GEOTECHNICAL INVESTIGATION OF CAUSES OF ROAD FAILURES IN IKOLE AREA

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(CVE/12/0831)

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ABSTRACT

The condition and adequacy of the highway is often judged by the smoothness or roughness of the pavement, deficient pavement conditions can result in increased user costs and travel delays, unnecessary braking and fuel consumption, vehicle repairs and probability of increased crashes. Pavement failure is usually a result of several factors which includes traffic impacts on the road, environmental factors, poor maintenance culture, geotechnical characteristics of the subgrade and other road construction material, to mention a few. In this study, four failed segments of roads in the Ikole local government area were selected for geotechnical studies. The following processes were carried out: reconnaissance survey, interview sessions with relevant parties, site observations and geotechnical laboratory tests of road subgrade soil in accordance to B.S. 1377. The geotechnical investigation involved grain size analysis, atterbergs limits, West African compaction test, specific gravity determination, natural moisture content determination, permeability tests and california bearing ratio (CBR) test. Compaction test showed a maximum dry density range of 2.0kg/m^3 to 1.86kg/m^3 and an optimum moisture content range between 10.5% and 15%. The values of the California Bearing Ratio (CBR) of un-soaked soil samples were within the range of 38.4% and 80.8%. Furthermore, the sieve analysis revealed that a substantial percentage of the soil samples passed through the No. 200 BS sieve suggesting the soil consist mostly of silty clayey material, which translates to a fair material rating according to AASHTO design standard (1986). The permeability test result indicates the samples are semi-permeable soils. From the tests result analysis of the Ikole-Omuo road stretch within the Ikole local Government area it appears the subgrade materials under the failed and unfailed segments of the road is suitable according to AASHTO design standard. Therefore, It is imperative that sustainable solutions addressing the issue of traffic impacts, adequate drainage provision, environmental factors and quality control should be focused on, to curtail the incessant occurrence of road failures on the Ikole Omuo road stretch.

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I would like to express my sincere gratitude to my project supervisor "Prof. J.B Adeyeri" for giving me the opportunity to work on this topic. It would never be possible for me to take this project to this level without his innovative ideas and his relentless support and encouragement.

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DEDICATION

I dedicate this project to GOD almighty my creator, my strong pillar, my source of inspiration, wisdom, knowledge, and understanding. I also dedicate this project to my parents Mr. Abayomi Omosebi and Mrs Deborah Omosebi for their undying supports and encouragement. Finally, I dedicate this project to everyone who played a part in ensuring it was a success.

CERTIFICATION

This is to certify that this project entitled "GEOTECHNICAL INVESTIGATION OF CAUSES OF ROAD FAILURES IN IKOLE AREA" by OMOSEBI ISAAC ANUOLUWAPO (CVE/12/0831), submitted in partial fulfillment of the requirements for the degree of Bachelor of Engineering (B. ENG.) in CIVIL ENGINEERING of Federal University Oye-Ekiti, during the academic year 2012-2017, is a bonafide record of work carried out under my guidance and supervision.

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CHAPTER ONE

INTRODUCTION

1.1 General Background

Generally, every work of construction in civil engineering is built on soil or rock and in many instances, they are used as raw materials for construction of infrastructures, such as buildings, earth dams, liners and covers for landfills and road pavement. Therefore, a good understanding of the properties of the soil and its behavior under load before usage is highly essential in civil engineering and road pavement construction according to Agbede and Osuolale, (2005).

Failure of road pavement is a major experience that occurs on Nigeria roads. The failure of road pavements is dated as far back as to the colonial period, Chukweze, (1988), Jegede, (2004) and Pollit, (1950). The failures had been attributed to some factors, such as properties of construction materials, subgrade conditions, environmental conditions, traffic loading, lack of drainage and poor workmanship according to Arumala, and Akpokodje, (1987); Jegede, (2004); Madedor, (1992); Ogundipe (2008).

In recent times, road pavement failure has been a very serious problem that cause unnecessary delay in traffic flow, distorts pavement aesthetics, breakdown of vehicle and most significantly, causes road traffic accident that has resulted into loss of lives and properties amounting to millions of Naira, Jegede (2000) and Ogundipe (2008).

The impact caused by the lack of proper attention being paid to roads in Nigeria is further accentuated by a report given by the Federal Road Safety Corps recently as researched by World Health Organization: out of 192 countries ranked, Nigeria came 191 in number of deaths caused by road accidents, coming as the second worst country in the world. It states further that 162 people die from road accidents from every 100,000 Nigerian, this is a pandemic. This report indicates that road accident is gradually overtaking deaths from Malaria and Tuberculosis and it is fast consuming the human resources we have at our disposal. Hence, there is a lot of concern about the state of disrepair of all categories of roads and the need to reappraise the construction materials and method used on roads within the country in order to check and overcome all the results of road pavement failures already mentioned.

Investigations had been conducted on possible causes of road failures in Nigeria, especially in Ekiti, Ondo, and some parts of Osun State, Jegede, (2000); Jegede, (2004); Ogundipe, (2008) but there is insufficient research on the geotechnical investigation of the causes of road failure in Ikole area.

1.2 Description of Study Area

Geographically, Ikole Local Government is entirely within the tropics. It is located between longitude 5°30'52.17" East of Greenwich and latitude 7°47'53.76" North of the Equator (distancetos.com). Its neighbours are Kwara State to the North, Kogi State to the North east, Ekiti East to the East, Gboyin Local Government in the South and Oye Local government in the West as shown in fig 1.1. The headquarters of the local government, Ikole Ekiti which shall be considered in this research work is about 48.7 kilometres from Ado – Ekiti, the Ekiti State capital and 27.2 km from Omuo (a town located at the end of the Ikole – Omuo road stretch with longitude 5°43'28.98" East of Greenwich and latitude 7°45'46.85" North of the Equator, sourced from distancetos.com). The local government is mainly on the upland zone rising to about 250 metres above the sea level.

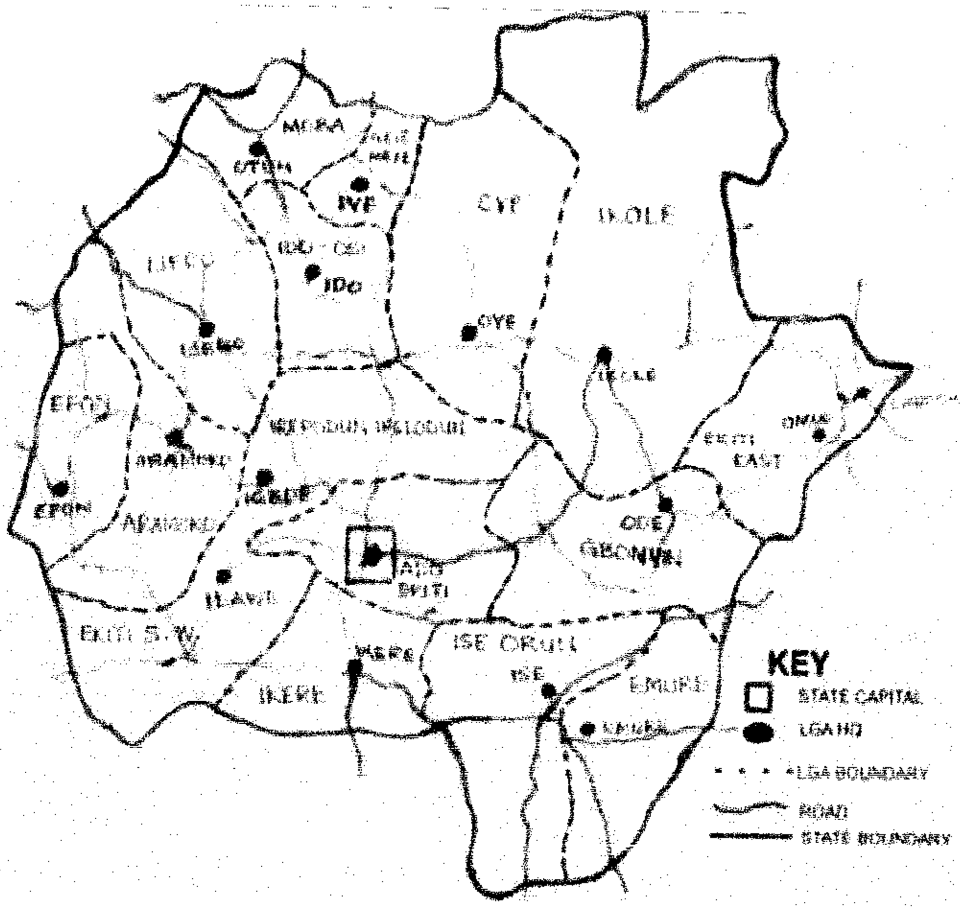

Figure 1.1 Map showing Ikole area and its surrounding local government

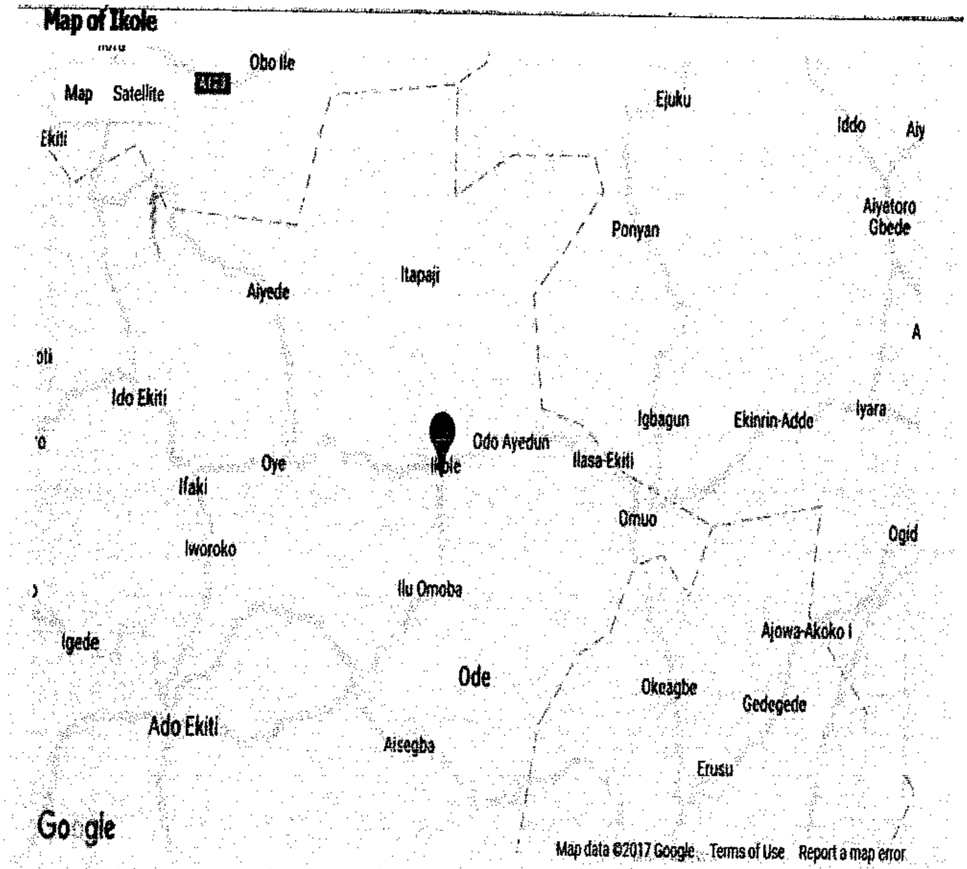

Figure 1.2 Map showing Ikole-Omuo road (Googlemap)

The Local Government occupies an area of about 374,940kms^2 of land and according to the 2006 National Population Census figure, the total population of the local government was 168,436; Male: 87,976; Female: 80,460 (ekitistate.org.ng, 2017).

1.3 Problem Statement

In Nigeria today, the issue of road accidents has become a persistently enduring problem. It has been found that factors responsible for most of the failures experienced on our roads are usually: poor design and construction, heavy traffic, poor maintenance culture, poor highway facilities, use of low quality materials, poor workmanship, poor supervision, low knowledge base, lack of local standard of practice, inadequate sanctions for highway failures. Geotechnical factors from researches around the world have also been found to be an underlying contributing factor which is often overlooked. A sound knowledge of how much impact these geotechnical factors have on the overall serviceability of asphaltic roads can help in making better decisions regarding road design and maintenance.

1.4 Justification of the Study

The study is considered very important, as it will investigate the geotechnical property of soils around Ikole area, which will in turn provide a basis for the determination the actual causes or causes of pavement failures in Ikole area.

1.5 Aim and Objectives

1.5.1 Aim of the Study

The purpose of the research work is to investigate the geotechnical causes of asphaltic road failures in Ikole area particularly the Ikole-Omuo road stretch.

1.5.2 Objectives of the Study

i. To conduct a reconnaissance survey of the Ikole Omuo road stretch
ii. To obtain soil samples at specific failed segments of the road stretch
iii. To subject the soil samples to the following geotechnical tests: specific gracity test, natural moisture content test, permeability test, West Africa compaction test, grain size distribution test, CBR test, and Atterbergs Limit test.
iv. To draw informed conclusion and give recommendations based on the results obtained from the tests

1.6 Scope and Limitation of The Study

The samples of disturbed soils were collected from four (4) locations namely: Kota/Omuo –Ekiti, Ilasa-Ekiti, Ayebode-Ekiti and Ikole-Ekiti all within the Ikole-LGA and will be subjected to the following tests;

a) Permeability test;
b) Specific gravity;
c) Sieve analysis;
d) Consistency/Atterbergs Limit test;
e) Natural Moisture Content;
f) West African Compaction test;
g) California Bearing Ratio (CBR) test.

The study is a geotechnical investigation of the causes of road failures in Ikole area of Ekiti state. The research is limited to only Ikole-Omuo road.

CHAPTER TWO

LITERATURE REVIEW

2.1 Road Pavement

A road pavement is a structure of superimposed layers of selected and processed materials that is placed on the basement soil or subgrade. The main structural function of a pavement is to support the wheel loads applied to the carriageway and distribute them to the underlying subgrade. The term subgrade is normally applied to both the in-situ soil exposed by excavation and to added soil that is placed to form the upper reaches of an embankment.

Modern pavement design is concerned with developing the most economical combination of pavement layers that will ensure that the stresses and strains transmitted from the carriageway do not exceed the supportive capacity of each layer, or of the subgrade and it can withstand the action of the climate with minimal deterioration during the design life of the road. Major variables affecting the design of a given pavement are therefore the volume and composition of traffic, the subgrade environment and strength, the materials economically available for use within the pavement layers, and the thickness of each layer. For discussion purposes, pavements can be divided into two broad types; flexible pavements and rigid pavements (see Fig 2.1).

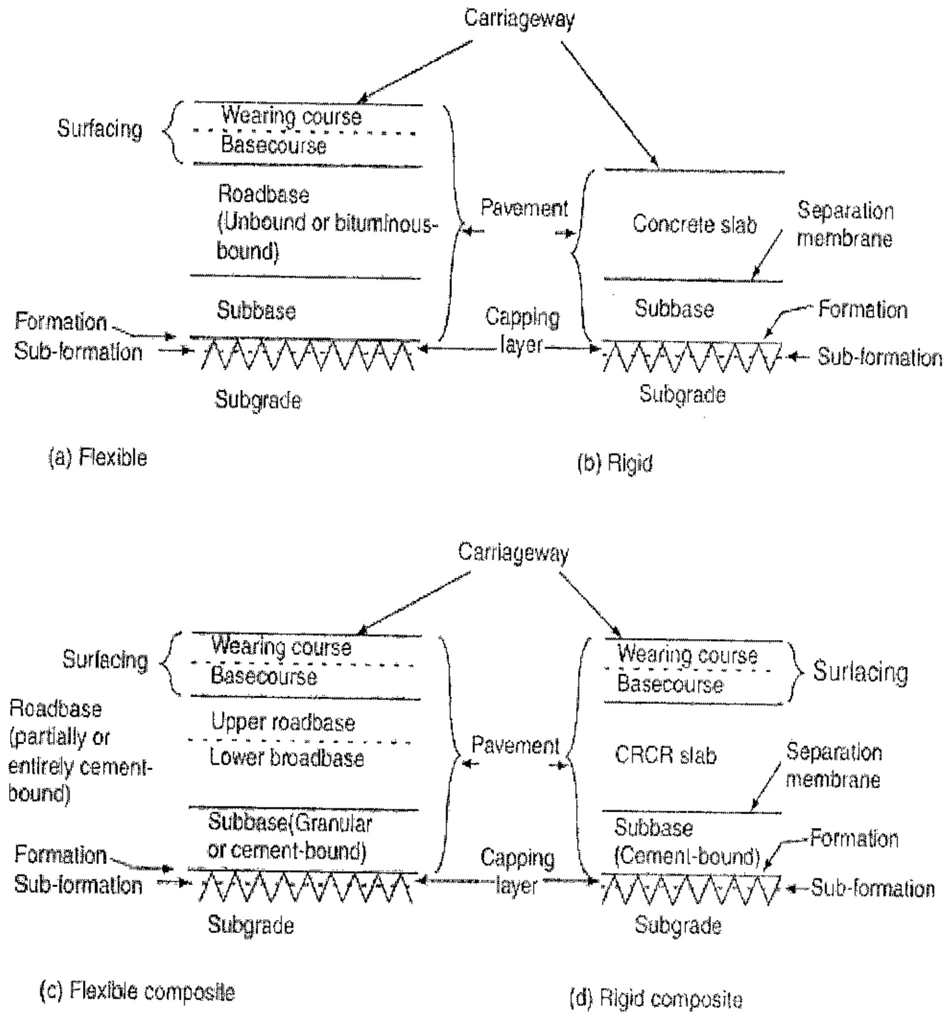

Figure 2.1 Basic Elements of Flexible and Rigid Types of Pavement

2.2 Flexible Pavements

Highway pavements are divided into two main categories namely: rigid and flexible. But the concern of this study is flexible pavements. Flexible pavements usually consist of a bituminous surface underlaid with a layer of granular material and a layer of a suitable mixture of coarse and fine materials. Traffic loads are transferred by the wearing surface to the underlying supporting materials through the interlocking of aggregates, the frictional effect of granular materials, and cohesion of fine materials.

Flexible pavements are further divided into three subgroups: high type, intermediate type, and low type. High-type pavements have wearing surfaces that adequately support the expected traffic load without visible distress due to fatigue and are not susceptible to weather conditions. Intermediate-type pavements have wearing surfaces that range from surface treated to those with qualities just below that of high type pavements. Low-type pavements are used mainly for low-cost roads and have wearing surfaces that range from untreated to loose natural materials to surface treated earth.

2.2.1 Structural Component of a Flexible Pavement

1. **Subgrade (Prepared Road Bed)**: The subgrade is usually the natural material located along the horizontal alignment of the pavement and serves as the foundation of the pavement structure. It also may consist of a layer of selected borrow materials, well compacted to prescribed specifications. It may be necessary to treat the subgrade material to achieve certain strength properties required for the type of pavement being constructed.
2. **Subbase Course**: This is located immediately above the subgrade, the subbase component consists of material of a superior quality to that which is generally used for subgrade construction. The requirements for subbase materials usually are given in terms of the gradation, plastic characteristics, and strength. When the quality of the subgrade material meets the requirements of the subbase material, the subbase component may be omitted. In cases where suitable subbase material is not readily available, the available material can be treated with other materials to achieve the necessary properties. This process of treating soils to improve their engineering properties is known as stabilization.

3. **Base Course**: The base course lies immediately above the subbase. It is placed immediately above the subgrade if a subbase course is not used. This course usually consists of granular materials such as crushed stone, crushed or uncrushed slag, crushed or uncrushed gravel, and sand. The specifications for base course materials usually include more strict requirements than those for subbase materials, particularly with respect to their plasticity, gradation, and strength. Materials that do not have the required properties can be used as base materials if they are properly stabilized with Portland cement, asphalt, or lime. In some cases, high-quality base course materials also may be treated with asphalt or Portland cement to improve the stiffness characteristics of heavy-duty pavements.
4. **Surface Course**: The surface course is the upper course of the road pavement and is constructed immediately above the base course. The surface course in flexible pavements usually consists of a mixture of mineral aggregates and asphalt. It should be capable of withstanding high tire pressures, resisting abrasive forces due to traffic, providing a skid resistant driving surface, and preventing the penetration of surface water into the underlying layers. The thickness of the wearing surface can vary from 3 in. to more than 6 in., depending on the expected traffic on the pavement. (Garber & Hoel, 2009)

2.3 Soils in Pavement Design

Soil is defined as a natural aggregate of mineral grains, with or without organic constituents, that can be separated by gentle mechanical means such as agitation in water, V.N.S Murthy. Soil is mainly formed by weathering and other geologic processes that occur on the surface of the solid rock at or near the surface of the earth. Weathering is the result of physical and chemical actions, mainly due to atmospheric factors that change the structure and composition of the rocks. Weathering occurs through either physical or chemical means. Physical weathering, sometimes referred to as mechanical weathering, causes the disintegration of the rocks into smaller particle sizes by the action of forces exerted on the rock. These forces may be due to running water, wind, freezing and thawing, and the activity of plants and animals. Chemical weathering occurs as a result of oxidation, carbonation, and other chemical actions that decompose the minerals of the rocks.

2.3.1 General Types of Soils

It has been discussed earlier that the process of physical and chemical weathering forms soil. The individual size of the constituent parts of even the weathered rock might range from the smallest state (colloidal) to the largest possible (boulders). This implies that all the weathered constituents of a parent rock cannot be termed soil. According to their grain size, soil particles are classified as cobbles, gravel, sand, silt and clay. Grains having diameters in the range of 4.75 to 76.2 mm are called gravel. If the grains are visible to the naked eye, but are less than about 4.75 mm in size the soil is described as sand. The lower limit of visibility of grains for the naked eyes is about 0.075 mm. Soil grains ranging from 0.075 to 0.002 mm are termed as silt and those that are finer than 0.002 mm as clay. This classification is purely based on size which does not indicate the properties of fine grained materials.

2.3.2 Residual and Transported Soils

On the basis of origin of their constituents, soils can be divided into two large groups:

1. Residual soils, and
2. Transported soils.

Residual soils are those that remain at the place of their formation as a result of the weathering of parent rocks. The depth of residual soils depends primarily on climatic conditions and the time of exposure. In some areas, this depth might be considerable. In temperate zones residual soils are commonly stiff and stable. An important characteristic of residual soil is that the sizes of grains are indefinite. For example, when a residual sample is sieved, the amount passing any given sieve size depends greatly on the time and energy expended in shaking, because of the partially disintegrated condition.

Transported soils are soils that are found at locations far removed from their place of formation. The transporting agencies of such soils are glaciers, wind and water. The soils are named according to the mode of transportation. Alluvial soils are those that have been transported by running water. The soils that have been deposited in quiet lakes are lacustrine soils. Marine soils are those deposited in seawater. The soils transported and deposited by wind are aeolian soils. Those deposited primarily through the action of gravitational force, as in land slides, are colluvial soils. Glacial soils are those deposited by glaciers. Many of these transported soils are loose and soft to a depth of several hundred feet. Therefore, difficulties with foundations and other types of construction are generally associated with transported soils.

2.3.3 Organic and Inorganic Soils

Soils in general are further classified as organic or inorganic. Soils of organic origin are chiefly formed either by growth and subsequent decay of plants such as peat, or by the accumulation of fragments of the inorganic skeletons or shells of organisms. Hence a soil of organic origin can be either organic or inorganic. The term organic soil ordinarily refers to a transported soil consisting of the products of rock weathering with a more or less conspicuous admixture of decayed vegetable matter.

2.3.4 Names of Some Soils that are Generally Used in Practice

Bentonite: This is a clay formed by the decomposition of volcanic ash with a high content of montmorillonite. It exhibits the properties of clay to an extreme degree.

Varved Clays: These consist of thin alternating layers of silt and fat clays of glacial origin. They possess the undesirable properties of both silt and clay. The constituents of varved clays were transported into fresh water lakes by the melted ice at the close of the ice age.

Kaolin: These are very pure forms of white clay used in the ceramic industry.

Boulder Clay: This is a mixture of an unstratified sedimented deposit of glacial clay, containing unsorted rock fragments of all sizes ranging from boulders, cobbles, and gravel to finely pulverized clay material.

Calcareous Soil: This is a soil containing calcium carbonate. Such soil effervesces when tested with weak hydrochloric acid.

Marl: This consists of a mixture of calcareous sands, clays, or loam.

Hardpan: This is a relatively hard, densely cemented soil layer, like rock, which does not soften when wet.

Caliche: This is an admixture of clay, sand, and gravel cemented by calcium carbonate deposited from ground water.

Peat: This is a fibrous aggregate of finer fragments of decayed vegetable matter. Peat is very compressible and one should be cautious when using it for supporting foundations of structures.

Loam: This is a mixture of sand, silt and clay.

Loess: This is a fine-grained, air-borne deposit characterized by a uniform grain size, and high void ratio. The size of particles ranges between about 0.01 to 0.05 mm. The soil can stand deep vertical cuts because of slight cementation between particles. It is formed in dry continental regions and its color is yellowish light brown.

Shale: This is a material in the state of transition from clay to slate. Shale itself is sometimes considered a rock but, when it is exposed to the air or has a chance to take in water it may rapidly decompose.

2.3.5 Soil Classification for Highway Use

Different soils with similar properties may be classified into groups and sub-groups according to their engineering behavior. Classification systems provide a common language to concisely express the general characteristics of soils, which are infinitely varied, without detailed descriptions. Most of the soil classification systems that have been developed for engineering purposes are based on simple index properties such as particle-size distribution and plasticity. Although several classification systems are now in use, none is totally definitive of any soil for all possible applications because of the wide diversity of soil properties, Garber & Hoel (2009).

Soils are classified as follows;

i. **Textural Classification**: In a general sense, texture of soil refers to its surface appearance. Soil texture is influenced by the size of the individual particles present in it. In most cases, natural soils are mixtures of particles from several size groups. In the textural classification system, the soils are named after their principal components, such as sandy clay, silty clay, and so forth.

ii. **Classification by Engineering Behaviour**: Although the textural classification of soil is relatively simple, it is based entirely on the particle-size distribution. The amount and type of clay minerals present in fine-grained soils dictate to a great extent their physical properties. Hence, the soils engineer must consider plasticity, which results from the presence of clay minerals, to interpret soil characteristics properly. Because textural classification systems do not take plasticity into account and are not totally indicative of many important soil properties, they are inadequate for most engineering purposes. Currently, two more elaborate classification systems are commonly used by soils engineers. Both systems take into consideration the particle-size distribution and Atterberg limits. They are the American Association of State Highway and Transportation Officials (AASHTO) classification system and the Unified Soil Classification System. The AASHTO classification system is used mostly by state and country highway departments. Geotechnical engineers generally prefer the Unified system. The AASHTO classification system will be employed in this study.

2.3.5.1 AASHTO Classification System

The AASHTO system of soil classification was developed in 1929 as the Public Road Administration classification system.

The AASHTO classification in present use is given in Table 2.1. According to this system, soil is classified into seven major groups: A-1 through A-7. Soils classified under groups A-1, A-2, and A-3 are granular materials of which 35% or less of the particles pass through the No. 200 sieve. Soils of which more than 35% pass through the No. 200 sieve are classified under groups A-4, A-5, A-6, and A-7. These soils are mostly silt and clay-type materials. This classification system is based on the following criteria:

1. Grain size
   a. Gravel: fraction passing the 75-mm (3-in.) sieve and retained on the No. 10 (2-mm) U.S. sieve
   b. Sand: fraction passing the No. 10 (2-mm) U.S. sieve and retained on the No. 200 (0.075-mm) U.S. sieve
   c. Silt and clay: fraction passing the No. 200 U.S. sieve
2. Plasticity: The term silty is applied when the fine fractions of the soil have a plasticity index of 10 or less. The term clayey is applied when the fine fractions have a plasticity index of 11 or more.
3. If cobbles and boulders (size larger than 75 mm) are encountered, they are excluded from the portion of the soil sample from which classification is made. However, the percentage of such material is recorded.

To classify a soil according to Table 2.1, one must apply the test data from left to right. By process of elimination, the first group from the left into which the test data fit is the correct classification. Figure 2.2 shows a plot of the range of the liquid limit and the plasticity index for soils that fall into groups A-2, A-4, A-5, A-6, and A-7. To evaluate the quality of a soil as a highway subgrade material,one must also incorporate a number called the group index (GI) with the groups and subgroups of the soil. This index is written in parentheses after the group or subgroup designation. The group index is given by the equation:

$\boldsymbol{GI = (F_{200} - 35)[0.2 + 0.005(LL - 50) + 0.01(F_{200} - 15)(PI - 10)]}$.....Equation 5.1

Where, G.I = Group Index

F_{200} = percentage passing the no. 200 sieve

LL = Liquid limit

PI = Plasticity Index

Table 2.1 AASHTO Classification of Highway Subgrade Materials

General classification	Granular materials (35% or less of total sample passing No. 200)						
	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 (percentage passing)							
No. 10	50 max.						
No. 40	30 max.	50 max.	51 min.				
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.
Characteristics of fraction passing No. 40							
Liquid limit				40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand			
General subgrade rating	Excellent to good						

General classification	Silt-clay materials (more than 35% of total sample passing No. 200)			
Group classification	A-4	A-5	A-6	A-7 A-7-5[a] A-7-6[b]
Sieve analysis (percentage passing)				
No. 10				
No. 40				
No. 200	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40				
Liquid limit	40 max.	41 min.	40 max.	41 min.
Plasticity index	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Silty soils		Clayey soils	
General subgrade rating	Fair to poor			

[a] For A-7-5, $PI \leq LL - 30$

[b] For A-7-6, $PI > LL - 30$

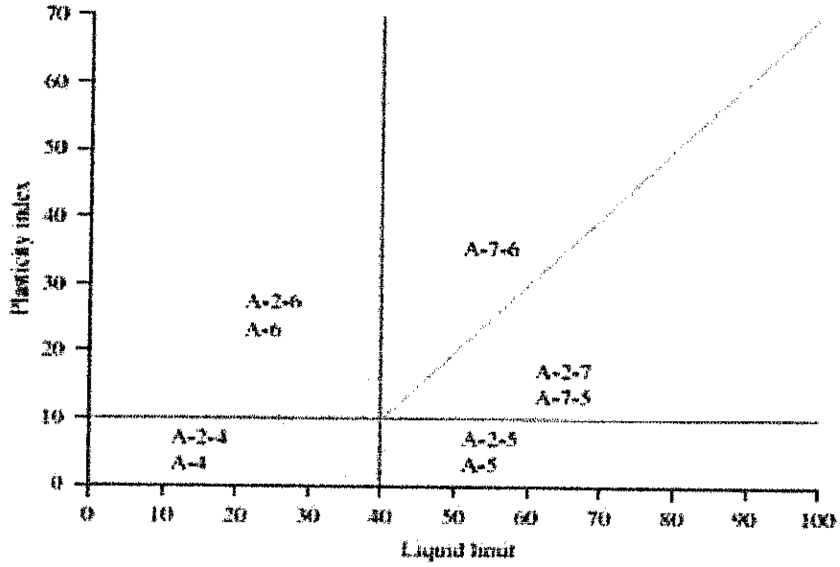

Figure 2.2 Range of liquid limit and plasticity index for soils in groups A-2, A-4, A-5, A-6, and A-7

2.4 Flexible Pavement Failure

Flexible pavement failure occurs when an asphalt surface no longer holds its original shape and develops material stress as mentioned by Lone Star Paving (2017) in an online article and this in turn results in failure noticeable as rutting, shoving, cracking, corrugation, settlement, etc. Okikbo in a journal published in 2012 referred to pavement failure as the visible evidence of undesirable condition in the pavement affecting the service ability, structural condition or appearance. Every vehicle, which passes over a road, causes a momentary, very small, but significant deformation of the road pavement structure. The passage of many vehicles has a cumulative effect, which gradually leads to permanent deformation and road surface deterioration Osadebe (2013).

There are essentially two ways in which pavement failure occurs, it may occur during;

1. Construction
2. Maintenance and Installation of Services

According to Woods and Adcox (2002), pavement failure was considered to be namely:

i. **Structural failure**

This is the loss of load carrying capability, where the pavement is no longer able to absorb and transmit the wheel loading through the fabric of the road without causing further deterioration.

ii. **Functional failure**

This is a broader term, which may indicate the loss of any function of the pavement such as skid resistance, structural capacity, and serviceability or passenger comfort.

iii. **Materials failure**

This occurs due to the disintegration or loss of material characteristics of any of the component materials.

2.5 Types of Flexible Pavement Failure

The common types of distresses in bituminous pavement are classified into the following four major groups namely;

2.5.1 Surface Deformation

The surface deformation occurs usually due to failure or weakness in one of the layers of the pavement due to traffic movement after construction. The common types of surface deformation includes the following, Kadyali and Lal, (2012);

a) **Corrugations:** Corrugation is a form of bituminous pavement distress which usually occurs due to formation of regular and shallow undulations in the form of ripples or small corrugations of depth up to 25mm on the bituminous surface or across the road on some stretches. The probable causes of corrugation include the following; Kadyali and Lal, (2012);

i. Lack of stability in the bituminous mix
ii. Excess binder content in the bituminous mix
iii. Excess proportion of fines in the mix
iv. Use of binder of low viscosity with respect to the temperature of the region
v. Faulty laying of surface course

b) **Rutting**: This is the longitudinal deformation or depression of the pavement surface along the wheel path of heavy vehicles formed due to repeated applications of heavy load along the same wheel path resulting in cumulative non-recoverable or pavement deformation of the pavement layers including subgrade and one or more of the pavement layers. The various causes of rutting may be summed up as;

   i. Inadequate stability of the subgrade or sub-base or base course or surface course or few of these pavement layers.
   ii. Inadequate compaction of the subgrade or any of the pavement layers
   iii. Channelized movement of heavy wheel loads causing significant vertical stress on the subgrade Improper design and specification of bitumen mix
   iv. Inadequate thickness of the pavement or weak pavement structure and the possible remedial measures for this type of distress include
   v. Cleaning the affected surface
   vi. Application of tack coat and covering the ruts
   vii. Filling the ruts using either a dense graded bituminous mix or open graded pre-mix followed by seal coat
   viii. Compaction by rolling
   ix. Providing a thin bituminous resurfacing course to achieve good riding quality.

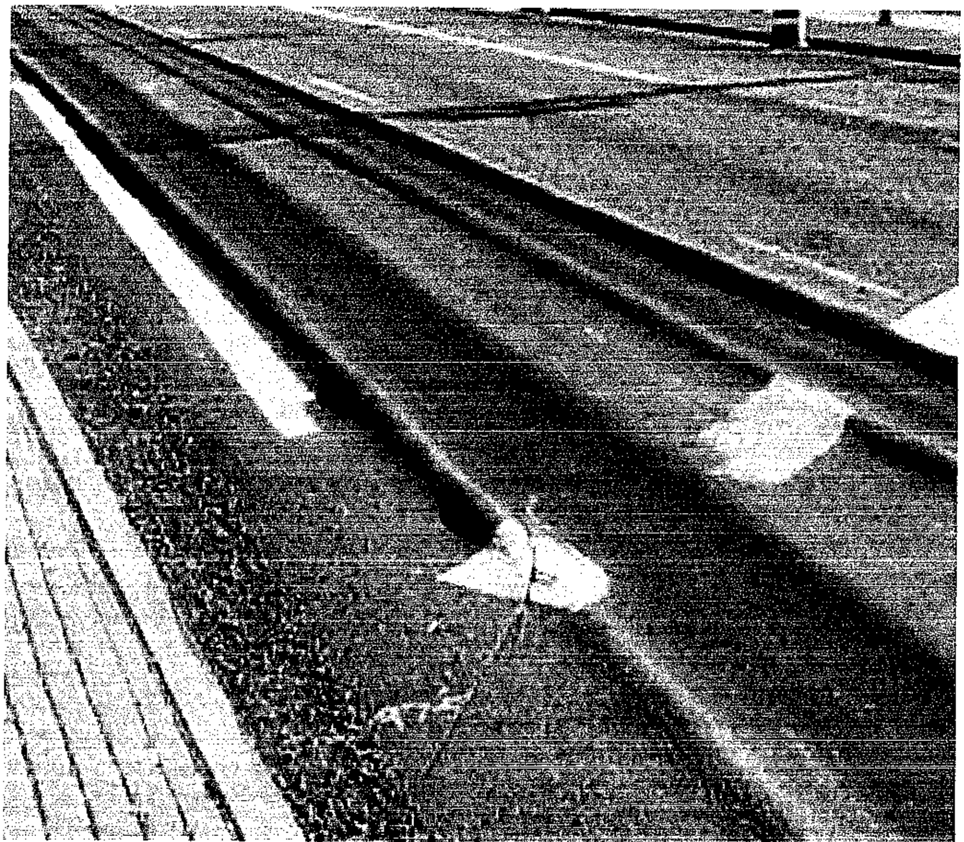

Plate 2.1 Rutting on a road, Smith (2004)

c) **Shoving**: Shoving is a form of plastic movement resulting in a localized bulging of the pavement surface. Shoving can take a number of different forms such as upheaval, "wash boarding" or ripples across the pavement surface, or even a crescent –shaped bulging.

The causes of shoving include the following;

i. Lack of stability in the bituminous mix
ii. Too much binder content in the hot mix
iii. Use of rounded and smooth textured aggregate particles in the mix
iv. Excess proportion of fines in the mix

Plate 2.2 Shoving on a trafficked road, (Smith, 2004)

d) **Shallow depressions:** Shallow depression are small localized bowl shaped area that may include cracking. Depressions usually causes the roughness on the bituminous pavement surface and are hazardous to automobiles, and they also allow collection of water on the pavement surface. The probable causes of depressions are the presence of inadequate compacted pocket or rather a localized consolidation or movement of the supporting layers beneath the surface course due to instability.

e) **Settlement and Upheaval:** Settlement and upheaval occurs due to large deformation of the pavement surface caused by expansion of the supporting layers beneath the surface course or the subgrade The causes due to this kind of distress include; poor compaction of fills, poor drainage, inadequate pavement or frost heave.

2.5.2 Cracking

The presence of surface cracks significantly reduces the life of bituminous pavements. This is because the surface cracks are one of the main contributors to the development of other different types of cracks in bituminous layers. They accelerate the development of cracking which would ultimately lead to early failure of the pavement (Halim et al., 1993). The common types of cracks include the following; (a) Fatigue cracking (b) Transverse cracking (c) Longitudinal cracking (d) Edge cracking and (e) Reflective cracking.

a) **Fatigue Cracking:** Fatigue cracks are a series of longitudinal and interconnected cracks caused by the repeated application of wheel loads. This type of cracking generally starts as short longitudinal cracks in the wheel path and progress to an alligator cracking pattern (Interconnected cracks) as shown in Plate 2.3 It happens due to repeated bending action of the hot mix asphalt HMA (surface layer) when the load is applied, this generates tensile stress that eventually creates cracks at the bottom of the asphalt layer. Cracks gradually propagates to the top of the asphalt layer and later progress and interconnect.

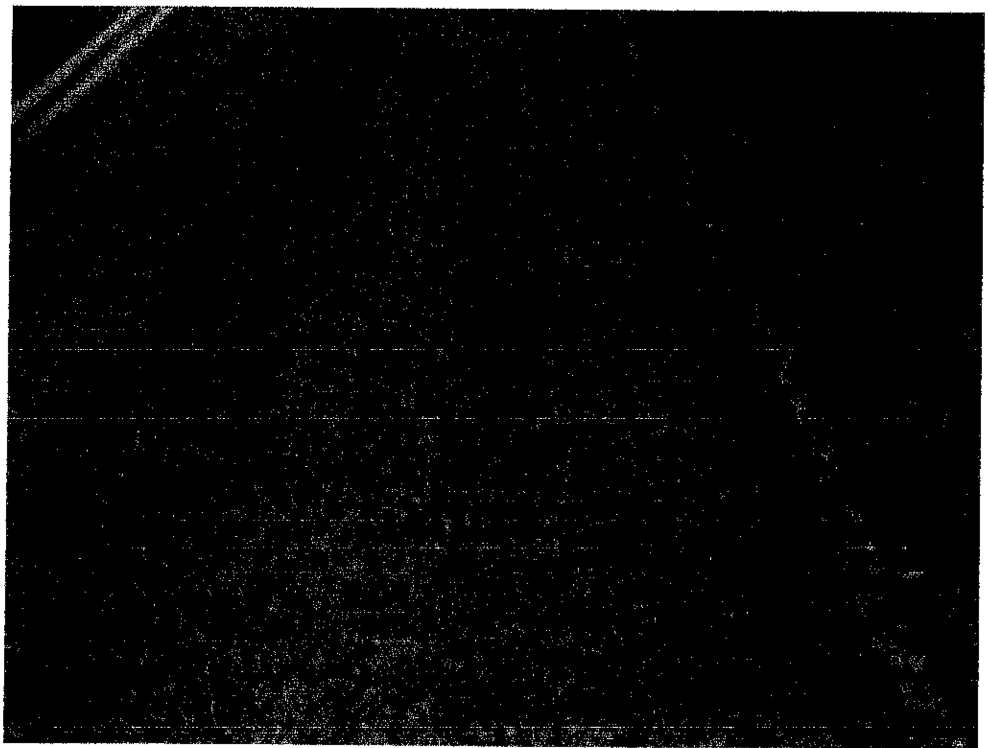

Plate 2.3 Fatigue cracking, Wada (2016)

b) **Transverse Cracking**: These are cracks perpendicular to the pavement centerline or lay down direction, they usually begin as hairline and widen with age. If not properly sealed and maintained, multiple cracks develop parallel to the initial crack.

c) **Longitudinal Cracking**: These are cracks parallel to the pavement centerline or lay down direction, which may eventually lead to moisture infiltration, roughness, and may indicate the possible onset of alligator cracking and structural failure. (c) Longitudinal Cracking: These are cracks parallel to the pavement centerline or lay down direction, which may eventually lead to moisture infiltration, roughness, and may indicate the possible onset of alligator cracking and structural failure.

The possible causes include poor drainage, shoulder settlement, weak joints between adjoining spread of pavement layers or differential frost heave.

d) **Edge Cracking**: Edge cracks typically starts as crescent shapes at the edge of the pavement. They will expand from the edge until they begin to resemble alligator cracking. They may occur in a curbed section when subsurface water causes weakness in the pavement. This type of cracking result from lack of support of shoulders due to weak materials or excess moisture. Frost heave and inadequate pavement width also contributes towards formation of this type of distress.

e) **Reflective Cracking**: Reflective cracking has been traditionally thought to initiate at the bottom of the lower pavement layers and then propagate to the surface (Abd El Halim ,1987) Reflective cracks generally develop in bituminous resurfacing or an overlay constructed over an existing cracked bituminous surface without resorting to appropriate measures.

The most common causative agents of this type of cracking are due to joints and cracks in the pavement layer underneath.

2.5.3 Disintegration

This the progressive breaking up of the pavement into small, loose pieces is called disintegration. The two most common types of disintegration are:

a) **Potholes**: Potholes are small, bowl-shaped depressions in the pavement surface that penetrate all the way through the hot mix asphalt (HMA) layer down to the base course. They generally have a sharp edges and vertical sides near the top of the hole.

Generally, potholes are the end result of fatigue cracking. As fatigue cracking becomes severe, the interconnected cracks creates small chunks of pavement which can be dislodged as vehicles pass over them. The remaining hole after the pavement chunk is dislodged is called a pothole.

Plate 2.4 Potholes (Wada, 2016)

b) **Patches**: An area of pavement that has been replaced with new materials to repair the existing pavement. A patch is considered a defect no matter how well they perform because it never completely meshes with the existing pavement nor is it structurally bound to it. The causes include the previous localized pavement deterioration that has been removed and patched, and also the utility cuts along the pavement.

2.5.4 Surface Defects

Surface defects are related to problems in the surface layer. The most common types of surface distress are:

a) **Raveling**: Raveling is the loss of material from the pavement surface as a result of insufficient adhesion between the asphalt, cement and the aggregate. Raveling typically tends to occur on an older pavement that have already oxidized. Raveling can be accelerated by traffic and other environmental conditions. A raveled pavement can be repaired with a wearing course or an overlay.

b) **Bleeding**: Bleeding occurs when the bituminous mix contains too much asphalt cement relative to the aggregates. In this case, the asphalt cement tends to bleed through the surface, hence, reducing the skid resistance of a pavement thereby making the pavement very slippery when wet, creating hazard to the road users. This problem is generally caused by the presence of excessive binder content in the mix and also using the binder with too low viscosity (too flowable) or an improperly applied seal coat. Bleeding occurs more often in hot weather when the bituminous binder is less viscous (more flowable) and the traffic forces the asphalt to the surface.

c) **Polishing**: Polishing is a failure mode of the pavement surface consisting of rough exposed aggregates which is caused by excess repeated traffic on an aging pavement system. It can result in a dangerous low friction surface with a decreased skid-resistance. Repair the surface by applying a skid-resistant slurry seal or a non-structural overlay.

2.6 Causes of Pavement Failures

It should be noted that the failure of road pavements is a product of both natural and anthropogenic factors. Abynayaka (1977) established that major factors responsible for road failures to include: poor road construction, poor road design, wrong clinic of construction material, collapse of drainage substructure. The Transport Road Research Laboratory (1991), argued that climatic factors can also affect the strength of road structure. Temperature fluctuations and acid rain attack on the base material of the road in water-logged area can weaken the sub-base of the road material as a result of capillary action, thereby reducing the supporting power of the road pavement. The various causes / causative factors identified in this review are as follows;

2.6.1 Geotechnical Characteristics of the Soil (Sub-grades), Subbases and Construction Materials.

Oglesby, Clarkson and Gary (1982) in their description of the characteristics of soils for highway pavements, denoted that it is very important to understand the basement soil (or sub-grade) and other materials used in construction of pavement structures for highways and other transportation facilities as the sub-grade is the supporting structure upon which the pavement surface and its special under courses rests and both the sub-grade and these special under courses (Base course, sub-base and wearing course) are of rock origin. According to them, the moisture content and moisture irrigation in soils is a function of the Geologic makeup (the sedimentology: texture and structure, porosity & permeability etc.) of the sub-grade soil and the moisture water content and migration characteristics of the soil mass and it's reaction to water affects it's strength and this is a function of its grain size and mineralogy.

Gidigasu (1983), in his review suggested that inadequate width of the shoulders which provide lateral support to the pavement would also lead to road failure especially when the shoulders are made of cohesive soils and worst still, when such soil are not properly compacted. Penetration of rain water during the wet season and higher water table weakens the material in the road pavement and during the dry season evaporation of soil water from the clayey shoulder material causes soil moisture suction under the road, both conditions tends to increase the deterioration of the pavement. Graham and Shields (1984) investigated the complex properties of postglacial clay at Winnipeg in Canada. After sampling and laboratory analysis, the geotechnical properties of this clay was confirmed to be troublesome causing major negative impacts on civil engineering structures and construction in the city. The identified problems include; house foundation movements, low stability of riverbanks, poor highway pavement performance, difficult excavation, and a high incidence of watermain breaks.

According to Akpokodje (1986), in his laboratory analysis of the soils within the entire length of the Port Harcourt —Enugu expressway classified them to consist of (1) concretionary laterite gravels, (2) non-lateritic tropical sandy/clayey soils which are gravelly in some places and (3) silty to fat clays formed from shales. The particle size distribution and the plasticity of the majority of the soils indicate that by standard acceptance specifications they are unsuitable for base materials. He stressed that although the pavement materials used for the expressway are inferior under conventional standards, mostly isolated rather than widespread pavement failures have so far occurred. Such failures are presumed to be more related to poor field compaction rather than the inferior quality of the construction materials. Also he noted that where the troublesome weathered shale forms the Sub-grade, severe pavement failure usually occurs. Arumala and Akpokodje (1987) in their investigation of the pavement conditions of roads in the Niger Delta and the geotechnical properties of the soil materials used in constructing them, an attempt to find permanent solutions to the recurrent widespread pavement failures in the region, most severe surface deformations, pavement cracking and failures occur in the seasonally flooded fresh/salt water swamps because of the high water table, poor drainage and the very fine-grained silty clays/clays used.

Okagbue and Uma (1988) in their geological and hydrogeological survey of a problematic section of the Port- Harcourt- Enugu expressway, covering Lokpaukwu, Lokpanta and Leru areas, were able to prove that the road problem is linked to the geological and hydrogeological conditions of the area. As it was evident that the problematic section of the road is built on a considerably jointed, fractured and weathered shale formation as a sub grade, at the foot of an escarpment having a high concentration of natural groundwater discharge resulting in increased groundwater storage, high water table, constant wetting of pavement Sub-grade and subsequent deterioration. Alexander and Maxwell (1996) worked on controlling shrinkages cracking from expansive clay sub-grades. They pointed out that pavements built on sub-grades of expansive clay soils are affected by volume changes through seasoned wetting and drying cycles. These clays are highly reactive to moisture which results in clays showing significant volume change as a direct result of moisture content variation.

Jegede (1997) investigated a case of long-term and frequent highway pavement failure induced by poor soil properties, at a locality along the F209 highway at Ado-Ekiti. After the laboratory soil mechanics tests carried out on the disturbed soil samples collected from the failed sections of the road identified poor soil bearing capacity, poor Sub-grade quality of materials like kaolinite and montemorrilonite (clays) as the root of the problem.

The results of the investigations of geotechnical properties of the Sub-grade soils in some sections of the Ibadan end of the Lagos –Ibadan expressway through laboratory analysis of collected samples by Adeyemi and Oyeyemi (1998) showed that the Sub-grade soils below the stable sections have a higher maximum dry density, unsoaked California bearing ratio (CBR) and uncured, unconfined compressive strength than those below unstable sections. In addition, the soils below stable sections have both a lower proportion of fines and clay-sized fraction and a lower optimum moisture content and linear shrinkage than the material below the unstable sections. Surprisingly, the soils below the unstable pavements not only have a lower plasticity index and higher soaked CBRs than those below the stable pavements but also are more mechanically stable. Thus they concluded that significant differences need not exist between the geotechnical properties of soils below stable zones and unstable sections before such parameters can serve as bases for predicting the stability of flexible highway pavements in the tropics.

Abam, Ofoegbu, Osadebe, and Gobo (2000) explored the impact of hydrology on the Port-Harcourt –Patani- Warri Road, by reviewing the hydro-meteorological, drainage and terrain peculiarities of the area against the backdrop of the design, alignment and performance of the road. In their findings, they ascribed the poor performance of the road to:

i. The southeast-northwest orientation of the road in a region with predominately northeast-southwest surface and subsurface flow, in which the road acts like a dam;
ii. The inferior construction aggregate composition;
iii. Changes in pavement condition due to interaction of local road aggregates with water.

Gupta and Gupta (2003) in their work on Highway Failure and Maintenance, made it clear that the Sub-grade soil is an integral part of a road pavement structure as it provides the support to the pavement from beneath; therefore should possess sufficient strength and stability under adverse climatic and loading conditions to avoid failure. Roy (2003) through theoretical considerations and empirical observations have demonstrated the occurrence of gravity ground-water flow systems in valleys where precipitation is high in adjacent mountains. In such systems the valley floor is often a ground-water sink and adjacent mountains contain ground-water sources. He was of the opinion that optimum conditions for growth of ice lenses beneath highway pavement consist of a frost-susceptible soil, a source of water, and the absence of high negative pore-water pressures. He thus suggested that proper selection of a highway route with respect to ground-water flow systems in mountain valleys may minimize pavement failure caused by frost heaving.

The Central Roads Research Institute (CRRI, 2004) in their report on the field survey investigation of the failed Rao Pitampur Toll Road near Indore, Madhya Pradesh, India, spotted that the road was constructed over a black cotton Sub-grade. The existing bituminous surface was observed to have extensive undulations. The other type of distress was in the form of settlement of the road pavement which might be attributed to the intrusion of sub base material into the soft black cotton Sub-grade soil, thereby adversely affecting the riding quality of the pavement. A general condition survey of the pavement surface was conducted to assess the type of distress. Special emphasis was laid on the drainage aspect and other relevant data were collected during the course of detailed investigation. The study indicated the failure of the surfacing, ravelling, extensive potholes, depressions, map cracks, edge failure and settlement of the surfacing accompanied by shoving of the surface layer. Considering the extent and severity of distress, the road was not expected to perform its intended function unless suitable remedial measures were suggested. The details of the suggestions are as below:

i. Geofabrics were recommended at only those portions, which required full depth reconstruction because of extensive failures caused by poor drainage and absence of a sand cushion layer over the existing Sub-grade.
ii. The drainage system is inadequate and should be improved by constructing side drains using geofabrics on either side of the road wherever required.

Akpan (2005) in his study carried out to relate the frequency of pavement failures, the engineering indices of the Sub-grade materials and the underlying geology. The results show a high variability in the indices such as the liquid limit, LL, the plasticity index, PI, the maximum dry density, MDD, the optimum moisture content, OMC, compressibility and the California Bearing Ratio, CBR, between the different geologic units. Engineering indices having significant correlation with CBR (the major criteria for assessing the quality of Sub-grade materials), are used to develop a scheme for evaluating the materials at different failure points along the Calabar-Itu Highway in Nigeria. The evaluation shows that locations exhibiting high failure rates are underlain by shaly or marly Sub- grade whereas locations characterised by low failure rates are underlain by weathered basement or sandy unit as Sub-grade. It is recommended that maintenance and provision of drainage facilities will go a long way to reducing the rate of failure.

Ajani (2006) in his review, commented that from available records, premature highway failures occur both in the northern and southern regions of Nigeria. However, it seems to be more prevalent and more extensive in the southern region. In the north, premature failure occurs mainly as washout on identifiable sections while in the south, it is usually extensive sometimes covering the entire highway pavement due to the geography and geological formation of the area.

Momoh, Akintorinwa and Olorunfemi (2008) used geophysical survey involving Schlumberger Vertical Electrical Sounding (VES) and dipole-dipole electrical resistivity, magnetic and Very Low Frequency Electromagnetic (VLF-EM) methods to investigate the significance of the geological factors in terms of the nature of the subsoil, the near surface structures and the bed rock structural disposition as possible causes of failures along the Ilesha-Owena highway located within the Lagos-Ibadan-Akure highway in the northern part of Osun State, Southwestern Nigeria. Detailing the subsurface geoelectric sequence, mapping the subsurface structural features within the sub-grade soil and delineating the bedrock relief as a means of establishing the cause(s) of the highway pavement failure. The magnetic profile, inverted VLF-EM model, dipole-dipole and geoelectric sections along the stable segment are diagnostic of generally resistive and homogeneous subsurface devoid of any geological feature. Along the failed segments low resistivity clay enriched, water absorbing substratum and linear features suspected to be faults, fracture zones, joints and buried stream channels were delineated. The clayey sub-grade soil below the highway pavement and identified suspected geological features are the major geologic factors responsible for the highway failure. Other factors include poor drainage and excessive cut into near-surface low resistivity water absorbing clay enriched substratum.

Adiat, Adelusi and Ayuk (2009), used integrated geophysical methods to investigate the courses of incessant road failure along some parts of Igbara-oke – Ibuji road – southwestern Nigeria. Results from the geophysical survey identified the causes of the road failure to include: Clayey nature of the topsoil / Sub-grade soil on which the road pavement is founded. Clay, though highly porous but less permeable owing to poor connectivity of its pores, retains water without releasing it thus makes it swell up and collapse at the exertion of pressure and this subsequently lead to road failure. Also reported by this group was the presence of near surface linear features such as faults, fractured zones, fissures and joints etc. in the subsoil beneath the road pavement as this creates structurally weak zones that enhance groundwater accumulation and hence pavement failure.

2.6.2 Faulty Design and Poor Road Construction

For any road pavement to be sound and stand the taste of time, it must be well designed and properly constructed. Many other factors of road failure can be taken care of at the design and construction stages.

Paul and Radnor (1976) in their work titled "Highway Engineering" stated that road design involves more than substituting data or taking values from a design chart, they also argued that many design methods in use are either entirely or partially empirical and may not give the desired result unless prior knowledge of the environment is known and rooms for adjustment in design created during construction. They disclosed that this has been discovered from many experimental roads. In addition they pointed out that all over the world, despite the level of technology, the number of design methods available have no hard and fast rule attached to them in designing flexible pavement.

Abynayaka (1977) who worked on the prediction of road construction failure in developing countries, reasoned the same way with Paul and Radnor (1976) by attributing faulty design to the fact that tests under which the specification for materials and equipment to be used are based and performed are in different environments. Again, he stated that there is a tendency of under-forecasting of the of traffic volume due to the developing nature of towns and cities in developing countries. Consequently this may result to under design and hence possible over stressing of the road pavement structure and eventually failure. They further disclosed that in Nigeria, award of contracts is most of the times based on no special ethics but on compassionate grounds. Thus constructions of roads, they said, is put in the hands of people with little or no technical know-how and hence early failure of roads. He further disclosed that majority of the specifications for a particular road contract is ignored during construction. This is for the contractor to maximize profit as against producing good quality road with longer life span. He particularly described how the dimension for roadways, pavement thickness and requirements for asphalt mixes are reduced in order to make profit ad save time in some Nigerian roads.

However, according to the World Bank (1981), even when road designers overcome the problems of design, the next problem that normally comes up is whether the constructor has the competence to execute the work according to specifications. They made it clear that the problem of poor road construction ranges from the selection of contractors (i.e. award of contracts) to the procedure of acceptance of the completed job through regular inspection of the job while work is in progress.

The United Nation Educational Scientific and Cultural Organization (UNESCO) (1991) described road design as the translation of field location, survey and other data into specific plan to guide construction. It further discussed that a faulty design leads to failure, and good road design must ensure flexibility to minimize erosion hazards under varying site conditions. In another development, the World Bank (1991) stated that choice of construction materials for a particular road should be a function of topography, climate, and edaphic factors. They established a standard, which said that an ideal road must be built on three layers, namely:

a) The structural bed layer
b) The sub-base layer
c) The pavement layer

Roads are bound to fail where any of the three layers is absent.

Again, the FMWH (1992) stated that in Nigerian road designs (flexible pavement) are based on Concrete Pavement Restoration (CPR) methods, using pavement design specification of the FMWH. They further disclosed that in the specification handbook, the quantitative values and qualities of materials to be used are given, buttressing further the need for proper road design. The federal Ministry of Works and Housing (FMWH) (1995) in a seminar on the importance of drainage system in all Nigerian roads, disclosed that faulty design is one of the causes of road failure. It again maintained that in the design of drainage structure, such data as soil conditions, rainfall intensity, and subsurface conditions are not collected prior to the design. Even if this rule is strictly followed, in some places, some are still neglected. It was also made clear that surface drainage is designed to remove storm waters from the travel roadway as rapidly as possible so that traffic may move safely and effectively. This means that drainage structure is a serious factor in road failure. Jain and Kumar (1998) in their report on the causes of cracks occurrence in Ramghat-Aligarh Road near Aligarh, commented that a team from IIT Roorkee visited the site and found that the density of the different layers was in conformity to the average value given by the records of construction time and the workmanship was of good quality. However, the cracks have appeared due to the following reason:

a) The top surface layer Mix Seal Surfacing (MSS) was constructed on existing BM layer. The layer of BM was exposed to unexpected rain and water percolated into lower portion through BM layer. The longitudinal drainage was not proper along the road and cross drainage works were also missing at various locations and the entire pavement was in saturated condition. That is why the water which had percolated into the lower layers could not escape through the sides. At one or two locations water was observed in BM layer while taking the density of BM and MSS layers by sand replacement method.
b) The road was widened on one side so there was differential settlement. It disturbed the camber position and the total thickness of the pavement crust was not uniform along the pavement width. This also caused variation in strength as well as load distribution of the pavement along the road width and results in cracks.
c) MSS itself is semi dense coat and during heavy rains, water was stagnated on the pavement surface. Due to continued stagnation of water, MSS layer also allowed percolation of water into lower layers. According to Jain and Kumar (1998) IIT Roorkee was also requested to estimate the overlay requirements of Meerut Bypass and the cause of its failure. It was found that there is a wide variation in overlay

thickness requirement of this 18.2 km road section. It varies from 25 mm (in terms of BM) at 15-16 km to 185 mm at 0-1 and 8-9 km. Here it was seen that water was stagnating along the road at some sections and side slopes were eroded. This resulted in the entry of water into the embankments and cracks had appeared in the pavement. These cracked sections required more strengthening in terms of layer thickness Kumar (2002) commented on the study assigned to IIT Roorkee to investigate the corrugations over the newly laid Semi Dense Bituminous Concrete (SDBC) in Army Area of Roorkee sometime in the year 2001. The investigating team observed that although the quantity of bitumen, aggregate quality and proportioning were good but the compaction was done after the temperature of the mix had cooled down. As a remedy, the contractor was asked to provide a sand bitumen layer. Comments on Erosion and Road Failures (krisweb) has it that Road failures can be triggered by several different mechanisms which may include:

i. Side Cast Material: When roads are being constructed, material excavated from the road bed is often side-cast off the edge of the road. This mound of material perched on a steep slope can become saturated and trigger a landslide that may take the road bed with it.

ii. Fill and Cut-Bank Failures: Forest roads are often built across weakly consolidated soils. If the road bed is not properly compacted and armored with a layer of rock, the road bed itself may fail as it becomes saturated. The exposed soil of a road cut can become saturated during a large storm event and trigger a landslide that takes out the road as well.

iii. Plugged Culvert: Each time a logging road crosses a stream, culverts are used to funnel the water under the road. The culvert is buried with fill material so that the logging road remains fairly level at the stream crossing. When culverts get plugged in large storm events, water backs up behind them and can take out the road. This type road failure is a human-induced debris torrent which contributes large amounts of sediment to the stream. If there are road crossings downstream from a culvert failure, they too will fail as the debris torrent extends downstream.

iv. Headwall or Mid-Slope Failure: When roads are constructed across steep headwater areas or across mid-slope areas with emergent ground water, they often fail. The compaction associated with road construction and use can block the flow of shallow ground water. This ground water pushes up near the surface and can saturate the road prism that in turn increases its likelihood of failure. Roads in headwater or mid-slope locations are often constructed on steep slopes and failures initiate debris torrents that can cause significant destruction.

2.6.3 Poor Maintenance Operations

According to Paul, and Radnor (1976), road maintenance includes both physical maintenance, activities such as patching, filling of joints, moving, and traffic services like painting, pavement markings, erecting signs and litter control. However, the Asphalt Institute (1976) in her manual series disclosed that road maintenance is limited and what the maintenance man does is just to make one dollar out of two dollar worth of job; this is not good and safe for our roads.

John and Gordon (1976) in their engineering manual captioned "A practical Guide to Earth Road Construction and Maintenance", noted that each road in which the natural soil is used as a running surface is not easy to maintain, particularly during the rainy season due to slippery surfaces, tendency to form corrugations that transverse the road or longitudinal rutting. They were also of the opinion that the need for road maintenance arises when the road develops cracks, ravelling or twisting pavement surface.

Paul and Radnor (1976) on the other hand argued that though traffic and climatic conditions and the soil characteristics of different regions vary, there are maintenance operations, which can be used equally well in all regions. They disclosed that this operation was launched as cost effective approach, otherwise what they called "six - step approach" to prevent maintenance. They emphasized further that the Department of Transport (DOT) of the state of Georgia in the United States of America calls it Concrete Pavement Restoration (CPR). They established and disclosed the "six -step approach" adopted by Georgia, DOT as:

a) Slab stabilization
b) Slab replacement (where necessary)
c) Repair of spalls
d) Resealing transverse and longitudinal joints
e) Shoulder restoration
f) Diamond grinding.

On the other hand, Oglesby and Garry (1978) were of the opinion that road maintenance starts immediately after construction and commissioning for public use in order for it to give maximum service. They argued that the basic requirement of a road is that it should provide a uniform skid resistance-running surface that has longer life span and needs little maintenance.

The TRRL (1991) said that the sustainability of a road facility depends on how well the roads is constructed and maintained. It stated that constant maintenance of a road facility ensures a lasting road and shows good management of a road facility. In order to have constant road maintenance, crew organization is needed in the form of direct labour to ensure efficient and regular road maintenance.

2.6.4 Traffic Effects and Human Impacts on the Roads

According to Paul and Radnor (1976) Traffic causes stress on road pavement as well as accelerating the distress caused by other factors. They are of the view that increased traffic flow repeatedly leads the road surface and the amount of pavement deformation increases as the number of load application increases.

The American Association of State Highway and Transportation Officials (AASHTO) (1976) in the manual they produced Bridge Maintenance, established some traffic characteristics responsible for the adverse effects of the road pavements. These characteristics include:

a) The traffic composition
b) The abrasive nature of loading
c) The speed of the vehicles
d) The vehicle wheel configuration
e) The tyre pressure
f) The Axle load and
g) The number and nature of repetition of the loading suffice to say that all these factors are rampant along the Onitsha/Enugu expressway because of its position as the major road through which goods move from north to the southeast and from the south to the north. The commercial value of Onitsha also enhanced this.

Similarly, the Anambra state Ministry of Works and Houing (ANSMWH), (1998) attributed road failure to human impacts on the roads. It was disclosed that corporate bodies like power Holding Company of Nigeria (PHCN), Nigerian

Telecommunication (NITEL), Water Corporation, etc impact the roads without proper repair. It was stated that for example Water Corporation, in an attempt to lay down water pipes for water supply to the consumers cut across the road and when the road is reinstalled the job will not be well done, thus leading to road pavement distress and causing discomfort to road users. It was clear that this indiscriminate cutting across the roads results to complete road failures in the long run.

Also FMWH (1995) remarked that some individuals construct big bumps or excavation across some roads in Nigeria in order to check over-speeding without knowing the effects of such constructions on the road pavements.

Again the former Commissioner for works and Housing in Anambra state, in a media chart delivered on December 12, 2004 disclosed that during social unrests like riot, rampage or celebrations, youths normally burn tyres on the asphalted roads. The asphalt used in paving the roads is made of petro-chemical material (Bitumen), which is inflammable and can burn when heated. The resultant effect is that the burnt portion s will become weak and limp when in contact with water. This later develops to cracks and eventually potholes and small ditches on the road pavement, resulting in failures.

Ibrahim (2011), a Nigerian Daily Times Online reporter, reported an interaction with the head of the Federal Road Maintenance Agency (FERMA), in Jos on the 30 th day of May 2011, Mr. Ifeanyi Nweke, who commented that "One other challenge is knowing the causes of the failure of our roads, considering the quantum of loads that ply the roads on a daily basis". "Our roads are constructed not to carry loads above 42 tones but the failure of our rail system has made it imperative for the transporters to carry goods above what is expected," he said. He thus called for the re-introduction of weigh-bridges on the roads to put a check on road users who carry goods beyond the limit. "Besides, many of these roads were constructed a long time ago and cannot stand the kind of weight being carried on them on daily basis". "If the Federal Government re -introduces the system, more revenue will be generated from law-breakers who will be taxed for the extra loads they carry," he said.

2.6.5 Environmental and Climatic Factors

Paul and Radnor (1976) pointed out that most of the defects credited to traffic are actually initiated by environmental and climatic factors, and are later developed by traffic. According to the shrinkage cracks which sometimes occur initially at the underside of a road pavement due to temperature and moisture changes are often found to increase in size on the last load applied to it by traffic. They concluded that temperature change, moisture differences and soil characteristic, which vary in different regions, contribute to the problems of road failure.

The TRRL (1991) in a report on road research disclosed that climatic factors can also affect the strength of road structure. It was stated that temperature fluctuation and acid rain attack on the base material of the road in water- logged areas can weaken the sub-base of the road materials through capillary action, thereby reducing the supporting power of the road pavement.

The World Bank (1991) in a paper titled "Nigeria Highway Sector Study" supported the view of TRRL (1991). Here it was stated that in some parts of Nigeria, temperature could rise as high as 35 0c in the day time and as low as 25 0c at Night. This fluctuation in temperature, according to it, induces stress on the road pavement. This results in cracking of poorly mixed asphalted road pavements. It was further stated that this high temperature could reduce the bond stiffness of the surface of the flexible road pavement leading to rutting under traffic. This means road failure.

Again, Abynayaka (1977) stated that when roads are poorly drained, such factors like erosion can take place leading to ejection of materials out of the road pavement.

CHAPTER THREE

METHODOLOGY

3.1 Preamble

The practice of testing soil samples in the geotechnical laboratory plays important role in soil mechanics and civil engineering practices. This is because the performance and durability of soil for any use is dependent on the strength characteristics of such soil. Therefore, evaluation of materials by various geotechnical tests to determine their suitability is highly essential. This will ensure a satisfactory performance when put into use.

3.2 Reconnaissance Survey

The Ikole – Omuo road was selected and surveyed within the case study areas. A detailed visual examination of the pavement surface including the taking of photographs was performed to document location, severity, extent of distress, the width of the road, the wearing course of the road, specification of drainage, the thickness of the asphalt and shoulder width as shown in the following pictures.


Plate 3.1 Reconnaissance survey being carried out on a failed road section

Plate 3.2 A pothole observed during the survey

Plate 3.3 A shoving defect observed during the survey

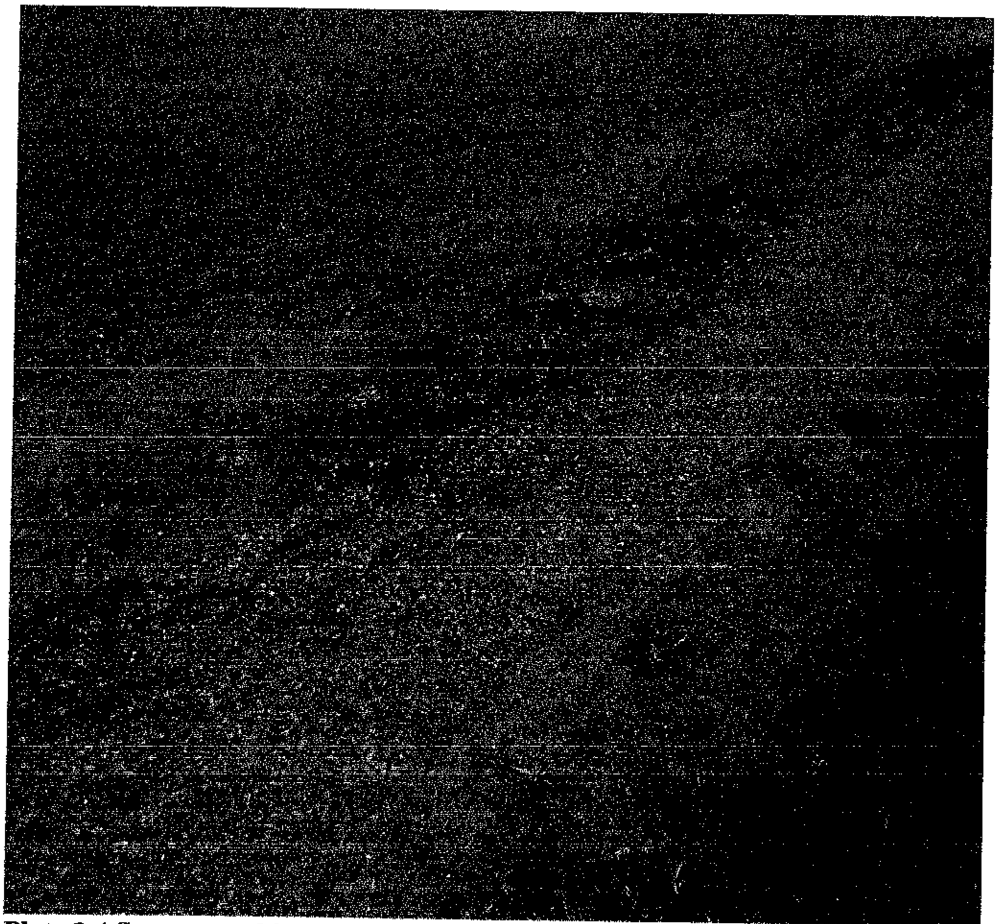

Plate 3.4 Severe cracking observed during the survey

3.3 Drainage Condition Assessment

Assessment of the surface and sub-surface drainage was conducted, as these elements contribute significantly to the overall performance of the pavement structure, Surface drainage is judged by the ability of the pavement surface to drain water as well as not allowing water to pond either on the bituminous surfacing or on the shoulder.

3.4 Interview

An interview was conducted with the staff of Ekiti State Ministry of Works, Federal Road Maintenance Agency (FERMA) and people within Ikole environs, to understand peoples opinion concerning design methodology, quality of design, traffic condition of road, and frequency of maintenance.

3.5 Sample Collection/ Laboratory Test

Disturbed soil samples were collected from one control stable segments and three failed segments, all were obtained from the roads shoulder. The soil samples were collected below the formation level of about 1.0 metre depth below the existing ground level and the overlying soil material as well as the top soil was discarded. The soil samples immediately after collection was contained in covered and labelled sacks to preserve the in-situ moisture content and taken to the laboratory for tests within 24 hours after collection. A hand held GPS device was used to determine the location of the trial pits, the coordinates of the location are given below;

Table 3.1 Coordinates of Collection Points

S/N	TRIAL PIT	COORDINATE IN DEGREE		COORDINATE IN METRIC	
		NORTHING	EASTING	NORTHING	EASTING
1	Kota Ekiti Road	7.764809°	5.509817°	862887.69	612294.94
2	Ilasa Ekiti Road	7.793713°	5.492262°	866099.74	610344.09
3	Ayebode Ekiti Road	7.804725°	5.641358°	867323.48	626912.83
4	Ikole Ekiti Road	7.796448°	5.479679°	866403.67	608945.77

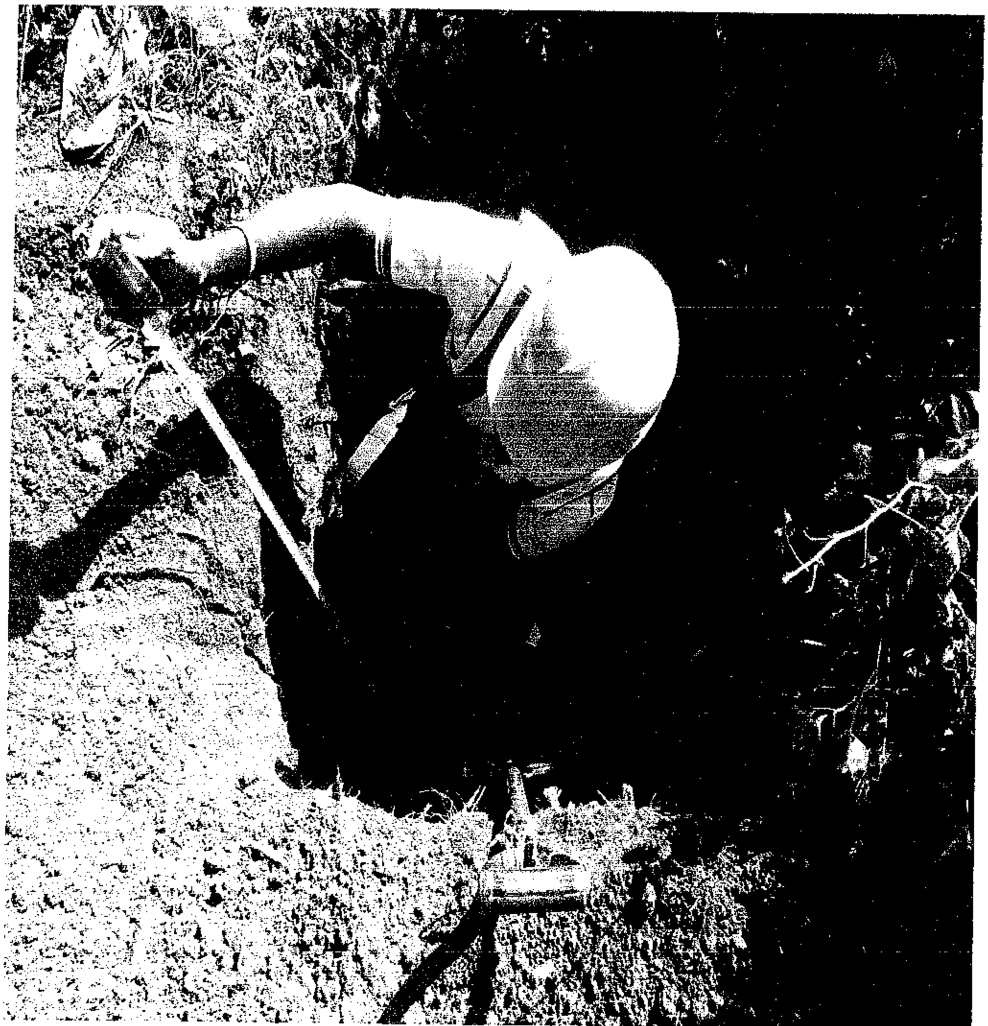

Plate 3.5 Measuring the depth of the trial pit with a measuring tape

The various methods of soil testing for engineering purposes was conducted in accordance with B.S. 1377 for all the soil samples and the soils was compacted to the West African Standard. The tests include Grain size analysis, Atterbergs Limit, Specific gravity, Permeability test, West African Compaction test, Natural moisture content determination and California Bearing Ratio.

3.5.1 Particle Size Distribution

This test is done to determine the particle size distribution of a soil sample

Tools

i. A set of fine IS Sieves of sizes – 2mm, 600μm, 425μm, 212μm and 75μm
ii. A set of coarse IS Sieves of sizes – 20mm, 10mm and 4.75mm
iii. Weighing balance, with an accuracy of 0.1% of the weight of sample
iv. Oven
v. Mechanical shaker
vi. Mortar with rubber pestle
vii. Brushes
viii. Trays

Preparation of Sample

i. Soil sample, as received from the field, should be dried in air or in the sun. In wet weather, the drying apparatus may be used in which case the temperature of the sample should not exceed 60°C. Clod may be broken with wooden mallet to hasten drying. Tree roots and pieces of bark should be removed from the sample.
ii. The big clods may be broken with the help of wooden mallet. Care should be taken not to break the individual soil particles.
iii. A representative soil sample of required quantity as given below is taken and dried in the oven at 105 to 120°C.

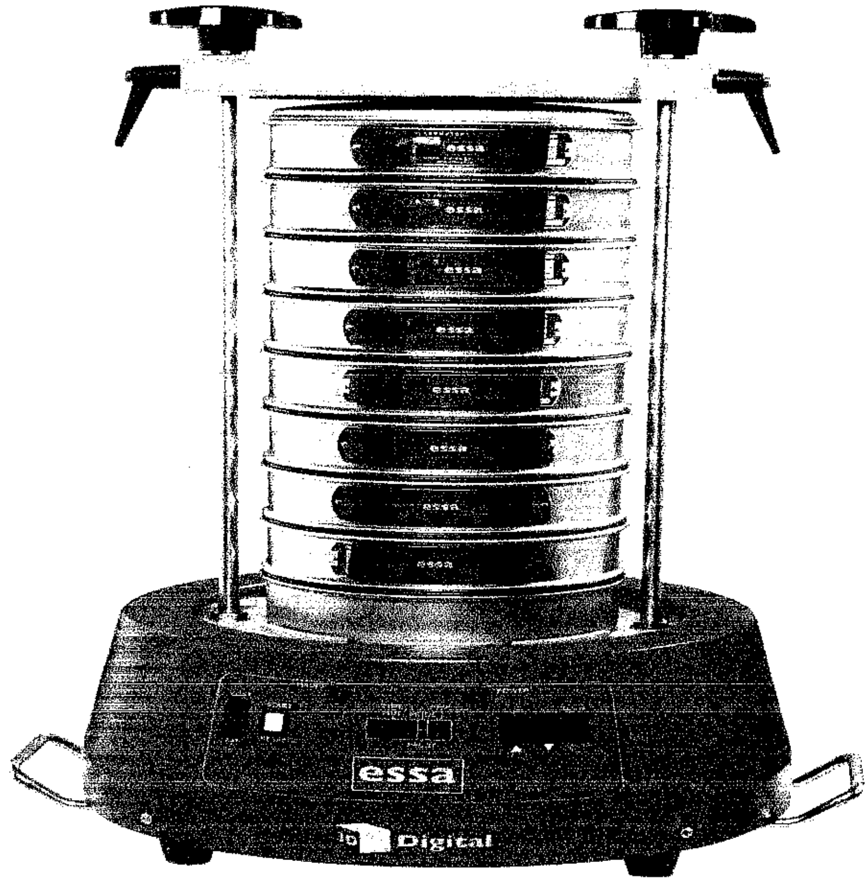

Plate 3.6 Mechanical Sieve Shaker

Procedure to Determine Particle Size Distribution Of Soil

i. The dried sample is taken in a tray, soaked in water and mixed with either 2g of sodium hexametaphosphate or 1g of sodium hydroxide and 1g of sodium carbonate per litre of water, which is added as a dispersive agent. The soaking of soil is continued for 10 to 12hrs.

ii. The sample is washed through 4.75mm IS Sieve with water till substantially clean water comes out. Retained sample on 4.75mm IS Sieve should be oven-dried for 24hrs. This dried sample is sieved through 20mm and 10mm IS Sieves.

iii. The portion passing through 4.75mm IS Sieve should be oven-dried for 24hrs. This oven-dried material is riffled and about 200g taken.

iv. This sample of about 200g is washed through 75μm IS Sieve with half litre distilled water, till substantially clear water comes out.

v. The material retained on 75μm IS Sieve is collected and dried in oven at a temperature of 105 to 120°C for 24hrs. The dried soil sample is sieved through 2mm, 600μm, 425μm and 212μm IS Sieves. Soil retained on each sieve is weighed.

vi. If the soil passing 75μm is 10% or more, hydrometer method is used to analyse soil particle size.

Hydrometer Analysis

i. Particles passed through 75μm IS Sieve along with water are collected and put into a 1000ml jar for hydrometer analysis. More water, if required, is added to make the soil water suspension just 1000ml. The suspension in the jar is vigorously shaken horizontally by keeping the jar in-between the palms of the two hands. The jar is put on the table.

ii. A graduated hydrometer is carefully inserted into the suspension with minimum disturbance.

iii. At different time intervals, the density of the suspension at the centre of gravity of the hydrometer is noted by seeing the depth of sinking of the stem. The temperature of the suspension is noted for each recording of the hydrometer reading.

iv. Hydrometer readings are taken at a time interval of 0.5 minute, 1.0 minute, 2.0 minutes, 4.0 minutes, 15.0 minutes, 45.0 minutes, 90.0 minutes, 3hrs., 6hrs., 24hrs. and 48hrs.

v. By using the nomogram given in IS: 2720 (Part 4) – 1985, the diameter of the particles for different hydrometer readings is found out.

Reporting of Results

After completing mechanical analysis and hydrometer analysis, the results are plotted on a semi-log graph with particle size as abscissa (log scale) and the percentage smaller than the specified diameter as ordinate.

3.5.2 Specific Gravity

This test is done to determine the specific_gravity of fine-grained soil by density bottle. Specific gravity is the ratio of the weight in air of a given volume of a material at a standard temperature to the weight in air of an equal volume of distilled water at the same stated temperature.

Tools

i. Two density bottles of approximately 50ml capacity along with stoppers
ii. Constant temperature water bath (27.0 + 0.2°C)
iii. Vacuum desiccator
iv. Oven, capable of maintaining a temperature of 105 to 110°C
v. Weighing balance, with an accuracy of 0.001g
vi. Spatula

Preparation of Sample

Soil sample (50g) should if necessary be ground to pass through a 2mm IS Sieve. A 5 to 10g sub-sample should be obtained by riffling and oven-dried at a temperature of 105 to 110°C.

Procedure to Determine the Specific Gravity of Fine-Grained Soil

i. The density bottle along with the stopper, should be dried at a temperature of 105 to 110°C, cooled in the desiccator and weighed to the nearest 0.001g (W_1).
ii. The sub-sample, which had been oven-dried should be transferred to the density bottle directly from the desiccator in which it was cooled. The bottles and contents together with the stopper should be weighed to the nearest 0.001g (W_2).
iii. Cover the soil with air-free distilled water from the glass wash bottle and leave for a period of 2 to 3hrs. for soaking. Add water to fill the bottle to about half.

iv. Entrapped air can be removed by heating the density bottle on a water bath or a sand bath.

v. Keep the bottle without the stopper in a vacuum desiccator for about 1 to 2hrs. until there is no further loss of air.

vi. Gently stir the soil in the density bottle with a clean glass rod, carefully wash off the adhering particles from the rod with some drops of distilled water and see that no more soil particles are lost.

vii. Repeat the process till no more air bubbles are observed in the soil-water mixture.

viii. Observe the constant temperature in the bottle and record.

ix. Insert the stopper in the density bottle, wipe and weigh (W_3).

x. Now empty the bottle, clean thoroughly and fill the density bottle with distilled water at the same temperature. Insert the stopper in the bottle, wipe dry from the outside and weigh (W_4).

xi. Take at least two such observations for the same soil.

Reporting of Results

The specific gravity G of the soil = $(W_2 - W_1) / [(W_{4-1})-(W_3-W_2)]$. The specific gravity should be calculated at a temperature of 27°C and reported to the nearest 0.01. If the room temperature is different from 27°C, the following correction should be done:-

G' = Kg where,

G' = Corrected specific gravity at 27°C

k = [Relative density of water at room temperature]/ Relative density of water at 27°C.

A sample for the record of the test results is given below. Relative density of water at various temperatures is taken from Table 3.

3.5.3 West African Compaction Test

This test is done to determine the maximum dry density and the optimum moisture content of soil. There are three (3) methods used for compaction, they include

a) Standard Proctor test
b) Modified AASHTO method
c) West Africa method

i. Cylindrical metal mould – it should be either of 100mm dia. and 1000cc volume or 150mm dia. and 2250cc volume.
ii. Balances – one of 10kg capacity, sensitive to 1g and the other of 200g capacity, sensitive to 0.01g
iii. Oven – thermostatically controlled with an interior of non-corroding material to maintain temperature between 105 and 110°C
iv. Steel straightedge – 30cm long
v. IS Sieves of sizes – 4.75mm, 19mm and 37.5mm

Preparation of Sample

A representative portion of air-dried soil material, large enough to provide about 6kg of material passing through a 19mm IS Sieve (for soils not susceptible to crushing during compaction) or about 15kg of material passing through a 19mm IS Sieve (for soils susceptible to crushing during compaction), should be taken. This portion should be sieved through a 19mm IS Sieve and the coarse fraction rejected after its proportion of the total sample has been recorded. Aggregations of particles should be broken down so that if the sample was sieved through a 4.75mm IS Sieve, only separated individual particles would be retained.

Procedure to Determine the Maximum Dry Density and the Optimum Moisture Content of Soil

a) **Soil not susceptible to crushing during compaction**

i. A 5kg sample of air-dried soil passing through the 19mm IS Sieve should be taken. The sample should be mixed thoroughly with a suitable amount of water depending on the soil type (for sandy and gravelly soil – 3 to 5% and for cohesive soil – 12 to 16% below the plastic limit). The soil sample should be stored in a sealed container for a minimum period of 16hrs.
ii. The mould of 1000cc capacity with base plate attached, should be weighed to the nearest 1g (W_1). The mould should be placed on a solid base, such as a concrete floor or plinth and the moist soil should be compacted into the mould, with the extension attached, in five layers of approximately equal mass, each layer being given 25 blows from the 4.9kg rammer dropped from a height of 450mm above the soil. The blows should be distributed uniformly over the surface of each layer. The amount of soil used should be sufficient to fill the mould, leaving not more

than about 6mm to be struck off when the extension is removed. The extension should be removed and the compacted soil should be levelled off carefully to the top of the mould by means of the straight edge. The mould and soil should then be weighed to the nearest gram (W_2).

iii. The compacted soil specimen should be removed from the mould and placed onto the mixing tray. The water content (w) of a representative sample of the specimen should be determined.

iv. The remaining soil specimen should be broken up, rubbed through 19mm IS Sieve and then mixed with the remaining original sample. Suitable increments of water should be added successively and mixed into the sample, and the above operations i.e. ii) to iv) should be repeated for each increment of water added. The total number of determinations made should be at least five and the moisture contents should be such that the optimum moisture content at which the maximum dry density occurs, lies within that range.

b) **Soil susceptible to crushing during compaction**Five or more 2.5kg samples of air-dried soil passing through the 19mm IS Sieve, should be taken. The samples should each be mixed thoroughly with different amounts of water and stored in a sealed container as mentioned in Part A)

c) **Compaction in large size mould**

For compacting soil containing coarse material up to 37.5mm size, the 2250cc mould should be used. A sample weighing about 30kg and passing through the 37.5mm IS Sieve is used for the test. Soil is compacted in five layers, each layer being given 55 blows of the 4.9kg rammer. The rest of the procedure is same as above.

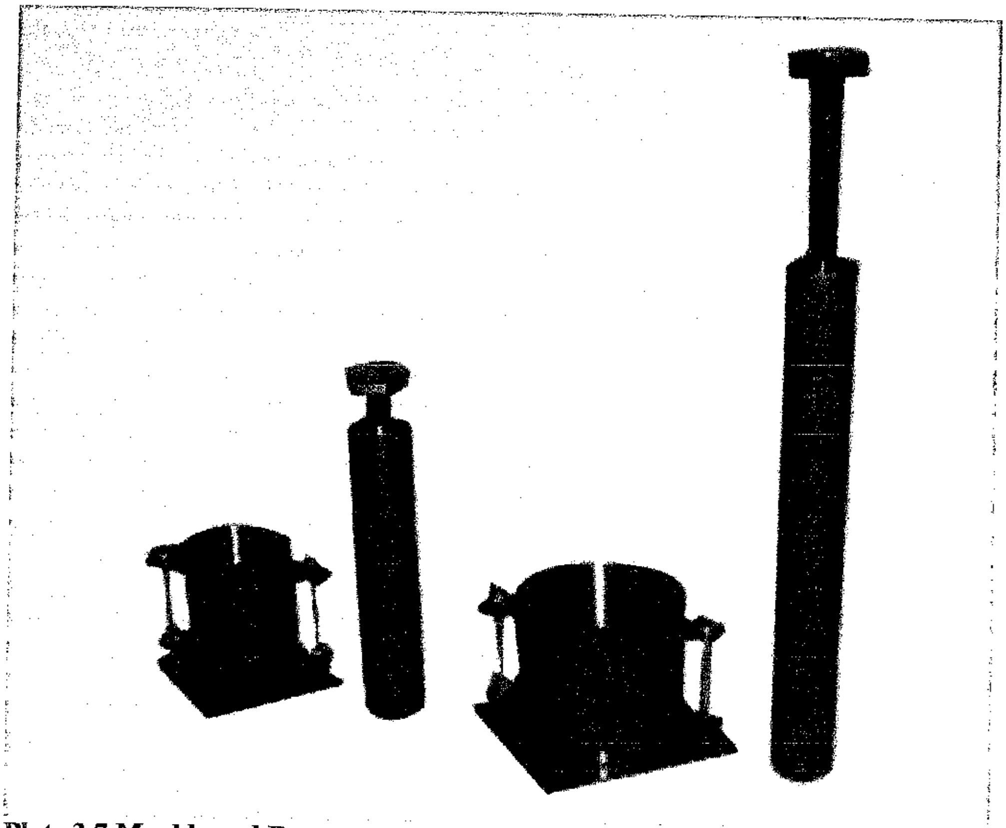
Plate 3.7 Moulds and Rammers

Reporting of Results

Bulk density Y(gamma) in g/cc of each compacted specimen should be calculated from the equation,

Y(gamma) = $(W_2-W_1)/V$

where, V = volume in cc of the mould.

The dry density Yd in g/cc

Yd = 100Y/(100+w)

The dry densities, Yd obtained in a series of determinations should be plotted against the corresponding moisture contents, w. A smooth curve should be drawn through the resulting points and the position of the maximum on the curve should be determined. The dry density in g/cc corresponding to the maximum point on the moisture content/dry density curve should be reported as the maximum dry density to the nearest 0.01. The percentage moisture content corresponding to the maximum dry density on the moisture content/dry density curve should be reported as the optimum moisture content and quoted to the nearest 0.2 for values below 5 percent, to the nearest 0.5 for values from 5 to 10 percent and to the nearest whole number for values exceeding 10 percent.

This test is done to determine the in-situ dry density of soil by sand replacement method as per IS: 2720 (Part XXVIII) – 1974. The apparatus needed is

i. Sand-pouring cylinder conforming to IS: 2720 (Part XXVIII) -1974
ii. Cylindrical calibrating container conforming to IS: 2720 (Part XXVIII) – 1974
iii. Soil cutting and excavating tools such as a scraper tool, bent spoon
iv. Glass plate – 450mm square and 9mm thick or larger
v. Metal containers to collect excavated soil
vi. Metal tray – 300mm square and 40mm deep with a 100mm hole in the centre
vii. Balance, with an accuracy of 1g

Procedure to Determine the In-Situ Dry Density of Soil by Sand Replacement Method

1. **Calibration of apparatus**

a) The method given below should be followed for the determination of the weight of sand in the cone of the pouring cylinder:

i. The pouring cylinder should be filled so that the level of the sand in the cylinder is within about 10mm of the top. Its total initial weight (W_1) should be maintained constant throughout the tests for which the calibration is used. A volume of sand equivalent to that of the excavated hole in the soil (or equal to that of the calibrating container) should be allowed to runout of the cylinder under gravity. The shutter of the pouring cylinder should then be closed and the cylinder placed on a plain surface, such as a glass plate.

ii. The shutter of the pouring cylinder should be opened and sand allowed to runout. When no further movement of sand takes place in the cylinder, the shutter should be closed and the cylinder removed carefully.

iii. The sand that had filled the cone of the pouring cylinder (that is, the sand that is left on the plain surface) should be collected and weighed to the nearest gram.

iv. These measurements should be repeated at least thrice and the mean weight (W_2) taken.

b) **The method described below should be followed for the determination of the bulk density of the sand (Ys):**

i. The internal volume (V) in ml of the calibrating container should be determined from the weight of water contained in the container when filled to the brim. The volume may also be calculated from the measured internal dimensions of the container.

ii. The pouring cylinder should be placed concentrically on the top of the calibrating container after being filled to the constant weight (W_1). The shutter of the pouring cylinder should be closed during the operation. The shutter should be opened and sand allowed to runout. When no further movement of sand takes place in the cylinder, the shutter should be closed. The pouring cylinder should be removed and weighed to the nearest gram.

iii. These measurements should be repeated at least thrice and the mean weight (W_3) taken.

2. Measurement of Soil Density

i. The following method should be followed for the measurement of soil density: A flat area, approximately 450sq.mm of the soil to be tested should be exposed and trimmed down to a level surface, preferably with the aid of the scraper tool.

ii. The metal tray with a central hole should be laid on the prepared surface of the soil with the hole over the portion of the soil to be tested. The hole in the soil should then be excavated using the hole in the tray as a pattern, to the depth of the layer to be tested upto a maximum of 150mm. The excavated soil should be carefully collected, leaving no loose material in the hole and weighed to the nearest gram (W_w). The metal tray should be removed before the pouring cylinder is placed in position over the excavated hole.

iii. The water content (w) of the excavated soil should be determined as discussed in earlier posts. Alternatively, the whole of the excavated soil should be dried and weighed (W_d).

iv. The pouring cylinder, filled to the constant weight (W_1) should be so placed that the base of the cylinder covers the hole concentrically. The shutter should then be opened and sand allowed to runout into the hole. The pouring cylinder and the surrounding area should not be vibrated during this period. When no further movement of sand takes place, the shutter should be closed. The cylinder should be removed and weighed to the nearest gram (W_4).

Reporting of Results

The following values would be reported:

i. Dry density of soil in kg/m3 to the nearest whole number; also to be calculated and reported in g/cc correct to the second place of decimal.

ii. Water content of the soil in percent reported to two significant figures.

3.5.4 Atterbergs Limit

3.5.4.1 Plastic Limit Test

This test is done to determine the plastic limit of soil as per IS: 2720 (Part 5) – 1985.The plastic limit of fine-grained soil is the water content of the soil below which it ceases to be plastic. It begins to crumble when rolled into threads of 3mm dia.

Tools

i. Porcelain evaporating dish about 120mm dia.

ii. Spatula

iii. Container to determine moisture content

iv. Balance, with an accuracy of 0.01g

v. Oven

vi. Ground glass plate – 20cm x 15cm

vii. Rod – 3mm dia. and about 10cm long

Preparation of Sample

Take out 30g of air-dried soil from a thoroughly mixed sample of the soil passing through 425µm IS Sieve. Mix the soil with distilled water in an evaporating dish and leave the soil mass for naturing. This period may be upto 24hrs.

Procedure to determine the Plastic Limit of Soil

i. Take about 8g of the soil and roll it with fingers on a glass plate. The rate of rolling should be between 80 to 90 strokes per minute to form a 3mm dia.

ii. If the dia. of the threads can be reduced to less than 3mm, without any cracks appearing, it means that the water content is more than its plastic limit. Knead the soil to reduce the water content and roll it into a thread again.

iii. Repeat the process of alternate rolling and kneading until the thread crumbles.

iv. Collect and keep the pieces of crumbled soil thread in the container used to determine the moisture content.

v. Repeat the process at least twice more with fresh samples of plastic soil each time.

Reporting of Results

The plastic limit should be determined for at least three portions of the soil passing through 425µm IS Sieve. The average water content to the nearest whole number should be reported.

3.5.4.2 Liquid Limit Test

This test is done to determine the liquid limit of soil as per IS: 2720 (Part 5) – 1985. The liquid limit of fine-grained soil is the water content at which soil behaves practically like a liquid, but has small shear strength. Its flow closes the groove in just 25 blows in Casagrande's liquid limit device.

Tools

i. Casagrande's liquid limit device

ii. Grooving tools of both standard and ASTM types

iii. Oven

iv. Evaporating dish

v. Spatula

vi. IS Sieve of size 425μm

vii. Weighing balance, with 0.01g accuracy bottle

viii. Air-tight and non-corrodible container for determination of moisture content

ix. Water bottle

Preparation of Sample

i) Air-dry the soil sample and break the clods. Remove the organic matter like tree roots, pieces of bark, etc.

ii) About 100g of the specimen passing through 425μm IS Sieve is mixed thoroughly with distilled water in the evaporating dish and left for 24hrs. for soaking.

Procedure to Determine the Liquid Limit of soil

i) Place a portion of the paste in the cup of the liquid limit device.

ii) Level the mix so as to have a maximum depth of 1cm.

iii) Draw the grooving tool through the sample along the symmetrical axis of the cup, holding the tool perpendicular to the cup.

iv) For normal fine grained soil: The Casagrande's tool is used to cut a groove 2mm wide at the bottom, 11mm wide at the top and 8mm deep.

v) For sandy soil: The ASTM tool is used to cut a groove 2mm wide at the bottom, 13.6mm wide at the top and 10mm deep.

vi) After the soil pat has been cut by a proper grooving tool, the handle is rotated at the rate of about 2 revolutions per second and the no. of blows counted, till the two parts of the soil sample come into contact for about 10mm length.

vii) Take about 10g of soil near the closed groove and determine its water content

viii) The soil of the cup is transferred to the dish containing the soil paste and mixed thoroughly after adding a little more water. Repeat the test.

ix) By altering the water content of the soil and repeating the foregoing operations, obtain at least 5 readings in the range of 15 to 35 blows. Don't mix dry soil to change its consistency.

x) Liquid limit is determined by plotting a 'flow curve' on a semi-log graph, with no. of blows as abscissa (log scale) and the water content as ordinate and drawing the best straight line through the plotted points.

Reporting of Results

Report the water content corresponding to 25 blows, read from the 'flow curve' as the liquid limit.

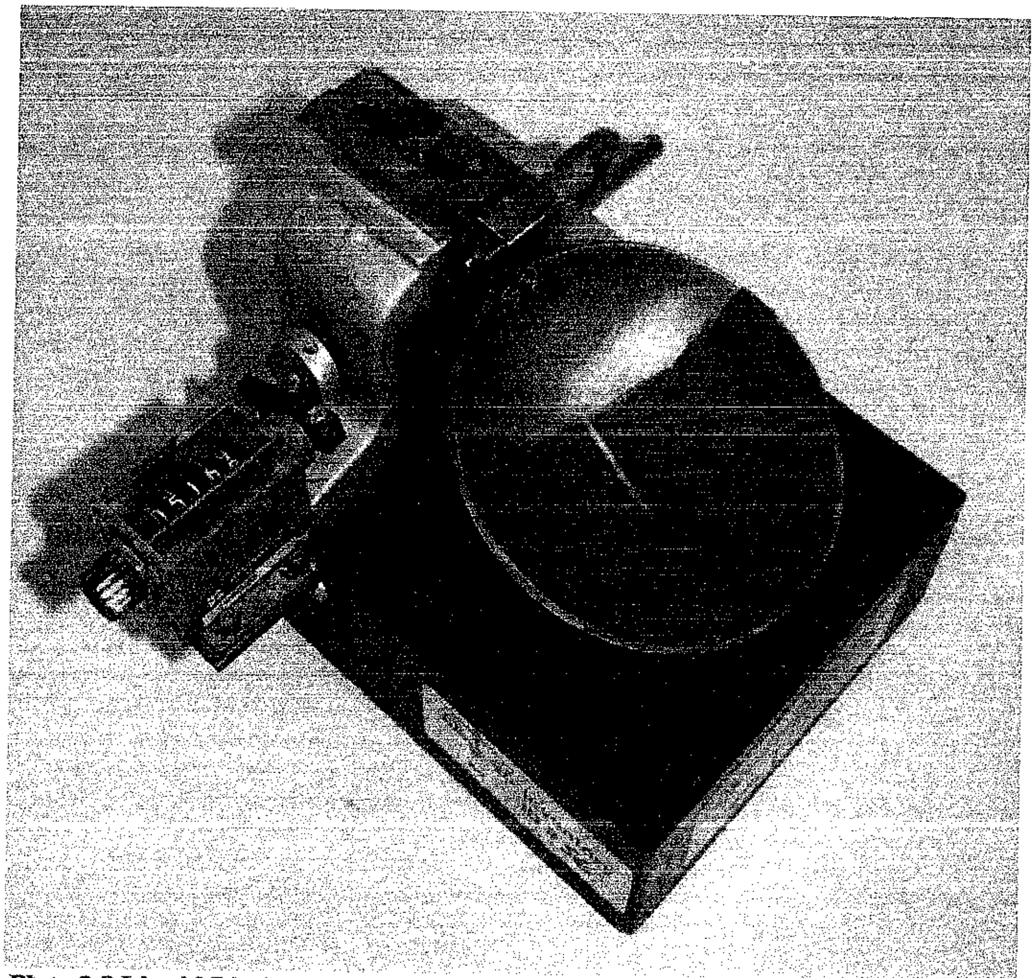

Plate 3.8 Liquid Limit apparatus

3.5.5 California Bearing Ratio Test

It is the ratio of force per unit area required to penetrate a soil mass with standard circular piston at the rate of 1.25 mm/min. to that required for the corresponding penetration of a standard material. The California Bearing Ratio Test (CBR Test) is a penetration test developed by California State Highway Department (U.S.A.) for evaluating the bearing capacity of subgrade soil for design of flexible pavement.

Tests are carried out on natural or compacted soils in water soaked or un-soaked conditions and the results so obtained are compared with the curves of standard test to have an idea of the soil strength of the subgrade soil.

Tools

i. Mould
ii. Steel Cutting collar
iii. Spacer Disc
iv. Surcharge weight
v. Dial gauges
vi. IS Sieves
vii. Penetration Plunger
viii. Loading Machine

CBR Test Procedure

Normally 3 specimens each of about 7 kg must be compacted so that their compacted densities range from 95% to 100% generally with 10, 30 and 65 blows.

i. Weigh empty mould
ii. Add water to the first specimen (compact it in five layer by giving 10 blows per layer)
iii. After compaction, remove the collar and level the surface.
iv. Take sample for determination of moisture content.
v. Weight of mould + compacted specimen.
vi. Take other samples and apply different blows and repeat the whole process.
vii. After four days, measure the swell reading and find percentage swell.
viii. Remove the mould from the tank and allow water to drain.
ix. Then place the specimen under the penetration piston and place surcharge load of 10lb.

x. Apply the load and note the penetration load values.

Reporting of Results

Draw the graphs between the penetration (in) and penetration load (in) and find the value of CBR. Draw the graph between the %age CBR and Dry Density, and find CBR at required degree of compaction.

Preparation of sample

i. A 2.5 kg sample shall be taken from a thoroughly mixed air dried or oven dried material.
ii. The initial moisture content of the 2.5 kg sample shall be determined. Then the soil shall be placed in the air tight container.
iii. Add required quantity of water to get the desired moisture content.
iv. Mix the soil thoroughly.
v. Weigh the empty permeameter mould.
vi. After greasing the inside slightly, clamp it between the compaction base plate and extension collar.
vii. Place the assembly on a solid base and fill it with sample and compact it.
viii. viii) After completion of a compaction the collar and excess soil are removed.
ix. ix) Find the weight of mould with sample.
x. x) Place the mould with sample in the permeameter, with drainage base and cap having discs that are properly saturated.

Procedure

i. For the constant head arrangement, the specimen shall be connected through the top inlet to the constant head reservoir.
ii. Open the bottom outlet.
iii. Establish steady flow of water.
iv. The quantity of flow for a convenient time interval may be collected.
v. Repeat three times for the same interval.

3.5.6 Moisture Content

The natural moisture content also called the natural moisture content is the ratio of the weight of water to the weight of the solids in a given mass of soil. This ratio is usually expressed as percentage. In almost all soil tests natural moisture content of the soil is to be determined. The knowledge of the natural moisture content is essential in all studies of soil mechanics. For example, natural moisture content is used to determine the bearing capacity and settlement. The natural moisture will give an idea of the state of the soil in the field. Water content is used in a wide range of scientific and technical areas and is expressed as a ratio which can range from 0 (completely dry soil) to porosity at saturation.

Tools

i. Non-corrodible air-tight container.
ii. Electric oven, maintain the temperature between 105^0C to 110^0C.
iii. Desiccator.
iv. Balance of sufficient sensitivity.

Natural Moisture Content Test Procedures

i. Weigh and record the weight of a clean and dry moisture content tin, W_1.
ii. Place it in the moisture content tin and weigh it. Record the weight as W_2.
iii. Carefully take the assembly to the drying oven and set the temperature at 105^0 C – 110^0C.
iv. After the temperature is attained, stop the drying oven and allow the samples cool naturally.
v. Weigh and record the final constant weight (W_3) of the container with dried soil samples.
vi. Repeat the procedure three more times.

CHAPTER FOUR

RESULTS AND DISCUSSION

Table 4.1 Summary of Test Results of Soil Samples

SAMPLE		A	B	C	D
Location		Kota - Ekiti	Ilasa-Ekiti	Ayebode - Ekiti	Ikole-Ekiti
Sieve Analysis	2.36	57.9	69.4	93.1	79.6
	0.6	40.2	54.9	81.6	66.2
	0.0075	23.2	39.5	54.3	42.1
Atterbergs Limit	LL %	31.0	30.2	31.0	30.5
	PL %	18.8	20.4	16.5	24.5
	PI %	12.2	9.8	14.5	6.0
Natural Moisture Content		13.1	17.0	22.9	15.4
AASHTO Classification		A-2-6	A-4	A-7-6^b	A-4
Specific Gravity		2.50	2.39	2.26	2.35
Compaction Test	OMC %	12.8	15.0	10.5	13.7
	MDD kN/m^3	2.00	1.86	1.99	1.89
CBR	2.5 mm	46.1	38.4	45.2	80.8
	5.0 mm	85.8	79.9	51.4	84.7
Permeability cm/s		5.96E-03	3.36E-03	4.05E-03	4.86E-03

4.1 Specific Gravity

The specific gravity of the soil samples ranges from 2.35 to 2.50. Sample A has the highest specific gravity value. According to specification, a good lateritic material should have specific gravity ranging from 2.6 to 2.9. Since Sample A, Sample B, Sample C, and Sample D are outside the range it suggests that soil consists of mostly Kaolinite clay mineral or potassium feldspar, according to, Braja M. Das (2010). See Appendix A3.

Table 4.2 Specific Gravity of Common Minerals, Braja M. Das (2010)

Mineral	Specific gravity, G_s
Quartz	2.65
Kaolinite	2.6
Illite	2.8
Montmorillonite	2.65–2.80
Halloysite	2.0–2.55
Potassium feldspar	2.57
Sodium and calcium feldspar	2.62–2.76
Chlorite	2.6–2.9
Biotite	2.8–3.2
Muscovite	2.76–3.1
Hornblende	3.0–3.47
Limonite	3.6–4.0
Olivine	3.27–3.7

4.2 Grain Size Distribution.

The percentage of the sample passing through No 200 BS for soil Sample A, Sample B, Sample C and Sample D are 23.2%, 39.5%, 54.3%, and 42.1%. The results show that many of the soil samples had a very high percentage finer than 0.0075 fractions (i.e. >35%). Hence, general rating as sub-grade in accordance with AASHTO (1986) is fair to poor materials. They have significant constituent materials of mainly clayey soils while few are silty or clayey gravel and sand where the percentage passing the No. 200 sieve is less than 35%. See Appendix A1 and B1.

4.3 Compaction

The West African compaction test method was used. The maximum dry density ranges between 2.0 kN/m^{3}and 1.86 kN/m^3, while the optimum moisture content ranges between 15.0% and 10.5%. Soil sample characterized by high value of maximum dry density and low optimum moisture content is best suitable as subbase and subgrade material. Sample C is one of the highest value of highest maximum dry density and lowest optimum moisture content. It can also be deduced from the result obtained that the soils have a medium organic content due to the values of maximum dry density and optimum moisture content observed. See Appendix A2 and B2.

4.4 California Bearing Ratio (CBR)

The un-soaked California bearing ratio of the sample ranges from 80.8% to 38.4% for 2.5 mm plunger penetration and 85.8% to 51.4%. Samples B has the lowest CBR value for 2.5 mm plunger penetration. All samples are suitable as a subgrade material except for Sample B which will require stabilization for satisfactory performance as shown in its plot of maximum dry density against optimum moisture content. See Appendix A6 and B3.

4.5 Permeability

The fallen head permeability method was adopted. The permeability value falls between 5.96 E-03 and 3.36 E-03, implying that the soil samples are semi-permeable. See Appendix A7.

4.6 Atterbergs Limit

The Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) values fall within 30.2% - 31.0%, 16.5% - 24.5% and 6.0% - 14.5% respectively. They were classified according to the AASHTO soil classification system and this revealed the soil samples to be organic clay of low to medium plasticity and inorganic sand of minimal plasticity. See Appendix A5 and B4.

4.7 Moisture Content

The result shows that moisture content value ranges from 13.1% to 22.9%. As a result of the moisture content values not exceeding 25%. It implies the soils in the study location have a low potential of water retention. See Appendix A4.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

All the soils in the study location have a low potential of water retention with their natural moisture content value not exceeding 25% and all the soils have clay contents based on the specific gravity result consisting mostly of Kaolinite clay mineral or potassium feldspar.

Sample A, B, C and D can generally be classified as silty-clay soil material with good percentage of sand and finer fractions and are fair to poor in general subgrade rating. Similarly, it was observed from the Atterbergs tests result showed that soil sample A is grouped as **A-7-6** class i.e. clayey soil, samples B and D are grouped as **A-4** class i.e. silty clay material and sample C is grouped as **A-2-6** class i.e. lateritic or granular soil and the group index value of samples A, B, C and D gives 0, 0.63, 5 and 0 respectively, according to AASHTO classification system.

For the compaction test using the maximum dry density and optimum moisture content values, it shows that all the soil samples from the trial pits are suitable as subgrade material except for sample A which may require soil stabilization for satisfactory performance.

The permeability test also shows that the soil samples are semi-permeable, this means they drain fairly well.

The assessment conducted shows that all the samples collected from the study area meet up with the standard specification for road by AASHTO. This information confirms that the soils samples are suitable as subgrade materials since they satisfy the recommended specifications. Hence, this shows that geotechnical properties of subgrades is not largely responsible for the road failure problem on the Ikole-Omuo road strectch.

5.2 Recommendations

Based on the finding and confirmations the following actions are recommended:

1. Attention should be given to quality control by appropriate government agencies to ensure roads are constructed to meet the desired standard.

2. Qualified engineering personnel should give adequate supervision during road constructions.
3. A proper maintenance culture should be adopted.
4. The provision of adequate surface drainage should be of topmost priority in any road construction project.

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APPENDICES

Appendix A1: Sieve Analysis Result

Table A1.1: Sieve Analysis Result of Sample A

	Sample A		
sieve size	wt. ret.	% ret.	% passing
9.50	100.60	20.12	79.90
4.75	65.90	13.18	66.70
2.36	44.00	8.80	57.90
1.18	45.30	9.06	48.90
0.600	43.10	8.62	40.20
0.300	36.80	7.36	32.90
0.150	30.30	6.06	26.80
0.75	18.30	3.66	23.20
Total	384.30		

Table A1.2: Sieve Analysis Result of Sample B

	Sample B		
sieve size	wt. ret.	% ret.	% passing
9.50	68.10	13.60	86.40
4.75	44.70	8.90	77.50
2.36	40.40	8.10	69.40
1.18	35.90	7.20	62.20
0.600	36.60	7.30	54.90
0.300	29.90	6.00	48.90
0.150	27.00	5.40	43.50
0.75	19.90	4.00	39.50
Total	302.50		

Table A1.3: Sieve Analysis Result of Sample C

	Sample C		
sieve size	wt. ret.	% ret.	% passing
9.50	20.60	4.10	95.90
4.75	5.60	1.10	94.80
2.36	8.60	1.70	93.10
1.18	18.70	3.70	89.40
0.600	38.90	7.80	81.60
0.300	40.80	8.20	73.40
0.150	45.40	9.10	64.30
0.75	49.80	10.00	54.30
Total	228.40		

Table A1.4: Sieve Analysis Result of Sample D

	Sample D		
sieve size	wt. ret.	% ret.	% passing
9.50	25.70	5.10	94.90
4.75	31.10	6.20	88.70
2.36	45.70	9.10	79.60
1.18	33.70	6.70	72.90
0.600	33.30	6.70	66.20
0.300	47.50	9.50	56.70
0.150	46.10	9.20	47.50
0.75	27.20	5.40	42.10
Total	290.50		

Appendix A2: West African Compaction Result and Calculation

Appendix A2: Compaction Analysis Result

Table A2.1: Compaction Result of Sample A

Sample A				
Trial No.	1	2	3	4
Weight of mould + soil (g)	5000	5200	5400	5350
weight of empty mould (g)	3150	3150	3150	3150
weight of wet soil	1850	2050	2250	2200
wet density of soil (Kg/m³)	1.9	2.1	2.3	2.2
container identification no.	A_7	A_8	A_9	A_{10}
weight of container (g)	17.8	19.8	20.1	20.0
weight of wet soil + container (g)	86.6	78.7	66.9	81.2
weight of dry soil + container (g)	82.8	74.0	61.6	72.9
weight of water (g)	3.80	4.70	5.30	8.30
weight of dry soil (g)	65.0	54.2	41.5	52.9
moisture content (%)	5.8	8.7	12.8	15.7
dry density (kN/m³)	1.75	1.89	1.99	1.90

Table A2.2: Compaction Result of Sample B

Sample B				
Trial No.	1	2	3	4
Weight of mould + soil (g)	5100	5250	5300	5200
weight of empty mould (g)	3150	3150	3150	3150
weight of wet soil	1950	2100	2150	2050
wet density of soil (Kg/m³)	1.95	2.10	2.15	2.05
container identification no.	B_7	B_8	B_9	B_{10}
weight of container (g)	26.60	10.70	26.70	26.90
weight of wet soil + container (g)	85.40	72.40	81.40	88.50
weight of dry soil + container (g)	73.40	65.70	74.10	79.30
weight of water (g)	5.10	6.70	7.30	9.20
weight of dry soil (g)	51.70	55.00	47.40	52.40
moisture content (%)	9.90	12.20	15.40	17.80
dry density (kN/m³)	1.77	1.83	1.86	1.74

Table A2.3: Compaction Result of Sample C

Sample C				
Trial No.	1	2	3	4
Weight of mould + soil (g)	5100	5300	5350	5250
weight of empty mould (g)	3150	3150	3150	3150
weight of wet soil	1950	2150	2200	2100
wet density of soil (Kg/m³)	1.95	2.15	2.20	2.10
container identification no.	C_7	C_8	C_9	C_{10}
weight of container (g)	20.10	26.70	26.60	18.00
weight of wet soil + container (g)	83.00	83.50	81.60	76.20
weight of dry soil + container (g)	79.20	78.60	76.10	68.30
weight of water (g)	3.80	4.90	5.50	7.90
weight of dry soil (g)	59.10	51.90	49.50	50.30
moisture content (%)	6.40	9.40	11.10	15.70
dry density (kN/m³)	1.83	1.97	1.98	1.82

Table A2.4: Compaction Result of Sample D

Sample D				
Trial No.	1	2	3	4
Weight of mould + soil (g)	4950	5150	5300	5200
weight of empty mould (g)	3150	3150	3150	3150
weight of wet soil	1800	2000	2150	2050
wet density of soil (Kg/m³)	1.80	2.00	2.15	2.05
container identification no.	D_7	D_8	D_9	D_{10}
weight of container (g)	17.80	26.60	18.50	18.30
weight of wet soil + container (g)	83.40	82.10	87.60	85.90
weight of dry soil + container (g)	77.80	76.30	79.30	75.10
weight of water (g)	5.00	5.80	8.30	10.80
weight of dry soil (g)	60.00	49.70	60.80	56.80
moisture content (%)	8.30	11.70	13.70	19.10
dry density (kN/m³)	1.66	1.79	1.89	1.72

Appendix A3: Specific Gravity Result and Calculation

Table A3.1: Specific Gravity Result of Sample A

Sample A		
Trial no.	1	2
weight of empty density bottle (g)	23.8	25.8
weight of density bottle + dry soil (g)	49.6	53.7
weight of density bottle + soil + water (g)	93.9	95.6
weight of density bottle + water (g)	78.3	79.0
Specific Gravity, S.G	2.53	2.47

W1 = weight of empty density bottle
W2 = weight of density bottle + dry soil
W3 = weight of density bottle + soil + water
W4 = weight of density bottle + water

$$S.G = \frac{W2 - W1}{(W4 - W1) - (W3 - W2)} = 2.50$$

Table A3.2: Specific Gravity Result of Sample B

Sample B		
Trial no.	1	2
weight of empty density bottle (g)	25.8	25.8
weight of density bottle + dry soil (g)	51.7	53.5
weight of density bottle + soil + water (g)	93.3	94.0
weight of density bottle + water (g)	78.1	78.0
Specific Gravity, S.G	2.42	2.37

W1 = weight of empty density bottle
W2 = weight of density bottle + dry soil
W3 = weight of density bottle + soil + water
W4 = weight of density bottle + water

$$S.G = \frac{W2 - W1}{(W4 - W1) - (W3 - W2)} = 2.39$$

Table A3.3: Specific Gravity Result of Sample C

Sample C		
Trial no.	1	2
weight of empty density bottle (g)	26.4	26.4
weight of density bottle + dry soil (g)	50.9	53.4
weight of density bottle + soil + water (g)	91.4	94.8
weight of density bottle + water (g)	77.6	79.9
Specific Gravity, S.G	2.29	2.23

W1 = weight of empty density bottle
W2 = weight of density bottle + dry soil
W3 = weight of density bottle + soil + water
W4 = weight of density bottle + water

$$S.G = \frac{W2 - W1}{(W4 - W1) - (W3 - W2)} = 2.26$$

Table A3.4: Specific Gravity Result of Sample D

Sample D		
Trial no.	1	2
weight of empty density bottle (g)	23.8	26.9
weight of density bottle + dry soil (g)	49.9	49.4
weight of density bottle + soil + water (g)	93.1	92.4
weight of density bottle + water (g)	78.2	79.4
Specific Gravity, S.G	2.33	2.37

W1 = weight of empty density bottle
W2 = weight of density bottle + dry soil
W3 = weight of density bottle + soil + water
W4 = weight of density bottle + water

$$S.G = \frac{W2 - W1}{(W4 - W1) - (W3 - W2)} = 2.35$$

Appendix A4: Natural Moisture Content Result and Calculation

Table A4.1: Natural Moisture Content Result of Sample A

Sample A		
Trial no.	1	2
weight of container (g)	26.6	19.7
weight of container + soil+ water (g)	88.8	79.3
weight of container + dry soil (g)	81.7	72.3
Moisture content (%)	12.9	13.3

A = weight of container
B = weight of container + water + soil
C = weight of container + dry soil

$$M.C = \frac{B - C}{C - A} \times 100 = 13.1$$

Table A4.2: Natural Moisture Content Result of Sample B

Sample B		
Trial no.	1	2
weight of container (g)	10.0	9.9
weight of container + soil+ water (g)	70.2	76.7
weight of container + dry soil (g)	60.7	67.9
Moisture content (%)	18.7	15.2

A = weight of container
B = weight of container + water + soil
C = weight of container + dry soil

$$M.C = \frac{B - C}{C - A} \times 100 = 17.0$$

Table A4.3: Natural Moisture Content Result of Sample C

Sample C		
Trial no.	1	2
weight of container (g)	26.7	26.9
weight of container + soil+ water (g)	95.3	92.8
weight of container + dry soil (g)	78.2	85.4
Moisture content (%)	33.2	12.6

A = weight of container
B = weight of container + water + soil
C = weight of container + dry soil

$$M.C = \frac{B - C}{C - A} \times 100 = 22.9$$

Table A4.3: Natural Moisture Content Result of Sample D

Sample D		
Trial no.	1	2
weight of container (g)	20.1	26.6
weight of container + soil+ water (g)	63.9	93.4
weight of container + dry soil (g)	57.9	84.7
Moisture content, M.C	15.9	15.0

A = weight of container
B = weight of container + water + soil
C = weight of container + dry soil

$$M.C = \frac{B - C}{C - A} \times 100 = 15.4$$

Appendix A5: Consistency Limit Test Result

Table A5.1: Consistency Limit Test Result of Sample A

Sample A							
trial no.	1	2	3	4			
no. of blows	44	30	22	12	**PLASTIC LIMIT**		
container identification no.	A_1	A_2	A_3	A_4	A_5	A_6	
weight of empty container (g)	11.5	6.7	17.2	8.1	12.1	7.1	
weight of container + wet soil (g)	37.3	34.7	49.8	42.4	32.1	29.4	
weight of container + dry soil (g)	30.0	28.2	41.7	33.4	28.9	25.9	
weight of water (g)	5.1	6.5	8.1	9.0	3.2	3.5	
weight of dry soil (g)	18.5	21.5	24.5	25.3	16.8	18.8	
moisture content (%)	27.6	30.2	33.1	35.6	19.0	18.6	PL = 18.8%

Table A5.2: Consistency Limit Test Result of Sample B

Sample B							
trial no.	1	2	3	4			
no. of blows	48	34	21	11	**PLASTIC LIMIT**		
container identification no.	B_1	B_2	B_3	B_4	B_5	B_6	
weight of empty container (g)	16.4	10.6	12.3	17.9	11.7	10.9	
weight of container + wet soil (g)	42.4	35.3	38.1	47.9	32.1	31.7	
weight of container + dry soil (g)	36.2	27.8	29.9	38.1	28.0	27.7	
weight of water (g)	6.2	7.5	8.2	9.8	4.1	4	
weight of dry soil (g)	19.8	17.2	17.6	20.2	16.3	16.8	
moisture content (%)	31.3	43.6	46.6	48.5	25.2	23.8	PL = 24.5%

Table A5.3: Consistency Limit Test Result of Sample C

Sample C							
trial no.	1	2	3	4			
no. of blows	46	34	20	12	**PLASTIC LIMIT**		
container identification no.	C_1	C_2	C_3	C_4	C_5	C_6	
weight of empty container (g)	25.2	19.1	26.8	2.6	18.3	10.9	
weight of container + wet soil (g)	42.6	39.6	49.4	41.7	40.9	26.9	
weight of container + dry soil (g)	38.4	34.1	42.9	32.7	38.2	24.3	
weight of water (g)	4.2	5.5	6.5	9.0	2.7	2.6	
weight of dry soil (g)	13.2	15.0	20.0	20.1	19.9	13.4	
moisture content (%)	32.0	36.7	40.4	44.8	13.6	19.4	PL = 16.5%

Table A5.4: Consistency Limit Test Result of Sample D

Sample D							
trial no.	1	2	3	4			
no. of blows	44	35	22	12	**PLASTIC LIMIT**		
container identification no.	D_1	D_2	D_3	D_4	D_5	D_6	
weight of empty container (g)	10	26.7	26.8	26.9	18.5	16.7	
weight of container + wet soil (g)	33	51.5	52.4	57.8	33	37.2	
weight of container + dry soil (g)	27.1	45.0	45.3	48.9	30.5	33.5	
weight of water (g)	5.9	6.5	7.1	8.9	2.5	3.7	
weight of dry soil (g)	17.1	18.3	18.5	22.0	12	16.8	
moisture content (%)	34.5	35.5	38.4	40.5	20.8	20	PL = 20.4%

Appendix A6: California Bearing Ratio Test Result

Table A6.1: California Bearing Ratio Test Result of Sample D

	CARLIFORNIA BEARING RATIO							
Plunger Penetration (mm)	Sample A		Sample B		Sample C		Sample D	
	Dial Gauge Reading	Load (kN)	Dial Gauge Reading	Load (kN)	Dial Gauge Reading	Load (kN)	Dial Gauge Reading	Load (kN)
0.5	109	2.73	76	1.90	89	2.23	201	5.00
1.0	132	3.30	91	2.28	128	3.20	289	7.20
1.5	176	4.40	127	3.18	155	3.88	330	8.30
2.0	205	5.13	168	4.20	197	4.93	378	9.50
2.5	244	6.10	203	5.08	239	5.98	426	10.70
3.0	310	7.75	251	6.28	276	6.90	478	12.00
3.5	348	8.70	302	7.55	298	7.45	538	13.50
4.0	483	12.08	465	11.63	332	8.30	587	14.70
4.5	595	14.88	536	13.40	375	9.38	628	15.70
5.0	685	17.13	638	15.95	410	10.25	675	16.90
5.5	725	18.13	662	16.55	442	11.05	710	17.80
6.0	769	19.23	695	17.38	468	11.70	762	19.10
6.5	802	20.05	719	17.98	491	12.28	801	20.00
7.0	838	14.90	741	18.53	513	12.83	820	20.50
7.5	876	21.90	768	19.20	538	13.45	838	21.00

Appendix A7: Permeability Test Result and Calculation

COEFFICIENT OF PERMEABILITY (Falling Head)

Location KORA EKITI

Sample No.: A

Soil Description : RED

Date : 21-02-2017

Sample Dimensions: Diam. 10.2 cm;

Area,A 85.10 cm^2

Vol. 1106.30 cm^3

Ht.L 13 cm

Sand pipe 1.0 cm Burette 50.1 ml Diam 10.2 Area,a 1.130 cm^2

Test no.	h_1 ,cm	h_2 ,cm	t, s	Q_{in}, cm^3	Q_{out}, cm^3	T, °C	Test no.	h_1 ,cm	h_2 ,cm	t, s	T, °C
1	94.5	50	17.5		44.50	21	1	94.5	50	17.5	21
2	94.5	50	18.2		44.50	21	2	94.5	50	18.2	21
3	94.5	50	18.9		44.50	21	3	94.5	50	18.9	21
4	94.5	50	19.1		44.50	21	4	94.5	50	19.1	21
						Average		94.5	50	18.425	21

$\alpha = \eta_T/\eta_{20} =$ 0.9761

$kT = (aL/At)\ln(h_1/h_2) =$ 5.96E-03 = 0.0059639716 cm/s

$k_{20} = \alpha k_T =$ 5.82E-03 = 0.0058214327 cm/s

Degree of Permeability: Medium (Soil testing for Engineerers by T. William Lambe, 1951)

Fig A7.1: Permeability Test Result of Sample A

COEFFICIENT OF PERMEABILITY (Falling Head)

Locaton ILASA EKITI

Sample No.: B

Soil Description : LIGHT RED

Date : 21-02-2017

Sample Dimensions: Diam. 10.2 cm;

Area,A 85.10 cm^2

Vol. 1106.30 cm^3

Ht.L 13 cm

Standpipe = Burette 50.1 ml Diam. 10.2 cm Area,a 1.130 cm^2

Test no.	h_1 ,cm	h_2 ,cm	t, s	Q_{in}, cm^3	Q_{out}, cm^3	T, °C	Test no.	h_1 ,cm	h_2 ,cm	t, s	T, °C
1	94.5	50	31.8		44.50	21	1	94.5	50	31.8	21
2	94.5	50	32.5		44.50	21	2	94.5	50	32.5	21
3	94.5	50	32.9		44.50	21	3	94.5	50	32.9	21
4	94.5	50	33.6		44.50	21	4	94.5	50	33.6	21
						Average		94.5	50	32.7	21

$\alpha = \eta_T/\eta_{20} =$ 0.9761

$kT = (aL/At)\ln(h_1/h_2) =$ 3.36E-03 = 0.0033604335 cm/s

$k_{20} = \alpha k_T =$ 3.28E-03 = 0.0032801192 cm/s

Degree of Permeability: Medium (Soil testing for Engineerers by T. William Lambe, 1951)

Fig A7.2: Permeability Test Result of Sample B

COEFFICIENT OF PERMEABILITY (Falling Head)

Location FAYEBODE EKITI

Sample No.: C

Soil Description : LIGHT RED

Date : 21-02-2017

Sample Dimensions: Diam. 10.2 cm;

Area,A 85.10 cm^2

Vol. 1106.30 cm^3

Ht.L 13 cm

Standpipe = Burette 50.1 ml Diam. 10.2 cm Area,a 1.130 cm^2

Test no.	h_1,cm	h_2,cm	t, s	Q_{in}, cm^3	Q_{out}, cm^3	T, °C	Test no.	h_1,cm	h_2,cm	t, s	T, °C
1	94.5	50	26.5		44.50	21	1	94.5	50	26.5	21
2	94.5	50	27.1		44.50	21	2	94.5	50	27.1	21
3	94.5	50	27.9		44.50	21	3	94.5	50	27.9	21
4	94.5	50	27.1		44.50	21	4	94.5	50	27.1	21
						Average		94.5	50	27.15	21

$\alpha = \eta_T/\eta_{20} =$ 0.9743

$kT = (aL/At)\ln(h_1/h_2) =$ 4.05E-03 = 0.0040473730 cm/s

$k_{20} = \alpha k_T =$ 3.94E-03 = 0.0039435313 cm/s

Degree of Permeability: Medium (Soil testing for Engineerers by T. William Lambe, 1951)

Fig A7.3: Permeability Test Result of Sample C

COEFFICIENT OF PERMEABILITY (Falling Head)

Location IKOLE EKITI

Sample No.: D

Soil Description : RED

Date : 21-02-2017

Sample Dimensions: Diam. 10.2 cm;

Area,A 85.10 cm^2

Vol. 1106.30 cm^3

Ht.L 13 cm

Standpipe = Burette 50.1 ml Diam. 10.2 cm Area,a 1.130 cm^2

Test no.	h_1,cm	h_2,cm	t, s	Q_{in}, cm^3	Q_{out}, cm^3	T, °C	Test no.	h_1,cm	h_2,cm	t, s	T, °C
1	94.5	50	20.8		44.50	22	1	94.5	50	20.8	22
2	94.5	50	22.1		44.50	22	2	94.5	50	22.1	22
3	94.5	50	23.2		44.50	22	3	94.5	50	23.2	22
4	94.5	50	24.4		44.50	22	4	94.5	50	24.4	22
						Average		94.5	50	22.625	22

$\alpha = \eta_T/\eta_{20} =$ 0.9499

$kT = (aL/At)\ln(h_1/h_2) =$ 4.86E-03 = 0.0048568476 cm/s

$k_{20} = \alpha k_T =$ 4.61E-03 = 0.0046135146 cm/s

Degree of Permeability: Medium (Soil testing for Engineerers by T. William Lambe, 1951)

Fig A7.4: Permeability Test Result of Sample D

APPENDIX B: GEOTECHNICAL TESTS RESULT GRAPHS

Appendix B1: Sieve Analysis Graphs

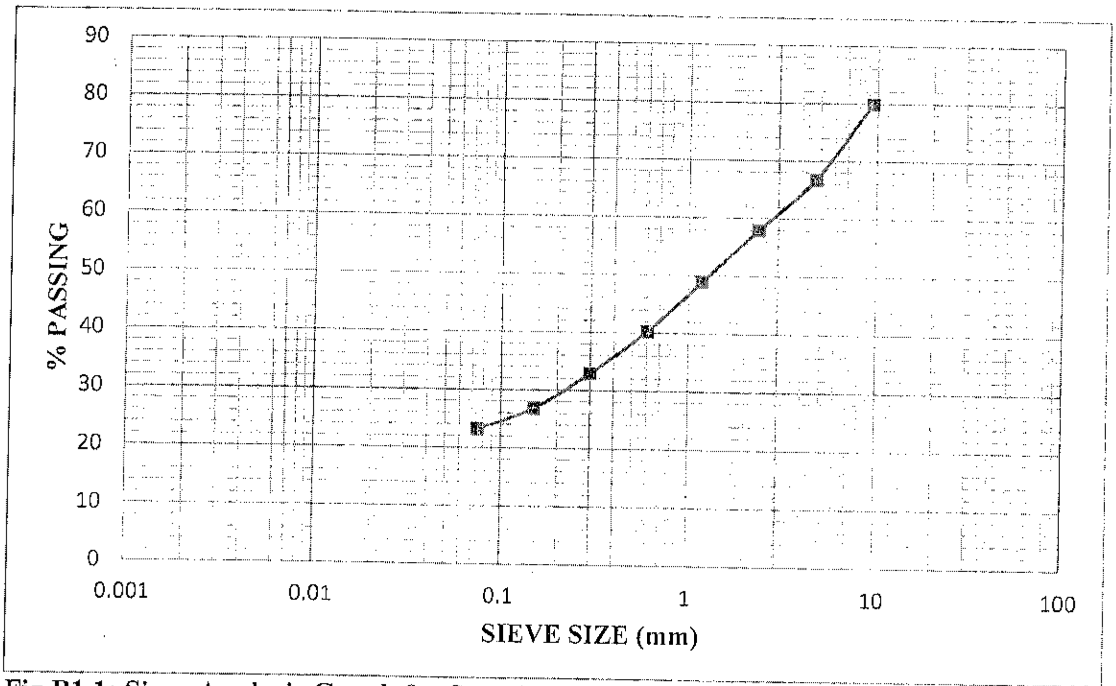

Fig B1.1: Sieve Analysis Graph for Sample A

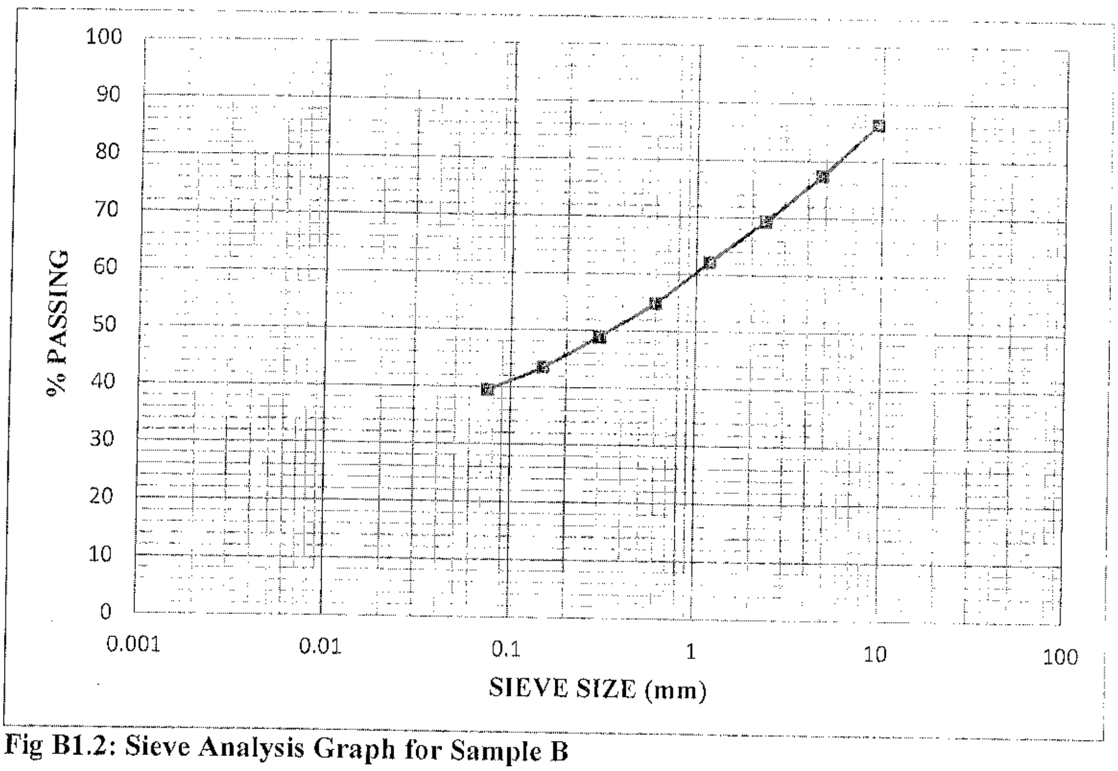

Fig B1.2: Sieve Analysis Graph for Sample B

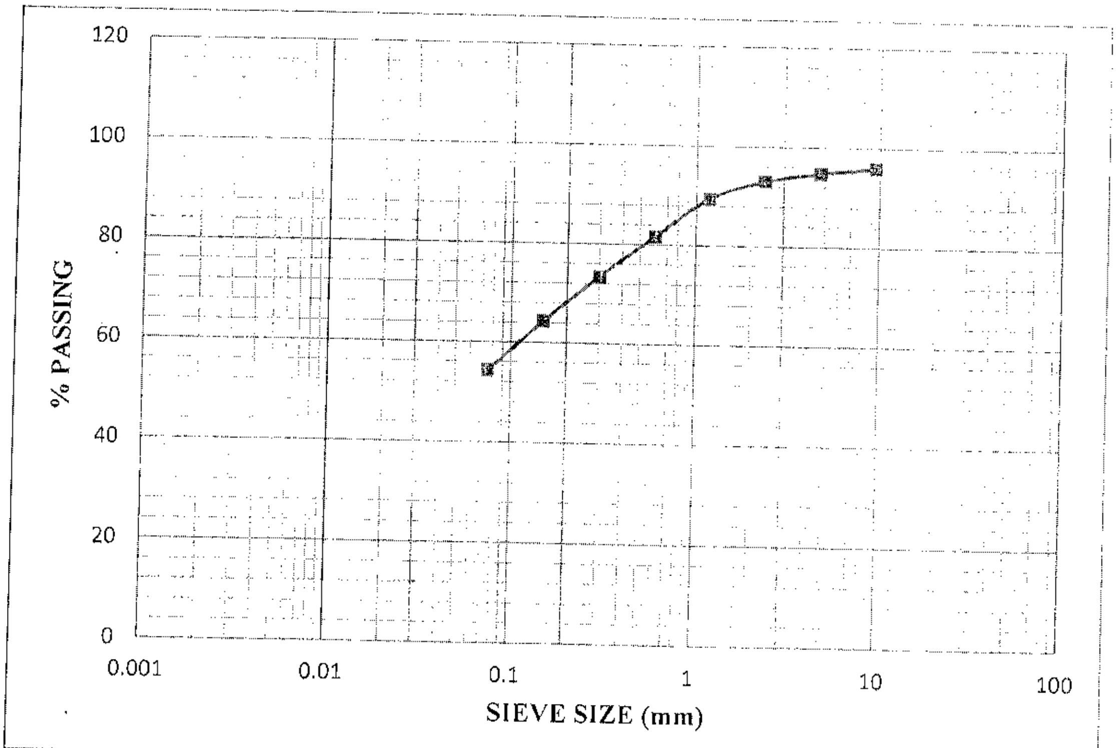

Fig B1.3: Sieve Analysis Graph for Sample C

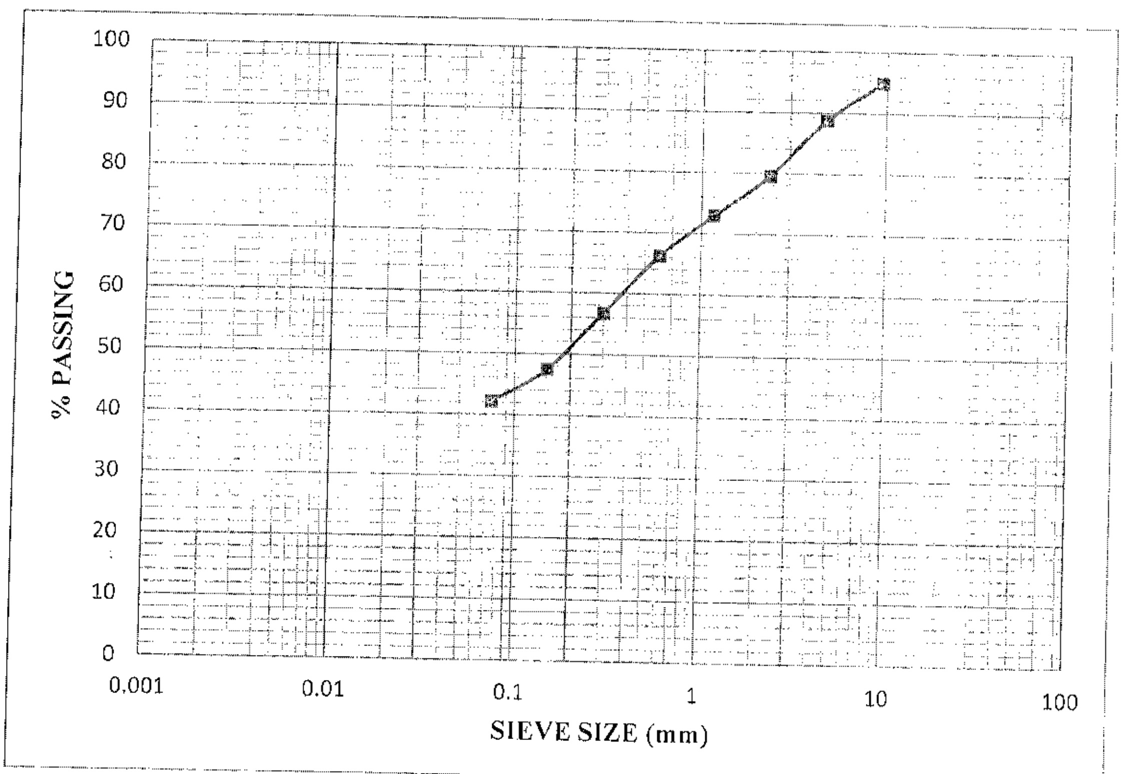

Fig B1.4: Sieve Analysis Graph for Sample D

Appendix B2: West African Compaction Graphs

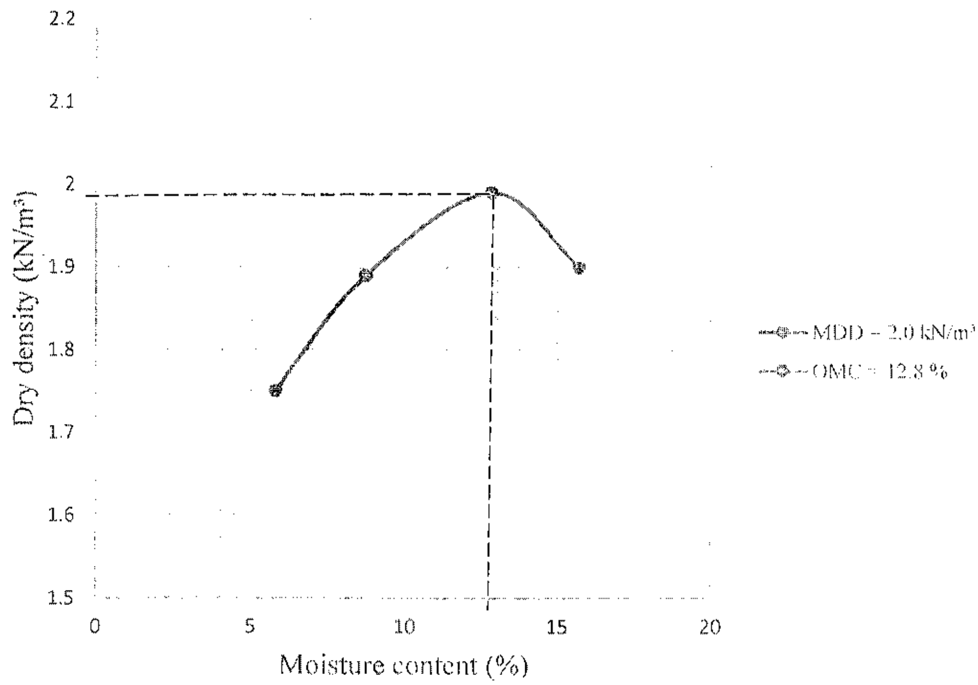

Fig B2.1: West African Compaction Graph for Sample A

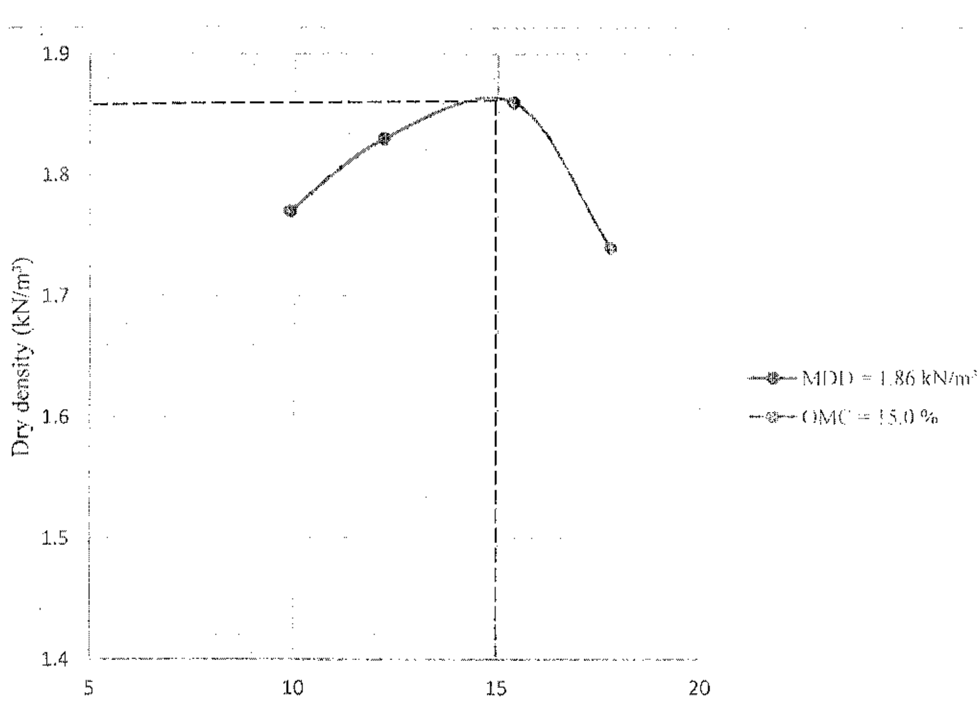

Fig B2.2: West African Compaction Graph for Sample B

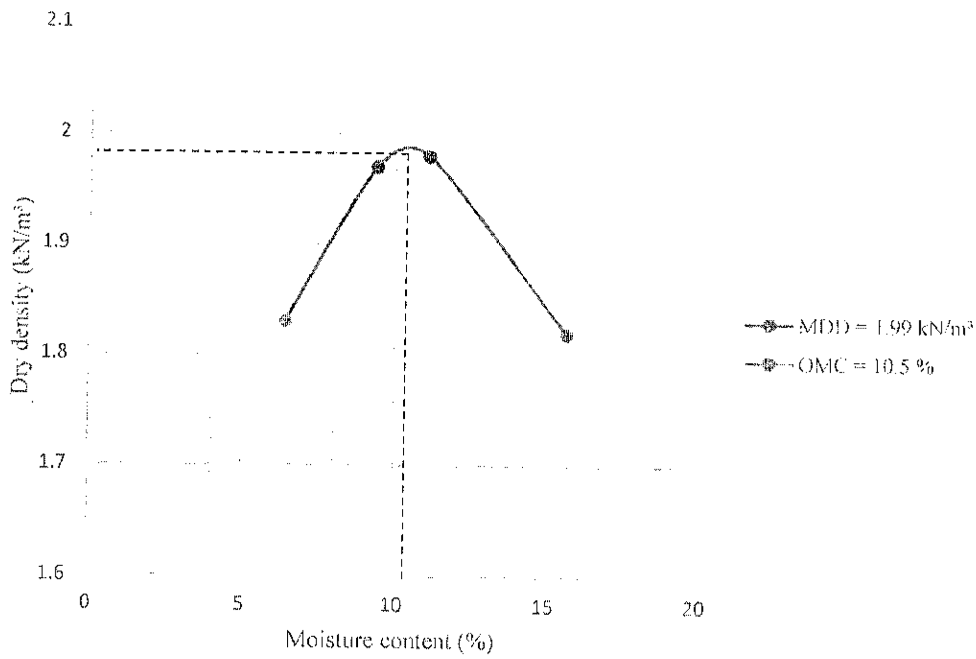

Fig B2.3: West African Compaction Graph for Sample C

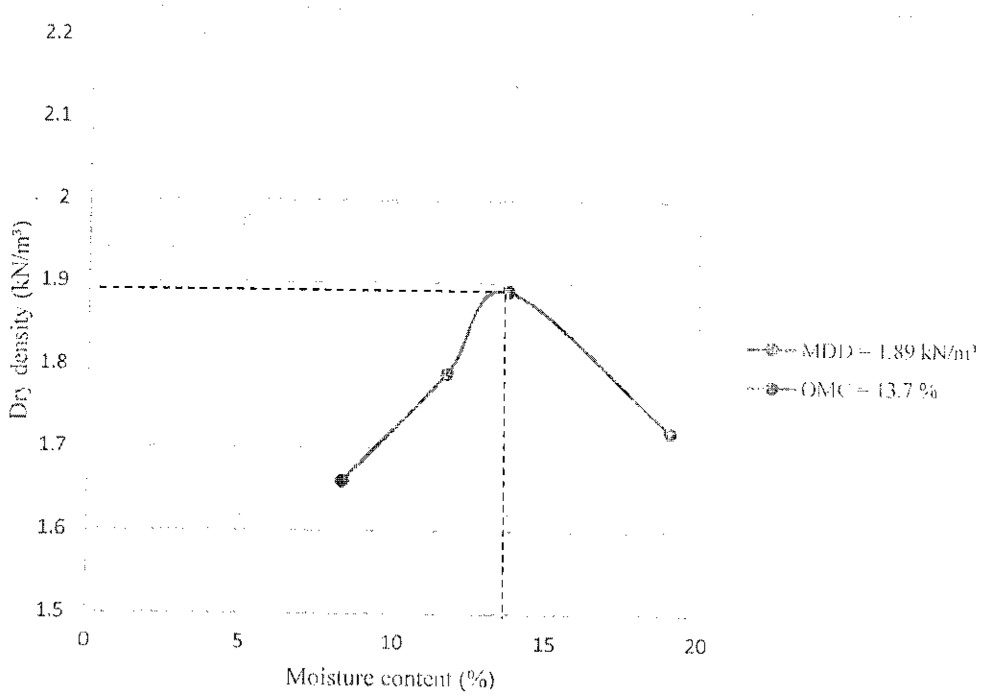

Fig B2.4: West African Compaction Graph for Sample D

Appendix B3: California Bearing Ratio Graphs

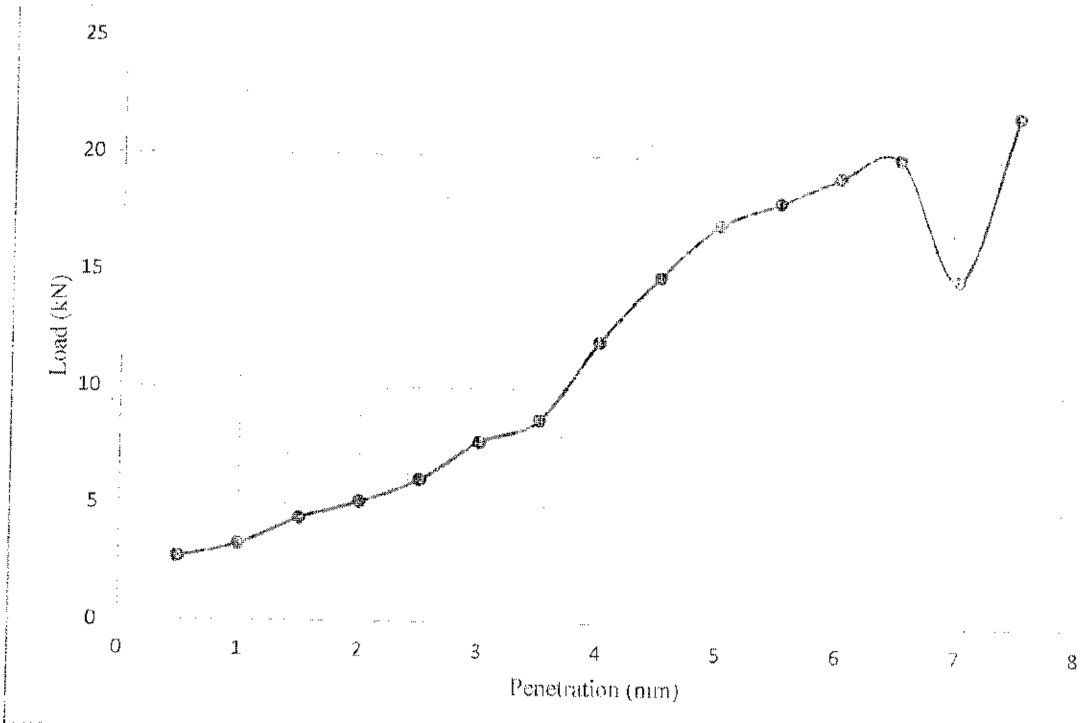

Fig B3.1: California Bearing Ratio Graph for Sample A

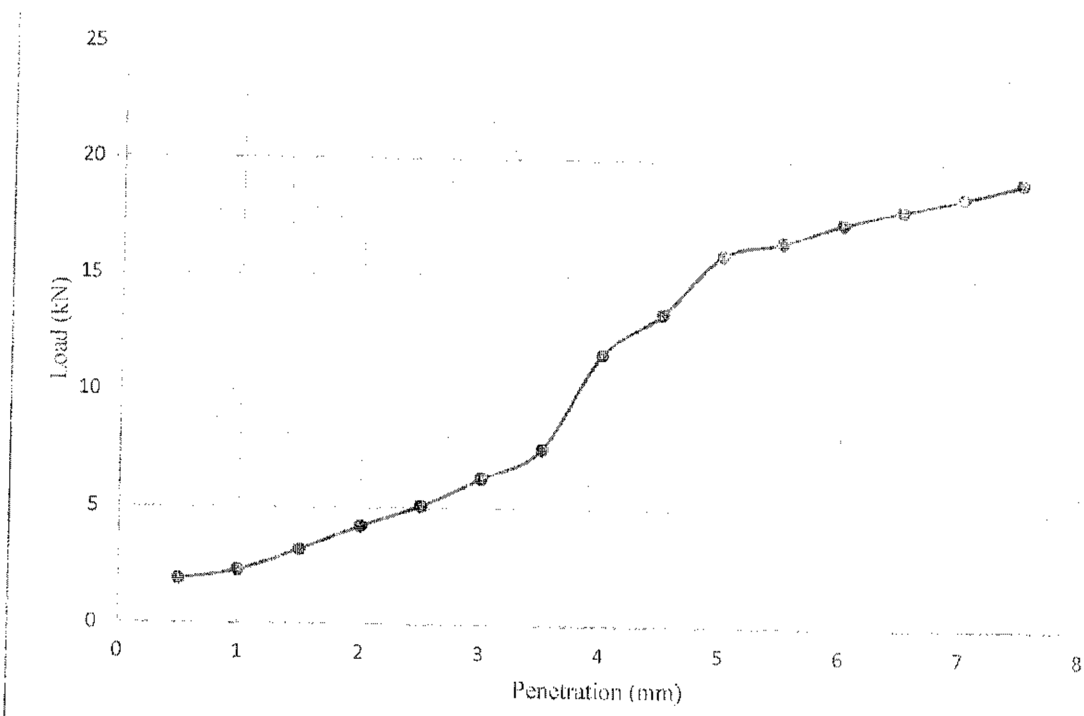

Fig B3.2: California Bearing Ratio Graph for Sample B

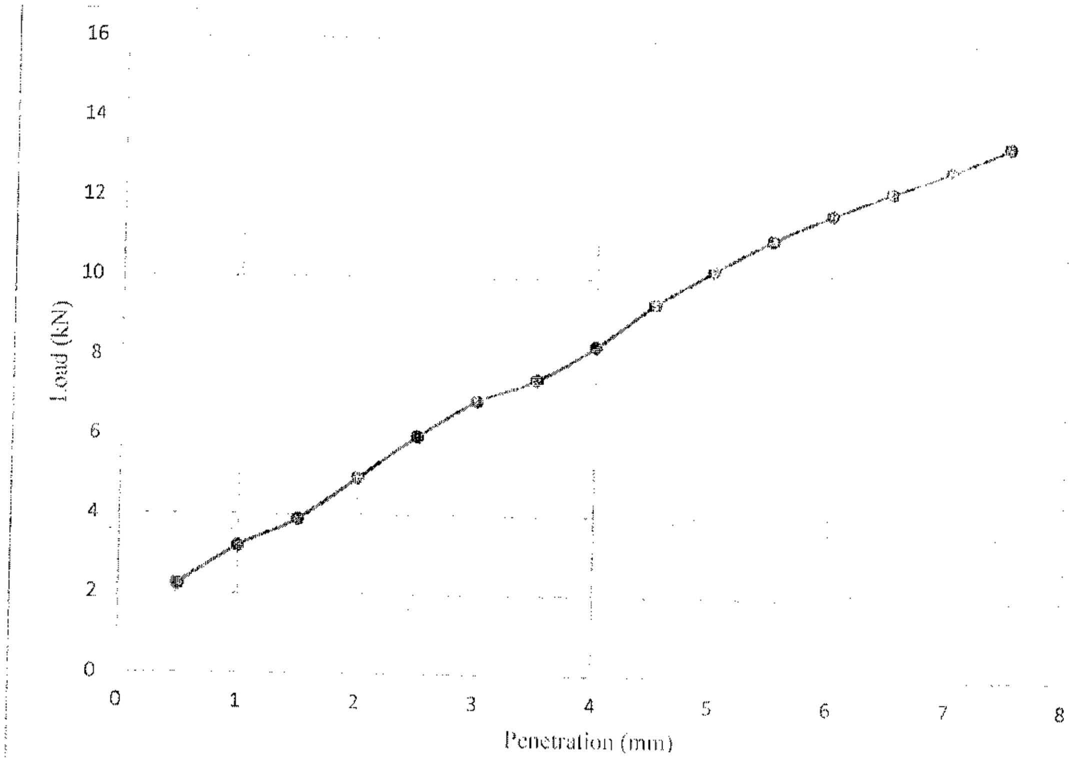

Fig B3.3: California Bearing Ratio Graph for Sample C

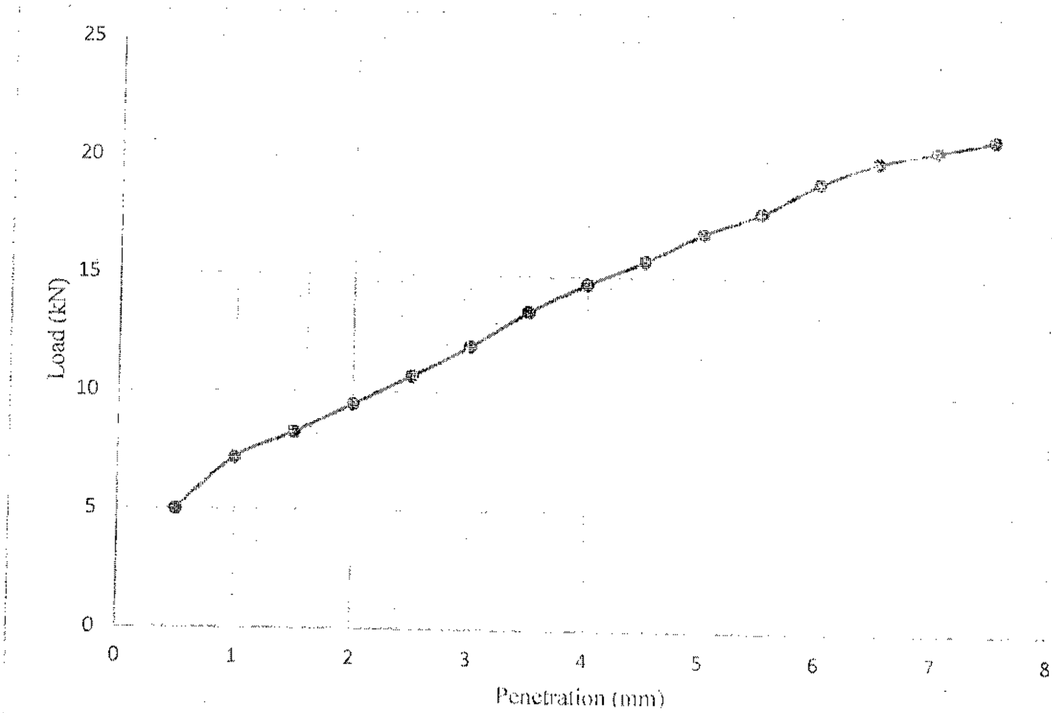

Fig B3.4: California Bearing Ratio Graph for Sample D

Appendix B4: Atterbergs Limit Graphs

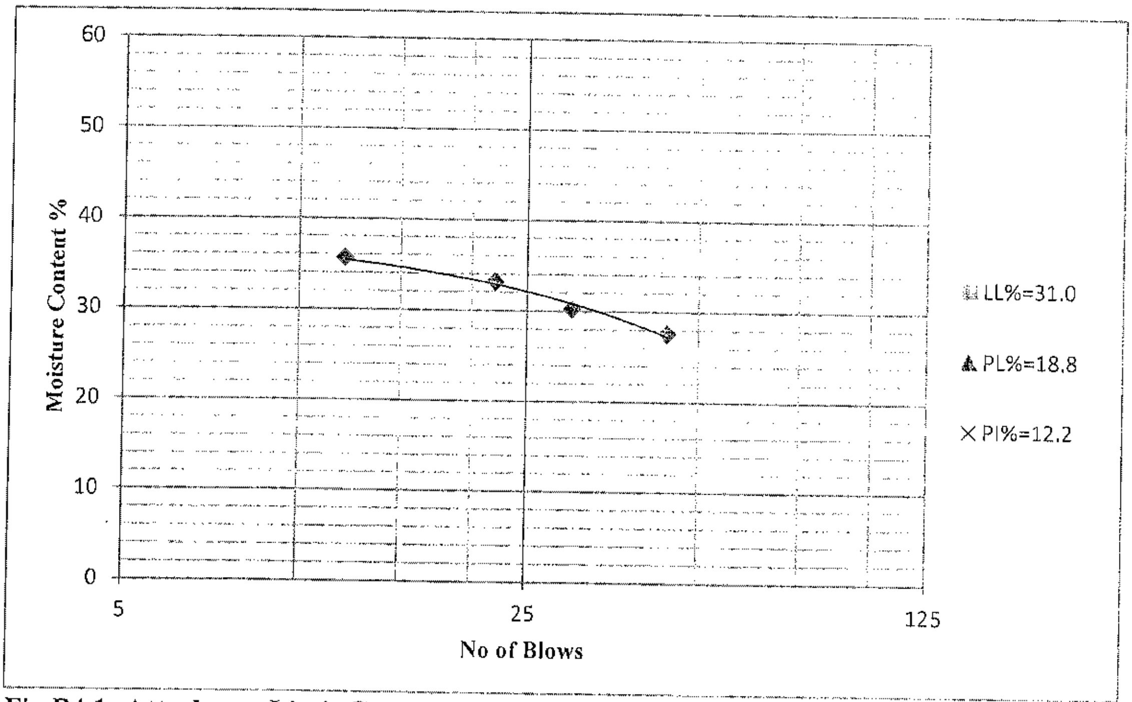

Fig B4.1: Atterbergs Limit Graph for Sample A

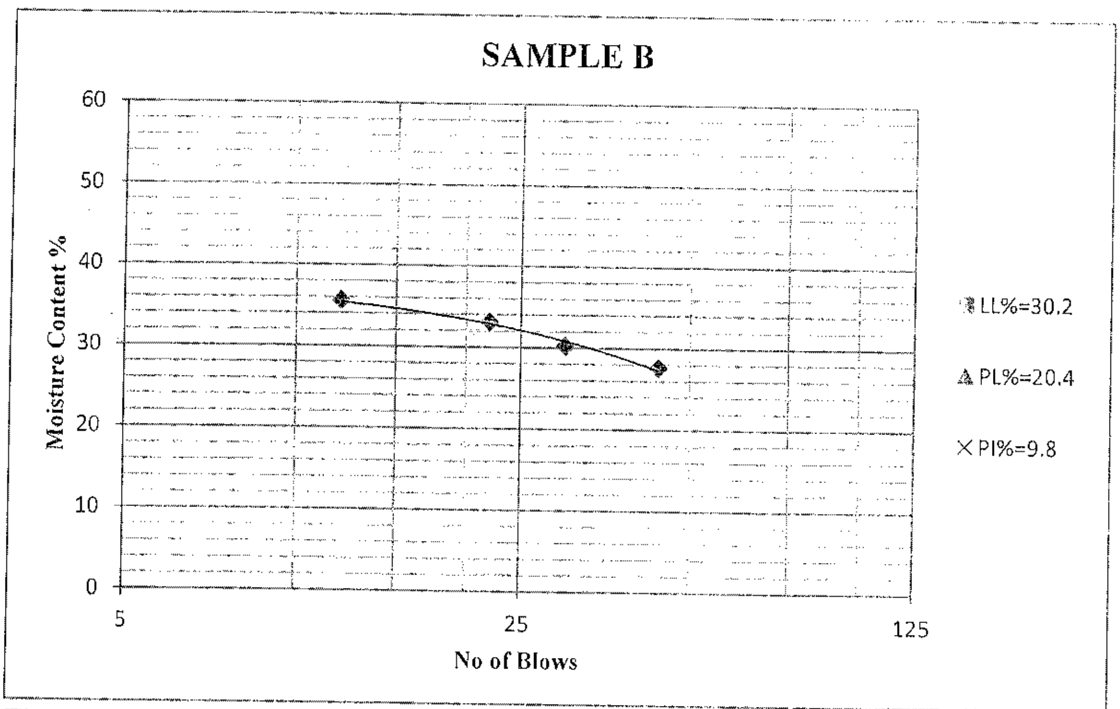

Fig B4.2: Atterbergs Limit Graph for Sample B

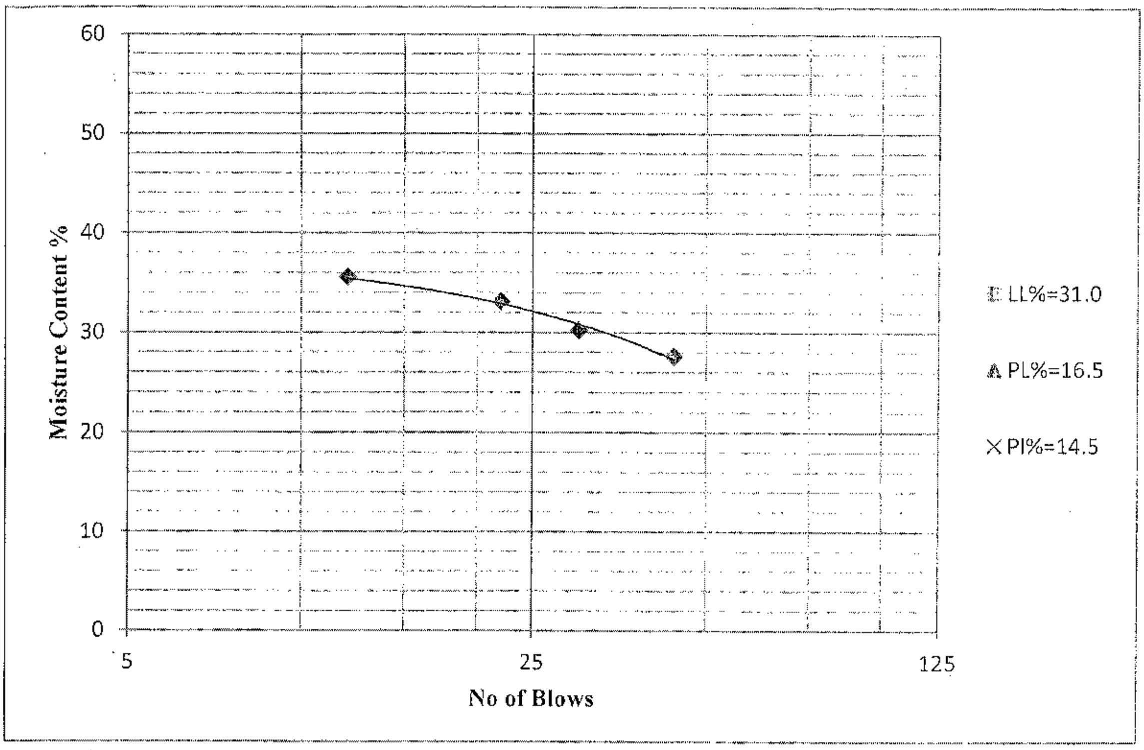

Fig B4.3: Atterbergs Limit Graph for Sample C

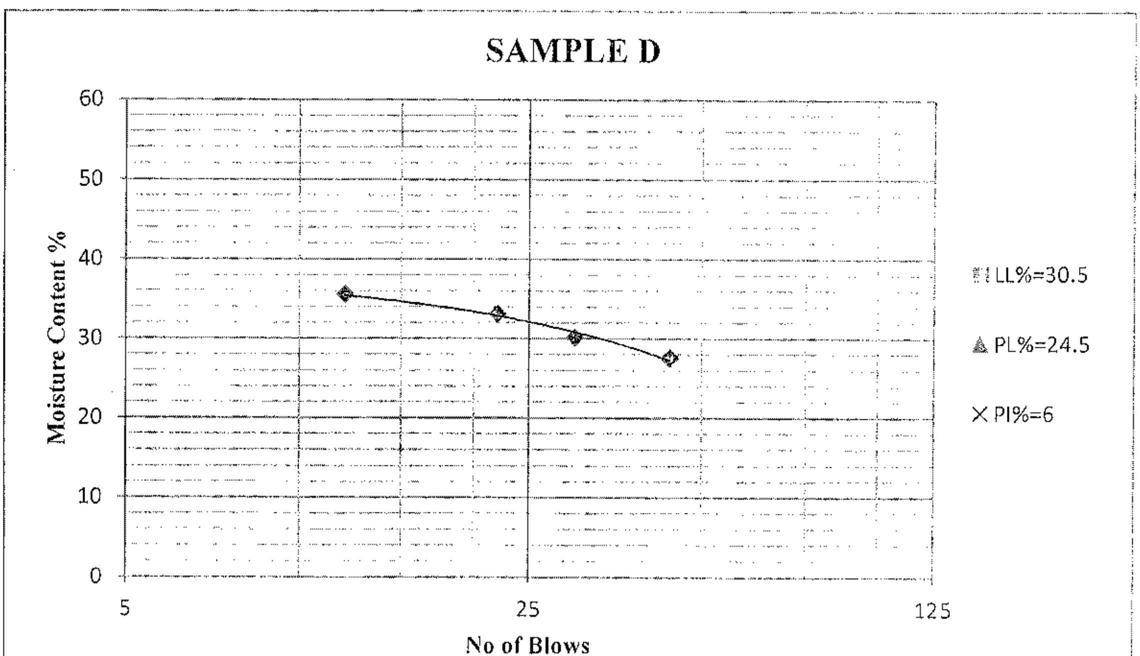

Fig B4.4: Atterbergs Limit Graph for Sample D