INVESTIGATING THE USE OF DENSITY POLYETHYLENE IN MODIFYING BITUMEN TO IMPROVE PERFORMANCE OF FLEXIBLE PAVEMENTS ALONG CLIMBING LANES

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ABSTRACT

This study investigated the application of bitumen modification in improving the performance of flexible pavements against rutting along climbing lanes. Such sections are characterized as severely loaded sites that experience deceleration of traffic and increased loading time. As the pavement temperature increases, the binder softens and is unable to withstand the increased loading hence causing shear deformation. A case study was evaluated at the climbing lane section along Bweyogerere – Jinja road that experienced premature deformation a few years after its reconstruction. The study deduced that the failure was caused by instability rutting within the asphalt layer. The properties of both aggregates and bitumen were evaluated to understand the cause of failure. The study employed mechanistic – empirical approach where the index properties of the materials were related to pavement performance using recommended asphalt models. Whereas the aggregates were found to possess excellent performance, the bitumen was found to be temperature sensitive hence causing reduced stiffness at high temperatures. Bitumen modification with LLDPE showed reduction in temperature susceptibility and increased stiffness modulus at high service operating temperatures and loading time. Asphalt mix specimen with 2% LLDPE increased the resistance to permanent deformation without affecting pavement durability. The modified specimen were subjected to ITS and PRD tests where an increase in tensile strength of 16.7% was recorded. The air voids at refusal density increased by 42% signifying increased flexibility upon secondary compaction under heavy loads thereby increasing service time and solving the issue of premature deformation.

DECLARATION

I hereby declare that the details in this report derived from efforts from my original work. This work has neither been plagiarized nor submitted to any institution of learning for any award.

Signature: …………………………………………. Date: ………………………………………………

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Registration number:

APPROVAL

This report has been submitted for examination with my approval as the university supervisor.

Signature: ……………………………………………   Date: …………………………………………………

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ABBREVIATIONS AND ACRONYMS

AASHTO	: American Association of State Highway and Transport Officials
AC	: Bituminous mix Class
ACV	: Aggregate Crush value
AIV	: Aggregate Impact Value
ASTM	: American Society for Testing and Materials
BS	: British Standards
HDPE	: High Density Polyethylene
HMA	: Hot Mixed Bituminous mix
LAA	: Los Angeles Abrasion
LLDPE	: Linear Low Density Polyethylene
LDPE	: Low density Polyethylene
OBC	: Optimum Bitumen Content
PMB	: Polymer-Modified Bitumen
MoWT	: Ministry of Works and Transport
VMA	: Voids in Mineral Aggregates
PI	: Penetration Index
PRD	: Percentage Refusal Density
ITS	: Indirect Tensile Strength

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CHAPTER ONE: INTRODUCTION

1.1 Introduction

Road transport in Uganda is the most used form of transport across the entire nation. A study conducted by ACODE to insinuate the state of roads in the country claimed the roads sector has always received the largest share of the national budget over the last decade; however, there is a persistent public outcry about the poor state of roads and premature deterioration of the works being executed before reaching their design life (Ggoobi, 2020). One of the major forms of deterioration is rutting which is caused by plastic deformation which manifests as irregularities along the wheel path (Lazar & Racanel, 2017). This may further cause fatigue cracking when the pavement deforms to a point of sudden fracture. These depressions also retain water on the pavement that may infiltrate and cause further damage. Rutting is the most common form of distress that occurs in asphalt pavements and is most prevalent along longitudinal slopes where there is slow moving traffic. This is associated with the decrease in average speed of vehicles while running up the slope thereby increasing the loading time on to the pavement (Chang, 2019).

In tropical areas like Uganda where maximum pavement temperatures can go up to 59^{0}C as highlighted by the meteorological center of Uganda, the viscosity of the binder reduces at such temperatures as well as the adhesion amongst on to the aggregates which decreases stiffness of the asphalt mixture (Rolt, 2022). Under slow moving traffic of heavy vehicles, the longer loading time, coupled with the reduction in stiffness, causes an accumulated permanent deformation under each repeated loading that leads to plastic deformation. The asphalt binder becomes too soft to withstand the shear stresses leading to rearrangement of aggregate particles caused by shear flow (Richard, 2009). This scenario is more influential along longitudinal slopes due the internal shearing mechanism that occurs in the middle layer of the asphalt pavement. This study focused on use linear low-density polyethylene to improve the rheological and aggregate binding properties of bitumen to increase stiffness of asphalt mixes used along longitudinal slopes to overcome shear deformation.

1.2 Background

In 2017, the Uganda National Roads Authority under the National Road Development and Maintenance Programme (NRDMP) embarked on the upgrading of the road section between Namanve and Bweyogerere along Kampala – Jinja highway. This comprised of removing the deteriorated asphalt layer and replacing it with a new asphalt overlay. Unfortunately, a few years after undertaking the upgrade, the asphalt has experienced rutting especially at the climbing lane section as shown in figure 1. It is essential to note that resistance to rutting is a combined effort between the aggregates which provide resistance to vertical compressive stress and the bitumen that carters for the internal shear stress distribution, which is the major cause of shear deformation especially along longitudinal slopes (Sun & Chao, 2021). The aggregate gradation can ensure aggregates size fraction are proportioned so as sufficient stone-to-stone contact between as many aggregate particles as possible (Rolt, 2022). The use of aggregates with higher angularity can also increase the inter-particle friction thereby increasing aggregate interlocking capability. This can help to improve the resistance of the asphalt pavement to vertical compressive loads however; the issue of internal shear deformation is a crucial aspect especially in tropical countries where the increase in temperature causes the binder to soften and deform under traffic loads.

The empirical approach which is widely used across Uganda for pavement design basically uses index tests which are restrictive to single point measures. This does not clearly account for the variation in field conditions hence may cause failures especially along severely loaded sections. This approach may be work effectively for low stress scenarios like flat sections where the amount of permanent strain or deformation in each load cycle is small, such that the compressive stress is proportional to gradual deformation over a long time. However, at higher stresses (severe loading conditions) such as slow-moving traffic at longitudinal slopes, the relationship between the stress and strain will not cause the same gradual deformation (Singh & Sahoo, 2021). As the material properties that define stiffness change due to high temperature, the asphalt will experience heavy loading rate and reduced loading time causing permanent deformation and shear flow as the binder softens at high temperatures thereby experiencing rutting at an earlier stage.

The use of additives to improve performance properties of asphalt in severe conditions has been proposed by researchers to be of greater advantage (Harun & Jones, 1992). Polymers often used as bitumen modifiers are categorized as; plastomers, elastomers and reactive polymers depending on their structure and chemical composition however, each has a different effect when used in bituminous binders. It is important to note that the rheological properties of bitumen are key in determining the index properties of binders, however, most of these are empirical and single point measures that may not be able to predict clearly the performance of the asphalt mixture as highlighted by (Richard, 2009). Relational derivatives were developed by several researchers that relate the empirical measures to mechanistic approach so as to predict the performance of the pavement at specified service conditions. Some of prediction models derived by researchers that relate index properties of bitumen to asphalt performance are highlighted in section 2.1. An approach by was recommended by many researchers and incorporated in ASTM specifications. This relates the index properties of softening point and penetration to temperature susceptibility through a parameter called penetration index. This parameter, in conjunction with expected loading time of the pavement, expected pavement temperature can be used to determine the expected stiffness modulus of the asphalt. This concept was used in this study to evaluate the performance of the LDPE modified bitumen.

1.3 Problem statement

Rutting is a major form of pavement deterioration which manifests as irregular depressions along the wheel path. These depressions may further retain water which infiltrates to the layers beneath escalating other defects such as cracking, potholes. Rutting was observed at the section along the climbing lane from Namanve to Bweyogerere along Jinja road, which has experienced premature deformation despite of the fact that an upgrade of the road section was conducted in 2022. A study conducted by (Chilukwa, 2019) deduced that rutting is most prevalent along climbing lanes characterized by a decrease in average speed while climbing up the longitudinal slope hence increasing the loading time.

In tropical regions, increase in temperature causes the binder to soften reducing the cohesion within the bitumen and adhesion between binder and aggregates hence reduced stiffness. This reduction in stiffness coupled with the increased loading time leads to plastic deformation. Asphalt mixtures suitable for such severely loaded sections require stiffer asphalt mixtures to withstand the stresses developed. The use of additives and modifiers was proposed by researchers to improve the performance of asphalt (Transport Research Laboratory, 2022). This study therefore was aimed at incorporating low-density polyethylene in bitumen to improve its adhesion and cohesion so as to enhance the performance properties of the asphalt pavements along severe loaded longitudinal slopes against plastic deformation.

1.4 Objectives

1.4.1 Main Objective

To investigate the use of low density polyethylene (LDPE) binder as a bitumen modifier in improving the performance of flexible pavements along climbing lanes.

1.4.2 Specific Objectives

1. To determine the index properties of the materials used in the neat asphalt mixture.
2. To determine the optimum binder content to use in the asphalt mixture.
3. To evaluate the rheological properties of the modified bitumen.
4. To evaluate the performance properties of the modified asphalt mixture.

1.5 Justification

Resistance to permanent deformation in asphalt concrete is a function of both aggregates and the binder which is bitumen. Aggregates provide the required strength and resist deformation under loading through aggregate interlocking. The bitumen acts as a viscoelastic binder which binds the aggregates and filler material together. According to (Richard, 2009), performance of bitumen is influenced by two major properties namely; temperature susceptibility and stiffness modulus. As temperatures increase this could lead to softening of the binder which results into deformation under heavy loading especially at severely loaded traffic sections such as climbing lanes. Several researchers have advised that at such severe locations, the use of bitumen modifiers can solve the problem of rutting (Harun & Jones, 1992). The use of high and low-density modified asphalt mixes has been well established to increase the stiffness of asphalt binder to be more resistant to rutting and fatigue (Alessandra & Paolo, 2017). When used as a modifier in bitumen, the material can ensure the resistance to deformation is improved without affecting the durability of the pavement.

LLDPE is a copolymer comprising of ethylene, butane and hexane with short chain branching linear structure that increases its surface area. When added to bitumen, it absorbs the oily fractions in bitumen and swells. The swollen particles when mixed in aggregates will increase the binding of aggregate particles. This improves the adhesion and cohesion within the asphalt mixture. Increase in adhesion within the binder makes it stiffer and less liable to soften and flow under high temperatures. This can increase the stability and resistance of the asphalt pavement to shear deformation when. The increased cohesion of between the aggregates increases the internal friction between the aggregates, which makes the pavement strong to resist deformation when subjected to axle loads from traffic.

1.6 Scope

1.6 1 Content scope

Several research studies have shown that the use of polyethylene (low-density and high density) improves the performance properties of bitumen to resist deformation under vertical loading. However, a research study by (Luo Yaolu, August, 2023) showed that change in longitudinal slope leads to an increase in loading where maximum vertical compressive stress occurs in the upper most layer of the asphalt causing a shear stress. Maximum shear stress occurs in the middle layer, which causes shear deformation as the longitudinal slope increases. The use of empirical approach in asphalt mix design was disregarded by many researchers as being obsolete. This has led to the introduction of mechanistic - empirical approaches that relate the empirical properties to performance properties of the asphalt mixtures during service conditions.

This study used the empirical data obtained from the various index tests commonly used for asphalt mix design in Uganda and related these to performance properties that affect cause permanent deformation in bituminous mixtures. This study applied the concept of bitumen modification using linear low-density polyethylene to assess its potential in improving the temperature susceptibility and stiffness modulus of asphalt concrete used along climbing lanes so as to resist that issue of rutting along such sections. The low density polyethylene was chosen since it exhibits better tensile strength properties than other low-density polyethylene to increase the resistance of the asphalt pavement to shearing stresses, and that cause shear flow and deformation that manifests as rutting especially along longitudinal slopes.

1.6.2 Geographical scope

The study area is along a section of Kampala to Jinja road at the climbing lane from Namanve to Bweyogerere that has experienced rutting 150m from the main intersection to Mandela national stadium.

Figure 1: Climbing lane section on Kampala - Jinja road.

CHAPTER TWO: LITERATURE REVIEW

2.1 Introduction

A good transport system is a crucial to the social and economic uplift of any developing nation and plays a vital role in the cost effective transportation of freight and passengers. The government of Uganda through the National Development Programme (NDP 3) seeks to improve the regional connectivity by increasing the stock of pave roads to at least 7500 km by 2025. This has induced efforts to set up new infrastructure around the country and improve the current available road network. Unfortunately there is a public outcry concerning the premature deterioration of constructed works even before reaching the design life. An essential component of any road network is the pavement structure which is notably affected by various environmental and induced factors that affect its performance over time hence causing deterioration. This has sparked the need for further research to address the elements that affect the performance of flexible pavements.

2.2 Road pavements

A road pavement is the hard surface of the road which carries traffic loads, transfers them gradually to the subsequent underlying layers such that the load does not the bearing capacity of the subgrade. Pavements can be categorized based on the materials used for construction as rigid or flexible pavements. A rigid pavement is one constructed with a single layer usually made of Portland cement concrete (PCC) or reinforced cement concrete (RCC). This type of pavement is usually durable and long lasting. Flexible pavements on other hand consists of several layers that accommodate flexing by allowing the pavement structure to bends or deflects under traffic and environment induced loads.

2.2.1 Elements of a flexible pavement

A typical flexible pavement structure consists of the subgrade which is the natural existing ground, sub base course, base and finally surface course arranged in order of deceasing strength. This is the most used type of pavement for road construction in Uganda. Flexible pavement structures have the capability to flex and deflect hence the term flexible pavement. Flexible pavements consist of different layers of different bearing capacity arranged from the strongest to the weakest as shown in figure 1 below.

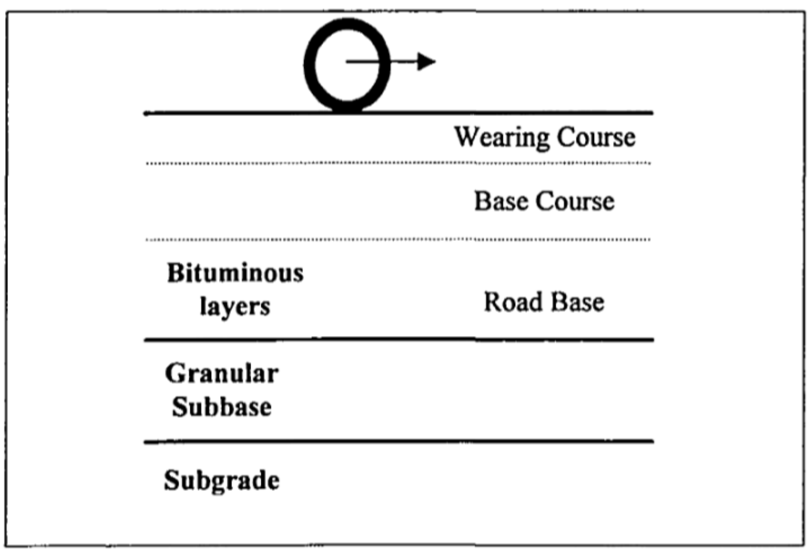


Figure 2: Layers of a flexible pavement

Subgrade

This consists of a layer of the natural existing soil prepared to receive the stresses from the layers above. It should be compacted to the desirable density and the optimum moisture content to achieve the required bearing capacity. In case of low quality soil such as swelling clay, the soil may be stabilized or replaced.

Subbase

The subbase typically comprises materials of lower quality compared to the base course but superior to the subgrade soils. Its inclusion may vary; for instance, a pavement built on a robust, high-quality subgrade might not necessitate a subbase course, thus it could be excluded from the design. However, if the pavement is laid over poor-quality soil like swelling clay, a subbase course may be essential to provide the necessary load distribution capabilities.

Base course

This is the layer beneath the surface course that provides provides additional load distribution and sub-surface drainage. It may be constructed from durable aggregates less susceptible to moisture attack that could be stabilized or unstabilized. In some situations where a highly stiff base is desired, it may be constructed using HMA mixes. However, HMA mixes used for base course mixes usually contain larger maximum aggregate sizes and are usually open graded as compared to surface course.

Surface course

The surface course is the layer in direct contact with traffic loads aimed at providing smooth ride, frictional resistance, noise control, rutting and shoving resistance. This layer is made of high quality materials and constructed with dense graded HMA. This layer is further subdivided into two layers:

- **Wearing Course**. This is the layer in direct contact with traffic loads. It is properly designed with high binder content to make it denser and have less air voids.
- **Binder Course**. This layer provides the bulk of the HMA structure. It is majorly meant to distribute load and hence consists of aggregates with less binder content as compared to the aggregates so as to make it more rigid.

2.2.2 Hot mix asphalt mixtures

An asphalt mixture is a composite material composed of a combination of aggregates, filler and binder (bitumen) commonly used as a road surfacing layer. The binder used in HMA is bitumen, also known as asphalt which binds the constituents of the mixture together. Bitumen is a non-crystalline solid or viscous liquid consisting of hydrocarbons and their derivatives that is soluble in trichloroethylene. It is black oily, viscous product of petroleum refinery but can also exist in natural deposits as naturally occurring organic by-product of decomposed plants (Hirst, 2019). The term bitumen as used in Europe may differ in the US to asphalt, or asphalt cement when the bitumen is refined to meet paving specifications and other special purposes. In this report the terms "asphalt binder," "asphalt cement" and "bitumen" will interchangeably be used to denote the material. In the same manner, "asphalt concrete," "asphalt mixture," "hot mix asphalt (HMA)" and "bituminous mixture" will be used to mean the mixture of aggregates and bitumen.

Different HMA mixtures may be produced from a range of aggregate combinations, nominal maximum aggregate size and asphalt binder content or type each having its own particular characteristics suited for a specific design and intended use depending on the application. The asphalt mixture performance is highly reliant on the constituents of the bituminous mixture which include; aggregates, bitumen, and air voids as shown in figure 3 below.

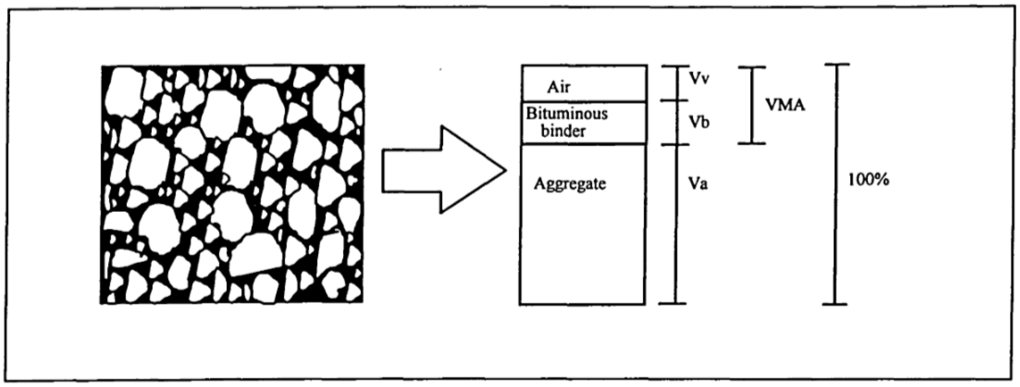


Figure 3: Constituents of bituminous mixtures

The figure 3 above shows the constituents of bituminous mixtures where Vb, and Va are volumetric proportion of air voids, bituminous binder, and aggregate respectively (in percentage). VMA is volumetric proportion of voids in mineral aggregates (Vv+ Vb) expressed as a percentage.

Asphalt mixtures may be classified based on the bituminous mixture composition as;

i. Dense-graded HMA

A dense-graded bituminous mixture is composed of well graded aggregates throughout the entire range of sieves used. In this, there are exist enough fine particles to effectively separate or fill the holes in between coarse particles therefore the distribution of stress within the pavement structure relies on both the coarse and fine particles. The dense graded mixtures contain a wide range of differently sized aggregate particles hence providing enough fines to fill the spaces within the coarse particles. This requires an asphalt binder content ranging in between 4.5% to 6% and VMA between 11% and 17% with air voids close to 4%. These are suitable for all pavement conditions that call for structural stability, friction resistance, levelling and patching needs.

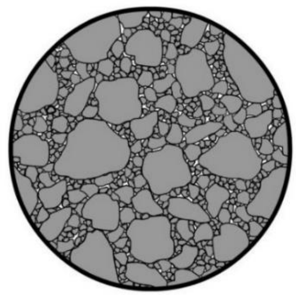


Figure 4: Dense graded HMA

ii. Gap-graded or Stone Matrix Asphalt (SMA)

Gap-graded or SMA is a bituminous mix mixture with a high-coarse aggregate content aimed at maximizing the resistance to rutting by ensuring that there is sufficient stone-to-stone contact within the asphalt mixture. This mix requires aggregates that are more durable, high bituminous mix content (typically more than 6%) and a high-filler content (approximately 10% by weight which could require modified asphalt binder. The resultant mix has a VMA above that for the dense graded more than 17%with asphalt binder content 6%+ and volume of voids close to 4%. This is however cost effective in severe loading conditions such as slow moving heavily loaded traffic due to the increased rutting resistance and improved durability required for it to be successfully placed. This mix contains fewer mid-sized particles therefore the stress distribute with in mixture is achieved through frictional resistance of coarse particles hence more resistance to deformation while the fine particles fill the spaces in between the coarse particles.

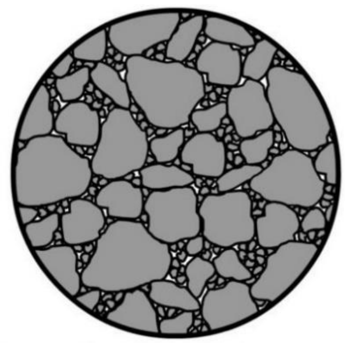


Figure 5: Gap graded HMA

iii. Open-graded HMA

An open-graded layer is a bituminous mixture designed to have a large volume of air voids (typically 18 to 22 percent) to achieve water permeability so that water readily drains through the pavement layer. The Open-Graded mixture require crushed stone or gravel and sand. These mixes can be designed as;

- Permeable pavement: This is where the open graded mix is used as a pavement layer to provide for positive drainage under either a bituminous mix or Portland cement concrete pavement surface.
- Open graded friction course (OGFC): This comprises of a surface course (about an inch thick) placed atop an impermeable dense graded asphalt surface to allow water to drain through it to the underlying layer.
- Asphalt treated permeable bases (ATPB): This is similar to the OGFC but with less restricting specifications and can be used as a drainage layer for dense graded mixture, stone matrix asphalt or Portland cement concrete.

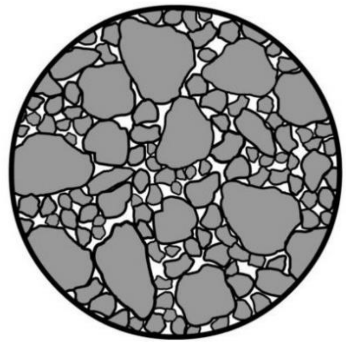


Figure 6: Open-graded HMA

2.3 Pavement design approaches

Pavement design is therefore aimed at economically building road pavements with better performance, which meets the criteria for both structural and functional conditions throughout the design life of the pavement. Functional conditions are aimed at achieving better serviceability in terms of the riding comfort of road users whereas the structural condition is concerned with the structural integrity of the pavement layers. Approaches used for the analysis and design of bituminous mixture include;

2.3.1 Empirical approach

This bases on empirical equations of the measurable pavement characteristics obtained from constructed test sections to predict the response of pavement materials. The observations obtained are used to establish correlations between the pavement design and the pavement performance. This approach does not address the exact cause and effect of the phenomenon and has no firm scientific basis. Henceforth, it can no longer accommodate the current changes and the development of new road materials and the increased demand in road traffic. This approach is however simple to apply and is based on actual data.

2.3.2 Analytical (mechanistic) approach

The analytical approach uses theoretical analysis of the mechanical properties of bituminous materials to assess a designed pavement structure. The analytical approach works by limiting the horizontal tensile strain induced in the pavement layer which determines the resistance to fatigue and the vertical compressive strain at the top of the subgrade which determines the pavement's response to permanent deformation.

2.3.3 Mechanistic-empirical approach

The mechanistic-empirical approach combines the features of both the mechanistic and empirical approaches. This mechanistic-empirical approach deals with two main components: response and performance. The pavement response such as the stresses, strains due to loading and environmental impacts are related to performance of the pavement through empirical distress models. An example is computing the tensile strains with in the pavement due to an applied load by using linear elastic mechanistic models. The computed strain is the related empirically to the accumulation of fatigue distresses within the pavement. A rough estimate of the pavement performance can then be obtained by developing an empirical relationship between the response and the rate of deterioration hence the damage caused by different combinations of loading and environmental conditions can be determined.

2.4 Performance of flexible pavements

In flexible pavements, tensile and compressive stresses induced within the pavement due to loading by heavy vehicle loads decrease with increasing depth. This phenomenon is basis for the gradation of materials where the relatively strong and costly asphalt layer is used as the surfacing and the less strong and cheaper layers used for the base and sub base. The distribution of loads with in the pavement structure is highly dependent on the aggregate interlocking capability and particle frictional resistance and cohesion stability in terms of stiffness of the layers. The layers are arranged with in descending order of structural strength with highest load bearing capacity material (most expensive) on top and the lowest load bearing capacity material (least expensive) at the bottom. The induced stresses within the pavement layers become smaller with the increase in depth of the pavement structure due to distribution of the induced stresses throughout the pavement layers as shown in fig 2.1 above.

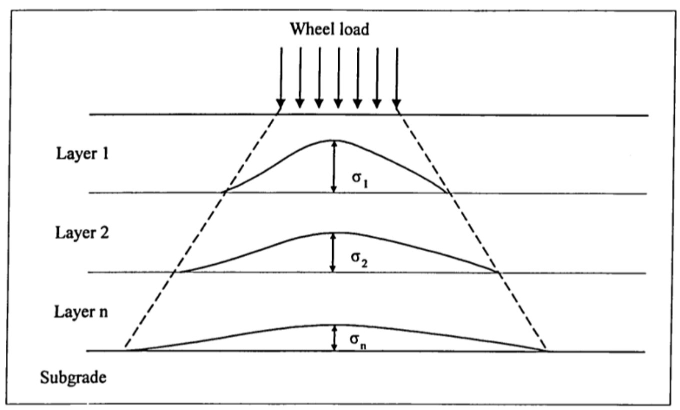


Figure 7: Load distribution in a flexible pavement

2.4.1 Failures in flexible pavements

Failures in flexible pavements can be classified into four major damage mechanisms namely; cracking, rutting, moisture damage, and age hardening.

Cracking

Cracking in bituminous pavements can be caused by thermal induced stresses under severe climatic conditions or by fatigue under repeated loading. Based on the causes of the cracking, it can be categorized as;

i. ***Thermal cracking***; this arises from asphalt being subjected to a single thermal cycle where the temperature drops to a critical low. Thermal volumetric stresses induced above this critical point can lead to cracking when the bituminous binder fails to flow rapidly enough to alleviate stress while accommodating the significant tensile strains caused by temperature fluctuations. Failure arises when these thermally induced stresses surpass the material's tensile strength. Typically, this mechanism manifests as:

- **Transverse cracks**: this manifest across the full width of bituminous pavements at regular intervals.
- **Longitudinal cracks**: These occur parallel to the centerline of bituminous pavements.
- **Block cracks**: These take place in the transverse and longitudinal direction of road pavement and are usually caused when the binder becoming too brittle due to high voids that allow oxidation of the bituminous surface.

ii. ***Fatigue cracking***: Fatigue cracking in bituminous pavements is the phenomenon of cracking is caused by tensile strains generated in the pavement due to traffic loading, temperature variations, and construction practices. The cracks can propagate to the surface as one or more longitudinal parallel cracks but under repeated traffic loading, the cracks connect and develop a pattern like the skin of alligator, therefore it is sometimes also termed alligator cracking. Alligator cracking occurs only in areas that are subjected to repeated traffic loading.

iii. ***Reflective cracking***: This is one of the main causes of premature pavement deterioration occurs when a layer of bituminous material is placed on top of a discontinuous base. Reflection cracking is produced by either traffic or thermally induced stresses, which may be initiated because of already existing cracks or joints in an underlying pavement.

Moisture damage

This usually involves the segregation of the binder film from the aggregate surface in the presence of water causing loss of cohesion and adhesion failure between the binder and aggregates bond. This results in a reduction of the strength and stiffness of the mixture hence reducing the ability of the pavement to withstand traffic induced stresses and strains. Moisture damage can occur in two major ways as; stripping and raveling.

- **Stripping**: This is caused by presence of moisture and is characterized by the loss of adhesion between the aggregates and the binder. This usually begins at the bottom of the bituminous layer and progresses upwards causing bleeding and resulting in reduction of cohesion in the stripped layer and instability in the upper part of the layer due to excessive bitumen (Scholz, 1995).
- **Raveling**: This is the wearing away of the pavement surface caused by factors which include; insufficient binder content that causes poor coating in aggregates thereby reducing adhesion between the binder and aggregates, insufficient amount of fine aggregates to hold coarse aggregate particles together, insufficient compaction that can lead to high void content which accelerates age hardening.

Permanent deformation

Permanent deformation or rutting propagates as surface depressions generally in the wheel path as a result of traffic loading. The first stage of deformation is characterized by a small amount of deformation that occurs in bituminous surfaces due to densification under traffic hence referred to as secondary compaction. Permanent deformation becomes more significant as the cumulative number of traffic loading increases causing major structural failures. There are three main types of permanent deformation namely;

i. **Wear rutting**: This occurs in the wheel path, which is caused by, combined environmental and traffic conditions that lead to the progressive loss of coated aggregate particles from the pavement. This could be caused by;
   - poor mix properties such as; low bitumen content, less durable aggregates, inappropriate mixture design
   - inadequate compaction
   - studded tyres

   This can be reduced by use of durable aggregates and good mixture design.

ii. **Structural rutting**: This is caused by permanent deformation of pavement structure due to repeated traffic loading and is majorly associated to permanent deformation in the subgrade. Structural rutting can be caused by:
   - poor drainage
   - overstressing pavement layers due to excessive traffic loading
   - poor construction practices such as inadequate compaction

- contribution from other pavement distress like fatigue cracking, stripping

Structural rutting can be prevented by providing good drainage, adequate pavement design, construction quality control and selection of good quality materials.

iii. **Instability rutting or plastic deformation**: This occurs within the asphalt layer due to lateral (shear) displacement of a single bituminous layer along the wheel path. Instability rutting can be caused by:

- Adverse hot temperature
- Severe loading conditions e.g. heavy loading, increased tyre pressure, increased axle loading, slow speeding heavy traffic.
- Mix design and material properties e.g.; aggregate and binder characteristics, air void content in mixture, selection of a suitable mixture design.

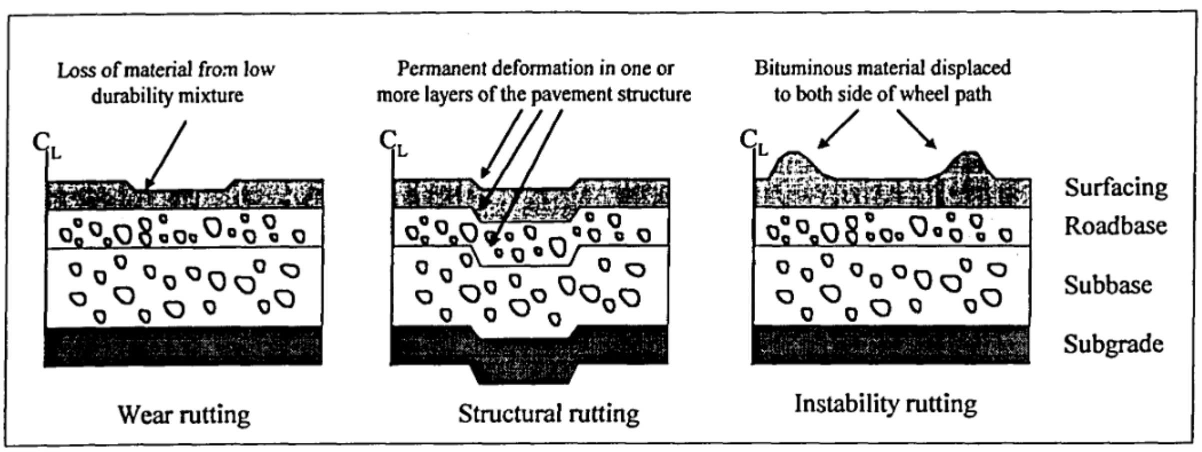


Figure 8: Forms of rutting in asphalt

The use of highly stiff binders can help to reduce instability rutting. This research was focused on the plastic deformation (instability rutting). It was intended to study the effect of polymer modified binder in resisting plastic deformation developed along climbing lanes due to slow moving traffic which reduces loading time on the pavement and increase in temperature that softens the binder hence causing shear deformation.

2.5 Deformation in asphalt mixtures along climbing lanes

Climbing lanes sections are characterized by a decrease in average speed as the vehicles navigate along the longitudinal slope hence causing an increasing in the loading time on the pavement (Chilukwa, 2019). In tropical regions like Uganda, increase in temperature causes the binder to soften hence reducing the stiffness of the asphalt mixture. This reduction in stiffness coupled with the increasing loading time leads to plastic deformation within the asphalt layer. According to (Transport Research Laboratory , 1993), asphalt mixtures designed for such severe sections need to be designed appropriately and with stringent specifications as compared to well channeled sections or flat sections in order to withstand the induced loads and shear stresses.

2.5.1 Asphalt mix variables to resist permanent deformation

Rutting resistance to permanent deformation is a combined effort between the asphalt binder properties, aggregate properties, aggregate size proportioning, volumetric requirements and compaction. Asphalt mix design procedures take in to consideration the variation of constituents of an asphalt mixture to improve its resistance to permanent deformation as highlighted below.

Aggregates

Aggregates should be strong enough to resist crushing when subjected to traffic loads. Weak aggregates tend to undergo wearing and abrasion which could lead to permanent deformation. Higher coarse aggregate content in the asphalt mixture can be used as to increase the aggregate interlocking capability in the mixture. According to (Transport Research Laboratory, 2022), mix specifications that may work for flat and well channeled sections may not be sufficient for climbing lanes and intersections. An aggregate grading specification highlighted in table 2.1 BC2 (base course 2) is suggested to be used for such severely loaded sections to increase the resistance to permanent deformation. The effect of particle shapes and textures is also an important in the performance of asphalt mixtures. The more angular shape and rougher surfaced aggregates are preferred due to their good interlocking capabilities due to increased friction. The super pave mix design specifies requirements for coarse and fine aggregate angularity and flakiness index to ensure good interlocking is met in such severe sections as shown in table 2.

Table 1: Recommended grading (BC2) for wearing courses along climbing lanes (TRL, 1993)

Mix designation	WC3	WC4	BC2
	Wearing Course (5)		Basecourse
BS test sieve (mm)	Percentage by mass of total aggregate passing test sieve		
28	-	-	100
20	100	-	95 - 100
14	95 - 100	100	65 - 85
10	70 - 90	95 - 100	52 - 72
6.3	45 - 65	55 - 75	39 - 55
3.35	30 - 45	30 - 45	32 - 46
1.18	15 - 30	15 - 30	-
0.3	-	-	7 - 21
0.075 (1)	3 - 7	3 - 7	2 - 8
Bitumen grade (2) (pen)	80/100 or 60/70	80/100 or 60/70	80/100 or 60/70
Bitumen content (3) (per cent by mass of total mix)	5.3 ± 0.5	5.5 ± 0.5	5.0 ± 0.6
Thickness (4) (mm)	40 - 55	30 - 40	50 - 80

Table 2: Super pave aggregate requirements to ensure good interlocking properties

Test	ASTM/ AASHTO Designation	Super pave requirements
Coarse aggregate angularity (CAA)	ASTM D-5821	Min. 95
Fine aggregate angularity (FAA)	AASHTO T-304	Min. 45
Flakiness index	ASTM D-4791	Max. 10

Bitumen

Bitumen acts as a lubricant during compaction and as a viscoelastic binder which binds the aggregates and filler material together. Due to thermal induced effects, temperature fluctuations could lead to softening of the binder at higher temperatures which results into deformation under traffic loads. Low temperature cracking may also occur due to rapid contraction when temperature drops significantly inducing thermal stresses in the HMA which may exceed the tensile strength of asphalt hence leading to cracking. Rheological properties of the bitumen are essential in analyzing the properties that govern way in which bitumen will deform or flow. The empirical approach as described in section 2.1.2 above employed index parameters such as softening point, penetration, ductility to describe the rheological performance the asphalt. These properties are however single point measures that may hardly predict the overall performance at varying operating conditions. In order to advance the modelling of asphalt, rheological derivatives have been developed by researchers to derive mechanistic performance measures that relate index properties to the pavement performance under service conditions. According to (Richard, 2009), these pseudo rheological properties were subdivided into; temperature susceptibility and shear susceptibility which relates to the loss of stiffness hence causing shear flow.

Temperature susceptibility

This relates to the consistency of the binder with variations in temperatures. All bituminous binders display temperature susceptibility properties such as softening when heated and hardening at low temperatures. Various researchers have carried out studies to relate the index properties of the binder in terms of its temperature susceptibility so as to predict how the asphalt mixture will perform under service temperature variations. The parameter that relates to temperature susceptibility is the penetration index.

Several researchers came up with approaches that combine different consistency measures relating them to temperature susceptibility. Some of these are shown below.

Penetration Index (PI)

An equation that relates the penetration ad softening point of bitumen to temperature susceptibility was developed by (Pfeiffer & Van, 1936) which presumes that for road paving binders the penetration index should be as close to zero as possible. The approach was introduced as the penetration index (PI) with the formulation that suggests that; if the logarithm of penetration, P, is plotted against temperature, T, a straight line is obtained such that:

$$\log P = \mathrm{AT} + \mathrm{K}$$

Where;

A is the temperature susceptibility that ranges from 0.0015 to 0.06

P is thepenetration at temperature T, and K is a constant

An equation for the temperature response of bitumen that assumes a value of zero for road bitumen was developed which defines the penetration index as;

$$PI = \frac{20(1 - 25\mathrm{A})}{(1 + 50\mathrm{A})}$$

The penetration index (PI) in terms of penetration and softening point can be obtained from the equation 2.1 below after obtaining the penetration of about 800 deci mm which was found to be the penetration at ASTM softening point temperature for most bitumen.

$$PI = \frac{1951.5 - 500\ \mathrm{logPen} - 20 * \mathrm{softening\ point}}{50\ Pen - 52 - 120.1} \quad \text{(Equation 2.1)}$$

The PI of each bitumen can be calculated using equation 1. According to (Pfeiffer & Van, 1936), the PI values ranges from -3 for high temperature susceptibility to around

+7 however for road paving application this was reduced to (-2 to +2). Low PI values are an indication of high temperature susceptibility. Low PI values below 0 indicate high temperature susceptibility.

Penetration-Viscosity Number (PVN)

Another approach related the penetration and viscosity of bitumen and used it to assess the temperature susceptibility of bitumen (Vincent Janoo, 1993). This led to the development of the penetration viscosity number which is based on the penetration at 25^0C and viscosity at 60^0C. The formula developed for obtaining the penetration viscosity number shown below was developed (Chang, 2019) (Kar, et al., 2019).

$$PVN = \frac{6.491 - 1.5 \log Pen\ 25 - \log \cap 60}{1.05 - 0.22 \log Pen\ 25} \quad \text{(Equation 2.2)}$$

Where;

Pen25 is penetration at 25^0C

$\cap\ 60$ is the viscosity at 60°C

Viscosity Temperature Susceptibility (VTS)

Another approach that used the viscosity temperature susceptibility (VTS) value as a measure of temperature susceptibility for bitumen sample was developed by (Traxler & Schweyer, 1936). The VTS value obtained using the equation;

$$VTS = \frac{\log(\log \cap 1) - \log(\log \cap 2)}{\log T1 - \log T2} \quad \text{(Equation 2.3)}$$

Where;

∩1 is the viscosity at T1

∩2 is the viscosity at T2

T1 and T2 are the temperature limits

Viscosity of bitumen was related to temperature via a graph of slope of log viscosity against temperature. The model has been effective over the years in the determination of mixing and compaction temperatures for asphalt mixture. The model developed was summarized in a graph as shown below which is used to derive mixing and compaction temperatures of asphalt mixtures.

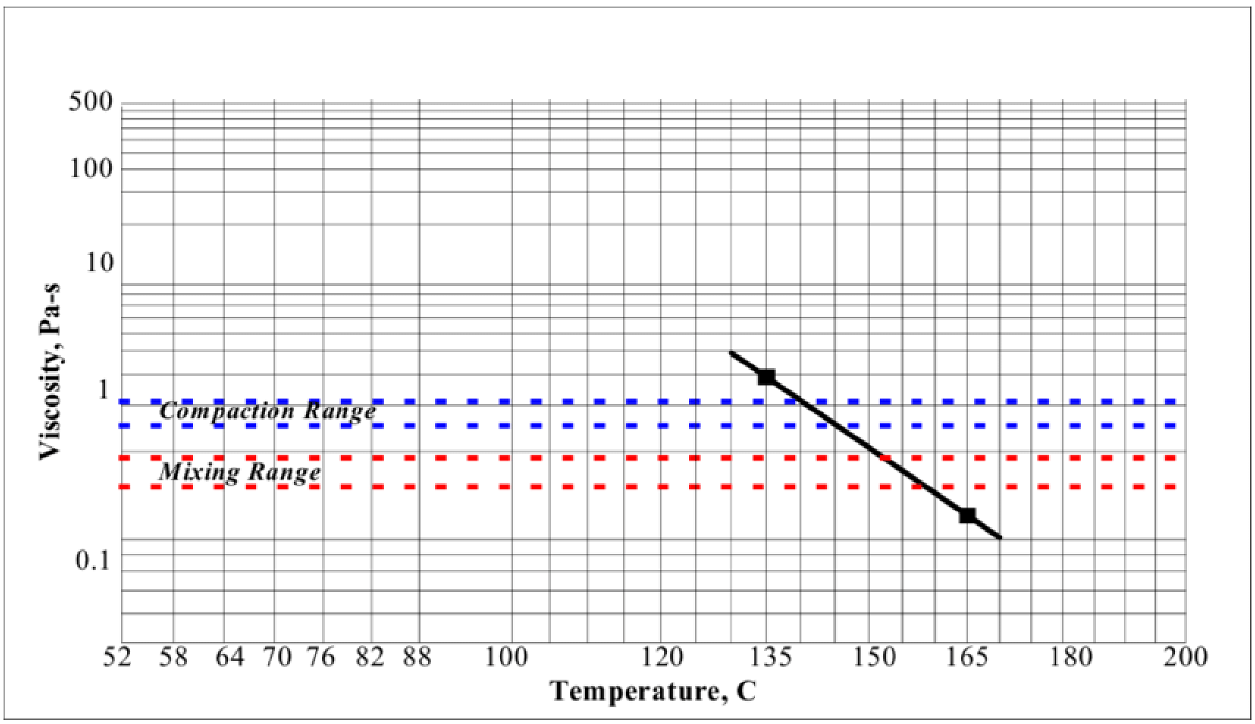


Figure 9: Variation of viscosity with temperature.

Bitumen stiffness modulus

Bituminous materials are rheologic in nature and portray Newtonian behavior at high temperatures. This means that their stress-strain relationship is dependent on both loading time and temperature. According to (Richard, 2009), the stiffness modulus is an essential parameter in determining the shear susceptibility of asphalt binders which impacts on the shear flow under induced traffic load and thermal stresses. This relationship is referred to as the stiffness modulus of the mixture by many researchers and it is and indicator of the elastic modulus or the ability to resist permanent deformation (Boris Radovskiy, 2018). The stiffness modulus of bituminous mixtures can either be measured using sophisticated laboratory equipment or predicted from the asphalt mixture properties of the aggregates and bitumen. The most commonly used and recommended stiffness prediction model by many researchers was developed by (Van der Poel, 1975). According to the prediction model, stiffness modulus of the bitumen is obtained from the penetration index, the softening point temperature, operating conditions of pavement temperatures, and loading time. The prediction model was summarized in form of a nomograph usually referred to as Van der Poel's nomograph. This has been used by many researchers to predict the resilience of the asphalt mixture to permanent deformation as explained further in section 3.7.1.

Climbing lane road sections are characterized by deceleration of traffic which leads to reduced traffic speeds hence inceasing in the loading time. In tropical areas like Uganada, the high temperatures can cause the binder to soften if its temperature susceptibility is high thereby reducing the binder stiffness modulus. This can consequently result into permanent deformation that exhibits as rutting. According to

(Transport Research Laboratory , 1993), it is recommended that bitumen with low temperature sensitivity and good resistance to hardening should be selected. Further suggestion by (Harun & Jones, 1992) suggests that for such severe locations, the use of bitumen modifiers is often advantageous to improve the rheological properties of the bitumen. Asphalt modification can therefore be done to improve the properties of the bitumen and the asphalt concrete used along such severe sections like climbing lanes.

2.6 Modification of asphalt mixtures

The significant rise in pavement distress has drawn attention to the critical challenges within road paving technology, spanning from material design to application techniques. Increased traffic loading has accelerated asphalt pavement damage, reducing road service life and escalating maintenance and repair costs. Enhancing aggregate and binder properties can improve asphalt mixtures' resistance to deformation. However, bitumen, being viscoelastic, undergoes property changes under adverse weather conditions like high temperatures and severe loading. Stiffer mixes, characterized by highly viscous, low penetration bitumen such as 35 pen grade, tend to become brittle at low temperatures, leading to susceptibility to cracking. Conversely, softer bitumen is effective in preventing cracking but struggles to withstand heavy loads at high temperatures. Therefore, modifying bitumen presents an alternative, cost-effective solution when an asphalt mix fails to meet permanent deformation requirements.

2.6.1 Reasons for modifying bituminous binders.

- To obtain softer mixtures to reduce cracking at low service temperatures and stiffer mixes to reduce permanent deformation at high temperatures.
- To increase structural strength especially for bituminous mixtures where the strength of the mixtures relies on the stiffness of mortar binder thereby requiring polymer modification.
- To improve workability and compaction in adverse weather conditions, such as strong winds and low temperatures, where road construction is a problem to extend the temperature range for mixing, laying, and compaction.
- To allow for thicker binder films on aggregate to gain better workability and fatigue resistance while achieving sufficient stiffness to resist permanent deformation.
- To improve bonding and to reduce stripping of bitumen and aggregate that would cause loss of adhesion between binder and aggregates, especially in the presence of moisture by addition of polymers reduces moisture damage potential.
- To reduce bleeding since increase in the stiffness at high temperatures, consequently reduces the bleeding potential
- To improve resistance to ageing or oxidation by addition of antioxidants into the mixtures hence improving the durability of bituminous mixtures.
- To reduce structural thickness of road pavement layers by addition of polymers to meet the same strength criteria as the road pavement structure without polymer modification

2.6.2 Properties of polymer modified binders

To achieve effectiveness in the use of modified bitumen (Iswandaru, 1998) suggested that following considerations be taken when choosing the modifier to use;

- The polymer to use should be readily available.
- It should have good solubility when blending with bitumen to avoid phase separation.
- Should resist degradation at asphalt mixing temperatures.
- Should improve resistance to flow at high road temperatures affecting workability while laying or making it too stiff and brittle at low temperatures

Additionally, in order to optimize effectiveness, the modified binders should be used in combination with high quality aggregate especially in regions, where exceptional resistance to permanent deformation is needed. The use of mechanistic-empirical design approaches is more suitable for polymer-modified asphalt.

2.6.3 Classification on bitumen modifiers

The Strategic Highway Research Program (SHRP) researchers (Jones & Kennedy, 1991) classified the modifiers into six groups that are based on the mode of addition as;

- Mineral fillers (dust, lime, portland cement, carbon black, sulphur)
- Elastomers (Styrene Butadiene Rubber (SBR)
- Plastomers or thermoplastics (polyethylene, polypropylene, Ethylene Vinyl Acetate (EVA))
- Oxidants (manganese and other metal salts)
- Antioxidants (lead compounds, carbon, calcium salts)

- Hydrocarbons (aromatic oils)

This study will focus only on thermoplastics/plastomers. Thermoplastic polymers soften on heating and harden on cooling which is similar to bitumen. The commonly used thermoplastic polymers are polyethylene, polypropylene, polyvinyl chloride, polystyrene, and ethylene vinyl acetate because of their flexibility at low temperature, and hence they are not brittle.

2.6.4 Polyethylene as a bitumen modifier

Polyethylene is a polymer derived from polymerization of the ethylene monomer. Polyethylene is also widely available as both virgin and recycled materials (Jew & Svazic, 1986). When polyethylene is blended with bitumen, at the temperature when the bitumen matrix becomes soft and deformable (temperature near 50°C), the dispersed polyethylene particles assume the characteristic of a filler hence increasing the resistance to flow (viscosity) and consequently shear deformation. Polyethylene is classified as; High Density Polyethylene (HDPE), Low Density Polyethylene (LDPE), and Linear Low Density Polyethylene (LLDPE). These copolymers usually melt in the temperature range between 100°C and 130°C and exhibit low glass transition temperature (around -120°C). This indicates the high flexibility at low temperature that can improve the toughness and ductility of the base bitumen at low temperatures

2.6.5 Material performance of polymer modified mixtures.

The main performance criteria for flexible pavements considers resistance to fatigue and to permanent deformation. However, it should also be noted that modified asphalt mixtures should exhibit advantageous properties that do relate to the durability and deformation of the pavement. Therefore, it is necessary for modified asphalt mixtures to satisfy the following properties;

- Good tolerance to adverse weather conditions
- Good resistance to cracking
- Good resistance to ageing
- Good resistance to effects of water
- Good resistance to permanent deformation

2.6.6 Blending of polymer modified bitumen

Blending of polymers with bitumen is a crucial aspect since it is not simple to achieve a good blend between polymer and bitumen. The major parameter considered here is the phase of dispersion for the polymer to be dispersed properly in a bitumen to cause swelling.

An investigation by (Tanguy, 1999) studied the extent of swelling of different bituminous binders modified with SBS copolymer. The extent of swelling was independent of temperature over the 80-160°C range, however the rate decreased as the polymer content increased. They value of polymer content beyond which a polymer was no longer swollen was recorded and termed the Colloidal Instability (Cl) index. The colloidal instability index is a parameter used to characterize the state of dispersion and is denoted as:

$$\text{Colloidal Instability Index} = \frac{\text{asphaltenes} + \text{saturated oils}}{\text{resins} + \text{aromatic oils}} \quad \text{(Equation 2.4)}$$

A study by (Locher, et al., 2006) showed that a colloidal index is associated with a reduced critical polymer concentration (CPC), resulting in a non-uniform structure, diminished cohesiveness in tensile strength, and a decrease in the maximum phase angle. Conversely, a lower colloidal index corresponds to an elevated CPC, fostering a finely dispersed structure with heightened cohesion energy and maximum phase angle. Figure 10 demonstrates that the most favorable rheological performance of polymer-modified binders occurs at a colloidal index of 0.3, corresponding to a polymer concentration of 6 to 8%.

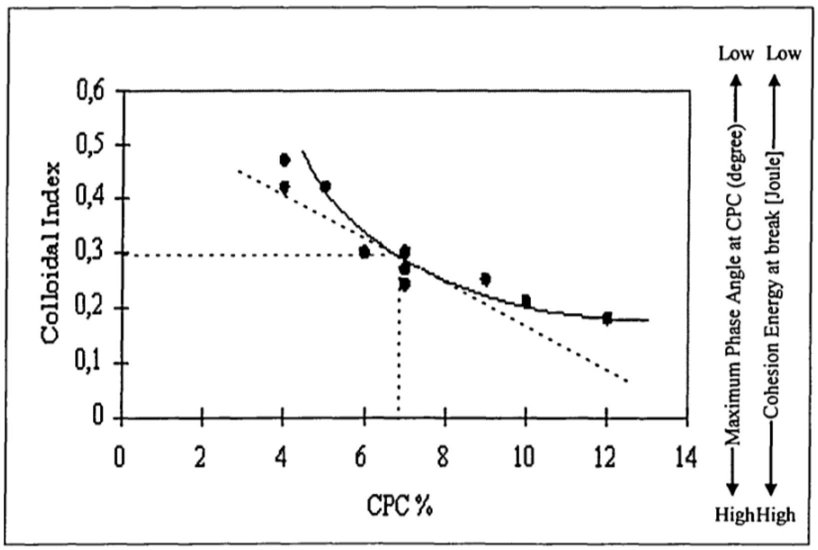


Figure 10: Polymer concentration against colloidal index

The above study helps us analyze that blending of thermoplastics with bitumen can be done by varying the polymer concentration between 2% to 10% where the colloidal index remains constant.

A study by (Sabzoi Nizamuddin, 2020) evaluated the suitability of LLDPE in the road sector, comparisons were made between base bitumen and LLDPE-modified bitumen blends through physical, chemical, rheological, and thermal assessments. It was observed that higher concentrations of LLDPE increased viscosity and raised the softening point from 0.62 Pa s and 44.1°C to 5.75 Pa s and 122.3°C, respectively. Penetration values, however, decreased from 59.3 to 14.3 (0.1mm). Thermo gravimetric analysis (TGA) indicated that the modified bitumen exhibited less evaporation and higher thermal stability compared to the base bitumen. Rheological evaluation illustrated how the addition of LLDPE significantly impacted the bitumen's thermo susceptibility, enhancing resistance to permanent deformation at high temperatures and elastic recovery. Overall, the study findings suggest that correctly dosed LLDPE, obtained from homogeneous sources, can improve asphalt's resistance to permanent deformation.

The mechanical behavior of Low-Density Polyethylene was examined by (Jessica Adaluz, 2022). Asphalt cement (AC) was modified using LDPE residue at 5%, 7%, and 10% of the AC mass via the wet method. Evaluation of the physical-rheological properties revealed that 5% LDPE yielded the best performance. ANOVA analysis was conducted to determine if the changes in mechanical response compared to the control mixture were statistically significant. Overall, the findings suggest that LDPE-modified mixtures are suitable for thick layers in high-temperature climates to mitigate rutting.

2.7 General discussion

From the vast literature above, several deductions were derived which shaped the direction and proceedings of this research study. Some of the major aspects to be noted include;

The empirical approach of pavement design cannot be relied on alone to determine the performance of the pavement. Therefore mechanistic-empirical approach was used where the index properties of the bituminous mixtures obtained from empirical laboratory tests are related to rheological derivatives that have been developed by researchers to derive mechanistic performance measures of the pavement performance under service conditions. Analysis of the rheological derivatives and performance prediction.

Linear low-density polyethylene was dosed in percentages from 2% - 10% since the colloidal stability index studied by (Locher, et al., 2006) showed the colloidal stability index remained constant below polymer concentration of 2% - 10%. This research will be focused on ensuring that the modified asphalt mixture achieves resistance to permanent deformation caused by shearing stresses and the laboratory assessment of the performance of bituminous mixtures will be conducted.

The asphalt mixture obtained from the modification of bitumen with linear low density polyethylene was tested to assess its potential in the improving the resistance of the asphalt mixture against permanent deformation without impacting on the durability of the asphalt mixture.

CHAPTER THREE: METHODOLOGY

3.1 Introduction

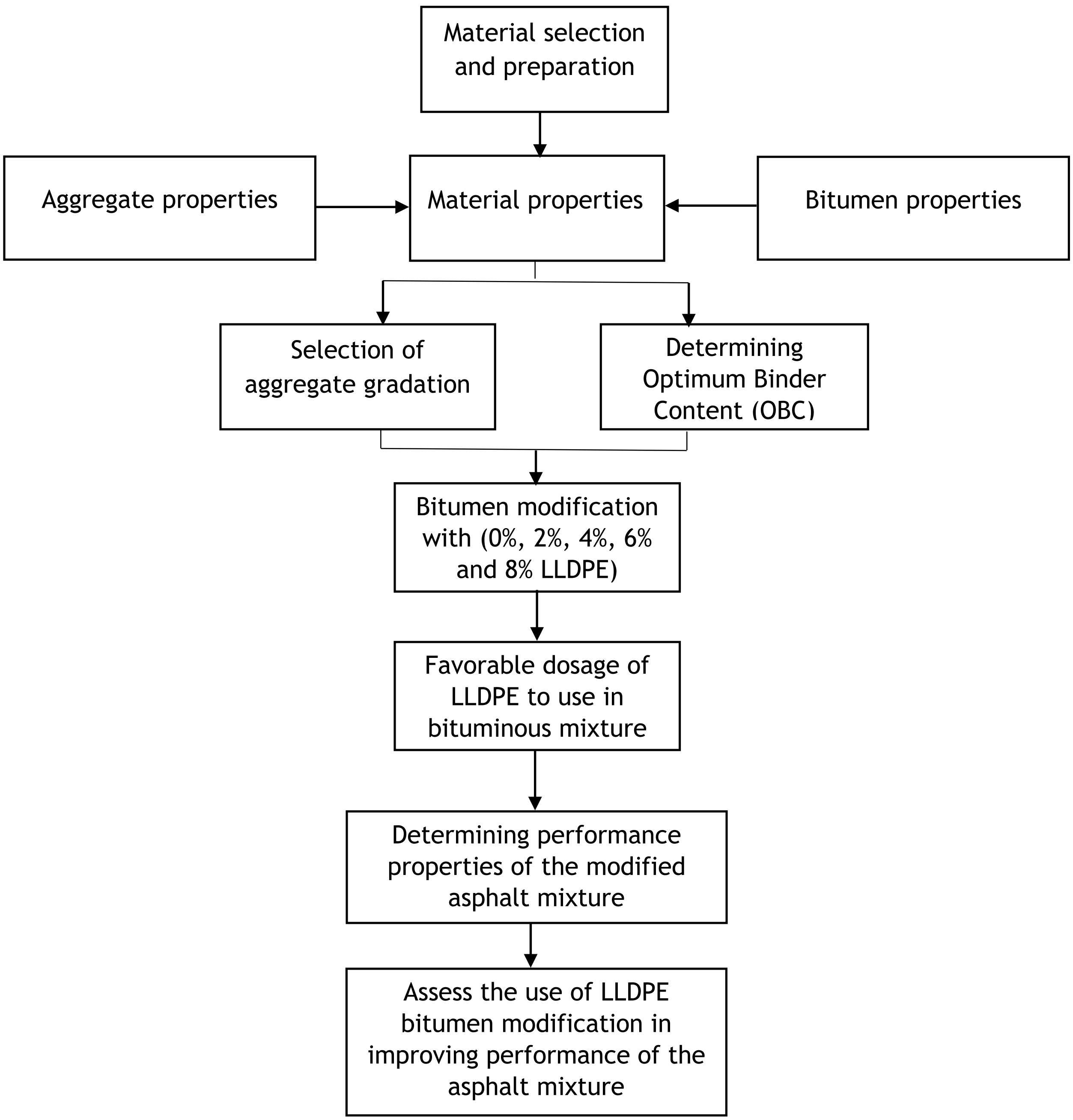
This chapter describes the methods and procedures that were used to collect and analyze the data so as to achieve the objectives of this research study. This involved visual inspection to ascertain the condition of the pavement, conducting laboratory tests to come up with meaningful data relating the research to the cause of premature deterioration of the pavement. Based from previous researchers, several techniques were borrowed to aid in the relation of the empirical data obtained from the index laboratory tests to the mechanistic behavior of the pavement. The information obtained was used to analyze and assess the possibility of improvement in resistance against permanent deformation in the bituminous mixtures.

3.2 Research approach

The approach used to conduct the research was a quantitative approach which involved experimental based studies to come up with a cause and effect relationship. This was attained by conducting experimental tests on the aggregates sourced from the same environment where the aggregates used at the geographical scope were attained. The same grade of bitumen used for the construction of the road was also subjected to experimental tests. These helped in ascertaining the most possible cause of the premature deformation at the road section. A solution aimed at modifying the binder used in the asphalt pavement was suggested which comprised the basis of this research. Descriptive and correlational analysis was conducted to relate the results obtained from experimental tests conducted on the modified asphalt specimen to the possibility of improving the resistance of the asphalt against permanent deformation.

3.3 Research methodology

This section presents the sequence of methods that aided the execution of this research. This was presented in form of a flow chart as shown below.



3.4 Material selection and preparation

The selection of materials for the asphalt mix design is a crucial aspect in asphalt mix design. The scope of this research study is a climbing lane section which classified as severely loaded road section according to (Transport Research Laboratory , 1993).This therefore necessitates the need for material selection to be conducted while ensuring that the resistance to permanent deformation of bituminous mixtures is improved by considering factors such as;

- Aggregates with coarse surface texture, angular shape to improve the inter-particle frictional resistance so as to reduce on the plastic deformation under heavy slow moving traffic.
- Binder with high stiffness and low temperature susceptibility to resist shear flow under high temperatures and longer loading time.
- Considering aggregates maximum nominal sizes (AC 20) and proper gradation and mix design procedure to ensure that the asphalt mixture has sufficient airvoids, VMA content, and VFB capable of resisting deformation while ensuring the durability of the asphalt mixture is not compromised.

3.4.1 Aggregates selection

The Virginia Asphalt Association recommends that road sections with an expected increase in traffic loading (severely loaded sections such as climbing lanes), the aggregates and aggregate blending must meet higher standards (Virginia Department of Transportation, 2018). This therefore requires tough and durable aggregates that are strong enough to withstand heavy loads to avoid disintegration and crushing whichwould lead to pavement failure. The aggregates selected for this study were of granite rock deposits sourced from Wankoba quarry site located in Kasenge parish in Nama sub-county Mukono district. The source of the aggregates was close to Halai quarry site in Namuyenje parish which was the source for the granite rocks used for the construction of the Bweyogerere to Jinja road section which is the scope of this researchstudy. The quarry was under the management of Stirling engineering company limited. The aggregates were acquired after having been crushed and graded in different hot bins of sieve sizes ranging from; 0-6, 6-10, 10-14 and 14-20-mm. Granite aggregates were selected for this study basing on their good geological properties such as good hardness and durability, roughness and angularity, and less hydrophobic nature make it good for use in road construction. Before using the aggregates, they were tested toensure that they meet the necessary requirements for use in bituminous mixtures.

Figure 11: Acquiring aggregates from Wankoba quarry site

3.4.2 Binder selection

Whereas the aggregate structure provide the strength and toughness of the asphalt mixture, the bitumen binds the pavement together and gives additional stiffness. The grade or type of binder used is therefore important in influencing the properties of the asphalt mixture. The binder used for this study was classified as penetration grade

60/70 based on paving grade classification (ASTM-D946, 2020). The penetration grade relates to the stiffness of the asphalt which is crucial in improving the capability of the asphalt mixture to resist deformation hence requiring a highly stiff binder (lower penetration grade). The use of 40–50 and 60-70 pen bitumen (based on ASTM classification) is recommended in tropical regions since it provides a suitable compromise between workability, deformation resistance and potential hardening (Transport Research Laboratory , 1993). The bitumen was acquired from Stirling Civil engineering Ltd laboratory in Nama sub-county Mukono district.

Figure 12: Acquiring bitumen sample

3.4.3 Linear low density polyethylene

The Linear Low-density polyethylene used for this study was acquired from Africa Polysack Industry Limited which is located in Namanve industrial area. The material was classified as linear low density owing to its density of 0.917 g/cm^3 that lies within the range of 0.915 – 0.925 for all linear low density polyethylene plastics as recommended by requirements for low density polyethylene and linear low density polyethylene by IS 3395:1997 (Bureau of Indian Standards, 1997) since Uganda does not have standards for the material.

Figure 13: Linear-Low Density Polyethylene used for bitumen modification

3.5 Index properties of materials in the asphalt mixture.

Index properties a set of fundamental properties that determine the physical and mechanical properties of the aggregates. These properties are crucial in ensuring the materials used to prepare the asphalt mixture possess good physical and mechanical properties for effective performance of the asphalt mixture. The tests conducted to characterize the index properties of the materials are explained in this section.

3.5.1 Aggregate tests

The aggregates were tested to ensure they conform to they possess the required characteristics for use in the bituminous mixtures. The tests carried out to ensure that the aggregates were good enough to be used for the study are highlighted below with the test methods used.

i. Aggregate Crushing Value (ACV)

The Aggregate Crushing Value test was performed to determine the resistance of aggregates to crushing under gradually applied compressive load. This was done by subjecting the aggregates to a gradual load of 400kN and the ACV determined by measuring the material passing 2.36mm sieve in accordance to **BS 812: Part 110:1990.**

Sample preparation

The test sample used consisted of a riffled sample of 8 kg aggregates sized from 14 mm to 10 mm. The sample was divided in to 3 approximately equal samples of aggregates placed on separate trays. Each of the samples was heated at 105 ± 5^0C for about four hours to ensure that all the moisture is eliminated then cooled at room temperature before testing.

Test Procedure

- The first sample of aggregates was poured in to a cylinder in 3 layers of equal approximate depth with each layer tamped with 25 blows by allowing the tamping rod to fall freely from a height of about 50 mm or more.
- The excess material was removed and surface was carefully levelled using the straight edge of the tamping rod. The weight of the cylinder and aggregates were measured and recorded as the mass of the test specimen used.
- The specimen was the transferred into the test mold in the same manner as for the cylinder, the surface levelled and the plunger placed on top so that it rests horizontally on the surface.

- This assembly was placed between the plates of the testing machine loaded at uniform rate so that the total load of 400kN was reached in 10 minutes.
- The load was then released and the crushed material poured on to a tray, weighed and its mass recorded as $M1$.
- The material was sieved through a 2.36 mm sieve and the weight of the sample passing (M2) through and that retained (M3) recorded.
- This procedure was repeated for the second and third sample using the same sample mass.

Computation and recording of results

The ACV was obtained from the equation below as a percentage.

$$ACV = \frac{M2}{M1} \times 100\ \% \text{ (Equation 3.1)}$$

Where;

- M2 – Mass of material passing through 2.36mm sieve
- M1 – Mass of test specimen

The ACV obtained from each of the three samples was used to obtain the average. This was recorded as ACV of the aggregates recorded as a percentage with a standard deviation.

ii. Aggregate Impact Value (AIV)

The aggregate impact value test was performed to determine the resistance of the aggregates to sudden shock or impact. The test was be performed for a fraction of aggregates passing a 14mm sieve and retained on 10 mm sieve in dry condition. The test was performed in accordance to BS 812: Part 112:1990.

Sample preparation

The test sample used consisted of a riffled sample of 2 kg aggregates from the aggregate batch of 14 to 10 mm. the sample was divided to 3 approximately equal samples of aggregates. Each of the samples was heated at 105 ± 5 ^{0}C for about four hours to ensure that all the moisture is eliminated then cooled at room temperature before testing.

Test Procedure

- The first test sample was put in the cylindrical steel cup in three equal layers and tamping done (25 blows) for each layer evenly on the aggregate using a tamping rod falling freely from a height above 50 mm above the surface of the aggregate.
- The surplus aggregates were removed by rolling the tamping rod across the top ofthe steel cup.
- The net mass of the aggregate was measured and recorded (M1) and with the cylindrical cup of the impact machine fixed firmly in position on the base of the machine, the whole test specimen was placed in the cup.
- The hammer was raised to the top of the hammer guide columns and allowed to fall freely on to the aggregate to give 15 such blows.
- The crushed specimen was removed from the cup, weighed and its mass recorded as M1.
- The sample was then be sieved through a sieve of 2.36 mm, weight of sample that passing and that retained recorded as M2 and that retained as M3 respectively.
- The same procedure was repeated for the other two samples for consistency.

Calculation and recording of results obtained

The AIV was then determined for each sample using the formula below.

$$AIV = \frac{M1-M2}{M1} \times 100\% \text{ (Equation 3.2)}$$

The procedure was conducted for the other two samples to obtain two sets of results. The AIV of the aggregates was obtained as an average of the three results obtained from the three sample and recorded with the standard deviation.

iii. Los Angeles Abrasion Test

This test was selected to assess the aggregate resistance to wear and abrasion when used as a wearing course. The Los Angeles value is a measure of degradation of mineral aggregates of standard grading resulting from a combination of actions such as abrasion, impact and grinding in a rotating steel drum containing a specified number of steel spheres to create abrasive action when rotated in a drum for a specified number of revolutions. The procedure was done in accordance to ASTM C131 – 89.

Sample preparation

A sample of the aggregates was sieved through sieves 20, 12.5 and 10 mm to attain two sample sets each weighing 5 kg conforming to the grading B shown in table 3.1.

The grading was selected such that it conforms to AC 20 which was used in this study. The sample specimen was dried in oven at 105° – 110°C for 4 hours then left to cool at room temperature. The grading type B used the test was selected based on tables 3 and 4 below; since aggregates used were for the test were 14 – 10 grading type B was chosen.

Table 3: Selection of the grading type for the LAAV test.

Sieve size (mm)		Weight of sample (g)						
Passing (mm)	Retained on sieve(mm)	A	B	C	D	E	F	G
80	63					2500		
63	50					2500		
50	40					5000	5000	
40	25	1250					5000	5000
25	20	1250						5000
20	12.5	1250	2500					
12.5	10	1250	2500					
10	6.3			2500				
6.3	4.75			2500				
4.75	2.36				5000			

Table 4: Selection of abrasion charge based on grading type chosen

Grading	Number of steel balls	Weight of charge (g)
A	12	5000 ± 25
B	11	4584 ± 25
C	8	3330 ± 25
D	6	2500 ± 25
E	12	5000 ± 25
F	12	5000 ± 25
G	12	5000 ± 25

Test procedure

- The abrasive charges (11) were selected as per the table shown below which conformed to the grading chosen.
- The aggregates and abrasive charges were placed in the cylinder and the cover fixed.
- The machine was started and rotated at a speed of 30 to 33 revolutions per minute for 500 revolutions.
- The machine is stopped after the desired 500 revolutions and material was discharged to a tray and sieved through the 1.70 mm sieve.
- The material retained on the 1.7mm size was weighed and recorded.
- The procedure was repeated for the other aggregate sample set to attain two sets of results to ensure consistency.

Computation and recording of the results

The percentage of wear (Los Angeles Abrasion Value) from the equation below;

$$LAAV = \frac{(W1 - W2)}{(W1 \text{ X } 100)} \text{ X } 100\% \qquad \text{(Equation 3.3)}$$

Where;

- W1 is the original weight of aggregate sample, (5000g)
- W2 is the weight of the sample specimen retained on 1.70 mm sieve.
- (W1 – W2) is the weight of the sample specimen that passed 1.7mm sieve.

The procedure was repeated for the second sample specimen to obtain two LAAV data sets. The LAAV of the aggregates was recorded as an average of the two data sets which was recorded as a percentage with a standard deviation.

iv. Ten Percent Fines Value (TFV)

This test was selected to determine the resistance of an aggregate to crushing under a gradually applied load. This was done both in the dry and soaked condition. The 10 % Fines Value test involved determining the load required to crush theaggregates of standard size fraction 14 mm to 10 mm to give 10 % material passing the 2.36 mm sieve after crushing. The TFV test is more like the Aggregate Crushing Value (ACV) test, which requires a force equal to 400 kN to be applied on the test sample. This was conducted in reference to BS 812: Part 111: 1990.

Sample preparation

For dry condition

- The sample was reduced to produce a test portion of sufficient mass to produce three test specimens of 14 mm to 10 mm size fraction.
- The test specimens were oven-dried at 105 ± 5°C for 4 hours and allowed to cool beforetesting. Record the mass of material comprising the test specimens.

For soaked condition

- Each of the three test specimens were placed in a separate wire basket and immersed in water while ensuring that the water level was at least 50 mm above the top of the baskets.
- The entrapped air was removed by lifting the baskets 25 mm above the base of the container and allowing it to drop 25 times. The aggregates were immersed for 24 hours to soak.
- The specimen was removed from the basket and surface dried with a towel to

remove the water from the surface of the aggregates.

Test Procedure

- The cylinder was placed i on the base plate the aggregates of the first test sample placed in three layers with each layer being compacted by 25 strokes using a tamping rod
- The surface of the aggregate was carefully levelled and the plunger inserted so that it rests horizontally on this surface.
- The apparatus with the specimen and plunger in position were placed between the plates of the compression testing machine.
- The machine was started and a force applied at a uniform rate to cause a total penetration of the plunger in 10 minutes ± 30 seconds.
- The maximum force (f) reached after the full penetration of the plunger within 10 minutes was recorded.
- The force was released and the crushed material aggregate removed by holding the cylinder over a clean tray of known mass and hammering on the outside with the rubber mallet.
- A wire brush was used to remove the fine particles adhering on to the inside.
- The tray and aggregates were weighed and the mass of aggregate used (M1) was recorded.
- The whole specimen was sieved on the 2.36 mm sieve until no further significant amount passed. The masses of the fraction passing and retained on the sieve were weighed and recorded as M2 and M3 respectively.

Computation and recording of results

The percentage of the material (**m**) passing the sieve was calculated from;

$$m = \frac{M2}{M1} \times 100\%$$

When this percentage did not fall within the range 7.5 % - 12.5 %, the test was repeated with the maximum target force adjusted using the formula below to bring the percentage of fines within the range.

The force F (in kN), required to produce 10 % fines for each test specimen was obtained from the equation:

$$F = \frac{14f}{m+4} \quad \text{(Equation 3.4)}$$

Where;

- **f** is the maximum force (in kN), **m** is the percentage of material passing the 2.36 mm sieve at the maximum force.

The test procedure was repeated for the other two specimen of the same mass as the first specimen at approximately the same force (f) that gave a percentage fines value within the range 9% - 11 %.

The AIV for the aggregates was obtained and recorded as the average ± SD for the three values of F obtained from the sample results.

v. Flakiness Index

Objective

Flakiness Index test was conducted determine the presence and percentage of flaky aggregates in the aggregates sample. Aggregates are considered as flaky when theyhave a thickness of less than 60% of the nominal length or size. Presence of flaky particles is considered undesirable since they may cause inherent weakness with possibilities of breaking down under heavy loads aggregates as referenced from (BS812-P105:1-1989).

Sample preparation

The aggregates were riffled and sampled to obtain a two samples of mass of about 2kg each. These were washed to clean away any dust and dried in an ovenat 105 – 110^{0}C. The sample specimen was then weighed and its mass recorded.

Test procedure

- The sample was sieved through a set of different sieve sizes namely; 63mm, 50mm, 37.5mm, 28mm, 20mm, 14mm, 10mm and 6.3mm and all material retained on the 63 mm sieve and passing 6.3 mm sieve was discarded.
- The weights of aggregates retained on the different sieves was determinedand recorded then placed on marked trays and the individual percentageretained on each of the various sieves was calculated m2 from the sums ofmasses of the fractions in the trays m1.
- Each fraction using the thickness gauge was gauged and all particles passing the gauges were combined and weighed m3.

Computation and recording of results

The flakiness index of the aggregates was determined from the formula;

Flakiness index = $\frac{m2}{m3} \times 100\%$ (Equation 3.5)

vi. Relative density (specific gravity) And Water absorption of aggregates

Objective

This test was aimed at determining the relative densities of the aggregates for use in the asphalt mix design computations.

The test was used to determine the;

1. Relative density on oven dry basis which is mainly required in asphalt mix design.
2. Relative density on a saturated surface-dry basis mainly used in concrete mix design.
3. Apparent relative density, which may be used in production control to check if the density of the aggregate varies.
4. Water absorption of the aggregates.

This test was done in accordance to ASTM C127-88 and is suitable for aggregate sizes 5mm to 40 mm.

Sample preparation

- Two aggregate test samples of about 2-2.5 kg each were used.
- The samples were thoroughly washed through 10mm test sieve to remove finer particles e.g. clay, silt and dust while allowing it to drain.
- The samples were placed in the wire basket and immersed in water for 24 hours.
- The entrapped air was removed by lifting the basket slightly up and down under

water with gentle agitation to remove entrapped air.

Procedure

- The basket with the first sample was weighed in water and the mass C recorded.
- The aggregates were removed and placed on the dry cloth then gently surface-dried until all visible films of water were removed.
- The aggregates were then weighed and the mass recorded (mass A).
- The aggregates were placed on a tray and put in an oven to dry at 105 ± 5°C for 24 hours then left to cool.
- The sample mass was then weighed and recorded as (mass D).
- The procedure was repeated for the second sample and the relative densities were calculated as an average of the two test results obtained from the equation below.

Computation and recording of results

1. Relative Density at oven-dry basis = $\frac{D}{A-(B-C)}$ (Equation 3.6.1)
2. Relative Density on saturated surface dry basis = $\frac{A}{A-(B-C)}$ (Equation 3.6.2)
3. Apparent relative density = $\frac{D}{D-(B-C)}$ (Equation 3.6.3)
4. Water absorption = $\frac{(B-A)}{A} \times 100$ (Equation 3.6.4)

where;

- A is the mass of the saturated surface-dry aggregate in air (g)

- B is the apparent mass in water of the basket + sample of saturated aggregate(g)
- C is the apparent mass in water of the empty basket (g)
- D is the mass of the oven-dry aggregate in air (g)

The procedure was conducted for the two samples to obtain two sets of results for each of the parameters. Their average value was determined and recorded.

vii. Sieve analysis

Objective

The sieve test was chosen so as to aid in the classification of the particle size distribution of the different aggregate proportions from the individual hot bins. The grading of the different size fractions was used proportioning and blending of the different size fraction into a combined aggregate mix blend. The blending is important in ensuring that the aggregate grading developed was able to improve asphalt mixture properties such as air voids, VMA, interlocking properties to improve rutting resistance and others.

Sample preparation

- The sample of aggregates from the hot bin was riffled to produce a test sample specimen of about 2kg.
- The test specimen was heated at 105^{0}C for about 12 hours, allowed to cool and weighed again and its mass recorded as M1.

Test procedure

- The set of sieves mounted on a mechanical shaker was assembled in descending order from the largest diameter on top and smallest diameter down with a pan at the bottom.

- The oven dried sample was placed on it and the mechanical sieve shaker started until it was evident that no more material was passing.
- The material retained on each sieve was weighed and its mass recorded.
- The material was then placed on a pan and sufficient water added until it was half filled then agitated a little bit such that the fines were completelyseparated from coarse.
- The suspended fine solids were placed on the 75µm sieve and washed thoroughly allowing them n to a tray.
- The washed aggregates and fine suspended solids were decanted off the water and placed in an oven to dry at $105^{0}C$ ±5 for at least 12 hours, allowed to cool and its mass weighed and recorded as M2.
- The mass of material passing the 75µm sieve can be determined as; M1- M2.

Computation and recording of results

The percentage retained on each sieve was determined as a percentage of the original mass M1.

The mass of material passing each sieve was obtained as a cumulative percentage from the equation.

(% passing) = (% passing of previous sieve) – (% retained on this sieve) (Equation3.7)

viii. Developing an aggregate gradation curve

A graph showing the particle size distribution curve of the material was then plotted on a semi-logarithmic chart. This was done for each of the size fraction or aggregate from different hot bins. The particle size distribution curves for each of the size fraction (0-6, 6-10, 10-14, 14-20) were plotted on the same graph. Using the trial and error method, an aggregate gradation was developed that lies within the lower and upper limits specified by (Ministry of Works and Transport, 2005). The type of AC mix was chosen based on the Nominal Maximum Aggregate size which was the first sieve to retain at least 10% of the material. The AC mix design chosen was AC20.

3.5.2 Rheological properties of bitumen

i. Bitumen penetration test

The penetration test was selected to determine the relative measure of consistency of bituminous materials expressed as the distance in tenths of a millimeter that a standard needle vertically penetrates a sample of the material under known conditions of loading, loading time and temperature. This test aids in classifying the bitumen and is an indirect measure of viscosity of the bituminous material. A lower penetration indicates that the bitumen has a low viscosityand thus it is very hard. The test was performed in accordance to **ASTM D5-86**.

Sample preparation

- The sample of bitumen in a metallic bucket was heated carefully on a gas cookeruntil it was fluid.
- A metallic scoop was used to fetch a portion of the sample and then poured into

thefour well labelled penetration tins/cups to such a depth that it was greater than thedepth the needle is expected to penetrate.

- The container was loosely covered to protect it against dust and allowed to cool atroom temperature one hour.
- The sample was then be cured by putting the container in a water bath maintainedat 25℃ for one hour after which the test can be started.

Test procedure

- The cup with the sample was placed in a glass beaker containing water maintainedat 25^0C which was the placed at the base plate of the penetration apparatus.
- The penetration needle load was adjusted with the specified load to just touch thesurface of the sample.
- While reading the dial gauge, the pointer was brought to zero and the needle loadedfor 5 seconds to penetrate the bitumen sample.
- The dial gauge was then adjusted to read the distance moved.
- The needle was then removed and switched to another position then penetrated again and the penetration distance read and recorded on the dial gauge.
- This was done the third time to obtain three penetration distance for the same sample.

Recording of results

An average of the three penetration distances in deci-millimeters was obtained and recorded as the penetration for the first sample.

The test was repeated for the other four test samples to ensure consistency in results.

The penetration of the bitumen was recorded as the average ± SD of the penetration got from each of the four test samples in deci-millimeters.

ii. Softening point test

The test was aimed at determining the softening point of the binder and is an indirect measure of viscosity of a bituminous material. The softening point value of a material indicates the temperature susceptibility of the material. A high softening point shows lower tendency to flow at high temperature while a low softening point shows higher tendency to flow. The test was conducted in reference to **ASTMD36-95** standard procedure.

Sample preparation

The sample of bitumen in a metallic bucket was heated carefully on a gas cookeruntil it was fluid. The rings of the ring and ball apparatus were smeared with a thin coat of glycerol then placed on a metal plate also well smeared. The 2 rings were then filled with the bitumen and left to cool at room temperature forone hour.

Procedure

- A glass container containing the ring and ball apparatus and thermometer was filledwith distilled water to a depth of 108 mm.
- The balls were then placed onto the rings in the centering guide using forceps.

- The glass container was placed onto a heating unit.
- Heat was applied to obtain a constant temperature raise of 5℃ per minute.
- The temperature was recorded every 30 seconds.
- The temperature reading when the specimen of bitumen in each ball touched thebase was recorded.

Recording of results

The average temperature of two recorded temperatures from the two specimen was reported as the softening point of that sample. The test was repeated once again by picking another sample of bitumen to obtain twosets of results. The softening point of the bitumen was recorded as the average ± SD of the softeningpoints of the two test results obtained from the two samples picked.

iii. Ductility test

This test was selected to determine the relative measure of cohesive strength of bitumen. The ductility of bitumen was determined as the distance up to which the bitumen sample will elongate before breaking when stretched apart in a ductility testing machine at a speed of 5cm/min at a temperature below its softening point. The test was performed in accordance to **ASTM D113**.

Equipment

- Briquette moulds for ductility testing device
- Ductility testing machine with a water bath and thermometer
- Brass plate

Test procedure

- The bitumen sample was heated to molten state.
- The molten bitumen was poured it into the set of three briquette molds, which wereplaced on a brass plate coated with a mixture of glycerin to prevent sticking of bitumen to obtain three specimen in three molds of the same sample.
- The specimen was left to cool for one hour at room temperature.
- After cooling, the briquettes were moved apart with the sides and the plate removed.
- The mold assembly was the placed in the water bath for another one hour and attached to the mold clips of the machine carefully without applying any additionalforce to the bitumen.
- With the pointer of the machine calibrate to zero, the machine was started and theclips pulled horizontally at a constant speed of 50 millimeters per minute.

Recording of results

The ductility value for each specimen was recorded as the distance at which the bitumen thread stretches and then ruptures. The average for the three specimen was determined and recorded as the ductility forthe first sample. The procedure was done for a second sample and an average ± SD of the two samplesrecorded as the ductility of the bitumen.

iv. Temperature susceptibility (penetration index)

The penetration index is a measure of the consistency response of bitumen to temperature variation. The penetration index of particular bitumen is important in predicting its behavior (temperature susceptibility) during application. This parameter can be obtained from the penetration and softening point of a particular bitumen hence helping to relate the traditional or index tests that are more empirical to the mechanistic behavior of the bitumen. This way it is possible to predict how the bitumen consistency will be affected by temperature rises hence causing rutting. One of the best known equations to derive the penetration index was developed by (Pfeiffer & Van, 1936). This is explained in detail in section 2.5.1.

According to (Pfeiffer & Van, 1936), the PI value ranged from -3 for high temperature susceptibility to around +7 for highly blown (low temperature susceptibility) however for road paving application this was reduced to (-2 to +2). The temperature susceptibility A and penetration index (PI) can be a penetration of about 800 deci mm which was found to be the penetration at ASTM softening point temperature for most bitumen from the equation 3.8 below.

$$PI = \frac{1951.5 - 500\ \text{logPen} - 20 * \text{softening point}}{50\ Pen - 52 - 120.1} \qquad \text{(Equation 3.8)}$$

The equation 3.8 shown above was applied in this study where the data obtained from the softening point temperatures and penetration tests of the modified bitumen tests were used to calculate for the penetration index which was used to determine the temperature susceptibility of bitumen. The values obtained which were related to the ASTM specification that recommends a PI range of -1.5 to +0.7.

v. Wet process of bitumen modification

Wet process of asphalt modification was preferred in this study. This involved heating the bitumen to temperatures above the softening point so that it could melt. Five metallic trays were labelled based on the percentages of LLDPE that was used in modification as; 0%, 2%, 4%, 6%, and 8%. The trays were one by one placed on the weighing scale and bitumen added depending on the required mass for each percentage modification as showed in the table 4 below. The LLDPE of the required mass was also by spreading over the bitumen so that it was evenly distributed. The mixture was then transferred on to a heating gas cooker (high shear mixture is also recommended) and the mixture heated while maintaining the mixing temperature between 145 – 160 ^{0}C. The mixing was done for about one hour until the mixture achieved a homogenous mixture. This was done for all the samples after which the modified bitumen was allowed to cool before being subjected to bitumen tests. Modified bitumen was prepared by mixing LLDPE with bitumen at 145 – 160 ^{0}C to obtain1000g grams of each sample modified bitumen as shown in the table 5 below where;

Mass of LLDPE = % LLDPE X Mass of bitumen (1000) in g.

Table 5: Mix proportions of bitumen and LLDPE used for bitumen modification

Mass in total modified bitumen	**% of LLDPE**				
	0%	**2%**	**4%**	**6%**	**8%**
Mass of LLDPE (g)	0	20	40	60	80
Mass of bitumen (g)	1000	980	960	940	920
Total (g)	1000	1000	1000	1000	1000

Figure 14: LLDPE mixed in molten bitumen to acquire homogenous mix

3.6 Optimum Binder Content for the asphalt mixture

The Marshall mix design method was used to determine the optimum modified bitumen content to use in the asphalt mix as described in (Asphalt Institute, 1979). Using the Asphalt Institute procedure (Asphalt Institute, 2014), the optimum bitumen content was determined from the graphs showing the stability, flow, density and voids against binder content.

3.6.1 Marshall Mix design procedure (Asphalt Institute, 1979)

i. ***Sample preparation of asphalt specimen (ASTM D6926)***

With the aggregate size gradation pre-determined as described in the procedure above, 3 series of aggregate mixtures (for G_{mm}, G_{mb}, and Stability and flow) each comprising of 2 sample test specimens or duplicates (for consistency) were prepared. This was done for each percentage of binder to obtain 15 sets of specimen in duplicates for consistency. Modified bitumen was added to the aggregate mixtures in varying percentages with increments of 0.5% (i.e.; 3.5%, 4%, 4.5%, 5%, and 5.5%) and heated whilemaintaining temperatures at 145 – 160^{0}C while mixing until the aggregates had completely been coated with bitumen. The mixture was transferred to a compaction mold and compacted with 75 blows on either side of the sample.

Specimen for determination of the maximum specific gravity of un-compacted asphalt mixture were not compacted by spread evenly while separating fines from coarse material so that they cooled at room temperature then placed in separate containers.

ii. Testing of the asphalt mix specimen

The compacted specimens were also left to cool at room temperature and cure for 24 hours. They were later cleaned using a wire brush to remove any loose material to prepare them for the Marshall tests. The specimen were subjected to different tests as described in detail in section 4.4.1 so as to determine the;

- Bulk specific gravity (G_{mb}) in accordance to (AASHTO T 166 or ASTM D 2726)
- Maximum specific gravity (G_{mm}) in accordance to ASTM155-89
- Voids in mineral aggregates analysis (VMA) in accordance to (ASTM D6995-21)
- Stability and flow test in accordance to (ASTM D6927)

Computation of results

From the data obtained, graphs were plotted showing;

- Stability against bitumen content
- Flow against bitumen content
- Air voids in the asphalt mixture against bitumen content

- Bulk density against bitumen content

The optimum binder content was obtained from the average of the bitumen content at maximum stability, maximum density and 4% air voids. The obtained average bitumen content was plotted on the graph showing flow against bitumen content and if the flow value plotted lied within 2-4 mm, the average value was considered as the optimum bitumen content as described further according to the Asphalt Institute procedure.

3.7 Evaluating the rheological properties of modified bitumen.

This was aimed at evaluating the effect of modifying the bitumen with the LLDPE on the rheological properties that would influence the performance properties of the asphalt mixture. The mixture of polymers with bitumen leads to formation of a bi-phasic interaction between two functional groups that are formed; the asphaltenic phase and the polymeric phase. For effective polymer modification, it is essential that the amount of polymer added is essential to form an ideal homogenous structure.

In order to achieve this, the modified bitumen with varying percentages of LLDPE was subjected to bitumen tests to determine the rheology properties for each percentage modification. The modified bitumen was subjected to penetration, softening point, and ductility tests. The procedure used for each of these tests is the same procedure highlighted in section 3.5.2 above henceforth reference can be made to those particular sections.

3.7.1 Tests conducted on modified bitumen

i. Penetration test

The penetration test was conducted for each of the 2%, 4%, 6% and 8% LLDPE modified bitumen in quadruplicates following the procedure described in section 3.5.2.

Reporting of results

For each of the different percentages of polymer modified bitumen, the penetration was conducted in triplicates and result recorded as an average ± SD. The results were analyzed and a graph of penetration against percentage LLDPE added was plotted showing the trend of penetration with addition of LLDPE to the bitumen. The results were further analyzed to evaluate the influence of the polymer modification on the penetration and how it can influence the performance properties of the asphalt mixture against rutting.

ii. Softening point test

The softening point test conducted on the modified bitumen was conducted based on the procedure described in section 3.5.2 in accordance to ASTM D36-95. The test was conducted for each of the 2%, 4%, 6% and 8% LLDPE modified bitumen in duplicates.

Reporting of results

For each percentage of polymer modified bitumen, the softening point was taken as the average ± SD of the two specimen. The obtained results were plotted in form of a graph showing the trend of softening point with addition of LLDPE. The results were further analyzed to evaluate the influence of the polymer modification on the softening point and how it can influence the performance properties of the asphalt mixture against rutting.

iii. Temperature susceptibility of modified bitumen

The temperature susceptibility of the modified bitumen was determined from the penetration index. Having obtained the penetration and softening point for each of the different percentages of modified bitumen as described above in sections 3.5.2, the penetration index for each of the different modified bitumen of varying LLDPE percentage was determined using the equation 3.7.

Reporting of results

Using the data for the penetration index obtained, a graph was plotted showing the trend of penetration index with varying percentages of LLDPE. The results were further analyzed in relation with the recommended specifications for the requirements of penetration index as described in (ASTM-D946, 2020). This was based on so as to determine influence of polymer modification on the temperature susceptibility of the bitumen and how it can influence the performance properties of the asphalt mixture against rutting.

iv. Stiffness modulus of modified bitumen

Based on experimental data, Van der Poel developed a research model summarized in form of a nomograph for determination of stiffness modulus of bitumen (Van der Poel, 1975). According to the prediction model, having determined the softening point and penetration index for each of the different modified binders with varying LLDPE percentages as described above, the bitumen stiffness modulus was determined based on the prevailing operating conditions of loading time and maximum pavement temperatures expected in the pavement.

Depending on penetration index obtained, the difference of temperatures (Tsp − T) (where Tsp is the softening point temperature of the bitumen and T is the expected operation temperature in the field) and the load duration t in seconds obtained from the vehicle speed at the road section ranging from (10^{-6} and 10^{10} seconds), the stiffness of the bitumen can be obtained. Van der Poel's nomograph was included into reference books such as The Shell Handbook by (Robert N. Hunter, 2015), as well as several research technical papers. Van der Poel's prediction model was not published in form of empirical equations hence stiffness modulus can be obtained only graphically as shown below.

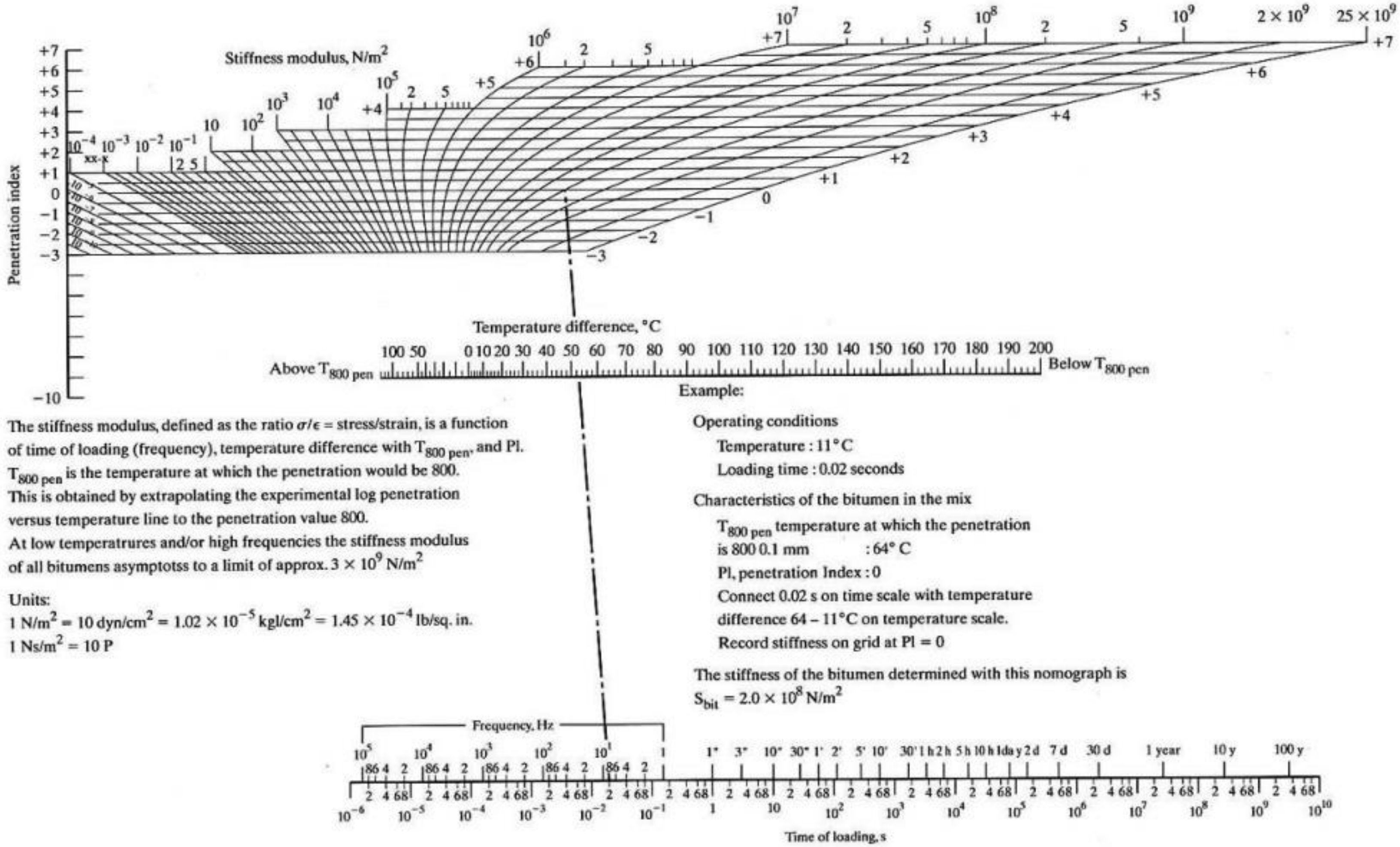


Figure 15: Obtaining the bitumen stiffness modulus from Van Der Poel's nomograph.

Computation of Stiffness modulus from the nomograph.

From the Figure 14 above, assumptions for the operating conditions of the pavement below were made for purpose of illustration;

- Loading time = 0.02 seconds
- Pavement temperature T = 11^{0}C

Experimental data obtained from bitumen tests;

- Softening point temperature, Tsp = 64^{0}C
- Penetration index obtained = 0
- Difference in temperature = Tsp – T = (64 – 11) = 53^{0}C

Given the above information, the bitumen stiffness was obtained by plotting the 0.02s from the loading time axis, which is matched with the temperature difference of 53^{0}C to touch the axis of Penetration index of 0. The bitumen stiffness modulus was then obtained by interpolating along the axis of stiffness modulus in N/m^{2}. This was done for the data obtained for all the different modified bitumen and the stiffness modulus converted in to kPa.

Recording of results

Having obtained the stiffness modulus for each of the modified bitumen with varying percentages of LLDPE, a graph was plotted showing the variation of the stiffness with addition of LLDPE. The results were further analyzed in order to determine influence of polymer modification on the stiffness modulus of the bitumen and how it can influence the performance properties of the asphalt mixture against rutting.

v. Ductility of modified bitumen

Having determined the stiffness modulus of the bitumen, it was essential to conduct a ductility test to evaluate whether the increase in stiffness of the bitumen does not impede on the flexibility of the bitumen and its ability to withstand sudden fracture when subjected to loading. The ductility test on the modified bitumen was conducted based on the procedure described in section 3.5.2 in accordance to ASTM D113. The test was conducted for each of the 2%, 4%, 6% and 8% LLDPE modified bitumen in triplicates.

Reporting of results

The ductility for each sample of modified binder was recorded as an average ± SD of the ductility for the three test specimen. A graph showing the variation in the ductility of the bitumen with the addition of LLDPE was plotted. The results were further analyzed in relation with the recommended specifications for ductility of modified bitumen as described in (Ministry of Works and Transport, 2005). This was based on so as to determine influence of polymer modification on the ductility of the bitumen and how it can influence the performance properties of the asphalt mixture.

3.8 Performance properties of the modified asphalt mixture

This objective was aimed at assessing the performance properties of the modified asphalt mixture. Asphalt mix specimen comprising of modified bitumen of varying LLDPE percentage were prepared and subjected to Marshall, indirect tensile strength and refusal density tests as shown below.

Preparation of modified asphalt specimen

The modified bitumen with different percentages of LLDPE was prepared using the wet bitumen modification process as described in section 3.6.1. The modified bitumen was then used to prepare asphalt mixtures using bitumen content obtained as described in section 3.6 (OBC=4.4%) and the aggregate size fractions from the aggregate gradation shown in Figure 15. The trial mix that was used is shown in the table 6 below.

Table 6: Asphalt mix design used to prepare the modified asphalt specimen

Aggregate sizes	% of aggregate sizes	Neat asphalt mixture (0% LLDPE)	Modified asphalt mixture (2%, 4%, 6% & 8% LLDPE)	Mass in total mix (g)	Mass in total mix Gmm (g)
20/14	25	25	25	286.8	478
14/10	12	12	12	137.7	229.4
10/6	8	8	8	91.8	153.0
3/6	53	53	53	608.0	1013.4
Filler	2.0	2.0	2.0	22.9	38.2
Total Agg	100	100	100	1147.2	1912
Bitumen	-	4.4	4.4	52.8	88
Total mix	**100%**			**1200.0**	**2000**

For each percentage of binder modification, 14 specimen were prepared which comprised of 6 specimen for Marshal tests (three for volumetric properties and bulk specific gravity (G_{mb}), three for Marshall stability and flow tests), 6 specimen for ITS (dry and wet) and 2 specimen for the un-compacted theoretical maximum specific gravity (G_{mm}) as shown in the table 7.

Table 7: Number of specimen prepared for each performance test conducted

Test conducted	Number of specimen prepared for each percentage sample of modified bitumen
Marshall tests	6
ITS dry and wet	6
PRD	2
G_{mm}	2

The sample specimen were then prepared for the respective tests by compaction using a Marshall compactor and subjecting them to performance tests as described in the procedure for the different test procedure showed below.

3.8.1 Tests conducted on the modified asphalt mixture

i. Marshall tests

Objective

Marshall testing was aimed at determining the stability and flow values, bulk density of compacted asphalt mixture and air voids in the total mix (VIM), voids in the mineral aggregate (VMA) and voids filled with bitumen (VFB).

Sample preparation of asphalt specimen (ASTM D6926)

The asphalt mixture was placed in an oven maintained at 175^{0}C to carter for temperature losses during transportation of the samples. The specimens were removed once they had acquired the required mixing and compaction temperature and compacted using a Marshall compactor offering 75 blows on each face at a temperature 155 – 165^{0}C. The compacted asphalt specimen were then allowed to cool and cure for about 24 hours.

Determining the diameter and thickness of compacted specimen

Determining the average diameter and thickness of the compacted asphalt specimen is crucial since these values are used to ascertain the different correctional factors that are applied to the measured values of stability. The average diameter was also crucial in determination of the ITS value as stated in equation 3.16 The diameter and height of each of the specimen was measured venier using a caliper across three different dimensions of the specimen. The average thickness and average diameter was then determined as an average of the three dimensions measured and recorded in mm to one decimal place. The specimen were then ready to be subjected to the respective Marshal tests as described below.

Determining bulk specific gravity (Gmb) (AASHTO T 166 or ASTM D 2726)

The bulk specific gravity was determined using the SSD method in accordance to AASHTO T166 or AST D2726.

Test procedure

- The specimen was weighed using a weighing balance and mass in air recorded as

M1.

- The specimen was then placed in a water bath maintained at $25^0C \pm 5$ directly belowthe weighing scale and its mass under water was determined and recorded as M2.
- The specimen was then submerged in water for about 5 minutes after which it was surface dried using a damp cloth and its mass at surface dry condition determined within 30 seconds after removing it from the water andrecorded as M3.

Computation of results

The bulk specific gravity of the compacted asphalt mixture was then determined using the formula;

$$G_{mm} = \frac{A}{B-C} \quad \text{(Equation 3.9)}$$

Where:

A = mass of the specimen in air

B = mass of specimen at saturated surface dry (SSD) condition

C = mass of the specimen in water at 25°C.

The G_{mb} for all the compacted specimen (Marshall, PRD, ITS) was determined using this procedure and the obtained value recorded to the nearest 3dp.

Theoretical maximum specific gravity (G_{mm}) (ASTM D2041 orAASHTO T209)

The maximum specific gravity of the asphalt mixture was determined for the un-compacted asphalt specimenin accordance to ASTM D2041 or AASHTO T209 as described further in (Asphalt Institute, 2014).

Test procedure

- The loose asphalt mixture (about 2kg each specimen) was warmed and separated into loosely so that the coarse and fine coated aggregates were clearly distinguished to avoid sticking together.
- The specimen was weighed and its mass in air recorded as mass A.
- The sample was then placed in a calibrated pycnometer (of known volume) and water was added to submerge the sample.
- The vacuum lid of the pycnometer was fitted tightly then placed on a vibratory shaker.
- A vacuum pump was connected to the pycnometer and started while ensuring the pressure reading on the manometer was almost (27.5 mm Hg) vacuum is obtained toprovide gentle agitation to remove of any air between particles.
- After 15 minutes, the pycnometer and submerged sample were filled with water andthe lid placed on top while ensuring that there were no air voids.
- The mass of the pycnometer and specimen filled with water was the determined asthen recorded as C.

Computation of results

The theoretical maximum specific gravity was then calculated using the equation;

$$G_{mm} = \frac{A}{A+B-C} \quad \text{(Equation 3.10)}$$

Where;

A = Mass of uncompacted asphalt specimen in air (g)

B = Mass of pycnometer filled with water (g)

C = Mass of pycnometer + specimen + water

The procedure was done for the two specimen and the G_{mm} recorded as an average of the two test specimen up to 3 dps. The procedure above was used to obtain the G_{mm} of all the modified asphalt samples with varying percentage of LLDPE.

Determining of percentage of voids in the asphalt mixture

Having obtained the bulk specific gravity of the compacted asphalt mixture (G_{mb}) and the theoretical maximum specific gravity of the un-compacted asphalt mixture (Gmm), the volumetric properties of the asphalt mixture were obtained using the formulae below;

$$\text{Air voids (\%)} = \frac{\text{Gmm} - \text{Gmb}}{\text{Gmm}} \quad \text{(Equation 3.11)}$$

$$\text{VMA (\%)} = 100 - \frac{\text{Gmm} \times \text{Ps}}{\text{Gsb}} \quad \text{(Equation 3.12)}$$

$$\text{VFB (\%)} = 100 \times \frac{VMA - Pa}{VMA} \quad \text{(Equation 3.13)}$$

Where;

G_{mb} = bulk specific gravity of the compacted asphalt mixture

P_s = Percent aggregate in the asphalt mixture specimen

G_{sb} = Combined bulk specific gravity of the aggregate blend

VFB = Percent volume of VMA filled with bitumen

VMA = Voids in Mineral Aggregates

P_a= Percent Air Voids of the Total mix volume

Stability and flow test (ASTM D6927)

Objective

Marshall Stability and flow test was aimed at measuring the maximum load sustained by the bituminous mixture and deformation experienced when the maximum load is reached. The test was conducted in accordance to ASTM D6927 procedure.

Equipment required

- Marshall compression testing device
- Calibrated Marshall flow meter
- water bath maintained at 60^0C

Test procedure

- The asphalt specimen was immersed in a water bath maintained at a temperature of 60° ± 1°C for 30 minutes.
- The specimen was then placed on the Marshall Stability testing machine and loaded at a constant rate of deformation of 5 mm per minute until failure.
- The maximum load in kN reached at maximum deformation was read off he proving ring and recorded.

- The stability value was obtained by multiplying the maximum load by the proving ring factor.
- The stability was then corrected depending on the average thickness and average diameter of the specimen (corrected for volume).
- The deformation that occurred at maximum load was measured off the flow meter in units of 0.25 mm and recorded as the flow value in mm.

Recording of results

The stability was obtained from the equation;

Stability (kN) = maximum load × ring factor × volume correction factor (Equation 3.14)

The stability and flow values for the three specimen of the same percentage of LLDPE was determined was obtained and the average value determined and recorded as the stability of the modified asphalt mixture.

ii. Indirect Tensile Strength test

Objective

The test was aimed at determining the resistance of the modified asphalt mixture to moisture induced damage. This was crucial in evaluating whether the modified asphalt would be subject to stripping. This test involved subjecting the asphalt specimen to conditioned and unconditioned state after which the tensile strength was determined. The relation of the strength values of the conditioned specimen and the unconditioned specimen was determined as the Indirect Tensile Strength Ratio (ITSR).

Sample preparation

- The asphalt test specimen were prepared using the Marshall procedure described in ASTM 6926 but the compaction effort used for indirect tensile strength samples was 33 blows which was aimed at attaining an expected in place percentage of 7 % air voids after which it was left to cure at room temperature for 24hours.
- After curing, the sample specimen were tested to determine the maximum specific gravity (Gmm), average thickness, bulk specific gravity (Gmb) and the air voids.

For the conditioned set

- The volume of air voids in the specimen was first determined in cubic centimeters.
- The specimen were then subjected to vacuum saturation in a pycnomter for about 7 minutes.
- They were then wrapped in leak proof plastic bags and subjected to freezing at -18^{0}C and warm water conditions.

For the dry set

- The sample specimen were saturated in a pycnometer after which the volume of water absorbed was determined by SSD method as stated in AASHTO T166**.**
- The degree of saturation was then determined from the equation;

$$degree\ of\ saturation = 100 \times \frac{vol.of\ water\ absorbed}{volume\ of\ air\ voids} \quad \text{(Equation 3.15)}$$

- If the degree of saturation of saturation lied between 70 and 80 %, the freezing conditioning was continued. If it lied below, saturation was repeated using more vacuum. If the degree of saturation was greater, the sample was discarded.

- The specimen that passed were wrapped in a water tight plastic bag and frozen at -18^{0}C for 24 hours.
- The specimen were then placed in water bath maintained at 25^{0}C ± 1 for 2 hours.

Test procedure

- The specimen were removed from the bath, thickness determined then placed on the Marshall testing equipment and a load applied at constant rate of 50mm per minute until the specimen cracked.
- The maximum load that caused cracking is read off the dial gauge and recorded.

Computation and reporting of results

The tensile strength and tensile strength ratio was calculated using the formula;

$$St = \frac{2P}{\pi td} \quad \text{(Equation 3.16)}$$

Where;

St = tensile strength (kPa)

P = maximum load (kN)

t = thickness of specimen (mm)D = diameter of specimen (mm)

Tensile Strength Ratio (TSR) = $\frac{Average\ tensile\ strength\ in\ conditioned\ state}{Average\ tensile\ strength\ in\ dry\ state}$ (Equation 3.17)

iii. Percentage Refusal Density (PRD)

Objective

The percentage refusal density test was aimed at determining whether the modified asphalt mixtures was capable of retaining the minimum required air voids content after secondary compaction. The test was also aimed at determining whether there would be a possible improvement in the ability to retain more air voids at refusal density by the modified asphalt mixture as compared to the unmodified mixture. The test was conducted in accordance to **BS 598 104 1989** standard procedure.

Equipment used

- Vibrating hammer method
- Moulds of diameter 152-153 mm
- Electric vibrating hammer operating at a frequency of 20 to 50 hz.
- Two tamping (102 mm and 146 mm respectively)
- Stop clock
- Heating oven maintained at 150 - 160^{0}C
- Filter paper

Test procedure

- Six asphalt mix specimen comprising (3 comprised of neat bitumen and the other 3 modified bitumen with 2% LLDPE) of were prepared and kept in an oven maintained at 150 to 165 ^{0}C
- The moulds and tamping feet were pre-heated in an oven before starting the test.
- The mould and base plate were well assembled and smeared with wax or glycerine to aid effective removal of the specimen.

- A filter paper was placed at the bottom and the mixture placed in the mould.
- The vibrating hammer was held firmly in a vertical position over the specimen and stop clock started immediately when compaction was started.
- The vibrating hammer was moved from position to position in the prescribed order; S, W, E, NW, SE, SW and NE while ensuring at each point, compaction continued for about 2s and 10 seconds
- The compaction sequence was continued until a total of 2 minutes ± 5 seconds of compaction time was reached.
- The large tamping foot was then used to smoothen the surface of the sample.
- The base plate was turned and placed on top of the mould which was then turned over and the sample turned over to the other side.
- The compaction sequence was repeated.
- After compaction was finished, the cores were allowed to cool in air for at least 2 hours and then removed from the mould and placed in open air to cure for 24hours
- The average thickness of the samples and the sample was weighed, in air (mass A), and in water (mass B) then also weighed at SSD (mass C) after being immersed in water at 25 ±1 ^{0}C.

Computation of results

The Refusal density of the specimen and the voids in the mix at refusal density were obtained using the equation;

$$G = \frac{A}{C-B} \quad \text{(Equation 3.18)}$$

$$VIM = 100 \times \frac{Gmm - G}{Gmm} \quad \text{(Equation 3.19)}$$

Where;

G = Refusal density of the specimen

VIM= Voids in the mix at refusal density

A = Mass of specimen in air

B = Mass of specimen in water

C= Mass of specimen at SSD

Gmm = Maximum specific gravity of un-compacted asphalt mixture

CHAPTER 4: RESULTS AND DISCUSSIONS.

This chapter presents data obtained from the tests carried out in this study. The data was analyzed and discussed in relation to the research study to deduce relationships between the different parameters and how well they can solve the issue of rutting.

4.1 Index properties of aggregates and bitumen.

Index properties of materials are a set of fundamental properties that determine the physical and mechanical properties of the aggregates. The results obtained from tests to identify the index properties of materials are discussed further in this section.

4.1.1 Results for aggregate tests

Table 8: Results obtained from the aggregate tests

Property	Test	Test standard	unit	Average measured value ± SD	Specification (MoWT)
Strength	ACV	BS 812: Part 110: 1990	%	16.7 ± 0.1	Max 25
	AIV	BS 812: Part 112: 1990	%	15.2 ± 0.1	Max 25
	TFV dry	BS812: Part 111:1990	kN	282.8 ± 0.2	Min 160
	TFV wet	BS812: Part 111:1990	kN	296.3 ± 0.2	Min 110
Abrasion	LAA	ASTM C535-89	%	17.2 ± 0.1	Max 30
Particle Shape and texture	flakiness index	BS812:105.1:1989	%	19.3 ± 0.1	Max 35
	CAA	ASTM D-5821			Min 95 (super pave)
	FAA	AASHTO T-96			Min 95 (super pave)
	Sand Equivalent	ASTM D2419			
Combined Specific Gravity	Specific gravity	BS1377:Part 2:1990	g/cc	2.6 ± 0.05	2.0 – 3.0
Water Absorption	Water absorption	BS812: Part 2:1975	%	0.3 ± 0.05	Max 2%

Under severe loading conditions such as slow moving heavy traffic, asphalt mixes along such sections are expected to be subjected to severe secondary compaction along the wheel path (Transport Research Laboratory , 1993). Whereas the bitumen binds the pavement together and gives additional stiffness, it is the role of the aggregate structure to carry the load hence making the aggregate selection and blending key constituents in an asphalt mix design. The Virginia Asphalt Association recommends that for road sections with an expected increase in traffic loading on the pavement, the aggregates and aggregate blending must meet higher standards (Virginia Department of Transportation, 2018). Basing on the results obtained from aggregate tests as showed above in table 5, the aggregate were of high standards when compared to the specifications by Ministry of Works and Transport (MoWT). The results obtained were also compared to the super pave aggregate requirements for resistance against permanent deformation.

Tough and durable aggregates are necessary since the aggregates should be strong enough to withstand crushing under gradual loading and sudden impact loading to avoid disintegration and crushing which would lead to rutting. The ACV, AIV and TFV values obtained for the aggregates were extremely good implying that the aggregates were strong to resist disintegration. The aggregate blend is also expected to have a high internal friction so as to exhibit a degree of interlocking needed to resist shearing and rutting. The more angular and rougher the aggregates are, the greater the interlocking properties due to frictional resistance developed when subjected to loading. The super pave requirements provide minimum requirements for CAA, FAA and clay content to ensure good degree of interlocking is achieved. The results obtained from the aggregate test when compared to the minimum requirements were also very good implying the aggregate had a capability to portray good aggregate interlocking to resist rutting.

Durable aggregates are also required to avoid issues of wear rutting which could occur due to wearing off of surface material. Abrasion value obtained from the LAAT was far less when compared to the maximum abrasion expected which meant that the chances of wearing were extremely low. The particle shape is also a crucial aspect since flat and elongated aggregates are very weak and less capable of resisting the heavy loads at such severely loaded sections. The flakiness index value obtained for the aggregates as shown in table 5 was very good to resist disintegration due to particle shape.

Water absorption within the aggregates accelerates aging and causes stripping since the water weakens the adhesion between the aggregates and the bitumen. Less water absorbent aggregates are therefore preferred and as shown in table 5, the water absorption obtained was extremely low hence aggregates would be less susceptible to water susceptibility.

It is essential to note that resisting to in asphalt is a combined effort between the aggregates and the binder properties. The results obtained from the aggregate tests revealed that the aggregates were good enough to resist rutting. Analysis was then conducted on the bitumen tests results obtained to assess its potential in resisting permanent deformation.

Combined aggregate grading

Individual grading of the different aggregate sizes was conducted and a combined aggregate gradation of the different aggregate size fractions obtained using the trial and error numerical procedure after which a grading curve shown in Figure 15 below was developed.

Aggregate gradation on a semi-logarithmic grading chart												
Sieve size (mm)	28	20	14	10	5	2.36	1.18	0.6	0.3	0.15	0.075	Spec. Table 1.1
Upper limit % passing	100	100	80	70	56	44	34	27	20	13	6	
Lower limit % passing	100	80	60	50	36	28	20	15	10	5	2	
Grading attained (%)	100	100	76	65	47	31	22	17	12	8	5	

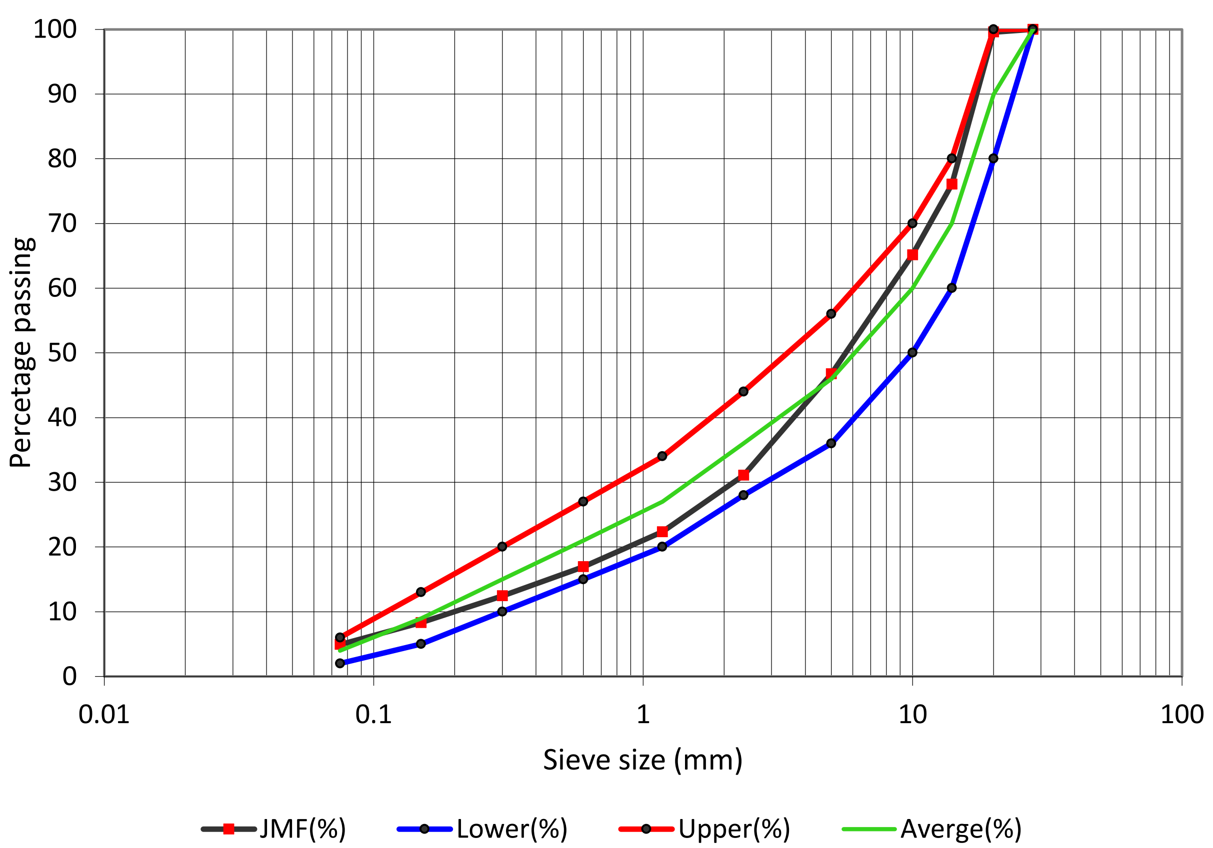


Figure 16: Combined aggregate gradation developed.

The graph above shows the AC20 individual gradation and it also shows that the grading curve lies within the specified grading limits. This was chosen to incorporate larger aggregate sizes which tend to be more resistant to rutting. The larger aggregates not only lead to an increase of Voids in mix which is important in reducing the rutting performance but also have a lower surface area to volume ratio hence reduces the binder content . A higher binder content is good for improved fatigue life and durability but enhances rutting performance. The incorporation of filler material increases the surface area of interaction of the binder to improve the adhesion towards aggregates. The filler also makes the binder stiffer. Finer aggregates however are more resistant to deformation henceforth the gradation curve was scaled to incorporate this in mind.

4.1.2 Results for unmodified bitumen tests

Table 9: Results obtained from unmodified bitumen tests

Property	Test standard	unit	Average measured value ± SD	Specification
Penetration	ASTM D-5	deci mm	64 ± 0.5	60 - 70
Softening point	ASTM D36	^0C	52 ± 0.5	49 – 56
viscosity	ASTM D4402-91	mm^2/s	340 ± 0.5	> 295
Specific gravity	ASTM 1429	g/cc	1.027 ± 0.01	1.10 - 1.06
Penetration index	Annex A	--	-0.7	-1.5 to +0.7 (EN 12591)
Ductility	ASTM D113	mm	1165 ± 0.5	> 1000

The index rheological properties commonly used to assess the rheological properties of bitumen were tested for and results obtained as shown in the table above. The results were further analyzed to assess how the related to the permanent deformation of the asphalt mixture.

Penetration; this is an index for consistency of the binder at intermediate service temperatures. According to paving grade classification (ASTM-D946, 2020), the penetration obtained in table 6 above indicates that the bitumen conforms to pen grade

60-70. The penetration grade relates to the stiffness of the asphalt and to resist permanent deformation, a highly stiff binder (lower penetration grade) is required. The use of 40–50 and 60-70 pen bitumen (based on ASTM classification) is recommended in tropical regions since it provides a suitable compromise between workability, deformation resistance and potential hardening (Transport Research Laboratory , 1993). However the penetration is a one point temperature test and cannot be justified as an indicator of consistency in bitumen stiffness as the temperature rises. The authors however recommend that if possible, suitable bitumen should be selected with low temperature sensitivity so as to resist rutting susceptibility.

Softening point; this is an index of consistency of bitumen at elevated service temperatures. According to EN 12591, the requirements for softening point of pen grade 60-70 should lie within 46 - 54 ^{0}C (European Committee for Standardization, 2009). The MoWT specifications however suggest a requirement of 49 – 56 ^{0}C which implies that the value obtained in table 6 above lies within the required specifications. However, it is essential to note that this index property is widely used around the world for compliance but should not be considered reliable for assessing the high temperature performance or temperature susceptibility in paving application (Richard, 2009).

Several researchers have highlighted that the above mentioned traditional measures are restrictive due to their empirical nature and so laboratory based which contradicts the derivation of engineering performance properties in the field. Several models have been developed to correlate the traditional index properties with more mechanistic rheological properties. Since bituminous materials are rheological in nature, the stress

– strain relationships are dependent on properties such as temperature susceptibility and loading time (A Al-Suhaibani, 1995). Several researchers (section 2.5.1), have derived models that relate the traditional / index properties to the latter which informs the deformation behavior of asphalt hence making it possible to predict its performance in the field.

Temperature susceptibility; this relates with the change in consistency of the viscosity of the binder as the service temperatures change. The penetration index (explained further in section 2.5.1) was used to relate the temperature susceptibility of the bitumen which is directly related to deformation behavior especially in tropical regions like Uganda where pavement temperatures rise as high as 50^{0}C in the central region as highlighted by (Mpaata, et al., 2023).

Penetration index of unmodified bitumen = $\frac{1951.5-500\log(64)-20*52}{50\log(64)-52-120.1}$ = -0.7

In order to improve the performance of the asphalt mixture against rutting, (Fakhrur, 2013) explains that it is essential to keep the temperature susceptibility of the bitumen low especially for hot climatic regions. The range for paving grade bitumen recommended by EN 12591 ranges from -1.5 to +0.7 with low PI values indicating high temperature susceptibility and high values meaning low temperature susceptibility. However, several researchers recommend that the PI should be as close to 0 as possible to reduce the rutting susceptibility. This is because at this point, bitumen tends to equi-viscous state at which the viscosity will remain constant despite the changes in temperature. This study therefore intended to modify the bitumen used in the asphalt mixture to improve its pseudo rheological properties at higher temperatures (up to 50^0C) and increased loading time due to low vehicle speeds at the climbing lane section.

Viscosity; this relates with the measure of resistance to flow of the binder at production and construction temperatures. It is widely used to measure workability of bituminous mixtures. The test was conducted to determine the mixing and compaction temperatures for the production of the asphalt mixture. The dynamic viscosity obtained was 340 mm^2/s, which gives mixing, and compaction temperature of 146 ^0C that lies within the recommended range which is $145\text{-}160^0C$. The mixing and compaction temperatures were obtained from the equi-viscous temperatures ranges obtained from the Figure 9 showing variation of temperature with viscosity.

4.2 Optimum binder content of the asphalt mixture

To determine the optimum binder content of unmodified bitumen, six sets of trial mixtures were prepared and tested according to the Marshall mix design procedure. The trial mixes were composed of varying bitumen contents ranging from 3.5%, 4.0%, 4.5%, 5.0%, 5.5% and 6.0% of the entire asphalt mixture. The specimen were compacted using a Marshall compactor offering 75 blows on either sides or face of the specimen. For each trial mix, three specimen were prepared for stability and flow test and for the determination of bulk specific gravity (G_{mb}). Three other specimens composed of 2.0 kg each were prepared (un-compacted) for the determination of the theoretical maximum specific gravity (G_{mm}) of asphalt mixture. The test results of the trial mixtures are shown in Table 10 below.

Table 10: Results based on to determine the optimum bitumen content

Property	% LLDPE				
	3.5	4.0	4.5	5.0	5.5
Stability of Marshall specimen (kN)	12.5	13.5	13.8	13.2	12.5
Flow of Marshall specimen (mm)	2.5	2.5	3.3	2.0	2.3
Bulk specific gravity (G_{mb})	2.333	2.354	2.369	2.363	2.350
Air voids in Marshall specimens (%)	6.3	4.7	3.5	3.2	3.3

In order to determine the optimum bitumen content, the Asphalt Institute Procedure was followed as shown in the Figures 16 to 19. The OBC was obtained in accordance to the Asphalt institute procedure (Asphalt Institute, 2014).

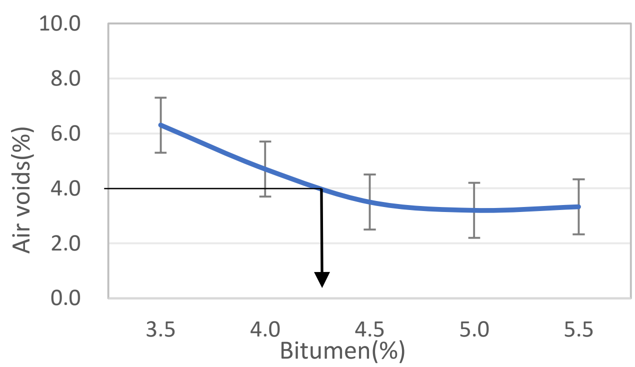


Figure 17: Air voids against bitumen content

The optimum bitumen content at air voids was obtained at 4% air voids was 4.3%.

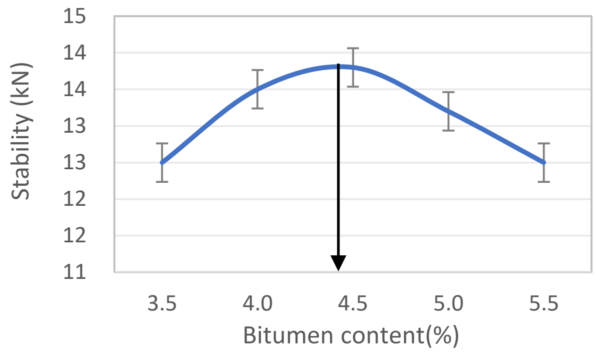


Figure 18: Stability against bitumen content

The optimum bitumen content using stability was obtained from the maximum stability obtaining 4.4%.

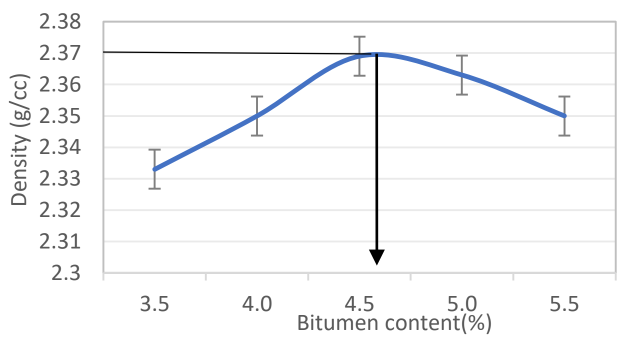


Figure 19: Bulk density against bitumen content

The optimum bitumen content was obtained from the maximum giving 4.6%.

Table 11: Obtaining the optimum binder content

Parameter	**Optimum binder content (%)**
AIR VOIDS	4.3
STABILITY	4.4
DENSITY	4.6
AVERAGE	**4.4 ± 0.15**

This OBC was checked to meet the criteria for flow as seen in the graph below;

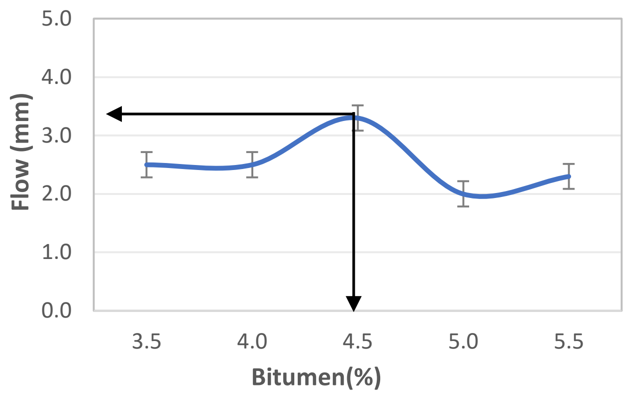


Figure 20: Flow against bitumen content

Since the flow value (3.3) obtained from the average optimum bitumen content (4.4%), is within the standard flow range of 2mm to 4mm, then the optimum bitumen content is okay and the mixture will be safe against plastic deformation.

4.3 Evaluating the rheological properties of modified bitumen.

Having modified the bitumen with varying percentages of LLDPE i.e.; 2%, 4%, 6% and 8%. Bitumen tests were conducted to determine the index properties of the modified bitumen. These tests were done in duplicates and triplicates and the average values with standard deviations were calculated. An asphaltmechanistic model by (Pfeiffer & Van, 1936) which has been recommended by several researchers and incorporated in the IS EN 12591 was used to relate the index propertiesto temperature susceptibility of the bitumen using penetration index (PI). The PI, in conjunction with loading time which are crucial properties that affect the mechanisticperformance of asphalt along climbing lanes were used to predict the stiffness modulusof bitumen using the Van Der Poel's model (Van der Poel, 1975). The results were analyzed and discussed further to determine the effect of the LLDPE on the rheological properties ofthe bitumen and ascertain the favorable dosage to use in the asphalt mixture.

i. Bitumen softening point

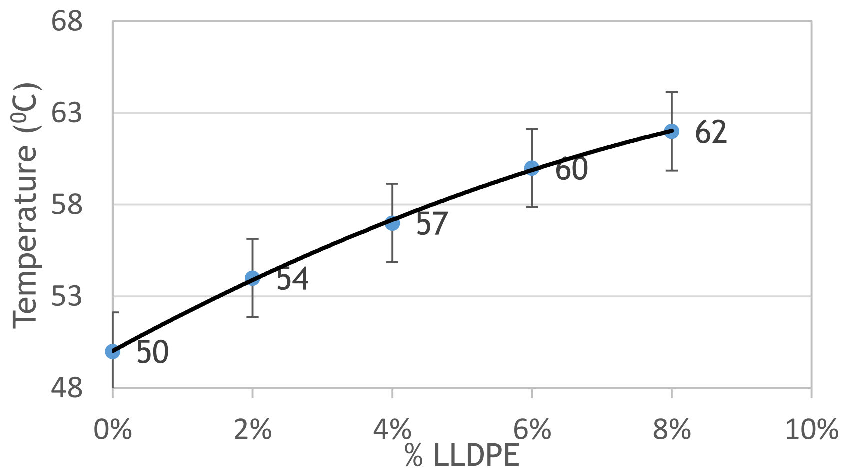


Figure 21: Softening point against % LLDPE

The data obtained indicates an increasing trend in the softening point of the bitumen. Further addition of LLDPE beyond 6% exceeded the specifications range of (49 – 56) °C by MoWT and EN 12591. It should however be noted that the compliance of the softening point values obtained to the specification should not be considered reliable for assessing the high temperature performance or temperature susceptibility in paving application. This therefore means that this parameter is only empirical and cannot be justified as an assurance that it will reduce the rutting performance of the asphalt.

ii. Bitumen Penetration

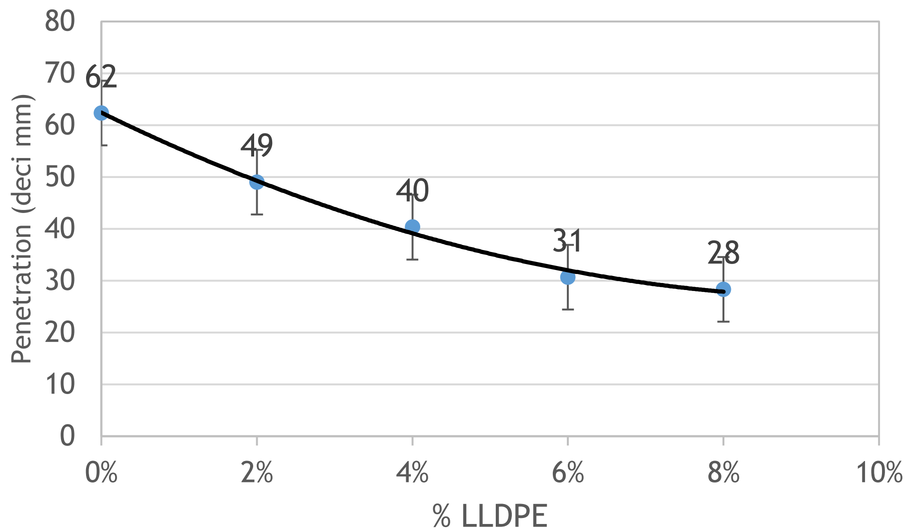


Figure 22: Bitumen penetration against % LLDPE

The data obtained showed a decreasing trend in bitumen penetration as the percentage of LLDPE was increased which indicates an increase in stiffness. The recommended penetration grade bitumen for use in paving application include 40-50, 50/70. Addition of 2% and 4% improved the bitumen penetration to pen 40-50 which would ideally mean an increase in stiffness and ability to resist rutting due to improved shear strength. Further addition of LLDPE up to 6% and 8% produced harder bitumen with penetration lying within pen grade 20-30. Bitumen of pen grade 20-30 is considered hard and unsuitable for paving application (Robert N. Hunter, 2015). However, whereas 2% and 4% LLDPE modification increased the hardness of the bitumen, this is not conclusive of the potential to resist rutting since penetration is a one point temperature test and cannot be justified as an indicator of consistency in bitumen stiffness at varying temperatures.

iii. Temperature susceptibility using penetration index (PI)

The penetration index for each of the different percentages of modified bitumen were determined using equation 3.7 and results obtained are shown in the table below.

Table 12: Penetration index obtained for the modified bitumen

% LLDPE	Penetration ± SD(d mm)	Softening point ± SD (^{0}C)	Penetration Index
0%	62 ± 2.3	50 ± 1.4	-0.67
2%	49 ± 1.7	54 ± 1.4	-0.29
4%	40 ± 0.6	57 ± 2.8	-0.09
6%	31 ± 0.6	60 ± 1.4	-0.08
8%	28 ± 1	62 ± 0.5	0.13

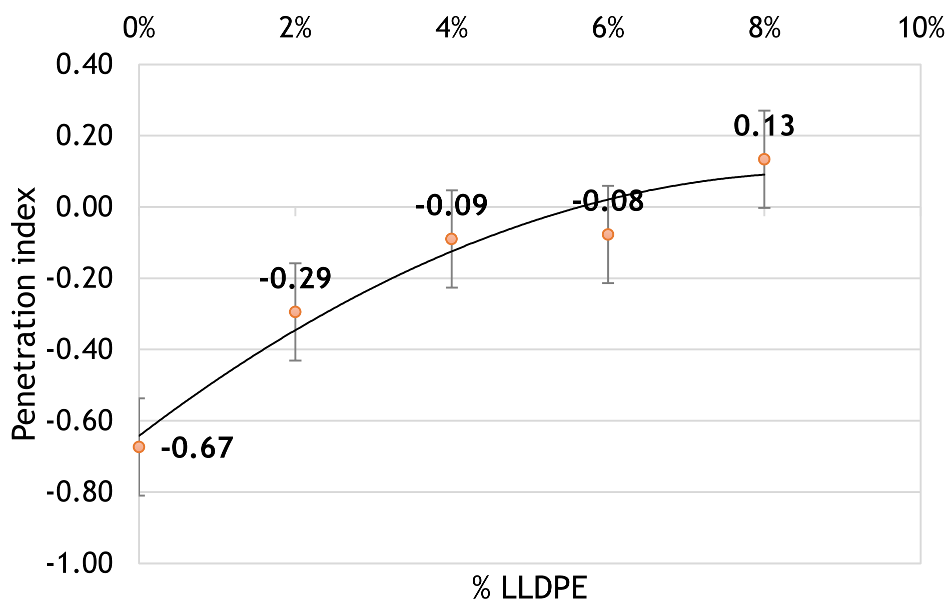


Figure 23: Variation of penetration index with addition of LDPE

According to the European Standards EN 12591, the for paving grade bitumen, the specification for penetration index is in the range, -1.5 to +0.7 with low PI values indicating high temperature susceptibility (higher possibility of rutting at high service temperatures) whereas high PI values indicate low temperature susceptibility (higher possibility of cracking due less flexibility at lower temperatures). However, (Pfeiffer & Van, 1936) recommends that the closer the PI values tends to 0, the better it is for paving grade bitumen since it safe guards against rutting susceptibility and thermo-volumetric cracking. This is because at this point, bitumen tends to equi-viscous state at which the viscosity will remain constant despite the changes in temperature. The trend obtained from the penetration index shows an increase in PI towards 0 for 2%, 4% and 6% LLDPE. Further addition of LLDPE to 8% exceed the 0 mark giving PI of +0.13. This shows that addition of LLDPE to bitumen reduced the temperature susceptibility of the bitumen which implies reduction in potential to soften and flow easily to cause rutting during high temperature.

iv. Bitumen stiffness modulus

The increased loading time and rise in temperature along climbing lanes are crucial factors that affect the bitumen stiffness modulus leading to pseudo rheological properties of asphalt concrete along climbing lanes that leads to rutting.

The stiffness modulus was for each modified binder samples was determined from Van Der Poel's prediction model as described in Figure 14. The operating conditions considered were;

- An average vehicle speed along the climbing lane = 5km/hr = loading time of 0.02s
- Penetration index obtained in table 10 above for each percentage of modified bitumen

- Expected maximum pavement temperature = 50^0C

These conditions were input in to the in order to predict the bitumen stiffness modulus at these service conditions and a graph showing the trend of stiffness modulus of the asphalt mixture was plotted based on the results obtained as shown in Figure 23 below.

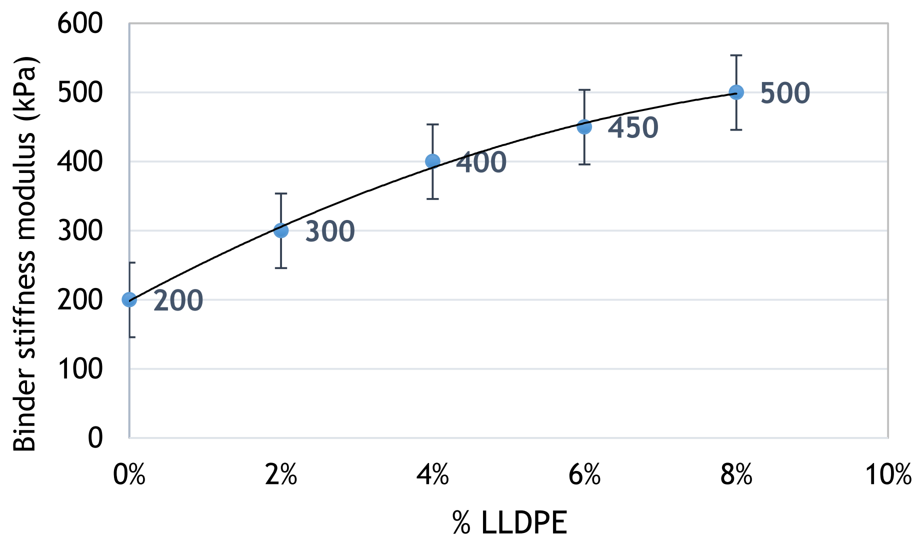


Figure 24: Variation of bitumen stiffness modulus with % LDPE

The graph shown above indicates an increase in the stiffness of the bitumen with addition of LLDPE. It is essential to understand that bituminous materials are rheologic in nature hence at high temperatures and longer loading times (expected along climbing lanes), the bitumen loses its stiffness and behaves as a viscous material hence leading to rutting (A. Al-Suhaibani, 1997). The results shown above indicate that addition of LLDPE has the potential to improve the stiffness of the bitumen at such service conditions expected along climbing lanes. However, it's also important to ensure that as the stiffness increases, the bitumen still has enough flexibility avoid cracking. This index property was assessed using the ductility test as shown below.

v. Bitumen Ductility

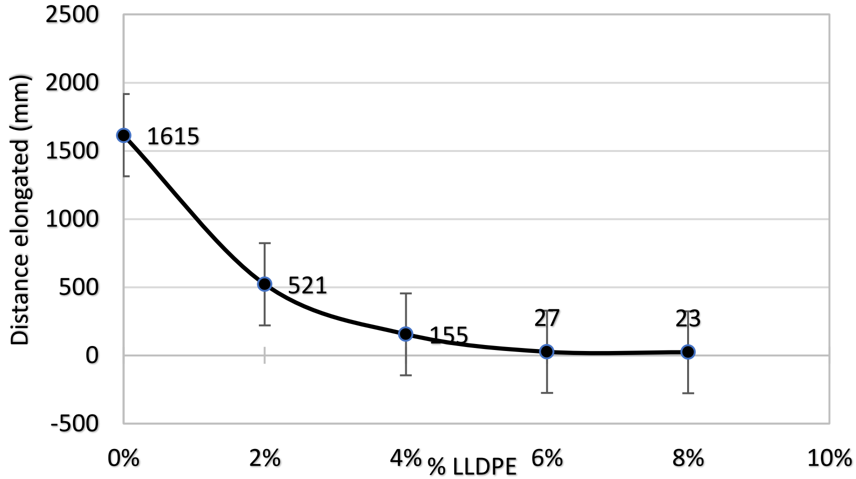


Figure 25: Variation of ductility with % LDPE

Ductility is an essential parameter used to ensure the binder is able to withstand high strain by ensuring it is flexible enough to avoid cracking. According to (Ministry of Works and Transport, 2005) the minimum ductility for SBR and SBS polymer modified bitumen should be 500mm while for EVA, which is plastomer just like LLDPE, the minimum ductility should be 300mm. In relation to these specification, only 2% LLDPE qualifies henceforth is the favorable dosage. However, addition of more polymer in the bitumen achieves a greater portionof the polymeric phase in the bitumen, which makes the binder stiffer and highly capable of resisting deformation. This study therefore considered to assess the performance properties of 2% and 4% as the favorable dosages of LLDPE in the asphalt mixture. These dosage percentages were used to prepare the modified bitumen, whichwas mixed with the aggregates to prepare asphalt specimen that were subjected to Marshall Stability and flow, indirect tensile strength and percentage refusal densitytests to analyze their performance against rutting.

4.4 Performance properties of the modified asphalt mixture

The modified asphalt mix samples were prepared as described in section 3.8.1 and subjected to Marshall test procedure. The asphalt mixture samples that were able to meet the criteria for the Marshall test procedure as described in (Asphalt Institute, 2014) were then subjected to indirect tensile strength test and percentage refusal density.

4.4.1 Tests conducted on the modified asphalt mixture

The results obtained for the performance tests that were conducted are described in this section.

i. Marshall Test results

The modified asphalt mixture samples were subjected to Marshall tests as described in section 3.8.1. This was aimed at obtaining the densities, volumetric properties, Marshall stability and flow values which were analyzed so as to determine the performance properties of the modified asphalt mixtures. The results obtained are shown in table 11 below and compared to the specifications by (Asphalt Institute, 2014).

Table 13: Results obtained from the Marshall tests

Properties	% LLDPE					
	0%	2%	4%	6%	8%	Spec.
Stability	13.27±1.57	16.67±0.31	19.65±1.22	22.62±0.64	23.30±0.83	9 – 18
Flow (mm)	2.8±0.08	2.7±0.22	2.4±0.24	2.1±0.18	1.8±0.38	2 – 4
G_{mb}	2.45±0.01	2.44±0.004	2.38±0.01	2.45±0.001	2.43±0.02	--
G_{mm}	2.456±0.005	2.448±0.001	2.384±0.01	2.454±0.01	2.433±0.02	--
VIM (%)	4.7±0.28	4.9±0.14	6.9±0.44	9.3±0.24	8.3±0.75	3 - 5
VMA (%)	15.2±0.26	15.6±0.14	18.7±0.36	18.5±0.24	18.4±0.69	Min. 14
VFB (%)	68.88±1.42	68.43±0.71	64.07±0.92	49.7±0.80	54.7±2.49	65 - 75

Voids analysis of the modified asphalt mixture

Voids help to signify the volumetric properties which are crucial parameters that influence the mechanical performance of bituminous mixtures. These usually include; voids in mineral aggregates (VMA), air voids (Vv), and voids filled with bitumen (VFB). The volumetric properties shown in table 11 above were ascertained using equations 3.11 to 3.13 and graph plotted to insinuate the trend as the percentage of LLDPE was increased as shown in the graphs below.

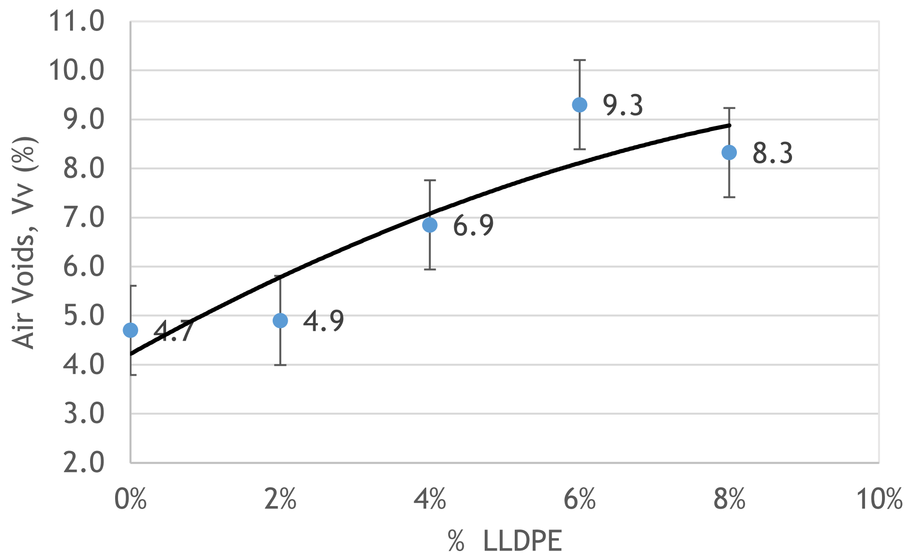


Figure 26: Air voids against % LDPE

The Figure 25 above shows an increasing trend in air voids as the percentage of LLDPE in the bitumen was increased. This is strongly related with the increase in the stiffness of bitumen shown in Figure 23. A research study by (Guangji Xu, 2019) suggests that as the stiffness of bitumen increases, the phenomenon leads to a more porous asphalt mixture with higher voids content due to the reduced flow ability of the bitumen to fill the voids in aggregates. The specification (Asphalt Institute, 1979) recommends a minimum and maximum air void content of 3% to 5% respectively to ensure stability and durability collectively. Research has reported that asphalt mixtures with air voids below 2% are susceptible to instability under loading due to pore pressure buildup and bleeding of the binder at high temperatures hence leading to permanent deformation (Efraem, 2014). Additionally, the study suggests that there would be a danger of loss of durability if voids exceed 6% as a result of voids being interconnected and allowing access to oxidants and moisture in to the asphalt layer. This therefore means only 2% LLDPE meets the requirement for sufficient air voids that will improve stability of the mixture against permanent deformation without impounding its durability.

Voids in Mineral Aggregates (VMA)

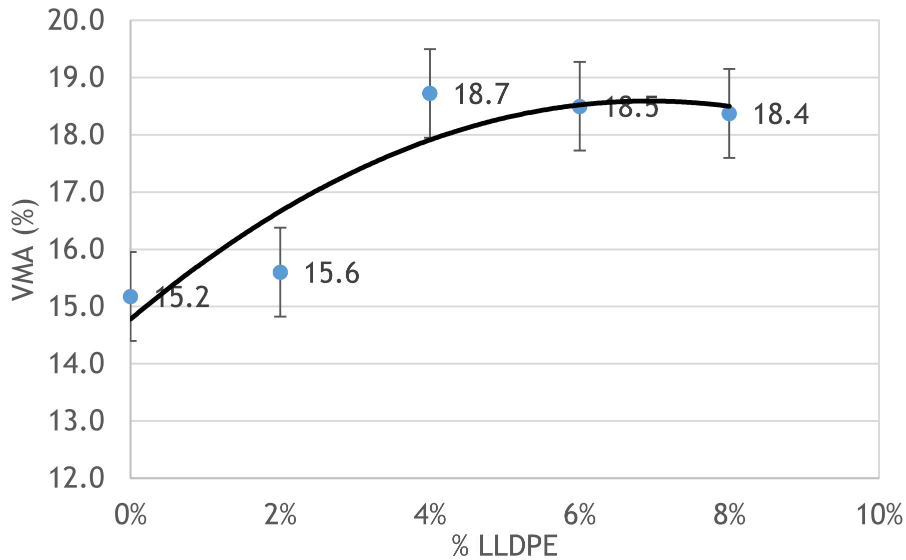


Figure 27: Variation of VMA with % LLDPE

Adequate VMA in the asphalt mixture is essential in ensuring that an adequate film thickness for the bitumen to coat the aggregates and bind the asphalt mixture (Bruce A. Chadbourn, 2000). This helps to prevent bleeding or flushing which would impound on the stability of the asphalt mixture causing deformation. The graph shown above indicates an increase in the VMA which implies there is sufficient voids in between the aggregates to allow for coating and binding of the asphalt mixture while leaving some room for thermo-volumetric expansion and contraction hence improving the stability of the asphalt mixture. Basing on the minimum requirement by the specification, all the percentages of LLDPE modification meet the requirements of at least 14% VMA.

Voids Filled with Bitumen (VFB)

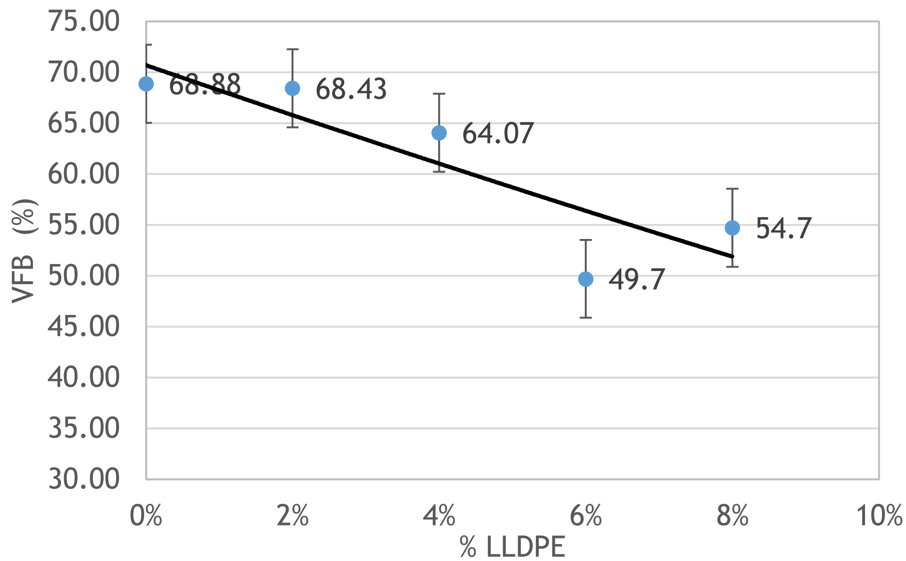


Figure 28: Voids Filled with Bitumen against % LLDPE

The Asphalt Institute specifies a minimum and maximum VFB of 65% and 75% respectively. The minimum VFB requirement ensures that there is sufficient bitumen within the asphalt mixture to coat the aggregates and fill the void spaces so as to meet the requirement for allowable maximum voids content that would not affect the durability of the mixture as stated by (Siswanto, 2016). The maximum VFB ensures that the binder is not excessive. The researcher further suggests that asphalt mixtures with excessive binder content tend to have rut faster majorly because the excessive bitumen contradicts with aggregate interlocking properties of the asphalt mixture hence causing rutting when pavement temperatures increase. The Figure 27 above shows a decreasing trend in the VFB as the percentage of LLDPE was increased. It is clear that only 2 % meets the criteria for minimum and maximum VFB.

Marshall Stability

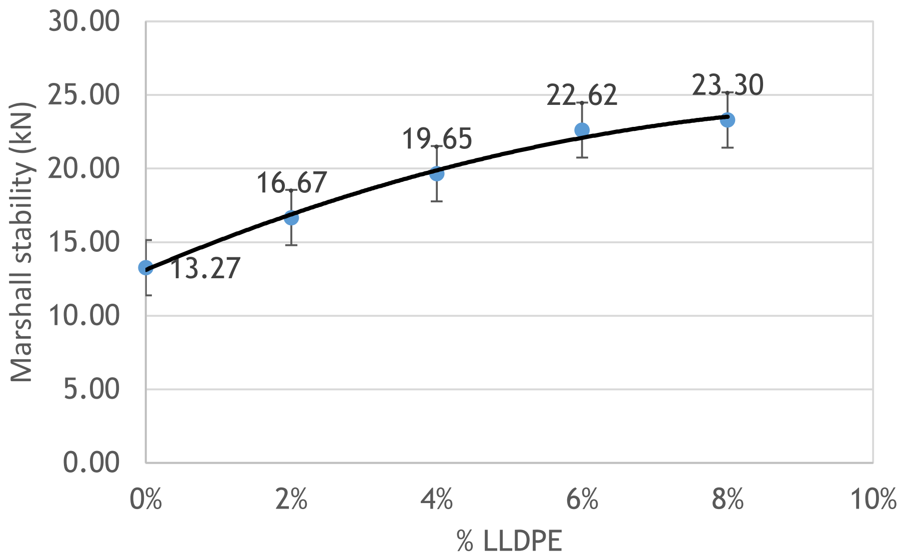


Figure 29: Marshall stability against % LLDPE

The Figure 28 above shows an increasing trend in stability of the asphalt mixture as the percentage of LLDPE was increased. This can be related to the increase in stiffness modulus of the binder and improved temperature performance (low temperature susceptibility portrayed by improved penetration index) due to addition of LLDPE since Marshall Stability test was carried out at elevated temperatures (60^{0}C). A research study by (Siswanto, 2016) suggested that binders with higher stiffness and better high temperature performance are stable and exhibit improved resistance to rutting in asphalt pavements. However, higher stability values are indicative of an overly stiff mixture which decrease the flexibility hence causing thermal, fatigue and reflective cracking. This impounds on the durability of the pavement as noted by (F. M. Nejad, 2018). The minimum and maximum requirements for Marshall stability of asphalt specimen is 9kN and 18kN respectively. This indicates that only 2% LLDPE meets the required specifications.

Flow

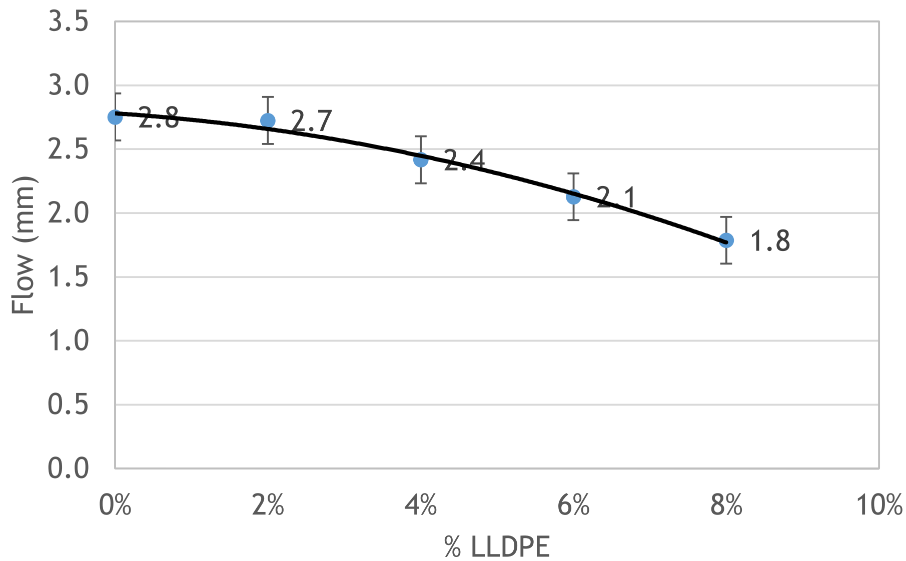


Figure 30: Flow against % LLDPE

The graph above indicates a decrease in the flow with addition of LLDPE which means a decreased ability of the asphalt mixture to deform and flow under elevated temperatures and loading. Binders with higher flow values tend to exhibit greater susceptibility to rutting when exposed to high temperatures and traffic loading. The decrease in the flow value is related to the increased stiffness to withstand heavy loads and the low temperature susceptibility of the bitumen. This therefore means the LLDPE was capable of increasing the rutting resistance of the asphalt mixture against rutting under elevated temperatures and loading. However, further modification up to 8% LLDPE lead to very low flow values below the minimum requirement by (Asphalt Institute, 1979) of 2mm. This means that the asphalt binder became overly stiff leading to decreased flexibility which would cause an increased susceptibility to cracking hence impairing on the durability of the asphalt pavement.

Based on the above Marshall tests, it was evident that addition of LLDPE in the bitumen improved the stiffness of the asphalt mixture and lowered the temperature susceptibility which speaks to the ability to maintain consistency under elevated temperatures. However further modification made the asphalt mixture overly stiff which would affect the durability of the asphalt pavement. Only 2% LLDPE produced an asphalt mixture that was able to improve the resistance to permanent deformation while maintaining the durability of the asphalt mixture. This study therefore deduced that 2% bitumen modification was the favorable dosage of LLDPE to use in the preparation of the asphalt mixture.

ii. Indirect Tensile Strength Test

According to (Asphalt Institute, 2014) having come up with an asphalt mix design and determined the stability and durability in the mixture design process, it is essential that the mixture is tested for moisture susceptibility. It is essential that the mixture meets all aggregate and volumetric requirements prior tothis test. For this case therefore, the only modified asphalt mixture sample that met all these requirements was the 2% LLDPE modified asphalt mixture. The results obtained are shown below and compared with the specifications for (Ministry of Works and Transport, 2005) and (Asphalt Institute, 2014).

Table 14: Test results for indirect tensile strength

Property	Unmodified asphalt	2% LLDPE modified asphalt	Specification
ITS unconditioned	901.8	1052.3	Min. 800 kPa
ITS conditioned	743.0	872.0	80% of unconditioned
TSR	0.82	0.83	0.80

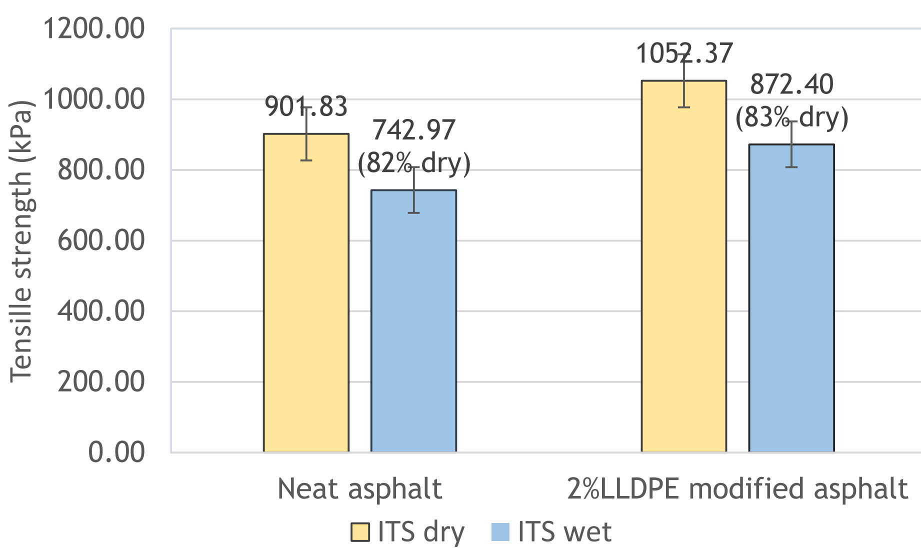


Figure 31: Indirect tensile strength for neat and modified asphalt mix

According to (Asphalt Institute, 2014), research shows that a minimum TSR value of 0.80 (80%) is considered acceptable and indicates an asphalt mixture that is not susceptible to moisture damage. This is similar to the specification by (Ministry of Works and Transport, 2005) which states that the ITS obtained for the conditioned specimen should be at least 80% that of the unconditioned (dry) specimen. This therefore implies the 2% LLDPE modified sample specimen is safe against moisture damage with a slight increase in resistance to moisture damage as compared to the unmodified sample as indicated by the 1% increase in the TSR value. The Asphalt institute further recommends that agencies set a minimum indirect tensile strength to carter for the concern that a mixture may have insufficient tensile strength to resist deformation under load. The MoWT specification states a minimum tensile strength value of 800kPa for the unconditioned state. The results obtained as shown above indicated that the modified asphalt mixture had sufficient tensile strength with an increase in tensile strength by about 16% when compared with the tensile strength obtained for the unmodified specimen.

iii. Percentage refusal density (Vibrating hammer compaction)

The refusal density compaction method which employs the electric vibrating hammer compaction was used as to attain the voids at refusal density in accordance to BS 598-104. Asphalt mixtures comprising of the neat asphalt mixture and the 2% LLDPE modified asphalt mixture. These were subjected to 2 minute vibration on each face to refusal density compaction as described in section 3.8.1 and the retained air voids at refusal density was determined using equation 3.19. The results obtained are described in Table 15 below.

Table 15: Air voids at refusal density

% LLDPE	Air voids at refusal (%)	Specification
0%	3.3 ± 0.15	Min. 3%
2%	4.7 ± 0.06	Min. 3%

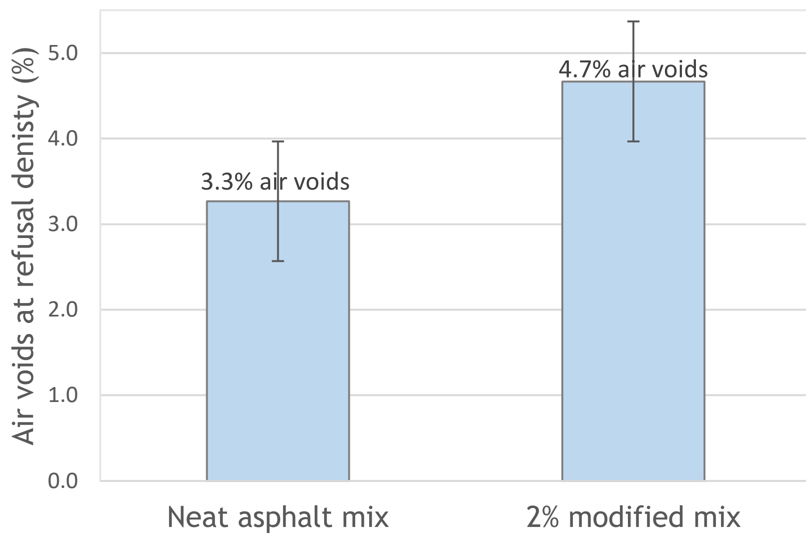


Figure 32: Voids at refusal density plotted against %LLDPE

From the Figure 31 above, it is evident that 2% LLDPE modified asphalt mixture was capable of retaining the minimum required 3.0 percent air voids as recommended by the MoWT specification for Uganda. When compared to the control mixture which was the neat asphalt mixture, the amount of retained air voids at refusal density increased by 42%. This indicates that when subjected to secondary compaction, the asphalt mix will retain a good amount of air voids to allow for flexibility under heavy loading hence reducing the likelihood of permanent deformation or rutting. This also is indicative of a prolonged pavement life hence avoiding the scenario of premature deformation as such severely loaded asphalt sections along climbing lanes (Eri Susanto Hariyadi, 2007).

However, it is crucial to note that such an asphalt mixture with higher air voids at refusal compaction is prone to age hardening since the voids allow for room for oxidants which in the long run could impact on the durability of the pavement (Transport Research Laboratory, 2022). This research study therefore recommends that a surface dressing should be laid on top to protect the mix from severe age hardening especially for cases where secondary compaction may not occur for areas which are not along the wheel path and may not be heavily trafficked thereby retain air voids above 5% which affects pavement durability.

4.5 Asphalt mix design

From the trial mixes prepared and test results analyzed from the performance properties of the various asphalt mixes, it was observed that the only 2% modified asphalt mixture was capable of improving the resistance to permanent deformation of the asphalt mixture while meeting all the various criteria specified by the test specifications. This research was therefore capable of establishing a job mix formula for the asphalt mixture that could be used as base course for such severely loaded pavements along climbing lanes comprising of 2% LLDPE as a bitumen modifier as stated in table

Table 16: Asphalt mix design to deter rutting at the climbing lane

Aggregate size fractions	% composition of constituents (%)	Mass composition in 1200g (g)
20/14	25	286.8
14/10	12	137.7
10/6	8	91.8
3/6	53	608.0
Filler	2.0	22.9
Total Agg	100	1147.2
2% LLDPE modified Bitumen	4.4	52.8
Total mix		**1200.0**

Where;

Mass of modified bitumen = 98% mass of total bitumen required + 2% LLDPE

For example in order to prepare 1 kg of the modified bitumen, it would require;

1000g of modified bitumen = 980g of bitumen + 20g of LLDPE

It should be noted that based on the voids at refusal density obtained, the voids increased which shows the good ability of the asphalt mixture to resist pavement deformation however, to ensure smooth riding quality, prevention of the mixture from rapid hardening due to ageing, this asphalt mixture should be used as a base course ans a surface dressing placed on top to protect the base course asphalt mixture.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The tests conducted on bitumen and aggregates showed the aggregates exhibited good properties hence capable of resisting permanent deformation. However considering the resistance to rutting in asphalt mixture is a combined effort of the binder proving stiffness and resistance to shear flow and the aggregate interlocking capability and strength. This research identified that the bitumen was susceptible to temperature fluctuations hence causing leading to high capability to soften and weaken the stiffness of the asphalt mixture hence leading to shear flow that manifests as rutting in flexible pavements. This initiated the study to modify the bitumen and improve the rheological properties of the binder.

The asphalt mix design procedure requires determining an optimum binder content that will ensure the durability and stability are not compromised. The Marshall mix procedure was conducted in accordance with the asphalt institute procedure and an optimum bitumen content of 4.4% was obtained. This was used in preparing the bituminous mix samples.

Having modified the bitumen with varying percentages of LLDPE, the modified bitumen was evaluated to assess the influence of the LLDPE on the rheological properties of the bitumen. The study identified that the temperature susceptibility of the bitumen was lowered and the stiffness improved with addition of LLDPE. This meant that the binder was capable of resisting softening at high service temperatures while maintaining consistency and high stiffness. This would improve its resistance to rutting at reduced traffic speeds (longer loading times) and increase in pavement temperatures.

The performance of the modified asphalt mixture was evaluated using Marshall test, indirect tensile strength and percentage refusal density. The results obtained from the Marshall tests indicated that only 2% LLDPE was capable of improving the resistance to higher loads and increasing pavement temperatures while maintaining the durability of the asphalt mixture. The modified asphalt mixture with 2% LLDPE was subjected to Indirect Tensile Strength testing to evaluate its performance against moisture induced damage. The study identified that the Tensile Strength was improved by 16.7% as compared to the neat asphalt mixture. The air voids at refusal density were determined and indicated an increase by from 3.3% air voids to 4.7% when compared to the unmodified asphalt mixture. This was indicative that the asphalt mixture was capable of retaining a good amount of air voids in the asphalt mixture to ensure stability and flexibility after secondary compaction. This study therefore concluded that modification of asphalt mixtures with 2% modification are capable of improving the performance of flexible pavements against heavy loading and increased temperatures which could cause permanent deformation especially along severely loaded sections such as climbing lanes.

5.2 Recommendations

1. The study recommends that whereas LLDPE used for this study was in form of pellets, it could further be turned in to powder to increase its surface area and workability. This would make the mixing process even easier.
2. The use of other bitumen modifiers such Ethylene Vinyl Acetate (EVA) can be further researched about to identify their potential in improving the resistance to permanent deformation along such severely loaded sections.
3. The effect of oxidation and other ageing elements on the LLDPE modified bitumen canbe further studied to ascertain its effect on the asphalt mixture. The storage stability of the LLDPE and other polymer modified binders can also be further studied to identify how this aspect can be improved.

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APPENDIX A

Mixing aggregates and bitumen

Compacting specimen to refusal density

Compacted asphalt specimen

Un-compacted asphalt mixture for Gmm

Testing for maximum specific gravity of asphalt mixture

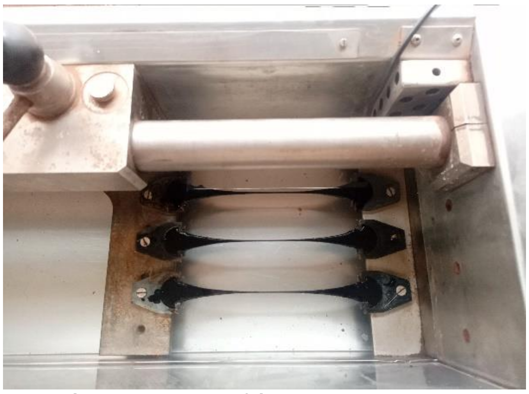
Determining sample thickness and diameter

Mixing bitumen and LLDPE homogenously

Conducting softening point test of bitumen


Conducting the bitumen penetration test


determining Marshall stability and flow


Ductility testing of bitumen


ITS wet samples sealed in air tight bags

APPENDIX B: LABORATORY TESTS RESULTS

INSTITUTION	CLIENT	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling
PROJECT:	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF	

A.C.V. LABORATORY TEST RESULT FORM

(BS 812PART 110:1990)

LOCATION:	MUKONO SITE	Oparator	15/Jan/24
MATERIAL DESCRIPTION:	AGGREGATES FOR ASPHALT	Date	16/Jan/24

A.C.V

(A) WT BEFORE CRUSHING (gm)	2860.5	2826.2	2799.6
(B) WT AFTER CRUSHING (gm)	2859.5	2825.8	2799.1
(C) WT RETAINED AFTER CRUSHING (gm)	2371.8	2363.5	2335.7
(D) WT PASSING SIEVE 2.36 mm	488.7	462.7	463.9
A.C.V(%) (D/B)*100	17.1	16.4	16.6
AVERAGE RESULTS %	16.7		

NB more than B by 10gms repeat the test

A.I.V

(A) WT BEFORE TEST (gm)	356.2	358.1	367.5
(B) WT AFTER TEST (gm)	355.5	357.9	367.5
(C) WT RETAINED AFTER TEST (gm)	300.5	305.9	311.1
(D) WT PASSING SIEVE 2.36 mm	55.7	52.2	56.4
A.I.V(%) (D/B)*100	15.7	14.6	15.3
AVERAGE RESULTS %	15.2		

NB If c+d is more than B by 1gms repeat the test

SPECIFIED LIMITS IN ACCORDANCE WITH TYPE OF MATERIAL

FOR TESTING LAB

Lab Technician

Materials Engineer

STIRLING CIVIL ENGINEERING LTD
29 JAN 2024
P. O. BOX 796, KAMPALA (U)

INSTITUTION	NAMES	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT: INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

DETERMINATION OF AGGRGATE'S 10% FINES VALUE DRY AND SOAKED

(BS 812PART 111:112:1990)

LOCATION:	Lab	OPERATOR	
		DATE SAMPLED	15 January 2024
MATERIAL DESCRIPTION:	AGGREGATES FOR ASPHALT	DATE TESTED	16 January 2024

10% FINE VALUE DRY

TEST NO	1	2	3
CRUSHING FORCE (KN)	263	263	263
WT. OF AGGREG (gm)after crushing (M1)	2832.9	2868.7	2,896 3
WT. OF AGGREG. RETAINED ON SIEVE 2.36 mm (M3)	2570 1	2610.7	2641.2
WT.AGGREG.(gm) PASSING SIEVE 2.36 mm (M2)	262.8	258.0	255.1
TEN % FINE VALUE (M=M2/M1*100)	9.3	9.0	8.8
AVERAGE RESULTS % (M)		9.0	
AVERAGE CRUSHING FORCE (F)		263.2	

F= $\frac{14F}{M+4}$ 282.8 DRY 282.8 KN

10% FINE VALUE SOAKED

TEST NO	1	2	3
CRUSHING FORCE (KN)	263	263	263
WT. OF AGGREG (gm)after crushing (M1)	2863.0	2864.0	2851 8
WT. OF AGGREG. RETAINED ON SIEVE 2.36 mm (M3)	2621.3	2625.3	2608 5
WT.AGGREG.(gm) PASSING SIEVE 2.36 mm (M2)	241.7	238.7	243 3
TEN % FINE VALUE (M=M2/M1*100)	8.4	8.3	8 5
AVERAGE RESULTS % (M)		8.4	
AVERAGE CRUSHING FORCE (F)		263 2	

F= $\frac{14F}{M+4}$ 296.3 SOACKED WET/DRY(%)= 105

SPEC >110 SPEC >75%

f= Maximum force (KN)

of material passing the 2.36mm sieve at the maximum force

SPEC REQUIREMENT 7.5%-12.5% (BS 812-111) (if <or> discard

TESTING LAB

Materials Engineer

STIRLING CIVIL ENGINEERING LTD
19 JAN 2024
P. O. BOX 796, KAMPALA (U)

INSTITUTION	CLIENT	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY A Centre of Excellence in the Heart of Africa	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT	**INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES**

RESISTANCE TO DEGRADATION
BY ABRASION AND IMPACT TO
LOS ANGELES MACHINE
(AASHTO T96 - 99)

JOB:	MUKONO SITE	OPERATOR	
LOCATION :	MUKONO CRUSHER	TOTAL BY DRY WT. OF THE SAMPLE:1	5,000.0
SUPPLIER:	STIRLING	TOTAL BY DRY WT. OF THE SAMPLE:2	5,000.0
MATERIAL:	AGGREGATES FOR ASPHALT	DATE SAMPLED:	15/Jan/2024
SPECIFICATION...		DATE TESTED:	16/Jan/2024

Test 1 Grading of Test Samples

SIEVE SIZE		Mass of indicated Sizes,g			Grading
Passing	Retained on	A 12 balls	B 11balls	C 8 balls	D 6balls
mm 20	**10**				
37.5 (1 1/2in)	25.0 (1 in)	1250 ± 25	......	......	......
25.0 (1 in)	19.0 (3/4 in)	1250 ± 25	......	......	......
19.0 (3/4 in)	12.5 (1/2 in)	1250 ± 10	2500 ± 10	......	......
12.5 (1/2 in)	9.5 (3/8 in)	1250 ± 10	2500 ± 10	......	......
9.5 (3/8 in)	6.3 (3/4 in)	.....	.....	2500 ± 10	......
6.3 (3/4 in)	4.75 (No. 4)	.....	.....	2501 ± 10	......
4.75 (No. 4)	2.36 (No. 8)	.....	.....	......	5000 ± 10
TOTAL:........................		5000 ± 10	5000 ± 10	5000 ± 10	5000 ± 10

Speed of Rotation: 33Rev/min. Max. 500 Rev.

Max.Duration 15 min			Wt after crushing:	4,972.3
GRADING USED FOR TEST:	SAMPLE: 1	SAMPLE: 2	Wt after crushing :	4,982.5
Wt of Mat. Retained on 1.7mm sieve :	4,144.5	4,132.0		
Wt of fine material _	855.5	868.0	Average: %	17.2
Percentage of wear_ %	17.1	17.4	Spec Req	40%

FOR TESTING LAB

LAB TECHNICIAN

MATERIALS ENGINEER

STIRLING CIVIL ENGINEERING LTD
JAN 2024
P. O. BOX 795, KAMPALA (U)

INSTITUTION	STUDENTS	CONTRACTOR
UGANDA CHRISTIAN UNIVERSITY A Centre of Excellence in the Heart of Africa	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling
PROJECT	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES	

FLAKINESS INDEX OF AGGREGATES (BS 812 PART 105.2 1990)

Location: STIRLING LAB

Operator:

Material: COMBINED AGGREGATES FOR AC 20 ASPHALT

Date: 1/20/2024

weight of sample (gms) 2107.4

BS sieve Size	Weight Retained (gm) (from grading sheet)	% Retained
28mm	0	0.0
20mm	0	0.0
14mm	87.0	4.1
10mm	141.0	6.7

s less than 5% that size is not tested for flakiness

BS sieve size(mm)			28.0	20.0	14.0	10.0		Total
Weight retained gm (A)					87.0	141.0		228
iffled weight (if needed) gm (B)					255.2	285.8		
Correction factor a/b (C)					0.3	0.5		
Wt. Passing sieve gm (D)					33.0	66.2		
Wt. Retained on sieve gm (E)					54.0	74.8		
Corrected Wt. Passing (dxc) (F)					11.3	32.7		43.9

Flakiness Index $= \frac{TOTAL F}{TOTAL A} * 100$ 19.3

FOR STUDENTS

Lab Technician

Materials Engineer

STIRLING CIVIL ENGINEERING LTD P.O. BOX 796, KAMPALA (U)

INSTITUTION	STUDENTS	TESTING LAB		
UGANDA CHRISTIAN UNIVERSITY	**AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)**	**Stirling**		
PROJECT	**INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES**			
SAMPLE DISCRIPTION	STONE DUST	Sampling Date		1-Mar-24
TEST METHOD	DETERMINATION OF SAND EQUIVALENT			
TEST METHOD	AASHTO D2419			
S.no	**Description**	**Sample 1**	**Sample 2**	**Sample 3**
1	Sand Reading (A)mm	20.2	18.55	14.78
2	Clay reading (B)mm	48.36	45.22	30.89
3	Sand Equivalent SE % (A/B)*100	**42**	**41**	**48**

FOR TESTIN LAB

Lab Technician *Materials Engineer*

STIRLING CIVIL ENGINEERING LTD
P. O. BOX 796, KAMPALA (U)

INSTITUTION	CLIENT	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT	INVESTIGATING THE ÚSE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES
TEST	**SPECIFIC GRAVITY**
TEST METHOD	**ASTM:C128-97**

Sample Ref:	**AC 14 MM**	Technician :	
SOURCE:	**Mukono Stirling quarry**	Sampling date:	1/15/2024
Aggregate size :	**COMBINED**	Testing date:	1/17/2024
Description of aggregates:	**HOT BINS**		

Aggregate size :	**20-14**	**14-10**	**10-6.0**	**6.0-0**	**FILLER**
GS bulk :	2.623	2.618	2.614	2.607	2.635
PROPORTIONS:	5	7	18	66	4
COMBINED SG :	2.611				
WATER ABSOPTION	0.2	0.3	0.3	0.4	
COMBINED WATER ABSOPTION	0.3				

REMARKS

FOR TESTNG LAB

Lab Technician

Materials Engineer

STIRLING CIVIL ENGINEERING LTD
P. O. BOX 796, KAMPALA (U)

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT:	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

SPECIFIC GRAVITY FILLER (AASHTO T100-95 (1995))

LOCATION: Mukono Lab	OPERATOR:
SAMPLE No	SAMPLE DATE: 1/15/2024
TYPE: Filler	TESTING DATE: 1/17/2024

	Beaker K		Beaker 1
[A) Wt. OVEN dry sample (gm	490.43		483.28
[B) Wt. of Pycnometer containing water alone (gm)	1805.95		1768.7
[C] Wt of Pyconometer containing Sample and water (	2109.49		2069.36
SPECIFIC GRAVITY OF FILLER $\frac{A}{A+(B-C)}$	2.624		2.646
AVERAGE	2.635		

FOR TESTING LAB

Lab Technician

STIRLING CIVIL ENGINEERING LTD

Materials Engineer

P. O. BOX 795, KAMPALA (U)

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY A Centre of Excellence in the Heart of Africa	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

SPECIFIC GRAVITY & WATER ABSORPTION FINE AGGREGATES

(AASHTO ; T84—00)

ASTM DESIGNATION ; C128—97

LOCATION:		OPERATOR:	
SAMPLE No		SAMPLE DATE:	1/15/2024
TYPE:	QUARRY DUST (O/6 mm)	TESTING DATE:	1/17/2024

TEST NO		1		K
[A] wt. of oven dry sample in air	(gm)	567.54		547.12
[B] wt. of pycnometer filled with water	(gm)	1805.71		1771.19
[C] wt. of pycnometer with specimen and water	(gm)	2157.22		2110.82
[S] wt of saturated surface dry sample	(gm)	569.44		549.28
Bulk Specific Gravity on oven dry basis	(B-C) / (B+S-C)	2.604		2.610
Bulk Specific Gravity on saturated surface dry basis	S / (B+S-C)	2.613		2.620
Apparent Specific Gravity $\frac{A}{(C-B)+A}$ $\frac{A}{A-(C-B)}$	A / 100(B-A)	2.627		2.637
Water Absorption(%)= $\left(\frac{S-A}{A}\right)100$	A / A	0.3		0.4

BULK SPECIFIC GRAVITY	2.607
BULK SPECIFIC GRAVITY ON SATURATED SURFACE DRY BASIS	2.616
APPARENT SPECIFIC GRAVITY	2.632
WATER ABSORPTION	0.4

FOR TESTING LAB

Lab Technician

Materials Engineer

STIRLING CIVIL ENGINEERING LTD
P. O. BOX 796, KAMPALA (U)

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY A Centre of Excellence in the Heart of Africa	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling
PROJECT	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES	

SPECIFIC GRAVITY & WATER ABSORPTION COARSE AGGREGATES

(AASHTO ; T85—91)

ASTM DESIGNATION ; C127—88

LOCATION:	Mukono Quarry	OPERATOR:	
SAMPLE No		SAMPLE DATE:	1/15/2024
TYPE:	10 - 6 mm	TESTING DATE:	1/17/2024

TEST NO		A	B	C
[A] wt. of oven dry sample in air	(gm)	2088.4		2033.8
[B] wt. of saturated surface dry sample in air	(gm)	2094.2		2041.8
[C] wt of saturated sample in water	(gm)	1296.1		1262.8
Bulk Specific Gravity on oven dry basis	A / (B-C)	2.617		2.611
Bulk Specific Gravity on saturated surface dry basis	B / B-C	2.624		2.621
Apparent Specific Gravity	A / A-C	2.636		2.638
Water Absorption(%)=	100(B-A) / A	0.3		0.4

AVERAGE RESULTS	
BULK SPECIFIC GRAVITY	2.614
BULK SPECIFIC GRAVITY ON SATURATED SURFACE DRY BASIS	2.623
APPARENT SPECIFIC GRAVITY	2.637
WATER ABSORPTION	0.3

FOR TESTING LAB

Lab Technician

Materials Engineer

STIRLING CIVIL ENGINEERING LTD
P. O. BOX 795, KAMPALA (U)

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY A Centre of Excellence in the Heart of Africa	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling
PROJECT	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES	

SPECIFIC GRAVITY & WATER ABSORPTION COARSE AGGREGATES

(AASHTO ; T85—91)

ASTM DESIGNATION ; C127—88

LOCATION:	Mukono Quarry	OPERATOR:	
SAMPLE No		SAMPLE DATE:	1/15/2024
TYPE:	14-10 mm	TESTING DATE:	1/17/2024

TEST NO		A	B	C
[A] wt. of oven dry sample in air	(gm)	2021.4		2014.0
[B] wt. of saturated surface dry sample in air	(gm)	2026.6		2019.7
[C] wt of saturated sample in water	(gm)	1254.6		1250.3
Bulk Specific Gravity on oven dry basis	A / (B-C)	2.618		2.618
Bulk Specific Gravity on saturated surface dry basis	B / B-C	2.625		2.625
Apparent Specific Gravity	A / A-C	2.636		2.637
Water Absorption(%)=	100(B-A) / A	0.3		0.3

AVERAGE RESULTS	
BULK SPECIFIC GRAVITY	2.618
BULK SPECIFIC GRAVITY ON SATURATED SURFACE DRY BASIS	2.625
APPARENT SPECIFIC GRAVITY	2.637
WATER ABSORPTION	0.3

FOR TESTING LAB

STIRLING CIVIL ENGINEERING LTD

Lab Technician

Materials Engineer

P. O. BOX 795, KAMPALA (U)

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY A Centre of Excellence in the Heart of Africa	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling
PROJECT	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES	

SPECIFIC GRAVITY & WATER ABSORPTION COARSE AGGREGATES

(AASHTO ; T85—91)

ASTM DESIGNATION ; C127—88

LOCATION: Mukono Quarry	OPERATOR:
SAMPLE No	SAMPLE DATE: 1/15/2024
TYPE: 14-20 mm	TESTING DATE: 1/17/2024

TEST NO			A	B	C
[A] wt. of oven dry sample in air	(gm)		2427.1		2184.8
[B] wt. of saturated surface dry sample in air	(gm)		2431.0		2188.5
[C] wt of saturated sample in water	(gm)		1506.9		1354.6
Bulk Specific Gravity on oven dry basis		A / (B-C)	2.626		2.620
Bulk Specific Gravity on saturated surface dry basis		B / B-C	2.631		2.624
Apparent Specific Gravity		A / A-C	2.638		2.632
Water Absorption(%)=		100(B-A) / A	0.2		0.2

AVERAGE RESULTS	
BULK SPECIFIC GRAVITY	2.623
BULK SPECIFIC GRAVITY ON SATURATED SURFACE DRY BASIS	2.628
APPARENT SPECIFIC GRAVITY	2.635
WATER ABSORPTION	0.2

FOR TESTING LAB

Lab Technician

Materials Engineer

STIRLING CIVIL ENGINEERING LTD
P. O. BOX 795, KAMPALA (U)

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling
PROJECT	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES	

JOB ASPHALT MIX DESIGN

LOCATION MUKONO LAB

18-Jan-24

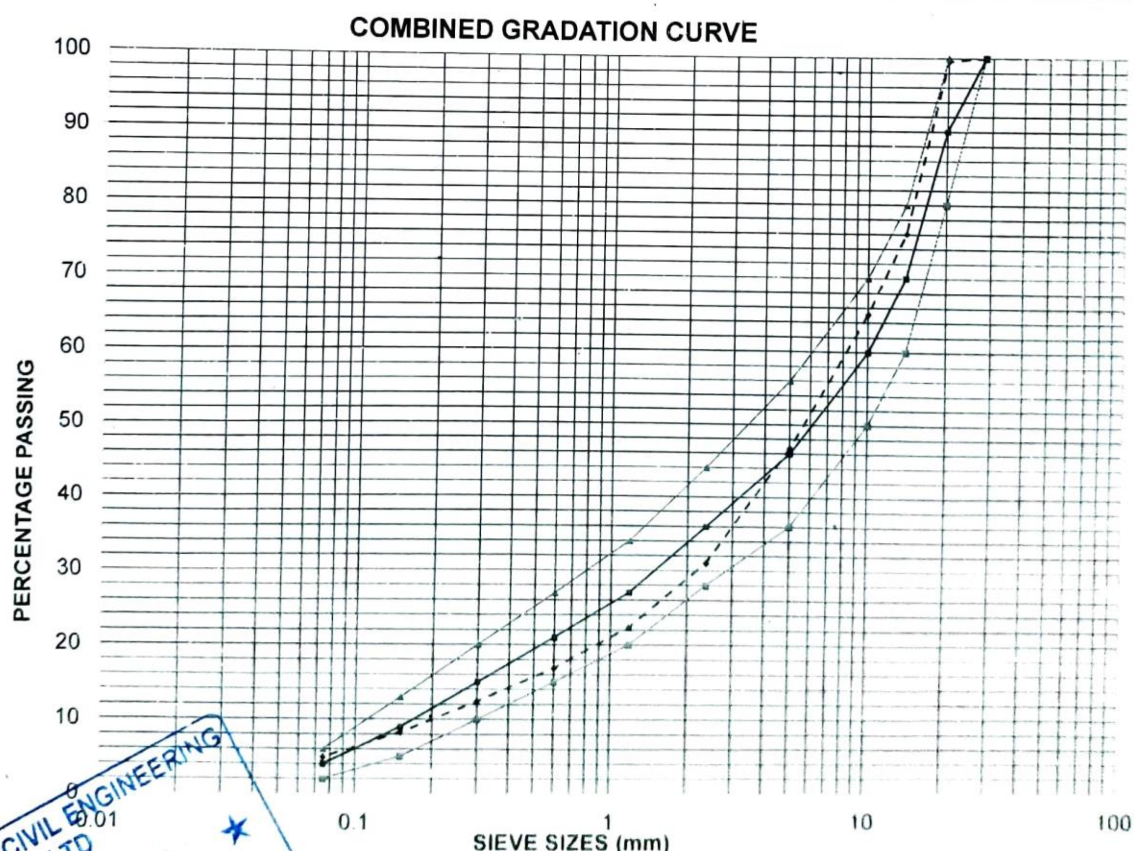
SUPPLIER HOTBIN No1

SAMPLE No

MATERIAL AC 20 INDIVIDUAL GRADATION

	14/20MM		10/14MM		6/10MM		0/6MM		FILLER		actual	TARGET GRADING	SPEC
		25.0		12.0		8.0		53.0		2.0	100.0		
28	100.0	25.0	100.0	12.0	100.0	8.0	100.0	53.0	100.0	2.0	100	100	100
20	98.6	24.7	99.4	11.9	100.0	8.0	100.0	53.0	100.0	2.0	100	90	80—100
14	18.7	4.7	70.3	8.4	99.4	8.0	100.0	53.0	100.0	2.0	76	70	60—80
10	1.6	0.4	18.7	2.2	93.9	7.5	100.0	53.0	100.0	2.0	65	60	50—70
5	0.4	0.1	1.3	0.2	6.6	0.5	83.0	44.0	100.0	2.0	47	46	36—56
2.36	0.4	0.1	0.9	0.1	5.1	0.4	53.7	28.5	100.0	2.0	31	36	28—44
1.18	0.4	0.1	0.8	0.1	4.3	0.3	37.4	19.8	100.0	2.0	22	27	20—34
0.6	0.4	0.1	0.7	0.1	3.6	0.3	27.3	14.4	99.8	2.0	17	21	15—27
0.3	0.4	0.1	0.6	0.1	3.0	0.2	19.1	10.1	95.8	1.9	12	15	10—20
0.15	0.4	0.1	0.5	0.1	2.5	0.2	12.0	6.3	79.7	1.6	8	9	5—13
0.075	0.4	0.1	0.4	0.0	2.2	0.2	6.5	3.4	58.4	1.2	5	4	2—6

COMBINED GRADATION CURVE



FOR TESTING LAB

FOR STUDENTS

STIRLING CIVIL ENGINEERING LTD
JAN 2024
P. O. BOX 796, KAMPALA (U)

INSTITUTION	STUDENT'S NAME	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	**AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)**	**Stirling**

PROJECT	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

LOCATION	**BITUMEN TESTS**	OPERATOR Lab team
SUPPLIER		CONTAINER/DELIVERY NO
NEAT BITUMEN 12-Jan-24		
MATERIAL TYPE 60/70		DESTINATION

TEST NO	IFF		3		L	D	S2	R	AVERAGE	REMARKS
PENETRATION 100gr 5 sec 25 C	62 65 63		60 62 61		62 63 63	65 62 65	60 62 61	62 61 63	62	60-70
SOFTENING POINT (°C)		49		51					50	(49-56)°C
BITUMEN AFFINITY										>95
SPECIFIC GRAVITY	1.029		1.028		1.024	1.028	1.028	1.028	1.027	1.01-1.06

Materials Engineer.

STIRLING CIVIL ENGINEERING LTD P. O. BOX 795, KAMPALA (U)

INSTITUTION	STUDENT'S NAME	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY A Centre of Excellence in the Heart of Africa	**AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)**	**Stirling**

PROJECT	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

MODIFIED WITH 2% PLASTIC

BITUMEN TESTS

LOCATION	OPERATOR Lab team
SUPPLIER	CONTAINER/DELIVERY NO
16-Jan-24	DESTINATION
MATERIAL TYPE 60/70	

TEST NO	2B		CM		RL		RS		AVERAGE	REMARKS
PENETRATION 100gr 5 sec 25 C	50 48 49		47 49 52		50 50 50		48 47 48		49	60-70
SOFTENING POINT (°C)		53		55					54	(49-56)°C
BITUMEN AFFINITY										>95
SPECIFIC GRAVITY	1.018		1.019		1.019		1.016		1.018	1.01-1.06

Materials Engineer

STIRLING CIVIL ENGINEERING LTD P. O. BOX 793, KAMPALA (U)

INSTITUTION	STUDENT'S NAME	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY A Centre of Excellence in the Heart of Africa	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT: INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

MODIFIED WITH 4% PLASTIC

LOCATION

SUPPLIER

16-Jan-24

BITUMEN TESTS

OPERATOR: Lab team

CONTAINER/DELIVERY NO

DESTINATION

MATERIAL TYPE 60/70

TEST NO	NG		4I		RM		IFF		AVERAGE	REMARKS
PENETRATION 100gr 5 sec 25 C	39 40 41		40 40 42		39 42 39		40 39 38		40	60-70
SOFTENING POINT (°C)		59		55					57	(49-56)°C
BITUMEN AFFINITY										>95
SPECIFIC GRAVITY	1.008		1.010		1.010		1.007		1.009	1.01-1.06

Materials Engineer. ____

STIRLING CIVIL ENGINEERING LTD, P. O. BOX 796, KAMPALA (U)

INSTITUTION	STUDENT'S NAME	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT: INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

MODIFIED WITH 6% PLASTIC

LOCATION	BITUMEN TESTS	OPERATOR: Lab team
SUPPLIER		CONTAINER/DELIVERY NO
16-Jan-24		DESTINATION
MATERIAL TYPE 60/70		

TEST NO	O8		I		4B		D		AVERAGE	REMARKS
PENETRATION 100gr 5 sec 25 C	32 30 30		30 30 28		31 33 30		30 31 33		31	60-70
SOFTENING POINT (°C)		59		61					60	(49-56)°C
BITUMEN AFFINITY										>95
SPECIFIC GRAVITY	1.010		1.000		1.009		1.008		1.006	1.01-1.06

Materials Engineer.____

STIRLING CIVIL ENGINEERING LTD P.O. BOX 2[illegible] KAMPALA (U)

INSTITUTION	STUDENT'S NAME	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY A Centre of Excellence in the Heart of Africa	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

MODIFIED WITH 8% PLASTIC

BITUMEN TESTS

LOCATION		OPERATOR Lab team
SUPPLIER		CONTAINER/DELIVERY NO
16-Jan-24		DESTINATION
MATERIAL TYPE 60/70		

TEST NO	6		S2		K		2'		AVERAGE	REMARKS
PENETRATION 100gr 5 sec 25 C	28 30 29		28 28 27		27 27 29		30 30 28		28	60-70
SOFTENING POINT (°C)		62		62					62	(49-56)°C
BITUMEN AFFINITY										>95
SPECIFIC GRAVITY	0.995		0.997		1.001		0.999		0.998	1.01-1.06

Materials Engineer.

STIRLING CIVIL ENGINEERING LTD
P. O. BOX 703, KAMPALA (U)

INSTITUTION	STUDENT'S NAME	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY A Centre of Excellence in the Heart of Africa	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	CML CENTRAL MATERIALS LABORATORY

PROJECT	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

LOCATION		OPERATOR	Lab team
MATERIAL TYPE	BITUMEN	DESTINATION	

DUCTILITY BITUMEN TEST

TEST NO	0% LLDPE				AVERAGE	REMARKS (mm)	Testing method
Mould No.	1	2	3				
Time poured	12:10pm	12:10pm	12:10pm				
Time place in water bath	1:10pm	1:10pm	1:10pm				
Distance elongated (mm)	1610	1610	1625		1615	>1000	ASTM D113

TEST NO	2% LLDPE				AVERAGE	REMARKS (mm)	Testing method
Mould No.							
Time poured	9:15am	9:15am	9:15am				
Time place in water bath	10:15am	10:15am	10:15am				
Distance elongated (mm)	494	548	522		521	>500	DIN 52013 (For modified bitumen)

TEST NO	4% LLDPE				AVERAGE	REMARKS (mm)	Testing method
Mould No							
Time poured	12:20pm	12:20pm	12:20pm				
Time place in water bath	1:20pm	1:20pm	1:20pm				
Distance elongated (mm)	180	158	126		155	>500	DIN 52013 (For modified bitumen)

TEST NO	6% LLDPE				AVERAGE	REMARKS (mm)	Testing method
Mould No							
Time poured	10:02am	10:02am	10:02am				
Time place in water bath	11:02am	11:02am	11:02am				
Distance elongated (mm)	30	25	25		27	>500	DIN 52013 (For modified bitumen)

TEST NO	8% LLDPE				AVERAGE	REMARKS (mm)	Testing method
Mould No							
Time poured	1:03pm	1:03pm	1:03pm				
Time place in water bath	2:05pm	2:05pm	2:05pm				
Distance elongated (mm)	25	22	23		23	>500	DIN 52013 (For modified bitumen)

STIRLING CIVIL ENGINEERING LTD
P.O. BOX 796, KAMPALA (U)
FEB 2024

....
For Stirling

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	**AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)**	Stirling
PROJECT	**INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES**	

PROPERTY TESTS FOR BITUMEN

Testing method	**EN 12596**	VISCOCITY TEST	Operator : Lab Team
Date sampled:	16-Jan-24		
Date tested:	16-Jan-24		
Material Type	60/70		

Test No	1		2	3	AVERAGE	REMARKS
Viscocity at 135°C (mm²/Sec)	340		340	341	**340**	Min 295

Remarks:

FOR TESTING LAB

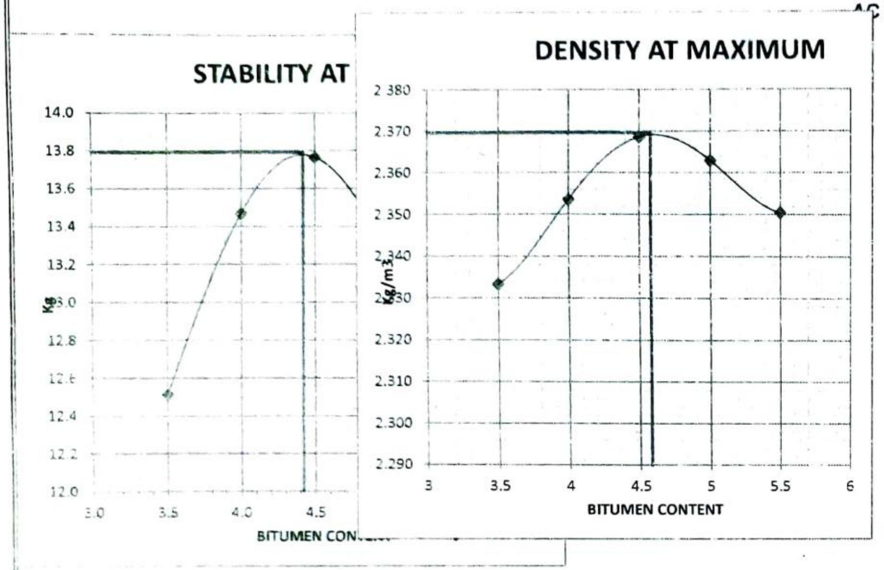
Lab Technician

Materials Engineer

STIRLING CIVIL ENGINEERING LTD
05 FEB 2024
P.O. BOX 796, KAMPALA (U)

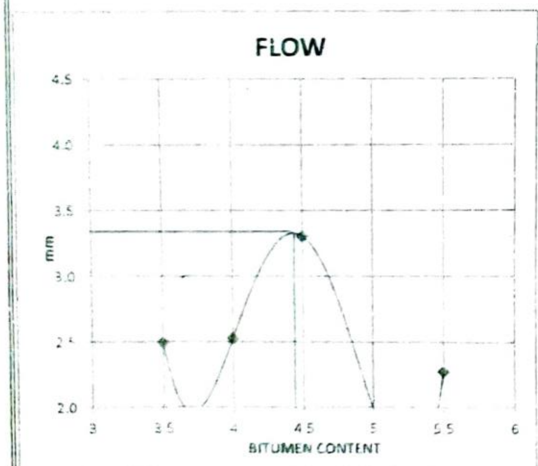
AC 20 JOB MIX DESIGN

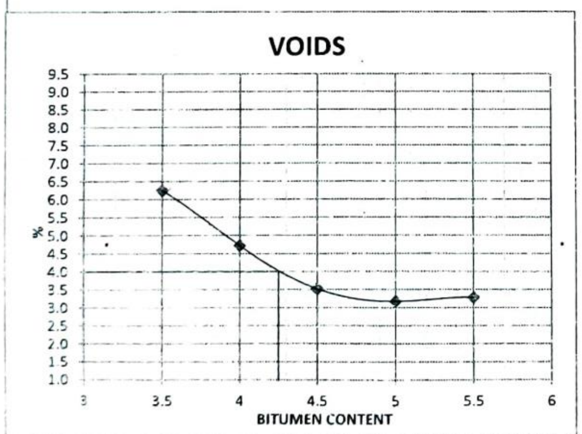
DATE: 1/20/2024



				MIX PROPORTION	
Stability	4.4	%	FILLER	2	%
Density	4.6	%	Q / Dust	53	%
Flow	4.4	%	10mm	8	%
Voids	4.3	%	14mm	12	%
Average	4.4		20mm	25	%
			TOTAL	100	

3





BITUMEN %	3.5	4	4.5	5	5.5
DENSITY	2.333	2.354	2.369	2.363	2.350
STABILITY	12.5	13.5	13.8	13.2	12.5
FLOW	2.5	2.5	3.3	2.0	2.3
VOIDS	6.3	4.7	3.5	3.2	3.3

Signatures:

For Testing Lab ....

For Students ....

STIRLING CIVIL ENGINEERING LTD
P. O. BOX 796, KAMPALA (U)

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT :	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

NEAT ASPHALT

SUMMARY OF A/C 20TEST RESULTS

BITUMEN CONTENT		4.9
MARSHALL MIX TEST RESULTS AFTER MIX	ACHIEVED PLANT PRODUCTION	SPECIFIED
MARSHALL FLOW	2.8	2—4
MARSHALL STABILITY 75BLOWS	13.9	9-18
MARSHALL AIR VOIDS 75BLOWS	4.7	3—5
VOIDS IN MINERAL AGGREGATES	15.2	>14%
VOIDS FILLED WITH BINDER	68.9	65—75%
INDIRECT TENSILE STRENGTH @ 25C	743	>800kpa
INDIRECT TENSILE WET STRENGTH	82	>80% of dry
RATIO STABILITY/FLOW	4.8	>2.5

TESTING LAB

Lab technician

Materials Engineer

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT: INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

Field Ref. No.:	Lab. no.	MIX	NEAT ASPHALT	Sampling date:	02-Feb-24
Sample grade: AC 20	Compaction: 75 blows			Testing date:	03-Feb-24
Sample Description:			AC 20		

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/m^3)	% Air Voids	% VMA	% VFB
1	1200.0	689.50	1201.00	2.346	2.335	4.9	15.3	67.9
2	1187.0	685.00	1190.00	2.350	2.339	4.7	15.2	68.8
3	1183.0	684.00	1185.00	2.361	2.350	4.3	14.8	70.9
4	1183.5	681.00	1185.50	2.346	2.335	4.9	15.4	67.9
Average Sample 1				**2.351**	**2.340**	**4.7**	**15.2**	**68.9**
-				-	-	-	-	-
Average Sample 2								
Average Sample 1 & 2				**2.351**	**2.340**	**4.7**	**15.2**	**68.9**

ASTM D6927Standard 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)
65.6	66.0	65.8	65.8	0.95	15.5	14.69	2.86	5.135
66.3	65.7	66.0	66.0	0.94	11.7	11.04	2.68	4.118
65.8	64.9	64.7	65.1	0.96	14.0	13.46	2.72	4.948
64.7	65.4	64.9	65.0	0.96	14.5	13.87	2.75	5.044
Average Sample 1			**65.5**	**0.95**	**13.9**	**13.26**	**2.8**	**4.8**
-	-	-	-	-	-	-	-	-

EXTRACTED BITUMEN CONTENT	4.5	%

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

SAMPLE 1 (Pycnometer with Water)			SAMPLE 2 (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	1020.1		Asphalt	1036	-
Pycn - Water	8552.3		Pycn - Water	8552.3	-
Pycnometer - Asph + Water	9157.6		Pycnometer - Asph. + Water	9165.8	-
Volume of asphalt	414.8	-	Volume of asphalt	422.5	-
G_{mm}	2.459		G_{mm}	2.452	-
Av. G_{mm}	**2.459**		**Av. G_{mm}**	**2.452**	

Av. Gmm (Kg/m3) Sample 1& 2	2.456

Comment:

FOR TESTING LAB

Materials Engineer

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT: INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

INDIRECT TENSILE STRENGTH

BITUMINOUS MIXTURE SAMPLED ON 2/2/2024 | 102 | GMM 2.456 | Bit.content,% 4.5 | NO OF BLOWS 54

SAMPLE NO.	THICKNESS: HEIGHT 1 (mm)	HEIGHT 2 (mm)	HEIGHT 3 (mm)	Av. Thickness (mm)	Compacted material parameters: Weight of Core in Air (g) A	Weight of Core in Water (g) B	Weight of Core in SSD condition (g) C	Volume of Core (cc) D=(C-B)	Bulk Density (g/cm^3) E=(A/D)	GMM (maximum theoretical density) (g/cm^3) F	VOLUME OF AIR	SATURATED SPECIMEN	VOLUME OF WATER	DEGREE OF SATURATION	VIM, AIR VOIDS (%) =100*(F-E)/F spec min 3.0% G
MIX	NEAT ASPHALT														
	WET														
1	65.1	65.4	65.2	65.2	1184.2	680,5	1192.3	511.8	2.291	2.456	34.390	1204.5	20.300	59.028	6.7
4	66.0	66.3	66.4	66.2	1189.5	677.0	1193.5	516.5	2.280	2.456	36.954	1208.0	18.500	50.063	7.2
3	67.2	66.8	67.0	67.0	1199.7	681.2	1202.6	521.4	2.278	2.456	37.741	1216.5	16.800	44.513	7.2
	DRY														
2	66.8	66.5	67.0	66.8	1178.4	675.8	1186.4	510.6	2.285	2.456	35.529				7.0
5	66.5	66.9	66.7	66.7	1185.1	675.1	1188.4	513.3	2.286	2.456	35.527				6.9
6	67.2	67.0	66.8	67.0	1183.5	673.4	1187.4	514.0	2.280	2.456	36.872				7.2

INDIRECT TENSILE STRENGTH

DRY: SPECIMEN No	GAUGE READING	LOAD RING FACTOR	MAXIMUM LOAD,P	SINGLE TENSILE STRENGTH,S_t	AVERAGE TENSILE STRENGTH,S_t	WET: SPECIMEN No.	GAUGE READING	LOAD RING FACTOR	MAXIMUM LOAD,P	SINGLE TENSILE STRENGTH,S_t	AVERAGE TENSILE STRENGTH,S_t		WET/DRY
	div	kn/div	kn	spec min 800 kPa			div	kn/div	kn	kPa	kPa	S_t =2P/∏tD where P= maximum load(N) t=specimen thickness(mm) D=specim	spec 80%
2	44	0.213	9.4	896.3	901.8	1	35	0.213	7.5	713.0	743		82
5	48	0.213	10.2	956.3		4	37	0.213	7.9	742.4			
6	43	0.213	9.2	852.9		3	39	0.213	8.3	773.5			

FOR TESTING LAB

Lab Technician

Materials Engineer

ENGINEER	EMPLOYER	CONTRACTOR
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling
PROJECT	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES	

PERCENTAGE REFUSAL DENSITY TEST (BS 594)

BITUMINOUS MIXTURE SAMPLED ON 3/28/2024 | SG of agg 2.621 | Bit.content,% 4.6

SAMPLE NO.	THICKNESS				PARAMETERS AT OPTIMUM BITUMEN CONTENT(ASPHALT PLANT PRODUCTION)								
	HEIGHT 1 (mm)	HEIGHT 2 (mm)	HEIGHT 3 (mm)	Av. Thickness (mm)	*Weight of Core in Air (g) A*	*Weight of Core in Water (g) B*	*Weight of Core in SSD condition (g) C*	*Volume of Core (cc) D=(C-B)*	*Bulk Density (g/cm^3) E=(A/D)*	*GMM (maximum theoretical density) (g/cm^3) F*	*VIM, AIR VOIDS (%) =100*(F-E)/F spec min 3.0% G*	*VMA, VOIDS IN MINERAL AGGREGATE (%) =100-(E*(100-%BITUMEN CONTENT)/Sg aggregtes) H*	*VFB, VOIDS FILLED WITH BINDER (%) =[(H-G)/H]*100 I*
MIX	*NEAT ASPHALT*												
1	52.3	51.9	52.1	52.1	2198.1	1274.5	2200.0	925.5	2.375	2.459	3.4		
2	50.1	52.3	51.3	51.2	2179.0	1263.5	2179.5	916.0	2.379	2.459	3.3		
3	53.9	52.1	52.3	52.8	2198.3	1277.5	2200.5	923.0	2.382	2.459	3.1		
AVERAGE											3.3		

Remarks:

FOR CLIENT

Lab Technician — Materials Engineer

FOR SUPPLIER

Lab Technician — Materials Enginee

STIRLING CIVIL ENGINEERING LTD P.O. BOX 703 KAMPALA (U)

ENGINEER	EMPLOYER	CONTRACTOR
UGANDA CHRISTIAN. UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling
PROJECT	RECYCLING AND OVERLAY OF LUKULI ROAD (7.71KM) INCLUNDING SIGNALIZATION OF LUKULI/NAMASOOLE/ KAYEMBA JUNCTION	

PERCENTAGE REFUSAL DENSITY TEST (BS 594)

BITUMINOUS MIXTURE SAMPLED ON 3/28/2024 | *SG of agg* 2.621 | Bit.content,% 4.6

SAMPLE NO.	THICKNESS				*PARAMETERS AT OPTIMUM BITUMEN CONTENT(ASPHALT PLANT PRODUCTION)*								
	HEIGHT 1 (mm)	HEIGHT 2 (mm)	HEIGHT 3 (mm)	Av. Thickness (mm)	*Weight of Core in Air (g) A*	*Weight of Core in Water (g) B*	*Weight of Core in SSD condition (g) C*	*Volume of Core (cc) D=(C-B)*	*Bulk Density (g/cm³) E=(A/D)*	*GMM (maximum theoretical density) (g/cm³) F*	*VIM, AIR VOIDS (%) =100*(F-E)/F spec min 3.0% G*	*VMA, VOIDS IN MINERAL AGGREGATE (%) =100-(E*(100-%BITUMEN CONTENT)/Sg aggregtes) H*	*VFB, VOIDS FILLED WITH BINDER (%) =[(H-G)/H]*100 I*
MIX	*A/C 20 ASPHALT BITUMEN MODIFIED WITH 2% PLASTICS*												
1	56.5	58.5	58.6	57.9	2184.2	1229.3	2190.7	961.4	2.272	2.383	4.7		
2	56.8	57.3	58.3	57.5	2174.1	1226.5	2182.5	956.0	2.274	2.383	4.6		
3	55.8	52.8	55.3	54.6	2185.6	1226.3	2188.2	961.9	2.272	2.383	4.7		
	AVERAGE										**4.6**		

Remarks:

FOR CLIENT

Lab Technician ______ Materials Engineer ______

FOR SUPPLIER

Lab Technician ______ Materials Engineer

STIRLING CIVIL ENGINEERING LTD P. O. BOX ... KAMPALA (U)

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT :	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

2% LLDPE

SUMMARY OF A/C 20 TEST RESULTS

BITUMEN CONTENT		4.9
MARSHALL MIX TEST RESULTS AFTER MIX	**ACHIEVED PLANT PRODUCTION**	**SPECIFIED**
MARSHALL FLOW	2.7	2—4
MARSHALL STABILITY 75BLOWS	18.1	9-18
MARSHALL AIR VOIDS 75BLOWS	4.9	3—5
VOIDS IN MINERAL AGGREGATES	15.6	>14%
VOIDS FILLED WITH BINDER	68.4	65—75%
INDIRECT TENSILE STRENGTH @ 25C	872	>800kpa
INDIRECT TENSILE WET STRENGTH	83	>80% of dry
RATIO STABILITY/FLOW	6.1	>2.5

TESTING LAB

Lab technician

Materials Engineer

STIRLING CIVIL ENGINEERING LTD P.O. BOX 796, KAMPALA (U)

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT :	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

4% LLDPE

SUMMARY OF A/C 20 TEST RESULTS

BITUMEN CONTENT		4.9
MARSHALL MIX TEST RESULTS AFTER MIX	ACHIEVED PLANT PRODUCTION	SPECIFIED
MARSHALL FLOW	2.4	2—4
MARSHALL STABILITY 75BLOWS	21.0	9-18
MARSHALL AIR VOIDS 75BLOWS	6.9	3—5
VOIDS IN MINERAL AGGREGATES	18.7	>14%
VOIDS FILLED WITH BINDER	63.4	65—75%
INDIRECT TENSILE STRENGTH @ 25C	677	>800kpa
INDIRECT TENSILE WET STRENGTH	74	>80% of dry
RATIO STABILITY/FLOW	8.2	>2.5

TESTING LAB

Lab technician

Materials Engineer

STIRLING CIVIL ENGINEERING LTD P.O. BOX 796, KAMPALA (U)

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT : INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

6% LLDPE

SUMMARY OF A/C 20 TEST RESULTS

	BITUMEN CONTENT		4.9
MARSHALL MIX TEST RESULTS AFTER MIX		**ACHIEVED PLANT PRODUCTION**	**SPECIFIED**
MARSHALL FLOW		2.1	2—4
MARSHALL STABILITY 75BLOWS		23.9	9-18
MARSHALL AIR VOIDS 75BLOWS		9.3	3—5
VOIDS IN MINERAL AGGREGATES		18.5	>14%
VOIDS FILLED WITH BINDER		49.7	65—75%
RATIO	STABILITY/FLOW	10.7	>2.5

TESTING LAB

Lab technician

Materials Engineer

STIRLING CIVIL ENGINEERING LTD P.O. BOX 795, KAMPALA (U)

INSTITUTION	STUDENTS	TESTING LAB
UGANDA CHRISTIAN UNIVERSITY	AINEBYONA MARVIN (S20B32/017) & ADRIKO DERRICK (S19B32/629)	Stirling

PROJECT :	INVESTIGATING THE USE OF LINEAR DENSITY POLYETHYLENE IN BITUMEN TO IMPROVE THE PERFORMANCE OF ASPHALT PAVEMENTS ALONG LONGITUDINAL SLOPES

8% LLDPE

SUMMARY OF A/C 20 TEST RESULTS

BITUMEN CONTENT		4.9
MARSHALL MIX TEST RESULTS AFTER MIX	ACHIEVED PLANT PRODUCTION	SPECIFIED
MARSHALL FLOW	1.8	2—4
MARSHALL STABILITY 75BLOWS	25.5	9-18
MARSHALL AIR VOIDS 75BLOWS	8.3	3—5
VOIDS IN MINERAL AGGREGATES	18.4	>14%
VOIDS FILLED WITH BINDER	54.7	65—75%
RATIO STABILITY/FLOW	13.5	>2.5

TESTING LAB

Lab technician

Materials Engineer

STIRLING CIVIL ENGINEERING LTD
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