - WP}{IP} \quad (\text{Craig, 2005}).$$

2.3.4.1 The plasticity Chart and Classification

In the British standard for clay and silt (BSCS), fine soil is divided into ten classes based on their measured plasticity index and liquid limit values: CLAYS are distinguished from SILTS and five divisions of plasticity are defined.

Low Plasticity	WL = <35%
Intermediate Plasticity	WL = 35% – 50%
High Plasticity	WL = 50% – 70%
Very High Plasticity	WL = 70 %– 90%
Extremely High Plasticity	WL = >90%

The plasticity chart is provided to aid classification (Atkinson, 2000)

2.3.4.2 Activity

Atkinson, (2000) pointed out the fact that the so-called “Clay” is not 100% clay. He further stated that the proportion of clay mineral flakes (<2µm size) in a fine soil affects its current state, particularly its tendency to swell and shrink with changes in water content and degree of plasticity related to clay content is called ACTIVITY of the soil. Craig (2005) also opined that the degree of plasticity of the clay size fraction of a soil is expressed by the ratio of the plasticity index to the percentage of clay-size particles in the soil is called the ACTIVITY. That is

$$\text{Activity} = \frac{\text{IP}}{\% \text{ clay particles}}$$

Source: (Atkinson, 2000; Craig 2005)

Some typical values of activity are shown below:

Table 2.3: Typical value of activity of soil

MINERAL	ACTIVITY	SOIL	ACTIVITY
Muscovite	0.25	Kaolin clay	0.4 – 0.5
Kaolinite	0.40	Glacial clay and loess	0.5 – 0.75
Illite	0.90	Most British clays	0.75 – 1.25
Montmorillonite	>1.25	Organic estuarine clay	>1.25

Source: Atkinson (2000)

According to Craig (2005) plasticity of fine soils can be assessed by means of the toughness and dilatancy tests and an assessment of dry strength may also be useful.

2.3.4.3 Toughness test

A small piece of soil is rolled out into a thread on flat surface or on the palm of the hand, moulded together and rolled out again until it has dried sufficiently to break into lumps at a diameter of around 3mm. In this condition, inorganic clay of high liquid limit is fairly stiff and tough; those of low liquid limit are softer and crumble more easily (Craig, 2005).

2.3.4.4 Dilatancy test

A pat of soil with sufficient water added to make it soft but not sticky, is placed in the open (horizontal) palm of hand, the side of the hand is then struck against the other hand several times. Dilatancy is indicated by the appearance of a shiny film of water on the surface of the pat; if the pat is then squeezed or pressed with the fingers, the surface becomes dull as the pat stiffens and eventually crumbles. These reactions are pronounced only for predominantly silt size material and for very fine sands, but plastic clay gives no reaction (Craig, 2005).

2.3.4.5 Dry strength test

A pat of soil about 6mm thick is allowed to dry completely, either naturally or in an oven, the strength of the dry soil is then assessed by breaking and crumbling between the fingers. Inorganic clays have relatively high dry strength, the greater the strength the higher the liquid limit. Inorganic silts of low liquid limit have little or no dry strength, crumbling easily between the fingers (Craig, 2005).

2.4 Clays and Clay Mineral Deposits

The term “clay” is applied both to materials having a particle size of less than 2 micrometers (25, 400micrometers = 1inch) and to the family of minerals that has similar chemical compositions and common crystal structural characteristics (Velde, 1995).

According to Ranjan and Rao (2005) clay is normally understood to mean a clay soil whose grains are predominantly composed of clay minerals and which has plasticity and cohesion. Though the clay soils are fine-grained, not all fine-grained soils possess plasticity and cohesion (Ranjan and Rao 2005). The presence of water, which is relatively unimportant in coarse-grained soils, plays a decisive role in the engineering behavior of a clay soil. On the other hand, grain size distribution and grain shape influence the engineering behavior of granular soils and hardly affect the behavior of clay soils. (Ranjan and Rao, 2005).

Clays are distinguished from other fine-grained soils by various differences in composition. Silts which are fine-grained soil which do not include clay mineral, tend to have larger particle sizes than clays, but there is some overlap in both particle size and other physical particles properties, and there are many natural deposits which include both silts and clays (Guggenheim and

Martin, 1995). The distinction between silt and clay varies by discipline. Geologist and Soils Scientists usually consider the separation to occur at a particle size of 2 μ m and colloid chemists use 1 μ m. Geotechnical engineers distinguished between silt and clays based on the plasticity properties of the soils' Atterberg limit (Guggenheim and Martin, 1995).

Primary clays are known as kaolin and are located at the site of formation, while secondary clays deposit have been moved by erosion and water from their primary location (Guggenheim and Martin, 1995). Ranjan and Rao (2005) confirmed that clay materials are basically composed of tiny crystalline substances of one or more members of small group of minerals commonly known as CLAY MINERALS. He further discussed that clay minerals are evolved mainly from the chemical weathering of certain rock-forming minerals, these minerals are hydrous aluminosilicates with other metallic ions, their particles are very small in size, very flaky in shape and thus have considerable surface area and they can only be viewed with an electron microscope.

Clays and minerals have been mined since the Stone Age; today they are among the most important minerals used by manufacturing and environmental industries (Nora, 1999). Clay minerals have a wide range of particle sizes from 10's of angstroms to millimeters. (An angstrom (A) is a unit of measure at the scale of atoms), thus clay may be composed of mixtures of finer-grained clay minerals and clay-sized crystals of other minerals such as quartz, carbonate, and metal oxides. Clays and clay minerals are found mainly on or near the surface of the Earth.

Ranjan and Rao (2005) indicated that clay minerals, on the basis of their crystalline arrangement, can be divided into three main groups. They further observed that more or

less similar engineering properties are exhibited by all clay minerals belonging to a particular group.

Clay minerals are typically formed over long periods of time by gradual chemical weathering of rocks (usually silicate-bearing by low concentrations of carbonic acid and other diluted solvents) these solvents (usually acidic) migrate through the weathering rock after leaching through upper weathered layers ((Guggenheim and Martin, 1995). Clay deposits may be formed in place as residual deposits. But thick deposits usually are formed as the result a secondary sedimentary deposition process after they have been eroded and transported from their original location of formation and these deposits are typically associated with very low energy depositional environments such as large lake and marine deposits (Guggenheim and Martin, 1995).

According to Nora (1999), clays and clay minerals occur under a fairly limited range of geologic conditions. He then stated that the environments of formation include soil horizons, continental and marine sediments. Geothermal field, volcanic deposits and weathering rock formation. Most clay minerals form where rocks are in contact with water, air or stream, examples of these situations include; weathering boulders on a hillside, sediments on sea or lake bottoms, deeply buried sediments containing pore water, and rocks in contact water heated by magma (molten rock). All these environments may cause the formation of clay minerals from pre-existing minerals (Nora, 1999). Extensive alteration of rocks to clay minerals can produce relatively pure clay deposits that are of economic interest (for example, bentonites, primarily montmorillonite) used for drilling mud and clays used in ceramics (Nora, 1999).

2.4.1 Classes of Clay Minerals

Smith and Smith (2003) established the fact that for the production of particles smaller than 0.002mm, some form of chemical action is generally necessary before breakdown can be achieved and such particles, although having a chemical content similar to the parent rock, have a different crystalline structure and are known as clay particles.

The minerals comprising clay are invariably the result of the chemical weathering of rock particle and are hydrates of aluminum ion or magnesium silicate generally combined in such a manner to create sheet-like structures only a few molecules thick (Smith and Smith, 2003). These sheets are built from two basic units the tetrahedral unit of silica and the octahedral unit of the hydroxide of aluminum, ion or magnesium. The main dimension of clay particle is usually less than 0.002mm and the different types of minerals have been created from the manner in which these structure were stacked together (Smith and Smith, 2003). The three main groups of clay according to Smith and Smith (2003) are as follows:

2.4.1.1 Kaolinite group

This mineral is the most dominant part of clay deposits and is made up of large stacks of alternating single tetrahedral sheets of silicate and octahedral sheets of aluminum. Kaolinites are very stable with a strong structure and absorbed little water. They have low swelling and shrinkage responses to water content variation (Smith and Smith, 2003).

2.4.1.2 Illite Group

This group consists of series of single octahedral sheets of aluminum sandwiched between two tetrahedral sheets of silicon. In the octahedral sheets some of the aluminum is replaced by ion and magnesium and in the tetrahedral sheets there is partial replacement of silicon

by aluminum. Illites tend to absorb more water than kaolinites and have higher swelling and shrinkage characteristics (Smith and Smith, 2003).

2.4.1.3 Montmorillonite Group

This mineral has a similar structure to the illites group in the tetrahedral sheets; some of the silicon is replaced by iron, magnesium and aluminum. Montmorillonite exhibits extremely high water absorption, swelling and shrinkage characteristics. Bentonite is a member of this mineral group and is usually formed from weathered volcanic ash because of its large expansive properties when it is mixed with water, it is much in demand as general grout in plugging of leaks in reservoirs and tunnels, it is also used as a drilling mud for soil borings (Smith and Smith, 2003).

Ranjan and Rao (2005) are of the same opinion in the classes of clay minerals. The classification is shown in the table 2.4 below.

Table 2.4: Clay Minerals

Name of Mineral and Group	Structural Formula
Kaoline Group	
a) Kaolinite	$(OH)_4Al_4Si_4O_{10}$
b) Halloysite	$(OH)_4Al_4Si_4O_{10}.4H_2O$
Montmorillonite Group	
Montmorillonite	$(OH)_4Al_4Si_yO_{20}.nH_2O$
Illite Group	
Illite	$(OH)_4K_y(Si_x-yAl_y)(Al_4.Mg_6.Fe_4.Fe_6)O_{20}$

Source: Ranja and Rao (2005)

2.4.2 Structure of a Clay Deposit

2.4.2.1 Macrostructure

The visible features of a clay deposit collectively form its macrostructure and include such features as fissures, root holes, bedding patterns. Silt and Sand seams or lenses and other discontinuities. A study of the of macrostructure is important as it usually has an effect on the behavior of the soil mass. For example the strength of an unfissured clay mass is much stronger than along a rock (Smith and Smith, 2003).

2.4.2.2 Microstructure

The structural arrangement of microscopic sized clay particle defines the microstructure of a clay deposit. Clay deposits have been laid down under water and were created by the settlement and deposition of clay particles out suspension. Often during their deposition, the action of Van der Waal forces attracted clay particles together and created flocculent, or honey combed structures which, although still microscopic are of considerably greater volume than single clay particles. Such groups of clay particles are referred to as CLAY FLOCS (Smith and Smith, 2003)

2.4.3 Physical and Chemical Properties of Clays

The characteristics common to all clay minerals derived from their chemical composition, layer, structure and size (Nora, 1999). Ranjan and Rao (2005) submitted that the physical properties of clays are considerably influenced by the amount of water present in them. Depending upon the water content, the liquid state, the plastic state, the semi-solid state and the solid state are the four stages or states of consistency used to describe the consistency of clay soils (Ranjan and Rao, 2005).

All clay minerals have a great affinity for water. Some swell easily and may double in thickness when wet and most have the ability to soak up ions (electrically charged atoms and molecules) from a solution and release the ions later when conditions change (Nora, 1999).

Water molecules are strongly attracted to clay minerals surfaces. When a little clay is added to Water, a slummy forms because the clay distributes itself evenly throughout the water. This property of clay is used by the paint industry to disperse pigment (color) evenly throughout paint. Without clay to act as a carrier, it would be difficult to evenly mix the paint base and colour pigment (Nora, 1999).

A mixture of a lot of clay and a little water results in a mud that can be shaped and dried to form a relatively rigid solid, therefore, this property is exploited by potters and the ceramics industry to produce plates, cups, bowls, pipes and so on and also environmental industries use both these properties to produce homogeneous liners for containment of waste (Nora, 1999).

The process by which some clay minerals swell when they take up water is reversible, swelling clay expands or contracts in response to changes in environmental factors (wet and dry conditions, temperature) (Velde, 1995). Hydration and dehydration can vary the thickness of a single clay particle by almost 100% (e.g. a 10Å – thick clay mineral can expand to 19.5Å in water) (Velde, 1995). Nora (1999) asserted that houses, offices, schools and factories built on soil containing swelling clays may be subject to structural damage caused by seasonal swelling of the clay portion of the soil.

Another important property of clay minerals, is the ability to exchange ions, relates to the charged surface of clay minerals. Ions can be attracted to the surface of a clay particle or

taken up within the structure of these minerals (Nora, 1999). The property of clay minerals that causes ion in solution to be mixed on clay surfaces or within internal sites applies to all types of ions, including organic molecules like pesticides (Nora, 1999).

2.4.4 Consistency of Clay: Atterberg Limit

Consistency of clays as viewed by Ranjan and Rao (2005) is a term which is used to describe the degree of firmness of soil in a qualitative manner by using descriptions such as soft, medium, firm, stiff or hard and it indicates the relative ease with which a soil can be deformed. Ranjan and Rao (2005) said that in practice, the property of consistency is associated only with fine-grained soils, especially clays.

According to Ranjan and Rao (2005), a Swedish soil scientist, Atterberg in 1911, first demonstrated the significance of the CONSISTENCY LIMIT also known as the ATTERBERG LIMIT. They continued by saying that, these limits of water content, though empirical in nature, are of great significance in understanding the behavior of clays. When a fine-grained soil is mixed thoroughly with a large quantity of water, the resulting suspension is in liquid state, and offers practically no resistance to flow, thus the soil has virtually no shear strength (Ranjan and Rao, 2005). If the water content of the suspension is gradually reduced keeping the consistency of the sample uniform, a stage comes when it just begins offering resistance to flow. This is the stage when the sample changes from possessing no shearing strength to having infinitesimal shear strength and changes from the liquid to the plastic state (Ranjan and Rao, 2005). Smith and Smith (2003) are also of the same opinion that Atterberg (1911) determined the various values of water content at which changes in a soil's strength characteristics occur.

With a cohesion-less soil, i.e. a gravel or sand, both parameters are only slightly affected by a change in water content where as a cohesive soil, i.e. silt or a clay, tends to become considerably stronger and less compressible, i.e. less easy to mould, as it dries out (Smith and Smith, 2003). It was reported by Smith and Smith (2003) that particle size analysis will completely define a gravel and sand but it is necessary to carry out plasticity tests in order to fully classify a clay or fine silt and that the test was first carried out by Atterberg in 1911.

Ranjan and Rao (2005) pointed out the fact that the boundary water content between the liquid state and the plastic is called the LIQUID LIMIT. In the plastic state, the soil can be moulded to different shapes without rupturing it, due to plasticity but if the water content is further reduced, the clay sample changes from the plastic state to semi-solid state at a boundary water content which is called the PLASTIC LIMIT. In the semi- solid state, the soil does not have plasticity, it becomes brittle and when pressure is applied, the soil simply crumbles (Ranjan and Rao, 2005). Up to the semi-solid state, the soil remains fully saturated and any reduction in the volume of water will result in an almost equal reduction in the volume of the soil mass (Ranjan and Rao, 2005). A further reduction in the water content, however, brings about a state when with a decrease in moisture, the volume of the soil mass does not decrease any further but remains the same; the sample changes from the semi-solid to the solid state. The boundary water content is called the SHRINKAGE LIMIT. Below this limit, the sample begins to dry up at the surface and the soil is no longer fully saturated and the colour of the sample also begins to change as shown in the figure below (Ranjan and Rao, 2005).

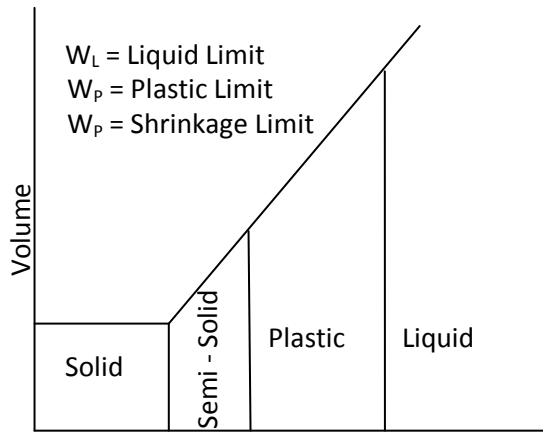


Fig 2.3: Atterberg Limits
 Source: Ranjan and Rao (2005).

The limits described above are all expressed in percentages of water content and the different states of consistency through which a soil mass passes through, with the decrease in water content is depicted in fig 2.5 below (Ranjan and Rao, 2005).

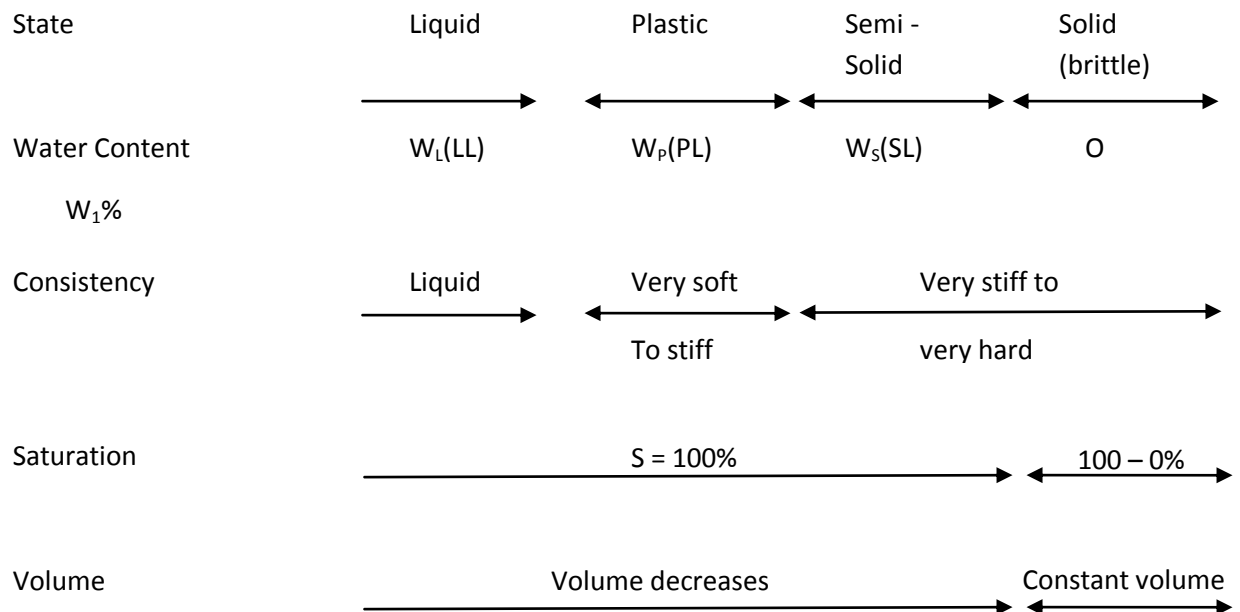


Fig 2.4: Different stages, consistencies and Atterberg limits.
 Source: Ranjan and Rao (2005)

2.4.5 Determination of Liquid Limit, Plastic Limit and Shrinkage Limits

Liquid limit test: according to Smith and Smith (2003) BS 1377, part 2 (1990) specifies the following three methods for determining the liquid limit of soil.

2.4.5.1 Cone Penetrometer Method (Definitive Method)

The soil to be tested is usually air dried under this method and thoroughly mixed. At least 200g of soil is sieved through a 425 μ m sieve and placed on a glass plate. The soil is then mixed with distilled water into paste (Smith and Smith, 2003). A metal cup, approximately 55mm in diameter and 40mm deep is filled with the paste and the surface struck of level. The cone, of mass 80g, is next placed at the center of the smoothed soil surface level with it (Smith and Smith, 2003).

The cone is released so that it penetrates into the soil and the amount of penetration over a period of 5seconds, is measured. The test is now repeated by lifting the cone clear, cleaning it and filling up the dression in the surface of the soil by adding more of the wet soil. If the difference between the two measured penetrations is less than 0.5mm, then the tests are considered valid. The average penetration is noted and a moisture content determination is carried out on the soil tested (Smith and Smith 2003).

The procedure is repeated at least four times with increasing water contents. The amount of water used should be such that the penetrations obtained lie within a range of 15 to 25mm. to obtain the liquid limit the variation of cone penetration (plotted vertically) to moisture content (plotted horizontally) is drawn out (both scales being natural). The best straight line is drawn through the experimental points and the liquid limit is taken to be the moisture content corresponding to a cone penetration of 20mm (expressed as a whole number) (Smith and Smith, 2003).

a) **One-point cone penetrometer method:** In this test, the procedure is similar to that described above, with exception that only one point is required. The test is thus fairly rapid. Once the average penetration for the point is established, the moisture content of the soil is determined. The moisture content is then multiplied by a factor to give the liquid limit. The value of the factor is dependent on both the cone penetration and the range of moisture content falls. The factors were determined through experimental work performed by Clayton and Jukes (1978) in Smith and Smith (2003).

b) **Method using the Casagrande Apparatus:**

According to Smith and Smith (2003), until 1975 this was the only method for determining liquid limit that was recognized by the British Standard Institution. He established the fact that although still used worldwide, the test is now largely superseded by cone penetration techniques.

Casagrande apparatus for the determination of liquid limit as explained by Ranjan and Rao (2005), is designed by A. Casagrande ; the apparatus consists of a vulcanized rubber compound base with a brass cup suitably mounted. The brass cup can be raised and made to fall on the rubber base through a cam arrangement operated by a handle. The height of a fall of the brass cup can be adjusted to 10mm. a grooving tool is used to cut a groove in the pat of soil placed in the cup (Ranjan and Rao, 2005). Two types of grooving tools are needed; these are

(i) Casagrande tools which cuts a groove of 2mm width at the bottom, 11mm width at the top and 8mm deep and

(ii) ASTM tool which cuts a groove 2mm wide at the bottom, 12mm wide at the top and 10mm deep. The selection of the grooving tools depends upon the type of soil. For a soil of low plasticity, the ASTM grooving tool is preferred (Ranjan and Rao, 2005).

About 120g of air-dried soil passing through IS sieve 425 micron is taken and mixed with water such that the soil attains a putty-like consistency. A portion of the paste is placed in cup and leveled so as to have a maximum depth of about 10mm. A groove is cut in the soil placed in the cup, using the grooving tool. In cutting the groove, the grooving tool is drawn through the sample along the symmetrical axis of the cup, holding the tool perpendicular to the cup. The handle is rotated at the rate of 2 revolutions per second and the number of blows necessary to close the groove for a distance of 13mm is noted. The groove is then closed by flow and not by slippage of soil. Then, about 10g of soil near the close groove is taken to determine its water content (Ranjan and Rao, 2005). By altering the water content of the soil and repeating the operations mentioned above, four to five readings water content in the range of 10 to 40 blows are obtained. A graph is then plotted between number of blows, N on a logarithmic scale and the water content, W on a natural scale as shown in figure 2.7. It will be seen that the semi-logarithmic plot is a straight line called the "*flow curve*" (Ranjan and Rao, 2005). The liquid limit is determined by reading the water content corresponding to 25, blows on the flow curve.

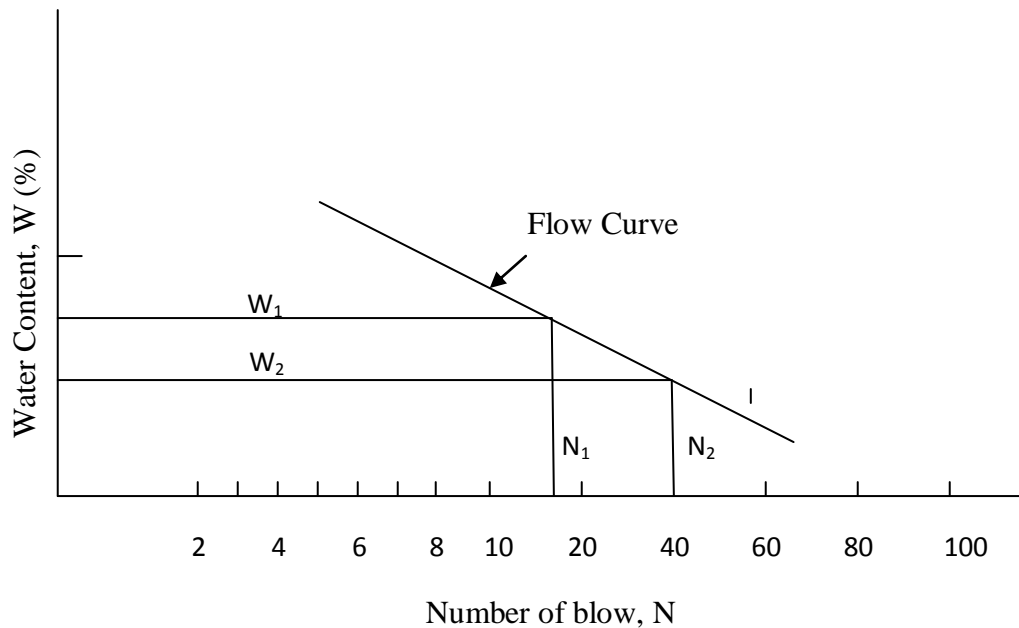


Fig. 2.5: Flow Curve
 Source: Ranjan and Rao (2005).

a) Plastic limit test: Smith and Smith (2003) carried out a research to determine the plastic limit of soil sample in which about 20g of soil prepared as in the liquid limit test are used. The soil is mixed on the glass plate with just enough water to make it sufficiently plastic for rolling into a ball, which is then rolled out between the hand and the glass to form a thread. The soil is said to be at its plastic limit when it just begins to crumble at thread diameter of 3mm. At this stage a section of the thread is removed for moisture content determination (Smith and Smith, 2003). However, it was suggested by test should be repeated at least once more.

According to Smith and Smith (2003), it is interesting to note that, Russia and P.R China, the cone penetrometer is used to determine both liquid limit (WL) and plastic limit (WP) and that the apparatus used consists of a 30° included angle cone with a total mass of 76g. Smith and Smith (2003) further explained that the test is the same as the liquid limit test in

Britain, a penetration of 17mm giving WL and a penetration of 2mm giving plastic limit (Wp).

In a similar development, Ranjan and Rao, (2005) also embarked on a research on plastic limit determination. The research was conducted with soil sample of about 15g of air-dried soil passing through IS sieve 425micron. The air-dried soil sample is taken for plastic limit determination and is mixed with a sufficient quantity of water which would enable the soil mass to become plastic enough to be easily shaped into a ball. A portion of the ball is taken and rolled on a glass plate with the palm of the hand into a thread of uniform diameter throughout its length. When a diameter of 3mm is reached, the soil is remolded into a ball. The process of making the thread and remolding is continued till the thread at a diameter of 3mm, just starts crumbling. Some of the crumbled portion of thread is kept in the oven for water content determination. The test is repeated twice with fresh samples and the average of the three values water contents is taken as the plastic limit (Ranjan and Rao, 2005).

b) Shrinkage limit test: Ranjan and Rao, (2005) conducted a research on shrinkage limit of clay soil. At the beginning of the research work about 30g of soil passing through IS: 425micron sieve is taken in an evaporating dish. The soil is mixed with sufficient quantity of water to bring the soil to a consistency that it may flow. The soil mixture is placed in the shrinkage dish in three equal quantities so as to fill the dish. The excess soil is removed and the dish is weighed with soil. The soil pat is allowed to dry in the air till the colour of the pat changes from dark to light. The dish is then placed in the oven at 110⁰C till its weight becomes constant. The shrinkage dish is weighed with the dry sample and the dry weight of soil is determined and the volume of dry pat of soil is measured by the MERCURY DISPLACEMENT METHOD (Ranjan and Rao, 2005).

The shrinkage limit is calculated as shown in fig 2.6 below.

- a) Represent the soil sample in plastic state which fills the container of known value V_1 and has a weight W_1 . As the sample is gradually dried, the water content at certain stage becomes equal to the shrinkage limit as shown in fig 2.6(b).
- b) At this point, the volume decreases to V_2 , the corresponding weight being W_2 . The sample is still in a saturated state. Beyond the shrinkage limit, the sample continues to dry with no further volume decrease, until it reaches the dry state represented in figure 2.6(c)
- c) At this stage, the dry weight of the soil sample W_s and its volume V_2 are measured

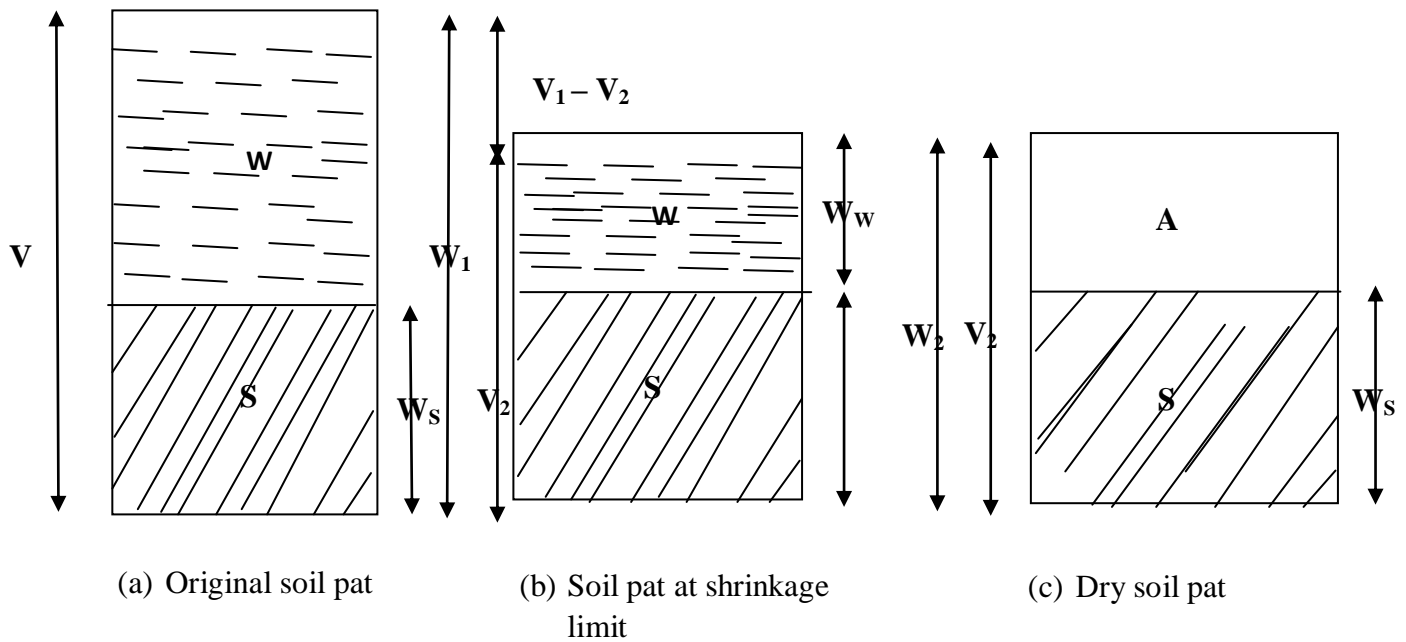


Fig 2.6: Shrinkage limit determination

Source: Ranjan and Rao (2005)

2.5 Soil Stabilization

Mukerji and Strulz (1988) submitted that, soils that do not possess the desired characteristic for a particular construction can be improved by adding one or more

stabilizers. Mukerji and Strulz (1988) then stated that each stabilizer can fulfill one (or at the most two) of the following functions:

- i) Increase the compressive strength and impact resistance of the soil construction and also reduce binding the particles of soil together.
 - ii) Reduces or completely excludes water absorption (causing swelling, shrinkage and abrasion) by seating all voids or pores, and covering the clay particles with a water proofing film.
 - iii) Reduces cracking by imparting flexibility which allows the soil to expand and contract to some extent.
- c) Reduces excessive expansion and contraction by reinforcing the soil with fibrous material (Mukerji and Strulz, 1988).

According to Arora (2005) soil stabilization is the process of improving the engineering properties of the soil and thus making it more stable. He further observed that, it is required when the soil available for construction is not suitable for the intended purpose. However, the term stabilization is generally restricted to the processes which alter the soil material itself for improvement of its properties (Arora, 2005). The effect of stabilization is usually increased when the soil is compacted and sometimes compaction alone is sufficient to stabilize, the effect may not be permanent, particularly in case of increased exposure to water (Mukerji and Strulz, 1988)

Soil stabilization can be used to serve different purposes and different functions depending on the intention of the researcher. As opined by Arora (2005), soil stabilization is used to reduce the permeability and compressibility of the soil mass in earth structures and to increase its shear strength. He also stated that soil stabilization is required to increase the

bearing capacity of foundation soils and also improve the natural soils for the construction of highways and airfield. The principles of soil stabilization are used for controlling the grading of soils and aggregates in the construction of bases and sub-bases of highways and airfields (Arora, 2005). Soil stabilization is also used to make an area trafficable within a short period of time for military and other emergency purposes and sometimes soil stabilization is used for city and suburban streets to make them more noise absorbing (Arora,2005).

Afjman (1994) asserted that qualities are the two common improvement and thickness reduction established through soil stabilization under the quality improvement includes better soil gradation, reduction of plasticity index or swelling potential, and increases in durability and strength (Afjman, 1994).

Under the thickness reduction, the strength and stiffness of a soil layer can be improved through the use of additives to permit a reduction in design thickness of the stabilized material compared with un-stabilized or unbound material (Afjman, 1994).

Mukerji and Strulz (1988) say before considering the use of stabilizer the following points must be investigated:

i) Does the available soil satisfy the main requirements even without stabilization?

This is largely dependent on the local climate, natural hazards and type of construction (Mukerji and Strulz, 1988).

ii) Does the building design take into account the characteristics and limitations of the material?. Building on a high level and incorporating damp-proof course (to minimize damage by rising water) and providing against rain and solar radiation are examples of appropriate design (Mukerji and Strulz, 1988).

iii) Is the stabilization of the entire construction really necessary, or can a good surface protection (e.g. stabilized render) be sufficient? (Mukerji and Strulz 1988).

2.5.1 Kinds of Stabilizers

A great number of substances may be used for soil stabilization and much research is going on to find the most suitable stabilizer for each soil type. But despite these research efforts, there is no “miracle” stabilizer that can be used in all cases (Mukerji and Strulz, 1988).

Stabilizer is not the exact science, so that it is up to the builder to make trial blocks with various kinds and amounts of stabilizers which can be tested (Mukerji and Strulz, 1988).

The most common naturally available stabilizers used in traditional constructions occurring to Mukerji and Strulz (1988) are:

- a) Sand and clay
- b) Straw, plant fibres
- c) Plant juice (sap, latexes, oils)
- d) Wood ashes (cinders)
- e) Animal excreta (mainly cow dung, horse urine)
- f) Other animal products (blood, hair, glues, termite hills)

Also the most common manufactured stabilizers (i.e. products or by-products of local village industries or large industrial processes) according to Mukerji and Strulz (1988) are:

- i) Lime and pozzolanas
- ii) Portland cement
- iii) Gypsum
- iv) Bitumen
- v) Commercial soil stabilizers

- vi) Sodium silicate (water glass)
- vii) Resins
- viii) Whey (casein)
- ix) Molasses

2.5.2 Lime Stabilization

According to Arora (2005), lime is produced by burning of limestone in kilns. He said the quality of lime obtained depends upon the parent material and the production process.

There are basically five (5) types of lime according to Arora (2005). These are:

- i) High Calcium, quick lime (CaO)
- ii) Hydrated, high Calcium lime [Ca(OH)₂]
- iii) Dolomitic lime (CaO +MgO)
- iv) Normal, hydrated dolomitic lime [Ca(OH)₂ + MgO₂]. Afjman (1994) also opined that various forms of lime have been successfully used as soil stabilizing agents for many years and the most commonly used products are hydrated high-calcium lime, mono hydrated dolomitic lime, calcitic quick lime and dolomitic quick lime. Afjman (1994) further stated that hydrated lime is used most often because it is much less caustic than quick lime. However, the use of quick lime for soil stabilization has increased with slurry type applications. Arora (2005) established the fact that quick lie is more effective as stabilizer than the hydrated lime; but the latter is more safe and convenient to handle.

Arora (2005) said lime stabilization is done by adding lime to soil and this is useful for stabilization of clayey soils. Lime stabilization refers to stabilization of the soils by the addition of burned lime stone products, either calcium oxide or calcium hydroxide (Bell 1988 cited in Gokceoglu, Tonoç & Ulusay, 2004). Mukerji and Strulz (1988) argued that

clayey soil (with liquid limits in the region of 40% or more) can be stabilized only with lime, as it reacts with the clay particles in the soil to form a binder. Lime stabilization occurs when lime is added to reactive soil to generate long-term strength gain through a pozzolanic reaction, this reaction produces stable calcium silicate hydrates and calcium aluminate hydrates as the calcium from the lime reacts with the aluminates and silicates solubilized from the clay (NLA, 2001). When lime reacts with soil, there is exchange of cations in the adsorb water layer and a decrease in plasticity of the soil occurs, the resulting material is more friable than the original clay, and is, therefore, more suitable as sub-grade (Arora, 2005).

The effects of lime stabilization on clay soils have been investigated by numerous researchers. Because the properties of soil lime mixtures depend upon character of clay soil, type and period of curing, the method and quality of construction, and the proper amount of lime to be used should be investigated before the application of lime stabilization (Gokceoglu et al, 2004).

Gokceoglu et al, (2004) investigated the performance of quicklime in powder form on laboratory-scaled models to improve physical, swelling and strength characteristics of the clay. The lime-soil mixture design technique was employed in the laboratory, and five different types of lime-clay mixture between 2% and 10% by weight were prepared. Engineering properties of the clay determined from natural and lime-treated samples cured for 3, 7, 14 and 28 days were compared. As a result of reactions, Uniaxial Compressive Strength (UCS) increased about 84% after 28days. However, the results indicated that if curing period is less than 28days, the UCS values of the lime-treated samples are higher than those of natural samples. This suggests that the pozzolanic reaction is slower than

flocculation Ankara (a town in Turkey) clay, which achieves optimum stabilization at 4% lime content and 28 days curing time when swelling is considered (Gokceoglu et al, 2004). Osinubi (1995) also carried out a research on lime modification of black cotton soil. The results obtained shows that the addition of lime improved the engineering properties of black cotton soil which was classified as an A – 7 – 6 soil according to the AASHTO (1986) classification system. The liquid limit of the natural soil reduced from 62.3% to 54% when treated with 4% lime content while the plasticity index reduced from 38.1% to 23% and the percentage passing BS sieve No 200 reduced from 60% to 20%. It was however concluded by him that the modified soil can be stabilized with cement since its workability was improved by addition of 4% lime content.

Apart from lime stabilization, there are various other methods of soil stabilization. These are:

i) Mechanical stabilization (Granular stabilization): This method is achieved by gradation and compaction and does not include the addition of another material. Mechanical stabilization is the process of improving the properties of the soil by changing the gradation (Arora, 2005). Two or more types of natural soils are mixed to obtain a composite material which is superior to any of its components. To achieve the desired grading sometimes the soils with coarse particles are added or the soils with fine particles are removed (Arora, 2005). Mechanical stabilization is the simplest method of soil stabilization and it is generally used to improve the sub-grades of low bearing capacity. It is extensively used in the construction of bases, sub-bases and surfacing of roads (Arora, 2005).

ii) **Cement stabilization:** Cement stabilization is done by mixing pulverized soil and Portland cement with water and compacting the mix to attain a strong material (Arora, 2005). Soils with low clay contents are best stabilized with Portland cement, which binds the sand particles and gravel in the same way as in concrete, that is it reacts with the water in the soil mixture to produce a substance which fills the voids, forming a continuous film around each particle, binding them all together (Mukerji and Strulz, 1988). The reaction of cement and water (known as hydration) liberates calcium hydroxide (slaked lime) which reacts with of pozzolanic binder. If the clay content is too low the lime remains free, this can be remedied by replacing a proportion (15 to 40%) by weight usually cheaper than cement (Mukerji and Strulz, 1988).

The material obtained by mixing soil cement is known as *soil – cement*. The soil cement becomes a hard and durable structural material as the cement hydrates and develops strength (Arora, 2005). Just as in cement- sand mortar, soil-cement mixes become more workable by adding lime. Michell and Freitag (1959) cited in Arora (2005) divided soil-cement into three categories:

a) **Normal soil-cement:** It consists of 5 to 14% of cement by volume. The quantity of cement mixed with soil is sufficient to produce a hard and durable construction material and the quantity of water used should be just sufficient to satisfy hydration requirements of the cement and to make the mixture workable. The normal soil-cement is quite weather resistant and strong. It is commonly used for stabilizing sandy and other low plasticity soils.

b) **Plastic soils-cement:** this type of soil cement also contains cement 5 to 14% by volume, but it has more quantity of water to have wet consistency similar to that of plastering

mortar at the time of placement. The plastic soil cement can be placed on steep or irregular slopes where it is difficult to use normal road making equipment, and it has also been successfully used for water-proof lining of canals and reservoirs. The plastic soil-cement can be used for protection of steep slopes against erosive action of water.

c) Cement- modified soil: It is a type of soil-cement that contains less than 5% of cement by volume. It is a semi- hardened product of soil and cement. It is quite inferior to the other two types. As the quantity of cement used is small, it is not able to bind all the soil particles into a coherent mass. However, it interacts with the silt and clay fractions and reduces their affinity for water; it also reduces the swelling characteristic of soil. The use of cement- modified soil is limited.

According to Afjman (1994), Portland cement can be used as a cement stabilizer, either to modify and improve the quality of soil or to transform of soil into a cemented mass with increased strength and durability. The amount of cement used will depend upon whether the soil is to be modified or stabilized (Afjman, 1994).

iii) Bituminous Stabilization: Bitumen are non-aqueous systems of hydrocarbons that are soluble in carbon disulphide. Tars are obtained by the destructive distillation of organic materials such as coal while asphalts are materials in which the primary components are natural or refined petroleum bitumen (Arora, 2005). Bituminous stabilization is generally done with asphalt as binder, as asphalt are normally too viscous to use directly, these are used as cut-back with some solvent, such as gasoline and they are also used as emulsion, but in this form they require a longer drying period (Arora, 2005).

Afjman (1994) explained that stabilization of soils and aggregates with asphalt differs greatly from cement and lime stabilization. Afjman (1994) further observed that the basic

mechanism involved in asphalt stabilization of fine-grained soils is a waterproofing phenomenon. Soil particles or soil agglomerates are coated with asphalt that prevents or slows the penetration of water which could normally result in a decrease in soil strength (Afjman, 1994). Dahunsi (2008) also of the same opinion that the use of bitumen/asphalt prevent percolation of water i.e. it waterproofs the soil, hence this allows the soil to retain its moisture content for a long time. With the absence of water, the moisture content will be kept constant, hence the soil quality will be fairly stable, this is so because the higher the moisture content, the lower the strength of the soil (Dahunsi, 2008). Asphalt stabilization can improve durability characteristics by making the soil resistant to the detrimental effects of water such as volume (Afjman, 1994). The amount of bitumen required generally varies between 4 to 7% by weight and the actual amount is determined by trial (Arora, 2005). Important facts to be known when using bitumen are the quantity and the quality of the C or asphalt (Dahunsi, 2008).

iv) Chemical stabilization: According to Dahunsi (2008), chemical stabilization method involves the addition of chemicals to soil in order to improve the quality of the soil. Arora (2005), also opined that in chemical stabilization, soils are stabilized by adding different chemicals and the main advantage of chemical stabilization is that setting time and curing time can be controlled. Arora, (2005) posited that chemical stabilization is however generally more expensive than other types of stabilization.

v) Thermal stabilization: This is carried out by heating the soil, in the case of clay soil, heating enhances reduction of plasticity of the soil and thus improves the quality of the soil (Dahunsi, 2008). Arora, (2005) said thermal change causes a marked improvement in the properties of the soil and that thermal stabilization is done either by heating the soil or by

cooling it. Subsequent to the thermal stabilization of clay soil, there is reduction in liquid limit and plastic limit. If these fall below 45% and 20% respectively, the lay soil could then be stabilized with Portland cement (Dahunsi, 2008).

For thermal stabilization, the temperature used must be greater than 500⁰C for heating thermal stabilization (Dahunsi, 2008). Arora, (2005) stated in his own opinion that when soil is heated, to temperature of 400⁰C to 600⁰C, some irreversible changes occur which make the soil non-plastic and non-expansive, the clay clods (lumps) are assert that with further increase in temperature, there is some fusion and vitrification, and can be used as an artificial aggregate for mechanical stabilization. This method of stabilization is quite expensive because of large heat input and it is rarely used in practice (Arora, 2005). Freezing of thermal stabilization is done by cooling which causes small losses of strength of clayey soils due to an increase in inter- particle repulsion. However, if the temperature is reduced to the freezing point, the pore water freezes and the soil is stabilized thus ice so formed acts as cementing agent (Arora, 2005).

iv) Electrical Stabilization: electrical stabilization of clayey soils is done by a process known as electro-osmosis. As a direct current (D.C) is passed through a clayey soil ad pore water migrates to the negative electrode (cathode). The stabilization occurs because of the attraction of positive ions (cat ions) that are present in water towards cathodes (Arora, 2005). The strength of the soil is considerably increased due to removal of water (Arora, 2005).

2.5.3 Rules Governing the Use of Stabilizers

Some general rules governing the use of stabilizer according to Mukerji and Strulz (1988) are as follow:

- a) The full benefit of using a stabilizer is achieved only if it makes contact with each particles of soil, hence thorough mixing is necessary.
- b) Much preparation and testing is required to find the best combination and proportion of stabilizers for a given soil. It is certainly worth the time and effort, even if it takes one or two month of preparation.
- c) The only way to determine the correct proportion of stabilizers is to make 5 to 7 trial blocks from each mix and subject them to a series of tests, such as compression strength tests after different periods of drying, prolonged wetting and drying tests, and immersion in water.
- d) Portland cement and lime stabilized blocks need to be moist cured for at least 7days to gain strength.
- e) Testing programs should take into account, the local climatic condition, the possible occurrence of frost, and the like. The choice of stabilizer will also differ between arid and humid regions.
- f) It should be remembered that trial blocks need only a small amount of soil which is easy to mix. During the actual construction or mass block production, the mixing of large quantities of the soil is more difficult, so that a slightly higher proportion of stabilizer should be added (except in the case of cement)
- g) The aim of the tests should always be to find the lowest amount of stabilizer to satisfy the requirement and very often the specified requirements are unjustifiably high leading to unnecessarily high costs.

2.6

Bricks

The word “brick” is used to describe a small block of burned clay of such size that can be conveniently held in one hand and it is slightly longer than twice its width (Stephen and Christopher, 2005). A brick is a block of ceramic material used in masonry construction, usually laid using mortar (Wikipedia Encyclopedia, 2009). Bricks are usually made from clay (BS 3921) or from sand and lime (BS 187) and are available in a wide variety of strengths, types, textures, colors and special shapes to BS 429 (Chudly and Greeno, 2005). Bricks were often used, even in areas where stone was available for reasons of speed and economy. It may be made from clay, shale, soft slate, calcium silicate concrete, or shaped from quarried stone (Wikipedia Encyclopedia, 2009).

The oldest shaped bricks found dated back 7,500 B.C and they have been found in ‘Cayonu’ in the upper ‘Tigris’ region and south east Anatolia close to Diyarbakar (Wikipedia Encyclopedia, 2009). Other more recent findings dated between 7,000 and 6,395 B.C, come from ‘Jericho’ and ‘Catal Huyuk’. From archeological evidence, invention of fired bricks (as opposed to the considerably earlier sun-dried mud brick) is believed to have arisen in about the third millennium BC in Middle East. Being much more resistance to cold and moist weather conditions, bricks enabled the construction of permanent buildings in regions where the harsh climate precluded the use of mud bricks (Wikipedia Encyclopedia, 2009). Bricks have the added warmth benefit of storing heat energy from the sun during the day and continuing the release heat after sunset (Wikipedia Encyclopedia, 2009).

The trend of building upwards for offices that emerged towards the end of 19th century displaced brick in favor of cast and wrought iron and later steel and concrete (Wikipedia

Encyclopedia, 2009). Some early 'Skyscrapers' were made in masonry and demonstrated the limitations of the material, for example, the 'Monadnock Building' in Chicago (opened in 1896) is masonry and just seventeen stories high, the ground walls are almost 6feet (1.8m) thick, clearly building any higher would lead to excessive loss of internal floor space on lower floors (Wikipedia Encyclopedia, 2009). Brick was revived for high structure in the 1950s following work by Swiss Federal Institute of Technology and the Building Research Establishment in Watford, UK (Wikipedia Encyclopedia, 2009).

The great majority of bricks in use today are made from clay, although bricks can also be made from sand and lime or concrete. In the UK the standard brick size is 215 x 102.5 x 65mm (Stephen and Christopher, 2005). Bricks may be manufactured to other shapes and sizes and are usually known as 'special' and can be of three types such as clay brick, calcium silicate brick (sand-lime) and concrete bricks (Stephen and Christopher, 2005).

CHAPTER THREE

MATERIALS AND METHODS

3.1

Materials

All materials used for this research were obtained within the Sokoto State. The laboratory works were carried out in three laboratories. The initial particle size distribution of two different red clay soil samples were carried out at Department of Soil Science, Faculty of Agriculture, Usman Danfodiyo University, Sokoto. This was done in order to confirm the red clay soil sample that has the highest silt content. The liquid limit, particles size distribution, plastic limit, optimum moisture content (OMS) and the compressive strength tests were carried out at Road Nigeria Plc central laboratory besides Sokoto Cement, Kalambaina road, Sokoto. Batching, moulding and curing of the bricks took place at the laboratory of the Department of Building Technology, the Polytechnic of Sokoto State, Sokoto. All the tests were carried out with the relevant British Standard Code of Practice (BS).

The materials used for the research work are red clay soil, lime and water. The nature and quality of these materials are as follows:

3.1.1 Red Clay Soil

The red clay soil sample was obtained from a borrow pit at “AMANAWA” in Dange Shuni local government area of Sokoto state, Nigeria, using method of disturbed sampling. The clay soil was crushed manually and size graded by sieve of 4.00mm to remove leave and other unwanted materials from the clay soil.

3.1.2 Lime

High quality hydrated lime manufactured by a member of the Aglo American Plc group was used. The name of the product is “LIMBUX” manufactured in Great Britain. The lime was obtained from Sokoto State Water Board, Sokoto.

3.1.3 Water

Tap water fit for drinking that has been chemically treated from Sokoto state water board was used for the research.

3.2 Study Area

Sokoto State is a balkanized entity of what used to be part of the extensive Sokoto Caliphate founded since 1804. The then Sokoto Caliphate was founded by a group of committed men led by the universally acknowledged Islamic scholar Shehu Usmanu Danfodio, who sought to establish a political system based on the principles of universal justice. The Caliphate flourished until the British conquered it in 1903. In 1976, when more states were created, Northwest State was split in to Niger and Sokoto State (Sokoto State Online, 2013).

The state is located on the extreme northwestern part of Nigeria between longitude $4^{\circ}8'E$ and $6^{\circ}54'E$ and latitude $12^{\circ}N$ and $13^{\circ}58'N$. It shares common borders with Niger Republic to the north, Kebbi state to the Southwest and Zamfara state to the east. The total land area is about 32,000 Sq. km, while the total population in 1999 (projected from the 1991 census) is about 2,809, 168. The state, like every other state in federation, has three senatorial districts, namely Sokoto East, Sokoto North and Sokoto South (Sokoto State Online, 2013).

Amanawa, where the Red Clay Soils were obtained falls in Sokoto South senatorial district, under Dange Shuni Local Government Area.

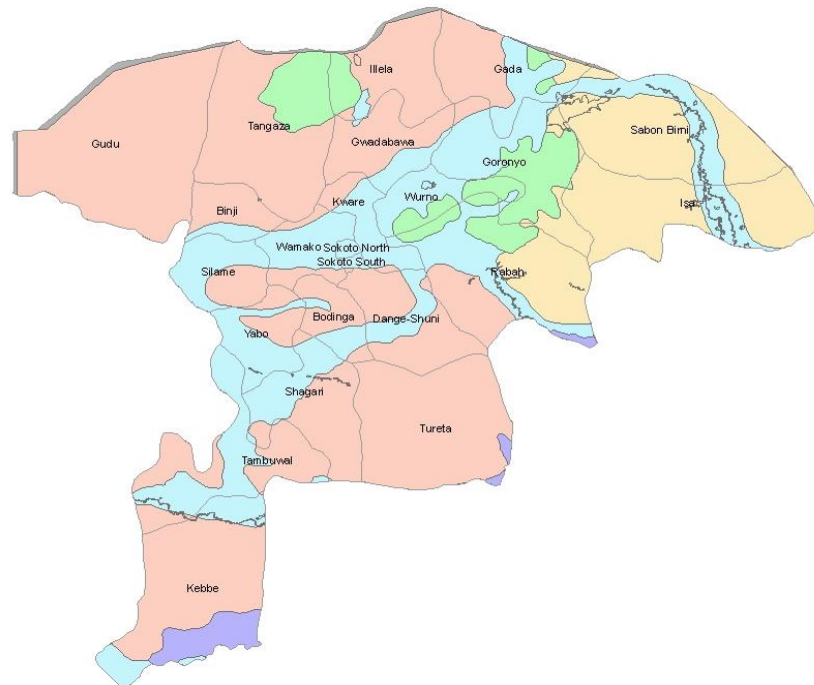


Plate 1: Map of Sokoto State Showing Dange Shuni L.G

3.3 Description of Research Methodology

Hydrometer method of Soil analysis was initially conducted on two different Red Clay Soil (RCS) within the same area so as to confirm the RCS with the highest percentage of clay and silt content. After the analysis, field tests and laboratory tests such as; particle size distribution, liquid limit, plastic limit, optimum moisture content of both RCS and Stabilized RCS were carried out. Bricks were produced at 0%, 3%, 6%, 9%, 12%, 15%, 18% and 21% stabilization. After the stipulated curing period, the brick samples were then tested for compressive strength.

3.4 Physical Properties of the Red Clay Soil Sample.

Red clay soil sample was obtained from two different portions within the same locality of the study area and thereafter soil analysis was conducted.

3.4.1 Hydrometer Method of Soil Mechanism Analysis.

The quality of the sample was determined through soil mechanism analysis conducted on the samples. Hydrometer method of soil mechanism analysis was used to determine the percentage of sand, clay and silt in each soil sample. In hydrometer method of soil mechanism analysis, clay sample with the highest percentage of silt is considered to be the best clay soil. Therefore, after the experimental work, the clay soil sample with highest silt content was selected and used for this research work.

51g of red clay soil was measured from each sample. The samples were transferred into two different measuring cylinders. Thereafter, 5g of sodiumhexa-metaphosphate powder ((NaPO₃)₆) was measured and diluted in 100ml of water to form calgon solution. The solution was prepared twice; each solution was added to the measuring cylinder containing the soil sample. The samples were allowed to stay for initial 40seconds. The hydrometer and temperature reading of the two samples were measured respectively. After 3hours, the final hydrometer and temperature reading was taken. Based on the result of the hydrometer and temperature reading, silt was determined. The percentage of sand, clay and silt were computed using the formulae in equation 3.1, 3.2, and 3.3 respectively.

$$\% \text{ of sand} = 51 - (H_1 - 2) - 0.3 (T_1 - 2) \div 51 \times 100 \quad \dots\dots(3.1)$$

$$\% \text{ of clay} = (H_2 - 2) + 0.3 (T_2 - 20) \div 51 \times 100 \quad \dots(3.2)$$

$$\% \text{ of slit} = 100 - (\%sand + \%clay) \quad \dots\dots(3.3)$$

The first soil sample was denoted by letter A and the second soil sample was denoted by B and H and T represent hydrometer and temperature. According to the result of this experiment, the first soil sample has the highest silt content. Hence, the first soil sample was adopted for the research work.

3.4.2 Particle Size Analysis of the Soil

500g of red clay soil sample was weighed on an electrical balance. The sample was then washed with cleaned tap water so that the soil particles will be freely separated. The soil sample was washed in a sieve and therefore the method of washing is referred to as wet sieving.

After wet sieving, the sample was placed in an oven for 24 hours. When it was removed from the oven, it was allowed to cool to standard room temperature. Therefore, it was then re-weighed so as to compare the new weight with the initial weight of the sample before washing. Since the Soil sample is clay soil, the sample was sieved through the sieves, 7, 36, 52, 72, 100, 200 and passing 200 μ m of BS sieve. This was done in accordance with BS 1377-2: 1990 and BS 812-103.1(1985). However, the percentage of material passing sieve was determined.

3.4.3 Liquid Limit and Plastic Limit

The soil passing sieve 36 μ m was used for both liquid and plastic limit experiments. Certain quantity of water was added to the soil and mixed thoroughly to make it plastic. It was then covered and left for about 24 hours. The soil was later moisturized and samples were taken and placed on liquid limit apparatus. After the sample had been neatly placed on the bowl of the Casagrande Apparatus, the middle part of the sample was then cut off. Thereafter, the blowing commences and immediately the two edges met, the number of blows that lead

to the meeting of the two edges was noted and recorded. The method used to determine the liquid limitation (Atterberg's limit) in this research is referred to as "Four point Method". After the blows, the samples were packed from the bowl one after the other and put in containers. Immediately they were placed in the containers, the containers were then moved into a desiccator in order not to allow fresh air get in contact with the samples. At the end of the four experiments, the samples were transferred from desiccator to oven for 24 hours. On removal from oven, the samples were weighed and the result was recorded accordingly. The plastic limit test was also conducted without a casagrande machine. About 10g of the plastic soil mass was taken in one hands and a ball was formed. The ball was rolled with fingers on a glass plate to form a soil thread of 10mm form diameter. The diameter of the thread like soil was kept at about 3mm without crack formation. The thread-like soil were later sampled in two containers and weighed on an electrical balance. The containers were then transferred into an oven after the weight has been noted. They were left for 24 hours and re-weighed. The plastic limit of the soil sample was determined from the average samples in the two containers.

3.5 Moisture Content Dry Density Relationship for the Red Clay Soil Sample

3000g of the soil sample were weighed each in five different places. 10%, 12%, 14%, 16% and 18% of water were added to each soil samples. Immediately, percentage of water is added to the soil sample, it was then mixed thoroughly on a tray before transferred to the cylindrical container.

Each sample was compacted inside the cylindrical container in three layers. Each layer was compacted twenty seven times. After the third compaction, the weight of the compacted wet sample was determined on 20kg Ohaus balance. And the wet density of the wet sample

was also determined. The proctor test method of soil compaction was adopted in accordance with BS 1377 (Part 4) [2] (1990).

To determine the moisture content of the soil sample, the soil was sampled twice from each compacted soil sample. These were carried out so as to get average moisture content of each soil sample compacted in the cylinder. The samples were then weighed before being transferred into an oven. After 24 hours, they were removed from the oven and weighed to know the weight of the soil samples at the dried stage. Therefore, the optimum moisture content (OMC) of the red clay soil sample was determined in accordance with BS ordinary for soil.

Optimum Moisture Content (OMC) was also determined for lime-stabilized red clay soil, at 3%, 6% and 9%. Since the optimum moisture content of the red clay soil was determined to be 14.6%. Therefore, the percentage of water added to the stabilized soil samples ranged from 14% to 18% (i.e. 14%, 16% and 18%). The procedure used in determining the OMC of the stabilized red clay soil is the same as that of red clay soil sample discussed above. At 6% and 9% stabilization, it was observed that the percentage of the OMC was the same. Hence, the percentage of OMC used for all the stabilized red clay soil in this research was based on that of 6% and 9% stabilizations.

3.6 Test Sample Production

According to Vazquez et al, (2005), for soil stabilization in Mexico, 3% to 6% by weight of lime is commonly added to the soil and the percentage of lime varies according to the strength required and the original characteristics of the soil. Mukerji and Strulz (1988) also confirmed that the required amount of lime to be stabilized clay can arrange between 3% and 14% by dry weight, depending largely on clay content. Based on the fact above, the

percentage of lime that was applied as stabilizer by weight for this research were 0%, 3%, 6%, 9%, 12%, 15%, 18% and 21% respectively.

The brick samples were moist cured for 3, 7, 14, 21 and 28 days respectively. Three samples of brick were tested at each curing period to obtain the average compressive strength of the samples. The standard size of brick according to BS 3921 (1985) was adopted.

The test samples were produced using hydraform interlocking machine. The machine is specially designed for brick making. The length of the mould was designed in accordance with the BS 3921 (1985). The dimension of the mould of the machine is 225mm x 112mm. The thickness of mould is adjustable. Since BS 3921 (1985) provides the standard thickness of the brick to be 75mm, therefore, the mould was adjusted to 75mm thickness.

After batching and proper mixing of the red clay soil with lime, some quantities of water were added based on the optimum moisture content (OMC) determined.

3.6.1 Identification of Samples

The brick samples were identified based on the percentage of stabilization and hydration period. A, B, C, D, E, F, G, H, represent percentages of stabilization and 3, 7, 14, 21 and 28days represent hydration period. Table 3.1 depicts the samples' identification as used in the research.

Table 3.1: Samples Identification

Batch	Percentage of Stabilization and Hydration Periods.				
	Samples with 0%				
1	A3	A7	A14	A21	A28
	A3	A7	A14	A21	A28
	A3	A7	A14	A21	A28
	Samples with 3%				
2	B3	B7	B14	B21	B28
	B3	B7	B14	B21	B28
	B3	B7	B14	B21	B28
	Samples with 6%				
3	C3	C7	C14	C21	C28
	C3	C7	C14	C21	C28
	C3	C7	C14	C21	C28
	Samples with 9%				
4	D3	D7	D14	D21	D28
	D3	D7	D14	D21	D28
	D3	D7	D14	D21	D28
	Samples with 12%				
5	E3	E7	E14	E21	E28
	E3	E7	E14	E21	E28
	E3	E7	E14	E21	E28
	Samples with 15%				
6	F3	F7	F14	F21	F28
	F3	F7	F14	F21	F28
	F3	F7	F14	F21	F28
	Samples with 18%				
7	G3	G7	G14	G21	G28
	G3	G7	G14	G21	G28
	G3	G7	G14	G21	G28
	Samples with 21%				
8	H3	H7	H14	H21	H28
	H3	H7	H14	H21	H28
	H3	H7	H14	H21	H28

3.6.2 Batching, Mixing and Moulding

The quantities of materials were determined by using absolute weight method (i.e. Batching by weight). The red clay soil, lime and water were batched by weight. And the

quantities of red clay soil used to produce 15 brick samples at 0% stabilization was noted and used as the basis for other stabilized red clay soil materials.

Mixing of the materials was done manually by the used of shovel and hand trowel. The two materials (red clay soil and lime) were properly mixed before measured quantity of water was added.

After water had been added, the material was then re-mixed thoroughly to ensure that the materials are properly mixed with water. Henceforth, it was the packed and filled in a container close to mould before it finally transferred into the mould gently as shown in plate 12 below. When the material is finally transferred into the mould it was compacted by the upper hydraulic ram with flat-like surface. After compaction the brick sample is formed and released out from the mould.

A total of 120 bricks were produced for the research. This was done within two days.

3.6.3 Curing of the Samples

The samples were allowed for 24hours after moulding before curing commenced. Moist curing method was adopted as described by Oti, Kinuthia and Bai (2009). All the stabilized bricks were moist cured for 3, 7, 14, 21 and 28days respectively at the room temperature of about 20⁰C before testing for compressive strength in accordance with BS 1924-2(1990) and BS EN 771-1(2003).

3.7 Compressive Strength Test

Compressive strength test is the only destructive method of testing used in this study. A total of 120 bricks of 225mm x 112.5mm x 75mm were tested for compressive strength. The crushing was carried out at 3, 7, 14, 21 and 28days, using hydraulic crushing machine

and the failure load was divided by the cross-sectional area of the sample to obtain the strength of the brick.

CHAPTER FOUR

DATA PRESENTATION AND DISCUSSIONS

4.1 Preliminary Analysis of the Soil

This chapter presents the result of soil test, physical properties and mechanical properties tests. The results were based on the tests carried out on the soil and brick samples. The analysis and discussion of the results were also presented in this chapter.

Table 4.1(a): Soil analysis of two different red clay soils

After 40seconds		
Sample	Hydrometer reading, H_1	Temperature reading ($^{\circ}C$) T_1
A	33	30
B	28	30
After 3hours		
	H_2	T_2
A	12	32
B	10	32

Table 4.1(b): Percentage of Sand, Clay and Silt

Sample	% Sand	% Clay	% Silt
A	33.3	26.7	40.0
B	43.1	22.7	34.2

Table 4.1 presented the results of preliminary soil analysis conducted on two different red soil samples. The hydrometer and temperature reading at 40seconds and 3hours were presented in table 4.1 (a). At 40seconds, the hydrometer reading of sample A was more than that of sample B, while the temperature reading of the two samples was the same. After 3hours, the hydrometer reading for the two soil samples reduced. But sample A, still has the highest value than sample B.

Table 4.1(b) shows the result of the percentage of sand, clay and silt determined based on the results obtained from table 4.1(a).

The formulae in equations 3.1, 3.2 and 3.3 were used to determine the percentage of sand, clay and silt respectively. The result is presented in Table 4.1(b). Soil sample A, has the lowest percentage of sand, highest percentage of clay and highest percentage of silt. The sample B has the highest percentage of sand, lowest percentage of clay and lowest percentage of silt.

Table 4.2: Particle Size Analysis of the Red Clay Soil Sample

BS Sieve Size		Weight retained	% retained	Total passing	Remark
18 INCH	3.35mm			100.0	
No. 7	2.36mm	12.7	2.54	97.46	
No. 14	1.18mm				
No.25	600µm				
No.36	425 µm	26.9	5.38	92.08	
No.52	300 µm	11.7	2.34	89.74	
No.72	212 µm	32.8	6.56	83.18	64%
No.100	150 µm	7.1	1.42	81.72	
No.200	75 µm	87.2	17.44	64.32	
Passing 200	23.4+298.2	321.6	64.32		

4.2 Result of Particle Size Distribution

The weight of soil retained in each sieve was determined and the percentage by weight passing each sieve was noted and presented in Table 4.2. However, at sieve 200µm, it was observed that the sieve retained highest percentage of soil sample followed by sieve 72µm. Thus, at sieve 200, 64.32% of the clay soil sample passing the sieve. Hence, 64% of the soil sample passed the sieve 200.

4.3 Determination of Plasticity Index

As noted by Craig (2005) plasticity is an important characteristic in the case of fine soils, the term plasticity describing the ability of a soil to undergo unrecoverable deformation without cracking or crumbling. From Appendix 1, L_L denoted liquid limit and P_L denoted Plastic limit. It shows from Appendix 1 that the number of blows required to close the bottom groove increases from plastic soil sample 1 to plastic soil, sample 4, and the percentage of moisture content (m%) decreases. It is however showed that the percentage of moisture content decreases as the number of blows required increases. The percentage

of moisture content was determined by dividing weight of moisture by weight of dried soil multiply by 100.

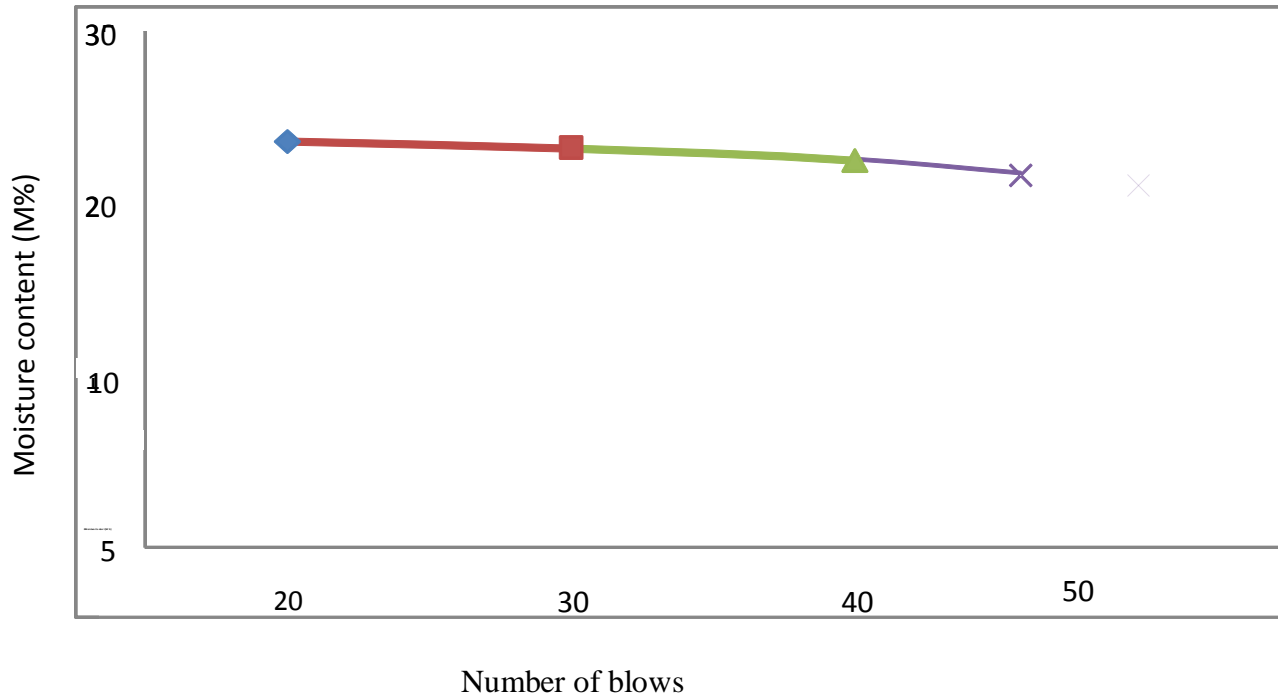


Figure 4.1 : Four points method of liquid limit.

The liquid limit of the soil was derived in Figure 4.1. Craig (2005) asserted that the liquid limit is defined as the water content at which 25 blows are required to close the bottom of the groove over a distance of 13mm. Therefore from Figure 4.1, 25 numbers of blows corresponded with 28 moisture content. Hence, the liquid limit of the soil sample is determined to be 28. Craig (2005) further established the fact that the upper and lower limit of the range of water content over which the soil exhibited plastic behavior is defined as the liquid limit (LL) and the plastic limit (PL), and the water content range itself is defined as the plasticity Index (PI), $PI = LL - PL$. Vazquez et al (2005) posited that when the plasticity index is at least 10, according to Atterberg limits; the soil can be stabilized with

lime. The result from Appendix 1 and Figure 4.1 indicated that the plasticity index is 10.4. Since the plasticity index of the clay soil sample is in conformity with Vazquez et al (2005), therefore, the red clay soil can be stabilized with lime.

4.4 Optimum Moisture Content (OMC) of the Soil and stabilized Soil

With reference to Appendix 2, it was shown that the weight of compacted wet soil increases as the percentage of water added increases from 10% to 14%. But at 16% and 18% water added, the weight of compacted wet soil decrease. Consequently, the wet density of the compacted soil decreases at 16% and 18%. Sehgal (2002) and Craig (2005) reported that compaction of a soil is the process of increasing density of a soil by packing the particles closer together with a reduction in the volume of air. This shows that the result of wet density presented in Appendix 2 is in conformity with Sehgal (2002) and Craig (2005) stated above from 10% to 14% of water added. Moreover, the average moisture content of the soil sample increases from 10% to 18% of water added. But the dry densities of the soil sample started reducing from 16% of water added.

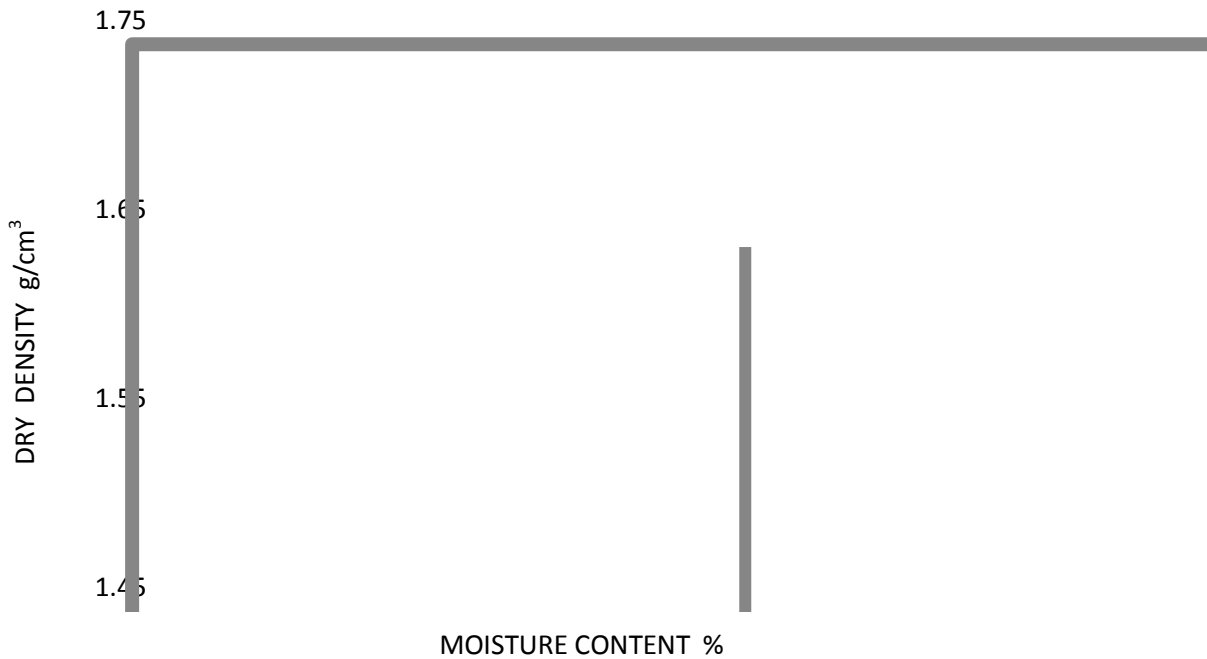


Figure 4.2: Optimum Moisture Content of the Soil at 0% stabilization

The relationship between dry density and moisture content of the soil sample is depicted in Figure 4.2, the optimum moisture content (OMC) and maximum dry density of the red clay soil sample was derived to be 14.6% and 1.69% respectively. The value derived for optimum moisture content of the red clay sample agreed with Arora (2005) who noted that optimum water content of clay soil ranges from 14% to 20%. He also explained that the water content corresponding to maximum dry density is known as the optimum moisture content (OMC).

The results of the moisture content and dry density at 3%, 6% and 9% stabilization are respectively presented in Appendix 3, 4 and 5. The percentage of water added starts from 14%. This was based on the OMC realized for the red clay soil sample. However, the

percentages of water added to the stabilized red clay soil sample are 14%, 16%, 18% respectively.

At 3% of lime stabilization, the wet density of the stabilized red clay soil increases from 14% to 18% of water added. Also, the weight of wet compacted soil increases as indicated in Appendix 3. As it was presented in Appendix 3, the average percentage of moisture content increased a little compared to percentage of water added. And the dry density reduced at 18% of water added. This implies that the dry density of the stabilized red clay soil started reducing at a point when the optimum moisture content is attained. From the figure 4.3, the optimum moisture content (OMC) and the maximum dry density at 3% of stabilization are 16.4% and 1.71g/cm^3 respectively.

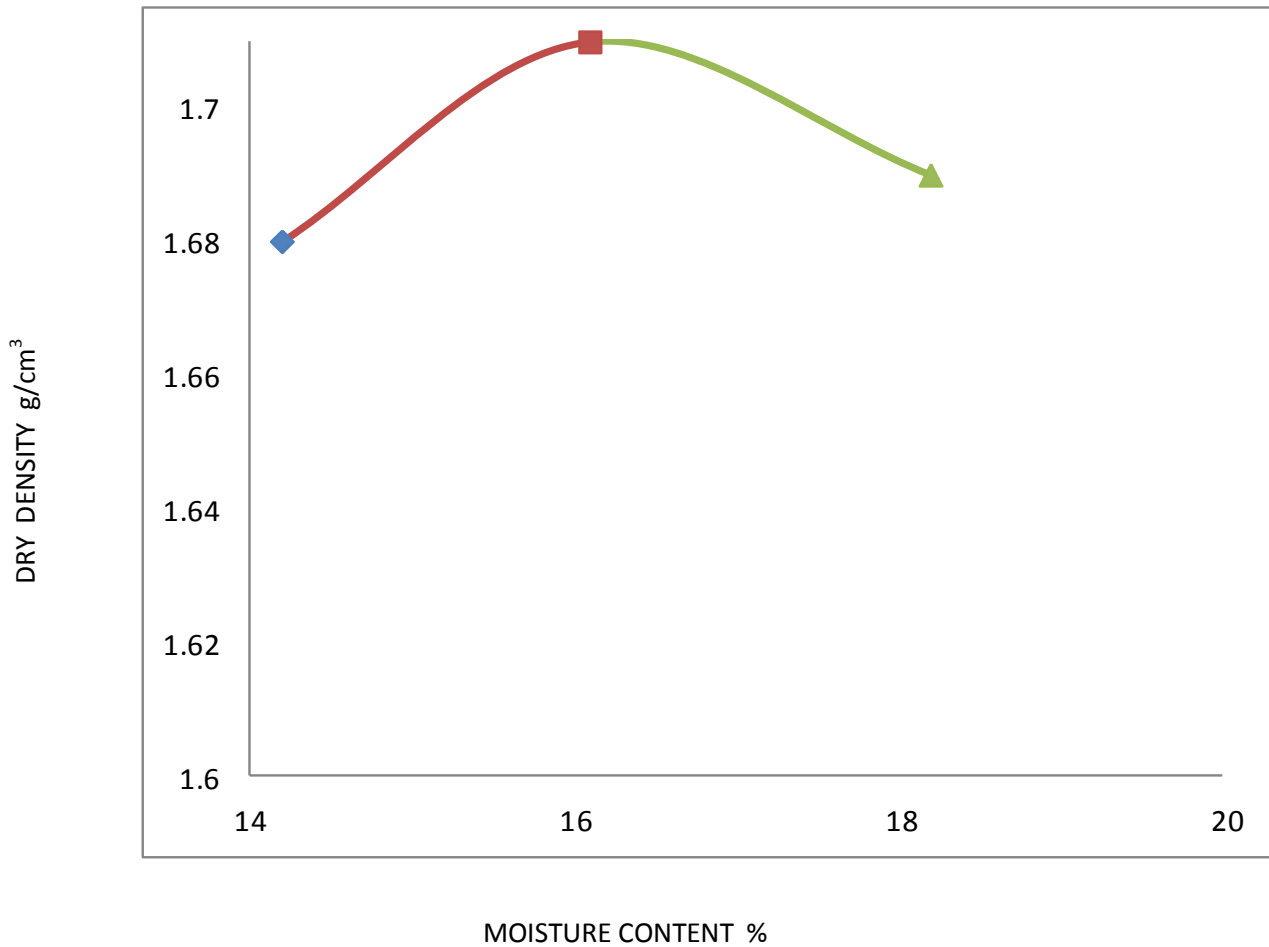


Figure 4.3: Optimum Moisture Content of the Soil at 3% stabilization

It was however observed that at 6% and 9% of stabilization, the optimum moisture content was determined to be the same as shown in Figure 4.4 and 4.5 and the value of the OMC derived is 16.8%. Therefore, since the OMC at 6% and 9% stabilization are the same and the OMC at 3% stabilization is close to that of 6% and 9% stabilization. Hence, the OMC of 6% and 9% stabilization was adopted for all the stabilized red clay soil used for the production of bricks in this study. This was ensure so as to comply with Sehgal (2002) who explained that addition of water beyond the optimum moisture content, reduces the dry

density because the extra water starts occupying the space which the soil could have occupied.

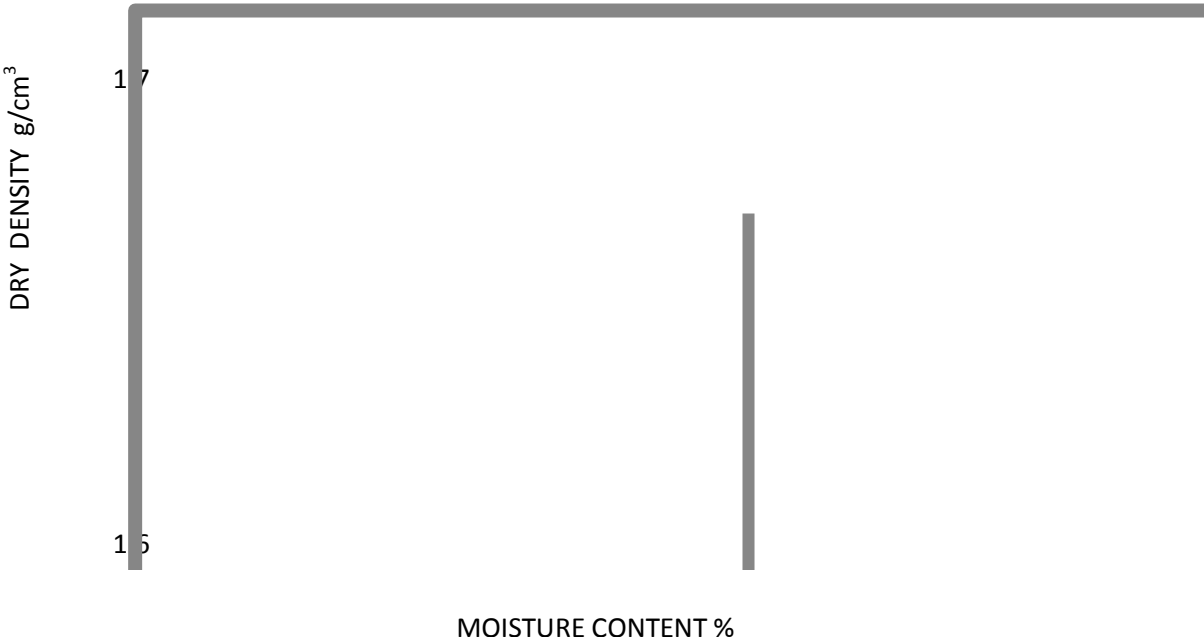


Figure 4.4: Optimum Moisture Content of the Soil at 6% stabilization

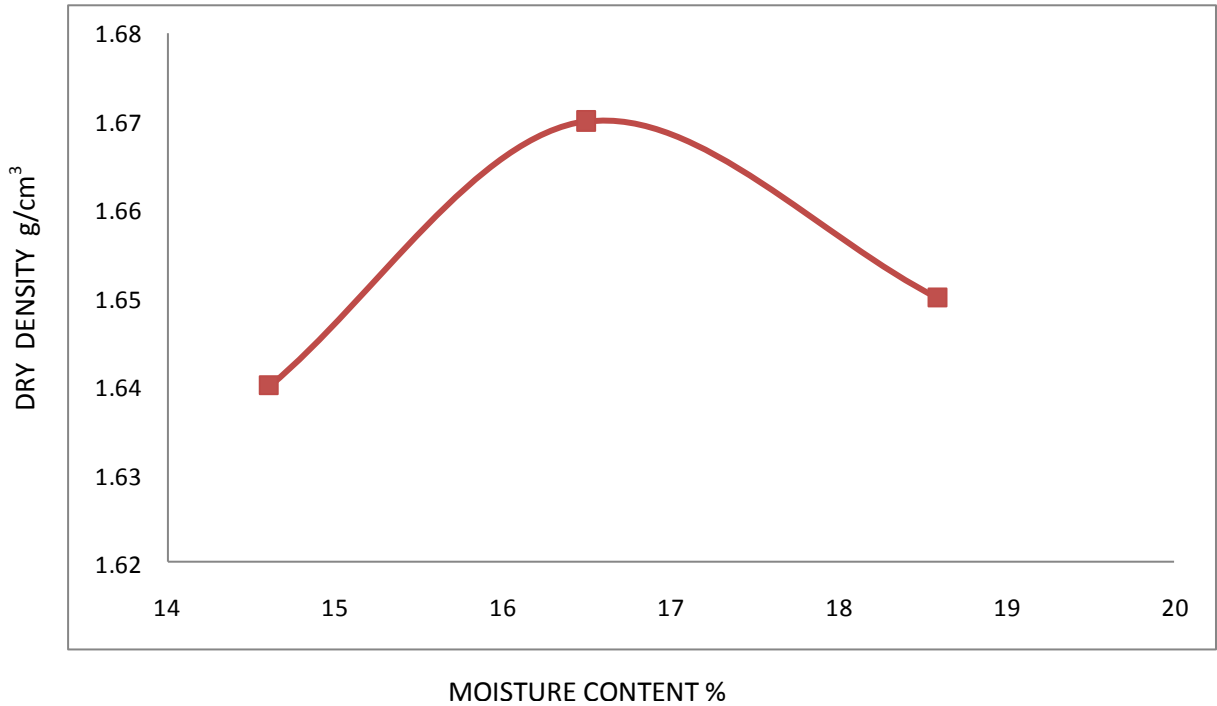


Figure 4.5: Optimum Moisture Content of the Soil at 9% stabilization

Table 4.3: Compressive Strength Tests of Bricks after Curing

Table 4.3.1: After 3 day of curing

S/No	Percentage	Average Weight (g)	Average Density (g/cm ³)	Average Load (KN)	Average Compressive strength
1	0	5923	1.62	18	0.71
2	3	6434	1.75	28	1.11
3	6	6229	1.70	21	0.82
4	9	6172	1.68	21	0.88
5	12	6218	1.70	21	0.82
6	15	6099	1.66	23	0.92
7	18	5960	1.62	16	0.63
8	21	6232	1.70	15	0.59

Table 4.3.2: After 7 days of curing

S/No	Percentage	Average Weight (g)	Average Density (g/cm ³)	Average Load (KN)	Average Compressive strength
1	0	6739	1.83	21	0.83
2	3	6830	1.86	31	1.22
3	6	6229	1.85	29	1.16
4	9	6792	1.70	20	0.96
5	12	6255	1.68	24	0.94
6	15	5962	1.62	20	0.82
7	18	6190	1.69	15	0.60
8	21	6160	1.68	13	0.50

Table 4.3.3: After 14 days of curing

S/No	Percentage	Average Weight (g)	Average Density (g/cm ³)	Average Load (KN)	Average Compressive strength
1	0	5831	1.59	25	0.99
2	3	6708	1.83	32	1.27
3	6	6571	1.79	28	1.18
4	9	6414	1.75	21	0.93
5	12	6157	1.68	23	0.91
6	15	6065	1.65	24	0.94
7	18	6289	1.71	15	0.59
8	21	6098	1.66	14	0.54

Table 4.3.4: After 21 days of curing

S/No	Percentage	Average Weight (g)	Average Density (g/cm ³)	Average Load (KN)	Average Compressive strength (N/mm ²)
1	0	6269	1.71	28	1.11
2	3	7029	1.91	33	1.30
3	6	6594	1.80	31	1.21
4	9	6165	1.68	23	0.98
5	12	6185	1.68	25	0.98
6	15	5880	1.60	20	0.79
7	18	6345	1.73	11	0.42
8	21	6003	1.63	7	0.29

Table 4.3.5: After 28 days of curing

S/No	Percentage	Average Weight (g)	Average Density (g/cm ³)	Average Load (KN)	Average Compressive strength
1	0	6514	1.78	13	0.50
2	3	6959	1.90	34	1.35
3	6	6476	1.76	31	1.24
4	9	6139	1.68	16	0.85
5	12	6041	1.65	21	0.82
6	15	5966	1.63	16	0.63
7	18	5957	1.62	12	0.47
8	21	5820	1.59	9	0.37

4.5 Compressive Strength Test of Bricks

Table 4.3 presents the average compressive strength of bricks made with Sokoto red clay soil at different percentages of stabilization. Moreover, it was observed that the average compressive strength of brick samples at 0% stabilization increases from 3days up till 21days hydration period. At 21days hydration period, the brick samples attained the highest average compressive strength of 1.11N/mm². But the average compressive strength

of bricks at 0% stabilization in 28days of moist curing reduced drastically to 0.50N/mm². This might be as a result of cracks showed on the surface of the bricks at 28days which may be attributed to lack of stabilization.

The results presented in Table 4.3.1 to 4.3.5 also indicates that at 3% stabilization, average compressive strength of the bricks is higher than other percentages used in this research. After 28day of moist curing, 3% stabilization has the highest average compressive strength of 1.35N/mm², followed by 6% stabilization with 1.24N/mm². However, it was showed from Table 4.3.1 to 4.3.5 that at 21% stabilization, the lowest average compressive strength was recorded. Table 4.3.1 presented the highest value of the average compressive strength at 21% stabilization in 3days. Figure 4.6, 4.7 and 4.8 described the compressive strength of bricks at different stabilization and different hydration periods.

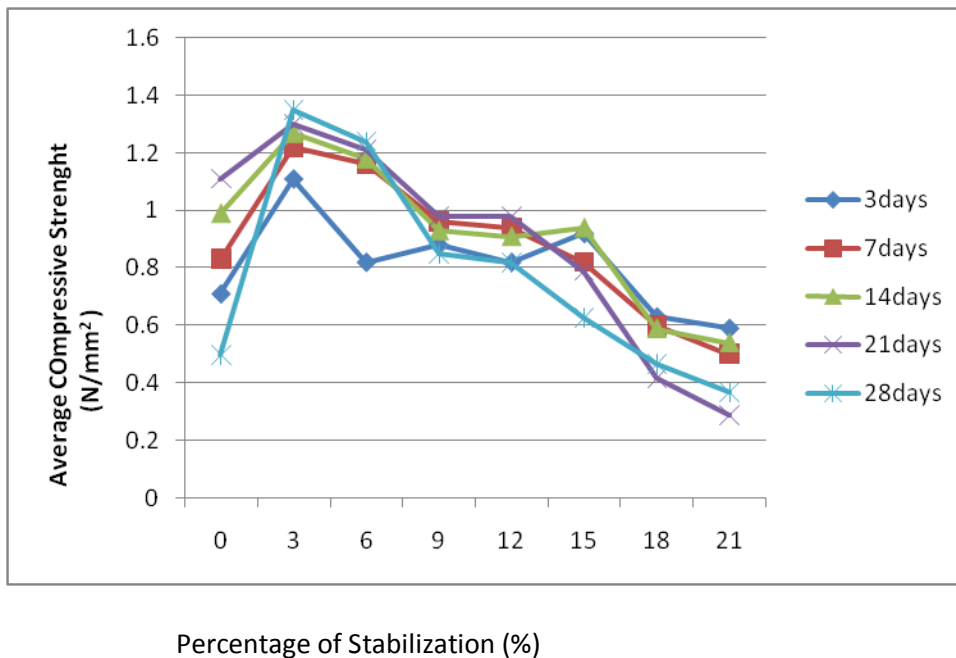
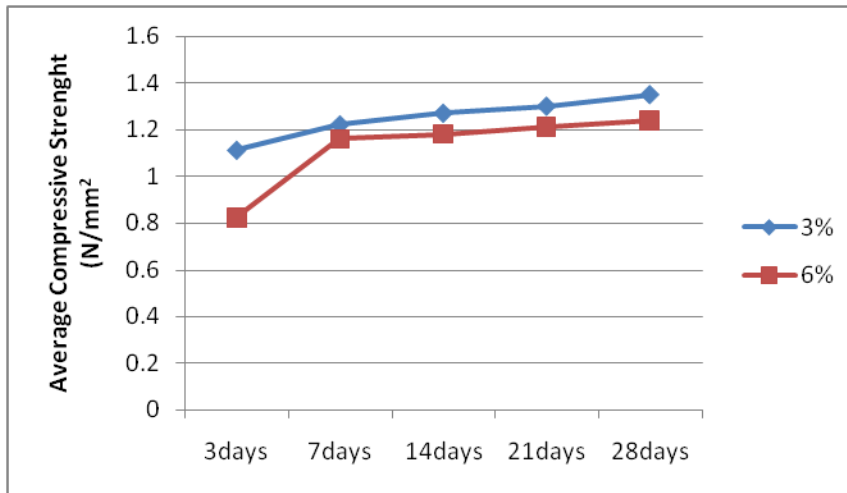


Figure 4.6: Compressive Strength of Bricks at different Stabilization

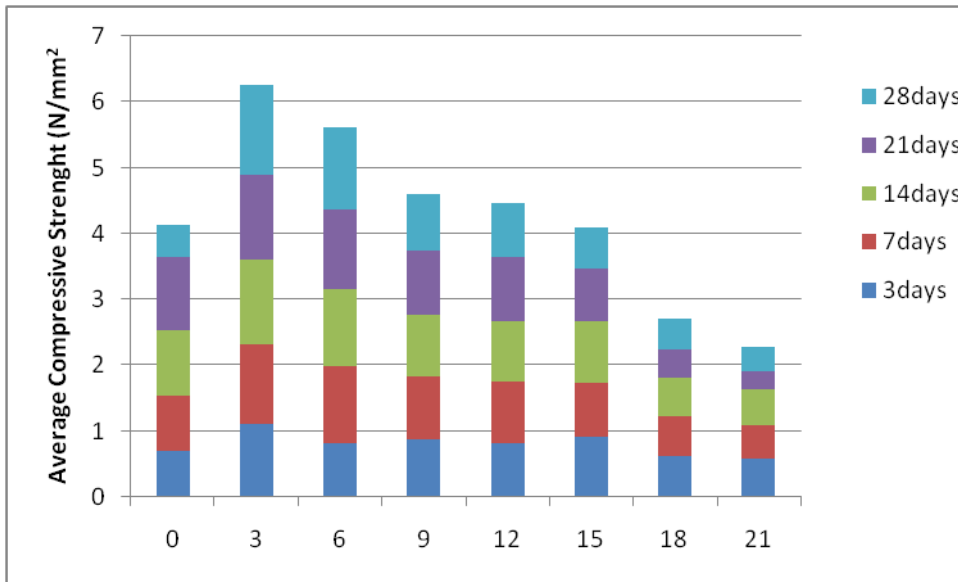


Hydration Period (days)

Figure 4.7: Variation in Compressive Strength at 3% and 6% Stabilization

BS 5628 part 1 (1978) specified that minimum compressive strength for masonry (bricks) is 2.8N/mm^2 . The highest average compressive strength derived in this research is 1.35N/mm^2 at 3% stabilization in 28days. This does not conform to the British standard. Moreover, Makunza (2006) established the fact that the plasticity index of red clay soil (i.e. 10 above) has influence on the compressive strength of the stabilized red clay bricks. He further explained that the higher the plasticity index, the higher the compressive strength of the stabilized red clay bricks.

Therefore, low results of compressive strengths derived in this research may be attributed to low plasticity index of the soil sample (i.e. 10.4) and the stabilizer used.



Percentage of Stabilization (%)
 Figure 4.8: Effect of Hydration on Compressive Strength of Stabilized Bricks

CHAPTER FIVE

SUMMARY, CONCLUSION AND RECOMMENDATIONS

5.1 Summary of the Findings

Summary of findings of this study are as follows:

- i) The total percentage of soil passing the sieve 200 during particle size analysis is 64%. This gives a satisfactory total percentage passing.
- ii) The plasticity index of the “Sokoto” red clay soil used for this research was determined to be 10.4. This is slightly greater than 10 as specified by Atterberg limits.
- iii) Optimum moisture content (OMC) of the red clay soil sample was determined to be 14.6% and that of stabilized red clay soil was also determined to be 16.8%.
- iv) At 0% stabilization, the highest average compressive strength value was derived to be 1.11 N/mm² in 21 days hydration period. However, for the stabilized red clay bricks the highest compressive strength was recorded to be 1.35N/mm² at 3% stabilization at 28 days. Thus, the result is not up to the value stipulated in British standard and this may be attributed to low value of plasticity index recorded.

5.2 Conclusion

This research shows that lime alone can not be used to stabilize red clay soil for brick production. The highest compressive strength recorded in this research is 1.35N/mm² 28 days of hydration. However, the value is not up to the value stipulated in British standard. Although, lime-stabilization has effect on red bricks produced in this study. At 0% stabilization, the highest compressive strength was derived in 21days with value 1.11N/mm². However, 1.30N/mm² was also obtained at 3% Stabilization in the same

21days. The value derived at 0% stabilization in 21days is the same to the value derived at 3% stabilization in 3days as a result of the effect of lime-stabilization. It was also observed that stabilization of bricks is environmental friendly compared to burnt brick. The reason is, it does not involve deforestation and pollution.

5.3 Recommendations

5.3.1 Recommendations Based on Findings

Based on the results of this research, the following are recommended:

- i) It is recommended that lime should be used together with one or more stabilizer to stabilize Red Clay Soil before using for brick making.
- ii) Red Clay Soil with higher plasticity index should be adopted for stabilization because the value of plasticity index influences the compressive strength of bricks.
- iii) Since there is an increase in compressive strength of bricks from 3days to 28days at 3% and 6% Stabilization. It is suggested that hydration period of lime-stabilized bricks should be allowed for more than 28days.
- iv) In area where different types of Clay Soil are available in abundance, mechanical stabilization of two different Clay Soil could be carried out so as to determine the plasticity index value of the Soil before recommended for lime Stabilization.

5.3.2 Recommendations for Further Studies

- i) Further research should be carried out on how to improve the plasticity index of red clay soil if found low.
- ii) Alternative method of curing could be adopted for lime-stabilized bricks instead of moist method in further research.

- iii) Further research should also be conducted to determine the required hydration period needed to obtain appreciable compressive strength when compared to standard.

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Appendices

Appendix 1: Liquid and Plastic Limits of the Clay Soil.

<u>Test Numbers</u>	1	2	3	4	1	2
Type of Test	L _L	L _L	L _L	L _L	P _L	P _L
Container Numbers	16	35	5B	78B	94	1
Wt of wet soil+ container, w ₁ (g)	29.5	32.5	33.5	31.5	24.4	25.2
Wt of driedsoil+container, w ₂ (g)	25.0	27.4	27.9	26.8	22.0	22.8
Wt of moisture w ₁ - w ₂ (g)	4.5	5.1	5.1	4.6	2.4	2.4
Wt of container, w ₃ (g)	9.3	9.3	9.4	9.1	8.5	9.0
Wt of dried soil , w ₂ – w ₃ (g)	15.7	18.1	18.5	17.7	13.5	13.8
Moisture content, m(%)	28.7	28.2	27.6	26.0	17.8	17.4
Number of blows	19	28	37	44	Avrg	=17.6

Appendix 2: Moisture Content and Dry Density of the Red Clay Soil Sample.

Water added	10%	12%	14%	16%	18%
Wt of cylinder + wt of soil(g)	3743	3823	3882	3850	3794
Wt of cylinder(g)		208			
Wt of wet soil(g)	1662	1742	1801	1769	1713
Vol. of cylinder(cm ³)		927			
Wet density(g/cm ³),DW	1.79	1.88	1.94	1.91	1.85

Moisture Content Determinations

Container No.	171	X3	149	T	38	41	80	82	99	74
Wt of soil +tin(g)	97.3	91.3	91.7	86.0	76.5	89.9	94.7	75.7	87.1	90.0
Wt of dried soil +tin(g)	89.4	84.0	83.2	78.1	68.6	80.5	83.5	67.1	75.8	78.5
Wt of tin(g)	15.8	16.2	16.6	15.5	15.1	16.1	16.2	16.0	15.7	16.3
Wt of dry soil(g)	73.6	67.8	66.6	62.6	53.6	64.4	67.3	51.1	60.1	62.2
Wt of moisture	7.9	7.3	8.5	7.9	7.9	9.4	11.2	8.6	11.3	11.5
Moisture content, M(%)	10.7	10.8	12.8	12.6	14.8	14.6	16.6	16.8	18.8	18.5
Average (%)	10.8		12.7		14.7		16.7		18.7	
Dry density(g/cm ³) $DS=\frac{100DW}{100+M}$	1.62		1.67		1.69		1.64		1.56	

Appendix 3: Moisture Content and Dry Density at 3% of Stabilization

Water Added		14%	16%	18%		
Wt of cylinder + wt of soil (g)		3863	3922	3935		
Wt of cylinder (g)			2 0 81			
Wt of wet soil (g)		1782	1841	1854		
Volume of cylinder (cm ³)			9 2 7			
Wet Density (g/cm ³) - DW		1.92	1.99	2.00		
Moisture Content Determinations						
Container number	x3	93	14 ^E	45	6 ^W	10 ^Z
Wt of soil + tin (g)	79.6	87.0	86.2	89.0	69.0	87.9
Wt of dried soil + tin (g)	71.8	78.1	76.5	78.9	60.8	76.8
Wt of tin (g)	16.2	16.1	16.0	16.1	15.7	15.7
Wt of dry soil (g)	55.6	62.0	60.5	62.8	45.1	61.1
Wt of Moisture (g)	7.8	8.9	9.7	10.1	8.2	11.1
Moisture content M(%)	14.0	14.4	16.0	16.1	18.2	18.2
Average (%)		14 .2		16 .1		18 .2
Dry Density (g/cm ³)						
DS = $\frac{100DW}{100+M}$		1 .68		1 .71		1 .69

Appendix 4: Moisture Content and Dry Density at 6% of Stabilization

Water Added	14%		16%		18%	
Wt of cylinder + wt of soil (g)	3865	20 81	3927		3940	
Wt of cylinder (g)	1784		1846		1859	
Wt of wet soil (g)		9 2 7				
Volume of cylinder (cm ³)	1.92		1.99		2.01	
Wet Density (g/cm ³) – DW						
Moisture Content Determinations						
Container number	20 ^A	41	2 ^F	5 ^A	70 ^B	65
Wt of soil + tin (g)	67.4	77.0	81.4	90.0	87.3	85.7
Wt of dried soil + tin (g)	61.2	69.3	72.1	79.7	76.3	75.0
Wt of tin (g)	16.2	16.1	15.7	16.2	16.2	16.2
Wt of dry soil (g)	44.8	53.2	56.4	63.5	60.1	58.8
Wt of Moisture (g)	6.4	7.7	9.3	10.3	11.0	10.7
Moisture content M(%)	14.3	14.5	16.5	16.2	18.3	18.2
Average (%)	14 .4		16 .4		18 .3	
Dry Density (g/cm ³)	1 .68		1 .71		1 .70	
$D = \frac{100DW}{100+M}$						

Appendix 5: Moisture Content and Dry Density at 9% of Stabilization

Water Added	14%		16%		18%	
Wt of cylinder + wt of soil (g)	33827		3890		3901	
Wt of cylinder (g)	1746		1809		1820	
Wt of wet soil (g)		20	81			
Volume of cylinder (cm ³)	1.88		1.95		1.96	
Wet Density (g/cm ³) - DW						
Moisture Content Determinations						
Container number	28^A	19	177	17^D	71	58
Wt of soil + tin (g)	91.8	78.9	96.0	89.4	81.6	85.4
Wt of dried soil + tin (g)	82.3	70.9	84.6	79.1	71.3	74.6
Wt of tin (g)	16.2	16.1	15.8	16.0	15.8	16.1
Wt of dry soil (g)	66.1	54.8	68.8	63.1	55.5	58.5
Wt of Moisture (g)	9.5	8.0	11.4	10.3	10.3	10.8
Moisture content M(%)	14.4	14.6	16.6	16.3	18.6	18.5
Average (%)	14.6		16.6		18.6	
Dry Density (g/cm ³)	1.64		1.67		1.65	
$DS = \frac{100DW}{100+M}$						