# THE CLASSIFICATION OF LATERITIC SOILS IN IKOLE

# AS A ROAD CONSTRUCTION MATERIAL

By

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(CVE/13/1055)

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## ABSTRACT

The performance of a pavement depends on the quality of its base, sub-base layers and sub grade. As the foundation for the pavement's upper layers, the subgrade layer plays a key role in mitigating the detrimental effects of static and dynamic stresses generated by traffic. Therefore, building a stable base course is vital for constructing an effective and long lasting pavement system. This study proceeds from the basis that, construction of foundations of most engineering structures, requires that adequate information about the engineering properties of the soil and sub-soil condition of the area are established prior to the conception of works. This is necessary for the engineering planning, design and construction of such foundations to be based on sound geotechnical parameters. To achieve the objectives of the study, different soil test such as CBR test, compaction test, sieve analysis, triaxial and specific gravity tests were carried out on soil samples taken at Asin, Oke-orin, Ikoyi and FUOYE ikole campus soils. The test results aided in the classification of these soil samples using AASHTO soil classification system to determine the suitability of these lateritic soils to be used as road construction material. The soils were classified as A-2-7, A-7-6, A-2-6 and A-2-7 for Asin, Campus, Ikoyi and Oke orin samples respectively.

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My gratitude goes to Almighty God, for his unending love upon my life and for always being faithful to me in my academic rigors.

A sincere gratitude to the entire academic and non-academic staff of the department of Civil Engineering, Federal University Oye Ekiti for their unending efforts in imbibing knowledge into me and my colleagues and equipping us with the necessary tools needed to excel. I am also most grateful to my parents Mr and Mrs BADA, Mr Dennis Okorofor and my supervisor Prof. J.B Adeyeri for their guidance and care.

## DEDICATION

I dedicate this work to GOD almighty my creator, my strong pillar, my source of inspiration, the reason behind my breath... author of wisdom, knowledge, and understanding. I also dedicate this report to my parents and siblings for their support and their encouragement.

## CERTIFICATION

This is that this proposal was written by **BADA ADEOLA OLUWADUNSIN (CVE/13/1055)** under supervision and is approved for the contribution to knowledge and literary presentation. All sources of information are specifically acknowledged by means of references, in partial fulfillment for the award of Bachelor of Engineering (B. Eng) degree in civil engineering, Federal University Oye Ekiti.

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## CHAPTER ONE

## INTRODUCTION

### 1.1 Background

In many countries in Africa there is a growing realization of the cost-effectiveness of upgrading gravel roads to a sealed standard even at relatively low traffic levels, often less than 200 vehicles per day. This has challenged road authorities to make optimum use of naturally occurring materials which are often rejected by traditional specifications for use in the upper layers of road pavements. One such naturally occurring material is laterite – a type of residual soil that occurs extensively in the humid tropical and sub-tropical zones of the world, including much of central, southern and western Africa. Fortunately, research carried out in the late 1960s in a number of countries, notably in Angola, Mozambique, Brazil, Australia and Nigeria indicates that the performance of laterite has often been better than expected on the basis of traditional specifications. However, if successful use is to be made of this material, the conditions under which it can be successfully used must be carefully specified – one of the key objectives of this report.

It is noteworthy at the outset that terms such as "laterite", "lateritic soils" and "ferricretes" are often used synonymously. However, such terms convey different meanings to different practitioners in that their application ranges from strict conformity to Buchanan's original definition (Buchanan, 1807) which confines the material to a fairly small group of red soils that harden irreversibly on exposure (described by Buchanan1807 as "red clay used for air dried brick production"), to any variety of reddish, iron-rich, tropical residual soils. As a result, the confidence with which "laterite" can be used for road construction is diminished largely because the term may apply to a material with a wide range of geotechnical properties. Nonetheless, certain types of this material are eminently suitable for use in the construction of road pavements in Africa for which a commonly agreed definition is desirable so as to enable their engineering behaviour to be predicted.

1. Unfortunately, laterites have not been used to their fullest extent in the upper (base and sub base) layers of low volume paved roads (LVSRs) in the African region for a number of reasons including: The variability in their engineering properties and their failure to meet traditional specifications. For example, these materials commonly exhibit gaps in the grading curve (e.g. in the sand coarse fraction); high

plasticity indices (PIs 15-20) and soaked CBR values lower than the minimum of 80 per cent normally specified (Netterberg, 2014).

2. Lack of awareness of the more appropriate specifications that were first developed by the Portuguese in the 1950s and 1960s in countries such as Angola and Mozambique and subsequently adapted for use in other countries, notably Brazil and Australia In view of the above, the use of neat (untreated) laterites for the construction of low volume sealed roads (LVSRs) in some African countries has been limited as the road authorities continue to use much tighter, restrictive standards that greatly suppress the use of this type of material. As a result, other more expensive options are adopted such as hauling over long distances other natural gravels which meet the traditional specifications; stabilizing the laterites with cement and lime or using crushed stone for the base (Netterberg, 2014).

Fortunately, there are some relatively recent examples of the use of laterites in a number of Southern African countries, such as Angola, Botswana, Kenya, Malawi, Mozambique, Nigeria, Zambia and Zimbabwe, where this type of material has been successfully used in the upper layers of both low and high-volume roads, despite its non-compliance with traditional specifications. Despite the excellent performance of these roads, and of similar examples in other countries, the likelihood of adopting these designs in practice is limited, largely because the national standards of many countries in the region do not contain appropriate specifications for the use of laterites in road construction.

## 1.2 Statement of Problem

The Civil Engineer is faced with the practical problems raised by use of soil as a foundation and construction material. In Nigeria, the non-availability of generalized relevant data in this area, particularly for initial preliminary engineering planning and designs, has been the major cause of failure of most of highway construction projects, such that, failure occurs almost immediately after the project is commissioned or even before. The construction material (laterite), which is used for engineering highway projects, is therefore as important, as other engineering design factors. Thus in road pavement design, the soil materials used in the pavement construction transmit the axle-load to the sub-soil or sub grade. Hence, the durability of a highway pavement is a function of the ease and rigidity of the pavement soil to transmit the stress induced in it to the sub-soil such that unnecessary deformation is avoided.

### 1.3 Justification for the Study

This study is conducted so as to raise awareness of the performance-based specifications that have been developed specifically for the use of laterites in road construction in Nigeria taking Ikole Ekiti as a study area. This study's primary audience is the Nigerian Civil Engineers.

The results of this study will aid engineers and contractors maximize the use of lateritic soils in Ikole area for the construction of roads especially for use as a base and sub-base course material. Also the results of this study will provide reliable technical information on the geotechnical properties of the lateritic soils in Ikole- Ekiti. It will also provide useful guidelines for Civil Engineers in selection of materials for the construction and rehabilitation of roads in Ekiti State and Nigeria as a whole.

### 1.4 Aims and Objectives of the Study

#### 1.4.1 Aims of the Study

This study is aimed at classifying lateritic soils in Ikole as road construction materials (sub base and base course).

#### 1.4.2 Objectives of the Study

1. To test, measure and ascertain the geotechnical properties of the lateritic soils in Ikole
2. To assess the suitability of lateritic soils found in Ikole for use as road sub base and base course material.
3. To raise awareness of the existence of the performance-based specifications that have been developed specifically for the use of laterites in road construction in Nigeria

### 1.5 Scope of the Study

From the field surveys conducted, the study focuses on specific laboratory tests on the soil samples obtained from the site location(s), Ikole Area. The test results are used to assess the suitability of the lateritic soils as base and sub base material with reference to standards on various manuals.

### 1.6 Study Area

Ikole is a Local Government Area of Ekiti State, Nigeria. Its headquarters are in the town of Ikole. It has an area of 321 km² and a population of 168,436 at the 2006 census, it can be found on coordinates 7°47′0″N 5°31′0″E.


**Figure 1.1:** The map of Nigeria showing Ekiti State

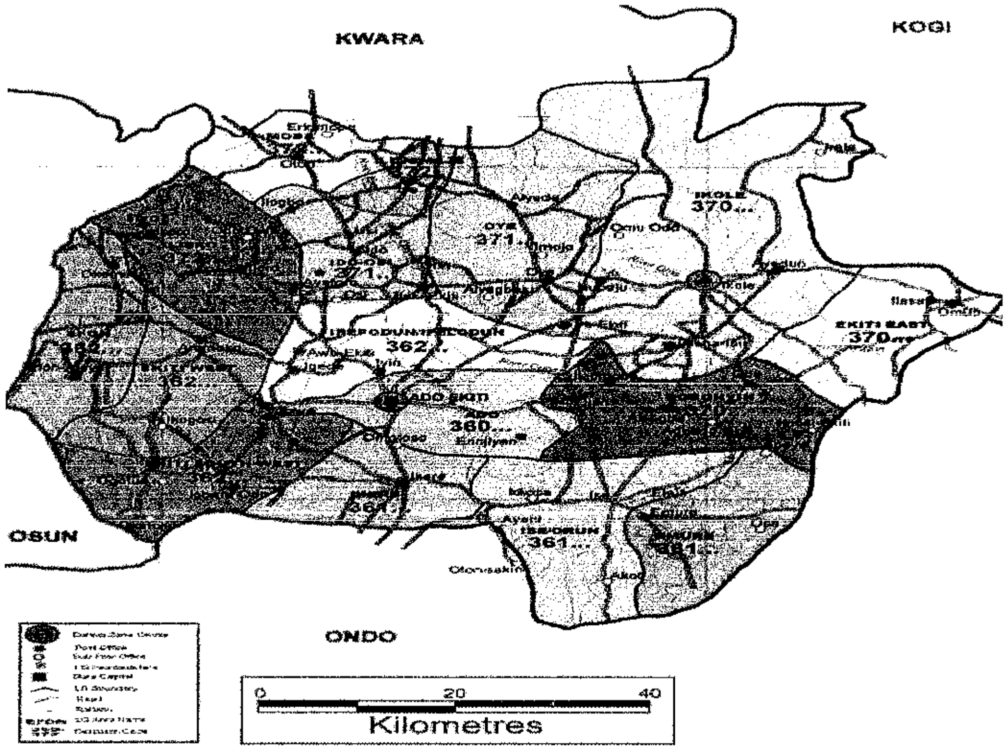



**Figure 1.2:** Map showing the study area

# CHAPTER TWO

# LITERATURE REVIEW

## 2.1 Background

A road pavement is a structure consisting of superimposed layers of processed materials above the natural soil sub-grade, whose primary function is to distribute the applied vehicle loads to the sub-grade. The pavement structure should be able to provide a surface of acceptable riding quality, adequate skid resistance, favorable light reflecting characteristics, and low noise pollution. The ultimate aim is to ensure that the transmitted stresses due to wheel load are sufficiently reduced, so that they will not exceed bearing capacity of the subgrade.

Two types of pavements are generally recognized as serving this purpose, namely flexible pavements and rigid pavements. This chapter gives an overview of pavement types, layers, and their functions, design and construction and pavement failures. Improper design of pavements leads to early failure of pavements affecting the riding quality.

Pavements can be classified based on the structural performance into two, flexible pavements and rigid pavements. In flexible pavements, wheel loads are transferred by grain-to-grain contact of the aggregate through the granular structure. The flexible pavement, having less flexural strength, acts like a flexible sheet (e.g. bituminous road). On the contrary, in rigid pavements, wheel loads are transferred to sub-grade soil by flexural strength of the pavement and the pavement acts like a rigid plate (e.g. cement concrete roads). In addition to these, composite pavements are also available. A thin layer of flexible pavement over rigid pavement is an ideal pavement with most desirable characteristics. However, such pavements are rarely used in new construction because of high cost and complex analysis required.

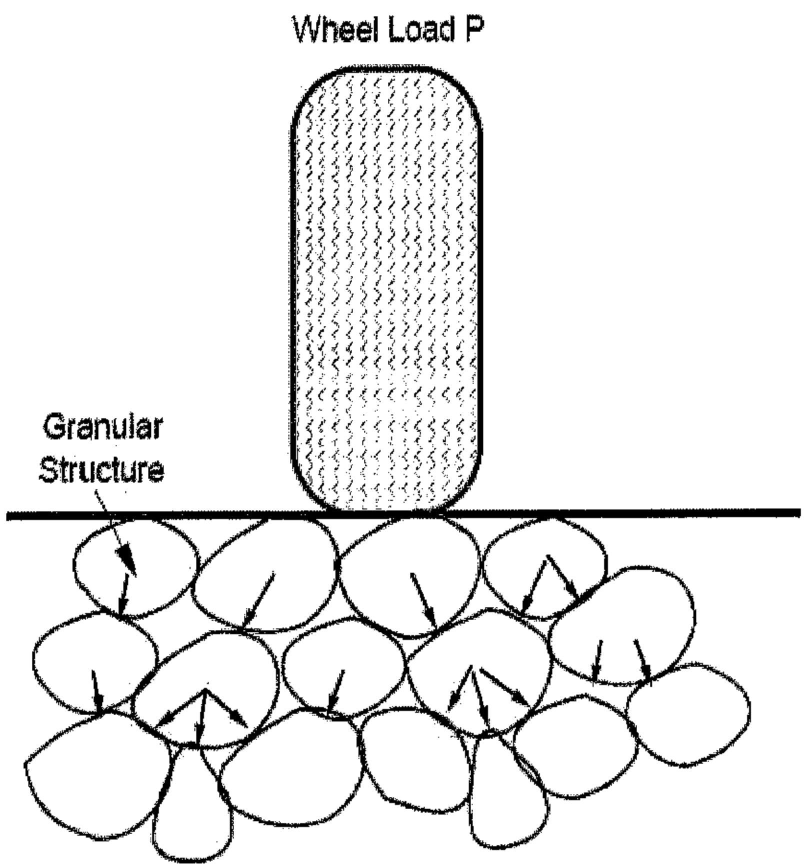



**Figure 2.1:** Load transfer in granular structure

Flexible pavements transmit wheel load stresses to the lower layers by grain-to-grain transfer through the points of contact in the granular structure (see Fig. 2:1). The wheel load acting on the pavement will be distributed to a wider area, and the stress decreases with the depth. Taking advantage of this stress distribution characteristic, flexible pavement normally has many layers. Hence, the design of flexible pavement uses the concept of layered system. Based on this, flexible pavement may be constructed in a number of layers and the top layer has to be of best quality to sustain maximum compressive stress, in addition to wear and tear. The lower layers experience lesser magnitude of stress and low quality material can be used. Flexible pavements are constructed using bituminous materials. These can be either in the form of surface treatments (such as bituminous surface treatments generally found on low volume roads) or, asphalt concrete surface courses (generally used on high volume roads such as national highways). Flexible pavement layers reflect the deformation of the lower layers on to the surface layer (e.g. if there is any undulation in sub-grade then it will be transferred to the surface layer). The design flexible pavement is based on overall performance of the pavement, the stresses produced are kept well below the allowable stresses of each pavement layer (Wikipedia).

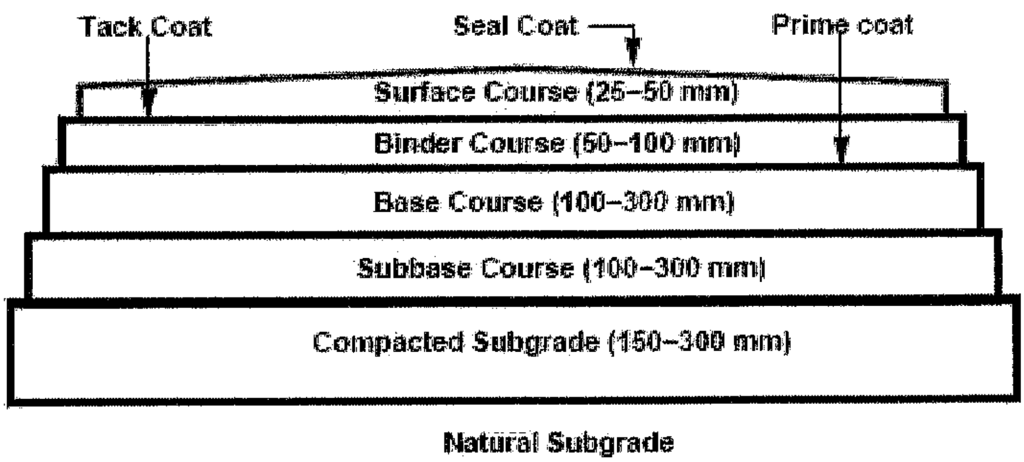



**Figure 2.2:** Typical cross section of a flexible pavement

Typical layers of a conventional flexible pavement includes seal coat, surface course, tack coat, binder course, prime coat, base course, sub-base course, compacted sub-grade, and natural sub-grade (Fig 2:2).

1. **Seal Coat**: Seal coat is a thin surface treatment used to water-proof the surface and to provide skid resistance.
2. **Tack Coat:** Tack coat is a very light application of asphalt, usually asphalt emulsion diluted with water. It provides proper bonding between two layers of binder course and must be thin, uniformly cover the entire surface, and set very fast.

3. **Prime Coat**: Prime coat is an application of low viscous cutback bitumen to an absorbent surface like granular bases on which binder layer is placed. It provides bonding between two layers. Unlike tack coat, prime coat penetrates into the layer below, plugs the voids, and forms a water tight surface.
4. **Surface course:** Surface course is the layer directly in contact with traffic loads and generally contains superior quality materials. They are usually constructed with dense graded asphalt concrete (AC). The functions and requirements of this layer are:
   1) It provides characteristics such as friction, smoothness, drainage, etc. Also it will prevent the entrance of excessive quantities of surface water into the underlying base, sub-base and sub-grade,
   2) It must be tough to resist the distortion under traffic and provide a smooth and skid- resistant riding surface,
   3) It must be water proof to protect the entire base and sub-grade from the weakening effect of water.
5. **Binder course:** This layer provides the bulk of the asphalt concrete structure. Its chief purpose is to distribute load to the base course. The binder course generally consists of aggregates having less asphalt and doesn't require quality as high as the surface course, so replacing a part of the surface course by the binder course results in more economical design.
6. **Base course;** The base course is the layer of material immediately beneath the surface of binder course and it provides additional load distribution and contributes to the sub-surface drainage. It may be composed of crushed stone, crushed slag, and other untreated or stabilized materials.
7. **Sub-Base course:** The sub-base course is the layer of material beneath the base course and the primary functions are to provide structural support, improve drainage, and reduce the intrusion of fines from the sub-grade in the pavement structure If the base course is open graded, then the sub-base course with more fines can serve as a filler between sub-grade and the base course A sub-base course is not always needed or used. For example, a pavement constructed over a high

quality, stiff sub-grade may not need the additional features offered by a sub-base course. In such situations, sub-base course may not be provided.

8. **Sub-grade:** The top soil or sub-grade is a layer of natural soil prepared to receive the stresses from the layers above. It is essential that the soil sub-grade is not overstressed. It should be compacted to the desirable density, near the optimum moisture content. (Wikipedia, 2018)

## 2.2 Soils

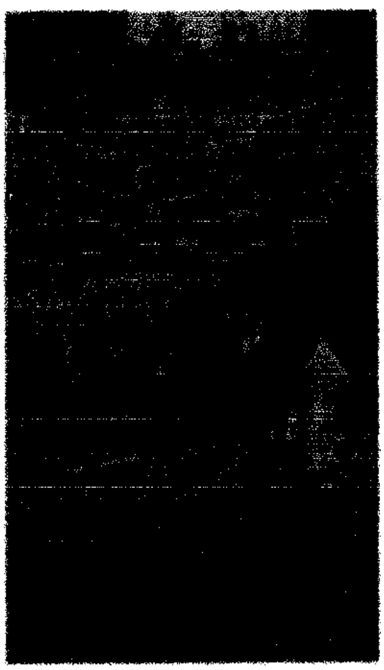
Soils are aggregates of mineral particles, and together with air and/or water in the void spaces, they form three-phase systems (Das braja, 1996). A large portion of the earth's surface is covered by soils, and they are widely used as construction and foundation materials. Soil mechanics is the branch of engineering that deals with the engineering properties of soils and their behavior under stress.

To the civil engineer, soil is any uncemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks as part of the rock cycle, the void space between the particles containing water and/or air. Weak cementation can be due to carbonates or oxides precipitated between the particles, or due to organic matter. Subsequent deposition and compression of soils, combined with cementation between particles, transforms soils into sedimentary rocks (a process known as lithification). If the products of weathering remain at their original location they constitute a residual soil. If the products are transported and deposited in a different location they constitute a transported soil, the agents of transportation being gravity, wind, water and glaciers. During transportation, the size and shape of particles can undergo change and the particles can be sorted into specific size ranges. Particle sizes in soils can vary from over 100 mm to less than 0.001 mm. In the UK, the size ranges are described.

The terms 'clay', 'silt' etc. are used to describe only the sizes of particles between specified limits. However, the same terms are also used to describe particular types of soil, classified according to their mechanical behavior (Das braja, 1996).

## 2.3 Laterite

The term laterite first appeared in the scientific literature more than 200 years ago and seems to have been first used by Dr. Francis Buchanan-Hamilton to denote a building material used in the Malabar district of India (Buchanan, 1807). Its appearance was described as "that of a ferruginous deposit of vesicular structure, apparently unstratified, and occurring not far below the surface". Moreover, Buchanan observed that "when fresh, it can readily be cut into regular blocks with a cutting tool. However, on exposure to the air, it rapidly hardens and becomes highly resistant to weathering". Thus, the unusual feature of the material first described by Buchanan under the name laterite, from the Latin word later, which means a brick, was that it had a soft consistency in situ but hardened rapidly on exposure – a phenomenon which led to the use of this material as a building brick. Modern pedological terminology would now perhaps describe Buchanan's laterite as plinthite (Wikipedia,2018).



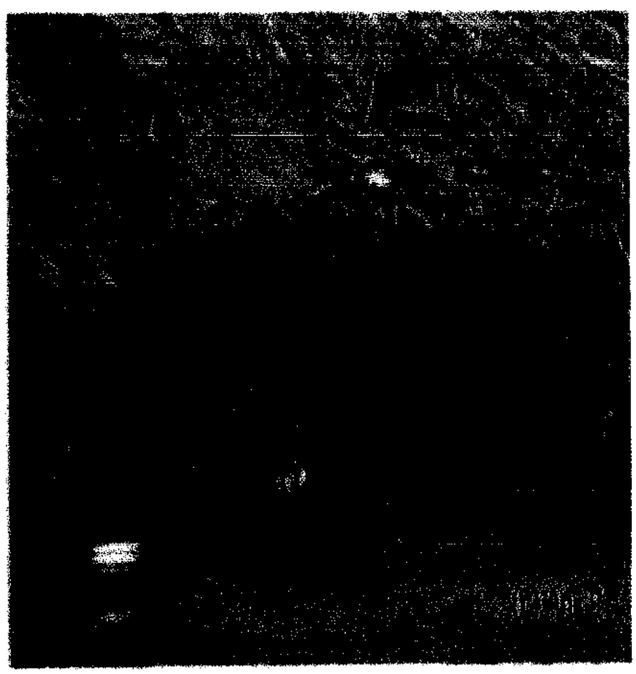


**Plate 2.1**: Cutting of laterite bricks in quarry (Left) and Use of laterite bricks for house construction (Right)

Despite its widespread occurrence and use in many African countries and, indeed, in many other tropical countries of the world, much confusion still exists concerning the general characteristics of what is often referred to loosely as "laterite". Moreover, the numerous attempts made at definition and classification, to enable engineering behaviour to be predicted, have been unsatisfactory because the term has been so loosely applied without a detailed soil description related to the formation of the material. What is better understood is that laterite is the result of a decomposition or weathering process, the consequences of which are of overriding significance in the formation of this material in the various tropical regions of the world. Thus, an appreciation of the tropical weathering process is fundamental to any system of classification or any attempt to identify the significant engineering characteristics of laterites.

### 2.4 Laterite Formation

Laterite is the product of a humid tropical weathering process, current or past, which has the following effects:

1. The parent material is chemically enriched with iron and aluminum oxides and hydroxides (sesquioxides)
2. The clay mineral component is largely kaolinitic
3. The silica content is reduced

The above processes usually produce yellow, brown, red or purple materials, with red being the predominant colour. While tropical weathering in oxidizing conditions generally leads to reddening, this does not necessarily produce a lateritic material – hence the widespread confusion concerning laterite and its behavior.

Laterite formation requires particular conditions which concentrate the iron- and aluminum- rich weathering products sufficiently to allow concretionary development, often progressing to a cemented horizon within the weathering profile. Three phases of action are necessary to produce concretionary laterite:

1. Humid tropical weathering to produce the minerals of laterite
2. Concentration of these minerals in a discrete zone
3. Concretionary development within the horizon.

According to Charman (1988), before the concretionary development of true laterite can take place, an additional process is required – the concentration of the weathering products within the residual soil/completely weathered zones.

In the investigation conducted by Charman (1988), "a mean annual temperature of around 25°C is needed for laterite formation, and in seasonal situations there should be a coincidence of the warm and wet periods. If there is high rainfall during the cold season, laterites do not develop freely, but the minimum annual rainfall required for laterite formation is generally at least 750 mm.

Ackroyd (1967) suggested a possible upper limit on rainfall of 1500 – 2000 mm per annum for the formation of concretionary laterites in Nigeria, while the work of Newill and Dowling (1970) found laterites in Malaysia in areas with a current annual rainfall of about 2000 mm. However, under higher rainfall conditions, concretions do not appear to form and the result is rather a highly leached sandy, fersiallitic soil in which the silt and clay are weakly cemented into "pseudosand" or "pseudosilt" particles (Ackroyd, 1967).

**2.5 Composition of Laterite**

Laterites are essentially two-component mixtures of the original host or parent material and the authigenic cementing, replacing or relatively accumulated minerals (mostly sesquioxides but also certain clay minerals). As the laterite develops, so the authigenic mineral content increases until it may constitute almost the whole material. Thus, hardpan laterite can be expected to have a higher content of sesquioxides (Al2O3 + Fe2O3) than a nodular laterite. Table 2.1 shows the typical chemical composition of materials described as laterites (or ferricretes in southern Africa). The citrate-carbonate-dithionite (CBD)-extractable iron content (a measure of the total free iron oxide and hydroxide minerals present) of hardpans laterites ranges between 43 and 77 % (Fitzpatrick, 1978; Fitzpatrick and Schwertmann, 1982). In the case of lateritic soils and gravels the content of Fe2O3 increases and that of Al2O3 decreases with particle size, while SiO2 is highest in intermediate fractions (LNEC et al, 1969).

**Table 2.1:** Typical chemical composition of lateritic material (Netterberg, 1985)

| Component | % By Mass | Main form of occurrence |
|---|---|---|
| $SiO_2$ | 5 - 70 | Quartz, feldspar, clay minerals |
| $Al_2O_3$ | 5 - 35 | Feldspar, clay minerals, gibbsite |
| $Fe_2O_3$ [2] | 5 - 70 | Goethite, hematite |
| $TiO_2$ | 0 - 5 | Anatase, rutile |
| MnO | 0–5? | ? |
| $P_2O_5$ | 0-1 | |
| $H_2O$ + | 5 - 20 | Clay minerals, goethite, gibbsite |
| Loss on ignition | 5 - 30 | Clay minerals, goethite, gibbsite, organic matter |
| Organic matter | 0,2 - 2 | Organic matter |

**Notes:**

[1] Bauxites are excluded.
[2] Total iron as $Fe_2O_3$.

**Table 2.2:** The minerals usually found in laterites **(Netterberg, 2013).**

| Major Element | Mineral [1] | Composition [2] | Colour [2] |
|---|---|---|---|
| Fe | limonite [3] | $Fe{\cdot}OH{\cdot}nH_2O$ | yellow to brown |
| | goethite | $\alpha - FeO(OH)$ | yellow to brown to black |
| | lepidocrocite | $\gamma - FeO(OH)$ | orange |
| | haematite | $\alpha - Fe_2O_3$ | red, reddish brown to black |
| | maghemite | $\gamma - Fe_2O_3$ | reddish brown |
| | magnetite | $Fe_3O_4$ | iron black |
| | ferrihydrite | $Fe_5HO_8{\cdot}H_2O$ [4] | reddish brown |
| Al | gibbsite | $\gamma - Al(OH_3)$ | white, greyish, greenish or reddish white |
| | boehmite | $\gamma - AlO(OH)$ | white, grey, pale lavender, yellow-green |
| | diaspore | $\alpha - AlO(OH)$ | white grey, pale lavender, yellow-green |
| Mn | pyrolusite? | $MnO_2$ | iron black |
| | manganite? | MnOOH | grey to black |
| Ti | anatase | $TiO_2$ | red, reddish brown to black |
| | rutile | $TiO_2$ | red, reddish brown to black |
| | ilmenite | $FeTiO_3$ | iron black |

**Notes:**

[1] Compiled from the various authors quoted in the text and Dixon and Weed (1989). Other non-sesquioxide minerals include kaolinite, halloysite, metahalloysite, illite, smectite, chlorite, and allophane; whilst significant organic matter may also be present.
[2] Mostly from Klein and Hurlbut (1993) and Dixon and Weed (1989).
[3] A field term used to refer to natural hydrous iron oxides of uncertain identity (Klein and Hurlbut, 1993).
[4] Also given as $Fe_5O_7(OH){\cdot}4H_2O$, $Fe_2O_3{\cdot}2FeOOH{\cdot}2.6H_2O$, $Fe_5HO_8{\cdot}4H_2O$, etc.

Studies on the mineralogy of laterites in Angola, and Mozambique (LNEC et al, 1959 1969), South Africa (Van der Merwe and Heystek, 1952; Maud, 1965; Frankel and Bayliss 1966, Fitzpatrick, 1974, 1978, 1983) all show the iron and aluminum in these materials to be dominantly in the form of goethite FeO(OH) (yellow-brown) with lesser haematite (Fe2O3), (red) and gibbsite (Al (OH)3) (white), and rarely maghemite (Fe2O3) (reddish-brown). Traces of anatase (TiO2) and rutile (TiO2) may also be present. No information is available on the S/R ratios of the laterites and lateritic soils actually used as road building materials in southern Africa. It is therefore recommended that as an interim measure, this ratio be determined both on the fraction passing 2.00 mm and on that passing 2 μm of a selection of such materials and that it also be determined before the relaxed Angolan or Brazilian specifications are applied to a particular case.

### 2.6 Geotechnical Properties

The geotechnical properties of lateritic materials generally depend on three factors:

1. The nature of the host or parent material (e.g. whether it was predominantly clay, sand or rock);
2. The stage of development (i.e. the extent to which the host material has been cemented or replaced); and
3. The nature of the cementing and/or replacing sesquioxide minerals

During development, the finer particles, such as clay, silt and sand, tend to become flocculated, aggregated, and cemented into silt to gravel-sized particles of varying strength and porosity (Netterberg, 1971; various authors cited in Gidigasu, 1976; and Morin and Todor, 1976). These particles or aggregations may or may not be broken down during laboratory testing and during construction. Moreover, both the clay mineral and the cementing and replacing minerals are different from the minerals in the temperate zone soils consisting of discrete particles from which much of our geotechnical experience and specifications have been derived. Laterites can therefore be expected to exhibit certain differences in behaviour.

The presence of porous particles found in laterite, for example, will tend to increase all moisture content determinations, including Atterberg limits, whereas in traditional soil mechanics it is usually assumed that all the water is outside the particles. Kaolinite, the dominant clay in most lateritic materials, has a non-expansive lattice which, compared to other clay mineral types such as smectite, makes the material less susceptible to volumetric expansion in the presence of moisture. Moreover, the sesquioxides in laterites may be hydrated and/or amorphous, while clays such as hydrated halloysite and allophane may be present. The possible effects of these minerals have been well reviewed by Morin and Todor (1976) and Gidigasu (1976) and, to a large extent, account for the so-called "relaxed" specifications adopted for selecting laterites, compared with the more traditional specifications such as those of AASHTO (2011).

In essence, the differences between traditional and pedogenic materials (e.g. laterite) render the geotechnical behaviour of the latter less predictable for the interpretation of the results of fundamental engineering tests such as Atterberg limits and grading.

In the investigation conducted by Kekere A.A. and Ifabiyi I.B. (2013) also revealed in their findings on a research on "Geotechnical Investigation of Road Failure along Ilorin-Ajase – Ipo Road Kwara State, Nigeria" that the Effort to maintain the road along Ilorin-Ajase Ipo road by government agency have not yielded any result because the maintenance carried out was approached wrongly. It is evidently clear from the findings that poor foundation materials like the subgrade and sub-base constitute the foundation of the road for instance, the presence of clayey soil and sandy soil have contributed to road failure witnessed on the road. It was recommended that areas badly affected should be scooped out and replaced with stabilizing agent like lateritic soils to ensure stability of foundation.

The work of Mukerji and Bahlmann (1978) indicated that laterites are difficult to use in road construction as their properties vary considerably and this variability makes their use difficult. They do note the fact that the materials achieve relatively high strengths and water resistance on drying. Similarly when excavated in "vertical cuts, highly laterized soils can be self-stabilizing (after hardening on exposure to the air)."

Nogami and Villibor (1991) indicate that fine grained lateritic soils were used only for sub base (or stabilized with cement for base) until the early 1970s when trial sections were done with neat soils. Routine use of such lateritic soils for base for low to medium traffic in Sao Paulo State started in the 1980s. In many cases they were blended with gap-graded crushed stone for more heavily trafficked roads. The area has rain in all months with between 1000 and 2000 mm annually (Thornthwaite 5 – 100). Although Charman (1988) suggested their use for low volume road bases only, they are used in Brazil for up to 1500 vpd and 5 MESA (Villibor, 2006) – their use clearly not being limited to low volume roads.

In the work of Bolarinwa et al.(2017), from the soil exploration and laboratory analysis of ikole ekiti, it was inferred that the soils encountered from 300mm to about 12m depth are mostly lateritic soils because they possess both cohesive and cohensionless soil properties.

### 2.7 Classification of Soil for Highway Use

Soil classification is a method by which soils are systematically categorized according to their probable engineering characteristics. It therefore serves as a means of identifying suitable sub base materials and predicting the probable behavior of a soil when used as subgrade material. The classification of a given soil is determined by conducting relatively simple tests on disturbed samples of the soil; the results are then correlated with field experience. Note, however, that although the engineering properties of a given soil to be used in highway construction can be predicted reliably from its classification, this should not be regarded as a substitute for the detailed investigation of the soil properties. Classifying the soil should be considered as a means of obtaining a general idea of how the soil will behave if used as a subgrade or sub-base material. The most commonly used classification system for highway purposes is the American Association of State Highway and Transportation Officials (AASHTO) Classification System. The Unified Soil Classification System (USCS) also is used to a lesser extent. A slightly modified version of the USCS is used fairly extensively in the United Kingdom.

### 2.8 AASHTO Soil Classification System

The AASHTO Classification System is based on the Public Roads Classification System that was developed in 1929 from the results of extensive research conducted by the Bureau of Public Roads, now known as the Federal Highway Administration. Several revisions have been made to the system since it was first published. The system has been described by AASHTO as a means for determining the relative quality of soils for use in embankments, subgrades, sub-bases, and bases. In the current publication, soils are classified into seven groups, A-1 through A-7, with several subgroups, as shown in Table 2.1. The classification of a given soil is based on its particle size distribution, LL, and PI. Soils are evaluated within each group by using an empirical formula to determine the group index (GI) of the soils, given as

$$GI = (F - 35) [0.2 - 0.005(LL - 40)] - 0.01(F - 15) (PI - 10)$$

Where,

1. GI -group index
2. F - Percent of soil particles passing 0.075 mm (No. 200) sieve in whole number based on material passing 75 mm (3 in.) sieve
3. LL - liquid limit (expressed in whole number)
4. PI - plasticity index (expressed in whole number)

The GI is determined to the nearest whole number. A value of zero should be recorded when a negative value is obtained for the GI. Also, in determining the GI for A-2-6 and A-2-7 subgroups, the LL part of above is not used—that is, only the second term of the equation is used.

Under the AASHTO system, granular soils fall into classes A-1 to A-3. A-1 soils consist of well graded granular materials, A-2 soils contain significant amounts of silts and clays, and A-3 soils are clean but poorly graded sands.

Classifying soils under the AASHTO system will consist of first determining the particle size distribution and Atterberg limits of the soil and then reading Table 2.3 from left to right to find the correct group. The correct group is the first one from the left that fits the particle size distribution and Atterberg limits and should be expressed in terms of group designation and the GI. Examples are A-2-6(4) and A-6(10).

In general, the suitability of a soil deposit for use in highway construction can be summarized as follows.

1. Soils classified as A-1-a, A-1-b, A-2-4, A-2-5, and A-3 can be used satisfactorily as subgrade or sub-base material if properly drained .In addition, and such soils must be properly compacted and covered with an adequate thickness of pavement (base and/or surface cover) for the surface load to be carried.
2. Materials classified as A-2-6, A-2-7, A-4, A-5, A-6, A-7-5, and A-7-6 will require a layer of sub-base material if used as subgrade. If these

are to be used as embankment materials, special attention must be given to the design of the embankment.

3. When soils are properly drained and compacted, their value as subgrade material decreases as the GI increases. For example, a soil with a GI of zero (an indication of a good subgrade material) will be better as a subgrade material than one with a GI of 20 (an indication of a poor subgrade material)

**Table 2.3**: AASHTO Classification System

| General classification | Granular materials (35% or less of total sample passing No. 200 sieve) | | | | | | |
|---|---|---|---|---|---|---|---|
| | A-1 | | | A-2 | | | |
| Group classification | A-1-a | A-1-b | A-3 | A-2-4 | A-2-5 | A-2-6 | A-2-7 |
| Sieve analysis (% passing) | | | | | | | |
| No. 10 sieve | 50 max | | | | | | |
| No. 40 sieve | 30 max | 50 max | 51 min | | | | |
| No. 200 sieve | 15 max | 25 max | 10 max | 35 max | 35 max | 35 max | 35 max |
| For fraction passing No. 40 sieve | | | | | | | |
| Liquid limit (LL) | | | | 40 max | 41 min | 40 max | 41 min |
| Plasticity index (PI) | 6 max | | Nonplastic | 10 max | 10 max | 11 min | 11 min |
| Usual type of material | Stone fragments, gravel, and sand | | Fine sand | Silty or clayey gravel and sand | | | |
| Subgrade rating | Excellent to good | | | | | | |

| General classification | Silt-clay materials (More than 35% of total sample passing No. 200 sieve) | | | |
|---|---|---|---|---|
| Group classification | A-4 | A-5 | A-6 | A-7 |
| | | | | A-7-5[a]<br>A-7-6[b] |
| Sieve analysis (% passing) | | | | |
| No. 10 sieve | | | | |
| No. 40 sieve | | | | |
| No. 200 sieve | 36 min | 36 min | 36 min | 36 min |
| For fraction passing No. 40 sieve | | | | |
| Liquid limit (LL) | 40 max | 41 min | 40 max | 41 min |
| Plasticity index (PI) | 10 max | 10 max | 11 min | 11 min |
| Usual types of material | Mostly silty soils | | Mostly clayey soils | |
| Subgrade rating | Fair to poor | | | |

[a]If PI $\leq$ LL $-$ 30, the classification is A-7-5.

[b]If PI $>$ LL $-$ 30, the classification is A-7-6.

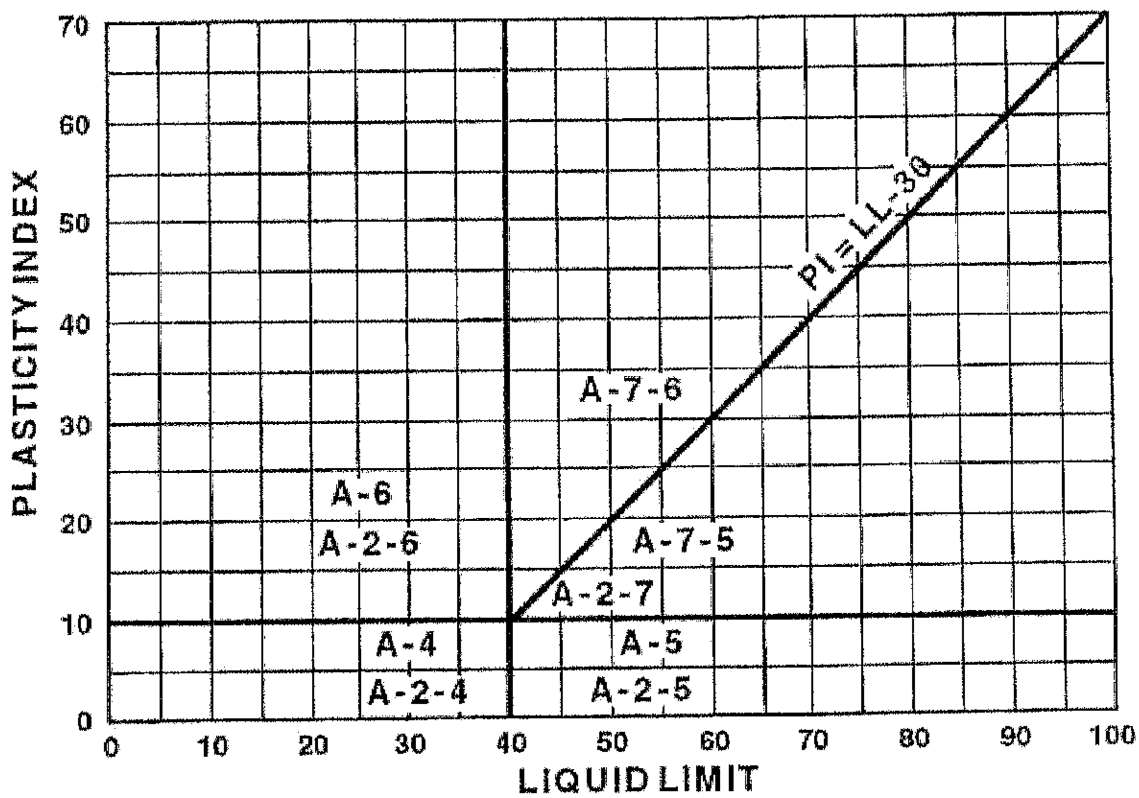



**Figure 2.3:** Relationship between Liquid Limit and Plasticity Index for Silt-Clay Groups (AASHTO M 145-91)

## 2.9 Unified Soil Classification System (USCS)

The original USCS system was developed during World War II for use in airfield construction. That system has been modified several times to obtain the current version which also can be applied to other types of construction such as dams and foundations. The fundamental premise used in the USCS system is that the engineering properties of any coarse-grained soil depend on its particle size distribution, whereas those for a fine-grained soil depend on its plasticity. Thus, the system classifies coarse-grained soils on the basis of grain size characteristics and fine-grained soils according to plasticity characteristics.

Table 2.4 lists the USCS definitions for the four major groups of materials, consisting of coarse-grained soils, fine-grained soils, organic soils, and peat. Material that is retained in the 75 mm (3 in.) sieve is recorded, but only that which passes is used for the classification of the sample. Soils with more than 50 percent of their particles being retained on the No. 200 sieve are coarse-grained, and those with less than 50 percent of their particles retained are fine grained soils. The coarse grained soils are subdivided into gravels (G) and sands (S). Soils having more than 50 percent of their particles larger than 75 mm—that is, retained on the No. 4 sieve—are gravels and those with more than 50 percent of their particles smaller than 75mm—that is, passed through the No. 4 sieve—are sands. The gravels and sands are further divided into four subgroups—each based on grain-size distribution and the nature of the fine particles in them. They therefore can be classified as either well graded (W), poorly graded (P), silty (M), or clayey (C). Gravels can be described as either well- graded gravel (GW), poorly graded gravel (GP), silty gravel (GM), or clayey gravels (GC), and sands can be described as well-graded sand (SW), poorly graded sand (SP), silty sand (SM), or clayey sand (SC).

**Table 2.4:** Classification of Four major groups of materials

| **Soil Identification** | **First Letter of Group Symbol** | **Second Letter of Group Symbol** |
|---|---|---|
| Coarse grained soil | G: gravel, S: sand | W: Well graded<br>P: Poorly graded |
| Fine grained soil | M: silt, C: clay | L: Low plasticity (LL less than 50)<br>H: High plasticity (LL more than 50) |
| Organic soil | O | L: Low plasticity (LL less than 50)<br>H: High plasticity (LL more than 50) |
| Highly organic soils | Pt | No second letter |

A gravel or sandy soil is described as well graded or poorly gr6aded, depending on the values of two shape parameters known as the coefficient of uniformity, Cu, and the coefficient of curvature, Cc, given as

$$cu = \frac{D60}{D10}$$

And

$$cc = \frac{(D30)^2}{D60 * D10}$$

Where

$D60 =$ Grain diameter at 60% passing

$D30 =$ Grain diameter at 30% passing

$D10 =$ Grain diameter at 10% passing

Gravels are described as well graded if Cu greater than four and Cc is between one and three. Sands are described as well graded if Cu greater than six and Cc is between one and three. The fine-grained soils, which are defined as those having more than 50 percent of their particles passing the No. 200 sieve, are subdivided into clays (C) or silt (M),depending on the PI and LL of the soil. A plasticity chart, shown in Figure 2.2, is used to determine whether a soil is silty or clayey. The chart is a plot of PI versus LL, from which a dividing line known as the "A" line, which generally separates the more clayey materials from the silty materials, was developed.
Soils with plots of LLs and PIs below the "A" line are silty soils, whereas those with plots above the "A" line are clayey soils. Organic clays are an exception to this general rule, since they plot below the "A" line. Organic clays, however, generally behave similarly to soils of lower plasticity.

Classification of coarse-grained soils as silty or clayey also depends on their LL plots. Only coarse-grained soils with more than 12 percent fines (that is, passes the No. 200 sieve) are so classified (see Fig. 2.3). Those soils with plots below the "A" line or with a PI less than four are silty gravel (CM) or silty sand (SM), and those with plots above the "A" line with a PI greater than seven are classified as clayey gravels (GC) or clayey sands (SC). The organic, silty, and clayey soils are further divided into two groups, one having a relatively low LL (L) and the other having a relatively high LL (H). The dividing line between high LL soils and low LL soils is arbitrarily set at 50 percent. Fine-grained soils can be classified as either silt with low plasticity (ML), silt with high plasticity (MH), clays with high plasticity (CH), clays with low plasticity (CL), or organic silt with high plasticity (OH).

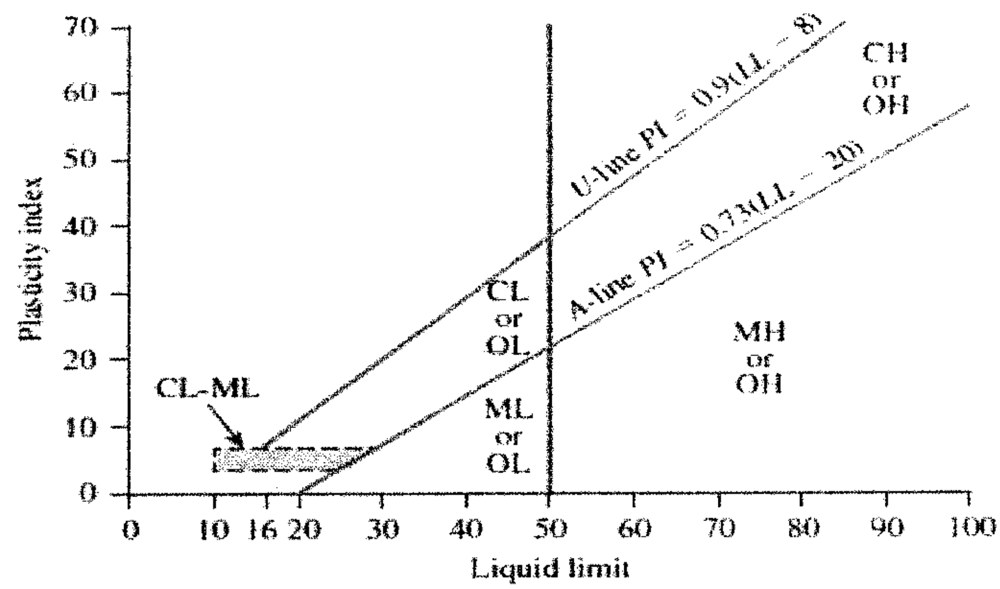
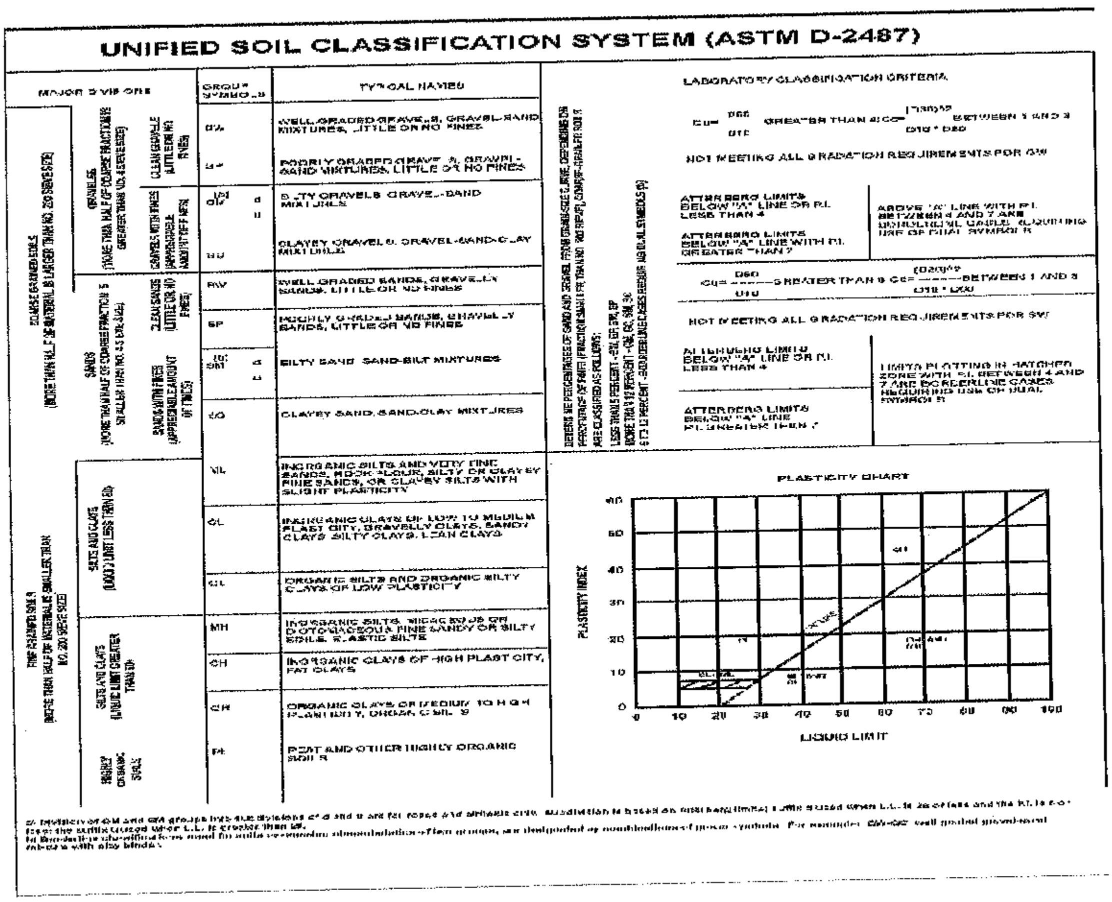



**Figure 2.4:** Plasticity Chart of USCS, Joseph E (1978)

The plasticity chart shown above in is a graphical representation of the USCS based solely on the plastic and liquid limits (Section 4-2.06.02) of the material passing the 0.425mm (No. 40) sieve. Clays will plot above the "A-line" and silts below. The chart further divides the clays and silts into low (less than 50) and high liquid limits.

Table 2.5 shows the USCS classification system along with the criteria utilized for associating the group symbol, such as "CL," with the soil. In this chart, $D_{60}$ refers to the diameter of the soil particles that 60 percent of the sample would pass on a sieve, as indicated on the gradation curve. Similarly, $D_{10}$ relates to the maximum diameter of the smallest 10 percent, by weight.

**Table 2.5:** Unified Soil Classification System chart (after U.S. Army Corps of Engineers, Waterways Experiment Station, TM 3-357, 1953).




## 2.1.0 Correlation of the Classification Systems

The AASHTO and USCS classification systems are attempts to associate pertinent engineering properties with identifiable soil groupings. However, each system defines soil groups in a slightly different manner. For example, AASHTO classification systems distinguish gravel from sand at the 2.0 millimeters (No.10) sieve, whereas the USCS uses a break at the 4.76 millimeters (No. 4) sieve. The same coarse-grained soil could, therefore, have different percentages of gravel and sand in the USCS classification systems.

### 2.1.1 Classification of Laterites and Lateritic Soils

There have been several attempts to classify laterites and lateritic soils for many years, but none of the proposed classification systems has been accepted universally. According to Maignien (1942), these classification systems can be grouped as;

(a) Analytical classifications which are based mainly on morphological characteristics with a bias toward soil genetic considerations, and

(b) Synthetic classifications which are based on genetic factors or soil-genetic processes or on properties of pedogenetic factors or processes.

As Mchr and Van Baren (1945) point out every classification system should have some predetermined purposes. None of the classification systems mentioned above has an aim to classify the soils according to their engineering behavior. Although there are some popular engineering classification systems, such as the Unified system or the American Association of State Highway Officials system, which have been used satisfactorily in the temperate environments of the world for years, they have not been so successful in the case of tropical soils. These classification systems are based on plasticity and gradation data of the soils; but as discussed previously, such characteristics of tropical soils are not reproducible by standard laboratory tests. The reasons for this, once again, are the influences of sample preparation and handling which disrupt the natural structure of the soil.

In order to avoid such problems, several authors have advocated a classification of laterites and lateritic soils for engineering purposes, based on parent material and degree of weathering.

Fish (1923) and Gidigasu (1926) made attempts to use pedalogical classifications for engineering purposes. Ruddock (1953) has suggested an engineering classification based on topographic position, sample depth and depth to water table which are, in fact, factors influencing the degree of weathering. Lohnes and Demirel (1937) have suggested to use specific gravity, void ratio and degree of weathering for engineering classification of tropical soils. None of these proposed engineering classification systems, however, has found a broad acceptance yet.

From the above discussion, it becomes evident that an appropriate classification of laterites and lateritic soils for engineering purposes is still nonexistent.

## CHAPTER 3

## METHODOLOGY

### 3.1 Theoretical Background/Framework

The project involves sample collection and laboratory tests. Each test was conducted several times and the averaged results considered. These are red soils obtained from earlier sampling and fieldwork. The aim is to attain geotechnical information, which is the composition and properties of the soil, which is essential to proper design and execution of engineering works (Capper, 1963). In this study, experimental design were employed and deductions derived purely from the obtained results. The size of each sample is sufficient for the following tests to be carried out.

1. Atterberg Limits
2. Compaction test (Standard Compaction: 2.5 kg rammer)
3. California Bearing Ratio Test
4. Triaxial Test
5. Sieve Analysis
6. Linear Shrinkage

### 3.2 Materials Design and Preparation

A total of four samples were collected from four different locations. The soil samples were taken at a depth of 1200mm-1500mm. Both disturbed and undisturbed samples were taken at these locations. The locations are Asin Ekiti, Oke-Orin Ekiti, Ikoyi Ekiti, and Federal University Ikole Campus Ikole Ekiti.

The project involves sample collection and laboratory tests. Each test was conducted thrice and the averaged results is considered.

After collection, soil samples were stored in polythene bags to prevent loss of moisture contents. The samples were then take to the laboratory where the deleterious materials such as roots were removed. The samples were air dried, broken down with mortar and pestle and passed through a set of sieve (i.e. from Sieve No. 10 (18.75mm) to Sieve No. 1 (75mm) to remove large particles. Molding of test specimens were started as soon as possible after completion of identification. All tests were performed according to standard methods contained in BS 1377 (1990). Their properties were studied and determined to ensure that all relevant factors would be available for establishment of correlations among them. The tests carried out on each of the selected samples will be Grain size analysis, Consistency test (i.e. Liquid Limits (LL), Plastic Limit (PL) and Plasticity Index (PI)), Compaction test (i.e. Optimum Moisture Content (OMC) and Maximum Dry Density (MDD)), Natural moisture content, Specific Gravity, Consolidation test and California Bearing Ratio (CBR). The results were compared to the standard specified values and grouped in accordance with General Specification for roads and bridges FMWH, (1997) and American Association of State Highway and Transportation Officials AASHTO, (1986) respectively.

## 3.3 Experimental Investigations

### 3.3.1 Preliminary Investigations

**1. Particle Size Distribution**

The samples were air dried for about 24 hours and those with cluster particle were pulverized to their natural sizes. 500 gram of each sample were weighed and wet sieve using 425 μm sieve. Residues of the washed samples were oven-dried and pulverized into fine mass. Each sample will be placed in sieve stack and shaken using mechanical shaker. The weight retained in each sieve were recorded for further computations and plotting.

**2. Linear Shrinkage**

Linear Shrinkage method covers the determination of the total linear shrinkage from linear measurements on a bar of soil of the fraction of a soil sample passing a 425 mm test sieve, originally having the moisture content of the Liquid Limit.

### 3.3.2 Main Investigations

**1. California Bearing Ratio (CBR) Test**

Fresh sets of air-dried samples were compacted in a 152mm diameter 173mm height CBR mode following already described procedure but at 27 blows per layer. A piece of filter paper was placed on the compacted sample and the base was replaced by a perforated plate and immersed in water for 48hours. The socked sample were taken to the CBR machine and readings of force were taken at interval of penetration of 0.625mm.

**2. Compaction Test**

The dried soil sample passing the 20mm BS sieve of about 8kg was used. The sample was mixed thoroughly with suitable amount of water of 2.5% initially and later increased to 5%, 7.5% and 10% on subsequent tests. The soil was compacted using British Standard and Western African Standard. The British Standard Method of compaction test make use of a small mould of volume 1000cm3, small rammer of mass 2.5kg and the sample is divided into three (3) layers, each layer being compacted with 27 blows per layer at a falling height of 300mm while the West Africa compaction method makes use of big mould of volume 2305cm3, a big rammer 4.5kg in mass. The sample was divided into five layers and each layer is compacted with 27 blows per layer at a falling height of 450mm. A reprehensive sample of the specimen was taken and the moisture content determined. From the graph of the dry density against moisture content, the maximum dry density (MDD) and optimum moisture content (OMC) were determined.

**3. Consistency Test**

Consistency limits test is also known as Atterberg limits test where Liquid limit test and Plastic limit test were carried out. Also the plasticity index was determined. The limits were determined for the soil in its natural state.

**A. Liquid Limit**

The cone penetrometer method is the preferred method to the Casagrande test as it is essentially a static test depending on soil shear strength. This method covers the determination of the liquid limit of a sample in its natural state, or a sample from which material retained on a 425 mm test sieve has been removed, it is based on the measurement of penetration into the soil of a standardized cone.

**B. Plastic Limit and Plasticity Index**

This method covers the determination of the liquid limit of a sample in its natural state, or a sample from which material retained on a 425 mm test sieve has been removed.

## 4. Triaxial Test

The triaxial compression test, introduced by Casagrande and Terzaghi in 1936, is by far the most popular and extensively used shearing strength test, both for field application and for purposes of research. As the name itself suggests, the soil specimen is subjected to three compressive stresses in mutually perpendicular directions, one of the three stresses being increased until the specimen fails in shear. Usually a cylindrical specimen with a height equal to twice its diameter is used. The desired three-dimensional stress system is achieved by an initial application of all-round fluid pressure or confining pressure through water. While this confining pressure is kept constant throughout the test, axial or vertical loading is increased gradually and at a uniform rate. The axial stress thus constitutes the major principal stress and the confining pressure acts in the other two principal directions, the intermediate and minor principal stresses being equal to the confining pressure.

# CHAPTER FOUR

## RESULTS AND DISCUSSION

The results of classification tests (grain size analysis, natural moisture content, Atterberg's limits and soil classification) as well as the compaction, triaxial and CBR tests are discussed below;

### 1. Particle Size Distribution

The percentage of material passing through no 200BS sieve ranges between 22%-44.8%. The particle size analysis shows that the percentages passing number 200BS sieve are 32%, 22%, 44.8% and 34% for samples at oke orin, ikoyi, campus and asin respectively. These are shown in figures below. According to federal ministry of works general specification requirements for roads and bridges (1994), samples S1 and S2 can be deduced as suitable for sub-grade, sub-base and base materials as the percentage by weight finer than N0 200BS test sieve is less than 35%.

**Table 4.1**- Sieve Analysis Results from Oke-Orin

| Sieve size | Weight retained | % retained | % passing |
|---|---|---|---|
| 9.50 | 12.65 | 2.50 | 97.50 |
| 4.75 | 28.43 | 5.70 | 91.80 |
| 2.36 | 45.78 | 9.20 | 82.60 |
| 1.18 | 46.76 | 9.40 | 73.20 |
| 600 | 41.06 | 8.30 | 64.90 |
| 300 | 73.2 | 14.60 | 50.30 |
| 150 | 60.60 | 12.10 | 38.20 |
| 75 | 35.02 | 7.00 | 31.20 |

**Table 4.2**- Sieve Analysis Results from Asin

| Sieve size | Weight retained | % retained | % passing |
|---|---|---|---|
| 9.50 | 3.40 | 0.70 | 97.20 |
| 4.75 | 20.50 | 4.10 | 95.20 |
| 2.36 | 40.00 | 8.00 | 87.20 |
| 1.18 | 54.00 | 10.80 | 76.40 |
| 600 | 64.80 | 13.00 | 63.40 |
| 300 | 67.60 | 13.50 | 49.90 |
| 150 | 50.50 | 10.10 | 39.80 |
| 75 | 28.80 | 5.80 | 34.00 |

**Table 4.3**- Sieve Analysis Results from Ikoyi

| Sieve size | Weight retained | % retained | % passing |
|---|---|---|---|
| 9.50 | 20.75 | 4.20 | 95.80 |
| 4.75 | 57.50 | 11.50 | 84.30 |
| 2.36 | 63.84 | 12.80 | 71.50 |
| 1.18 | 49.31 | 9.90 | 61.60 |
| 600 | 38.83 | 7.70 | 53.90 |
| 300 | 65.91 | 13.20 | 40.70 |
| 150 | 64.80 | 13.00 | 27.70 |
| 75 | 29.24 | 5.80 | 21.90 |

**Table 4.4**- Sieve Analysis Results from Campus

| Sieve size | Weight retained | % retained | % passing |
|---|---|---|---|
| 9.50 | 9.51 | 1.90 | 98.10 |
| 4.75 | 11.06 | 2.20 | 95.90 |
| 2.36 | 15.31 | 3.10 | 92.80 |
| 1.18 | 28.42 | 5.70 | 87.10 |
| 600 | 51.28 | 10.30 | 76.80 |
| 300 | 68.32 | 13.70 | 63.10 |
| 150 | 59.47 | 11.90 | 51.20 |
| 75 | 31.92 | 6.40 | 44.80 |

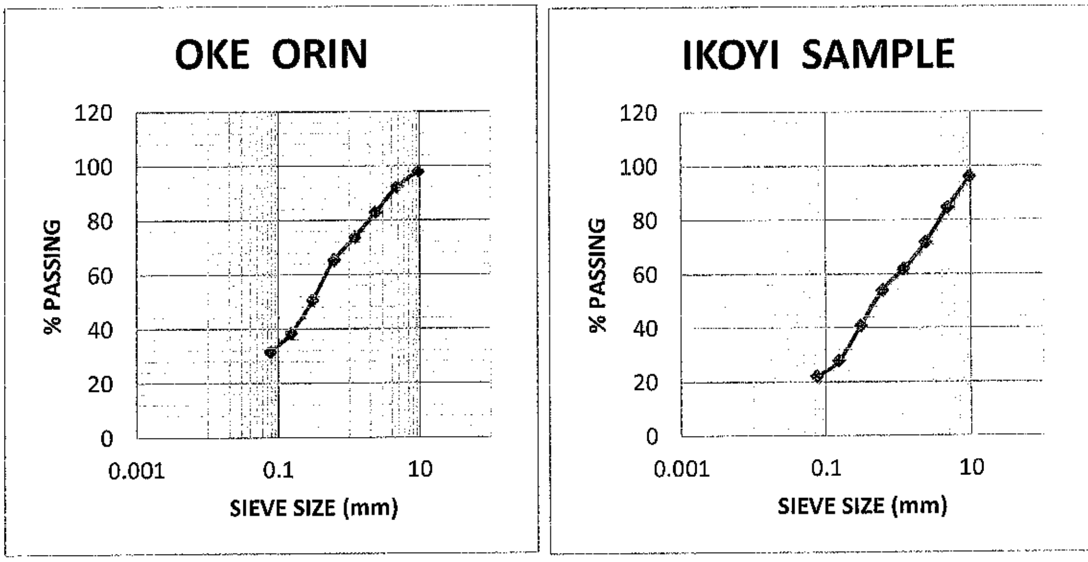



**Figure 4.1**- Particle size distribution for Oke-orin and Ikoyi

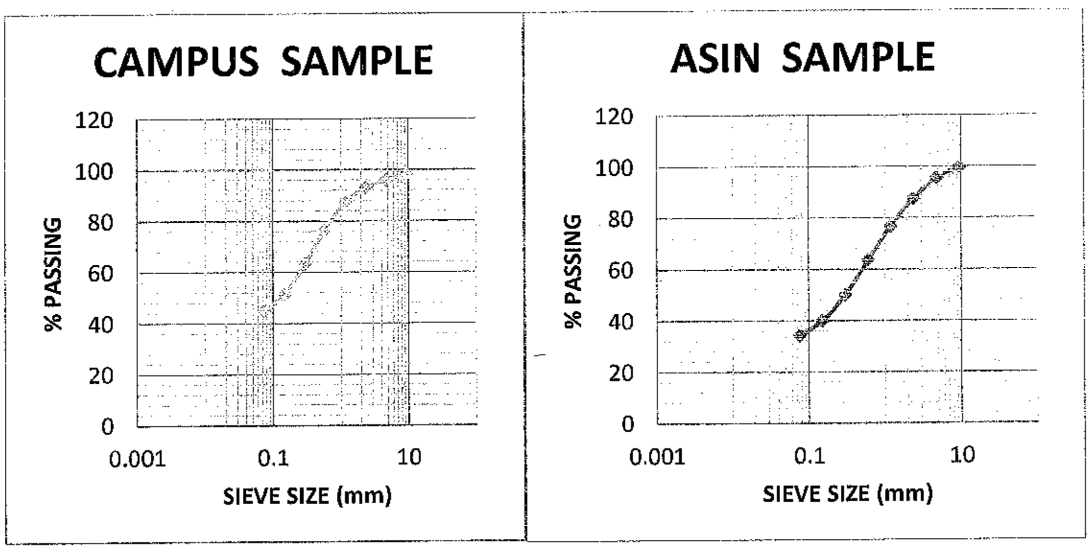



**Figure 4.2**- Particle size distribution for Campus and Asin

## 2. California Bearing Ratio

The soaked CBR value for the lateritic soil samples ranges from 34.50% - 75.65%. Federal ministry (1994) recommended soaked CBR for sub-grade and sub-base soils not less than 5% and 30% respectively. For the base (unsoaked CBR) not less than 80%. The result for soil samples shows that all the soils are suitable for sub-grade and sub-base course. The summary of the CBR results are shown in table below and the figures below show the graphs for the CBR.

**Table 4.5**- Summary of the California bearing ratio (CBR)

| Sample Location | | ASIN | CAMPUS | IKOYI | OKE-ORIN |
|---|---|---|---|---|---|
| CBR | 2.5mm | 75.50 | 34.50 | 36.70 | 67.22 |
| | 5.0mm | 75.68 | 40.10 | 52.10 | 72.64 |

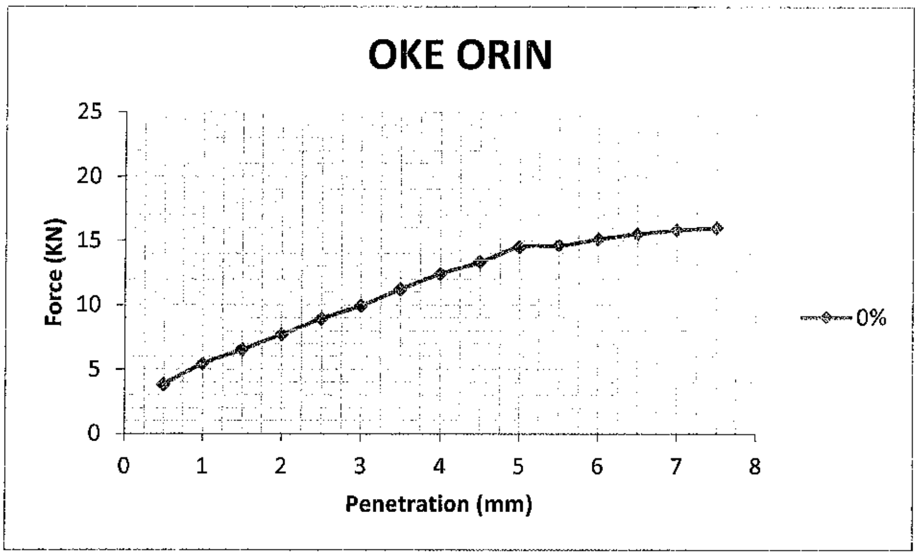



**Figure 4.3-** CBR Graph for Oke-Orin

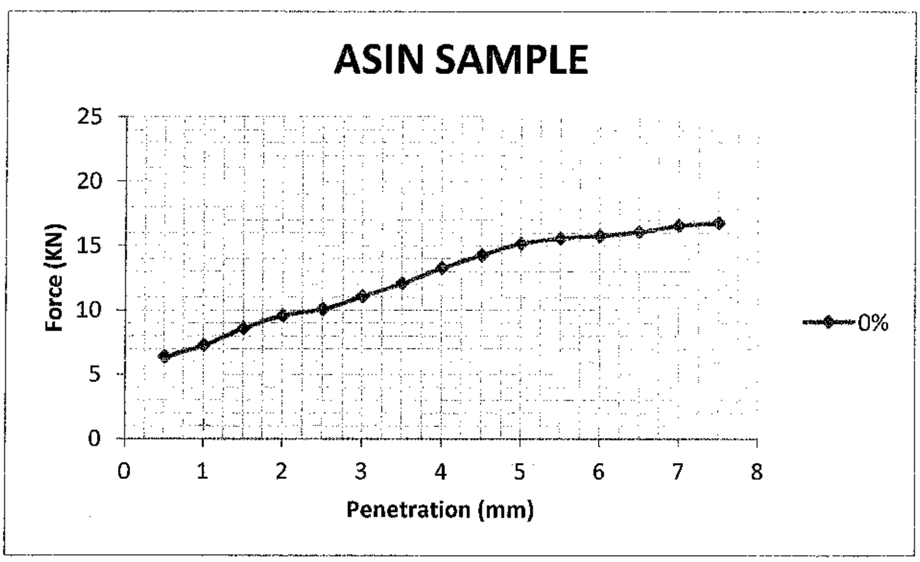



**Figure 4.4-** CBR Graph for Asin

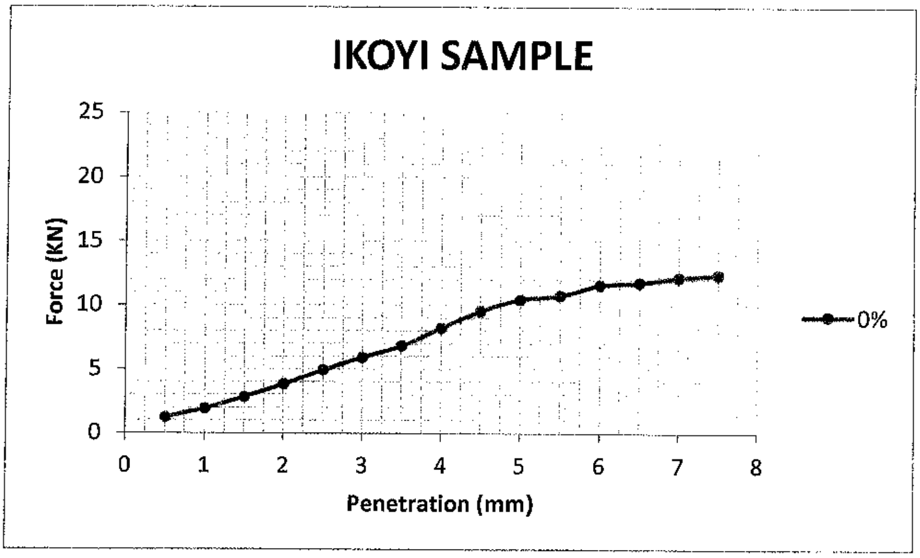



**Figure 4.5-** CBR Graph for Ikoyi

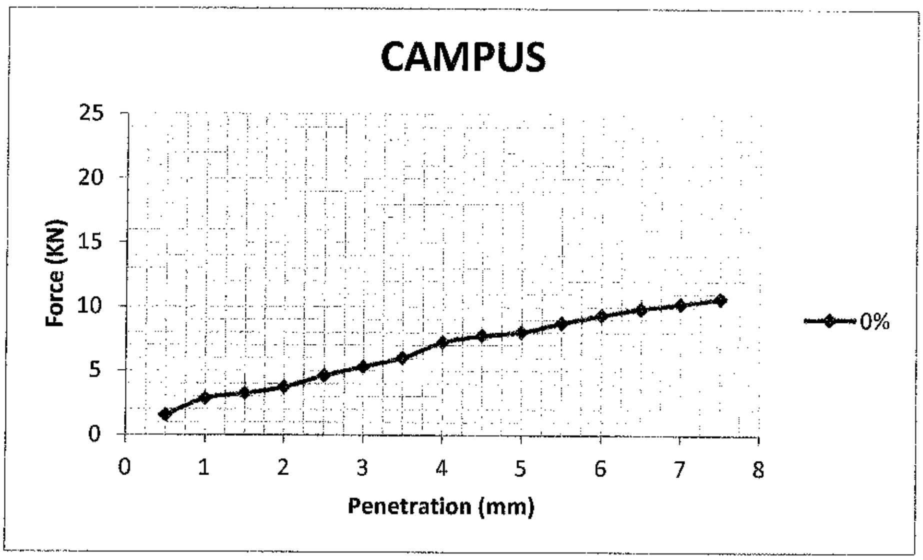



**Figure 4.6-** CBR Graph for Campus

**3. Atterberg Limit**

The result shows that the samples have silty particle sizes predominating. Hence the soil sample as a result of its particle size composition happens to be a cohesive soil with a considerable amount of plasticity. Federal ministry of works general specification requirements for roads and bridges (1994) recommend liquid limit not greater than 80% for sub-grade and not greater than 35% for sub-base and base course. Also plasticity index not greater than 55% for sub-grade and not greater than 12% for both sub-base and base. From the studied soil, the samples at locations asin, ikoyi, oke orin and campus fall within this specifications for the liquid limit but fail to fall within the specifications for the plasticity index making them poor for sub-base and base material but excellent as a subgrade material.

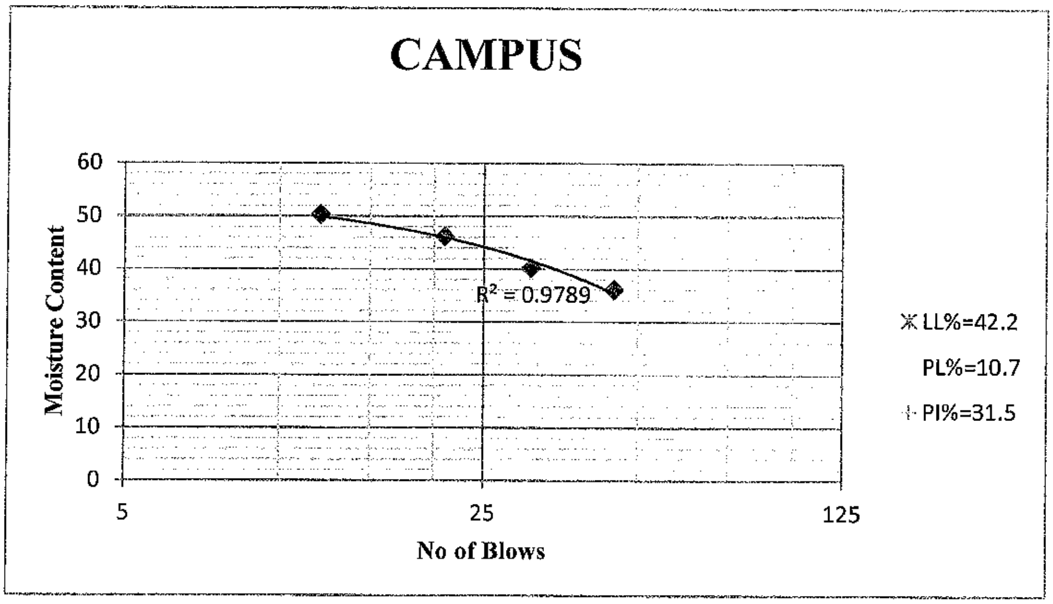



**FIGURE 4.7-** Graph for Campus showing the plastic limit, liquid limit and plasticity index

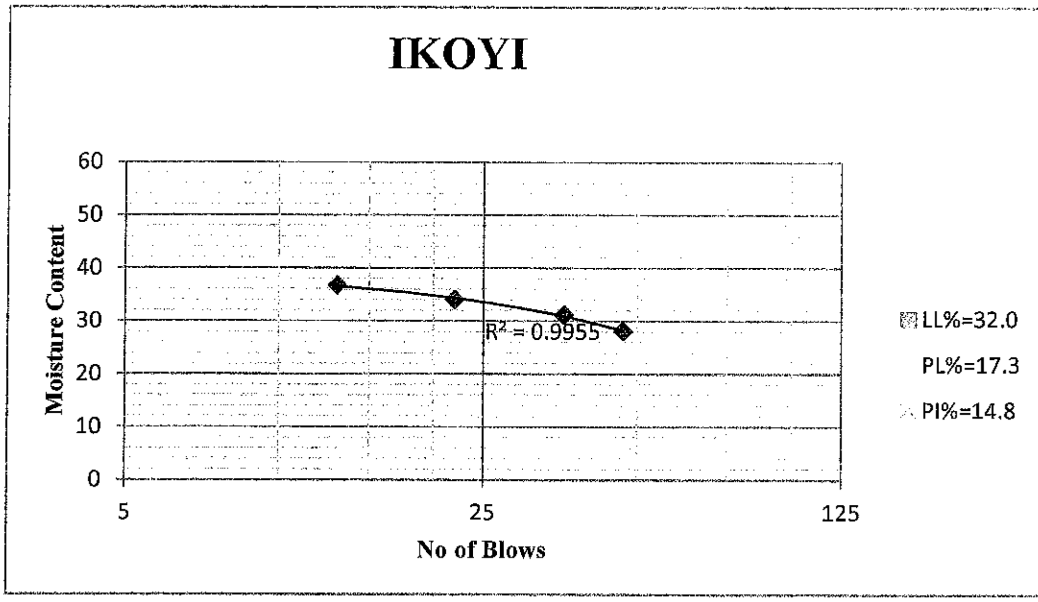



**Figure 4.8-** Graph for Ikoyi showing the plastic limit, liquid limit and plasticity index

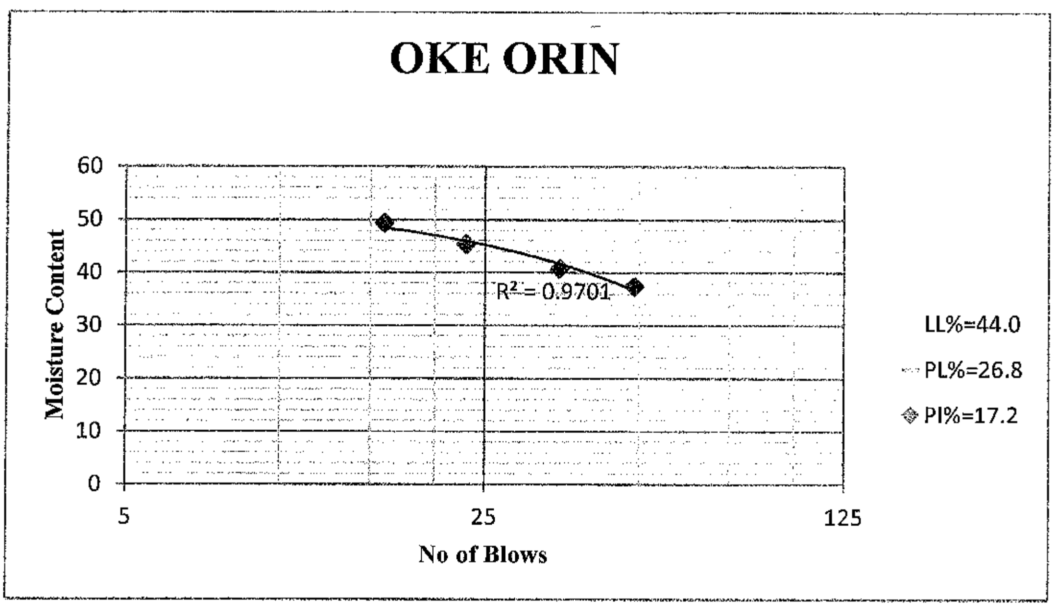



**Figure 4.9-** Graph for Oke-orin showing the plastic limit, liquid limit and plasticity index

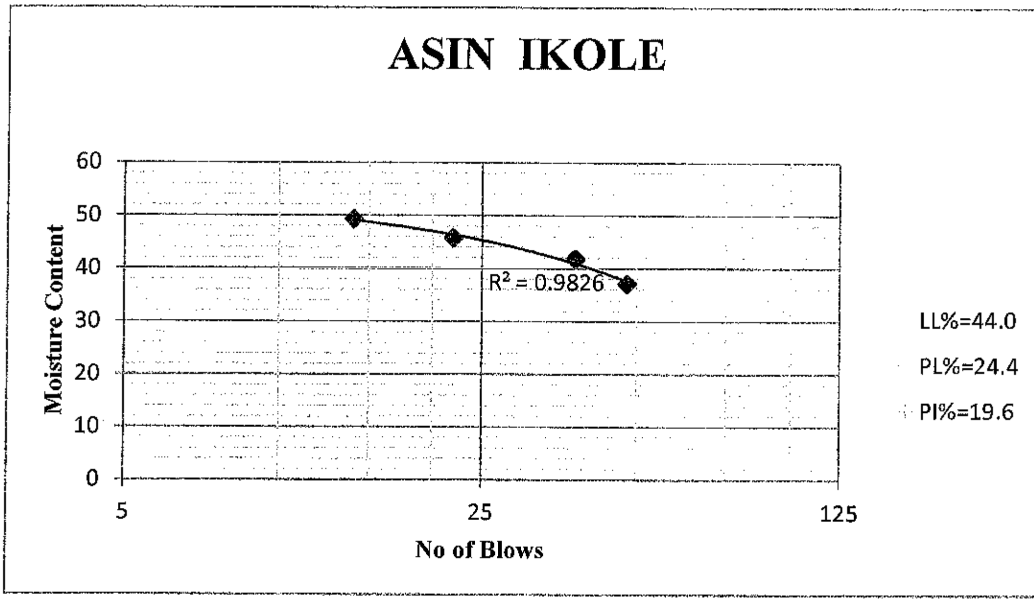



**Figure 4.10-** Graph for Asin showing the plastic limit, liquid limit and plasticity index

**4. Compaction Test**

The maximum dry density for the soil samples varies between 1.56 Mg/m3 and 1.78Mg/m3 while that of optimum moisture content ranged between 19.60% to 24%.The summary of the results are shown in table below. The figure below show the behavior of the soil for compaction. According to O'flaherty (1988) the ranges of values that may be anticipated when using the standard proctor test methods are: for clay, maximum dry density fall between 1.44Mg/m3 and 1.685Mg/m3 and optimum moisture content may fall between 20-30%. For silty clay maximum dry density is between 1.6Mg/m3 and 1.845Mg/m3 and optimum moisture content ranges between 15- 25%. For sandy clay maximum dry density usually ranges between 1.75 and 2.165Mg/m3 and optimum moisture content between 8 and 15%. Looking at the results of the soil samples, it could be deduced that they are clay.

**Table 4.6**- Summary of the compaction results

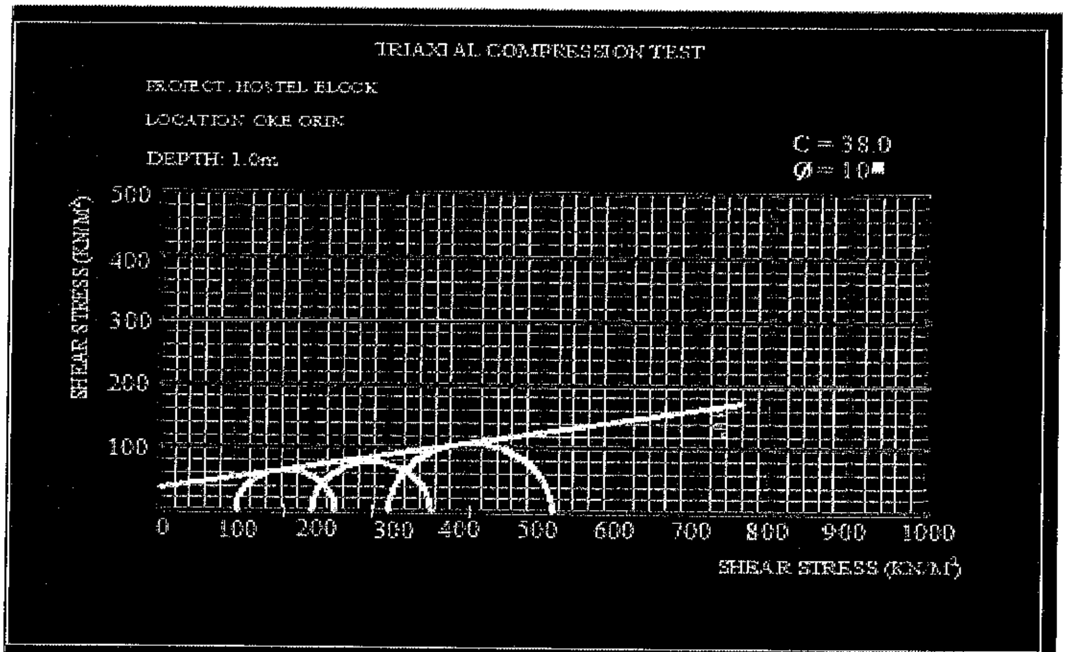
| SAMPLE LOCATION | ASIN | CAMPUS | IKOYI | OKE-ORIN |
|---|---|---|---|---|
| Optimum Moisture Content (%) | 19.60 | 22.90 | 22.2 | 24.00 |
| Maximum Dry Density kN/m3 | 1.78 | 1.67 | 1.56 | 1.65 |

**5. Triaxial Test**

The unconfined compressive test was conducted on the undisturbed soil samples for oke-orin and campus soils. The Figures below shows the behavior of the soil samples for the test. The unconfined compressive strength for the soil samples are 159.20 and 130.6 kN/m2. This shows that the shear strength of the soil samples are good and these materials can be used as either a sub-base or sub-grade material but best when stabilized.

**Table 4.7**- Summary of the triaxial test

| | C | Ø |
|---|---|---|
| **CAMPUS SOIL** | **37.00** | **18.00 °** |
| **IKOYI SOIL** | **38.00** | **10.00 °** |



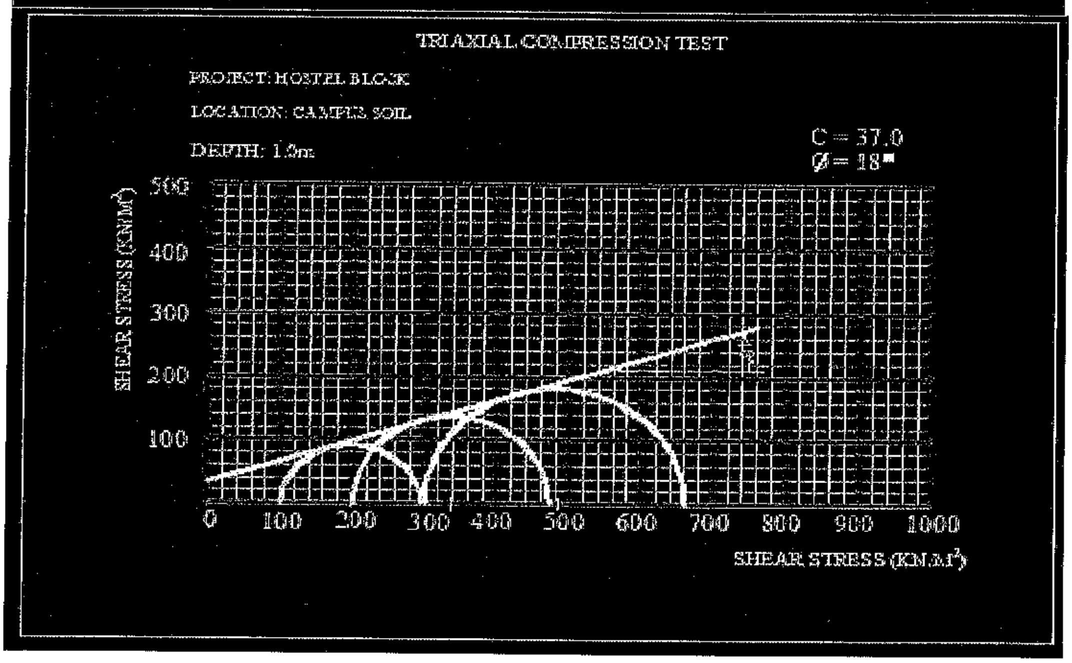




**Figure 4.11**- Graph for Triaxial test

## 4.1 Classification of the Soil Samples

According to AASHTO soil classification samples at Asin can be classified as A-2-7 materials which contain mostly silt and sand and is good as a subgrade material, and Campus soil can be classified as A-7-6 material which consist mainly of clayey soils and is poor for sub-base but good as a subgrade material. While samples at Ikoyi and Oke-Orin can be classified as A-2-6 and A-2-7 materials respectively which is rated good as a sub-base material and excellent as a subgrade material.

**Table 4.8**- Summary table

| SAMPLES LOCATION | | A ASIN | B CAMPUS | C IKOYI | D OKE-ORIN |
|---|---|---|---|---|---|
| Sieve Analysis | 2.36 | 57.20 | 92.80 | 91.50 | 82.50 |
| | 0.6 | 63.40 | 76.80 | 53.90 | 64.90 |
| | 0.0075 | 34.00 | 44.80 | 21.90 | 31.20 |
| Atterberg Limit | LL% | 43.50 | 43.43 | 32.48 | 43.65 |
| | PL% | 24.40 | 23.40 | 17.30 | 26.80 |
| | PI% | 14.10 | 20.02 | 15.18 | 16.85 |
| | SL% | 9.20 | 10.7 | 8.60 | 14.60 |
| Natural Moisture Content (%) | | 17.75 | 15.00 | 14.30 | 14.60 |
| Compaction Test | OMC (%) | 19.60 | 22.90 | 22.2 | 24.00 |
| | MDD kN/m3 | 1.78 | 1.67 | 1.56 | 1.65 |
| CBR | 2.5mm | 75.50 | 34.50 | 36.70 | 67.22 |
| | 5.0mm | 75.68 | 40.10 | 52.10 | 72.64 |
| Specific Gravity | | 2.35 | 2.42 | 2.59 | 2.59 |
| AASHTO Classification | | A-2-7(2) | A-7-6(3) | A-2-6(0) | A-2-7(1) |

# CHAPTER 5
# CONCLUSION AND RECOMMENDATION

## 5.1 Conclusion

Subsequent upon the tests carried out on the soil samples the following conclusion can be made:

1. The sieve analysis shows that only samples taken at Oke-orin and Ikoyi has less than 35% passing 200BS sieve while samples from Campus and Asin has more than 35% passing 200BS sieve.
2. The CBR test shows that the entire studied soil most of the specified values required by the Federal ministry of works general specification requirements for roads and bridges (1994).
3. Samples from Asin, Ikoyi and Oke-orin contain mostly silt and sand which can be used as a subgrade, sub-base and if well stabilized can be used as base materials.
4. Sample from Campus location is a clayey soil which cannot be used as a subbase material except if stabilized.

This research work has:

1. Provided data for engineers, planners, designers and contractors; and
2. Prevented possible difficulties, delays and additional expenses during construction due to inadequate geotechnical information.

## 5.2 Recommendation

In order to prevent constant reoccurrence of road failure, there is need to properly evaluate the engineering properties of the materials used for road construction such as laterite, asphalt etc. The values derived from this research will help engineers appreciate the use of lateritic soils for road construction. It is therefore important for Engineers, government bodies and private construction firms to be informed of present conditions soils used for road construction especially lateritic soils. After adequate conclusion drawn from this research I therefore recommend that:

1. The soils found in these research locations should be stabilized before road construction use to ensure maximum road strength after road construction

2. Further research should be conducted on lateritic soils around the country to create awareness about the effects, uses and performance based specification of lateritic soils.

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