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FACULTY OF ENGINEERING

DEPARTMENT OF CIVIL AND BUILDING ENGINEERING

**ASSESSING THE IMPACT OF BITUMEN TYPES ON THE
PERFORMANCE OF ASPHALT CONCRETE ROAD
PAVEMENT CONSTRUCTION IN UGANDA**

By

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CERTIFICATION

The undersigned certifies that he has read and hereby recommends for acceptance by Kyambogo University, a research thesis entitled: **Assessing the impact of bitumen types on the performance of asphalt concrete road pavement construction in Uganda**; in fulfilment of the requirements for the award of a degree of Master of Science in Construction Technology and Management of Kyambogo University.

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Date: 5/12/18

DECLARATION

I, Steven Mpaata, hereby declare that this research thesis is my own work and that, to the best of my knowledge and belief, it contains no material previously published or written by another person nor material which has been accepted for the award of any other degree of a university or other institution of higher learning, except where due acknowledgement has been made in the text and reference list.

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LIST OF ACRONYMS

AASHTO	American Association of State Highways and Transportation Organization
AC	Asphalt Concrete
ASTM	American Society for Testing and Materials
AR	Aged Residue
BBR	Bending Beam Rheometer
BS	British Standard
CSIR	Council for Scientific and Industrial Research
CML	Central Materials Laboratory
DSR	Dynamic Shear Rheometer
DTT	Direct Tension Test
ESAL	Equivalent Standard Axial Load
Eqn.	Equation
FWD	Falling Weight Deflectometer
G*	Complex Shear Modulus
GAMES	General Analysis of Multi-layered Elastic Systems
GSRBW	General Specifications for Roads and Bridge Works
HMA	Hot Mix Asphalt
KTR	Kamuli Township Roads
kPa	Kilo Pascal
LTPP	Long Term Pavement Performance

MBP	Mbarara Bypass
M-E	Mechanistic Empirical
MKN	Mukono – Kayunga - Njeru
MoWT	Ministry of Works and Transport
NCR	Northern Corridor Route
PAV	Pressurised Aging Vessel
PG	Performance Grade
RDF	Relative Damage Factors
RTFO	Rolling Thin Oven Film
RV	Rotational Viscosity
SHRP	Strategic Highway Research Program
SP	Superpave
SPSS	Statistical Package for Social Scientists
SI	System International
TanRoads	Tanzania National Roads Agency
UNRA	Uganda National Roads Authority
US	United States
USMID	Uganda Support to Municipal Infrastructure Development

ABSTRACT

This research was conducted to assess the impact of local asphalt binders on the performance of asphalt concrete road pavement construction in Uganda. The bitumen samples were collected from ongoing road construction projects where asphalt concrete made part of the pavement. The sampling was done in accordance with procedures provided in BS EN 58: 2012 and BS EN 475: 2000. Testing was carried out in Central Materials Laboratory, Tan Roads, Dar Es Salaam, Tanzania to confirm binder classification and obtain performance properties. The penetration tests, fluidity and pumping tests conducted on the bitumen samples confirmed compliance with specifications. It was noted that all penetration grade bitumen samples gave softening points lower than the allowable limits implying in-service performance challenges. Most of the samples did not comply with physical property requirements when compared with the binder specifications implying that they are susceptible to rutting and fatigue damage. Binder Temperature zoning based on Strategic Highway Research Program of asphalt grading system was completed in fourteen zones and presented on a map. Three temperature zones were confirmed incorporating Performance Grade (PG) 70+3 in most of the Northern, far Eastern and Kasese regions, PG 64+6 for most of Central and PG 58+3 for far Western regions bordering with Rwanda and Democratic Republic of Congo. This study introduced a robust testing regime and acceptance criteria to be used in future asphalt designation and construction in Uganda. Accordingly, value engineering was achieved based on increased time intervals of road maintenance expected based on performance of binders zoned.

CHAPTER ONE: INTRODUCTION

1.1 BACKGROUND

Bitumen is a complex thermoplastic and viscous material produced through the fractional distillation of crude oils. It's composed of asphaltenes and maltenes with majority Carbon and Hydrogen elements. It is extremely difficult to predict the chemical properties.

Bitumen occurs in different forms such as cut back bitumen, emulsions, polymer modified binders and penetration binders.

Cut back bitumen

These are usually used in priming surfaces of bases before wearing courses are applied. They create the bond between base layer and the surfacing course. Common types of cut back bitumen include Moist curing (MC) 70, MC 30 and MC 40.

Penetration Bitumen

Penetration bitumen is material used as binder in asphalt concrete surfacing or in chip sealing. The common binders on the local market include penetration grade (Pen) 20/30, Pen 35/50, Pen 50/70, Pen 70/100. They are assigned those numbers based on their penetration values at 25 degrees centigrade in a cup by a needle.

Emulsion Bitumen

Emulsions can occur as anionic or cationic depending on whether they are positively or negatively charged. Emulsions are binders that are common on local market used in road surfacing as a tack coat providing bondage between bituminous base course and

wearing course. Similarly, emulsions are also used as base course primers where they provide good bondage between the base course and the wearing course.

In gravel base construction, emulsions have been used as stabilisation materials especially for the low volume traffic roads. In Uganda, emulsions have been used in stabilization during the construction of Matuga – Semuto – Kapeeka road from gravel to paved bituminous standard.

Polymer modified binders

Polymer modified binders are not common on Ugandan market but they are used in cases where desired asphalt concrete pavement performance may not be achieved when lean binders are used.

In Uganda, bitumen is majorly used in road construction as binder in asphalt concrete surfacing and as a binder sealing material in surface dressing pavements using crushed stone chipping.

The increase in the traffic volume, loading rates and durability requirements, local roads has greatly impacted on the roads surfaced roads with stone chipping. Based on this finding, there has been a trend of introducing asphalt concrete surfacing to give additional mechanical strength and flexibility in the pavement to achieve desired pavement life.

However, the newly completed roads surfaced with asphalt concrete have experienced pre-mature deterioration of the asphalt concrete raising concerns in the design base, materials specification, construction methods, construction equipment, quality of binder, traffic volume, loading rates, laxity in supervision and poor choice of binder.

This research provides a general overview of the local bitumen binders used in asphalt concrete construction in Uganda and their impact on the pavement performance based on local environmental and climatic conditions.

Below is an overview of the failure modes observed on the newly constructed Mbarara – Ntungamo – Kabale road.



Figure 1.1: Cracking along newly completed Mbarara – Kabale – Katuna road

1.2 STATEMENT OF THE PROBLEM

There has been a trend of moving from the traditional surface dressing of roads using stone chipping to surfacing using asphalt concrete in road pavement construction in Uganda largely due to increased traffic volume and speed plying on the roads and durability requirements.

However, the completed asphalt concrete paved roads have experienced noticeable pre-mature deterioration, rising lots of concerns regarding the quality of the materials used, materials specifications, design, construction methods, laxity in supervision, etc (Zou, 2017). The road projects with premature defects include Bugiri to Malaba, the newly constructed Northern corridor routes – lots 1, 2 and 3. Based on the failure modes, this research presents the types of local binders used as and their impacts on pavement performance.



Figure: 1.2 - Pictorial view of road sections from Mbarara – Ntungamo – Kabale – Katuna

1.3 OBJECTIVE OF THE STUDY

1.3.1 MAIN OBJECTIVE

To assess the impacts of bitumen types on the performance of asphalt concrete road pavement construction in Uganda.

1.3.2 SPECIFIC OBJECTIVES

The specific objectives of the study are:

- i. To determine the consistency properties of the binders on the local market;
- ii. To determine the physical properties of the bitumen types on the local market;
- iii. To determine the temperature binder zones in different regions of the country based on local conditions;

1.4 RESEARCH QUESTIONS

The research was aimed at addressing the following research questions;

- i. What are the consistency properties of the binders on the local market?
- ii. What are the physical properties of the binders on the local market?
- iii. What are the temperature binder zones distribution in the country based on local conditions?

1.5 JUSTIFICATION OF THE STUDY

There is an increasing trend of asphalt concrete pavements experiencing pre-mature deterioration on road projects in Uganda. This has largely been attributed to the fact that the quality of bitumen on local market has not been exhaustively investigated. Not all bitumen produced is good for road pavement construction (Sabita, 2016). The provisions in Table 4202/1 of the standard specifications for roads and bridge works (MoWT, 2010) do not elaborate on the physical properties that inform the performance of the binders.



Figure 1.3: MBP – Repair of cracks MN - Shear Cracking Shear Failure and Cracking

Table 1.2 below presents the requirements for Penetration and performance graded binders provided in the Ministry of Works and Transport, General Specifications for roads and bridge works. It was also noted that on the sites visited, no adequate physical property testing equipment was available and acceptance was based on empirical tests. Interaction with testing team indicated that there was inadequate expertise in carrying out physical property testing locally.

Table 1. 1 – Specification for Bituminous Binders (MoWT, 2010)

Type of Binder	Specification
Penetration grade bitumen	BS EN 12591: 2000
Performance grade	SP-1*)
Cut back bitumen	BS EN 12591: 2000
Bitumen emulsions, anionic	BS 434: 1984
Bitumen emulsions, cationic	BS 434: 1984
*) Appendix A. Performance Graded Asphalt Binder Specification and Testing, Superpave series No. 1 (SP – 1) Asphalt Institute.	

This research focused on classifying, determining and analysing the physical properties and performance criteria of bitumens on the local market and provide binder temperature zoning for the road pavement construction in various regions of the country based on local conditions.

1.6 SIGNIFICANCE OF THE STUDY

This study has provided the bitumen grades that are best suited for our local environmental and climatic conditions such as traffic volume, speed and temperatures variations. The research provided adequate information required for Ministry of Works and Transport to update the General Specifications for Roads and Bridges regarding bitumen binders by introducing a more robust testing regime and acceptance criteria. Zoning of binder application consequently would increase time intervals of maintenance and thus value engineering.

1.7 SCOPE AND LIMITATIONS OF THE STUDY

1.7.1 GEOGRAPHICAL, TIME AND CONTENT SCOPE

The research was conducted in Uganda from December 2017 to June 2018. Representative bitumen samples were obtained from ongoing road construction projects like the capacity improvement of Kampala Northern Bypass (Penetration grade bitumen 35/50), Mbarara Bypass project lot 1 (performance grade bitumen 76-10), Kamuli township roads (penetration grade bitumen 50/70), and Mukono – Kayunga - Njeru road (performance grade bitumen 70-16) where such materials were available. The criteria for binder selection depended on material availability. Consistency and Rheological properties of the binders were carried out at Central

Materials Laboratory, TanRoads, Dar Es Salam, Tanzania to confirm binder classification performance properties. Field air, pavement temperatures and latitudes were measured, recorded and a model was developed to predict maximum pavement design temperatures that enabled regional zoning of binder application and traffic characterization.

The research was limited to locally available binders currently being used. No mixtures are discussed in this study.

1.7.2 FINANCIAL SCOPE

Four thousand seven hundred U.S dollars was spent to deliver this research. There was 50% partial Employer's career development policy contribution and personal savings that covered allowances for field teams, logistics, transporting and testing of materials, stationery and report production.

1.8 CONCEPTUAL FRAMEWORK

Identification of the ongoing road construction projects where asphalt binder was available and was being used in asphalt concrete surfacing. Sampling of asphalt binders was in line with procedures provided in BS EN 58: 2012 and BS 2000-474: 2012. Samples were extracted from the containers at CML and conditioned before testing. Determination of the consistency properties like flash point, specific gravity, ductility and penetration were done to confirm the classification of the binders and ensure safe handling during further physical property testing.

Testing for Rheological and physical properties of the asphalt binders on local market were done to determine the early and in-service hardening of binders through

laboratory simulations involving rolling thin film oven test and pressure ageing vessel respectively. Dynamic shear rheometer tests were tested from virgin and asphalt binder residues to measure the resistance to Rutting and fatigue damage. Subsequently, rotational viscosity tests were carried out to measure the sufficiency in pumping and mixing of the binder. The results obtained from the consistency tests were able to confirm the binder classifications. The rheological and physical property tests were used to predict the asphalt early and in-service hardening during mixing, transportation, placing and compaction. Consequently, intermediate temperature tests using PAV aged samples were used aged to predict the rutting and fatigue cracking potential of the binders during in-service performance and sufficiency of the binders in fluid mixing and pumping confirmed eventual uniform mixture productions that improve void ratios. Zoning of binder application to guide future utilization of the various bitumen types on the local market in different regions of the country based on environmental and climatic conditions such as traffic speed and traffic volume and temperature variations. The research improved on the testing regime through introduction of physical property tests and thus an improved binder acceptance criteria that meet local environmental and climatic conditions such as temperatures variation and traffic volume and speed. Consequently, the reduced life cycle costing of asphalt resulting from increased time intervals for maintenance brought about value engineering. Figure 1.10 presents schematic diagram of the conceptual framework.

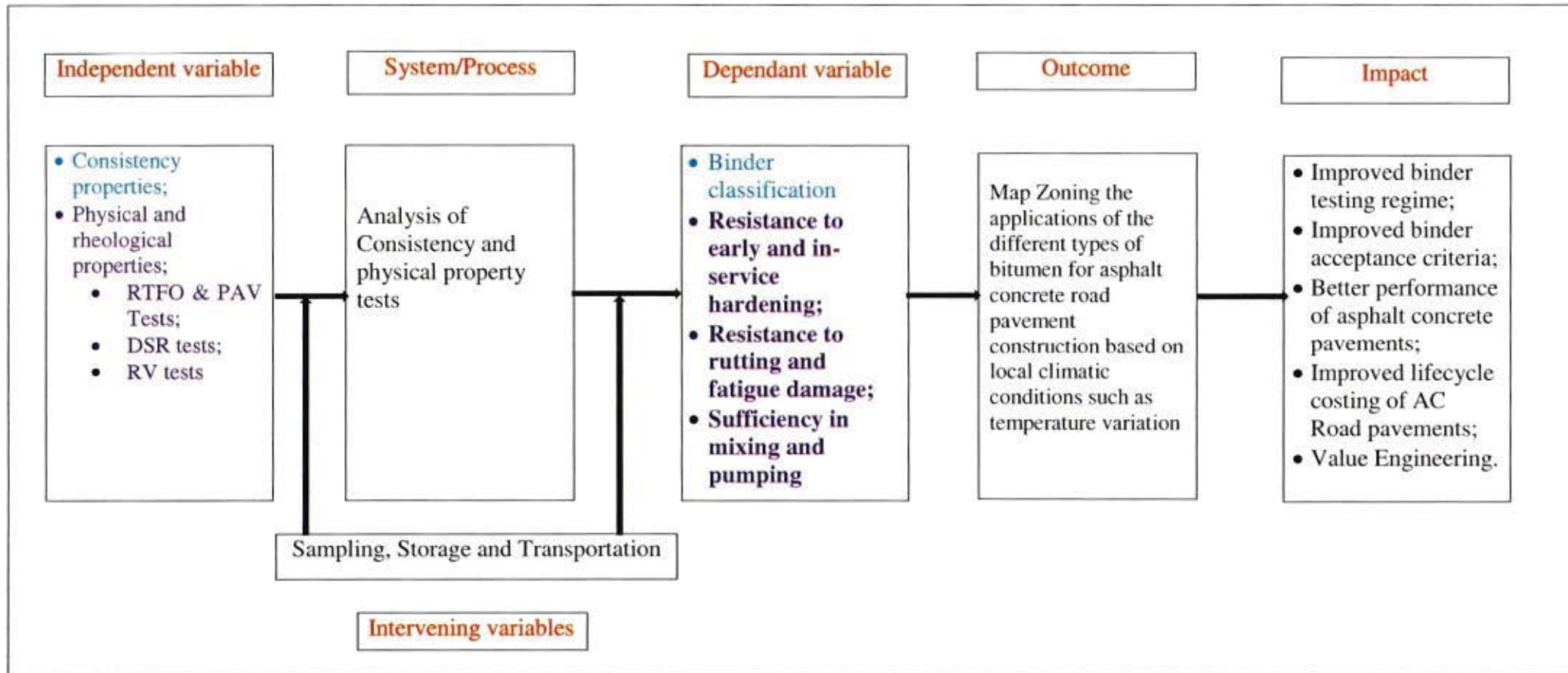


Figure 1.4; Conceptual Framework

CHAPTER TWO: LITERATURE REVIEW

2.1 INTRODUCTION

This chapter provided most of the already available data about the consistency and physical properties of bitumen as reported by researchers in recent studies. Air temperatures in most of weather zones that influence the pavement temperature and previous case studies where zoning of binders such as in Thailand, South Africa and Sudan. Previous test results carried out in evaluating the performance of binders when exposed to high and intermediate temperatures to compare with my findings has been presented.

Chemistry of Bitumen

The composition of bitumen is highly dependent on the crude oil source because of different geological conditions while the crude oil is being formed. According to Read and Whiteoak, 2003, Asphalt Academy, 2007 stated that bitumen has a highly complex composition which comprises of mostly hydrocarbons with small quantities of sulphur, nitrogen, oxygen and very small amounts of metal such as iron, calcium, magnesium, nickel and vanadium. The exact composition of bitumen is different for each crude oil source because of the origin, modifications developed during the manufacturing process and in-service ageing but it usually varies between certain percentages as can be seen in Table 1.2. These elements form four constituents in bitumen namely asphaltenes, saturates, aromatics and resins. Figure 1.1 shows how these four constituents are separated from each other (Zyl, 2018).

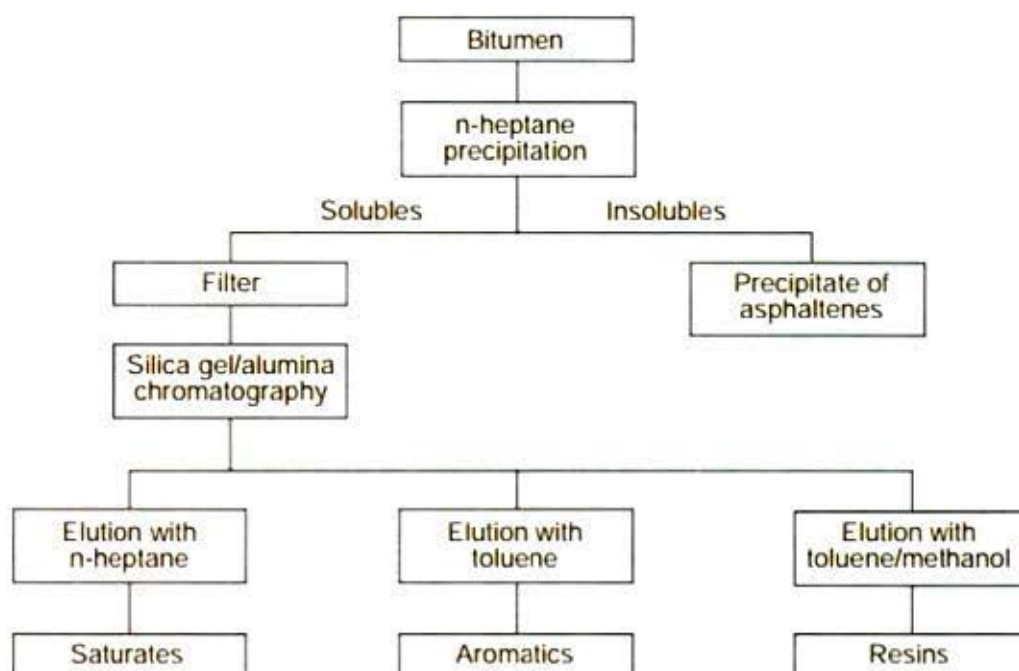


Figure 2.1: Separation of Bitumen constituents (Zyl, 2018)

Table 2.1: Elementary Bitumen Composition (Zyl, 2018)

Element	Percentage
Carbon	82-88
Hydrogen	8-11
Sulphur	0-6
Oxygen	0-1.5
Nitrogen	0-1

Asphaltenes

Asphaltenes are insoluble black to brown amorphous solids that contain carbon and hydrogen with some oxygen, sulphur and nitrogen. Asphaltenes have a fairly high molecular weight which ranges from 1,000 to 100,000 with a particle size of 5 to 30 nm and are considered to be highly polar. Rheological properties of bitumen are highly dependent on the asphaltene content. An increase in the asphaltenes would produce a harder bitumen with a lower penetration and a higher softening point and will make the bitumen more viscous. Figure 1.2 shows a typical asphaltene molecular structure.

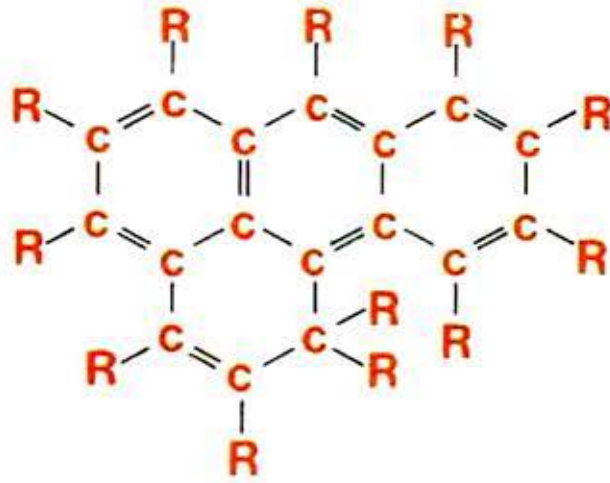


Figure 2.2: Asphaltenes molecular structure (Zyl, 2018)

Where:

C = Carbon

R = Aliphatic, naphthenic or aromatic carbon chains

Saturates

Saturates are non-polar viscous oils that have the lowest molecular weight with a range of 300 to 2000g/mol. They consist of branched or straight chain aliphatic hydrocarbons with alkyl-naphthenes and some alkyl-aromatics. Figure 1.3 shows the two types of saturate structures in bitumen, namely waxy and non-waxy.

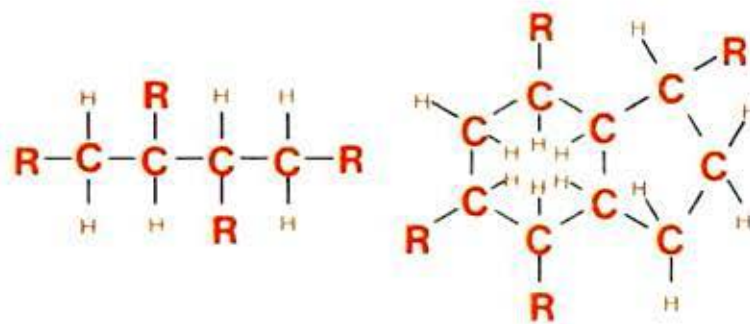


Figure 2.3: Saturate molecular structure (Zyl, 2018)

Where:

C = Carbon

H = Hydrogen

R = Aliphatic, naphthenic or aromatic carbon chains

Aromatics

Aromatics comprise of naphthenic aromatic compounds which represent the biggest portion of bitumen. It is a dark brown liquid and has the same molecular weight as saturates. Aromatics are also non-polar carbon chains and have a high dissolving ability for other high molecular weight hydrocarbons. Figure 1.4 shows a typical aromatic molecular structure.

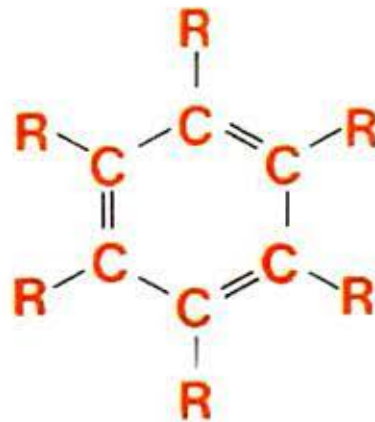


Figure 2.4: Aromatic molecular structure (Zyl, 2018)

Resins

Resins are largely composed of carbon and hydrogen and contain small quantities of nitrogen, sulphur and oxygen. Resins have a dark brown colour, are polar by nature and can be either solid or semi-solid. Because of their polar nature, resins are a very strong adhesive. The molecular weight of resins range from 500 to 50,000 g/mol and they have a particle size of 1 to 5 nm.

Bitumen Structure

Bitumen has a type of colloidal dispersion system which consists of two different phases namely the dispersion phase and the continuous liquid phase. Asphaltene micelles, dispersed or dissolved, form the disperse phase and the saturates and aromatics form the continuous liquid phase. The resins promote a good dispersion of the solid-state asphaltenes in the saturates and aromatics. When the bitumen has a sufficient quantity of resins and aromatics the asphaltene micelles can move freely in the bitumen. These types of bitumen are known as SOL type bitumens and are illustrated in Figure 1.5. When there are not enough resins and aromatics in the bitumen, the micelles can bind together further and form a network of asphaltenes. This network is then filled with an intermicellar fluid of mixed constituents. These types of bitumen are known as GEL type bitumens and are shown in Figure 1.5. Aggregation and solvation provide the colloidal behaviour of the asphaltenes in bitumen. The degree to which the asphaltenes are dispersed will have a considerable influence on the viscosity. The viscosity is also dependent on changes in temperature. According to Asphalt Academy (2007) unmodified bitumen has a Newtonian behaviour due to weak interaction between molecules at high temperatures (above the bitumen's softening point), whereas the temperature change is directly proportional to the viscosity.

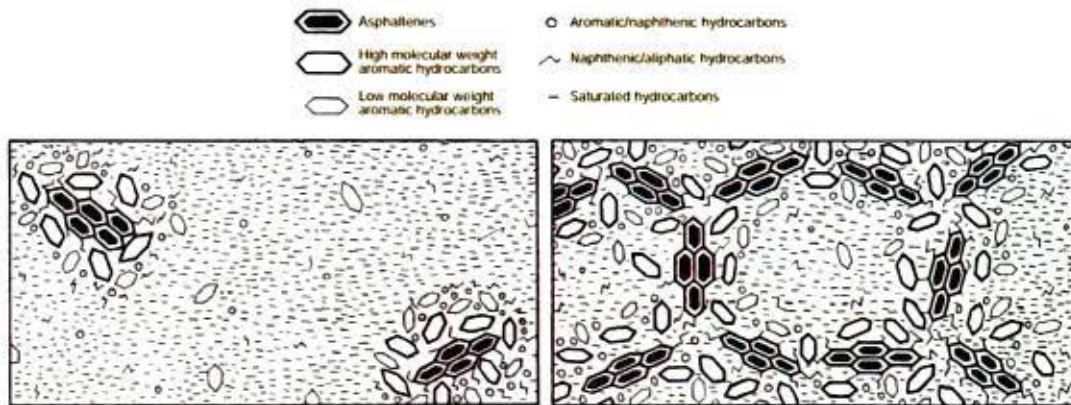


Figure 2.5: Bitumen type structures: SOL type (Left) and GEL type (Right) (Zyl, 2018)

Bitumen varies widely in consistency from solid to semisolid at normal air temperatures (AI, 2005). When bitumen is heated, it softens and is able to coat around particles during asphalt hot mix production. It has been used in road construction as a binder in pavement construction and in surfacing. When discussing bitumen, attention regarding its rheology is paramount. Rheology is the science that deals with deformation and flow of materials. Rheology of bitumen is important in determining the properties that give qualitative and quantitative relations to performance of asphalt mixtures, specifying bitumen grades, estimating effects of ageing and in developing new binders. Research has shown that Superpave performance graded binders perform better than penetration types in asphalt pavement construction (AI, 2003) because they tend to incorporate environmental and climatic conditions. The Performance graded binders usually include measures describing stress-strain relationships that can link laboratory-measured parameters to in-service pavement performance. PG considers pavement conditions like temperature, traffic speed and volume and pavement structure. Extensive research has been conducted on the topic of asphalt performance testing which has demonstrated that use of such laboratory simulation tests like rolling thin film oven tests, dynamic shear tests on virgin and aged bitumen can improve the

longevity of asphalt pavements and indeed reduce on the risk of early deterioration (Eshan V. Dave, 2011). The PG system measures the physical properties of binders through tests that are intended to be directly related to field performance by engineering principles. These tests are conducted under the temperature extremes that a pavement experiences to simulate the three critical stage of a binder's life, namely:

- First stage – transportation, storage and handling prior to mixing with aggregate;
- Second stage – after mixing, production and construction (Rolling Thin Film Oven Test treatment is used to approximate this stage of short-term ageing);
- Third stage – binder ageing on the road (the pressure ageing vessel (PAV) is used to simulate long-term ageing of the binder).

The United States of America under the Strategic Highway Research Programme (SHRP), introduced performance properties incorporating the complex shear modulus and shear angle that help grade different bitumens in terms of performance (FHWA, 2011). The relationships between the shear modulus and phase angles gave a guidance on the performance criteria. In a few areas of the Gulf, modifications of the binders were not required in applications where the pavement temperatures were less than 64⁰C. It was however studied that more than 50% of the Gulf countries experienced maximum pavement temperatures of 76⁰C implying that asphalt binder modifications were required to achieve maximum pavement performance (Wahabb, 1998).

In China, rutting has been one of the major distresses observed on asphalt pavement due to increasing traffic volume, heavy axle load, continuous hot weather, etc., especially in long-steep-slope sections, bus stops, etc. Many factors would affect

rutting resistance of asphalt pavement, including material properties, climatic condition, traffic volumes, speed, and axle types, and construction quality (Zou, 2017).



Figure 2.6: Pavement distress in China (Zou, 2017)

In Uganda, we have experienced pre-mature deterioration of asphalt concrete pavements on newly constructed road pavements. Such road links include the Jinja - Bugiri - Malaba road, the Northern Corridor Route - Kampala - Masaka - Mbarara - Kabale - Katuna road. below is a pictorial view of some defects captured during the site surveys;



Figure 2.7: Asphalt concrete surfacing along Mbarara - Kabale section

These failures have largely been attributed to inadequate designs, construction methods, increased axle loads, traffic volumes and speed, laxity in supervision and quality of materials used during production and construction of asphalt concrete

pavements. Based on the failure mechanism, this research focused on the quality of asphalt binders used since it was the only material which was not locally available. It has also been observed that there have been replacements of the Design Consultant specified performance grade binders on some road projects by “equivalent bitumens”. For example, on the Mbarara – Ntungamo – Kabaale – Katuna road, Lot 2, penetration grade bitumen 20/30 was used as an alternative of the PG 76-10 originally specified by the Design Consultant. The criteria for replacing the PG 76-10 was the similarity in penetration values obtained and brittleness resistance as indicated by fraas breaking point testing provided by the supplier of up to -7°C . It was worth noting that the asphalt pavement experienced pre-mature deterioration as presented in Figure 1.7 above. Therefore, this research focused on going beyond the bitumen consistency property testing and determined the actual rheological and physical properties of the bitumens.

2.2 CLASSIFICATION PROPERTIES OF BINDERS USED FOR AC PAVEMENT

Bitumens can be classified using their consistence properties. The consistency properties were determined through penetration testing at 25°C , flash and fire points, solubility, ductility at 25°C , softening points, specific gravity and viscosity tests.

In Thailand, a study was carried out to measure the consistency properties of common penetration graded binders since there was lack of capital investment to acquire the performance graded properties like complex shear modulus and phase angles that provide high temperatures of the binder and predict the hardening, rutting, fatigue damage and thermal resistance (Charoentham, 2012). The Table below presents the test results obtained during the study;

Table 2.2: Current Asphalt Binders Used in Thailand (Charoentham, 2012)

Source	Pen	PG	Pen @ 25°C			T _{R&B}			PI		
			Original	RTFO - AR	PAV	Original	RTFO - AR	PAV	Original	RTFO - AR	PAV
I	60/70	64-22	63	38	23	47.1	52.3	58	-1.4	-1.2	-1.0
II	60/70	64-22	64	39	24	46.3	52.3	57.3	-1.6	-1.2	-1.1
III	60/70	64-22	70	43	29	45.6	50.1	54.6	-1.6	-1.5	-1.3
IV	60/70	64-22	69	42	28	46.7	50.7	55.9	-1.3	-1.4	-1.1
V	60/70	64-22	68	38	24	48.5	55	62	-0.9	-0.6	-0.2
Minimum			63	38	23	45.6	50.1	54.6	-1.6	-1.5	-1.3
Maximum			70	43	29	48.5	55	62	-0.9	-0.6	-0.2
Average			66.8	40.0	25.6	46.8	52.1	57.6	-1.4	-1.2	-0.9
STDEV			3.1	2.3	2.7	1.1	1.9	2.8	1.3	0.3	0.4
85% Percentile			69	42	28	47.7	53.4	59.6	-1.1	-1.0	-0.7

In West Virginia, studies were performed to evaluate the precision of test results of performance graded asphalt binder testing equipment and testing protocol provided residues of rolling thin film oven test less than 0.5% (Zaniewski, 2004). An overview is presented in Table 2.2 below. It was noted that the mass loss obtained while using the rolling thin film oven test ranged between 0.04 to 0.31%. These values were within the permissible limits (not exceeding 0.5%). Details of the test results are provided in Appendix I of this report. The decision was made to proceed with the testing program and to note any differences observed between the Dynamic Shear Rheometer (DSR) tests conducted from drained Rolling Thin Film Oven Test residue and from scraped Rolling Thin Film Oven residue. No significant differences were observed in the Dynamic Shear Rheometer test results on residue collected by the different methods.

In Uganda, consistency testing has been carried out on asphalt binders conforming to the requirements of Table 4202/1 of the General Specifications for Roads and Bridge Works which largely relies on BS EN 12591: 2009. Table 2.1 provides an overview of the requirements provided in BS EN 12591: 2009. It is noted that physical property testing is not emphasized by this requirement and yet it is the only basis of predicting field performance based on laboratory simulations.

Table 2. 3 - Penetration Binder Specifications (BS EN 12591: 2000)

Penetration	Test Method	Grade								
		20/30	30/45	35/50	40/60	50/70	70/100	100/150	160/220	250/330
Penetration at 25°C	EN 1426	20-30	30-45	35-50	40-60	50-70	70-100	100-150	160-220	250-330
Softening Pt °C Min	EN 1427	55	52	50	48	46	43	39	35	30
(Max)		63	60	58	56	54	51	47	43	38
Resistance to hardening	EN 12607-1									
- Change of mass % Max.		0.5	0.5	0.5	0.5	0.5	0.8	0.8	1.0	1.0
- Retained Penetration Min.		55	53	50	50	50	46	43	37	35
- Softening point after hardening, Min.		57	54	49	49	48	45	41	37	32
Flash Point, Min. °C	EN 22592 (b)	240	240	240	230	230	230	230	220	220
Solubility, Min. %	EN 12592	99.0	99.0	99.0	99.0	99.0	99.0	99.0	99.0	99.0
Kinematic Viscosity at 135°C, Min. mm ² /s	IP 319	530	400	370	325	295	230	175	135	100

2.3 PHYSICAL PROPERTIES OF AVAILABLE ASPHALT CONCRETE BINDERS

Physical properties of asphalt binders are useful in predicting the in-service asphalt binder performance. These properties include resistance to hardening, thermal resistance, resistance to rutting and fatigue damage. Recognising the limitations of empirical testing, the US Strategic Highway Research Program (SHRP) sponsored a major research effort to develop performance based tests and specifications for binders and hot mix asphalt mixes. This resulted in the development of the Superpave Performance Grading (PG) system which has had significant impact on binder specifications outside the US (Chin, 2010). However, the Superpave PG system has been shown to have a strong correlation with the Australian viscosity grading. Superpave PG is essentially a surrogate of viscosity. For this reason, the viscosity testing at high service temperature should be highly considered as the performance based test many are looking for in their binders' specifications (Chin, 2010).

Under the Performance Grading system, (M320-05), Strategic Highway Research Program researchers introduced two important properties to determine the performance grading of a binder over a range of temperatures and loading rates that is to say, complex modulus (G^*) and phase angle (δ) that are measured using a Dynamic Shear Rheometer. Table 2.2 overleaf indicates properties of the commonly available Performance Graded bitumen types;

Performance Grade	PG 46			PG 76						PG 82					
Average seven day maximum pavement design temperature, °C	< 46			< 76						< 82					
Minimum Pavement Design Temp. °C	>(-34), >(-40), >(-46)			>(-10), >(-16), >(-22) >(-40)						>(-34), >(-40), >(-46)					
Design Shear (T315), G*/Sin δ, min 1kPa test temp @ 10 rad/s, °C	46			5 76						82					
Design Shear (T315), G*/Sin δ, min 2.2kPa test temp @ 10 rad/s, °C	46			5 76						82					
PAV ageing temperature, °C	90			9 100 (110)						100 (110)					
Dynamic shear (T315), G*Sin δ, min 5000 kPa test temp @ 10 rad/s, °C	10	7	4	25	22	19	34	31	28	25	40	37	34	31	28

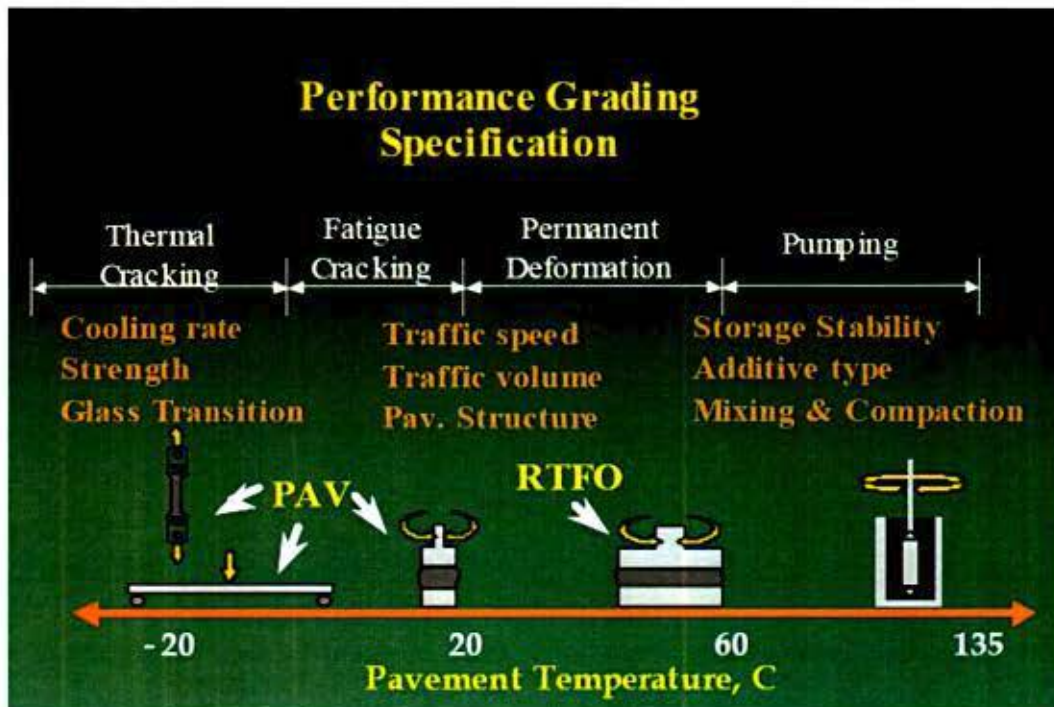


Figure 2.8: Performance grading (PG) specification (Sabita, 2005)

The PG grading system is based on climate, so the grade notation consists two portions, that is to say, high and low pavement service temperatures (Asphalt Institute, 2005). The major concern for high temperature performance is rutting, which typically takes time to accumulate, therefore an average of 7 day maximum pavement temperature is used for describing the high temperature climate (AASHTO, 2012). On the low temperature side, thermal cracking can happen during one really cold night. Therefore the minimum pavement temperature is used for describing the low temperature climate. For both high and low temperature grade, PG grades are graded in 6°C increment. The average 7 day maximum pavement temperature typically ranged from 46 to 82°C, and minimum pavement temperature typically ranged from -46°C to -10°C. A binder identified as PG 64-10 must meet performance criteria at an average 7 day maximum pavement temperature of 64°C and also at a minimum pavement temperature of -10°C. Please note that maximum pavement temperature is typically

higher than the air temperature by about 20°C since the dark colour pavement absorbs the heat and retains it. The maximum pavement temperature is typically measured at about 25mm below the pavement surface. However, the minimum pavement temperature occurs on the surface of the pavement and is equal to the air temperature.

The common minimum reliability used is 98%, so that means when the PG 64-10 binder is selected, the asphalt binder in the AC pavement should perform satisfactorily under normal traffic condition at the location where the extreme pavement temperature are within the range of -10°C and 64°C throughout its service life with a minimum 98% confidence level. Where the traffic condition is not typical, such as the really heavy traffic like interstate highway, or slow traffic such as bus stop or intersection area, one or two grades stiffer asphalt binder may be used to help prevent rutting.

Physical properties of the PG binders are measured at various temperatures both before and after laboratory aging (FHWA, 2011). The laboratory aging is conducted to simulate field conditions imposed during the asphalt pavement production process as well as from long-term environmental exposure. Physical property measurements include;

Permanent deformation or rutting resistance

This is a complex phenomenon influenced by packing of aggregate, stiffness and quality of the binder and underlying support, and is compounded by high surface temperatures and high traffic volumes. Rutting usually occurs at higher temperatures and is measured by applying an oscillating shear stress and also measure the corresponding strain using the Dynamic Shear Rheometer (Sabita, 2005). It is a common type of distress in hot mix asphalt layers, and can lead to ponding of water in

wheel tracks, which can be a serious road pavement hazard in wet weather. Rutting can also lead to poor riding quality, which in turn results in increased vehicle operating costs. From the stress-strain relationship, the Materials Modulus will be deduced from the ratio of stress-strain which is indeed a measure of the material stiffness. The DSR test provides the Complex Shear Modulus, denoted as G^* and phase angle, δ . The ratio between Complex Shear Modulus and the sine of phase angle $= \frac{G^*}{\sin\delta}$ corresponds to rutting resistance.

It is specified that the rutting specification $\frac{G^*}{\sin\delta} > 1.00 \text{ kPa on unaged binder}$ and also $\frac{G^*}{\sin\delta} > 2.20 \text{ kPa on RTFO aged binder}$.

In order to resist rutting, an asphalt binder should be stiff (it should not deform too much) and it should be elastic (it should be able to return to its original shape after load deformation). Therefore, the complex shear modulus elastic portion, $G^*/\sin \delta$ should be large. When rutting is of greatest concern (during an HMA pavement's early and mid-life), a minimum value for the elastic component of the complex shear modulus is specified. Intuitively, the higher the G^* value, the stiffer the asphalt binder is (able to resist deformation), and the lower the δ value, the greater the elastic portion of G^* is (able to recover its original shape after being deformed by a load) (Princeton, 2003).

Another way to look at this is that rutting is basically a cyclic loading phenomenon. With each traffic cycle, work is being done to deform the pavement surface. Part of this work is recovered by the elastic rebound of the pavement surface, while part is dissipated in the form of permanent deformation, heat, cracking and crack propagation.

Therefore, in order to minimize rutting, the amount of work dissipated per loading cycle should be minimized. The work dissipated per loading cycle at a constant stress can be expressed as:

$$W_c = \pi \sigma_o^2 \left(\frac{1}{G^* \sin \delta} \right) \dots \dots \dots \text{Equation (2.1)}$$

Where:

W_c = work dissipated per load cycle

σ_o = stress applied during load cycle

G^* = complex modulus

δ = phase angle

In order to minimize the work dissipated per loading cycle, the parameter $G^*/\sin \delta$ should be maximized. Similarly, the resistance to thermal cracking shall be determined by proper choice of binders taking into consideration the maximum temperatures that a pavement is likely to experience at least in 7 days in its entire life.

Resistance to fatigue damage

This usually occurs at intermediate temperatures. In the DSR test, the relationship, $G^* \cdot \sin \delta$ correlates to fatigue resistance and it is specified that $G^* \cdot \sin \delta < 5000$ kPa on PAV aged binder. In order to resist fatigue cracking, an asphalt binder should be elastic (able to dissipate energy by rebounding and not cracking) but not too stiff (excessively stiff substances will crack rather than deform-then-rebound). Therefore, the complex shear modulus viscous portion, $G^* \cdot \sin \delta$ should be a minimum. When fatigue cracking is of greatest concern (late in an HMA pavement's life), a maximum value for the viscous component of the complex shear modulus is specified. Another way to look at

this is that fatigue cracking can be considered a stress-controlled phenomenon in thick HMA pavements and a strain-controlled phenomenon in thin HMA pavements. Since fatigue cracking is more prevalent in thin pavements, the parameter of most concern for fatigue resistance can be considered a strain-controlled one. With each traffic cycle, work is being done to deform the pavement surface. Part of this work is recovered by the elastic rebound of the pavement surface, while part is dissipated in the form of permanent deformation, heat, cracking and crack propagation. The lower the amount of energy dissipated per loading cycle, the less likely fatigue cracking is. Therefore, in order to minimize fatigue cracking the amount of work dissipated per loading cycle should be minimized. The work dissipated per loading cycle at a constant strain can be expressed as (Princeton, 2003);

$$W_c = \pi \varepsilon_0^2 (G^* \sin \delta) \dots \dots \dots \text{Equation (2.2)}$$

Where:

W_c = work dissipated per load cycle

ε_0 = strain during load cycle

G^* = complex modulus

δ = phase angle

This relationship between $G^* \cdot \sin \delta$ and fatigue cracking is more tenuous than the rutting relationship. In order to minimize the work dissipated per loading cycle, the parameter $G^* \cdot \sin \delta$ should be minimized. Therefore, maximum values for $G^* \cdot \sin \delta$ for the DSR tests conducted on PAV aged asphalt binder are specified.

Table 2. 5- Dynamic Shear Binder Specifications (AASHTO, 2012)

Material type	Value	Specification (AASHTO T 315)	HMA Distress of Concern
Un-aged binder	$G^*/\sin \delta$	1.0 kPa (0.145 psi) Min.	Rutting
RTFO Asphalt Residue	$G^*/\sin \delta$	2.2 kPa (0.319 psi) Min.	Rutting
PAV Residue	$G^*, \sin \delta$	5000 kPa (725 psi) Max.	Fatigue Cracking

Studies were performed to evaluate the precision of test results of performance graded asphalt binder testing equipment and testing protocol provided high and low temperature dynamic shear rheometer test results on virgin and aged asphalt binders as presented in the Table 2.4 below (Zaniewski, 2004).

Low Temperature Cracking Resistance

The low temperature characterization is analysed using the bending beam rheometer (BBR) test that determines the creep stiffness (s) of an asphalt binder at low temperatures. This therefore implies that if a binder is too stiff at service temperatures, then low temperature cracking shall be expected (Princeton, 2003).

There are two types of thermal cracking, namely low temperature cracking and thermal fatigue cracking. Low temperature cracking develops due to the entire pavement contracting and as a result, the pavement is subjected to tensile stresses within the full depth of the asphalt layer. Transverse cracks suddenly appear that run the full depth of the pavement due to these stresses. Thermal fatigue cracking, on the other hand, is when cracks develop from the surface downwards into the asphalt layer due to the properties of the surface course (Zyl, 2018). The high temperature range test results that predict pavement resistance to rutting and fatigue damage are presented in Appendix I of this report.



Figure 2.9: (Left) Fatigue cracking, (Middle) Rutting and (Right) Low temperature cracking (Zou, 2017)

Test results to measure accuracy of low temperature binder testing equipment provided the test results presented in Appendix I of the report. The results obtained indicated that the binder would resist low temperature cracking.

Resistance to pre-mature aging

Asphalt binders undergo aging through the loss of volatiles (always known as loss of light ends) and oxidation (CITGO, 2003). Aging usually takes place during storage or transportation, during construction in the plant and placement at the site. Aging also occurs in the life of the pavement usually 7 – 10 years in service depending on the loading criteria and service temperatures (Princeton, 2003).

2.4 ZONING OF BINDER APPLICATION IN VARIOUS REGIONS IN THE COUNTRY

In the United States of America, the performance grading system of specifying asphalt binders has been adopted since it covers all types of environmental conditions. The system recognizes that the main distress mechanisms affecting pavement performance include rutting, fatigue damage and thermal cracking. Bitumen plays a major role in resisting those failure mechanisms mentioned above. The system recognizes that the resistance to those distresses are not similar and are measured at different temperatures

that seem to simulate field conditions (M.F.C. van de Ven1, 2004). Acceptance of binder is based on specific pavement temperature and traffic conditions.

In Thailand, a relationship between high temperature and low temperatures was developed by simply measuring the air temperature and pavement temperature at the surface and 50mm below the surface. This activity was done in five regions of Thailand. A linear regression was developed using the measured air temperatures as the independent variable and pavement temperatures as the dependent variable. In this relationship, wind speed, sun radiation, moisture in the atmosphere were not considered (A.Charoentharn, 2012).

Table 2.6: Maximum Pavement Design Temperatures for Thailand (A.Charoentharn, 2012)

Region	% Reliability	T _{air} (7 days - Max) °C	T _{PAV @surface max.} °C	T _{PAV @ 50mm (Max)} °C
North	50	41	42	44
	98	45	51	54
North East	50	41	47	48
	98	43	55	57
Central	50	40	48	49
	98	44	57	59
East	50	38	47	47
	98	42	56	57
South	50	38	39	44
	98	40	47	53

The low pavement temperatures at the surface was obtained by using the Super pave's low temperature model at the pavement surface given by;

$$T_{PAV@SURFACE(mm)} = 0.286 + 0.692T_{AIR(min)} \dots\dots\dots \text{Equation (2.3)}$$

However, at any specified depth, the low temperature can be obtained through

$$T_{D(min)} = T_{PAV6.5mm(Min)} - ((0.00123T_{PAV6.4mm(Min)}(d-6.4)) + 0.0146(d-6.4)) \dots\dots \text{Equation (2.4)}$$

Where

$T_{d(\min)}$ = Minimum Pavement Temperature at Depth, d

d = Depth from surface

$T_{PAV6.5mm(\min)}$ = Minimum Pavement Temperature at 6.4mm

$$= 2.2 + 0.778T_{AIR(\min)} \text{ for air temperature below } 0^{\circ}\text{C} \dots \text{Equation (2.5)}$$

$$= 6.83 + 1.014T_{AIR(\min)} \text{ for air temperature above } 0^{\circ}\text{C} \dots \text{Equation (2.6)}$$

The Table below shows the minimum pavement temperatures for Thailand;

Table 2.7: Minimum Pavement Design Temperatures for Thailand (A.Charoentham, 2012)

Region	% Reliability	$T_{air(\min)}$ $^{\circ}\text{C}$	T_{PAV} @surface Min. $^{\circ}\text{C}$	T_{PAV} @ 50mm (Min) $^{\circ}\text{C}$	Temperature Zoning
North	50	5	4	12	-2 to 54 $^{\circ}\text{C}$
	98	-1	-2	6	
North East	50	6	4	13	0$^{\circ}\text{C}$ to 57 $^{\circ}\text{C}$
	98	2	0	9	
Central	50	10	7	17	1$^{\circ}\text{C}$ to 59$^{\circ}\text{C}$
	98	4	1	11	
East	50	12	9	19	4$^{\circ}\text{C}$ to 57$^{\circ}\text{C}$
	98	8	4	15	
South	50	16	11	22	5$^{\circ}\text{C}$ to 53$^{\circ}\text{C}$
	98	10	5	16	

Based on the volume and speed of traffic, a performance grade with maximum temperature of 70 $^{\circ}\text{C}$ was considered.

Table 2.8:- Proposed Performance Graded Binders for Thailand (A.Charoentham, 2012)

Proposed Grade	PG 58			PG 64			PG 70		
High Temperature Grade	58			64			70		
Low Temperature Grade	+8	+2	-4	+8	+2	-4	+8	+2	-4

In South Africa, zoning of asphalt binder application has been done in a way to achieve maximum pavement performance. The principle rules included;

- Performance grading zoning should rely on bitumen specific constituent models of stress – strain relationships under loading conditions experienced in the field;
- Consider pavement conditions including temperature, traffic volume and speed;
- Acceptance limits derived from experience and documented field performance.

In the case of the first criteria, the existing van der Poel nomograph which allows estimation of creep stiffness at a range of temperatures and loading times using the penetration index can be used to derive the bitumen constituent models based on Penetration and softening point measures (M.F.C. van de Ven1, 2004)..

For the case of pavement conditions like temperatures, details were available at the weather bases in South Africa to obtain the maximum and minimum design pavement temperatures. These were used to zone the performance grades needed for various regions in the country.

Based on experience and past field performance, several binders would suit the temperature ranges in the different regions. An overview is presented in figure 2.2. (M.F.C. van de Ven1, 2004).

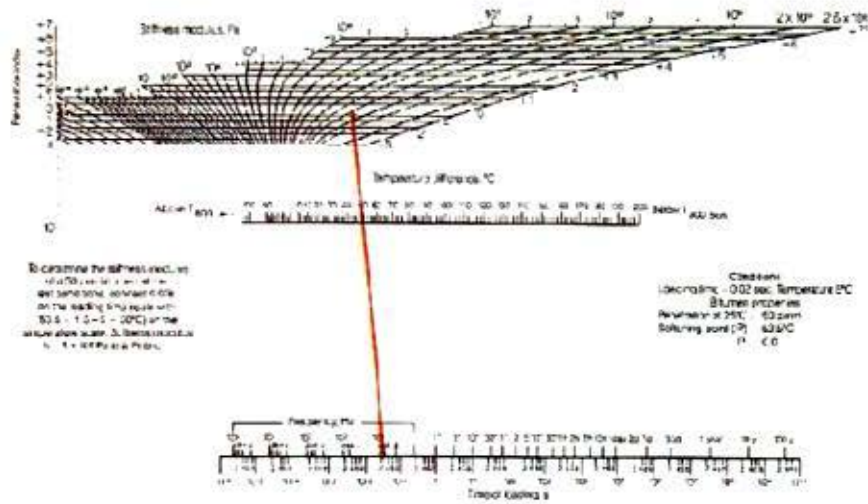


Figure 2.10: Van der Poel Nomograph

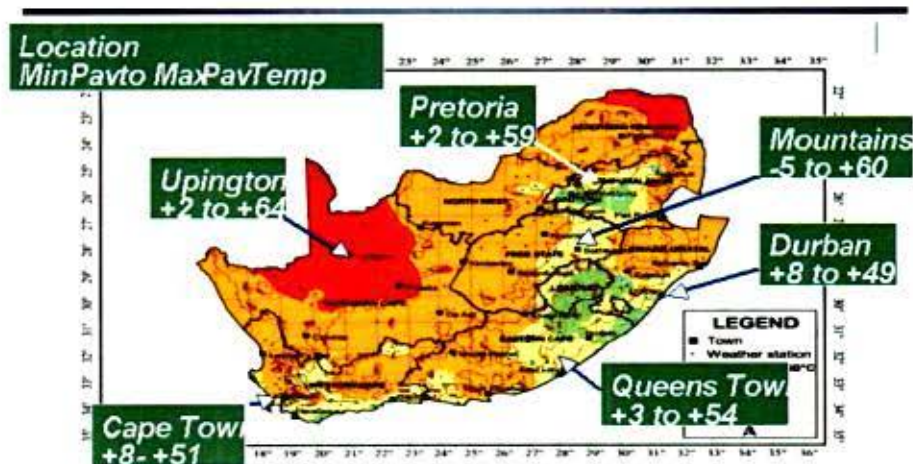


Figure 2.11: Design Pavement Temperatures in South Africa (M.F.C. van de Ven1, 2004).

Regarding the acceptance limits, the performance ought to be selected such that the acceptance limits of the binders are fully defined in relation to workability, rutting resistance and fatigue resistance.

In Sudan, based on the equation developed by SHRP researchers, determination of the highest and minimum pavement design temperatures has been done and divided the country into two major categories of performance graded binders (Kobbail, 2005). The categories recommended are PG 70-10 and PG 64-10. The data was obtained from

eighteen weather stations (Kobbail, 2005). They used theoretical analysis of actual conditions performed with models for net heat flow, energy balance, and assuming many factors each with a typical value such as solar absorption (0.9), radiation transmission through air (0.81), atmospheric radiation (0.7) and wind speed (4.5 m per sec). The equation for Maximum Pavement Temperatures at 20mm depth is as follows;

$$T_{20mm} = (T_{air} - 0.00618Lat^2 + 0.2289 Lat + 42.2) (0.9545) - 17.78 \dots\dots\dots\text{Equation. (2.7)}$$

Where,

- T_{20mm} = High pavement design temperature at a depth of 20 mm
- T_{air} = Seven-day average high air temperature, °C
- Lat = The geographical latitude of the project

Similarly, the Canadian Strategic Highway Research Program measured and designed low pavement temperatures based on data captured from the eighteen weather stations. The Equation developed for the Low Temperature is provided below;

$$T_{min} = 0.859T_{air} + 1.7 \dots\dots\dots\text{Equation (2.8)}$$

Where;

- T_{min} = Low Pavement Design Temperature at 20mm depth;
- T_{air} = Air Temperature.

The tables below show the maximum and minimum pavement temperatures measured in the various states in Sudan.

Table 2.9: Sudan Average Maximum and Minimum Temperatures (Kobbail, 2005)

ST. No.	year Station Name	2001		2002		2003		2004		2005	
		Avg max (7-day)	Min (1-day)	Avg max (7-day)	Min (1-day)	Avg max (7-day)	Min (1-day)	Avg max (7-day)	Min (1-day)	Avg max (7-day)	Min (1-day)
		Highest C°	Lowest C°	Highest C°	Lowest C°	Highest C°	Lowest C°	Highest C°	Lowest C°	Highest C°	Lowest C°
1	Wau	39.64	11.70	40.00	12.00	39.66	15.20	39.93	15.00	40.29	15.00
2	Atbra	45.41	12.00	45.50	7.50	45.17	10.00	45.54	8.50	45.27	8.20
3	Elfasher	40.49	5.00	40.87	4.30	40.64	8.20	40.71	6.40	40.51	4.20
4	Elgenena	39.84	7.00	40.14	5.50	39.71	8.00	39.47	9.20	38.99	8.00
5	Sennar	42.06	9.00	43.51	4.70	42.65	7.70	42.93	9.00	43.09	7.40
6	Malakal	40.67	11.50	41.00	12.50	41.54	13.60	42.14	13.50	42.10	15.50
7	Nyala	39.96	11.20	41.04	9.00	40.33	12.00	40.50	11.50	40.56	10.90
8	Wadi Halfa	44.03	7.00	44.99	6.00	44.69	6.50	44.24	4.00	44.06	4.50
9	El Damazin	41.29	10.50	41.86	9.50	41.79	11.00	41.25	11.50	42.16	11.70
10	Dongla	46.19	7.30	46.17	3.60	46.06	5.30	46.14	4.60	46.10	6.00
11	Port Sudan	44.04	14.00	42.77	12.00	43.40	12.00	43.12	13.00	43.04	15.30
12	Kassala	41.99	13.50	43.41	8.20	42.71	11.50	42.74	10.30	42.90	11.30
13	Juba	39.54	13.70	39.40	14.50	39.94	14.00	39.56	14.50	41.36	13.80
14	Rashad	37.86	12.00	38.63	9.00	39.30	14.20	39.00	14.50	39.06	11.50
15	Elgedaref	42.26	13.60	43.66	10.50	43.07	14.00	42.63	13.10	42.96	12.00
16	Elobied	40.30	9.00	41.54	7.00	41.51	10.50	41.43	9.80	41.53	9.00
17	Khartoum	43.36	10.50	44.14	8.60	43.36	12.00	44.01	11.00	43.46	10.30
18	Kadugli	41.29	11.60	41.71	7.50	41.97	7.60	41.76	8.50	41.97	10.00

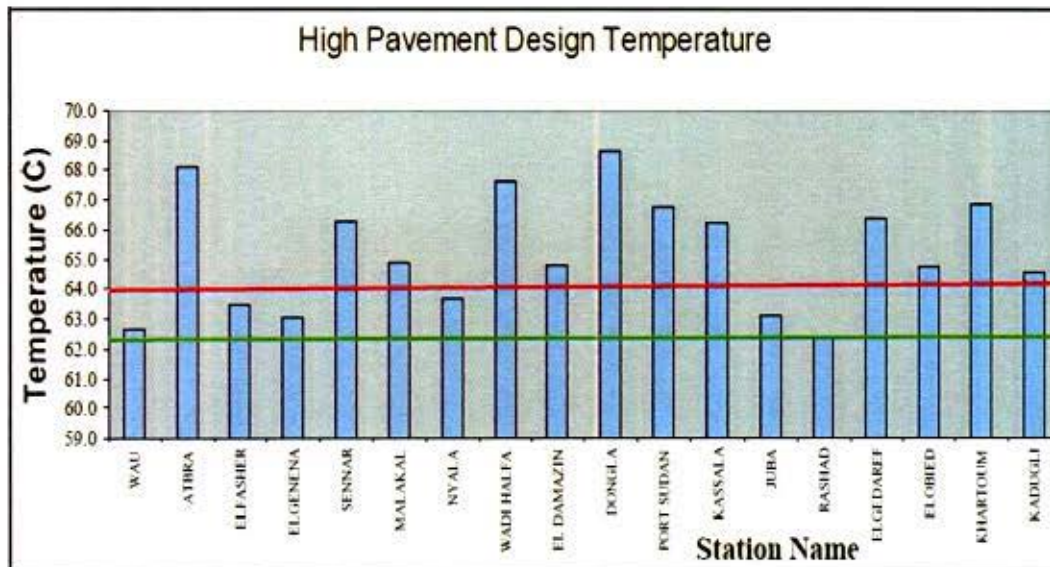


Figure – 2.12: High Pavement Design Temperatures in Sudan (Kobbail, 2005)

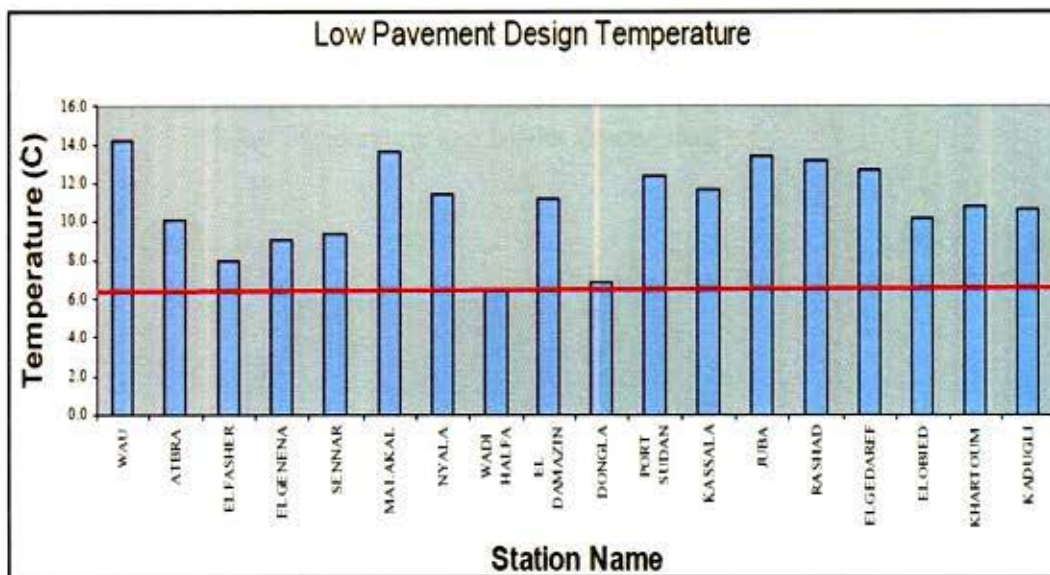


Figure – 2.13: Low Temperature Design Temperature (Kobbail, 2005)

The Maps below show the typical Binder application based on pavement temperatures measured in the various states in Sudan;

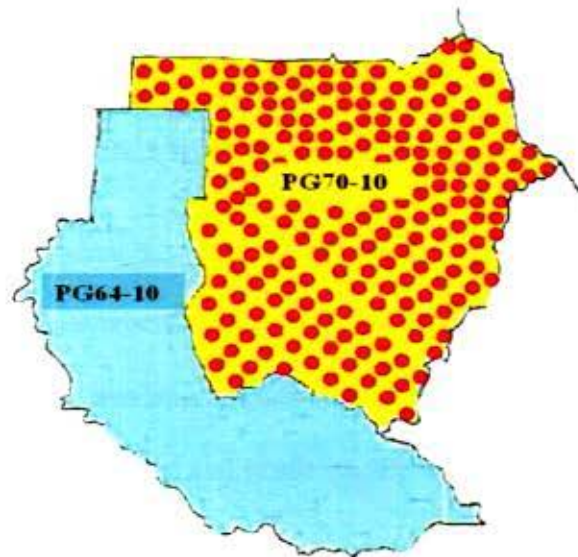




Figure 2.14  PG 70-10  PG 64-10
Sudan Temperature and Binder Zoning map

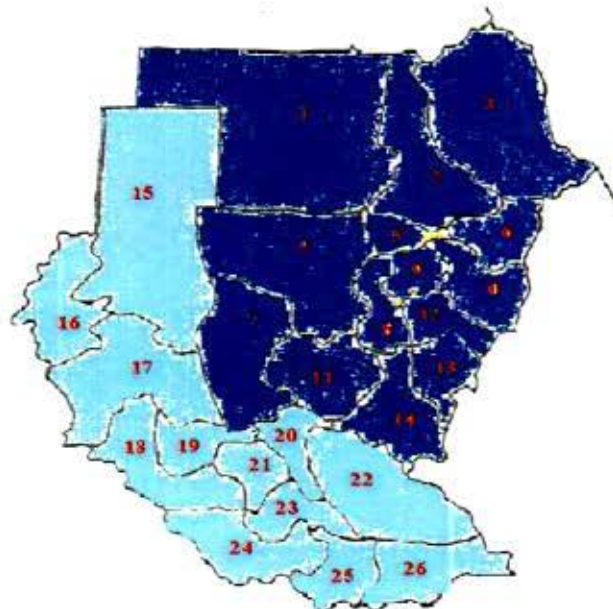


Figure 2.15: Zoning of Binder application in each state in Sudan (Kobbail, 2005)

Uganda experiences generally higher temperatures that no low temperature risk is envisaged to affect the eventual asphalt pavement. The performance of bituminous mixtures varies from region to region based on the variation of environmental factors. Based on record from the meteorological centre the Northern districts experienced highest temperatures with isolated areas in Kasese. These varied from 40 to 37°C (Bataze, 2016).

To achieve satisfactory pavement performance, an asphalt binder that conforms to the daily average 7 day maximum temperatures and traffic loading is key in determining the most economical binder type and grade for the asphalt concrete design mix. The figure below shows the maximum and minimum 7 day temperature variation in Uganda.

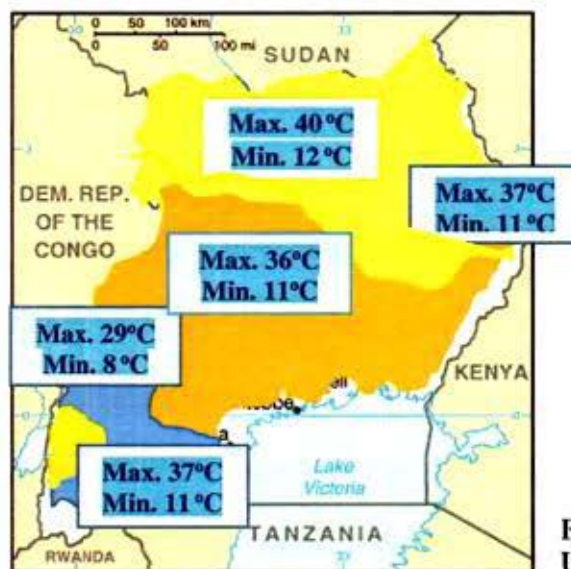


Figure 2.16: Air Temperatures in Uganda (Bataze, 2016)

A linear regression model in the form $y = a + bx_1 + cx_2 + \epsilon$ was sought for Uganda to relate latitudes, air and pavement temperatures, where a is a constant, b and c are regression coefficients, y is the dependent variable (pavement design temperature) and x_1 and x_2 represent the maximum or minimum temperatures and latitudes respectively and ϵ is the error in the model.

CHAPTER THREE: METHODOLOGY

3.1 INTRODUCTION

This chapter provides the necessary equipment and test procedures to achieve the objectives. It further describes the scope, conditioning of asphalt binders before testing, the Standards followed to achieve the test results and the temperature measurements that informed the temperature zoning of the binders in the different regions of the country.

3.2 STUDY AREA

The study was conducted in Uganda and bitumen samples were obtained from ongoing road construction sites. The selected binder sources included;

- Capacity improvement of the Kampala Northern Bypass project involving use of penetration grade bitumen 35/50. The Civil works contractor is Mota-Engil Africa. The Supervision Consultant is COWI A/S;
- Upgrading of Kamuli, Gulu and Arua township roads where penetration grade bitumen 50/70 was being used. The Civil works Contractor is China Chongqing International Construction Company (CICO) and Supervision Consultants is the in-house Uganda National Roads Authority (UNRA);
- Mbarara Bypass project where performance grade bitumen, PG 76-10 was being used. The Civil works contractor was China Railway Engineering Group Seven. The Supervision Consultant is COWI A/S;
- Upgrading of Mukono – Kayunga – Njeru road project involving use of performance grade bitumen, PG 70-10. The Civil works contractor was SBI International Holdings AG and Supervised by M-PEP and Multi-plan.

3.3 SAMPLING TECHNIQUES, SIZE AND INSTRUMENTS

Three representative samples of bitumen from each selected road project were obtained following procedures provided in BS EN 58: 2004 and BS 2000: Part 474: 2004 (BSi, 2004). The samples from the Capacity Improvement of Kampala Northern Bypass Project, Mukono – Kayunga – Njeru project and Mbarara Bypass Project were obtained using permanently installed systems. The samples from Kamuli, Gulu and Arua Township road projects were obtained using hand tools like chisels and hammers from drums.

The samples were clearly and durably labelled and delivered to the Central Materials Laboratory, TanRoads Agency, Dar Es Salaam, Tanzania for testing.

3.4 TESTING PROCEDURE

Testing commenced with consistence tests to confirm the Binder Classifications. The classification tests included penetration tests, ductility, specific gravity, viscosity, softening point, flash and fire points. The Binders were then subjected to physical property testing starting with virgin samples and ending with aged samples. The physical property testing provided in-service properties using laboratory simulation tests. These tests included rolling thin film oven tests, Pressure ageing vessel, rotational viscosity and dynamic shear. Testing was done at CML TanRoads, Dar es salaam, Tanzania.

3.5 DATA PRESENTATION

Test results were presented on Standard Test Forms and summarised accordingly as shown in Appendix 1 to give a quicker overview in interpreting results. Further

presentation were made in form of figures with representative graphs showing the overall behaviour of the materials.

3.6 DATA ANALYSIS

The test results obtained were analysed using Micro soft excel.

3.7 RESEARCH APPROACH AND DESIGN

This research was experimental since different bitumen samples were taken for laboratory testing to classify them and determine the consistency and physical properties. Air and pavement temperature measurements were taken to obtain the relationship between air temperature, pavement temperature and locations of the weather stations.

3.8 CONSISTENCY TESTS

Samples were tested for handling properties to determine the penetration value and specific gravity and thus confirm the classification of the binders. Binders were further tested for ductility to measure their tensile resistance, flash point to understand the safety in handling the various bitumens, softening point to determine the tendency to flow during service in elevated temperatures, viscosity for fluid in mixing and pumping and ductility to measure the cohesive ability of a binder. These were tested in accordance with the relevant AASHTO testing procedures.

3.8.1 PENETRATION TESTS (ASTM D 5-86)

A: OBJECTIVE

Penetration test was carried out to measure the consistency of bituminous materials expressed as the distance in tenths of a millimetre that a standard needle vertically

penetrates a sample of the material under known conditions of loading, loading time and temperature. The distance the needle penetrates in units of 1/10 mm is termed as the penetration value. This method however applies to penetration values in the range of 2-500. Penetration tests is usually carried out on virgin binders to confirm the classification of penetration grade asphalt binders.

B: SIGNIFICANCE

The significance of the penetration test was to confirm the binder nomenclature and thus classification.

C: SAMPLE PREPARATION;

Heated bitumen was poured in to two penetration tins filled to at least 10mm above the expected penetration depth. They were immediately covered to protect them from dust using lids and allowed to cool. These samples were later placed in a transfer containers and placed in a water bath to achieve the test temperatures.

D: TEST APPARATUS

- Standardized Penetrometer;
- Standard penetration needle;
- Penetration cups;
- Stop Watch with accuracy of 0.1s;
- Perforated plate supported minimum 50mm above bottom of the bath and minimum 100mm below the water surface;
- Water bath
- Standardized thermometer;

- Calibrated thermometers to calibrate the water bath;
- Source of light

E: TEST PROCEDURE

- Well prepared samples were placed under the penetration needle and levelled vertically below the spindle;
- The spindle was lowered for the tip of the needle to just touch the top of the bitumen sample in the can;
- The needle was released for about five seconds and later the reading on the gauge was recorded;
- The test was conducted each time using two cans and an average value was taken as the accepted penetration value;
- This data was recorded in a standard form for analysis

F: REPORTING

The depth of penetration of the needle in the cup containing the bitumen sample was read off the scale and recorded to the nearest whole number.

3.8.2 FLASH AND FIRE POINT TESTS (ASTM D 92-90 & AASHTO T 48)

A: OBJECTIVE

Flash point is defined as the lowest temperature at which the vapour from a liquid is ignited by an open flame. Fire point is the lowest temperature at which the liquid will continue to burn without further heat supply for at least 5 seconds. Fire point temperatures are usually higher than flash point temperatures.

The major objective of the test was to measure the binders' combustibility. Flash and fire points give temperature ranges for safe handling of asphalt binders.

B: SIGNIFICANCE

The importance of carrying out the flash and fire point tests was to determine the temperature above which asphalt binders shall not be heated and ensure safe handling of binders.

C: SAMPLE PREPARATION;

The material in the container was heated to about 40°C or just below the expected flash point to allow it to be placed in a cup. The cup was filled with then sample until the required mark.

D: APPARATUS

- Cleveland open cup apparatus;
- Square shield, 460mm wide by 610mm tall, with open front;
- Gas supply with Test flame with radius not exceeding 152mm;
- Temperature measuring thermometer, ASTM 11C;
- Barometer for confirming the atmospheric pressure.

E: PROCEDURE

- i. The apparatus was set up in a dust free room shielded from strong light to ensure the accurate determination of flash point;
- ii. A thermometer was placed vertically and the tip in the sample in the cup with the bulb about 6.4mm above the bottom of the cup and located at a point half

- between the centre and the side of the cup on a diameter perpendicular to the arc of the sweep of the test flame and placed on the opposite side;
- iii. Asphalt binders were obtained from their containers, pre-heated just below the expected flash point and placed in the sample cup to the required mark.
 - iv. The samples were then heated starting with high rate and then gradually reducing towards the flash point with a test flame between 3.2 to 4.8mm diameters being passed over the surface of the bitumen sample in cup usually 2mm above the surface of the binder;
 - v. The rate of heating was approximately 17^oC/minute initially and 5-6^oC before attaining the expected flash point;
 - vi. When a flame just started to show up at the top of the bitumen in the sample cup, the temperature was recorded as the flash point;
 - vii. Further heating of bitumen and passing of the test flame over the sample cup provided a continuous flame for about 5 seconds;
 - viii. This temperature was recorded as the fire point;
 - ix. Care was taken to remove any bubbles on the surface of the binder in cup to ensure false flames were not recorded;
 - x. There was no need to apply the conversions during the test result calculations since we maintained atmospheric pressures throughout the testing process. Otherwise, Corrected Value = $C + 0.03x (760-P)$ would apply.



Figure 3.1; Flash and Fire Point Test

It was worth noting that a significant difference between flash point and fire point temperatures indicate generally the presence of highly volatile and flammable materials in a relatively non-volatile or non-flammable material.

F: REPORTING

Three trials were recorded to the nearest 1^oC and the average temperature was taken as the flash point and fire point.

3.8.3 SOFTENING POINT (ASTM D 3461-14)

A: OBJECTIVE

Softening point is a consistency test carried out to measure the temperatures at which asphalt binders tend to flow at elevated temperatures during in-service performance of asphalt concrete pavements.

B: SIGNIFICANCE

The test was important in describing the maximum temperatures at which the bitumen binders can be heated during road operations.

C: SAMPLE PREPARATION

Samples were obtained from their original containers and heated in the oven to ensure that they could be retrieved and filled in the softening containers. The samples were filled in the containers and allowed to cool down just below 15°C.

D: APPARATUS

- Softening point apparatus;
- Ring and balls
- Temperature measuring thermometer
- Heat source;
- Stirling rod

E: PROCEDURE

- i. The prepared samples in the ring were carefully placed in the water bath with balls placed on them;
- ii. The samples were subjected to gentle heating, rising the temperature gently;
- iii. The temperature of the water in the bath was adequately maintained by continuous mixing or stirring using a glass rod to ensure uniform distribution of temperatures and thus bitumen softening at the same rate;
- iv. The balls were observed passing through the rings and when they reached the both disc, such temperatures were noted;
- v. An average temperature was calculated for such balls that fell at different periods of time;
- vi. The average value was recorded as the softening point of each binder.

Below are figures showing how different binders softened;

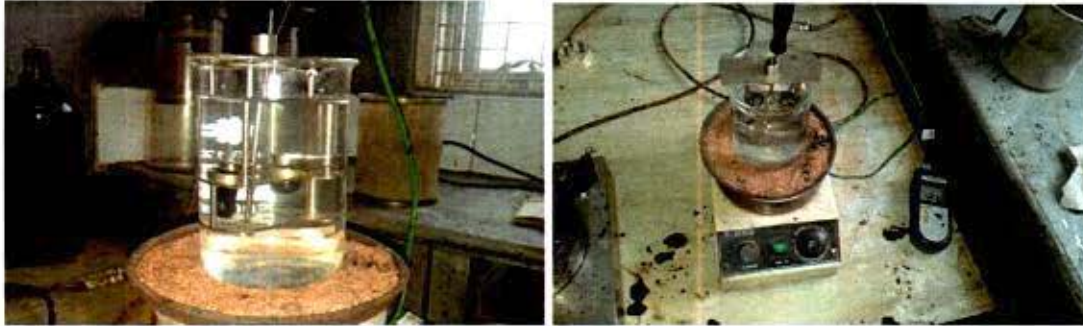


Figure 3.2; Softening Point in Progress

F: REPORTING

An average of two trials was recorded as the softening point to the nearest whole number.

3.8.4 SPECIFIC GRAVITY (ASTM T228/D70)

A: OBJECTIVE

To determine the ratio of volume of semi-solid bitumen to the volume of water at room temperature.

B: SIGNIFICANCE

- To determine the quantities of materials in the job mix formula;
- To classify the materials according to the Specific gravity;
- To determine the foreign material present.

C: SAMPLE PREPARATION

The Bitumen samples were carefully heated in an oven until they become sufficiently fluid to pour out. The heating did not exceed 30 minutes.

D: APPARATUS

- Specific gravity bottles of 25ml capacity;
- Water bath with thermometer;

- Weighing Balance.

E: PROCEDURE

- i. the asphalt binder was heated to (75 – 100)°C;
- ii. the asphalt binder was mixed during the heating continuously for getting a uniform and with no voids asphalt;
- iii. the pycnometer was cleaned and weighed;
- iv. the pycnometer was filled with pure water at 25 °C and weight it;
- v. the pycnometer was half filled with asphalt binder then cooled for half hour in water at 25°C and weighed;
- vi. the rest of the space remaining was filled from the pycnometer in step (v) with water at 25°C and weight it;
- vii. the results were then computed.

F: REPORTING OF RESULTS

The Specific Gravity was recorded to the nearest three decimal places.

3.8.5 DUCTILITY (AASHTO T 51-94 & ASTM D 113-86)

A: OBJECTIVE

Ductility is the measure of the cohesive strength of a binder. It is done by stretching a standard-sized briquette of asphalt binder until its breaking point. The stretched length in centimetres at breaking is then reported as the ductility. This is an empirical test and is only tested at 25 degrees centigrade.

B: SIGNIFICANCE

The test provided the visco-elastic behaviour of a binder. This test was important as it measures the binder's resistance to temperature change cracking and fatigue cracking.

C: SAMPLE PREPARATION

Bitumen binders were heated to ensure they can flow into the test apparatus and then cooled down in a water bath at a temperature of about 27°C. The excess bitumen is cut off or trimmed off using a hot knife. The binder in the moulds were then kept in the water bath of the ductility machine for about 90 minutes.

D: APPARATUS

- Water bath capable of maintaining 25 degrees centigrade;
- Ductility testing apparatus;
- Briquette mould including base plates;
- Thermometer;
- Release agent;
- Oven;
- Trimming tool;
- Specific gravity additive;
- 0.300mm sieve;



Figure 3.3: Ductility test

E: PROCEDURE

- i. Assembled the ductility testing machine;

- ii. Applied the release agent on the base and sides of the mould to prevent the binder from adhering on the surfaces;
- iii. The sides were then removed, the clips hooked on the machine and the ductility machine operated to elongate or stretch the binder until breaking point.
- iv. Great care was taken to ensure uniform rate of pulling, controlled pouring temperature and temperature of testing was controlled;
- v. The distance by which the binder spread or extended just until breaking was recorded as the ductility value in centimetres;
- vi. The values that were obtained were compared to those specified in BIS.

F: REPORTING

The distance at which breaking of the binder occurred was recorded in centimetres to the nearest whole number as the ductility value.

3.9 PHYSICAL PROPERTY TESTING

3.9.1 MASS LOSS TEST (AASHTO T 240 & ASTM D 2872)

A: OBJECTIVE

The rolling thin oven test was conducted on virgin asphalt binder to measure its resistance to early hardening potential during transportation to the asphalt mixing plant, during mixing and on-site compaction.

B: SIGNIFICANCE

The loss of volatiles increases the binder's viscosity. This test provided simulated short term aged asphalt binder for physical property testing (TRL, 2002). The purpose of

this test was to determine the early hardening potential of the binder and predict its susceptibility to cracking and fatigue damage.

C: SAMPLE PREPARATION

The sample was inspected to establish whether it was free of water. The virgin binder in its container was heated not exceeding 150°C to ensure that the sample is completely fluid. Manually stir the sample while avoiding adding air bubbles. In three RTFO glass containers, a weight of about 35g was taken in each container.

D: APPARATUS;

- RTFO Glass containers;
- Balance readable to 0.01g;
- Forced ventilated Oven with a vertical turntable
- Flow-meter;
- Cooling rack

E: PROCEDURE

- i. 35g of the sample were poured into each of the required glass containers, providing sufficient material for characterizing tests which are to be run on the residue;
- ii. Immediately after pouring the sample in the glass containers, the container was turned to a horizontal direction;
- iii. Rotated to pre-coat its cylindrical surface;
- iv. Care to be taken not to pour out the sample;

- v. Placed the containers horizontally in a clean cooling rack that is maintained in a draft-free, room-temperature located away from ovens and other sources of heat;
- vi. Allowed the glass containers to cool in the cooling rack for a minimum of 60 minutes and a maximum of 180minutes;
- vii. The prepared samples were weighed and allowed to cool down in the RTFO glass containers;
- viii. The samples were then kept in an Oven capable of attaining 163.5°C in ten minutes;
- ix. After 85 minutes of keeping in the oven, the samples were removed, allowed to cool down and weighed to determine the final mass of binder and glass container;
- x. The difference between the original binder mass and RTFO mass is due to a partial distillation and oxidation to which the binders were subjected.
- xi. This change in mass simulates the short-term ageing which occurs during heating, pumping, spraying, and mixing with aggregates in the plant, during road layer finishing and compaction;
- xii. The mass loss was compared with the specification to confirm adherence with general requirements for use in the road asphalt pavement construction as presented below;



Figure 3.4; Rolling Thin Film Oven Test in Progress

F: REPORTING

The mass loss after ageing were measured to the nearest 0.001g and percentage loss computed and recorded in standard rolling thin film oven test forms.

3.9.2 PRESSURE AGEING VESSEL, PAV (AASHTO T 316)

A: OBJECTIVE

An Asphalt binder ages from the time it is transported to the Asphalt Mixing Plant, during mixing, placement and in pavement performance. The in-service hardening will govern how long the asphalt concrete pavement remains intact before deterioration.

These Asphalt binders were exposed to heat and pressure to simulate in-service ageing over a 7 to 10 year period.

B: SAMPLE PREPARATION

The residual binders from the rolling thin film oven test samples were used for the pressure ageing vessel test.

C: APPARATUS

- Pressure ageing vessel complete with all accessories;
- Weighing scale capable of measuring up to 0.01g
- RTFO glass containers;
- Stainless steel pans for PAV testing

D: SIGNIFICANCE

To determine the in-service hardening potential of binders and lowest temperatures at which the binder can be exposed to before asphalt concrete pavement failure can be witnessed.

E: PROCEDURE

- i. RTFO aged asphalt binder samples were placed in stainless steel pans and then aged them for 20 hours in a heated vessel at 110 °C pressurized to 305 *psi* (2.10 MPa or 20.7 atmospheres);
- ii. The temperature of 110°C other than 100°C for moderate climate and 90°C for cold climate was chosen to simulate the in-service hot climate where the binders were being applied in the asphalt concrete mixtures;
- iii. The temperatures and pressures subjected to the binder were adequate to completely oxidise the binder thereby increasing its viscosity until such a point when there was no more oxidation through immobilization of the most chemically reactive elements;
- iv. After the ageing period, the samples were cooled in a vacuum degassing oven to ensure all the entrapped air bubbles dry out and then stored for use in Dynamic shear testing.



Figure 3.5; Pressure Ageing Vessel (PAV) Test Machine



Figure 3.6; PAV Aged samples

It is evident that oven tests usually rely on high temperatures and thin asphalt films to accelerate the oxidation process. This provides a higher volatile loss than in service HMA does and Pressure tests work by using high pressure to increase the diffusion rate of oxygen into an asphalt binder sample. In general, this approach limits the loss of volatiles while aging the asphalt binder sample.

3.9.3 DYNAMIC SHEAR TESTS (AASHTO T 320)

A: OBJECTIVE

The dynamic shear tests were carried in order to characterize the viscous and elastic behaviour of asphalt binders at medium to high temperatures which simulates in-service resistance of eventual mixture to rutting. The tests were conducted on both virgin and aged binders. The tests were largely software controlled but procedures that were followed complied with those set out in AASHTO M332 – 14.

B: SIGNIFICANCE

The test allowed prediction of early and in-service hardening potential of the binders when used in the mixtures through laboratory simulations. The laboratory simulations were used to mimic field or operational conditions.

C: SAMPLE PREPARATION

For virgin samples, the original binder was heated in its container to 140°C and a small sample obtained for the test.

D: APPARATUS

- DSR apparatus complete with all accessories;
- Computer Monitor;
- Weighing scale capable of measuring up to 0.01g;

E: PROCEDURE FOR TESTING

- i. The Dynamic Shear tests were conducted on both virgin and aged binders;
- ii. These samples were obtained after pre-heating each sample to about 140°C to flow;
- iii. The DSR machine was set up in both cases noting that for virgin binders, a disc of 8mm was used and for aged binders, a disc of 2mm was used;
- iv. Further still, when testing virgin binders, the machine is set to 1.0 SHRP mark and for aged or residue bitumen, the SHRP Point is set at 1.5 mark;
- v. The testing for the performance graded bitumen types is commenced at temperatures significantly below the specified maximum temperatures and oscillations at different temperatures are measured until sample failure with frequency maintained at 1.59Hz or 10 radians per second. The purpose of maintaining the frequency at 10 radians per second is to simulate the shearing action that corresponds to a traffic speed of about 90km/hr;
- vi. At sample failure, I were able to conclusively define the grade of the binder since failure maximum temperature is recorded.



Figure 3.7: Dynamic Shear Tests in progress

The combination of the complex shear modulus G^* and phase angle δ were used to characterise the visco-elastic properties of the binders. DSR software used in these testing was Bohlin Software – Integrated DSR 10M that performs the necessary calculations automatically as it uses the following equations:

$$\tau_{\max} = \frac{2T}{\pi r^3} \dots\dots\dots\text{equation 3.1}$$

$$\gamma_{\max} = \frac{\theta r}{h} \dots\dots\dots\text{equation 3.2}$$

Where;

τ_{\max} = maximum applied stress

γ_{\max} = maximum resultant strain

T = maximum applied torque

r = specimen radius (either 4 or 12.5 mm)

θ = deflection (rotation) angle (in radians)

h = specimen height (either 1 or 2 mm)

Then, the complex modulus (G^*) and phase angle (δ) are determined by:

$$G^* = \frac{\tau_{\max}(\text{stress})}{\gamma_{\max}(\text{strain})} \dots\dots\dots\text{Equation 3.3}$$

Where;

G^* = Complex shear modulus

In order to resist rutting, the asphalt binders were tested for stiffness to ensure they do not deform greatly and yet remain elastic. Therefore, the complex shear modulus elastic portion, $G^*/\sin \delta$ should be large. When rutting is of greatest concern (during an HMA pavement's early and mid-life), a minimum value for the elastic component of the complex shear modulus shall be determined and compared with the specified value. Intuitively, the higher the G^* value, the stiffer the asphalt binder is (able to resist deformation), and the lower the δ value, the greater the elastic portion of G^* is (able

to recover its original shape after being deformed by a load. The results of the DSR on both virgin and aged binders are summarised in the table 3.3 below;

F: REPORTING

The dynamic shear tests were conducted and the phase angles and respective complex shear modulus values were recorded. The relationship between the ratios of complex shear modulus and the sine of the phase angles were recorded to predict the binder's early hardening during mixing, placement and onsite compaction. Similarly, the products of the complex shear modulus values and the sine of the phase angles were recorded to predict in-service hardening.

3.9.4 ROTATIONAL VISCOSITY TESTING (AASHTO T 316)

A: OBJECTIVE

Rotational viscosity was carried out and the apparent viscosity of bitumen at elevated temperatures was measured. The torque on the apparatus measuring geometry, rotating in a thermostatically controlled sample holder containing a sample of bitumen was used to measure the relative resistance to rotation.

B: SIGNIFICANCE

To analyse the ability of the binder to be pumped and mix well with aggregates in the asphalt mixture thereby reducing the volume of voids in the mix and improving the overall density of the mix.

C: APPARATUS

- Rotational viscometer, capable of measuring the torque required to rotate the selected apparatus-measuring geometry at selected constant temperature while

submerged in asphalt binder at constant desired test temperature, and with the capability to convert the torque measurement to viscosity in Pascal seconds (Pa-s).

- Apparatus measuring-geometry;
- Temperature-controlled Thermal Chamber Heater; for heating the sample of asphalt at the test temperature;
- Re-usable Sample-chambers;
- Temperature Controller, capable of maintaining the specimen temperatures to $\pm 1.0^{\circ}\text{C}$ for the test temperature between 150 and 260°C ;
- Balance readable to 0.1g for determining the mass of the sample;
- Platinum Resistant Thermometer (PRT)

D: SAMPLE PREPARATION

The material was obtained in its original state or aged state. In the original state, adequate sample to fill the disc was used. The rest of the excess binder was cleaned off the disc.

E: PROCEDURE

- i. Instrument's electronics were warmed up for at least 15 minutes before conducting the analysis;
- ii. The Temperature controller was set to the desired test temperature;
- iii. An apparatus measuring geometry that would develop a resisting torque between 10 to 98% of the instrument capacity at the selected speed was selected;
- iv. Higher torque readings for accurate measurements were maintained,

- v. The sample was pre-heated in sample apparatus-measuring geometry until temperature equilibrium was obtained for at least 15 minutes;
- vi. An adequate volume of sample was added to the apparatus-measuring geometry that was used to the sample chamber;
- vii. This amount was obtained by calculating from the approximate density data for the sample and then returning the sample chamber to the temperature controlled chamber heater;
- viii. The sample was thoroughly stirred to obtain a representative sample before weighing. Care should be taken not to overfill the sample chamber bearing in mind that the apparatus-measuring geometry will be completely immersed;
- ix. The selected pre-heated apparatus-measuring geometry was inserted into the liquid in the chamber, and couple it to the viscometer;
- x. the asphalt sample was brought to the desired temperature in 30 minutes and allow it to equilibrate at the desired test temperature;
- xi. The mortar rotation of the viscometer was started at a speed that would develop a resisting torque that is between 10 to 98% the full scale instrument capacity;
- xii. The speed was maintained and allowed the sample to equilibrate for an additional 5 minutes;
- xiii. Either the viscometer or torque was measured at 1 min intervals for a total of three minutes;
- xiv. The above was repeated for each test temperature required.

F: REPORTING

The test results were recorded to the nearest significant figure.

3.10 PAVEMENT AND AIR TEMPERATURE MEASUREMENT

Pavement and air temperatures were measured during the month of August and September where it is expected to be very hot in most parts of the country. Low Air temperatures were measured between 0530 and 0630 hours and high temperatures, between 1330 and 1500 hours. The record was recorded in pre-determined templates and analysis done.

3.10.1 AIR TEMPERATURE MEASUREMENT

The temperatures were measured by hand held gadgets that were able to give minimum and maximum temperatures during morning hours and afternoon periods respectively. It was important to ensure that no air interference prevailed at the sites of measurements. Presence of traffic whose affect the air temperatures was taken into consideration during measuring and no such records have been considered. The temperature readings read from the gadgets were recorded after steady values were obtained. Care was taken to measure low temperatures on typically rainy and cold nights to obtain the estimated low temperatures the pavement may be subjected to.

3.10.2 PAVEMENT TEMPERATURE MEASUREMENT

Pavement temperatures were measured both on the surface and 20mm in the surfacing. Before readings could be taken, the sites were cordoned off to ensure no interference with environment. The Temperature gadgets were simply lowered for the tip to contact the road surface until readings became steady. For the depth measurements, the surfaces were drilled using a hammer and 2 inch nails to provide a test depth required. The temperature gadgets were lowered to the entire drilled depths and steady readings were recorded. Several readings were recorded to even out effects due to moving air.

It was observed that surface temperatures were generally lower than depth temperatures largely due to absorption of heat.



Figure 3.8: Pavement Temperature Measurements

CHAPTER FOUR: PRESENTATION, ANALYSIS AND DISCUSSION OF TEST RESULTS

4.1 INTRODUCTION

In this Chapter, the test results obtained through the testing processes were presented in Table 4.1, Table 4.2, Table 4.3 and Table 4.4 and Figure 4.1 and Figure 4.2. Test Results Analysis was carried out to give the relationship between the findings and the standard construction requirements. Explanations have been deduced to give a qualitative understanding between the findings and standard requirements, their impact on the eventual asphalt mixtures and thus leading to conclusions and recommendations.

4.2 CONSISTENCY AND CLASSIFICATION TEST RESULTS

Table 4.1 below presents the results from consistency tests that were performed with an aim of confirming the asphalt binder classification and appreciate the handling procedures during conditioning, mixing and placing. The test results in Table 4.1 overleaf presents the ductility, penetration, softening point, flash and fire, kinematic viscosity at 60 degrees, rotational viscosity, mass loss and specific gravity.

Table 4. 1 - Binder Test Results

Binder Source	Binder Type	Sample No.	Ductility BIS		Penetration at 25°C ASTM D 5-86		Softening Point ASTM D 3461-14		Flash Point ASTM D 36 °C		Specific gravity ASTM D 70-18		Kinematic Viscosity at 135°C cSt (ASTM D 2170)		Rotational Viscosity at 135°C (Pa-s) ASTM D 7741		Rolling Thin Film Oven ageing AASHTO T 240 and ASTM D2872		Density ASTM D 70-97	Fire Point
			Value (cm)	Spec (cm)	Value	Spec	Value	Spec	Value	Spec	Value	Spec	Value	Spec	Value	Spec	Mass loss (%)	Spec	g/cc	°C
MKN	PG 70-16	1	140	Min. 75	30.3	N.A	48.05	N.A	327	N.A	1.028	N.A	740.2	N.A	0.759	Max. 3.0	0.62	Max. 0.5	1.025	368
		2	140	Min. 75	30.7	N.A	45.50	N.A	346	N.A	1.016	N.A	731.7	N.A	0.743	Max. 3.0	0.71	Max. 0.5	1.016	384
		3	140	Min. 75	29.0	N.A	46.4	N.A	331	N.A	1.016	N.A	731.7	N.A	0.743	Max. 3.0	0.87	Max. 0.5	1.016	376
MBP	PG 76-10	1	140	Min. 75	19.7	N.A	55.25	N.A	288	N.A	1.018	N.A	1360	N.A	1.380	Max. 3.0	0.27	Max. 0.5	1.015	352
		2	140	Min. 75	20.3	N.A	54.60	N.A	333	N.A	1.018	N.A	1360	N.A	1.380	Max. 3.0	0.25	Max. 0.5	1.014	362
		3	140	Min. 75	22.3	N.A	54.60	N.A	333	N.A	1.018	N.A	13709	N.A	1.390	Max. 3.0	0.28	Max. 0.5	1.017	362
KTR	Pen 50/70	1	140	Min. 75	61.3	50-70	42.10	46-54	316	Min. 230	1.027	N.A	409.9	Min. 295	0.420	Max. 3.0	0.01	Max. 0.5	1.024	352
		2	140	Min. 75	62.7	50-70	41.75	46-54	315	Min. 230	1.029	N.A	442.7	Min. 295	0.454	Max. 3.0	0.01	Max. 0.5	1.026	357
		3	140	Min. 75	62.0	50-70	41.75	46-54	316	Min. 230	1.029	N.A	442.7	Min. 295	0.454	Max. 3.0	0.02	Max. 0.5	1.024	356
KNB	Pen 35/50	1	140	Min. 75	45.7	35-50	41.70	50-58	345	Min. 240	1.029	N.A	433.2	Min. 370	0.444	Max. 3.0	0.028	Max. 0.5	1.026	369
		2	140	Min. 75	46.3	35-50	40.80	50-58	345	Min. 240	1.029	N.A	446.8	Min. 370	0.460	Max. 3.0	0.025	Max. 0.5	1.029	379
		3	140	Min. 75	45.3	35-50	41.95	50-58	343	Min. 240	1.034	N.A	431.3	Min. 370	0.450	Max. 3.0	0.027	Max. 0.5	1.031	382

4.2.1 ANALYSIS AND DISCUSSION OF CONSISTENCY TEST RESULTS

PENETRATION TEST RESULTS

The test results as observed from Table 4.1 for Pen Grade 50/70 ranged between 61.3 and 62.7 and Pen Grade 35/50 ranged between 45.3 and 46.3 indicating compliance with the classification requirements for the penetration grade binders to which they infer. The penetration test results for PG 70-16 varied between 29.0 and 30.7 implying that the binder tends to Pen Grade 30/45. The values of penetration obtained for PG 76-10 varied between 19.7 and 22.3 implying that the binder tends to Pen Grade 20/30. The PG samples were stiffer than the pen grade samples implying that there is presence of increased amount of asphaltenes making the binder harder. Higher temperatures for mixing of binder with aggregates and compaction are required for the eventual mixtures where PG samples are used.

SOFTENING POINT

The softening point test results from Table 4.1 above for the Pen Grade 50/70 varied between 41.75 and 42.10°C. When compared with the provisions in ASTM D 3461-14, these values were found to fall below the acceptable range of 46-54°C. The softening values for the Pen Grade 35/50 obtained varied between 40.8 and 41.95°C. When compared with the provisions in ASTM D 3461-14, these values were found to fall below the acceptable range of 50-58°C. This therefore implied that such binders can only be used in regions where high pavement temperatures do not exceed 40°C to achieve satisfactory pavement performance. Otherwise, mixture will be prone to damage during service when the pavement temperatures exceed the binder softening points.

The softening point test results obtained for PG 70-16 varied between 45.5 and 48.05°C. When compared with the provisions in ASTM D 3461-14, for equivalent Pen Grade 30/45, these values were found to fall below the acceptable range of 52-60°C. The softening point values obtained for PG 76-10 varied between 54.60 and 55.26, giving an average of 54.8°C. When compared with the provisions in ASTM D 3461-14, for equivalent Pen Grade 20/30, these values were found to fall below the acceptable range of 55-63°C. This therefore implied that the PG binders can only be applied in regions whose high pavement temperatures do not exceed 45°C for PG 70-16 and 54°C for PG 76-10 binders to achieve satisfactory pavement performance.

FLASH AND FIRE POINTS

The flash point test results obtained for Pen Grade 50/70 (Table 4.1) varied between 315 and 316°C and for Pen Grade 35/50; varied between 342 and 345°C. These values were far above the specified minimum of 230°C. This therefore implied that the binders could be handled safely without causing any fire hazards during laying operations.

The flash point test results obtained for the PG 70-16 varied between 327 and 346 and for PG 76-10; between 288 and 333°C. When compared with equivalent Pen Grade binders, it was concluded that both binders exceeded the minimum specification of 240°C. The findings confirmed that the binders complied with all penetration grade specifications for flash points and would be safely handled especially during mixing and laying.

SPECIFIC GRAVITY

The specific gravity test results obtained for PG76-10 was 1.018 (Table 4.1). This value was within the acceptable range 0.97 to 1.02 (BSi, 2007) implying that the binder did not contain increased aromatic type mineral impurities that would bring about stripping of aggregates in the mix. Specific Gravity test results obtained for PG70-16 varied between 1.016 and 1.028, Pen Grade 50/70 varied between 1.027 and 1.029 and Pen Grade 35/50 varied between 1.029 and 1.034. This implies that the binders contained traces of increased aromatic mineral type impurities that have potential to surface stripping as binder flow is slowed down. This ultimately affects the flow of binder around aggregates.

DUCTILITY

The ductility test results obtained for penetration and performance graded bitumen samples were all greater than the minimum specified value of 75cm (Table 4.1). This therefore implied that the binders could effectively extend or elongate under local conditions and fill the voids in the mix, easily compacted to reduce air voids and adequately coat around aggregates forming a dense asphalt concrete mixture.

KINEMATIC VISCOSITY

The test results obtained for kinematic viscosity at 135⁰C for Pen Grade 50/70 varied between 409.9 and 442.7 cSt and for pen grade 35/50, varied between 431.3cSt and 446.8 cSt (Table 4.1). When compared with ASTM D 2170, the values were above the minimum 295 and 370 cSt specified respectively. The test results obtained for kinematic viscosity at 135⁰C for PG 70-16 varied between varied between 731.7 and 740.2cSt and for PG 76-10, varied between 1360 and 1369 cSt. When compared with ASTM D 2170, the values were above the minimum 400 cSt for Pen Grade 30/45 and

530 cSt for Pen Grade 20/30 respectively. The results obtained indicated that the binders complied with all penetration grade specifications for Kinematic Viscosity and thus could flow around the aggregates to form a dense mixture when compacted.

ROTATIONAL VISCOSITY

The test results obtained when binders were subjected to elevated temperatures are presented in Table 4.1 above. It was observed that PG 70-16 gave rotational viscosity test results varying from 0.743 to 0.759 Pa-s, PG 76-10 gave test results varying from 1.380 to 1.390 Pa-s, Pen Grade 35/50 gave test results varying from 0.450 to 0.454 Pa-s and Pen Grade 50/70 gave test results varying from 0.440 to 0.450 Pa-s. These test results were all within the permissible maximum limit of 3.0 Pa-s when analysed in accordance with requirements provided in ASTM D 7741. This therefore implied that the binder can effectively be pumped and mixed with aggregates and any additives to form a uniform and dense asphalt concrete mixture.

Having observed that all the binders have rotational viscosity values less than 3.0 Pa-s, it is worth noting that the binders with very low values are susceptible to lower mixing and compacting temperatures as compared to the binders with higher rotational viscosity values.

MASS CHANGE TEST

The mass change obtained when the bitumen samples were subjected to 85 minutes of drying in the oven indicated that mass loss varying between 0.62 to 0.87% for PG 70-16 was recorded (Table 4/1). When compared with AASHTO T 240 and ASTM D2872, it was found to be higher than the specified maximum of 0.5%. This implied that the binder had been oxidized and thus susceptible to early hardening bringing

about brittleness. During service, brittle binders are susceptible to surface cracking that leads to pavement deterioration.

For PG 76-10, a mass change between 0.25 and 0.28% was recorded. The mass change obtained for Pen Grade 50/70 varied between 0.10 to 0.20% and Pen Grade 35/50 mass loss obtained varied between 0.025 and 0.028%. The values obtained in these three cases indicated that the binders would resist early hardening when used in asphalt concrete mixtures. The binder would retain flexibility properties under various loading when used in the asphalt concrete mixture.

4.3 PHYSICAL PROPERTY TEST RESULTS

The physical property test results were conducted on all Binders using high temperatures and intermediate temperatures to simulate field conditions from laboratory testing procedures. The findings are presented in Table 4.2, Table 4.3, Table 4.4, Figure 4.1 and Figure 4.2 below;

Table 4. 2-Un aged High Temperature Rheometer Test Results

HIGH TEMPERATURE RANGE TEST RESULTS – UNAGED BINDER								
Source	Grade	Freq. (rad/sec)	Final Temp. °C	Confirmed Binder grade	Modulus, G* (kPa)	Phase angle (Degrees)	G*/sin δ (kPa)	Min. G*/sin δ (kPa)
MBP	PG76-10(1)	10	87.0	82	3.617	78.3	0.96	1.00
MBP	PG76-10(2)	10	87.9	82	0.926	70.0	0.99	1.00
MBP	PG76-10(3)	10	86.4	82	3.11	78.7	0.83	1.00
MKN	PG70-16(1)	10	74.6	70	0.846	81.3	0.86	1.00
MKN	PG70-16(2)	10	74.5	70	0.826	81.5	0.84	1.00
MKN	PG70-16(3)	10	75.0	70	1.770	83.1	0.89	1.00
KTR	Pen 50/70(1)	10	65.4	64	1.149	81.4	1.16	1.00
KTR	Pen 50/70(2)	10	66.3	64	0.621	79.7	0.63	1.00
KTR	Pen 50/70(3)	10	66.6	64	0.656	83.7	0.66	1.00
KNB	Pen 35/50(1)	10	70.0	64	0.987	82.5	1.00	1.00
KNB	Pen 35/50(2)	10	69.3	64	0.893	83.1	0.90	1.00
KNB	Pen 35/50(3)	10	68.7	64	0.836	83.5	0.84	1.00

Table 4.3 – Aged high Temperature Dynamic Rheometer Test Results

HIGH TEMPERATURE RANGE TEST RESULTS – RTFO – AGED BINDER								
Source	Grade	Freq. (rad/sec)	Final Temp. °C	Confirmed Binder grade	Modulus, G* (kPa)	Phase angle (Degrees)	G*/sin δ (kPa)	Min. G*/sin δ (kPa)
MBP	PG76-10(1)	10	88.1	88	2.17	75.9	2.24	2.20
MBP	PG76-10(1)	10	88.2	88	2.21	77.4	2.27	2.20
MKN	PG70-16(1)	10	77	76	1.21	76.6	1.25	2.20
KTR	Pen 50/70(1)	10	68.7	64	1.81	82.1	1.83	2.20
KTR	Pen 50/70(1)	10	68.9	64	1.85	81.5	1.87	2.20
KTR	Pen 50/70(1)	10	69.7	64	2.06	80.1	2.09	2.20
KNB	Pen 35/50(1)	10	67.8	64	1.64	84.6		2.20
KNB	Pen 35/50(1)	10	67.9	64	1.69	75.7	1.75	2.20

Most of the Binders failed to meet the High Temperature range requirements

Table 4.4 – PAV Aged Dynamic Shear Rheometer Test Results

INTERMEDIATE TEMPERATURE RANGE TEST RESULTS – PAV – AGED BINDER								
Source	Grade	Freq. (rad/sec)	Final Temp. °C	Binder grade	Modulus, G* (kPa)	Phase angle (Degrees)	G* sin δ (kPa)	Max. G* sin δ (kPa)
MBP	PG76-10(1)	10	37.4	-10	8,783	28.6	4,209	5,000
MBP	PG76-10(2)	10	34.1	-10	9,273	26.5	4,143	5,000
MKN	PG70-16(1)	10	35.2	-16	7,686	33.9	4,290	5,000
MKN	PG70-16(2)	10	33.5	-16	9,295	30.9	4,775	5,000
KTR	Pen 50/70(1)	10	35.6	-10	7,984	33.6	4,417	5,000
KNB	Pen 35/50(1)	10	29.3	-10	6,088	43.3	4,175	5,000

The intermediate temperature range test results were marginal if compared to studies in Thailand that were done when determining the super pave grading system

Variation of Complex Shear and Phase angle with Type of Binder

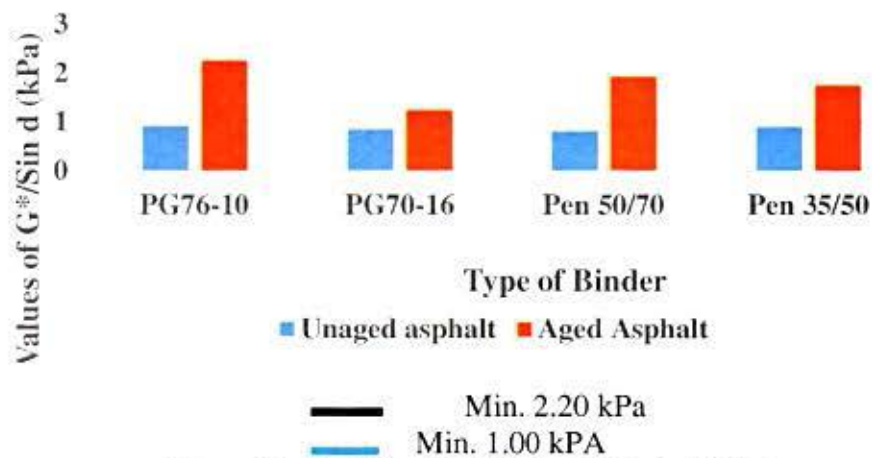


Figure 4.1: Variation of G*/Sin δ with Asphalt Binders

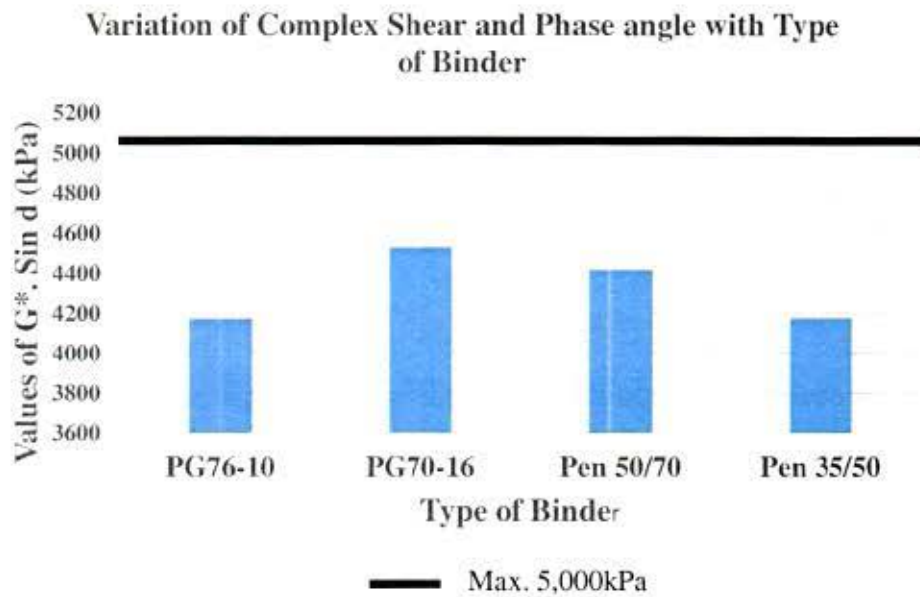


Figure 4.2: Variation of values of G^* , $\text{Sin } \delta$ with Asphalt Binders

Figures 4.1 and 4.2 present graphical overviews of performance properties of the binders.

4.3.1 ANALYSIS AND DISCUSSION OF PHYSICAL PROPERTY TEST RESULTS

The Dynamic shear tests conducted on virgin binders and residues from rolling thin film oven tests and Pressure Ageing Vessels are summarised in Table 4.2, Table 4.3 and Table 4.4 above.

DYNAMIC SHEAR RHEOMETER TEST RESULTS

Performance Grade 70-16

The ratio of complex shear modulus to phase angle obtained varied from 0.84kPa to 0.89kPa (Table 4.2). These values obtained were below the specified minimum of 1.00 kPa when analysed in accordance with AASHTO T315. This therefore implied that the binder had probably been oxidised. Such binder is susceptible to early hardening leading to formation of a brittle pavement susceptible to rutting under traffic. However, the ratio of complex shear modulus to phase angle obtained for RTFO residue binders

varied from 2.24 kPa to 2.27 kPa (Table 4.3). These values were above the minimum specified of 2.20 kPa though marginal under ambient conditions when compared to similar studies carried out in Thailand regarding development of a performance grading system for asphalt binders used in the country (Charoentham, 2012).

The values obtained when product of the complex shear modulus value and the phase angle performed on pressure aged vessel binder residues varied between 4,140 kPa to 4,410 kPa. These values were less than the maximum specified value of 5000 kPa (Table 4.4). Under ambient temperatures, these values are marginal (Charoentham, 2012).

Performance Grade 76-10

The ratio of complex shear modulus to phase angle obtained varied from 0.83kPa to 0.99kPa (Table 4.2). These values were below the specified 1.00kPa when analysed in accordance with AASHTO T315. The ratio of complex shear modulus to phase angle obtained for RTFO residue binders varied from 2.06 kPa to 2.24 kPa giving an average value of 2.16kPa (Table 4.3). These values were also below the minimum specified value of 2.20 kPa when analysed in accordance with AASHTO T315. This therefore implied that the binder had been oxidized and would suffer from short term ageing before the expected service life and cause the mixture to suffer from rutting under traffic. The values obtained when product of the complex shear modulus value and phase angle performed on pressure aged vessel binder residues varied between 4,290 kPa to 4,782 kPa (Table 4.4). These values obtained were below the maximum specified value of 5,000 kPa. However, these values were too marginal when

compared with the similar studies conducted to developing performance graded binders in Thailand (Charoentham, 2012).

Penetration Grade 35/50

The ratio of complex shear modulus to phase angle obtained varied from 0.84 kPa to 0.995 kPa. These values were less than the 1.00kPa when analysed in accordance with AASHTO T315. The ratio of complex shear modulus to phase angle obtained for RTFO residue binders varied from 1.65 kPa to 1.75 kPa. These values were below the minimum specification of 2.20 kPa for RTFO aged asphalt binders when analysed in accordance with AASHTO T315. This therefore implied that the binder would become brittle and would cause the asphalt concrete pavement to deform or crack under traffic loading.

The values obtained when product of the complex shear modulus value and phase angle performed on pressure aged vessel binder residues varied between 4,144 kPa to 4,175 kPa which were below the maximum specified of 5000 kPa. These values were found to be marginal when compared to studies carried out while developing binder performance grading for Thailand (Charoentham, 2012).

Penetration Grade 50/70

The ratio of complex shear modulus to phase angle obtained varied from 0.611 kPa to 0.660 kPa. These values were far below the 1.00kPa when analysed in accordance with AASHTO T315. This implied that the binder is susceptible to early hardening, forming a brittle binder that can make the eventual mixture liable to crack development. The ratio of complex shear modulus to phase angle obtained for RTFO residue binders

varied from 1.830 kPa to 2.06 kPa. These values were also below the minimum specification of 2.20 kPa. This therefore implied that the binder would be liable to rutting failure during ambient temperatures and traffic loading.

The values obtained when product of the complex shear modulus value and phase angle performed on pressure aged vessel binder residues varied between 4,031 kPa to 4,788 kPa which were below the maximum specified of 5000 kPa. However, these values were too marginal when compared with the standard requirements. This therefore implied that under ambient weather and in-service, fatigue damage is predicted when the binder is used in the asphalt mixture.

4.4 ZONING OF BINDER APPLICATION IN UGANDA

4.4.1 DETERMINATION OF PAVEMENT DESIGN TEMPERATURE

Historical air temperatures were obtained from the Meteorological Centre to understand the air temperature distribution in the Country. The data was obtained backwards from 14 weather zones with an assumption that no great variation occurred between individual stations within a zone for the years 2017, 2016, 2015 and 2014. The hottest seven-day maximum and the lowest one-day temperatures for each year were calculated and recorded respectively. The data for all the four years was standardised as presented in Table 4.5 below with a purpose of studying the variation of the individual readings from the average reading taken. The air temperatures that were measured in the field were not used directly in the determination of the temperature zoning because the pavement temperatures are usually higher than air temperatures. Pavement temperatures were measured between August and September 2018 with a purpose of determining the highest and lowest pavement temperatures

possible in different regions of Uganda. These pavement temperatures were correlated with air temperatures and latitudes (North or South of the Equator) of the stations to provide an equation derived from regression analysis having neglected the effect due to solar absorption, atmospheric radiation and wind speed. Latitudes were considered because they influence temperatures depending on the North or South distance from the Equator. Areas closer to the Equator receive high sunlight as compared to those quite away. Based on SHRP Superpave grading system, the highest and lowest pavement temperatures were measured at 20mm below the pavement surface. Table 4.5 below presents results showing the highest and lowest air temperatures. Figure 4.3 and Figure 4.4 provide Maximum and Minimum Air and Pavement Temperatures.

4.5 ZONING OF BINDER APPLICATION IN UGANDA

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Historical air temperatures were obtained from the Meteorological Centre to understand the air temperature distribution across the different regions in the Country. The data was obtained backwards from 14 weather zones with an assumption that no great variation occurred between individual stations within a zone for the years 2017, 2016, 2015 and 2014. The hottest seven-day maximum and the lowest one-day temperatures for each year were calculated and recorded respectively. The data for all the four years was standardised as presented in Table 4.5 below with a purpose of studying the variation of the individual readings from the average reading taken. The air temperatures that were measured in the field were not used directly in the determination of the temperature zoning because the pavement temperatures are usually higher than air temperatures. Pavement temperatures were measured between

August and September 2018 with a purpose of determining the highest and lowest pavement temperatures possible in different regions of Uganda. These pavement temperatures were correlated with air temperatures and locations (North or South of the Equator) of the stations to provide an equation derived from regression analysis having neglected the effect due to solar absorption, atmospheric radiation and wind speed. Latitudes specify distance of a station from the North or South of the Equator. Latitudes influence precipitation, prevailing winds and temperatures. The areas closest to the Equator receive the highest amount of sunlight which then impacts on the pavement temperatures. That's why I considered latitudes other than altitudes and longitudes. Based on SHRP Superpave grading system, the highest and lowest pavement temperatures were measured at 20mm below the pavement surface. Table 4.5 below presents results showing the highest and lowest air temperatures. Figure 4.3 and Figure 4.4 provide Maximum and Minimum Air and Pavement Temperatures.

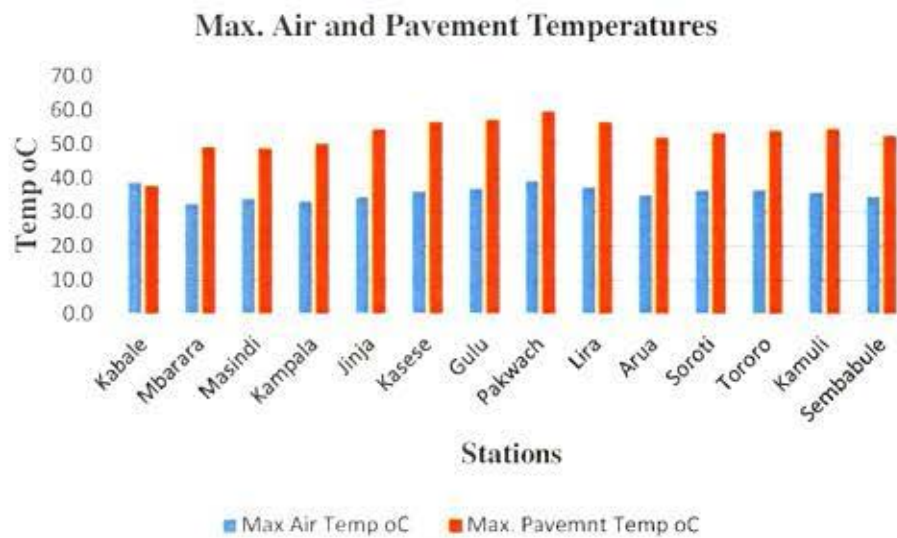


Figure 4.3: Max Air and Pavement Temperatures

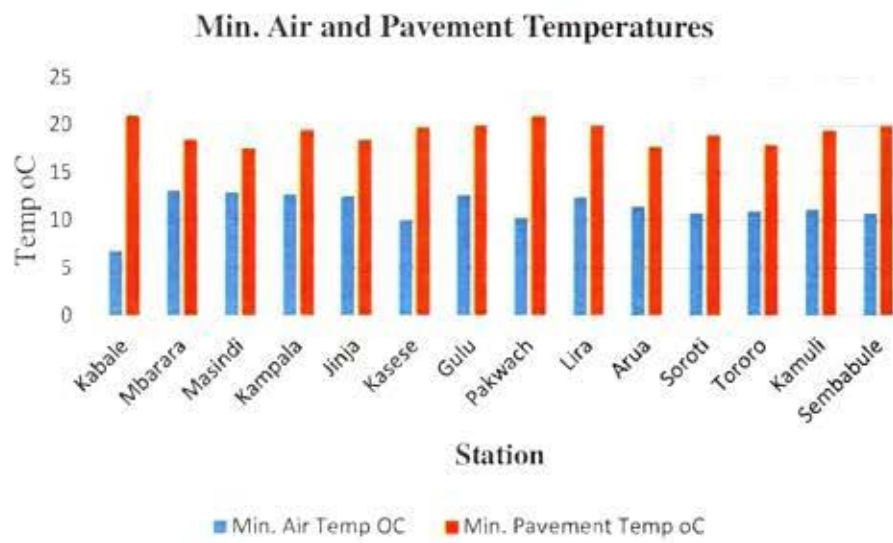


Figure 4.4: Minimum Air and Pavement Temperatures

Table 4.5 – Air and Pavement Temperatures

Air Temperatures, Pavement Temperatures measured at 20mm depth at selected Stations in Uganda																	
Station/ Year	Average High Air Temperature °C					Average Low Air Temperature °C					Latitudes	Measured High Pavement Temp °C	Measured Low Pavement Temp °C	Standard Deviation		Mean + 3*StDev	
	7 Day Max Air Temperature					1Day Minimum Air temperatures								Highest Temp °C	Lowest Temp °C	Highest °C	Lowest °C
	2014	2015	2016	2017	High	2014	2015	2016	2017	Low							
Kabale	28.4	28.8	29.5	29.5	29.5	7.8	7.5	6.8	7	6.8	-1.24857	38.0	21.0	0.54	0.46	31.13	8.17
Mbarara	32.8	33.5	33.7	33	33.7	13.8	13.4	13.3	13.2	13.2	0.6057	49.3	18.5	0.42	0.26	34.96	13.99
Masindi	34.9	34.8	36.7	35.8	36.7	14.2	13.0	13.8	14.6	13.0	1.6444	48.9	17.6	0.89	0.68	39.37	15.05
Kampala	33.0	34.3	33.2	34.0	34.3	12.8	14.2	16.0	16.4	12.8	0.31628	50.3	19.5	0.62	1.67	36.17	17.80
Jinja	33.8	36.0	34.2	37.7	37.7	13.5	12.8	13.2	12.6	12.6	0.43902	54.5	18.5	1.79	0.40	43.08	13.81
Kasese	35.1	37.4	36.7	36.5	37.4	13.5	13.5	12.0	10.1	10.1	0.1833	56.7	19.8	0.96	1.61	40.29	14.94
Gulu	36.8	38.4	37.6	37.6	38.4	16.4	12.7	14.4	12.7	12.7	2.77466	57.4	20.0	0.65	1.76	40.36	17.98
Pakwach	39.3	40.1	40.3	40.0	40.3	11.7	11.0	10.3	10.3	10.3	2.45716	59.7	21.0	0.43	0.67	41.60	12.31
Lira	39.0	36.8	37.6	36.0	39.0	14.3	13.8	12.5	12.5	12.5	2.23333	56.6	20.0	1.28	0.92	42.84	15.25
Arua	34.7	35.5	36.0	35.0	36.0	11.7	11.5	13.3	11.5	11.5	3.02013	52.1	17.8	0.57	0.87	37.71	14.12
Soroti	36.9	37.7	37.2	37.2	37.7	15.8	14.4	10.8	10.8	10.8	1.71464	53.4	19.0	0.33	2.55	38.69	18.44
Tororo	36.1	37.2	37.2	37.2	37.2	11.5	13.2	13.2	11.0	11.0	0.69299	54.0	18.0	0.55	1.14	38.85	14.43
Kamuli	36.7	37.4	36.5	37.2	37.4	11.2	12.6	13.2	11.2	11.2	0.9403	54.5	19.5	0.42	1.01	38.66	14.23
Sembabule	34.0	35.4	35.0	34.0	35.4	12.5	12.3	12.5	10.8	10.8	-0.07722	52.5	20.0	0.71	0.82	37.54	13.27

4.5.2 TRAFFIC SPEED AND LOADING EFFECT IN SELECTION OF PERFORMANCE GRADING

Selection of performance grade is greatly influenced by the speed and loading of traffic. Two cases involving traffic speed and traffic loading influence the binder selection criteria. Based on loading criteria, the Strategic Highway Research Program researchers set adjustments in the selection of high temperature binder grade which was included in the standard specification for Superpave Volumetric Mix Design AASHTO MP-2 for slow and standing load conditions to offset the effect of slower loading speed as shown in Table 4.6 below. One grade shift for high temperature design equals 6°C, such as the PG 64 instead of PG 58. It should be noted that when the high temperature grade increases by one grade, the stiffness of the binder will be approximately double. Usually, one grade shift is used for slow moving traffic and two grades shift such as PG76 instead of PG 64 is used for standing design load conditions. For extraordinary high numbers of heavy traffic loads, there is an additional shift which suggests where the design lane traffic expected to be more than 10,000,000 equivalent standard axle loads which is defined as 80KN, four-tired dual axle. Also if the designed traffic is expected to be between 10,000,000 and 30,000,000 ESALs, then you may consider selecting one high temperature binder grade higher than the selection based on climate. If the designed traffic is expected to exceed 30,000,000 ESAL, then the binder should be selected one high temperature grade higher (Kobbail, 2005). These adjustments can be used in all regions where such cases exist. The Table below presents the Superpave specifications for high temperature design grade adjustment based on traffic volume and speed conditions.

Table 4.6: High Temperature Binder Grade adjustments based on Traffic Speed and Level

Design ESALs (Million)	Adjustment to Binder PG Grade		
	Traffic Load Rate		
	Standing (Avg. speed <20km/hr)	Slow (Avg. speed 20 to 70km/hr)	Standard (Avg. speed > 70km/hr)
<0.3	-	-	-
0.3 to < 3	+2	+1	-
3 to < 10	+2	+1	-
10 to < 30	+2	+1	-
> 30	+2	+1	+1

Design ESALs are anticipated project traffic level expected on the design lane over 20 years period. Adjustments of one grade for high temperatures for the case that applies keeping the low temperature grades. Its also advisable that care shall be taken in not specifying binders stiffer than PG 82-XX. In such a case, if the design ESALs is within 10 to < 30, increase the traffic designation to > 30 ESALs.

4.5.3 DEVELOPMENT OF REGRESSION EQUATIONS

Relationship between maximum air temperatures, latitudes and maximum pavement temperatures was developed in form of a linear regression model. It was developed to formulate a relationship between high or low air temperature, latitudes and high or low pavement temperature. Equation 4.1 below shows the developed model with correlation = 0.95

$$\text{Design Pavement temp} = -21.75 + 2.13 \times \text{temp} - 0.35 \times \text{latitude} \dots \text{Eqn. 4.1}$$

Figure 1 below shows the predicted and measured maximum pavement temperature.

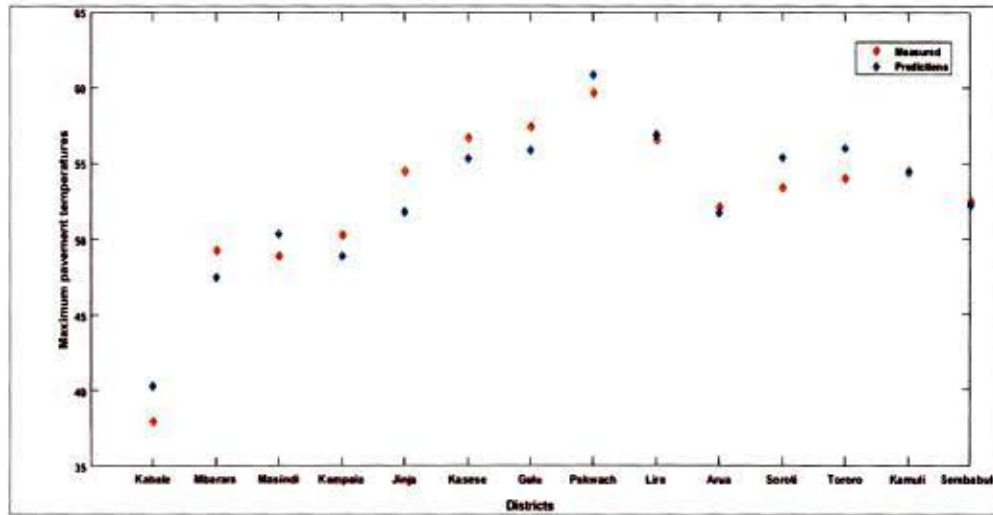


Figure 4.5: Measured and predicted maximum pavement temperatures

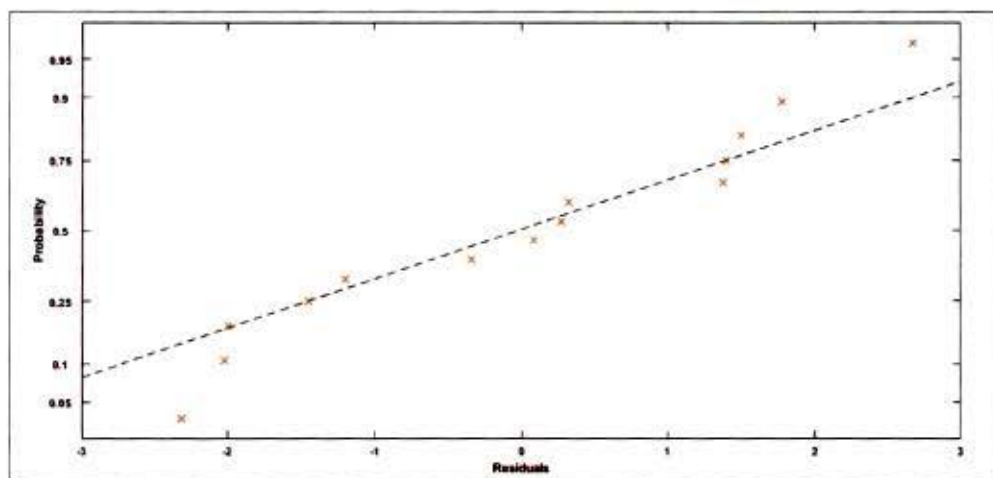


Figure 4.6: Normality plot of the residuals

Based on the Equations obtained from the developed regression model, the Design High and Low Pavement Temperatures were calculated for each station as shown in the Table 4.8 and presented in Figure 4.7 and Figure 4.8. Following the high and low temperature determinations, binder temperature zoning based on performance grading system was developed as presented in Table 4.9. It is also recommended that based on traffic characterization, speed and loading, adjustments using factors provided in Table 4.6 above from these temperature zoning can be done to meet the desired level of pavement performance.

Table 4.7: Pavement Design Temperatures

Station	High Temp. °C	Min. Temp. °C	Latitudes	High Pavement Temp. °C	Low Pavement Temp. °C
Kabale	31.13	8.17	-1.24857	45.00	-3.91
Mbarara	34.96	13.99	0.6057	52.50	7.83
Masindi	39.37	15.05	1.6444	61.53	9.73
Kampala	36.17	17.80	0.31628	55.18	16.06
Jinja	38.87	13.81	0.43902	60.89	7.51
Kasese	40.29	14.94	0.1833	64.01	10.01
Gulu	40.36	17.98	2.77466	63.24	15.57
Pakwach	41.60	12.31	2.45716	66.01	3.61
Lira	42.84	15.25	2.23333	68.71	9.96
Arua	37.71	14.12	3.02013	57.53	7.26
Soroti	38.69	18.44	1.71464	60.07	16.93
Tororo	38.85	14.43	0.69299	60.76	8.75
Kamuli	38.66	14.23	0.9403	60.27	8.24
Sembabule	37.54	13.27	-0.07722	58.23	6.53

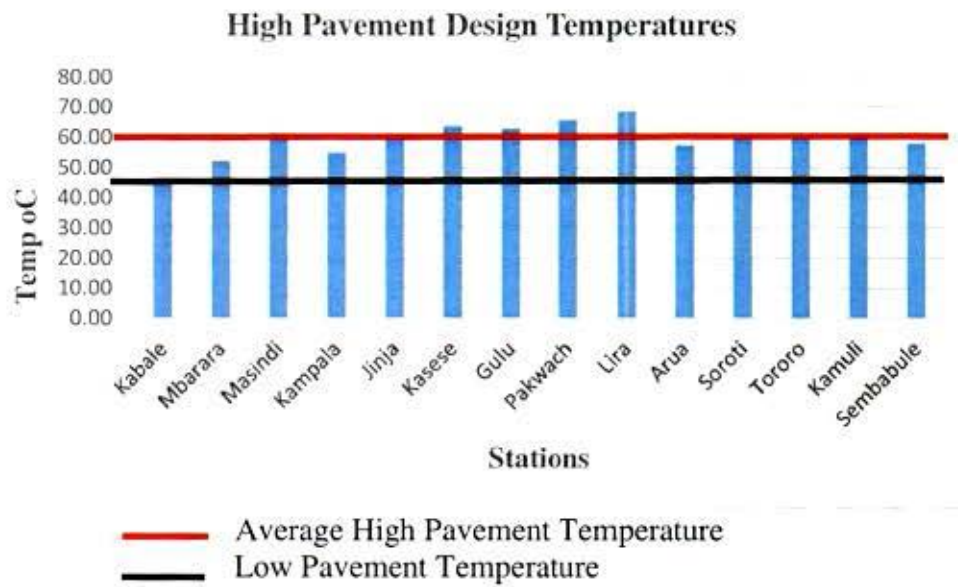


Figure 4.7: High Pavement Design Temperatures

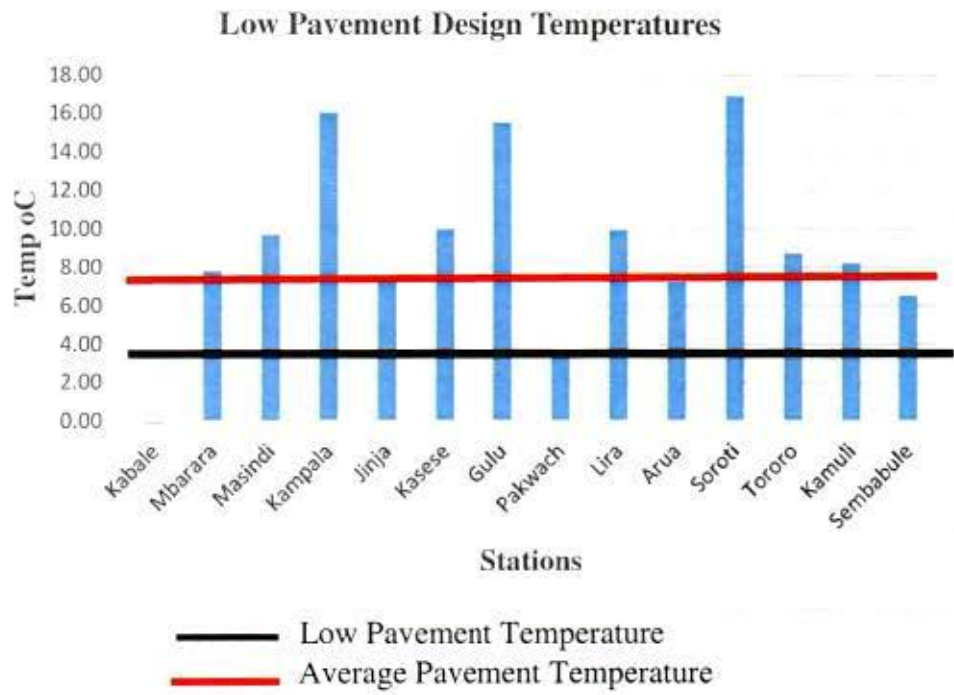


Figure 4.8: Low Pavement Design Temperatures

Table 4.8: Binder Temperature Zoning in Uganda

Station	Zoning of Binder Application				
	High Pavement Temperature °C	Low Pavement Temperature °C	Proposed Binder Grade	Binder Selection	
				High Temp Grade	Low Temp Grade
Kabale	45.00	3.91	PG 58	58	+3
Mbarara	52.50	7.83	PG 58	58	+7
Masindi	61.53	9.73	PG64	64	+9
Kampala	55.18	16.06	PG64	64	+16
Jinja	60.89	7.51	PG64	64	+7
Kasese	64.01	10.01	PG70	70	+10
Gulu	63.24	15.57	PG70	70	+15
Pakwach	66.01	3.61	PG70	70	+3
Lira	68.71	9.96	PG70	70	+9
Arua	57.53	7.26	PG64	64	+7
Soroti	60.07	16.93	PG64	64	+16
Tororo	60.76	8.75	PG64	64	+8
Kamuli	60.27	8.24	PG64	64	+8
Sembabule	58.23	6.53	PG64	64	+6

Uganda does not experience challenges with low pavement temperatures. The greatest challenge comes with high pavement temperatures. As presented in

Table 4.8 above, the high pavement temperatures vary from 45°C in Kabale to 66°C in Pakwach. We have observed from the consistency testing that the softening point values for pen grade binders ranged between 40.8 and 42°C. This therefore implies that the pen grade binders will flow under traffic causing the eventual mixture to rut especially under the wheel paths of vehicles during service conditions.

The PG 70-16 binders had softening points ranging between 45 and 48°C. This implies that the binders can selectively be used in Kabale and Kisoro regions. Their application in central region will not provide the level of pavement performance expected. Rutting as a result of flow of binders in the mix is expected especially under the wheel paths. The PG76-10 binder whose softening point ranged between 54 and 56°C best suites the regions from Mbarara and Kabale. It has indeed been noted that the binder has been applied along the Mbarara Bypass project. Binder selection criteria is based on high pavement temperatures since the risk associated with low pavement temperatures are very low.

As presented in the map of Uganda below, it is clearly observed that PG70 was zoned for generally northern Uganda and Southern Rwenzori areas – Kasese with maximum pavement temperatures obtained as 66°C in Pakwach. In Central Uganda and Arua sub region, because of the maximum pavement temperatures measured as 61°C, PG64 was zoned. In Western Uganda, covering the districts of Mbarara, Kabale, Rukungiri, Kisoro, Isingiro,

Kabarole, Ibanda, PG58 was zoned since the maximum pavement temperature obtained within these areas was 52°C .

As presented in Table 4.6 above, adjustments to these temperature zoning based on performance grading can be made depending on the traffic loading rate and design equivalent standard axial loads. The traffic loading rates include standing traffic with average speed not exceeding 20km/hr, slow traffic with average speed ranging between 20 to 70km/hr and standard rate where the average speed exceeds 70km/hr.

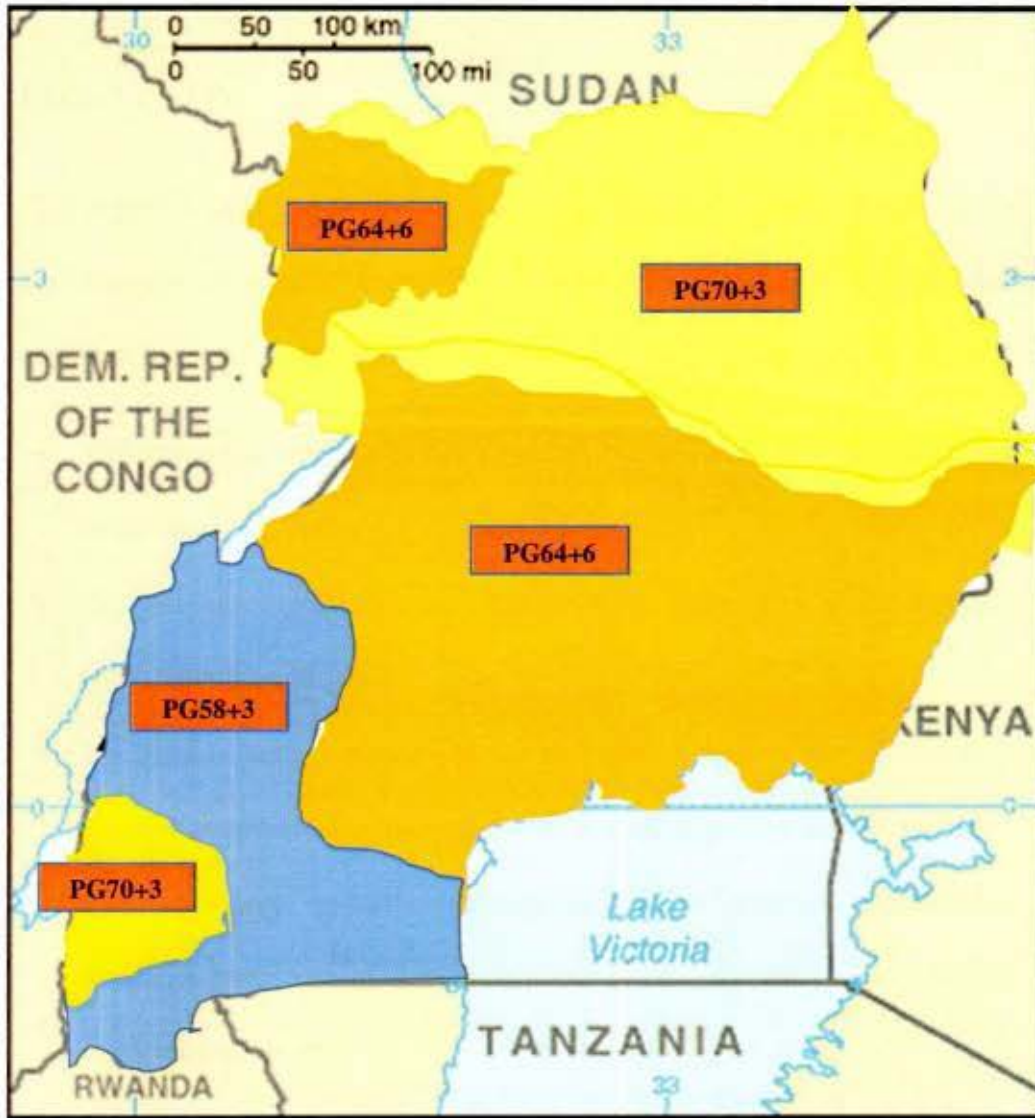


Figure: 4.9: Binder Temperature Zoning Map of Uganda

CHAPTER FIVE: CONCLUSION AND RECOMMENDATIONS

CONCLUSION

The objective of this research; to assess the impact of bitumen types on the performance of asphalt concrete pavement construction in Uganda was accomplished;

- The classification of penetration graded bitumen used on local market has been found satisfactory;
- All penetration graded binders had probable traces of increased aromatic type mineral impurities;
- The performance temperatures for all types of binders were found to be lower than specified as observed from softening point values obtained;
- The laboratory in-service simulation of performance requirements indicated that most of the binders exhibited potential to early and in-service hardening;
- Most of the binders exhibited potential to rutting and fatigue damage during high and intermediate temperatures when used in the mixture;

Based on the available data from the Meteorological Centre in the fourteen out of sixteen zones in Uganda, and the subsequent analysis based on the Strategic Highway Research Program Superpave system of analysis, I deduce the following;

- The lowest air temperatures ranged from 6.8°C at Kabale to 13.2°C in Mbarara and highest air temperatures ranged from 29.5°C in Kabale to 40.3°C in Pakwach;
- The study of the environmental conditions in Uganda showed that the highest pavement design temperature is 66°C and the minimum pavement design temperature is -4°C;
- The High Temperature zoning in Uganda has been distributed in three Temperature zones including PG70, PG64 and PG58.

RECOMMENDATION

- Historical air temperatures were obtained from each of the fourteen zones in Uganda. It is recommended that each station within a zone should be measured separately and data analysed to provide accurate air temperatures which indeed impact on the pavement temperatures;
- Measurement of Pavement Temperatures should be done between January and March where maximum air temperatures are recorded each year. This is because air temperatures impact on pavement temperatures;
- Further research by varying the test frequencies to simulate the various field speeds of traffic to enable binder master curves development;
- A robust testing regime and acceptance criteria for asphalt concrete binders should be embraced by extending the current consistency property testing that only give empirical values and precautionary handling properties to physical property testing that provide in-service performance using laboratory simulations;

- Chemical composition of the binders on the local market should be determined to analyse the amount of saturates, aromatics, resins and asphaltenes present since they impact on the colloidal structure of bitumen and hence performance of the binders.

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**APPENDIX I: SUMMARY OF TABLES OF TEST RESULTS FROM
PREVIOUS STUDIES**

Rolling Thin Film Oven Test Results (Zaniewski, 2004)

Sample Set	Pint No.	RTFO Bottle No.	Pre-Conditioned Mass (grams)	Post-Conditioned Mass (grams)	Mass Loss (grams)	Average Mass Loss (%)
1	10	Bottle 1	34.992	34.821	0.00489	0.3069
		Bottle 2	35.056	35.012	0.00126	
2	11	Bottle 1	35.399	35.387	0.00034	0.0411
		Bottle 2	35.211	35.194	0.00048	
3	12	Bottle 1	34.826	34.796	0.00086	0.1064
		Bottle 2	34.713	34.669	0.00127	
4	9	Bottle 1	34.852	34.800	0.00149	0.0933
		Bottle 2	34.785	34.772	0.00037	

Thermal Cracking Investigation (Zaniewski, 2004)

Test No.	Temp. ¹ (°C)	Load, P (mN)	Deflection, δ (mm)	Creep Stiffness, S(60) (MPa)	Max. Allow. S(60) ² (MPa)	S(60) Test Results	m-value, m(60)	Min. Allow. m(60) ¹	m(60) Test Results
11	-12.4	10.01	2.546	322.257 2	300	Failed	0.311	0.300	Passed
12	-11.2	9.89	2.564	246.392 4	300	Passed	0.329	0.300	Passed
15	-12.2	10.01	2.636	301.202 6	300	Failed	0.308	0.300	Passed
16	-12.5	10.02	2.714	302.951	300	Failed	0.301	0.300	Passed

Physical test values analysis (Zaniewski, 2004)

HIGH TEMPERATURE RANGE TEST RESULTS: RTFO-AGED BINDER								
Test Number	Frequency (Rad/sec)	Final Temp. (°C)	Strain Amplitude (Percent)	Modulus, G' (kPa)	Phase Angle, (Degrees)	G'/sin (kPa)	Minimum Allowable G'/sin (kPa)	Test Status
1	10.08	70	10.01	2.546	83.2	2.564	2.2	Passed
2	10.08	70	9.89	2.564	83.6	2.580	2.2	Passed
3	10.08	70	10.01	2.636	83.1	2.656	2.2	Passed
4	10.08	70	10.02	2.714	82.7	2.737	2.2	Passed
5	10.08	70	10.29	2.463	83.1	2.481	2.2	Passed
6	10.08	70	9.93	2.120	83.4	2.134	2.2	Failed
6b	10.08	70	9.98	2.019	84.3	2.029	2.2	Failed
7	10.08	70	10.06	2.410	83.3	2.427	2.2	Passed
8	10.08	70	10.06	2.548	83.0	2.567	2.2	Passed

INTERMEDIATE TEMPERATURE RANGE TEST RESULTS: PAV-AGED BINDER								
Test Number	Frequency (Rad/sec)	Final Temp. (°C)	Strain Amplitude (Percent)	Modulus, G' (kPa)	Phase Angle, (Degrees)	G'/sin (kPa)	Maximum Allowable G'/sin (kPa)	Test Status
9	10.08	28	1.03	3879	50.6	2999	5000	Passed
10	10.08	28	1.01	3722	50.5	2873	5000	Passed
13	10.08	28	1.01	3641	50.4	2804	5000	Passed
14	10.08	28	1.00	3226	50.4	2486	5000	Passed

HIGH TEMPERATURE RANGE TEST RESULTS: UNAGED BINDER								
Test Number	Frequency (Rad/sec)	Final Temp. (°C)	Strain Amplitude (Percent)	Modulus, G' (kPa)	Phase Angle, (Degrees)	G'/sin (kPa)	Minimum Allowable G'/sin (kPa)	Test Status
33	10.08	70	12.06	1.207	84.4	1.213	1.0	Passed
34	10.08	70	12.07	1.200	84.6	1.205	1.0	Passed
35	10.08	70	11.99	1.224	85.1	1.229	1.0	Passed
36	10.08	70	11.83	1.074	85.1	1.078	1.0	Passed

**APPENDIX II: SUMMARY OF CONSISTENCY AND PHYSICAL TEST
RESULT SUMMARIES**



SUMMARY SHEET
Bituminous Binder
PERFORMANCE GRADING SYSTEM OF ANALYSIS



PROJECT: Assessment of the impact of bitumen types on the performance of asphalt concrete pavement road construction in Uganda	Date: 29 April 2018	Date: 29 April 2018
Client: Mr. Mpele Steven Student from Kyambogo University.	Checked:	Approved:
Contract No.: 2017/2018/115		
Responsible Technician: Clement Mdeghe		

Location: Rehabilitation of Mukono - Kayunga - Njeru Road				RESULTS												Specification
Binder Type (as labelled by Client) - PG 70-16				Trial 1				Trial 2				Trial 3				AASHTO M320
Test name	CML test	Ref.	Unit	64 °C	70 °C	76 °C	82 °C	64 °C	70 °C	76 °C	82 °C	64 °C	70 °C	76 °C	82 °C	
AASHTO M320 Performance Graded Specifications																
Virgin Binder																
Pass/Fail Temp (°C) [G*/sinδ @ 10 rad/s]		ASTM D7175 / AASHTO T315	kPa		1.67	0.86			1.64	0.84				1.78	0.89	G* sinδ ≥ 1.00
Dynamic Viscosity @ 135 °C	3.8	ASTM D4402	Pa.s	0.759	0.759	0.759	0.759	0.750	0.750	0.750	0.750	0.743	0.743	0.743	0.743	3 max
Kinematic Viscosity @ 135 °C	3.8	ASTM D4402	cSt	740.2	740.2	740.2	740.2	738.3	738.3	738.3	738.3	731.7	731.7	731.7	731.7	
Relative Density	3.2	ASTM D70-97		1.028	1.028	1.028	1.028	1.019	1.019	1.019	1.019	1.021	1.021	1.021	1.021	
Density	3.2	ASTM D70-97	g/cm ³	1.025	1.025	1.025	1.025	1.016	1.016	1.016	1.016	1.016	1.016	1.016	1.016	
Softening Point	3.8	ASTM D36-70	°C	48.05	48.05	48.05	48.05	45.5	45.5	45.5	45.5	46.4	46.4	46.4	46.4	
Flash Point	3.3	ASTM D92-90	°C	327	327	327	327	346	346	346	346	331	331	331	331	
Fire point	3.3	ASTM D92-90	°C	368	368	368	368	384	384	384	384	376	376	376	376	
Penetration	3.5	ASTM D5-86	1/10mm	30.3	30.3	30.3	30.3	30.7	30.7	30.7	30.7	29.0	29.0	29.0	29.0	
Ductility	3.7	ASTM D113-86	mm	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	
After RTFOT - Ageing																
Mass change (% m/m)	3.4	ASTM D2872	%	-0.62	-0.62	-0.62	-0.62	-0.71	-0.71	-0.71	-0.71	-0.87	-0.87	-0.87	-0.87	1.00 max
Pass/Fail Temp (°C) [G*/sinδ @ 10 rad/s]		ASTM D7175 / AASHTO T315	kPa		5.20	2.49	1.25	10.07	4.57	2.18		9.11	4.23	2.06		G* sinδ ≥ 2.20
After PAV - Ageing			°C	28	31	34	37	28	31	34	37	28	31	34	37	
Pass/Fail Temp (°C) [G*/sinδ @ 10 rad/s]		ASTM D7175 / AASHTO T315	kPa	8965	8915	5471	4290	7544	6042	4775		5532	4552			G* sinδ ≥ 5000
Creep stiffness		ASTM D6648 / AASHTO T313	MPa	NT	NT											S ≤ 300
m-value				NT	NT											m ≥ 0.300

Comments: 1) The results relate only to the samples actually supplied to CML.
2) Retained material will be disposed after 3 months.
3) NT=Not Tested, low temperature cracking not a concern



SUMMARY SHEET
Bituminous Binder
PERFORMANCE GRADING SYSTEM OF ANALYSIS



PROJECT: Assessment of the impact of bitumen types on the performance of asphalt concrete pavement road construction in Uganda Client: Mr. Mpaala Steven Student from Kyambogo University Contract No.: 2017/2018/115 Responsible Technician: Clement and Asha	Date: 29 April 2018 Checked:	Date: 29 April 2018 Approved:	
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Location: Construction of Mbarara Town Bypass Project				RESULTS															Specification	
Binder Type (as labelled by Client) - PG 76-10:				Trial 1					Trial 2					Trial 3					AASHTO M320	
Test name	CML test	Ref.	Unit	64 °C	70 °C	76 °C	82 °C	88 °C	64 °C	70 °C	76 °C	82 °C	88 °C	64 °C	70 °C	76 °C	82 °C	88 °C		
AASHTO M320 Performance Graded Specifications																				
Virgin Binder																				
Pass/Fail Temp (°C) [G' min @ 10 rad/s]		ASTM D7175 / AASHTO T315	kPa		7.67	3.09	1.82	0.98			3.39	1.77	0.99			3.17	1.81	0.83	G' min ≥ 1.00	
Dynamic Viscosity @ 135 °C	3.8	ASTM D4402	Pa.s	1.390	1.390	1.390	1.390	1.390	1.390	1.390	1.390	1.390	1.390	1.378	1.378	1.378	1.378	1.378	1.378	3 min
Kinematic Viscosity @ 135 °C	3.8	ASTM D4402	cSt	1369.9	1369.9	1369.9	1369.9	1369.9	1369.9	1369.9	1369.9	1369.9	1369.9	1369.9	1369.9	1369.9	1369.9	1369.9	1369.9	
Relative Density	3.2	ASTM D70-87		1.018	1.018	1.018	1.018	1.018	1.018	1.018	1.018	1.018	1.018	1.021	1.021	1.021	1.021	1.021	1.021	
Density	3.2	ASTM D70-87	g/cm ³	1.019	1.019	1.019	1.019	1.019	1.014	1.014	1.014	1.014	1.014	1.017	1.017	1.017	1.017	1.017	1.017	
Softening Point	3.8	ASTM D36-70	°C	55.25	55.25	55.25	55.25	55.25	54.6	54.6	54.6	54.6	54.6	53.7	53.7	53.7	53.7	53.7	53.7	
Flash Point	3.3	ASTM D92-80	°C	328	328	328	328	328	333	333	333	333	333	324	324	324	324	324	324	
Fire point	3.3	ASTM D92-80	°C	359	359	359	359	359	362	362	362	362	362	362	362	362	362	362	362	
Penetration	3.5	ASTM D5-86	1/100mm	19.7	19.7	19.7	19.7	19.7	20.3	20.3	20.3	20.3	20.3	22.3	22.3	22.3	22.3	22.3	22.3	
Ductility	3.7	ASTM D113-88	mm	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	1400	
After RTFOT - Ageing																				
Mass change (% min)	3.4	ASTM D2872	%	0.27	0.27	0.27	0.27	0.27	0.25	0.25	0.25	0.25	0.25	0.28	0.28	0.28	0.28	0.28	0.28	1.00 max
Pass/Fail Temp (°C) [G' min @ 10 rad/s]		ASTM D7175 / AASHTO T315	kPa			9.54	4.83	2.24			9.82	4.99	2.27			10.10	4.92	2.45	G' min ≥ 2.20	
After PAV - Ageing																				
Pass/Fail Temp (°C) [G' min @ 10 rad/s]		ASTM D7175 / AASHTO T315	kPa	8558	7325	6154	5108	4209	7144	6032	5022	4143		7186	6065	5189	4401		G' min ≥ 5000	
Creep stiffness m-value		ASTM D6948 / AASHTO T313	MPa	NT	NT															S ≤ 300 m ≥ 0.300

Comments: 1) The results relate only to the samples actually supplied to CML.
 2) Retained material will be disposed after 3 months.
 3) NT=Not Tested, low temperature cracking not a concern



SUMMARY SHEET Bituminous Binder

PERFORMANCE GRADING SYSTEM OF ANALYSIS

CML

PROJECT: Assessment of the impact of bitumen types on the performance of asphalt concrete pavement road construction in Uganda				Date: 29 April 2018		Date: 29 April 2018					
Client: Mr. Mpaala Steven Student from Kyambogo University.				Checked		Approved					
Contract No. 2017/2018/115											
Responsible Technicians: Clemet and Asha											
Location: Upgrading of Kamuli Township Roads											Specification
Binder Type (as labelled by Client) - PEN 50/70				Trial 1		Trial 2		Trial 3			AASHTO M320
Test name	CML test	Ref.	Unit	64 °C	70 °C	64 °C	70 °C	88 °C	64 °C	70 °C	
AASHTO M320 Performance Graded Specifications											
Virgin Binder											
Pass/Fail Temp (°C) [G*/sin θ @ 10 rad/s]		ASTM D7175 / AASHTO T315	kPa	1.16	0.61	1.30	0.63	3.08	1.36	0.66	G*/sin θ \geq 1.00
Dynamic Viscosity @ 135 °C	3.8	ASTM D4402	Pa.s	0.420	0.420	0.454	0.454	0.455	0.455	0.455	3 max
Kinematic Viscosity @ 135 °C	3.8	ASTM D4402	cSt	409.9	409.9	442.7	442.7	444	444	444	295 Min.
Relative Density	3.2	ASTM D70-97		1.027	1.027	1.029	1.029	1.028	1.028	1.028	
Density	3.2	ASTM D70-97	g/cm ³	1.024	1.024	1.026	1.026	1.024	1.024	1.024	
Softening Point	3.6	ASTM D36-70	°C	42.1	42.1	41.7	41.7	42.0	42.0	42.0	46-54
Flash Point	3.3	ASTM D92-90	°C	316	316	315	315	317	317	317	230 Min
Fire point	3.3	ASTM D92-90	°C	352	352	357	357	356	356	356	
Penetration	3.5	ASTM D5-86	1/10mm	61.3	61.3	62.7	62.7	63	63	63	50-70
Ductility	3.7	ASTM D113-86	mm	1400	1400	1400	1400	1400	1400	1400	-
After RTFOT - Ageing											
Mass change (% m/m)	3.4	ASTM D2872	%	-0.01	-0.01	-0.01	-0.01	-0.02	-0.02	-0.02	0.5 max
Pass/Fail Temp (°C) [G*/sin θ @ 10 rad/s]		ASTM D7175 / AASHTO T315	kPa	4.10	1.83	4.15	1.87		4.54	2.09	G*/sin θ \geq 2.20
After PAV - Ageing											
Pass/Fail Temp (°C) [G*.sin θ @ 10 rad/s]		ASTM D7175 / AASHTO T315	kPa		6977		6484		5768	4788	G*.sin θ \leq 5000
Creep stiffness		ASTM D6648 / AASHTO T313	MPa	NT	NT						S \leq 300
m-value				NT	NT						m \geq 0.300

Comments: 1) The results relate only to the samples actually supplied to CML.
 2) Retained material will be disposed after 3 months.
 3) NT=Not Tested, low temperature cracking not a concern



SUMMARY SHEET
Bituminous Binder
PERFORMANCE GRADING SYSTEM OF ANALYSIS

PROJECT: Assessment of the impact of bitumen types on the performance of asphalt concrete pavement road construction in Uganda	Date: 29 April 2018	Date: 29 April 2018
Client: Mr. Mpeala Steven - Student from Kyambogo University.	Checked	Approved
Responsible Technician: Clement Ndagwa		

Location: Capacity Improvement of the Kampala Northern Bypass Project				Specification									
Binder Type (as labelled by Client) - PEN35/50				Trial 1			Trial 2			Trial 3			AASHTO M320
Test name	CML test	Ref.	Unit	58 °C	64 °C	70 °C	58 °C	64 °C	70 °C	58 °C	64 °C	70 °C	
AASHTO M320 Performance Graded Specifications													
Virgin Binder													
Pass/Fail Temp (°C) [G* _{sinδ} @ 10 rad/s]		ASTM D7175 / AASHTO T315	kPa	4.53	2.05	1.00	4.40	1.92	0.90	3.80	1.74	0.84	G* _{sinδ} ≥ 1.00
Dynamic Viscosity @ 135 °C	3.8	ASTM D4402	Pa.s	0.444	0.444	0.444	0.400	0.400	0.400	0.445	0.445	0.445	3 max
Kinematic Viscosity @ 135 °C	3.8	ASTM D4402	cSt	433.2	433.2	433.2	446.8	446.8	446.8	431.3	431.3	431.3	370 Min.
Relative Density	3.2	ASTM D70-97		1.029	1.029	1.029	1.032	1.032	1.032	1.034	1.034	1.034	
Density	3.2	ASTM D70-97	g/cm ³	1.028	1.028	1.028	1.029	1.029	1.029	1.031	1.031	1.031	
Softening Point	3.6	ASTM D36-70	°C	41.7	41.7	41.7	40.8	40.8	40.8	42.0	42.0	42.0	50-58
Flash Point	3.3	ASTM D92-90	°C	317	317	317	345	345	345	343	343	343	230 min
Fire point	3.3	ASTM D92-90	°C	309	309	309	379	379	379	382	382	382	
Penetration	3.5	ASTM D5-86	1/10mm	45.7	45.7	45.7	46.3	46.3	46.3	45.3	45.3	45.3	35-50
Ductility	3.7	ASTM D113-86	mm	1400	1400	1400	1400	1400	1400	1400	1400	1400	-
After RTPOT - Ageing													
Mass change (% m/m)	3.4	ASTM D2872	%	0.028	0.028	0.028	0.025	0.025	0.025	0.027	0.027	0.027	0.5 max
Pass/Fail Temp (°C) [G* _{sinδ} @ 10 rad/s]		ASTM D7175 / AASHTO T315	kPa		3.54	1.85		3.54	1.85		3.78	1.78	G* _{sinδ} ≥ 2.20
After PAV - Ageing													
Pass/Fail Temp (°C) [G* _{sinδ} @ 10 rad/s]		ASTM D7175 / AASHTO T315	kPa	7408	5647	4175	7408	5647	4175	7434	5590	4144	G* _{sinδ} ≤ 5000
Creep stiffness m-value		ASTM D6848 / AASHTO T313	MPa		NT	NT							S ≤ 300 m ≥ 0.300

Comments: 1) The results relate only to the samples actually supplied to CML.
2) Retained material will be disposed after 3 months.
3) NT=Not Tested, low temperature cracking not a concern