GEOTECHNICAL INVESTIGATION OF SUBGRADE SOIL AND STRUCTURAL DESIGN FOR RECONSTRUCTION OF ELIAS EGBOKA-NWAKPADOLU ROADS

BY

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CERTIFICATION

This is to certify that this project topic titled "Geotechnical Investigation of Subgrade Soil and Structural Design for Reconstruction of Elias Egboka-Nwakpadolu Roads" was undertaken by Nwanga Michael Chidera with registration number (NAU/2017224041) in the Department of Civil Engineering, Nnamdi Azikiwe University Awka, Anambra State. Nigeria

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This research work "Geotechnical Investigation of Subgrade Soil and Structural Design for Reconstruction of Elias Egboka-Nwakpadolu Roads" has been assessed and approved by Department of Civil Engineering Nnamdi Azikiwe University Awka, Anambra State. Nigeria.

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DEDICATION

This work is dedicated to Almighty God for the gift of life and also for guiding me through school. I also want to dedicate this work to my lovely foster family, Onugbolu's who served as a real source of inspiration toward my academic pursuit.

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ABSTRACT

The need to design a pavement structure that will facilitate easy movement of goods and services and also minimize the difficulties faced by both private and commercial car owners and motorcyclist plying Elias Egboka-Nwakapadolu roads particularly during the rainy season formed the basis for the study. The study was undertaken to design Elias Egboka- Nwakapadolu roads using AASHTO method and detail using ArchiCad. Three different samples of soils collected within the study area designated as A, B and C was subjected to geotechnical testing. The tests conducted on the samples were sieve analysis test, specific gravity test, atterberg limit (liquid and plastic limit) test, compaction and CBR test. Result obtained from sieve analysis test classified the samples as A-2-6, A-2-6 and A-2-4 according to AASHTO Soil Classification System, according to Unified Soil Classification System; the samples were classified as SC (sand mixed with clay). The specific gravity of the samples ranged from 2.6-2.66, the liquid, plastic and plasticity index of the samples ranged from 37.8-42.4%, 17.4% - 24.2% and 16.5% - 24.2% respectively, plasticity rating of the three samples revealed that the samples were medium – highly plastic. The maximum dry unit weight and optimum moisture content of the samples ranged from 18.53kN/m^3 to 20.23kN/m^3 and 9.85% - 11.54% respectively, the soaked and un-soaked CBR of the samples ranged from 11% - 15% and 25% - 31% respectively. The soaked CBR of 11% was used to support a total equivalent single axle load (ESAL) of 25.8 x 10^6 for a design period of 20 years. Results obtained from the design of Elias Egboka-Nwakpadolu roads using AASHTO method revealed that the thickness of the sub-base, base course and surface course were 150mm, 200mm and 160mm respectively with a total thickness of 510mm superimposed on the sub-grade layer. Detailing of the designed pavement layers was done using architectural software commonly referred to as Archicad.

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LIST OF SYMBOL & ABBREVIATION

CBR—California Bearing Ratio

G_S – Specific Gravity

AASHTO – American Association of State Highway and Transportation Officials

USCS – Unified Soil Classification System

ASTM – American Society for Testing and Material

BSL – British Standard Light

BSH – British Standard Heavy

MDUW- Maximum Dry Unit Weight

OMC – Optimum Moisture Content

LL – Liquid Limit

PL – Plastic Limit

SL – Shrinkage Limit

PI – Plasticity Index

D_{10} – Particle Size such that 10% is finer than the Size

D_{30}- Particle Size such that 30% is finer than the Size

D_{60}- Particle Size such that 60% is finer than the Size

C_U – Coefficient of Uniformity

C_C – Coefficient of Curvature

SC – Clayey Sand

SM – Silty Sand

SG – Silty Gravel

GC—Clayey Gravel

GW—Well Graded Gravel

GP—Poorly Graded Gravel

SP—Poorly Graded Sand

SW—Well Graded Sand

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CHAPTER ONE

INTRODUCTION

1.1 Background of Study

Roads infrastructure plays a paramount role in the socio-economic development of any nation (Chegenizadeh et al., 2016). The impacts of road infrastructure in the development of any country cannot be overemphasized (World Bank, 1991). (Burnett, 2011) opined that there are countless advantages associated with the construction of good road networks particularly within the confines of socioeconomic and environmental benefits. Highway pavement is an integrated system comprising of layers of superimposed processed materials above the natural compacted sub-grade soil which function primarily to distribute the wheel load of vehicle to the supporting soil in such a way that the bearing capacity of the supporting soil is not exceeded (Kadeyali, 1997). Pavement types ranges from flexible, rigid to semi-rigid and composite pavement (Kadeyali, 1997).

One of the widely used pavement types in Nigeria is flexible pavement. Performance of flexible pavement is influenced by a number of factors ranging from traffic load, moisture, construction quality, strength of supporting soil, maintenance and environmental factors (Adlinge and Gupta, 2014). One of the root causes of the aforementioned factors influencing the performance of flexible pavement during service is structural analysis and design deficiencies. Pavement design is a complex field requiring sound knowledge of both soil and paving materials and particularly their response under various loadings and environmental conditions (Ekwulo, 2017). Pavement design methods can vary and have evolved over the years in response to changes in traffic and loading conditions, construction material and procedures. Design methods have advanced from rule of thumb methods to empirical methods and hitherto, towards mechanistic approach.

Elias Egboka road is a paved 3km one-lane single carriageway located at the outskirt of Nnamdi Azikiwe University Campus that adjoins to Nwakpadolu road. After few years of construction of Elias Egboka road, there is a visible loss in the structural integrity of the wearing course which is evident by the wearing away of the asphalt course a condition known as raveling. This could be attributable to the dislodgment of the aggregate particles and also, aging of the asphalt mix. This development is a major source of concern to motorists as this distress has adversely affected riding quality resulting to vehicular damage and could deteriorate if not fully addressed.

Nwakpadolu road is an unpaved road originating from both Elias Egboka road and leading to AguAwka all situated outside the Nnamdi Azikiwe University Complex. The Nwakpadolu road is a characterized by deep depression in the forms of gullies and absence of drainage channel. This features causes severe difficulty to both pedestrians and motorist as large volume of water resulting from rainfall and discharge from drainage channel of Elias Egboka road accumulate within this depression during rainy season thereby affecting mobility. Moreover, this features manifested in the road also contribute to damage of essential parts of mobility aid (motorcycle, cars) and must be addressed.

To tackle the above mentioned problem, it is however important that both roads (Elias Egboka and Nwakpadolu) should be consolidated and designed as a single entity. The design of both roads will adopt the use of American Association of State Highway and Transportation Official (AASHTO) design method. The choice of AASHTO design method was justified by a study conducted by (Yahaya et al., 2018) on comparative study of contemporary flexible pavement design methods in Nigeria based on costs, California Bearing Ratio (CBR), Asphalt Institute and AASHTO method was adopted for the study. Key findings suggest that CBR method was recommended as the most economical method while AASHTO and Asphalt Institute methods were recommended as technically reliable and as a result, will provide the most durable pavement when constructed.

1.2 Statement of Problem

The growing deterioration of Elias Egboka road after few years of construction and difficulty experienced by pedestrians and motorist plying the Nwakpadolu road is a major source of concern. Elias Egboka road is characterized visible wearing away of the asphalt mix a condition referred to as reveling. This situation could be attributable to aging of the asphalt mix and dislodgment of aggregate particles due to heavy wheel loads which constitute a problem to motorist such as reduction in vehicular speed thereby affecting travelling time, damage to essential part of mobility aids and also affect the aesthetics requirement of the road. Nwakpadolu road originating from Elias Egboka road is characterized by large depression resulting to accumulation of surface water through rainfall and discharge from Elias Egboka road. The presence of this depression in the form of gullies has not only caused driving difficulty but weaken the strength of the existing soils. Therefore, it is extremely important that both roads should be consolidated and designed as a single entity such as driving difficulty experienced by motorist resulting from several factors is minimized.

The study will therefore carry out the design and detailing of Elias Egboka-Nwakpadolu roads using AASHTO design method.

1.3 Aim and Objectives of Study

The aim of the study is to carry out the analysis and design of Elias Egboka-Nwakpadolu roads using AASHTO design method while the objectives include:

1. To undertake a geotechnical assessment of the subgrade soil.
2. To conduct traffic assessment necessary for the pavement design.
3. To undertake the structural design of the pavement using AASHTO design method.
4. Drawing and detailing of the cross-sections of designed flexible pavement layers using Archicad software.

1.4 Scope of Study

The study is essentially centered at analysis and design of Elias Egboka-Nwakpadolu roads located at the outskirt of Nnamdi Azikiwe University, Awka, Anambra State. Nigeria. Traffic volume count survey so as to estimate the average daily traffic (ADT) will be done manually. Index and CBR properties of sample collected at different location within the study area will be determined. Some of the laboratory test to be conducted so as to determine these properties include: Sieve analysis test, Atterberg limit (plastic and liquid limit) test, Compaction and California bearing ratio test. The roads will be designed using AASHTO method and thickness of the pavement layers will be specified. Detailing of the designed pavement layers will also be presented using Archicad software.

1.5 Significance of Study

Key findings obtained from the analysis and design of Elias Egboka-Nwakpadolu roads using AASHTO method will be significant in the following ways:

1 Promote the use of AASHTO design method as against the CBR method preferred by pavement design engineers in highway construction industry for the design of flexible pavement.
2 Tackle issue of flexible pavement distresses resulting from structural analysis and design deficiency.
3 Promote the use of computer aided detailing in presentation of design outcome.
4 Partly serve as reference for subsequent design and construction of flexible pavement within the study environment.
5 Promote the importance of efficient analysis and design in the construction of flexible pavement.

CHAPTER TWO

LITERATURE REVIEW

2.1 Overview

An arrangement comprising of layers of overlay materials above the usual sub-grade which acts as foundational layer is referred to as pavement (Kadeyali, 1997). The mechanism of load transfer in a pavement structure regardless of the pavement types is surface or wearing course to base course to sub-base and ultimately to the natural compacted sub-grade which acts primarily as the pavement foundation, this sub-grade thereafter, transfer the load to the underlying soil or soil of sufficient bearing capacity (Kadeyali, 1997). The bearing capacity of the underlying soil must not be exceeded otherwise, failure of the pavement might occur thereby compromising the design life of such pavement design (Neeraj and Kumar, 2019). The pavement structure is expected to posses' desirable characteristics for it to perform satisfactorily during its service life and this feature includes: adequate skid resistance, durability, favorable light reflecting characteristics, minimum noise pollution, good riding surface, long design life with low repair cost, impermeable surface and structural efficiency (Kadeyali, 1997).

The primary aim of providing superimposed layers in a pavement structure is to ensure that the stress generated due to traffic load are considerably reduced, such that it is transferred to a soil of sufficient bearing capacity (Neeraj and Kumar, 2019). Flexible and rigid pavements are the commonly used type of pavement structure (Kadeyali, 1997). Flexible pavement are usually composed of bitumen (binder), aggregate (crushed gravel and sand) and filler materials (quarry dust or stone dust) with the bitumen acting as the most chemically active and energetically expensive constituents while rigid pavement are composed of aggregate (sand and crushed rock), binder (cement), water and reinforcement with the cement acting as the most chemically active and energetically expensive constituents (Kadeyali, 1997).

Rigid pavement possess flexural rigidity such that the stress generated due to traffic load are not transferred from grain to grain to the lower layers but rather depends solely on the concrete slab which transfers the load to a wider area in accordance with the design (Kadeyali, 1997). However, in flexible pavement wheel load transfer does not depend on the flexural strength but rather depends on the grain to grain contact of the aggregate (Kadeyali, 1997). Therefore the flexural strength of flexible pavement is relatively low compared to that of the rigid pavement. This chapter will therefore focus essentially on design of Elias Egboka-Nwakpadolu roads using AASHTO method.

2.2 History of Road Evolution

2.2.1 Road Transport and Societal Needs

Movement of people and goods have been an eternal need through the centuries, continuous and consistent need for survival have coerced people to migrate from one place to another either to hunt, explore new land or shifting shelter. Some of these activities have forced mankind to construct safe and secure trackways through the years, to stone roads and thereafter modern highways (Britannica Encyclopedia, 2012). Taking a retrospect as regards the history of roads, it is evident that technological development, but also every extension of the pre-existing network, took place under stable social conditions, especially during thriving states: Persian reign, Roman Empire, Byzantine Empire. In Europe, the extended network of Roman roads was practically abandoned during the middle Ages without any systematic maintenance or upgrading recorded. In fact, since the fall of the Roman Empire, in the fifth century, till the end of the eighteen century, there was no progress in road technology and only a limited extension of pre-existing networks in Europe. Following the Renaissance, and mainly during the age of Enlightenment, commerce and mobility of people strived to regain a place in the list of human activities. Indeed, gradual technological improvements in the seventeenth and eighteenth centuries saw increased commercial traffic and improved vehicles. These factors created an incessant demand for better roads, a real challenge to road engineers and scientists of the period.

2.2.2 Ancient Pathway, the Need of Mobility

The means of movement for people over centuries have been the ancient pathway. This pathway was used as route for travelling during the middle bronze era and era where metal tools was used to process stone. Whether the roads were used by wild animals for passage or whether from inception was shaped by men still holds some iota of reservation. However, after many centuries, the nature of surface and breadth of the path have restricted and reserved mobility to travelers, soldiers and horsemen under risky conditions (Egnatia, 2014). Bumps and holes were eliminated to produce an earth pavement as a result of the continuous and consistent movement of people. Subsequently, wheel vehicle was invented which necessitated the demand for better road.

2.2.3 Stone Road, the Need of Accessibility

The stone road was referred to as the road of the second generation and was said to have rule for approximately 4,000 years. The Minoans on the island of Crete, in Greece, constructed a 50 km road from Gortyna on the south coast, over the mountains at an elevation of about 4,300 feet (1,300 m), to Knossos on the north coast at about 1000BC. The roadway took account of the necessity of drainage by a crown throughout its length and even gutters along certain sections. The pavement, which was approximately 12 feet wide, comprised of sandstone bound by a clay–gypsum mortar. The surface of the central portion comprised of two rows of basalt slabs, 2 inches thick. The centre of the roadway seems to have been used for foot traffic and the edges for animal and carts.

The Persian Royal Road was an ancient roadway in the Middle East joining the royal city of Susa in Mesopotamia to Sardes in Asia Minor. The road was originally drawn by the Assyrians and used for centuries as a broad, earth trackway. It was in the fifth century BC that King Darius rebuilt it and built a high-performing stone road.

A unique stone-made road during the classical period was the ancient Diolkos of Corinth, in Greece. The ancient Diolkos was a 6 km stone trackway which allowed the overland transfer of cargos and boats across the Isthmus of Corinth, thus giving out the long and dangerous navigation round the peninsula of Peloponnese. It is supposed that the Diolkos had been constructed as early as eighth century BC and re-built by Periander, tyrant of Corinth, in the sixth century BC. Having a differing breadth from 4 m to 6 m, the paved trackway had two deep parallel grooves, 1.6 m apart, to ease movement of transporting wheeled carts. This pioneer form of a guided movement along a paved road is regarded to be the forerunner of the railway.

Romans have been the great road builders of the antiquity. Under the Roman Empire, a dense and extended network of roads was paved, covering, practically, the entire area of the conquered land. They provided effective and efficient means for the overland movement of armies, officials and civilians, and the inland carriage of official communications and trade goods. Romans introduced, in the art of road making, several innovative principles that remained acceptable and valuable until modern times. The first principle defined the general layout, the road itinerary. Striving to prevent their routes from flooding, they drew track ways uphill. At the same time, they were concerned about snow and ice, so they did not go up too high and chose an intermediate itinerary (Dulac and Cuenot, 2012).

The pavement structure was also a technical accomplishment. The pavement comprised of three separate layers: the first (lower) layer stood for the foundation of the whole structure. The second layer from sand and gravel constituted a resistant leveling course to bear the slab stones of the upper layer (summum dorsum). These hexagonal stones of the upper layer pinned in a stable, partly deformable material were very resistant and hardly breakable. In rare cases of pavement distress, repair by replacement of several stones seemed a rather easy task. Special care was also paid to the shape of the pavement surface. The crowned form facilitated surface drainage, while the raised edge stones prevented deviation of moving carriages.

In northern Greece, Via Egnatia was the most important road built by the Romans. Built up by Gaius Egnatius on a formerly existing trackway, Via Egnatia constituted the most important link from the Adriatic Sea to the Bosphorus, traversing Illyria, Macedonia and Thrace. Along the 860 km of stone-paved road, Via Egnatia was fully equipped by milestones, landmarks, post stations and roadhouses to offer shelter to soldiers and travelers. It remained an overland link of major significance during the early Byzantine times and fell in decline after the sixth century.

2.2.4 Road in the Age of the Automobile (The Smooth Road)

The development of the art of road building has been agonizingly slow through the years. In fact, from the Greek and Roman stone road to the traditional asphalt road of the nineteenth century, little progress has been recorded. Reasons must be looked for in the societal structure and its evolution. For quite a long period of history, people were mostly organized in city-states, fairly self-sufficient, exhibiting limited need for trade and movement. A road in good shape would invite potential enemies to conquer and ravage the city. During the middle Ages, the organization of the population in small states (feudalism), as well as aggressiveness and threat to life inflicted by diseases, calamities and warfare between states, did not constitute acceptable ground to road technological development. Movement of individuals was judged, in most cases, needless and potentially harmful, while pre-existing routes were deserted and left to perish. The very few cases of development of road network in Western Europe under the reign of Charlemagne, and in Eastern Europe where the Varangian Road (its main part was water road) served as a transnational route of commerce, do not change the general scenery of decline.

It was not until the end of the eighteenth century that the innovative ideas of Tresaguet, Telford and Mc-Adam altered the layout and the usual practice in road building. Crushed stone materials and grading of aggregates were the main principles introduced by the aforementioned road engineers. Soil enhancement and drainage assets were also incorporated in the new road design at the end of the nineteenth century. Crushed aggregate of 0/50 (mm), instead of large stones of 50cm, were applied on the pavement surface. In the beginning of the nineteenth century, several kilometers of roads were constructed in Europe and America on the basis of these revolutionary principles. Road ride-ability proved to be acceptable but the issues of dust and mud had to be faced and dealt with. In fact, industrialization, social progress and exploding trade pushed forward to the era of ‘smooth road’, the road of the third generation.

During the late 1800s, and especially in the beginning of the twentieth century, the asphalt road appeared and developed rapidly. Gradually, the asphalt pavement surface, starting from France and the United States (US), was made the rule for all new important overland links. In the US, the famous Route 66, leading from Chicago to California with a length of 3,700 km, bore an asphalt pavement surface and was completed in 1937. This new generation of road surfacing, offering incessant traffic-ability and ride comfort, was the key to the development of new roads during the later nineteenth and the twentieth century’s. At the same time, inventive road engineers suitably adapted technological achievements to produce some amazing works of art in road making. To date, and despite the ongoing and persistent pressure for highways and freeways, the traditional two-lane asphalt roads of the third generation still are the rule. In most countries, the national and the local two-lane road networks constitute the greater part of the total length of roads, whereas in some countries, freeways simply do not exist. No matter the transport needs for better and safer roads, the old two-lane network must not be abandoned, since it will make up a multipurpose, reliable and safe transport infrastructure for many years.

2.2.5 The Age of Motorway

The industrial development and the need for trade in Europe after World War I led to the exploration of improved transport infrastructure. The first application of the intrinsic principles of the motorway was performed in Italy (Vogias, 1992). The ‘autostrada’ linking Milan to Varese, over a length of 42 km, had limited access and toll payment. Nevertheless, this highway had no additional lanes for safe overtaking. An appreciable rise in traffic volume during the third decade of the twentieth century pushed forward the funding of the project. The highway of a length of 20 km was completed in four years (1929–32) and had two lanes per direction and no level crossings. No median lane or separation barrier was initially installed and this resulted in numerous accidents due to overtaking maneuvers. This project constituted a starting point for Germany to construct an extended network of 2x2 highways with separated traffic. The representative section comprised of dual carriageways, each one of a width of 7.5 m and a median lane of 5 m. By 1942, the network was stretched to a total length of 6,500 km (Britannica Encyclopedia, 2012).

The first complete motorway was built in the US. It was in 1940 that the ‘Pennsylvania Turnpike’, of a length of 320 km, was opened to traffic. This was the first full application of the concept of the motorway: separated traffic; no level crossings; controlled access; two lanes per direction; and a median strip. Smooth grades and sweeping curves completed the arrangement of the first motorway in the history of roads. The Pennsylvania Turnpike was later extended to a length of 500 km linking Philadelphia to Ohio.

A modern motorway is defined with regard to the basic concern for safer, faster and easier driving. Median lane, removal of grade crossings and controlled access provide safety, while additional traffic lanes and smooth grades contribute to fast driving. Innovative information devices and service stations make travel more comfortable and pleasant. While the great challenge of completion of the ongoing concession projects is always firm and insistent, the issue of effectively and efficiently operating and managing this network of motorways subsequently arises. As sources of financing road repair become rarer, it is of utmost importance to provide reliable plans for the welfare of the transport infrastructure, this being a major issue for all industrialized countries. The stake for the roads of the new generation lies mainly in the domain of operation and repair, far from searching new concepts in highway construction to accomplish a clearly improved performance.

2.3 Flexible Pavement

A flexible pavement is one of the commonly constructed pavement structures due to its relative economy in construction and service life (Neeraj and Kumar, 2019). In a flexible pavement, stresses generated by wheel load are transmitted to a lower layer (usually sub-grade) by grain to grain transfer mechanism as it does not rely on its flexural strength for load transfer (Kadeyali, 1997). Flexible pavement consist of a bituminous surface placed over a layer of granular material and a layer of suitable mixture of fine and coarse aggregate resting on the natural compacted sub-grade acting as the pavement foundation. Flexible pavements are mostly of bituminous material acting as the binding agent such that it remains in contact with the underlying materials even when minor deficiencies occurs.

Flexible pavements are further divided into three sub-groups namely: high type, intermediate and low type (Kadeyali, 1997). High type pavements have surface course that sufficiently support the anticipated traffic load without visible distress due to fatigue and they are not vulnerable to unfavorable weather condition. Intermediate type pavements have surface course that moderately support anticipated traffic load with likelihood of the pavement developing distress and this mainly due to the quality of the treated surface as their treated surface are low compared to the high type pavements. Low type pavements are highly vulnerable to environmental conditions, marginally support expected traffic load with high possibility of distress been developed (Neeraj and Kumar, 2019). This type of flexible pavement are used mainly for low cost roads and have wearing course ranging from untreated to loosed natural materials to surface treated earth.

2.3.1 Structural Component of a Flexible Pavement

The structural component of a flexible pavement includes: sub-grade or prepared road bed, sub-base, base course and wearing course, all these components are superimposed on each other and perform distinct functions (Kadeyali, 1997). The performance of each component largely dictates the performance of the flexible pavements and as a result proper evaluation of these components is required for effective pavement performance and service life.

2.3.1.1 Sub-grade

The sub-grade is one of the most important structural components of a flexible pavement. It is a natural compacted earth lying beneath other layers and act as interface between the pavement and the underlying soil. It is referred to as the pavement foundation as it transmits the expected traffic load to the underlying soil of sufficient bearing capacity (Neeraj and Kumar, 2019). The sub-grade consists mainly of earth material and must be compacted to the desirable density, near the optimum moisture content. The stiffness is referred to as the degree of resistance upon loading and it depends primarily upon the soil properties, existing stress conditions and soil stress history (Susanka, 2016).

2.3.1.2 Sub-base

The sub-base is a layer of material above the sub-grade and beneath the base course and they function primarily to provide structural support, improve drainage and reduce intrusion of fines from the sub-grade to the pavement structure (Neeraj and Kumar, 2019). The structural capacity of the wearing course can be used to determine whether a sub-base can be dispensed with. If the pavement is constructed over a high quality wearing surface, stiff sub-grade may not require additional features offered by a sub-base layer and in such condition, sub-base may be dispensed with.

2.3.1.3 Base-course

The base-course is a layer of materials directly above the sub-base and beneath the wearing or surface course, it reduces the magnitude or intensity of the load transmitted to the underlying pavements layers (Neeraj and Kumar, 2019). It provides additional load distribution mechanism and contributes to sub-surface drainage. It may be composed of crushed stone, crushed slug and other untreated or stabilized materials.

2.3.1.4 Wearing Course

Wearing course is a layer directly in contact with traffic loads and generally contains superior quality materials. They are usually constructed with dense graded asphalt concrete (Neeraj and Kumar, 2019). The functions and requirements of this layer are: It provides characteristics such as friction, smoothness, drainage. Also, it prevents the entrance of excessive quantities of surface water into the underlying base, sub-base and sub-grade (Kadeyali, 1997). It must be tough to resist the distortion under traffic and provide a smooth and skid resistant riding surface, it must be water proof to protect the entire base and sub-grade from the weakening effect of water (Kadeyali, 1997).

2.3.2 Typical Coat used in Flexible Pavement

Typical coats of a conventional flexible pavement include seal coat, prime coat and tack coat (Kadeyali, 1997). Seal coat is a thin surface treatment used to water-proof the surface and to provide skid resistance while tack coat is a very light application of asphalt, usually asphalt emulsion diluted with water, it provides proper bonding between two layers of binder course and must be thin, uniformly cover the entire surface and set very fast (Kadeyali, 1997). Prime coat is an application of low viscous cutback bitumen to an absorbent surface like granular bases on which binder layer is placed (Kadeyali, 1997). It provides bonding between two layers. Unlike tack coat, prime coat penetrates into layer below, plugs the voids, and forms a water tight surface.

2.4 Flexible Pavement Design History

Pavement design is a complex field requiring knowledge of both soil and paving materials, and especially, their responses under various loadings and environmental conditions. Pavement design methods can vary, and have evolved over the years in response to changes in traffic and loading conditions, construction materials and procedures. Design methods have advanced from rule-of-thumb methods, to empirical methods and at present, towards a mechanistic approach.

In the United States, the majority of pavement designers use the AASHTO (American Association of State Highway and Transportation Official) Guide for design of Pavement Structures (AASHTO, 1993). The AASHTO Guide was developed from empirical performance equations based on observations from the AASHTO Road Test conducted in Illinois from October, 1958 to November, 1960. Many significant changes in loading conditions, construction materials and methods, and design needs have occurred since the time of AASHTO Road Test, prompting development of new mechanistic-empirical design procedures. This procedure allows the designer to consider current site conditions such as realistic loading, climatic factors such as temperature and moisture, material properties and existing pavement condition in the design of a new pavement, rehabilitation of an existing pavement, or evaluation of an existing pavement. This approach is described in more details in the Guide for mechanistic-empirical Design of New and Rehabilitated Pavement Structures (NCHRP, 2004). Additionally, mechanistic-empirical design procedure was developed such that improvement could be made as technology advances.

Empirical methods of analysis are derived from experimental data and practical experience. The mechanistic-empirical (M-E) design approach considers the three necessary elements of rational design (Yoder and Witczak, 1975). The element of rational design include: an assumed failure or distress parameter predictive theory, evaluation of material properties in relationship to the theory selected and relationship determination between the performance level desired and the appropriate parameter magnitude. The mechanistic-empirical design approach applies engineering mechanics principles to consider these rational design elements.

The initial phase of the mechanistic design approach consists of proper structural modeling of pavement structures (NCHRP, 2004). Pavement is modeled as multilayered elastic or viscoelastic on elastic or viscoelastic foundation. These models are used in analysis to predict critical pavement responses (deflections, stresses and strains) due to traffic loading and environmental conditions for selected trial or initial design. The accuracy of the chosen model is validated by data from controlled vehicle tests or other types of tests where actual loading and environmental conditions are simulated. Where predicted values agree with measured values, the level of confidence in the model increases with increase data available for validation. Once an accurate structural response model is developed, the responses are input into distress models to determine pavement damage throughout the specific design period. Failure criteria are then evaluated, and an iterative process continues until a final design is reached.

2.5 Flexible Pavement Design Principle

Before the 1920s, pavement design consisted fundamentally of defining the thickness of layered materials that would provide strength and protection to a soft sub-grade. Pavements were designed against sub-grade shear failure. Engineers used their experience based on successes and failures of past projects. As experience evolved, several pavement design methods based on subgrade shear strength were developed. Ever since, there has been a change in design criteria as a result of increase in traffic volume. As important as providing sub-grade support, it is equally important to evaluate pavement performance through ride quality and other surface distress that increase the rate of deterioration of pavement structure. For this reason performance became the focus of pavement designs. Methods based on serviceability (an index of the pavement service quality) were developed based on test track experiments. The AASHTO Road Test in 1960s as a seminal experiment from which the AASHTO design guide was developed. Methods developed laboratory test data or test track experiments in which model curves are fitted to data are typical example of empirical methods. Although they may exhibit good accuracy, empirical methods are valid for only the materials and climate conditions for which they were developed.

Meanwhile, new materials started to be used in pavement structures that provide better sub-grade protection, but with their own failure modes. New designs criteria were required to incorporate such failure mechanisms such as fatigue cracking and permanent deformation in the case of asphalt concrete. The Asphalt Institute method (Asphalt Institute, 1982, 1991) and the Shell method (Claessen et al., 1977) are examples of procedures based on asphalts concrete’s fatigue cracking and permanent deformation failure modes. These methods were the first to use linear elastic theory of mechanics to compute structural response in combination with empirical models to predict number of loads to failure for flexible pavements. The problem in the use of the elastic theory is that pavement material do not exhibit the simple behaviour assumed in isotropic linear elastic theory. Nonlinearities, time and temperature dependency, and anisotropy are some of the complicated features often observed in pavement materials. Therefore to predict pavement performance mechanistically, advanced modeling is required. The mechanistic design approach is based on the theories of mechanics and relates pavement structural behaviour and performance to traffic loading and environmental influences. Progress has been made on isolated cases of mechanistic performance prediction problem, but the reality is that fully mechanistic methods are not yet available for practical pavement design (Schwartz and Carvalho, 2007).

2.6 Review of Flexible Pavement Design Methods

Studies in pavement engineering have shown that the design procedure for highway pavement is either empirical or mechanistic. An empirical approach is one which is based on the results of experiments or experience or both. This means that the relationship between design inputs and pavement failure were arrived at through experience, experimentation or a combination of both. The mechanistic approach involves selection of proper materials and layer thickness for specific traffic and environmental conditions such that certain identified pavement failure modes are minimized. The mechanistic approach involves the determination of material parameters for the analysis, at conditions as close as possible to what they are in the road structure. The mechanistic approach is based on the elastic or visco-elastic representation of the pavement structure. In mechanistic design, adequate control of pavement layer thickness as well as material quality are ensured based on theoretical stress, strain or deflection analysis. The analysis also enables the pavement designer to predict with some amount of certainty regarding the life of the pavement (Schwartz and Carvalho, 2007). Below are outline and description of the pavement design methods.

2.6.1 CBR Design Method

The almost universal parameter used to characterize soils for pavement design purpose is the California Bearing Ratio (CBR). This empirical index test was abandoned in California over 50 years ago but, following its adoption by the US. Corps of Engineers in World War II, it was gradually accepted World-wide as the appropriate test (Brown, 1997). Given that the test is at best, an indirect measurement of un-drained shear strength and the pavement design requires knowledge of soil resilience and its tendency to develop plastic strains under repeated loading, the tenacity exhibited by generation of highway engineers in regard to the CBR is somewhat surprising. Jim

Porter, a Soil Engineer for the State of California, introduced the "Soil Bearing Test" in 1929 commented nine years later, that the bearing values are not direct measure of the supporting value of materials (Porter, 1938). In recognition that the CBR design curves give a total thickness of pavement to prevent shear deformation in the soil. CBR is an index of shearing strength (Turnbull, 1950). The shear strength of soil is not of direct interest to the road engineer, the soil should operate at stress levels within the elastic range (Brown, 1997). The pavement engineer is therefore more concerned with the elastic modulus of soil and the behaviour under repeated loading.

The CBR method of pavement design is an empirical design method and was first used by the California Division of Highways as a result of extensive investigations made on pavement failures during the years 1928 and 1929 (Corps of Engineers, 1958). To predict the behaviour of pavement materials, the CBR was developed in 1929. Tests were performed on typical crushed stone representative of base course materials and the average of these tests designated as a CBR of 100 percent. Samples of soil from different road conditions were tested and two design curves were produced corresponding to average and light traffic conditions. From these curves the required thickness of sub-base, base and surfacing were determined. The investigation showed that soils or pavement material having the same CBR required the same thickness of overlying materials in order to prevent traffic deformation. So, once the CBR for the sub-grade and those of other layers are known, the thickness of overlying materials to provide a satisfactory pavement can be determined. The US corps of Engineers adopted the CBR method for airfield at the beginning of the Second World War, since then, several modifications of the original design curves have been made (Oguara, 2005). Some of the common CBR design methods include the Asphalt Institute (Asphalt Institute, 1981) method, the National Crushed Stone Association (NCSA) design method (NCSA, 1972), the Nigerian (CBR) design procedure (Highway Manual, 1973) etc.

2.6.2 Asphalt Institute Methods

Although the Asphalt institute has developed a new thickness design procedure based on the mechanistic approach (Asphalt Institute, 1981), the original asphalt institute thickness design procedure is based on the concept of full depth asphalt that is using asphalt mixtures for all courses above the sub-grade or improved sub-grade. Traffic analysis is in terms of 80kN equivalent single axle load in the form of a Design Traffic Number, DTN. The DTN is the average daily number of equivalent 80kN single-axle estimated for the design period. The CBR, Resistance value or bearing value from plate loading test is used in sub-grade strength evaluation.

2.6.3 National Crushed Stone Association Methods

The National Crushed Stone Association (NCSA) empirical design method (NCSA, 1972) is based on the US Corps of Engineers pavement design. Traffic analysis is based on the average number of 80kN single-axle loads per lane per day over a pavement life expectancy of 20 years. The method incorporates a factor of traffic in the design called Design Index (DI).

2.6.4 Nigeria CBR Methods

The Nigerian (CBR) design procedure is an empirical procedure which uses the California Bearing Ratio and traffic volume as the sole design inputs. The method uses a set of design curves for determining structural thickness requirement. The Nigerian (CBR) design method is a CBR-Traffic volume method, the thickness of the pavement structure is dependent on the anticipated traffic, the strength of the foundation material, the quality of pavement material used and the construction procedure. This method considers traffic in the form of number of commercial vehicles/day exceeding 29.89kN (3 tons). Sub-grade strength evaluation is made in terms of CBR.

2.6.5 AASHTO Pavement Design Methods

The AASHTO method (AASHTO, 1993) was based on the AASHTO Road Test from the late 1950s, is the most widely used pavement design method today. The AASHTO design equation is a regression relationship between the number of load cycles, pavement structural capacity, and performance measured in terms of serviceability. The concept of serviceability was introduced in the AASHTO method as an indirect measure of the pavement’s ride quality. The serviceability index is based on surface distress commonly found in pavements. The AASHTO (1993) method has been adjusted several times over the years to incorporate extensive modifications based on theory and experience that allowed the design equations to be used under conditions other than those of the AASHTO Road Test.

The AASHTO Guide for Design of Pavement Structures is the primary document used to design new and rehabilitated highway pavements. The Federal Highway Administration's 1995-1997 National Pavement Design Review found that some 80 percent of states use the 1972, 1986, or 1993 AASHTO Guides (AASHTO, 1972; 1986; 1993), of the 35 states that responded to a 1999 survey (Newcomb and Birgisson, 1999), 65% reported using the 1993 AASHTO Guide for both flexible and rigid pavement designs.

All versions of the AASHTO Design Guide are empirical methods based on field performance data measured at the AASHO Road Test in 1958-60, with some theoretical support for layer coefficients and drainage factors. The overall serviceability of a pavement during the original AASHO Road Test was quantified by the Present Serviceability Rating (PSR; range = 0 to 5), as determined by a panel of highway raters. This qualitative PSR was subsequently correlated with more objective measures of pavement condition (e.g., cracking, patching, and rut depth statistics for flexible pavements) and called the Pavement Serviceability Index (PSI). Pavement performance was represented by the serviceability history of a given pavement by the deterioration of PSI over the life of the pavement. Roughness is the dominant factor in PSI and is, therefore, the principal component of performance under this measure.

2.7 Review of Flexible Pavement Design Parameters

2.7.1 Input Parameters

2.7.1.1 Sub-Grade Resilient Modulus (M_R)

Resilient modulus is an important parameter for design of flexible pavements. It generally corresponds to the degree to which a material recovers from external shock or disturbance. This property of the material is actually an estimate of the modulus of elasticity (Davich et al., 2004). In case of slowly applied loads, slope of the stress- strain curve in a linearly elastic region yields modulus of elasticity where for rapidly applied loads (traffic loads), this would yield resilient modulus (M_R). The resilient modulus can be expressed as:

$$M_R = \frac{\emptyset}{Er} \quad \text{..} \; (2.0)$$

Resilient modulus describes the mechanical response of a pavement sub-grade to applied load and hence, it is regarded to be an essential parameter for pavement design (Durham et al., 2003). Knowledge of the resilient modulus for the sub-grade soil and the pavement materials can be used in ascertaining the structural behaviour of the pavement against traffic load (Siwosoebrotho et al., 2005). However, obtaining resilient modulus is a very difficult task and it can be determined by laboratory testing of the sub-grade soil (Davich et al., 2004: Durham et al., 2003: Siwosoebrothoet al., 2005). As such, the long term pavement test protocol (LTTP) is widely used for determining resilient modulus which in turn requires dynamic triaxial testing on cylindrical cores (Durhamet al., 2003). Several other modified research program (NCHRP) method and federal highway administration (FHWA) method are also employed for determining sub-grade resilient modulus.

2.7.1.2 Analysis Period (AP)

Analysis period is usually equal to the functional period for which the pavement will have to deliver its functional service (FMWH, 2013). It is fundamentally determined by factors such as speed, travelling time, delays, freedom to change position in the traffic stream and driving comfort (FMWH, 2013). According to AASHTO (1993), analysis period refers to the overall duration that the design strategy must cover. It is identical to the performance period however; realistic performance limitation may require planned rehabilitation within the desired analysis period in which case, the analysis period may encompass multiple performance periods. Analysis period in this context is synonymous with the design life of flexible pavement recommended in AASHTO 1993 Guide. AASHTO recommendations for analysis period for different types of roads are summarized in Table 2.0.

Table 2.0: Guidelines for Length of Analysis Period (AASHTO, 1993)

Highway Conditions	Analysis Period (Years)
High – Volume Urban	30 – 50
High – Volume Rural	20 – 50
Low – Volume Paved	15 – 25
Low – Volume Aggregate Surface	10 – 20

2.7.1.3 Performance Period (PP)

Performance period refers to the time a pavement design is intended to last before it needs rehabilitation (AASHTO, 1993). It is equivalent to the time elapsed as a new reconstructed or rehabilitated pavement structure deteriorates from its initial serviceability to its terminal serviceability.

2.7.1.4 Equivalent Single Axle Loads (ESALs)

18 kips equivalent single axle loads means the measure used in pavement design based on the number of 80kN equivalent single axle loads associated with vehicular traffic on a certain road in a particular period of time (AASHTO, 1993). Highway pavement structures are designed to withstand a number of standardized loadings derived from a known mix of truck types in the traffic

stream and this standardized vehicle loadings used in pavement design are termed equivalent single axle load (ESALs). The pavement structure will be subjected to a specific number of equivalent single axle load (ESALs) during its design life. Traffic analysis requires the evaluation of initial traffic volume, traffic growth, directional distribution and traffic type (AASHTO, 1993). According to (AASHTO, 1993) design guide on cumulative 18 kip (80kN) equivalent single axle loads (ESALs). Equivalent single axle loads (ESALs) can be calculated using the following equation.

$$\text{ESALs} = (\text{ADT0})\,x(\text{T})x\,(\text{Tf})\,x(\text{G})\,x(\text{D})\,x(\text{L})\,x(365)x\,(\text{Y}) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.1)$$

Where:

ADT0 = Average daily traffic at the start of the design period.

T = Percentage of truck in the ADT.

Tf = Truck factor or the number of 18 kip ESAL per truck.

G = traffic growth factor.

D = directional distribution factor.

L = lane distribution factor.

Y = Design period in years.

2.7.1.5 Reliability

Design reliability is defined as the probability that a pavement section will perform satisfactorily given the design period (AASHTO, 1993). It account for uncertainties in traffic loading, environmental conditions and construction materials (AASHTO, 1993). The AASHTO design methods account for this uncertainties by incorporating a reliability level to provide a factor of safety and thereby increase the probability that the pavement will perform satisfactorily as intended over its design life. The levels of reliability recommended by (AASHTO, 1993) for various classes of roads are summarized in Table 2.1.

Table 2.1: Suggested Levels of Reliability for Various Functional Classifications (AASHTO, 1993).

Functional Classification	Recommended Level of Reliability (%)	
	Urban	Rural

Interstate and other Freeways	85 – 99.9	80 – 99.9
Principal Arterials	80 – 99	75 – 95
Collectors	80 – 95	75 – 95
Local	50 – 80	50 – 80

The reliability level is not included directly in the (AASHTO, 1993) design equation rather, it is used to determine the standard normal deviate (Z_R). Values of standard normal deviate (Z_R) corresponding to selected levels of reliability are summarized in Table 2.2

Table 2.2: Standard Normal Deviate for Various Levels of Reliability (AASHTO, 1993).

Reliability (%)	Standard Normal Deviate (Z_R)	Reliability (%)	Standard Normal Deviate (Z_R)
50	0.000	93	-1.476
60	-0.253	94	-1.555
70	-0.524	95	-1.645
75	-0.674	96	-1.751
80	-1.841	97	-1.881
85	-1.037	98	-2.054
90	-1.282	99	-2.327
91	-1.340	99.9	-3.090
92	-1.405	99.99	-3.750

The AASHTO design equation also requires specification of the overall standard deviate (S_0). For flexible pavement design, values of S_0 typically range between 0.35 and 0.50 with a value of 0.45 commonly used for design purposes (AASHTO, 1993).

2.7.1.6 Serviceability

Serviceability is quantified by the present serviceability index (PSI). Although present serviceability index (PSI) theoretically ranges between 5 and 0, the actual range for flexible pavement design is between 4.5 to 1.5 (AASHTO, 1993). The initial serviceability index (P_0) corresponds to the road condition immediately after construction. A typical value of P_0 for flexible pavement is 4.2. The terminal serviceability will be tolerated before rehabilitation or reconstruction becomes necessary. A terminal serviceability index of 2.5 or higher is recommended for design of major highways. Thus allowable serviceability loss due to traffic for flexible pavement can be expressed as follows:

$\Delta PSl = PE - P0 = 4.2 - 2.5 = 1.7$ **…………………………………..** (2.2)

2.7.2 Output Parameters

2.7.2.1 Structural Layer Coefficient (a_1, a_2, a_3)

The layer coefficient represents the strength of the material, it is a primary variable obtained in the type of material used for the construction of each pavement layers (AASHTO, 1993). Structural layer coefficients are typically determined empirically based on the performance of the material. (AASHTO, 1993) design guide recommended specific structural layer coefficient values for commonly used pavement materials. These coefficients are presented in Table 2.3 below.

Table 2.3: Recommended Values of Structural Coefficient Layers for Pavements Materials (AASHTO, 1993).

Pavements Materials	Structural Coefficient Values
Hot Mix Asphalt (HMA)	0.44
Road Mix (Low Stability)	0.20
Aggregate Base	0.13
Engineered Fill	0.10

2.7.2.2 Drainage Coefficient (m_2, m_3)

Drainage coefficient value assigned to a pavement layer represents its relative loss of strength due to drainage characteristics and exposure to moisture saturation (AASHTO, 1993). Layers that drains slowly and are often saturated would have a lower drainage coefficient while layers that drains quickly and do not become saturated would have a relatively higher drainage coefficient. For most pavement design, drainage coefficient recommended by most design methods are unity indicating normal drainage coefficient.

2.7.2.3 Structural Number (SN)

Structural number is a critical element of flexible pavement design equation. It represents the overall structural requirement needed to sustain the traffic loads anticipated during the service life of the pavement (AASHTO, 1993). The required structural number depends on a combination of existing soil supports, total traffic loads and environmental condition. Although design equation can be used in different ways depending on the inputs available, one of the common applications is to effectively determine the structural number. Determination of the structural number is valuable in specifying appropriate thickness for each of the pavement layers (AASHTO, 1993). It is the major output in the flexible pavement design equation. The structural number can be represented mathematically using the relation below:

$$SN = a1D1 + a2D2m2 + a3D3m3 \quad \text{.............} \quad (2.3)$$

Where:

D1, D2 and D3 represent the thickness of the pavement layers.

m1 and m2 represent the drainage coefficient.

a1, a2 and a3 represent the structural layer coefficients.

2.8 Review of Pavement Design Approach

2.8.1 Mechanistic Design Approach

The mechanistic design approach represents the other end of the spectrum from the empirical methods. The mechanistic design approach is based on the theories of mechanics to relate pavement structural behavior and performance to traffic loading and environmental influences. The mechanistic approach for rigid pavements has its origins in Westergaard's (Westergaard, 1926) development during the 1920s of the slab on sub-grade and thermal curling theories to compute critical stresses and deflections in a PCC slab. The mechanistic approach for flexible pavements has its roots in Burmister's (Burmister, 1945) development during the 1940s of multilayer elastic theory to compute stresses, strains, and deflections in pavement structures.

A key element of the mechanistic design approach is the accurate prediction of the response of the pavement materials - and, thus, of the pavement itself. The elasticity-based solutions by Boussinesq, Burmister, and Westergaard were an important first step toward a theoretical description of the pavement response under load. However, the linearly elastic material behavior assumption underlying these solutions means that they will be unable to predict the nonlinear and inelastic cracking, permanent deformation, and other distresses of interest in pavement systems. This requires far more sophisticated material models and analytical tools. Much progress has been made in recent years on isolated pieces of the mechanistic performance prediction problem. The

Strategic Highway Research Program during the early 1990s made an ambitious but, ultimately, unsuccessful attempt at a fully mechanistic performance system for flexible pavements. To be fair, the problem is extremely complex; nonetheless, the reality is that a fully mechanistic design approach for pavement design does not yet exist. Some empirical information and relationships are still required to relate theory to the real world of pavement performance.

2.8.2 Empirical Design Approach

An empirical design approach is one that is based solely on the result of experiment or experience. Observations are used to establish correlations between the inputs and the outcomes of a process, for example pavement design and performance. These relationships generally do not have firm scientific basis, although they must meet the tests of engineering reasonability. Empirical approaches are often used as an expedient when it is too difficult to define theoretically the precise cause and effect relationships of a phenomenon.

The principal advantages of empirical design approaches are that they are usually simple to apply and are based on actual real-world data. Their principal disadvantage is that the validity of the empirical relationships is limited to the conditions in the underlying data from which they were inferred. New materials, construction procedures, and changed traffic characteristics cannot be readily incorporated into empirical design procedures. The first empirical method for flexible pavement design date to the mid-1920s when the first soil classification were developed. One of the first to be published was the Public Roads (PR) soil classification system (Huang, 2004). In 1929, the California Highway Department developed a method using the California Bearing Ratio (CBR) strength test (Porter, 1938; Huang, 2004). The CBR method relates the material's CBR value to the required thickness to provide protection against subgrade shear failure. The thickness computed was defined for the standard crushed stone used in the definition of the CBR test. The CBR test was improved by the US Corps of Engineers (USCE) during the World War II and later became the most popular design method. In 1945 the Highway Research Board (HRB) modified the PR classification. Soils were grouped in seven categories (A-1 to A-7) with indexes to differentiate soils within each group. The classification was applied to estimate sub-base quality and total pavement thickness (Huang, 2004).

2.8.3 Mechanistic-Empirical Design Approach

The development of mechanistic-empirical design approaches dates back at least four decades. As its name suggests, a mechanistic-empirical approach to pavement design combines features from both the mechanistic and empirical approaches. The induced state of stress and strain in a pavement structure due to traffic loading and environmental conditions is predicted using theory of mechanics. Empirical models link these structural responses to distress predictions. Huang (1993) notes that Kerkhoven and Dormon (1953) were the first to use the vertical compressive strain on top of the sub-grade as a failure criterion to reduce permanent deformation. Saal and Pell (1960) published the use of horizontal tensile strain at the bottom of the asphalt bound layer to minimize fatigue cracking. The concept of horizontal tensile strain at the bottom of the asphalt bound layer was first used by Dormon and Metcaff (1965) for pavement design. The Shell method (Claussenet al., 1977) and the Asphalt Institute method (Asphalt Institute, 1992) incorporated strain-based criteria in their mechanistic-empirical procedures. Several studies over the past fifteen years have advanced mechanistic-empirical techniques. Most of the works, however, were based on variants of the same two strain-based criteria developed by Shell and the Asphalt Institute. The Washington State Department of Transportation (WSDOT), North Carolina Department of Transportation (NCDOT) and Minnesota Department of Transportation(MNDOT), to name but a few developed their own Mechanistic-Empirical procedures (Schwartz and Carvalho, 2007). The National Cooperative Highway Research Program (NCHRP) 1-26 project report, Calibrated Mechanistic Structural Analysis Procedures for Pavements (1990), provided the basic framework for most of the efforts made by state DOTs. WSDOT (Pierce et al., 1993).

2.9 Pavement Design Case Studies

(Yahaya et al., 2018) carried out a comparative study of contemporary flexible pavement design methods in Nigeria based on costs, California Bearing Ratio (CBR) method, Asphalt Institute methods and American Association of State Highway and Transportation Official (AASHTO) method was adopted for the study. Key findings suggest that California Bearing Ratio (CBR) method was recommended as the most economical method while American Association of State Highway and Transportation Official (AASHTO) and Asphalt Institute methods were recommended as technically reliable and as a result, will provide the most durable pavement when constructed.

In another research, Choudhary and Joshi, (2014) conducted a detailed study of CBR method for design of flexible pavement. Values of CBR of two different soil samples and their correlation design purpose of flexible pavement as per guidelines of IRC: SP: 37-2001 determined. Index properties and CBR characteristics of the two different samples of soils and traffic index was determined. Results obtained suggest that the thickness of sub-base, base course and wearing course of the two samples at 5 and 10% CBR value were 300, 250, 110mm and 200, 250 and 90mm. It was concluded that both samples meet the requirement as per guidelines of IRC: SP: 37-2001 for pavement layer thickness.

(Apata et al., 2022) conducted a study on pavement design of Ilaro – Papa Alanto highway, Ogun State Nigeria. Index, compaction and CBR properties of samples collected at failed section of both Ilaro and Papa Alanto road was determined. Present axle load, bitumen content and marshal stability of the existing asphalt samples was also determined. Asphalt Institute method was used for the design. Result obtained showed that the required combined thickness of the sub-base and base course layer of Ilaro and Papa Alanto roads were 580 and 320mm respectively. It was also deduced that the existing road pavement with thickness of 120mm was grossly inadequate for a subsoil of 3.08% CBR value and traffic count of 505.

(Meena et al., 2021) carried out a comparative study of IRC and AASHTO design methods for urban roads design. Geometric design of the study area was using Civil 3d software. Results obtained were compared from each other to ascertain the discrepancy in design outputs for both crust and geometric design. It was inferred that both AASHTO and IRC design methods do not have any substantial difference as they are relatively the same.

Kumar, (2014) carried out traffic analysis and design of flexible pavement with cemented base and sub-base. The pavement was designed for a period of 15 years and IRC 37: 2012 was used for the design. Result obtained showed that the thickness of sub-base and base course for both conventional and non-conventional materials were 230, 50 and 250 and 50mm respectively.

(Ekwulo and Agwuwamba, 2014) developed a modified Nigeria CBR method for design of cement stabilized lateritic base to low volume asphalt pavement in Nigeria. Fatigue cracking and rutting deformation was the major consideration for the development of the design method. Layered elastic procedure was used for selection of materials for specific traffic conditions. Analyses were performed for hypothetical asphalt pavement sections using the layered elastic analysis program known as EVERSTRESS. Result suggest that the developed modified Nigeria CBR method have a potential of reducing rutting deformation and fatigue cracking in pavement as the sub-grade soil modulus was relatively higher that of pavement designed with the conventional CBR method.

2.10 Summary of Findings on Flexible Pavement Analysis and Design

Table 2.4 presents the design methods, factors, consideration and findings obtained from design of flexible pavement layers undertaken by past researchers. Below is a presentation of the above stated information.

Table 2.4: Summary of Past Research on Flexible Pavement Design

Researchers	Design Methods	Consideration	Factors	Results	Remark
(Yahaya et al., 2018)	CBR, AASHTO, Asphalt Institute and Group Index.	Relative cost and Reliability.	Traffic Index, CBR of soil sample and design life.	Thickness of sub-base, base course and wearing course for CBR method were 100, 125 and 75mm at a cost of 61, 701, 412.50, 150, 260 and 150 mm at a cost of 71, 539, 240.00 for AASHTO, 120, 200 and 80mm at a	The CBR method was adjudged to be relatively economical while AASHTO and Asphalt Institute method was adjudged to be reliable since the stress- strain properties of the soil was taken into consideration.

				cost of 71, 195, 040.00 for Asphalt Institute and 100, 300 and 100mm at a cost of 67, 823, 595.00 for Group Index	
Choudhary and Joshi, (2014).	CBR	Structural reliability.	Traffic Index and CBR value of sample.	Thickness of sub-base, base course and wearing course of the two samples were 300, 250, 90mm and 200, 250 and 90mm.	Both samples meet the pavement layer thickness as specified by guidelines of IRC: SP: 37-2001.
Kumar, (2014)	IRC 37: (2012)	Durability	Traffic Index and CBR value of sample	Thickness of sub-base and base course for both conventional and non conventional materials were 230, 50 and 250 and 50mm respectively	Samples used met the requirement specified by IRC 37: 2012
(Apata et al., 2022)	Asphalt Institute method	Pavement distress investigation	Traffic index, marshal stability, CBR of samples and bitumen content.	Combined thickness of the sub-base and base course layer of both roads was 580 and 320mm respectively.	Initial pavement thickness of 120mm for the existing road was grossly inadequate to cater for increase in traffic volume.

It is against the backdrop of cost and poor structural reliability of pavement designed with method other than AASHTO that the study will carry out the analysis, design and detailing of pavement layers of Elias Egboka-Nwakpadolu roads located at the outskirt of Nnamdi Azikiwe University Awka, Anambra State. Nigeria.

CHAPTER THREE

MATERIALS AND METHODS

These section presents both the empirical and analytical methods employed to achieve the research aim. Empirical method used involves the laboratory investigation of soil samples collected at varying location and depths. Laboratory testing performed on the samples include: sieve analysis test, Atterberg limit test, Compaction and California Bearing Ratio Test. Result obtained from CBR test was used as basis for the design of Elias Egboka-Nwakpadolu roads.

3.1 Soil Sample Collection and Preparation

Soil samples collected at three different locations by method of bulk disturbed sampling. The samples are designated as A, B and C. These samples were collected at a depth of 0.5 – 1.5m with the aid of digger and shovel after removal of the top soil. These samples were stored and kept in the laboratory for test BS 1377 (1990) and the Nigeria general specification for roads and bridges (1997) was used to evaluate sample properties such as Atterberg limit, Compaction and California bearing ratio.

3.1.1 Traffic Count Survey

Two personnel were stationed at both Elias Egboka and Nwakpadolu roads with tally sheets, stop watch and pen for recording. The number of vehicles plying each of the roads was observed and recorded on the tally data sheet on 12 hours (7am – 7pm) daily basis for a particular vehicle category. The exercise was repeated for one full week.

3.2 Description of Study Area

Elias Egboka and Nwakpadolu roads which is object of this research is situated outside Nnamdi Azikiwe University Awka Anambra State. Elias Egboka is a tarred 3km one-lane carriage way adjoined to Nwakpadolu road and is surrounded by private residence. An off shoot of Elias Egboka road is the Ahaneku road which also adjoins to Nwakpadolu road.

Nwakpadolu road on the other hand is a 13km unpaved road that connects Agu-Awka to Book Foundation, this roads is characterized by deep depression in the form of gullies and constitute severe driving difficulty especially during the rainy season. Both roads are lies within longitude 6^o45lE to 7^o30lE and latitude 6^o00lN to 6^o30lN.

Plate 3.0: Photograph of Elias Egboka Road

Plate 3.1: Photograph of Nwakpadolu Road

3.3 Laboratory Investigation

This section presents the experimental procedure and laboratory tests that were adopted for the project work. The tests was conducted for the soil samples collected at three different locations and this tests include: Sieve analysis test, Atterberg limit test, compaction test and California bearing ratio test. The above listed tests were carried out at Nnamdi Azikiwe University Civil Engineering Laboratory located inside the school campus. Below is a description of test procedures and apparatus:

3.3.1 Sieve Analysis Test

Sieve analysis is a procedure used to assess the particle size distribution of a granular material (Atkinson, 2000). The size distribution is often of critical importance to the behaviour of the material during use. Sieve analysis can be performed on any type of non-organic or organic granular material including sand, crushed rock, clay, granite, feldspar and a wide range of manufactured powders, grains and seed down to minimum size depending on the exact method. The standard grain size analysis test determines the relative proportion of different grain sizes as they are distributed among certain size ranges.

Soil possesses a number of physical characteristics which can be used as aid to identify it sizes in the field. A handful of soil rubbed through the finger can yield the following:

1. Sand and other coarser particle are visible to the naked eye.
2. Silt particle becomes dusty and are easily brushed off.
3. Clay particle are greasy and sticky when wet and hard when dry and have to be scrapped or washed off hand and boot

For a soil to be well graded the value of coefficient of uniformity (Cu) has to be greater than 4 and 6 for gravel and sand respectively, while the Coefficient of Curvature (C_v) should be in the range of 1 to 3.

Very coarse soils	Boulder size		> 300 mm
	Cobble size		80 - 300 mm
Coarse soils	Gravel size (G)	*Coarse*	20 - 80 mm
		Fine	4.75 - 20 mm
	Sand size (S)	*Coarse*	2 - 4.75 mm
		Medium	0.425 - 2 mm
		Fine	0.075 - 0.425 mm
Fine soils	Silt size (M)		0.002 - 0.075 mm
	Clay size (C)		< 0.002 mm

Figure 3.0: Ranges for grain Sizes of different Soil type (Atkinson, 2000).

The apparatus needed for this experiment is listed below:

1. Stack of sieves including pan and cover.
2. Mechanical sieve shaker.
3. Weighing balance of 0.01g sensitivity.
4. Hand brush
5. Mortar and pestle (Used for crushing if the sample is conglomerated or lumped)
6. Thermostatically controlled Oven (With temperature of about $80^{O}C$-$110^{O}C$).
7. Masking tape for identification of sample.

8. Exercise book and pen for recording of result.
9. The calculation for attaining Coefficient of uniformity and Coefficient of curvature are outlined below.

Percentage retained (%) $=\frac{\text{mass of soil retained in the sieve(g)}}{\text{total mass of soil sample(g)}} \times 100$ **…………………..** (3.1)

Cumulative percentage retained = $\sum$ Percentage retained (%) **……………..………** (3.2)

Cumulative Percentage Finer (%) = 100-Cummulative percentage retained. **…………..** (3.3)

Coefficient of Curvature $=\frac{D60}{D10}$ **………………………..……….** (3.4)

Coefficient of Uniformity $=\frac{(D30)\times 2}{D10\times D60}$ **…………………..…………….** (3.5)

Where

D10= particle size such that 10% of the soil is finer than the size

D30= particle size such that 30% of the soil is finer than the size.

D60= particle size such that 60% of the soil is finer than the size**.**

Test Procedure

i. The stack of sieves to be used for the experiment was properly cleaned using hand brush.
ii. About 500g of air-dried soil sample was weighed with the aid of a weighing balance.
iii. The weighed soil sample was poured into 75um sieve and wash under a steady supply of water until clear water start coming out from the sieve after passing through the soil sample.
iv. After washing pour the washed soil sample into a pre-weighed plate and dry it inside the thermostatically controlled oven at a controlled temperature of 80-110^{O}C for 16-24hrs.
v. The sample was removed from the oven and the weight was determine (net weight) by deducting the weight of plate from the weight of plate and soil.
vi. The stacks of sieve was arranged in the ascending order, placed in a mechanical sieve shaker, and thereafter the sample was poured and connected to the shaker for about 10-15 minute.

vii. The sieve shaker was disconnected and the mass retained on each of the sieve sizes was determined.

viii. The percentage retained, Cumulative percentage retained and Cumulative percentage finer was determined.

ix. The graph of sieve Cumulative percentage finer against sieve sizes was plotted.

x. D10, D30 and D60 were determined from the plotted graph.

xi. The Coefficient of Curvature and Coefficient of Uniformity was determined and used to classify the soil adopting the American Association of State Highway and Transportation Official (AASHTO) and Unified Soil Classification System (USCS) respectively.

Plate 3.2: Segregation of Clods Prior to Sieve Analysis Test

Plate 3.3: Washing of Soil Samples for Particle Size Determination

Plate 3.4: Arrangement of Stacks of Sieve on a Mechanical Sieve Shaker

3.3.1.1 AASHTO Soil Classification System

AASHTO system was developed by the American Association of State Highway and Transport Officials, Specifically for highway construction and is still widely used for that purpose. This system of soil classification has undergone several revisions, with the present version proposed by the Committee on Classification of materials for sub-grade and granular type road of the Highway research Board in 1945 (ASTM Designation D-3282: AASHTO METHOD M145). The AASHTO Classification in present use is given in table. According to the system, soil is classified into seven (7) major groups A-1 to A-7. Soils classified under group A-1, A-2 and A-3 are granular material of which 35% or less of the particle passes through Sieve No: 200 (0.075mm). Soils of which more than 35% passes through Sieve No: 200 (0.075mm) are classified under A-4, A-5, A-6 and A-7 respectively. These soils are mostly clay and silt type materials.

Table 3.0: AASHTO Classification System (ASTM D3282)

General Classification	Granular Materials (35% or less passing 0.075mm Sieve)			Silt-Clay Materials (>35% Passing 0.075mm Sieve)			
	A-1	A-3	A-2	A-4	A-5	A-6	A-7-6

Soil Classification	A-1-a	A-1-b		A-2-4	A-2-5	A-2-5	A-2-6				A-7-5, A-7-6
Sieve Analysis											
2.00mm (No. 10)	50 max	…..	…..	…..	…..	…..	……	....	…….	....	……..
0.425mm (No 40)	30 Max	50 Max	51 max	…….	……	……	…….	....	…….	……	………
0.075mm (No 200mm)	15 max	25 max	10 max	35max				36min			
Characteristics of fraction passing 0.425mm (No.40)											
Liquid Limit	……..			40 max	41 max	40 max	41 max	40 max	41 max	40 max	41 max
Plasticity	6 max	N.P	10 max		11 min					11min	
Usual types of significant constituent materials	Stone Fragment Gravel and sand	Fine Sand	Silty or Clayey Gravel and Sand					Silty Soils		Clayey Soils	
General rating as a sub-grade	Excellent to Good							Fair to Poor			

3.3.2 Specific Gravity Test

Specific gravity is the ratio of mass of unit volume of soil at a stated temperature to mass of equal volume of gas-free distilled water at the same temperature (Krishna, 2002). Also as defined by (Braja, 2006), Specific gravity can be defined as the ratio of unit weight of a material to unit weight of water. The specific gravity of soil solids is often needed for various calculations in soil mechanics. It can be determined accurately in the soil laboratory.

The apparatus employed for this experiment includes:

1. Density bottle of 50ml capacity and a stopper.
2. Desiccator containing anhydrous silica gel.
3. Thermostatically controlled oven with temperature of about 80-110^{O}C.

4. Weighing balance of 0.01g sensitivity.
5. Mantle heater.
6. Plastic wash bottle.
7. Distilled water.
8. Funnel
9. Thin glass rod for stirring.
10. 425um Sieve.
11. Dry piece of cloth for cleaning.
12. Masking tape for identification of sample.
13. Exercise book and pen for recording of result.

Test Procedure

i. The density bottle properly cleaned and rinsed with distilled water, thereafter oven- dried and then cooled it in a desiccator so as to remove any moisture present.

ii. The empty clean and dry density bottle was weighed and recorded as (M_1).

iii. About 10-15g of soil passing through 425um sieve was placed inside the density bottle, weigh and the weight of density bottle +dry soil + stopper was recorded as (M_2).

iv. Distilled water was added to fill about half to three-fourth of the density bottle, and then the sample was soaked for 24hrs (The time stated is to enable complete settlement of the soil particle which is evident when clear water appears above the submerged soil).

v. The density bottle was gently stirred using thin glass rod and thereafter connected to a mantle heater to de-air the sample, the sample was not allowed to boil over.

vi. After agitation, the sample was allowed to cool at room temperature and then filled with distilled water up to the specified mark (at lower meniscus level), the exterior surface of the density bottle was cleaned with a clean dry cloth and the weight of the density bottle + stopper +soil filled with water was determined and recorded as (M_3).

vii. The density bottle was emptied, cleaned and rinsed with distilled water, then filled with distilled water up to the same mark. The exterior surface of the density bottle was cleaned with a clean dry cloth and the weight of the density bottle filled with distilled water + stopper was determined and recorded as (M_4).

viii. The test procedure was repeated for two more trials and the average specific gravity value was obtained from the total no of trial, the variation in the specific gravity result obtained for each trial must not exceed 2%, otherwise repeat the experiment.

The Procedure for Computation of result obtained is as follows:

Specific gravity (G_S) = $\frac{(M2-M1)}{(M2-M1)-(M3-M4)}$ **…………………** (3.6)

Where M_1= weight of density bottle + stopper

M_2= Weight of density bottle + air-dried soil + stopper.

M_3= Weight of density bottle filled with water + wet soil + stopper.

M_4= Weight of density bottle filled with water + stopper

3.3.3 Atterberg Limit Test

The behavior of soils especially fine grained soils differs considerably in the presence of water. Clay in the presence of water may almost take a liquid or can be quite hard. Consistency is the property of soil that offers resistance to deformation, it denote the degree of firmness of a soil and can be explained in terms of plasticity and stickiness of soil. Stickiness is the ability of soil especially fine grained soil to adhere to other materials while plasticity on the other hand is the ability of soils to undergo a change in shape under the action of an impressed force without a change in volume.

Stickiness of soils especially fine grained soils can be identified practically by mixing of an air-dried soil with a given quantity of water and then interposing the soil between the thumb and the fore finger (index finger), thereafter the following inference are made as it regards to the observation and this includes:

1. **Non-Sticky:** If the wet soil falls freely between the thumb and the forefinger without leaving any remain or without stretching.
2. **Slightly Sticky:** If the wet soil falls slowly with an infinitesimal traces of remains but without stretching.
3. **Sticky:** If the wet soil falls quite slowly with visible remains and apparent stretching.

4. **Very Sticky:** If the wet soil stretches between the thumb and the fore finger without falling.

The plasticity of soils can be identified practically by rolling a known weight of wet soil into a 3mm uniform diameter thread and the following inferences based on the observation are made and they are as follows:

1. **Non-Plastic:** If the wet soil cannot be rolled into thread.
2. **Slightly Plastic:** If the wet soil can be rolled into thread but crumbles easily under application of little pressure.
3. **Plastic:** If the wet soil can be rolled into 3mm thread but crumbles under intense application of pressure and cannot be reformed.
4. **Very Plastic:** If the wet soil can be rolled into 3mm diameter thread but crumbles under intense application of pressure and can be reformed.

The Atterberg Limit is a limit characterized by visible transition of soil (especially fine grained soils) from liquid-plastic-semi-solid-solid state consequent upon the variation of moisture content. This test was developed by Albert Atterberg a Swedish agricultural scientist in 1911. This test is divided into three limits namely:

1. Liquid Limit (LL)
2. Plastic Limit (PL)
3. Shrinkage Limit

I. Liquid Limit Test

It is the water content at which the soil has a small shear strength that it flows to close a groove of standard width when jarred in a specified manner. It is the minimum water content at which the soil tends to flow like a liquid. When a soil is mixed with an excessive amount of water, it will be in a liquid state and flow like a viscous liquid. When the viscous liquid dries gradually due to loss of moisture it will pass into a plastic state. With further loss of moisture, the soil will pass into a semi-solid state. With even further reduction of moisture, the soil will pass into a solid state. The moisture content (%) at which a cohesive soil will pass from liquid state to plastic state is referred to as the liquid limit of the soil.

In order to study the liquid limit of the soil Casagrande test was conducted. Liquid limit is generally determined by the mechanical method using Casagrande apparatus or the standard liquid limit test apparatus. With respect to this method, the liquid limit is defined as the moisture content at which 25 blows or drop in standard liquid limit apparatus will just close a groove of standardized dimension cut into sample by a grooving tool at a specified amount (Aroja et al., 2017).

The apparatus used for liquid limit determination is outlined below:

1. Liquid limit device (Cassagrande type)
2. Grooving tool
3. Moisture content tins
4. Porcelain evaporating dish
5. Spatula or pellet knife
6. Thermostatically controlled oven
7. Weighing balance sensitive to 0.01g
8. Plastic wash bottle containing distilled water
9. Paper towels
10. Masking tape for identification of tin.
11. Exercise book and pen for recording of data
12. 425um Sieve
13. Airtight container

Test Procedure

i. Prepare the sample by weighing about 150g of soil passing through 425um sieve, mix the sample with distilled water in a glass plate mixing with pellet knife, remove any coarse particle by hand and mix to form a thick homogenous paste, place the mixed soil in an airtight container and leave to mature for 24hrs.
ii. Determine the mass of four moisture content tins say (W_1)
iii. Place the matured sample on an evaporating dish and add little water using the plastic squeeze bottle, mix the soil properly to ensure uniform distribution of moisture.
iv. Place a portion of the paste (mixed soil) on the liquid limit device and level the mixture so as to obtain a maximum depth of 1cm.

v. Using the grooving tool, cut a groove along the symmetrical axis of the cup holding the tool perpendicular to the cup.

vi. Turn the crank or rotate the handle of the liquid limit device at the rate of 2 revolution per second and count the no of blows required to close the groove at a distance of 13mm. Closing of the groove should be as a result of plastic flow of the soil and not by sliding, if sliding occurs repeat the test.

vii. Take about 10g of soil in the closed groove and put in the moisture content tins for moisture content determination, weigh the sample say (W_2)

viii. Remove the rest of the soil in the cup and use paper towel to clean the cassagrande cup.

ix. Alter the water content of the soil and the repeat the process to get the no of blows in the range of 15-40 blows.

x. Plot the graph of moisture content against the log of no of blows, the moisture content corresponding to 25 blows on the abscissa gives the value of the liquid limit.

The Procedure employed for the Computation of the Result obtained is as Follows:

Moisture content $= \frac{\text{Weight of water}}{\text{weight of dry soil}} \times 100 = \frac{W2-W3}{W3-W1} \times 100$ .... (3.7)

Where W_1 = Weight of empty tin.

W_2 = Weight of tin + wet soil.

W_3 = Weight of tin + oven-dried

Plate 3.5: Determination of Liquid Limit of Samples Collected Within the Study Area

II. Plastic Limit Test

The plastic limit of a soil is the moisture content expressed as a percentage of the weight of oven-dried soil at the boundary between the plastic and the semi-solid state of consistency. It is the moisture content at which a soil will just begin to crumble when rolled into a uniform 3mm diameter thread using a glass plate or other recommended surface for rolling. Soil used for Atterberg limit test can be classified based on the plasticity index of the soil. The plasticity index is the amount of water required to change a soil from its plastic limit to liquid limit, in other word it is the numerical difference between the liquid limit and the plastic limit of soil. Table 3.1 is used to classify soil based on the ranges of it plasticity index.

Table: 3.1 Plasticity Ratings for Fine grained Soil (Braja, 2002).

Plasticity Index	Plasticity
0	Non-Plasticity
<7	Low Plasticity
7-17	Medium Plasticity
17-35	High Plasticity
>35	Very High Plasticity

The apparatus used for this experiment includes:

1. A smooth glass plate about 300mm square and 10mm thick.
2. A palette knife or spatula
3. A short length of 3mm metal rod
4. Moisture content tins
5. Plastic squeeze bottle
6. Weighing balance with 0.01g sensitivity
7. Veneer caliper
8. Masking tape for tin identification

Test Procedure

i. Prepare the sample by the method described in the liquid limit using the sample passing 425um sieve.
ii. Identify and weigh the empty moisture content tins say (W1).
iii. Take about 20g of the prepared soil paste on a porcelain evaporating dish, add water from the plastic squeeze bottle and mix thoroughly until the paste is plastic enough to be rolled into a ball.
iv. Take a portion of the ball and roll it on a glass plate with the palm of the hand into a thread of uniform diameter throughout its length by rolling forward and backward.
v. Continue rolling and remolding until the thread just start to crack at a distance of 3mm.
vi. Collect the small crumbed pieces, place in a moisture content tin and weigh say (W2).
vii. Place the tin in the oven at a constant temperature of 80-110^{O}C for a period of 16-24hrs.
viii. After 24hrs, remove the tin from the oven and determine the weight of the dry soil plus the tin say (W_3).
ix. Repeat the test for at least two trials and take the average plastic limit value for all the trials.

The Computation for Plastic Limit is as follows:

Plastic limit = $\frac{\text{Weight of water}}{\text{Weight of oven-dried soil}} \times 100 = \frac{W2-W3}{W3-W1} \times 100$ .... (3.8)

Where W_1 = Weight of empty tins.

W_2 = Weight of tin plus wet soil

W3 = Weight of tin plus oven-dried soil

3.3.4 Compaction Test

Compaction is the process of increasing the bulk density of the soil by driving out air. It involves the densification of soils by mechanical means thereby increasing the dry density of the soil. According to (Shruthi, 2017) Compaction of soil is the process by which the soil solid are packed more closely together by mechanical means, thus increasing it dry density. It could also be stated as the process of packing the soil particle more closely together usually by tamping, rolling or other mechanical means, thus increasing the dry density of the soil. It is achieved through the reduction of the volume of air void in the soil with little or no reduction in water content. The process must not be confused with consolidation in which water is squeezed out under the action of steady static load. Consolidation is a natural process and result in dense packing of the soil.

In civil engineering practice soil compaction is essential for the following reasons:

1. Increasing the bearing strength of foundation
2. Provide stability to slope and foundation.
3. Prevention of undesirable settlement of structures
4. Reduction of water seepage from structure

The compaction methods to be adopted for this research is British Standard Light for the disturbed soil samples collected at three different locations within the study area.

Details of British Standard Compaction Process

Table 3.2 Details of Compaction Mould.

Type	Diameter (mm)	Height (mm)	Volume(cm^3)
British Standard	105	115.5	1000

Table 3.3 Details of Compaction Procedure

Type of test	Mould (cm^3)	Rammer(kg)	Drop (mm)	No of layers	Blow per layer
BS light	1000	2.5	300	3	27
BS heavy	1000	4.5	450	5	27

The mechanical energy applied in each type of British Standard in term of work done is given as follows:

British Standard Light

Mechanical energy = $\frac{\text{Weight of rammer} \times \text{no of layers} \times \text{no of blows} \times \text{height of drop}}{\text{Volume of mould}}$ **…………** (3.9)

$=\frac{2.5\text{g}\times 3\text{ layers}\times 27\text{ blows}\times 300\text{mm}}{1000}$ = 60.75kgm =60.75× 9.81Nm=596g

Work done per unit volume of soil $=\frac{596}{1000}$ =596kg/m^3

The apparatus used for the test are as follows:

1. Compaction mould with a detachable base plate and removable extension collar.
2. Metal rammer (either 2.5kg or 4.5kg)
3. Measuring Cylinder 200ml or 500ml
4. Large Metal tray (600mm×600mm ×600mm)
5. Balance up to 10kg readable to 1g
6. Small tools such as palette knife, steel straight edge about 300mm long.
7. Drying oven temperature of 105-110^{O}C
8. Apparatus for moisture content determination

Test Procedure.

i. Check to see if the mould, extension collar and base plate are clean and dry. Measure the dimension and weigh to the nearest 1kg check if the rammer falls freely.
ii. Grease the internal surface of the mould
iii. Attach the extension collar to the mould.
iv. Weigh about 3kg of the soil sample on a weighing balance
v. Add about 4% water to the soil sample, mixing it thoroughly and separating the soil into three layers for British Standard Light and five layers for British Standard Heavy.
vi. Pour the wet soil into the mould and compact by applying the required no of blow using either a 2.5kg or 4.5kg rammer falling freely from a height of 300mm. The blow must be distributed uniformly over the surface of the mould.
vii. After completion of the compaction operation remove the extension collar and level carefully the top of the mould by means of a straight edge.
viii. Weigh the mould with the compacted soil to the nearest 1kg, record the weight as W_2.
ix. Determine the moisture content of the representative sample of the specimen; record the moisture content as M.
x. Repeat the procedure for 8%, 12%, 16% and 20% of water to be added and record the value obtained.
xi. Plot the graph of dry density against moisture content and determine the maximum dry density (MDD) of the soil at the corresponding optimum moisture content (OMC).

The Computation of the result obtained is as follows:

Determination of Dry Density (P_d).

Wt of mould (kg) = W_1

Wt of mould + wet soil (kg) = W_2

Wt of wet soil (kg) = W_2-W_1

Volume of mould (M^3) = W_4

Bulk Density (kg/m^3) = $\frac{\text{Wt of wet soil (kg)}}{\text{Vol of mould (m3)}} = \frac{W2-W1}{W4}$ .... (3.10)

Moisture Content (%) = $\frac{\text{mosture content (top)+ moisture content (bottom)}}{2}$ .... (3.11)

Dry Density (kg/m^3) = $\frac{\text{Bulk density}}{\text{1+moisture content (\%)}} = \frac{Pb}{1+w/100}$ .... (3.12)

Determination of Moisture Content (w) for top and bottom respectively.

Wt of tin (kg) = W_1

Wt of tin + wet soil = W_2

Wet of wet soil (kg) = $W_3 = W_2 - W_1$

Wt of tin + dry soil (kg) = W_4

Wt of dry soil (kg) = $W_5 = W_4 - W_1$

Wt of water (kg) = $W_6 = W_3 - W_5$

Moisture Content (%) = $\frac{\text{Wt of water}}{\text{Wt of dry soil}} \times 100 = \frac{W6}{W5} \times 100$

Plate 3.6: Manual Compaction of Soil Samples Collected Within the Study Area

3.3.5 California Bearing Ratio Test

The California bearing ratio test was originally developed by the California division of highway in 1938, for the design of highway thickness. The test is used for evaluating the suitability of materials used in sub-grade, sub-base and base course respectively. The test result has been correlated with the thickness of various materials required for flexible pavement construction. The test may be conducted on a prepared specimen in a mould or on the soil in-situ condition.

In the test the load required to push a plunger into a soil specimen at a controlled rate is measured, then the load on the plunger at a certain depth is recorded as a percentage of a standardized load. The load necessary to push a plunger to a certain depth into the soil is expressed as a percentage of the load required to force the same plunger to the same depth into a standard sample of compacted crush stone. The construction of highway pavement requires a California Bearing Ratio value for 2.5mm and 5mm penetration respectively, with that of 2.5mm penetration being comparatively higher than that of 5mm penetration. The Federal Ministry of work Standard Specification for roads and bridges (1997) state that road construction material should have a CBR value of 10%, 20% and 80% for use as sub-grade, sub-base and base course respectively. The material to be used for the test will be subjected to 48 hours soaking in other to ascertain it behavior under worst condition (flooding as a result of intense rainfall).

Table 3.4: Standard load adopted for different penetration on a standard material with CBR value of 100%

Penetration of plunger (mm)	Standard Load (kg)
2	1150
2.5	1320
4	1760
5	2000
6	2220
7.5	2630
8	2650
10	3180
12.5	3600

The apparatus used for the test are outlined below:

1. A cylindrical corrosion resistant mould 152mm×127mm having a diameter of 150-152mm with a detachable base plate and a removable extension collar.
2. A compressive device for static compaction of applying a force of at least 300KN
3. Metal plugs 150mm ± 0,5mm and 50mm thick.
4. Metal rammer 2.5kg or 4.5kg.
5. Dial gauge of 0.01g sensitivity.
6. Soaking tank.
7. A steel rod of about 16mm diameter and 600mm long and a straight edge of 300mm steel stripe and 3mm thick with one beveled edge.
8. Weighing balance of 25kg accuracy and a spatula.
9. Filter paper
10. Apparatus for moisture content determination.
11. Masking tape used for identification of moisture content tins.
12. Exercise book and pen for recording.

Test Procedure

The methods used for California Bearing Ratio Test are:

i. Compression with tamping.
ii. Recompaction with known maximum dry unit weight (MDUW) and optimum moisture content (OMC).
iii. For this course of study the method for recompacted sample with known maximum dry unit weight (MDUW) and optimum moisture content (OMC) is to be adopted and the procedure is outlined below:
iv. Carry out Compaction test using 6kg of soil sample, varying the moisture content at a particular percentage say 4%, determine the maximum dry density and optimum moisture content.
v. Clean properly and grease the internal surface of the CBR mould.
vi. Weigh 6kg of soil mixing with the optimum moisture content determined from compaction test.
vii. Divide the soil into 5 equal layer (CBR Heavy) and seal in an airtight container until requested for use.

viii. Stand the mould assembly in a solid base, place the first soil portion and compact using 4.5kg rammer for 62 even blows.
ix. Repeat using the remaining four portion of soil in turn so that the level of the soil is not more than 6mm above the top of the mould body.
x. Remove the collar and trim the soil flush with mould with the scrapper or knife edge.
xi. Weigh the mould, soil and base plate to the nearest kg.

Preparation for Soaking

Soil may soften when load is placed on it due to flooding or increase in moisture content. Soaking of the sample is done primarily to determine the strength (load bearing strength) of the soil under worst condition (rainy season).Below are the list of apparatus used for CBR Soaking:

i. Perforated base plate fitted to CBR mould in place of normal base plate.
ii. Perforated swell plate with an adjustable stem to provide a sealing for the stem of the dial gauge.
iii. Tripod mounting to support dial gauge
iv. Soaking tank
v. Annular Surcharge discs with internal diameter of 52-54mm and external diameter of 145mm to 150mm.
vi. Petroleum jelly.
vii. The Soaking procedures are enumerated as follows:
viii. Remove the base plate and replace with perforated base plate.
ix. Fit the collar to the other end of the mould, pack the screw thread with petroleum jelly to make it water tight.
x. Place the mould assembly in soaking, place the filter paper in the sample, the perforated swell plate, and then annular surcharge disc.
xi. Mount dial gauge on top of the extension collar, secure the dial gauge in place and adjust the stem in the perforated base plate to give zero.
xii. Fill the immersion tank with water just below the extension collar. Start the timer when water has just covered the base plate.
xiii. Record the time taken for water to appear at the top of the sample if it does occur within two days. Flood the top of the sample and leave to soak for a day.

xiv. Plot the swelling against elapsed time or square root of time. Flattening curve indicates that swelling is complete.

xv. Take off the dial gauge and its support; remove the mould assembly and leave to drain for 15min.

xvi. Remove the Surcharge discs, perforated plate and collar, then fit the other base plate.

xvii. Weigh the sample + mould + base plate if density is required after soaking is completed.

xviii. If the sample has swollen, trim it to the level of the mould and reweigh

xix. Test the sample by adjusting the dial gauge to start at zero and take the reading at interval of 0.5mm for every 30seconds till 7mm penetration.

xx. Record the load at penetration of 0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5 and 7.0mm and express this force as percentage of the standard load.

xxi. Calculate the CBR for 2.5 and 5mm penetration; repeat the same procedure for top and bottom, the higher CBR value will be used as the CBR for the material.

xxii. Plot the graph of force (KN) against penetration (mm).

xxiii. The normal curve is convex upward, but if the initial part is concave upward applies the necessary correction to the curve.

Mathematically it is expressed as $\frac{\text{test load}}{\text{standard load}} \times 100$ …………………. (3.13)

Where

$\text{Test load} = \text{dial gauge reading} \times \text{proof ring constant}$ …………………. (3.14)

3.4 Procedure for Design of Flexible Pavement Using AASHTO Design Method

AASHTO method employs the use of CBR value of the pavement as a design parameter. Result obtained from CBR test will be used for design. However, this study will adopt the lowest soaked and un-soaked CBR value as the CBR of the sub-grade and sub-base soil while that of the base course will be based on by Nigeria general specification for roads and bridges (1997). Below are steps recommended by AASHTO for design of flexible pavement.

1 The traffic survey count was conducted so as to determine the average annual daily traffic (AADT).

2 The CBR of the sub-grade, sub-base and base course was determined through laboratory testing.

3. The resilient modulus of the pavement layers was determined using a correction factor (1500CBR) specified by AASHTO.
4. The design life and annual growth traffic of the pavement was determined.
5. The cumulative number of axle load at the end of the pavement design life was calculated.
6. The equivalent standard axle load (ESAL) was determined.
7. The reliability level, standard deviation and design serviceability loss was determined.
8. The structural number was obtained graphically through interpretation of charts specified by AASHTO for flexible pavement design based on mean values for each inputs, the charts shows the relationship between equivalent axle loads and the structural number or mathematically the structural number can be determined from the expression below:

$$\mathrm{Log10}\,(W18) = ZR\,x So + 9.36 \ x \ \log10\,(SN + 1) - 0.20 + \log10\left[\frac{\frac{\Delta PSI}{4.2-1.5}}{\frac{0.4+1094}{(SN+1)5.19}}\right] + 2.32\ x\log10\,(MR) - 8.07 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.15)$$

Where:

$W18 =$ Predicted number of $18 -$ kipequivalent axle load application.

$ZR =$ Standard normal deviate.

$S0 =$ Combined standard error of traffic prediction and performance prediction.

ΔPSI
$=$ Difference between the initial design serviceability index $(P0)$and the design terminal index (Pt)

$MR =$ Resilien tmodulus.

9. The structural layer coefficients (a1, a2 and a3) and drainage coefficient (m1 and m2) was determined.

10. The thickness of the sub-base, base course, wearing course and combined thickness of layer super imposed on the sub-grade soil (pavement foundation was determined using the relation below:

$$SN = a1D1 + a2D2m2 + a3D3m3 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.16)$$

Where:

SN is the structural number.

$D1, D2$ and $D3$ represent the thickness of the pavement layers.

$m1$ and $m2$ represent the drainage coefficient.

$a1, a2$ and $a3$ represent the structural layer coefficient

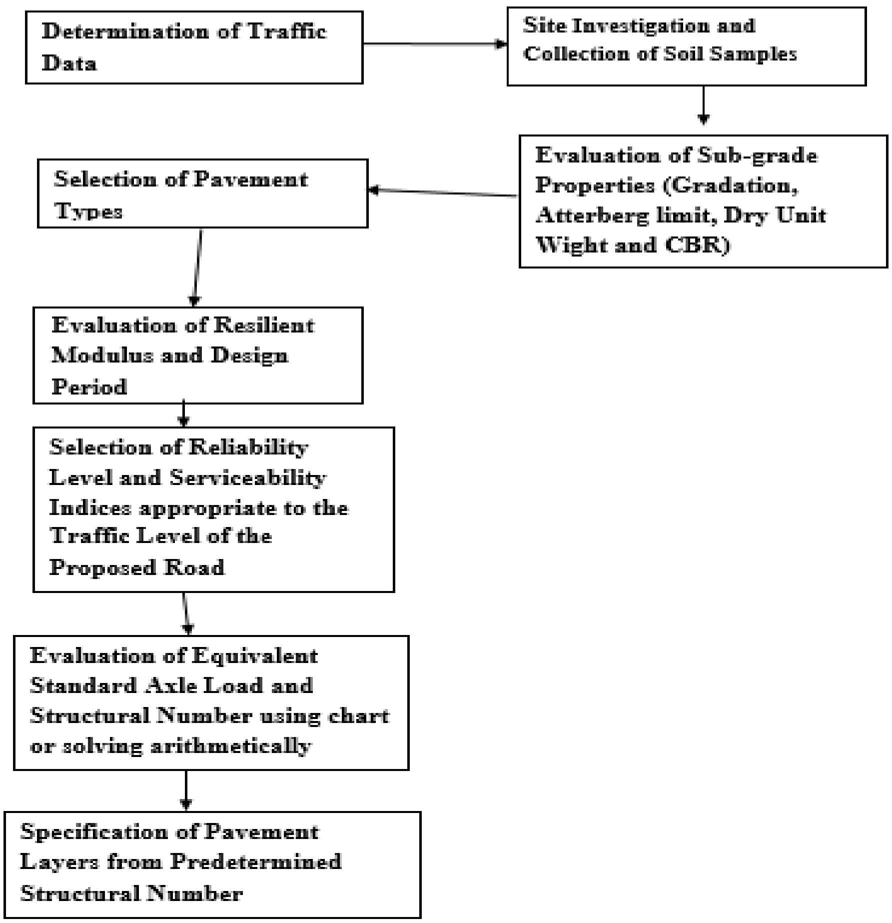

Figure 3.1: Flowchart for the Design Process

3.6 Flexible Pavement Design

The parameters needed for the analysis and design of Elias Egboka-Nwakpadolu roads are presented below:

Table 3.5: Guidelines for Length of Analysis Period (AASHTO, 1993)

Highway Conditions	Analysis Period (Years)
High – Volume Urban	30 – 50
High – Volume Rural	20 – 50
Low – Volume Paved	15 – 25
Low – Volume Aggregate Surface	10 – 20

Table 3.6: Conversion factor to obtain the Equivalent Number of Standard Axle from the Number of Commercial Vehicles (AASHTO, 1993).

Type of Road	Number of Axle per Commercial Vehicle (a)	Number of Standard Axle per Commercial Vehicle (b)	Number of Standard Axle per Commercial Vehicle (a) x (b)
Motorways and truck roads designed to carry over 1000 commercial vehicle per day in each direction at time of construction.	2.7	0.4	1.08
Road designed to carry between 250 and 1000 commercial vehicles per day in each direction at time of construction	2.4	0.3	0.72
All other public roads	2.25	0.2	0.45

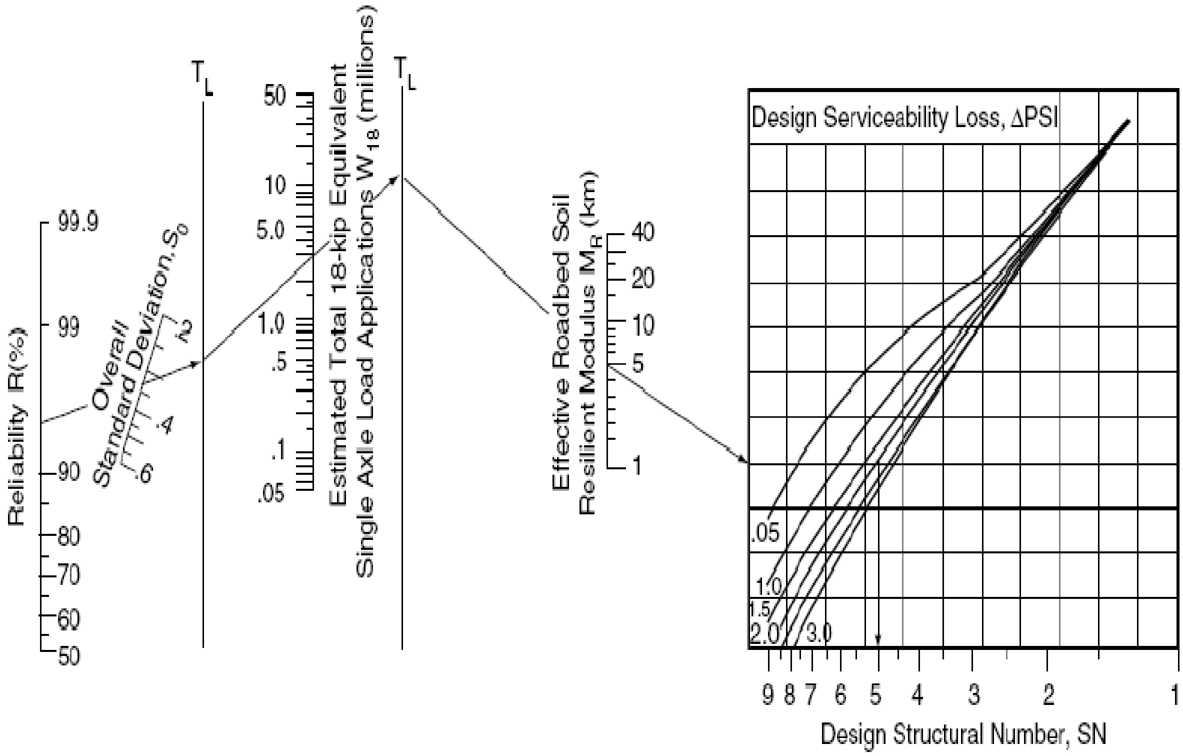

Figure 3.2: Design Chart for flexible pavement Based on Using Mean Values for Each Inputs (AASHTO, 1993).

3.7: Design of Pavement Using AASHTO Method

The traffic count survey result revealed that the average daily traffic for the road under design was 831.72pcu/hr/day with the highest percentage traffic composition of 12.0% recorded on day 1 (Monday) and the lowest percentage traffic composition of 15.7% recorded on day 7 (Sunday). It can be therefore inferred that weekends particularly Sunday's accounts for larger chunk of the traffic volume for the road under design.

The design was done based on the 1993 AASHTO guide procedure.

AASHTO gives the correction factor:

$Mr = 1500x\,CBR$ **………………………………………………..** (3.17)

CBR value of base course material = 100% (assumed)

Resilient Modulus of base course material = 31,000N/mm^3 (assumed)

CBR value of sub-base material = 25%

Resilient Modulus of sub-base material = 25x1500 = 37,500

CBR value of the sub-grade soil = 11%

Resilient Modulus of the sub-grade soil = 11x1500 = 16,500

Resilient Modulus of asphalt concrete = 300x1500 = 450,000N/mm^3 (assumed)

Design Life of Pavement = 20 years

Present traffic = 832CV/day

Cumulative repetition at the end of 20 years (N) is given by the following:

$N = (1 + r)^n$ x Ax 365 x n ……………….. (3.18)

Where:

N =cumulatibve number of standard axles

r = annual growth rate of commercial vehicle

A = traffic on each lane

n = design life in years

Annual growth rate of traffic = 7.5% (assumed)

Since we are designing a single lane carriage way**:**

Traffic on single lane = 832pcu/hr/day

$N = (1+ 0.075)^{20}$ x 832x365x20 (derived from equation 3.18)

Therefore, N becomes 25.8 $x10^6$

The number of standard axle to be used for design purpose is 0.72 (Table 3.6)

Equivalent number of standard axle (ESAL) = 0.72x 25.8 x 10^6 = 18.6 x 10^6

Reliability Level (R) = 99% (AASHTO; 1993 Guide)

Standard Deviation (S_0) = 0.49 (ranges from 0.4 to 0.5)

Initial serviceability index Pi = 4.5

Terminal serviceability Index = 2.5

ΔPsi = (4.5-2.5) = 2.0

MR = Sub-grade resilient modulus = 16500

$$\text{Log10 (W18)} = \text{ZRxSo} + 9.36 \times \log10\ (SN + 1) - 0.20 + \log10 \left[\frac{\frac{\Delta PSI}{4.2-1.5}}{\frac{0.4+1094}{(SN+1)5.19}} \right] + 2.32 \times \log10\ (MR) - 8.07$$

Solving mathematically we have:

$$\text{Log 10 (18600000)} = 2.327 \times 0.49 + 9.36 \times \log10\ (SN + 1) - 0.20 + \log10 \left[\frac{\frac{2}{4.2-1.5}}{\frac{0.4+1094}{(SN+1)5.19}} \right] + 2.32 \times \log10\ (16500) - 8.07$$

$$7.2695 = 1.14023 + 9.36 \times \log10\,(SN + 1) - 0.20 + \log10\left[\cancelto{0}{\frac{\frac{2}{4.2-1.5}}{0.4+\frac{1094}{(SN+1)5.19}}}\right] + 9.78455 - 8.07$$

Solving using excel solver

Structural Number (SN) = 4.0

Alternatively determining the structural number graphically using charts shown in Figure 3.2

Parameters needed:

Equivalent standard axle load = 25.8 x 10^6

Reliability level = 99%

Roadbed modulus = 16,500

ΔPsi = 2

With this parameter determined, the structural number can be read from the charts shown in Figure 3.2

Therefore, structural number (SN) = 4.4

Adopting the value of 4.4 for the structural number, appropriate layer coefficient for each construction material is determined as follows:

Resilient modulus of asphalt layer = 450,000N/mm^3

Structural layer coefficient for asphalt layer (a_1) = 0.44

Resilient modulus of base course = 31,000N/mm^3

Structural layer coefficient for base course layer (a_2) = 0.14

Resilient modulus of sub-base layer = 37,500N/mm^3

Structural layer coefficient for sub-base layer (a_2) = 0.11

To determine the drainage coefficient assuming the percentage of time the pavement structure is exposed to moisture levels approaching saturation is greater than 25% and drainage quality is good therefore, drainage coefficient for base course and sub-base level (m_2 and m_3) = 1 and 0.8 respectively (Table 2.4: AASHTO, 1993).

The structural number of 4.4 is used to obtain the values of D1, D2 and D3. Using the appropriate value of resilient modulus obtained for each layer:

For asphalt layer with resilient modulus of 450,000N/mm^3, structural number (SN_1) = 2.8 (Figure 3.2)

Base course layer with resilient modulus of 31,000N/mm^3, structural number (SN_2) = 3.9 (Figure 3.2)

Structural number of sub-base layer (SN_3) = 4.4 (determined from calculation)

3.8: Determination of Thickness of Pavement Layers:

$D_1 \geq \frac{SN1}{a1} = \frac{2.8}{0.44} =$ 6.4 in (160mm) for thickness of surface course.

$SN_1 = a_1D_1 = 0.44x6.4 = 2.82 \geq 2.8$

$D_2 = \frac{SN2-SN1}{a2m2} = \frac{3.9-2.82}{0.14x1} =$ 7.7 in (200mm) for thickness of base course layer

$D3 = \frac{SN3-SN2}{a3m3} = \frac{4.4-3.9}{0.11x0.8} =$ 5.7 in (150mm) for thickness of sub-base layer

CHAPTER FOUR

RESULTS AND DISCUSSION

This section presents the results valuable to the design of the pavement structure obtained during the experimentation phase of the study. Also presented, is the analysis of the experimental results, design and detailing of the pavement. The design was done using AASHTO method while the detailing of the designed pavement layers was done using Archicad. Below is a presentation of the aforementioned information.

4.1 Assessment of Geotechnical Properties of the soil samples

4.1.1 Result of Seive Analysis

The result of the sieve analysis of the fine aggregate used in the study is presented in Figure 4.0

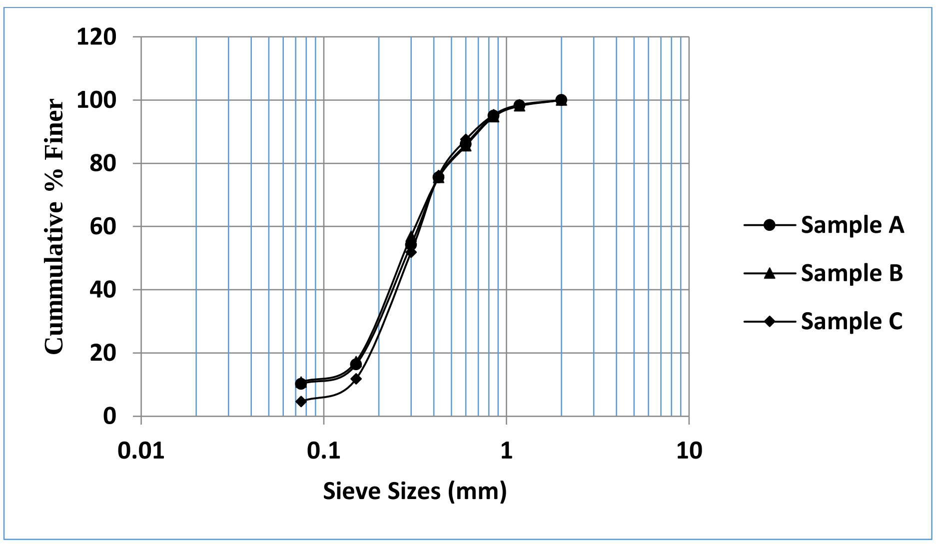

Figure 4.0: Particle Size Distribution Curve for the soil samples

The chart depicts the particle size distribution curve for the three different samples. The percentage passing sieve No 200 (0.075mm) for sample A, B and C were 10.12%, 10.76% and were 10.2%, 10.7% and 4.6% while the sand 4.61% respectively, the results therefore suggest the fine content present in the three samples content were 89.9%, 89.2% and 95.4% respectively. The coefficient of uniformity of the three samples were 4.4, 3.7 and 2.8 while the coefficient of curvature were 1.7, 1.1 and 1.2 respectively. Since more than 35% was retained on sieve No 200(0.075mm), the samples were classified as A-2-6, A-2-6 and A-2-4 according to AASHTO Soil Classification System and since more than 50% was retained on sieve No 200 (0.075mm), the three samples were classified as SC (sand mixed with clay). Gradation assessment of the three different samples revealed that the samples were poorly graded. Based on AASHTO sub-grade rating for highway construction materials, the samples constitute excellent to good sub-grade materials which justifies their use in design and construction of the proposed road.

4.1.2 Specific Gravity Test

The result of the Specific Gravity of the soil sample used in the study is shown in Figure 4.1

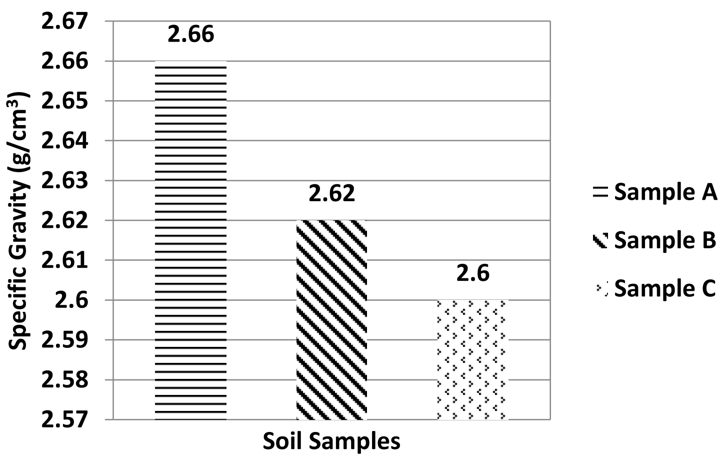

Figure 4.1: Specific Gravity of the samples

The specific gravity of the three samples were 2.66, 2.62 and 2.6 respectively, this results met the specification given by Federal Ministry of Works and Housing, (1999) which state that the specific gravity of sub-grade materials must fall between 2.5 to 2.75. Sample A yielded the highest specific gravity value while sample C yielded the lowest specific gravity value. The range of specific gravity (2.6-2.66) obtained for the three samples suggest the presence of clay and silt which is of advantage at the sub-grade level of pavement construction.

4.1.3 Atterberg Limit Test

4.1.3.1 Liquid Limit Values Test

The result of the Liquid Limit of the soil sample used in the study is shown in Figure 4.2

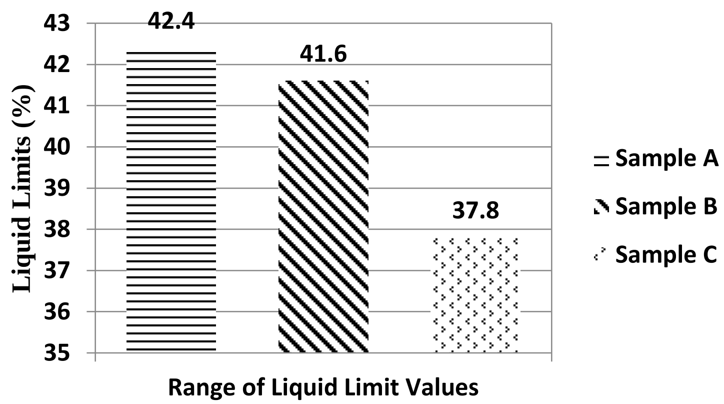

Figure 4.2: Liquid Limit values of the three Different Samples

4.1.3.2 Plastic Limit Test

The result of the Specific Gravity of the soil sample used in the study is presented in Figure 4.3

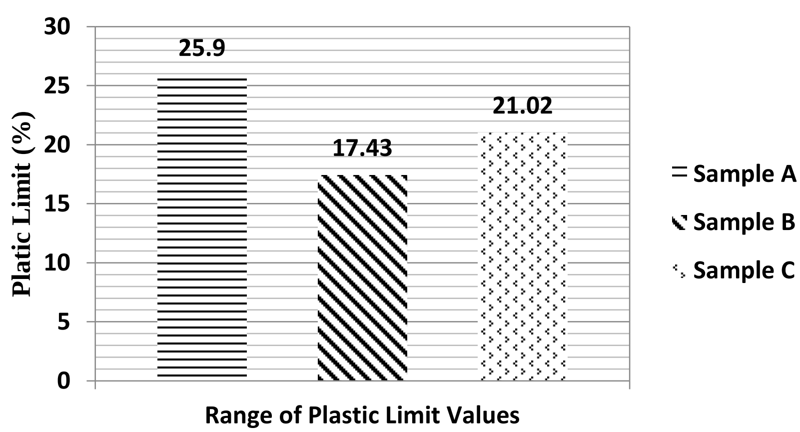

Figure 4.3: Plastic Limit values of the three Different Samples

4.1.3.3: Plasticity Index Test

The result of the Specific Gravity of the soil sample used in the study is shown in Figure 4.4

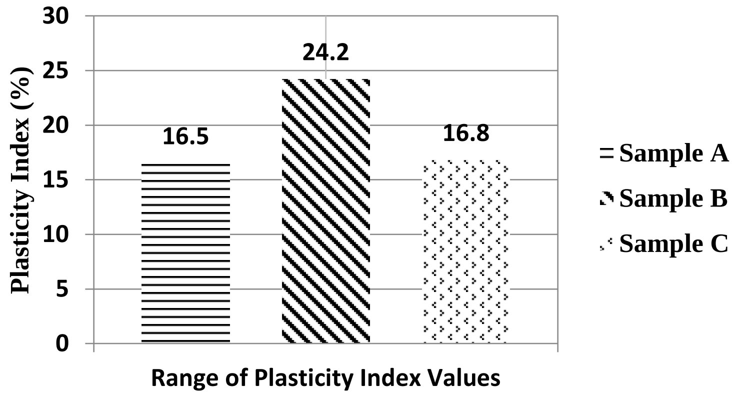

Figure 4.4: Plasticity Index values of the three Different Samples

The charts depicts the liquid limit, plastic limit and plasticity index of the three different samples. The liquid limit of sample A, B and C were 42.4%, 41.6% and 37.8% respectively, this suggests that sample A contains significant amount of fines than sample B and C while sample C contains the lowest amount of fines as supported from grain size analysis test results. Evaluation of the plastic limit and plasticity index of the three different samples revealed that sample A yielded the highest plastic limit and lowest plasticity index while sample C yielded the least plasticity index. Plasticity rating of the samples showed that sample A and C were medium plastic, while sample B was highly plastic. The range values obtained for the liquid limit and plasticity index of the samples indicate that the samples met the requirements given by Federal Ministry of Works and Housing, (1999) which state that the liquid limit and plasticity index of sub-grade material must not exceed 80% and 55% respectively.

4.1.4 Compaction Test

The result of the Compaction Test of the soil sample used in the study is presented in Figure 4.5

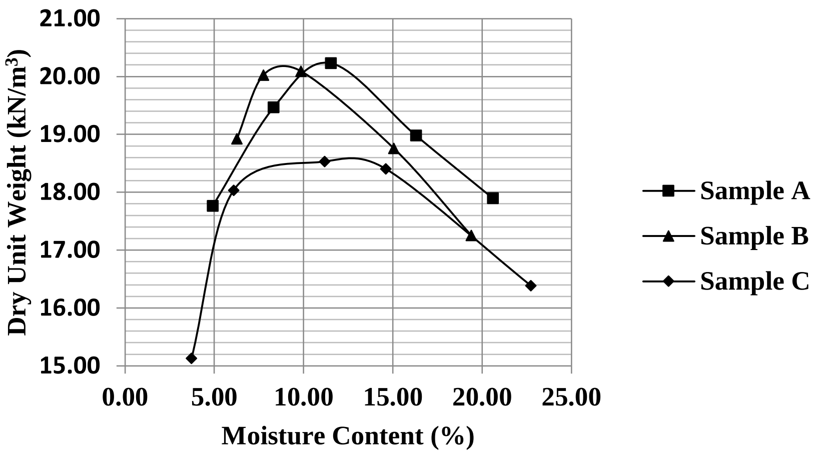

Figure 4.5: Compaction Curve of the soil samples

The charts depicts the compaction curve obtained for the three different samples collected within the study area. Assessment of the compaction characteristics of the samples revealed that sample C which has the lowest fine content (4.6%) yielded the least maximum dry unit weight while sample A yielded the peak maximum dry unit weight. It was observed that the compaction characteristics of sample A and B were somewhat the same as their respective maximum dry unit weight falls within a close range. Assessment of the optimum moisture content of the samples showed that sample A which yielded the highest maximum dry unit weight also yielded the peak optimum moisture content while sample B yielded the lowest optimum moisture content, this implies that less water content will be required to achieve maximum dry unit weight during field compaction for sample A than that of sample A and C, in other words, sample B can be easily compacted dry of optimum during field compaction. The maximum dry unit weight obtained for the three different samples shows that the samples satisfied the requirement for use at the sub-grade level of pavement construction as the maximum dry unit weight exceeds $18kN/m^3$ as recommended by Federal Ministry of Works and Housing, (1999). The peak value of optimum moisture content obtained for sample A contradicts the findings obtained by Venkatramaiah (2006), Rowe (2000) where it was found that the optimum moisture content decreased with increase in maximum dry unit weight.

4.1.5 CBR Test

The result of the CBR Curve for the soaked soil sample (Top)used in the study is presented in Figure 4.6

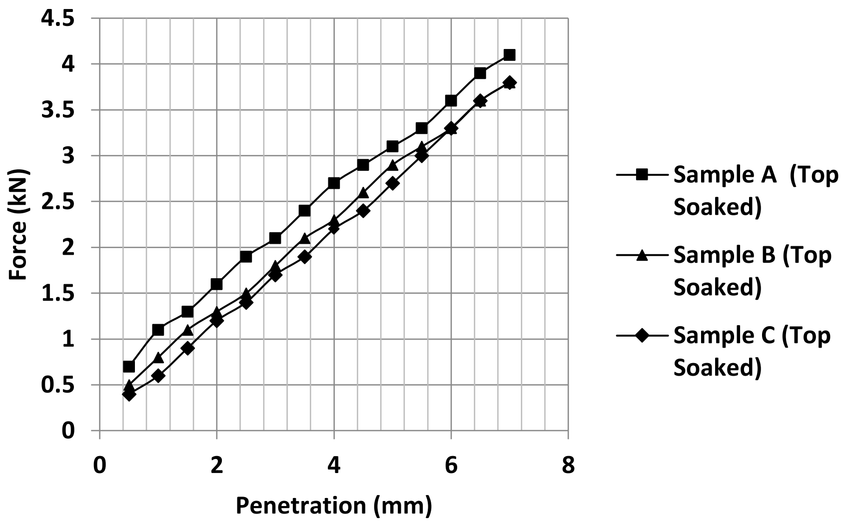

Figure 4.6: CBR Curve for the soaked sample (Top)

The result of the CBR Curve for the soaked soil sample (Bottom)used in the study is presented in Figure 4.7

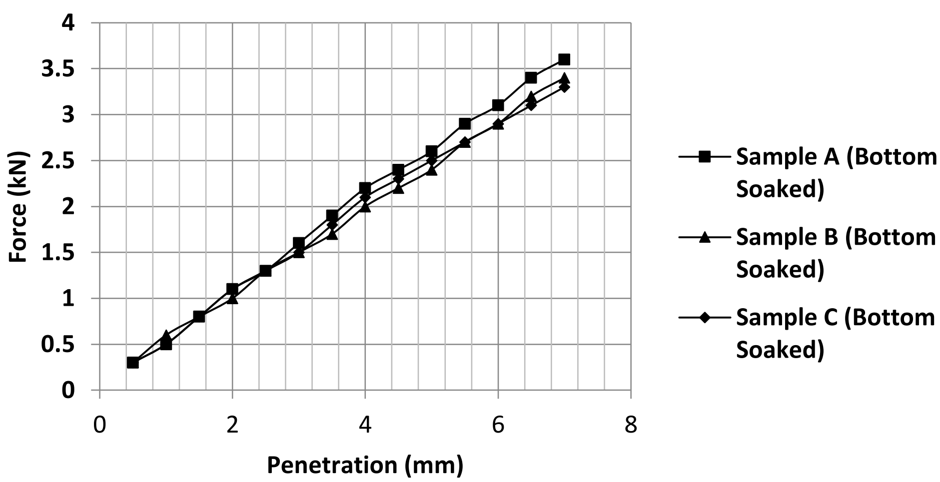

Figure 4.7: CBR Curve for the soaked sample (Bottom)

The result of the CBR Curve for the Un-soaked Sample (Top) used in the study is presented in Figure 4.8

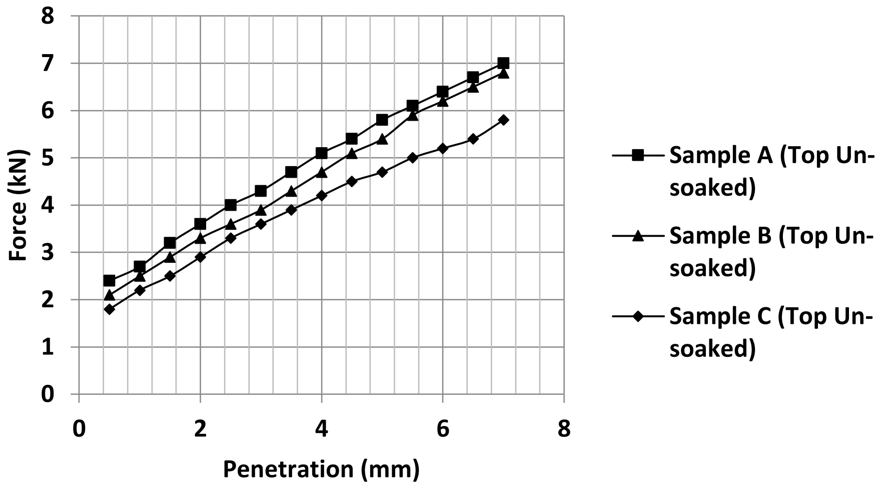

Figure 4.8: CBR Curve for the Un-soaked sample (Top)

The result of the CBR Curve for the Un-soaked Sample (Bottom) used in the study is presented in Figure 4.9

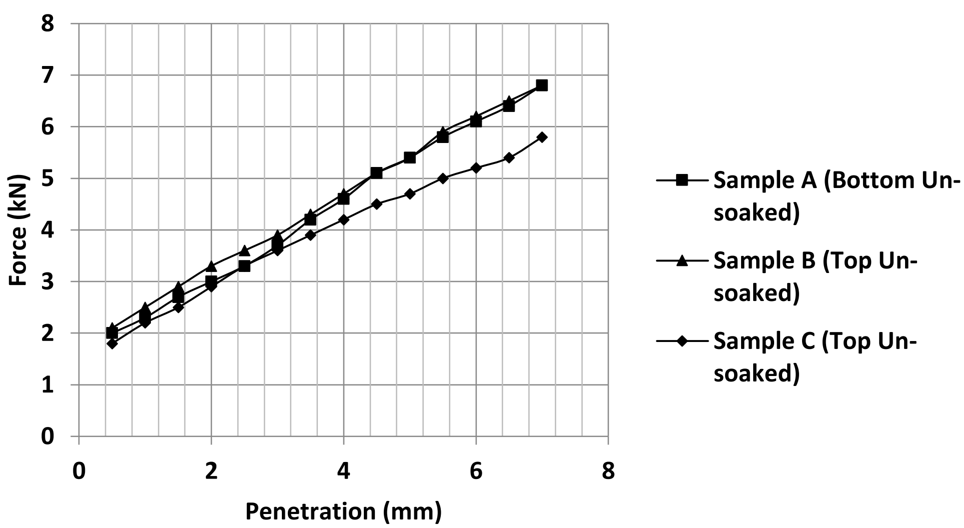

Figure 4.9: CBR Curve for the Un-soaked sample (Bottom)

The charts depicts the CBR curve for the different samples of soils. The soaked CBR of the samples were 15%, 12% and 11% while the un-soaked CBR of the samples were 31%, 28% and 25% respectively as depicted in Table 4.1. These results shows that sample A yielded the peak CBR value in both soaked and un-soaked state while sample C yielded the lowest CBR value in both soaked and un-soaked state. The three different samples satisfied the requirement for use at the sub-grade level of pavement construction as their CBR exceeds 10% as recommended by Federal Ministry of Works and Housing, (1999). Comparative assessment of the CBR and compaction characteristics of the three different soil samples revealed that sample A with the highest maximum dry unit weight also yielded the highest CBR value, this therefore suggest that the CBR of soils shares a direct relationship with the maximum dry weight in other words, the maximum dry unit weight is indices of the CBR of soils.

Table 4.1: Engineering Properties of Natural Soil Samples

Summary of all index properties of the soil sample for the design of the flexible pavement is shown in the table.

Sample Number	**A**	**B**	**C**
Sample Description	Reddish Brown	Reddish Brown	Reddish Brown
% Passing 0.075mm Sieve	10.2	10.7	4.6
Fine Content (%)	10.2	10.7	4.6
Sand Content (%)	89.9	89.2	95.4
Coefficient of Uniformity	4.4	3.7	2.8
Coefficient of Curvature	1.7	1.1	1.2
USCS Classification System	SC	SC	SC
AASHTO Classification System	A-2-6	A-2-6	A-2-4

AASHTO Sub-grade Rating	Excellent - Good	Excellent – Good	Excellent –Good
Gradation	SP	SP	SP
Specific Gravity	2.66	2.62	2.6
Liquid Limit (%)	42.4	41.6	37.8
Plastic Limit (%)	25.9	17.4	21.02
Plasticity Index	16.5	24.2	16.78
Plasticity Rating	Medium Plastic	High Plastic	High Plastic
Maximum Dry Unit Weight (kN/m^3)	20.23	20.09	18.53
Optimum Moisture Content (%)	11.54	9.85	11.17
Soaked CBR (%)	15	12	11
Resilient Modulus (N/mm^3)	22,500	18,000	16,500
Un-soaked CBR (%)	31	28	25
Resilient Modulus (N/mm^3)	46,500	42,000	37,500

The soil sample obtained from the location of the study was observed to be Reddish Brown which suggests that the sample is a mixture of fine sand and clay which also implies the sub-grade soil is laterite

Percentage passing 0.075mm sieve for the sample implies the samples contains significant quantity of coarse fraction and less quantity of fine fractions. This favours the pavement design process particularly when the sample index constitute the sub-grade level.

Assessment of the design shape parameters of the sample based on the Coefficient of Uniformity (4.4, 3.7, 2.8) and Coefficient of Curvature (1.7, 1.1, 1.2) suggests that the sample were poorly graded. To achieve uniformity in the soil size, more fine fractions will be introduced to the coarse fractions.

AASHTO classification system recorded as A-2-6, A-2-6, A-2-4, and USCS classification system recorded as Soil mixed with clay (SC) suggests that the sample will constitute Excellent to good sub-grade material which is described in AASHTO sub-grade Rating

The Specific Gravity of the soil samples (2.66, 2.62, 2.6) falls in the range of 2.5 to 2.75 which is approved and appropriate range verified by Federal Ministry of Works and Housing (1999).

The Atterberg Limit Tests (Liquid Limit, Plastic Limit, Plastic Index and Plasticity Index) indicates the samples Plastic Limit is high while the Plasticity Index is medium. The Plasticity Rating indicates the samples are medium plastic thereby meeting the requirements approved by the government.

Maximum Dry Unit Weight of the samples indicates adequacy of the parameter. This shows little water content will be required to achieve maximum dry unit weight during field compaction

The moisture content of the samples recorded as 11.54, 9.85, 11.17 confirms the Venkatramaih (2006), Rowe (2000) finding which suggests the optimum moisture content decrease with increase in maximum dry unit weight.

The CBR results satisfies the requirement approved by Federal Ministry of Works and Housing (1999) as their CBR exceeds 10%

Table 4.2: Summary of Traffic Count

Summary of the traffic count data and result collected in the space of 1 week (each day) for the design of the road is presented in the table.

Days/Equivalent P. C. U.	**Cars and Taxi**	**Buses**	**Bikes**	**Trucks and Lorries**	**Total Vehicle (P. c. u)**
Equivalent P. C. U.	1.0	2.8	0.75	2.0	
Monday	156	330.4	102	112	700.4
Tuesday	177	434	94.5	128	833.5
Wednesday	168	442.4	90	170	870.4
Thursday	148	462	88.5	104	802.5
Friday	178	504	64.5	116	862.5
Saturday	209	414.4	82.5	144	849.9
Sunday	218	462	78.8	160	918.8

Average Daily Traffic (ADT) = 831.72pcu/hr/day

The average daily traffic evaluated from the traffic count survey was 832pcu/hr/day with the highest percentage of traffic composition recorded on day 7 (Sunday) while the lowest percentage of traffic composition was recorded on day 1 (Monday). The analysis was conducted in other to determine factors such as peak traffic hours, congestion levels, vehicle classifications and changes in traffic patterns over time.

4.2 Assessment of Pavement Design Layers

Table 4.3: Summary of Design Results

Results of the thickness of each layer of pavement is presented in the table

Pavement Layers	**Pavement Layer Thickness (mm)**
Surface course	160
Base Course	200
Sub-base	150
Total layer thickness super imposed on sub-grade layer	510

Table 4.3 depicts the output obtained from design of Elias Egboka-Nwakpadolu roads using AASHTO design method. From the outcome, it was observed that the thickness of the sub-base, base course and surface layer were 150mm, 200mm and 160mm respectively while the total thickness of the pavement superimposed on the sub-grade layer was 510mm.

Figure 4.10 is a cross-section of the designed pavement layers, their thickness sections, dimension of the pavement layers.

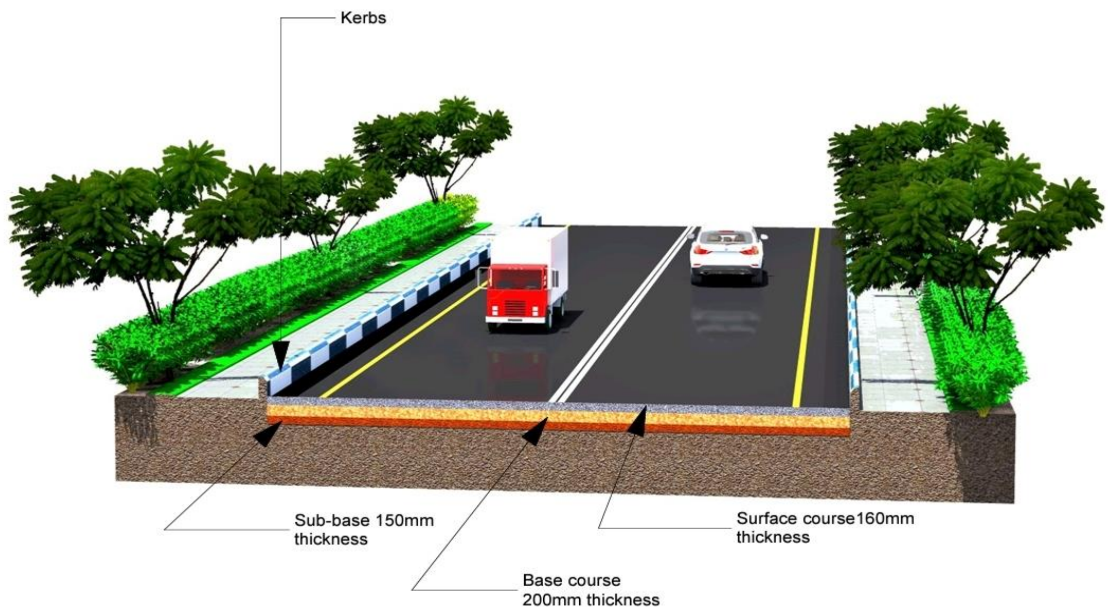

Figure 4.10: General Arrangement of Designed Pavement Layers

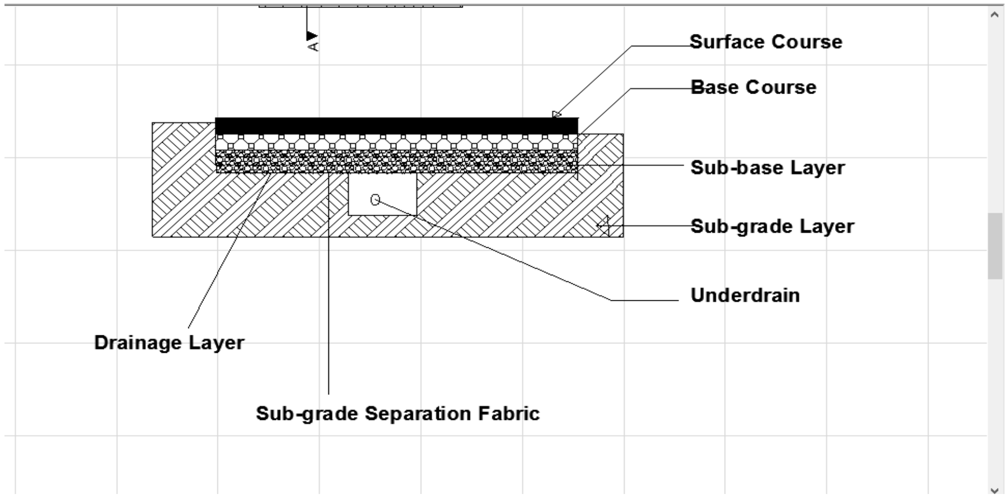

Figure 4.11: Section A-A/B-B

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The following conclusion in the light of the findings obtained on design of Elias Egboka-Nwakpadolu roads, the following deductions can be drawn:

1 Preliminary testing of the three different samples collected at different locations within the road under study classified the samples as A-2-4 and A-2-6 according to AASHTO soil classification system, the specific gravity of the samples ranged from 2.62 to 2.66, liquid limit of the samples ranged from 41 -45%, while the plastic limit and plasticity index ranged from 17.4-25.9% and 16.5%- 24.2% respectively.

2 The samples satisfied most of the specification given by Federal Ministry of Works and Housing, (1999) for road construction materials.

3 The average daily traffic evaluated from the traffic count survey was 832pcu/hr/day with the highest percentage of traffic composition recorded on day 7 (Sunday) while the lowest percentage of traffic composition was recorded on day 1 (Monday).

4 The design CBR adopted for the sample tested were 11% as this will give the most critical condition the soil will be subjected to during service.

5 The thickness of the surface course, base course and sub-base course obtained from design of Elias Egboka-Nwakpadolu road were 160mm, 200mm and 150mm respectively.

6 The total thickness of the pavement layers superimposed on the sub-grade soil was 510mm.

7 The design output was adjudged to be structurally reliable due to the fact that the designed pavement layers provided can withstand both the present and anticipated traffic loads and other effects during service, in other words, they account for the stress-strain properties of the pavement structure.

5.2 Recommendation

The following recommendations based on the findings gleaned from the design of Elias Egboka-Nwakpadolu roads can be made:

1 AASHTO method of pavement design should be used in contemporary design of Nigeria roads since it takes into account, the stress –strain properties of the soil which subsequently, will require less maintenance cost after construction.

2 Further studies should be undertaken to compare the structural reliability of other pavement design methods as a means of validating the recommendation stated in step 1.

3 I recommend that the government construct the road using AASHTO method for longer design life, also to ease movement of students into the university.

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APPENDICES

APPENDIX A

SPECIFIC GRAVITY TEST

Table A1. Specific Gravity Result for Sample A.

Determinants	**Trial 1**	**Trial 2**	**Trial 3**
Wt of density bottle, W_1 (g).	24.76	25.64	25.90
Wt of bottle + dry soil, W_2 (g).	34.74	35.63	35.90
Wt of bottle + soil + water, W_3 (g).	84.33	85.15	85.79
Wt of bottle + water, W_4 (g).	78.07	78.94	79.56

The Specific gravity of the sample is calculated as follows:

Specific Gravity for Sample A.

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.74-24.76)}{(34.74-24.76)-(84.33-78.07)} = 2.68$

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.63-25.64)}{(35.63-25.64)-(85.15-78.94)} = 2.64$

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.90-25.90)}{(35.90-25.90)(85.79-79.56)} =2.65$

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.68+2.64+2.65)}{3} = 2.66$

Table A2. Specific Gravity Result for Sample B

Determinants	**Trial 1**	**Trial 2**	**Trial 3**
Wt of density bottle, W_1 (g).	23.87	25.21	25.54
Wt of bottle + dry soil, W_2 (g).	33.86	35.21	35.54
Wt of bottle + soil + water, W_3 (g).	82.91	81.13	79.94
Wt of bottle + water, W_4 (g).	76.72	74.95	73.77

The Specific gravity of the sample is calculated as follows:

Specific Gravity for Sample B.

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(33.86-23.87)}{(33.86-23.87)-(82.91-76.72)} = 2.63$

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.21-25.21)}{(35.21-25.21)-(81.13-74.95)} = 2.62$

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.54-25.54)}{(35.54-25.54)(79.94-73.77)} =2.61$

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.63+2.62+2.61)}{3} = 2.62$

Table A3: Specific Gravity Result for Sample C

Determinants	**Trial 1**	**Trial 2**	**Trial 3**
Wt of density bottle, W_1 (g).	26.14	25.88	25.26
Wt of bottle + dry soil, W_2 (g).	36.14	35.87	35.25
Wt of bottle + soil + water, W_3 (g).	86.14	84.26	82.14
Wt of bottle + water, W_4 (g).	79.96	78.13	76.02

The Specific gravity of the sample is calculated as follows:

Specific Gravity for Sample C.

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.14-26.14)}{(36.14-26.14)-(86.14-79.96)}$ = 2.62

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.87-25.88)}{(35.87-25.88)-(84.26-78.13)}$ = 2.59

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.25-25.26)}{(35.25-25.26)(82.14-76.02)}$ = 2.58

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.62+2.59+2.58)}{3}$ = 2.60

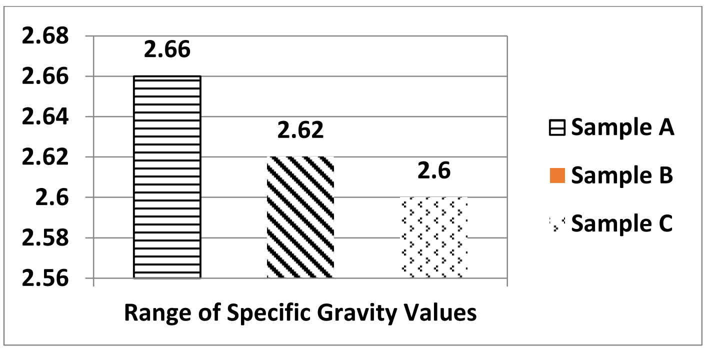

Figure A1: Result Showing the Specific Gravity Values of the Samples

APPENDIX B

SIEVE ANALYSIS TEST

Table B1: Sieve Analysis Test Result for Sample A

Sieve Sizes (mm)	Mass Retained (g)	% Mass Retained	Cum% Retained	Cum % Finer
2	0.03	0.006	0.006	99.994
1.18	8.42	1.684	1.69	98.31
0.85	16.53	3.306	4.996	95.004
0.6	44.8	8.96	13.956	86.044
0.425	52.43	10.486	24.442	75.558
0.3	106.9	21.38	45.822	54.178
0.15	189.4	37.88	83.702	16.298
0.075	30.6	6.12	89.822	10.178
Tray	9.87	1.974	91.796	8.204

Figure B1: Particle Size Distribution Curve for Sample A

Table B2: Sieve Analysis Result for Sample B

Sieve Sizes (mm)	Mass Retained (g)	% Mass Retained	Cum% Retained	Cum % Finer
2	0.04	0.008	0.008	99.992
1.18	9.23	1.846	1.854	98.146
0.85	17.21	3.442	5.296	94.704

0.6	45.63	9.126	14.422	85.578
0.425	50.41	10.082	24.504	75.496
0.3	92.98	18.596	43.1	56.9
0.15	198.3	39.66	82.76	17.24
0.075	32.4	6.48	89.24	10.76
Tray	11.2	2.24	91.48	8.52

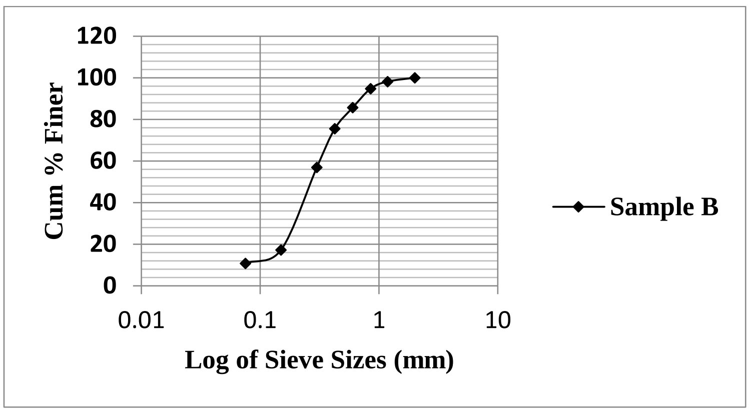

Figure B2: Particle Size Distribution Curve for Sample B2

Table B3: Particle Size Distribution Curve for Sample C

Sieve Sizes (mm)	**Mass Retained (g)**	**% Mass Retained**	**Cum% Retained**	**Cum % Finer**
2	0.02	0.004	0.004	99.996
1.18	7.48	1.496	1.5	98.5
0.85	15.93	3.186	4.686	95.314
0.6	38.94	7.788	12.474	87.526
0.425	56.87	11.374	23.848	76.152
0.3	121.6	24.32	48.168	51.832
0.15	200.4	40.08	88.248	11.752
0.075	35.7	7.14	95.388	4.612
Tray	10.08	2.016	97.404	2.596

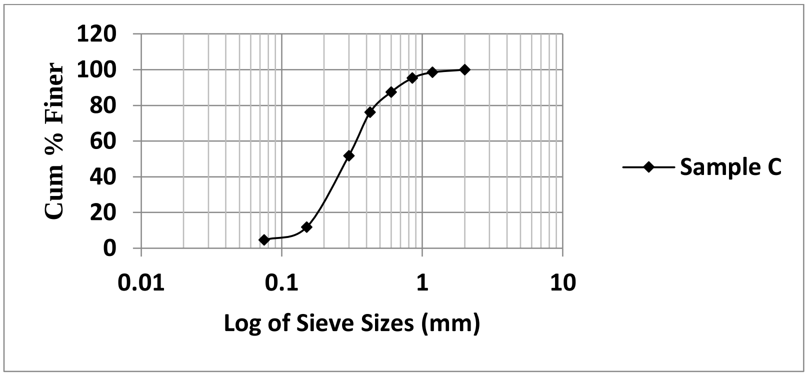

Figure B3: Particle Size Distribution Curve for Sample C

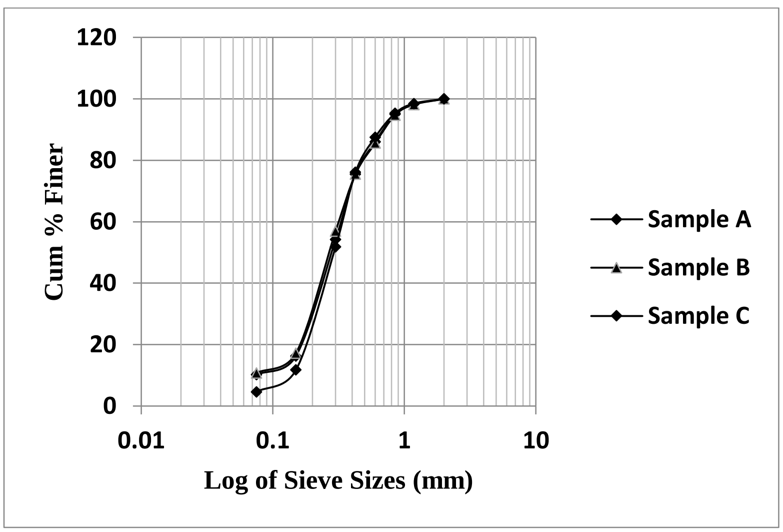

Figure B4: Particle Size Distribution Curve for all Samples.

APPENDIX C

LIQUID AND PLASTIC LIMIT TEST

Table C1: Liquid Limit Result for Sample A

Blows	**32**	**27**	**21**	**18**	**13**
Wt of empty tin (g)	15.24	14.86	15.46	17.81	17.04
Wt of tin + wet soil (g)	35.04	40.44	56.18	60.4	45.66
Wt of wet soil	19.8	25.58	40.72	42.59	28.62
Wt of tin + oven dried soil (g)	29.96	33.18	43.02	46.08	35.22
Wt of oven dried soil (g)	14.72	18.32	27.56	28.27	18.18
Wt of water (g)	5.08	7.26	13.16	14.32	10.44
Moisture Content (%)	34.51086957	39.62882096	47.75036	50.6544	57.42574

Table C2: Plastic Limit Results for Sample A

Sample A	**Test 1**	**Test 2**	**Test 3**
Wt of empty tin (g)	14.68	13.58	14.21
Wt of tin + wet soil	31.77	35.84	38.31
Wt of wet soil (g)	17.09	22.26	24.1
Wt of tin + dry soil (g)	28.41	31.14	33.28
Wt of oven dried soil (g)	13.73	17.56	19.07
Wt of water (g)	3.36	4.7	5.03
Plastic Limit (%)	24.47	26.77	26.38

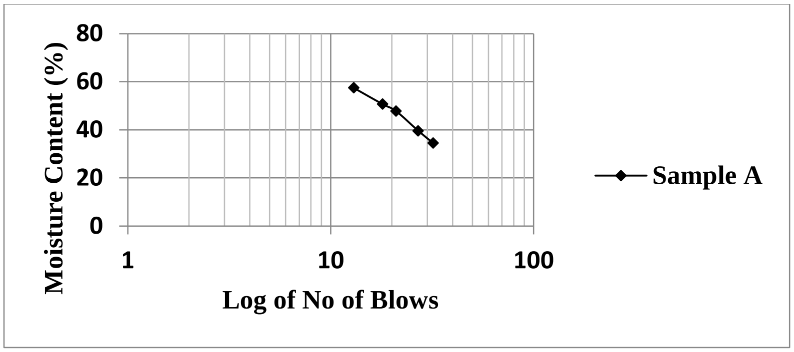

Figure C1: Liquid Limit Graph for Sample A

Table C3: Liquid Limit Test Result for Sample B

Blows	**33**	**28**	**22**	**18**	**13**
Wt of empty tin (g)	16.34	14.52	15.46	14.44	16.82
Wt of tin + wet soil (g)	35.34	40.44	56.18	60.4	45.66
Wt of wet soil	19	25.92	40.72	48.41	28.84
Wt of tin + oven dried soil (g)	30.67	33.18	43.12	46.08	35.02
Wt of oven dried soil (g)	14.33	18.66	27.66	31.64	18.2
Wt of water (g)	4.67	7.26	13.06	16.77	10.64
Moisture Content (%)	32.58897418	38.90675241	47.2162	53.00253	58.46154

Table C4: Plastic Limit Test Result for Sample B

Sample B	**Test 1**	**Test 2**	**Test 3**
Wt of empty tin (g)	13.51	14.48	15.21
Wt of tin + wet soil	40.03	32.14	30.08
Wt of wet soil (g)	26.52	17.66	14.87
Wt of tin + dry soil (g)	36.49	29.14	27.98
Wt of oven dried soil (g)	22.98	14.66	12.77
Wt of water (g)	3.54	3	2.1
Plastic Limit (%)	15.40	20.46	16.44

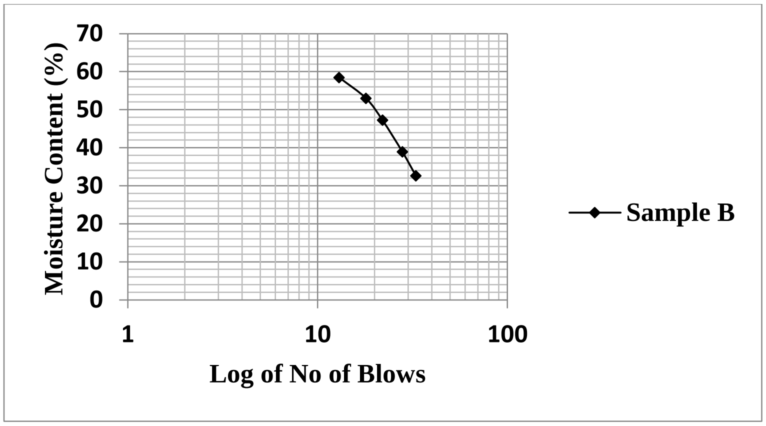

Figure C2: Liquid Limit Graph for Sample B

Table C5: Liquid Limit Test Result for Sample C

Blows	**32**	**27**	**23**	**17**	**13**
Wt of empty tin (g)	15.21	13.64	16.41	15.32	16.22
Wt of tin + wet soil (g)	30.04	35.57	52.18	41.6	32.33
Wt of wet soil	14.83	21.93	35.77	48.41	16.11
Wt of tin + oven dried soil (g)	27.04	30.21	41.56	47.58	26.34
Wt of oven dried soil (g)	11.83	16.57	25.15	32.26	10.12
Wt of water (g)	3	5.36	10.62	16.15	5.99
Moisture Content (%)	25.35925613	32.34761617	42.22664	50.062	59.18972

Table C6: Plastic Limit Test Result for Sample C

Sample C	**Test 1**	**Test 2**	**Test 3**
Wt of empty tin (g)	14.21	13.82	15.21
Wt of tin + wet soil	28.84	25.22	28.91
Wt of wet soil (g)	14.63	11.4	13.7
Wt of tin + dry soil (g)	26.51	23.18	26.41
Wt of oven dried soil (g)	12.3	9.36	11.2
Wt of water (g)	2.33	2.04	2.5
Plastic Limit (%)	18.94	21.79	22.32

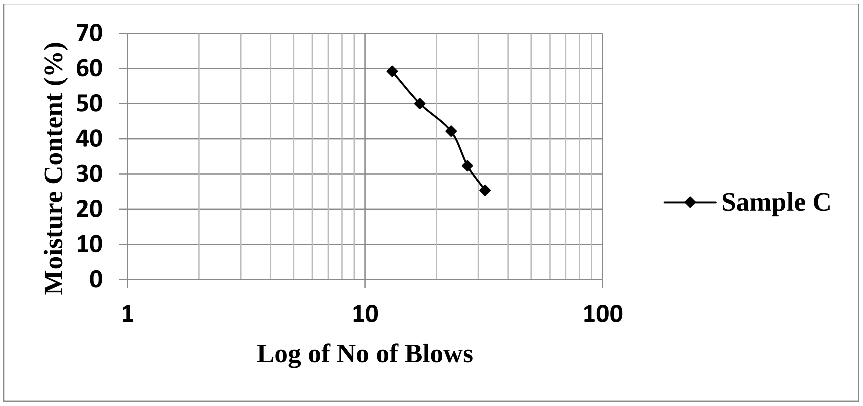

Figure C3: Liquid Limit Graph for Sample C

APPENDIX D

COMPACTION TEST

Table D1: Dry Unit Weight Result for Sample A

Percentages of Water	**Vol of Mould**	**Wt of Mould**	**Wt of Mould + Wet Soil**	**Wt of Wet Soil**	**Bulk Density**	**Moisture Content**	**Dry Unit Weight**
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.9	1.9	18.64	4.91	17.77
8	0.001	4	6.15	2.15	21.09	8.33	19.47
12	0.001	4	6.3	2.3	22.56	11.54	20.23
16	0.001	4	6.25	2.25	22.07	16.31	18.98
20	0.001	4	6.2	2.2	21.58	20.61	17.89

Table D1.1: Moisture Content Result for Sample A (Top)

Percentages of Water	**Wt of tin**	**Wt of tin + wet**	**Wt of wet Soil (g)**	**Wt of tin + dry Soil**	**Wt of dry**	**Wt of Water (g)**	**Moisture**
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.48	44.58	30.1	43.17	28.69	1.41	4.91
8	17.12	51.93	34.81	49.25	32.13	2.68	8.34
12	15.38	46.88	31.5	43.66	28.28	3.22	11.39
16	14.45	48.36	33.91	43.91	29.46	4.45	15.11
20	15.12	54.28	39.16	47.45	32.33	6.83	21.13

Table D1.2: Moisture Content Result for Sample A (Bottom)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	13.48	45.09	31.61	43.56	30.08	1.53	5.09
8	14.52	52.63	38.11	49.7	35.18	2.93	8.33
12	15.16	55.76	40.6	51.51	36.35	4.25	11.69
16	16.68	48.29	31.61	43.58	26.9	4.71	17.51
20	15.12	64.92	49.8	56.59	41.47	8.33	20.09

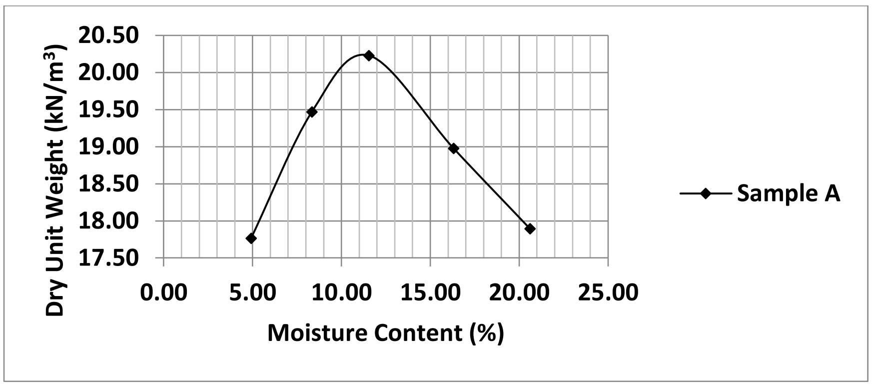

Figure D1: Compaction curve for Sample A

Table D2: Dry Unit Weight Results for Sample B

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	6.05	2.05	20.11	6.27	18.92
8	0.001	4	6.2	2.2	21.58	7.76	20.03
12	0.001	4	6.25	2.25	22.07	9.85	20.09
16	0.001	4	6.2	2.2	21.58	15.06	18.76
20	0.001	4	6.1	2.1	20.60	19.39	17.26

Table D2.1: Moisture Content Determination for Sample B (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.46	48.62	34.16	46.83	32.37	1.79	5.53
8	15.38	49.52	34.14	47.11	31.73	2.41	7.60
12	13.78	51.64	37.86	48.44	34.66	3.2	9.23
16	16.33	52.95	36.62	48.69	32.36	4.26	13.16
20	17.48	66.72	49.24	58.87	41.39	7.85	18.97

Table D2.2: Moisture Content Determination for Sample B (Bottom)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.24	52.87	38.63	50.34	36.1	2.53	7.01
8	14.86	55.74	40.88	52.74	37.88	3	7.92
12	15.13	58.26	43.13	54.17	39.04	4.09	10.48
16	14.54	60.82	46.28	54.11	39.57	6.71	16.96
20	16.38	65.72	49.34	57.56	41.18	8.16	19.82

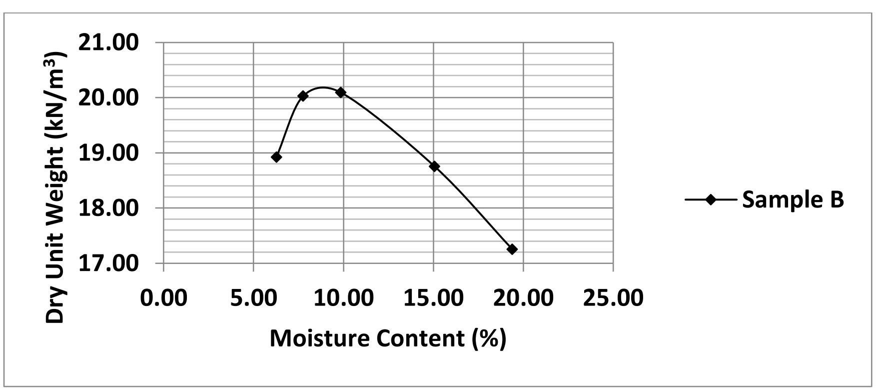

Figure D2: Compaction Curve for Sample B

Table D3: Dry Unit Weight Result for Sample C

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.6	1.6	15.70	3.73	15.13
8	0.001	4	5.95	1.95	19.13	6.08	18.03
12	0.001	4	6.1	2.1	20.60	11.17	18.53
16	0.001	4	6.15	2.15	21.09	14.61	18.40
20	0.001	4	6.05	2.05	20.11	22.73	16.39

Table D3.1: Moisture Content Result for Sample C (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.71	28.34	13.63	27.85	13.14	0.49	3.73
8	15.03	40.04	25.01	39.04	24.01	1	4.16
12	14.31	46.22	31.91	43.06	28.75	3.16	10.99
16	17.82	65.65	47.83	59.54	41.72	6.11	14.65
20	13.56	63.34	49.78	52.78	39.22	10.56	26.93

Table D3.2: Moisture Content Result for Sample C (Bottom)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	15.83	32.73	16.9	32.12	16.29	0.61	3.74
8	14.44	46.43	31.99	44.06	29.62	2.37	8.00
12	14.06	45.44	31.38	42.24	28.18	3.2	11.36
16	17.45	55.99	38.54	51.09	33.64	4.9	14.57
20	13.79	55.75	41.96	49.19	35.4	6.56	18.53

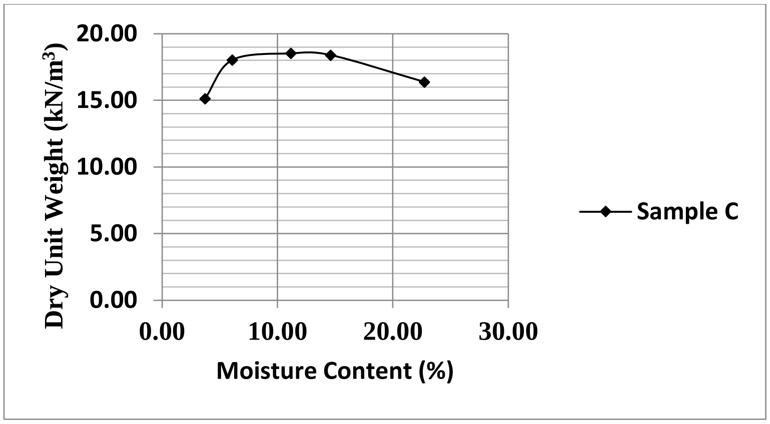

Figure D3: Compaction Curve for Sample C

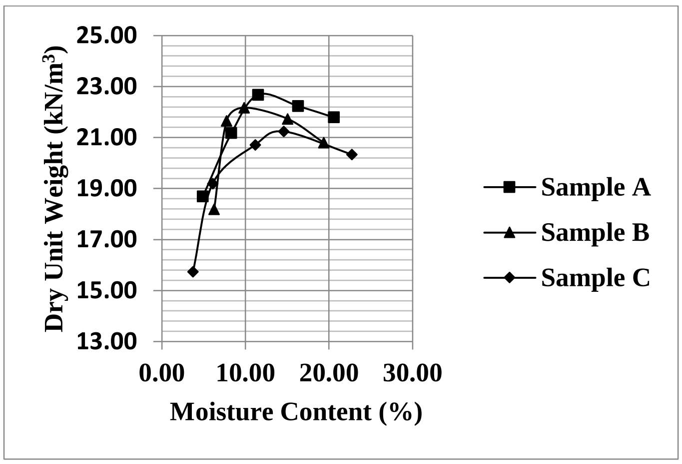

Figure D4: Combined Compaction Curve for the Samples

APPENDIX E

CALIFORNIA BEARING RATIO TEST

Table E1: Un-soaked CBR Test Result for Sample A

Penetration (mm)	Dial Reading Top	Top Force (kN)	Dial Reading (Bottom)	Bottom Force (kN)
0.5	36	2.4	10	2
1	54	2.7	25	2.3
1.5	71	3.2	38	2.7
2	84	3.6	52	3
2.5	96	4	67	3.3
3	108	4.3	82	3.7
3.5	115	4.7	99	4.2
4	122	5.1	110	4.6
4.5	126	5.4	126	5.1
5	130	5.8	137	5.4
5.5	136	6.1	148	5.8
6	140	6.4	157	6.1
6.5	154	6.7	166	6.4
7	158	7	174	6.8

Table E2: Un-soaked CBR Test Result for Sample B

Penetration (mm)	Dial Reading Top	Top Force (kN)	Dial Reading (Bottom)	Bottom Force (kN)
0.5	36	2.1	10	1.6
1	54	2.5	25	2
1.5	71	2.9	38	2.3
2	84	3.3	52	2.6
2.5	96	3.6	67	2.9
3	108	3.9	82	3.3
3.5	115	4.3	99	3.7
4	122	4.7	110	4.1
4.5	126	5.1	126	4.4
5	130	5.4	137	4.7
5.5	136	5.9	148	5
6	140	6.2	157	5.4
6.5	154	6.5	166	5.8
7	158	6.8	174	6.1

Table E3: Un-soaked CBR Test Result for Sample C

Penetration (mm)	Dial Reading Top	Top Force (kN)	Dial Reading (Btm)	Bottom Force (kN)
0.5	36	1.8	10	1.3
1	54	2.2	25	1.6
1.5	71	2.5	38	1.9
2	84	2.9	52	2.3
2.5	96	3.3	67	2.6
3	108	3.6	82	2.9
3.5	115	3.9	99	3.4
4	122	4.2	110	3.7
4.5	126	4.5	126	4
5	130	4.7	137	4.3
5.5	136	5	148	4.5
6	140	5.2	157	4.8
6.5	154	5.4	166	5.1
7	158	5.8	174	5.4

Table E4: Soaked CBR Test Result for Sample A

Penetration (mm)	Dial Reading Top	Top Force (kN)	Dial Reading (Bottom)	Bottom Force (kN)
0.5	36	0.7	10	0.3
1	54	1.1	25	0.5
1.5	71	1.3	38	0.8
2	84	1.6	52	1.1
2.5	96	1.9	67	1.3
3	108	2.1	82	1.6
3.5	115	2.4	99	1.9
4	122	2.7	110	2.2
4.5	126	2.9	126	2.4
5	130	3.1	137	2.6
5.5	136	3.3	148	2.9
6	140	3.6	157	3.1
6.5	154	3.9	166	3.4
7	158	4.1	174	3.6

Table E5: Soaked CBR Test Result for Sample B

Penetration (mm)	Dial Reading Top	Top Force (kN)	Dial Reading (Bottom)	Bottom Force (kN)
0.5	36	0.5	10	0.3
1	54	0.8	25	0.6
1.5	71	1.1	38	0.8
2	84	1.3	52	1.0
2.5	96	1.5	67	1.3
3	108	1.8	82	1.5
3.5	115	2.1	99	1.7
4	122	2.3	110	2.0
4.5	126	2.6	126	2.2
5	130	2.9	137	2.4
5.5	136	3.1	148	2.7
6	140	3.3	157	2.9
6.5	154	3.6	166	3.2
7	158	3.8	174	3.4

Table E6: Soaked CBR Test Result for Sample C

Penetration (mm)	Dial Reading Top	Top Force (kN)	Dial Reading (Bottom)	Bottom Force (kN)
0.5	36	0.4	10	0.3
1	54	0.6	25	0.5
1.5	71	0.9	38	0.8
2	84	1.2	52	1.1
2.5	96	1.4	67	1.3
3	108	1.7	82	1.5
3.5	115	1.9	99	1.8
4	122	2.2	110	2.1
4.5	126	2.4	126	2.3
5	130	2.7	137	2.5
5.5	136	3.0	148	2.7
6	140	3.3	157	2.9
6.5	154	3.6	166	3.1
7	158	3.8	174	3.3

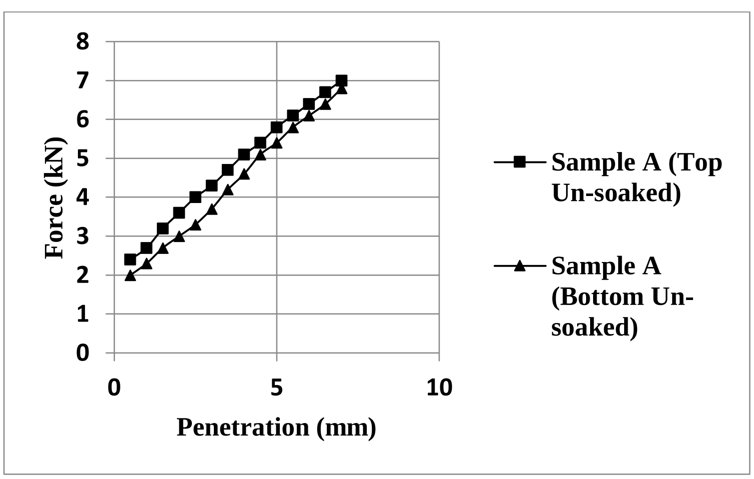

Figure E1: Un-soaked California Bearing Ratio Curve for Sample A

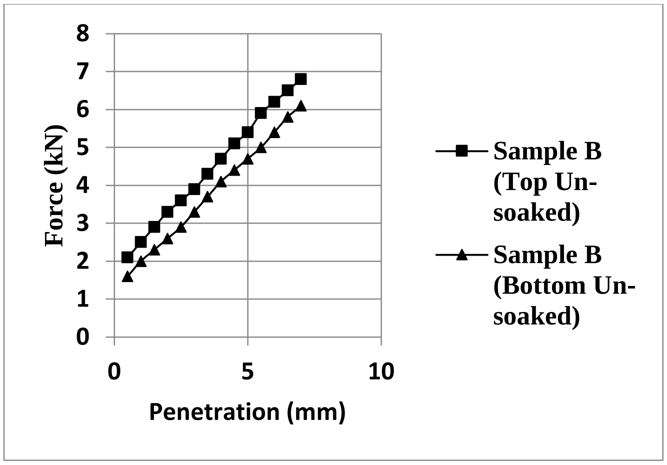

Figure E2: Un-soaked California Bearing Ratio Curve for Sample B

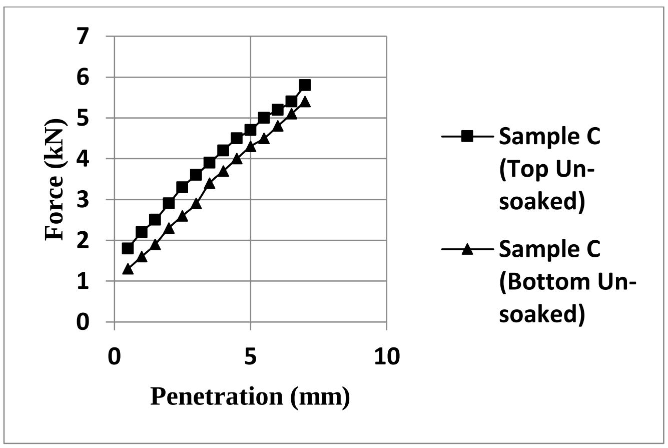

Figure E3: Un-soaked California Bearing Ratio Curve for Sample C

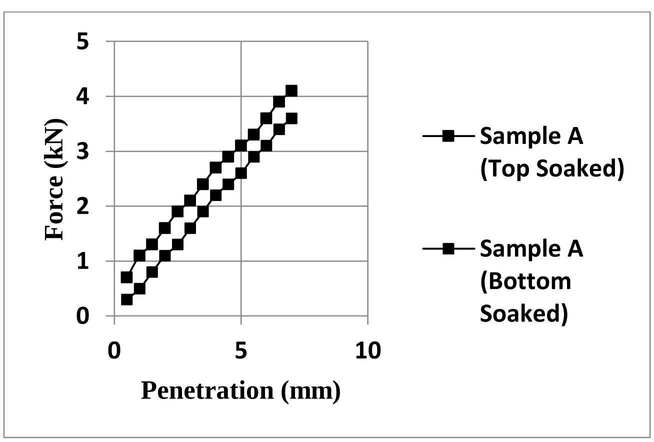

Figure E4: Soaked California Bearing Ratio Curve for Sample A

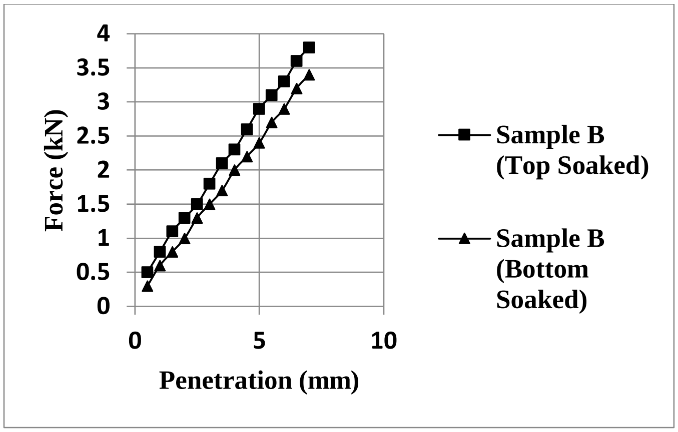

Figure E5: Soaked California Bearing Ratio Curve for Sample B

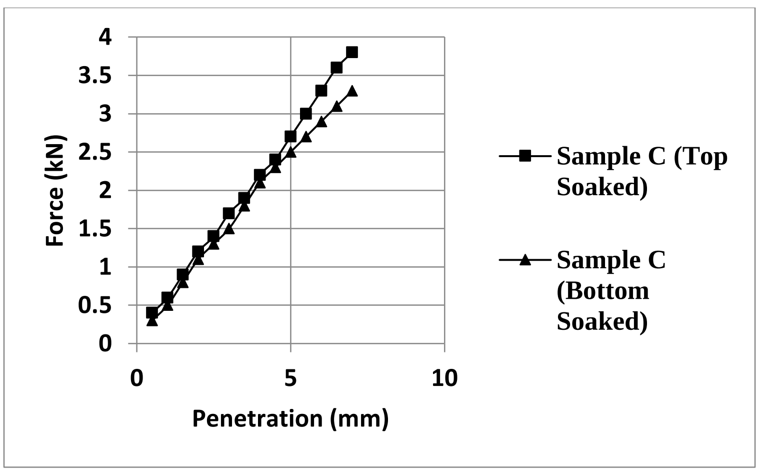

Figure E6: Soaked California Bearing Ratio Curve for Sample C

APPENDIX F

TRAFFIC COUNT DATA

Table F1: Summary of Traffic Count Survey Data

Days/Equivalent P. C. U.	**Cars and Taxi**	**Buses**	**Bikes**	**Trucks and Lorries**	**Total Vehicle (P. c. u)**
Equivalent P. C. U.	1.0	2.8	0.75	2.0	
Monday	156	330.4	102	112	700.4
Tuesday	177	434	94.5	128	833.5
Wednesday	168	442.4	90	170	870.4
Thursday	148	462	88.5	104	802.5
Friday	178	504	64.5	116	862.5
Saturday	209	414.4	82.5	144	849.9
Sunday	218	462	78.8	160	918.8

Average Daily Traffic (ADT) = $\frac{5838 \times 52}{365}$ **= 831.72pcu/hr/day**