EVALUATION OF GEOTECHNICAL PROPERTIES OF IKOLE-IFAKI ROAD

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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.

DEPARTMENT OF CIVIL ENGINEERING

FACULTY OF ENGINEERING

FEDERAL UNIVERSITY OYE- EKITI, EKITI STATE.

MARCH, 2019

ABSTRACT

This project is aimed at evaluating the geotechnical properties of soils around Ikole-Ekiti to Ifaki-Ekiti, Nigeria. The objectives of this project are to carry out the preliminary analysis and Engineering analysis of soils, evaluate the geotechnical properties of soils along Ikole – Ifaki road and compare the results with acceptable standards. The research work was conducted at selected points along Ikole - Ifaki road where soil samples were taken at a depth of 1.2m below the existing ground level from seven trial pits. After collection, the soil samples were stored in polythene bags to prevent moisture content loss. 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 BS sieves to remove large particles. Molding of test specimens were started as soon as possible after completion of identification. All tests were performed according to standard methods contained in BS 1377 (1990).The seven soils samples collected were subjected to the following laboratory tests: natural moisture content, grain size analysis, Atterberg limits, specific gravity, compaction test, california bearing ratio and direct shear test in accordance with the British standard 1377 of (1990). The results of the specific gravity of samples ranged from (2.38-2.53). The grain size analysis results showed the percentage passing No. 200 ranged from (15.4%-62.1%). Compaction test showed a maximum dry density range of (1.51kg/m3-1.72kg/m3) and an optimum moisture content between (16%-19.5%). The California bearing ratio (C.B.R) ranged from (13.8%-71.4%) for 2.5mm plunger penetration and (23.4%-80.4%) for 5mm plunger. In conclusion the results obtained shows that the materials along the 34.4km distance between point A (IkoleEkiti) and point G (Ifaki-Ekiti) needs to be stabilized properly in order for it to be suitable for future road way construction to prevent rapid road way deterioration as it is observed by road users. The geotechnical data obtained in this project will be useful for future road foundation design and construction in the study area.

ACKNOWLEDGEMENT

We would like to express our sincere gratitude to the Almighty for His protection, good health and guidance during the whole duration of our study. We wish to express our gratitude to our supervisor **Prof. O.O Amu** for the excellent support, direction, suggestions and above all his patience in guiding us at every stage of this project.

Our sincere thanks to the entire Academic and Non-Academic staffs of the Department of Civil Engineering, Federal University Oye–Ekiti for their efforts in imparting knowledge to us the and equipping us with the necessary tools needed to thrive in this world. We also want to thank all whose names were not mentioned in this work but played a significant role for the success of this study.

DEDICATION

We dedicate this project to the Almighty who has been our guidance and source throughout our study years in this great institution and to our wonderful families (Mr & Mrs Bukola-Ojumu), (Alh. & Mrs. Williams and Engr. O. Bakare) for all the love, care and support they have given. We are indeed grateful.

CERTIFICATION

This is to certify that this project was written by BUKOLA-OJUMU VICTOR **(CVE/13/1057)** and DABIRI ABDULQUDUS BOLAJI **(CVE/13/1058)** under my supervision, in partial fulfilment of the requirements for the award of Bachelor of Engineering (B.Eng) degree in Civil Engineering, Federal University Oye- Ekiti, Ekiti State, Nigeria.

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CHAPTER ONE

INTRODUCTION

1.1 General Introduction

The relevance of road to life and its activities cannot be over emphasized. Road is as old as the origin of man and plays a vital role in the life of the people in communities, states, countries. It is quite obvious that life would have been very difficult without the existence of roads. Nigeria for instance acknowledged the importance of road such that the local, state and federal government are all involved in the development and maintenance of road network. History has shown that provision of roadway is a necessary tool for civilization. It advances prosperity increase and this bring about demand for better and faster communication facilities especially roads. The continuing increase in Nigeria's population has made the daily movement of passengers and goods an increasing complex problem. 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. 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). The current state of the road under consideration in this project is bad and is characterized at different sections by failed sections. The traffic capacity of the road is increasing because of the influence of the university community to the economic environment of Oye-Ekiti and Ikole-Ekiti towns respectively. Because the road has not been properly maintained, motorist prefer going through the longer way by passing through Ado - Ilumoba - Ijeshaisu road. When the road is properly improved, all these problems will be eliminated and a lot of benefit will be accrued, benefit to the intending business center as a result of more people using the road and reduction in travel time.

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 evaluate the geotechnical properties along Ikole-Ifaki, Ekiti State, Nigeria.

1.3 Justification

No previous attempt has been made to evaluate the geotechnical properties of the road section between the towns of Ikole-Ekiti and Ifaki-Ekiti , however, due to the impending increase in traffic on the road due to the influence of a new university community on the road it becomes imperative to improve the standard of the road and maintain it accordingly.

1.4 Description of Study Area

The study area is the road between Ifaki and Ikole Ekiti. This road is a federal road (Truck A) which runs 34.4km.There are seven towns (Ikole, Osin, Itapa, Ilupeju, Oye, Ayegbaju, Ifaki) Ekiti along this road. 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.

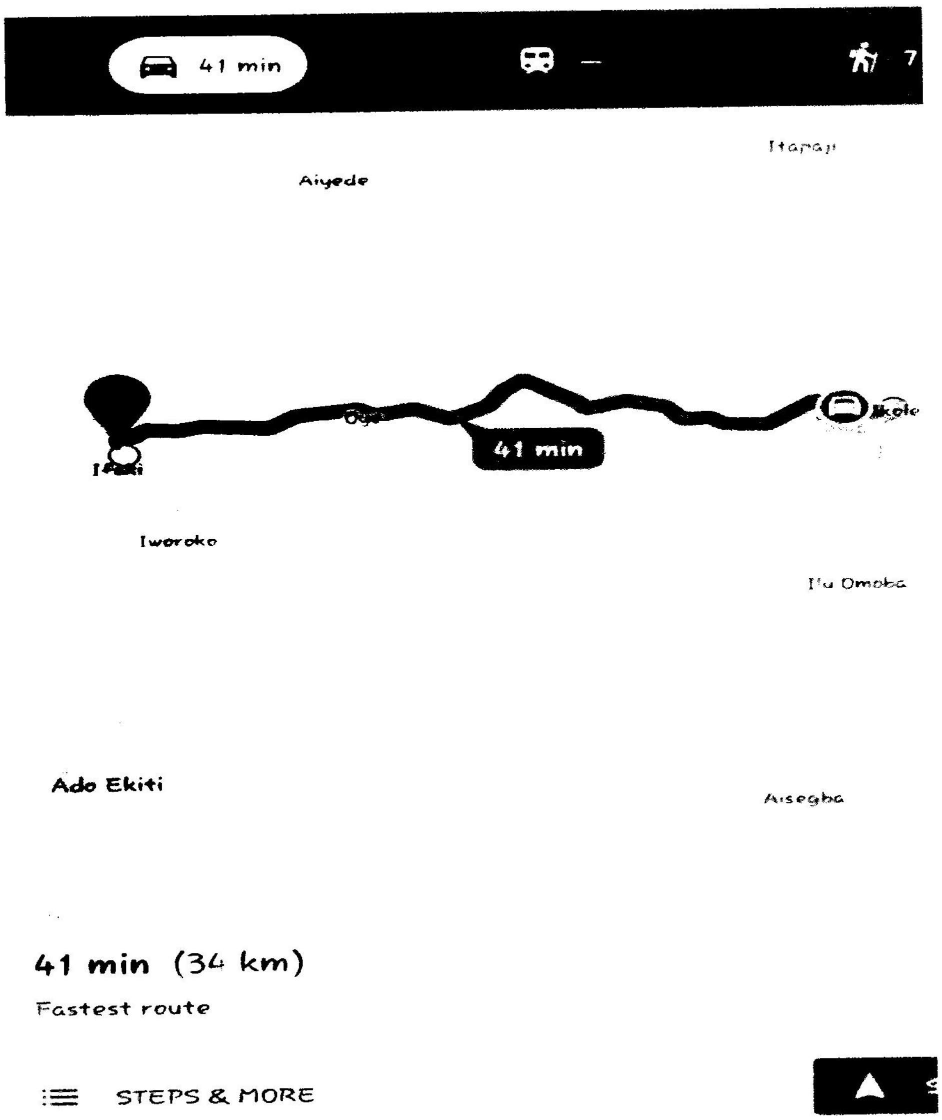


Plate 1.4: Distance from Ikole-Ekiti to Ifaki-Ekiti

1.5 Aim and Objectives

The aim of this research is to evaluate the geotechnical properties along Ikole-Ifaki, Ekiti State, Nigeria.

The objectives of this research are:

i. To carry out the preliminary analysis of the soil samples,
ii. To carry out engineering properties of the soil samples, and
iii. Evaluate the Geotechnical properties of the soil samples and compare with acceptable standards.

1.6 Significance of the Study

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

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

Engineering can be defined as a profession which applies the knowledge of science and technology to proffer solutions to societal problems facing humanity. The development of engineering over the years has been driven by various discoveries by scientist and inventions of modern tools by technologist which are always results of proven hypothesis through different researches with different degrees of comprehensiveness. Transportation engineering is not an exception to this general norm, where engineers have been highly occupied with search for ways of improving the existing transportation systems in their immediate environment. Improvement in transportation system has brought rapid economic growth especially in developed countries. Contrarily, the stagnant development of transportation system in developing countries as a result of many factors has not only dwindled economic growth but has also lowered the dignity of transportation engineering. As earlier said, the nucleus of every discovery or invention is the research carried out to yield a positive or negative result. The procedure of executing this essential building block of development includes:

1. Review of the related literature to the hypothesis at stake
2. Accumulation of unpublished information through seminars, lecture notes and so on.
3. Experimentation
4. Conclusion and Recommendation which may either be positive or negative

An effective researcher is one who studied available resources to know the level of his knowledge on his hypothesis (project topic) before embarking on a journey of bringing about discovery or invention. In essence, a research will mean a waste of time if it has been done before. In view of the above, it becomes essential to carry out an exhaustive review of the available literature relating to the subject of study so as to make the entire exercise effective and efficient. This chapter is entirely dedicated to reviewing what has been done before. The assurance of effectiveness of a transportation system starts with good planning followed by accurate engineering design, well carried out construction using good materials and proper maintenance. The problem of highway engineering in Nigeria has always been associated with the failure of the foundation material (subgrade) and the embankment material (base and sub base) when exposed to moisture (water). Therefore, major researches on improving the highway transportation in Nigeria have tried to proffer solutions in different forms by studying different factors that ensure effectiveness of the road, most especially the materials which have been stabilized over the years to yield desired results.

2.2 Road Network

Road network the road network is the system of interconnected roads designed to accommodate wheeled road going vehicles and pedestrian traffic.

Plate 2.2: A road network

2.3 Description

The road network consists of a system of interconnected paved carriageways which are designed to carry buses, cars and goods vehicles; the road network generally forms the most basic level of transport infrastructure within urban areas, and will link with all other areas, both within and beyond the boundaries of the urban area.

A road network can be divided into parts such as:

i. intersections
ii. controlled or uncontrolled intersections
iii. roundabouts
iv. urban roads
v. rural roads
vi. motorways
vii. bicycle lanes

viii. footpaths and pedestrian areas
ix. pedestrian crossings
x. bridges
xi. tunnels

Furthermore, several road-side systems (or Intelligent Transportation Systems, ITS) and monitoring systems are used to control the traffic, such as

i. intersection control with traffic lights
ii. Variable Message Signs (VMS)
iii. Dynamic Road Information Panels (DRIPs)
iv. loop detectors

2.4 Functions

There are several functions of the road, the functions are briefly explained.

2.4.1 Social

The road network facilitates the movement of people allowing for social interaction. A high quality road network is essential not only for connecting key urban centers but for improving connectivity of more isolated local communities for whom many public transport options are limited or not available. Roads connect remote communities with the areas where employment options are more concentrated and services and facilities more readily available.

2.4.2 Economic

By connecting geographic locations, road networks facilitate the transport and movement of people, goods, and services, creating welfare. Road networks played a crucial role in the economic development of the 20th century, enabling relatively fast individual transportation for the masses from the second part of the 20th century. And although the development of air transportation and telecommunication networks started to compete with road networks during the latter part of the 20th century, in most EU countries road transport still plays a crucial role in the national and local transportation networks. Investments in road networks reduce the travel time between two locations, increase the robustness of the transportation network and hence reduce the travel costs. These kind of effects are referred to as the so-called direct effects of road networks. The economic impact of road networks extend in most cases beyond these direct effects due to the further rounds of economic activity as a result of the efficient transportation of goods, skills and persons, the so-called indirect economic effects. The investment in road networks, however, do not just lead to positive effects. Apart from the necessary investments in terms of time and money, road networks fill up land and have negative social and environmental impacts such as congestion, traffic accidents, light and noise pollution, and (of course) air pollution. In order to assess if an investment in a road network has a positive effect on society, or to compare different alternatives of transport infrastructure, economic tools can be used to value the positive and negative direct, indirect and external effects of these alternatives. The difficulty with this kind of economic appraisal is first of all that it is not easy to measure the valuation of travel time, and secondly that new road infrastructure will generate road use that would not have been made without the investments, the so-called induced demand. A third problem is that especially external effects such as quality of life, the value of unique nature, the value of no air pollution, are very difficult to be expressed in monetary terms. Although road networks are hardly affected by security threats (crime, terrorism), it can happen that roads get blocked due to e.g. riots, bomb explosions, traffic flow management, etc. The economic impact of these kind of threats can be significant, especially in indirect terms since roads facilitate economic activities. Security measures can prevent these negative economic effects, for instance, by ensuring there are alternatives routes for traffic (traffic flow management). These security measures, however, also generate economic impact, especially when these measures limit the mobility of road users.

2.4.3 Mobility

The function of a road network is to facilitate movement from one area to another. As such, it has an important role to play in the urban environment to facilitate mobility. It furthermore determines the accessibility of an (urban) area (together with public transport options). The capacity of a network is determined by its roads. The capacity of a road is the maximum number of vehicles that can pass a certain road section per hour. The capacity of a road is determined e.g. by its width, number of lanes and speed limit. If the traffic demand is larger than the road capacity, congestion will occur. When congestion is present, the road network cannot longer fulfill its task. Therefore, one tries to prevent or reduce congestion with traffic management measures. Developing a good road network has many positive effects, such as stimulating the development of certain areas (commercial activities, urban development, creating jobs etc.). For security, good accessibility by the road network is also important, for example for good accessibility in case of incidents (see also incident management). However, the road network may also facilitate criminals to reach their target. This could be controlled with access control.

2.4.4 Safety

Safety of road users is typically focused on road safety (prevention of accidents through speed control, seatbelt enforcement, etc.). Proper planning is critical in ensuring road safety: In the case of national roads, where the speed limit can exceed 50-60 km per hour, the proliferation of roadside development should be avoided. The intensification of or the development of new accesses to national routes can generate additional turning movements which in turn can introduce additional safety risks to road users. The lay-out of the road should help to improve traffic safety, for example by providing separate bicycle lanes and physically separated driving directions for motorways and larger urban roads. In The Netherlands one uses a uniform lay-out for different road types which should help to increase recognizability and safety.

2.4.5 Security Issues

The presence or absence of routes from one place to another can influence the mobility of the public, but also of criminals. This can have a direct effect on the perceived attractiveness of a location to criminals. Security issues influence by the pervasiveness of the road network because an easy escape adds to the attractiveness of targets, are:

i. Burglary
ii. Robbery
iii. Raid
iv. Vehicle theft

The layout of the road network and its associated potential to ram a car through a window front is essential to the attractiveness of objects for

2.4.6 Ram-raiding

Disturbance of important traffic nodes can attract a lot of public attention. This can make these nodes (like important or prominent bridges or tunnels) attractive objects for destruction of property by fanatics. The measures for each type of security issue can be found on the respective pages. There are few measures that are specifically suited or unsuited to this kind of urban object, but some general considerations can be mentioned: Directing traffic flows and access control should only be taken if it doesn't hamper the primary function of the road network, providing mobility, too much.

2.5 Types of Road

Many types of road exist around the world, all of which are thoroughfares which can be used by motorized traffic. Roads are not necessarily available for use by the general public without permission, highways are available to the public. Although a toll may be charged. In places the term highway is reserved for high capacity roads, in older English there was a distinction between highways and byways. Different terms are used in different countries for broadly the same design, although there are differences. For example, freeway USA, motorway UK and autobahn Germany all of which are broadly similar. A road can consist of one or more carriageways.

2.5.1 Generally private roads

These are roads not maintained by us and are usually unadopted. These roads are generally in a condition not meeting the standard of other adopted roads within the borough. The responsibility for private roads and streets lies with the road owners. Example is a Driveway.

2.5.1.1 Driveway

A driveway is a type of private road for local access to one or a small group of structures, and is owned and maintained by an individual or group. Driveways rarely have traffic lights, but some that bear heavy traffic, especially those leading to commercial businesses and parks, do.

2.5.2 Lower capacity highways

This is any highway public or private road or any other public way on land. It is used for major roads, but also includes other public tracks. Types include the following;

2.5.2.1 Alley

An alley or alleyway is a narrow lane, path, or passageway, often reserved for pedestrians, which usually runs between, behind, or within buildings in the older parts of towns and cities. It is also a rear access or service road (back lane), or a path or walk in a park or garden.

Plate 2.5.2: An alley

2.5.2.2 Backroad

A back road is a secondary type of road, usually found in rural areas. In North Carolina, where they are also referred to as "blue roads", the roads are often constructed of gravel, and are one or two-lane roads off larger roads such as parkways.

Plate 2.5.2.2: A back road

2.5.2.3 Byway

A byway in the United Kingdom is a track, often rural, which is too minor to be called a road. These routes are often unsurfaced, typically having the appearance of 'green lanes'. Despite this, it is legal (but may not be physically possible) to drive any type of vehicle along certain byways, the same as any ordinary tarmac road.

2.5.2.4 Frontage road

A frontage road (also known as an access road, service road, parallel road, etc.) is a local road running parallel to a higher-speed, limited-access road. A frontage road is often used to provide access to private driveways, shops, houses, industries or farms. Where parallel high-speed roads are provided as part of a major highway, these are also known as local-express lanes. A frontage lane is a paved path that is used for the transportation and travel from one street to another. Frontage lanes, closely related to a frontage road, are common in metropolitan areas and in small rural towns. Frontage lanes are technically not classified as roads due to their purpose as a bridge from one road to another, and due to the architectural standards that they are not as wide as a standard road, or used as commonly as a standard road, street, or avenue.

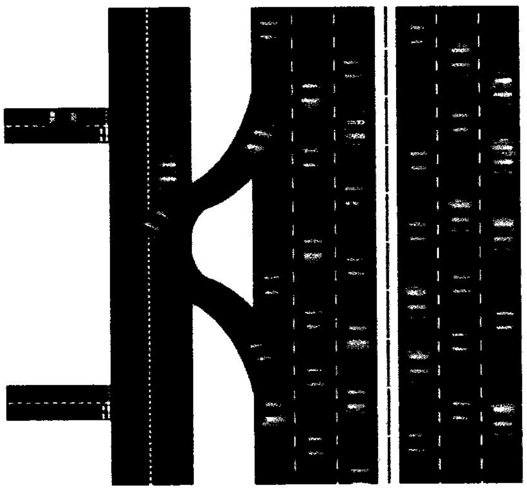


Plate 2.5.2.4: A frontage road

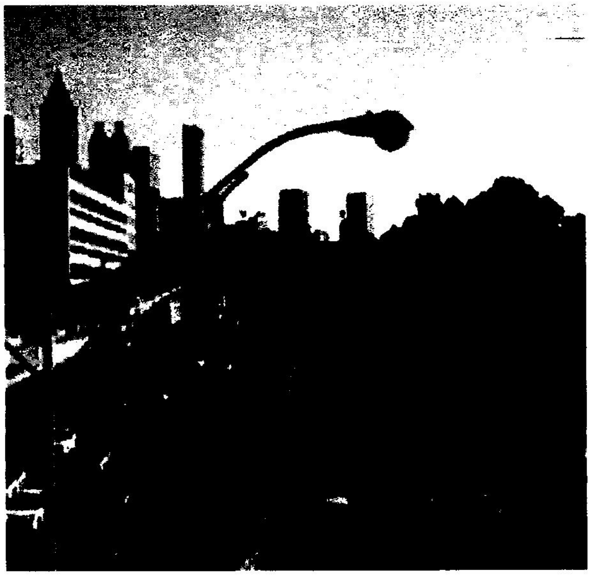
2.5.2.5 Gravel road

A gravel road is a type of unpaved road surfaced with gravel that has been brought to the site from a quarry or stream bed. They are common in less-developed nations, and also in the rural areas of developed nations such as Canada and the United States. In New Zealand, and other Commonwealth countries, they may be known as 'metal roads'. They may be referred to as 'dirt roads' in common speech, but that term is used more for unimproved roads with no surface material added. If well-constructed and maintained, a gravel road is an all-weather road.

Plate 2.5.2.5: A gravel road

2.5.3 Highway

A highway is any public or private road or other public way on land. It is used for major roads, but also includes other public roads and public tracks: It is not an equivalent term to controlled-access highway, or a translation for autobahn, AutoRoute, etc. 9 In North American and Australian English, major roads such as controlled-access highways or arterial roads are often state highways (Canada: provincial highways). Other roads may be designated "county highways" in the US and Ontario. These classifications refer to the level of government (state, provincial, county) that maintains the roadway. In British English, "highway" is primarily a legal term. Everyday use normally implies roads, while the legal use covers any route or path with a public right of access, including footpaths etc.



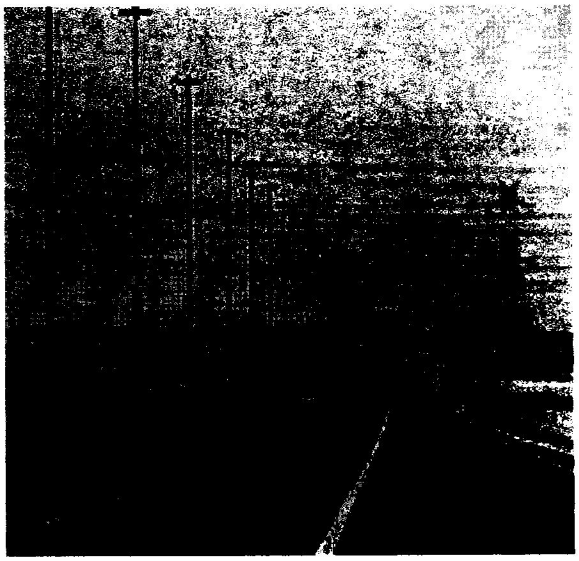


Plate 2.5.3: A highway diagram

2.5.3.1 Dual carriageway

A dual carriageway (British English) or divided highway (American English) is a class of highway with carriageways for traffic travelling in opposite directions separated by a central reservation. Roads with two or more carriageways which are designed to higher standards with controlled access are generally classed as motorways, freeways, etc., rather than dual carriageways. A road without a central reservation is a single carriageway regardless of the number of lanes. Dual carriageways have improved road traffic safety over single carriageways and typically have higher speed limits as a result. In some places, express lanes and local/collector lanes are used within a localexpress-lane system to provide more capacity and to smooth traffic flows for longer-distance travel.

Plate 2.5.3.1: A dual carriage way

2.5.3.2 Expressway

A limited-access road, known by various terms worldwide, including limited-access highway, dual carriageway, expressway, and partial controlled access highway, is a highway or arterial road for high-speed traffic which has many or most characteristics of a controlled-access highway (freeway or motorway), including limited or no access to adjacent property, some degree of separation of opposing traffic flow, use of grade separated interchanges to some extent, prohibition of some modes of transport such as bicycles or horses, and very few or no intersecting cross-streets. The degree of isolation from local traffic allowed varies between countries and regions. The precise definition of these terms varies by jurisdiction.

Plate 2.5.3.2: An expressway

2.6 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.

2.7 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)

2.7.1 Sub-grade

In transport engineering, subgrade is the active material underneath a constructed road, it is also called formation level. They are commonly compacted before the construction of a road, pavement or rail track, and are sometimes stabilized by the addition of asphalt, lime, Portland cement or other modifiers. 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;

1. Proper compaction
2. stabilization
3. Proper drainage

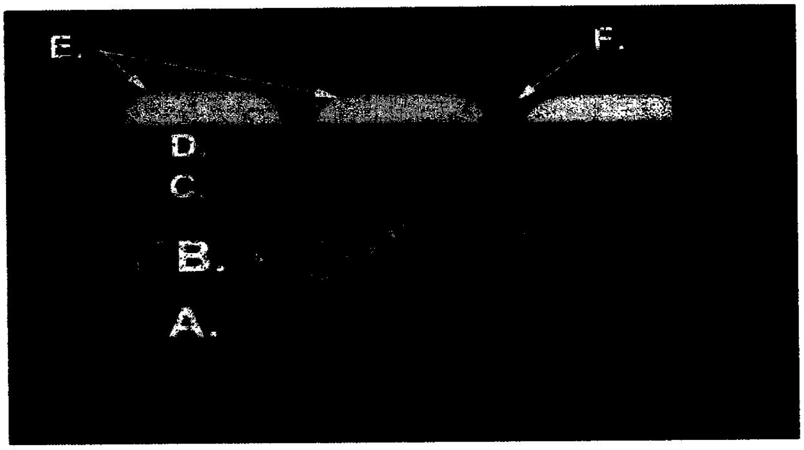


Figure 2.7.1: Diagram showing the subgrade

2.7.2 Sub-base

In highway engineering, sub base is the layer of aggregate materials laid on the subgrade, on which the base course layer is located. It may be omitted when there will be only foot traffic on the pavement, but it is necessary for surfaces used by vehicles. Its role is to spread the load evenly over the subgrade. The materials used maybe either unbound granular, or cement bound. 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).

2.7.3 Base course

The layer immediately beneath the surface course. It provides additional load distribution and contributes to drainage. Base courses are usually out of crushed aggregate or HMA. It may be composed of crushed stones, slags and other untreated or stabilized materials. Shahin et al. (1984).

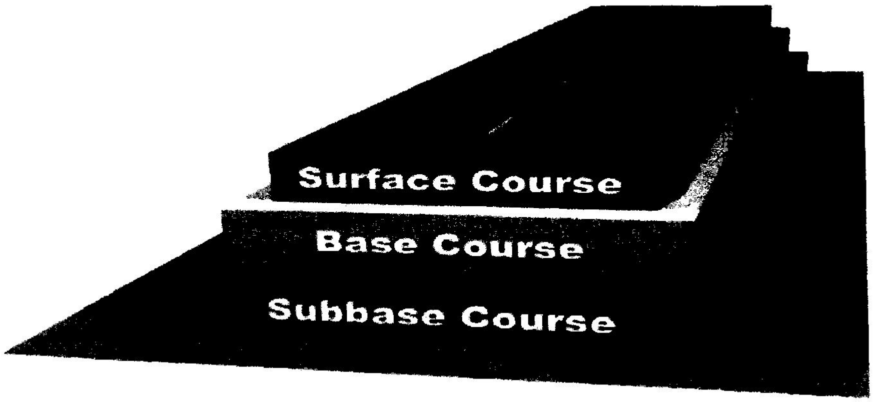


Figure 2.10.3: Diagram showing various layers of a flexible pavement

2.7.4 Surfacing (wearing course)

The layer in contact with traffic loads. It provides characteristics such as friction, smoothness, noise control, rut resistance and drainage. In addition, it prevents entrance of surface water into the underlying base, sub base and subgrade (NAPA, 2001). This top structural layer of material is sometimes subdivided into two layers: the wearing course (top) and binder course (bottom). Surface courses are most often constructed out of HMA. 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)

2.7.6 Types of road pavement

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

i. Flexible pavement
ii. Rigid pavement

2.8 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 nonhomogenous 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.

2.9 Soil

(Das braja)Soils are aggregates of mineral particles, and together with air and/or water in the void spaces, they form three-phase systems. A large portion of the earth's surface is covered by soils, and they are widely used as construction and foundation materials. Soil mechanics is the branch of engineering that deals with the engineering properties of soils and their behavior under stress. (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. The terms 'clay', 'silt' etc. are used to describe only the sizes of particles between specified limits. However, the same terms are also used to describe particular types of soil, classified according to their mechanical behavior.

2.10 Laterites and Lateritic Soils

In this section, a brief literature review on laterites and lateritic soils will be presented. The information which will be compiled here was selected to be directly related to the scope of this particular study. More detailed information obtained from the literature on these soils can be found in Paulson's (49) and Fish's (23) theses.

2.10.1 Definition of genesis of laterites and lateritic soils

The recognition of laterite as an earth material, with unique properties, dates back to 1807 when Buchanan first encountered a material in India which he called laterite and defined as "soft enough to be readily cut into blocks by an iron instrument, but which upon exposure to air quickly becomes as hard as brick, and is reasonably resistant to the action of air and water". Since Buchanan's time; the word laterite has been used to describe a wide variety of tropical soils without reaching an agreement on the exact origin, composition and properties of laterites. If one attempts to find the definition of laterite by searching the literature, he will encounter several different definitions. Among them, the one of Alexander and Cady is widely accepted; "Laterite is a highly weathered material rich in secondary oxides of iron, aluminum, or both. It is nearly void of bases and primary silicates, but it may contain large amounts of quartz and kaolinite. It is either hard or capable of hardening on exposure to wetting and drying." Among those characteristics listed in the above definition, hardness is the only one which makes laterite unique. Later on, Lohnes and Demirel used the same definition in their studies on tropical soils, with the slight modification that "hardness means there is sufficient induration of the soil that it cannot be readily excavated by a shovel or spade."

There are certain tropical soils which have not weathered as severely as laterites, but still have high sesquioxide and kaolinite contents, and low base and primary silicate contents, however, they are neither hard nor capable of hardening. According to Lohnes and Demirel, such soils can be referred to as lateritic soils. In this thesis, the terms laterite and lateritic soils will be used in accordance with the definitions made by Lohnes and Demirel. A complete accumulation of information on laterites prior to 1966 can be found in Maignien's UNESCO report, "Review of Research on Laterites," in which he condensed the information contained in more than 2000 bibliographical references.There are numerous hypotheses on the genesis of laterites, differing from each other one way or another. The following points, however, remain common in most of them. The weathering process involves leaching of silica, formation of colloidal sesquioxides, and precipitation of the oxides with increasing crystallinity and dehydration as the rock becomes more weathered. The parent rock which contains primary feldspars, quartz, and ferromagnesian minerals is transformed to a porous clayey system containing kaolinite, sesquioxides, and some residual quartz. The primary feldspars are converted to kaolinite and then, kaolinite is transformed to gibbsite. Primary ferromagnesian minerals, on the other hand, are eventually converted to diffuse goethite, followed by well crystallized goethite, and finally hematite. The crystallization leads to the formation of iron and/or aluminum oxide concretions, coalescence of concretions and their cementation by iron and/or aluminum colloids, until the entire system is a continuous iron and/or aluminum oxide cemented crust. The weathering process of soils is very much dependent on the environmental conditions in which soils are occurring. There are five major factors influencing the formation of soils and they are: parent material, climate, topography, vegetation, and time. From this point of view, the tropical regions with high temperature and humid conditions provide a favorable environment for intense weathering.

It is quite difficult to differentiate which factors have more influence on weathering than the others. One may intuitively say that the properties of a soil, which is the product of weathering, should be directly dependent on the properties and features of the parent material from which the soil is derived. The influence of climate on weathering, on the other hand, is a commonly accepted fact. In discussing the factors influencing soil weathering, Mohr and Van Baren (1945) put considerable emphasis on the rainfall distribution. In his study on Hawaiian soils, Sherman (1954) observed the influence of rainfall intensity on the mineral composition of soils. Although many investigators reported certain relationships between rainfall and mineral composition, there are some discrepancies. Dean (1918) and Tanada (1965), for example, observed that in Hawaiian soils, high kaolinite content occurs in regions receiving small amount of rainfall (65-90 cm annually), and that the kaolinite content decreases with increasing rainfall. Some other publications on Hawaiian soils, on the other hand, present cases in which kaolinite does not exist at all under a rainfall of 90 cm per year; instead, high amounts of sesquioxides of secondary origin occur. This difference in occurrence is attributed to the age of the soils the influence of topography on weathering, also, cannot be overlooked. Since topography has influence on ground and surface water movement, it has a direct influence on the development of soil profile. Mohr and Van Baren (1945) point out that the same type of rock may yield a weathering product of completely different composition if different topography and accordingly different drainage processes are involved. Vegetation is another important factor in the formation of laterites. As Sherman et al. (1959) point out dehydration of the colloidal hydrated oxides of the soil has a very important role in the development of laterite; and vegetation is one of the factors which determine the rate of dehydration. Vegetation also has a protective effect on runoff and erosion which are important environmental factors influencing the genesis of soils. Finally, the role of time in the formation of laterites and lateritic soils, is a well-recognized point introduced by many investigators Sherman (1954) states that "since the geological ages of the parent materials vary greatly, the time of exposure of the parent material to soil forming processes will also have had a major effect on soil development." As a result of the preceding discussions, it is concluded that because of the combined effects of those several soil forming factors, laterites and lateritic soils exhibit a very complex pattern of soil development.

2.10.2 Engineering properties of laterites and lateritic soils

Many studies have shown that plasticity and grain size distribution data for lateritic soils are extremely varied and erratic the reasons for this are discussed in detail by several investigators when soils are manipulated their characteristics vary a lot. Pre-testing drying cause's variations in some properties of lateritic soils and this behavior is commonly attributed to the dehydration of the colloidal hydrated oxides occurring in these soils. In most of the cases the variation, resulting from drying, is irreversible and results in a soil with more granular characteristics. To disperse such a system for plasticity and grain size determinations is almost impossible (1974). Because of such difficulties it is extremely difficult to derive an acceptable generalization for lateritic soils with regard to plasticity and gradation. Lohnes and Demirel (1937) are the first investigators who have put emphasis on using specific gravity as an indicator for engineering behavior of lateritic soils. By definition specific gravity is the weighted average of the specific gravities of the minerals which comprise the soil. In the weathering process of lateritic soils, it is always stated that the contents of high specific gravity minerals increase with age of formation. This fact, of course, should be reflected in the value of specific gravity, that is, specific gravity of lateritic soils should increase with increasing degree of weathering. Lohnes and Demirel made an attempt to verify this thought by plotting extractable iron content versus specific gravity for several selected Puerto Rican soils and ended up with a good correlation between increasing specific gravity and increasing iron content. They also used the data presented by Trow and Morton (1970) on Dominican Republic soils to show increasing specific gravity with increasing amount of goethite. Thus it appears that specific gravity of lateritic soils can be regarded as a parameter which can be used for a better understanding of the engineering behavior of tropical soils in relation to degree of weathering. Other engineering properties of lateritic soils, such as wet and dry densities, moisture content, and void ratio (or porosity), have not been taken into account in the majority of studies. There are very limited data on such properties of lateritic soils in the literature. This is an unfortunate situation, because these properties have an advantage over plasticity and gradation, in that, the majority of them are determined by bulk measurements and as such are not influenced by degree of manipulation. Specific gravity, which is used in determining the void ratio, is also a parameter not affected much by the manipulation of soils prior to testing. In addition, the bulk properties, reflect the behavior of undisturbed soils; so, from the engineering point of view, they provide better information on laterites and lateritic soils. In their study on Puerto Rican soils, Lohnes and Demirel (1937) observed a relationship between void ratio and specific gravity, indicating a decrease in void ratio as specific gravity increases. Besides that, they observed increasing cohesion with decreasing void ratio. By making use of these relationships, they suggested the possibility of an engineering classification system for lateritic soils which relates void ratio, strength and degree of weathering to each other.

2.10.3 Structure of laterites and lateritic soils

The size, shape, and arrangement of mineral grains which form the soil mass is known as soil structure. The importance of soil structure in explaining the engineering behavior of soils Is emphasized by many investigators In this section of the literature review the soil properties which have been inferred as having direct relationships with the soil structure will be introduced.

2.10.4 Soil mineralogy

Soil minerals occurring in the soil mass apparently have direct influence on the size, shape and arrangement of the soil aggregates. According to many investigators the predominant minerals occurring in lateritic soils are kaolinite, gibbsite, arid Iron It has also been inferred that the occurrence of kaolinite in large amounts comes first in the course of weathering. With continued weathering the kaolinite content decreases, while the sesquioxides of iron and aluminum become larger in amount Peterson (1950) made an attempt to measure the capacity of kaolinite to form water stable aggregates under the influence of cyclic wetting and drying, and found out that kaolinite is very inert as a binding agent which has very little effect on aggregation. Oxides of iron and aluminum, on the other hand, are reported as being very active as binding agents by many investigators Area and Weed point out that the relationship between aggregate occurrence and free iron oxide content remains highly significant and fairly constant at all sizes studied (0.1-2.0 mm diameter). These observations suggest that the soils with high kaolinite content would show low aggregation, and soils with large amount of sesquioxides would exhibit good soil aggregation. In his study on some residual soils from the highlands of Papua, New Guinea, Wallace (1975) made an attempt to idealize the structure of lateritic soils. According to him, soil aggregates are cemented together at their contacts to form a continuous three-dimensional structural framework and the precipitation of iron and aluminum hydroxides is responsible for the cementation. This is verified somewhat by the scanning electron microscope photos of Lohnes and Demirel (1937). As it is stated by many investigators, the ultimate end product of laterization in tropical soils could be either iron oxide rich laterite or aluminum oxide rich laterite. According to Sherman (1954) the end product of weathering is closely related to the distribution of rainfall. He states that an alternating wet and dry season climate results in the stabilization of the iron oxide. Under continuously wet conditions; however, the alumina becomes the stabilized free oxide, while iron oxide becomes unstable and leaches away. This explains the difference in formation of so called ferruginous and aluminous, or bauxite, laterites. Titanium oxide, in the form of anatase, is occasionally encountered in tropical soils. Sherman (1956) studied the titanium content of Hawaiian soils and discussed its significance in the weathering process of soils. The data presented by him suggest that the occurrence of titanium element usually takes place in the surface of soils, more specifically in A-horizon, under a climate which has definite wet and dry seasons.

2.10.5 Soil pore structure

Besides studying the size, shape and arrangement of solid phase in a soil mass, it should be worthwhile to study the pore phase of soils as well. Mercury Injection technique is a method recently developed, and can be used for analyzing several aspects of porous materials. Diamond (1920) introduced the method first for studying the pore size distribution of soils. After him, several pore size studies have been performed on temperate soils, but not on tropical soils. It is the belief of the author that such a useful tool should be utilized in studying the pore structure of lateritic soils in order to be able to understand the behavior of these soils better, with regard to soil weathering and soil strength. The most common methods utilized in studying the structure of lateritic soils have been light microscopy and recently, scanning electron microscopy. It is extremely difficult to define the soil structure quantitatively from the micrographs obtained from microscopic studies. The mercury porosimetry, however, gives the opportunity to generate several parameters from the pore size distribution curves and to quantify, at least, the pore structure of soils.

2.10.6 Strength of laterites and lateritic soils

A literature review on the strength characteristics of undisturbed samples of laterites and lateritic soils reveals that the investigation of these soils from an engineering point of view has been greatly overlooked, although such soils have been used as a primary engineering construction material in tropical and equatorial countries for many years. Some studies performed recently on the strength behavior of laterites and lateritic soils have added little because either they are incomplete or they deal with localized problems and in restricted areas. The available reported test results can be summarized in the following manner. Lateritic soils usually have relatively high to very high cohesion and internal friction angle. This behavior is generally attributed to the cementation which is taking place among the individual soil grains by the binding action of sesquioxides. The strength behavior of lateritic soils was observed to be very much dependent on the moisture content of the sample tested. Decreasing moisture content usually causes an increase both in cohesion and internal friction angle. This behavior is attributed to the variation in soil structure with varying moisture content, such that, as the soil dries out part of the hydrated colloidal iron and aluminum oxides dehydrates and forms strong bonds among certain soil grains which, in turn, causes an increase in strength. In their study on Puerto Rican soils, Lohnes and Demirel (1937) made an attempt to relate cohesion to degree of weathering and they observed that cohesion increases with increasing weathering.

2.10.7 Classification of laterites and lateritic soils

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

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

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

As Mchr and Van Baren (1945) point out every classification system should have some predetermined purposes. None of the classification systems mentioned above has an aim to classify the soils according to their engineering behavior. Although there are some popular engineering classification systems, such as the Unified system or the American Association of State Highway Officials system, which have been used satisfactorily in the temperate environments of the world for years, they have not been so successful in the case of tropical soils. These classification systems are based on plasticity and gradation data of the soils; but as discussed previously, such characteristics of tropical soils are not reproducible by standard laboratory tests. The reasons for this, once again, are the influences of sample preparation and handling which disrupt the natural structure of the soil. In order to avoid such problems, several authors have advocated a classification of laterites and lateritic soils for engineering purposes, based on parent material and degree of weathering. Fish (1923) and Gidigasu (1926) made attempts to use pedalogical classifications for engineering purposes. Ruddock (1953) has suggested an engineering classification based on topographic position, sample depth and depth to water table which are, in fact, factors influencing the degree of weathering. Lohnes and Demirel (1937) have suggested to use specific gravity, void ratio and degree of weathering for engineering classification of tropical soils. None of these proposed engineering classification systems, however, has found a broad acceptance yet. From the above discussion, it becomes evident that an appropriate classification of laterites and lateritic soils for engineering purposes is still nonexistent.

2.11 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, geomorphologies 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. It has been observed that problem-soils poses a serious threat to civil engineering projects which results to defect or collapse of infrastructures such as roads, buildings, dams among others, Kekere et al. (2012)

2.11.1 Specific gravity

Specific gravity is the ratio of the mass of soil solids to the mass of an equal volume of water. It is an important index property of soils that is closely linked with mineralogy or chemical composition and also reflects the history of weathering. It is relatively important as far as the qualitative behavior of the soil is concerned and useful in soil mineral classification, for example iron minerals have a larger value of specific gravity than silicas. It gives an idea about suitability of the soil as a construction material; higher value of specific gravity gives more strength for roads and foundations. It is also used in calculation of void ratio, porosity, degree of saturation and other soil parameters. Typical values of specific gravity are given in Table 2.11.1.

Table 2.11.1: Typical values of specific gravity (Bowles, 2012)

Type of soil	Specific gravity
Sand	2.65-2.67
Silty sand	2.67-2.70
Inorganic clay	2.70-2.80
Soil with mica or iron	2.75-3.00
Organic soil	1.00-2.60

2.11.2 Compaction

Compaction is the process of densification of soil by the application of mechanical energy. Generally, compaction is done at specific moisture content to achieve maximum densification of soil. Compaction condition can be determined by Standard Proctor and Reduced Proctor Test. However, different indirect approaches were initiated to observe compaction condition. Several researchers utilized electrical resistivity to evaluate compaction condition. Compaction is associated with the decrease of void ratio and increase of degree of saturation. Good correlations between electrical resistivity and compaction condition were observed in several studies. A laboratory scale test was performed by Rinaldi and Cuestas (2002) to evaluate relationship between electrical conductivity and compaction. Samples were sieved through No 40 sieve and compacted at 18% moisture content. Compaction was conducted using Standard Proctor method in a rectangular mixing pan. After compaction, conductivity was measured using four probe electrode device. Iso-conductivity contour obtained from the test is presented in Figure 2.14.2.

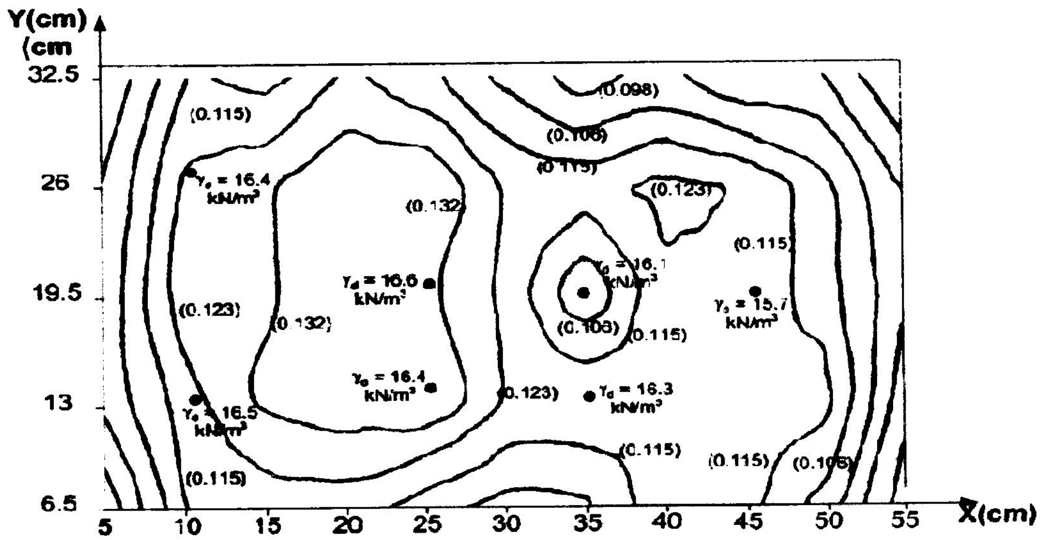


Figure 2.11.2: Iso conductivity contour of compacted sample, parentheses showed electrical conductivity in mho/m (Rinaldi and Cuestas, 2002)

From the **Figure 2.11.2**, it is depicted that conductivity at central portion is greater than right hand side and border. Author indicated that variation of conductivity was attributed due to the variation of soil unit weight. Unit weight was higher at left hand side and decreased at right hand side and border due to the low stiffness of the wall of the mixing pan. McCarter (1984) conducted a study on the effect of air void ratio in soil resistivity. He indicated that reduction in air void ratio in soil structure had significant effect on soil resistivity. Tests were conducted on Cheshire and London clay. With the increase of degree of compaction or degree of saturation decrease of soil resistivity was observed for both samples.

Author concluded that only moisture content could not be a criterion in resistivity measurement. Compaction condition also played an important role in resistivity. Abu Hassanein et al., (1996) performed a comprehensive study on the effect of molding water content and compactive effort in soil resistivity. Samples were compacted at three different compaction methods: Standard, Modified and Reduced Proctor. Observed test results on the study are presented in Figure 2.11.3.

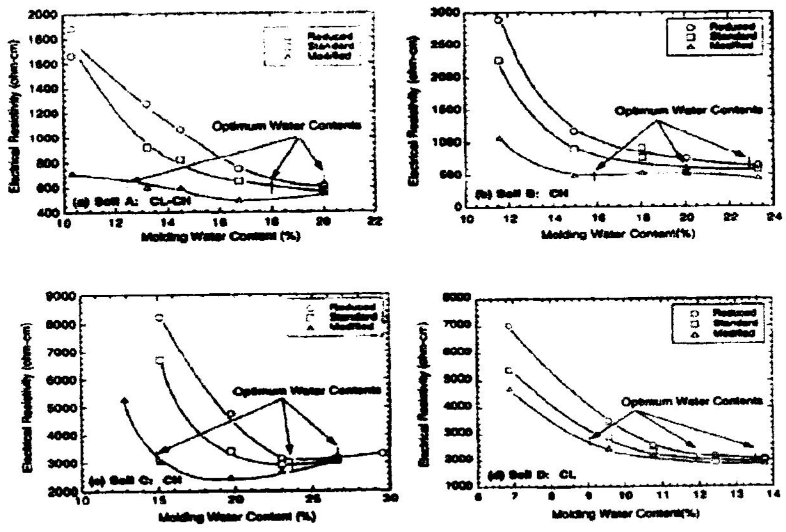


Figure 2.11.3: Relationship among electrical resistivity, molding water content and compactive effort for different soils (Abu Hassanein Et Al., 1996)

Observed resistivity was high when soil was compacted at dry optimum and low when compacted at wet optimum. Resistivity was sensitive of molding water content when water content was below optimum. At wet optimum, resistivity had become almost independent of molding water content. Authors indicated that this relation could be used to evaluate compaction condition of soil. Relationship between resistivity and compaction was discussed in the light of structural change of soil during compaction. At low compactive effort and dry of optimum water content, clay clods are difficult to remold. Interclod pores are also relatively large in this condition. Many pores are filled with dielectric air and inter particle contacts are poor. Furthermore diffuse double layers are not fully developed. Therefore, soil shows high resistivity. In contrast, when soil is compacted at wet optimum and high compactive effort, clods of clay are easily remolded. At this condition, pores are nearly saturated and smaller in size compare to previous case. Better particle-to-particle contact and formation of bridge between particles improve conductivity. Thus, lower resistivity is attained when compacted at wet optimum water content and high compactive effort (Abu Hassanein et al., 1996). Moreover, study showed that change in compactive effort did not affect resistivity significantly when compacted at wet optimum.

2.11.3 Density index

The degree of compaction of fine grained soils is measured in relation to maximum dry density for a certain compactive effort, like 90% of light compaction density or proctor density. But in case of coarse grained soils, a different sort of index is used for compaction. Depending upon the shape, size, and gradation of soil grains, coarse grained soils can remain in two extreme states of compaction, namely in the loosest and densest states. Any intermediate state of compaction can be compared to these two extreme states using an index called relative density or density index. The soil characteristics based on relative density are shown in Table 2.11.3

Table 2.11.3: Characteristics of soils based on relative density

Relative density (%)	Soil compactness	Angle of shearing resistance (°)
0-15	Very loose	<28
15-35	Loose	28-30
35-65	Medium	30-36
65-85	Dense	36-41
85-100	Very dense	>41

Density index is expressed in percent and is defined as the ratio of the difference between the void ratio of a cohesionless soil in the loosest state and any given void ratio to the difference between its void ratios in the loosest and the densest states. It is a measure of the degree of compactness, and the stability of a stratum.

As per Apparao and Rao, relative density is an arbitrary character of sandy deposit. In real sense, it expresses the ratio of actual decrease in volume of voids in a sandy soil to the maximum possible decrease in volume of voids i.e. how far the sand under investigation can capable to the further densification beyond its natural state. Its determination is helpful in compaction of coarse grained soils and in evaluating safe bearing capacity of sandy soils.

2.11.4 Consolidation

Consolidation means dissipation of excess pore water pressure with time. The consequence of consolidation is settlement. With the settlement, pore water is dissipated and contact between soil particles increases. It was observed that reduction of water in consolidation affects soil resistivity. Various studies were conducted to determine the effect of consolidation in soil resistivity. McCarter and Desmazes (1997) investigated changes in electrical conductivity of clay soil in response to consolidation. Modified consolidation cell was utilized to conduct tests on soil samples having moisture content of 71%. Test results are presented in Figure 2.11.4.

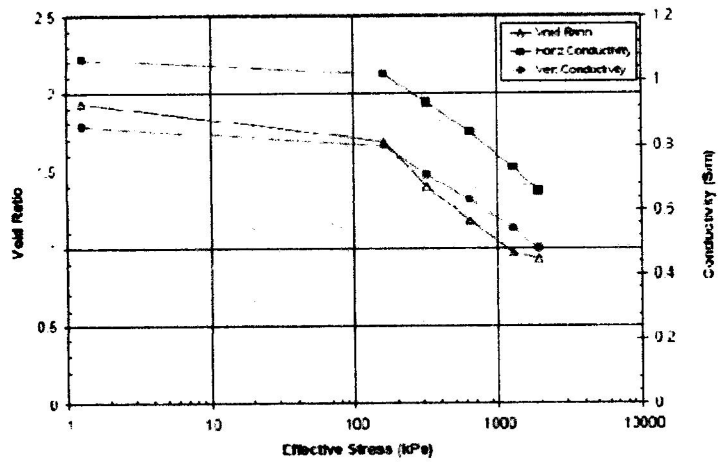


Figure 2.11.4: Relationship of conductivity with consolidation (Mccarter And Desmazes, 1997).

It indicates that the changes of void ratio and conductivity with effective stress are very similar. According to the authors, conduction in saturated soil occurred through continuous interstitial water. Thus, fractional volume of water and composition of pore fluid influenced electrical properties significantly. Conductivity of soil decreased with consolidation process due to the dissipation of pore water. Bryson (2005) linked void ratio with conductivity from the curve obtained by McCarter and Desmazes (1997). Volumetric strain in one dimensional consolidation occurs due to the vertical strain. Reduction in sample height is associated with the change in void space in vertical direction. Thus, change in the vertical conductivity is related to the change in void space in vertical direction. According to the Figure 2.23, vertical conductivity is a function of void ratio.

2.11.5 Consistency limits

The consistency of a fine-grained soil is largely influenced by the water content of the soil. A gradual decrease in water content of a fine-grained soil slurry causes the soil to pass from the liquid state to a plastic state, from the plastic state to a semi-solid state, and finally to the solid state. The water contents at these changes of state are different for different soils. The water contents that correspond to these changes of state are called the Atterberg limits. The water contents corresponding to transition from one state to the next are known as the liquid limit, the plastic limit and the shrinkage limit. The liquid limit of a soil is the water content, expressed as percentage of the weight of the oven dried soil, at the boundary between the liquid and plastic states of consistency of the soil. The soil has negligibly small shear strength. The plastic limit of a soil is the water content, expressed as a percentage of the weight of oven dried soil, at the boundary between the plastic and semi-solid states of consistency of the soil. The plastic limit for different soils has a narrow range of numerical values. Sand has no plastic stage, but very fine sand exhibits slight plasticity. The plastic limit is an important soil property. Earth roads are easily usable at this water content. Excavation work and agricultural cultivation can be carried out with the least effort with soils at the plastic limit. Soil is said to be in the plastic range when it possesses water content in between liquid limit and plastic limit. The range of the plastic state is given by the difference between liquid limit and plastic limit and is defined as the plasticity index. The plasticity index is used in soil classification and in various correlations with other soil properties as a basic soil characteristic. Based on the plasticity index, the soils were classified by Atterberg, shows the correlations between the plasticity index, soil type, degree of plasticity and degree of cohesiveness (Table 2.11.5).

Table 2.11.5: Types of soils based on plasticity index

Plasticity index (%)	Soil type	Degree of plasticity	Degree of cohesiveness
0	Sand	Non-plastic	Non-cohesive
<7	Silt	Low plastic	Partly cohesive
7-17	Silt clay	Medium plastic	Cohesive
>17	Clay	High plastic	cohesive

Skempton observed that the plasticity index of a soil increases linearly with the percentage of the claysized fraction. Laskar and Pal found that plasticity depends on grain size of soil. With the increase of sand content plasticity index of soil decreases, which might be due to decrease of inter molecular attraction force. Due to decrease of attraction force, liquid limit of the soil decreases and accordingly plasticity index decreases. But as the clay content increases inter molecular attraction force increases and liquid limit increases. The shrinkage limit is the maximum water content expressed as a percentage of oven-dried weight at which any further reduction in water content will not cause a decrease in volume of the soil mass, the soil mass being prepared initially from remolded soil. The finer the particles of the soil, the greater are the amount of shrinkage. Soils that contain montmorillonite clay mineral shrink more. Such soils shrink heterogeneously during summer, as a result of which cracks develop on the surface. Further, these soils imbibe more and more water during the monsoon and swell. Soils that shrink and swell are categorized as expansive soils. Indian black cotton soils belong to this group. According to Prakash and Jain, the value of shrinkage limit is used for understanding the swelling and shrinkage properties of cohesive soils. It is used for calculating the shrinkage factors which helps in the design problems of the structures made of the soils or/and resting on soil. It gives an idea about the suitability of the soil as a construction material in foundations, roads, embankments and dams. It helps in knowing the state of given soil. As per Ersoy et al., consistency is an important property and is a useful measure for the processing of very fine clayey soils. Plasticity and cohesion reflect the soil consistency and workability of the soils. However, these properties of the soils play an essential role in many engineering projects, such as the construction of the clay core in an earth fill dam, the construction of a layer of low permeability covering a deposit of polluted material, the design of foundations, retaining walls and slab bridges, and determining the stability of the soil on a slope. Agbede et al. conducted the study at University of Ibadan (UI), Nigeria. The building under study was two storey with basement complex and housed offices, classrooms, a laboratory, library and a computer room. This building is located in a flat, low terrain with an upper layer of loose lateritic clayey soils while the underlying soil is sandy soil mixed with silty clay material. The cracks were observed due to expansive soil supporting the foundation of the building. The soil foundation contains high amount of clay with high plasticity index.

2.11.6 Shear strength

The shear resistance of soil is the result of friction and the interlocking of particles and possibly cementation or bonding at the particle contacts. The shear strength parameters of soils are defined as cohesion and the friction angle. The shear strength of soil depends on the effective stress, drainage conditions, density of the particles, rate of strain, and direction of the strain. Thus, the shearing strength is affected by the consistency of the materials, mineralogy, and grain size distribution, shape of the particles, initial void ratio and features such as layers, joints, fissures and cementation. The shear strength parameters of a granular soil are directly correlated to the maximum particle size, the coefficient of uniformity, the density, the applied normal stress, and the gravel and fines content of the sample. It can be said that the shear strength parameters are a result of the frictional forces of the particles, as they slide and interlock during shearing. Soil containing particles with high angularity tend to resist displacement and hence possess higher shearing strength compared to those with less angular particles. Different researchers explained that the capability of a soil to support a loading from a structure, or to support its overburden, or to sustain a slope in equilibrium is governed by its shear strength. The shear strength of a soil is of prime importance for foundation design, earth and rock fill dam design, highway and airfield design, stability of slopes and cuts, and lateral earth pressure problems. It is highly complex because of various factors involved in it such as the heterogeneous nature of the soil, the water table location, the drainage facility, the type and nature of construction, the stress history, time, chemical action, or environmental conditions. As per Prakash and Jain, confining pressures play the significant role in changing the behavior of soils in deep foundations. Similarly in high rise earth dams, the confining pressures are of very high magnitude. Triaxial test is the only test to simulate these confining pressures. For short term stability of foundations, dams and slopes, shear strength parameters for unconsolidated undrained or consolidated undrained conditions are used, while for long term stability shear parameters corresponding to consolidated drained conditions give more reliable results. Akayuli et al. found that the friction angle is high for a sandy soil than its cohesion and vice versa for clayey soil. Shanyoug et al. In their study concluded that there is a general increase in cohesion with clay content. As more clay is introduced into the sandy materials, the clay particles fill the void spaces in between the sand particles and begin to induce the sand with interlocking behavior.

Hence, clayey sand soils are expected to exhibit low cohesion whereas the cohesion increases with high clay content. Dafalla observed that the mineralogy can have a major role in the shearing strength capacity of clays. The cementation between particles can either be due to a chemical bond or physicochemical bond. Swelling and shrinkage in expansive soils are of two extreme opposite effects on the shearing strength. The shear strength is generally low for fully expanded clay while dry shrinking clay is capable of developing higher cohesion and angle of internal friction. The study indicated that choosing the appropriate mix or using appropriate quantity of clay, can help to achieve required shear strength. Very moist clay-sand mixture showed steep drop in both cohesion and angle of internal friction when the clay content is high. According to Murthy and El-Maksoud, cohesion is mainly due to the intermolecular bond between the adsorbed water surrounding each grain, especially in fine-grained soils. As per Mollahasani et al., the soils with high plasticity like clayey soils have higher cohesion and lower angle of shearing resistance. Conversely, as the soil grain size increases like sands, the soil cohesion decreases. During two case studies of embankment dams in Iran, Karmi et al. found that for large dams, internal friction angle has more critical role in stability analysis than cohesion parameter. Soil shear strength are used to determine the load on soil. Some of these tests are;

i. Direct shear test
ii. Triaxial shear test
iii. Vane shear test
iv. Unconfined compression test

2.11.6.1 Direct shear test

A direct shear test is a laboratory or field test used by geotechnical engineers to measure the shear strength properties of soil or rock material, or of discontinuities in soil or rock masses. The U.S. and U.K. standards defining how the test should be performed are ASTM D 3080, AASHTO T236 and BS 1377-7:1990, respectively. For rock the test is generally restricted to rock with (very) low shear strength. The test is, however, standard practice to establish the shear strength properties of discontinuities in rock. The test is performed on three or four specimens from a relatively undisturbed soil sample. A specimen is placed in a shear box which has two stacked rings to hold the sample; the contact between the two rings is at approximately the mid-height of the sample. A confining stress is applied vertically to the specimen, and the upper ring is pulled laterally until the sample fails, or through a specified strain. The load applied and the strain induced is recorded at frequent intervals to determine a stress– strain curve for each confining stress. Several specimens are tested at varying confining stresses to determine the shear strength parameters, the soil cohesion (c) and the angle of internal friction, commonly known as friction angle. The results of the tests on each specimen are plotted on a graph with the peak (or residual) stress on the y-axis and the confining stress on the x-axis. The y-intercept of the curve which fits the test results is the cohesion, and the slope of the line or curve is the friction angle. Direct shear tests can be performed under several conditions. The sample is normally saturated before the test is run, but can be run at the in-situ moisture content. The rate of strain can be varied to create a test of undrained or drained conditions, depending whether the strain is applied slowly enough for water in the sample to prevent pore-water pressure buildup. Direct shear test machine is required to perform the test. The test using the direct shear machine determinates the consolidated drained shear strength of a soil material in direct shear. The advantages of the direct shear test over other shear tests are the simplicity of setup and equipment used, and the ability to test under differing saturation, drainage, and consolidation conditions. These advantages have to be weighed against the difficulty of measuring pore-water pressure when testing in undrained conditions, and possible spuriously high results from forcing the failure plane to occur in a specific location

2.11.6.2 Triaxial test

A triaxial shear test is a common method to measure the mechanical properties of many deformable solids, especially soil (e.g., sand, clay) and rock, and other granular materials or powders. There are several variations on the test. In a triaxial shear test, stress is applied to a sample of the material being tested in a way which results in stresses along one axis being different from the stresses in perpendicular directions. This is typically achieved by placing the sample between two parallel platens which apply stress in one (usually vertical) direction, and applying fluid pressure to the specimen to apply stress in the perpendicular directions. (Testing apparatus which allows application of different levels of stress in each of three orthogonal directions are discussed below, under "True Triaxial test".) The application of different compressive stresses in the test apparatus causes shear stress to develop in the sample; the loads can be increased and deflections monitored until failure of the sample. During the test, the surrounding fluid is pressurized, and the stress on the platens is increased until the material in the cylinder fails and forms sliding regions within itself, known as shear bands. The geometry of the shearing in a triaxial test typically causes the sample to become shorter while bulging out along the sides. The stress on the platen is then reduced and the water pressure pushes the sides back in, causing the sample to grow taller again. This cycle is usually repeated several times while collecting stress and strain data about the sample. During the test the pore pressures of fluids (e.g., water, oil) or gasses in the sample may be measured using Bishop's pore pressure apparatus. From the triaxial test data, it is possible to extract fundamental material parameters about the sample, including its angle of shearing resistance, apparent cohesion, and dilatancy angle. These parameters are then used in computer models to predict how the material will behave in a largerscale engineering application. An example would be to predict the stability of the soil on a slope, whether the slope will collapse or whether the soil will support the shear stresses of the slope and remain in place. Triaxial tests are used along with other tests to make such engineering predictions. During the shearing, a granular material will typically have a net gain or loss of volume. If it had originally been in a dense state, then it typically gains volume, a characteristic known as Reynolds' dilatancy. If it had originally been in a very loose state, then contraction may occur before the shearing begins or in conjunction with the shearing. Sometimes, testing of cohesive samples is done with no confining pressure, in an unconfined compression test. This requires much simpler and less expensive apparatus and sample preparation, though the applicability is limited to samples that the sides won't crumble when exposed, and the confining stress being lower than the in-situ stress gives results which may be overly conservative. The compression test performed for concrete strength testing is essentially the same test, on apparatus designed for the larger samples and higher loads typical of concrete testing.

2.11.6.3 Vane shear test

The test can be conducted either from the ground surface or from the bottom of a borehole or a test pit. If conducted from the bottom of a bore-hole, the test area should be should be at the depth of least three times the borehole diameter lower that the borehole bottom in order to avoid the borehole disturbance effects. The test starts by pushing the vane and the rod vertically into the soft soil. The vane is then rotated at a slow rate of 6° to 12° per minute. The torque is measured at regular time intervals and the test continues until a maximum torque is reached and the vane rotates rapidly for several revolutions. At this time, the soil fails in shear on a cylindrical surface around the vane. The rotation is usually continued after shearing and the torque is measured to estimate the remoulded shear strength.

2.11.6.4 Unconfined compression test

The primary purpose of the Unconfined Compression Test is to quickly determine a measure of the unconfined compressive strength of rocks or fine-grained soils that possess sufficient cohesion to permit testing in the unconfined state. This measure is then used to calculate the unconsolidated undrained shear strength of the clay under unconfined conditions. In general, The UCT can be conducted on rock samples or on undisturbed, reconstituted or compacted cohesive soil sample. In the unconfined compression test, the sample si placed in the loading machine between the lower and upper plates. Before starting the loading, the upper plate is adjusted to be in contact with the sample and the deformation is set as zero. The test then starts by applying a constant axial strain of about 0.5 to 2% per minute. The load and deformation values are recorded as needed for obtaining a reasonably complete load-deformation curve. The loading is continued until the load values decrease or remain constant with increasing strain, or until reaching 20% (sometimes 15%) axial strain. At this state, the samples is considered to be at failure. The sample is then removed for measurement of the water content. As for the results, the axial stress is usually plotted versus the axial strain. The maximum axial stress, or the axial stress at 20% (sometimes 15%) axial strain if it occurs earlier, is reproted as the unconfined compressive strength σc. The undrained shear strength then reads

$S_u = \sigma_c / 2$

S_u: ndrained shear strength σ_c: unconfined compressive strength

2.11.7 Particle size analysis

The percentage of different sizes of soil particles coarser than 75 μ is determined by sieve analysis whereas less than 75 μ are determined by hydrometer analysis. Based on the particle size analysis, particle size distribution curves are plotted. The particle size distribution curve (gradation curve) represents the distribution of particles of different sizes in the soil mass. It gives an idea regarding the gradation of the soil i.e. it is possible to identify whether a soil is well graded or poorly graded. In mechanical soil stabilization, the main principle is to mix a few selected soils in such a proportion that a desired grain size distribution is obtained for the design mix. Hence for proportioning the selected soils, the grain size distribution of each soil is required to be known. Apparao and Rao explained that the grain size analysis is widely used in classification of soils. The data obtained from grain size distribution curves is used in the design of filters for earth dams and to determine suitability of soil for road construction, air fields, etc. Raj stated that the particle size of sands and silts has some practical value in design of filters and in the assessment of permeability, capillarity, and frost susceptibility. Very relevant and useful information may be obtained from grain size curve such as (i) the total percentage of larger or finer particles than a given size and (ii) the uniformity or the range in grain-size distribution. Bowles found that particle-size is one of the suitability criteria of soils for roads, airfield, levee, dam, and other embankment construction. Information obtained from particle-size analysis can be used to predict soil-water movement, although permeability tests are more generally used. The susceptibility to frost action in soil, an extremely important consideration in colder climates, can be predicted from the particle-size analysis. Very fine soil particles are easily carried in suspension by percolating soil water, and under drainage systems are rapidly filled with sediments unless they are properly surrounded by a filter made of appropriately graded granular materials. The proper gradation of this filter material can be predicted from the particle-size analysis. Particle-size of the filter materials must be larger than the soil being protected so that the filter pores could permit passage of water but collect the smaller soil particles from suspension. As per Dafalla, the sand shape whether rounded, subrounded, or angular will affect the shearing strength of soil. Angular grains provide more interlock and increased shear resistance. The gradation and size of the sand affect the shear resistance. Well-graded materials provide more grain to grain area contact than poorly graded materials. Porosity and spaces available for clay within the sand is an important while considering the mixtures of clays and sands.

2.11.8 Permeability

The amount, distribution, and movement of water in soil have an important role on the properties and behavior of soil. The engineer should know the principles of fluid flow, as groundwater conditions are frequently encountered on construction projects. Water pressure is always measured relative to atmospheric pressure, and water table is the level at which the pressure is atmospheric. Soil mass is divided into two zones with respect to the water table: (I) below the water table (a saturated zone with 100% degree of saturation) and (ii) just above the water table (called the capillary zone with degree of saturation ≤ 100%). Data from field permeability tests are needed in the design of various civil engineering works, such as cut-off wall design of earth dams, to ascertain the pumping capacity for dewatering excavations and to obtain aquifer constants. The permeability of soils has a decisive effect on the stability of foundations, seepage loss through embankments of reservoirs, drainage of subgrades, excavation of open cuts in water bearing sand, and rate of flow of water into wells. Prakash and Jain explained that water flowing through soil exerts considerable seepage forces, which have direct effect on the safety of hydraulic structures. The rate of settlement of compressible clay layer under load depends on its permeability. The quantity of stored water escaping through and beneath an earthen dam depends on the permeability of the embankment and the foundation respectively. The rate of drainage of water through wells and excavated foundation pits depends on the coefficient of permeability of the soils. Shear strength of soils also depends indirectly on its permeability, because dissipation of pore pressure is controlled by its permeability. According to U. S. Bureau of Reclamation, soils are classified as (I) Impervious: k (coefficient of permeability) less than 10-6 cm/sec, (ii) Semi pervious: k between 10-6 to 10-4 cm/sec (iii) Pervious: k greater than 10-4 cm/sec. The Hsinchu is located from north to south along the west coastal plain of Taiwan. Taiwan is a seismically active region and has governing seismic design criteria similar to those used in the International Building Code (IBC). At the foundation construction site, different layers were found at different depth like fill (soft, silty clay with variable amounts of sand, gravel, and organic material) clay (medium stiff to stiff, silty clay) Gravel/Cobble. The hydraulic conductivity (permeability) varied accordingly. The use of permanent drainage systems under the floor slab to draw down the groundwater table allowed the buildings to be supported on the more cost effective shallow footings and slab-on-grade floors.

2.11.9 Bearing capacity

In geotechnical engineering, bearing capacity of soil to support the loads applied to the ground. The bearing capacity of soil is the maximum average contact pressure between the foundation and the soil which should not produce shear failure in the soil. Ultimate bearing capacity divided by a factor of safety. Sometimes, on soft soils sites, large settlements may occur under loaded foundations without actual shear failure occurring; in such cases, the allowable bearing capacity is based on the maximum allowable settlement. There are three modes of failure that limit bearing capacity: shear failure, local shear failure, and punching shear failure.

2.11.10 California bearing ratio (CBR)

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 2.14.9.1. 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.

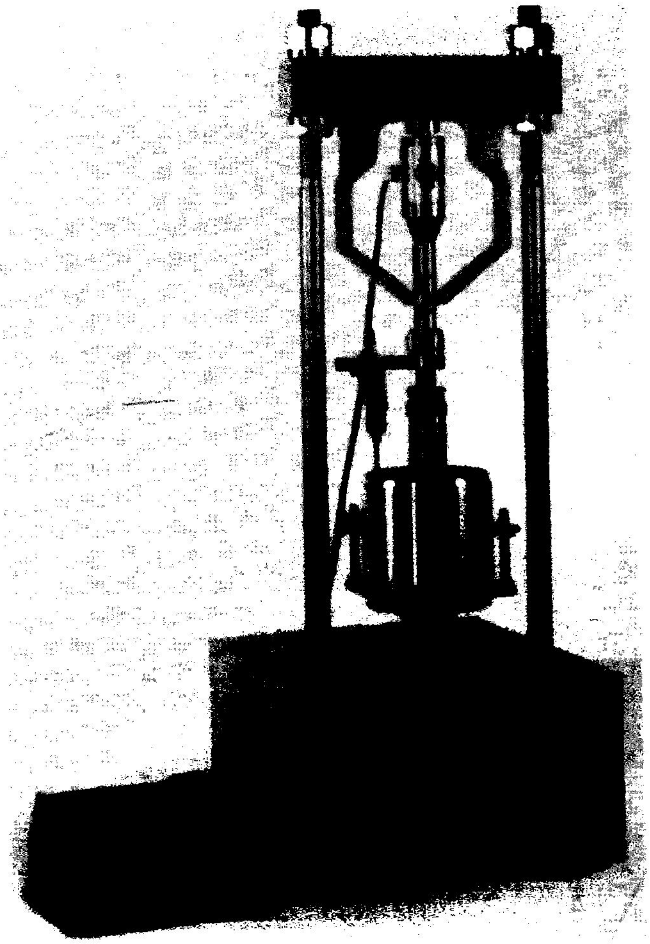


Plate 2.11.10: CBR testing equipment

2.12 Physical Properties of Soil

The physical properties of the soil involves all the preliminary investigations of the soils which includes; Atterberg's limit, optimum moisture content, particle size analysis, cohesion and plasticity etc.

2.12.1 Collapse and swelling

Certain soil formations are prone to volume change due primarily to variation in moisture content. For example, loess deposits are characterized by high void ratio, low unit weight and are incompressible when dry. However, when wet, or subject to dynamic loading or shock they can be prone to sudden collapse. Inundation collapse is also a common phenomenon associated with loose man-made fills. Soils can swell due to rebound after a period of compression or as a result of the introduction of water. Montmorillonite clays, for example, characteristically swell when saturated leading to significant changes in volume. Swelling may also occur in soil due to the action of frost or from the exposure to air and moisture as in the case of some shale. Here expansion results from the formation of clay minerals. Swelling test requirements also exist for stabilised soils, MacNeil and Steele (2001).

2.12.2 Particle size

Particle size is defined as the percentages of various grain sizes present in a material as determined by sieving and sedimenta-tion (British Standard BS 1924: Part 1: 1990). BS 1924: Part 1: 1990 identified three classes of stabilized 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 stabilized 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 2.12.2: Classification of materials based on particle size distribution, source: BS 1924: part 1: 1990

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, ferromagnesium minerals	Quartz, feldspars, ferro- magnesium minerals, heavy minerals	Quartz, feldspars, secondary clay Minerals
General Characteristics	Loose grained, nonsticky, 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, noncohesive, 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.

Table 2.12.3: Soil classifications and properties, source: Townsend, (1973)

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.

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 behavior 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.

2.12.3 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 Stabilization/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 finegrained 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 2.12.3.** 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 nonplastic (Sherwood, 1993).

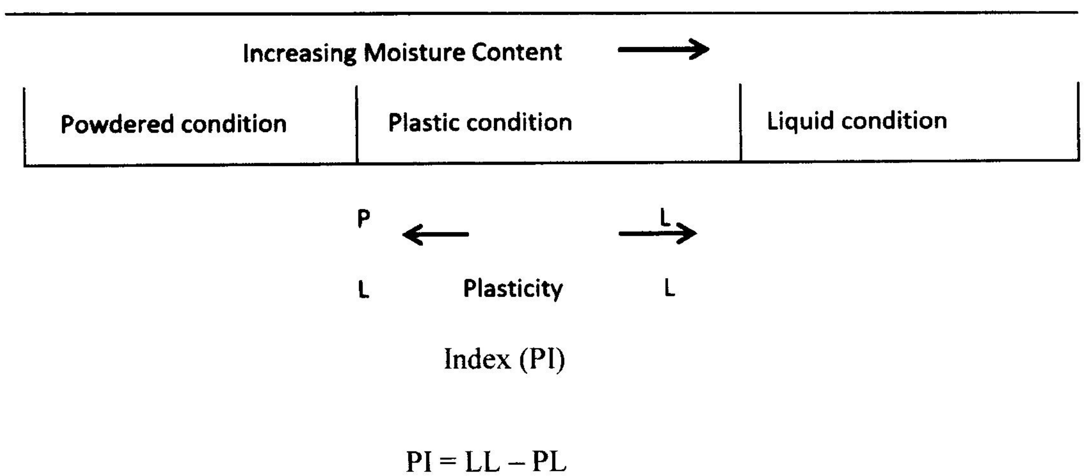


Figure 2.12.3: Definitions of soil plasticity, Sherwood (1993)

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.

2.12.4 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 claylime 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 longterm 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 minimize the flow of water through the stabilized material. \

2.12.5 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.

2.12.6 Frost heave and frost shattering

Sherwood (1992) described frost heave as an effect that can occur when temperatures are subzero for several days. If it is possible for water to move from the water table to the frozen zone easily, it will be drawn up into this zone where it will freeze to form ice lenses. Once this has occurred, further water may be drawn up and be subsequently frozen. It is expansion due to the freezing of transported water that is the primary cause of frost heave, rather than freezing of water originally present. Permeability of the material is identified as the leading factor behind frost heave susceptibility.Frost shattering is identified as the result of expansion of excess water present in voids of the surface of the material as it freezes, (Sherwood, 1992).

2.12.7 Temperature

The disruption of structure of a waste form can result from the action of frost on freshly solidified materials. Guidance on using concrete in cold weather (American Concrete Institute, 1994) can be used to help mitigate the effects of cold weather on waste form placement. Whilst temperature does not affect the improvement of plasticity in a lime modified soil, it can adversely affect early age strength development. Sherwood (1992) stated for lime, a minimum temperature of 7°C is stipulated whereas for cement, temperature is less important but a mini0mum of 3°C is given.

2.12.8 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.

2.12.9 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.

2.13 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

2.12.5 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.

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2.12.7 Temperature

The disruption of structure of a waste form can result from the action of frost on freshly solidified materials. Guidance on using concrete in cold weather (American Concrete Institute, 1994) can be used to help mitigate the effects of cold weather on waste form placement. Whilst temperature does not affect the improvement of plasticity in a lime modified soil, it can adversely affect early age strength development. Sherwood (1992) stated for lime, a minimum temperature of 7°C is stipulated whereas for cement, temperature is less important but a mini0mum of 3°C is given.

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). Kekere et al. (2012) also revealed in their findings on a research on "Geotechnical Investigation of Road Failure along Ilorin-Ajase – Ipo Road Kwara State, Nigeria" that the Effort to maintain the road along Ilorin-Ajase Ipo road by government agency have not yielded any result because the maintenance carried out was approached wrongly. It is evidently clear from the findings that poor foundation materials constitute the foundation of the road for instance, the presence of clayey soil and sandy soil have contributed to road failure witnessed on the road. It was recommended that areas badly affected should be scooped out and drainage should be provided to enable discharge of runoff because concentration of runoff during precipitation affects compaction level of the foundation where drainage facility is provided, debris and sediments should be cleared regularly to avoid blockage of culvert and drainage channels to enable free flow of water from the surface of foundation because concentration of run-off affects stability of foundations (2013).

2.14 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 Subbase 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. In this project we will be classifying soils in accordance with the (AASHTO) classification system and Federal Ministry of Works and Housing (1997).

2.14.1 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.

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

ii. 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.

iii. 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 2.14.1: 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 $\leq$ LL $-$ 30, the classification is A-7-5.
[b]If PI $>$ LL $-$ 30, the classification is A-7-6.

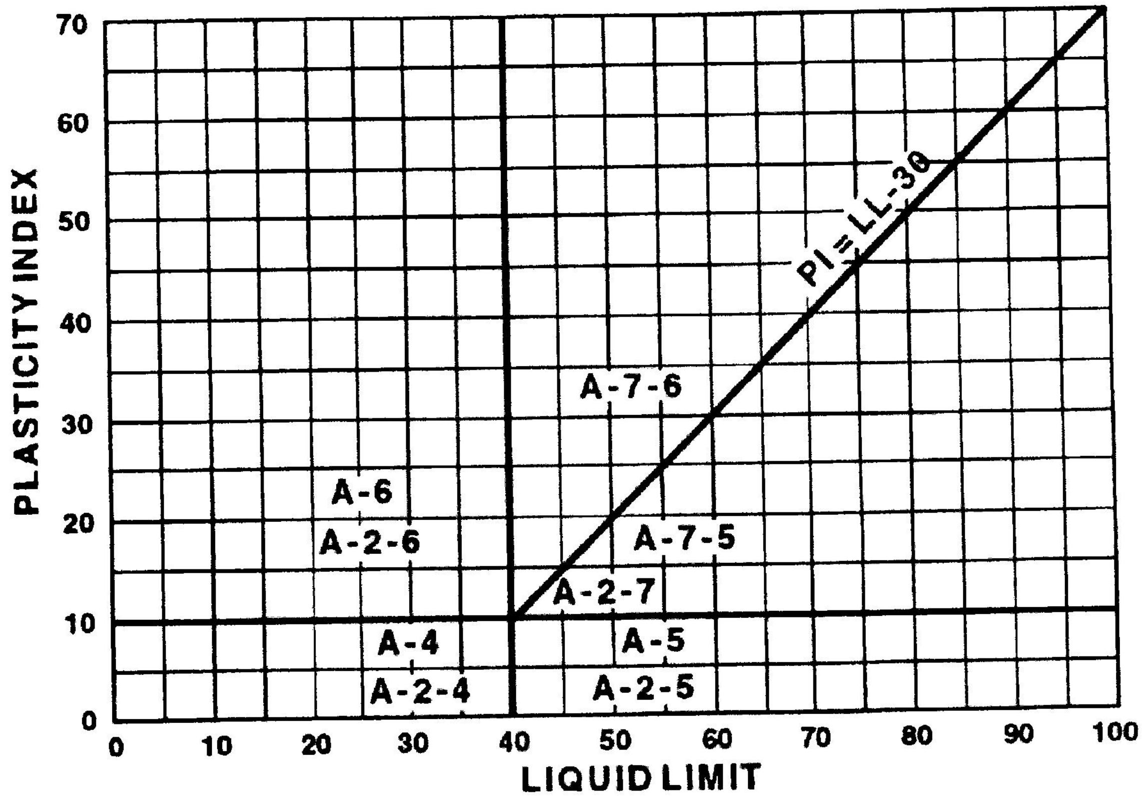

Figure 2.14.2: Relationship between liquid limit and plasticity index for silt-clay groups (aashto m 145-91)

2.15 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

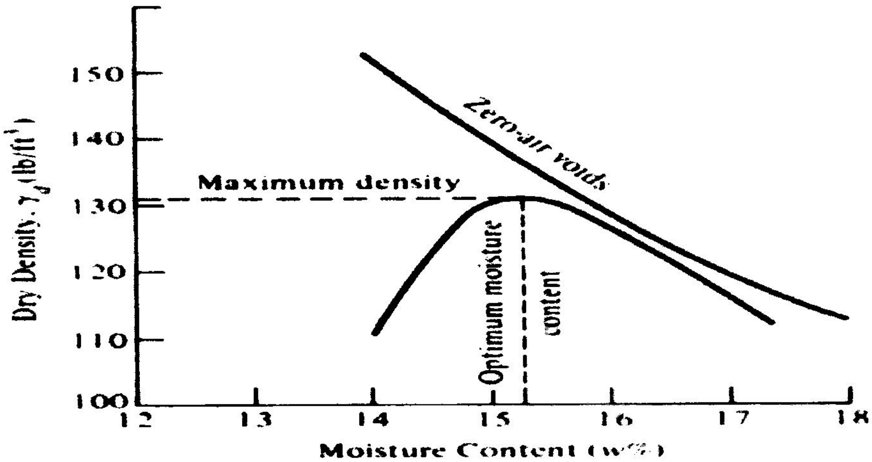

Figure 2.15: Typical moisture-density relationship for soils, Joseph E (1978)

CHAPTER THREE
METHODOLOGY

3.1 Preamble

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

3.2 Research Overview

The research work was conducted along selected points at five kilometer distance apart along Ikole - Ifaki road where soil samples were taken at a depth of 1.2m below the existing ground level from seven trial pits. After collection, the soil samples were stored in polythene bags to prevent loss of moisture content. 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 BS sieves to remove large particles. Molding 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). The properties of the soil samples were studied and determined to ensure that all relevant factors were available for establishment of correlations among them. The tests carried out on each of the selected samples were 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, Direct Shear test and California Bearing Ratio (CBR). 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.

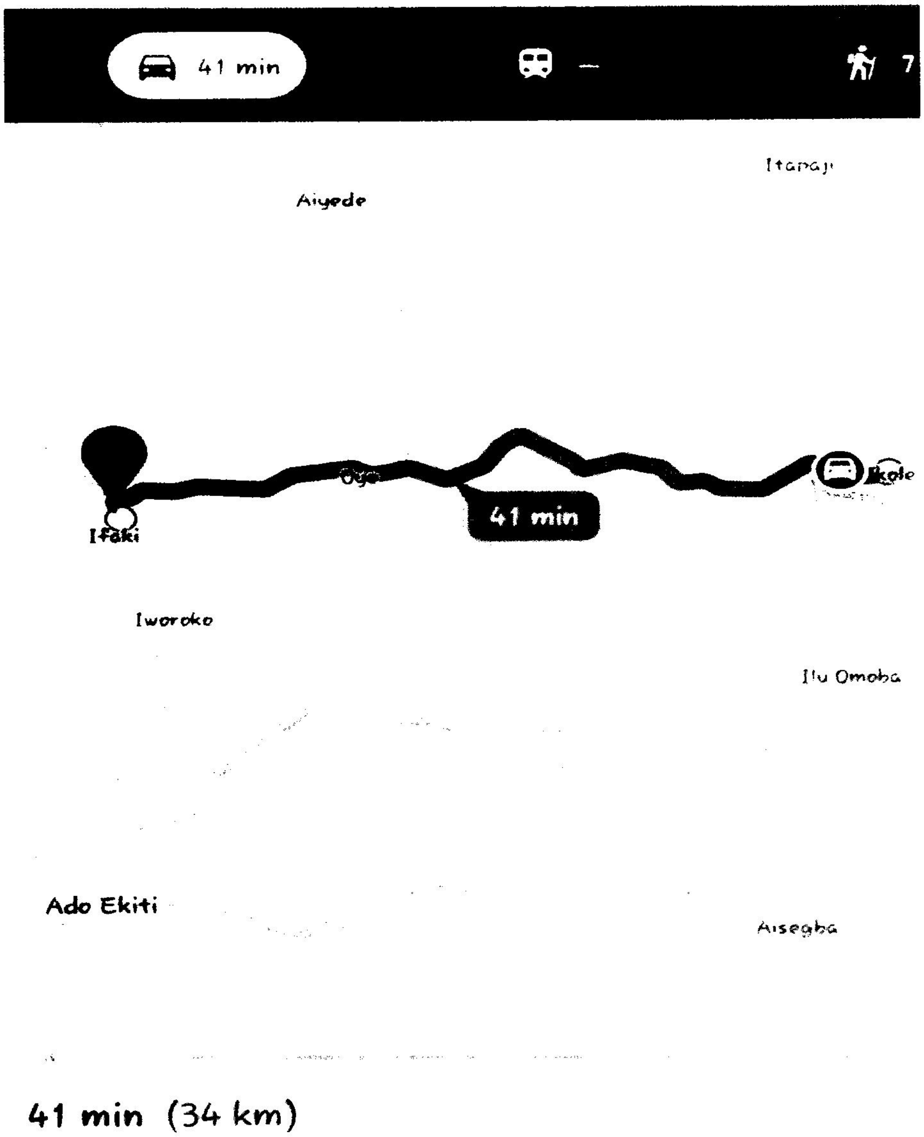

Plate 3.1: Picture showing the distance in km between Ikole and Ifaki

3.3 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. The researchers 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 and orientation of the drainage channels were done.

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

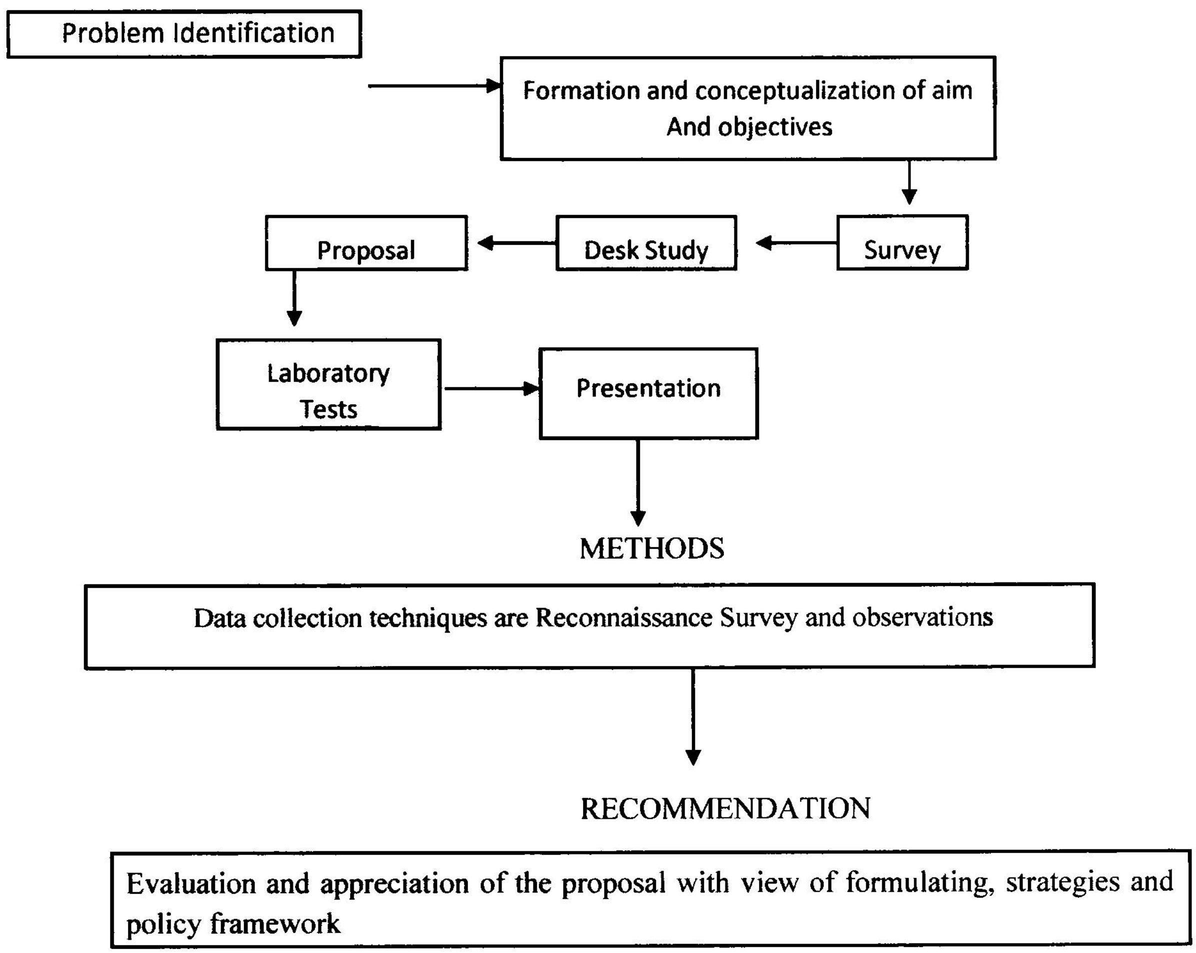

Figure 3.1: Diagrammatic Representation of the Research Design

3.4 Desk Study

The researchers 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- Ifaki road, the types of soil variation and the terrain of the area.

3.5 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 – Ifaki Ekiti. The researcher visited and explored the site for the purpose of investigating the soil conditions at the location of study (Ikole-Ifaki 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.

3.6 Laboratory Research

The following tests below were carried out in the laboratory at Federal polytechnic Ado – Ekiti, in order to classify and determine the properties of the soil samples collected. Listed below are the tests carried out on each layer of the road in the laboratory

3.6.1 Grain size analysis

This test was performed to determine the percentage of different grain sizes contained within a soil. The distribution of different grain sizes affects the engineering properties of soil. Specific gravity.

3.6.2 Consistency test

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

3.6.3 Natural moisture content

This test was performed to determine the natural moisture content contained within a soil.

3.6.4 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.

3.6.5 Compaction test

The main objective of this test was to determine the relationship between the moisture content and the density of a soil for a specified compaction effort. \

3.6.6 Permeability test

Permeability is a measure of the ease in which water can flow through a soil volume. It is one of the most important geotechnical parameters. However, it is probably the most difficult parameter to determine. In large part, it controls the strength and deformation behavior of soils.

3.6.7 California bearing ratio

This is a special test on soil to determine its suitability for use as sub-grade or base materials.

3.6.8 Direct shear test

This is a test use by geotechnical engineers to measure the shear strength properties of soil or rock material.

3.7 Sampling of Materials

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 were collected from seven different locations along Ikole- Ifaki Ekiti road, Ekiti State. This involved the digging of pits to a depth of 1200mm (1.2m) 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 3.7 below

Table 3.7: Sampling Point and their Respective Coordinates

S/N	SAMPLING POINT	COORDINATE IN DEGREE	
		NORTHING	**EASTING**
1	A	7.801274°	5.494107°
2	B	7.800392°	5.407323°
3	C	7.809864°	5.394132°
4	D	7.800042°	5.368914°
5	E	7.800522°	5.315929°
6	F	7.793276°	5.284588°
7	G	7.792541°	5.253878°

Plate 3.7: Sampling Points across Seven Locations

3.8 Sample Preparation

After collection, the 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 collected were 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 were stirred at regular intervals during the period of air drying.

3.9 Experimental Investigations

The conventional tests for evaluation of soil suitability for engineering purposes were carried out on collected samples from the study area.

3.9.1 Preliminary Investigations

This section entails all the physical carried out on samples. The tests are explained below.

3.9.1.1 Particle size distribution

The soil samples as received from the field were air dried for about 24hrs, larger particles in the soil samples were broken down to hasten drying. About 500g of each sample were weighed and wet sieved through the 4.75mm BS sieve. The retained washed samples were then oven dried at 120°C for 24hrs. The oven dried soil samples were then placed in a set of standard BS sieves and shaken using a mechanical sieve shaker. The weight in each sieve was then weighed and recorded for further computations

3.9.1.2 Specific gravity

The soil samples were riffled to pass through the 2mm BS sieve. 10g of the samples passing the 2mm sieve were then oven dried at 110°C for 24hrs. The density bottle was then weighed to the nearest 0.001g as W1, then the oven dried sample poured into the density bottle and weighed to the nearest 0.001g as W2. After which distilled water was poured into the density bottle. Entrapped air was removed by stirring the mixture with a clean glass rod until no further loss of air is observed. The density bottle was then weighed with its content as W3. The density bottle was then emptied, cleaned and filled with distilled water and weighed as W4. The process was repeated twice for each soil sample.

3.9.1.3 Atterberg's limits

1. Plastic limit: A representative soil sample passing through the 425µm BS sieve was mixed with distilled water and left to soak for about 24hrs. A portion of the soaked soil was taken and rolled on a tray with the fingers to form a roll of moist soil of about 3mm diameter. The soil sample was further rolled until cracks appeared when rolled below 3mm diameter. The moisture content of the cracked sample was determined. The process is then repeated twice for each soil sample.
2. Liquid limit: A representative soil sample passing through the 425µm BS sieve is mixed with distilled water in and evaporating dish and left for 24hrs to soak. A portion of the soil paste was then placed on the liquid limit device after which a grooving tool was used to cut the soil. The handle of the liquid limit apparatus was then rotated and the number of blows counted till the two parts of the soil sample come into contact. A portion of the soil paste was then taken near the groove and its water content determined. The remaining soil was then mixed with the original sample paste and the test repeated for the each sample while altering the water content.

3.9.1.4 Natural moisture content

Portions of the soil samples were placed in polythene bags from the field in order to preserve the moisture contents. The weight of the empty cans to be used were weighed as W1, the soil samples were then placed in the cans and weighed as W2. The cans were placed in the oven and dried at 105°C for 24hrs. The weights of the cans and dry soil were then determined as W3. The values of W1, W2 and W3 were then used to compute the natural moisture contents of the soil samples.

3.9.2 Main tests

This section consists of the Engineering tests carried out on the soil samples.

3.9.2.1 Compaction test

A 6kg representative sample of air dried soil passing through the 19mm sieve was taken and mixed thoroughly with about 2.5% water and then stored in a sealed container for 24hrs. The mold of 1000cc capacity with the base plate attached was weighed to the nearest 1g as W1. The moist soil sample was then compacted into the mold in five layers and given 25 blows distributed uniformly over the surface of the sample from a 4.5kg rammer from a drop height of 450mm above the soil. After the extension of the mold is removed and the top levelled with a straight edge. The mold and the compacted soil was then weighed as W2. The compacted soil was then removed from the mold after which the moisture content and density was determined.

3.9.2.2 California bearing ratio test (C.B.R)

A representative soil sample of about 6kg was taken and compacted in a compaction mold in five layers with 10 blows per layer. After compaction, the compacted soil is left for about two days and then taken under the penetration piston and a surcharge load was placed on it. The load was applied and the penetration load values were recorded

3.9.2.3 Direct shear test

After the shear box was assembled, the soil sample was compacted in the mold and the dial gauges set and the readings set to zero. The desired load was then placed and the reading on the dial gauge reset to zero. The direct shear test machine was then started to produce a constant rate shearing, after which the readings of the shear force, normal force and the applied normal load were recorded.

3.9.2.4 Permeability test

2.5kg of each sample was taken from thoroughly mixed air dried sample material. Initial moisture content of sample was recorded. Empty parameter mould was recorded. Mould was greased, soil sample was then placed and compacted. The excess soil was removed and the weight was determined. After the experiment procedures the coefficient of permeability was then recorded on cm/sec at 27°.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Introduction

This chapter presents the results of various physical and geotechnical tests carried out on seven locations on the 34.4km road span from Ikole-Ekiti to Ifaki-Ekiti. The laboratory tests carried out on the samples are grouped into physical properties and Engineering properties. Physical tests include (Particle size distribution, Atterberg limits, Natural Moisture Content) and Engineering tests (permeability, Compaction test, direct shear test and California Bearing Ratio). The summary of the results are discussed below.

4.2 Summary of Result

The results, analysis and discussion from the preliminary tests (Grain size analysis, Natural moisture contents, Specific gravity and Atterberg's limits) as well as the Engineering properties (Compaction, California Bearing Ratio (CBR), direct shear test and Permeability) are shown below in Table 4.2.

Table 4.2: Summary results table for preliminary analysis tests and engineering test

SAMPLE		A	B	C	D	E	F	G
LOCATION		**IKOLE**	**OSIN**	**ITAPA**	**ILUPEJU**	**OYE**	**AIYEGBAJU**	**IFAKI**
SIEVE ANALYSIS (% PASSING)	**2.36mm**	92.0	68.9	43.8	86.4	89.5	48.2	94.3
	0.6mm	82.2	51.2	32.2	60.7	67.0	35.2	73.4
	0.0075mm	62.1	23.8	15.4	33.6	34.7	16.6	46.5
ATTERBERG LIMITS (%)	**L.L**	51.0	29.0	30.2	32.6	36.6	37.0	41.6
	P.L	25.2	17.0	20.7	20.9	19.3	21.0	23.8
	P.I	25.8	12.0	9.5	11.7	15.6	16.0	17.8
	S.L	11.4	8.3	8.6	7.1	7.9	7.1	9.3
NATURAL MOISTURE CONTENT (%)		22.3	13.6	15.4	16.2	19.2	18.5	22.1
AASHTO CLASSIFICATION		A-7-5(14)	A-2-6(0)	A-2-4(0)	A-2-6(0)	A-2-6(1)	A-2-6(0)	A-7-6(5)
SPECIFIC GRAVITY		2.53	2.38	2.42	2.52	2.42	2.51	2.51
COMPACTION TEST	**OMC %**	16.0	17.1	16.0	17.0	16.5	18.0	19.5
	MDDKn/M^3	1.52	1.63	1.68	1.72	1.51	1.62	1.52
C.B.R	**2.5mm (%)**	13.8	17.5	25.7	71.4	19.8	52.9	20.74
	5.0mm (%)	23.4	39.4	44.67	80.4	29.4	65.0	23.5
	Final (%)	23	39	44	80	29	65	23
DIRECT SHEAR	**(Kn/mm2)**	36	24	61	42	36	35	28
	(°)	18	21	20	15	15	21	18
PERMEABILITY (CM/S)		1.68E-3	1.10E-3	9.95E-4	1.24E-4	5.87E-7	1.27E-3	9.15E-4

4.2.1 Specific gravity

The specific gravities of sample A, B, C, D, E, F and G are **2.53, 2.38, 2.42, 2.52, 2.42, 2.51** and **2.51** respectively. This values fall within that given in **Das, (2010)** for Clay minerals, as Halloysite (2.0-2.55) shown in Table 4.2.1.1. The results are shown in Table 4.2.1 below.

Table 4.2.1: Specific gravity values of soil samples

Soil samples	Specific gravity values
A	2.53
B	2.38
C	2.42
D	2.52
E	2.42
F	2.51
G	2.51

Table 4.2.1.1: Specific gravity of common minerals, Braja M. Das (2010)

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

4.2.2 Grain size distribution

Table 4.2.2 and Figure 4.2.2 shows the results of the grain size distribution. The percentage of the sample passing through No 200 BS for soil Sample A, B, C, D, E, F and G are 62.1%, 23.8%, 15.4%, 33.6%, 34.7%, 16.6 and 42.1% respectively . The results show that soil samples A and G had a high percentage finer than 0.075 fractions (i.e. >35%) while the remaining five (5) samples have a low percentage finer than 0.075 fractions (i.e. <35%). Hence, sample A and G have a general rating as sub-grade in accordance with AASHTO (1986) of fair to poor materials while samples B, C, D, E and F have a general subgrade rating of excellent to good. Samples A and G have significant constituent materials of mainly clayey soils while samples B,C,D,E and F are silty or clayey gravel and sand where the percentage passing the No. 200 sieve is less than 35%. Sample A is classified as **A-7-5(14)** , sample B as **A-2-6(0)** , sample C as **A-2-4(0)** , sample D as **A-2-6(0)** , sample E as **A-2-6(1)** , sample F as **A-2-6(0)** and sample G, as **A-7-6(5).** The figure below shows the grain size distribution of sample point G under Appendix B. According to the Federal Ministry of Works and Housing (1997) specification, all soils tested except A and G are suitable for subgrade, subbase and base materials as the percentage finer than No. 200 BS test sieve is less than 35%.

Table 4.2.2: Results from grain size distribution

Soil sample	**(%) passing 0.0075mm**	**AASHTO classification**
A	62.1%	A-7-5(14)
B	23.8%	A-2-6(0
C	15.4%	A-2-4(0)
D	33.6%	A-2-6(0)
E	34.7%	A-2-6(1
F	16.6%	A-2-6(0)
G	42.1%	A-7-6(5)

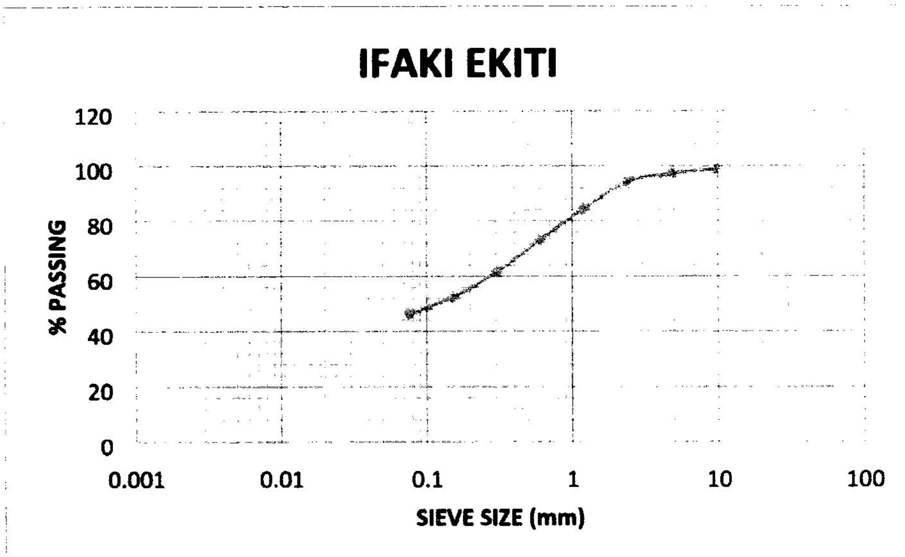

Figure 4.2.2: Grain size analysis graph for sample point G

4.2.3 Moisture content

The natural moisture content of samples taken along the stretch of road at the towns of Ikole, Osin, Itapa ,Ilupeju ,Oye,Aiyegbaju and Ifaki were **22.3, 13.6, 15.4, 16.2,19.2, 18.5** and **22.1(%)** respectively as shown in Table 4.2.3. Sample B (Osin) had the lowest natural moisture content while sample A (ikole) had the highest. This is a function of the void ratios and the specific gravities of the samples. This showed that the soil samples contained appreciable amount of moisture which is largely affected by the climatic condition. The result shows that moisture content value ranges from 13.6% to 22.3%. As a result of the moisture content values not exceeding 25%, it implies the soils in the study location have a low potential of water retention. See appendix A

Table 4.2.3: Results of natural moisture content

Soil sample	Moisture content (%)
A	22.3
B	13.6
C	15.4
D	16.2
E	19.2
F	18.5
G	22.1

4.2.4 Atterberg's limit

From Appendix D, the results of the Atterberg's limit test is shown the following variations in values of Liquid limit, Plastic Limit and plasticity index. LL: 29.0- 51%, PL: 17.0- 25.2% and PI: 9.5-25.8% for sample materials between Ikole and Ifaki. Results of the samples is shown in the Table 4.2.4 and Figure 4.2.3 below. According to AASHTO (1986) plasticity index not greater than 55%, liquid limit not greater than 80% is suitable enough. It is therefore observed that samples A-G fall below 55% for plasticity index and below 80% for liquid limit. The Federal Ministry of Works and Housing (1997) specified 50% maximum value of liquid limit for sub base and base materials, Simeon et al. (1973) recommended plastic limit of 25% maximum for sub grade tropical soils. It is therefore good as a subgrade, sub base and base material for construction because it meets the acceptable standards. The soil materials are classified as a cohesive clay with a high degree of plasticity.

Table 4.2.4: Results of Atterberg's limits test for all the samples (A-G)

SAMPLE		A	B	C	D	E	F	G
LOCATION		IKOLE	OSIN	ITAPA	ILUPEJU	OYE	AIYEGBAJU	IFAKI
ATTERBERG	L.L	51.0	29.0	30.2	32.6	36.6	37.0	41.6
LIMITS	P.L	25.2	17.0	20.7	20.9	19.3	21.0	23.8
(%)	P.I	25.8	12.0	9.5	11.7	15.6	16.0	17.8
	S.L	11.4	8.3	8.6	7.1	7.9	7.1	9.3

Table 4.2.4.1: Classification of soil according to plasticity index

Plasticity index (%)	Soil type	Degree of plasticity	Degree of cohesiveness
0	Sand	Non-plastic	Non-cohesive
<7	Silt	Low plastic	Partly cohesive
7-17	Silt clay	Medium plastic	Cohesive
>17	Clay	High plastic	cohesive

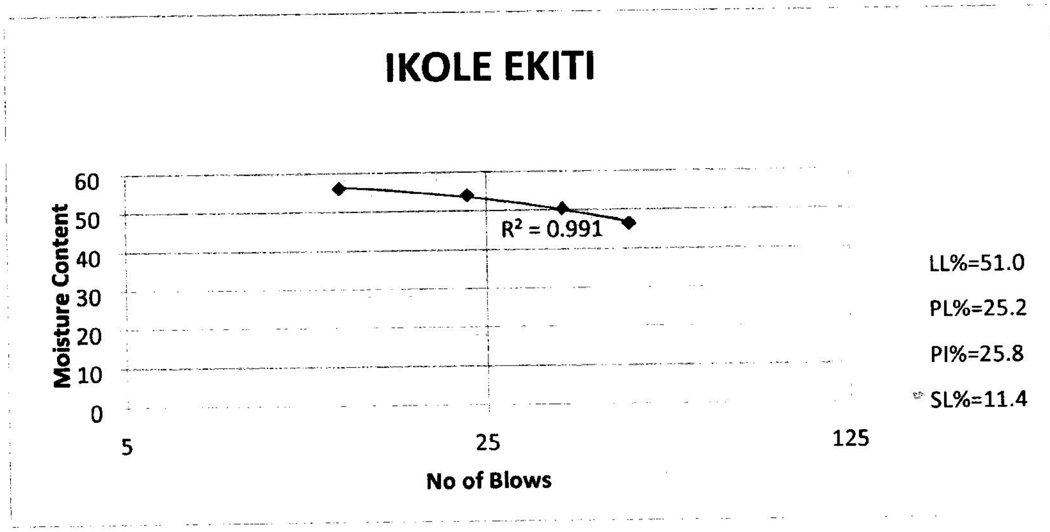

Figure 4.2.4: Atterberg's limit graph for sample A

4.2.5 Permeability test (falling head)

Results of permeability tests are shown in Appendix H and Table 4.2.5 the results indicate coefficients of permeability of the soils in the range of 9.15*10-4 to 1.10*10-3cm/sec: this is due to the low fines content in the studied soil samples, AASHTO (1986). The soil samples B, C, F and G can therefore be classified to be of good drainage characteristics and medium permeability classification while samples A, D, and E are of poor drainage and low permeability classification.

TABLE 4.2.5: Summary of permeability test results

SAMPLE	K(cm/sec)	Drainage Characteristics	Permeability Classification
A	5.87E-4	POOR	LOW
B	1.24E-3	GOOD	MEDIUM
C	1.68E-3	GOOD	MEDIUM
D	9.95E-4	POOR	LOW
E	9.15E-4	POOR	LOW
F	1.27E-3	GOOD	MEDIUM
G	1.10E-3	GOOD	MEDIUM

4.2.6 Compaction test

The results of the compaction test are shown in Appendix F with their respective graphs. From Table 4.2.6, Figure 4.2.6 and 4.2.6.1, it was observed that the OMC values varies from 16% - 19.5% for soil sample materials, the MDD values varies from 1.51 - 1.72kg/m3 for the soil sample materials while the optimum moisture content ranges between 16.0% and 19.5%. According to O‟Flaherty (1988) the range of values that may be anticipated when using the modified proctor test methods are: For clay, maximum dry density (MDD) may fall before 1.44Mg/m3 and 1.685Mg/ m3 and optimum moisture content (OMC) may fall between 20-30%. For silty clay MDD is usually between 1.6 and 1.845Mg/m3 and OMC ranged between 15-25%. For sandy clay, the samples from sampling point A, B, C, D, E, F and G did not meet AASHTO (1986) specification which state the MDD values most not be less than 1.76kg/m3 for subgrade sample, this implies that the samples are not suitable for subgrade materials. The 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 must not be less than 2.0kg/m3, hence they are also not suitable for base and sub base materials.

Table 4.2.6: Results of compaction test

SAMPLE	COMPACTION	
	MDD (KG/M3)	OMC (%)
A	1.52	16.0
B	1.63	17.1
C	1.68	16.0
D	1.72	17.0
E	1.51	16.5
F	1.62	18.0
G	1.52	19.5

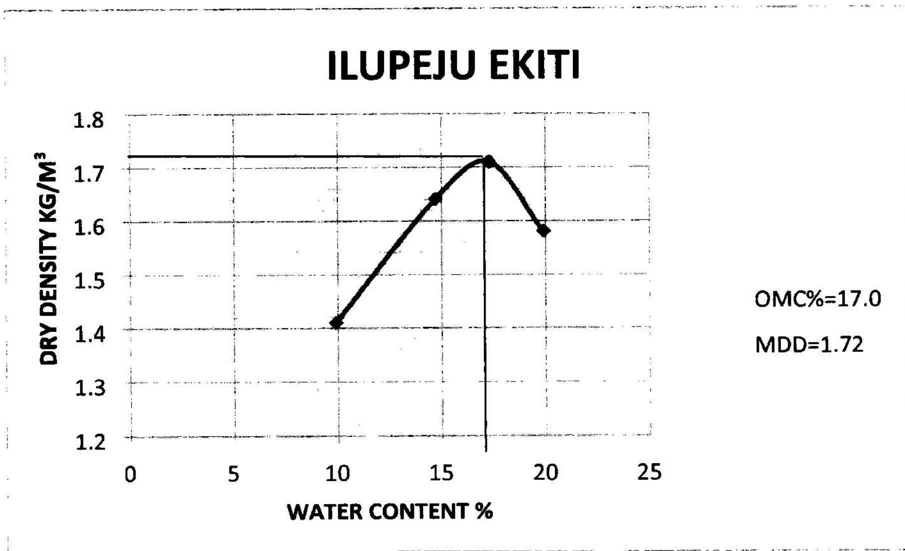

Figure 4.2.6: Plot of compaction test graph for sample D

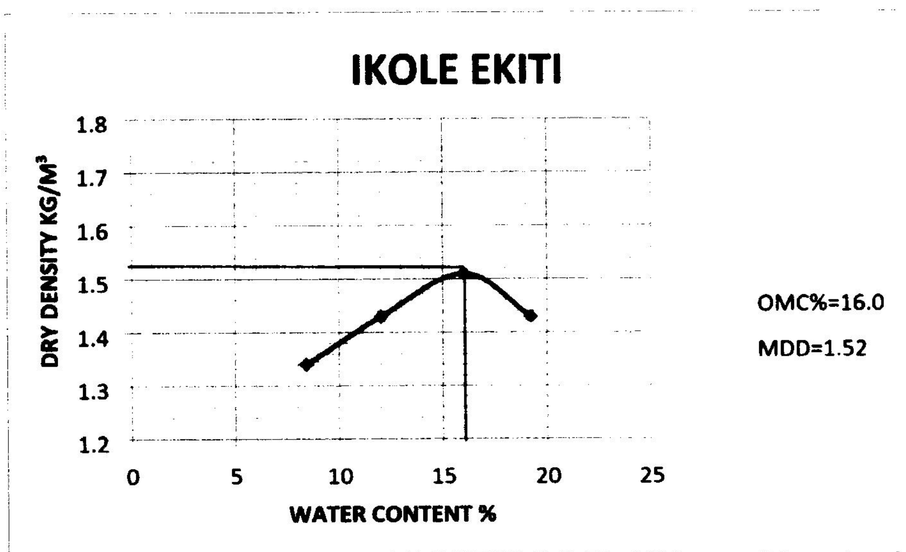

Figure 4.2.6.1: Plot of compaction test graph for sample A

4.2.7 California bearing ratio (CBR)

The un-soaked California bearing ratio of the sample ranges from 13.8% to 71.4% for 2.5 mm plunger penetration and 23.4% to 80.4% for 5mm plunger penetration. Sample A has the lowest CBR value for 2.5 mm plunger penetration. According to AASHTO (1986), the specified value for unsoaked CBR should not be less than or equal to 10% for sub-grade materials, and from the results it was deduced that the unsoaked CBR values varied between 23.4 % and 80.4% for materials in all the locations, which are greater than 10% specified by AASHTO (1986).All samples are not suitable as a subgrade material and therefore will require stabilization for satisfactory performance as shown in its plot of maximum dry density against optimum moisture content. The above results implied that almost all the soil materials used for subgrade courses along the chainages of the road are not suitable. Thus, these factors may have contributed to the widespread failure observed on the roadway. Also according to the Federal Ministry of Works and Housing (1997) subgrade, subbase, and base soils should be less than or equal to 10%, 30%, and 80% respectively. Hence A, G and E are good for subbase materials all samples have the recommended value for use as base materials. (Federal Ministry of Works and Housing, 1997).See Appendix G and Table 4.2.7 and Figure 4.2.7.

Table 4.2.7: Unsoaked california bearing ratio test results

SAMPLES	**C.B.R**	
	2.5mm (%)	**5.0mm (%)**
A	13.8	23.4
B	17.5	39.4
C	25.7	44.67
D	71.4	80.4
E	19.8	52.9
F	29.4	65.0
G	20.74	23.5

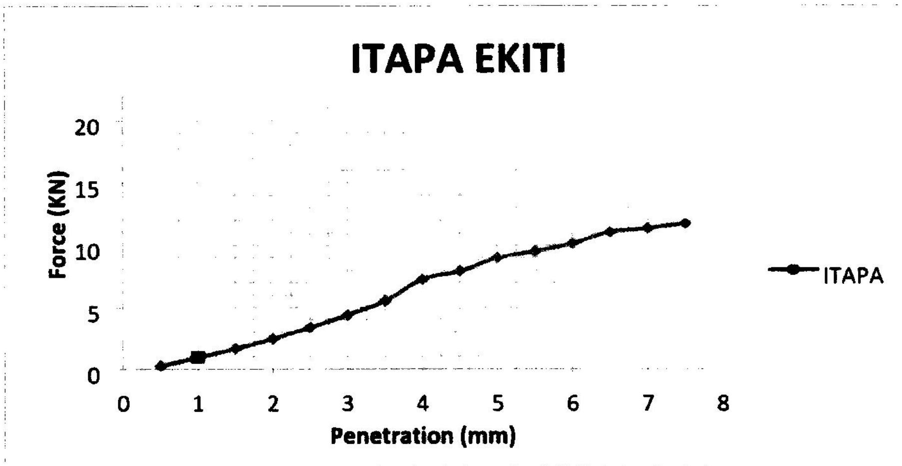

Figure 4.2.7: Plot of the CBR on sampling point F

4.2.8 Direct shear test

The direct shear test is used to determine the angle of internal friction of the soil, the cohesion of the soil and the effective pressure. Table 4.2.8, Figure 4.2.8 and 4.2.8.1 shows the results for the direct shear test. The test shows that the soil samples A, B, C, D, E, F and G compacted in a modified proctor exhibit cohesion values of 36Kn/mm2, 24 Kn/mm2, 61 Kn/mm2, 42 Kn/mm2, 36 Kn/mm2, 35 Kn/mm2, and 28 Kn/mm2 respectively and angle of internal friction of 18°, 21°, 20°, 15°, 15°, 21°, and 18° (degrees) respectively. The graphical representation is shown below. It indicates that soil samples A, B, C, D, E, F and G are clays because the angle of internal friction ranges from 0-20 degree. Soil particles with high angularity tend to resist displacement and hence possess higher shearing strength when compared to those with less angular particles.

Table 4.2.8: Results of the direct shear test

Soil samples	Cohension values (Kn/mm2)	Angle of internal friction (°)
A	36 Kn/mm2	18°
B	24 Kn/mm2	21°
C	61 Kn/mm2	20°
D	42 Kn/mm2	15°
E	36 Kn/mm2	15°
F	35 Kn/mm2	21°
G	28 Kn/mm2	18°

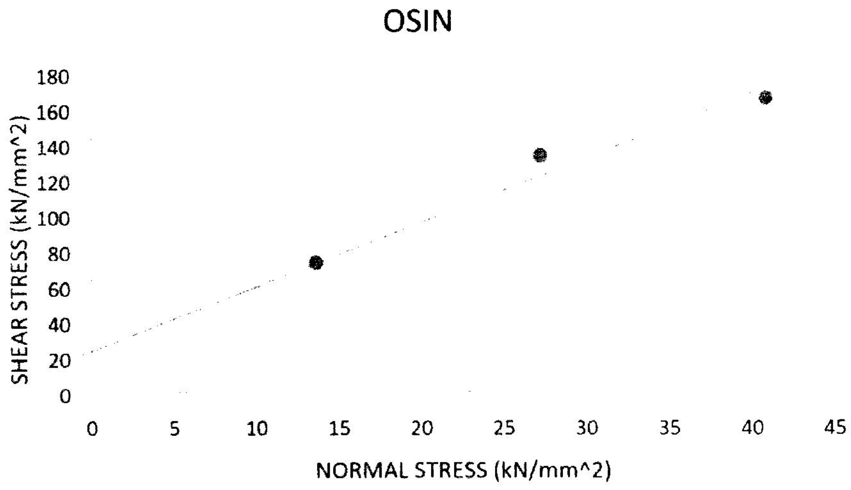

Figure 4.2.8: Plot of shear stress vs normal stress for sample B

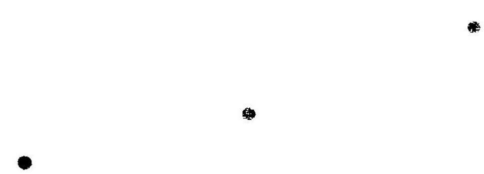

Figure 4.2.8.1: Plot of shear stress vs. normal stress for sample A

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

From the experiments carried out to determine the geotechnical properties of the soil, which includes Particle size distribution, Atterberg limits, Natural Moisture Content, permeability, Compaction test, direct shear test and California Bearing Ratio. The following conclusions can be stated. It has been examined that any soil which exceeds 25% in its natural moisture content has a very high water retaining capacity which is not good for the construction of a roadway. This can be observed in samples A (Ikole) and Samples G (Ifaki). Also samples A and G are of clayey materials because they pass through the 0.0075mm sieve. Sample A and G have a general rating as sub-grade in accordance with AASHTO (1986) of fair to poor materials while samples B, C, D, E and F have a general subgrade rating of excellent to good. According to AASHTO classification system (1986) in our categorization soil samples A-G were grouped as follows. Sample A is classified as **A-7-5(14)**, sample B as **A-2-6(0)**, sample C as **A-24(0)** , sample D as **A-2-6(0)** , sample E as **A-2-6(1)** , sample F as **A-2-6(0)** and sample G as **A-76(5).** The optimum moisture content from the analysis in this project from samples A to G are as follows **16, 17.1, 16.0, 17.0, 16.5, 18.0** and **19.5(%).** The MDD from the compaction test varies from (1.52 to 1.72) and according to AASHTO (1986) specification which state the MDD values most not be less than **1.76kg/m3** for subgrade samples therefore the soil samples taken from each location are therefore not suitable and need to be noted before beginning any road construction. The Atterberg's limit test shows that the soil samples from point A to G are good as a subgrade material for road construction. Also the results indicate coefficients of permeability of the soils in the range of 9.15*10-4 to 1.10*10-3 cm/sec: this is due to the low fines content in the studied soil samples, AASHTO (1986).). The soil samples B, C, F and G can therefore be classified to be of good drainage characteristics and medium permeability classification while samples A, D, and E are of poor drainage and low permeability classification. From the compaction test, samples from sampling point A, B, C, D, E, F and G did not meet AASHTO (1986) specification which state the MDD values must not be less than 1.76kg/m3 for subgrade sample, this implies that the samples are not suitable for subgrade materials. According to AASHTO (1986), the specified value for unsoaked CBR should be less than or equal to 10% for sub-grade materials, and from the results it was deduced that the unsoaked CBR values varied between 23.4 % and 80.4% for materials in all the locations, which are more than 10% specified by AASHTO (1986) (i.e. Unsoaked CBR $\leq$ 10%), $\leq$30% for subbase and $\leq$ 80% for base material except for sample D which has an unsoaked CBR value of 80.4% and can be used for base course. With all these analyses put in mind, it should be considered that soils (subgrade) along the 34.4km span length between point A (IKOLE-EKITI) and point G (IFAKI-EKITI) need to be stabilized properly in order for it to be suitable for future road way construction in order to prevent rapid road way deterioration as it is observed by road users

5.2 Recommendation

In order to prevent to constant reoccurrence of failed roads in the country and the study area considered in this project, it is necessary to critically evaluate the geotechnical properties of the soil so as to prevent future road failure. These valuable data obtained from the geotechnical analysis can be useful for civil engineers in the design and construction of roads in the Ikole and Ifaki environs for maximum durability and efficiency. It is recommended that engineering confirmatory test be carried out before embarking on any construction such as road. Geotechnical evaluation of soils is an important factor in Civil Engineering fields as it is a baseline for any construction of the roadway of any span length. With the results from geotechnical tests, Engineers, sponsors, government bodies can be adequately informed about present conditions of the soils if it can adequately perform to the required expectation. Also in recommendation adequate supervision of test experimentation and analysis needs to be done with adequate care so as not to begin construction of roadway along a poor soil area. If these are properly considered and recommended, the issues of damaged roadways will be controlled and in future times be improved on. The government should also find policies to find alternative means of transportation so as to reduce the influx of transport on roadway most especially heavy duty trucks and machines which tends to reduce the life span of the road. It is 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

Natural moisture content (N.M.C)

Table A1: Results of natural moisture content for (Oye, Ifaki, Osin, Ikole) Ekiti.

	OYE-EKITI		**IFAKI-EKITI**		**OSIN-EKITI**		**IKOLE-EKITI**	
Can Number	**1**	**2**	**1**	**2**	**1**	**2**	**1**	**2**
Weight of Can (g)	26.8	19.1	19.9	28.8	26.8	20.2	26.9	26.9
Weight of Can + wet soil (g)	54.2	49.7	65.8	64.9	62.9	57.6	59.5	62.7
Weight of Can + dry soil (g)	49.6	45.0	57.6	58.3	58.5	53.0	53.7	56.0
Weight of water (g)	4.6	4.7	8.2	6.6	4.4	4.3	5.8	6.7
Weight of dry soil (g)	22.8	25.9	37.7	29.5	31.7	32.8	26.8	29.1
Moisture content (%)	20.2	18.1	21.8	22.4	14.0	13.1	21.6	23.0
Average (%)	19.2		22.1		13.6		22.3	

Table A2: Results of natural moisture content for (Aiyegbaju, Ilupeju, Itapa) Ekiti.

	AIYEGBAJU-EKITI		**ILUPEJU-EKITI**		**ITAPA-EKITI**	
Can Number	**1**	**2**	**1**	**2**	**1**	**2**
Weight of can (g)	26.7	19.9	19.8	18.3	9.9	11.7
Weight of can + wet soil (g)	70.7	59.6	64.1	63.1	62.8	57.1
Weight of can + dry soil (g)	63.8	53.5	56.2	55.5	55.6	51.2
Weight of water (g)	6.9	6.1	7.9	7.6	7.2	5.9
Weight of dry soil (g)	36.9	33.6	36.4	37.2	45.7	39.5
Moisture content (%)	18.7	18.2	11.9	20.4	15.8	14.9
Average (%)		18.5	16. 2			15.4

Appendix B

Particle size distribution

Table B1: Results of particle size analysis of (Ifaki, Oye, Ikole) Ekiti.

	IFAKI-EKITI			OYE-EKITI			IKOLE-EKITI		
SIEVE OPENING	**WEIGHT RETAINED**	**% RETAINED**	**% PASSING**	**WEIGHT RETAINED**	**% RETAINED**	**% PASSING**	**WEI GHT RETAINED**	**% RETAINED**	**% PASSING**
3/8	5.6	1.1	98.9	4.7	0.9	99.1	4.1	0.8	99.2
4	8.4	1.7	97.2	19.6	3.9	95.2	17.1	3.4	95.8
8	14.3	2.9	94.3	28.7	5.7	89.5	18.8	3.8	92.0
16	49.4	9.9	84.4	42.9	8.6	80.9	17.6	3.5	88.5
30	54.8	11.0	73.4	69.4	13.9	67.0	31.3	6.3	82.2
60	58.5	11.7	61.7	82.7	16.5	50.5	45.6	9.1	73.1
100	45.7	9.1	52.6	53.8	10.8	39.7	34.1	6.8	66.3
200	30.3	6.1	46.5	24-.9	5.0	34.7	21.1	4.2	62.1

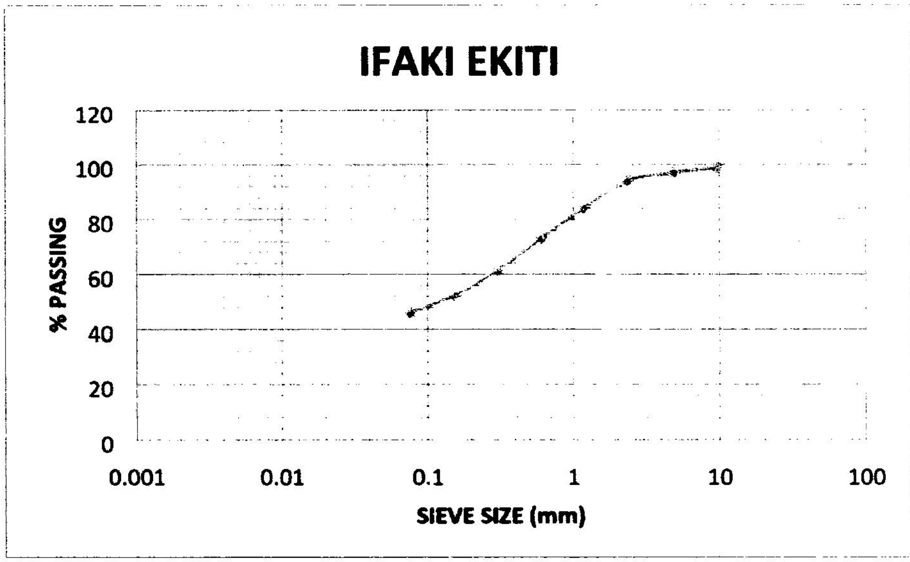


Figure B1: Diagram of particle size distribution for Ifaki-Ekiti

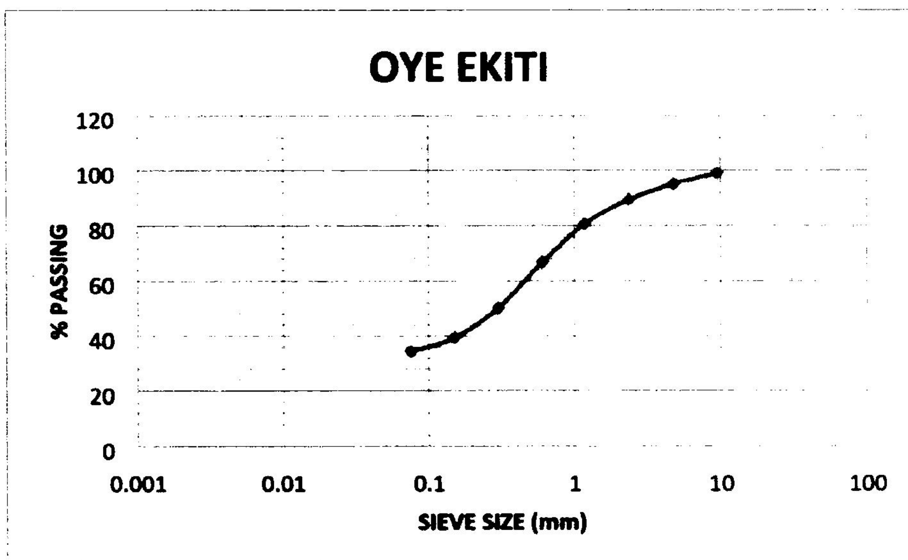


Figure B2: Diagram of particle size distribution for Oye-Ekiti

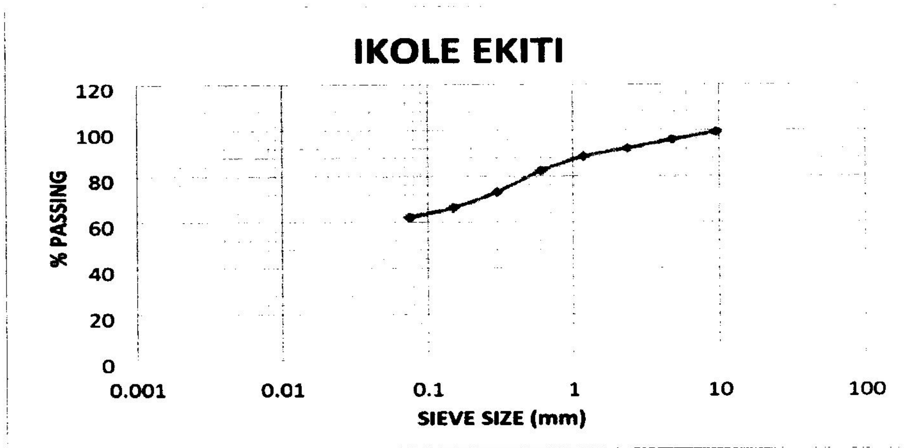
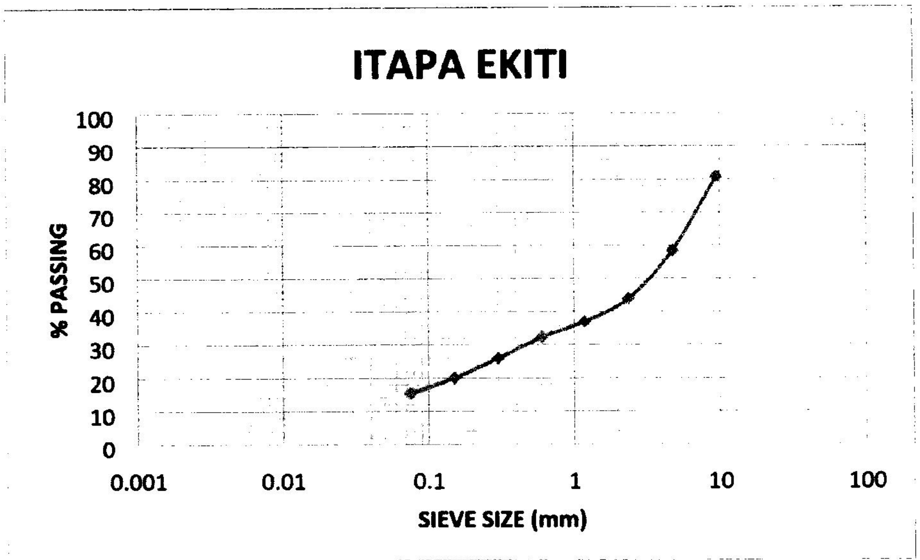


Figure B3: Diagram of particle size distribution for Ikole-Ekiti

Table B2: Results of particle size analysis of (Itapa, Ilupeju, Aiyegbaju) Ekiti.

	ITAPA-EKITI			ILUPEJU-EKITI			AIYEGBAJU-EKITI		
SIEVE OPENING	**WEIGHT RETAINED**	**% RETAINED**	**% PASSING**	**WEIGHT RETAINED**	**% RETAINED**	**% PASSING**	**WEI G HT RETAINED**	**% RETAINED**	**% PASSING**
3/8 INCH	96.5	0.8	99.8	9.0	1.8	98.2	103.0	20.6	79.4
4	112.2	3.4	95.8	24.5	4.9	93.3	102.2	20.4	59.0
8	72.4	3.8	92.0	34.5	6.9	86.4	53.8	10.8	48.2
16	34.6	3.5	88.5	51.0	10.2	76.2	30.4	6.1	42.1
30	23.7	6.3	82.2	77.3	15.5	60.7	34.4	6.9	35.2
60	31.5	9.1	73.1	81.1	16.2	44.5	42.9	8.6	26.6
100	29.3	6.8	66.3	40.8	8.2	36.3	30.6	6.1	20.5
200	23.0	4.2	62.1	13.5	2.7	33.6	19.3	3.9	16.6



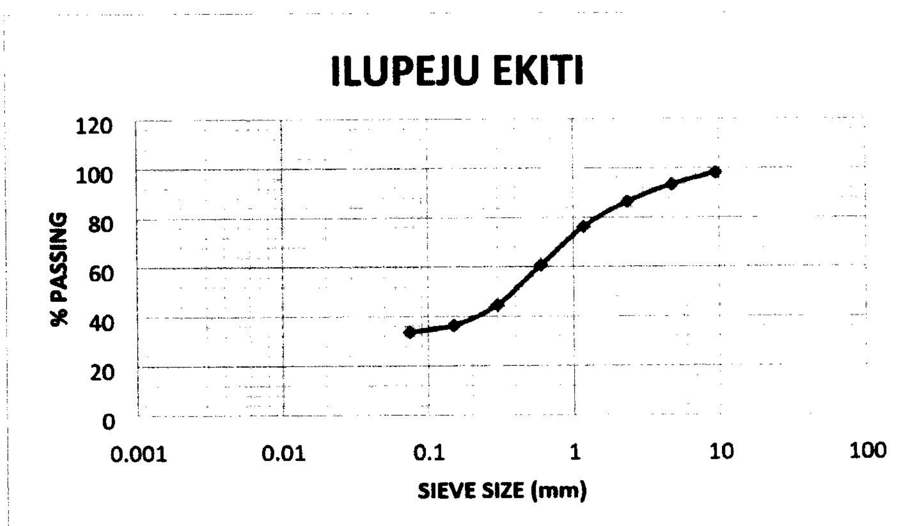


Figure B4: Diagram of particle size distribution for (Itapa, Ilupeju) Ekiti

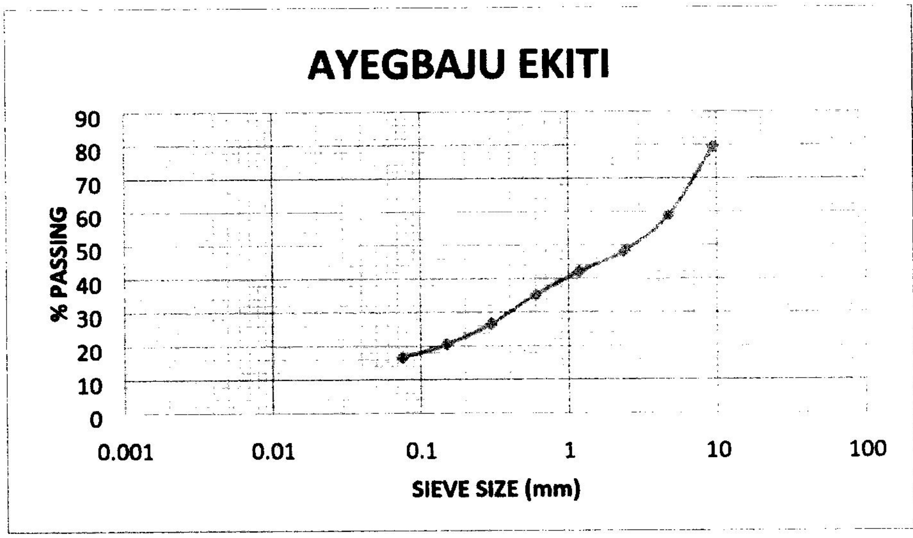


Figure B5: Diagram of particle size distribution for Aiyegbaju-Ekiti

Table B3: Results of particle size analysis of Osin-Ekiti

	OSIN-EKITI		
SIEVE OPENING	**WEIGHT RETAINED**	**% RETAINED**	**% PASSING**
3/8 INCH	100	20.0	80.0
4	18.2	3.6	76.4
8	37.5	7.5	68.9
16	48.3	9.7	59.2
30	39.9	8.0	51.2
60	70.5	14.1	45.1
100	87.4	17.5	27.6
200	18.9	3.8	23.8

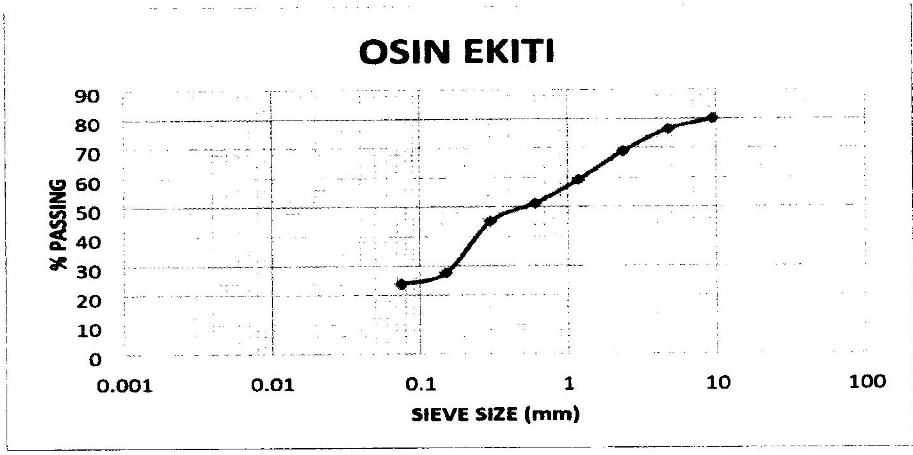


Figure B6: Diagram of particle size distribution for Osin Ekiti

Appendix C

Direct shear

Table C1: Results of direct shear test for Ifaki-Ekiti

Test no	Normal stress applied (kg/cm^2)	Normal force (kN)	Normal strength (kg/m^2)	Max DR DIV	Shear force (kN)	Shear stress (kN/m^2)
1	5	0.04905	13.625	28.00	0.168	46.7
2	10	0.0981	27.25	36.20	0.2172	60.3
3	15	0.14715	40.875	52.80	0.3168	88.0

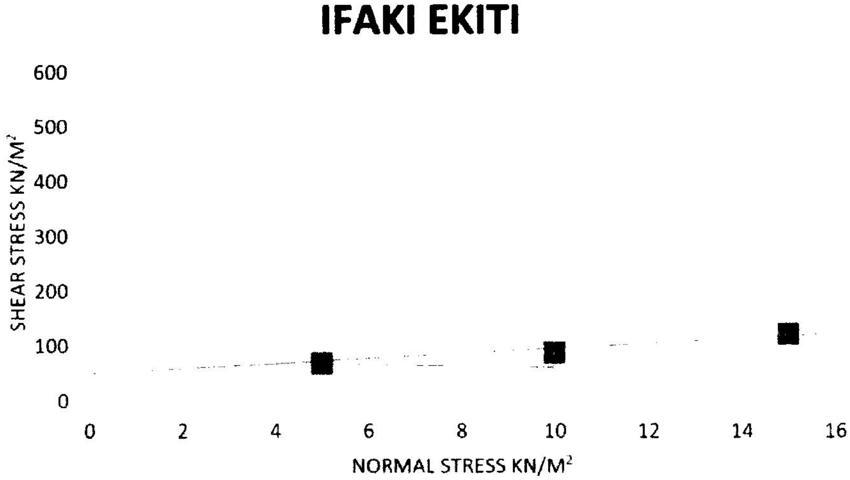


Figure C1: Diagram of direct shear test for Ifaki-Ekiti

AIYEGBAJU-EKITI

Table C2: Results of direct shear test for Aiyegbaju-Ekiti

Test no	Normal stress applied (kg/cm^2)	Normal force (kN)	Normal strength (kg/m^2)	Max DR DIV	Shear force (kN)	Shear stress (kN/m^2)
1	5	0.04905	13.625	39.00	0.168	46.7
2	10	0.0981	27.25	57.00	0.2172	60.3
3	15	0.14715	40.875	75.00	0.3168	88.0

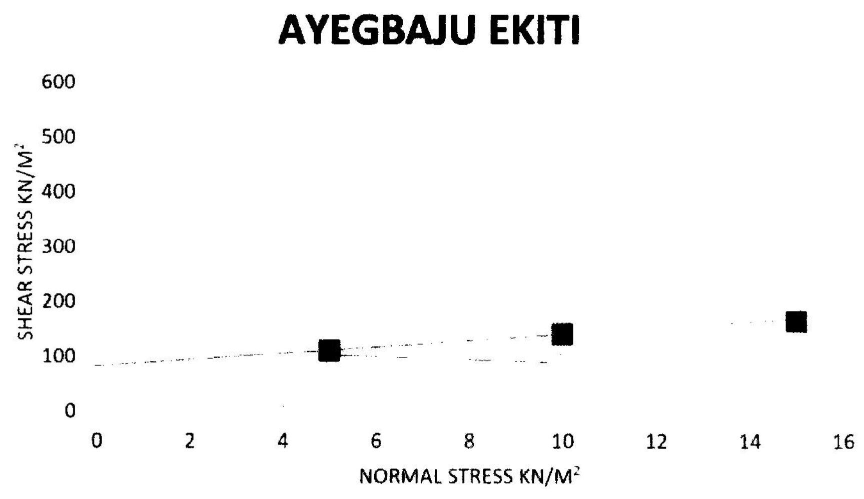


Figure C2: Diagram of direct shear test for Aiyegbaju-Ekiti

ITAPA-EKITI

Table C3: Results of direct shear test for Itapa-Ekiti

Test no	Normal stress applied (kg/cm^2)	Normal force (kN)	Normal strength (kg/m^2)	Max DR DIV	Shear force (kN)	Shear stress (kN/m^2)
1	5	0.04905	13.625	58.00	0.348	96.0
2	10	0.0981	27.25	75.40	0.452	125.6
3	15	0.14715	40.875	97.00	0.582	161.7

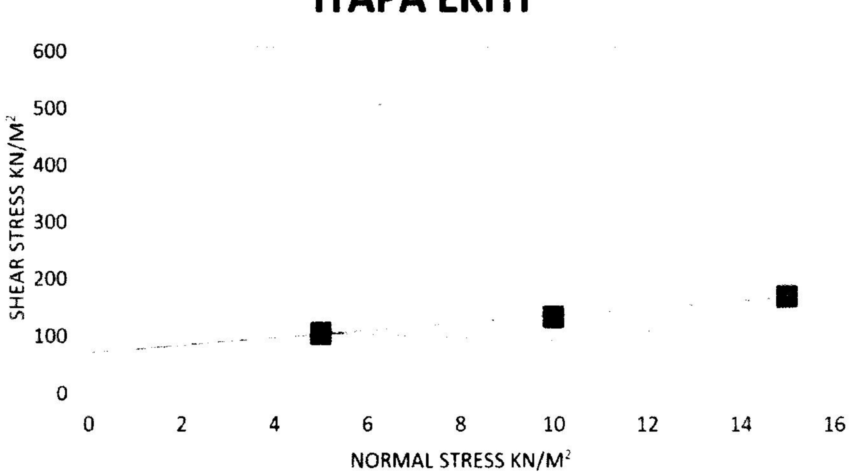


Figure C3: Diagram of direct shear test for Itapa-Ekiti

IKOLE-EKITI

Table C4: Results of direct shear test for Ikole-Ekiti

Test no	Normal stress applied (kg/cm^2)	Normal force (kN)	Normal strength (kg/m^2)	Max DR DIV	Shear force (kN)	Shear stress (kN/m^2)
1	5	0.04905	13.625	36.20	0.2172	60.3
2	10	0.0981	27.25	47.60	0.2856	79.3
3	15	0.14715	40.875	67.80	0.4068	113.0

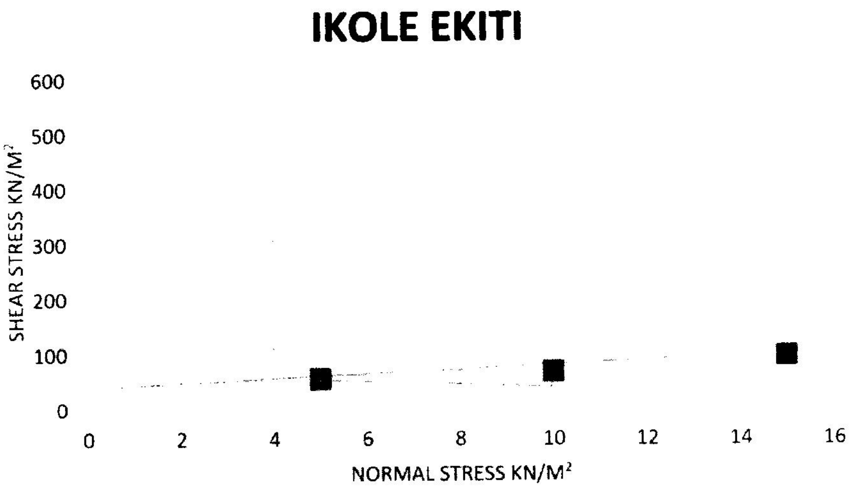


Figure C4: Diagram of direct shear test for Ikole-Ekiti

ILUPEJU-EKITI

Table C5: Results of direct shear test for Ilupeju-Ekiti

Test no	Normal stress applied (kg/cm^2)	Normal force (kN)	Normal strength (kg/m^2)	Max DR DIV	Shear force (kN)	Shear stress (kN/m^2)
1	5	0.04905	13.625	39.80	0.2388	66.3
2	10	0.0981	27.25	59.0	0.354	92.8
3	15	0.14715	40.875	69.4	0.4164	115.6

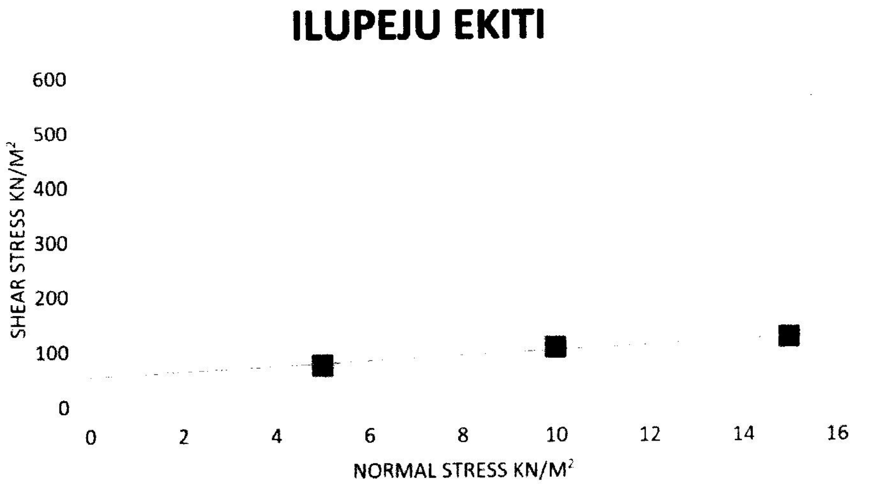


Figure C5: Diagram of direct shear test for Ilupeju-Ekiti

OSIN-EKITI

Table C6: Results of direct shear test for Osin-Ekiti

Test no	Normal stress applied (kg/cm^2)	Normal force (kN)	Normal strength (kg/m^2)	Max DR DIV	Shear force (kN)	Shear stress (kN/m^2)
1	5	0.04905	13.625	43.90	0.2634	73.2
2	10	0.0981	27.25	80.00	0.48	133.3
3	15	0.14715	40.875	99.2	0.5952	165.3

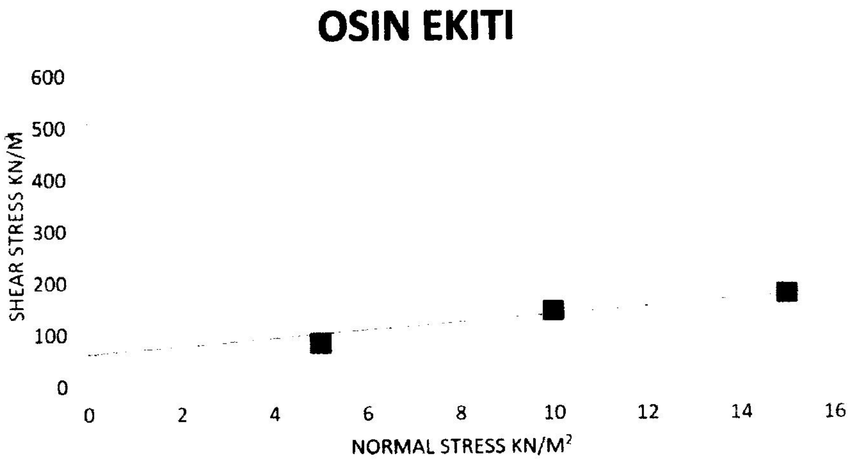


Figure C6: Diagram of direct shear test for Osin-Ekiti

Appendix D

Atterberg's limit

Table D1: Table of results for Aiyegbaju-Ekiti

	Liquid limit				Plastic limit	
NO OF BLOWS	**45**	**35**	**22**	**15**		
CONTAINER NO	**A**	**B**	**C**	**D**	**E**	**F**
Mass of container (g)	19.9	9.3	21.6	10.6	27.0	26.8
Mass of container + wet soil (g)	41.1	35.9	47.8	39.5	44.3	44.0
Mass of container + dry soil(g)	35.9	28.9	40.4	30.8	41.3	41.0
Mass of water (g)	5.2	6.9	7.4	8.7	3.0	3.0
Mass of dry soil (g)	16.0	19.6	18.8	20.2	14.3	14.2
Moisture content %	32.5	35.2	39.4	43.1	20.9	21.1

PL=21.0%

SL=7.1%

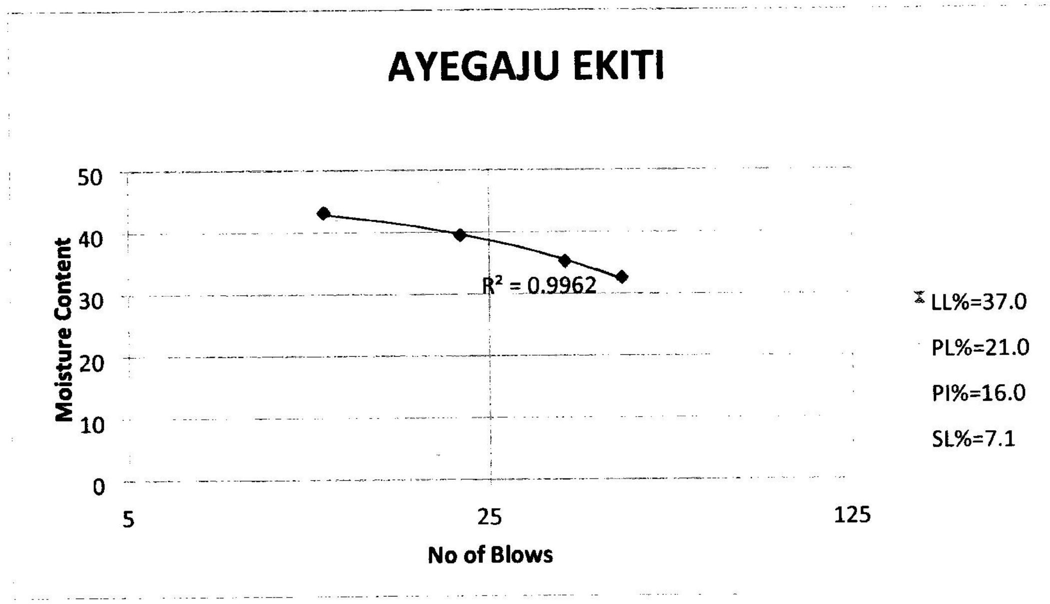


Figure D1: Diagram of results for Aiyegbaju-Ekiti

Table D2: Table of results for Oye-Ekiti

	Liquid limit					
NO OF BLOWS	**49**	**38**	**24**	**15**	**Plastic limit**	
CONTAINER NO	**A1**	**B1**	**C1**	**D1**	**E1**	**F1**
Mass of container (g)	26.7	26.7	17.9	14.9	19.8	26.9
Mass of container + wet soil (g)	47.8	48.1	40.4	40.0	32.2	41.7
Mass of container + dry soil(g)	42.3	42.1	33.9	32.0	30.2	39.3
Mass of water (g)	5.5	6.0	6.9	8.0	2.0	2.4
Mass of dry soil (g)	15.6	15.4	16.0	17.1	10.4	12.4
Moisture content (%)	35.3	39.0	43.1	46.8	19.2	19.4

PL=19.3%

SL=7.9%

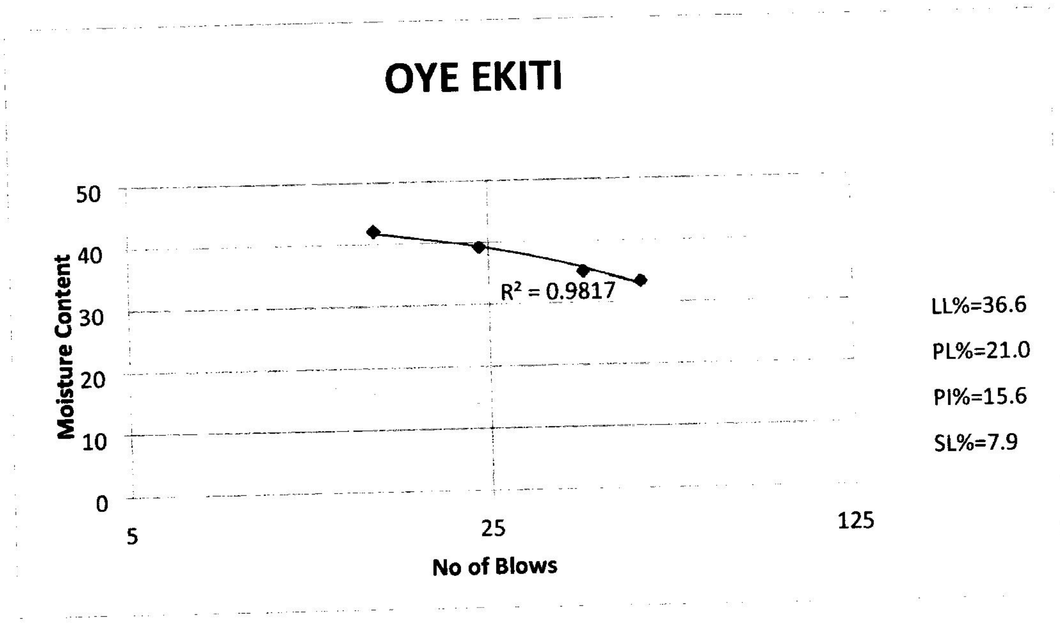


Figure D2: Figure of results for Oye-Ekiti

Table D3: Table of results for Ikole-Ekiti

	Liquid limit				Plastic limit	
NO OF BLOWS	**47**	**35**	**23**	**13**		
CONTAINER NO	**A2**	**B2**	**C2**	**D2**	**E2**	**F2**
Mass of container (g)	20.3	13.7	19.7	21.0	11.6	8.3
Mass of container + wet soil (g)	40.5	35.2	46.8	52.2	26.8	23.9
Mass of container + dry soil(g)	34.1	28.0	37.3	41.0	23.7	20.8
Mass of water (g)	6.4	7.2	9.5	11.2	3.1	3.1
Mass of dry soil (g)	13.8	14.3	17.6	20.0	12.1	12.5
Moisture content (%)	46.4	50.3	54.0	56.0	25.6	24.8

PL=25.2%

SL=11.4%

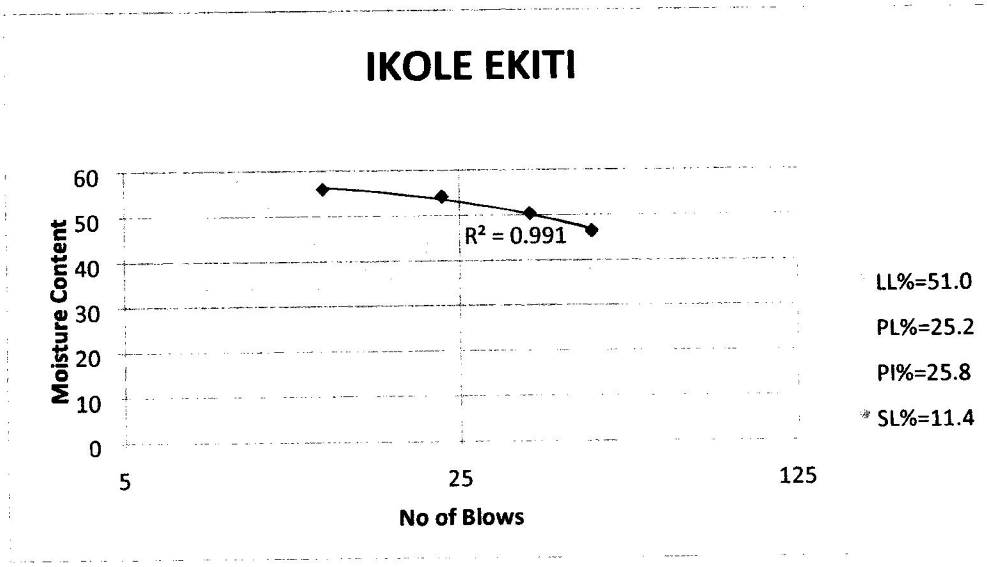


Figure D3: Figure of results for Ikole-Ekiti

Table D4: Table of results for Ifaki Ekiti

	Liquid limit					
NO OF BLOWS	**49**	**38**	**21**	**12**	**Plastic limit**	
CONTAINER NO	**A3**	**B3**	**C3**	**D3**	**E3**	**F3**
Mass of container (g)	21.4	12.2	16.5	21.6	17.8	21.8
Mass of container + wet soil (g)	43.0	36.6	41.6	48.2	31.9	34.3
Mass of container + dry soil(g)	37.4	29.7	33.9	39.6	29.2	31.9
Mass of water (g)	5.6	6.9	7.7	8.6	2.7	2.4
Mass of dry soil (g)	16.0	17.5	17.4	18.0	11.4	10.1
Moisture content (%)	35.0	39.4	44.3	47.8	23.7	23.8

PL=23.8%

SL=9.3%

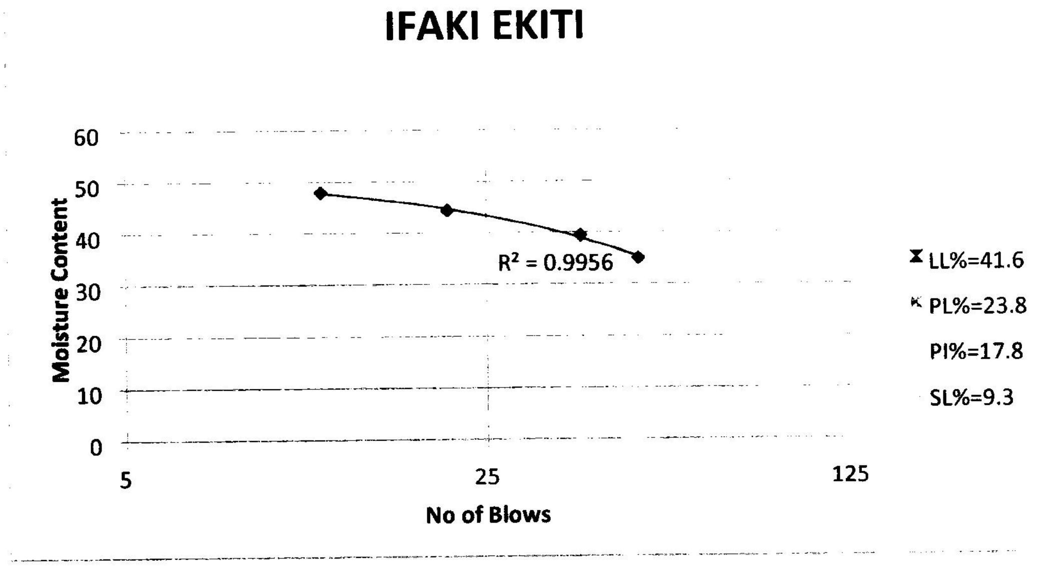


Figure D4: Diagram of results for Ifaki Ekiti

Table D5: Table of results for Osin-Ekiti

	Liquid limit				Plastic limit	
NO OF BLOWS	**46**	**33**	**22**	**12**		
CONTAINER NO	**A4**	**B4**	**C4**	**D4**	**E4**	**F4**
Mass of container (g)	26.3	16.7	26.3	17.1	16.8	11.7
Mass of container + wet soil (g)	46.9	49.2	50.7	43.9	33.9	26.6
Mass of container + dry soil(g)	42.7	44.2	42.8	37.1	31.3	24.4
Mass of water (g)	4.2	5.0	5.9	6.8	2.4	2.2
Mass of dry soil (g)	16.4	17.5	18.5	20.0	14.5	12.7
Moisture content (%)	25.6	28.7	31.9	34.0	16.6	17.3

PL=17.0%

SL=8.7%

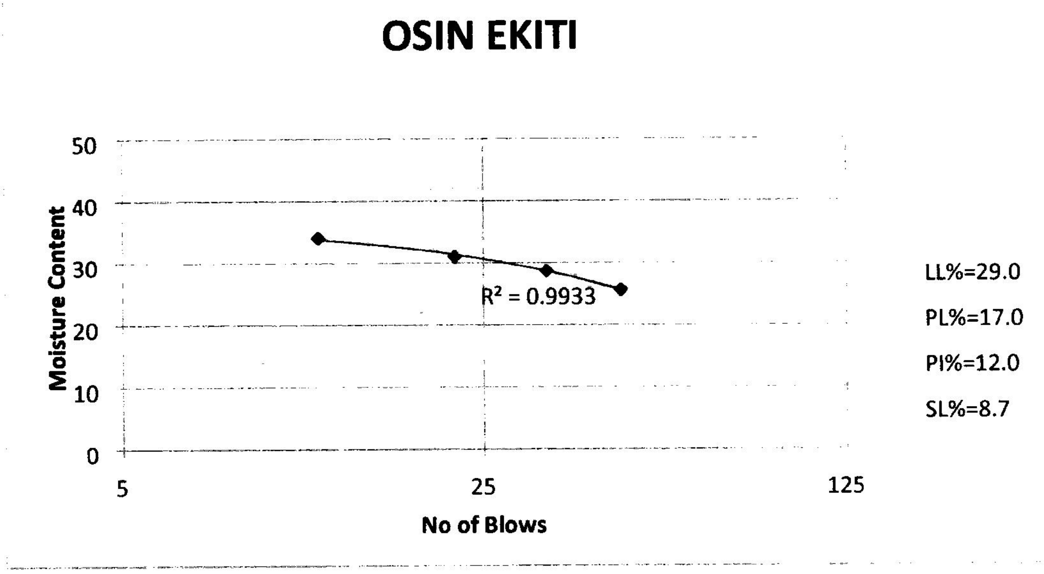


Figure D5: Diagram of results for Osin-Ekiti

Table D6: Table of results for Ilupeju-Ekiti

	Liquid limit				Plastic limit	
NO OF BLOWS	**47**	**33**	**22**	**12**		
CONTAINER NO	**A5**	**B5**	**C5**	**D5**	**E5**	**F5**
Mass of container (g)	21.3	6.7	17.8	13.2	16.9	10.6
Mass of container + wet soil (g)	45.3	33.8	46.4	41.7	29.4	25.3
Mass of container + dry soil(g)	40.0	27.2	38.8	33.5	27.3	22.7
Mass of water (g)	5.3	6.6	7.6	8.2	2.1	2.6
Mass of dry soil (g)	18.7	20.5	21.0	20.3	10.4	12.1
Moisture content (%)	35.3	39.0	43.1	46.8	20.2	21.5

PL=20.9%

SL=7.1%

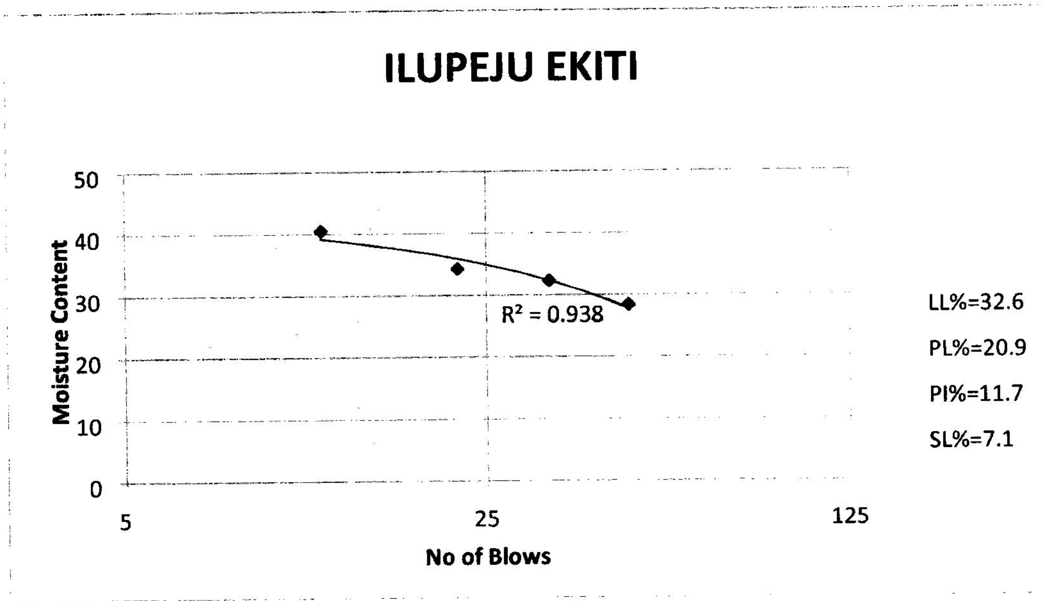


Figure D6: Diagram of results for Ilupeju-Ekiti

Table D7: Table of results for Itapa-Ekiti

	Liquid limit					
NO OF BLOWS	**46**	**35**	**21**	**11**	**Plastic limit**	
CONTAINER NO	**A6**	**B6**	**C6**	**D6**	**E6**	**F6**
Mass of container (g)	26.8	26.5	17.7	13.6	14.0	16.0
Mass of container + wet soil (g)	41.7	45.9	40.5	37.7	26.8	30.1
Mass of container + dry soil(g)	38.8	40.6	33.6	30.5	24.5	27.8
Mass of water (g)	12.0	14.1	15.9	16.9	2.3	2.3
Mass of dry soil (g)	3.9	5.7	6.4	7.2	10.5	11.8
Moisture content (%)	32.5	36.3	40.3	43.4	21.9	19.5

PL=20.7%

SL=8.6%

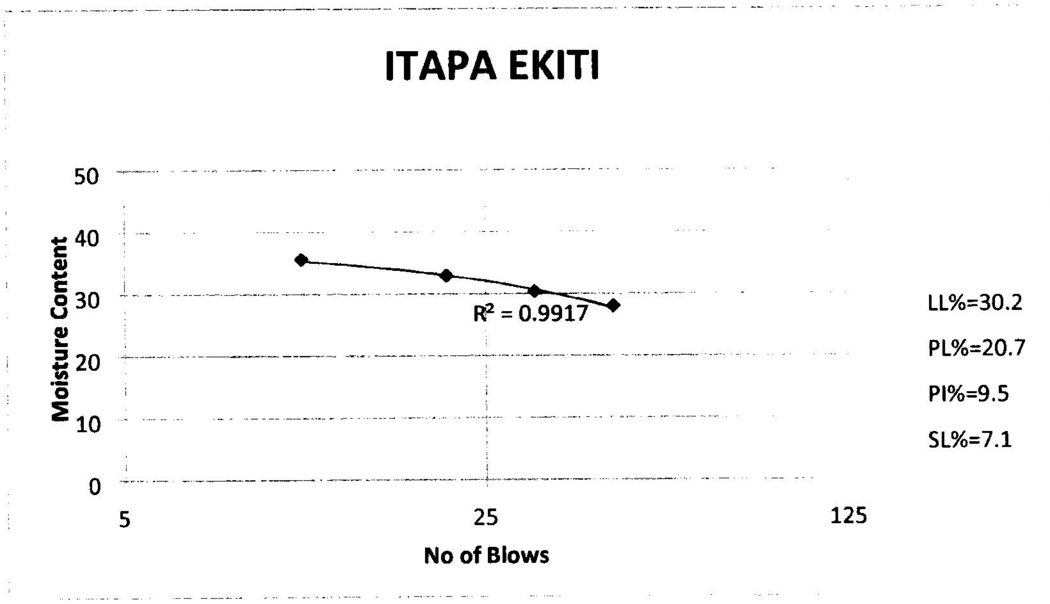


Figure D7: Diagram of results for Itapa-Ekiti

Appendix E

Permeability test

Table E1: Results of permeability test for Itapa-Ekiti

ITAPA-EKITI					AVERAGE TOTAL
TIME	21.8	22.3	22.9	23.2	22.6

Table E2: Results of permeability test for (Oye, Aiyegbaju) Ekiti

OYE-EKITI					AVERAGE TOTAL
TIME	38.5	37.9	38.2	38.2	38.2

AiYEGBAJUEKITI					AVERAGE TOTAL
TIME	22.8	23.2	24.1	23.1	23.3

Table E3: Results of permeability test for Ikole-Ekiti

IKOLE EKII					AVERAGE TOTAL
TIME	59.9	61.8	62.3	61.4	61.4

Table E4: Results of permeability test for Osin-Ekiti

OSIN EKITI					AVERAGE TOTAL
TIME	29.8	30.5	31.2	31.2	30.7

Table E5: Results of permeability test for Ilupeju-Ekiti

ILUPEJU EKITI					**AVERAGE TOTAL**
TIME	36.8	37.5	36.9	37.3	37.1

Table E6: Results of permeability test for Ifaki-Ekiti

IFAKI-EKITI					AVERAGE TOTAL
TIME	45.9	46.1	47.0	46.2	46.3

Appendix F

Compaction test

Table F1: Results of compaction test for Ilupeju-Ekiti

MASS OF MOULD + WET SOIL (g)	5500	5850	6000	5900
MASS OF MOULD	4000	4000	4000	4000
MASS OF COMPACTED SOIL (g)	1550	1850	2000	1900
WET DENSITY (KG/M3)	1.55	1.85	2.00	1.90
CONTAINER NUMBER	A	B	C	D
MASS OF CONTAINER (g)	26.7	23.3	18.1	20.4
MASS OF CONTAINER + WET SOIL (g)	43.4	48.0	44.5	45.1
MASS OF CONTAINER + DRY SOIL (g)	41.9	45.2	40.6	41.0
MASS OF WATER (g)	1.5	2.8	3.9	4.5
MASS OF DRY SOIL (g)	15.2	21.9	22.5	20.6
MOISTURE CONTENT (%)	9.9	13.0	17.3	19.9

DRY DENSITY (kg/M3)	1.41	1.64	1.71	1.58

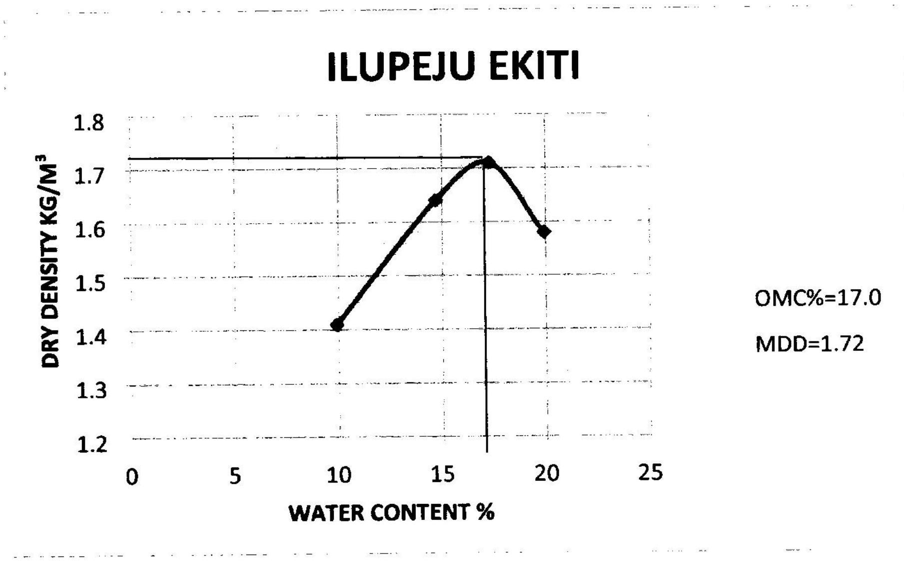


Figure F1: Diagram of compaction test for Ilupeju-Ekiti

Table F2: Results of compaction test for Ikole-Ekiti

MASS OF MOULD + WET SOIL (g)	5450	5600	5750	5700
-MASS OF MOULD	4000	4000	4000	4000
MASS OF COMPACTED SOIL (g)	1450	1600	1750	1700
WET DENSITY (KG/M3)	1.45	1.60	1.75	1.70
CONTAINER NUMBER	A5	B5	C5	D5
MASS OF CONTAINER (g)	26.7	19.6	26.8	19.5
MASS OF CONTAINER + WET SOIL (g)	43.4	45.7	68.8	53.7
MASS OF CONTAINER + DRY SOIL (g)	42.1	42.9	56.1	48.2
MASS OF WATER (g)	1.3	2.8	4.7	5.5
MASS OF DRY SOIL (g)	15.4	23.3	29.3	28.5
MOISTURE CONTENT (%)	8.4	12.0	16L0	19.2

DRY DENSITY (kg/M3)	1.34	1.43	1.51	1.43

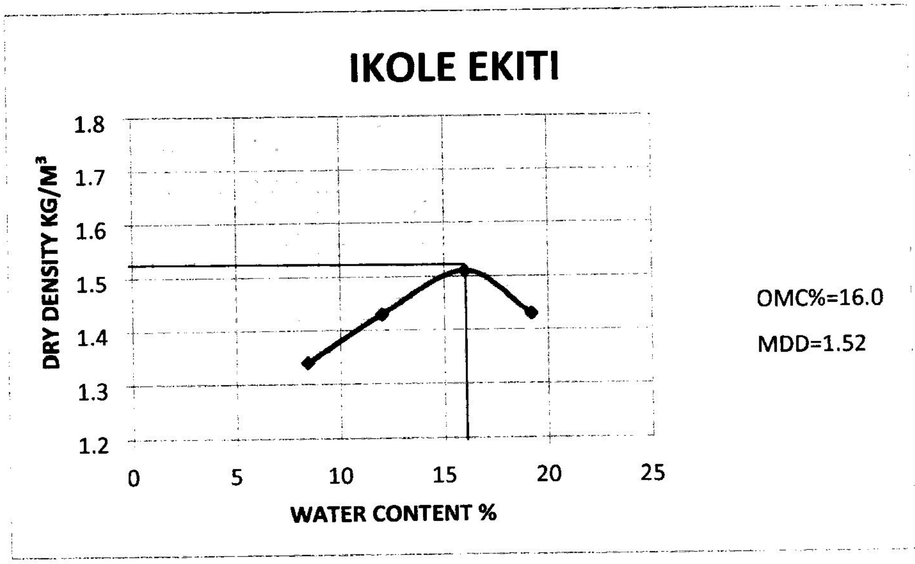


Figure F2: Diagram of compaction test for Ikole-Ekiti

Table F3: Results of compaction test for Aiyegbaju-Ekiti

MASS OF MOULD + WET SOIL (g)	5600	5750	5900	5800
MASS OF MOULD	4000	4000	4000	4000
MASS OF COMPACTED SOIL (g)	1600	1750	1900	1800
WET DENSITY (KG/M3)	1.60	1.75	1.90	1.80
CONTAINER NUMBER	A1	B1	C1	D1
MASS OF CONTAINER (g)	19.8	20.0	19.8	18.9
MASS OF CONTAINER + WET SOIL (g)	70.1	63.7	63.9	61.0
MASS OF CONTAINER + DRY SOIL (g)	65.4	58.3	57.1	53.2
MASS OF WATER (g)	4.7	5.4	6.8	7.8
MASS OF DRY SOIL (g)	45.8	38.3	37.3	34.3
MOISTURE CONTENT (%)	10.3	14.1	18.2	22.2

DRY DENSITY (kg/M3)	1.45	1.53	16.1	1.47

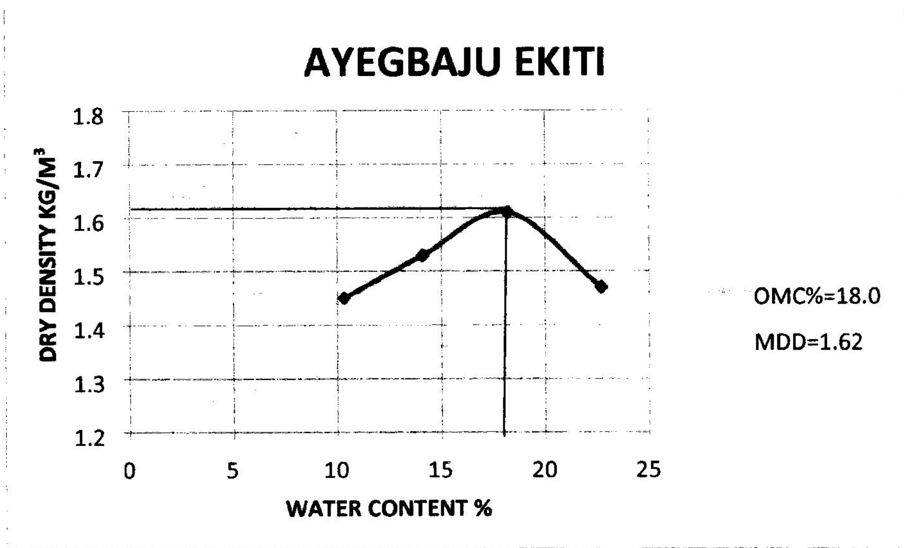


Figure F3: Diagram of compaction test for Aiyegbaju-Ekiti

Table F4: Results of compaction test for Osin-Ekiti

MASS OF MOULD + WET SOIL (g)	5650	5750	5900	5850
MASS OF MOULD	4000	4000	4000	4000
MASS OF COMPACTED SOIL (g)	650	1750	1900	1850
WET DENSITY (KG/M3)	1.65	1.75	1.90	1.85
CONTAINER NUMBER	A2	B2	C2	D2
MASS OF CONTAINER (g)	26.9	10.1	21.7	15.8
MASS OF CONTAINER + WET SOIL (g)	58.0	36.4	66.4	58.1
MASS OF CONTAINER + DRY SOIL (g)	55.6	33.1	60.0	50.8
MASS OF WATER (g)	2.4	3.3	6.4	7.3
MASS OF DRY SOIL (g)	28.7	26.9	38.3	35.0
MOISTURE CONTENT (%)	8.4	12.3	16.7	20.9

DRY DENSITY (kg/M3)	1.50	1.56	1.63	1.53

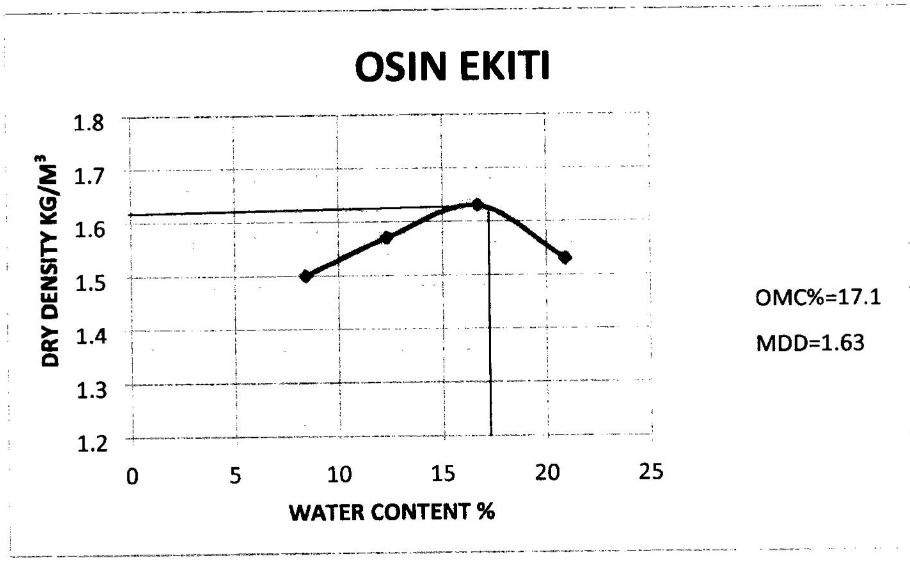


Figure F4: Diagram of compaction test for Osin-Ekiti

Table F5: Results of compaction test for Itapa-Ekiti

MASS OF MOULD + WET SOIL (g)	5600	5800	5950	5900
MASS OF MOULD	4000	4000	4000	4000
MASS OF COMPACTED SOIL (g)	1600	1800	1950	1900
WET DENSITY (KG/M3)	1.60	1.80	1.95	1.90
CONTAINER NUMBER	A3	B3	C3	D3
MASS OF CONTAINER (g)	22.0	15.6	21.1	12.5
MASS OF CONTAINER + WET SOIL (g)	49.4	53.4	57.8	48.8
MASS OF CONTAINER + DRY SOIL (g)	47.2	49.2	52.7 -	42.5
MASS OF WATER (g)	2.2	4.2	5.1	6.3
MASS OF DRY SOIL (g)	25.2	33.6	31.6	30.0
MOISTURE CONTENT (%)	8.7	12.5	16.1	21.0

DRY DENSITY (kg/M3)	1.47	1.60	1.68	1.57

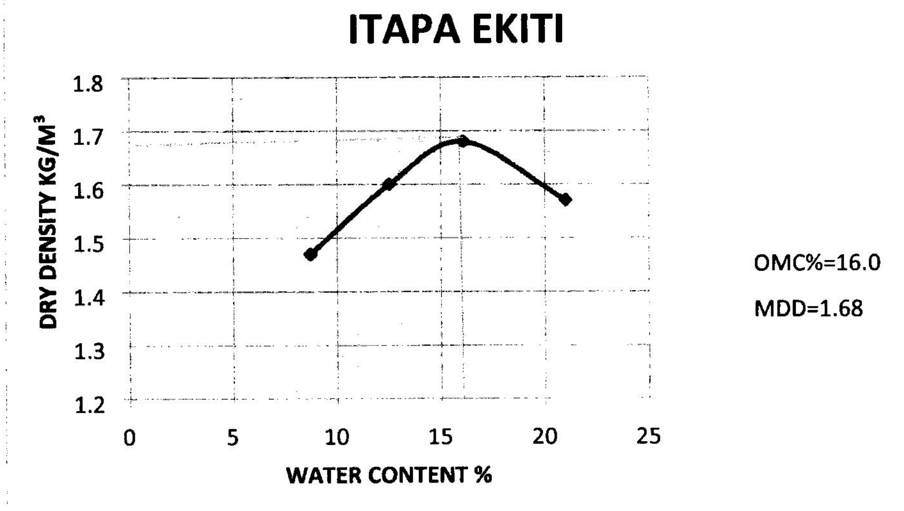


Figure F5: Diagram of compaction test for Itapa-Ekiti

Table F6: Results of compaction test for Ifaki-Ekiti

MASS OF MOULD + WET SOIL (g)	5500	5700	5800	5650
MASS OF MOULD	4000	4000	4000	4000
MASS OF COMPACTED SOIL (g)	1500	1700	1800	1650
WET DENSITY (KG/M3)	1.50	1.70	1.80	1.65
CONTAINER NUMBER	A4	B4	C4	D4
MASS OF CONTAINER (g)	10.1	20.7	28.9	13.5
MASS OF CONTAINER + WET SOIL (g)	34.3	46.7	63.2	42.5
MASS OF CONTAINER + DRY SOIL (g)	32.1	43.2	56.7	36.0
MASS OF WATER (g)	2.3	3.5	5.5	6.3
MASS OF DRY SOIL (g)	22.0	22.5	28.8	28.0
MOISTURE CONTENT (%)	10.5	15.6	19.1	22.5

DRY DENSITY (kg/M3)	1.36	1.47	1.51	1.35

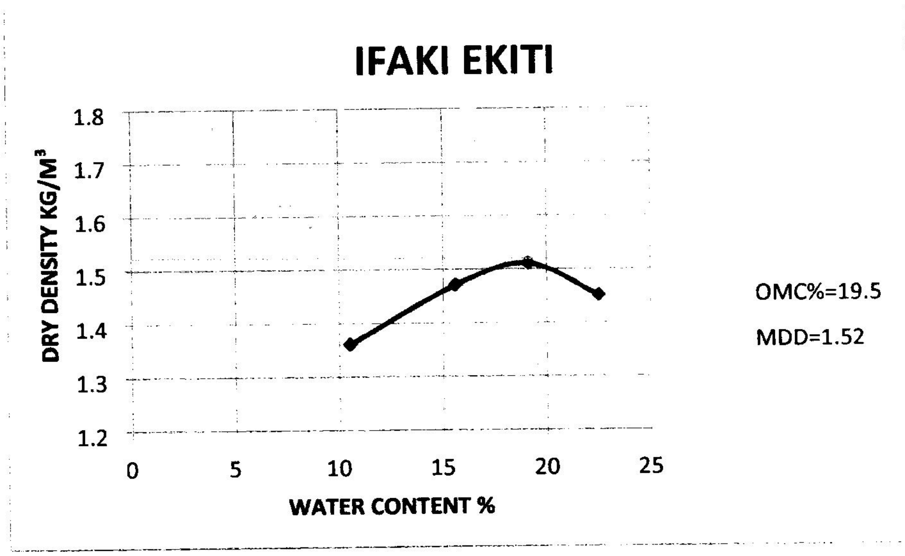


Figure F6: Diagram of compaction test for Ifaki-Ekiti

Table F7: Results of compaction test for Oye-Ekiti

MASS OF MOULD + WET SOIL (g)	5500	5600	5750	5650
MASS OF MOULD	4000	4000	4000	4000
MASS OF COMPACTED SOIL (g)	1500	1600	1750	1650
WET DENSITY (KG/M3)	1.50	1.60	1.75	1.60
CONTAINER NUMBER	A5	B5	C5	D5
MASS OF CONTAINER (g)	26.9	26.2	20.2	19.8
MASS OF CONTAINER + WET SOIL (g)	46.4	55.2	50.8	55.1
MASS OF CONTAINER + DRY SOIL (g)	45.1	51.4	46.5	47.8
MASS OF WATER (g)	1.8	3.8	4.3	5.5
MASS OF DRY SOIL (g)	18.2	29.0	26.3	28.0
MOISTURE CONTENT (%)	9.9	13.1	16.3	19.6

DRY DENSITY (kg/M3)	1.36	1.41	1.50	1.38

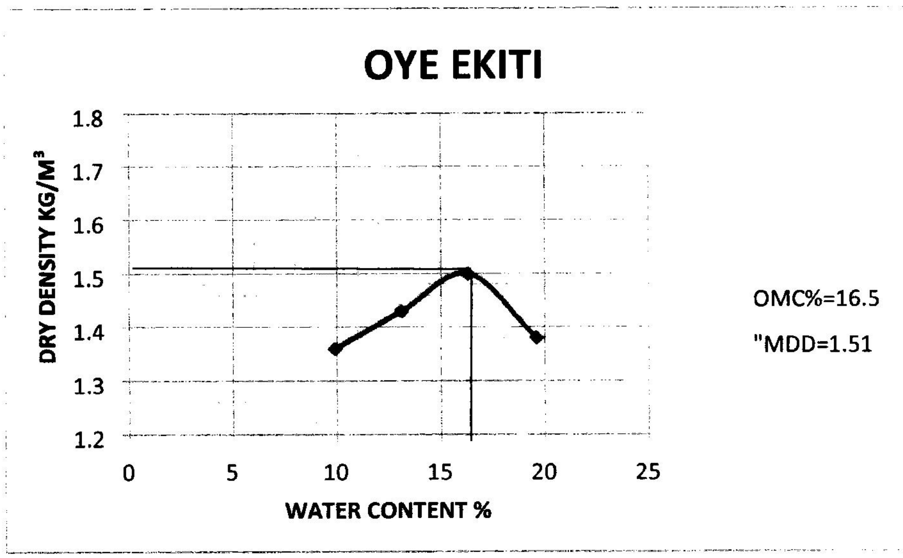


Figure F7: Diagram of compaction test for Oye-Ekiti

Appendix G

California bearing ratio

Table G1: Results of California bearing ratio of Itapa-Ekiti

NUMBER OF BLOWS	DIAL READING	LOAD APPLIED
50	10	0.25
100	38	0.95
150	66	1.65
200	98	2.45
250	134	3.35
300	175	4.38
350	220	5.50
400	289	7.23
450	315	7.88
500	358	8.89
550	379	9.48
600	403	10.08
650	438	11.00
700	452	11.30
750	467	11.68

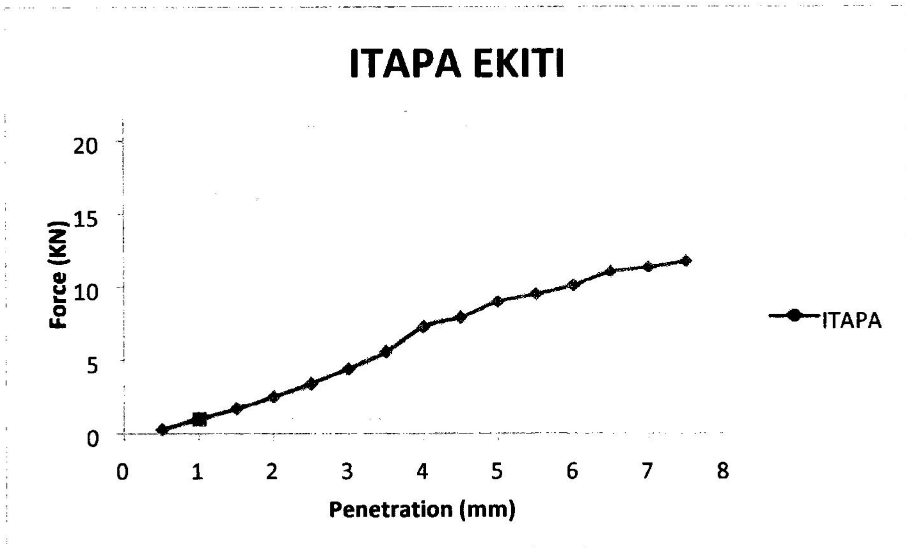


Figure G1: Diagram of California bearing ratio of Itapa-Ekiti

Table G2: Results of California bearing ratio of Aiyegbaju-Ekiti

NUMBER OF BLOWS	DIAL READING	LOAD APPLIED
50	87	2.18
100	121	3.03
150	169	4.23
200	225	5.63
250	279	6.98
300	326	3.15
350	387	9.68
400	423	10.58
450	468	11.17
500	507	12.68
550	534	13.35
600	571	14.28
650	600	15.00
700	623	15.58
750	634	15.85

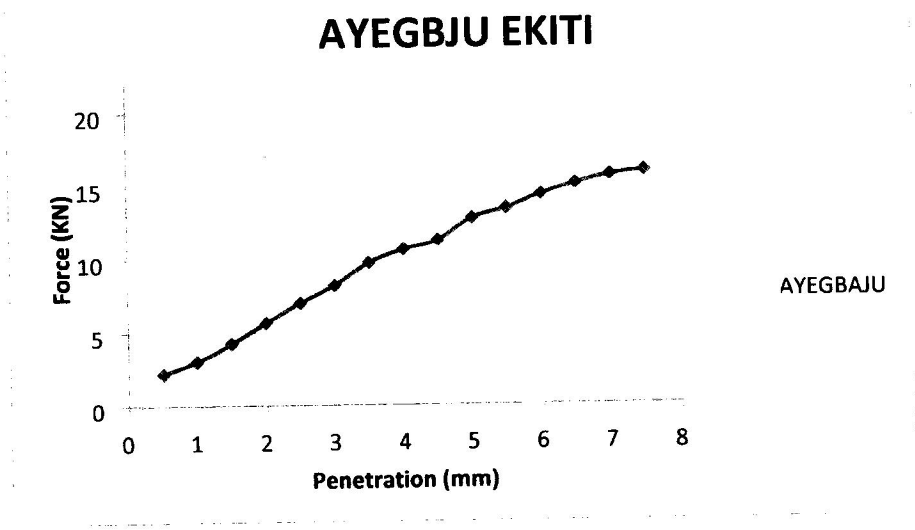


Figure G2: Diagram of California bearing ratio of Aiyegbaju-Ekiti

Table G3: Results of California bearing ratio of Iluepju-Ekiti

NUMBER OF BLOWS	DIAL READING	LOAD APPLIED
50	214	5.35
100	249	6.25
150	297	7.43
200	333	8.33
250	372	9.30
300	426	10.65
350	478	11.95
400	521	13.03
450	603	15.08
500	639	16.0
550	652	16.3
600	681	17.03
650	701	17.53
700	718	17.95
750	727	18.18

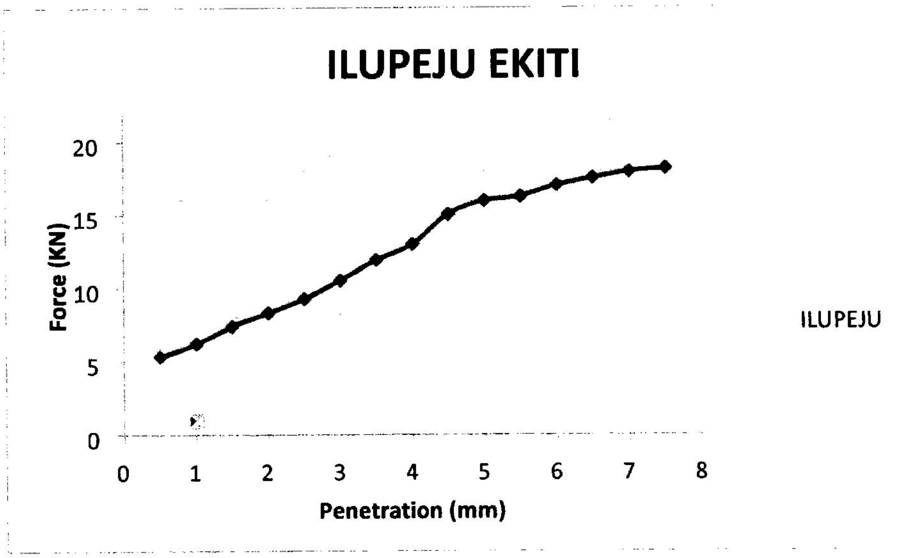


Figure G3: Diagram of California bearing ratio of Iluepju-Ekiti

Table G4: Results of California bearing ratio of Oye-Ekiti

NUMBER OF BLOWS	DIAL READING	LOAD APPLIED
50	16	0.4
100	35	0.9
150	66	1.65
200	86	2.15
250	103	2.58
300	120	3.00
350	148	3.70
400	174	4.35
450	202	5.05
500	234	5.85
550	251	6.28
600	269	6.73
650	288	7.20
700	301	753
750	323	8.08

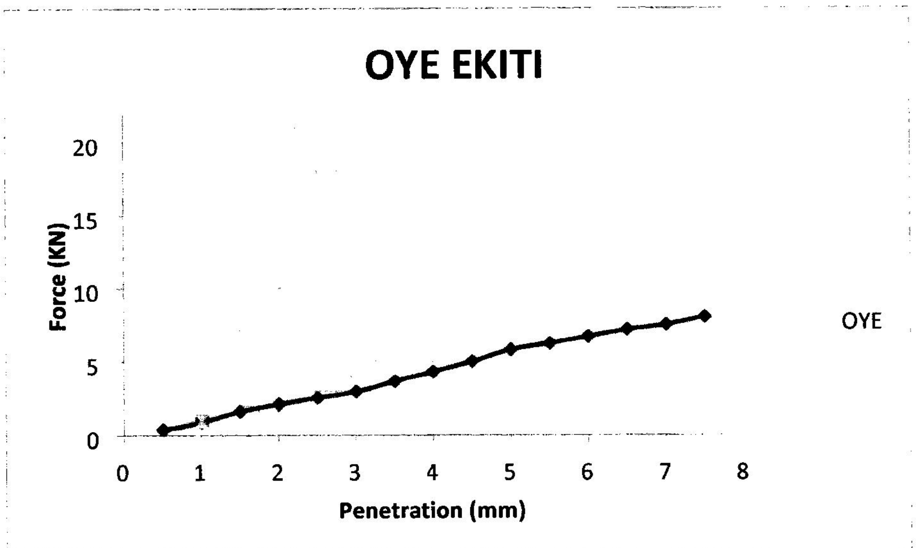


Figure G4: Diagram of California bearing ratio of Oye-Ekiti

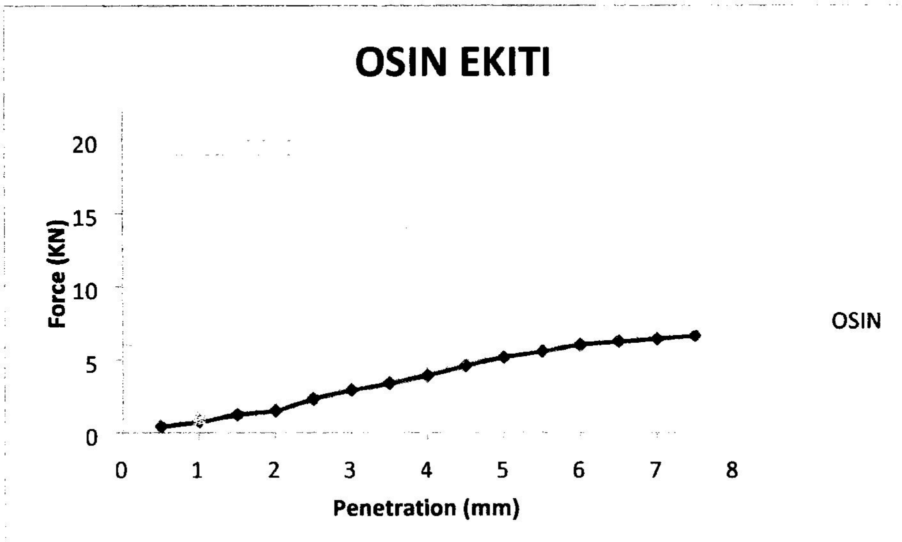


Figure G5: Diagram of California bearing ratio of Osin-Ekiti

Table G6: Results of California bearing ratio of Ikole-Ekiti

NUMBER OF BLOWS	DIAL READING	LOAD APPLIED
50	8	0.20
100	23	0.58
150	43	1.08
200	61	1.53
250	72	1.80
300	89	2.23
350	101	2.53
400	109	2.73
450	114	2.58
500	122	3.05
550	126	3.15
600	131	3.28
650	134	3.35
700	137	3.43
750	141	3.53

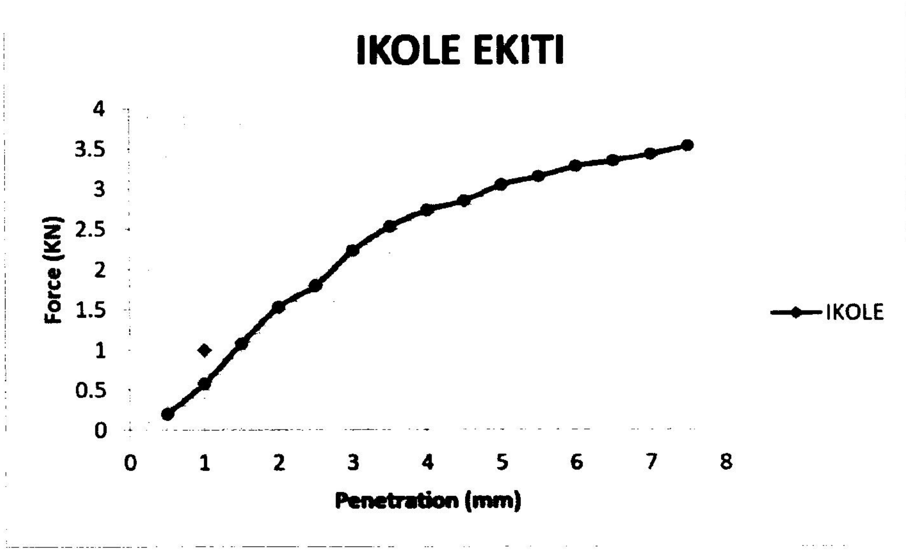


Figure G6: Diagram of California bearing ratio of Ikole-Ekiti

Table G7: Results of California bearing ratio of Ifaki-Ekiti

NUMBER OF BLOWS	DIAL READING	LOAD APPLIED
50-	13	0.33
100	31	0.78
150	70	1.75
200	87	2.08
250	108	2.70
300	124	3.110
350	139	3.48
400	150	3.75
450	167	4.18
500	175	4.68
550	184	4.60
600	194	4.85
650	201	5.03
700	208	5.20
750	218	5.45

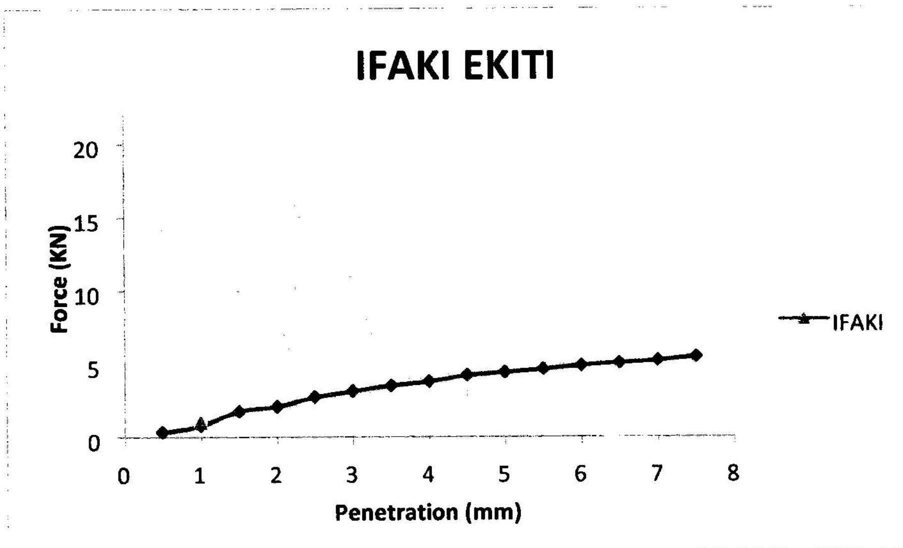


Figure G7: Diagram of California bearing ratio of Ifaki-Ekiti

COEFFICIENT OF PERMEABILITY (Falling Head)

Locaton IFAKI Sample No G

Soil Description: BROWNISH

Date:

Sample Dimensions: Diam. 10.2 cm;

Area,A 81.75

Vol. 915.55

Ht.L 13

Stand Pipe Burette 50.1 ml Diam. 10.2 cm Area,a

Test no.	h_1 ,cm	h_2 ,cm	t, s	Q_{in}, cm^3	Q_{out}, cm^4	T, °C	Test no.
1	100	75	45.9		44.50	22	1
2	100	75	46.1		44.50	22	2
3	100	75	47,0		44.50	22	3
4	100	75	46.2		44.50	22	4
						Average	

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

= 0.9761

kT = (aL/At)ln

(h_1/h_2) = 1.12E-03 =

$k_{20} = \alpha k_T$ = 1.10E-03 =

Degree of Permeability: Medium

COEFFICIENT OF PERMEABILITY (Falling Head)

Location ILUPEJU EKITI   Sample D

Soil Description: LIGHT RED

Date:

Sample Dimensions: Diam. 10 cm Area,A 85.10

Vol.   1106.30

Ht.L   13

Standpipe =   Burette 50 ml Diam. 10.2 cm   Area,a

Test no.	h_1 ,cm	h_2 ,cm	t, s	Q_{in}, cm^3	Q_{out}, cm^4	T, °C	Test no.
1	100	75	36.8		44.50	22	1
2	100	75	37.5		44.50	22	2
3	100	75	36.9		44.50	22	3
4	100	75	37.3		44.50	22	4
						Average	

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

kT = (aL/At)ln

1.05E-03   =

(h_1/h_2) =

$k_{20} = \alpha k_T =$ 9.95E-04   =

Degree of Permeability: Low

COEFFICIENT OF PERMEABILITY (Falling Head)

Location OSIN EKITI   Sample No B

Soil Description: LIGHT DARK BROWN

Date:

Sample Dimensions: Diam. 10.2 cm; Area,A 85.10

Vol.   1106.30

Ht.L   13

Standpipe =   Burette 50 ml Diam. 10.2 cm   Area,a

Test no.	h_1 ,cm	h_2 ,cm	t, s	Qin, cm3	Qout, cm4	T, °C	Test no.
1	100	75	29.8		44.50	21	1
2	100	75	30.5		44.50	21	2
3 100 75 31.2	44.50	21			3		

4	100	75	31.2		44.50	21	4
						Average	

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

kT = (aL/At)ln

(h_1/h_2) $=$ 1.27E-03   =

$k_{20} = \alpha k_T =$ 1.24E-03   =

Degree of Permeability: Medium

COEFFICIENT OF PERMEABILITY (Falling Head)

Location: IKOLE -EKITI Sample No A

Soil Description: Light brown

Sample Dimensions: Diam. 10.2 cm; Area,A 85.10

Vol. 915.55

Ht.L 13

Standpipe = Burette 50 ml Diam. 10.2 cm Area,a

Test no.	h_1 ,cm	h_2 ,cm	t, s	Q_{in}, cm^3	Q_{out}, cm^4	T, °C	Test no.
1	100	75	59.9		45.00	23	1
2	100	75	61.8		45.00	23	2
3	100	75	62.3		45.00	23	3
4	100	75	61.4		45.00	23	4
						Average	

$\alpha = \eta T/\eta 2=$ 0.9267

kT = (aL/At)ln 6.34E-04 =

(h /h) = 6.34E-04 =

(h_1/h_2) =

$k_{20} = \alpha k_T =$ 5.87E-04 =

Degree of Permeability: Low

COEFFICIENT OF PERMEABILITY (Falling Head)

Project: AYEGBAJU EKITI Sample No F

Soil Description: DARK RED

Tested by: Date:

Sample Dimensions: Diam. 10.2 cm; Area,A 81.75

Vol. 915.55

Ht.L 11.2

Standpipe = Burette (50 ml) Diam. 1.06 cm Area,a

Test no.	h_1 ,cm	h_2 ,cm	t, s	Q_{in}, cm^3	Q_{out}, cm^4	T, °C	Test no.
1	100	75	22,8		22.07	26	1
-	100	75	23 2		22.07	26	2
3	100	75	24.1		22.07	26	3
4	100	75	23.1		22.07	26	4
						Average	

$\alpha = \eta_T/\eta_{20} =$ 0.8642

kT = (aL/At)ln

(h_1/h_2) = 1.47E-03 =

$k_{20} = \alpha k_T =$ 1.27E-03 =

Degree of Permeability: Medium

COEFFICIENT OF PERMEABILITY (Falling Head)

Project: OYE EKITI Sample No E

Soil Description:

Sample Dimensions: Diam. 10.2 cm; Area, A 81.75

Dat

e:

Vol. 915.55

Ht.L 13

Stand pipe 1.0cm Burette (50 ml) Diam. 1.06 cm Area,a

Test no.	h_1 ,cm	h_2 ,cm	t, s	Q_{in}, cm^3	Q_{out}, cm^4	T, °C	Test no.
1	100	75	38 5		4.41	26	1
2	100	75	37.9		22.07	26	2
3	100	75	38.2		22.07	26	3
4	100	75	38.3		22.07	26	4
						Average	

$\alpha = \eta_T/\eta_{20} =$ 0.8642

kT = (aL/At)ln

1.06E-03 =

(h_1/h_2) =

$k_{20} = \alpha k_T =$ 9.15E-04 =

Degree of Permeability: Medium

Appendix I

Specific gravity

$$\frac{W2-W1}{(W4-W1)-(W3-W2)}$$

Table I1: Results of specific gravity of Osin-Ekiti

	A	B
W1	26.4	26.3
W2	54.8	55.1
W3	96.9	96.7
W4	79.8	79.7

Table I2: Results of specific gravity of Oye-Ekiti

	A	B
W1	24.2	24.1
W2	53.0	54.1
W3	95.8	96.0
W4	78.4	79

Table I3: Results of specific gravity of Ilupeju-Ekiti

	A	B
W1	26.0	26.0
W2	52.6	53.2
W3	94.6	95.1
W4		
	78.6	78.7

Table I4: Results of specific gravity of Aiyegbaju-Ekiti

	A	B
W1	25.6	25.6
W2	52.4	53.0
W3	94.8	95.3
W4	78.3	79.0

Table I5: Results of specific gravity of Ifaki-Ekiti

	A	B
W1	25.9	25.5
W2	49.8	50.1
W3	89.2	90.1
W4	75.0	75.0

Table I6: Results of specific gravity of Ikole-Ekiti

	A	B
W1	43.3	42.9
W2	87.8	88
W3	187.5	188.1
W4	161.1	160.4

Table I7: Summary of results of specific gravity

S/N	SOIL SAMPLES	AVERAGE SPECIFIC GRAVITY VALUES
1	A	2.53
2	B	2.38
3	C	2.42
4	D	2.52
5	E	2.42
6	F	2.51
7	G	2.51