GEOTECHNICAL CHARACTERIZATION OF FAILED SECTIONS ALONG IKOLE – OYE EKITI ROAD, EKITI STATE, NIGERIA

BY

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A RESEARCH PROJECT SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENT FOR THE AWARDS OF BACHELOR OF ENGINEERING (B.ENG), DEGREE IN CIVIL ENGINEERING,

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ABSTRACT

This research project assesses the geotechnical characterization of soils used for the construction of the failed sections along Ikole- Oye Ekiti Road, Ekiti State, Nigeria. This road spans 23kms. The study area falls within coordinates 7.7979306° N (Oye) - 7.7982660 N (Ikole) and 5.3285505°E (Oye) - 5.5144930°E (Ikole). It is a known fact throughout the world that the conditions of any road largely depend upon its geotechnical properties such properties includes soil strength, compaction, particle size classification, etc. Four different locations were considered in which sampling point method was used in taking disturbed samples from the depth of 750mm.The coordinates of the locations are sampling point 1 (7.474693° N, 5.280855° E), sampling point 2 (7.483325° N, 5.240643° E), sampling point 3 (7.479756° N, 5.220363° E), sampling point 4 (7.480405° N, 5.189875° E). Laboratory tests was carried on the samples collected in order to determine soil geotechnical properties of Ikole- Oye road. The tests carried out are; Compaction test, Sieve analysis , Permeability test, Natural moisture content, Specific gravity, California bearing ratio test and Consistency limit test.

The results showed that the subgrade soil materials used along the road have high percentages of clayey and silt/gravel/sand materials respectively (rated as fair to poor materials for road use) based on AASHTO design standard (1986). The Natural and Optimum Moisture Contents are low for all the Base, Sub-base and Sub-grade courses samples which ranges between 7.7% and 21.8%. The Maximum Dry Density (MDD) values ranges between 1.59-2.0kg/m3 for subgrade , 1.66-2.02kg/m3 for sub-base and 1.70-2.02kg/m3 and Optimum Moisture contents values ranges between 14.5-19.6% for subgrade, 10-20.20% for sub-base and 12-15.5% for base materials. Subgrade course samples met the specified specification of MDD of not less than 1.76kg/m3 but sub-base and base materials does not meet the specification by AASHTO (1986) of MDD not less than 2.0kg/m3. From the tests result analysis of the Ikole-Oye road stretch within Ikole and Oye local Government areas, it appears the subgrade materials under the failed sections of the road is suitable according to AASHTO design standard (1986) while those of sub-base and base materials are not suitable. Therefore, It is imperative that sustainable solutions addressing the issue of using sub-standard materials for sub-base and base materials should be focused on, to curtail the incessant occurrence of road failures on the Ikole Oye road stretch.

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DEDICATION

I dedicate this project to Almighty God who has been my guidance and my provider throughout my study years in this great institution and to my wonderful family for all the love, care and support they have given me. I am forever grateful.

CERTIFICATION

This is to certify that this project was written by JIMOH, Wasiu Segun (CVE/12/0829) under my supervision and is approved for its contribution to knowledge and literary presentation. All sources of information are specifically acknowledged by means of references, in partial requirements for the award of Bachelor of Engineering (B.Eng) degree in civil engineering, Federal University Oye- Ekiti, Ekiti, Nigeria.

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Table of Contents

LIST OF ABBREVATIONS

AASHTO- American Association of State Highway and Transportation Officials

USCS- Unified Soil Classification System.

PL- Plastic Limit

LL- Liquid Limit

PI- plasticity Index

SL- Shrinkage Limit

SP- Sampling Point

LIST OF FIGURE

LIST OF PLATE

LIST OF TABLE

CHAPTER ONE

INTRODUCTION

1.1 Background to the Study

The first forms of road transport were horses, oxen or even humans carrying goods over tracks that often followed game trails, such as the Natchez Trace. In the Stone Age humans did not need constructed tracks in open country. The first goods transport was on human backs and heads, but the use of pack animals, including donkeys and horses, developed during the Stone Age. To date, road pavement is being used, road consist of pavement which is generally defined as the structural material placed above a sub-grade layer. In asphaltic pavement, it is typically a multi-layered system comprising the sub-grade (support), sub-base, base course and surfacing. Its principal function is to receive load from the traffic and transmits it through the layers to the sub-grade.

In the investigation conducted by Adeyeri et al. (2017) concluded that the soils around Ikole areas are mostly lateritic and are suitable as subgrade, sub-base and base course materials in highway construction.

The economy of any nation depends on the quality of her mode of transportation which involves movement of people and goods from one location to another. In countries where the development of these transportation infrastructures has followed a rational, coordinated and harmonized path, economic growth normally received a big boost, Beesley (1973). The problem of failed road pavement has made it difficult and expensive to move products and services from producer to consumers. This often leads to loss of human life, man-hours and high cost of goods and services. Road pavement failure can be attributed to engineering properties of subgrade materials, geology of the road route, hydrology/hydrogeology and geomorphology of the area traversed and usage factor.

Bad roads have become one of the identifying characteristics of Nigerian infrastructures. From the east to West and North to South, there has been tales of woes regarding the state of Nigerian roads. Reports and findings reveal that billions of naira is spent yearly on road construction and maintenance, Akinrolabu (2004). By the end of the first three years of the third republic for example, it was recorded that over three hundred and twenty billion naira had been awarded for road construction and maintenance, Ogundipe (2003). The problem of has been attributed mostly to the geotechnical property of the soils used for the construction. It is important to note that in many cases, the materials on which these roads were constructed were not in harmony with the road sub-grade

For several years now the Ikole- Oye road has failed and the reconstruction has lingered for so long. Several lives and properties of inestimable value have been lost as a result of frequent vehicle accidents, caused by this failed highway pavement. Factors responsible for road failures may include geological, geomorphological, geotechnical, road usage, design and construction inadequacies, and maintenance factors, Adegoke et al. (1980).

Field observations and laboratory experiments carried out by Adegoke et al (1980); Mesida (1981) and Ajayi (1987) showed that road failures are not primarily caused by usage or design construction problems alone, but can equally arise from inadequate knowledge of the characteristics and behaviour the subgrade on which the roads are built and non-recognition of the influence of geology and geomorphology during the design and construction phases thus the design of the roadway should be able to accommodate these factors, mainly climate and geology as they determine the actual behaviour of the roadway.

Failure or destruction of roads in Ekiti has become a serious challenge to the Ekiti government. Motor vehicles are forced to use other routes due to closure of roads. A good example is a road section along Ikole- Oye road Due to varying soil structures and different geological setting of the construction locations, it has been challenging for government road engineers to develop standard road designs, drainage design as well as maintenance techniques that can overcome these challenges. Salcon (1997) argue that, the strength of the road is depended on the load bearing capacity of the underlying soil which transmits the load to the parent rock. The type of soil is a very important determinant of road design procedures, general layout and construction methods. Therefore, to achieve reliable strata, excavation is usually carried out to allow transmission of the design load to the parent rock. When constructing roads, engineers are advised to consider building roads that can withstand low, moderate as well as heavy traffic levels for purposes of providing access for residents, timber harvesting, reforestation, rangeland management, and other multiple use activities such as the availability of some basic heavy equipment, such as bulldozers and graders. Soil structure is essential when constructing roads to reduce impairment of natural soil structure and interception of subsurface flow by the road cut slopes. It also increases or decreases shear strength that contributes to failure of roads. The soil type, strength and hardness determine the severity of road failure. The severity of road failure also depends on the drainage density of the watershed. Therefore, it is important to note that, the greater the intensity of storm events and the more drainage dissects the landscape, the more acute the necessity to plan for avoiding water impacts in constructing and stabilizing roads. The durability and stability of roads after construction depends on the proper soil compatibility. Low soil densities leads to structural failure of roads due to increased settlement and reduced shearing resistance.

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

A pavement section may be generally defined as structural materials placed above a subgrade layer, wood and Acdox (2002). In asphaltic pavement, it is typically a multi-layered system comprising the subgrade (support), sub base, base course and surfacing. Its principal function is to receive load from the traffic and transmits it through its layers to the subgrade, Kadiyali (1989). A pavement is said to have failed, when it can no longer perform this function during its design life. This Ikole- Oye road is characterized by failure of all kinds like potholes, cracks, depression, ruts, etc. and there is not just one reason for each type of failure. This makes it difficult for people to meet their access needs, because they are conformed to delays resulting from accident that have claimed lives of breadwinners of many homes.

In precious research, it was discovered that roads failed due to the following reasons:

1. traffic effects and human impacts on the roads, Paul and Radnor (1976)
2. Negligence of road maintenance, Madedor (1992).
3. Poor soil properties like low CBR, Jegede (2004)
4. Inadequacies in design and poor workmanship, Ogundipe (2007)
5. Poor drainage system, Lewis and Andrew (2007).

1.2 Statement of Problem

Travelling by road in most parts of Nigeria especially in the study area can be very worrisome as there exists many failed sections along the road. The road is made of flexible pavement and it is characterized by failure of all kinds like potholes, cracks, depression, ruts etc. and there is not just one reason for each type of failure. Generally, previous researches show that roads failed due to negligence of road maintenance, inadequacies in design and poor workmanship, poor soil properties like low CBR and high liquid limits etc. among others. This research intended to analyze, whether there are any relationships between the poor performance of the road and the geotechnical properties of the materials on which and with this road was constructed.

1.3 Justification

However, no previous attempt has been made to investigate the geotechnical characterization of failed sections along Ikole- Oye Ekiti road, Ekiti State. Hence, the justifications for this project research work. This investigation is being initiated because, there is little or no work done on the geotechnical characterization of failed sections along Ikole – Oye Ekiti road.

1.4 Study Area Accessibility and its Local Geology

The study area is the road between Oye and Ikole Ekiti highway, which is located within Latitudes 7.7979306° N (Oye) - 7.7982660 N (Ikole) and longitude of 5.3285505°E (Oye) - 5.5144930°E (Ikole). The geographical map of Ekiti State is shown in Figure. 1.1. This road is a federal road (Truck A) which runs 23km.There are three villages (Usin, Itapa and Ilupeju Ekiti) along this road as shown in Figure 1.2. This road connects several towns, villages, farm settlements and markets, it also serve as link to Abuja and other states which makes the road very busy all year round.

The study area is underlain by the Precambrian Basement Complex rocks as concluded by Oladapo and Ayeni, (2013). In their own work Adeyeri et al (2017), investigated the stratigraphic profile and geotechnical properties of soils in Ikole area of Ekiti State.

The site investigation revealed a subsoil stratification consisting of reddish brown granitic clayey sand (Laterite) top layer from existing ground level to about 12.0m depth.

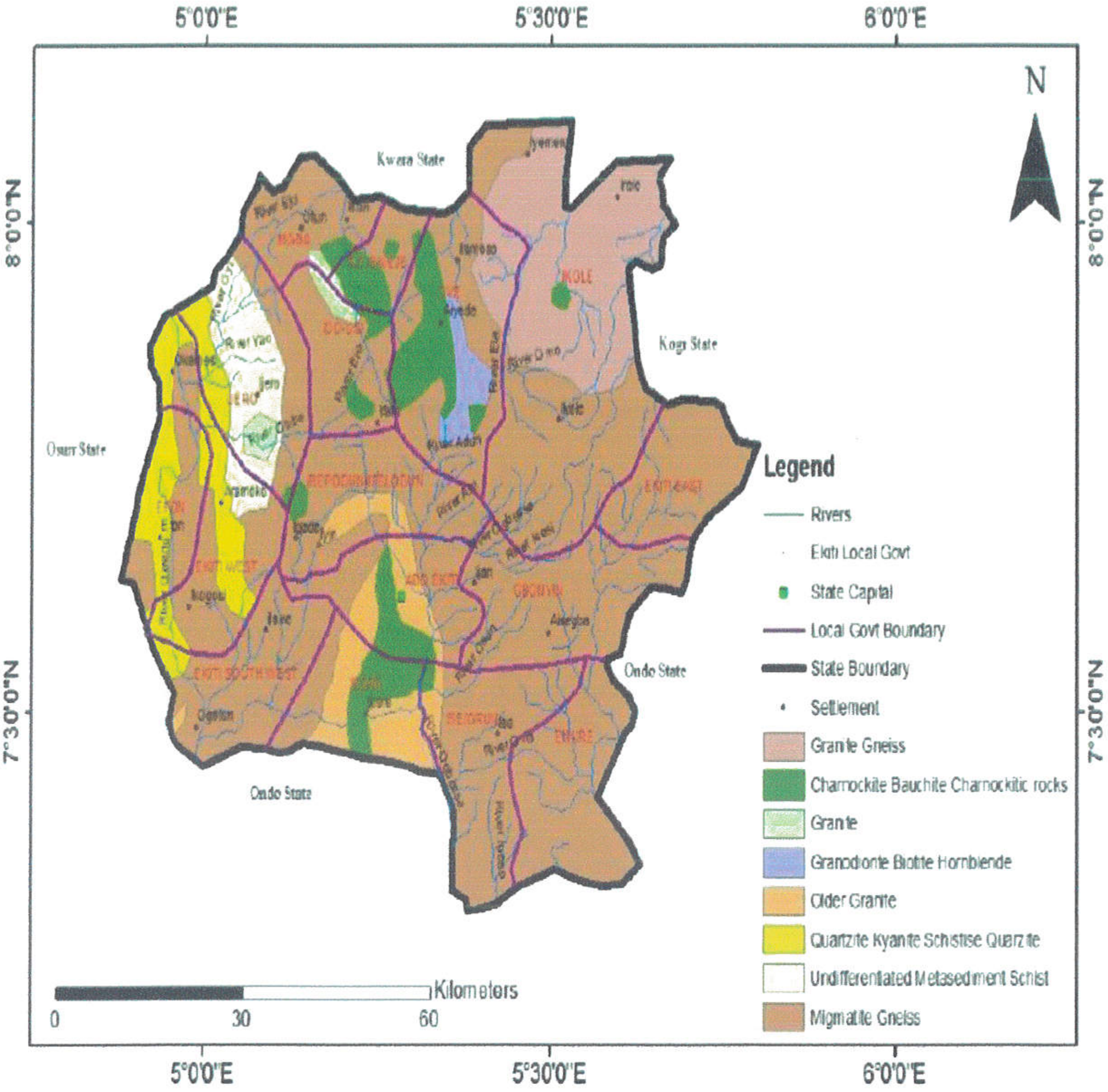


Figure 1.1 The Geological Map of Ekiti State, Ademilua (1997)

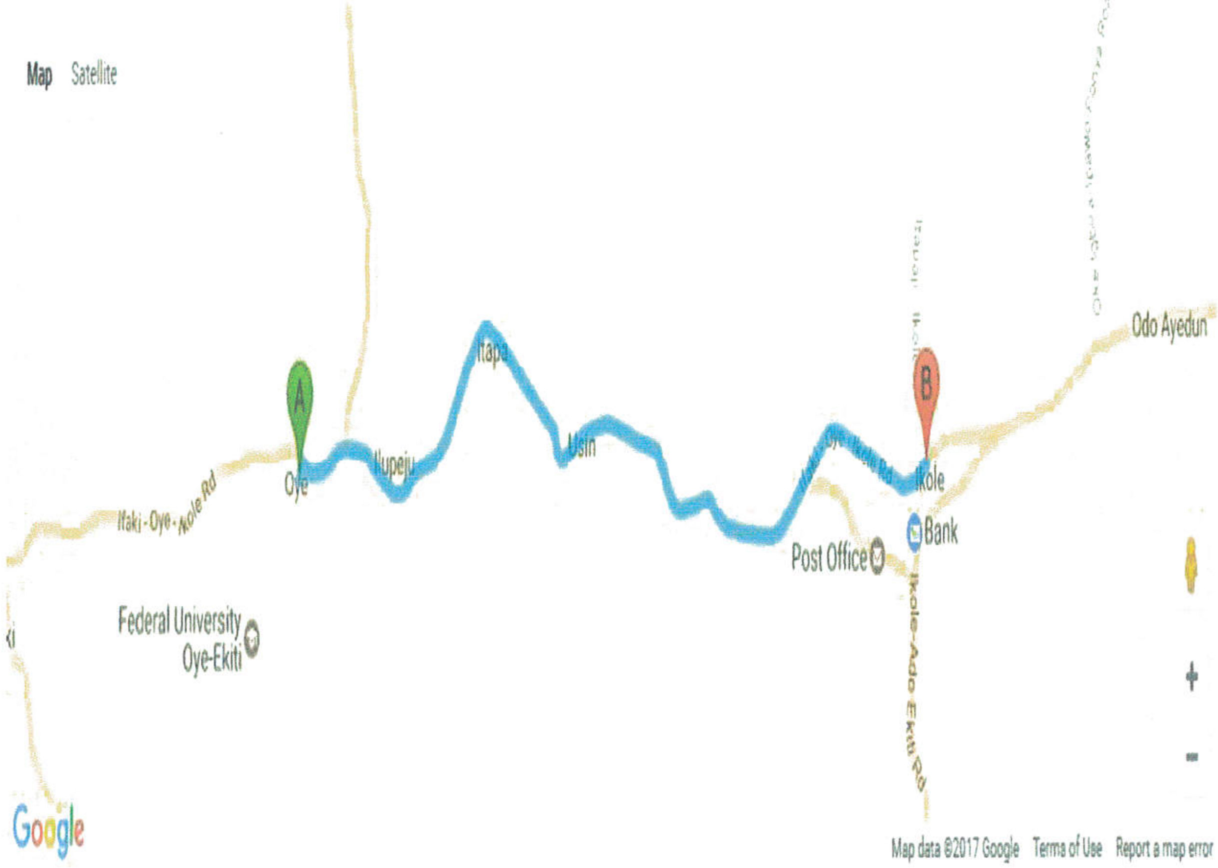


Figure 1.2 Ikole – Oye Ekiti highway map showing the study area (Online source distancesfrom.com, (2017)

1.5 Aim and Objectives

The aim of this research is to assess and analyze the geotechnical characterization of failed section along Ikole-Oye Ekiti, Ekiti State, Nigeria.

The objectives of this research are;

i. To carry out some geotechnical tests such as, specific gravity, particle size analysis, moisture content determination test, atterberg limit, direct shear, consistency test, compaction and California Bearing Ratio tests.
ii. To determine the constituent of materials used for the construction of this road
iii. To establish whether the geotechnical characteristics of the soil of the area is a factor for the road failure or not.

iv. To classify the soils according to the American Association of State Highway and Transport Officials (AASHTO).
v. To make recommendation based on the outcome of the tests and analysis.

1.6 Scope of study

This project research is divided into five chapters; the first chapter basically gives the general background of the study, problem statement, study area accessibility and its local geology, aim and objectives, scope of work, significance of the study and finally justification.

Chapter two contains a comprehensive review of previous works and similar studies mostly done in the same geological formation. Chapter three discusses the research methods applied in this project, breakdown of activities involved in the research work and their respective completion duration, mode of data collection, relevant laboratory or in-situ tests, analysis and discussion of results, observations from the obtained results, recommendations and seminar presentations.

Chapter four basically discusses the obtained results and analysis of same. Inferences are also drawn from the obtained results.

Lastly, chapter five gives the conclusions and recommendations based on results obtained from chapter four

1.7 Significance of the study

The results of this study will provide reliable technical information on the geotechnical properties of soil materials along Ikole- Oye Ekiti and their environs. It will also provide useful guidelines for Civil Engineers and Builders in selection of materials for the construction and rehabilitation of roads along Ikole – Oye Ekiti, Ekiti State.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

Geotechnical engineering is the branch of civil engineering concerned with the engineering behavior of earth materials. Geotechnical engineering is important in civil engineering, but also has applications in military, mining, petroleum and other engineering disciplines that are concerned with construction occurring on the surface (superstructure) or within (substructure) the ground. Geotechnical engineering uses principles of soil mechanics and rock mechanics to investigate subsurface conditions and materials; determine the relevant physical/mechanical and chemical properties of these materials; evaluate stability of natural slopes and man-made soil deposits; assess risks posed by site conditions; design earthworks and structure foundations; and monitor site conditions, earthwork and foundation construction, Holtz et. al. (1981)

A typical geotechnical engineering project begins with a review of project needs to define the required material properties. Then follows a site investigation of soil, rock, fault distribution and bedrock properties on and below an area of interest to determine their engineering properties including how they will interact with, on or in a proposed construction. Site investigations are needed to gain an understanding of the area in or on which the engineering project will be built. Investigations can include the assessment of the risk to human being, property and the environment from natural hazards such as earthquakes, landslides, sinkholes, soil liquefaction, debris flows and rock falls, Jon W. et al. (1989) involve foundation and anchor systems for offshore structures such as oil platforms.

The fields of geotechnical engineering and engineering geology are closely related, and have large areas of overlap. However, the field of geotechnical engineering is a specialty of soil engineering, where the field of engineering geology is a specialty of geology, Bowles (1981).

2.2 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, Moulton (1980). 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.

2.3 Structural Layers of Road Pavement

The road layers consist of three tiers; a surface course, a binder course and a base course together these constitute the top layer of the road structure, Serfass and Courteille (1980). There is a wide range of surface course products available, and these wearing mixtures must be designed to have sufficient stability and durability to withstand the appropriate traffic loads and the detrimental effects of environmentally induced stresses such as air, water and temperature changes, while in other cases the wearing course should be impermeable, to keep water out of the road structure Moulton (1980). Moulton notes further that, the binder course is an intermediate layer. It is designed to reduce rutting and withstand the highest stresses that occur at about 50-70mm below the surface course layer.

The sub base and sub grade layers constitute the foundations of the road structure, and since the formation and subsoil often comprise of relatively weak materials, it is of utmost importance that the damaging loadings are effectively eliminated by the layers above. These sub base layers consist of unbound materials, such as indigenous soils, crushed or uncrushed aggregate, or reused secondary material.

Moulton (1980) stated that the layers of road comprises of;

i. The Sub- Grade

ii. The Sub-Base

iii. Road Base

iv. Surfacing (wearing course)

2.3.1 Sub-grade

According to Youder and Witczack (1975) subgrade is describe as a natural materials on site that bears the load of pavement and traffic load in other to reduce the effective thickness, the bearing capacity is reduced by

a. Proper compaction
b. stabilization
c. Proper drainage

2.3.2 Sub- base

This is introduced due to poor bearing capacity of the sub grade soil or high traffic density.it is made to improve earths. Its functions is to transmit the traffic load from the road and spreading as jet over a large area of the sub grade formation level, Youder and Witczack (1975).

2.3.3 Road base

The function of the base is bearing the late load from the traffic and long heed from the over wearing surface and spreading it uniformly over a large area of the sub-grade or sub-base, Shahin, et al. (1984).

2.3.4 Surfacing (wearing course)

The wearing course is to spread the wheel load to the road base against surface water. The presence of bitumen improves the water proofing property. It also provide skid resistance, Youder and Witczack (1975). 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)

2.3.5 Types of Road Pavement

According to Mwangi (2013) he stated that there are two main types of pavement

i. Flexible pavement
ii. Rigid pavement

2.4 Performance and Serviceability of a Road

Road performance is defined as the ability of a pavement to satisfactorily serve traffic over time, AASHTO (2003). Serviceability on the other hand, refers to the ability of a road to serve the traffic for which it was designed. Integrating both definitions will give a new understanding of the performance which can be interpreted as the integration of the serviceability over time, Youder and Witczack (1975).

Performance is a broad, general term describing how road condition changes or how pavement structures serve their intended functions with accumulating use, George, et al (1989). To measure and predict the performance of any road, a repeatable, well established and field calibrated condition rating system must be adopted, Shahin, et al. (1984). Several methods and approaches have been developed to measure the pavement performance. A road is a very sophisticated physical structure that responds in a complex manner to the external traffic and environmental loading. This is mainly due to the non-homogenous composition of the asphalt mixture, aggregate and sub grade soil, and the vast variation in traffic and environmental characteristics from a region to another. In the study area, asphalt roads and pavements demonstrated different types of both structural and functional distresses as a result of the combined effect of traffic and climate. In Kenya most roads deteriorate due to high axle loading and lack of proper drainage and road maintenance (Mwangi, 2013).

Therefore it's important that roads and drainage systems monitored, scheduling the maintenance and rehabilitation works. Road performance depends on several factors but this project concentrates on the effects that inadequate drainage systems has on roads and the surrounding environment.

2.5 Mechanism of Failure of Flexible Pavement

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 capillary action, thereby reducing the supporting power of the read pavement. According to Pavement Design and Evaluation Committee (1965), the performance and life of flexible pavements are governed by failures which may attributed to:

2.5.1 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, are put in the hands of people with little or not 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.

2.5.2 Poor Maintenance Operations/Functions

According to Paul, and Radnor (1976), road maintenance includes both physical maintenance, activities such as patching, filling of joints, moving, and also 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 the maintenance man is 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.

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.

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 labor to ensure efficient and regular road maintenance.

2.5.3 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 (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

The number and nature of repetition of the loading suffice to say that all these factors are rampant along the Ikole-Oye expressway because of its position as the major road through which goods move from southeast to north.

2.5.4 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 waterlogged 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 250c 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. All these causes lead to cracking on the pavement which leads to potholes.

2.5.5 Poor Drainage System

Drainage according to Oxford English Dictionary (2007) refers to emptying accomplished by draining. Drainage system on the other hand according to the English Dictionary, refers to a system of watercourses or drains for carrying off excess water. Adequate drainage is essential in the design of roads since it affects the highways serviceability and usable life, including the roads and pavement's structural strength. If pounding on the travelled way occurs, hydroplaning becomes an important safety concern, Lewis and Andrew (2007).

Drainage design involves providing facilities that collect, transport and remove water from the road. The design must also consider the water reaching the roadway embankment through natural stream flow or manmade ditches, Shahin et al. (1984) Proper road drainage is absolutely critical if we expect roads to stand up to the damaging effects of weather and traffic. For long term non deteriorating roads cannot be built without providing good drainage. However not all water can be termed to be bad for the road, UNH, (2009).

2.5.6 Poor Geotechnical Characteristics of the Soil (subgrades, Sub-bases 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 subgrade) 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.

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.

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 montmorillonite (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.

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.

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.

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.

Adiat et al. (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 Soil

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

2.7 Geotechnical Properties of Soil

Geotechnical properties include all geologic earth materials which may undergo laboratory analysis before any civil engineering construction takes place. Geotechnical analysis is required because it provides useful information on foundation soils before any civil engineering projects are carried out. Engineering geologist, geotechnical engineers, geomorphologist among other professionals play an integral role in modern engineering project this is because report on geotechnical analysis make them aware of problem- soil with a view to avoid structural failure, defects or collapse of civil engineering projects.

2.7.1 Strength

The strength of a soil measures its ability to withstand stresses without collapsing or becoming deformed, Brady and Weil (1996). Soil strength can be considered in terms of the capacity of a soil to withstand normal and/or shear stresses. Shear stress can be resisted only by the skeleton of solid particles, by means of the forces developed at the inter-particle contacts. Normal stress may be resisted by the soil skeleton due to an increase in the inter-particulate forces. If the soil is fully saturated, the water filling the voids can also withstand normal stress by an increase in pressure, Craig (1992). A soil's ability to withstand normal stresses can be influenced by a number of related soil characteristics, amongst which are:

- Bearing resistance
- Soil compressibility; and
- Soil compactability.

These factors in turn are determined by parameters such as soil moisture content, particle size distribution and the mineralogy of the soil particles. In general, coarser textured materials have greater soil strengths than those with small particle size, Brady and Weil (1996). For example, quartz sand grains are subject to little compressibility, whereas silicate clays are easily compressed.

The bearing capacity of the material can be important both in terms of long-term engineering performance to carry loads and also supporting heavy plant in the short-term.

2.7.2 Compaction

According to A. Bolarinwa et al. (2017), Compaction is an artificial process, which basically involves densification of the soil mass through reduction of air in voids of the soil mass while the latter is a natural process of gradual reduction in volume of the soil mass (settlement) through expulsion of the excess pore water in the soil over a period of time.

Compaction is the process of increasing the density of a soil by packing the particles closer together with a reduction in the volume of air; there is no significant change in the volume of water in the soil. In general, the higher the degree of compaction, the higher the shear strength will be and the lower the compressibility of the soil Craig (1992).

The bulk density of a material is defined as the mass of a material (including solid particles, any contained water and any fluid stabiliser) per unit volume including voids. The dry density (ρ_d) is the mass of material after drying to constant mass at 105°C, and after removal of any fluid stabilisers, contained in unit volume of un-dried material (BS 1924: Part 1: 1990). The dry density of a material can be determined for a given compaction at varying moisture contents. This will determine the optimum moisture content at which a specified amount of compaction will produce a maximum dry density.

2.7.3 Particle Size

Particle size is defined as the percentages of various grain sizes present in a material as determined by sieving and sedimentation (British Standard BS 1924: Part 1: 1990). BS 1924: Part 1: 1990 identified three classes of stabilised material depending on their particle size. These are shown in Table 2.1. Any material is regarded as belonging to the finest-grained group appropriate under the definitions given. Materials that contain large or irregular shaped particles can be difficult to test in the laboratory, and in the field they are likely to cause damage to the mixing plant. BS 1924: Part 1: 1990 stated that materials containing greater than 10% retained on the 37.5mm test sieve cannot be fully examined by the majority of test procedures given in that standard. This problem can be overcome by pre-screening to remove the large pieces or crushing the larger particles to within acceptable limits. The fine and medium-grained materials can be further classified as shown in Table 2.1. The grading of the material to be stabilised can influence the strength gain properties of the treated material. Well-graded materials have been found to exhibit a linear increase in unconfined compressive strength (UCS) with increased addition of cement binder (and lime binder before all the clay minerals have reacted)

Table 2.1 Classification of Materials Based on Particle Size Distribution, Source: BS 1924: Part 1: 1990

Class	Definition
Fine-grained materials	Containing less than 10% retained on a 2mm test sieve
Medium-grained materials	Containing more than 10% retained on a 2mm test sieve but not more than 10% retained on a 20mm test sieve
Coarse-grained materials	Containing more than 10% retained on a 20mm test sieve but not more than 10% on a 37.5mm test sieve.

Table 2.2 Soil Classifications and Properties, Source: Townsend, (1973)

Grain size	Coarse sand	Fine sand	Silt	Clay
Maximum (mm)	2	0.2	0.06	0.002
Average number of particles per g	350	350 000	3 × 10^8	3 × 10^11
Average surface area per g (cm^2)	40	400	4000	60 000
Typical mineralogical make-up	Quartz, feldspars, rock fragments	Quartz, feldspars, ferro-magnesium minerals	Quartz, feldspars, ferro-magnesium minerals, heavy minerals	Quartz, feldspars, secondary clay Minerals
General Characteristics	Loose grained, non-sticky, air in pore space of moist sample Visible to the naked eye.	Loose grained, non-stick, no air in pore space of moist sample, visible to the naked eye.	Smooth and flourlike, non-cohesive, Microscopic	Sticky and plastic, microscopic to sub microscopic, exhibit Brownian movement
Implications for Stabilization/Solidification (s/s)	Likely to be easily mixed. Potential for increased permeability (over	Likely to be easily mixed. Potential for Increased permeability (over well graded/fine grained	Sensitivity to moisture change needs to be addressed at design.	Uniform mixing may be difficult, but clay is easily stabilized. Clay minerals can react with binders to form cementitious products.

The mean particle size is not reported to affect this phenomenon; therefore a linear increase in strength can be expected for either clays or gravels. However, uniformly graded materials are identified as the exception to this linear behaviour when smaller quantities of binder are added. Sherwood (1993) suggested that this is due to the binder acting as filler in uniformly graded materials. Once the binder has improved the grading of the material Sherwood (1993) reported a linear increase again.

2.7.4 Cohesion and Plasticity

The properties of clay minerals give unique engineering properties to clay soils: cohesion and plasticity. Cohesive material can be defined as all material which, by virtue of its clay content, will form a coherent mass. Non-cohesive (granular) material will not form a coherent mass (BS 1924: Part 1: 1990). Where soils that are predominantly coarse-grained contain sufficient fine grains to show apparent cohesion and plasticity, they will be classified as fine soils (BS 5930: 1999). As a consequence, a cohesive soil can comprise less than 10% clay-sized particles.

Knowledge of the cohesiveness of a soil assists in the selection of Stabilisation/Solidification (S/S) treatment methods. Due to the poor mixing characteristics of cohesive material, treatment using ex-situ (e.g. pug mill) S/S techniques may not be possible, without the inclusion of a lime-treatment step. The addition of lime to cohesive soils can result in a decrease in plasticity due to the flocculation of clay particles as well as a longer-term pozzolanic reaction. The initial change in plasticity can significantly improve the workability of the material, enabling existing treatment techniques to be used. The plasticity of a fine-grained soil can be measured by its Atterberg limits. The plastic limit is defined as the moisture content at which soil changes in texture from a dry granular material to a plastic material that can be moulded. With increasing moisture content a cohesive material becomes increasingly sticky, until it behaves as a liquid. The point at which this phenomenon occurs is known as the liquid limit. The range of moisture content between the plastic limit (PL) and the liquid limit (LL) is defined as the plasticity index (PI) i.e. LL – PL = PI. These concepts are illustrated in Figure 2.1.

The transition points are fairly arbitrary, determined by index tests described in BS 1377-2:1990, but they do serve a valuable function in the classification of cohesive soils. With an increase in moisture content, granular soils pass rapidly from a solid to a fluid condition. In these circumstances the PL and LL cannot be identified and such soils are classified as non-plastic (Sherwood, 1993).

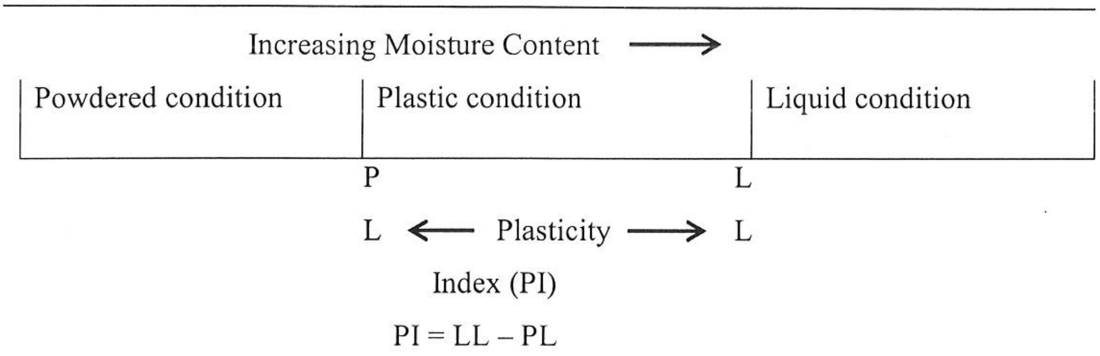


Figure 2.1 Definitions of soil Plasticity, Sherwood (1993)

Cohesive soils may be classified according to their plasticity properties. Silts have low plasticity indices, which mean that they quickly become difficult to handle once the moisture content exceeds the plastic limit. With increasing clay content in a soil, both the plastic limit and the liquid limit increases. The difference between the two limits may widen due to the activity of the clay minerals present, Sherwood (1993) and Cernica (1995). The activity of clay minerals can be related to plastic index, fineness of clay particles and behavioural tendency to volume changes Cernica (1995).

Cohesive soils characteristically have high plasticity indices. Stavridakis and Hatzigogos (1999), state that in soils containing expansive clay minerals with high liquid limits (40- 60%), the liquid limit can be used to gauge the amount of cement required to stabilise the soil. Although soils with liquid limits >60% can be stabilised, the amounts of cement required can be uneconomical and result in unacceptable volume increase.

2.7.5 Moisture Content

The moisture content of a soil is the ratio of the mass of water to the mass of solids in the soil, Craig (1992). The moisture content is determined as the mass of free water that can be removed from a material, usually by heating at 105°C, expressed as a percentage of the dry mass (BS 1924: Part 1: 1990). If a soil or waste contains too much water then the porosity and permeability are likely to increase. If the amount of moisture present in a soil is above optimum then the density of the compacted product is reduced and this may have an impact on the strength achieved in an S/S product. It is often necessary to adjust the moisture content in soils prior to

S/S and this can be achieved by stockpiling and draining with time, by the addition of lime or by blending the soil with other materials. Alternatively, water can be added to soil that is too dry. Drying soils with lime is commonly undertaken and it was traditional practice to allow a clay-lime mix to stand for a period of typically 24 h, either in a stockpile or for single layer treatment in situ, in order that complete lime distribution could occur. Current thinking, however, suggests that immediate water content adjustment and compaction is more beneficial in achieving a long-term strength gain .Glendenning et al. (1998). Boardman (1999) stated that immediate compaction would undoubtedly be beneficial for contaminated soil treatment, as long as thorough mixing is possible, since the pozzolanic reaction bonds that form at an early stage would assist with contaminant retention and minimise the flow of water through the stabilised material.

2.7.6 Permeability

The term permeability expresses the coefficient of permeability or hydraulic conductivity, describing the rate (ms^{-1}) at which water can flow through a permeable medium. Permeability is related to the distribution of particle sizes, particle shape and soil structure. In general, the smaller the particles, the smaller the average size of the pores and the lower is the coefficient of permeability. The transport of water through a soil will be faster if the soil has a higher coefficient of permeability than if it has a lower value, Craig (1992). However, it should be noted that the rate of transport of contaminants depends upon a number of factors including solubility and the rate at which contaminants are attenuated in a soil. The determination of the coefficient of permeability using the constant head method or in a cell under known effective stress conditions are described in BS 1377: 1990.

2.7.7 Specific Gravity

Specific gravity of a substance denotes the number of times that substance is heavier than water. In simpler words it can be defined as the ratio between the mass of any substance of a definite volume divided by mass of equal volume of water. In case of soils, specific gravity is the number of times the soil solids are heavier than an equal volume of water.

2.7.8 Consistency

Chew et.al. (2004) examined the relationship between the microstructure and engineering properties (Atterberg limits and unconfined compressive strength among others) of cement – treated marine clay. It has been concluded that the multitude of changes in the properties and behavior of cement – treated marine clay can be explained by four microstructural mechanisms. In soils, strength is measured in terms of shear strength. Soils do not generally have much, if any, strength in tension due to the particulate composition of soils. Shear strength in soils is the resistance to shear deformation of the soil mass and is described by internal angle of friction and cohesion. Shear strength in soils results from particle interlocking, particle interference, and sliding resistance, Terzaghi and Peck (1948).

Internal angle of friction (φ) is a function of mineralogical composition, shape, gradation, void ratio, and organic content of the soil and is measured in degrees, Holtz and Kovacs (1981) , Coduto (1999). The contribution of friction angle to the shear strength of a soil is a function of the vertical effective stress at a given point in the soil.

2.7.9 Effect of Geotechnical Properties on Soils

Cyril et al. (2016) were able conclude based on a study performed on the "Geotechnical Investigation and 2D Electrical Resistivity Survey of a Pavement Failure in Ogbagi Road, Southwestern Nigeria" that the possible causes of the highway pavement failure in a typical basement complex area result from Clayey topsoil/subgrade soils tendency of absorbing water which makes them swell and collapse under imposed wheel load stress which subsequently lead to road failure (July 2016).

Kekere et al. (2012) mentioned conclusively in a research conducted on "Relationship between Geotechnical Properties and Road Failures along Ilorin – Ajase Ipo Road Kwara State, Nigeria" that geotechnical properties of the foundation of the road have significantly affected the rate of road failure along Ilorin- Ajase-Ipo road. Results have indicated that geotechnical properties were not properly analyzed before construction started to identify areas with problem soils which are threatening the road today with various forms of failures. It is also evidently clear that, the presence of clayey soil and sandy soil which were poorly graded have caused cracks, bulges which result to series of potholes and depression on the road.

However, poor engineering construction also contribute to the rate of failure, it has been observed that the bituminous pavement of the road falls between 45-50mm which is far below engineering specification of 150-200mm British standard for flexible pavement cited in O'Flaherty (2001). Absence of drainage facility to discharge concentration of run-off especially during wet season and where drainage facilities is present, it is completely covered with sediments, the concentration of run off on the road also affects compaction rate of the road foundation hence weaken the stability of the foundation of the road (2012).

2.8 Classification of Soil for Highway Use

Soil classification is a method by which soils are systematically categorized according to their probable engineering characteristics. It therefore serves as a means of identifying suitable sub-base materials and predicting the probable behavior of a soil when used as subgrade material. The classification of a given soil is determined by conducting relatively simple tests on disturbed samples of the soil; the results are then correlated with field experience. Note, however, that although the engineering properties of a given soil to be used in highway construction can be predicted reliably from its classification, this should not be regarded as a substitute for the detailed investigation of the soil properties.

Classifying the soil should be considered as a means of obtaining a general idea of how the soil will behave if used as a subgrade or sub-base material. The most commonly used classification system for highway purposes is the American Association of State Highway and Transportation Officials (AASHTO) Classification System. The Unified Soil Classification System (USCS) also is used to a lesser extent. A slightly modified version of the USCS is used fairly extensively in the United Kingdom.

2.8.1 AASHTO Soil Classification System

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

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

where GI -group index

F - Percent of soil particles passing 0.075 mm (No. 200) sieve in whole number based on material passing 75 mm (3 in.) sieve

LL - liquid limit expressed in whole number

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

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

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

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

1. Soils classified as A-1-a, A-1-b, A-2-4, A-2-5, and A-3 can be used satisfactorily as subgrade or sub-base material if properly drained. In addition, such soils must be properly compacted and covered with an adequate thickness of pavement (base and/or surface cover) for the surface load to be carried.
2. Materials classified as A-2-6, A-2-7, A-4, A-5, A-6, A-7-5, and A-7-6 will require a layer of sub-base material if used as subgrade. If these are to be used as embankment materials, special attention must be given to the design of the embankment.
3. When soils are properly drained and compacted, their value as subgrade material decreases as the GI increases. For example, a soil with a GI of zero (an indication of a good subgrade material) will be better as a subgrade material than one with a GI of 20 (an indication of a poor subgrade material)

Table 2.3 AASHTO Soil Classification System

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

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

[a] If PI ≤ LL − 30, the classification is A-7-5.

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

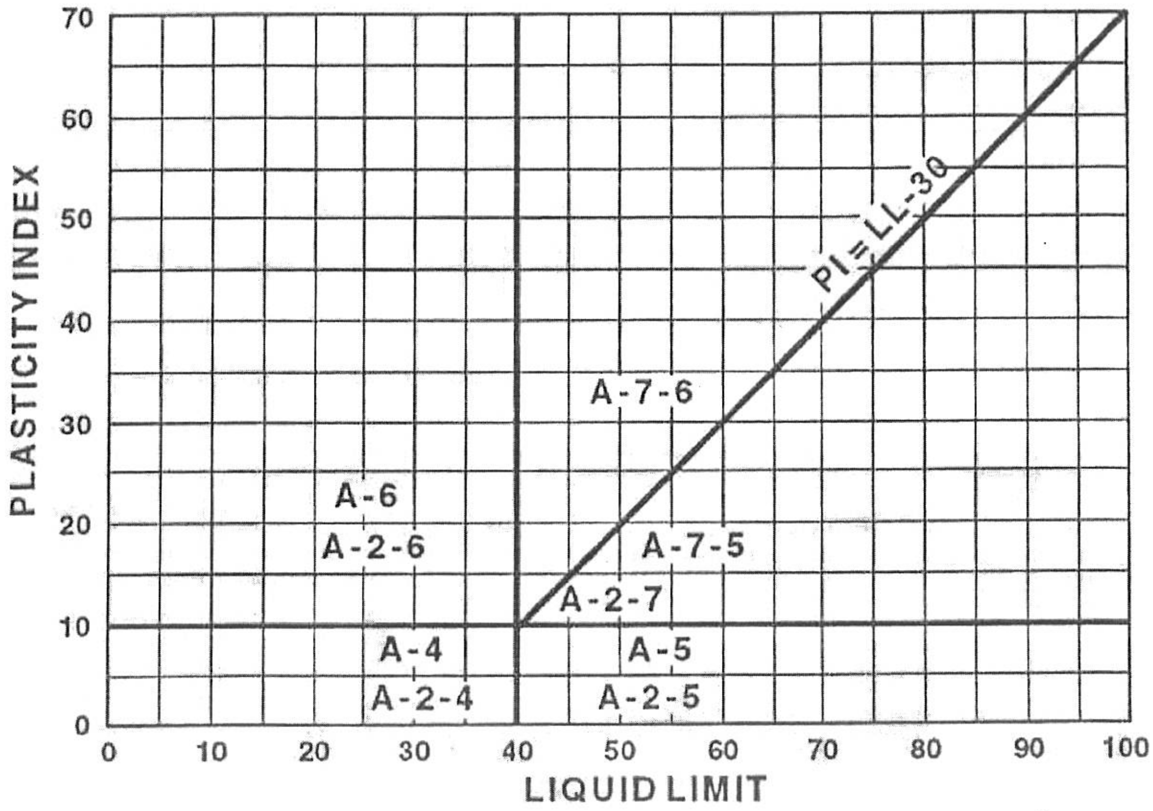


Figure 2.2 Relationship between Liquid Limit and Plasticity Index for Silt-Clay Groups (AASHTO M 145-91)

2.8.2 Unified Soil Classification System (USCS)

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

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

Table 2.4 Classification of Four major groups of materials

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

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

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

and

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

Where

$D60$ = Grain diameter at 60% passing

$D30$ = Grain diameter at 30% passing

$D10$ = Grain diameter at 10% passing

Gravels are described as well graded if Cu greater than four and Cc is between one and three. Sands are described as well graded if Cu greater than six and Cc is between one and three. The fine-grained soils, which are defined as those having more than 50 percent of their particles passing the No. 200 sieve, are subdivided into clays (C) or silt (M),depending on the PI and LL of the soil. A plasticity chart, shown in Table 2.3, is used to determine whether a soil is silty or clayey. The chart is a plot of PI versus LL, from which a dividing line known as the "A" line, which generally separates the more clayey materials from the silty materials, was developed. Soils with plots of LLs and PIs below the "A" line are silty soils, whereas those with plots above the "A" line are clayey soils. Organic clays are an exception to this general rule, since they plot below the "A" line. Organic clays, however, generally behave similarly to soils of lower plasticity.

Classification of coarse-grained soils as silty or clayey also depends on their LL plots. Only coarse-grained soils with more than 12 percent fines (that is, passes the No. 200 sieve) are so classified (see Fig. 2.3). Those soils with plots below the "A" line or with a PI less than four are silty gravel (CM) or silty sand (SM), and those with plots above the "A" line with a PI greater than seven are classified as clayey gravels (GC) or clayey sands (SC).

The organic, silty, and clayey soils are further divided into two groups, one having a relatively low LL (L) and the other having a relatively high LL (H). The dividing line between high LL soils and low LL soils is arbitrarily set at 50 percent. Fine-grained soils can be classified as either silt with low plasticity (ML), silt with high plasticity (MH), clays with high plasticity (CH), clays with low plasticity (CL), or organic silt with high plasticity (OH).

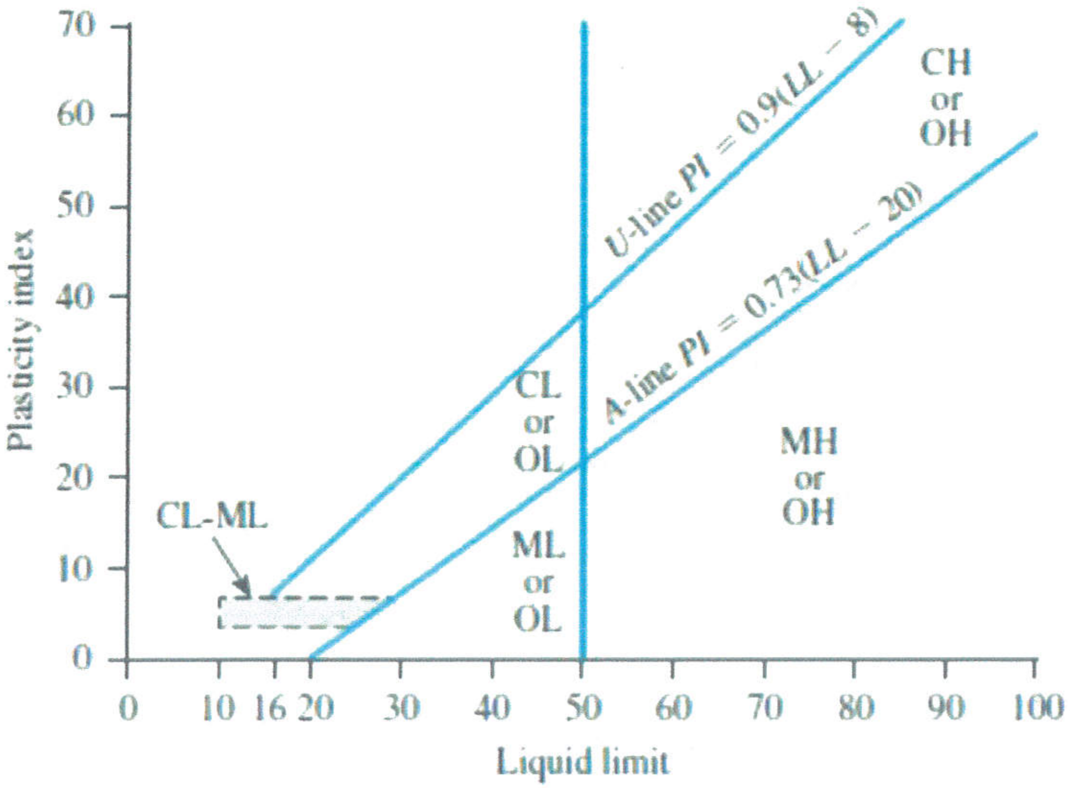
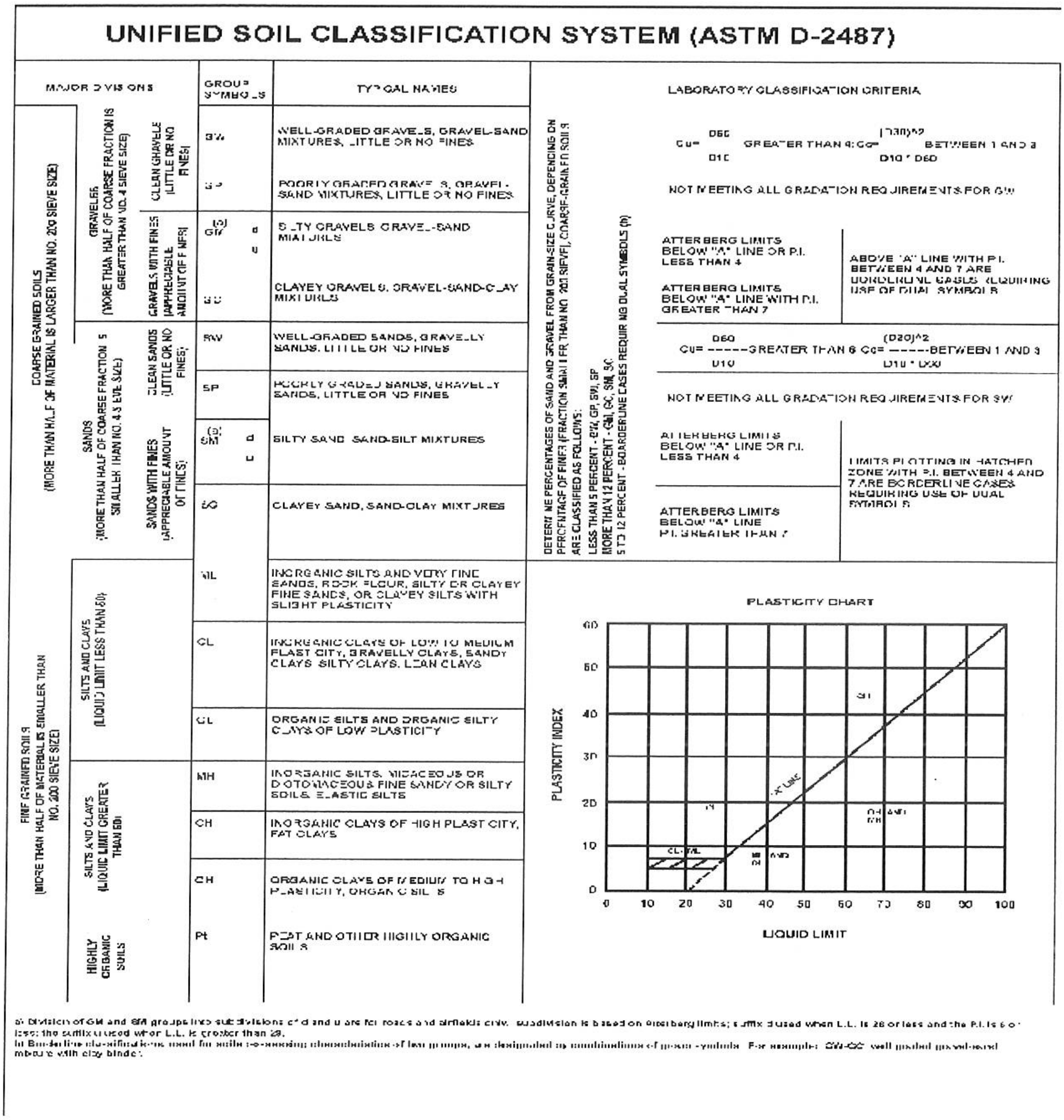


Figure 2.3 Plasticity Chart of USCS, Joseph E (1978)

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

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

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



UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES		LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE)	GRAVELS (MORE THAN HALF OF COARSE FRACTION IS GREATER THAN NO. 4 SIEVE SIZE)	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	DETERMINE PERCENTAGES OF SAND AND GRAVEL FROM GRAIN-SIZE CURVE, DEPENDING ON PERCENTAGE OF FINES (FRACTION SMALLER THAN NO. 200 SIEVE), COARSE-GRAINED SOILS ARE CLASSIFIED AS FOLLOWS: LESS THAN 5 PERCENT - GW, GP, SW, SP MORE THAN 12 PERCENT - GM, GC, SM, SC 5 TO 12 PERCENT - BOARDERLINE CASES REQUIRING DUAL SYMBOLS (b)	$C_u = \frac{D60}{D10}$ GREATER THAN 4; $C_c = \frac{(D30)^2}{D10 * D60}$ BETWEEN 1 AND 3	
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		NOT MEETING ALL GRADATION REQUIREMENTS FOR GW	
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM (a) d u	SILTY GRAVELS, GRAVEL-SAND MIXTURES		ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4	ABOVE "A" LINE WITH P.I. BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS BELOW "A" LINE WITH P.I. GREATER THAN 7	
	SANDS (MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE)	CLEAN SANDS (LITTLE OR NO FINES)	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		$C_u = \frac{D60}{D10}$ GREATER THAN 6 $C_c = \frac{(D20)^2}{D10 * D30}$ BETWEEN 1 AND 3	
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		NOT MEETING ALL GRADATION REQUIREMENTS FOR SW	
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM (a) d u	SILTY SAND, SAND-SILT MIXTURES		ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4	LIMITS PLOTTING IN HATCHED ZONE WITH P.I. BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
			SC	CLAYEY SAND, SAND-CLAY MIXTURES		ATTERBERG LIMITS BELOW "A" LINE P.I. GREATER THAN 7	
FINE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE)	SILTS AND CLAYS (LIQUID LIMIT LESS THAN 50)		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY		PLASTICITY CHART	
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS			
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY			
	SILTS AND CLAYS (LIQUID LIMIT GREATER THAN 50)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS			
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS			
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS			
	HIGHLY ORGANIC SOILS		Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS			

a) Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.

b) Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well graded gravel-sand mixture with clay binder.

2.9 Optimum Moisture Content

The determination of the optimum moisture content of any soil to be used as embankment or subgrade material is necessary before any field work is commenced.

Most highway agencies now use dynamic or impact tests to determine the optimum moisture content and maximum dry density. In each of these tests, samples of the soil to be tested are compacted in layers to fill a specified size mold. Compacting effort is obtained by dropping a hammer of known weight and dimensions from a specified height a specified number of times for each layer. The moisture content of the compacted material is then obtained and the dry density determined from the measured weight of the compacted soil and the known volume of the mold. The soil is then broken down or another sample of the same soil is obtained. The moisture content is then increased and the test repeated. The process is repeated until a reduction in the density is observed. Usually a minimum of four or five individual compaction tests are required. A plot of dry density versus moisture content is then drawn from which the optimum moisture content is obtained. The two types of tests commonly used are the standard AASHTO or the modified AASHTO.

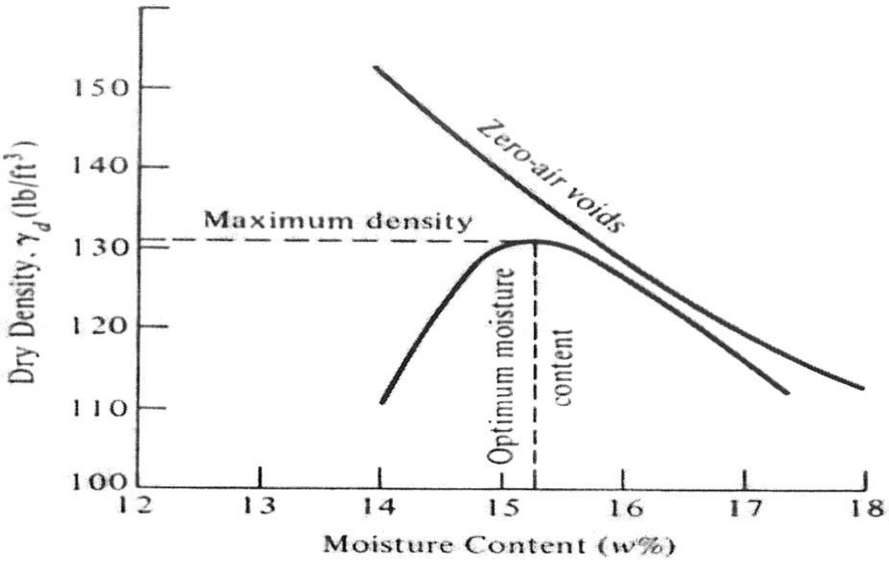


Figure 2.4 Typical Moisture-Density Relationship for Soils, Joseph E (1978)

2.10 Special Soil Tests for Pavement Design

Apart from the tests discussed so far, there are a few special soil tests that are sometimes undertaken to determine the strength or supporting value of a given soil if used as a subgrade or sub-base material. The results obtained from these tests are used individually in the design of some pavements, depending on the pavement design method used. The two most commonly used tests under this category are the California Bearing Ratio Test and Hveem Stabilometer Test.

2.10.1 California Bearing Ratio (CBR)

Test This test is commonly known as the CBR test and involves the determination of the load-deformation curve of the soil in the laboratory using the standard CBR testing equipment shown in Figure 2.5. It was originally developed by the California Division of Highways prior to World War II and was used in the design of some highway pavements. The test is conducted on samples of soil compacted to required standards and immersed in water for four days, during which time the samples are loaded with a surcharge that simulate the estimated weight of pavement material the soil will support. The objective of the test is to determine the relative strength of a soil with respect to crushed rock, which is considered an excellent coarse base material. This is obtained by conducting a penetration test on the samples still carrying the simulated load and using a standard CBR equipment. The CBR is defined as the unit load for 0.1 piston in standard crushed rock is usually taken as 1000 lb/in.2, which gives the CBR as

$$CBR = \frac{measured\ pressure\ for\ site\ soils\ (N/mm)^2}{pressure\ to\ achieve\ equal\ penetration\ on\ standard\ \ soils \left(\frac{N}{mm}\right)^2}.$$

The test is fully described in Standard Specifications for Transportation Materials and Methods of Sampling and Testing by AASHTO and is standardized under the AASHTO designation of T193. The main criticism of the CBR test is that it does not correctly simulate the shearing forces imposed on sub-base and subgrade materials as they support highway pavements. For example, it is possible to obtain a relatively high CBR value for a soil containing rough or angular coarse material and some amount of troublesome clay if the coarse material resists penetration of the piston by keeping together in the mold. When such a material is used in highway construction, however, the performance of the soil may be poor, due to the lubrication of the soil mass by the clay, which reduces the shearing strength of the soil mass.

Figure 2.5 CBR Testing Equipment

CHAPTER THREE

METHODOLOGY

3.1 Research Overview

The research work was within the failed section of Ikole-Oye Ekiti road whereby soil samples were taken from the subgrade, sub base and base layers of the road for four different locations. The sampling points with their appropriate coordinates are sampling point 1 (7.474693° N, 5.280855° E), sampling point 2 (7.483325° N, 5.240643° E), sampling point 3 (7.479756° N, 5.220363° E), sampling point 4 (7.480405° N, 5.189875° E) and the locations are shown in Table 3.1.The soil samples from sampling point were selected as representative samples. After collection, soil samples were stored in polythene bags to prevent loss of moisture contents. The samples were then taken to the laboratory where the deleterious materials such as roots were removed. The samples were air dried, broken down with mortar and pestle and passed through a set of sieve (i.e. from Sieve No. 10 (18.75mm) to Sieve No. 1 (75mm) to remove large particles. Moulding of test specimens was started as soon as possible after completion of identification. All tests were performed according to standard methods contained in BS 1377 (1990). Their properties were studied and determined to ensure that all relevant factors would be available for establishment of correlations among them. The tests carried out on each of the selected samples are Grain size analysis, Consistency test (i.e. Liquid Limits (LL), Plastic Limit (PL) and Plasticity Index (PI)), Compaction test (i.e. Optimum Moisture Content (OMC) and Maximum Dry Density (MDD)), Permeability test, Natural moisture content, Specific Gravity, Consolidation test and California Bearing Ratio (CBR). The results were compared to the standard specified values and grouped in accordance with General Specification for roads and bridges FMWH, (1997) and American Association of State Highway and Transportation Officials AASHTO, (1986) respectively.

3.2 Research Design

This is the outline, plan or scheme used to generate answers to the research problem. It is basically the plan and structure of investigation. I used field, desk and laboratory research in working towards the set objectives. Field research involved observations while laboratory research involved collection and testing of soil samples. Observation of the side drains, orientation of the drainage channels as well as the slope and gradient of the embankment was done.

This research project employed both observation and photography as tools for which data would be collected. This involved observation and taking of photographs to show the current state of the drainage system along the road. From observation also; a brief description of what was observed was given with the help of photographs.

3.3 Desk Study

I did an in-depth desk research which involved analyzing information that already existed in print or published media and on the internet. It involved reading of the geotechnical resources, geological resources and any other relevant material which could be helpful towards achieving the research objectives. The information from all these sources give a general background of Ikole-Oye road, the types of soil variation and the terrain of the area.

3.4 Reconnaissance Survey

Reconnaissance survey involved observations and inspections of the site including taking of photographs to show the current state of the road along Ikole – Oye Ekiti. From observation also; a brief description of what was observed is given with the help of photograph as shown in plate 3.1. I visited and explored the site for the purpose of investigating the soil conditions at the location of study (Ikole-Oye ekiti road). The site topography was used to determine the nature of the geological deposits underlying the soil as well as determining their engineering properties.

Plate 3.1 Current State of Ikole – Oye Ekiti Road.

3.5 Laboratory Research

The following tests; Grain Size Analysis, Permeability, Consistency test, Specific Gravity test, Natural Moisture Content, California Bearing Ration test, Compaction test was carried out in the laboratory at Federal polytechnic Ado – Ekiti, to help in classification and also to determine properties of the soil samples collected.

All laboratory tests was carried out in accordance to the BS 1377-part2:1990 (BRITISH STANDARDS) except for the compaction test which will be carried out in accordance to A.A.S.T.H.O standards, the results will be compared to the standard specified values and grouped in accordance with General Specification for roads and bridges FMWH, (1997) and American Association of State Highway and Transportation Officials AASHTO, (1986) respectively.

3.6 Sampling of Materials

The three layers of road (subgrade, sub base and base course) was subjected to geotechnical tests, In order to carry out the geotechnical examination work, a borehole was excavated at the locations chosen for collection of soil sample. The disturbed samples for this project was collected from four different locations along Ikole- Oye Ekiti, Ekiti State. This involved the digging of pits to a depth of 150cm at every change of strata below the existing ground level and the overlying soil material as well as the top soil was discarded. Diggers, cutlass and hoe was used to dig the ground to the collect soil samples, some of the samples was sealed in polythene bags to preserved the insitu moisture condition of the soil. The soil samples was taken to the laboratory for tests.

The sampling points with their appropriate coordinates are shown in Table 3.1 below.

Table 3.1 Sampling Point and their Respective Coordinates

S/N	SAMPLING POINT	COORDINATE IN DEGREE		COORDINATE IN METRIC(m)	
		NORTHING	**EASTING**	**NORTHING**	**EASTING**
1	Ikole - Oloko Road	7.474693°	5.280855°	830647.60	586850.80
2	Osin –Aparigi Road	7.483325°	5.240643°	831606.87	582382.12
3	Itapa - Ilupeju Road	7.479756°	5.220363°	831210.25	580128.45
4	Oye Road	7.480405°	5.189875°	831282.37	576740.38

3.7 Sample Preparation

After collection, soil samples was stored in polythene bags to prevent loss of moisture contents. The samples was then taken to the laboratory where the deleterious materials such as roots was removed. The sample collected was air-dried for weeks before being subjected to laboratory test except those of moisture contents which were immediately carried out in the laboratory. The samples was stirred at regular intervals during the period of air drying. After air-dried, part of the soil was sieved through 425umbs sieves. The un-sieved soil was used for Atterberg limit and other tests.

3.8 PROCEDURE OF SOIL TESTS.

3.8.1 Particle Size Distribution

This test is done to determine the particle size distribution of a soil sample, the distribution of different grain sizes affects the engineering properties of soil and it was carried out according to AASHTO Grain size analysis provide the grain size distribution and it is required in classifying the soil

Tools

i) A set of fine BS Sieves of sizes – 2mm, 600μm, 425μm, 212μm and 75μm
ii) A set of coarse BS 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.

Figure 3.3 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 BS Sieve with water till substantially clean water comes out. Retained sample on 4.75mm BS Sieve should be oven-dried for 24hrs. This dried sample is sieved through 20mm and 10mm BS Sieves.

iii) The portion passing through 4.75mm BS 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 BS Sieve with half litre distilled water, till substantially clear water comes out.

v) The material retained on 75µm BS 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 BS 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 BS 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 monogram given in BS: 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

3.8.2 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 BS 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.

3.8.3 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

- Standard Proctor test
- Modified AASHTO method
- 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) BS 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 BS Sieve (for soils not susceptible to crushing during compaction) or about 15kg of material passing through a 19mm BS Sieve (for soils susceptible to crushing during compaction), should be taken. This portion should be sieved through a 19mm BS 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 BS 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 BS 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 leveled 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 BS 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.

Figure 3.4 Moulds and Rammers

Reporting of Results

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

$$Y(\text{gamma}) = (W_2 - W_1) / V$$

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.

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

A. 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.

B. Measurement of soil density

The following method should be followed for the measurement of soil density:

i) 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.

3.8.4 Plastic Limit Test

This test is done to determine the plastic limit of soil as per BS: 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 BS 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 BS Sieve. The average water content to the nearest whole number should be reported.

3.8.5 Liquid Limit Test

This test is done to determine the liquid limit of soil as per BS: 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

viii) Wash bottle

ix) Air-tight and non-corrodible container for determination of moisture content

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 BS 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.

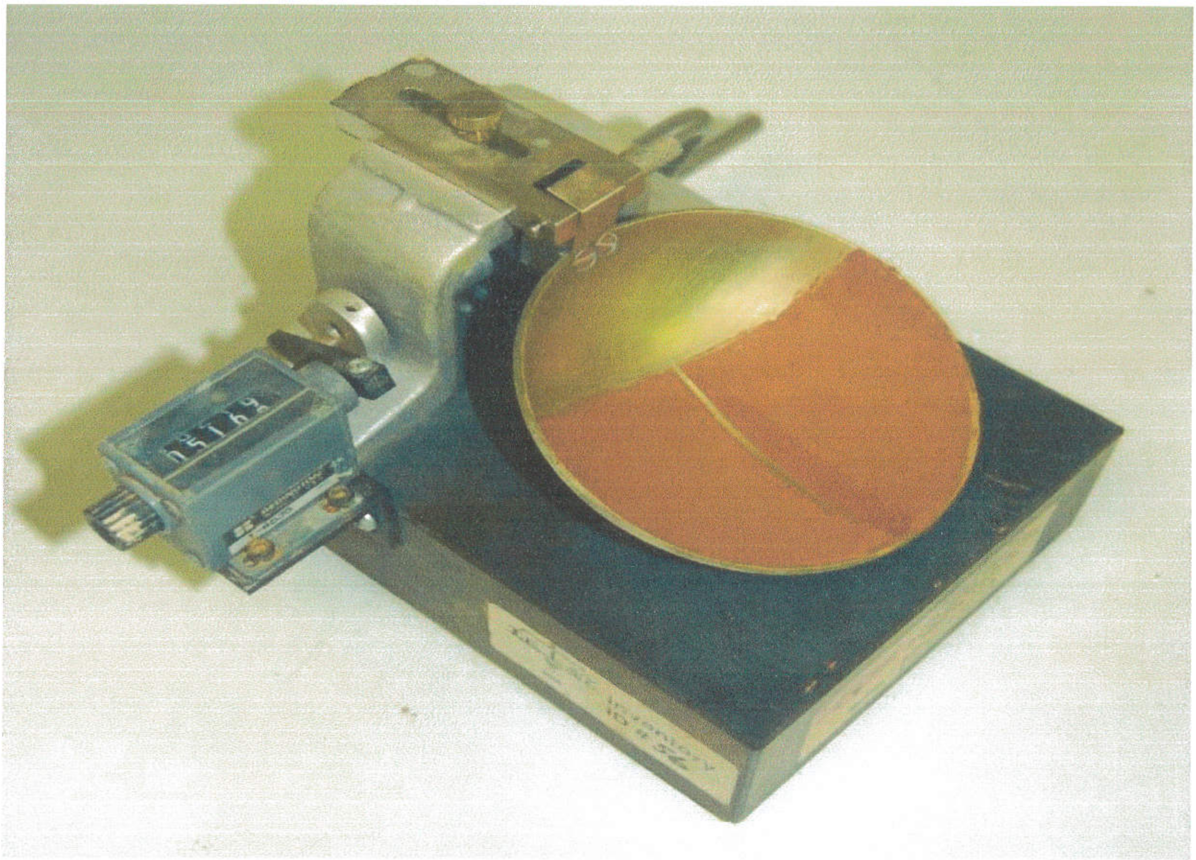


Figure 3.5 Liquid Limit apparatus

3.8.6 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.

AIM: to determine the suitability of a soil for use as sub-grade or base materials

Tools

i) Mould
ii) Steel Cutting collar
iii) Spacer Disc
iv) Surcharge weight
v) Dial gauges
vi) BS Sieves
vii) Penetration Plunger
viii) Loading Machine
ix) Filter paper

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 of 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 %age 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.

3.8.7 Permeability Test

The knowledge of this property is much useful in solving problems involving yield of water bearing strata, seepage through earthen dams, stability of earthen dams, and embankments of canal bank affected by seepage, settlement etc. For disturbed soil sample

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) After completion of a compaction the collar and excess soil are removed.
ix) Find the weight of mould with sample.
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.

Presentation of data

The coefficient of permeability is reported in cm/sec at 27o C. The dry density, the void ratio and the degree of reported in cm/sec at 27o C. The dry density, the void ratio and the degree of saturation shall be reported.

CHAPTER FOUR

RESULT AND DISCUSSION

This chapter presents the results of various geotechnical test carried out on four sampling points used. The laboratory tests carried out on the samples are Particle size distribution, Atterberg limits, Natural Moisture Content, Permeability, Specific Gravity, Compaction test and California Bearing Ratio.

Table 4.1 Summary Results Table for Compaction test, Atterberg limit, Sieve Analysis and AASHTO Soil Classification

Sampling point	Pavement section	Compaction		Atterberg limit				Sieve Analysis	AASHTO Classification
		MDD kg/m3	OMC %	LL %	PL %	PI %	SL %	% Passing Sieve 200	
1	Subgrade	1.83	14.5	34.2	21.7	12.5	7.1	22.8	A-1-b
	Sub-base	1.72	16.8	33.8	20.05	13.8	9.3	27.3	A-2-7
	Base	2.01	12.0	28.1	13.8	14.3	9.3	21.9	A-1-b
2	Subgrade	1.77	15.8	34.6	21.9	21.7	10.7	36.8	A-7-5
	Sub-base	1.66	20.20	35.5	21.9	13.6	10.7	38.6	A-7-5
	Base	1.70	15.2	34.0	18.0	16.0	9.3	39.5	A-7-5
3	Subgrade	2.0	15.8	31.0	26.0	4.9	7.1	17.0	A-1-b
	Sub-base	2.02	15.1	29.1	19.0	10.1	5.0	21.6	A-1-b
	Base	2.1	15.5	32.2	21.8	10.4	8.8	18.8	A-1-b
4	Subgrade	1.59	19.6	51.0	22.0	29.0	11.6	56.0	A-7-6
	Sub-base	1.95	10	40.9	20.9	19.1	7.1	33.0	A-2-5
	Base	1.87	15	34.0	25.0	9.0	7.1	45.6	A-5

Table 4.2 Summary Results table for Specific Gravity, Natural Moisture content, Permeability test and California Bearing Ratio.

Soil samples location	Pavement section	Average Specific Gravity	Natural moisture content %	Permeability mm/sec	CBR	
					2.5mm	5.0mm
1	Subgrade	2.33	17.2	$4.71 * 10^{-2}$	0.4	1.03
	Sub-base	2.46	12.7		3.13	5.15
	Base	2.56	7.7		8.0	16.20
2	Subgrade	2.05	13.3	$2.9 * 10^{-2}$	0.13	0.45
	Sub-base	2.43	18.4		3.0	4.33
	Base	2.32	16.9		8.98	16.00
3	Subgrade	2.41	16.0	$4.7 * 10^{-2}$	0.65	2.08
	Sub-base	2.32	15.7		4.10	9.70
	Base	2.38	14.9		9.63	17.13
4	Subgrade	1.79	21.8	$1.77 * 10^{-2}$	0.02	0.2
	Sub-base	2.59	14.6		2.53	4.30
	Base	2.42	20.5		9.78	16.43

4.3 DISCUSSION

4.3.1 Particle Size Distribution

Summary of results of respective properties of selected samples is presented in Table 4.1 and Table 4.2. It was observed from Table 4.1 and Fig. 4.1 that the percentage range of soil passing through No. 200 BS sieve is between 17.0 and 56% for subgrade materials, 21.6 and 38.6% for sub-base materials and 18.6 and 45.6% for base materials. Two of the sampling points (i.e. SP 1 and SP3) had a percentage finer less than 0.0075 fraction (i.e. < 35%) which varies between 17.0% and 22.8% and the two other sampling points (i.e. SP 2 and SP 4) had a high percentage finer than 0.0075 fraction (i.e. > 35%) which varies between 36.8% and 56.0% for sub grade course. 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 A.

For the sub base course, the results show that three of the sampling points (i.e. SP 1,3 and 5) had a percentage finer less than 0.0075 fraction (1.e. < 35%) which varies between 21.60% and 33.0% which implies that the sub-base for these sampling points are of granular materials and the other sampling point (i.e. SP 4) had a high percentage finer than 0.0075 fraction (i.e. > 35%) equal to 38.6% which implies that the sub-base for this sampling point is of silt- clay materials.

For the base course, the results shows that two of the sampling point (i.e. SP1 and 3) had a percentage finer less than 0.0075 fraction (1.e. < 35%) which varies between 18.0% and 21.9% which implies that the base course for these sampling points are of granular materials and the other two sampling points (i.e. SP 2 and 4) had a high percentage finer than 0.0075 fraction (i.e. > 35%) which varies between 39.5% and 45.6% which implies that the base for this sampling points are of silt- clay.

Figure 4.1 Grain Size Analysis Graph for Sampling Point 1

4.3.2 Atterberg Limits

The result of Atterberg limits are shown in Appendix B with their appropriate graphs. Table 4.3 revealed the following variations in values of Liquid limit, Plastic Limit and plasticity index. LL: 31.0- 51%, PL: 21.7- 26% and PI: 7.1-12.5% for subgrade course materials, LL: 29.1-40.9%, PL: 19.0- 20.9% and PI: 10.1-13.8% for sub-base course materials, LL: 28.1-34.0% , PL: 13.8-25.0% and PI: 9.0-16.0% for base course materials. AASHTO (1986) recommended liquid limit not greater than 80% for subgrade and not greater than 35% for sub-base and base course materials. Also, plasticity index not greater than 55% for subgrade and not greater than 12% sub-base and base course materials. From the values above, the subgrade soils fall within these specification specified by AASHTO (1986), thus making the subgrade soils suitable for subgrade course materials while those of sub-base and base course materials did not meet the specifications thus make them not suitable for sub-base and base course materials.

Table 4.3 Results of Atterberg Limits Test for all the samples

Soil samples location	**Pavement section**	**Atterberg limit**			
		Liquid Limit %	**Plastic Limit %**	**Plasticity Index %**	**Shrinkage Limit %**
1	Subgrade	34.2	21.7	12.5	7.1
	Sub-base	33.8	20.05	13.8	9.3
	Base	28.1	13.8	14.3	9.3
2	Subgrade	34.6	21.9	21.7	10.7
	Sub-base	35.5	21.9	13.6	10.7
	Base	34.0	18.0	16.0	9.3
3	Subgrade	31.0	26.0	4.9	7.1
	Sub-base	29.1	19.0	10.1	5.0
	Base	32.2	21.8	10.4	8.8
4	Subgrade	51.0	22.0	29.0	11.6
	Sub-base	40.9	20.9	19.1	7.1
	Base	34.0	25.0	9.0	7.1

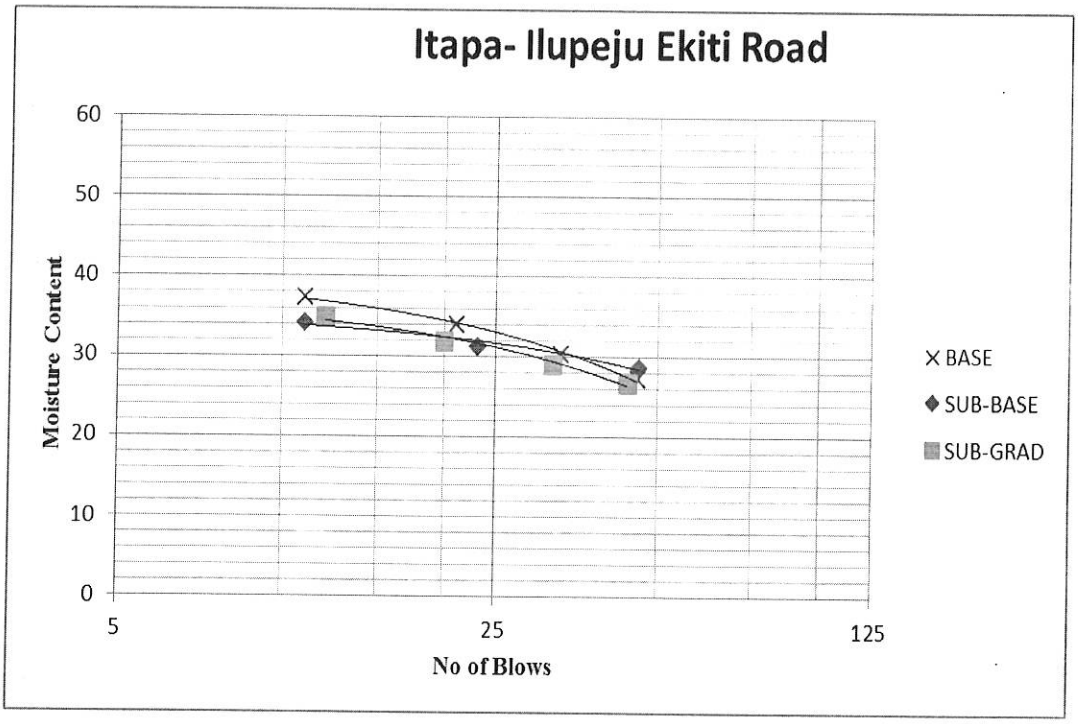


Figure 4.2 Atterberg Limit Graph for Sampling Point 3

4.3.3 Natural Moisture Content Test

The result of natural moisture content were shown in Appendix D with their respective graphs and on Table 4.4. From the results all sampling points have low values of moisture content which show the soils have low potential of water retention. The natural moisture values ranging between 7.7% and 21.8%

Table 4.4 Average Natural Moisture Content for the samples

Sampling Point	Pavement section	Average Natural moisture content %
1	Subgrade	17.2
	Sub-base	12.7
	Base	7.7
2	Subgrade	13.3
	Sub-base	18.4
	Base	16.9
3	Subgrade	16.0
	Sub-base	15.7
	Base	14.9
4	Subgrade	21.8
	Sub-base	14.6
	Base	20.5

4.3.4 Permeability Test (Falling Head)

Results of permeability tests are shown in Appendix G and Table 4.5. The results indicate low coefficients of permeability of the soils in the range of 1.77*10-2 to 4.77*10-2 mm/sec: this is due to the high fines content in the studied soil samples, AASHTO (1986). The soils can therefore be classified to be of low permeability with relatively poor to fair drainage characteristics.

Table 4.5 Summary of the Permeability Test Results

S/N	Sample Location	K (mm/sec)	Grading Type
1	Ikole- Oloko Ekiti Road	$4.71 * 10^{-2}$	Low
2	Osin– Aparigi Ekiti Road	$2.9 * 10^{-2}$	Low
3	Itapa- Ilupeju Ekiti Road	$4.7 * 10^{-2}$	Low
4	Oye Ekiti Road	$1.77 * 10^{-2}$	Low

4.3.5 The Specific Gravity

The results of the specific gravity are shown in Table 4.6 and Appendix C, Table 4.7 shows the variation in the results, the specific gravity varies from 1.79 to 2.41 for subgrade materials, also varies from 2.32 to 2.59 for sub-base materials and varies from 2.32 to 2.56 for base materials.

Table 4.6 Average Specific Gravity of Soil Samples

Soil samples location	Pavement section	Average Specific Gravity
1	Subgrade	2.33
	Sub-base	2.46
	Base	2.56
2	Subgrade	2.05
	Sub-base	2.43
	Base	2.32
3	Subgrade	2.41
	Sub-base	2.32
	Base	2.38
4	Subgrade	1.79
	Sub-base	2.59
	Base	2.42

4.3.6 Compaction Test

The results of the compaction test is shown in Appendix E with their respective graphs. From Table 4.7, it was observed that the OMC values varies from 14.5% - 19.6% for subgrade materials, 10% - 20.20% for sub-base materials and 12% -15.5% for base materials while the MDD values varies from 1.59 - 1.83kg/m3 for subgrade materials, 1.66 – 2.02kg/m3 for sub-base materials and varies from 1.7 - 2.1kg/m3 for base materials. The subgrade samples from sampling point 1, 2 and 3 met AASHTO (1986) specification which state the MDD values most not be less than 1.76kg/m3 for subgrade sample, this implies that the subgrade samples are suitable.

The sub-base and base samples from most of the sampling point did not meet AASHTO (1986) specification which state that the MDD values for both base and sub-base course most not be less than 2.0kg/m3, with exception of sub-base material used sampling point 3 with MDD value of 2.02kg/m3.

The above results implied that the soil materials used for subgrade courses are suitable according to AASHTO (1986) with the exception of subgrade materials used in sampling point 4. The result also shows that the soil materials used for sub-base and base courses in all sampling points are not suitable with the exception of sub-base materials used in sampling point 3 which is considered suitable.

Table 4.7 Result of Compaction Test

SOIL SAMPLES LOCATION	PAVEMENT SECTION	COMPACTION	
		MDD kg/m3	OMC %
1	Subgrade	1.83	14.5
	Sub-base	1.72	16.8
	Base	2.01	12.0
2	Subgrade	1.77	15.8
	Sub-base	1.66	20.20
	Base	1.70	15.2
3	Subgrade	2.0	15.8
	Sub-base	2.02	15.1
	Base	2.1	15.5
4	Subgrade	1.59	19.6
	Sub-base	1.95	10
	Base	1.87	15

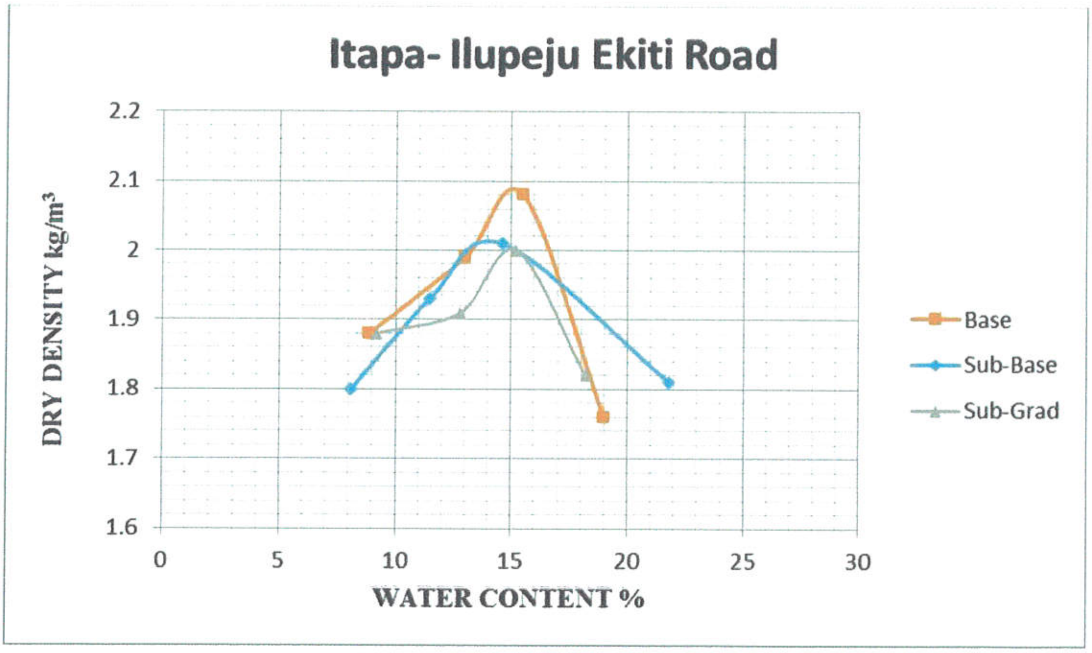


Figure 4.3 Compaction Test Graph for Sampling Point 3 (SP3)

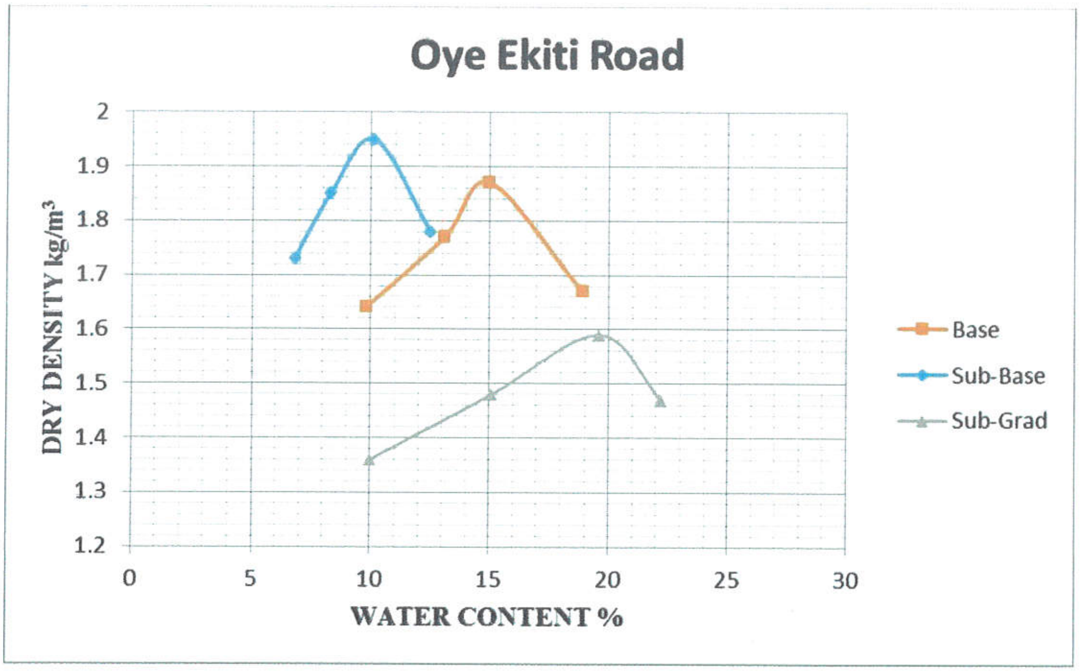


Figure 4.4 Compaction Test Graph for Sampling Point 4 (SP4)

4.3.7 California Bearing Ratio Test

From Table 4.8, it was observed that the soaked CBR values for sub-grade course materials varied between 0.2% and 2.08% in all the locations, all the sampling points samples met AASHTO (1986) specification of not less than 5% of soaked CBR value for subgrade materials. According to AASHTO (1986) , the specified value for unsoaked CBR is not less than 80% for sub-base materials , and from the results it was deduced that the unsoaked CBR values varied between 8% and 9.78% for base course materials in all the locations, which are less than 80% specified by AASHTO (1986) .(i.e. Unsoaked CBR ≥ 80%). While the specified soaked CBR values for sub-base course materials most not be less than 30% for base materials , AASHTO (1986), the soaked CBR values for base materials varied between 4.30% and 9.70% in all the sampling points, these values did not meet the specified value of not less than 30% (i.e. Soaked CBR ≥ 30%)).

The above results implied that all the soil materials for base and sub-base courses along the chainages are not suitable, while the soil materials used for subgrade courses along the chainages are suitable. Thus, these factors may have contributed to the widespread failure observed on the roadway.

Table 4.8 California Bearing Ratio Results

Soil samples location	Pavement section	CBR	
		2.5mm	5.0mm
1	Subgrade	0.4	1.03
	Sub-base	3.13	5.15
	Base	8.0	16.20
2	Subgrade	0.13	0.45
	Sub-base	3.0	4.33
	Base	8.98	16.00
3	Subgrade	0.65	2.08
	Sub-base	4.10	9.70
	Base	9.63	17.13
4	Subgrade	0.02	0.2
	Sub-base	2.53	4.30
	Base	9.78	16.43

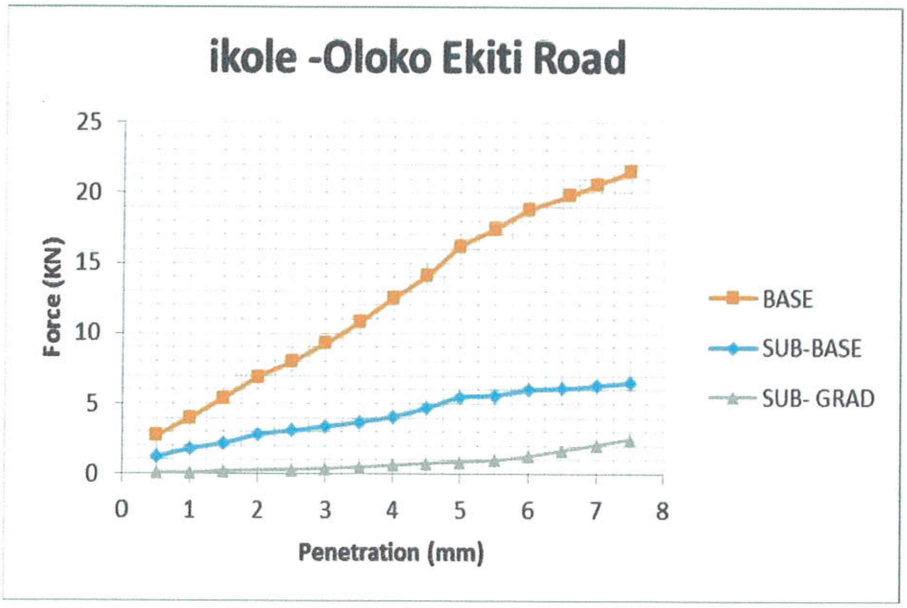


Figure 4.5 California Bearing Ratio Test for Sampling Point 1 (SP1)

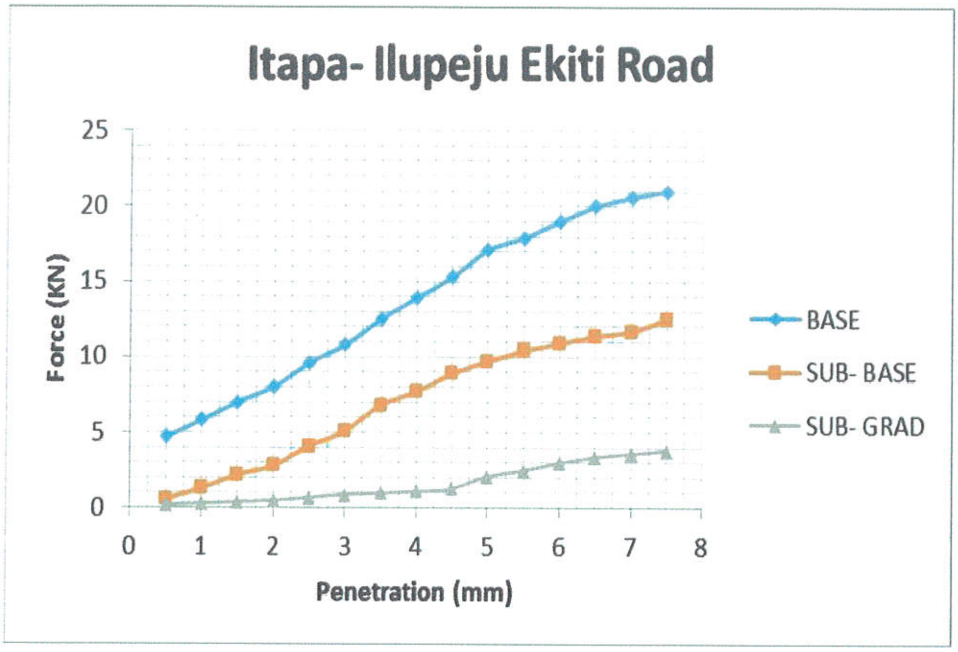


Figure 4.6 California Bearing Ratio Test for Sampling Point 3 (SP3)

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

One of the greatest assets of any nation is its road network and can only be preserved through maintenance and a good drainage structure such as culverts, drains etc. From the results obtained from tests carried out (particle size distribution, atterberg limits, specific gravity, natural moisture content, permeability test, California bearing ratio, and compaction test) the following conclusions were drawn.

All the soils in the study location have low potential of water retention with their natural moisture content not exceeding 22% and most of the subgrade soils are of clayey materials because greater than 35% of the soil passed through the 0.0075mm sieve.

The subgrade soils from sampling points 1, 2, 3 and 4 are grouped into A-1-b, A-7-5, A-1-b and A-7-6 respectively, while the sub-base soils are also grouped into A-2-7, A-7-5, A-1-b, A-2-5 respectively and the base soils are grouped into A-1-b, A-7-5, A-1-b, A-5 respectively, according to AASHTO classification system (1986).

The OMC values varies from 14.5% - 19.6% for subgrade materials, 10% - 20.20% for sub-base materials and 12% -15.5% for base materials while the MDD values varies from 1.59 - 1.83kg/m3 for subgrade materials, 1.66 – 2.02kg/m3 for sub-base materials and varies from 1.7 - 2.1kg/m3 for base materials. The subgrade samples from sampling point 1, 2 and 3 met AASHTO (1986) specification which state the MDD values most not be less than 1.76kg/m3 for subgrade sample, this implies that the subgrade samples are suitable. The sub-base and base samples from most of the sampling point did not meet AASHTO (1986) specification which state that the MDD values for both base and sub-base course most not be less than 2.0kg/m3, with exception of sub-base material used sampling point 3 with MDD value of 2.02kg/m3.

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

The soaked CBR values for sub-grade course materials varied between 0.2% and 2.08% in all the locations, all the sampling points samples met AASHTO (1986) specification of not less than 5% of soaked CBR value for subgrade materials. According to AASHTO (1986) , the specified value for unsoaked CBR is not less than 80% for sub-base materials , and from the results it was deduced that the unsoaked CBR values varied between 8% and 9.78% for base course materials in all the locations, which are less than 80% specified by AASHTO (1986) .(i.e. Unsoaked CBR ≥ 80%). While the specified soaked CBR values for sub-base course materials most not be less than 30% for base materials , AASHTO (1986), the soaked CBR values for base materials varied between 4.30% and 9.70% in all the sampling points, these values did not meet the specified value of not less than 30% (i.e. Soaked CBR ≥ 30%)).

Geotechnical investigation shows that the materials used for sub-grade materials met AASHTO specifications while those materials used for sub-bases and bases courses did not meet the specifications specified for highway materials (needed for good and stable road). This cause the major failure of the road. Thus, the result calls for proper geotechnical analysis of materials for the construction of each pavement layer if the road is to be reconstructed.

In addition, the road suffers rapid deterioration due to inadequate design, according to Federal Ministry of Works, the specified thickness for wearing course is 40mm, but the wearing course of this road is 35mm, which might have also contribute to the road failure due to heavy duties vehicles that ply the road daily.

5.2 RECOMMENDATIONS

In order to prevent reoccurrence of road failure of this type, adequate investigation of the geotechnical properties of the soil materials to be used for pavement construction should be given high priority so that sub-standard materials is not used and the asphalt to be used for surface course should be well blended in the right proportion.

The federal or state government should as much as possible try to improve or develop alternative means of transportation e.g. railways and water ways to help in the conveyance of heavy haulage. The development of railway transportation will even further help in quick delivery of goods to long distance and thereby improving the economic situation of the country.

Attention should be given to quality control by appropriate government agencies to ensure roads are constructed to meet the desired standard.
Qualified engineering personnel should give adequate supervision during road constructions.
A proper maintenance culture should be adopted.
The provision of adequate surface drainage should be of topmost priority in any road construction project.

I also recommended that soil stabilization test should be carried out on sub-grade course due to the type of lateritic soils (i.e. clayey soils).

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APPENDICES

APPENDIX A

PARTICLE SIZE DISTRIBUTION

Results of Particle Size Distribution

Sampling Point 1

BASE			SUB BASE			SUB GRADE			
Sieve Size	Weight retained	% retained	% passing	Weight retained	% retained	% passing	Weight retained	% retained	% passing
9.50	22.4	4.5	95.5	0.1	0	100	9.7	1.9	98.1
4.75	56.8	11.4	84.1	41.9	8.4	91.6	48.6	9.7	88.4
2.36	128.3	25.7	58.4	113.3	22.7	68.9	79.9	16.0	72.4
1.18	61.4	12.3	46.1	82.8	16.6	52.3	80.1	16.0	56.4
600	40.3	8.2	37.9	45.2	9.0	43.3	62.5	12.5	43.9
0.30	38.9	7.8	30.1	35.8	7.2	36.1	47.4	9.5	34.4
0.15	26.7	5.3	24.8	27.2	5.4	30.7	35.1	7.0	27.4
0.0075	14.7	2.9	21.9	17.0	3.4	27.3	23.1	4.6	22.8

Sampling Point 2

BASE			SUB BASE			SUB GRADE			
Sieve Size	Weight retained	% retained	% passing	Weight retained	% retained	% passing	Weight retained	% retained	% passing
9.50	68.1	13.6	86.4	24.4	4.5	95.5	15.6	3.1	96.9
4.75	44.7	8.9	77.5	32.4	6.5	89.0	52.0	10.4	86.5
2.36	40.4	8.1	69.4	57.2	11.4	77.6	70.6	14.1	72.4
1.18	35.9	7.2	62.2	53.5	10.7	66.9	60.7	12.1	60.3
600	36.6	7.3	54.9	37.0	7.4	59.5	32.5	6.5	53.8
0.30	29.9	6.0	48.9	35.5	7.1	52.4	29.5	5.9	47.9
0.15	27.0	5.4	43.5	37.1	7.4	45.0	31.1	6.2	41.7
0.0075	19.9	4.0	39.5	32.2	6.4	38.6	24.5	4.9	36.8

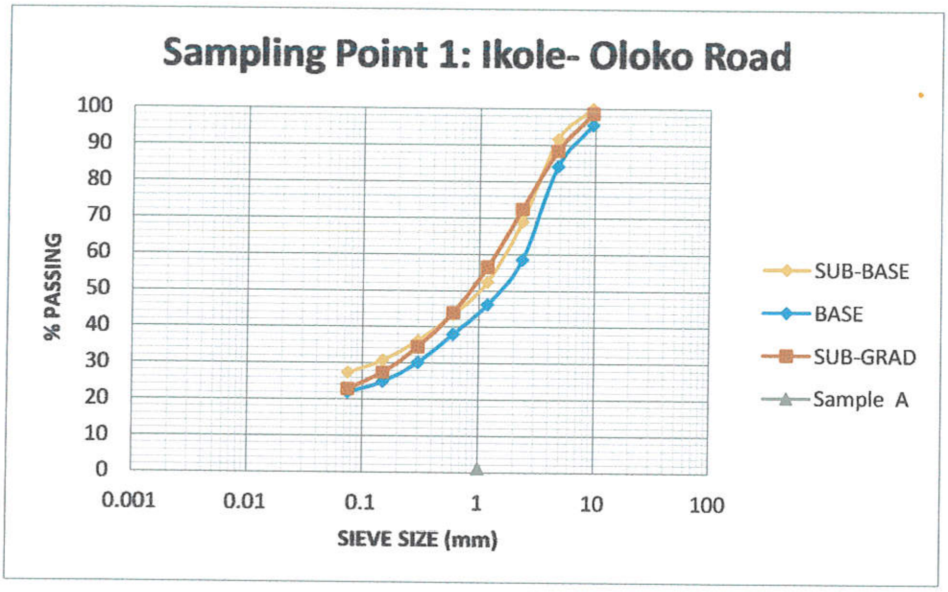
Sampling Point 3

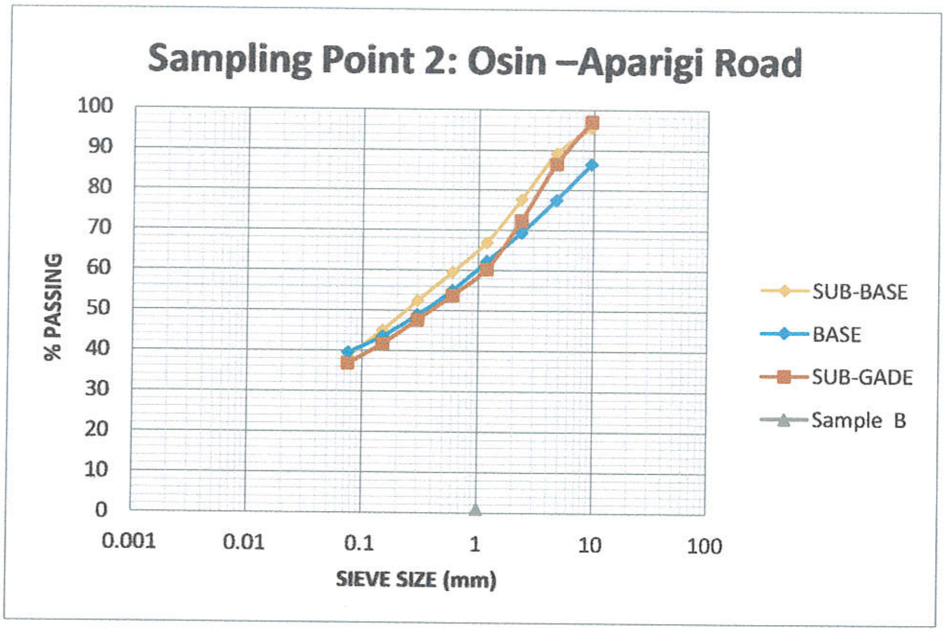
BASE			SUB BASE			SUB GRADE			
Sieve Size	**Weight retained**	**% retained**	**% passing**	**Weight retained**	**% retained**	**% passing**	**Weight retained**	**% retained**	**% passing**
9.50	4.2	0.8	99.2	0	0	0	11.1	2.2	97.8
4.75	78.3	15.7	83.5	83.1	16.6	83.4	91.2	18.2	79.6
2.36	108.0	21.6	61.9	94.8	19.0	64.4	101.0	20.2	59.4
1.18	83.5	16.7	45.2	70.4	14.1	50.3	70.2	14.0	45.4
600	44.0	8.8	36.4	46.7	9.3	41.0	42.4	8.4	37.0
0.30	34.8	7.0	29.4	41.4	8.2	32.8	37.7	7.5	29.5
0.15	31.4	6.3	23.1	35.1	7.0	25.8	36.7	7.3	22.2
0.0075	21.7	4.3	18.8	26.8	4.2	21.6	26.1	5.2	17.0

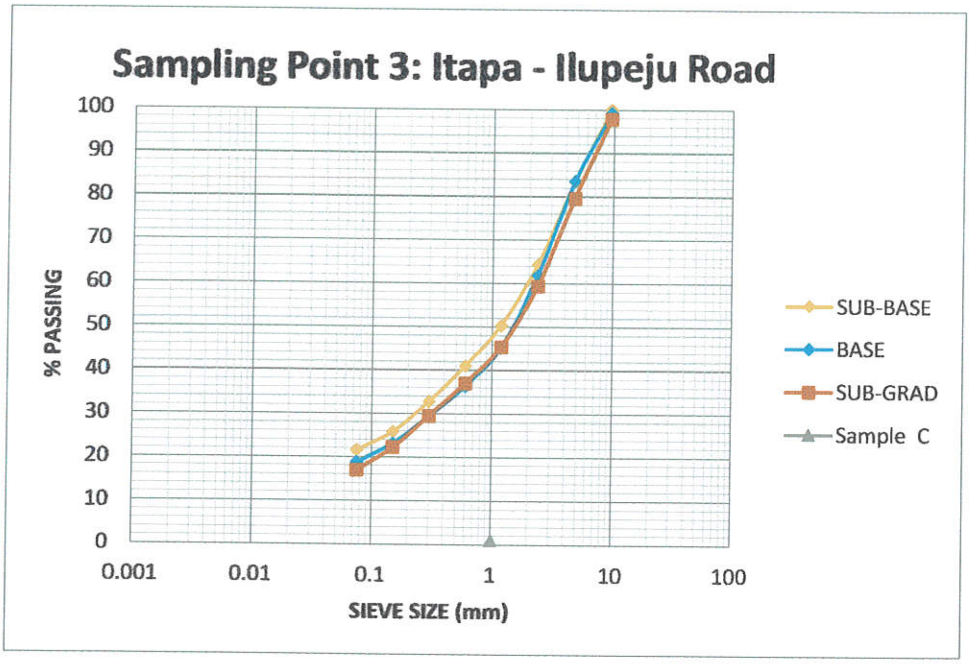
Sampling Point 4

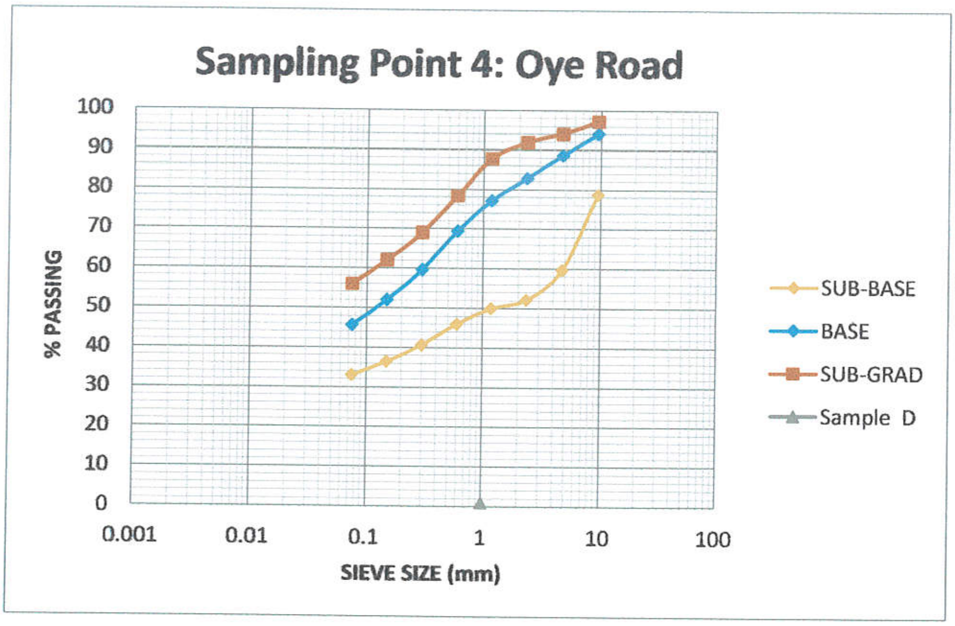
BASE			SUB BASE			SUB GRADE			
Sieve Size	**Weight retained**	**% retained**	**% passing**	**Weight retained**	**% retained**	**% passing**	**Weight retained**	**% retained**	**% passing**
9.50	30.1	6.0	94.0	106.5	21.3	78.7	14.1	2.8	97.2
4.75	27.1	5.4	88.6	95.2	19.0	59.7	15.4	3.1	94.1
2.36	29.2	5.8	82.8	37.9	7.6	52.1	11.2	2.2	91.9
1.18	28.4	5.7	77.1	11.1	2.2	49.9	20.9	4.2	87.7
600	38.5	7.7	69.4	19.8	4.0	45.9	47.2	9.4	78.3
0.30	48.9	9.8	59.6	26.5	5.3	40.6	46.4	9.3	69.0
0.15	37.9	7.6	52.0	20.9	4.2	36.4	34.8	7.0	62.0
0.0075	32.2	6.4	45.6	16.8	3.4	33.0	30.1	6.0	56.0

Graphs of Particle Size Distribution






Sampling Point 3: Itapa - Ilupeju Road
% PASSING
100
90
80
70
60
50
40
30
20
10
0
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)
SUB-BASE
BASE
SUB-GRAD
Sample C


Sampling Point 4: Oye Road
% PASSING
100
90
80
70
60
50
40
30
20
10
0
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)
SUB-BASE
BASE
SUB-GRAD
Sample D

APPENDIX B

Consistency Test

Results of Atterberg Limits Test

B1: Consistency Test by Cassagrande Method for Base layer Ikole –Oloko Road

Trial no.	1	2	3	4		
No. of blows	48	36	22	12	PLASTIC LIMIT	
Container identification no.	A	B	C	D	E	F
Weight of empty container (g)	11.2	16.4	16.6	19.7	19.9	7.1
Weight of container + wet soil (g)	39.5	45.3	48.5	53.1	40.0	30.0
Weight of container + dry soil (g)	33.9	39.3	41.4	45.2	37.5	27.3
Weight of water (g)	5.6	6.0	7.1	7.9	2.5	2.7
Weight of dry soil (g)	22.7	22.9	24.8	25.5	17.6	20.2
Moisture content	24.7	26.2	28.6	31.0	14.2	13.4
						PL=13.8

B2: Consistency Test by Cassagrande method for sub base layer Ikole –Oloko Road

Trial no.	1	2	3	4		
No. of blows	48	37	23	13	PLASTIC LIMIT	
Container identification no.	A2	B2	C2	D2	E2	F2
Weight of empty container (g)	26.7	26.8	24.5	19.6	16.7	19.8
Weight of container + wet soil (g)	56.7	59.9	58.3	55.4	37.5	39.5
Weight of container + dry soil (g)	48.7	51.9	49.4	45.2	34.2	36.0
Weight of water (g)	7.0	8.0	8.9	10.2	3.2	3.5
Weight of dry soil (g)	22.1	24.1	24.9	26.2	17.5	16.2
Moisture content	31.7	33.2	35.7	38.9	18.5	21.6
						PL=20.05

B3: Consistency Test by Cassagrande Method for Subgrade Layer Ikole –Oloko Road

Trial no.	1	2	3	4		
No. of blows	45	33	22	13	PLASTIC LIMIT	
Container identification no.	A3	B3	C3	D3	E3	F3
Weight of empty container (g)	18.7	18.6	20.0	12.3	11.5	16.7
Weight of container + wet soil (g)	35.2	39.5	47.4	37.9	30.6	39.7
Weight of container + dry soil (g)	31.4	34.5	38.8	30.7	27.2	35.6
Weight of water (g)	3.8	5.0	6.6	7.2	3.4	4.1
Weight of dry soil (g)	12.7	15.9	18.4	18.4	15.7	18.9
Moisture content	29.9	31.4	35.1	39.1	21.7	21.7
						PL=21.7

B4: Consistency Test by Cassagrande Method for Base Layer Osin-Aparigi Ekiti Road

Trial no.	1	2	3	4		
No. of blows	46	33	23	12	Plastic limit	
Container identification no.	G	H	I	J	K	L
Weight of empty container (g)	26.8	26.9	12.1	19.9	12.1	11.1
Weight of container + wet soil (g)	53.2	55.8	40.9	48.9	29.9	33.3
Weight of container + dry soil (g)	47.3	48.8	33.1	40.2	26.9	30.3
Weight of water (g)	5.9	7.0	7.8	8.7	3.0	3.0
Weight of dry soil (g)	20.5	21.9	21.0	20.3	14.8	19.2
Moisture content	28.8	32.0	37.1	42.9	20.3	15.6
					PI=18.0%	

B5: Consistency Test by Cassagrande Method for Sub-base Layer Osin-Aparigi Ekiti Road

Trial no.	1	2	3	4		
No. of blows	49	38	22	12	Plastic limit	
Container identification no.	G1	H1	I1	J1	K1	L1
Weight of empty container (g)	14.8	14.2	9.8	10.8	8.1	10.0
Weight of container + wet soil (g)	34.7	36.2	33.4	37.6	27.6	31.2
Weight of container + dry soil (g)	29.8	30.4	26.7	29.7	24.1	27.4
Weight of water (g)	4.9	5.8	6.7	7.9	3.5	3.8
Weight of dry soil (g)	15.0	6.2	16.9	18.9	16.0	17.4
Moisture content	32.7	35.8	39.6	41.8	21.9	21.8
					PL=21.9%	

B6: Consistency Test by Cassagrande Method for Subgrade Layer Osin-Aparigi Ekiti Road

Trial no.	1	2	3	4		
No. of blows	45	32	23	14	Plastic limit	
Container identification no.	G2	H2	I2	J2	K2	L2
Weight of empty container (g)	12.4	16.8	7.6	13.2	11.6	26.8
Weight of container + wet soil (g)	33.2	39.7	33.2	39.9	34.2	47.6
Weight of container + dry soil (g)	28.4	34.0	26.3	32.2	30.1	43.9
Weight of water (g)	4.8	5.7	6.9	7.7	4.1	3.7
Weight of dry soil (g)	16.0	17.2	18.7	19.6	18.5	17.1
Moisture content	30.0	33.1	36.9	40.5	22.2	21.6
					PL= 21.9%	

B7: Consistency Test by Cassagrande Method for Base layer Itapa- Ilupeju Ekiti Road

Trial no.	1	2	3	4		
No. of blows	**46**	**33**	**21**	**11**	**Plastic limit**	
Container identification no.	**Z1**	**Z2**	**Z3**	**Z4**	**Z5**	**Z6**
Weight of empty container (g)	**26.6**	**27.7**	**23.1**	**19.8**	**11.7**	**16.3**
Weight of container + wet soil (g)	**54.2**	**55.2**	**53.0**	**49.9**	**30.4**	**35.6**
Weight of container + dry soil (g)	**48.3**	**48.8**	**45.4**	**41.7**	**27.3**	**31.9**
Weight of water (g)	**5.9**	**6.4**	**7.6**	**8.2**	**3.1**	**3.7**
Weight of dry soil (g)	**21.7**	**21.1**	**22.3**	**21.9**	**15.6**	**15.6**
Moisture content	**27.2**	**30.3**	**34.0**	**37.4**	**23.7**	**23.7**
					PL=21.8%	

B8: Consistency Test by Cassagrande Method for Sub-base Layer Itapa- Ilupeju Ekiti Road

Trial no.	1	2	3	4		
No. of blows	**46**	**34**	**21**	**11**	**Plastic limit**	
Container identification no.	**M2**	**N2**	**O2**	**P2**	**Q2**	**R2**
Weight of empty container (g)	**15.5**	**20.0**	**25.7**	**26.6**	**7.1**	**11.7**
Weight of container + wet soil (g)	**35.8**	**41.4**	**50.4**	**60.3**	**25.8**	**32.9**
Weight of container + dry soil (g)	**31.5**	**36.3**	**44.1**	**51.4**	**22.4**	**30.0**
Weight of water (g)	**4.3**	**5.1**	**6.3**	**9.0**	**3.4**	**2.9**
Weight of dry soil (g)	**16.0**	**16.3**	**17.4**	**24.8**	**15.3**	**18.3**
Moisture content	**28.6**	**31.3**	**34.2**	**36.3**	**22.2**	**15.8**
					PL= 19.0%	

B9: Consistency Test by Cassagrande Method for Su grade Layer Itapa- Ilupeju Ekiti Road

Trial no.	1	2	3	4		
No. of blows	**44**	**32**	**20**	**11**	**Plastic limit**	
Container identification no.	**M3**	**N3**	**O3**	**P3**	**Q3**	**R3**
Weight of empty container (g)	**26.6**	**26.7**	**20.1**	**26.9**	**11.6**	**19.8**
Weight of container + wet soil (g)	**45.6**	**55.2**	**48.2**	**59.2**	**31.9**	**42.7**
Weight of container + dry soil (g)	**41.6**	**47.7**	**41.4**	**50.9**	**27.5**	**38.6**
Weight of water (g)	**4.0**	**6.1**	**6.8**	**8.3**	**4.4**	**4.9**
Weight of dry soil (g)	**15.0**	**21.0**	**21.3**	**24.0**	**15.9**	**17.0**
Moisture content	**26.7**	**29.0**	**31.9**	**34.6**	**27.8**	**24.1**
					PL=26.0%	

B10: Consistency Test by Cassagrande Method for Base Layer Oye Ekiti Road

Trial no.	1	2	3	4		
No. of blows	**42**	**32**	**21**	**11**	**Plastic limit**	
Container identification no.	**S1**	**T1**	**U1**	**V1**	**W1**	**X1**
Weight of empty container (g)	**15.3**	**9.6**	**16.4**	**26.4**	**18.5**	**19.8**
Weight of container + wet soil (g)	**42.0**	**40.4**	**46.3**	**61.0**	**39.8**	**36.0**
Weight of container + dry soil (g)	**34.6**	**32.0**	**37.1**	**49.6**	**35.9**	**32.4**
Weight of water (g)	**7.4**	**8.4**	**9.2**	**11.4**	**3.9**	**3.6**
Weight of dry soil (g)	**21.3**	**22.4**	**20.7**	**23.2**	**17.4**	**13.1**
Moisture content	**34.7**	**37.5**	**44.4**	**49.1**	**22.4**	**27.5**
					PL=25.0%	

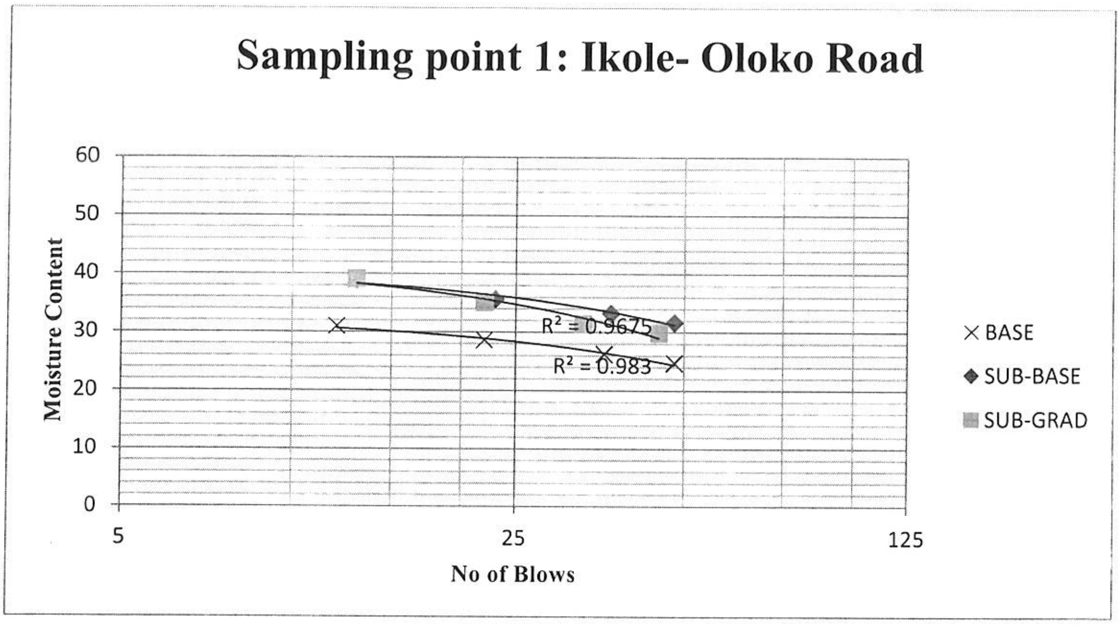
B11: Consistency Test by Cassagrande Method for Sub-base Layer Oye Ekiti

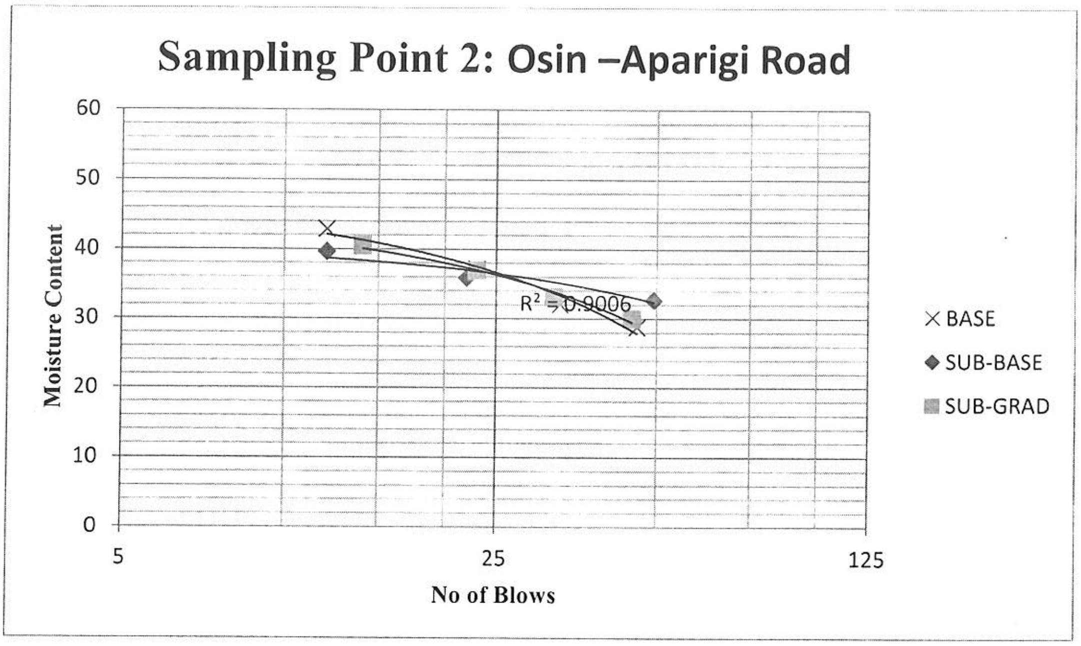
Trial no.	1	2	3	4		
No. of blows	46	34	22	14	**Plastic limit**	
Container identification no.	S2	T2	U2	V2	W2	X2
Weight of empty container (g)	26.8	17.7	19.8	19.2	13.6	15.3
Weight of container + wet soil (g)	44.5	39.7	44.6	46.3	31.5	33.8
Weight of container + dry soil (g)	40.4	34.2	37.7	38.7	28.4	30.6
Weight of water (g)	4.1	5.5	6.9	7.6	3.1	3.2
Weight of dry soil (g)	13.6	16.5	17.9	19.0	14.8	15.3
Moisture content	30.1	33.3	36.5	40.0	20.9	20.9
					PL=20.9%	
					SL=7.1%	

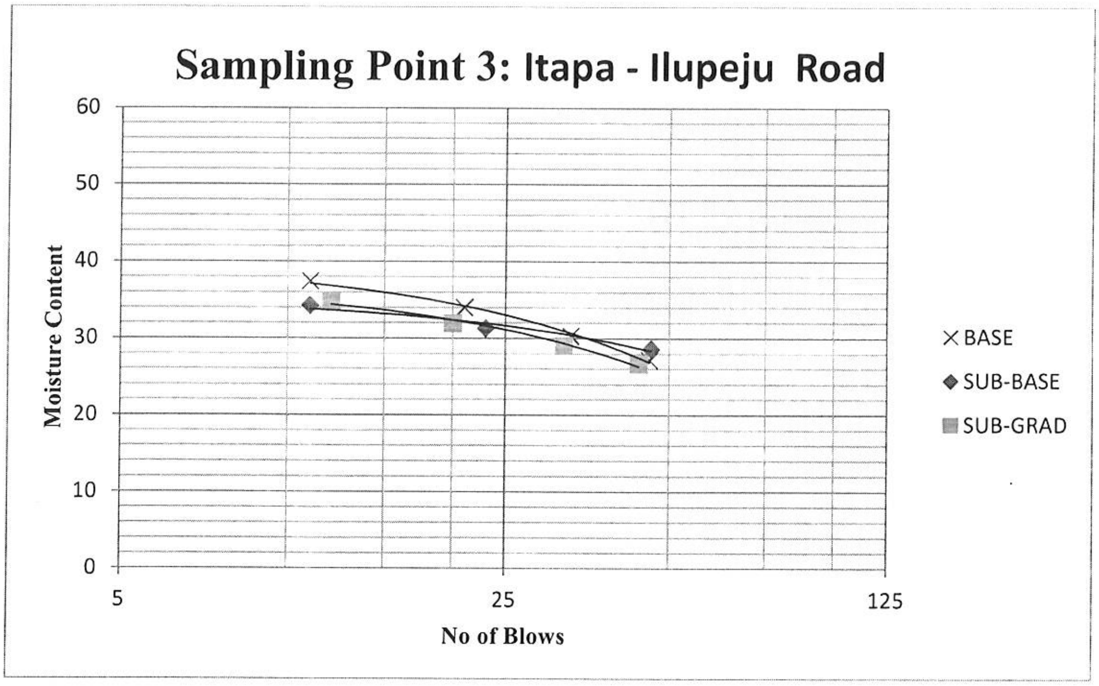
B12: Consistency Test by Cassagrande method for subgrade layer Oye Ekiti Road

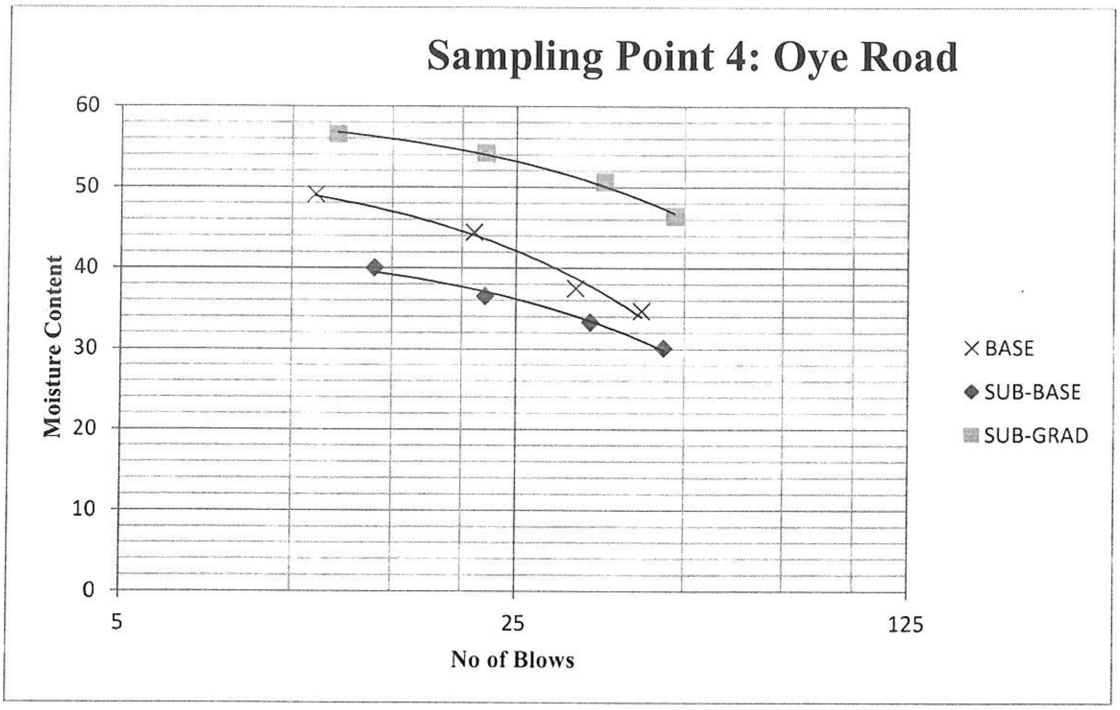
Trial no.	1	2	3	4		
No. of blows	46	34	22	14	**Plastic limit**	
Container identification no.	S3	T3	U3	V3	W3	X3
Weight of empty container (g)	26.9	10.0	26.8	26.9	7.7	11.7
Weight of container + wet soil (g)	49.0	36.2	58.1	61.7	24.2	26.2
Weight of container + dry soil (g)	42.0	27.4	47.1	49.5	21.2	23.6
Weight of water (g)	7.0	8.8	11.0	12.2	3.0	2.6
Weight of dry soil (g)	15.11	17.4	20.3	22.6	13.5	11.9
Moisture content	46.4	50.6	54.2	56. 5	22.2	21.8
					PL= 22.0%	
					SL=7.9%	

Graphs of Consistency Tests






Sampling Point 3: Itapa - Ilupeju Road
60
50
40
30
20
10
0
Moisture Content
5
25
125
No of Blows
× BASE
◆ SUB-BASE
■ SUB-GRAD


Sampling Point 4: Oye Road
60
50
40
30
20
10
0
Moisture Content
5
25
125
No of Blows
× BASE
◆ SUB-BASE
■ SUB-GRAD

APPENDIX C

Specific Gravity Test

Results Specific Gravity Test

Sampling Point 1

Road layers	**BASE**		**SUB BASE**		**SUB GRADE**	
Trial no.	**1**	**2**	**1**	**2**	**1**	**2**
Weight of empty density bottle (g)	**23.8**	25.8	26.4	26.4	26.4	26.4
Weight of density bottle + oven dried soil (g)	56.2	56.9	53.8	56.3	52.5	53.0
Weight of density bottle + oven dried soil + water (g)	98.1	97.9	93.9	97.6	92.5	95.0
Weight of density bottle + water full (g)	78.3	79.0	77.6	79.8	77.6	79.8
Specific Gravity						
Average specific gravity	2.56		2.46		2.33	

Sampling Point 2

Road layers	**BASE**		**SUB BASE**		**SUB GRADE**	
Trial no.	**1**	**2**	**1**	**2**	**1**	**2**
Weight of empty density bottle (g)	23.1	25.7	25.8	25.8	23.8	26.9
Weight of density bottle + oven dried soil (g)	50.2	50.6	52.3	52.1	54.5	58.9
Weight of density bottle + oven dried soil + water (g)	90.1	92.0	94.0	93.2	95.8	93.1
Weight of density bottle + water full (g)	74.8	77.8	78.0	78.1	78.2	79.4
Specific Gravity						
Average specific gravity	2.32		2.43		2.05	

Sampling Point 3

Road layers	BASE		SUB BASE		SUB GRADE	
Trial no.	1	2	1	2	1	2
Weight of empty density bottle (g)	23.8	25.8	25.7	25.8	23.8	25.8
Weight of density bottle + oven dried soil (g)	49.6	51.0	54.2	49.0	47.5	51.1
Weight of density bottle + oven dried soil + water (g)	93.2	93.6	94.4	91.1	92.2	93.7
Weight of density bottle + water full (g)	78.3	79.0	78.0	78.1	78.3	79.0
Specific Gravity						
Average specific gravity	2.38		2.32		2.41	

Sampling Point 4

Road layers	BASE		SUBBASE		SUB GRADE	
Trial no.	1	2	1	2	1	2
Weight of empty density bottle (g)	25.8	25.8	23.8	26.9	23.1	25.9
Weight of density bottle + oven dried soil (g)	53.4	51.4	51.7	56.7	51.9	51.5
Weight of density bottle + oven dried soil + water (g)	94.4	92.9	95.2	97.8	91.3	92.6
Weight of density bottle + water full (g)	78.1	78.0	78.2	79.4	74.8	87.8
Specific Gravity						
Average specific gravity	2.42		2.59		1.79	

APPENDIX D

Natural Moisture Content Test

Results of Natural Moisture Content Test

Sampling Point 1

Road layers	BASE		SUB BASE		SUB GRADE	
Trial no.	**1**	**2**	**1**	**2**	**1**	**2**
Container no.	**A1**	**A2**	**A3**	**A4**	**A5**	**A6**
Weight of container	18.4	18.5	16.3	11.9	14.1	13.2
Weight of container + soil+ water	88.4	88.5	75.9	65.2	87.0	62.5
Weight of container + soil	83.4	83.5	69.5	59.2	76.3	55.3
Weight of water	5.0	5.0	6.4	6.0	10.7	7.2
Weight of dry soil	65.0	65.0	53.2	45.1	62.2	42.1
Moisture content	7.7	7.7	12.0	13.3	17.2	17.1
AVERAGE MOISTURE CONTENT	7.7%		12.7%		17.2%	

Sampling Point 2

Road layers	BASE		SUB BASE		SUB GRADE	
Trial no.	1	2	1	2	1	2
Container no.	**B1**	**B2**	**B3**	**B4**	**B5**	**B6**
Weight of container	16.7	17.8	11.6	12.0	11.7	10.8
Weight of container + soil+ water	79.8	81.5	70.5	57.7	61.6	51.5
Weight of container + soil	70.5	72 .6	61.4	50.6	55.5	46.9
Weight of water	9.3	8.9	9.1	7.1	6.1	4.6
Weight of dry soil	53.8	53.8	49.8	38.6	43.8	36.1
Moisture content	17.3	16.5	18.3	18.4	13.9	12.9
AVERAGE MOISTURE CONTENT	16.9%		18.4%		13.3%	

Sampling Point 3

Road layers	BASE		SUB BASE		SUB GRADE	
Trial no.	1	2	1	2	1	2
Container no.	C1	C2	C3	C4	C5	C6
Weight of container	26.6	26.7	10.5	7.1	19.7	19.7
Weight of container + soil+ water	90.4	82.9	58.2	55.4	87.0	83.0
Weight of container + soil	82.0	75.8	51.7	48.9	77.8	74.2
Weight of water	8.4	7.1	6.5	6.5	9.2	8.2
Weight of dry soil	55.4	49.1	41.2	41.8	58.1	54.5
Moisture content	15.2	14.5	15.8	15.6	15.8	16.1
AVERAGE MOISTURE CONTENT	14.9%		15.7%		16.0%	

Sampling Point 4

Road layers	BASE		SUB BASE		SUB GRADE	
Trial no.	1	2	1	2	1	2
Container no.	D1	D2	D3	D4	D5	D6
Weight of container	19.8	12.2	8.4	8.1	9.6	11.6
Weight of container + soil+ water	83.6	60.9	55.4	53.9	49.4	57.8
Weight of container + soil	72.8	52.6	48.8	48.7	42.3	49.5
Weight of water	10.8	8.3	6.6	5.2	7.1	8.3
Weight of dry soil	53.0	40.4	40.4	40.6	32.7	37.9
Moisture content	20.4	20.5	16.3	12.8	21.7	21.9
AVERAGE MOISTURE CONTENT	20.5%		14.6%		21.8%	

APPENDIX E

COMPACTION TEST

Results of Compaction Test

E1: Compaction test Result for Base Sample Ikole- Oloko Ekiti Road

TEST NO	L1	L2	L3	L4
WT. OF MOULD + WET SOIL (g)	6250	6600	6800	6750
WT. OF MOULD (g)	4550	4550	4550	4550
WT. OF WET SOIL (g)	1700	2050	2250	2200
WET DENSITY(kg/m3)	1.7	2.05	2.25	2.20
CONTAINER NO	**J1**	**J2**	**J3**	**J4**
WT. OF CONTAINER + WET SOIL (g)	13.2	43.2	13.6	23.5
WT. OF CONTAINER + DRIED SOIL (g)	60.7	49.4	76.7	87.8
WT. OF EMPTY CONTAINER (g)	57.6	45.0	70.1	79.3
WT. OF WATER (g)	3.1	4.4	6.8	8.5
WT. OF DRY SOIL (g)	44.4	42.6	56.5	55.8
MOISTURE CONTENT (%)	7.0	10.3	12.0	15.2
DRY DENSITY (kg/m3)	1.59	1.86	2.01	1.91

E2: Compaction test Result for Sub-base Sample Ikole- Oloko Ekiti Road

TEST NO	L1	L2	L3	L4
WT. OF MOULD + WET SOIL (g)	6250	6350	6550	6450
WT. OF MOULD (g)	4550	4550	4550	4550
WT. OF WET SOIL (g)	1700	1800	2000	1900
WET DENSITY(kg/m3)	1.70	1.80	2.00	1.90
CONTAINER NO	**K1**	**K2**	**K3**	**K4**
WT. OF CONTAINER + WET SOIL (g)	8.4	17.4	6.7	6.6
WT. OF CONTAINER + DRIED SOIL (g)	42.9	69.3	48.6	71.0
WT. OF EMPTY CONTAINER (g)	38.9	62.2	38.8	59.2
WT. OF WATER (g)	4.0	7.1	9.8	11.8
WT. OF DRY SOIL (g)	30.5	44.8	32.1	64.4
MOISTURE CONTENT (%)	13.1	15.8	16.6	18.3
DRY DENSITY (kg/m3)	1.50	1.55	1.72	1.61

E3: Compaction test Result for Subgrade Sample Ikole- Oloko Ekiti Road

TEST NO	L1	L2	L3	L4
WT. OF MOULD + WET SOIL (g)	6250	6550	6650	6550
WT. OF MOULD (g)	4550	4550	4550	4550
WT. OF WET SOIL (g)	1700	2000	2100	2000
WET DENSITY(kg/m3)	1.70	2.00	2.10	2.00
CONTAINER NO	**L1**	**L2**	**L3**	**L4**
WT. OF CONTAINER + WET SOIL (g)	10.4	12.4	9.7	78
WT. OF CONTAINER + DRIED SOIL (g)	70.2	79.6	78.8	80.8
WT. OF EMPTY CONTAINER (g)	65.8	72.4	69.8	69.4
WT. OF WATER (g)	4.4	7.1	9.0	11.2
WT. OF DRY SOIL (g)	50.4	60.0	60.1	61.6
MOISTURE CONTENT (%)	7.9	11.8	15.0	18.3
DRY DENSITY (kg/m3)	1.58	1.79	1.83	1.69

E4: Compaction test Result for Base Sample Osin– Aparigi Ekiti Road

TEST NO	L1	L2	L3	L4
WT. OF MOULD + WET SOIL (g)	4850	5050	5150	5050
WT. OF MOULD (g)	3200	3200	3200	3200
WT. OF WET SOIL (g)	1650	1850	1950	1850
WET DENSITY(kg/m3)	1.65	1.85	1.95	1.85
CONTAINER NO	**A1**	**A2**	**A3**	**A4**
WT. OF CONTAINER + WET SOIL (g)	9.9	13.7	10.3	20.1
WT. OF CONTAINER + DRIED SOIL (g)	54.9	58.2	62.8	79.4
WT. OF EMPTY CONTAINER (g)	51.1	53.4	56.0	70.8
WT. OF WATER (g)	3.8	4.8	6.8	8.6
WT. OF DRY SOIL (g)	41.2	38.7	45.7	50.7
MOISTURE CONTENT (%)	9.2	12.1	14.9	17.0
DRY DENSITY (kg/m3)	1.51	1.65	1.70	1.58

E5: Compaction test Result for Sub base Sample Osin– Aparigi Ekiti Road

TEST NO	L1	L2	L3	L4
WT. OF MOULD + WET SOIL (g)	4850	4950	5200	5150
WT. OF MOULD (g)	3200	3200	3200	3200
WT. OF WET SOIL (g)	1650	1750	2000	1950
WET DENSITY(kg/m3)	1.65	1.75	2.00	1.95
CONTAINER NO	B1	B2	B3	B4
WT. OF CONTAINER + WET SOIL (g)	20.1	20.8	26.6	17.5
WT. OF CONTAINER + DRIED SOIL (g)	85.8	73.8	80.3	67.6
WT. OF EMPTY CONTAINER (g)	79.1	66.4	71.3	56.8
WT. OF WATER (g)	6.7	7.4	9.0	10.8
WT. OF DRY SOIL (g)	59.0	45.6	44.7	46.0
MOISTURE CONTENT (%)	11.4	16.2	20.2	23.5
DRY DENSITY (kg/m3)	1.48	1.51	1.66	1.58

E6: Compaction test Result for Subgrade Sample Osin– Aparigi Ekiti Road

TEST NO	L1	L2	L3	L4
WT. OF MOULD + WET SOIL (g)	4950	5150	5250	5100
WT. OF MOULD (g)	3200	3200	3200	3200
WT. OF WET SOIL (g)	1750	1950	2050	1900
WET DENSITY(kg/m3)	1.75	1.95	2.05	1.90
CONTAINER NO	C1	C2	C3	C4
WT. OF CONTAINER + WET SOIL (g)	9.9	13.7	12.2	21.6
WT. OF CONTAINER + DRIED SOIL (g)	72.9	68.8	72.6	86.6
WT. OF EMPTY CONTAINER (g)	67.9	62.7	64.4	76.3
WT. OF WATER (g)	58.0	49.0	52.2	54.7
WT. OF DRY SOIL (g)	5.0	6.1	8.2	10.3
MOISTURE CONTENT (%)	8.6	12.4	15.8	18.8
DRY DENSITY (kg/m3)	1.61	1.69	1.77	1.60

E7: Compaction test Result for Base Sample Itapa- Ilupeju Ekiti Road

TEST NO	L1	L2	L3	L4
WT. OF MOULD + WET SOIL (g)	5600	5800	5950	5650
WT. OF MOULD (g)	3550	3550	3550	3550
WT. OF WET SOIL (g)	2050	2250	2400	2100
WET DENSITY(kg/m3)	2.05	2.25	2.40	2.10
CONTAINER NO	D1	D2	D3	D4
WT. OF CONTAINER + WET SOIL (g)	20.3	19.5	19.8	19.8
WT. OF CONTAINER + DRIED SOIL (g)	71.8	77.8	77.8	84.4
WT. OF EMPTY CONTAINER (g)	67.6	71.1	70.0	74.1
WT. OF WATER (g)	4.2	6.7	7.8	10.3
WT. OF DRY SOIL (g)	47.3	51.6	50.2	54.3
MOISTURE CONTENT (%)	8.9	13.0	15.5	19.0
DRY DENSITY (kg/m3)	1.88	1.99	2.08	1.76

E8: Compaction test Result for Sub base Sample Itapa- Ilupeju Ekiti Road

TEST NO	L1	L2	L3	L4
WT. OF MOULD + WET SOIL (g)	5600	5800	5950	5650
WT. OF MOULD (g)	3550	3550	3550	3550
WT. OF WET SOIL (g)	1950	2150	2300	2200
WET DENSITY(kg/m3)	1.95	2.15	2.30	2.20
CONTAINER NO	E1	E2	E3	E4
WT. OF CONTAINER + WET SOIL (g)	26.7	27.1	21.3	19.7
WT. OF CONTAINER + DRIED SOIL (g)	72.2	80.5	94.1	81.2
WT. OF EMPTY CONTAINER (g)	68.8	75.0	84.8	70.2
WT. OF WATER (g)	3.4	5.5	9.3	11.0
WT. OF DRY SOIL (g)	42.1	47.9	63.5	50.5
MOISTURE CONTENT (%)	8.1	11.5	14.6	21.8
DRY DENSITY (kg/m3)	1.80	1.93	2.01	1.81

E9: Compaction test Result for Subgrade Sample Itapa- Ilupeju Ekiti Road

TEST NO	L1	L2	L3	L4
WT. OF MOULD + WET SOIL (g)	5600	5800	5850	7700
WT. OF MOULD (g)	3550	3550	3550	3350
WT. OF WET SOIL (g)	2050	2150	2300	2150
WET DENSITY(kg/m3)	2.05	2.15	2.30	2.15
CONTAINER NO	F1	F2	F3	F4
WT. OF CONTAINER + WET SOIL (g)	27.0	26.7	27.0	19.9
WT. OF CONTAINER + DRIED SOIL (g)	89.8	87.5	92.0	84.3
WT. OF EMPTY CONTAINER (g)	84.5	80.6	83.4	74.4
WT. OF WATER (g)	5.3	6.9	8.6	9.9
WT. OF DRY SOIL (g)	57.5	53.9	56.4	54.5
MOISTURE CONTENT (%)	9.2	12.8	15.2	18.2
DRY DENSITY (kg/m3)	1.88	1.91	2.00	1.82

E10: Compaction test Result for Base Sample Oye Ekiti Road

TEST NO	L1	L2	L3	L4
WT. OF MOULD + WET SOIL (g)	5600	5800	5950	5800
WT. OF MOULD (g)	3800	3800	3800	3800
WT. OF WET SOIL (g)	1800	2000	2150	2000
WET DENSITY(kg/m3)	1.8	2.0	2.15	2.00
CONTAINER NO	G1	G2	G3	G4
WT. OF CONTAINER + WET SOIL (g)	19.2	5.3	16.8	10.7
WT. OF CONTAINER + DRIED SOIL (g)	62.9	48.6	68.8	71.7
WT. OF EMPTY CONTAINER (g)	59.0	43.6	62.0	62.
WT. OF WATER (g)	3.9	5.0	6.8	9.7
WT. OF DRY SOIL (g)	39.8	38.3	45.2	51.3
MOISTURE CONTENT (%)	9.8	13.1	15.0	18.9
DRY DENSITY (kg/m3)	1.64	1.77	1.87	1.67

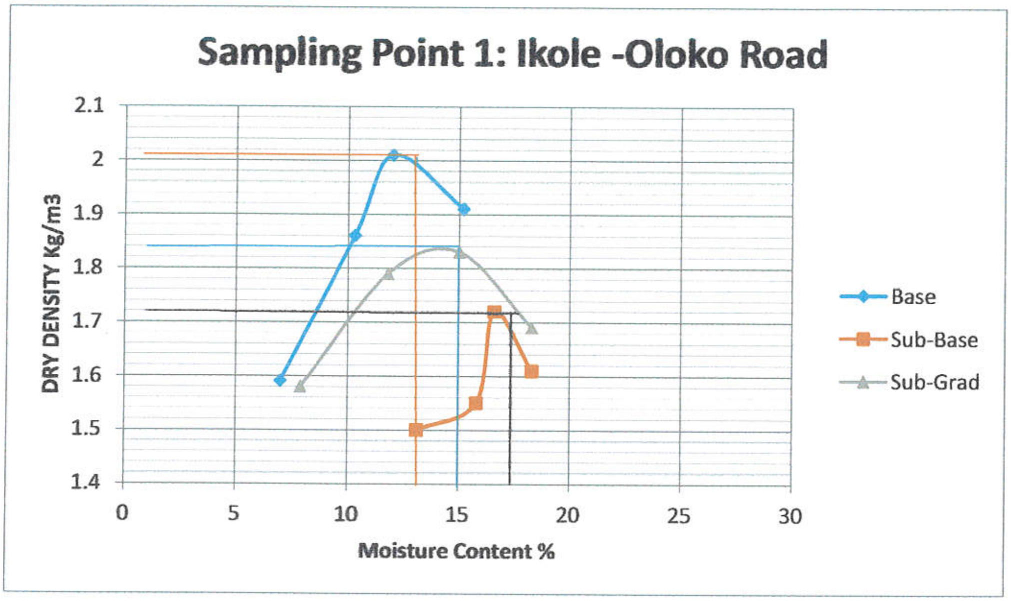
E11: Compaction test Result for Sub-base Sample Oye Ekiti Road

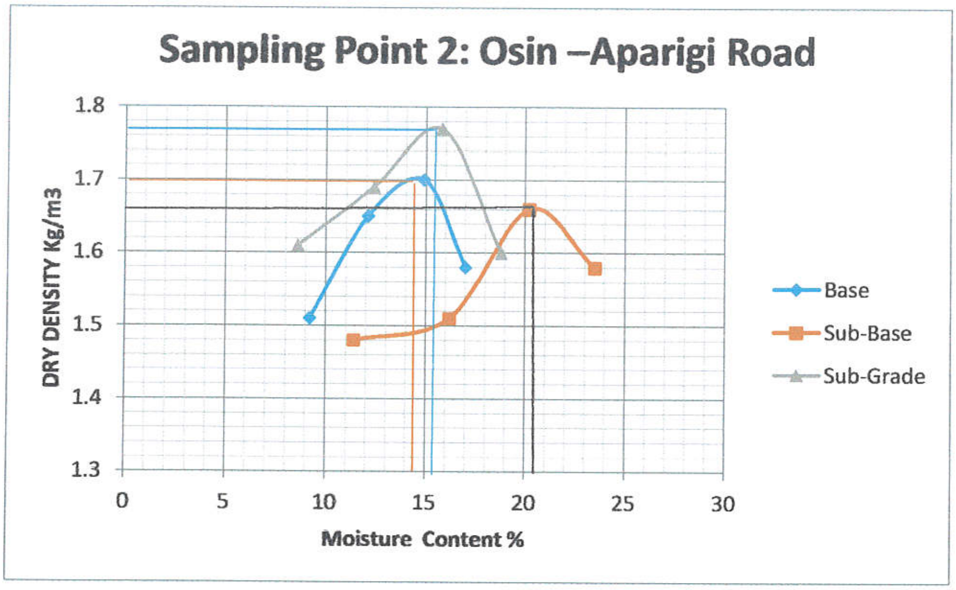
TEST NO	L1	L2	L3	L4
WT. OF MOULD + WET SOIL (g)	5650	5800	5950	5800
WT. OF MOULD (g)	3800	3800	3800	3800
WT. OF WET SOIL (g)	1850	2000	2150	2000
WET DENSITY(kg/m3)	1.85	2.00	2.15	2.00
CONTAINER NO	H1	H2	H3	H4
WT. OF CONTAINER + WET SOIL (g)	17.9	20.3	11.3	26.7
WT. OF CONTAINER + DRIED SOIL (g)	65.3	70.9	74.1	95.9
WT. OF EMPTY CONTAINER (g)	62.3	67.0	68.4	88.2
WT. OF WATER (g)	3.0	3.9	5.9	7.7
WT. OF DRY SOIL (g)	44.4	46.9	56.7	61.5
MOISTURE CONTENT (%)	6.8	8.3	10.1	12.5
DRY DENSITY (kg/m3)	1.75	1.85	1.95	1.78

E12: Compaction test Result for Subgrade Sample Oye Ekiti Road

TEST NO	L1	L2	L3	L4
WT. OF MOULD + WET SOIL (g)	53OO	5500	5700	5600
WT. OF MOULD (g)	3800	3800	3800	3800
WT. OF WET SOIL (g)	1500	1700	1900	1800
WET DENSITY(kg/m3)	1.50	1.70	1.90	1.8
CONTAINER NO	X1	X2	X3	X4
WT. OF CONTAINER + WET SOIL (g)	22.8	26.6	26.6	12.6
WT. OF CONTAINER + DRIED SOIL (g)	84.2	80.8	80.4	68.2
WT. OF EMPTY CONTAINER (g)	78.6	73.7	71.6	58.1
WT. OF WATER (g)	5.6	7.1	8.8	10.1
WT. OF DRY SOIL (g)	55.8	47.1	45.0	45.5
MOISTURE CONTENT (%)	10.0	15.1	19.6	22.2
DRY DENSITY (kg/m3)	1.36	1.48	1.59	1.47

Graphs for Compaction Test Results





APPENDIX F

CALIFORNIA BEARING RATIO TEST

Results of California Bearing Ratio Test

Sampling Point 1

Road layers	BASE		SUB BASE		SUB GRADE	
Plunger penetration reading (mm)	DR	LOAD (KN)	DR	LOAD (KN)	DR	LOAD (KN)
50	107	2.675	49	1.23	2	0.05
100	161	4.025	72	1.80	4	0.1
150	216	5.4	89	2.23	9	0.23
200	275	6.875	113	2.83	13	0.33
250	320	8.0	125	3.13	16	0.4
300	371	9.25	137	3.45	20	0.5
350	433	10.83	147	3.68	25	0.625
400	501	12.53	164	4.10	31	0.775
450	562	14.05	198	4.73	36	0.9
500	648	16.2	205	5.15	41	1.03
550	697	17.43	224	5.60	52	1.3
600	752	18.8	237	5.95	69	1.73
650	790	19.75	244	6.10	83	2.08
700	825	20.63	250	6.25	98	2.45
750	861	21.53	258	6.45	107	2.68

Sampling Point 2

Road layers	BASE		SUB BASE		SUB GRADE	
Plunger penetration reading (mm)	**DR**	**LOAD (KN)**	**DR**	**LOAD (KN)**	**DR**	**LOAD (KN)**
50	**136**	**3.4**	**42**	**1.05**	**0.2**	**0.005**
100	**184**	**4.6**	**63**	**1.58**	**0.6**	**0.015**
150	**236**	**5.9**	**80**	**2.0**	**1**	**0.025**
200	**288**	**7.2**	**99**	**2.48**	**2.5**	**0.063**
250	**359**	**8.98**	**120**	**3.00**	**5**	**0.13**
300	**402**	**10.05**	**139**	**3.48**	**7**	**0.18**
350	**466**	**11.65**	**145**	**3.63**	**10**	**0.25**
400	**514**	**12.85**	**157**	**3.93**	**13**	**0.33**
450	**579**	**14.475**	**163**	**4.08**	**15**	**0.38**
500	**640**	**16.0**	**173**	**4.33**	**18**	**0.45**
550	**687**	**17.18**	**180**	**4.5**	**20**	**0.50**
600	**720**	**18**	**193**	**4.83**	**23**	**0.58**
650	**771**	**19.28**	**199**	**16.25**	**26**	**0.65**
700	**802**	**20.05**	**210**	**5.25**	**29**	**0.73**
750	**818**	**20.45**	**216**	**5.4**	**32**	**0.8**

Sampling Point 3

Road layers	**BASE**		**SUB BASE**		**SUB GRADE**	
Plunger penetration reading (mm)	**DR**	**LOAD (KN)**	**DR**	**LOAD (KN)**	**DR**	**LOAD (KN)**
50	**188**	**4.7**	**23**	**0.58**	**7**	**0.175**
100	**231**	**5.78**	**50**	**1.25**	**10**	**0.25**
150	**278**	**6.95**	**89**	**2.23**	**16**	**0.4**
200	**320**	**8.0**	**110**	**2.75**	**20**	**0.5**
250	**385**	**9.63**	**164**	**4.10**	**26**	**0.65**
300	**431**	**10.78**	**205**	**5.13**	**31**	**0.78**
350	**498**	**12.45**	**271**	**6.78**	**38**	**0.95**
400	**555**	**13.88**	**307**	**7.68**	**42**	**1.05**
450	**610**	**15.25**	**355**	**8.88**	**53**	**1.33**
500	**685**	**17.13**	**388**	**9.70**	**83**	**2.08**
550	**715**	**17.88**	**415**	**10.38**	**99**	**2.48**
600	**760**	**19.0**	**437**	**10.93**	**118**	**2.95**
650	**800**	**20**	**457**	**11.43**	**135**	**3.38**
700	**825**	**20.63**	**468**	**11.70**	**142**	**3.55**
750	**841**	**21.03**	**500**	**12.5**	**153**	**3.83**

Sampling Point 4

Road layers	BASE		SUB BASE		SUB GRADE	
Plunger penetration reading (mm)	DR	LOAD (KN)	DR	LOAD (KN)	DR	LOAD (KN)
50	198	4.95	11	0.28	0.2	0.0005
100	255	6.38	21	0.53	0.4	0.010
150	303	7.58	47	1.18	0.5	0.013
200	347	8.68	85	2.13	0.6	0.015
250	391	9.78	101	2.53	0.8	0.02
300	439	10.98	128	3.2	0.9	0.023
350	499	12.48	143	3.58	2	0.05
400	549	13.73	157	3.93	4.5	0.11
450	600	15.0	164	4.1	6	0.15
500	657	16.43	172	4.30	8	0.2
550	702	17.55	179	4.48	9.5	0.24
600	748	18.70	185	4.63	11	0.28
650	788	19.70	200	5.0	13	0.33
700	810	20.25	205	5.13	14	0.35
750	831	20.78	212	5.30	16	0.4

APPENDIX G

PERMEABILITY TEST

Results of Permeability Test

Sampling Point 1

COEFFICIENT OF PERMEABILITY (Falling head)

Location: Ikole- Oloko Ekiti Road, Ekiti state

pavement layer: subgrade

Soil Description: Light Brown

Date: 21-09-2017

Sample Dimensions: diam. 10cm; Area, A 85.10cm^2; Vol. 1106.30cm^3 ; HtL 13cm

Sand pipe 1.0 cm **Burette 50.1ml** **diameter 10.2cm** **Area, a 1.130cm^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	21.8		44.50	22	1	94.5	50	21.8	22
2	94.5	50	22.3		44.50	22	2	94.5	50	22.3	22
3	94.5	50	22.9		44.50	22	3	94.5	50	22.9	22
4	94.5	50	24		44.50	22	4	94.5	50	24	22
							Average	94.5	50	22.75	22

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

$KT = \left(\frac{aL}{At}\right) In(h_1/h2) =$ 4.83E-03 = 0.0048301616 $cm/_s$

$K_{20} = \alpha K_T = 4.71E - 03$ = 0.0047147207 $cm/_s$

Degree of Permeability: LOW (Soil testing for Engineers by T. Williams lambe 1951)