EVALUATING THE RUTTING RESISTANCE OF ASPHALT CONCRETE MODIFIED WITH ORGANO-PHILIC MONTIMORILLONITE NANO CLAY (OMMT)

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M22B32/003

A DISSERTATION SUBMITTED TO THE FACULTY OF ENGINEERING, DESIGN AND TECHNOLOGY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE OF BACHELOR OF SCIENCE IN CIVIL AND ENVIRONMENTAL ENGINEERING OF UGANDA CHRISTIAN UNIVERSITY

February, 2026

UGANDA CHRISTIAN UNIVERSITY

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ABSTRACT

The long term performance of asphalt pavements is a major consideration in designing durable and safe road infrastructure, especially in low and middle income countries where flexible pavements dominate due to their lower cost and ease of maintenance. Rutting, a critical form of permanent deformation occurring in wheel paths, remains one of the most detrimental pavement failures, contributing to traffic accidents and accelerated deterioration. This study evaluated the effectiveness of organophilic montmorillonite (OMMT) nanoclay in improving the rutting resistance of asphalt concrete by enhancing its mechanical and volumetric properties. Unlike studies that use untreated montmorillonite, this research employed OMMT to achieve better binder filler interaction and dispersion within the asphalt matrix.

Marshall mix design results identified 1.0% OMMT by mass of aggregate as the optimum dosage, producing a Marshall Stability of 17.2kN, Flow of 3.6mm, Air Voids of 4.2%, VMA of 15.3%, VFB of 72.4%, and a bulk density of 3.352g/cm^3. Indirect tensile strength test confirmed that OMMT significantly reduces rutting susceptibility by strengthening the binder phase and improving stress distribution within the aggregate structure under heavy axle loading.

DECLARATION

I hereby declare that the project proposal report titled "Evaluating the Rutting Resistance of Asphalt Concrete Modified with Organo-philic Montmorillonite Nano clay" has been submitted to fulfill part of the requirements for the award of the Bachelor of Science degree in Civil and Environmental Engineering at Uganda Christian University. This is my original work and has not been wholly or partly submitted to any institution or university and is yet to receive any academic award. All sources of information used to develop the project title have been fully referenced.

Signature: ……………………………….

Date: ……………………………………….

ASONYA JOHN

APPROVAL

This research and design project report is hereby submitted to be examined, with the approval of the below mentioned university supervisor.

Signature: ……………………………….

Date: ……………………………………….

ENG. DR. MORRIS OLENG

ACADEMIC SUPERVISOR

DEDICATION

I dedicate this report to my parents, teachers, and mentors. They have been constantly backing me, and it has had a very important role to play in my academic career. You have always motivated me, and I am very grateful to you for believing so much in me. This accomplishment is not just mine but also yours. I am forever grateful to you.

ACKNOWLEDGEMENTS

First and foremost, I thank GOD Almighty for having granted me strength and persistence during my academic pursuits. In giving thanks, I am also expressing appreciation to my supervisor, ENG. DR. MORRIS OLENG, who has provided guidance and support during the completion of my study. In addition, I am grateful to MR. ROGERS TAYEBWA, my research coordinator, who offered his insights which were beneficial towards the completion of this project report. Lastly, I also thank the technical team, ENG. EDUARD MARTIN and ENG. ROBIN, who provided technical support and assistance during my lab consultations.

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LIST OF ABBREVIATIONS

AC- Asphalt Concrete

ACV – Aggregate Crushing Value

HMA- Hot Mix Asphalt

TFV- Ten Percent Fines

LAA- Los Angeles Abrasion

ITS- Indirect Tensile Strength

AIV- Aggregate Impact Value

MMT- Montmorillonite

OMMT- Organo-philic Montmorillonite

PCC- Portland Cement Concrete

VMA- Voids in Mineral Aggregate

VFB- Voids Filled with Bitumen

SG- Specific Gravity

AASHTO- American Association of State Highway and Transportation Officials

ASTM- American Society for Testing and Materials

CHAPTER ONE: INTRODUCTION

1.1 Background

Road infrastructure is an important catalyst not only of economic but also social progress. Specifically, the flexible pavement road infrastructure, which is mainly built using conventional bitumen, is the spine of the transport sector (UNRA, 2023). Nevertheless, the mechanical deterioration of the road pavement infrastructure, such as the formation of potholes, ruts, and cracks, continues to persist and has resulted in a threat to road safety, increased costs of road maintenance, and the failure to optimize economic activities (Mugo et al., 2021). The performance of flexible pavement road infrastructure is greatly dependent on the properties of asphalt concrete, particularly the ability to resist repeated axial loading generated by the constant passage of heavy vehicles. Bitumen should possess adequate strength and elasticity to counteract permanent deformation and develop fatigue cracks due to such loading. Unfortunately, conventional bitumen is not mechanically strong enough to effectively withstand such forces, thereby promoting early deterioration of pavement road infrastructure. In addition, the inability of conventional pavement bitumen to effectively withstand changes in temperatures and exposure to moisture has further undermined the technical integrity of pavement road infrastructure, which is often characterized by recurrent road repairs and shortened lifespan (Apeagyei et al., 2018; Kisaaka et al., 2022). Breakthroughs and new discoveries under Nano clay technology, specifically Montmorillonite, have recently emerged and show great promise to improve conventional asphalt materials. Specifically, MMT has a high surface area and a 2:1 layer silicate structure which enables easier dispersion and interaction with binder to form an intercalated nanostructure which also limits the free movement of molecules, thereby showing improvements over conventional asphalt binder mechanical strength, shear strength, and thermal stability.

In earlier studies, the predominantly unprocessed montmorillonite used has been largely incompatible with the hydrophobic bitumen phase. The novelty of the present study is, therefore, the utilization of the organo-philic montmorillonite (OMMT), which is an organically modified montmorillonite treated with organic cations to improve dispersions of the montmorillonites within the bitumen phase. This study is, therefore, intended to address what is often considered to date to be one of the most problematic failure modes of flexible pavement, namely the failure due to rutting, by examining the role of a new asphalt additive, namely the OMMT Nano clay, to improve asphalt specimens. The addition of the new material, OMMT Nano clay, has increased the stiffness, shear strength, and strength properties of the asphalt mix, which has enabled it to effectively contend against the high contact pressure and cyclic vertical loads to which the pavement is subjected by commercial traffic, besides showing improvements with regards to thermal softening.

1.2 PROBLEM STATEMENT

The asphalt road infrastructure, particularly in Uganda, is fast degrading, mainly because of the inability to support high axle loads, which causes rutting, fatigue, and moisture-related deterioration. Of great concern is the rate of rutting, which not only limits the pavement's lifespan but also threatens road safety to various road users. Axle loads on asphalt pavement cause point loads, which result in cyclic loading on the road pavement. When subjected to numerous loads, pavement materials accumulate plastic strain, reducing the pavement structure's integrity.

Despite the regular maintenance works, the road section between Mukono and Mbalala, which is along the Kampala-Jinja Highway, is riddled with pavement rutting. This road is characterized by frequent high peak axle loads associated with high traffic volumes of commercial vehicles. Moreover, trucks contribute to increased plastic strain buildup within the asphalt layer, therefore significantly increasing ruts' rate of formation (Kaloush, Rada, & Youcef, 2001).

Though high temperatures also contribute to increased binder softening, it is apparent on various case histories along Uganda's road pavement infrastructure that the rate of pavement deterioration, such as rutting, is largely dependent on pavement exposure to high axle loads. This study therefore aimed at increasing the road pavement's ability to resist various cyclic tire loads by modifying asphalt concrete with Organo-philic Montmorillonite (OMMT).

1.3 RESEARCH OBJECTIVES

1.3.1 Main Objective

To evaluate the rutting resistance of asphalt concrete modified with organo-philic montmorillonite (OMMT) Nano clay.

1.3.2 Specific Objectives

1. To characterize the properties of OMMT, Aggregates, and Bitumen.
2. To determine the optimum OMMT content to be used in the mix
3. To determine the mechanical properties and rutting performance of asphalt concrete modified with Montmorillonite Nano clay.

1.4 RESEARCH QUESTIONS

1. What are the physical and mechanical properties of Organo-philic Montmorillonite Nano clay, bitumen, and aggregates used in asphalt concrete production?
2. What is the optimum percentage of Organo-philic Montmorillonite Nano clay that can be added to asphalt mix?
3. How does the addition of Organo-philic Montmorillonite Nano clay affect the mechanical and performance properties of asphalt concrete?

1.5 SCOPE OF STUDY

Mukono- Mbalala section of the Kampala-Jinja High way

1.6 SCIENTIFIC JUSTIFICATION

Rutting is a critical issue associated with asphalt pavement due to cyclic axle loading, which causes permanent deformation and a shortened service life (AASHTO, 2017). Adding OMMT nano clay to asphalt mixtures has proved to increase their anti-rutting properties and improve pavement mechanical properties (You, Zhan, & Zhang, 2011). Organo-philic Montmorillonite nano clay is characterized by plate-shaped silicate layers, which, when mixed with bitumen, result in intercalated and exfoliated nano materials (Jahromi & Khodaii, 2009). These layers impede the movement of molecules, which leads to increased binder rigidity and reduced movement due to high loads, ultimately increasing the anti-rutting properties (Khattak, Khosravi, & Zareif Spartaki, 2013). Various studies show that the raw form of MMT nano clay has promising properties which, when mixed with asphalt, enables it to support high axle loads and could ultimately result in a marked decrease in the depth of rutting (Liu, Zhang, & Liu, 2018). Improved mechanical properties of nanoclay-modified asphalt mixtures result due to increased aggregate and binder bonding and increased rigidity and ability to resist deformation (Wu, Zhang, Han, Li, & Ye, 2020).

CHAPTER TWO: LITERATURE REVIEW

2.1 Introduction

This chapter highlights a literature review on existing studies associated with the performance of asphalt pavement, with specific attention to the issue of rutting. The review starts by covering road pavement, the difference between rigid and flexible pavement, and then moves to the design of asphalt pavement and the essential properties of materials used, including bitumen, aggregates, and filler. In addition, it continues by looking into the nature of rutting and various other pavement failures to establish the difficulties associated with asphalt pavement when subjected to traffic and environment. Moreover, it delves into various methods of modification used to improve asphalt mixtures, including the utilization of conventional materials and, more advanced, nano clay technology. In specific, it focuses on montmorillonite nano clay, which has been indicated to possess properties and to interact positively with bitumen to increase pavement stiffness and resistance to rutting.

2.2 Road Pavements

Road pavements are the top structural components of a road and are essential in ensuring that the road has a safe and smooth riding surface. The basic purpose of road pavements is to transfer the traffic-generated loads to the soil underneath without adversely affecting it. Thus, a good pavement design should support frequent traffic without compromising ride ability and skid-resistant capabilities, besides possessing the ability to resist various environmental factors, including temperatures, rainfall, and oxidation (Huang, 2012). In Uganda and most countries on the East Africa continent, flexible pavement is the most common road pavement, which is built using bitumen materials. According to the Uganda National Roads Authority (UNRA, 2023), more than 70% of the road pavement is composed of flexible pavement owing to the lower costs associated with their construction and easy maintenance, and also because such road pavement is most suitable to countries which are developing, and the traffic on such countries is moderately high and growing very fast.

Nevertheless, tropical climatic influences, such as high temperatures, high rainfall, and wet-dry periods, result in pavement deterioration. High temperatures result in softening and oxidation of the binder, with eventual permanent pavement deformation, such as rutting and bleeding (Kisaaka et al. 2022). High rainfall and improper drainage systems also compromise the pavement layers, increasing the susceptibility to water-related deterioration, stripping, and cracking. Moreover, overloading by heavyweight trucks on important transport arteries such as the Kampala-Jinja Highway has been found to contribute to increased pavement deterioration and failure due to ruts (Mugo, Kimario, and Sakina 2021). Owing to such challenges, there has been a growing focus on enhancing the material properties of asphalt mixtures by modification procedures to improve the strength, stability, and resistance to deformation of such materials under high traffic and temperatures. The utilization of advanced materials, such as organo-philic montmorillonite nanoclay, has been identified to act as a remedy to improve the lifespan and performance of road materials under the tropical conditions present in Uganda.

2.3 Types of Pavements

There are usually two major types of pavements namely flexible pavements and rigid pavements. The most common form in the world is flexible pavements, which are mainly based on asphalt, as it is cheaper to install and they are not as difficult to maintain. On the contrary, well-established rigid pavements are always built with the help of Portland cement concrete and are characterized by their high strength, longer lifespan, and their ability to withstand deformation due to the heavy traffic loads (Khanna and Justo, 2011).

2.3.1 Flexible Pavements

The most common type of pavement in the globe, especially in the developing encountered countries, is flexible pavements as they are relatively cheap to put in place and are also easy to repair when damaged as opposed to the rigid forms of pavements. The flexible term is attributed to the phenomenon that the pavement structure will bounce during the impact of the wheels and will assign the stresses to the lower layers over time without serious cracking (Huang, 2012).

2.3.1.1 Structure of Flexible Pavements

A normal flexible pavement includes four primary layers:

a) **Surface Course:** This is normally composed of asphalt concrete; it is the layer which is exposed to the traffic. It is skid proof, comfortable to ride on, and waterproofing on bottom layers.

b) **Base Course:** This is the layer that is laid underneath the surface and in the general case, it is crushed rock or gravel. It spreads the loads on the wheels and provides structural capacity.

c) **Sub-base Course:** This is an intermediate layer that is laid between the base and the subgrade, commonly in areas where the subgrade does not have strong structural foundation. It enhances drainage and lessens the frost activity.

d) **Subgrade:** This refers to the soil base that forms the natural foundation and which should be stable and well compacted to hold the whole pavement (Kadiyali and Lal, 2016).

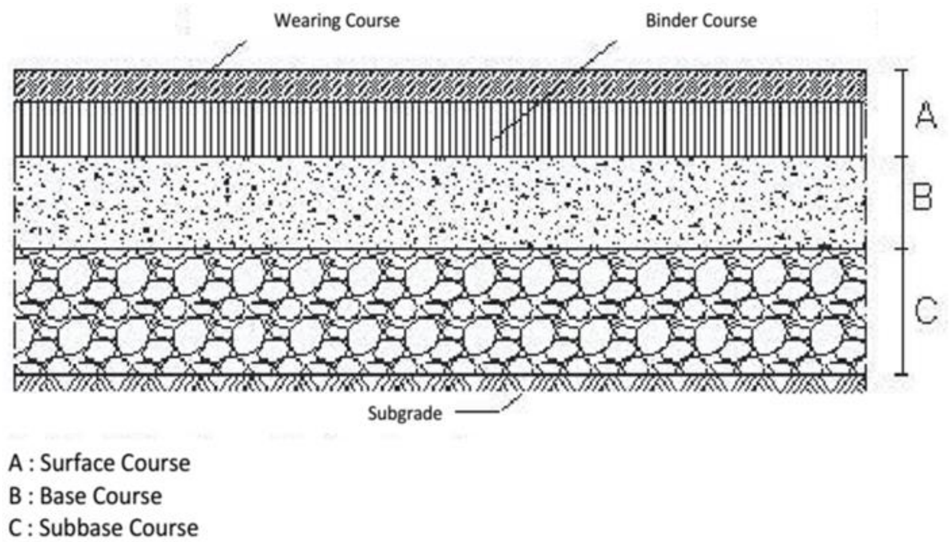


Figure 1: Shows layers of an asphalt flexible pavement

2.3.1.2 Load Distribution and Behavior

Under flexible pavements, the granular layers support loads on the edges of traffic by grain-to-grain contact to cause a funneling-shaped stress distribution pattern. This implies with depth the reduction in the intensity of stress and the quality of materials becomes the most vital in the highest depths (Khanna and Justo, 2011).

Elastic and plastic deformation of flexible pavements takes place as a result of repeated loading. Overloading or inadequate design may cause irreversible forms of deformations, which include rutting, shoving or surface cracking.

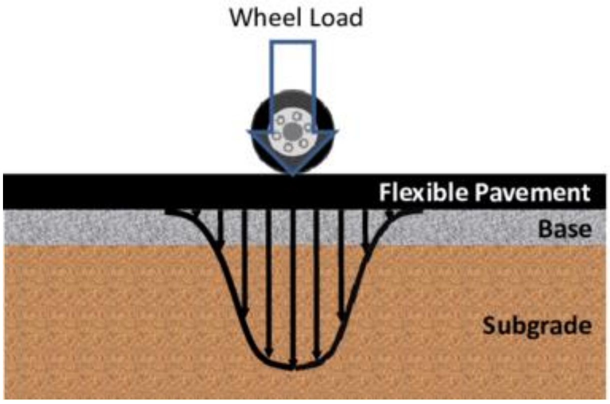


Figure 2: Shows load distribution in flexible pavements

2.3.1.3 Types of Flexible Pavements

Flexible pavements may be further classified based on the method of construction:

Conventional layered systems: These pavements are made up of surface, base, and sub-base layers.

Full-depth asphalt pavements: These ones consist of asphalt mixtures for all structural layers above the subgrade, used where high traffic loading is expected.

Contained rock asphalt mats (CRAM): These are specialized systems where large stone aggregates are bound with asphalt, used in special projects (Yoder & Witczak, 1975).

2.3.1.4 Performance Factors

The performance of flexible pavements is influenced by:

- Traffic loading (volume, axle load, frequency).
- Climatic conditions (temperature cycles, rainfall, and freeze-thaw).
- Material properties (bitumen grade, aggregate angularity, filler type).
- Construction quality (compaction, drainage, layer thickness control).

Good design must balance all these factors to minimize deformation and cracking while ensuring cost-effectiveness and serviceability.

2.3.1.5 Advantages of Flexible Pavements

- These pavements have lower initial construction cost compared to rigid pavements.
- They are quicker to construct and open for traffic.
- They are easier to repair and maintain since surface layers can be milled and replaced.
- The have the ability to accommodate minor settlements of the subgrade without significant cracking.

2.3.1.6 Limitations of Flexible Pavements

- They have a shorter design life compared to rigid pavements.
- They are more susceptible to rutting, fatigue cracking, and pothole formation under heavy traffic.
- They require frequent maintenance, which increases life-cycle costs.
- They are sensitive to variations in temperature and moisture, which affect bitumen properties (Huang, 2012).

2.3.2 Rigid Pavements

Rigid pavements are constructed using Portland cement concrete (PCC) and are referred to as “rigid” because they rely on the flexural strength of the concrete slab to distribute traffic loads over a wide area of the subgrade. Unlike flexible pavements, where load transfer is gradual through successive layers, rigid pavements spread the wheel loads over a much larger area due to the high stiffness and modulus of elasticity of concrete (Huang, 2012). This structural behavior makes rigid pavements less dependent on the strength of the underlying layers compared to flexible pavements.

2.3.2.1 Structure of Rigid Pavements

Rigid pavement is composed of the following parts:

Concrete Surface Course: This is the primary structural layer as it is resistant to flexural loads and is also durable to traffic and weather.

Base or Sub-base Layer: This layer will be laid in place directly under the concrete slab, it serves the purpose of to create uniform base, enhance load transfer and also to control pumping of fine materials.

Subgrade: It denotes the natural soil base, which however must still be compacted well although its effect in the flexible pavements is not as critical (Kadiyali and Lal, 2016).

Design of rigid pavement is most concerned with the thickness of the concrete slab because the slab is the one that supports and evenly distributes most of the applied load.

2.3.2.2 Types of Rigid Pavements

The rigid pavements are typically divided into the following three categories:

Jointed Plain Concrete Pavements (JPCP): These are concrete slabs that are spaced with closely spaced contraction joints in such a way that they curb cracking.

Jointed Reinforced Concrete Pavements (JRCP): JRCP uses steel reinforcement in addition to JPCP to minimize the joint spacing as well as the widths of the cracks.

Continuously Reinforced Concrete Pavements (CRCP): It is built using a continuous steel reinforcement such that there is no need of transverse joints. This kind of type offers high performance in heavy traffic (Khanna and Justo, 2011).

2.3.2.3 Performance Factors

The performance of rigid pavements depends on:

- Concrete mix design (cement type, aggregate properties, water-cement ratio).
- Joint design and reinforcement (spacing, dowel bars, tie bars).
- Sub-base quality (stability, drainage).
- Environmental factors (temperature cycles, moisture variations).
- Traffic loading (repetition and magnitude of axle loads).

Properly designed and built, rigid pavements would outperform flexible pavements in high-traffic corridors, access roads to industries, and locations subject to high axle loads.

2.3.2.4 Advantages of Rigid Pavements

- Longer service life, often 30 years or more with minimal maintenance.
- Higher resistance to rutting and permanent deformation.
- Better performance under very heavy traffic and high-temperature conditions.
- Lower maintenance requirements compared to flexible pavements.
- Provide smoother riding quality over time (Huang, 2012).

2.3.2.5 Limitations of Rigid Pavements

- High initial construction cost compared to flexible pavements.
- Longer curing time before opening to traffic.
- Require skilled labor and specialized equipment for construction.
- Repairs, when necessary, are more complex and expensive.
- Susceptible to cracking from shrinkage or temperature gradients if not properly designed (Kadiyali & Lal, 2016).

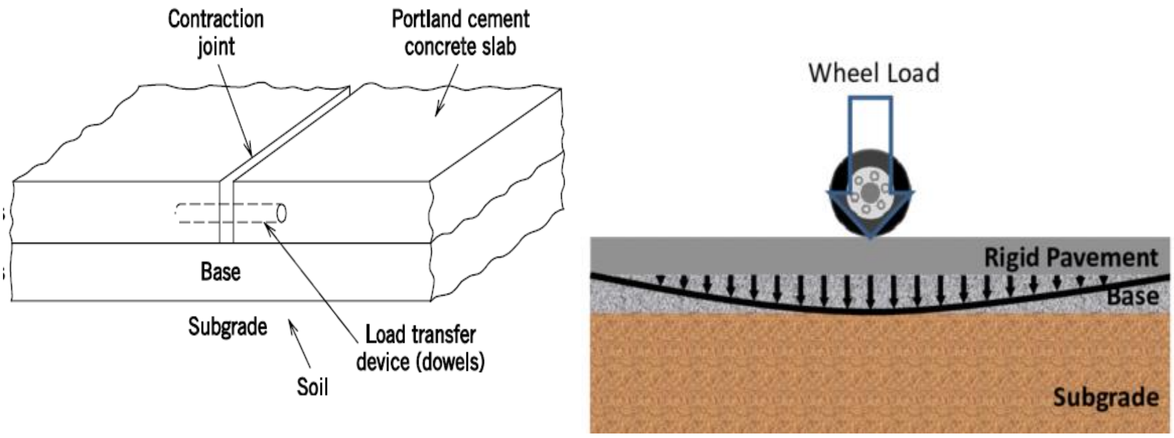


Figure 3: Shows the layers and load distribution of rigid pavements

Table 1: Indicates the comparison between Flexible and Rigid Pavements

Aspect	**Flexible Pavement**	**Rigid Pavement**
Primary Material	Bitumen and aggregates	Portland cement concrete
Structural Behavior	Distributes load gradually through multiple layers	Distributes load over a wide area due to high slab stiffness
Load Transfer Mechanism	Grain-to-grain contact between aggregate particles	Flexural action of the concrete slab
Initial Construction Cost	Relatively low	High
Maintenance Requirement	Frequent but easy and low-cost	Infrequent but costly and complex
Design Life	Typically 10–20 years	30 years or more
Resistance to Deformation (Rutting)	Moderate to low (temperature-sensitive)	High (minimal rutting)
Response to Temperature	Expands/softens with heat	Cracking possible due to thermal gradients
Typical Failures	Rutting, fatigue cracking, potholes	Cracking, joint failure, faulting
Suitability	Preferred for medium-traffic roads, easier rehabilitation	Ideal for heavy traffic corridors and industrial areas

Source: Adapted from Huang (2012) and Khanna & Justo (2011

2.4 Rutting in asphalt pavements

Rutting is also among the most severe cases of distress found in flexible pavements. It is manifested as long depressions along the tracks of the wheel, frequently with slight heaving on the edge of the rut. Rutting has a direct impact on eroding the ride quality, having a high risk of hydroplaning and hastens the overall Bad Road Network deterioration (Roberts et al., 1996). Rutting is also a problem in high traffic areas with environments that have a warm climate such as most of Africa where pavement performance and maintenance becomes a major problem.

2.4.1 Mechanism of Rutting

Asphalt pavements cause rutting due to the cumulative process of the permanent deformation which is caused by constant traffic loads. The asphalt binder becomes soft at high temperatures and becomes not rigid, and therefore the aggregates reorganize and move, creating shear flow to cause the instability of surface in the wheel paths. At initial stages of traffic densification of loosely compacted mixtures can be experienced as the air voids get minimized due to under loading though with time the plastic movement takes charge. Truck overloading and a large proportion of traffic contribute to the aggravation of the process (Zhang et al., 2019).

Beneath the asphalt layer rutting may also begin in the base soils and subgrade soils. Wet or damp foundations soften due to constant loads, which manifest themselves through depressions of the surface. Conditional influences also heighten rutting: elevated temperature of pavements raises the loss of binder viscosity, and moisture cracking decreases the bond between the aggregates and the binder, which results in less stability in the mixture. All in all, rutting is a complex combination of shear deformation of asphalt, compaction of mixture, and consolidation of the foundation, when subjected to accumulating traffic and environment loads (Rahman et al., 2020).

Table 2: Displays the Types of Rutting in Asphalt Pavements

Type	**Where it originates**	**Main causes**	**Typical diagnostic signs**
Mix rutting (surface rutting)	Surface asphalt layer	Inadequate mix stability, poor aggregate interlock, low binder stiffness at high temperature, poor compaction	Wheel-path depressions while subgrade remains sound
Structural / subgrade rutting	Subgrade and base layers	Weak or poorly compacted subgrade, high traffic loads, poor drainage	Rut depth through layers, whole-lane settlement, shoulder differential
Densification rutting	Layer consolidation (often early life)	Incomplete compaction during construction, post-construction consolidation under load	Progressive rutting that stabilizes after layer densification

2.4.2 Causes of Rutting

There are a number of reasons why rutting occurs in flexible pavement, the reasons may be classified under three categories, which are:

1. Material-Related Factors

Asphalt Binder Properties: A high temperature binder is prone to softening in hot conditions and its shear resistance capacity towards shear stress is limited (Read and Whiteoak, 2003).

Aggregate Gradation and Shape: Aggregate is rounded and thus decreases interlock and stability and poor gradation causes either voids or less stone-on-stone contact both of which contribute to rutting.

Filler and Additives: When there is lack of fillers or weak fillers, they lower the stiffness of asphalt mixture.

2. Structural and Construction Factors

Lack of Thickness: When the layers of the asphalt are very thin they are likely to deform very fast.

Poor Compaction: Mixtures that are not compacted well have excessive air cavities and thus they are prone to densification during traffic loading.

Weak Base/Subgrade: In the event that the layers of support are weak, the vertical deformation may be reflected at the surface in the form of ruts (Kadiyali & Lal, 2016).

3. Traffic and Environmental Factors

Giant Axle weights: Heavy Trucks create stress above capability of the pavement.

High Traffic Repetition: Despite the loads not going beyond the design limits millions of repetitions can slowly creep up and strain the plastic.

Temperature: Higher pavement temperatures will decrease the binder and increase the rutting (Khanna & Justo, 2011).

Effects of Rutting

Rutting has a number of implications to the road performance and user safety:

Drainage Problems: Water may collect in wheel paths and this poses a greater chance of hydroplaning during the rainy seasons.

Accelerated Degradation: Rutting normally rapid in nature and therefore cracks, potholes and collapse are experienced.

Increased Maintenance Costs: Earlier rutting has to be treated by regular resurfacing or using overlays thus increasing life-cycle costs (Huang, 2012)

2.4.3 Assessment of Rutting

The rutting may be evaluated both on the field and laboratory. Measurements Field assessment is done by a straight-edge or visual examination or automated profilometers of rut depth. Laboratory tests are the wheel tracking test, dynamic creep test and Hamburg wheel tracking device of which all test mimic repeated loading in order to determine the rutting resistance of a particular asphalt mixture (Brown and Cross, 1992).

2.4.4 Need for Rutting Resistance Improvement

Rutting resistance is another important design criterion of the present-day asphalt pavements, given its influence on the performance, safety, and costs. An improvement in rutting resistance needs a synergistic mix design, material adjustment, as well as, high quality construction. The performance has been enhanced by traditional techniques, including the addition of polymers or fillers, but current studies are in nanomaterial including montmorillonite nano clay as a more efficient and sustainable way to offer more effective options.

2.5 Asphalt Concrete Mix Design

Asphalt concrete mix design entails the choice and proportioning of aggregate, bitumen, filler and additives to come up with a mixture that matches the preferred performance specifications on the pavement construction. This will be aimed at optimal stability, durability, workability and economy. An effective asphalt mix should be resistant to typical distresses of pavement like rutting, fatigue cracking and moisture damage, as well as offering sufficient flexibility to meet traffic and environmental loads (Roberts et al., 1996; Zhang et al., 2018).

2.5.1 Conventional Asphalt concrete composition

The traditional asphalt concrete is a composite material which consists of aggregates, bitumen, and mineral filler in the proper ratio to obtain strength, durability, and flexibility. Aggregates make up a majority of the mixture which gives load-bearing skeleton and interlocking resistance. The aggregates are covered and bonded by the bitumen binder giving them adhesion and waterproofing. Mineral fillers fill the micro-voids and increase the interaction of the binder and the aggregates thereby, increasing the stability of the mixture. The combination of these three elements identifies the behavior of asphalt pavements in terms of traffic and the environment

2.5.2 Traditional Modifiers for Asphalt Concrete

Various modifiers have been introduced to enhance the mechanical and functional performance of the asphalt concrete by adding them to the conventional mixtures. Asphalt modifiers serve to reduce the material in the bitumen binder to attract certain qualities of the bitumen regarding its stiffness, performance with regard to elasticity and resistance to aging and rutting under the conditions of high temperatures and traffic loading (Whiteoak and Read, 2003). The modification process is intended to resolve shortcomings of pure bitumen which consist of the sensitivity to temperature, oxidation, and low shear resistance (Hassan et al., 2018).

The most widespread modifiers include polymers, rubber fiber and mineral fillers. Modification of polymer- especially with styrene-butadiene-styrene (SBS) and ethylene-vinyl-acetate (EVA)- has been extensively utilized as an enhancement of the polymer elasticity, and minimization of permanent deformation up to high service temperatures (Airey, 2004). SBS is a thermo plastic elastomer that improves the viscoelastic action of the binder, which allows it to pop better following a repeated traffic loading. Instead, EVA enhances thermal stability and deformation capacity but can elevate low-temperature rigidity, which can result in possible cracking (Liu and Zhang, 2019).

Another possible strategy that is being pursued as an environmentally friendly solution is rubber modification whereby waste tire crumb rubber is used. The modified crumb rubber damaged (CRM) asphalt portrays the better rutting and fatigue durability and contributes to waste recycling efforts (Lo Presti, 2013). Nonetheless, CRM binders demand increased mixing and compaction temperatures that may increase the cost of production and energy usage (Wang et al., 2018).

Cellulose, polyester, glass, and other types of fibers are employed as well to enhance the tensile and shear strength of asphalt mixtures, which involves reinforcement in the asphalt mixture (Shu & Huang, 2014). Similarly, mineral fillers such as lime, cement, and fly ash enhance the adhesion of binders and water resistance, but can exert minimal influence on permanent (deformation) (Behnood and Olek, 2017).

Notwithstanding these developments, a large majority of the traditional modifiers have the issue of cost, bitumen compatibility, and stability. An example is the polymer modification whereby there is strict control of temperature and blending machines to enable a homogeneous dispersion in the binder (Airey, 2004). These constraints have promoted the investigation of nano materials, which can boost their performance at a significantly reduced dose, because they have very large surface area and reactivity. Italianation of nanoclay modifiers, especially montmorillonite (MMT) has hence offered a viable alternative of replacing the conventional additives in order to enhance rutting resistance and durability in asphalt pavements (Kumar & Gope, 2012).

2.6 Nano Clay Technology

The development of nanotechnology has led to emergence of new generation of modifiers that are targeted to improve the performance of asphalt binders. Nanoclay is one of them and it has aroused great interest because of its layered nature and high surface area that makes it enhance mechanical, thermal and rheological characteristics of bituminous materials (Kumar & Gope, 2012). Phyllosilicate group of minerals are nanoclays, which are mainly made up of hydrated aluminum silicates structurally organized into layers of nanoscale. Montmorillonite (MMT) is the type of silicate mineral that is the most widely used in pavement modification that is a 2:1

layered mineral made of one alumina octahedral sheet encased between two silica tetrahedral sheets (Park et al., 2015).

Montmorillonite nanoclay when well dispersed in bitumen stiffening, less temperature-dependent and high rutting resistance are improved. The elevated aspect ratio of clay platelets creates a barrier network of the binder matrix and limits the mobility of the molecules and reduces the deformation under traffic loads (Chen et al., 2011). Additionally, the stratified form enhances good interfacial interaction with bitumen molecules, resulting in high load distributions and shearing forces at high temperatures (Abdullah, Aziz, and Ansari, 2022).

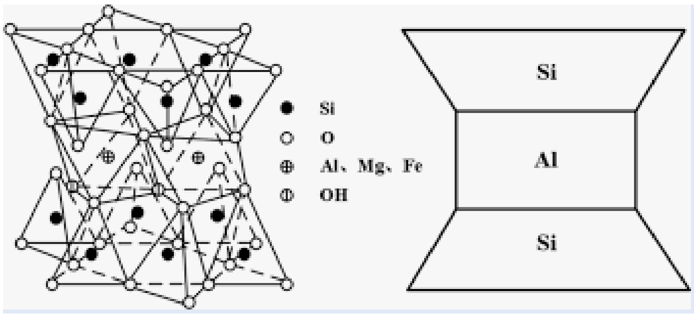


Figure 4: Shows the crystal structure of Montmorillonite clay

2.6.1 Organo-Philic Montmorillonite (OMMT) Modification

When raw montmorillonite is used, the compatibility because of the hydrophilic properties of that compound frequently becomes a constraint with the hydrophobic bitumen. To address this drawback, the montmorillonite may be functionalized by cation exchange reaction where inorganic interlayer cations are substituted with organic ones where (Na^+ or Ca^{2+}) can be substituted with quaternary ammonium compounds. The result is an organo-philic montmorillonite (OMMT) which is highly affinity with organic binders (Yao et al., 2012). The change enhances the distribution of the clay particles in the bitumen that create intercalated or exfoliated nanostructures depending on the extent of separation of the silicate layers (Zhang et al., 2019).

The features of the OMMT-modified binders are that they tend to have better viscosity, stiffness, and resistance to permanent deformation when high temperature and heavy traffic are concerned. The organically modified clay/bitumen molecules interplay and strengthen the colloidal structure of the binder to restrict its flow and increase the recoverability upon loading (Rahman et al., 2020). Besides this, the thermal stability of OMMT-modified binders is improved and, therefore, the performance remains constant over a wide temperature range, something that is especially useful in topics such as the tropical climate where pavements have to survive heavy surface temperatures.

Figure 5: Shows OMMT Nano clay powder

CHAPTER THREE: METHODOLOGY

3.1 Introduction

This chapter describes materials, standards of experiments, and testing protocols that were used in the study. These tests and methods were done to assess the engineering properties of the aggregates, bitumen and organo-philic montmorillonite (OMMT) Nano clay that were utilised in the study.

3.2 Characterization of Material Properties

3.2.1 Material Acquisition

Every material in this work was acquired at Stirling Engineering Limited, which is a licensed distributor of road construction material. Materials that were sourced were coarse and fine aggregates, bitumen and organo-philic montmorillonite (OMMT) Nano clay. The materials were also chosen according to their suitability to meet the standard requirements of asphalt concrete production (ASTM, 2017; BS 812, 1990).

a) Aggregates

The aggregates, which were sought in this research, were gathered in Stirling Construction Company Limited quarry in Mbalala-Mukono. The sampling was also done randomly across the aggregate stockpiles to get representative blend of different size properties such as 0-6 mm, 6-10 mm, 10-14 mm, and 14-20 mm. Aggregates, both coarse and fine, were examined visually to determine that they were clean, angular, and had no deleterious materials in them like clay, organic matter or dust until testing.

The aggregates were sieved according to BS 812: Part 103 (1990) categorization as sieved coarse and fine aggregate with regards to Hong Kong sieved aggregate, where the fraction that ascertained on 4.75 mm sieve and the fraction that ascertained to the sieved fractions included the coarse and fine fraction respectively. To determine the suitability of the aggregates with regards to asphalt concrete production, various laboratory tests were carried out on the strength, gradation of the aggregates and their overall suitability.

b) Bitumen

The grade of bitumen employed in this research was the penetration 60/70 grade; this is the wide spread grade of bitumen that is normally employed in road construction in Uganda. It was sourced out of Stirling Construction Company Limited which was the supplier of the aggregates. The bitumen was first visualized to improve it to be free of any impurities, lumps, and any evidence of age and contamination before testing.

The entire bitumen tests were made in relation to specific ASTM standards which ensure that it is viable to use in asphalt concrete. Tests done here were penetration (ASTM D5), softening point (ASTM D36), specific gravity (ASTM D70), and viscosity (ASTM D2170). The findings of these tests were then compared with the normal specification limits to establish that the binder was within the specification of producing a hot mix asphalt.

c) Organo-philic Montmorillonite (OMMT)

Organo-philic Montmorillonite (OMMT) was purchased in a reputable supplier of chemical compounds and kept in dry environment to avoid the absorption of moisture. The OMMT was visually checked before it was used in terms of uniformity, color, and lack of lumps. This was followed by drying of the material at a low controlled temperature to eliminate moisture remaining in it and sieving to gain the required size of particles. The ready OMMT was then utilized in the alteration of bitumen in different proportion to determine its influence on the mechanical and rutting characteristics of the asphalt concrete.

3.2.2 Laboratory Tests on Materials Used

All materials were tested in the laboratory to identify their physical, chemical, and mechanical characteristics. These tests were necessary in determining the viability of aggregates, bitumen, and OMMT as part of asphalt concrete. The processes incorporated the standard testing procedures to ascertain accuracy and reliability of findings.

3.2.2.1 Tests on Aggregates

a) Grading/Sieve Analysis

Gradation or sieve analysis test was conducted to ascertain the particle size distribution of the aggregates that are employed in the asphalt concrete. This fact was critical to make sure that the aggregates used reached the necessary requirements in terms of compaction, stability, and the durability of the altered asphalt mixture. The correct gradation also reduces the voids as well as enhances binding of bitumen and aggregates that have a direct effect on the rutting resistance.

The test was conducted on the basis of BS 812-P103-1 and BS 1377: Part 2: Clause 9:1990 wherein the aggregates were sieved using standard sieves and the mass that remained on the sieve was noted. The distribution data was then applied in coming up with an ideal combination of the aggregate that would be adopted in the modified asphalt concrete.

b) Specific Gravity

Specific gravity test was done with the aim of identifying the density of the aggregates in comparison with the density of water. This property is significant in the mix design, because it affects the volume calculation, content of void and stability of the asphalt concrete. Proper proportions of the materials need to be achieved with regard to desirable performance under inherent characteristics in the specific aggregate such as rutting resistance by possessing accurate knowledge of aggregate specific gravity.

This test was done according to BS 812: Part 2: 1975 guidelines whereby aggregates were dried in the oven and their mass in air and mass in water were taken. Specific gravity (bulk and apparent) was then determined using the results and it used to determine the mix design and assessment of the modified asphalt concrete.

c) Water Absorption

The water absorption test was conducted in order to know the water amount which the aggregates can take in under the normal conditions. The property is important in the determination of the porosity of aggregates, which influences the content of voids, the binder requirement, and the asphalt concrete durability. High water absorbing aggregates might need some modifications of the mix design in a bid to ensure adequate bonding with bitumen and to minimize premature rutting or stripping.

The test was conducted as per BS 812: Part 2: 1975 wherein the aggregates were dried in the oven and then placed in water under a given time and increment in weight was measured. It was followed by adjusting mix proportions and optimization of the performance of modified asphalt concrete using the results.

d) Los Angeles Abrasion Test (LAA)

Los Angeles Abrasion test was also carried out to identify the wear and mechanical degradation resistance of the aggregates. The property is essential in determining the durability and determining the performance of the asphalt concrete during the long run with respect to traffic loading. Low-abrasive aggregates are likely to result in premature raveling and poor rutting.

This was done as stated in BS 812-110: 1990, where the aggregates were put in a rotating drum using steel balls and the mass of the degraded stated. This was in the form of percentage loss which guided the choice of aggregates that will be used in the modified asphalt mix.

e) Ten Percent Fines Value (TFV)

In order to identify the load needed to generate 10 percent fines in aggregate samples under dry and wet (soaked) conditions the Ten Percent Fines Value (TFV) test was performed. The comparison measures the asphalt resistance to crushing when subjected to absolute moisture that is important in the performance of the asphalt in the different weather conditions. The test was conducted as BS 812-111: 1990, when aggregates were placed in compressive loading machine and loads applied on the aggregate until the production of 10 percent of aggregate. Both the TFV (Dry) and the TFV (Wet) were conducted in order to test the strength as well as moisture susceptibility of the aggregates employed in the amended asphalt blend.

f) Aggregate Crushing Value (ACV)

Aggregate crushing value test was conducted to establish the compressive strength of aggregates to compressive loads. The strength of the aggregates in withstanding traffic stresses in asphalt concrete is determined by this property. Reduced ACV values represent improved aggregates having superior rutting resistance.

It was conducted under the BS 812-110: 1990 where a compression machine was used to apply a load steadily on a cylindrical aggregate sample. The level of crushed material was determined and employed in determining the fitness of the aggregates in the modified asphalt mix.

g) Aggregate Impact Value (AIV)

Aggregate Impact Value test was done to determine the toughness of aggregates and their resistance to immediate shocks or impact. The significance of this property is in the fact that it guarantees the ability of the aggregates to survive dynamic loading in asphalt pavements.

The test was done in accordance with BS 812-112: 1990 in which aggregates were poured into a cylindrical mold and then crushed together by a series of controlled blows. The fines that resulted were performed as a percentage of the original mass to determine the resistance of the aggregates to impact.

h) Flakiness Index Test

Flakiness index test was carried out to establish the percentage of the flaky particles in the coarse aggregate. Flaky particles are very easy to crack under strain and may cause the asphalt structure to make it weaker giving it less stability and life-span.

The test was conducted on BS 812: Part 105.1: 1989 in a manner that a thickness gauge was used to separate flat sized particles form the sample. The flakiness index was given as a percentage of the total weight, contributing to the provision of control over the flakiness of the used aggregates being in good shape and well-suited to be used in the production of asphalt.

3.2.2.2Tests on Bitumen

1) Softening Point Test (ASTM D36-70)

The Ring and Ball method of testing bitumen as outlined in ASTM D36-70 was used in order to ascertain the softening point of the bitumen used. This experiment was performed in assessing the temperature at which the bitumen would become soft enough to allow a steel ball to fall through under regulated heating. The outcome showed that the binder was affected by temperature and capability of withstanding deformation in high temperatures of a pavement. A greater softening point meant that the bitumen could be stable and rutting or flowing at high temperature conditions that were encountered in road conditions.

2) Penetration Test (ASTM D5-97)

The penetration test was conducted as per the ASTM D5-97 in order to check the consistency and the hardness of the bitumen. The procedure used in this test was, a conventional needle was left to pierce the sample of bitumen when subjected to a load of 100 g in 5 seconds at an ambient temperature of 25 o C and the extent of penetration recorded in tenths of millimeters. The penetration value was used to identify the softness or hardness of the binder used relative. Lower penetration values made bitumen more difficult and fit to hot climatic conditions whereas higher penetration values had softer grades that were more applicable in the colder regions.

3) Viscosity Test (ASTM D2170-95)

The capillary viscometer was used in determining the viscosity of the bitumen according to the ASTM D2170-95. This test was done on the time required by a given amount of bitumen to pass through a calibrated glass capillary under gravity at a well-determined temperature. The value of viscosity derived indicated how the binder was resistant to flow and what could have been done with it during mixing and compaction. An increase in viscosity meant that the binder was stiffer and that it could afford a more ideal film thickness and a rutting resistance and a high viscous index would mean that it was undesirable in workability in asphalt production.

4) Specific Gravity Test (ASTM D70-18)

The specific gravity test was done using ASTM D70-18 to find the density of the bitumen in relation to the density of water. The sample was heated up until in the fluid state and was poured in a clean pycnometer that was weighed both before and after filling. The specific gravity was expressed based on the ratio of the mass of the bitumen that the same mass of water at 25 C had. This property was necessary to determine the relationships between weight and volume of the mixture during mix design, as well as the purity and consistency of binder

3.2.2.3 Tests on Organo-philic Montmorillonite

a) X-Ray Diffraction (XRD) Test

This was done by the use of X-Ray Diffraction (XRD) to identify the basal spacing and crystalline structure of the Organo-philic Montmorillonite (OMMT). Cu Ka (l= 1.5406 A) was used to measure the value with the diffractogram obtained at a 2th range of 10^0-80^0.

The observed results recorded that the raw MMT had a sharp basal reflection at 2th = 27.745^0 (which corresponds to interlayer spacing, d001, = 3.21 A). The OMMT after organo-modification had a distinct peak characteristic at 2th = 20.851deg, and the ratio of the d-spacing was higher with a diameter of 4.26 A. It was also found that in the modified sample data an extra sharp reflection is also present at = 2th = 26.632deg (d = 3.34 A). This change in the value of the basal reflection at 2^{nd} to a lower value (and the resulting increase in d-spacing) served to confirm that the organic surfactant molecules had been successfully intercalated into the clay galleries, so as to increase the spacing between the layers.

The above increment in the values of d-spacing meant that the organo-modification promoted the compatibility and dispersion of the clay in the bituminous structure. This enhanced interaction achieved enhanced exfoliation and uniform distribution of Nano clay layers in asphalt binder which led to improved rutting resistance and thermal stability of the modified asphalt concrete.

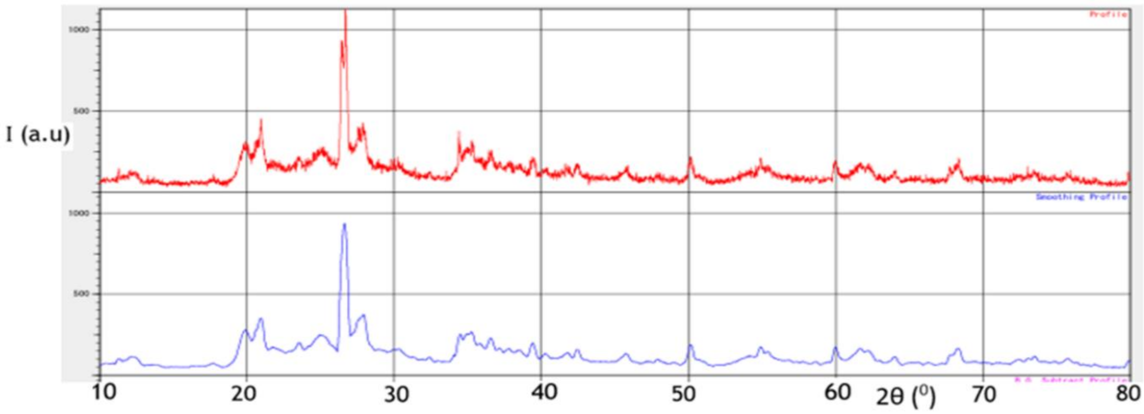


Figure 6: Shows the X-RD graph of OMMT

Group : Standard
Data : CLAY

Strongest 3 peaks

no.	peak no.	2Theta (deg)	d (A)	I/I1	FWHM (deg)	Intensity (Counts)	Integrated Int (Counts)
1	10	26.6318	3.34448	100	0.54290	320	8474
2	4	20.8513	4.25675	31	0.75430	99	3212
3	11	27.7451	3.21276	30	0.88670	96	4130

Figure 7: Shows the strongest peaks of the x-rd. graph

3.3 Determination of the Optimum OMMT Nano clay Content Used in the Mix

A series of trial mixes were done on different amounts of OMMT in order to identify the best percentage of the organo-philic montmorillonite (OMMT) nano clay to be incorporated in the asphalt. The selection adhered to the basic principles of the Marshall mix design approach, which had the performance parameters based on the stability, flow, bulk density, maximum theoretical density, and the volumetric properties of air voids, VMA, and VFB. Four OMMT doses of 0, 0.5, 1, and 1.5 percent in total mix weight were also conducted with an aim of determining the optimum content that gave maximum resisting to rutting and at the same time has no effects on mix workability.

3.3.1 Preparation of Modified Binder

To ensure sufficient fluidity to allow modification, 392 g of 60/70 penetration grade bitumen (which equates to 4.9 percent of the total 8, 000g aggregate) was gently heated to achieve a temperature of 150degC or 160degC to obtain the desired fluidity. Keeping this temperature range, the proportions of OMMT (0.5%, 1.0%, and 1.5%) were in turn added to the hot binder. A thorough stirring was done on the mixture over a period of about 25 to 35 minutes to ensure a homogeneous distribution of the particles and to reduce the agglomeration of the particles.

The strong necessity to maintain the temperature was crucial to the mixing to ensure the proper interaction of the bitumen and nano clay without thermal degradation and separation.

3.3.2 Mixing of Modified Binder and Aggregates

The modified binder was then mixed with 7,608 g of heated aggregates, to be kept at 135^0C up to 155^0C. Mixing was done manually in order to have a uniform distribution of all the aggregate particles. The mixture of the two was held at approximately 150-160^0C and stirred and stirred until the binder had been distributed uniformly. The same was done to all the four trial mixtures (0%, 0.5%, 1.0%, and 1.5% OMMT), each containing 8,000g.

3.3.3 Sample Preparation and Compaction

Out of every mix, 1,000 g of the mixture was sampled and sieved to obtain coarse and fine mixtures to test Maximum Theoretical Specific Gravity (Gmm). Besides, four samples each with an OMMT dosage (1200 g) were sampled and compacted using an automatic Marshall compactor. Compaction was done at temperatures of 160^0C and 75 blows were used on every side of the specimen to get target density, a total of 16 Marshall Specimens. The shrunken specimens were given room temperature to allow 24 hours of drying and then the specimens were further assessed.

3.3.4 Testing and Evaluation

The specimens were then conditioned and tested with the Marshall Stability and flow test that was carried out according to the relevant standards of ASTM/ AASHTO. The saturated surface dry method was used to determine the Bulk Specific Gravity (Gmb) and the volumetric properties Air Voids (Va), Voids in Mineral Aggregate (VMA), and Voids Filled with Bitumen (VFB), were calculated based on the acquired Gmb and Gmm values.

A comparison of the results at all the trial percentages made it possible to evaluate the role played by OMMT on the level of stiffness of the mixture, the resistance to deformation and the compaction nature. According to the Marshall approach to be adopted in establishing optimum binder/additive content, the end result was designed at a compromise based on good stability, reasonable flow characteristics, and satisfactory volumetric characteristics.

3.3.4 Selection of Optimum OMMT Content

The mechanical and volumetric results were analysed showing that the mix with 1.0 percent of OMMT nano clay offered the most desirable set of performance parameters. It was more stabilized and more densified than the other mixes with an acceptable flow and volumetric properties within specification parameters. As such, OMMT of 1.0 percent was chosen the ideal level of modification.

Chapter Four under Results and Discussion contains a detailed assessment of the findings of each trial mix and discussion.

Figure 8: Shows an asphalt sample Marshalls

Figure 9: Shows compacted asphalt

3.4 Determination of mechanical properties of the modified asphalt concrete

After calculating the optimum content OMMT (1.0% by total mass of mix), a new batch of asphalt mix was made with mechanical performance analysis purpose. The chosen mix was prepared with the same aggregate gradation, binder content and mixing conditions used in the trial mix phase so as to have consistency in evaluation.

3.4.1 Sample Preparation

The total mass of asphalt concrete 8,000 g of 1.0% OMMT nano clay was prepared. A modification was carried out on the binder (392 g of bitumen), by mixing it with OMMT at a temperature of between 150^0C and 160^0C, continually stirred to create fine dispersion. The modified binder was then added to 7,608 g pre-heated aggregates kept at temperatures ranging between 135^0C and 155^0C and mixing process was continued at high temperature until complete coating was realized. Out of this joint mixture, six Marshall Specimens were made each of which had a mass of some 1,200 g. Standard practice was then followed, with compaction of the specimens being done using an automatic Marshall compactor and giving 75 blows on each face of the specimen at a temperature of 160^0C.

The specimens were then left to cool at room temperature over a 24 hour period then mechanically tested.

3.4.2 Indirect Tensile Strength (ITS) Testing

The Indirect Tensile Strength (ITS) test was carried out to examine the tensile stresses resistance of the asphalt mix that correlates directly to durability of the material and the possibility of it being able to endure tensile rutting and moisture related damage. The testing was conducted as some of the procedures adhered to by the Central Materials Laboratory (CML), Ministry of works and Transport, Uganda, generally calculates to the AASHTO T283 on moisture susceptibility testing.

The six specimens were separated three, where:

- Three samples were experimented in the dry state, and
- Three of them were tested with the conditioning of moisture (wet test).

Moisture Conditioning (Wet ITS Specimens)

To test the wet conditions, specimens were smeared according CML conditioning guidelines:

- The samples were vacuum saturated partially to about 70 -80 %. Saturation.
- These were then placed in a water bath of 60degC temperature in 24 hours which mimicked unpleasant conditions of moisture action.
- The specimens after conditioning were cooled in 25 +- 1degC water bath during 2 hours and then tested.

Dry Condition Testing

The three dry specimens were left at the ambient laboratory temperature then exposed to a water bath at 25^0C of a 2 hour period before testing in order to make the samples of the same temperature.

3.4.3 Testing Procedure

The ITS test was conducted in the form of inserting all cylindrical specimen vertically between loading platens of the testing machine. The rate of deformation was kept constant at 50 mm/min until the specimen failure. It was measured and the maximum load at failure was utilized to get the indirect tensile strength using the formula;

ITS = (2P) / (p x D x h) Where:

P = maximum load at failure (N),

D = specimen diameter (mm),

h = specimen thickness (mm).

Tensile Strength Ratio (TSR) was also calculated in order to measure moisture vulnerability.

TSR = (ITS wet / ITS dry) x 100

An increased TSR means it is less exposed to moisture damage and is much performing in the long-term. In CML instructions, an acceptable minimum of 80% of TSR is usually regarded as widely accepted on asphalt mixtures to be used on high-volume roads.

3.4.4 Purpose of the Mechanical Evaluation

The ITS test was performed to evaluate the intra compaction cohesion and tensile value of modified asphalt mixture, and they are both factors that contribute to the shear deformation resistance. Whereas rutting takes place under compressive loads, mixtures that have greater values of ITS are more capable of withholding sideward movement of substances in wheel tracks later. Tensile strength is also high, which means that there is better bonding of binder-aggregate, and structural stability when traffic loads it. Both dry and wet testing also facilitate the assessment of moisture susceptibility because a weakened cohesion results in a rapid rutting. Thus, ITS offers the giving evidence of the capability of the mix to resist the deformation under the influence of the rut. An increase in ITS means that the rutting resistance of the OMMT-modified asphalt concrete is high.

CHAPTER FOUR: RESULTS AND DISCUSSION

4.1 Tests on Aggregates

These tests evaluate properties such as hardness, toughness, shape, and resistance to weathering. Their results ensure that aggregates can withstand traffic loads and environmental conditions without degrading.

4.1.1 Gradation of Aggregates

The gradation analysis procedure was conducted to assess whether the cumulative aggregates are within the suggested quantity of particles size and distribution to produce asphalt concrete. Based on the findings, the final blended gradation is close to the target grading envelope, meaning that it is a balanced blend of coarse and fine aggregates, as well as mineral filler.

The percentage passing at the larger sieve sizes (20 mm and 14 mm) fall within specification range that indicated that the coarse aggregates were not too large or not in the proper amount. The gradation at the intermediate sieve size like 10 mm, 5 mm and 2.36 mm was stable to the target curve that showed a smooth progression in particle sizes. This continuity is of significance since it provides the right interlocking of the aggregate particles, which leads to the strength and stability of the asphalt mixture.

As far as the smaller fractions were concerned, 0.6 mm, 0.3 mm and 0.075 mm sieve sizes, the retained material remained within the recommended ranges. The concentration of fines was not higher than the maximum value, which implies that the mixture should not experience a problem with an overload of binders, the inability to retain stability, or the inability to resist humidity destruction. Meanwhile there were abundant fines to fill spaces and encourage the well-companied network.

Table 3: Displays gradation results of aggregates

Sieve Size	14/20	10/14	6/10	0/6	FILLER	Theoretical	TARGET GRADING	BRITISH STANDARDS
28	100	100	100	100	100	100	100	100
20	98.6	99.4	100	100	100	100	90	80-100
14	18.7	70.3	99.4	100	100	76	70	60-80
10	1.6	18.7	93.9	100	100	65	60	50-70
5	0.4	1.3	6.6	83.0	100	47	46	36-56
2.36	0.4	0.9	5.1	53.7	100	31	36	28-44
1.18	0.4	0.8	4.3	37.4	100	22	27	20-34
0.6	0.4	0.7	3.6	27.3	99.8	17	21	15-27
0.3	0.4	0.6	3.0	19.1	95.8	12	15	10-20
0.15	0.4	0.5	2.5	12.0	79.7	8	9	5-13
0.075	0.4	0.4	2.2	6.5	58.4	5	4	2-6

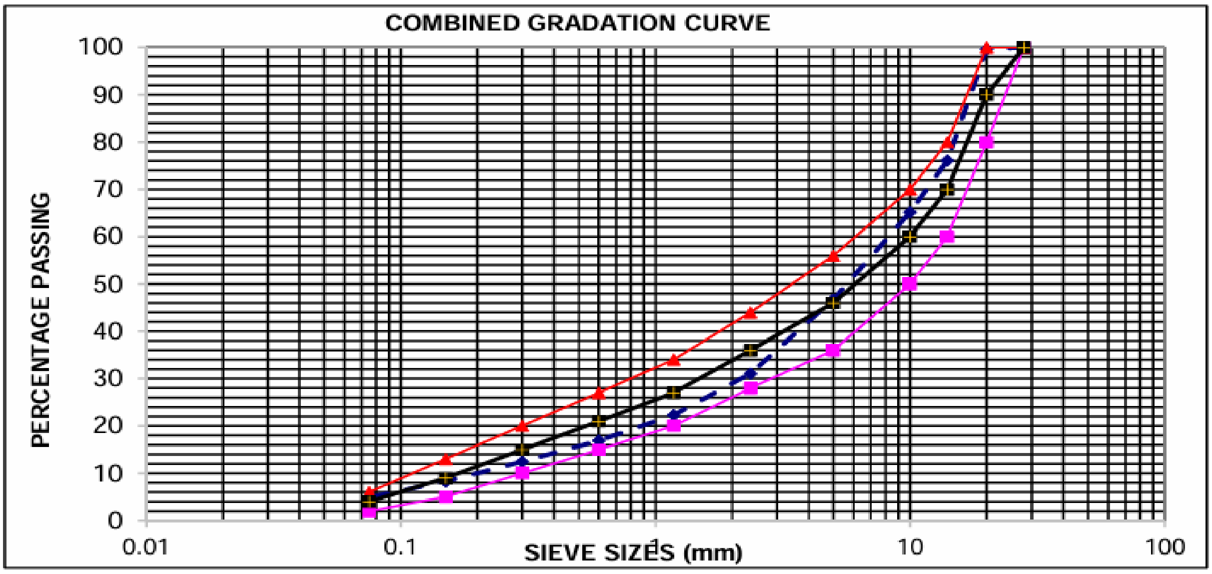


Figure 10: Shows the aggregate gradation curve.

4.1.2 Specific Gravity of aggregates

Specific gravity test was done to define the density and quality of coarse aggregates which were used in the mix of asphalt concrete. Based on the acquired results, the bulk specific gravity of coarse aggregates was 2.621, whereas the apparent specific gravity was 2.638. Bulk specific gravity value in saturated surface dry (SSD) basis, and water absorption were 2.627 and 0.2% respectively.

These values are in the acceptable range of 2.5-3.0 that is advised of the good aggregates that can be used in the production of asphalt concrete (Asphalt Institute, 2014). Specific gravity gives an idea of aggregate strength and porosity whereby high values tend to depict a stronger and denser material (Huang, 2012). The minor variation of bulk specific gravity and the apparent specific gravity signifies that the aggregates have a compact structure and have few internal pores that increase their crushing and wear resistance against traffic loading.

The very low value of water absorption which is 0.2% indicates that the aggregates are rather non-porous and unlikely to absorb moisture. It is stated that in BS 812: Part 2 (1995), aggregates whose water absorption values do not exceed 1% are regarded as high quality because of the cases of weak bonding when subjected to water conditions (Roberts et al., 1996). Therefore, aggregates in Mukono Quarry have good features of durability and resistance to moisture, which can be useful when attaining stability of pavement in the long-run.

Table 4: Displays the Summary of results of the specific gravity of different aggregate sizes

Aggregate size	20-14mm	14-10mm	10-6mm	6-0mm	Filler
Bulk SG	2.621	2.632	2.633	2.632	2.653
Combined SG	2.634				
Water Absorption (%)	0.2	0.4	0.5	0.6	
Combined (%)	0.4				
Standard (BS 812-2: 1995)	<2%				

4.1.3 Los Angeles Abrasion Test (LAA)

The Los Angeles Abrasion test was done to determine the resistance of aggregates to wear and mechanical degradation under the influence of traffic load. The findings indicated that the average abrasion level was 16.8% meaning it is below 40% which the ASTM C131 and BS 812-110 (1990) defines as a maximum limit of aggregates in asphalt concrete.

The value of abrasion is low, which verifies that the aggregates are tough and with high resistance against a mechanical break-up and can therefore be used in the applications of heavy-duty pavement. These aggregates are not prone to disintegration during the mixing and compaction processes, and/or traumas of traffic, which is helpful to preserve the structural stability and long-term rutting resistance of the asphalt mixture (Huang, 2012; Kadiyali & Lal, 2016).

Speed of Rotation: 33 Rev/min

Max: 500 Revolutions

Max Duration: 15 minutes

Table 5: Displays a summary of LAA values

Parameter	Sample 1	Sample 2
Weight of material retained on 1 mm sieve	4,155.0	4,165.0
Weight after crushing	4,968.2	4,946.1
Weight of fine material (g)	845.0	835.0
% of wear	16.9%	16.7%
Average	16.8%	
Standard (BS 812-110 1990)	<40%	

4.1.4 Ten Percent Fines Value (TFV)

High 10% Fines Value is preferred since it is the ability of the aggregate to resist crushing forces. The resistance to crushing of these aggregates makes them add to a stronger and more durable asphalt mixture that is more responsive to traffic loading.

Table 6: Displays results for 10% fine value dry

TEST NO	**1**	**2**
CRUSHING FORCE (KN)	267	267
WT. OF AGG AFTER CRUSHING,M1 (g)	2746	2754.2
WT. OF AGG RETAINED ON 2.36mm SIEVE, M3 (g)	2466	2463
WT. OF AGG PASSING THROUGH 2.36mm SIEVE, M2(g)	293	291.3
TEN % FINE VALUE (M=M2/M3*100)	10.7	10.6
AVERAGE RESULTS	10.7	
AVERAGE CRUSHING FORCE (KN)	267	
TFV DRY (KN)	255.6	
STANDARD (BS 812-111: 1990)	>110KN	

Table 7: Displays results for 10% fine value soaked

TEST NO	**1**	**2**
CRUSHING FORCE (KN)	267	267
WT. OF AGG AFTER CRUSHING,M1 (g)	2792.2	2770.6
WT. OF AGG RETAINED ON 2.36mm SIEVE, M3 (g)	2443	2425
WT. OF AGG PASSING THROUGH 2.36mm SIEVE, M2(g)	349.2	345.6
TEN % FINE VALUE (M=M2/M3*100)	12.6	12.5
AVERAGE RESULTS	12.5	
AVERAGE CRUSHING FORCE (KN)	267	
TFV SOAKED (KN)	226.6	
STANDARD (BS 812-111: 1990)	>110KN	

Table 8: Displays the Wet/Dry ratio

Sample	Dry	Wet	Wet/Dry ratio
Force (KN)	255.6	226.6	89
Standard (BS 812-111)			>75%

The soaked and dry TFV strength is greater than 110 KN, which is the required aggregate strength for asphalt concrete surfacing. Moreover, the wet/dry strength ratio, depicted in Table 8, is greater than the required standard of 75%, which is defined by the British Standards. This further exhibits the strength retention ability of the aggregate, which can withstand crushing due to traffic loading
(MoWHC, 2005).

4.1.5 Aggregate Crushing Value (ACV)

The Aggregate Crushing Value provides an estimate of the aggregate's resistance to compressive forces. It reflects how well the material can endure crushing when subjected to a steadily increasing load. During the test, a prepared aggregate sample is loaded until it breaks down, after which the amount of fines produced is measured. The procedure follows the guidelines set out in BS 812 Part 111:1990.

Table 9: Displays the ACV test results

SAMPLE	1	2
WT. BEFORE CRUSHING (g)	2751.1	2753.2
WT. WT. AFTER CRUSHING (g)	2751	2752.4
WT. RETAINED AFTER CRUSHING (g)	2316	2318.9
WT. PASSING SIEVE 2.36mm	435.1	434.3
A.C.V (%) (D/B)*100	15.8	15.8
AVERAGE RESULTS %	15.8	
BRITISH STANDARDS	<30%	

4.1.6 Aggregate Impact Value (AIV)

The results of the Aggregate Impact Value demonstrate average losses of approximately 15.0% which is considerably below the maximum of 30% that the British Standards indicate should be used in aggregates in asphalt concrete. These low values of AIV means that the aggregates are tough and can be used to repel any sudden shocks or impacts forces that may happen during the loading of traffic. The minimal disparity between the two groups also indicates the recurrence of material quality. All in all, the aggregates have sufficient strength to support the dynamic loads in the layers of pavements, which makes them applicable in asphalt concrete surfacing.

Table 10: Displays the results of the test on AIV.

SAMPLE	1	2
WT. BEFORE TEST (g)	347.5	337.7
WT. WT. AFTER TEST (g)	344.5	337.5
WT. RETAINED AFTER TEST (g)	296.2	286.8
WT. PASSING SIEVE 2.36mm	51.3	50.9
A.I.V (%) (D/B)*100	14.9	15.1
AVERAGE RESULTS %	15.0	
BRITISH STANDARDS	<30%	

4.2 Tests on Bitumen

4.2.1 Penetration of Bitumen

According to the analysis by the penetration test, the penetration value of 60/ 70-bitumen employed in this experiment is 68 in average, which is within the scope of 60/70 (British Standards) of this grade of binder. The low standard deviation of the individual readings is an indicator of a consistent and consistent binder. A penetration value of almost the upper range of the specification implies that the binder is still working at 25^0C, and it's still moderately soft enough to work at during mixing and compaction, but is also cohesive enough to coat the aggregate. This is common with what is known as binders that are applied in tropical climates where one needs flexibility to meet the daily changing of temperatures (Read and Whiteoak, 2003).

Table 11: Displays Bitumen Penetration test results

TEST NO	1	2
PENETRATION (5 SECONDS) AT 25^{0}C	66 68 68	69 67 66
AVERAGE RESULTS (^{0}C)	68	
BRITISH STANDARDS	60-70	

4.2.2 Softening Point of Bitumen

The softening point test is conducted to establish the point at which the bitumen changes from a semi-solid to a more fluid substance when subjected to controlled temperatures. Ring 1 softened at a temperature of 53°C, Ring 2 softened at 54°C, and the average softening point is 53.5°C. This is well within the desirable range of 49-56°C, which is suited to 60/70 penetration-grade bitumen, as set by the British Standards (BS 2000, cited by Read and Whiteoak, 2003).

A high degree of consistency between the readings obtained by the two rings suggests good thermal homogeneity and consistency of the sample, which further implies that either little oxidation has occurred, or the bitumen has not been contaminated.

The value of softening point obtained indicates a binder with the ability to retain enough stiffness at high pavement temperatures. In hot and dry climates, a high softening point is preferable, which assists the asphalt pavement to prevent high temperatures on the road surface (Huang, 2012).

Table 12: Indicates the result of Bitumen softening point

RING 1				**RING 2**		
TIME (MINUTES)	**TEMP (^{0}C)**	**TEMP RISE (^{0}C)**		**TIME (MINUTES)**	**TEMP (^{0}C)**	**TEMP RISE(^{0}C)**
0	2	2		0	2	2
1	7	5		1	7	5
2	11	4		2	11	4
3	15	4		3	15	4
4	20	5		4	20	5
5	24	4		5	24	4
6	29	5		6	29	5
7	34	5		7	34	5
8	38	4		8	38	4
9	44	6		9	44	6
10	49	5		10	49	5
11	50	1		11	50	1
12	53	3		12	54	4
SOFTENING POINT	53			SOFTENING POINT	54	
AVERAGE VALUE	53.5					
RECOMMENDED (BRITISH STANDARDS)	(49-56)^{0}C					

4.2.3 Specific Gravity of Bitumen

The result obtained by conducting the specific gravity test gave a value of 1.025, which is well within the 1.01 to 1.06 range specified by the British Standards for paving-grade bitumen. This result is an indication that the binder has the right density, which is not too high nor too low when compared to water, and is, therefore, of the right purity and has the right refining properties. Specific gravity is relevant during the design of the asphalt mixture, as it has an effect on volumetrics, binder contents, and overall mixture stability (Huang, 2012). A specific gravity close to 1.02 is, therefore, an indication that the bitumen is not too diluted with light fractions and has not been too aged, which has an effect on its properties. Since the result has fallen well within the right limits of the specifications, it is, therefore, assumed to be safe and can therefore be used to produce dense and well-compact asphalt concrete. Moreover, it serves as a correct basis to assess changes which may take place when it is mixed with OMMT nanoclay, which often has an effect on the specific gravity and volumetrics of the mixture (Read & Whiteoak, 2003).

4.3 Tests on Asphalt Samples

4.3.1 Maximum Theoretical Specific Gravity (Gmm) and Bulk Specific Gravity (Gmb)

The maximum theoretical specific gravity (Gmm) test is used to measure the density of the asphalt mixture, ignoring the air voids, which is helpful to assess the compaction and volumetric. The specific gravity of the compacted sample (Gmb), on the other hand, is used to measure the density of the sample, including the volume of the air voids. These are important to measure the air voids, along with determining mixture durability and susceptibility to deformation due to traffic (Roberts, et al., 1996).

Table 13: Presents the Gmm and Gmb results

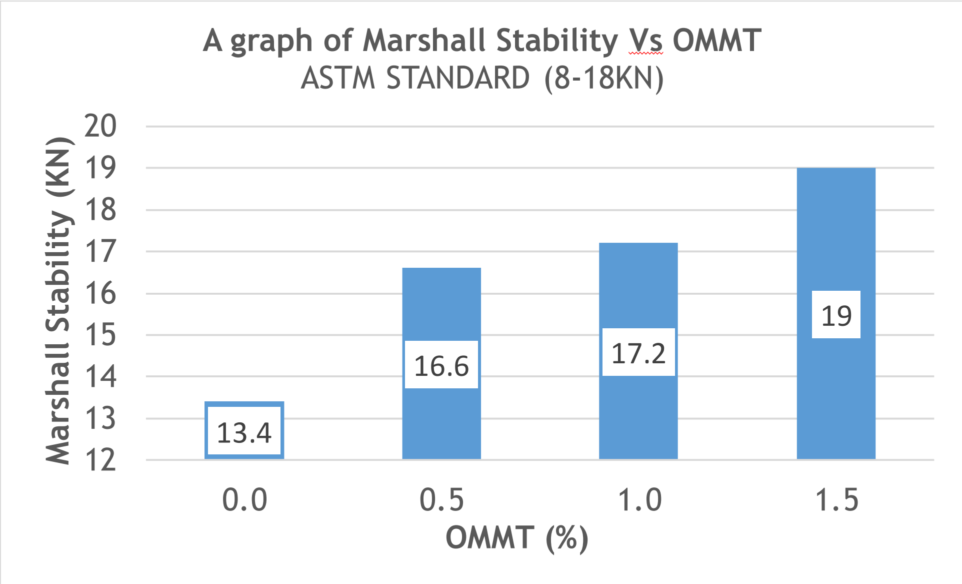
SPECIMEN NO.	OMMT %	Gmm	Gmb
1	0.0	2.450	2.342
2	0.5	2.443	2.346
3	1.0	2.437	2.352
4	1.5	2.438	2.322

From the obtained data, the value of Gmm slightly dropped from 2.450 to 2.437 at 1.0% OMMT, but it soon settled at 2.438 at 1.5%. On the other hand, the value of Gmb generally increased from 2.342 to a maximum of 2.352 at 1.0% OMMT, but decreased to 2.322 at 1.5%. The latter increment at 1.0% further reflects increased compaction and mixture densification, and is well associated with improved rutting resistance. This is supported by the fact that a dense mix with higher Gmb and moderate Gmm provides superior aggregate interlock and is less susceptible to deformation (Brown & Cross, 1992).

The slight decrease observed in Gmm and increase in Gmb to 1.0% OMMT could be attributed to the increased stiffness of the binder and homogeneity of the nano clay distribution in the matrix, thereby increasing the packing density. Nevertheless, with 1.5% OMMT, the increment in clay could cause slight agglomeration of the particles, which could have resulted in reduced compaction, thereby showing the decrease in Gmb

4.3.2 Marshall Stability

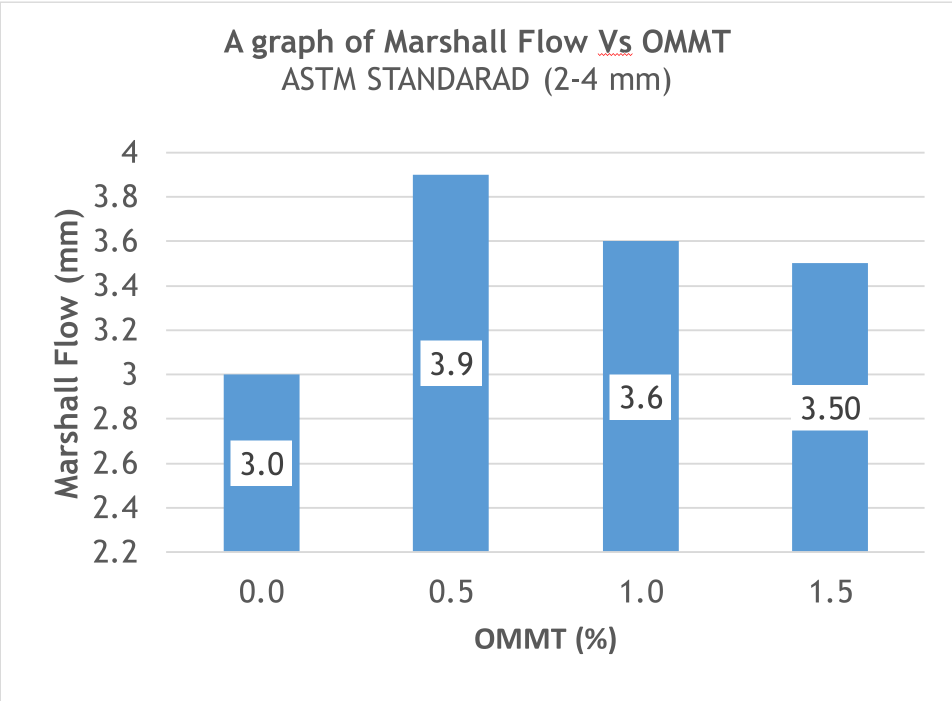
Marshall Stability test is used to determine the highest compressive load that the asphalt specimen may hold prior to failure that is, its ability to resist plastic deformation under traffic. An increase in stability means that the structure is stronger and the rutting performance is greater (Huang, 2012).



The findings demonstrated a gradual rise of 13.4kN (0% OMMT) 17.2kN (1.0% OMMT) and highest at 19.0kN (1.5% OMMT). All the values are between 8-18kN with exception of the utmost value, which slightly approximates above the upper limit. The value of 1.0% is a more suitable ratio of strength and flexibility, and the same value meets the design requirements, although the maximum balance of the latter is 1.5%. Greater stability at higher content of OMMT is due to enhanced rigidity and strength of the binder phase, higher aggregate bonding and shear resistance. But a high stability can be the sign of low flexibility, and thus, cracking can take place, particularly when subjected to repetitive loading (Khattak et al., 2013).

4.3.3 Marshall Flow

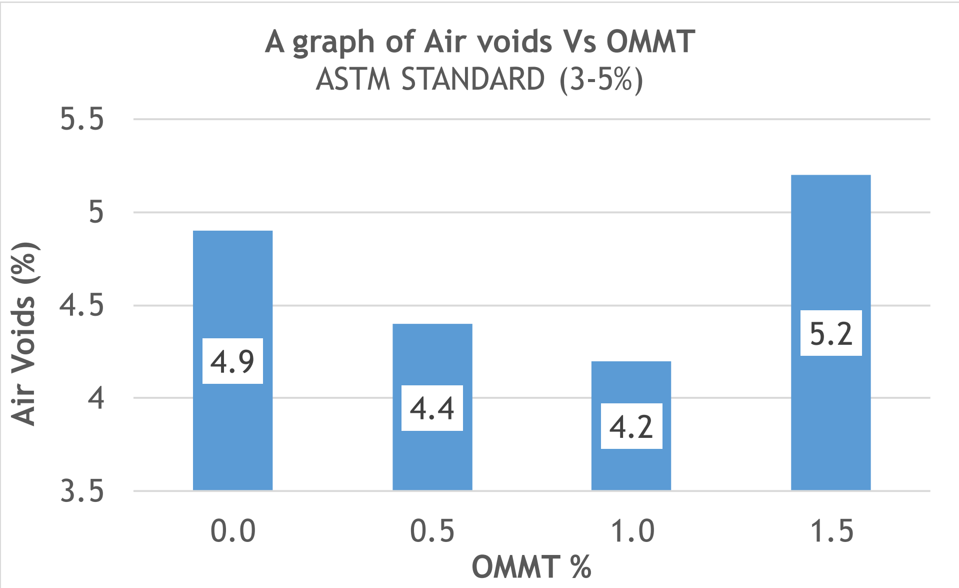
Marshall Flow measures how much deformation the mix can get at the time of failure and reflects the capacity of the mix to carry out any form of load displacement without cracking. A good combination must reflect a moderate flow of 2 to 4 mm, in line with the ASTM provisions, where it is necessary to have sufficient workability and resistance to over deformation (Read and Whiteoak, 2003).



Flow values of 3.0 mm at 0% OMMT, 3.6mm at 1.0 percent and 3.5mm at 1.5% were within standard ranges. The increase in flow at 1.0% indicates an opportune trade-off equilibrium between rigidity and deformability. Despite the fact that high content of the OMMT raises the binder viscosity and stiffness, it also raises the structural cohesion and makes the material behave in terms of controlled deformation under loading.

4.3.4 Air Voids (VA)

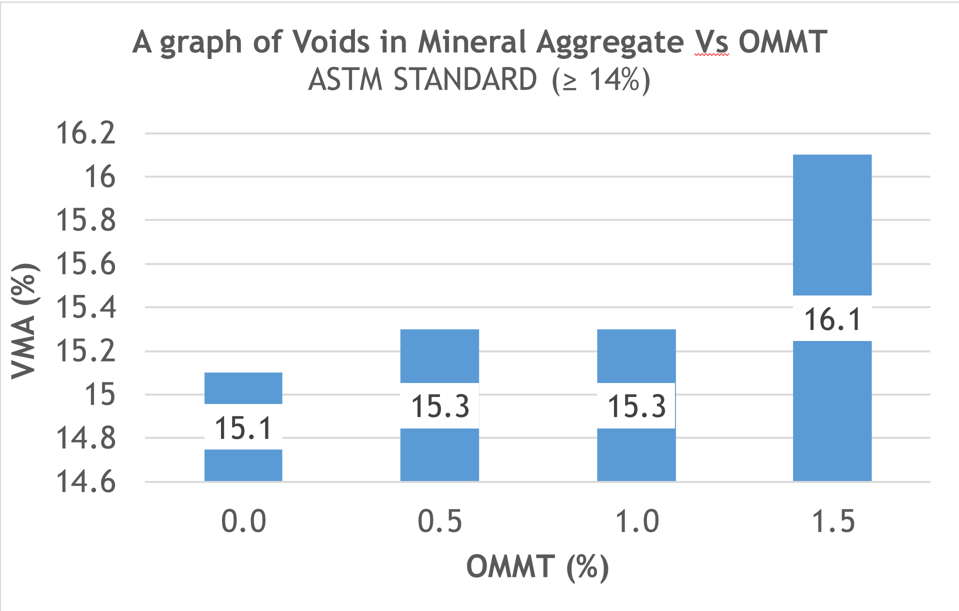
Air Voids point to the amount of percent of air pores present in the compacted asphalt mixture. They play a significant role in evaluating the properties of durability, repelling qualities to moisture and the ability to withstand irreversible deformation. The range of air void content that is better placed should be between 3 and 5% so as to achieve enough compaction and not excessive bleeding and permeability (Roberts et al., 1996).



There was a decrease in the air voids, which dropped to 4.2% at minimum of 1.0% and rose to 5.2% at 1.5%. The minimum at 1.0% is acceptable that suggests an increase in compaction and binder-aggregate interaction at that content. The diminution in air voids at 1.0% OMMT is probably because of enhanced binder stiffness and enhanced coating of aggregates, which enhances matrix densification. Nevertheless, it became stiff at 1.5% and the agglomeration of nano clays excessively, thereby preventing additional compaction resulting in a rise in air voids.

4.3.5 Voids in Mineral Aggregate (VMA)

The VMA is the sum of the voids in aggregate particles of compacted mix of both the air and the bitumen-filled ones. It shows the space that can be occupied by the binder and is critical in making it to last long and offer great resistance to rutting (Huang, 2012). High VMA permits in general higher accommodation to binder and fatigue resistance.

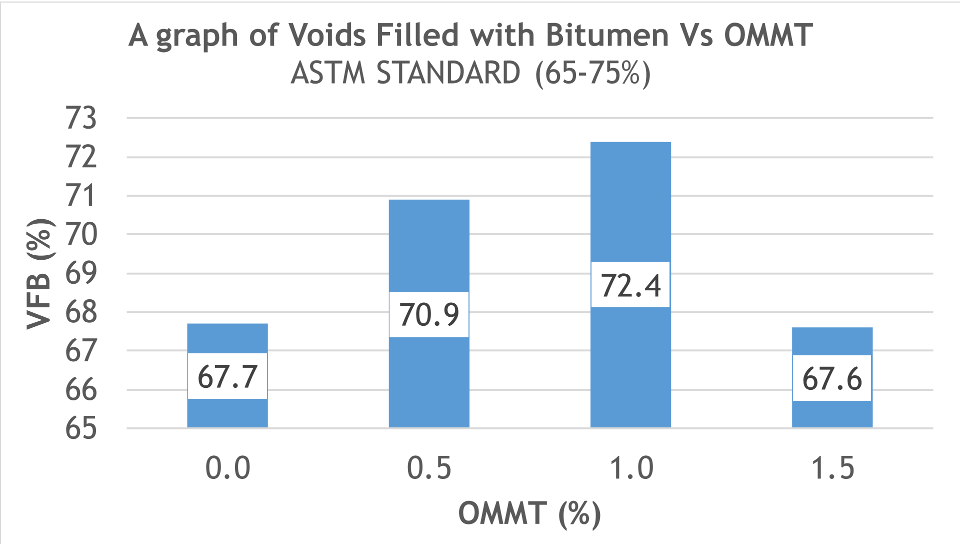


VMA values rose between 0 to 1.0 percentage OMMT in 16.1 to 15.1 percentage OMMT with little rise to 16.1% at 1.5 percent. These values meet the ASTM requirement of more than 14 per cent indicating sufficient binder space in all mixes.

The stable condition of VMA at the highest concentration of 1.0% OMMT is a sign of regulated aggregate structure and good binder filling. The slight rise at 1.5 percent can be attributed to a decrease in the compaction effectiveness achieved by heightened binder viscosity, preventing the aggregates in compaction movement (Khattak et al., 2013). This is a normal behavior in the increase in air voids at the same level of OMMT.

4.3.6 Voids Filled with Bitumen (VFB)

VFB is a percentage of VMA covered with bitumen and indicates the efficiency of binder to utilize a free space. An increase in VFB normally signals a better aggregate coating and durability behavior with the usual values between 65 and 75-percent optimum functioning (Read and Whiteoak, 2003).



VFB was higher than that of 0% OMMT (67.7) and reached to the highest level at 1.0% (72.4), and then declined to 67.6, 67.7 and 68.2 at 1.5, 2.0, and 3.0 respectively. The value at 1.0 percent is under upper optimum range, which is an indication of efficient use of binder and less prone to moisture penetration.

The highest VFB of 1.0% is probably caused by a better binding ability and a greater distribution of the OMMT in the matrix making it easy to produce the intended result of coating the aggregates. This could have stiffened and led to an uneven distribution of the binder at 1.5% hence the loss in VFB. The same tendencies were observed in the literature, with the optimum content of nano clay ensuring better absorption of the binder, and too much of it decreasing workability (Abdullah et al., 2022).

4.3.7 Indirect Tensile Strength (ITS)

The ITS test was conducted to assess the tensile strength of the asphalt mixture with 1.0% OMMT that was determined to be the optimal content of modifier. This is a test that determines the resistance of the mixture to cracking, internal cohesion and moisture sensitivity. Tensile strength is a factor that can also be used as an indicator of shear resistance and therefore ITS results can be effective predictors of rutting efficiency because mixtures with higher tensile strength tend to exhibit lower lateral deformation during repeated loading (Huang, 2012).

Table 14: Displays the ITS values of the asphalt samples

OMMT %	**ITS (DRY) Kpa**	**ITS (WET) %**
0.0 (CONTROL)	1066	89
1.0 (OPTIMUM)	1249	92
ASTM STANDARDS	>800Kpa	>80% of dry

The 1.0% OMMT blend registered a dry ITS of 1249kPa which was much higher than ASTM standard of 800kPa which is a sign of high internal bonding and a good tensile strength. The wet ITS was 92% of the dry, which exceeds the ASTM of having over 80% of the dry ITS remaining after the conditioning of moisture. The wet ITS of the modified mix was significantly higher than that of the control mix (1066kPa dry; 89% wet) and so was the wet performance. This implies that the tensile strength increased by 1.0% when OMMT which is presently at 1.0% was added but with the same moisture resistance being acceptable.

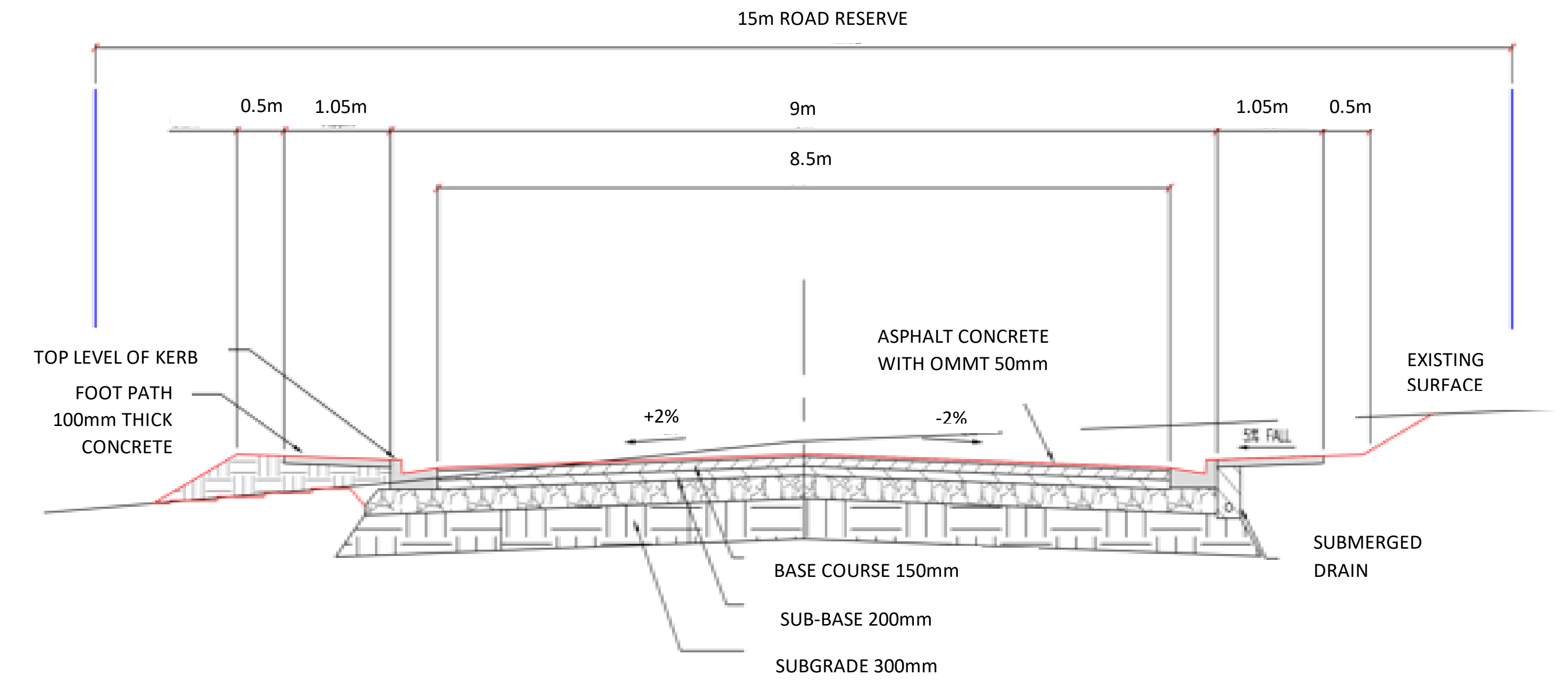
The enhanced dry mixture of the 1.0% OMMT mix of the ITS is explained by the strengthening effect of the platelets of nano clay on the bitumen. The OMMT not only increases the stiffness of the binder and enhances the strong performance of the aggregate-binder interface, but also improves the transfer of loads and the material cohesion (Chen et al., 2011). The wet ITS was also kept near the control since the adhesion on the wet ITS is affected by moisture in such way that no matter the modification, they always have weak adhesion which is not enough to meet performance requirements. In general, the ITS findings confirm the assumption that the 1.0% OMMT change improves shear resistance and positively affects rutting resistance because an increase in tensile strength is normally linked to a decrease in permanent deformation caused by traffic (Khattak et al., 2013).

4.4 DESIGN

The following table shows the detailed design of an altered asphalt mix, such as the proportions of the aggregates, the binder, and OMMT additive. It indicates the percentage of each component and the resulting mass in order to obtain an 8000g total mix.

Table 15: Shows the Mix Design of Asphalt Concrete with Optimum (1.0%) OMMT

Material in the mix	**% in the mix**	**Mass (g)**
14/20 mm	24	1825.9
10/14 mm	11	836.9
6/10 mm	18	1369.4
0/6 mm	46	3499.7
OMMT (1%)	1.0	76.1
Total Aggregates	100	7608
Binder	4.9	392
Total Mass		8000



TYPICAL ROAD CROSS SECTION

TITLE	EVALUATING THE RUTTING RESISTANCE OF ASPHALT CONCRETE MODIFIED WITH OMMT
NAME	ASONYA JOHN M22B32/003 GAMUKAMA MICHAEL S18B32/183
SCALE	1:50
DATE	10/12/2025

CHAPTER 5: CONCLUSION AND RECOMMENDATIONS

5.1 CONCLUSION

The experiment revealed that the addition of OMMT to asphalt concrete led to the significant increase of volumetric and mechanical performances. Having the optimal content of 1.0%, the mixture presented good balanced values of air voids, VMA and VFB all existing within recommended limits. These conclusions show that OMMT improved aggregate packing and distribution of binders, which improved compaction and declined permeability. The required stability and flow criteria were also achieved by the mixture, which showed a good balance of stiffness and flexibility which is crucial in preventing rutting and still allow the required accommodating deformations caused by the traffic.

The advantages of the nano clay modification were also further established with mechanical assessment using the ITS test. The 1.0% OMMT mixture realized dry and wet ITS which were superior to ASTM standards indicating enhanced cohesion and strength of aggregate-binder bonding even at wet condition. This increase in tensile strength implies opportunity to be resistant to cracking, moisture damage as well as shear deformation, which are critical elements influencing pavement durability.

In general, the results reveal that OMMT 1.0% gives a positive growth to the performance of asphalt concrete. The modifier enhanced binder dispersion, compact capacity and tensile strength without compromising on flexibility of the mixture.

5.2 RECOMMENDATIONS

1. Use 1.0% OMMT as the optimum dose of asphalt concrete, as it had given desired volumetric characteristics and high tensile strength, which directly boosts rutting resistance. This dosage needs to remain as a standard reference mix to be used during the production and performance assessment. Quick verification testing should be undertaken to indicate agreement of results in the event of any alterations in aggregate source or binder grade.

2. Ensure extremely tight quality control on dispersion of OMMT during the mixing process, the distribution pattern is to be the same to achieve the highest binder cohesion possible and avert negative effects caused by an increase in dosage. Average mixing time, temperature and blending sequence are to be recorded and used in all the batches. An occasional examination of the consistency of the binder and homogeneity of the mixture should be performed to identify and eliminate manner of dispersion issues beforehand.

3. Field-check lab tests, make use of wheel-tracking tests or pilot pavement areas to ensure rutting resistance in actual traffic and climatic conditions. Periodic surface inspection and rut depth monitoring should be incorporated in the performance monitoring. Field results need to be directly compared with laboratory predictions to help make large scale implementation decisions.

REFERENCES

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Abdullah, A., Aziz, G., & Ansari, M. M. (2022). The behaviour of nanoclay-modified asphalt binders and mixtures. University of Alberta Technical Report.

APPENDIX

MAKERERE UNIVERSITY

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Fax: + 256 - 414 - 531061

College of Natural Sciences
School of Physical Sciences
DEPARTMENT OF GEOLOGY AND PETROLEUM STUDIES

Tuesday, 30 September 2025

Gamukam Michael and Asonya John

Dear Sir,

RE: MINERALOGICAL ANALYSIS OF A SAMPLE BY XRD

The Department received one sample (1) in the laboratory, to be analysed for mineralogical composition. During analysis the sample was ground to fine powder (< 0.063 mm) and scanned using the XRD Spectrometer (Shimadzu -XRD-7000) for mineralogical composition. The results below were obtained.

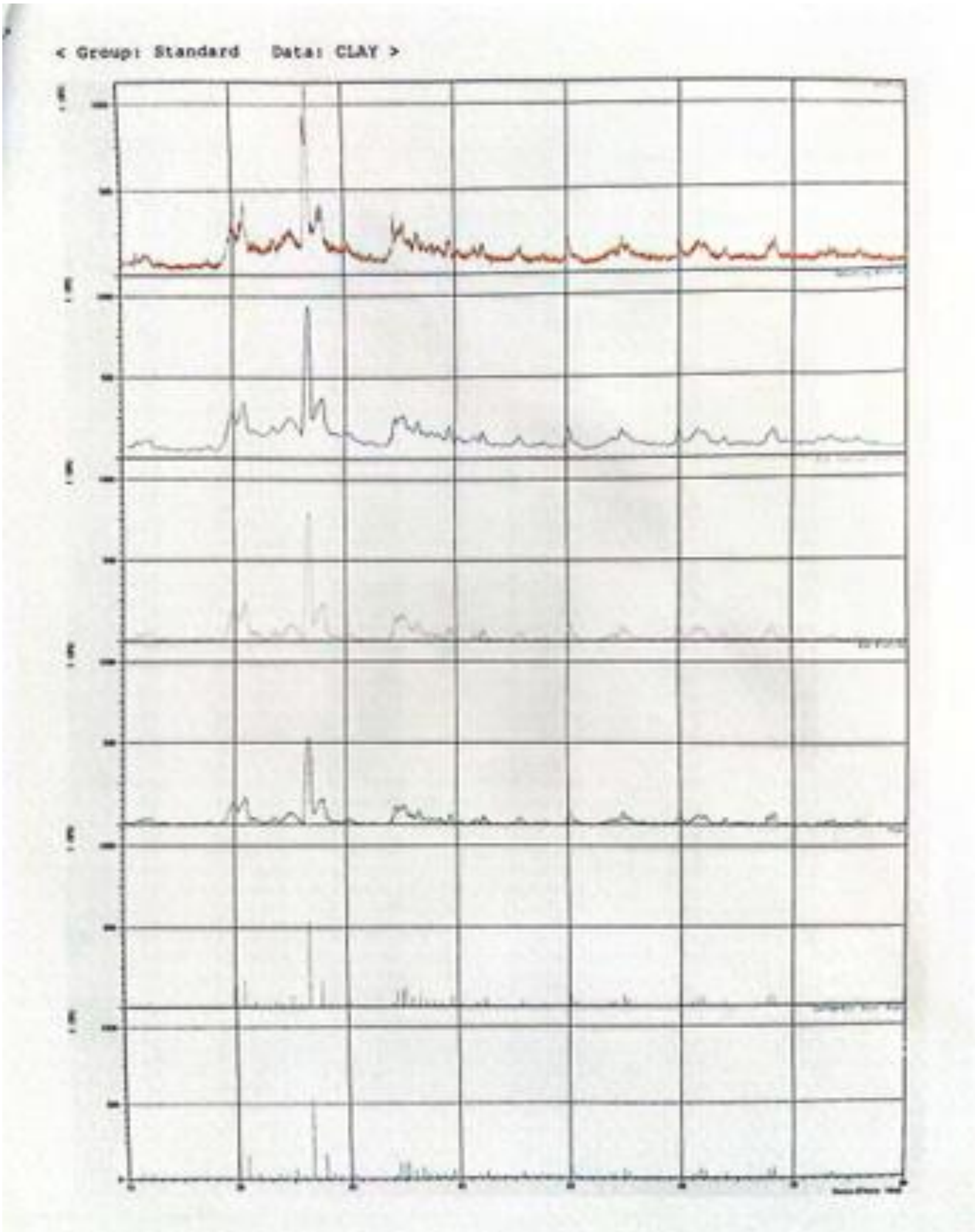
Minerals	Minerals
Quartz	Tridymite
Hallyoisite	Tetramajorites
Deckite	Anorthite
Gismon	Moganite
Nacrite	Albite
Kaolinite- Montomorinite	Grossite
Chrysotile	Anti Gorrite
Stishovite	Opal

Note: The details about the mineral as obtained from the XRD spectrometer are attached on the report.

Analyst: M. Kasaka

Kasaka Moses

Geochemistry Laboratory


< Group: Standard   Data: CLAY >

INSTITUTION	STUDENT	SUPPLIER
UGANDA CHRISTIAN UNIVERSITY	GAMUKAMA MICHAEL REG NO. S18B32/183 & ASONYA JOHN M22B32/003	Stirling

PROJECT	EVALUATING THE RUTTING RESISTANCE OF ASPHALT CONCRETE MODIFIED WITH MONMORILLONITE NANO CLAY

LOCATION	BITUMEN TESTS	OPERATOR Lab team
SUPPLIER		CONTAINER/DELIVERY NO
DATE TESTED: 10-7-2025		DESTINATION
MATERIAL TYPE 60/70		

TEST NO		3		5		2P		AX		AVERAGE	REMARKS
PENETRATION 5 sec 25 C	100gr	66 68 68		69 67 66		67 68 68		68 69 69		68	60-70
SOFTENING POINT (°C)		53		54						53.5	(49-56)°C
BITUMEN AFFINITY										>95	>95
SUPECIFIC GRAVITY										1.025	

STIRLING CIVIL ENGINEERING LTD

Lab Technician — Materials Engineer

P. O. BOX 798, KAMPALA (U)

4

INSTITUTION	STUDENT	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	GAMUKAMA MICHAEL REG NO. S18B32/183 & ASONYA JOHN M22B32/003	Stirling

PROJECT :	EVALUATING THE RUTTING RESISTANCE OF ASPHALT CONCRETE MODIFIED WITH MONMORILLONITE NANO CLAY

A/C 14 ASPHALT RESULTS

SUMMARY OF A/C 14 TEST RESULTS

			BITUMEN CONTENT		4.9
AGGREGATE TESTS	ACHIEVED	SPECIFIED	MARSHALL MIX TEST RESULTS AFTER MIX	ACHIEVED PLANT PRODUCTION	SPECIFIED
			MARSHALL FLOW	3.0	2—4
Sodium Soundness	2.1	<12%	MARSHALL STABILITY 75BLOWS	13.4	9-18
Water Absorption	0.4	< 2%	MARSHALL AIR VOIDS 75BLOWS	4.9	3—5
TFV Dry	255.6	>110kN	VOIDS IN MINERAL AGGREGATES	15.1	>14%
TFV Soaked Wet/Dry ratio	89%	>75%	VOIDS FILLED WITH BINDER	67.7	65—75%
Flakiness Index	19.5	< 25%	INDIRECT TENSILE STRENGTH @ 25C	1,066	>800kpa
Plastic Index	N/P	< 4%	INDIRECT TENSILE WET STRENGTH	89	>80% of dry
LAA	16.8		BITUMEN CONTENT AFTER EXTRACTION	5.0	±0.3
ACV	15.8		RATIO STABILITY/FLOW	4.4	>2.5
AIV	15.0				

For CONTRACTOR

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INSTITUTION	STUDENT	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	GAMUKAMA MICHAEL REG NO. S18B32/183 & ASONYA JOHN M22B32/003	Stirling

PROJECT :	EVALUATING THE RUTTING RESISTANCE OF ASPHALT CONCRETE MODIFIED WITH MONMORILLONITE NANO CLAY

A/C 14 ASPHALT WITH 0.5% MONMORILLONITE NANO CLAY

SUMMARY OF A/C 14 TEST RESULTS

			BITUMEN CONTENT		4.9
MARSHALL MIX TEST RESULTS AFTER MIX				ACHIEVED PLANT PRODUCTION	SPECIFIED
MARSHALL FLOW				3.9	2—4
MARSHALL STABILITY 75BLOWS				16.6	9-18
MARSHALL AIR VOIDS 75BLOWS				4.4	3—5
VOIDS IN MINERAL AGGREGATES				15.3	>14%
VOIDS FILLED WITH BINDER				70.9	65—75%
INDIRECT TENSILE STRENGTH @ 25C				1,111	>800kpa
INDIRECT TENSILE WET STRENGTH				89	>80% of dry
MASHAL DENSITY				2.3	....

For CONTRACTOR

STIRLING CIVIL ENGINEERING LTD

25 MAY 2025

P. O. BOX 795, KAMPALA (U)

INSTITUTION	STUDENT	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	GAMUKAMA MICHAEL REG NO. S18B32/183 & ASONYA JOHN M22B32/003	Stirling

PROJECT :	EVALUATING THE RUTTING RESISTANCE OF ASPHALT CONCRETE MODIFIED WITH MONMORILLONITE NANO CLAY

A/C 14 ASPHALT WITH 1.0% MONMORILLONITE NANO CLAY

SUMMARY OF A/C 14 TEST RESULTS

BITUMEN CONTENT		4.9
MARSHALL MIX TEST RESULTS AFTER MIX	ACHIEVED PLANT PRODUCTION	SPECIFIED
MARSHALL FLOW	3.6	2—4
MARSHALL STABILITY 75BLOWS	17.2	9-18
MARSHALL AIR VOIDS 75BLOWS	4.2	3—5
VOIDS IN MINERAL AGGREGATES	15.3	>14%
VOIDS FILLED WITH BINDER	72.4	65—75%
INDIRECT TENSILE STRENGTH @ 25C	1,249	>800kpa
INDIRECT TENSILE WET STRENGTH	92	>80% of dry
MASHAL DENSITY	2.3	....

For CONTRACTOR

Lab technician

STIRLING CIVIL ENGINEERING LTD

P. O. BOX 755, KAMPALA (U)

INSTITUTION	STUDENT	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	GAMUKAMA MICHAEL REG NO. S18B32/103 & ASONYA JOHN M22B32/003	Stirling

PROJECT :	EVALUATING THE RUTTING RESISTANCE OF ASPHALT CONCRETE MODIFIED WITH MONMORILLONITE NANO CLAY

A/C 14 ASPHALT WITH 1.5% MONMORILLONITE NANO CLAY

SUMMARY OF A/C 14 TEST RESULTS

	BITUMEN CONTENT		4.9
MARSHALL MIX TEST RESULTS AFTER MIX		ACHIEVED PLANT PRODUCTION	SPECIFIED
MARSHALL FLOW		3.5	2—4
MARSHALL STABILITY 75BLOWS		19.0	9-18
MARSHALL AIR VOIDS 75BLOWS		5.2	3—5
VOIDS IN MINERAL AGGREGATES		16.1	>14%
VOIDS FILLED WITH BINDER		67.6	65—75%
INDIRECT TENSILE STRENGTH @ 25C		724	>800kpa
INDIRECT TENSILE WET STRENGTH		78	>80% of dry
MASHAL DENSITY		2.3	....
For CONTRACTOR			

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29 NOV 2025

P. O. BOX 793, KAMPALA (U)

INSTITUTION	STUDENT	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	GABUKAIRA MICHAEL REG NO. [illegible] & ASONYA JOHN [illegible]	Stirling

PROJECT	EVALUATING THE RUTTING RESISTANCE OF ASPHALT CONCRETE MODIFIED WITH MONMORILLONITE NANO CLAY

Field Ref. No.		Lab no.		Chainage sample 1	A/C 14 ASPHALT WITH [illegible] MONMORILLONITE NANO CLAY	Sampling date	15-Nov-25	Test Type	Done by	Test Type	Done by
Sample grade	AC 14	Compaction	75 blows	Chainage sample 2		Testing date	16-Nov-25	B.R.D	lab team	A.C/Grad.	lab team
Sample Description				AC 14				T.M.R.D.	lab team	Stab. & Flow	lab team

ASTM D2726 - Standard Method for Bulk Specific Gravity and Density of Non-Absorptive Compacted Bituminous Mixtures.

Marshall specim.	Mass in air	Mass in Water	Saturated surface Dry in air	Bulk S.G (G_{mb})	Unit Wt. (Kg/m³)	% Air Voids	% VMA	% VFB
1	1184.4	687.40	1186.20	2.351	2.300	4.0	15.1	73.4
2	1189.6	685.20	1192.60	2.345	2.313	4.2	15.2	72.1
3	1192.0	688.70	1187.80	2.341	2.330	4.4	15.4	71.5
4	1189.4	686.70	1193.40	2.347	2.336	4.3	15.2	72.7
Average Sample 1				2.346	2.335	4.2	15.3	72.4
-				-	-	-	-	-
Average Sample 2								
Average Sample 1 & 2				2.346	2.335	4.2	15.3	72.4

ASTM D6927 Standard Method for Marshall Stability and Flow

Marshall Heights (mm)			Av. Hgt (mm)	Corr. Factor	Stability (KN) Measured	Stability (KN) Adjusted	Flow (mm)	Ratio (Stab./Flow)
64.3	63.1	64.6	63.9	0.99	17.3	17.17	3.41	5.034
64.2	64.4	64.6	64.4	0.98	17.1	16.77	3.52	4.763
64.5	64.2	65.2	64.6	0.97	17.6	17.06	3.75	4.545
64.3	64.3	64.3	64.4	0.98	16.6	16.31	3.71	4.397
Average Sample 1			64.3	1.0	17.2	16.8	3.6	4.7
-	-	-	-	-	-	-	-	-
Average Sample 2								
Average Sample 1 & 2								

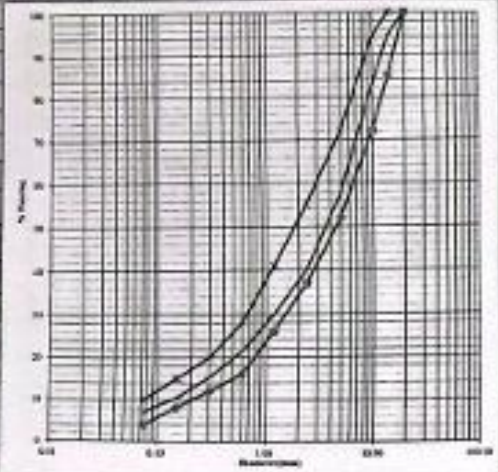
ASTM D 2172- Standard Test Method for Quantitative Extraction of Bitumen from Asphalt Mixtures

Mass (g)	Sample 1	Sample 2
Bowl	232.3	167.8
Bowl + Asphalt	1937.5	1869.0
Asphalt	1705.2	1701.2
Filter paper before extraction	29.3	31.8
Filter paper + Filler After extraction	30.8	33.1
Recovered Filler	1.5	1.3
Oven dry extracted [illegible] (dry)	1619.6	1614.1
Oven dry extr. roll + filler	1621.1	1615.4
Bitumen	84.1	85.8
% of Bitumen	4.9	5.0
Av. % of Bitumen	5.0	

ASTM D2041- Standard Test Method for Theoretical Maximum Specific gravity and Density of Bituminous Mixtures

SAMPLE 1			SAMPLE 2		
(Pycnometer with Water)			(Pycnometer with Water)		
Temperature of water (°c)	25°C		Temperature of water (°c)	25°C	
in pycnometer	in water bath		in pycnometer	in water bath	
Test No-	1	2	Test No-	1	2
Asphalt	1318		Asphalt	1278.5	-
Pycn. + Water	2479.7		Pycn. + Water	2479.7	-
Pycnometer + Asph. + Water	8256.5		Pycnometer + Asph. + Water	8224.1	-
Volume of asphalt	541.2	-	Volume of asphalt	526.1	-
G_{mm}	2.435	-	G_{mm}	2.430	-
Av. G_{mm}	2.435		Av. G_{mm}	2.430	
Av. Gmm (Kg/m3) Sample 1 & 2			2.437		

Sieve (mm)	Smpl1 Mass retained	Smpl2 Mass retained	Av. Mass retained	% Av. Retained	% Av. passing	JMF Lower	JMF Upper
20	0.0	0.0	0.0	0.0	100	100	100
14	87.9	90.4	89.2	5.5	94	85	100
10	185.0	196.7	190.9	11.8	83	72	94
5	388.0	420.0	404.3	21.0	[illegible]	32	71
2.36	287.9	282.1	285.0	17.6	[illegible]	37	55
1.18	170.0	160.2	165.1	10.2	[illegible]	26	41
0.600	146.6	127.2	136.8	8.5	[illegible]	16	28
0.300	106.8	89.8	98.3	6.1	11	12	20
0.150	76.0	83.2	79.6	4.9	[illegible]	8	15
0.075	55.0	52.4	53.7	3.3	7	4	10
Total filler	102.3	115.8	109.0				
Bot. Pan	3.0	5.00	4.0				
Ext. filler	1.5	1.30	1.4				
Sum of extr. Agg	1619.6	1614.1	1616.9				



Comment:

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