

**STUDY OF THE SUBGRADE FAILURE ON THE HIGHWAY PAVEMENT ALONG
ENUGU-ONITSHA EXPRESSWAY AT GARIKI LIVESTOCK MARKET AMANSEA**

BY

DOMINIC OBINNA GEORGE

2017224006

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CERTIFICATION

This is to certify that the research project on “EXAMINATION OF THE SUBGRADE FAILURE ON THE HIGHWAY PAVEMENT ALONG ENUGU-ONITSHA EXPRESSWAY AT GARIKI LIVESTOCK MARKET AMANSEA” was carried out by Dominic Obinna George with registration number 2017224006 of the Department of Civil Engineering in partial fulfilment of the requirement for the award of Bachelor’s Degree in Civil Engineering, Nnamdi Azikiwe University Awka under the close supervision of Engr. Dr. P. D. Onodagu of Department of Civil Engineering, Nnamdi Azikiwe University, Awka. This work has never been submitted either in part or in full for any degree in any university.

Dominic Obinna George
(Student)

Date

APPROVAL PAGE

This research thesis on “Examination of the subgrade failure on the highway pavement along Enugu-Onitsha expressway at Gariki livestock market Amansea” was carried out by Dominic Obinna George with registration number 2017224006 has satisfied all the requirements of this university for the award of Bachelor’s degree (B.Eng) in Civil Engineering, Nnamdi Azikiwe University, Awka.

Engr. Dr. P. D. Onodagu
(Project Supervisor)

Date

Engr. Prof. C. A. Ezeagu
(Head of Department)

Date

Engr. Prof. D. O. Onwuka
(External Supervisor)

Date

DEDICATION

This research project is wholeheartedly dedicated to God Almighty for his grace and guidance through my course of study and the completion of this project work and also to my parents, siblings, lecturers and well wishers that have contributed immensely to the successful completion of this project.

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ABSTRACT

This research addresses a comparative study and examination of the subgrade failure on the Highway along Enugu-Onitsha Expressway at Gariki Livestock Market Anambra. A highway pavement is composed of a system of overlaid strata of chosen processed materials that is positioned on the in-situ soil, termed the subgrade. The load-bearing capacity of a pavement depends chiefly on the subgrade features especially its strength. The major importance of this study is the resolution of the environmental impacts due to the pavement failure such as soil erosion and extreme traffic congestion. In this project, three locations were selected for sampling of the soil. Particle size distribution test was carried out on the soil samples to give an insight on the nature of the soil. Atterberg limit test was carried out to determine the plasticity of the soil from the clay content. The compaction test and the California Bearing Ratio (CBR) test was also carried out to determine the maximum dry density and load-bearing capacity of the subgrade soil. The sieve analysis test classified the soil as a poorly graded subgrade soil with a uniformity coefficient of 3.29 and a coefficient of curvature of 0.94. The Atterberg limit test which gave the plasticity index of 7.65% indicated the soil is of a moderate clay content and will therefore exhibit some plastic behaviour. With ranges between 2008–2041(kg/m³) of MDD and a range between 7.8% - 10.2% of OMC, the compaction test indicated that the soil can be effectively compacted to achieve a desirable density to enhance the strength and stability of the subgrade soil. The CBR test with an average value of 10% indicated a relatively low strength and load-bearing capacity of the subgrade soil. The results reveal the high degree of distress on the subgrade exposing some defects on the pavement like edge cracks and potholes. It was recommended that further investigation is required to give more insight into the soils behaviour and in adverse case additional measures such as soil stabilization, geosynthetic reinforcement or subgrade modification technique should be considered to improve the soils load-bearing capacity.

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CHAPTER ONE

INTRODUCTION

1.1 Background to Study

A highway pavement is composed of a system of overlaid strata of chosen processed materials that is positioned on the in-situ soil, termed the subgrade. Its basic requirement is the provision of a uniform skid-resistant running surface with adequate life and requiring minimum maintenance. The chief structural purpose of the pavement is the support of vehicle wheel loads applied to the carriageway and the distribution of them to the subgrade immediately underneath. If the road is in cut, the subgrade will consist of the in-situ soil. If it is constructed on fill, the top layers of the embankment structure are collectively termed the subgrade.

There are three basic components of a highway pavement;

1. Foundation
2. Roadbase
3. Surfacing

1.1.1 Foundation: the foundation consists of the native subgrade soil and the layer of graded stone (subbase and possibly capping) immediately overlaying it. The function of the subbase and capping is to provide a platform on which to place the roadbase material as well as to insulate the subgrade below it against the effects of inclement weather. These layers may form the temporary road surface used during the construction phase of the highway.

1.1.2 Roadbase

The roadbase is the main structural layer whose main function is to withstand the applied wheel stresses and strains incident on it and distribute them in such a manner that the materials beneath it do not become overloaded.

1.1.3 Surfacing

The surfacing combines good riding quality with adequate skidding resistance, while also minimising the probability of water infiltrating the pavement with consequent surface cracks. Texture and durability are vital requirements of a good pavement surface as are surface regularity and flexibility. For flexible pavements, the surfacing is normally applied in two layers – basecourse and wearing course – with the basecourse an extension of the roadbase layer but providing a regulating course on which the final layer is applied. In the case of rigid pavements, the structural function of both the roadbase and surfacing layers are integrated within the concrete slab. In broad terms, the two main pavement types can be described briefly as:

- I. **Flexible Pavements:** The surfacing and roadbase materials, bound with bitumen binder, overlay granular unbound or cement-bound material. Flexible pavements are called “flexible” since the total pavement structure “bends” or “deflects” due to traffic loads. A flexible pavement structure is generally composed of several layers of material to accommodate this “flexing” effect. The purpose of pavement is use for load support where flexible pavement uses more flexible surface course and distributes loads over a smaller contributing area. It relies on a combination of layers for transmitting load to the sub grade. Flexible pavements generally require some sort of maintenance or rehabilitation every 10 to 15 years.
- II. **Rigid Pavements:** Pavement quality concrete, used for the combined surfacing and roadbase, overlays granular cement-bound material. The concrete may be reinforced with steel.

The general layout of these two pavement types are shown in Fig. 1.0 and 1.1. Pavements are thus composed of several layers of material. They can consist of one or more bitumen or cement-bound

layers overlaying one or more layers of unbound granular material which in turn is laid on the in-situ soil (if the highway is in cut) or imported soil/granular material (if the highway is constructed in fill) which exists below formation level (HD 23/99) (DoT, 1999).

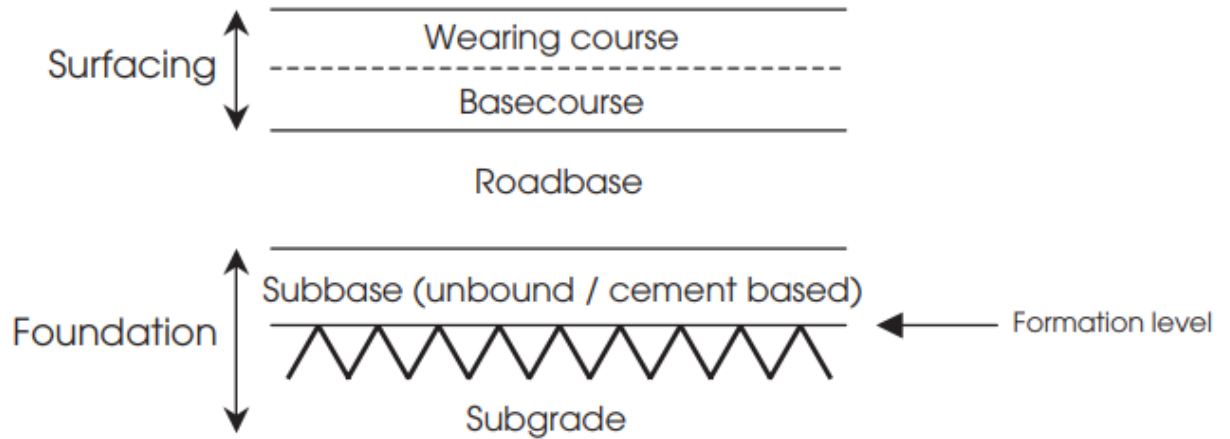


Fig 1.0 Layers within a typical flexible highway pavement

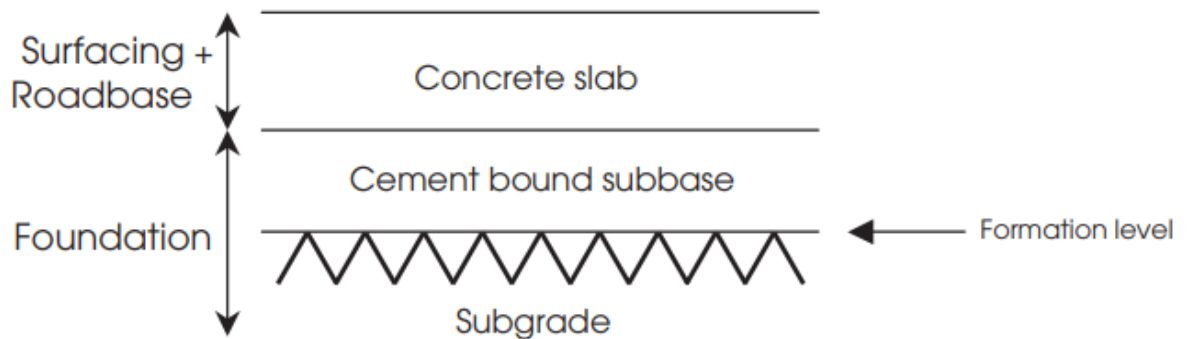


Fig 1.1 Layers within a typical rigid highway pavement

1.2 Statement of Problem

Continuos failure of the highway pavement at Gariki Livestock market along Onitsha-Enugu expressway, resulting to a hull of frequent traffic jam situated around the area. A pavement failure could be caused by a lot of factors having that it possesses different layer of well compacted

materials. One of the root cause which is the main focus of this project are features of the subgrade; the load bearing capacity, the moisture content and nature/type of the soil.

1.3 Aims And Objectives

The aim of this project is to examine the subgrade failure on the highway along Enugu – Onitsha expressway at Gariki Livestock Market, Amansea.

The specific objective of this study is:

- i. The collection of samples from the site to determine the type and nature alongside the load bearing capacity and moisture content of the subgrade soil.
- ii. The carrying out labouratory investigation on the cause of the consistent failure of the pavement.
- iii. To subject the laboratory results to certain codes of practice and direct a solution on preventing subsequent failure of the pavement if the subgrade posses as the root cause.

1.4 Significance of the Study

- i. This study gives an insight on the type of the soil, its nature, moisture content and the load bearing capacity of the subgrade soil.
- ii. The study reveals the cause of the continuous failure of the highway pavement as attributed to the subgrade features.
- iii. Understanding the causes of pavement failure can help engineers design pavements that are better suited to the underlying soil conditions. By considering the characteristics of the subgrade, such as its strength and stiffness, engineers can design pavements that are more resilient and better able to withstand the effects of traffic loads and environmental factors.

- iv. The major importance of this study is the resolution of the environmental impacts due to the pavement failure such as soil erosion and extreme traffic congestion. By designing pavements that are better suited to the underlying soil conditions, these impacts can be reduced and also make room for promotion of sustainable infrastructural development.

1.5 Scope of Study

This study is limited to the subgrade features; the type and nature of the soil and the load bearing capacity of the soil, of the intersection point at Gariki livestock market as it relates to the continuous failure of the highway pavement.

CHAPTER TWO
LITERATURE REVIEW

2.1 Flexible Pavement

2.1.1 Flexible Pavement Structure

Construction of Flexible Pavement by Riyanto (1996) was pavement using asphalt as a surface. The surface layers of the pavement carry and publish the traffic load to subgrade. A flexible pavement construction according to Sukirman (1999) consists of four parts, namely:

- i. Surface course
- ii. Base course
- iii. Subbase course
- iv. Subgrade

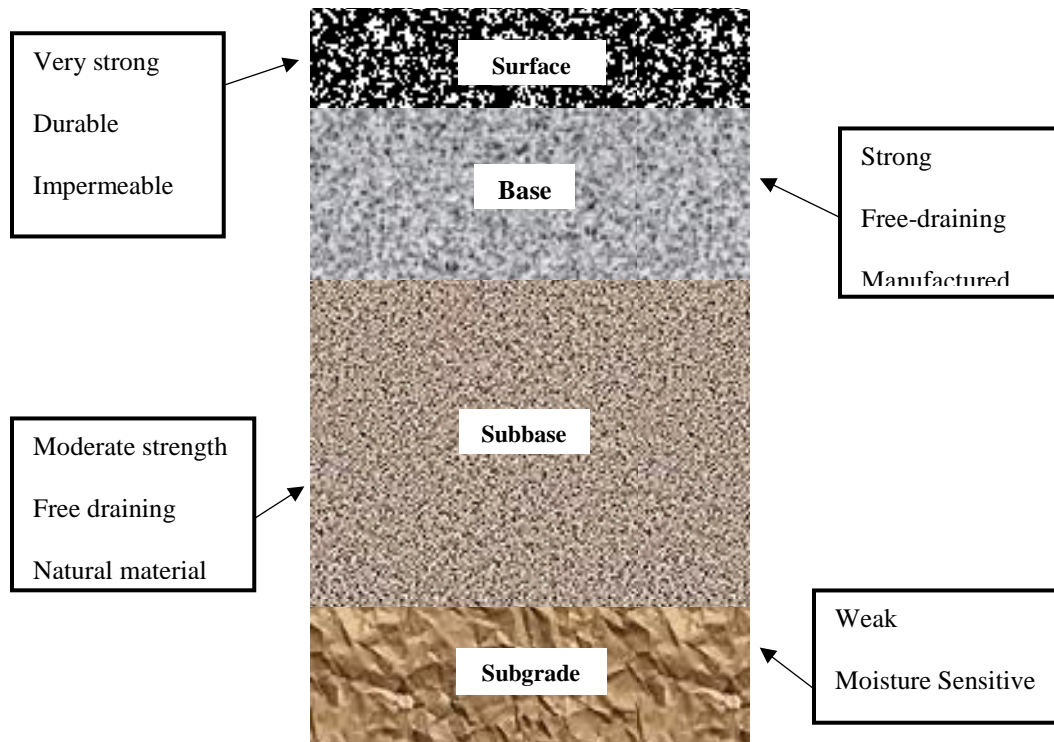


Plate 2.0 Flexible pavement layer composition (Riyanto, 1996).

According to Riyanto (1996), parts of the layer have the function to support the traffic load on it. The forces applied against the pavement are vertical forces (due to vehicle load), horizontally effective forces (due to shear or break of the wheels of the vehicle), and vibration force (collision).

i. Surface Course

Obviously, surface course is the layer in contact with traffic loads and normally contains the highest quality materials. Surface course play an important role in characteristics of friction, smoothness, noise control, rut and shoving resistance and drainage. Furthermore, surface course serves to prevent the entrance of excessive quantities of surface water into the underlying base, subbase and subgrade. This top structural layer of material is sometimes subdivided into two layers:

- 1) Wearing course: This is the toper layer in pavement structure and direct contact with traffic loads. A properly designed (and funded) preservation program should be able to identify pavement surface distress while it is still confined to the wearing course.
- 2) Binder course: The purpose of this layer is to distribute load from wearing course. This layer provides the bulk of the HMA (Hot Mix Asphalt) structure.

The layers are further described as follows:

1. Heavy-duty wheel hardening pavement, which has stability high to withstand load during service.
2. The waterproof coating to prevent the rain falling over it is from sinking under the coating and weakening the layer.
3. The wear layer, which is a layer that directly suffers from friction resulting from the wheel of the vehicle.
4. Layers that spread the load to the underlying layer, such as base, subbase, subgrade.

In order to fulfill the function above, generally the surface layer was made with asphalt binder so as to produce a waterproof coating with high stability and durability long (Riyanto, 1996).

ii. Base Course

The base course is a course of specified material and design thickness, which support the structural course, holds the latitude of the wheel loads and distributes the traffic loads to the sub base or subgrade. The base course is immediately beneath the surface course. It provides additional load distribution and contributes to drainage and frost resistance. Different base course material may have different thickness. Base courses are usually constructed out of:

- 1) Aggregate, are the most typically constructed from durable aggregates that will not be damage by moisture or frost action. Aggregates can be either stabilized or unsterilized.
- 2) HMA, used where high base stiffness is desired. In surface course HMA mixes, it usually contains larger maximum aggregate size (open graded) and subjected to more lenient specifications.

iii. Subbase Course

The subbase course is between the base course and the subgrade. The sub base consists of lower quality materials than the base course but better that subgrade soils. The sub-base consists of granular material – gravel, crushed stone, reclaimed material or a combination of these materials. The sub base is a layer of specified material and design thickness that support the base. This generally is limited to use with a composite base: For a pavement constructed over a high quality, stiff subgrade may not need the additional features offered by sub base course. However, a pavement constructed over a low quality soil such as swelling clay may require the additional load distribution characteristic that require subbase course to replace and support the poor quality subgrade. It functions primarily as structural support but it cans also:

- 1) Minimize the intrusion of fines from the subgrade into the pavement structure.
 - 2) Minimize frost action damage.
 - 3) Provide a working platform for construction. Spread the wheel load to the ground floor.
- 1) Prevent fine particles from the base soil rising to the upper layer of the foundation.

iv. Subgrade

The subgrade, the layer of soil on which the subbase or pavement is built, provides support to the remainder of the pavement system. It is crucial for highway engineers to develop a subgrade with a California Bearing Ratio (CBR) value of at least 10. Many Researches shows that if a subgrade has a CBR value less than 10, the subbase material will deflect under traffic loadings in the same manner as the subgrade and cause pavement deterioration. The subbase, the layer of aggregate material immediately below the pavement, provides drainage and stability to the pavement. Undrained water in the pavement supporting layers can freeze and expand, creating high internal pressures on the pavement structure. Moreover, flowing water can carry soil particles that clog drains and, in combination with traffic, pump fines from the subbase or subgrade. It is therefore crucial that highway engineers develop a stable, permeable subbase with longitudinal subdrains.

Layers of soil as thick as 50-100 cm where the layers will be laid on the bottom of the foundation are called the base ground layer. The base ground layer can be in the form of a compacted original soil if the original soil is good, or that land imported from another place and compacted or stabilized soil with lime or other materials. Good compaction is obtained if carried out at the optimum moisture content and cultivated the water content constant over the life of the plan (Riyanto, 1996).

2.2 Criteria for Surface Pavement Construction

According to Sukirman (1999), to provide a sense of security and comfort to the user of the road, the pavement construction must meet the certain requirements as follows:

i. Traffic Requirements

1. The flat surface is not wavy does not melt and not holey.
2. The surface is quite rigid so it is not easy to change shape due to loads that work on it.
3. The surface is sufficiently tight to provide good friction between the tires and road surface so it is not easy to skid.
4. The surface is not shiny, not glare when exposed to sunlight.

ii. Strength or Structural Requirements

Road pavement construction is in terms of carrying ability and spread the burden

- 1) The thickness is enough so that it can spread the load or charge traffic to the ground.
- 2) Wet to water so that water does not easily seep into the layer below it.
- 3) The surface easily drains the water so that rain falls on it can be quickly streamed.
- 4) Rigidity to carry a load that works without causing meaningful deformation.

2.3 Pavement Damage

Based on Road Maintenance Manual (1983) the causes of damage to the construction of flexible pavement include:

1. The imposition of traffic that may be an increase in expenses and repeated charges.
2. Water coming from rainwater, poor drainage system and rising water due to the nature of capillarity.
3. The pavement construction material, in this case, is due to the material's properties self and material processing system that is not good.

4. Climate, temperature, and rainfall are high enough to be one cause of damage.
5. Basic soil conditions are less good and unstable.
6. The moisture of vapor movements, layers of materials on the ground especially cohesive soils (silt clays), easily shrink and move.
7. The process of soil compaction is not good.
8. Emphasis (flushing) is a permanent extending dislocation of the region on a flexible road surface and caused by traffic.

2.3.1 Types of Damage on Flexible Pavement

According to road maintenance manual Number: 03 / MN / B / 1983 that was issued by the Directorate General of Bina Marga, road damage can be differentiated as:

1. Cracking, a type of damage which often occurs on the road surface. Cracks which often occur among others; fine cracks, longitudinal cracks, transverse cracks, crack edge, connection crack, crack slip, shrinkage crack and reflection crack.
2. Disintegration, another type of damage which leads to chemical and mechanical damage of the pavement layer. It includes potholes, raveling, and stripping.
3. Changes in shape (distortion) can occur due to the weak ground, less compaction on the base layer, resulting in additional compaction due to traffic load.
4. The wear (polished aggregate) of the road surface becomes slippery which endangers the vehicle. Wear occurs because the aggregate comes from materials that cannot withstand wear on the wheels of vehicles or aggregates used (round and slippery).
5. Bleeding or Flushing, caused by road surface showing slickness and on high temperature the asphalt will soften resulting in the occurrence of trace wheel damage causing obesity. Excessive use of asphalt can be overcome by spreading coarse aggregate.

The uses and application of modern technologies and new concepts have been devised to tackle problem of pavement failure by many organized bodies. In fairness, the Carthagians are generally credited with being the first to construct and maintain a road system (about 600 B.C) according to Tillson (1900). The Romans eventually decided that their neighbours across the Mediterranean were a bit of a threat to the empire destroying Carthage in 146 B.C. Apparently, there is no record of “traditional” roads in U.K prior to the Romans (Collins and Hart 1936). For the most part, the main Roman roads in the U.K (total of about 4100km) was for military purposes in that they connected camps which were about 30km apart (or about a day’s march). (Collins and Hart, 1936; Rose, 1935; Legar, 1875; Thomas and Hassan, 2018; Daibi-Oruene, Ebiloma and Bumaa, 2017). The Roman design for their primary U.K roads generally consisted of four layers (top to bottom) as follows (Collins and Hart, 1936): John macadam (born 1756) observed that most of the paved U.K roads in early 1800s were composed of rounded gravel (Smiles 1904). Several factors are responsible for road failure and other civil construction which include geological, geomorphological, geotechnical, land usage, construction practices and maintenance (Medisa, 1981) and (Ajayi, 1987) stated that any civil construction can arise from inadequate Thus, the treatment of the troublesome materials like clays are not being considered by the construction engineers. This was also supported by the work of (Gidigasu, 1983), (Graham and Shield, 1984), (Akpokodje, 1986).

2.4 Importance of Subgrade Soil Test in Flexible Pavement Design

2.4.1 Particle Size Distribution: The particle/grain size distribution of a soil is an important determinant of its geotechnical characteristics. In construction, clay materials are seen as troublesome Sowers and Sower (1970). This is because, clay is less permeable and to determine the percentage clay present in the natural soil of an area to know whether it will serve as a good

sub-grade or not is essential. This particle size distribution analysis becomes necessary. For grading test, good materials are materials having $\leq 35\%$ passing for sieve 75 μm or 200 sieves. Although other parameters are considered before the verdict can be given of which material is good.

2.4.2 Atterberg Limit: Soil with plasticity index > 35 is described as highly plastic soil, this is in line with the work of Sowers and Sower (1970). Such a soil usually has the ability to retain appreciable amount of total moisture in the diffuse double layer, especially by means of absorption. This fact was buttressed by the higher optimum moisture content recorded in soil sample. High plasticity materials are usually susceptible to high compressibility (Seed and Woodward, 1964); (Sowers and Sower, 1970); (Coduto, 1999). It should be noted that the more the clay, the more troublesome the material is. This is line with the works of Okagbue and Uma (1988), Jegede (1997), and Akpan (2005) amongst others.

2.4.3 Plasticity Index: The plasticity index (pi) is a measure of the plasticity of a soil. The plasticity index is the size of the range of water contents where the soil exhibits plastic properties. The plasticity index is the difference between the liquid limit and the plastic limit ($PI=LL-PL$). Soils with a high plasticity index tend to be clay, those with a lower plasticity index tend to be silt, and those with a plasticity index of 0 (non-plastic tend to have little or no silt or clay. Plastic index and their meanings; (0-3)–Non-plastic, (3-15)–Low plastic, (15-35)–slightly plastic, (35-50)–Medium plastic and >50 –Highly plastic.

2.4.4 Compaction: It should be noted that the density of the soil masses affects the strength of the soil. Generally, the strength of a soil increases as its dry density increases. (David, 2007). Also, the potential for the soil to take on water at later times is decreased by higher densities. This is due to the decreased presence of air space in the soil mass, (David, 2007).

2.4.5 California Bearing Ratio (CBR): It is the ratio of force per unit area required to penetrate a soil mass with standard circular piston at the rate of 1.25mm/min to that required for the corresponding penetration of a standard material. This is the parameter used by The California State Highway Department (U.S.A) for evaluating the bearing capacity of sub-grade soil for design of flexible pavement. Tests are carried out on natural or compacted soils in water soaked or unsoaked conditions and the results so obtained are compared with the curves of standard test to have an idea of the soil strength of the sub-grade soil. $CBR (\%) = \frac{[\text{Measured pressure for site soil (N/mm}^2)]}{[\text{Pressure to achieve equal penetration on standard soil (N/mm}^2)]}$

2.5 Construction, Maintenance and Management

2.5.1 Pavement Construction

Pavement construction is generally preceded by detailed surveys and technology for constructing highways has evolved over time and become increasingly sophisticated. This advancement in technology has raised the level of skill sets required to manage highway construction projects. This skill varies from project to project, depending on factors such as the project's complexity and nature, the contrast between new construction and reconstruction and differences between urban region and rural projects.

2.5.1.1 Surface Course Construction: There are two most commonly used types of flexible pavement surfacing used in highway construction: wearing course and a base course. These pavement surface courses provide a smooth and safe riding surface, while simultaneously transferring the heavy traffic loads through the various base courses and into the underlying sub grade soils.

2.5.1.2 Base Course Construction: The base course is the region of the pavement section that is located directly under the surface course. If there is a sub-base course, the base course is

constructed directly about this layer. Otherwise, it is built directly on top of the subgrade. Typical base course thickness ranges from 4 to 6 inches and is governed by underlying layer properties. Heavy loads are continuously applied to pavement surfaces, and the base layer absorbs the majority of these stresses. Generally, the base course is constructed with an untreated crushed aggregate such as crushed stone, slag or gravel. The base course materials will have stability under the construction traffic and good drainage characteristics.

2.5.1.3 Sub-Base Course Construction: A sub-base course is a layer designed of carefully selected materials that is located between the sub-grade and base course of the pavement. The sub-base thickness is generally in the range of 4 to 6 inches, and it is designed to withstand the required structural capacity of the pavement section. Common materials used for a highway sub-base include: gravel, crushed stone or sub-grade soil that is stabilized with cement, fly ash or lime. Permeable sub-base courses are becoming more prevalent because of their ability to drain infiltrating water from the surface. They also prevent subsurface water from reaching the pavement surface.

2.5.2 Pavement Maintenance

The overall purpose of pavement maintenance is to fix defects and preserve the pavement's structure and serviceability. Defects must be defined, understood and recorded in order to select an appropriate maintenance plan. Defects differ between flexible and rigid pavements. There are four main objectives of pavement maintenance:

1. Repair of functional pavement defects.
2. Extend the functional and structural service life of the pavement.
3. Maintain road safety and signage
4. Keep road reserve in acceptable condition.

Through routine maintenance practices, highway systems and all of their components can be maintained to their original as-built condition.

2.5.3 Project Management

Project management involves the organization and structuring of project activities from inception to completion. Activities could be the construction of infrastructure such as highways and bridges or major and minor maintenance activities related to constructing such infrastructure. The entire project and involved activities must be handled in a professional manner and completed within deadlines and budget. In addition, minimizing social and environmental impacts is essential to successful project management.

2.6 Summary of Flexible Pavement Failure

The pavement fails because the sub grade, base course or wearing surface course has failed. Sub-grade failure results when the applied load on soil expels all the water and air pressure in pores of the sub-grade combined with the force produced by the applied force such as plastic deformation is progressively under repetition of load and is the major course of road failure. In sufficient stability of soil which may be as a result of excessive moisture, improper compaction of soil it may be another cause of pavement failure.

CHAPTER THREE

MATERIALS AND METHODOLOGY

3.1 Materials

The material to be used in this project work include:

- i. The subgrade soil



Plate 3.0 The Subgrade soil made up of different particle sizes

This chapter exposes the methods used in conducting the experiments related to this study. After identification of the problem in previous chapter, we then proceeded to finding firstly, the basic properties of the subgrade through the sieve analysis test. Since the subgrade soil showed physical signs of water absorption, the Atterberg limit test was conducted next which comprised of both the liquid and plastic limit which helped in determining the water content of the soil. Finally, the compaction and CBR test were conducted for a conclusive confirmation of the assessment.

3.2 Collection of Soil Samples

The site where this study was carried out is strictly limited to the highway intersection at Gariki Livestock Market section of the Enugu-Onitsha Express Highway in Anambra state, Nigeria. I

visited the site to collect the representative samples (3) for the test by digging 2ft into one end of the pavement shoulder and each sample was taken 5 metres apart from the other.

3.3 Soil Property Tests

3.3.1 Sieve Analysis (Wet Sieving)

Wet sieving analysis was used to determine the particle size distribution of the soil samples.

Apparatus

- a) ASTM test sieves
- b) Weighing balance
- c) Mechanical sieve shaker
- d) Cleaning brush



Plate 3.1 Sieves of varying aperture mounted on the mechanical sieve shaker

Procedure

Preliminary separation

- i. Both sides of a 75 μ m test sieve was wetted and reserved for use in this test only.

- ii. The weighed oven dried test portion was placed in a container and sufficient water was added to half fill the container. The contents were agitated so that particles smaller than 75 μ m are completely separated from coarser particles.
- iii. The suspended fine solids were poured on to the guarded 75 μ m test sieve.
- iv. The coarse residue was washed continuously until the water passing the test sieve was clear and then the residues from the container and sieve were washed into the tray. Excess free water was removed by careful decantation through the test sieve, avoiding transfer of solids and dry the residue in the oven at 105 \pm 5 $^{\circ}$ C until constant mass was achieved.

Sieving the Dried Residue (By Use of The Mechanical Sieve Shaker)

- i. The clean and dry sieve were nested on a fitting receiver in order of increasing aperture size from bottom to top. The dried residue was placed on the top coarsest sieve and covered with a fitting lid. Using the mechanical sieve shaker, the sieves were shook for a sufficient time (5mins) to separate the test sample into the size fractions determined by the sieve apertures used.
- ii. When the mechanical sieve shaker was used, after sieving, the separation is checked if complete by briefly hand sieving.
- iii. Extraneous materials not representative of the bulk that will not readily break down into individual particles, such as clay lumps are recorded and removed from the sieve for separate weighing.
- iv. Pressure was not applied to the surface of the sieve to avoid forcing particles through the mesh. Light brushing with a soft brush on the underside of the sieve was used to clear sieve openings.

- v. In order to prevent blinding of the sieve apertures by overloading, it was ensured that the mass of soil retained on the sieve at completion of the operation did not exceed the value for that sieve.
- vi. The material retained on each sieve together with any material cleaned from the mesh on completion of sieving on that sieve was weighed.
- vii. The soil passing the sieve to the next sieve in the series was added before commencing the operation on that sieve.

3.3.2 Atterberg Limits Tests

The Atterberg limits tests establish the moisture contents at which fine-grained clay and silt soils transition between solid, semi-solid, plastic and liquid states. Each stage exhibits significant differences in strength, consistency and behaviour. This test accurately defines the boundaries between these states using moisture contents at the points where the physical changes occur.

3.3.3 Liquid Limit Test

The liquid limit of a soil is the moisture content, expressed as a percentage of the weight of the oven-dried soil, at the boundary between the liquid and plastic states of consistency. The moisture content at this boundary is arbitrarily defined as the water content at which two halves of a soil cake will flow together, for a distance of $\frac{1}{2}$ in. (12.7 mm) along the bottom of a groove of standard dimensions separating the two halves, when the cup of a standard liquid limit apparatus is dropped 25 times from a height of 0.3937 in. (10 mm) at the rate of two drops/second.

Apparatus

- a) Porcelain evaporating dishes or similar mixing dishes approximately $4\frac{1}{2}$ in. (114 mm) in diameter.

- b) Pulverizing apparatus - mortar and rubber-covered pestle.
- c) U.S. No. 40 (0.425 mm) sieve.
- d) Spatula, about 3 in. (75 mm) long and approximately $\frac{3}{4}$ in. (19 mm) wide.
- e) Balance sensitive to 0.01 g.
- f) Watering bottle, with distilled, demineralized or tap water.
- g) Drying tares with covers, such as metal cans with lids, which will prevent moisture loss.

The tares and covers should be marked and weighed as matched pairs.

- h) Mechanical Liquid Limit Device(s).

Manually operated - consisting of a brass cup and carriage, constructed according to the plan and dimensions.



Plate 3.2 The liquid limit apparatus

Preparation of Test Sample

1. It was preferable that soils used for the liquid limit determination were in their natural or moist state, because drying may alter the natural characteristics of some soils. Organic soils in particular undergo changes as a result of oven-drying or even extended air-drying. Other

soils containing clay may agglomerate, lose absorbed water which is not completely regained on rewetting, or be subject to some chemical change.

2. The soil contained sand and larger size particles and provision was made to separate the minus No. 40 (0.425 mm) material for testing despite the possibility that drying may alter the characteristics of some soils. The fine fraction of granular soil was free of organic matter but contained a minimal amount which did not affect the liquid and plastic limit results.

The soil was thoroughly dried in an oven at a temperature not exceeding $230\pm 9^{\circ}$ F ($110\pm 5^{\circ}$ C). The pulverizing apparatus and the No. 40 (0.425 mm) sieve was utilized for separation of the minus No. 40 (0.425 mm) fraction. Care was exercised to ensure that the pulverizing apparatus did not reduce the natural size of the individual grains. The ground soil was then be separated into two fractions by means of the No. 40 (0.425 mm) sieve. The plus No. 40 (0.425 mm) component was reground as before. When repeated grinding produced only a minimal quantity of minus No. 40 (0.425 mm) soil, the material retained on the No. 40 (0.425 mm) sieve was discarded and further pulverization of this fraction was suspended.

3. The material passing the No. 40 (0.425 mm) sieve obtained from the grinding and sieving operations described above was thoroughly mixed together and set aside for use in performing the physical tests. Approximately 0.3 lb. (150 g) generally sufficed for the liquid limit test.

Procedure

1. A sample of about 300g from the soil paste was taken and placed on the glass plate.
2. The prepared paste was mixed for at least 10 min using the two palette knives. Sufficient water was added to produce a consistency that will require 25 to 35 drops of the cup to

cause closure. With the cup of the apparatus resting on the base, a portion of the mixed soil was placed in the cup without entrapping air. The soil surface parallel to the base was leveled off.

3. The grooving tool was used to divide the soil into two equal parts by drawing the tool from the hinge towards the front in a continuous circular movement. The grooving tool was held normal to the surface of the cup, with the chamfered edge facing the direction of movement.
4. The crank handle was turned at the rate of 2 rev/sec so that the cup was lifted and dropped, counting the number of bumps (blows). The handle was turned continuously until the two parts of the soil came into contact at the bottom of the groove along a distance of 13 mm, measured with the end of the grooving tool or with a ruler. The number of bumps were recorded at which it occurred. A valid test is one in which 15 to 35 blows are required to close the groove.
5. A little more of the prepared soil from the glass plate was added and mixed with the soil in the cup and steps 2 to 4 were repeated until two consecutive runs gave the same number of bumps for closure.
6. About 10g of soil was taken with a spatula from the portions of the sample that had just flowed together. The soil was placed in a suitable container and the moisture content as specified was determined.
7. Steps 2 to 7 was repeated three more times using the same sample of soil to which further increments of distilled water had been added. Precedence was from the drier to the wetter condition of the soil. The objective of this procedure was to obtain samples of such consistency that at least one determination will be made in each of the following range of drops: 25-35, 20-30, 15-25, so the range in the three determinations was at least 10 drops.

The number of drops required to close the groove was above and below 25. Each time the soil is removed from the cup for the addition of water, the cup and grooving tool was washed and dried.

Table 3.3.3 Ranges of bumps/blows for liquid limit test

Trials	Range of bumps/blows for liquid limit test
1 st	15 – 20
2 nd	20 – 25
3 rd	25 – 30
4 th	30 - 35

8. At any time during the above procedure the soil was to be left for a while on the glass plate, the soil was covered with the evaporating dish or a damp cloth to prevent the soil drying out.

3.3.4 Plastic Limit Test

The plastic limit of a soil is the moisture content, expressed as a percentage of the weight of the oven-dry soil, at the boundary between the plastic and semisolid states of consistency. It is the moisture content at which a soil will just begin to crumble when rolled into a thread $\frac{1}{8}$ in. (3 mm) in diameter using a ground glass plate or other acceptable surface.

Apparatus

- 1) Evaporating dishes - porcelain or similar mixing dishes approximately 4½ in. (114 mm) in diameter.
- 2) Pulverizing apparatus - mortar and rubber covered pestle.
- 3) U.S. No. 40 (0.425 mm) sieve.
- 4) Spatula, about 3 in. (75 mm) long and approximately ¾ in. (19 mm) wide.
- 5) Balance sensitive to 0.01 g.

- 6) Watering bottle, with distilled water demineralized or tap water.
- 7) Drying tares with covers, such as metal cans with lids, which will prevent moisture loss.
The tares and covers should be marked and weighed as matched pairs.
- 8) Surface for rolling - a ground glass plate or piece of glazed or unglazed paper on which to roll the soil sample. (Unglazed refers to paper similar to that used for mimeographing).
Paper toweling is not satisfactory.
- 9) Oven - a thermostatically controlled drying oven capable of maintaining temperatures of $230\pm 9^{\circ}$ F ($110\pm 5^{\circ}$ C) for drying moisture samples.
- 10) A $\frac{1}{8}$ in. (3 mm) diameter rod may be used as a guide to help the operator estimate the thread size.

Preparation of Test Sample

1. The test was performed using material left over from the thoroughly mixed portion of the soil prepared for the liquid limit test, which normally was at a moisture content higher than the plastic limit. The sample was set aside and allowed to air dry until the liquid limit test had been completed. When the sample seemed too dry to permit rolling to a $\frac{1}{8}$ in. (3 mm) thread, water was added.
2. Where no leftover soil was available from the liquid limit test it was determined that the soil is organic or fine-grained, containing no plus No. 40 (0.425 mm) material, the plastic limits was run on the natural soil, brought to the approximate moisture content for the plastic limit determinations.

Procedure

1. An 8g test sample was squeezed and rolled into an ellipsoidal shaped mass. This mass was rolled between the fingers or palm of hand and the ground glass plate or satisfactory paper on a smooth horizontal surface with just sufficient pressure to roll the mass into a thread of uniform diameter throughout its length. The rate of rolling was between 80 and 90 strokes/min., counting a stroke as one complete motion of the hand forward and back to the starting position again.
2. When the diameter of the thread became $\frac{1}{8}$ in. (3 mm), the thread was broken into six or eight pieces. The pieces were squeezed together between the thumbs and fingers into a uniform mass roughly ellipsoidal in shape, and rerolled.
3. The alternate rolling to a thread $\frac{1}{8}$ in. (3 mm) in diameter, gathering together, kneading and rerolling continued until the thread crumbled under the pressure required for rolling and the soil could no longer be rolled into a thread.
4. Crumbling occurred when the thread had a diameter greater than $\frac{1}{8}$ in. (3 mm). This was considered a satisfactory end point, since the soil had been previously rolled into a thread $\frac{1}{8}$ in. (3 mm) in diameter.
5. The crumbling manifested itself differently with various soil types: some soils fell apart in numerous small aggregations of particles; others formed an outside tubular layer that started splitting at both ends. The splitting progressed towards the middle, and finally the thread fell apart in many small platy particles
6. When the plastic limit had been reached, a sample of the soil was immediately taken to determine its moisture content. The crumbled portions of the soil were placed together in

a suitable tared container. The container and wet soil were weighed and recorded. Weighing was to the nearest 0.01 g.

7. Steps 1 to 7 were repeated to obtain another plastic limit sample and also weighed and recorded.
8. The soil samples were oven-dried in the uncovered containers to constant weight at $230 \pm 9^\circ$ F ($110 \pm 5^\circ$ C). The samples were placed in a desiccator (1) and allowed to cool. The covers on the containers were replaced and weighed before hygroscopic moisture could be absorbed. (2) was weighed to the nearest 0.01 g and recorded. The loss in weight of the soil in each tare, due to drying, was recorded as the weight of water.

3.3.5 Plasticity Index: is the measure of the plasticity of a soil. The plasticity index is the size of the range of water contents where the soil exhibits plastic properties. The PI is the difference between the liquid limit and the plastic limit ($PI = LL - PL$).

3.3.6 Compaction Test (Method Using 4.5 Kg Rammer for Soils with Particles Up to Medium-Gravel Size)

This test covers the determination of the dry density of soil passing a 20 mm test sieve when it is compacted in a specified manner over a range of moisture contents. The range includes the optimum moisture content at which the maximum dry density for this degree of compaction is obtained. In this test a 4.5 kg rammer falling through a height of 450 mm is used to compact the soil in three layers into a 1 L compaction mould.

Apparatus

- 1) A cylindrical, corrosion-resistant metal mould i.e. the compaction mould, having a nominal internal volume of 1L. The mould shall be fitted with a detachable baseplate and a removable extension. The essential dimensions are shown in Figure 3 which also indicates

one suitable design of mould. The internal faces shall be smooth, clean and dry before each use.

- 2) A metal rammer having a 50 ± 0.5 mm diameter circular face, and weighing $4.5 \text{ kg} \pm 50\text{g}$.
The rammer shall be equipped with a suitable arrangement for controlling the height of drop to 450 ± 4 mm.
- 3) A balance readable to 1 g.
- 4) A palette knife or spatula.
- 5) A straightedge, e.g. a steel strip about 300 mm long, 25 mm wide, and 3 mm thick, with one beveled edge.
- 6) Test sieves, of aperture sizes 37.5 mm and 20 mm and a receiver.
- 7) A corrosion-resistant metal or plastics tray with sides, e.g. about 80 mm deep, of a size suitable for the quantity of material to be used.
- 8) Apparatus for moisture content determination.
- 9) Apparatus for extracting specimens from the mould (optional).
- 10) Preparation of sample.



Plate 3.3 The standard compaction test apparatus

Procedure

1. The mould was weighed with the baseplate attached to 1g (m1). The internal dimensions were measured to 0.1 mm.
2. The extension was attached to the mould and the mould assembly was placed on a solid base, e.g. a concrete floor or plinth.
3. A quantity of moist soil was placed in the mould such that when compacted it occupied a little over one-third of the height of the mould body.
4. 27 blows was applied from the rammer dropping from a height of 300 mm above the soil as controlled by the guide tube. The blows were distributed uniformly over the surface and it was ensured that the rammer always fell freely and not obstructed by soil in the guide tube.
5. Steps 3 and 4 were repeated twice more, so that the amount of soil used was sufficient to fill the mould body, with the surface not more than 6 mm proud of the upper edge of the mould body.

NOTE It was necessary to control the total volume of soil compacted, since it had been found that if the amount of soil struck off after removing the extension is too great, the test results will be inaccurate.

6. The extension was removed and the excess soil was struck and leveled off the surface of the compacted soil carefully to the top of the mould using the straightedge. The coarse particles were replaced, removed in the levelling process, by finer material from the sample, well pressed in.
7. The soil and mould was weighed with baseplate to 1 g (m2).

8. The compacted soil was removed from the mould and placed on the metal tray. A representative sample of the soil was taken for determination of its moisture content.
9. The remainder of the soil was broken up, rubbed through the 20 mm test sieve and mix with the remainder of the prepared test sample.
10. A suitable increment of water was added and mixed thoroughly into the soil.

NOTE The water added for each stage of the test was such that a range of moisture contents was obtained which included the optimum moisture content. In general, increments of 1 % to 2 % are suitable for sandy and gravelly soils and of 2 % to 4 % for cohesive soils. To increase the accuracy of the test it is often advisable to reduce the increments of water in the region of the optimum moisture content.

11. Steps 3 to 10 was repeated to give a total of at least five determinations. The moisture contents was such that the optimum moisture content at which the maximum dry density occurs, lay near the middle of the range.

Determination of Bulk Density and Dry Density

- i. Evaluation of the bulk density, ρ (in Mg/m^3) of each compacted specimen from the equation

$$\rho = \frac{m_1 - m_2}{V} \quad \text{Eqn. 1)}$$

where

m_1 is the mass of mould and baseplate (in g);

m_2 is the mass of mould, baseplate and compacted soil (in g).

V is the internal volume of the mould (in cm^3).

- ii. Evaluation of the dry density, ρ_d (in Mg/m^3) of each compacted specimen from the equation

$$\rho_d = \frac{100\rho}{100+w} \quad \text{Eqn. 2)}$$

where

w is the moisture content of the soil (in %).

3.3.7 California Bearing Ratio (CBR) Test

This method covers the laboratory determination of the California Bearing Ration (CBR) of a compacted or undisturbed sample of soil in accordance with the guidelines specified by BS 1997-4: 1990. The principle is to determine the relationship between force and penetration when a cylindrical plunger of a standard cross-sectional area is made to penetrate the soil at a given rate. At certain values of penetration, the ratio of the applied force to a standard force, expressed as a percentage, is defined as the California Bearing Ratio (CBR).

Preparation of Test Sample

The CBR test was carried out on a material passing the 20 mm test sieve. The moisture content of the soil was chosen to represent the design conditions for which the test results were required. Alternatively, where a range of moisture contents was to be investigated, water was added to or removed from the natural soil after disaggregation. After bringing it to the required moisture content the soil was thoroughly mixed. The mass of soil required for the test was calculated which depended upon the specified conditions and method of preparation.

Mass of Soil for Test

The density content of the compacted sample was specified and the exact amount of soil required for the test was calculated as;

Dry density specification.

The mass of soil, M_1 (in g), required to just fill the CBR mould of volume V_m (in cm^3) is given by the equation

$$M_1 = \frac{Vm}{100} (100 + w) \rho_d \quad \text{Eqn. 3)}$$

Where

w is the moisture content of the soil (in %); and

ρ_d is the specified dry density (in Mg/m^3).

Apparatus

1. Test sieves, of aperture sizes 20 mm and 5 mm.
2. A cylindrical, corrosion-resistant, metal mould, i.e. the CBR mould, having a nominal internal diameter of 152 ± 0.5 mm. The mould shall be fitted with a detachable baseplate and a removable extension. The essential dimensions are shown in Figure 12 and Figure 13, which also indicate one suitable design of mould. The internal faces shall be smooth, clean and dry before each use.
3. A compression device for static compaction. Horizontal platens shall be large enough to cover a 150 mm diameter circle and capable of a separation of not less than 300 mm. The device shall be capable of applying a force of at least 300 kN.
4. Metal plugs, 150 ± 0.5 mm in diameter and 50 ± 1.0 mm thick, for static compaction of a soil specimen [for methods (1) and (2)]. A handle which may be screwed into the plugs facilitates removal after compaction. The essential dimensions are shown in Figure 13. One is required for method (1) and three are required for method (2).
5. A metal rammer. These shall be either the 2.5 kg rammer or the 4.5 kg rammer, depending on the degree of compaction required.

6. A steel rod, about 16 mm in diameter and 600 mm long.
7. A spatula.
8. A balance, capable of weighing up to 25 kg readable to 5 g.
9. Apparatus for moisture content determination (at least 2 moisture cans).
10. Filter papers, 150 mm in diameter, e.g. Whatman No. 1 or equivalent.

Preparation of Test Sample by Dynamic Compaction (Rammer Compaction with Specified Effort).

The specified effort of compaction shall correspond to the 2.5 kg rammer method;

1. The prepared quantity of soil was divided into five portions equal to within 50 g and each portion was sealed in an airtight container until required for use, to prevent loss of moisture.
2. The mould assembly was stood on a solid base, e.g. a concrete floor or plinth.
3. The first portion of soil was placed into the mould and compacted, so that after 62 blows of the appropriate rammer the layer occupied about or a little more than one-third (one-fifth*) of the height of the mould. The blows were ensured to be evenly distributed over the surface.
4. Step 3 was repeated using the other four portions of soil in turn, so that the final level of the soil surface was not more than 6 mm above the top of the mould body.
5. The collar was removed and the soil flush with the top of the mould was trimmed with the scraper, checking with the steel straightedge.

6. The mould, soil and baseplate were weighed to the nearest 5 g (m3).



Plate 3.4 BS mould assembled for soaking

Soaking

Apparatus

1. A perforated baseplate, fitted to the CBR mould in place of the normal baseplate.
2. A perforated swell plate, with an adjustable stem to provide a seating for the stem of a dial gauge.
3. Tripod, mounting to support the dial gauge. A suitable assembly is shown in Figure 14.
4. A dial gauge, having a travel of 25 mm and reading to 0.01 mm.
5. A soaking tank, large enough to allow the CBR mould with baseplate to be submerged, preferably supported on an open mesh platform.
6. Annular surcharge discs, each having a mass known to ± 50 g, an internal diameter of 52 mm to 54 mm and an external diameter of 145 mm to 150 mm. Alternatively half-circular segments may be used.
7. Petroleum jelly.

Soaking Procedure

- 1.** The baseplate was removed from the mould and replaced with the perforated baseplate.
- 2.** The collar was fitted to the other end of the mould, packing the screw threads with petroleum jelly to obtain a watertight joint.
- 3.** The mould assembly was placed in the empty soaking tank. A filter paper was placed on top of the sample, followed by the perforated swell plate. The required number of annular surcharge discs was fitted around the stem on the perforated plate.
- 4.** The immersion tank was filled with water to just below the top of the mould extension collar.
- 5.** The time taken for water to appear at the top of the sample was recorded. (This did not necessarily indicate the end of the swelling stage.) If this had not occurred within 2 days (48hours), the top of the sample would have been flooded and left to soak for a further day, giving the normal soaking period of 2-4 days.
- 6.** The mould assembly was removed from the immersion tank and the sample was allowed to drain for 15 min.
- 7.** The surcharge discs, perforated plate and extension collar was removed. The perforated baseplate was removed and the original baseplate was refit.
- 8.** The sample with mould and baseplate was weighed to the nearest 5 g.
- 9.** The sample was then ready for test in the soaked condition.



Plate 3.5 BS moulds soaked in an immersion tank

Penetration Test Procedure

Apparatus

1. A cylindrical metal plunger, the lower end of which shall be of hardened steel and have a nominal cross-sectional area of 1935mm^2 , corresponding to a specified diameter of 49.65 ± 0.10 mm. A convenient size would be approximately 250mm long.
2. A machine for applying the test force through the plunger, having a means for applying the force at a controlled rate. The machine shall be capable of applying at least 45kN. The unloaded machine approach speed shall be 1.2 ± 0.2 mm/min.
3. A calibrated force-measuring device complying with **4.2.1.6** of BS 1377-1:1990. The device shall be supported by the crosshead of the compression machine so as to prevent its own weight being transferred to the test specimen.
4. A means of measuring the penetration of the plunger into the specimen, to within 0.01 mm. A dial gauge with 25 mm travel, reading to 0.01 mm and fitted to a bracket attached to the plunger is suitable.
5. A stop-clock or stopwatch readable to 1s.

6. The CBR mould.
7. Surcharge discs.

Procedure

1. The mould was placed with the baseplate containing the sample, with the top face of the sample exposed, centrally on the lower platen of the testing machine.
2. The appropriate annular surcharge discs was placed on top of the sample.
3. The cylindrical plunger and force-measuring device assembly was fitted into place with the face of the plunger resting on the surface of the sample.
4. A seating force was applied to the plunger, depending on the expected CBR value, as follows.

For CBR value up to 5 % apply 10 N

For CBR value from 5 % to 30 %, apply 50 N

For CBR value above 30 % apply 250 N

The reading of the force-measuring device was recorded as the initial zero reading (because the seating force is not taken into account during the test) or reset the force-measuring device to read zero.

5. The penetration dial gauge was secured in position. Its initial zero reading was recorded.
6. The test was started so that the plunger penetrated the sample at a nominal rate of 1 mm/min, and at the same instant the timer was started.
7. Readings of the force gauge at intervals of penetration of 0.25 mm, to a total penetration not exceeding 7.5 mm was recorded.

8. The plunger was raised and the surface of the sample was leveled by filling in the depression left by the plunger and cutting away any projecting material. Flatness was checked with the straightedge.
9. The baseplate was removed from the lower end of the mould and was fitted securely on the top end and the mould was inverted. The exposed surface was trimmed.
10. The test was carried out on the base by repeating **1** to **7**.



Plate 3.6 California Bearing Ratio (CBR) Test Machine



Plate 3.7 Observing and recording the penetration readings

CHAPTER FOUR

RESULTS AND ANALYSIS

4.1 INTRODUCTION

This chapter comprises of the results and analysis of all tests done in the process of this project. These tests include; Particle size distribution, Atterberg Limit Test (Liquid limit and Plastic limit), Compaction and California Bearing Ratio (CBR) and were performed on all three different samples (I, II and III).

4.2 Particle Size Distribution

The wet sieving analysis test was conducted on three samples of the subgrade soil to determine the particle size distribution of the soil and in turn classifying the soil. From the tables (4.2.0 – 4.2.2) we can see the result of the test on each sample.

i. Uniformity Coefficient $C_u = \frac{D_{60}}{D_{10}}$ Eqn. 4)

ii. Coefficient of curvature $C_c = \frac{D_{30}^2}{D_{60} \times D_{10}}$ Eqn. 5)

The uniformity coefficient C_u and the coefficient of curvature (gradation) are the measures of soil gradation. These coefficients will help in classifying the soil as well graded or poorly graded ones. A value of C_u greater than 4 – 6 classifies the soil as well graded. When the soil is less than 4, it is classified as poorly graded or uniformly graded soil. Uniformly graded soil has identical particles with C_u value approximately equal to 1. A uniformity coefficient value of 2 – 3 classifies the soil as poorly graded.

Table 4.2.0 Sieve Analysis Test (Wet Sieving) on Sample I

Sieve Size (mm)	Weight Retained (g)	% Weight Retained	Cumulative % Weight Retained	Cumulative % Weight Passing
2	5.32	2.48	2.48	97.52
1.18	6.28	2.93	5.41	94.59
0.85	7.39	3.45	8.86	91.14
0.6	14.52	6.78	15.64	84.36
0.425	22.51	10.51	26.15	73.85
0.3	30.68	14.33	40.48	59.52
0.15	73.87	34.50	74.98	25.02
0.075	39.97	18.66	93.64	6.86
TRAY	13.61	6.36	100	0

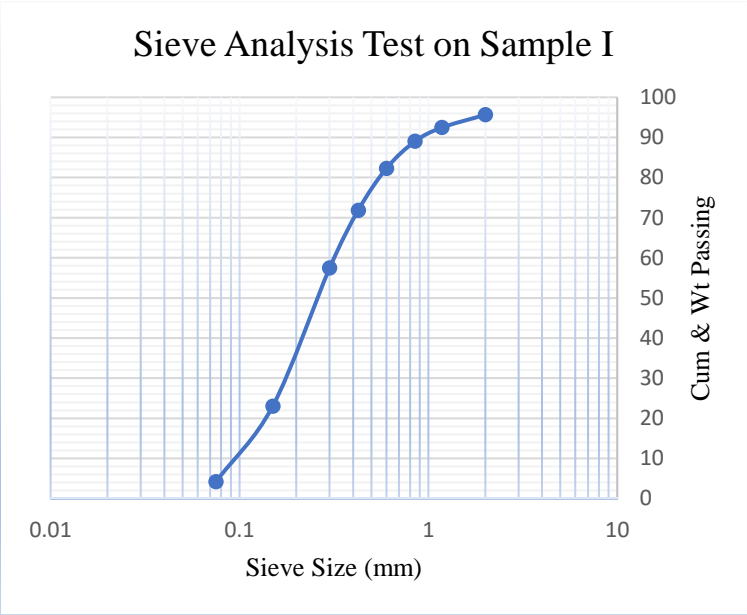


Fig. 4.0 Graph of Cum % Wt Passing against Sieve Size (mm)

i. Uniformity Coefficient $C_u = \frac{0.35}{0.1} = 3.50$

ii. Coefficient of curvature $C_c = \frac{0.18^2}{0.35 \times 0.1} = 0.93$

Table 4.2.1 Sieve Analysis Test (Wet Sieving) on Sample II

Sieve Size (mm)	Weight Retained (g)	% Weight Retained	Cumulative % Weight Retained	Cumulative % Weight Passing
2	10.56	4.37	4.37	95.63
1.18	7.66	3.17	7.54	92.46
0.85	8.30	3.43	10.97	89.03
0.6	16.29	6.73	17.70	82.3
0.425	25.32	10.47	28.17	71.83
0.3	34.83	14.40	42.57	57.43
0.15	83.22	34.40	76.97	23.03
0.075	45.60	18.85	95.82	4.18
TRAY	10.13	4.18	100	0

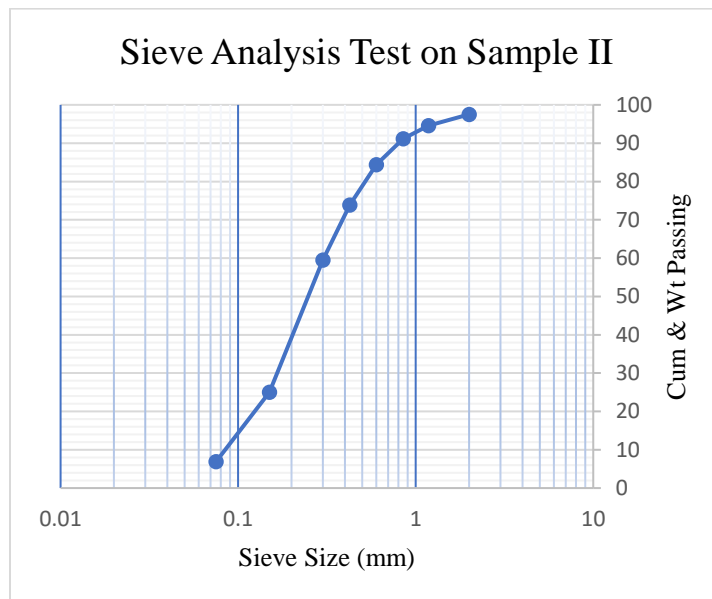


Fig. 4.1 Graph of Cum % Wt Passing against Sieve Size (mm)

i. Uniformity Coefficient $C_u = \frac{0.31}{0.09} = 3.44$

ii. Coefficient of curvature $C_c = \frac{0.17^2}{0.31 \times 0.09} = 1.04$

Table 4.2.2 Sieve Analysis Test (Wet Sieving) on Sample III

Sieve Size (mm)	Weight Retained (g)	% Weight Retained	Cumulative % Weight Retained	Cumulative % Weight Passing
2	27.02	8.81	8.81	91.19
1.18	14.51	4.73	13.54	86.46
0.85	15.63	5.10	18.64	81.36
0.6	26.68	8.70	27.34	72.66
0.425	39.75	12.97	40.31	59.69
0.3	49.51	16.15	56.46	43.54
0.15	91.41	29.82	86.28	13.72
0.075	31.83	10.38	96.66	3.34
TRAY	10.22	3.33	100	0

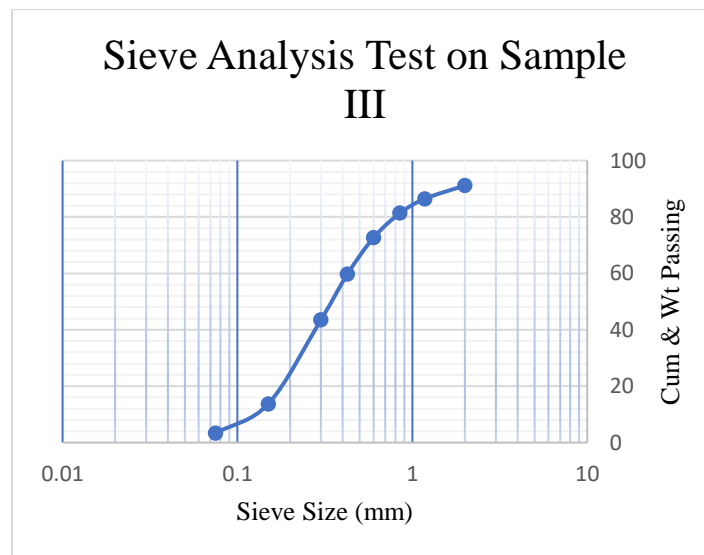


Fig. 4.2 Graph of Cum % Wt Passing against Sieve Size (mm)

i. Uniformity Coefficient $C_u = \frac{0.44}{0.15} = 2.93$

ii. Coefficient of curvature $C_c = \frac{0.24^2}{0.44 \times 0.15} = 0.87$

From the result of the computations above, the uniformity coefficients of the samples are 3.50, 3.44 and 2.93 with an average of 3.29 which is below the range for a well graded soil thus falls under the classification of a poorly graded soil indicating a significant range of particle size. The test results also yield the coefficient of curvature for the three samples which are 0.93, 1.04, and 0.87 with an average of 0.94. This also falls below and out of the range for a well graded soil.

4.3 Atterberg Limit Test

The liquid limit and plastic limit of the three samples were calculated and are presented in the table along with their various plasticity index for indicating and determining the clay content of the subgrade soil and its potential for volume changes.

Table 4.3 Liquid Limit of Sample A

No of blows A	15	20	25	30
Weight of tin (g) B	16.87	15.07	15.08	16.55
Weight of tin + wet soil (g) C	32.44	33.92	31.3	31.27
Weight of wet soil (g) $D = C - B$	15.57	18.85	16.22	14.72
Weight of tin + dry soil (g) E	29.51	30.77	28.76	29.08
Weight of dry soil (g) $F = E - B$	12.64	15.70	13.68	12.53
Weight of water $D - F$	2.93	3.15	2.54	2.19
Moisture Content (%) $(D - F/F) \times 100$	23.18	20.06	18.57	17.48

Table 4.3.1 Plastic Limit of Sample A

Weight of tin (g) A	15.59	15.61	17.67
Weight of tin + wet soil (g) B	16.77	17.43	19.71
Weight of wet soil (g) C = B – A	1.18	1.82	2.04
Weight of tin + dry soil (g) D	16.65	17.22	19.47
Weight of dry soil (g) E = D – A	1.06	1.61	1.8
Weight of water (g) F = C – E	0.12	0.21	0.24
Moisture Content (%) (C - E/E) x 100	11.32	13.04	13.33

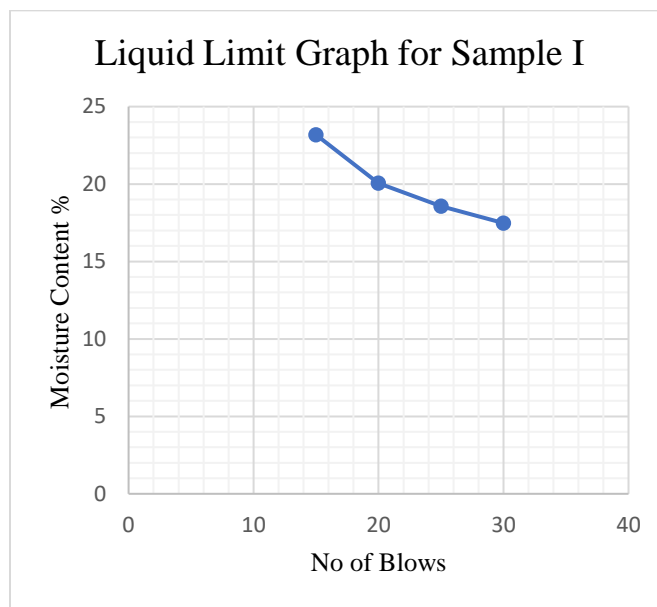


Fig. 4.3 Graph of Moisture Content % against No of Blows

Plasticity Index = LL – PL

Average MC of LL = 19.82%

Average MC of PL = 12.56%

PI = 19.82 – 12.56 = 7.26%

Table 4.3.2 Liquid Limit of Sample B

No of blows A	15	23	28	32
Weight of tin (g) B	16.56	16.39	14.79	15.84
Weight of tin + wet soil (g) C	28.70	27.57	24.93	28.71
Weight of wet soil (g) $D = C - B$	13.14	11.18	10.14	12.87
Weight of tin + dry soil (g) E	26.11	25.40	23.33	26.83
Weight of dry soil (g) $F = E - B$	9.55	9.01	8.54	10.99
Weight of water $D - F$	3.59	2.17	1.60	1.88
Moisture Content (%) $(D - F/F) \times 100$	37.59	24.08	18.74	17.11

Table 4.3.3 Plastic Limit of Sample B

Weight of tin (g) A	15.77	14.62	16.65
Weight of tin + wet soil (g) B	17.01	16.11	18.36
Weight of wet soil (g) $C = B - A$	1.24	1.49	1.71
Weight of tin + dry soil (g) D	16.84	15.94	18.15
Weight of dry soil (g) $E = D - A$	1.07	1.32	1.5
Weight of water (g) $F = C - E$	0.17	0.17	0.21
Moisture Content (%) $(C - E/E) \times 100$	15.89	12.88	14.00

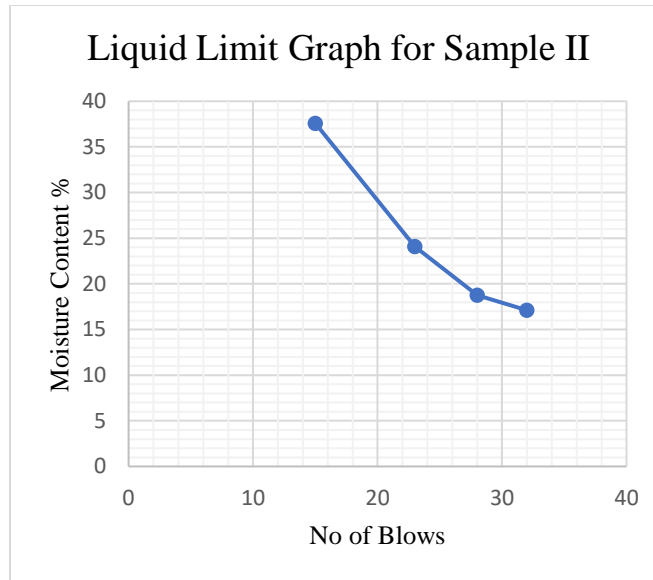


Fig. 4.4 Graph of Moisture Content % against No of Blows

Plasticity Index = LL – PL

Average MC of LL = $24.38 \times 1.014 = 24.72\%$

Average MC of PL = 14.26%

PI = $25.55 - 14.26 = 11.29\%$

Table 4.3.4 Liquid Limit of Sample C

No of blows A	18	24	28	34
Weight of tin (g) B	14.57	17.20	14.55	15.40
Weight of tin + wet soil (g) C	26.33	32.28	26.65	27.51
Weight of wet soil (g) D = C – B	11.76	15.08	12.10	12.11
Weight of tin + dry soil (g) E	24.24	29.85	24.76	25.64
Weight of dry soil (g) F = E – B	9.67	12.65	10.21	10.24
Weight of water D – F	2.09	2.43	1.89	1.87
Moisture Content (%) (D – F/F) x 100	21.61	19.21	18.51	18.26

Table 4.3.5 Plastic Limit on Sample C

Weight of tin (g) A	14.67	17.05	15.60
Weight of tin + wet soil (g) B	16.92	19.47	17.23
Weight of wet soil (g) C = B – A	2.25	2.42	1.63
Weight of tin + dry soil (g) D	16.65	19.15	17.03
Weight of dry soil (g) E = D – A	1.98	2.10	1.43
Weight of water (g) F = C – E	0.27	0.32	0.20
Moisture Content (%) (C - E/E) x 100	13.64	15.24	13.99

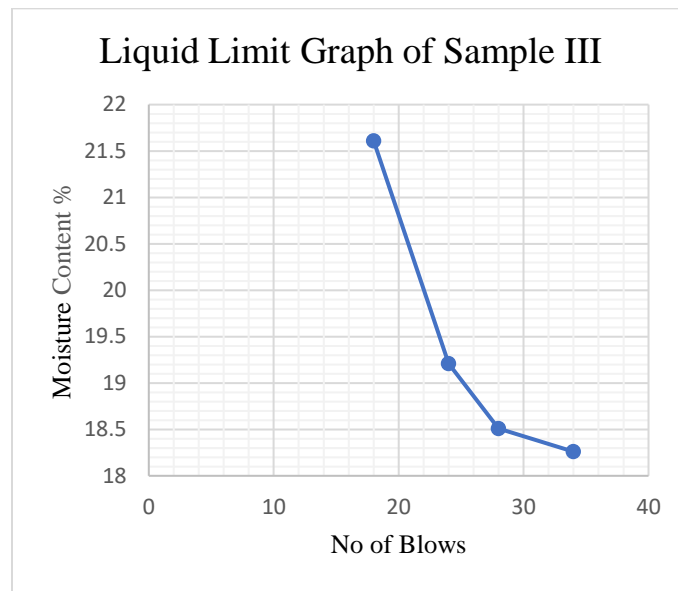


Fig. 4.5 Graph of Moisture Content % against No of Blows

Plasticity Index = LL – PL

Average MC of LL = $19.39 \times 0.995 = 19.29\%$

Average MC of PL = 14.29%

PI = $18.69 - 14.29 = 4.4\%$

The result of the Atterberg limit test of the three samples (I, II, and III) provided the plasticity indexes of 7.26%, 11.29% and 4.4% with an average of 7.65%. This indicates a soil with a moderate clay content and a limited range of moisture content within which it exhibits plastic behaviour.

4.4 Compaction Test

The compaction test conducted was to determine the maximum dry density and optimum moisture content of the subgrade soil of three samples. The optimum moisture content result from this test will also be used as a guideline for the California Bearing Ratio (CBR) test.

Table 4.4.1 Moisture content (*w*) % of Sample I

Test A	4%	8%	12%	16%	20%
Weight of tin (g) B	16.40	15.08	16.61	16.93	14.75
Weight of tin + wet soil (g) C	31.55	39.56	36.93	35.95	40.51
Weight of wet soil (g) $D = C - B$	15.15	24.48	20.32	19.02	25.76
Weight of tin + dry soil (g) E	31.00	37.83	34.96	33.64	36.66
Weight of dry soil (g) $F = E - B$	14.60	22.75	18.35	16.71	21.91
Weight of water (g) $G = D - F$	0.55	1.73	1.97	2.31	3.85
Moisture Content (%) $(D - F/F) \times 100$	3.77	7.60	9.54	10.52	12.44

Table 4.4.2 Compaction Test on Sample I

TEST A	4%	8%	12%	16%	20%
Volume of Soil (mould) (m ³) B	0.001	0.001	0.001	0.001	0.001
Weight of Mould (kg) C	4.20	4.20	4.20	4.20	4.20
Weight of Soil + Mould (kg) D	6.00	6.30	6.40	6.30	6.25
Weight of compacted Soil (kg) E = D – C	1.80	2.10	2.20	2.10	2.05
Bulk Density ρ (kg/m ³) F = E/B	1800	2100	2200	2100	2050
Moisture Content (%) G	3.77	7.60	9.54	10.52	12.44
Dry Density (kg/m ³) ρ _d H = (100 x F)/(100 + G)	1734.61	1951.67	2008.40	1900.11	1823.20

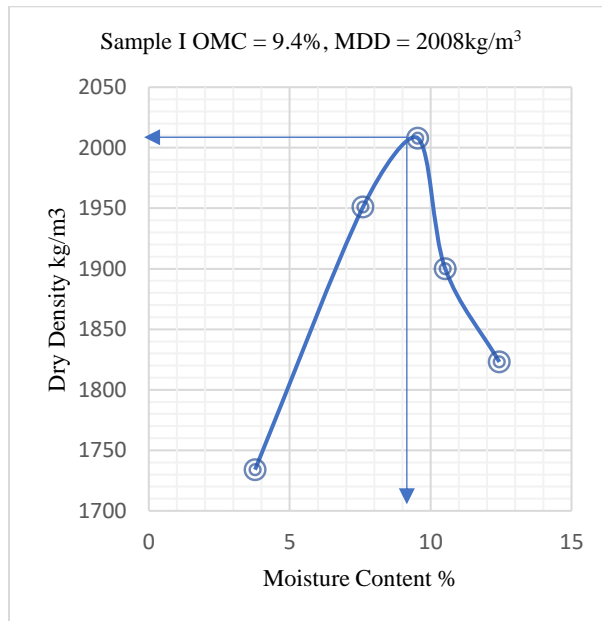


Fig. 4.6 Graph of Dry Density against Moisture Content %

$$\text{Bulk Density } \rho = \frac{\text{Weight of compacted soil}}{\text{Volume of soil}} \text{ (kg/m}^3\text{)}$$

$$\text{Dry Density} = \frac{100 \times \text{Bulk Density } \rho}{100 + \text{Moisture Content } W} \text{ (kg/m}^3\text{)}$$

Table 4.4.3 Moisture Content (W) % of Sample II

Test A	4%	8%	12%	16%	20%
Weight of tin (g) B	16.40	15.08	16.61	16.93	14.75
Weight of tin + wet soil (g) C	43.89	53.46	60.24	53.68	60.20
Weight of wet soil (g) $D = C - B$	27.49	38.38	43.63	36.75	45.45
Weight of tin + dry soil (g) E	42.39	50.45	55.22	49.63	55.09
Weight of dry soil (g) $F = E - B$	25.76	35.37	38.61	32.70	40.34
Weight of water (g) $G = D - F$	1.73	3.01	4.02	5.05	5.11
Moisture Content (%) $(D - F/F) \times 100$	6.72	8.51	10.15	12.39	12.67

Table 4.4.4 Compaction Test of Sample II

TEST A	4%	8%	12%	16%	20%
Volume of Soil (mould) (m ³) B	0.001	0.001	0.001	0.001	0.001
Weight of Mould (kg) C	4.20	4.20	4.20	4.20	4.20
Weight of Soil + Mould (kg) D	6.15	6.30	6.45	6.35	6.30
Weight of compacted Soil (kg) $E = D - C$	1.95	2.10	2.25	2.15	2.10
Bulk Density ρ (kg/m ³) $F = E/B$	1950	2100	2250	2150	2100
Moisture Content (%) G	6.72	8.51	10.15	12.39	12.67
Dry Density (kg/m ³) ρ_d $H = (100 \times F)/(100 + G)$	1827.21	1935.31	2042.67	1912.98	1863.85

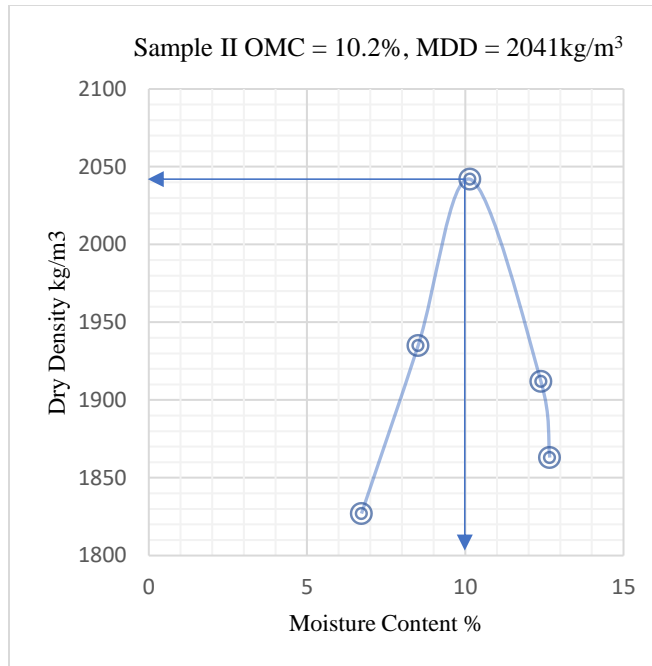


Fig. 4.7 Graph of Dry Density against Moisture Content %

Table 4.4.5 Moisture Content (W) % of Sample III

Test A	4%	8%	12%	16%	20%
Weight of tin (g) B	16.40	15.08	16.61	16.93	14.75
Weight of tin + wet soil (g) C	37.48	44.89	53.05	50.46	46.38
Weight of wet soil (g) D = C – B	21.08	29.81	36.44	33.53	31.63
Weight of tin + dry soil (g) E	35.77	42.96	50.34	46.89	42.61
Weight of dry soil (g) F = E – B	19.37	27.88	33.73	29.96	27.86
Weight of water (g) G = D – F	1.09	1.93	2.71	3.57	3.77
Moisture Content (%) (D - F/F) x 100	5.63	6.92	8.03	11.92	13.53

Table 4.4.6 Compaction Test of Sample II

TEST A	4%	8%	12%	16%	20%
Volume of Soil (mould) (m ³) B	0.001	0.001	0.001	0.001	0.001
Weight of Mould (kg) C	4.20	4.20	4.20	4.20	4.20
Weight of Soil + Mould (kg) D	6.05	6.35	6.40	6.35	6.25
Weight of compacted Soil (kg) E = D – C	1.85	2.15	2.20	2.15	2.00
Bulk Density ρ (kg/m ³) F = E/B	1850	2150	2200	2150	2000
Moisture Content (%) G	5.63	6.92	8.03	11.92	13.53
Dry Density (kg/m ³) ρ _d H = (100 x F)/(100 + G)	1751.40	2010.85	2036.47	1921.02	1805.69

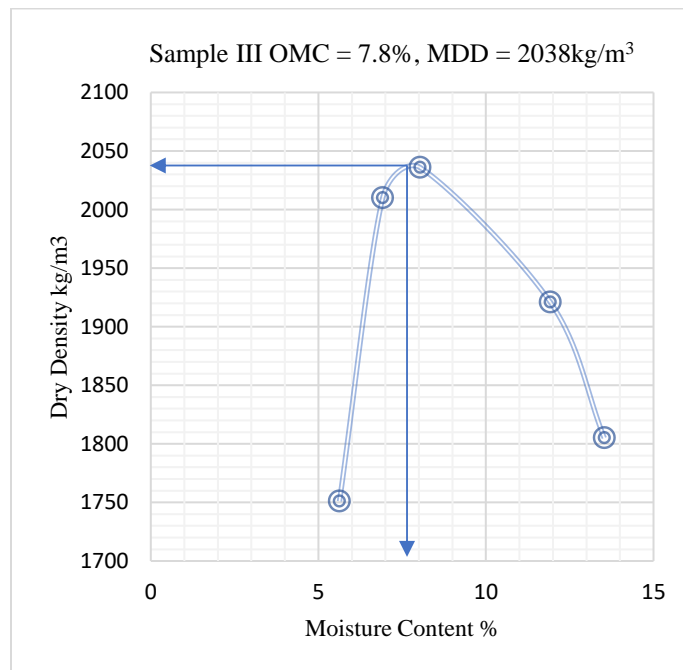


Fig. 4.8 Graph of Dry Density against Moisture Content %

The result from the compaction test of all three samples shows a maximum dry density (MDD) ranging between 2008 – 2041 (kg/m³) which suggests that the soil can achieve a considerable

degree of compaction with a fare load-bearing capacity. The optimum moisture content (OMC) obtained from the test shows a range between 7.8 – 10.2% indicating range of moisture content needed to achieve the maximum dry density of the subgrade soil during compaction.

4.5 California Bearing Ratio (CBR) Test

The laboratory California Bearing Ratio (CBR) test for this project was conducted on three soil samples of the subgrade soil in a soaked condition all for 48 hours each. This test is necessary to measure and determine the strength as well as the load bearing capacity of the subgrade soil in accordance with the standard procedures. The soil samples were all from the same site location and were taken 5 metres apart each from the other.

Table 4.5 California Bearing Ratio (CBR) Test on Sample I

			Dial Gauge Reading		Corrected Test Load (KN)	
Proving Ring Factor	Standard Load for same penetration (KN)	Penetration of plunger (mm)	Top	Bottom	Top	Bottom
A	B	C	D	E	D x A	E x A
0.04465		0.00	0.00	0.00	0.00	0.00
0.04465		0.50	11	7	0.49115	0.31255
0.04465		1.00	12.3	11	0.5492	0.49115
0.04465		1.50	16.6	16	0.7412	0.7144
0.04465		2.00	20	20	0.893	0.893
0.04465	13.2	2.50	24	24.4	1.0716	1.0895
0.04465		3.00	28	31	1.2502	1.38415
0.04465		3.50	32.2	34	1.4377	1.5181
0.04465		4.00	35.6	40.2	1.5895	1.7949
0.04465		4.50	41	45.1	1.83065	2.0137

0.04465	20.0	5.00	45	50.1	2.00925	2.23697
0.04465		5.50	52	56	2.3218	2.5004
0.04465		6.00	58	62.4	2.5897	2.78616
0.04465		6.50	64	69.3	2.8576	3.0943
0.04465		7.00	68.5	77.6	3.0585	3.46484
0.04465		7.50	74.5	83.4	3.3264	3.72381

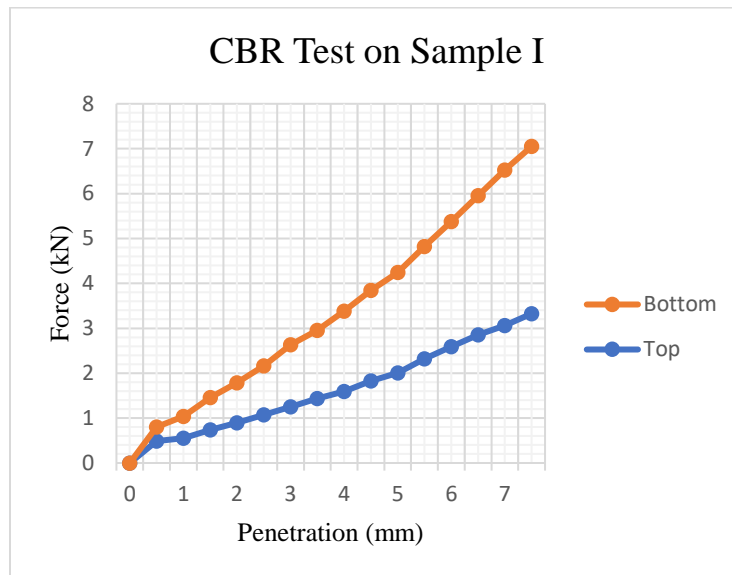


Fig. 4.9 Graph of Force (kN) against Penetration (mm)

CBR Calculation

Top:

$$CBR_{2.5} = \frac{1.0716}{13.2} \times 100 = 8.12\%$$

$$CBR_{5.0} = \frac{2.00925}{20.0} \times 100 = 10.05\%$$

Average = 9.09%

Bottom:

$$CBR_{2.5} = \frac{1.0895}{13.2} \times 100 = 8.25\%$$

$$\text{CBR}_{5.0} = \frac{2.23697}{20.0} \times 100 = 11.18\%$$

Average = 9.72%

CBR of sample I = 9.72%

Table 4.5.1 California Bearing Ratio (CBR) Test on Sample II

			Dial Gauge Reading		Corrected Test Load (KN)	
Proving Ring Factor	Standard Load for same penetration (KN)	Penetration of plunger (mm)	Top	Bottom	Top	Bottom
A	B	C	D	E	D x A	E x A
0.04465		0.00	0.00	0.00	0.00	0.00
0.04465		0.50	3	4	0.134	0.1786
0.04465		1.00	4	6.3	0.1786	0.2813
0.04465		1.50	6	9.2	0.2679	0.4108
0.04465		2.00	8	11.5	0.3572	0.5135
0.04465	13.2	2.50	10	14	0.4465	0.6251
0.04465		3.00	12	16	0.5358	0.7144
0.04465		3.50	14	19	0.6251	0.848
0.04465		4.00	17	21.4	0.759	0.9555
0.04465		4.50	20	24	0.893	1.0716
0.04465	20.0	5.00	23.7	27	1.0582	1.20555
0.04465		5.50	27	29.2	1.20555	1.3038
0.04465		6.00	31	33.1	1.38415	1.4779
0.04465		6.50	35	36.8	1.56275	1.6431
0.04465		7.00	39.2	40	1.7503	1.786
0.04465		7.50	43.5	44.2	1.9423	1.9735

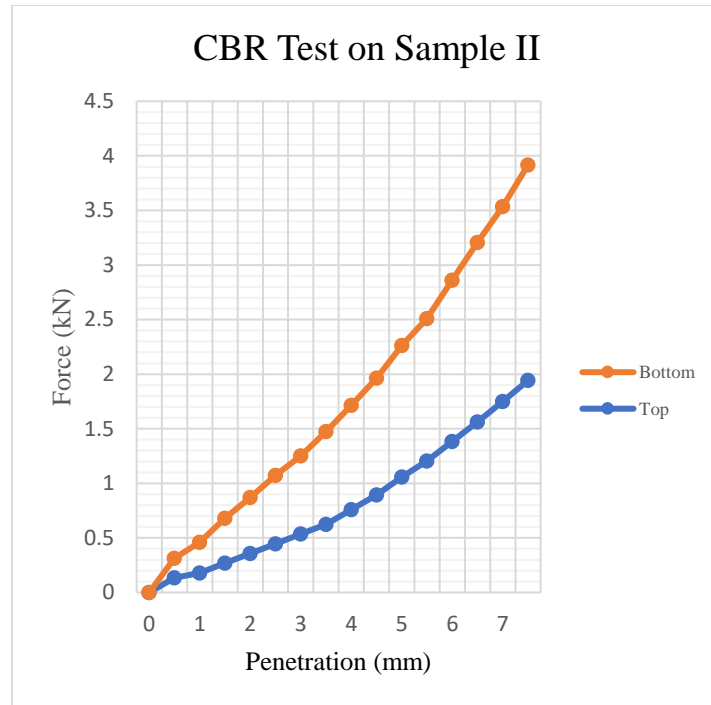


Fig. 4.10 Graph of Force (kN) against Penetration (mm)

CBR Calculation

Top:

$$CBR_{2.5} = \frac{0.4465}{13.2} \times 100 = 3.38\%$$

$$CBR_{5.0} = \frac{1.0582}{20.0} \times 100 = 5.29\%$$

Average = 4.34%

Bottom:

$$CBR_{2.5} = \frac{0.6251}{13.2} \times 100 = 4.74\%$$

$$CBR_{5.0} = \frac{1.20555}{20.0} \times 100 = 6.03\%$$

Average = 5.39%

CBR of sample II = 5.39%

Table 4.5.2 California Bearing Ratio (CBR) Test on Sample III

Proving Ring Factor	Standard Load for same penetration (KN)	Penetration of plunger (mm)	Dial Gauge Reading		Corrected Test Load (KN)	
			Top	Bottom	Top	Bottom
A	B	C	D	E	D x A	E x A
0.04465		0.00	0.00	0.00	0.00	0.00
0.04465		0.50	7	8	0.313	0.3572
0.04465		1.00	12	19	0.5358	0.84835
0.04465		1.50	16	25	0.7144	1.116
0.04465		2.00	20	33.4	0.893	1.4913
0.04465	13.2	2.50	23	38	1.02695	1.6967
0.04465		3.00	27	45.1	1.20555	2.0137
0.04465		3.50	32.2	52	1.4377	2.3218
0.04465		4.00	35	57	1.56275	2.54505
0.04465		4.50	38.3	60.4	1.7101	2.6969
0.04465	20.0	5.00	42	62.4	1.8753	2.7862
0.04465		5.50	45.5	66	2.0316	2.9469
0.04465		6.00	51.5	69.8	2.2995	3.1166
0.04465		6.50	55	73	2.45575	3.25945
0.04465		7.00	57.1	79	2.5495	3.52735
0.04465		7.50	64.3	83	2.871	3.706

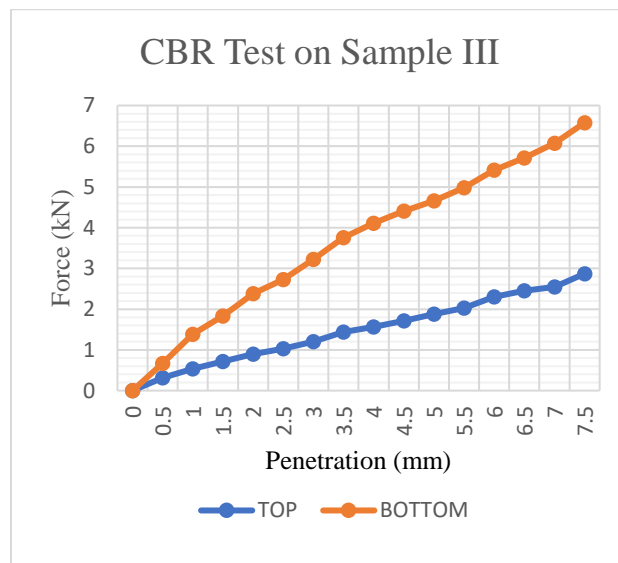


Fig. 4.11 Graph of Force (kN) against Penetration (mm)

CBR Calculation

Top:

$$\text{CBR}_{2.5} = \frac{1.02695}{13.2} \times 100 = 7.78\%$$

$$\text{CBR}_{5.0} = \frac{1.8753}{20.0} \times 100 = 9.38\%$$

Average = 8.58%

Bottom:

$$\text{CBR}_{2.5} = \frac{1.6967}{13.2} \times 100 = 12.85\%$$

$$\text{CBR}_{5.0} = \frac{2.7862}{20.0} \times 100 = 13.93\%$$

Average = 13.39%

CBR of sample II = 13.39%

The California Bearing Ratio (CBR) test of all three samples yielded peak values of 5.39, 9.72 and 13.58% with an average of 9.56% ~ 10% which suggests that the subgrade soil has relatively moderate strength. This implies that the subgrade will be suitable for design of highway pavement with lighter traffic loads.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

From the study that has been carried out, several conclusions can be derived, and also it was made in virtue of the aims and objective of this study. From the results and analysis of the various test results presented earlier, the following conclusions are drawn from the study:

1. The subgrade soil classifies as a poorly graded soil which lacks sufficient range of particle sizes. The absence of well-graded particle distribution will result in reduced strength and increased susceptibility to erosion and settlement. Further evaluation of the specific particle size fractions and their proportion is recommended to better understand the soils behaviour.
2. The plasticity index of 7.65% which indicates a low to moderate clay content in the soil will show a lesser proneness to volume changes and have a relatively lower potential for swelling and shrinkage. An adequate moisture control will be necessary during construction.
3. The maximum dry density and the optimum moisture content of the subgrade soil on indicates that the soil can be effectively compacted to achieve good load-bearing capacity soil stability.
4. The thickness of the subbase/capping layer is dependent on the CBR of the subgrade and the 10% value for the subgrade indicates a relatively low strength and load-bearing capacity for the soil.

The examination of the subgrade soil on the highway along Enugu-Onitsha Highway at Gariki Livestock Market Amansea section reveals that there is a high degree of distress on the subgrade exposing some defects on the pavement like edge cracks, fatigue cracks, potholes, patches, rutting

and longitudinal cracks. These defects occurred as a result of inadequate drainage, lack of design and potentially high moisture content of the sub-grade.

5.2 Recommendation

Based on the conclusions, the following recommendations can be made:

- i. Further investigation of the particle size distribution is recommended to provide better analysis that will give more insights into the soils behaviour and potential issues related to drainage, erosion and stability.
- ii. Considering the plasticity index, monitoring and control of moisture content during reconstruction of the pavement is essential to ensure proper compaction. Implementing suitable additives may be necessary depending on the project requirements.
- iii. Conducting a field compaction test to determine the project-specific MDD and OMC values is highly recommended. The test will provide accurate guidance on achieving the desired compaction level and moisture content during construction.
- iv. Base on the CBR value of 10%, it implies the need for a subbase/capping with varying thickness lying between the specification for a CBR range between 2.5% and 15% where two options are available:
 1. Using 150mm of subbase over a layer of capping material, the thickness of which depends on the subgrade CBR, or
 2. A layer of subbase varying between 150mm (at 15% CBR) and 350mm (at 2.5% CBR) in thickness.

Additional measures such as soil stabilization, geosynthetic reinforcement or subgrade modification technique should be considered to improve the soils load-bearing capacity.

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