RUBBER TYRE AND PLASTIC WASTE USE IN ASPHALT CONCRETE PAVEMENT

A dissertation submitted in the fulfilment of the requirements for the degree Magister Technologiae: Civil Engineering

in the Faculty of Engineering and Technology

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DECLARATION

I, Onyango Felix Odhiambo, hereby declare that this dissertation is my own original work, except where specific acknowledgement is made in the form of a reference. This dissertation has not been previously submitted to any institution for similar or any degree award.

Signature: _____________on this _________day of __________2015

ACKNOWLEDGEMENTS

I would like to express my sincere gratitude to my supervisor Dr Salim Wanjala and co-supervisors Prof. Maurice Ndege and Prof. Leonard Masu for their patience, support, inspiration and guidance throughout my research work.

Special thanks to the Civil engineering department staff and technicians. I would like to thank Mr. Ezra Nzama for his facilitation of my work by timely procurement of the equipment required for my research work.

My sincere gratitude goes to the Road Lab (PTY) staff, Mr Charel Van Biljon and Mr. Nico Herbst for allowing me to use their facilities for this research work and their technical input into my research work.

My sincere gratitude goes to Mr. Abdul Hajee of Afrisam (SA) who supplied me with aggregate materials for my research work at no cost. My gratitude also goes to Vidar rubber products and Welpac who supplied me with the crumb rubber and plastics for this research.

DEDICATION

To my late dad Hezekiah Onyango Jimbo.

ABSTRACT

Modified asphalt concrete is one of the important construction materials for flexible pavements. The addition of polymers and natural hydrocarbon modifiers to enhance the properties of asphalt concrete over a wide temperature range in paving applications has been the common practice. Currently these modified asphalt mixtures are relatively expensive. However, recycled polymers and rubber added to asphalt have also shown similar results in improving the performance of road pavements.

In this study, an attempt has been made to use low density polyethylene (LDPE) obtained from plastic waste and crumb rubber obtained from worn out vehicle tyres. The aim was to optimise the proportions of LDPE in the bitumen binder using the ‘wet process’ and crumb rubber aggregates in the hot mix asphalt (HMA) using the ‘dry process’. The Marshall method of bituminous mix design was carried out for varying percentages of LDPE namely 2%, 4%, 6%, 8% and 10% by weight of bitumen binder and 1%, 2%, 3%, 4% and 5% crumb rubber by volume of the mineral aggregates. The characteristics of bitumen modified with LDPE were evaluated. The modified asphalt mix was also evaluated to determine the different mix characteristics.

The results from laboratory studies in terms of the rheological properties of the LDPE modified bitumen binder showed an increase in viscosity, softening point and stiffness of the binder. The optimum Marshall stability values for HMA mixtures containing 2% crumb rubber tyre and 4% LDPE were found to be 30% higher than the conventional asphalt concrete mix. The wheel tracking test done at 50^{o}C was 9.81mm rut depth showing a good rutting resistance of the optimized mixture compared to the conventional asphalt mixes. The Modified Lottman test gave a Tensile Strength Ratio value of 0.979 which indicates a low degree of moisture susceptibility of the modified asphalt mix. The above results showed improved properties of the asphalt mixture. The economic assessment done using the present worth of costs indicated a reduction in maintenance cost due to the extended service life of the modified asphalt pavement.

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List of Abbreviations

4R: Reduce Reuse Recycle Recover

AASHTO: American Association of State Highway and Transportation Officials

AC: Asphalt Concrete

ACV: Aggregate Crushing Value

AIV: Aggregate Impact Value

ASTM: American Society of Testing and Materials

BRD: Bulk Relative Density

COLTO: Committee of Land Transport Officials

COTO: Committee of Transport Officials

CRM: Crumb Rubber Modified

CSIR: Council for Scientific and Industrial Research

FACT: Fines Aggregate Crushing Test

FHWA: Federal Highways Administration

GW: General Wastes

HDPE: High Density Polyethylene

HMA: Hot Mix Asphalt

ITS: Indirect Tensile Strength

LAA: Los Angeles Abrasion

LDPE: Low Density Polyethylene

MMLS: Model Mobile Load Simulator

MTRD: Maximum Theoretical Relative Density

NCHRP: National Cooperative Highway Research Program

PET: Polyethylene Terephthalate

PMB: Polymer Modified Binder

PWOC: Present Worth of Costs

RTFOT: Rolling Thin Film Oven Test

SABITA: Southern African Bitumen Association

SANRAL: South African National Roads Agency Limited

SANS: South African National Standards

SATRPC: South African Tyre Recycling Process Company

TG: Technical Guidelines

TMH: Technical Methods for Highways

TRB: Transportation Research Board

TRH: Technical Recommendations for Highways

TSR: Tensile Strength Ratio

VFB: Volume Filled with Binder

VMA: Voids in Mineral Aggregate

1 INTRODUCTION

1.1 Background of the Study

Roads are expensive to construct, operate and maintain. Construction of 1 km of road can cost as much as R25 million (Van Rooyen & Hilda, 2010, p. 7). Between 2006 and 2020, the South African government has planned to invest more than R63 billion for the development and maintenance of the country's road network. The majority of South Africa’s national road network is older than 25 years. This exceeds the design life of the roads and although well maintained, the national road network is experiencing unprecedented high traffic volumes (Van Rooyen & Hilda, 2010, p. 7). One consequence of growing freight volumes on the roads is that the overloading of freight trucks is causing not only rapid deterioration in road conditions, but also diminishes road capacity and safety (Development Bank of Southern Africa, 2012, p. 53). Construction and maintenance of the South Africa’s extensive road networks consume large amounts of natural non-renewable resources in form of virgin raw materials (CIB and UNEP-IETC, 2002, p. 24). The principal roads in South Africa are surfaced with hot-mixed asphalt (HMA) i.e. a mixture of coarse and fine aggregates bound together with asphalt cement used in a “hot mix” pavement composition (Department for International Development, 2002, p. 3; Liebenberg, et al., 2004).

In tropical and sub-tropical countries the performance of HMA has often been disappointing under severe climate conditions, with road surfaces sometimes failing within a few months of construction and rarely lasting the design life (Department for International Development, 2002, p. 3). The main distress contributing to asphalt pavement failures in South Africa are fatigue cracking, permanent deformation and thermal cracking (Mturi & O'Connell, 2011, p. 2). Such distresses are influenced by the rheological properties of the binder in asphalt pavement. Fatigue cracking and thermal cracking are associated with lower temperatures and aged binder of high viscosity, while permanent deformation is associated with higher temperatures where its rheology approaches Newtonian behaviour (Mturi & O'Connell, 2011, p. 2; Al-

Hadidy, et al., 2009, p. 1462). An ideal binder should, therefore display adequate elastic behaviour at higher temperatures to resist permanent deformation with a reduced age of ageing and lower viscosity at lower temperatures to prevent fatigue and thermal cracking.

Some improvements in asphalt properties have been gained by selecting the proper crude or tailoring the refinery process used to make asphalt (Kalantar, et al., 2012, p. 56). Unfortunately there are only a limited number of actions that can be taken to control the refining process to make improved asphalts. Air blowing makes asphalt binder harder. Fluxing agents or diluent oils are sometimes used to soften the asphalt. Another method used to significantly improve asphalt quality is the addition of polymers. To date two types of modifications have been proposed. These are crumb rubber modified (CRM) and polymer modified bitumen (PMB). Many studies have shown that modifying the asphalt with synthetic and natural polymers increases the viscosity and resistance to moisture damage and reduces the susceptibility to temperature and tendency to flow (Cao, 2007, p. 1011; Gawandea, et al., 2012, p. 1; Flynn, 1993, p. 41).

1.2 Motivation

Environmental sustainable development on a global level requires durable road infrastructures that consume a minimum of natural resources and energy during construction and allow reduced traffic induced atmospheric and acoustic pollution during its service life (Partl, et al., 2010, p. 283). Environmental concerns are becoming increasingly important and influence the techniques available; for example, encouraging the recycling of existing materials. The demand for higher pavement quality from users is ever increasing as the cost resulting from pavement failures continues to escalate. Hence, there is a strong desire to have a better performing asphalt mixture from highway agencies (Chui-Te & Li-Cheng, 2007, p. 1027).

Many approaches have been considered in recent years for treating and improving the conventional asphalts, such as the introduction of additives in order to improve their properties (Fontes, et al., 2010, p. 1193). Modified bitumen provides the technology to produce a bituminous binder with improved viscoelastic properties which remain in balance over a wider temperature range and loading conditions (Technical Guide 1, 2007, p. 5). Modified bitumen is expected to improve the life of surfacing up to 100% depending on the degree of modification and type of additives and modification process used (Rokade, 2012, p. 105).

1.3 Significance of the Study

Modification of asphalt concrete could lead to improved performance of the asphalt pavement and extension of its service life. This research is an attempt to improve the mechanical properties of asphalt concrete by using plastic waste and crumb rubber. The recycling of wastes in construction helps to save and sustain the natural resources base. It also decreases the pollution of the environment and it also helps to save and recycle energy in the production process (Hassani, et al., 2005). This will significantly reduce the cost of construction and maintenance thus saving government funds on infrastructure development which can be directed to other critical sectors of the economy. The landfill disposal sites for plastic and crumb rubber waste would be freed up for development.

1.4 Problem Statement

Solid waste management is one of the major environmental concerns all over the world including in South Africa (Department of Environmental Affairs, 2012). Plastics and rubber tyres form the bulk of the municipal solid wastes that are non-biodegradable. Approximately 11 million tyres or 275,000 tonnes of pneumatic waste tyres are generated in South Africa (SA) annually (SATRPC, 2011, p. 12). As a commodity a waste tyre has no economic value and therefore less than 4% of waste tyres are recycled at present. Tyre dealers and large end-users have virtually no satisfactory disposal mechanism for waste tyres leading to many waste tyres being illegally dumped, re-used or burnt.

The apparent annual consumption of plastic materials in South Africa was 1,250,000 tonnes in 2009 (Plastics SA, 2013). A major portion of plastics produced each year is used to make disposable items of packaging or other short lived products that are discarded within a year of manufacture. Once used, plastic materials are thrown out. They do not undergo bio-decomposition therefore they end up either in landfills or incinerators (Rokade, 2012, p. 105). Both processes are not eco-friendly as they pollute the land and the air. With the increase in demand for plastic and lack of suitable/appropriate sources of separation, collection and recycling facilities, plastic waste problem is expected to be even worse in future (Kajal, et al., 2007, p. 938).

Asphalt concrete pavements have a short life cycle, failing mainly due to temperature changes, traffic loading and ageing (Alonso, et al., 2010, p. 2592). The problem with asphalt is the tendency to become brittle at low temperatures and soft at high temperatures (Jain, et al., 2011, p. 233). These cause fatigue cracking and rutting which are the major forms of asphalt pavement failures (Al-Hadidy, et al., 2009, p. 1456). Numerous modifiers currently used to improve the properties of road surfaces are virgin materials (Kalantar, et al., 2012, p. 55). However the manufacture and application of these virgin modifiers is costly (Adhikari, et al., 2000, p. 943; Kalantar, et al., 2012, p. 56). The use of secondary (recycled), instead of primary (virgin), materials helps ease landfill pressures and reducing demand of extraction (Huang, et al., 2007, p. 59). Crumb rubber and recycled plastics have been used separately as binder modifiers and to replace a portion of the mineral aggregates in asphalt concrete mixtures (Al-Hadidy, et al., 2009, p. 1456; Huang, et al., 2007, p. 67). Polymer modified binders (PMB) contain small percentages of polymers to improve their physical properties. The principal source of raw material for producing crumb rubber modified (CRM) asphalt is scrap tyres (Liu, et al., 2009, p. 2701). However, concerns over inferior road performance and additional costs of construction have hindered the widespread use of such secondary binders and aggregates in such applications. For these reasons amongst others, research into improving the design and performance of asphalt road surfaces continue to be undertaken (Department for International Development, 2002, p. 3).

1.5 Study Objectives

1.5.1 Main objectives

The main objective of this research was to optimally utilise polymers from plastic waste and crumb rubber from waste tyres in making asphalt concrete.

1.5.2 Specific objectives

The specific objectives of this study were to:

i) Determine the optimum mix proportions of crumb rubber and plastic waste in modified asphalt concrete.

ii) Characterize the physical properties of the optimized crumb rubber -plastic waste modified asphalt concrete against conventional asphalt concrete, rubber asphaltic concrete and plastic asphaltic concrete.

iii) Evaluate the compatibility of crumb rubber and plastic waste in the asphalt mix.

iv) Evaluate the cost effectiveness of using plastic crumb rubber modified asphalt concrete against conventional asphalt concrete mixes.

1.6 Research Outline

This dissertation presents the methodology, results, analysis and discussion from an extensive laboratory investigation. The dissertation is divided into five chapters. Chapter one is an overview of the purpose and significance of this study. Chapter Two is a detailed literature review on the past and current use of modified asphalt concrete mixtures. Chapter Three outlines the test procedures carried out on bitumen and aggregates as well as asphalt concrete mixtures. Chapter Four gives the findings of the various tests carried out in the study as well as their analysis and discussions. Chapter Five gives the conclusions derived from the test analysis and recommendations from the study.

2 LITERATURE REVIEW

2.1 Introduction

Governments around the world rank infrastructure policy among their greatest concerns. The modernisation of infrastructure is seen as being critical to future economic competitiveness and crucial to accommodating expanding populations in urbanising environments (Urban Land Institute and Earnest and Young, 2011, p. 5). South Africa is no exception. Infrastructure lies at the heart of government's stimulatory fiscal package and is a pivotal component of the New Growth Path (Department of Economic Development, 2010, p. 1), accounting for just less than 8% of GDP in the 2012/13 fiscal year. South Africa's total road network consists of approximately 154 000 km of paved roads and 454 000 km of gravel roads, which are proclaimed as national, provincial or municipal roads (Development Bank of Southern Africa, 2012, p. 47). Un-proclaimed roads account for 140 000 km, or 33% of the total gravel network of 593 000 km. The un-proclaimed roads are predominantly in rural areas and have not been officially recorded in road inventories, hence no authority is responsible for their maintenance and upgrading. The total road network is in the order of 750 000 km in length (Development Bank of Southern Africa, 2012, p. 47).

According to the European Asphalt Pavement Association (European Asphalt Pavement Association, 2012, p. 2), South Africa's total production of hot and warm mix asphalt in the year 2011 and 2012 is 5.7 million tonnes. This translates to over 5.4 million metric tonnes of aggregates used in the production of the asphalt concrete mix in the respective years. A record 445,000 tonnes of bitumen was consumed in South Africa during 2010, showing an increase of some 10% on 2009 (Southern African Bitumen Association (Sabita), 2011, p. 3). Globally, crude oil sources are being depleted due to continuous extraction of these non-renewable sources. The high global demand for crude oil as an energy source is also driving the cost of this pavement material significantly higher. The cost of petroleum oil has increased ten-fold in the past 10 years. This has translated into high cost of bitumen binder used for construction.

2.2 Asphalt Concrete Road

A road pavement is composed of a system of overlaid strata of chosen processed materials that is positioned on the in-situ soil termed the sub-grade (Rogers, 2003, p. 192). Its basic requirement is the provision of a uniform skid resistant running surface with adequate performance life and requiring minimum maintenance. The structural function of the pavement is to support vehicle wheel loading applied and distribute them to the sub-grade underneath. Hence the pavement structure must ensure an economic and effective way that guarantees adequate dispersion of the incident wheel stresses so that the layers are not overstressed during the pavement’s design life. The pavement design must be economical and guarantee adequate dispersion of the incident wheel stresses to the subsequent layers. The layers within a typical flexible highway pavement are shown in Figure 2.1 below.

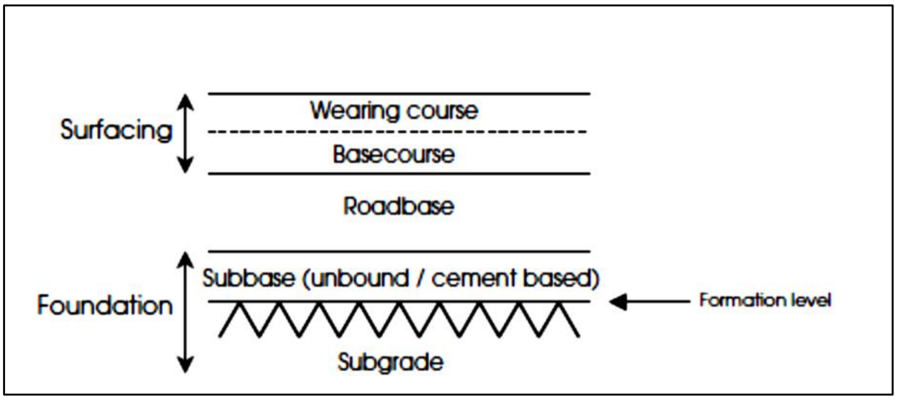

Figure 2.1: Layers within a typical flexible highway pavement *(Rogers, 2003, p. 193)*

The surfacing provides good riding quality by combining a regular surface with adequate skidding resistance as well as protecting the underlying layers from water infiltration. The durability, texture and flexibility of the surfacing are therefore key factors. Asphalt concrete is the most commonly used material in pavement surfacing because of its superior service performance in providing driving comfort, stability, durability and water resistance (Yilmaz, et al., 2011, p. 4279).

Asphalt concrete is a material that has been specially prepared for use in flexible road pavements by controlling its quality and consistency. Asphalt concrete is made of aggregate or solid materials such as sand, gravel, or recycled concrete bound together by bitumen. Bitumen is a thermoplastic material i.e. it gradually melts when heated. It is characterized by its stiffness, consistency, or ability to flow at different temperatures (Al-Hadidy, et al., 2009, p. 1456). It has an extremely diverse molecular structure depending on the crude oil source. Aggregate is the granular material used in asphalt concrete mixtures which make up 90-95 percent of the mixture weight and provides most of the load bearing characteristics of the mix. Asphalt concrete is ordinarily used in a 'hot mix' pavement composition that contains coarse and fine aggregates which are blended at specified temperatures, applied to the roadbed and compacted with rollers to produce a smooth driving surface (Al-Hadidy, et al., 2009, p. 1456).

2.3 Environmental Waste and Legislation

2.3.1 Rubber tyre waste

A tyre is a composite of complex elastomer formulations, fibres, textiles and steel cord (Siddique & Naik, 2004, p. 563). Tyres are made of plies of reinforcing cords extending transversely from bead to bead; on top is a belt which is located below the thread. Table 2.1 lists typical types of materials used to manufacture tyres.

Table 2.1: Rubber Tyre Composition *(Siddique & Naik, 2004, p. 564)*

1. Synthetic Rubber 2. Natural Rubber 3. Sulphur and sulphur compounds 4. Phenolic resin	
5. Oil	(i) Aromatic (ii) Naphthenic (iii) Paraffinic
6. Fabric	(i) Polyester (ii) Nylon etc.
7. Petroleum waxes	
8. Pigments	(i) Zinc oxide (ii) Titanium dioxide etc.
9. Carbon black 10. Fatty acids 11. Inert materials 12. Steel wires	

Large stockpiles of waste tyres exist and these discarded tyres are causing environmental, fire and health risks on mines, in townships and surrounding areas. The hazards of waste tyres include air pollution associated with open burning of tyres (particulates, odour, visual impacts and other harmful contaminants such as polycyclic aromatic hydrocarbon, dioxin, furans and oxides of nitrogen), aesthetic pollution caused by waste tyre stockpiles and illegal waste tyre collecting and other impacts such as alterations in hydrological regimes when gullies and water courses become waste sites (Rokade, 2012, p. 106). The disposal of tyres in a landfill is a problem, as they are hard to compact and tend to rise up through the waste (Department of Water Affairs and Forestry, 2005, p. A3-2). A further challenge is the management and control of casings that are worn down to the point where they are no longer suitable for re-treading. Figure 2.2 shows worn tyres being transported for re-treading and resale.

Figure 2.2: Worn out tyres transported on a major freeway

Innovative solutions to meet the challenge of tyre disposal problem have long been in development, and the promising options are;

(i) Use of tire rubber in asphalt mixes,

(ii) Thermal incineration of worn-out tires for the production of electricity or steam, and

(iii) Re-use of ground tire rubber in number of plastic and rubber products.

According to Saddique and Naik (2004, p.4) scrap tyres can be managed as a whole tyre, as slit tyre, as shredded/chipped tyre, crumb rubber or as ground rubber.

a) **Shredded/chipped tires-** Tire shreds or chips involve primary, secondary or both shredding operations. The size of the tire shreds produced in the primary shredding process can vary from as large as 300 to 460 mm (12–18 in.) long by 100–230 mm (4–9 in.) wide, down to as small as 100–150 mm (4–6 in.) in length, depending on the manufacturer's shredder model and the condition of the cutting edges. Production of tire chips, normally sized from 76 (3 in.) to 13 mm (0.5 in.), requires both primary and secondary shredding to achieve adequate volume (quantity) reduction.

b) **Crumb Rubber -** Crumb rubber consists of particles ranging in size from 4.75 mm (No. 4 Sieve) to less than 0.075 mm (No. 200 Sieve). Generally, these methods are used to convert scrap tires into crumb rubber. These methods are (i) cracker mill process, (ii) granular process, and (iii) micro mill process. The cracker mill process tears apart or reduces the size of tire rubber by passing the material between rotating corrugated steel drums. By this process irregularly shaped torn particles having large surface areas are produced. The size of these particles varies from 5 mm to 0.5 mm (No. 4 to No. 40 Sieve), and are commonly known as ground crumb rubber. Granular process shears apart the rubber with revolving steel plates, producing granulated crumb rubber particles, ranging in size from 9.5 mm (3/8 inch) to 0.5 mm (No. 40 Sieve).

c) **Ground Rubber**- Ground rubber for commercial applications may be nominally sized as large as 19 mm (or 3/4 in.) to as small as 0.15 mm (No. 100 sieve). It depends upon the type of size reduction equipment and intended applications. The processed used tires in ground rubber applications are typically subjected to two stages of magnetic separation and to screening. Various size fractions of rubber are recovered (Heitzman & Michael , 1992, p. 23). Some processes/markets term 30 mesh rubber as crumb rubber.

The most significant opportunity will be the recovery of all the products from the used tyres and to identify markets for re-use. Once this happens, used tyres will have an intrinsic value and thus will be items for both formal and informal collection (REDISA, 2009). Although the use of asphalt rubber is attractive from the viewpoint of environmental preservation, it is not widely used because its performance and cost effectiveness have not been conclusively proven (Hossain , et al., 1995, p. 189).

2.3.2 Plastic wastes

Plastics are cheap, lightweight and durable materials which can be readily moulded into various products with a wide range of applications (Hopewell, et al., 2009, p. 2115). The total global production of plastics reached 230 million tonnes in 2009 up from 1.5 million tonnes in 1950 (PlasticEurope, 2010, p. 9). Polyolefin is the collective description for plastics types that include polyethylene, namely: low-density polyethylene (LDPE), linear low-density polyethylene (LLDPE), high-density polyethylene (HDPE) and polypropylene (PP). Together they account for more than 57% (775 thousand tonnes) of South Africa's total consumption of 1.34 million tonnes of plastics each year (Plastics SA, 2013, p. 1). According to the Plastics SA Plastics Recycling Survey for 2011, South Africa currently recycles 18.9% of virgin plastics produced which has increased from 17.8% in 2009. The amount of plastic recycled in South Africa increased from 241,853 tonnes in 2010 to 245,696 tonnes in 2011 (Plastics SA, 2013, p. 4). The most common recyclable 'household' plastics are Polyethylene terephthalate (PET) and High Density Polyethylene (HDPE). PET is used in the manufacture of bottles for soft drinks and fruit juices bottles, pillow and sleeping bag filling, and textile fibres. Figure 2.3 shows the market applications for low density polyethylene by proportions.

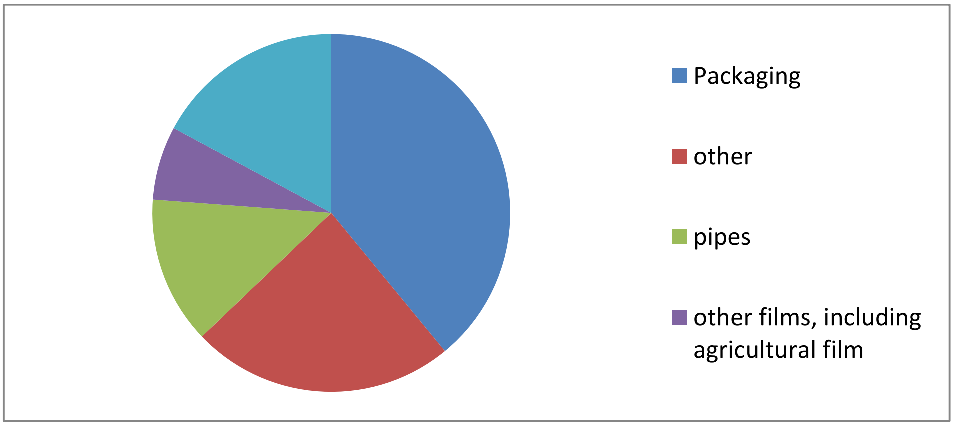

Figure 2.3: Market applications for Low Density Polyethylene- LDPE *(Plastics SA, 2013, p. 4)*

Around 4 percent of world oil and gas production, a non-renewable resource, is used as feedstock for plastics and a further 3-4 percent is expended to provide energy for their manufacture (Hopewell, et al., 2009, p. 2115). A major difficulty with the recycling of plastics is the need to separate different types of plastic, as these have different melting points (Department of Water Affairs and Forestry, 2005, p. A3-1). Mixed plastic will not form a uniform and stable material instead they tend to phase-separate like oil and water and set in different layers. Although packaging plastic accounts for the most recycled in form of HDPE, LDPE is among the main types that have the lowest recycling rates. As landfilling, the traditional source of waste disposal, has become more expensive because of closures and stricter operating requirements, recycling is becoming an alternative for waste disposal (Halstead, 1994, p. 4). Thus the use of LDPE plastics in asphalt pavements may provide an important outlet for such materials. The binding property of LDPE in its molten state has helped in finding out a method of safe disposal of waste polyethylene by using them in road making.

2.3.3 The South African climate

Air temperatures influence the road surface temperatures which in turn dictate the type and grade of binder to be used. Air and road temperatures are strongly affected by altitude. The average annual air temperatures in South Africa vary from less than 13˚C in the central mountain areas to 17˚C in the broader central and southern coastal areas and to 22˚C or more in the western, northern and eastern parts of the country (TRH 3, 2007, p. 3). In these areas, maximum air temperatures may exceed 35˚C (40˚C in the northern and eastern parts of South Africa) as shown in Figure 2.4. On the Highveld and in the mountainous regions, minimum temperatures may be as low as -8˚C (temperatures below -10˚C have been recorded). Minimum temperatures mainly occur during June and July (TRH 3, 2007, p. 3) as shown in Figure 2.5.

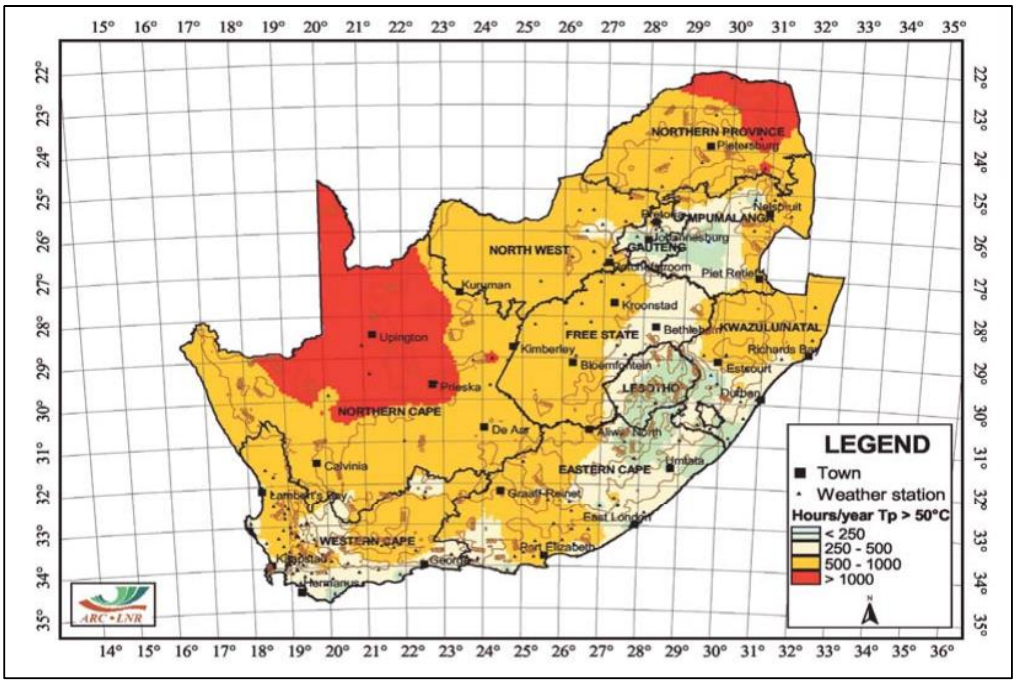

Figure 2.4: Estimated hours per year with asphalt surface temperatures above 50 ˚C *(The South African National Roads Agency, 2001, p. 2-10)*

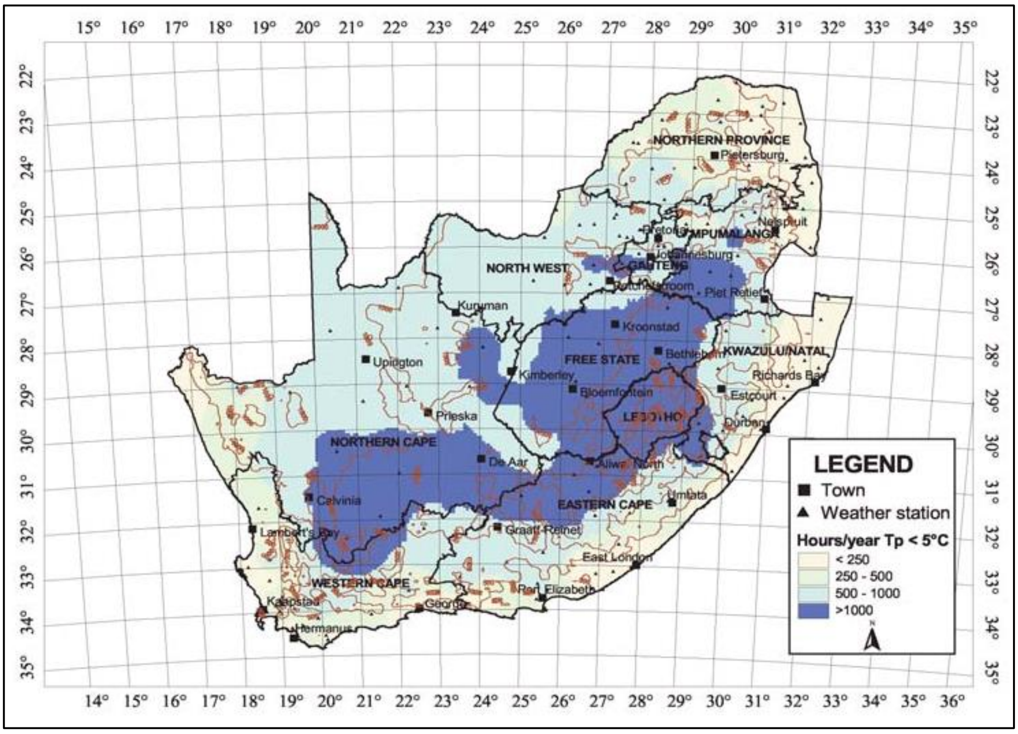

Figure 2.5: Estimated hours per year with asphalt surface temperatures below 5 ˚C *(The South African National Roads Agency, 2001, p. 2-10)*

The physical properties of asphalt binder vary tremendously with temperature (Lu & Isacsson, 2001, p. 82). At high temperatures, asphalt binder is a fluid with a consistency similar to that of motor oil. At room temperature most asphalt binders will have the consistency of putty or soft rubber. At sub-zero temperatures, asphalt binder can become very brittle, further asphalt stored in a freezer shatter like glass when dropped on a hard surface (NCHRP, 2011, p. 4). The purpose of modifying the asphalt is to extend its plastic phase. The viscosity range of asphalt will vary depending on the type, concentration and modifier used (Technical Guide 1, 2007, p. 5). Modified bitumen provides the technology to produce a bituminous binder with improved viscoelastic properties which remain in balance over a wider temperature range and loading conditions.

2.3.4 Legislation

Currently, despite alternatives in place, waste management services favour landfill sites because of the lack of incentives or punitive measures to act otherwise (Department of Environmental Affairs, 2010). Plastic wastes and rubber tyre wastes constitute a serious environmental problem as they accumulate rapidly and are not easily disposed of.

South Africa, though a signatory to Basel Convention, is not managing waste tyres as a hazardous waste. According to Department of Environmental Affairs, National Waste Information Baseline report (Department of Environmental Affairs, 2012, p. 18) plastics and tyres are classified as general wastes categories GW51 and GW54 respectively. Hence there is a need to research into better ways of disposal of these particular wastes (Hossain , et al., 1995, p. 189). The following regulations have been put in place by the Department of Environmental Affairs under the Environment Conservation Act, 1989 (Act No. 73 of 1989) to arrest or address the above waste generation:

(a) Waste Tyre Regulations, 2008, which regulate the management of waste tyres by providing for the regulatory mechanisms.

(b) Regulations under section 24(d) of the ECA – plastic carrier bags and plastic flat bags, 2003, which regulate the manufacture, trade and commercial distribution of domestically produced and imported plastic carrier bags and plastic flat bags.

(c) Regulations regarding waste disposal sites, 1994, which regulate the establishment and operation of landfill sites in the Republic of South Africa.

Integrated waste management requires the implementation of a hierarchical approach to waste management, i.e. a sequential application of waste prevention/minimisation, recycling and re-use, treatment, and ultimately disposal (Department of Environmental Affairs, 2005). This thinking is the basis of the 4Rs strategy in waste management parlance- in the order of decreasing environmental desirability -reduce, reuse, recycle (materials) and recover (energy), with landfill as the least desirable management strategy (Hopewell, et al., 2009, p. 2116). Hence, the 4R strategy is an integral activity in the way waste management will be implemented in the future in South Africa as illustrated in Figure 2.6.

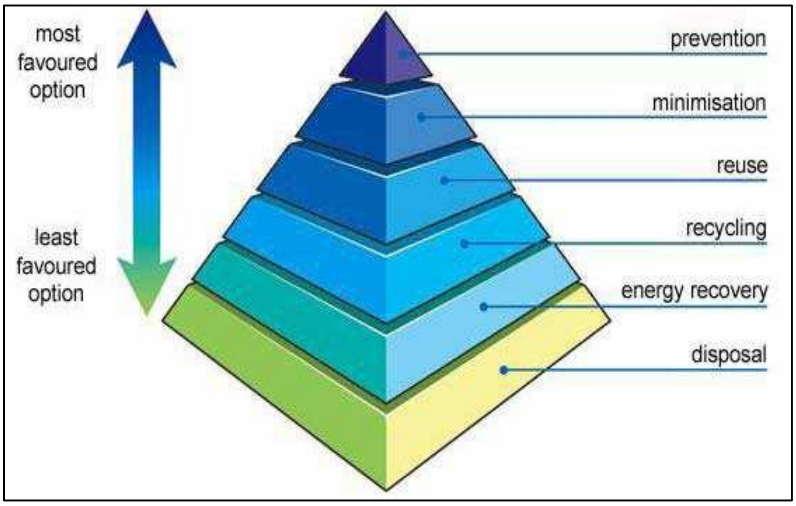

Figure 2.6: The Waste Management Hierarchy

The policy and strategy vision for these preventive and proactive waste management steps are that the rate of increase of waste disposed to landfill sites will be slowed down and informal salvaging at landfills will decrease. Natural resources (renewable and non-renewable) will be better conserved, landfill air-space will be more effectively utilised, and pollution and environmental degradation will be reduced. In addition, recycling has the potential for job creation, by promoting entrepreneurs to establish community collection systems and recycling centres.

2.4 Hot Mix Asphalt

Hot-mix asphalt has been used in South Africa since the 1920s (SABITA, 2005, p. 8). The term hot mix asphalt (HMA) is generally used to describe a variety of mixtures of aggregate, bitumen and mineral filler that are produced at an elevated temperature in an asphalt plant (SABITA, 2008, p. 1). Small amounts of additives and admixtures could be added to many HMA mixtures to enhance their performance or workability. These additives include fibres, crumb rubber, and anti-strip additives (NCHRP, 2011, p. 4). The types of HMA most frequently used in tropical countries are manufactured in an asphalt plant by hot-mixing appropriate proportions of the following materials:

2.4.1 Aggregates

There are a number of formal definitions of aggregate. In terms of road building, aggregate is defined as hard material which is generally derived from the crushing of solid rock or boulders (SANRAL, 2013, pp. 3-12).

- ***Course aggregates*** material have particles larger than 4.75 mm.
- ***Fine aggregates*** may be in form of sand, gravel, crushed stone or a blend of these having particles less than 4.75 mm and larger than 0.075 mm.
- ***Filler*** is mineral material less than 0.075 mm in size and may include stone dust, fly-ash, hydrated lime, Portland cement or a combination of these. Filler acts as an extender to the binder in an asphalt mix. It acts as a void-filling material to give a dense durable mix as well as stiffens the mix to improve resistance to plastic deformation.

Aggregates should be free of organic matter, clay lumps and soil which make it susceptible to binder stripping.

2.4.2 *Penetration grade bitumen*

Penetration grade bitumen is classified by its penetration, and is commonly supplied in the following grades; 40/50 pen, 60/70 pen, 80/100 pen and 150/200 pen (SANRAL, 2013, p. 15). Typically, the selection of penetration grade bitumen is made on the basis of climate, traffic volumes and speed, and aggregate shape. Higher values of penetration indicate softer consistency. Table 2.2 gives the general guide for the selection of binder type (SANRAL, 2013, p. 4-18).

Table 2.2: General guide for the selection of binder type *(The South African National Roads Agency, 2001, p. 2-19)*

Binder Type	Uses and Characteristics
40/50 pen bitumen	Mixes for high traffic applications, where increased stiffness is required. Typically not suitable for situations where support conditions are not of a high standard, or cold regions. Generally only used for thick layers and asphalt bases. Standard specifications apply.
60/70 pen bitumen	Typical for asphalt surfacings with light to medium traffic. Used for typical asphalt applications in most climatic zones. Standard specifications apply.
80/100 pen bitumen	Mixes for low traffic applications, where decreased stiffness is required. Typically not suitable for thick layers on a stiff support, or hot regions, unless stabilised (e.g. with fibres). Standard specifications apply.
Modified Binders & special binders	Used for heavy traffic applications or where special mix requirements exist (e.g. highly flexible or rut resistant mixes).

HMA is most commonly divided into three different generic types of mixes, i.e. continuous-graded, open-graded, and gap-graded – primarily according to the proportions of various aggregates used in the mix or their particle size distribution (gradation). Representative gradations of the main type are shown in Figure 2.7. Pavement designers specify these different mixture types to satisfy different pavement performance and application demands.

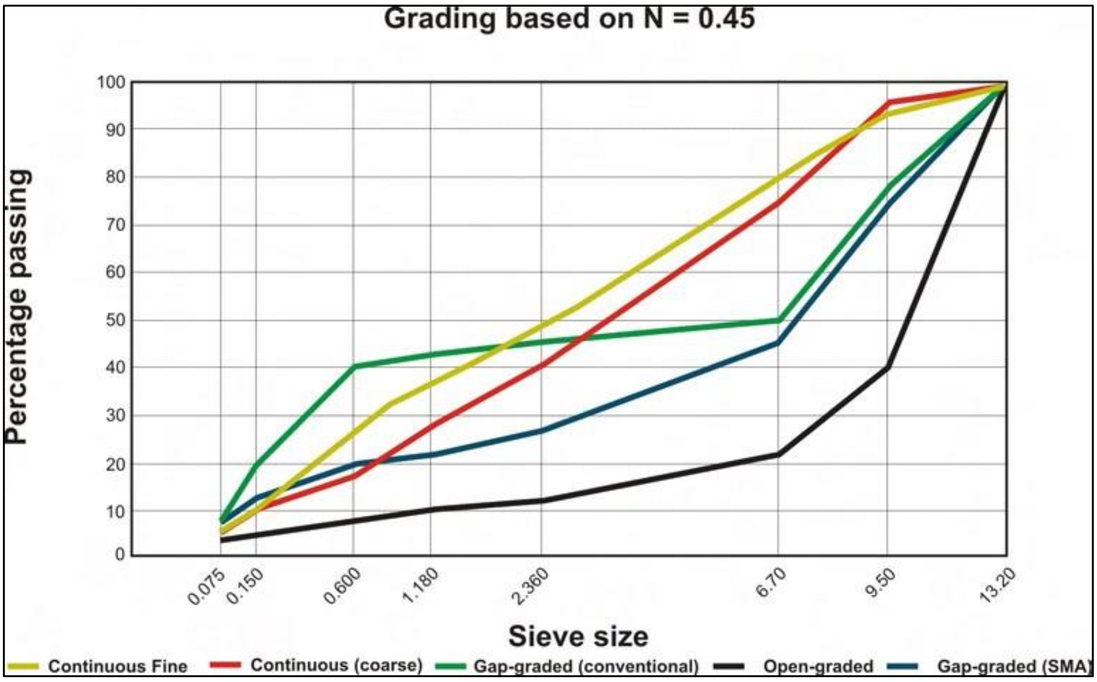

Figure 2.7: Representative continuous-graded, open-graded and gap-graded mixes of 13.2 mm maximum aggregate size *(SABITA, 2008, p. 1)*

The aim of specified grading envelopes e.g. continuous-graded, open-graded or gap graded is to ensure mixes generally comply with their overall intended function. However, these envelopes have often proved to be too restrictive because grading envelopes in themselves have a limited capacity to define aggregate packing. Consequently sound engineering practices are employed to determine the best relative proportions of the components. Such a process may result in the grading curves that fall beyond the limits of some envelopes in general use, while ensuring optimal mix design and the attainment of specified performance characteristics (SABITA, 2008, p. 2).

2.4.3 Manufacture of hot mix asphalt

Operations common to asphalt plants are drying and heating of aggregate, proportioning the components (aggregate, filler and binder) and mixing them. The mixing plant types can be divided into four categories; batch type, continuous mix type, drums mix type and modified type to allow for recycling.

In a batch plant aggregates are drawn from storage or stockpiles in controlled amounts in a cold feed unit and passed through a rotary dryer where they are dried and heated. The heated aggregates then pass over a screening unit which separates the material into different sized fractions and deposits them into bins for hot storage. The aggregates and mineral filler (when used) are then withdrawn from the bins in controlled amounts into a pug mill, where they are combined with binder and thoroughly mixed in a batch. The mixing temperatures for 60/70 bitumen penetration grade is 140 -170˚C (Department for International Development, 2002, p. 24). The mix is either stored in special bins or loaded into trucks and hauled to the paving site. Where aggregate gradations are consistent and the cold feed calibration system is in good order, the screening unit may be bypassed. In this case the mix composition is controlled directly from the cold feed, as for a continuous mixer. A simplified operational diagram of a typical continuous mixing plant is shown in Figure 2.8.

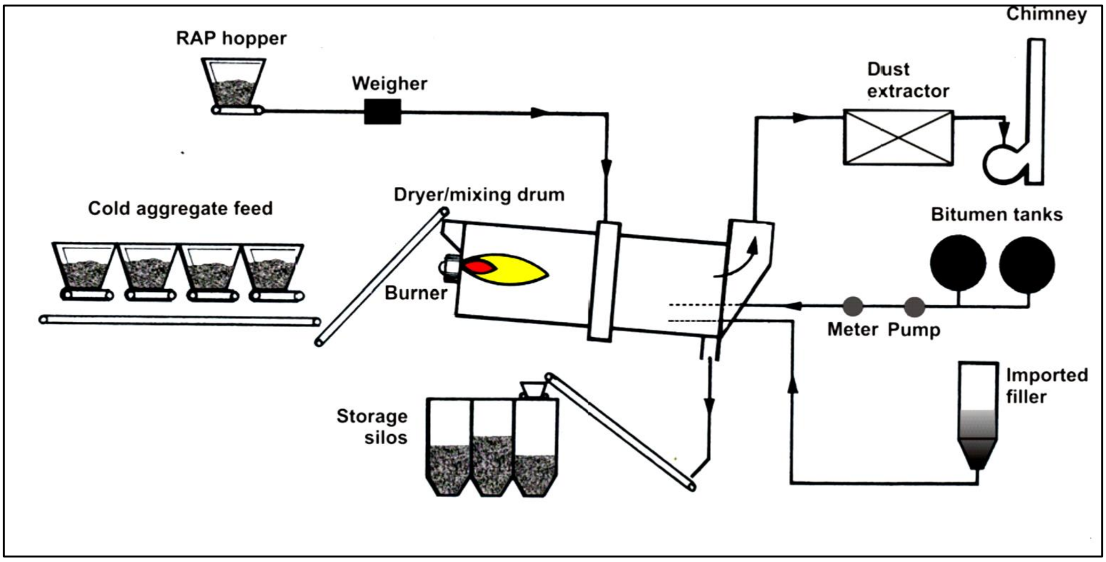

Figure 2.8: Drum mixer *(SABITA, 2008, p. 9)*

Conditions in the pug mill are such that the binder will harden excessively if the aggregate temperature is too high. The large mass of stone holds heat which could rapidly overheat the thin film of binder coating it; the action of the paddles in the pug mill allows air to come in contact with these thin films and hardening through oxidation takes place. This, in turn, reduces the flexibility and durability of the asphalt, shortening its effective life. Mixing should therefore take place at as low a temperature and as short a cycle as will provide complete coating of the aggregate particles.

Optimal mix cycles are between 45 – 50 seconds, of which the wet mixing times are 30 – 35 seconds to ensure adequate coating of the aggregate (SABITA, 2008, p. 10). The mixing times may have to be increased if additives such as bitumen modifiers or cellulose fibres are added. Too long a mixing cycle can result in excessive binder ageing. The modifiers used in this research tend to increase the viscosity and therefore may reduce workability. It is necessary to ensure that the binders are workable enough so that the mineral aggregates are properly coated during the production. To achieve the required workability the mixing and compaction temperatures are usually increased. This can result in increased production cost, loss of volatiles, increased oxidative ageing and possible degradation of the modifier (Bahia & Perdomo, 1996, p. 1192). The waste materials in the production process results to emissions/fumes/odour problems because HMA is produced at high temperatures. However very tight emission and environmental controls are already in place on HMA production facilities therefore there is no need for change in production equipment.

2.5 Pavement Deterioration

2.5.1 Introduction

The process of deformation of bituminous surfacing is accelerated by increase of pavement temperature, reduction in stiffness of the mix and increase in traffic loads (Jain, et al., 2011, p. 233). One of the initiatives aimed at increasing the design life of heavy-trafficked road sections is the high-modulus asphalt that combines superior permanent deformation resistance with high structural strength and good endurance. A higher performance pavement requires bitumen that is less susceptible to high temperature rutting or low temperature cracking (Chen, et al., 2003, p. 594). To a large extent, the stiffness and strength of mixtures determine the performance of asphalt pavements.

Structural failures in a flexible pavement are of two types, namely; surface cracking and rutting. Cracking is due to fatigue caused by repeated application of load in the bound layer generated by traffic. Rutting is developed due to accumulation of pavement deformation in various layers along the wheel path (Al-Hadidy, et al., 2009, p. 1462). The critical stresses and strains developed in bituminous pavement layers is illustrated in Figure 2.9. The process of deformation of bituminous surfacing is accelerated by fluctuation in pavement temperature, reduction in stiffness of mix and increase in traffic loads (Jain, et al., 2011, p. 233).

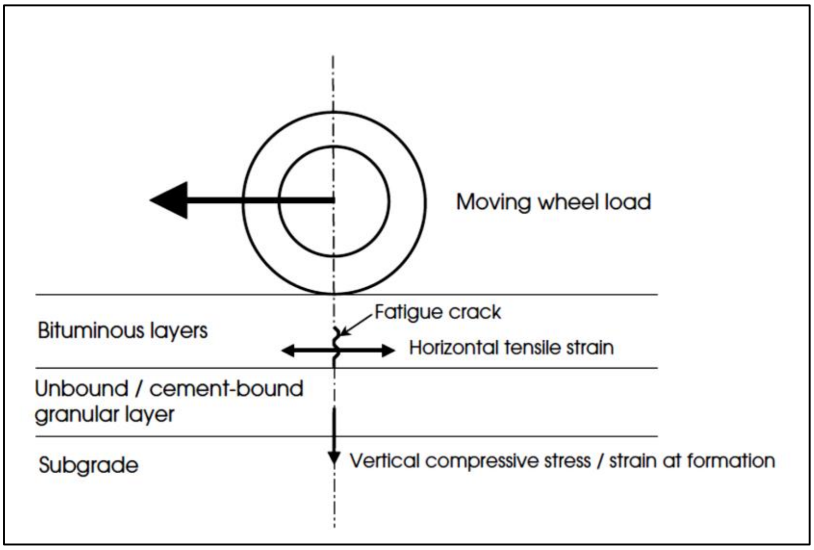

Figure 2.9: Critical Stresses/strains in a bituminous highway pavement slab *(Rogers, 2003, p. 232)*

2.5.2 Rutting

Rutting, often referred to as permanent deformation, is a common form of distress in flexible pavements. When truck tyres move across an asphalt concrete pavement, the pavement deflects a very small amount. These deflections range from much less than a tenth of a millimetre in cold weather when the pavement and subgrade are very stiff to a millimetre or more in warm weather when the pavement surface is hot and very soft. After the truck tyre passes over a given spot in the pavement, the pavement tends to spring back to its original position. Often, however, the pavement surface will not completely recover. Instead, there will be a very small amount of permanent deformation in the wheel path. After many wheel loads have passed over the pavement perhaps only a few thousand in a poorly constructed pavement, to 10 million or more for one properly designed and constructed for heavy traffic loads this rutting can become significant. Severely rutted pavements can have ruts 20 mm or more in depth. Rutting is a serious problem because the ruts contribute to a rough riding surface and can fill with water during rain or snow events, which can then cause vehicles traveling on the road to hydroplane and lose control (NCHRP, 2011, p. 7). Rut depths of about 10 mm or more are usually considered excessive and a significant safety hazard. Figure 2.10 shows a picture of rutting in an HMA pavement.

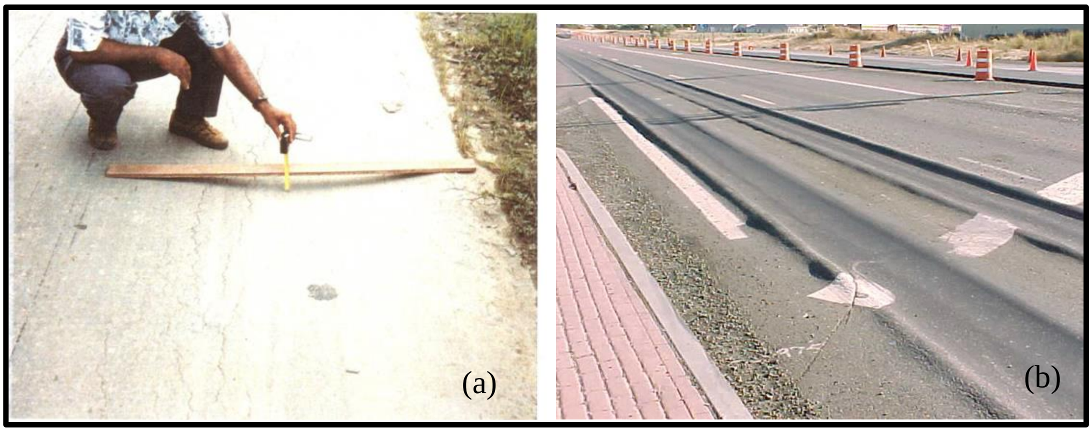

Figure 2.10: (a) Wide subgrade rutting, and (b) Narrow wheel path rutting *(SANRAL, 2013, p. 43)*

The modification of asphalts to resist permanent deformation can be done by increasing rigidity, elasticity or both. Rigidity can be increased by using stiffeners that will react with asphalt and change its consistency. Elasticity on the other hand requires creating an elastic structure using elastomeric materials which exhibits compatibility with the asphalt and resistant to changes due to oxidation, phase separation and unstable reaction (Bahia & Perdomo, 1996, p. 1192).

2.5.3 *Fatigue cracking*

Like rutting, fatigue cracking results from the large number of loads applied over time to a pavement subjected to traffic (NCHRP, 2011, p. 7). However, fatigue cracking tends to occur when the pavement is at moderate to low temperatures, rather than at the high temperatures that cause rutting. This is because the HMA at low temperatures is stiffer and more brittle than at high temperatures, it tends to crack under repeated loading rather than deform. Under the action of traffic loading, the initially microscopic cracks slowly grow in size and number into much larger cracks that can be visible to the naked eye and often referred to as “alligator cracks” such as the ones shown in Figure 2.11 below. These large cracks significantly affect pavement performance by weakening the pavement and allowing air and water into the pavement. The damage to pavement structure eventually leads to rough riding surfaces, large potholes, and total pavement failure.

Figure 2.11: Alligator cracks on HMA pavement

From a modification perspective, decreasing rigidity is simpler than increasing elasticity. Rigidity can be decreased with low cost hydrocarbons that are compatible with asphalt.

2.5.4 *Moisture susceptibility*

Water does not easily flow through properly constructed HMA pavements, but it will flow very slowly even through well-compacted material. Water can work its way between the aggregate surfaces and asphalt binder in a mixture, weakening or even totally destroying the bond between these two materials. This moisture damage is called stripping. Moisture damage can occur quickly when water is present underneath a pavement, as when pavements are built over poorly drained areas and are not properly designed or constructed to remove water from the pavement structure. Even occasional exposure to water can cause moisture damage in HMA mixtures prone to it because of faulty design or construction or poor materials selection. The physicochemical processes that control moisture damage are complex and only now are beginning to be understood (NCHRP, 2011, p. 9). Different combinations of asphalt binder and aggregate will exhibit widely varying degrees of resistance to moisture damage. It is very difficult to predict the moisture resistance of a particular combination of asphalt and aggregate. Hot Mix Asphalt produced with aggregates containing a large amount of silica, such as sandstone, quartzite, chert, and some granites, tend to be more susceptible to moisture damage (NCHRP, 2011, p. 9). Proper construction, especially thorough compaction, can help reduce the permeability of HMA pavements and so significantly reduce the likelihood of moisture damage. Anti-stripping additives can be added to HMA mixtures to improve their moisture resistance. Hydrated lime is one of the most common and most effective of such additives.

2.6 Modification Technology

2.6.1 Introduction

Road conditions are not static due to continuing developments in vehicle and tyre designs as well as traffic volumes and loading (Anochie-Boeteng, et al., 2011, p. 563; Department for International Development, 2002, p. 6) resulting into an increase in the stresses applied to the road. Heavier loads, higher traffic volume and higher tyre pressure demand higher performance pavements. The situation is worsened by the present drastic climatic variations that impact negatively on road pavements. This, in addition to the high cost of asphalt production has induced rigorous research in establishing a substitute or modification to improve the quality of the pavement.

The processes of asphalt modification involving natural and synthetic polymers were patented as early as 1843 (Kalantar, et al., 2012, p. 56). Test projects were underway in Europe in the 1930s, and neoprene latex began to be used in North America in the 1950s. In the late 1970s, Europe was ahead of the United States in the use of modified asphalts. This is because Europe used contractors who provided warranties thus motivated a greater interest in decreased life cycle costs, even at higher initial costs. The high preliminary expenses for polymer modified asphalt limited its use in the US. In the mid-1980s, newer polymers were developed and European technologies began to be used in the US (King, et al., 1993, p. 2). At the same time, the prevalence of a long-term economic outlook increased. In Australia, the current National Asphalt Specification includes guides and specifications regarding polymer modified binders (Yildirim, 2007, p. 66). More recently and South Africa's Asphalt academy and Southern African Bitumen Association (SABITA) have developed technical guidelines and manual for the use of modified binders in asphalt pavement applications.

The use of modifiers in asphalt concrete generally has two distinct approaches referred to as the 'wet' and 'dry' processes (Huang, et al., 2007, p. 64). The '***wet process'*** defines any method that blends the modifier with the asphalt binder prior to incorporating the binder into the asphalt mixture (Heitzman & Michael , 1992, p. 6). This process involves dissolving the polymer modifier in the bitumen at high temperatures to produce a viscous fluid through polymer-bitumen interaction (Gawandea, et al., 2012, p. 4; Flynn, 1993, p. 41). In the ***'dry process'*** a portion of mineral aggregates is replaced with ground rubber (0.85-6.4 mm) at typically a 1-3% replacement rate (Huang, et al., 2007, p. 66). In this process the reaction time between asphalt cement and crumb rubber is limited so that the crumb rubber can retain its physical shape and rigidity (Heitzman & Michael , 1992, p. 20; Huang, et al., 2007, p. 67)

Although the dry process presents some advantages in relation to the wet process, mainly concerning the costs involved and to the higher amount of modifier to be used, related research has concentrated mainly on the wet process. This choice may be explained by the irregular performance of some experimental sections built with the dry process, unlike the wet process, which has presented more satisfactory results (Cao, 2007, p. 1011). Considerable research into the wet process and the production of Crumb Rubber Modified (CRM) binders have been undertaken in South Africa over the past decade. The wet process is more popular because the binder properties are better controlled although its application requires modification of equipment (Heitzman & Michael , 1992, p. 25). On the other hand, the dry process is a far less popular method due to the need for specially graded aggregate to incorporate the reclaimed tyre crumb, construction difficulties and most importantly poor reproducibility and premature failures in terms of cracking and ravelling of the asphalt road surfacing. In addition, field trials have shown the performance of dry process CRM material used as a surface layer to be inconsistent and service life varies (Cao, 2007, p. 1011).

There are several reasons for this inconsistency and variation. The main assumption with the dry process is that rubber crumb is solely part of the aggregate and the reaction between bitumen and crumb rubber is negligible. However, recent research has showed that in the dry process, crumb rubber swells and reacts with bitumen at elevated temperatures and has an effect on the performance of the bitumen and, therefore, the asphalt mixture (Airey, et al., 2004, p. 455). The organic solvents in the bitumen diffuse into the bulk of the rubber increasing the dimensions of the rubber network until the concentration of the liquid is uniform and equilibrium is achieved (Heitzman & Michael , 1992, p. A-2). Hence the residual bitumen, i.e. bitumen not absorbed by the rubber, is the medium which binds the mineral aggregates and crumb rubber together (Airey, et al., 2001, p. 454). The interaction process depends on a number of variables, such as blending temperature, blending time, type and amount of mechanical mixing, polymer type, size and effective surface area of the polymer and the type of bitumen. Tyre properties also change with age and vary from manufacturer to manufacturer and this variability of scrap tyre source makes it even more difficult to control the consistency of the properties of the crumb rubber and consequently the properties of the mixture (Rahman , et al., 2011, p. 121). Therefore, the effect of crumb rubber particles in the dry process is different from the wet process as the binder modification and rubber swelling could change the mechanical properties of the mixtures.

Some of the above mentioned problems could be overcome by using both crumb rubber and polymers as surfacing layer modifiers. The binder viscosity can be increased in the wet process using LDPE in order to reduce the amount of binder absorbed by the crumb rubber in the dry process as illustrated in Figure 2.12. In this study, a larger crumb rubber size (2.36 mm) is used to further reduce the contact surface area in order to minimise absorption and swell of the crumb rubber in the mixture (Heitzman & Michael , 1992, p. 16).

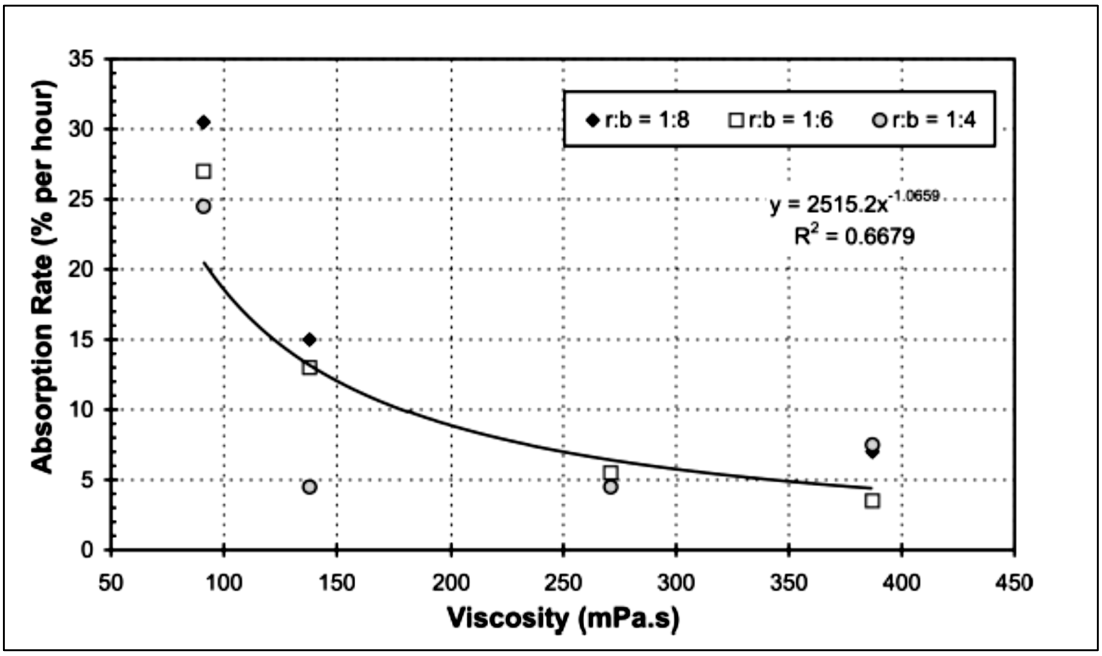

Figure 2.12: Relationship between the absorption rate and viscosity of different penetration grade bitumen at 160°C *(Airey, et al., 2001, p. 290)*

2.6.2 Polymer modified bitumen

Polymer-modified bitumen has been increasingly used to enhance pavement performance. Improvement in engineering properties including thermal cracking, stripping, rutting resistance, temperature susceptibility and fatigue damage have led to polymer modified binders to be a substitute for asphalt in paving and maintenance applications such as cold and hot crack filling, slurry seal, patching, hot mix, chip seals and recycling (Huang, et al., 2007; Yildirim, 2007, p. 70). The polymer in a polymer modified asphalt (PMA) road should neither make it too viscous at high service temperatures resulting in permanent deformation or too brittle at low service temperatures resulting in fatigue cracking (Alonso, et al., 2010, p. 2591).

The study by Chen et al. (2003, p. 594) was aimed at developing the procedure to determine the proper polymer content to be mixed with bitumen using two styrene butadiene-styrene (SBS) copolymers. The storage stability, shear rheometer and scanning electron microscopy (SEM) tests were conducted to investigate visco-elastic properties and microstructure of the polymer PMB. The investigation revealed that the addition of polymers increased the viscosity, softening point, toughness and complex modulus of bitumen. SEM results indicated that, as the polymer content increased, SBS gradually became the dominant phase and resulted in an increase in PMB's mechanical properties. Good compatibility produced an elastic network into the PMB up to 6% polymer concentration. The optimum polymer content was determined based on the rheological properties and formation of a critical network. However, adding higher polymer contents could lead to the separation of polymer and bitumen. In a paper by Jain et.al, (2011,p.237) it is reported that an increase in stiffness modulus of bituminous mixes is achieved by incorporation of waste polymeric packaging material (WPPM) through an optimum dose of WPPM of 0.3 to 0.4% by weight of bituminous mix for modification of bitumen and bitumen mix. There is little information to date about the use of PET in hot mix asphalt (Ahmadinia, et al., 2011, p. 4845).

Zoorob and Suparma (2000, p. 238) discusses the laboratory design of continuously graded asphaltic concrete mixtures containing recycled plastics aggregate replacement (Plastiphalt). Recycled waste plastics composed predominantly of low density polyethylene (LDPE) in pellet form were used in dense graded bituminous mixes to replace, by volume, a portion of the mineral aggregates of an equal size, i.e., 5.00.-

2.36 mm. The results obtained in this investigation indicated that at the same air-void content, the compacted Plastiphalt mix has lower bulk density than that a 30% aggregate replacement by volume with the LDPE, results in a reduction in bulk compacted mix density of 16%. This reduction in density is advantageous in terms of haulage costs. LDPE partial aggregate replacement also results in a 250% times increase in the Marshall Stability (strength) value and an improved Marshall Quotient value (resistance to deformation). The value of creep stiffness of the Plastiphalt mix after 1 hour loading at 60°C is found to be slightly lower than the control mix. However, the Plastiphalt gives 14% recovery after 1 hour unloading time compared to 0.6% for the control mix.

Some of the concerns about the use of recycled plastic as an asphalt cement modifier are performance and durability, cost effectiveness, availability, recyclability, health and environmental impacts (Tuncan , et al., 2003, p. 84). A number of asphalt properties when using recycled plastics are yet to be reported, nor are their certain cost and environmental implications due to limited practice so far (Huang, et al., 2007, p. 69). The literature on asphalt modification with plastomeric polymers is quite scarce, especially with respect to rheological properties (Al-Hadidy, et al., 2009, p. 1456).

2.6.3 Crumb rubber modifier

Used tyres require to be shredded before landfilling. In many countries, therefore, a levy is placed on tyres at point of sale to support tyre recycling technology and infrastructure. Current recycling and re-use options include re-treading of tyres and the use of tyres as fuel in cement kilns. Processes are also available for separating the steel from the rubber. The steel is then recycled and the rubber turned into rubber crumb.

In general crumb rubber has been used in asphalt mixtures to reduce cracking, improve durability and mitigate noise. It is generally agreed that crumb rubber improves durability and low temperature performance. On high temperature however there are mixed views from better, similar or comparable, to worse (Huang, et al., 2007, p. 69). Crumb rubber can be incorporated into asphalt paving mixtures using the wet process or the dry process. There are more studies and experiments on the use of crumb rubber in bituminous mixes related to the wet process than the dry process. This has made it possible to establish a set of reference values for different variables that affect its application with a view to obtain optimal results. In recent years however, the dry process has also captured the interest of researchers since it consumes larger quantities of waste than the wet process (Tuncan , et al., 2003, p. 84; Moreno, et al., 2012, p. 2323; Cao, 2007, p. 1011). In addition, the production of modified asphalt mixture by means of the dry process is logistically easier than the wet process and, therefore, the dry process is potentially available to a much larger market (Airey, et al., 2004, p. 455).

Liu et al. (2009, p. 2701) evaluated the performance of different crumb rubber modified (CRM) asphalt and reported that in general the crumb rubber content is the primary factor affecting the performance of CRM asphalt. Moreno (2012, p. 466), reports that the addition of crumb rubber significantly improved the performance of mixes and their response to plastic deformation. Hossain et al. (1995, p. 196) observed that since rubber is not as hard as the crushed stone aggregates, it follows that the Marshall stability of an asphalt aggregate chunk rubber mix would be lower than the mix without chunk rubber. However, it was also surmised that the larger rubber chunks tend to absorb some of energy imparted to compact a chunk rubber asphalt concrete (CRAC) sample, resulting in a weaker aggregate structure than a mix without any chunk rubber. According to Al-Abdul-Wahhab & Al-Amri (1991, p. 189), Mustafa et al., (2003, p.83) and Siddique & Naik, (2004, p. 568) pavements made of rubber and plastic modified asphalt concrete have better skid resistance, less cracking and a longer pavement life in comparison to conventional asphalt pavements. Early investigations on the use of discarded tyres in asphalt mixtures showed that rubberized asphalt had better skid resistance, reduced fatigue cracking and longer pavement life than conventional asphalt (Siddique & Naik, 2004, p. 565).

Cao (2007, p. 1011) in a study to minimize waste tyres pollution and improve properties of asphalt mixtures investigated the properties of recycled tyre rubber modified asphalt mixtures using the dry process. Tests on three types of asphalt mixture containing different rubber content (1%, 2% and 3% by weight of total mix) and a control mixture without rubber were conducted. Based on the results of rutting tests (60˚C), indirect tensile tests (-10˚C) and variance analysis, the addition of recycled tyre rubber in asphalt mixtures using dry process could improve engineering properties of asphalt mixtures, and the rubber content has a significant effect on the performance of resistance to permanent deformation at high temperature and cracking at low temperature. Cao (2007) concluded that the long term performance of recycled tyre rubber modified asphalt mixtures using the dry process required further study.

Research and feasibility studies have shown that bitumen rubber is the most cost effective modified binder in the South African bituminous product spectrum (Jooste, 2011, p. 597). In South Africa, crumb rubber modified (CRM) bitumen is manufactured through blending penetration grade bitumen (72-82%), rubber crumbs (18-24%) and extender oil (0-4%) (Technical Guide 1, 2007, p. 10) at elevated temperatures of between 190-210 ˚C. Thus large scale use of rubber from waste tyres in asphalt mixtures appears attractive and promising from the engineering and environmental point of view (Tuncan , et al., 2003, p. 83).

2.7 Conclusion

The philosophy of asphalt modification is expected to change from a general improvement of quality to focus more on using modifiers based on the most critical need as defined by two factors: (a) the application temperature domain, and (b) the type of distress to be remedied. Current research and practice tends to concentrate on the use of waste materials in the lower courses (base and sub-base) of the road as they absorb materials in larger quantities than the upper courses. However, highway authorities in South Africa are dealing more with the maintenance and repair works rather than new road construction. Such works are affecting mainly the upper layers of the pavement. Incorporating recycled waste polymers in pavement does not only have the potential of improving the performance of the pavement but will also reduce environmental pollution and need for extraction of mineral aggregates. The need to utilize virgin polymers in pavement will also be reduced, thus leading to possible cost savings in the long run. An improved asphalt pavement would mean extension in service life of the pavement. In addition there will be a significant reduction in density of the asphalt concrete mixture. This is economical in terms of haulage costs when tyre waste is used as partial replacement of aggregates and reduced bitumen used when plastic is used as binder modifier. This will further reduce the costs of construction materials.

3 RESEARCH METHODOLOGY

3.1 Introduction

This Chapter focuses on the materials, tests and standards that were used in this study. These tests are standards as normally applied to bituminous materials and mixtures used in road pavements. Authorities and publications relevant to test methods which were used are as follows:

a) TMH1, 1986: Standard Methods of Testing Road Construction Materials, superseded by SANS 3001
b) TRH8, 1987 used in parallel with Interim Guidelines for the Design of Hot-Mix Asphalt in South Africa.
c) TRH14, 1985 (reprinted 1989): Guidelines for Road Construction Materials
d) COLTO, 1998: Standard Specifications for Road and Bridge Works for State Road Authorities (currently under revision).
e) TG1, 2007: Technical Guideline: The use of Modified Bituminous Binders in Road Construction Second Edition.
f) ASTM International: Standards and Test Methods for Road Building Materials.
g) AASHTO Specifications and Test Methods

The HMA design method in South Africa that was used in this study is centred on the Marshall design method. The design process was divided into four phases namely;

- Preliminary considerations leading to mix selection and rating of design objectives
- Component evaluation (aggregate, binder and filler)
- Volumetric design leading to gradation and optimum binder content selection
- Performance testing

3.2 Hot Mix Asphalt

Figure 3.1 below shows the typical materials used in preparation of a conventional hot mix asphalt concrete mixture.

Figure 3.1: A compacted HMA briquette and the aggregates and asphalt binder used to prepare it *(NCHRP, 2011, p. 5)*.

The following materials were used in this research experiment:

3.2.1 Bitumen

The asphalt cement used in this investigation was obtained from Sasol South Africa. 60/70 penetration grade asphalt cement was used. This asphalt cement type was chosen because it is widely used in pavement construction in South Africa (SANRAL, 2013, p. 45 and Department for International Development, 2002, p. 22). Bitumen was sampled in accordance with Technical Methods for Highways procedures (TMH1, 1985). Bitumen tests were done to conform to the latest revision of SABS Method 307: Penetration Grade Bitumen.

3.2.2 Mineral aggregates

The coarse and fine aggregates used for this research was supplied by Afrisam (South Africa). The aggregates supplied and used in this research was crushed stones from dolerite rock for both the coarse aggregates fraction and crusher sand for the fines fraction. The aggregates supplied were of sizes 37.5 mm down to the crusher dust passing the 75 µm sieve size as shown in Table 3.1 below.

Table 3.1: Aggregate sizes as supplied by Afrisam (South Africa)

Aggregate Size (mm)	Quantity (kgs)
37.5	50
26.5	50
19	50
13.5	100
9.5	100
6.7	100
Crusher Sand (4.75 -0.075)	200

3.2.3 Waste polyethylene bags

Discarded plastic grocery bags, dry cleaning bags and household plastics were used. Low Density Polyethylene (LDPE) was targeted for use in this experiment to modify the bitumen. Figure 3.2 shows a sample of the waste LDPE bags used in this research.

Figure 3.2: Low Density Polyethylene

3.2.4 Rubber aggregate

A single size gradation of rubber aggregate normally referred to as crumb rubber of 2.36 mm standard sieve size was used from a single source in this experiment. The crumb rubber produced from rubber tyre waste and packed in 50 kg bag, was supplied from Vidar Rubber Products, Edenvale, South Africa. It can be noted that the sizes of the rubber particles lie in between ground and crumb rubber classification (see section

2.3.1). However, the terminology “crumb rubber” was adopted for this project to identify the rubber particles as it gives a narrow range in which the rubber used in this project falls.

3.3 HMA Component Evaluation

A series of tests were carried out on the aggregates, asphalt cement and modified binders as outlined in the subsequent subsections. The test methods used were largely drawn from the relevant South African National Standards (SANS) and American Society for Testing and Materials (ASTM) as discussed below. SANS 3001 series that succeeded the Technical Methods for Highways (TMH 1) for the standard methods of testing road construction materials was used.

3.3.1 Mineral aggregates

HMA mixtures are mostly aggregates, thus the aggregates used must be of good quality to ensure good pavement performance. The physical properties of aggregates are affected by the mineralogy of the parent rock, the extent to which the parent rock has been altered by leaching, oxidation etc., as well as by the processes required to produce graded and blended aggregate (The South African National Roads Agency, 2001, p. 3-1). The physical properties of aggregate are generally regarded as the most important aspect of aggregate selection. The required properties and acceptance criteria are shown in

Table *3.2*

Table 3.2: Required physical properties of aggregates fractions *(The South African National Roads Agency, 2001, p. 3-3).*

Property	Test	Designation	Acceptance Criteria
Particle Size	Grading: Sieve analysis	SANS 3001- GR1	
Hardness / Toughness	Aggregate Crushing Value (ACV)	SANS 3001-AG10; TMH1- B1	Maximum of: 25%: HMA base and surfacings,excl. open-graded and SMA mixes 21%:Open-graded surfacings and SMA
	Fines Aggregate Crushing Value (Dry 10% FACT)	SANS 3001-AG10; TMH1- B2	Minimum of: 160 kN: HMA surfacings and base, excluding open-graded and SMA mixes 210 kN: Open-graded surfacings and SMA
	Aggregate (Treton) Impact Value	SANS 3001-AG10; TMH1-B7	Maximum of: 25%
	Los Angeles Abrasion Test	ASTM C131 and ASTM C535	No Standard Specified Typical values are 10%: Very hard aggregate 60%: Very soft aggregate
Particle Shape and Texture	Flakiness Index Test	SANS 3001- AG4; TMH1- B3T	Maximum values for HMA surfacings 19 mm and 13.2 mm aggregate: 25 (grade 1*) or 30 (grade 2) 9.5 mm and 6.7 mm aggregate: 30 (grade 1*) or 35 (grade 2) HMA bases 35 (applies to -26.5 mm/+10.9 mm and -19.0 mm/+13.2 mm sieve fractions)
	Course Aggregate Fractured Faces	ASTM D5821	HMA surfacings At least 95% of all particles should have at least three fractured faces
Absorption	Water Absorption, coarse	TMH1- B14	Maximum of 1% by mass
	Water Absorption, fine aggregate	TMH1- B15	Maximum of 1.5% by mass
Cleanliness	Sand Equivalent Test	TMH1- B19	Minimum of: 50: total fines fraction 30: natural sand fraction to be mixed with aggregate (where permitted)

a) Grading of mineral aggregates

Grading was done in accordance with SANS 3001-GR1. A representative sample of 1200 g aggregate was passed through a stack of sieves of decreasing sizes. Grading was carried out on these aggregates to ascertain their gradation sizes as stipulated by the supplier. For this test, the sample of aggregate was sieved through a stack of the following standard sieve sizes: 37.5 mm, 26.5 mm, 19 mm, 13.2 mm, 9.5 mm, 4.75 mm, 2.36 mm, 1.18 mm, 600 µm, 300 µm, 150 µm, 75 µm and a receiving pan for materials passing the 75 µm sieve (Figure 3.3). The sieves were shaken mechanically until the aggregate had been separated completely on the various sieves. The aggregates retained on each sieve was then weighed and calculations are performed to determine the percentage passing each sieve size.

Figure 3.3: Test sieves

A continuously graded mix was chosen for this research. The grading limits used were derived from South African Pavement Engineering Manual (SANRAL, 2013, p. 26). A nominal maximum size of 13.2 mm was used for all design mixes. The selected gradation in this research within the limits was achieved by trial and error and the results are presented, analysed and discussed in Section 4.2.1.1.

b) Mineral Aggregate Crushing Value

This test was done in accordance with SANS 3001- AG10. The ACV determines the percent of fines produced under a load of 400kN. The test was done on oven dried aggregates passing the 13.2 mm and retained in 9.5 mm sieves. The aggregates were placed in the cylinder in three layers, each third being compacted by 25 strokes of the tamping rod evenly distributed over the surface of the layer. The plunger was inserted level on the surface of the aggregate in the cylinder and the apparatus with the specimen placed between the platens of the compression testing machine. A force of 400 kN was applied at a uniform rate of 0.67 KN/s for 10 minutes. The specimen was then sieved through a 2.36 mm sieve and the mass passing the sieve recorded. The apparatus used for this experiment is shown in Figure 3.4.

Figure 3.4: Compression machine setup

The results are presented, analysed and discussed in Section.4.2.1.

c) *Fines Aggregate Crushing Test (Dry 10% FACT)*

This test was done in accordance with TMH1-B2. The test procedure and equipment is similar to that of ACV test discussed above. The 10% FACT was determined by measuring the load required to crush the aggregate sample to give 10 percent of the test sample passing a 2.35 mm sieve. The test results are presented, analysed and discussed in Section 4.2.1.

d) Aggregate (Treton) Impact Value

This test, also known as Treton Impact value test, was done in accordance with TMH1 Method B7. The test was performed on oven dried aggregates passing 19.0 mm and retained in 13.2 mm sieve size. The cylindrical metal measure was filled with 15 to 20 pieces of aggregates and the weights recorded. The sample was then placed in the test cylinder as shown in Figure 3.5. The cylinder with specimen was then placed in the AIV equipment shown in Figure 3.6. The hammer weighing 15 kgs was let to drop 10 times on the sample. The sample was then sieved through 2.0 mm sieve and the sample passing weighed and reported as a percentage of the test sample mass.

Figure 3.5: AIV Sample in test cylinder

Figure 3.6: The aggregate impact value (Treton) apparatus

The AIV results are presented, analysed and discussed in Section 4.2.1

e) Los Angeles Abrasion (LAA) Test

This test was performed according to ASTM C 131-89 standards and procedures. The LAA test was done to assess the hardness of coarse aggregates. This gave a measure of degradation of mineral aggregates from a combination of traffic actions including abrasion, impact and grinding. Resistance to wearing action of moving traffic on the road is an essential property for aggregates used in road construction. The LAA test was done on washed and oven dried mineral aggregates of 9.5mm and 6.7 mm grading. The sample was placed in a rotating steel drum shown in Figure 3.7 containing 11 steel spheres at a speed of 33 revolutions per minute for 500 revolutions. The contents were removed after 500 revolutions, sieved through the 2.0 mm sieve, washed and weighed. The percentage passing the 2.0 mm sieve was recorded as the LAA value.

Figure 3.7: LAA apparatus

The results presented, analysed and discussed in Section 4.2.1.

f) Flakiness Index of mineral aggregates

This was done in accordance with SANS 3001–AG4. The presence of flaky particles are undesirable for base course and wearing course as they are likely to break down under heavy loads. Aggregates are classified as flaky when they have a thickness of less than 60% of their mean sieve size. This test was carried out by determining the percentage of the total mass of the aggregate that passes through slots of a specified width in a metal plate. The aggregates passing 37.5 mm and retained in 4.75 mm sieves (coarse aggregates) were passed through the flakiness sieves with slot width sizes 19.0 mm, 13.2 mm, 9.5 mm, 6.7 mm, 4.8 mm and 3.4 mm as specified in SANS 3001-AG4. Figure 3.8 shows the apparatus used during the flakiness index test.

Figure 3.8: Flakiness Index test sieve

The FI results are presented, analysed and discussed in Section 4.2.1.

g) *Course mineral aggregate fractured faces*

This test was done in accordance with ASTM D5821. Aggregate particles larger than 4.75 mm were visually examined to determine the percentage of particles that has at least three fractured faces.

h) *Density and water absorption of mineral aggregates retrieved on a 4.75 mm Sieve*

This test was done in accordance with SANS 3001 AG20. This test was used to determine the dry density of aggregates retrieved on a 4.74 mm sieve. The experiment was set up as shown in Figure 3.9. The dry bulk density and apparent density of aggregates retained on the 4.75 mm sieve were calculated from the loss in mass of saturated surface dry (SSD) aggregates when submerged in water (weight of aggregates in water). The water absorption was determined by calculating the mass of water absorbed as a percentage after 24 hrs immersion of the oven dried material.

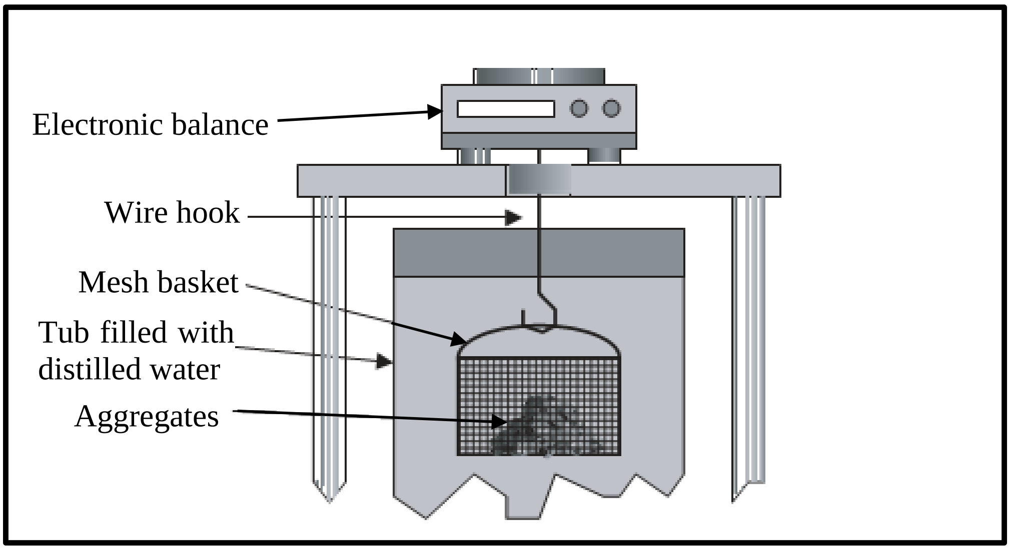

Figure 3.9: Weight in water set-up for determining specific gravity of coarse aggregate *(NCHRP, 2011, p. 35)*

The computation of the above values were accomplished by the following equations:

Bulk density, $\rho_{bd} = \frac{A}{\frac{(B-C)}{0.997}}$ (g/cm^3) (3.1)

Apparent density, $\rho_{ad} = \frac{A}{\frac{(A-C)}{0.997}}$ (g/cm^3) (3.2)

Water absorption (%) = $\frac{B-A}{A} x\ 100$ (3.3)

Where:

A= mass of oven dry sample in air, grams

B = mass of saturated surface dry sample in air, grams

C = mass of saturated sample in water at 25 °C, grams

The results presented, analysed and discussed in Section 4.2.1.

i) Density and water absorption of aggregates passing the 4.75 mm sieve

The bulk density and apparent relative density of material passing 4.75 mm sieve was calculated from the loss in mass of saturated surface-dry aggregate when submerged in water according with SANS 3001-AG21. The bulk density, apparent density and water absorption were calculated using the following equations:

Bulk density, $\rho_{bd} = \frac{A}{D-\frac{(C-B)}{0.997}}$ g/cm^3 (3.4)

$$Apparent\ density,\ \rho_{ad} = \frac{A}{D - \frac{(C-A-E)}{0.997}}\ (g/cm^3) \quad (3.5)$$

$$Water\ absorption = \frac{B-E-A}{A}\ x\ 100\ \% \quad (3.6)$$

Where:

A = mass of oven dry sample, grams

B = mass of saturated surface dry sample + pycnometer, grams

C = mass of saturated sample + pycnometer filled with water, grams

D = volume of pycnometer, cm^3

E = mass of clean dry pycnometer, grams

The results presented, analysed and discussed in Section 4.2.1.

j) Cleanliness of aggregate

The Sand Equivalent test was done in accordance to SANS 3001- AG5 procedures to evaluate the cleanliness of aggregates so as to identify when harmful clay-sized particles exist in an aggregate blend. The presence of dust or clay coatings on aggregates can prevent proper coating of the aggregates by the asphalt binder in the asphalt concrete mixture. This leads to water penetrating the asphalt binder film and stripping of the asphalt binder from the aggregate.

This procedure was conducted on the aggregate fraction of the blend that passes the 4.75 mm sieve. The aggregate sample was placed within a graduated transparent cylinder filled with a mixture of water and flocculating agent. The mixture of aggregate, water and flocculating agent was then agitated for 45±5 seconds. The mixture was then allowed to settle at room temperature for 20 minutes. The heights of the sand particles and the sand plus clay particles were measured after 20 minutes as shown in Figure 3.10. The sand equivalent value was calculated as the ratio of the height of the sand to the height of sand plus clay and expressed as a percentage.

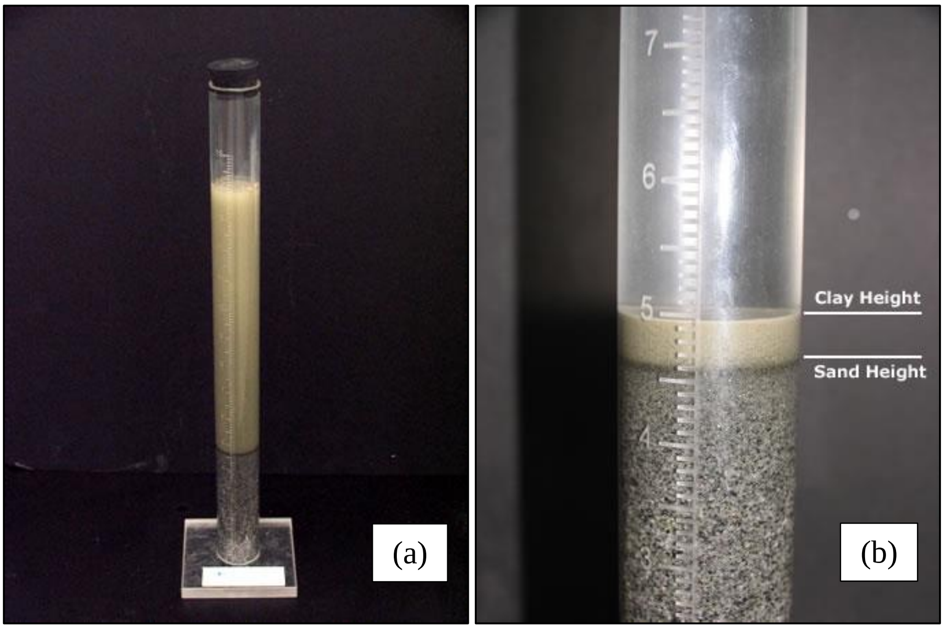

Figure 3.10: (a) Cylinder after shaking and irrigating - clay has not settled (b) Close up of cylinder showing clay and sand height.

The sand equivalent results are presented, analysed and discussed in Section 4.2.1.

3.3.2 Crumb rubber aggregate

Sieve analysis (grading) was performed on the crumb rubber sample shown Figure 3.11 in order to ascertain the size of the crumb rubber. The crumb rubber was passed through 2.36 mm and 1.18 mm standard sieve sizes to ensure the right crumb rubber size was used for this study.

Figure 3.11: Crumb rubber sample

3.3.3 60/70 penetration grade bitumen

a) Relative density

This test was done in accordance with ASTM D70-97. This test was used to determine the relative density and density of semi-solid bitumen. A sample of bitumen was filled into calibrated pycnometer of known mass and volume. The pycnometer was filled with distilled water at 25 °C and weighed. The results are presented, analysed and discussed in Section 4.2.3

b) Penetration

This test was performed in accordance with ASTM D5-6. The penetration test is an empirical method that was used to measure the consistency of bituminous materials. Usually penetration is measured at 25 °C which represents an approximate average in-service temperature. The value is used to classify the bitumen into standard penetration ranges in accordance with SANS 4001-BT1 and ASTM D5. The penetration value of the bitumen is measured and expressed as the distance in tenths of a millimetre (1/10-mm) that a standard needle vertically penetrates into the bitumen sample under a load of 100 g applied for five seconds at 25 °C. The test set up is shown in Figure 3.12. Results are presented, analysed and discussed in Section 4.2.3.

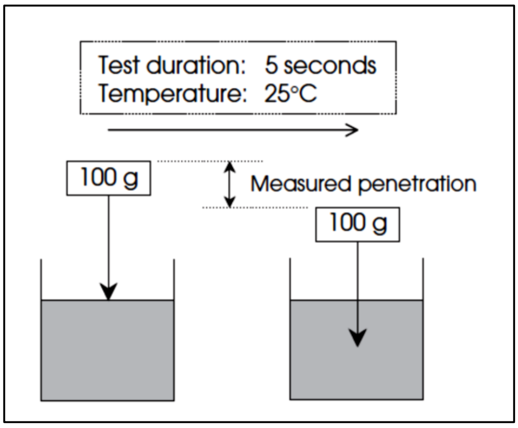

Figure 3.12: Penetration test for bitumen *(Rogers, 2003, p. 210)*

c) Softening Point

This test was done in accordance with ASTM D36. This test was used to determine the specific temperature at which the bitumen is transformed from a solid to a liquid phase. It also gives an indication of the tendency of the material to flow at elevated temperatures encountered in service. It is also known as the Ring-and-Ball softening point test. This test was used for bituminous binders in the temperature range 30 °C to 200 °C. A steel ball of 3.5 g was placed on the sample of binder contained in a brass ring which was suspended in a water bath. The water bath temperature was raised at 5 °C per minute as the binder gradually softened as the ball fell through the ring. The temperature of the water was recorded at the moment the bitumen and steel ball touched the base plate 25 mm below the ring. The Figure 3.13 below shows the ring-and –ball test. The results are presented, analysed and discussed in Section 4.2.3.

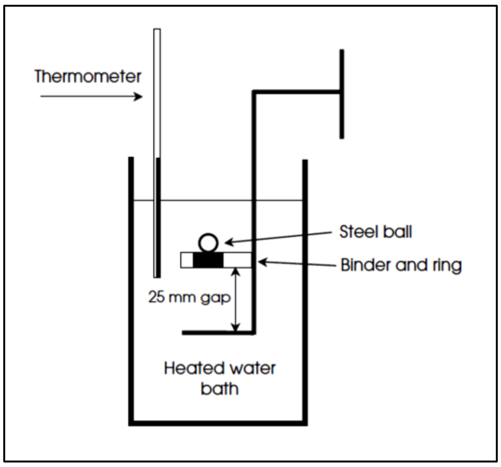

Figure 3.13: Softening point test *(Rogers, 2003, p. 211)*

d) Ductility

The ductility test was done in accordance with ASTM D 113-86. This test was used to describe the ductile and tensile behaviour of bituminous binders. The test reflects the homogeneity of the binder and its ability to flow. A dumbbell shaped specimen was placed in a water bath at 25 °C for 30 min. The sample was then placed in the ductilometer and made to stretch at 50 mm/min until it reached a breaking point. The distance at rupture was reported in cm as the ductility. Figure 3.14 shows ductility test setup.

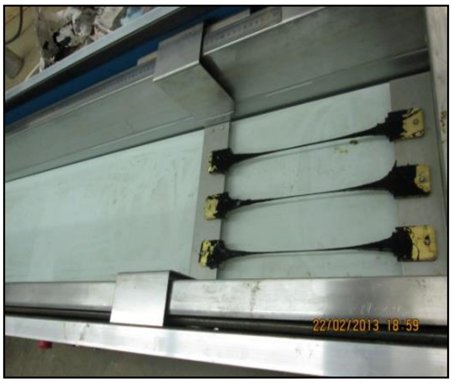

Figure 3.14: Ductility test setup

Three samples were tested and the average calculated. The results are presented, analysed and discussed in Section 4.2.3.

e) Dynamic viscosity

The dynamic viscosity test was done according to ASTM D4402. This test was used to measure the resistance to flow or shear due to internal friction and is expressed in units of stress required to overcome this friction. The dynamic viscosity was determined by measuring the torque required to rotate a spindle which was immersed in bitumen at 60 °C. The apparatus used was the Brookfield model DV with Thermosel system using SC4-24 type spindles shown in Figure 3.15.

Figure 3.15: Brookfield viscometer

These temperatures of the bitumen determine its temperature-viscosity characteristics at high in-service and application temperatures respectively. The appropriate temperature range for compaction is derived from the temperature-viscosity relationship of the aged binder. Three tests were carried out and the average calculated. The results are presented, analysed and discussed in Section 4.2.3.

f) Rolling Thin Film Oven Test

Rolling Thin Film Oven (RTFOT) test was done in accordance with ASTM D2872. This test was done to simulate asphalt binder age hardening as it occurs during mixing, transport, and placement. In this procedure, 35g of asphalt was carefully weighed into glass bottles which were then placed in a circular rack in a specially designed oven shown in Figure 3.16. A rack holding a series of glass bottles rotates in a vertical plane so that a fresh surface of bitumen is continuously being exposed to air while the oven maintains the test temperature of 163 °C. A jet of air was blown into the bottles from a single nozzle for a few seconds once every rotation. This test was continued for 75 minutes after which the bottles were then removed from the oven, cooled, and weighed. The percentage mass loss was calculated from the initial and final weight of the asphalt binder in the bottle. The average of three samples was calculated and the results were presented, analysed and tabulated in Section 4.2.3.

Figure 3.16: Rolling Thin Film Oven (RTFO) equipment

After mass loss determination, the bottles were heated, and the asphalt was poured into tins for further rheological testing. The penetration, softening point, ductility and viscosity were assessed in terms of the requirements of the relevant specifications as outlined in Section 3.3.2 above. The results are presented, analysed and discussed in Section 4.2.3.

3.3.4 LDPE modified bitumen

The penetration test, softening point test and dynamic viscosity tests were carried out in the same manner as the conventional bitumen test discussed in Section 3.3.2. The results are presented, analysed and discussed in Section 4.2.4. The RTFOT and ductility test methods on modified bitumen in accordance with Technical Guideline 1 (2007) and are given as follows.

a) Modified Rolling Thin Film Oven Test

This test was done in accordance with TG1 MB-3. This test has a similar purpose and procedure to that described under the Rolling Thin Film Oven Test carried out on conventional penetration grade bitumen. The effects of the LDPE modifier on age-hardening were evaluated by measuring the rheological properties of asphalt binders before and after selected levels of ageing. Short-term ageing was simulated using the rolling thin film oven at 163°C. Three samples were prepared for each percentage of LDPE content in the bitumen i.e. 2%, 4%, 6%, 8% and 10%. The percentage mass loss was calculated from each of the glass bottles. The results are presented, analysed and discussed in Section 4.2.4.

The LDPE modified bitumen samples from the RTFOT were used in the dynamic viscosity (TG MB-13), penetration, softening point (TG MB-17) and elastic recovery by ductilometer (TG MB-4) tests. The dynamic viscosity, penetration and softening point test procedures are similar to those carried out on conventional bitumen and were conducted as discussed in Section 3.3.2 above. The results are presented, analysed and discussed in Section 4.2.4

b) Elastic Recovery of Polymer Modified Binders by Ductilometer

This test was done on samples from the RTOFT in accordance with TG MB-4. This test method was used to assess the elastic recovery properties of a polymer modified binder. Moulded specimens were stretched for a distance of 200 mm in a ductilometer at 25 °C at a rate of 50 mm/min. The elongated thread was cut and the extent of recovery of the thread was measured after an hour. Three sample tests were done on three samples and an average calculated. The results are presented, analysed and discussed in Section 4.2.4.

3.4 Volumetric Design

The Marshall mix design method was used for this research. The design of asphalt paving mix was done by selecting and proportioning constituent materials to obtain the desirable properties in the pavement structure. Two major features of the Marshall method of design namely, density-voids analysis and stability-flow test were considered. The following procedure was followed for the Marshall mix design method:

- i) The aggregates to be used were selected;
- ii) The proportion of each aggregate size to fit the design grading curve was determined;
- iii) The specific gravity of the aggregates and asphalt cement was determined;
- iv) Trial specimens with varying asphalt contents were prepared;
- v) The specific gravity of each compacted specimen was determined;
- vi) The percentage voids in mix, voids filled with binder in each specimen was calculated;
- vii) The optimum binder content was selected and
- viii) Evaluating the mix design with the design performance requirements

3.4.1 *Preparation of asphalt concrete briquettes*

This was done in accordance with SANS 3001-AS1. The coarse aggregate, fine aggregate and filler material was proportioned to the required grading as in Section 3.3.1. Six kilograms of the mixture was taken to produce five compacted briquettes of approximately 63.5 mm thickness as shown in Figure 3.17. Approximately 1200 g of aggregates was used to produce the desired thickness.

Figure 3.17: Briquette Sample

The aggregates were heated to 175 °C in a thermoelectric controlled oven shown in Figure 3.18**.** The compaction mould assembly was kept pre-heated to 145 °C. The bitumen was heated to 160 °C and added to the mixture in 0.5 percent increments. The initial percentage was of the total aggregate weight of aggregates. The mixing was done thoroughly at 145 °C as shown in Figure 3.18 and placed in the standard Marshall mould and compacted with 75 Marshall blows on either side of the briquette giving a rough estimate of typical construction compactive effort for most densely graded mixes (The South African National Roads Agency, 2001, p. 4-7) and in accordance with ASTM D 1559.

Figure 3.18: (a) Heating up the bitumen (b) Preparation of the asphalt concrete mixture.

The samples were taken out of the mould after cooling to room temperature using an extruder. The following properties were determined using material prepared from the above procedure:

i. Bulk density
ii. Stability and flow
iii. Relative density and void analysis.

The tests and analysis methods are described in the following subsections.

3.4.2 Bulk relative density of the briquettes

The bulk relative densities (BRD) of the briquettes were determined by weighing the sample in air and in water as shown in Figure 3.19 below. The bulk density was calculated as follows;

Volume of test sample: $V_1 = \frac{m_3 - m_2}{\rho_{water}}$ (3.7)

Bulk relative density: $\rho_{bd} = \frac{m_1}{V_1}$ (3.8)

Where:

m_1 = mass in air (dry specimen)

m_2 = mass submerged in water

m_3 = mass of saturated surface dry specimen

ρ_{water} = density of water at 25 °C, set to be 0.997 g/cm^3

Calculation of water absorption was done by the following equation:

$$Water\ absorption = \frac{m_3 - m_1}{V_1}\ x\ 100 \quad (3.9)$$

The results are presented, analysed and discussed in Section 4.4.1.

Figure 3.19: Buoyancy balance apparatus

3.4.3 Maximum theoretical relative density of aggregate mixture

The maximum theoretical relative density (MTRD) of aggregate mixture was obtained by Rice's method as per ASTM D 2041-95. This method was used to determine the theoretical density (void free density) of asphalt mixes. A sample of oven dry sample in loose condition was placed in a conical flask with a rubber stopper and completely submerged in water at 25 °C. A vacuum pump was added to the setup shown in Figure 3.20 below to remove entrapped air within the sample. The void free volume of the sample was weighed with the container and sample immersed in water. The maximum theoretical density was used to calculate the air void content of asphalt mixes. It was also used to determine the amount of binder absorbed by the aggregates in the asphalt mix.

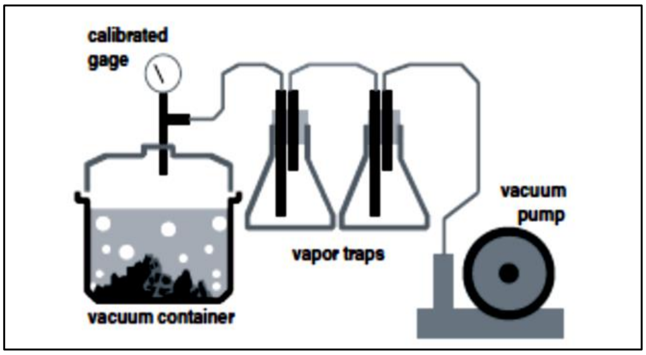

Figure 3.20: Diagramatic representation for determining maximum theoretical relative density

Calculations:

The calculation of maximum theoretical density to the nearest 0.001 g/cm^3 was done using the following formula:

Maximum theoreticalrelative density, $\rho_{d,\ max} = \frac{A}{\frac{A-(B-C)}{\rho(water)}}$ g/cm^3 (3.10)

Where:

A = mass of dry sample in air, grams

B= mass of flask and sample immersed in water, grams

C= mass of flask immersed in water, grams

ρ_{water} = density of water (0.997 g/cm^3 at 25 °C)

The maximum theoretical density was used in the calculations of the voids in the asphalt mixtures as presented in Section 4.3.

Calculation of binder absorption:

Calculation of the binder absorption was done by the following steps:

Step 1: The mass of binder in the sample was determined by:

$D = \frac{p\ x\ A}{100}$ grams (3.11)

Where:

D= mass of binder in the sample, grams

A = mass of dry sample in air, grams

p = percentage of binder in mixture, %

Step 2: The mass of aggregates in the sample was determined by:

$E = \frac{(100-p) x\ A}{100}$, grams (3.12)

Where:

E= mass of aggregates in the sample, grams

A= mass of dry sample in air, grams

p = percentage of binder in the mixture, %

Step 3: The bulk volume of aggregates and binder in the sample was determined by:

$$Vol_{bulk} = \frac{E}{\rho_{bd}} + \frac{D}{\rho_{bit}}, \text{ cm}^3 \tag{3.13}$$

Where:

Vol_{bulk} = bulk volume of aggregates and binder in the sample, cm^3

D = mass of binder in the sample, grams

E = mass of aggregates in the sample, grams

ρ_{bd} = density of the aggregate, g/cm^3

ρ_{bit} = density of the binder, g/cm^3

Step 4: The volume of void-free sample was determined by:

$$Vol_{voidfree} = \frac{A-(B-C)}{\rho_{water}}, \text{ cm}^3 \tag{3.14}$$

Where:

$Vol_{voidfree}$ = void-free volume of the sample, cm^3

A = mass of dry sample in air, grams

B = mass of flask and sample immersed in water

C = mass of flask immersed in water, grams

ρ_{water} = density of water (0.997 g/cm^3 at 25 °C)

Step 5: The percentage of the binder absorbed by the aggregates was determined by:

$$P_{abs} = \frac{(Vol_{bulk} - Vol_{voidfree}) x \, \rho_{bit}}{E} \; x \; 100, \text{ \%} \tag{3.15}$$

Where:

Vol_{bulk} = calculated bulk volume of aggregates and binder in the sample, cm^3

$Vol_{void\text{-}free}$ = void-free volume of the sample, cm^3

E = mass of aggregates in the sample, grams

The values gotten from the calculations were used in the calculation of voids in the concrete mixture. The results are presented, analysed and discussed in Section 4.3

3.4.4 Calculation of void content in bituminous mixes

This was done in accordance with ASTM D3203 and AASHTO PP19-93. This method was used to calculate void content, voids filled with binder and voids in mineral aggregate for bituminous mixes. The void content was calculated using the density and the maximum theoretical density of the samples. The voids in mineral aggregate were determined from the bulk volume of asphalt mix sample and the bulk volume of aggregates in the mix. The voids filled with binder were determined from the volume of effective binder in the mix and the volume of voids in the aggregate.

The mass of aggregates in the sample was calculated by:

$$m_{agg} = \rho_{bd\ mix} \text{ x } (100\text{-p}) \tag{3.16}$$

The mass of binder in the sample was calculated by:

$$m_{bit} = \rho_{bd\ mix} \text{ x p} \tag{3.17}$$

The mass of absorbed binder was calculated by:

$$m_{abs} = \frac{P_{abs}\ x\ m_{agg}}{100} \tag{3.18}$$

The mass of effective binder was calculated by:

$$m_{eff} = m_{bit} - m_{abs} \tag{3.19}$$

The volume of aggregates was calculated by:

$$V_{agg} = \frac{m_{agg}}{\rho_{bd\ agg}} \tag{3.20}$$

The volume of effective binder was calculated by:

$$V_{bit} = \frac{m_{eff}}{\rho_{bit}} \tag{3.21}$$

Where:

$\rho_{bd,\ mix}$ = bulk density of asphalt mix sample, g/cm^3

P = binder content, %

P_{abs} = binder absorbed by the aggregates, %

$P_{bd,\ agg}$ = average bulk density of total aggregates, g/cm^3

P_{bit} = density of the binder, g/cm^3

Calculation of void content

The void content was calculated using the following equation:

$$V_0 = \frac{\rho_{d\ max} - \rho_{bd\ mix}}{\rho_{d\ max}} x100 \quad (3.22)$$

Where:

$\rho_{d\ max}$ = maximum theoretical density of sample, g/cm^3

$\rho_{bd\ mix}$ = bulk density of asphalt mix sample, g/cm^3

Calculation of voids in mineral aggregate

The voids in mineral aggregate were calculated using the following equation:

$$VMA = 100 - V_{agg} \quad (3.23)$$

Where V_{agg} = volume of aggregates, cm^3

Calculation of voids filled with binder

The voids filled with binder were calculated using the following equation;

$$V_{fb} = \frac{V_{bit}}{VMA} x\ 100 \quad (3.24)$$

Where:

V_{bit} = volume of effective binder, cm^3

VMA = voids in mineral aggregate, %

The results are presented, analysed and discussed in Section 4.3.

3.4.5 Calculating target binder content

The target value for VBE (effective binder content by volume) was calculated by subtracting the design air void content—normally 4%—from the target VMA value. For example, a standard 13.5 mm mixture with a target VMA value of 15% would have a target VBE value of 15 − 4 = 11%.

The amount of binder absorbed by the aggregate was also added to come up with the required binder content in the asphalt mix. A quick approximate estimate, suitable for developing trial batches of 1% was added to the target VBE value. In the example above, this would result in a total target binder content of 12%. A more accurate estimate would be to calculate the volume of water absorbed by the aggregate, divide this by two, and add it to the target VBE value:

$$V_b = VBE + \left(1 - \frac{VMA}{100}\right)\left(\frac{G_{sb}P_{wa}}{2}\right) \tag{3.25}$$

Where;

V_b = total asphalt content by volume, %

VBE = effective asphalt content by volume, %

VMA = voids in the mineral aggregate = V_{be} + air void content

G_{sb} = aggregate bulk specific gravity

P_{wa} = water absorption of the aggregate, weight, %

3.4.6 Marshall Flow, Stability and Quotient

This test was done in accordance with SANS 3001-AS2. The Marshall briquette specimen prepared as described in Section 3.4.1 was preconditioned at temperature of 60 °C for 30 minutes in a water bath. The specimen was taken straight from the water bath and inserted in a preheated breaking head assembly then loaded in a direction perpendicular to the cylindrical axis at a steady rate of 50.8 mm/minute. The maximum load at failure and the deformation was recorded as the Marshall Stability and flow respectively. The ratio of stability to the flow gave the Marshall quotient. The apparatus used in the test is shown in Figure 3.21.

The results are presented, analysed and discussed in Section 4.3.

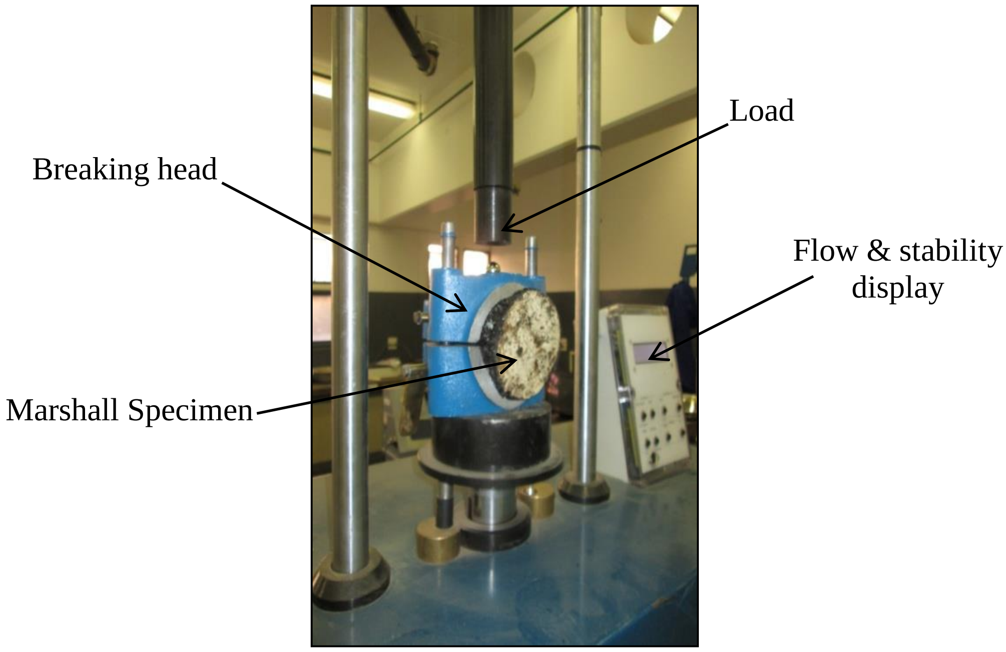

Figure 3.21: Marshall stability and flow test setup

3.4.7 Determination of optimum binder content

The optimum asphalt binder content was determined based on the combined results of Marshall Stability and flow, density analysis and void analysis. Optimum asphalt binder content was arrived at as per the following procedure:

I. The following curves were plotted with asphalt percentage increments on the x-axis:
   (a) Asphalt binder content against density.
   (b) Asphalt binder content against Marshall stability.
   (c) Asphalt binder content against flow.
   (d) Asphalt binder content against air voids.
   (e) Asphalt binder content against VMA.
   (f) Asphalt binder content against VFA.

The above graphs are presented in Section 4.3

II. The asphalt binder content that corresponds to the specifications median air void content of 4 % was determined. This is the optimum asphalt binder content.

III. The properties of the asphalt binder at this optimum binder content was determined by referring to the plot and each compared against specification values to ensure all are within specification.

The optimum binder content was selected as the average binder content for maximum density, maximum stability and percent air voids in the total mix. It was calculated by equation 3.26 below;

$$B_0 = \frac{B_1 + B_2 + B_3}{3} \tag{3.26}$$

Where;

B_0= optimum bitumen content

B_1= % asphalt content at maximum density

B_2 = % asphalt content at maximum stability

B_3 = % asphalt content at 4% air voids in total mix

3.5 Optimization Process

3.5.1 LDPE modified binder

Asphalt cement was heated in an oven at a temperature of 160 °C (Cao, 2007). The required amount of asphalt was weighed into a steel beaker, then the amount of plastic required to yield the desired plastic to asphalt ratio was added from 2-10 % by weight of bitumen (Kalantar, et al., 2012, p. 57). The beaker was placed on a hot plate to maintain a mixing temperature of at least 165 °C. The laboratory mixer was placed so that the propeller was about 15 mm above the bottom of the beaker and started. The prepared amount of plastic was added gradually to the beaker while stirring. The mixer was continued for 5-15 minutes until a homogeneous plastic modified binder was obtained (Hinislioglu & Agar, 2004, p. 269).

3.5.2 Rubber aggregate

Crumb rubber of sieve fraction 2.36 mm was used to substitute a fraction of the fine mineral aggregates of similar sieve size (2.36 mm) so that the overall grading was maintained. Proportions of 0%, 1% 2%, 3%, 4% and 5% of crumb rubber by weight of the aggregates were used in the asphalt mix (Cao, 2007, p. 1012). The ground rubber was added to the hot aggregate mixture just before the polymer modified bitumen was introduced into the mixture.

3.5.3 Preparation of modified Marshall briquettes

A percentage of optimum asphalt content by weight of the total mix established from the Marshall mix design was used in preparing the LDPE modified bitumen to be used in preparing asphalt concrete mixes throughout the study. The aggregates were heated to 160 °C in a thermostatically controlled oven and LDPE modified binder heated to 180 °C. The crumb rubber was introduced to the aggregate mix and mixed evenly into the aggregate mix for 10 seconds then modified binder was introduced into the mixture and the sample mixed thoroughly for 2 to 3 minutes at a temperature of 135 °C. The mixture was then placed into a standard Marshall mould with base and collar attached and compacted using the Marshall compactor according to test method specified in ASTM D1559.

Waste plastic content was varied from 2% to 10% at 2% intervals (Ahmadinia, et al., 2011, p. 4846; Al-Hadidy, et al., 2009, p. 1458) while the rubber content in the aggregates was varied from 1% to 5% percent by volume at 1% intervals with the corresponding zero percentages being the control specimens. Three samples were prepared for each polyethylene-binder and rubber-aggregate contents in the mix. The three samples prepared were used for the Marshall triplicate testing to ASTM D1559. In total 108 briquettes were prepared for the experiment. The temperature of the mix was increased from the conventional 165°C to 175 °C to keep the modified binder viscous enough to improve the workability of the mix. The manufacture of the mix involved 10 seconds agitation of the aggregate with the crumb rubber to ensure a homogeneous dispersion of the particles in the mix. LDPE modified bitumen was added followed by a 3 minutes of agitation to form a homogeneous mix. Preparation of the test specimens as well as determination of the bulk densities, voids in the mix and voids in the mineral aggregate of the asphalt concrete was done as described in Section 3.4**.**

3.6 Modified Hot Mix Asphalt

Wheel tracking tests are recommended for the evaluation of rutting performance on mixes involving modified binder as experience has shown that these mix types cannot be properly evaluated by means of conventional methods such as the dynamic creep test (CSIR & SABITA, 2001, p. 6-5). The indirect tensile strength test also provides an indication of mix stability in low to mid-temperature range (10° to 40°C) and can relate to rutting resistance as well as durability and stripping potential.

3.6.1 Permanent deformation test

The permanent deformation performance and susceptibility to moisture damage of bituminous road pavement mixtures was evaluated using simulated traffic loading with the third scale Model Mobile Load Simulator (MMLS3) shown in Figure 3.22 under controlled environmental conditions. The test involved the use of an MMLS3 machine which is equipped with one axle with 300 mm diameter inflatable pneumatic wheel, circulating in a vertical closed loop. This configuration enables 7200 load repetitions per hour to be applied to the test bed, which consist of briquettes prepared on laboratory prepared slabs as shown in Figure 3.23. Cross-sectional profiles were measured at 0, 2500, 5000, 10000, 25000 50000 and 100000 axle load repetitions to determine the depth of rutting. The temperatures were kept relatively constant at 50±2°C. The results are presented, analysed and presented in Section 4.5.

Figure 3.22: MMLS3 Testing Equipment

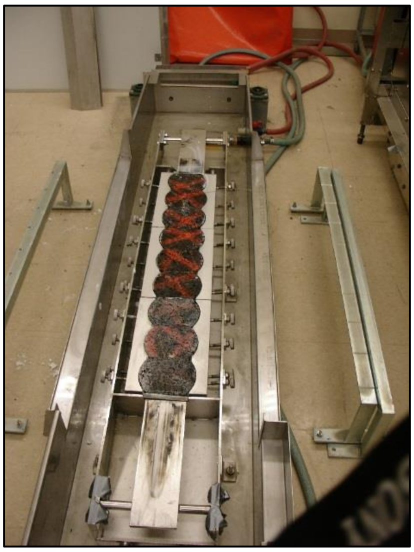

Figure 3.23: MMLS3 samples

3.6.2 Indirect Tensile Strength Test

For the design of wearing courses where expensive performance testing like the four point bending beam fatigue test is not warranted, the indirect tensile strength (ITS) test parameters coupled with the evaluation of binder durability, can be used to determine whether there is a risk of premature fatigue failure (CSIR & SABITA, 2001, p. 7-2).

The ITS test was done in accordance with ASTM D6931-07. Specimens for the ITS were prepared using Marshall compaction technique. The ITS test was carried out on four cylindrical samples, compacted to 4% air void content and partially saturated with water. Two samples were "conditioned" by freezing them for at least 15 hours and subsequently immersing them in a water bath set at 60 °C for 24 hours.

A cylindrical asphalt specimen was loaded on the diametral axis at a constant rate as shown in Figure 3.24 until a significant loss in applied load was noted. The peak load is used to calculate the ITS. The ratio of the ITS values of the conditioned and unconditioned samples, termed the tensile strength ratio (TSR), was used to assess the susceptibility to moisture damage. The results are presented, analysed and discussed in Section 4.5.2.

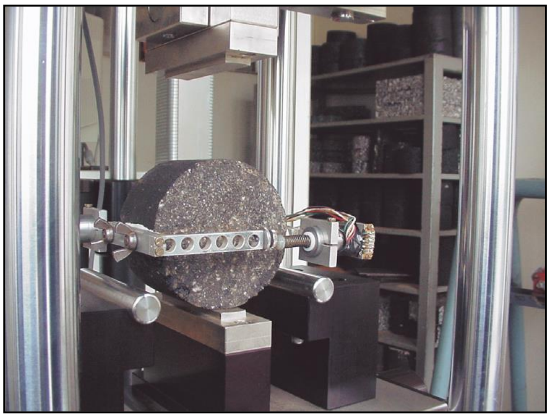

Figure 3.24: Sample positioned for ITS testing

3.6.3 Moisture sensitivity (Modified Lottman) test

This test was done in accordance to ASTM D4867. The Modified Lottman test (ASTM D4867) is generally regarded as the best test for evaluation of the stripping potential of an aggregate (NCHRP, 2011, p. 79; CSIR & SABITA, 2001, p. 8-5).

In this test, six cylindrical HMA specimens were compacted in the laboratory. Two of these were subjected to conditioning (vacuum saturation, freezing and thawing) while the other two were not conditioned (NCHRP, 2011, p. 9). Both sets of specimens were then tested using the indirect tension test. The percentage of strength retained after conditioning is called the tensile strength ratio (TSR), and is an indication of the moisture resistance of that particular mixture. The results are presented, analysed and discussed in Section 4.5.2.

3.7 Cost Analysis

The modified pavement costs was compared with conventional pavements on the basis of the present worth of the life cycle costs analysis (LCCA). The main economic factors used to determine the cost of the facility were the analysis period, structural design period, construction cost, maintenance costs and the real discount rate (COLTO, 1996, p. 67). The total cost of a project over its life was given by the construction cost plus maintenance cost plus road user costs minus the salvage value. The total cost was then expressed in terms of the present worth of costs (PWOC).

PWOC was used to determine the relative cost difference between pavement structures in the same road category and was calculated as follows (COLTO, 1996, p. 68):

$$PWOC = C + [M_1(1+r)]^{-X_1} + \cdots \ldots + [M_j(1+r)]^{-X_j} - S(1+r)^{-Z} \qquad (3.27)$$

Where:

PWOC = Present worth of costs

C = Present cost of initial construction

M_j = Cost of the j^{th} maintenance measure expressed in terms of current costs

r = Real discount rate

X_j = Number of years from the present for the j^{th} maintenance measure within the analysis period (where X_j = 1 to Z)

Z = Analysis period

S = Salvage value of pavement at the end of the analysis period expressed in terms of present values.

The LCCA was done based on the same road category with two scenarios: one built with conventional (unmodified) asphalt and one with optimum crumb rubber and LDPE modified asphalt concrete mix. An analysis period of 30 yrs was chosen for and the costs evaluated over this period (SANRAL, 2013, p. 3-11). An 8% discount rate was used for base case scenario however a sensitivity analysis with discount rates of 6%,8% and 10% was done (COLTO, 1996, p. 69). Salvage value at the end of the analysis period and road user costs over the project life was not considered in this case because they did not vary between different pavement types and therefore were not included in the calculation. The results, analysis and discussions are presented in Section 4.6.

4 RESULTS, ANALYSIS AND DISCUSSION

4.1 Introduction

This chapter presents the results, their analysis and the discussions that connects them to the specific objectives of this research. The significance of the experimental results, collected data and references to the outcome of the literature review will be blended in this chapter to connect the various experimental and other results to give credible answers to the objectives of this research.

4.2 Materials for Hot Mix Asphalt

Hot mix asphalt mixtures are composed primarily of aggregate and asphalt binder. Aggregates make up about 95% of a hot mix asphalt (HMA) mixture by weight whereas the asphalt binder makes up the remaining 5% (CSIR & SABITA, 2001, p. 4). By volume a typical HMA mixture is about 85% aggregate, 10% asphalt binder and 5% air voids. These components were tested, analysed and benchmarked to the standard specifications. The individual properties of the constituent materials were characterised and their influence on the overall performance of the HMA discussed.

4.2.1 Mineral aggregate physical properties

Pavement engineers have been able relate specific aggregate properties to HMA performance such as rutting, ravelling, fatigue cracking, skid resistance and moisture resistance (NCHRP, 2011, p. 28). The mineral aggregate physical properties test results and specifications are summarized in Table 4.1. The results reported for hardness and toughness are ACV, 10% FACT, AIV (Treton), and LAA tests. While for particle shape and texture are flakiness index and course aggregate fractured faces. Other physical properties tests included are grading, course and fine aggregate specific gravity, water absorption and cleanliness.

Table 4.1: Physical properties of Aggregates

Physical properties of crushed aggregate			
Properties	**Test Value**	**Specification** (CSIR & SABITA, 2001, pp. 3-3)	**Remark**
Aggregate grading	See Section 4.2.1.1	-	-
Crushing value, %	10	Maximum of : 25%: HMA base and surfacings 21%: Open-graded surfacings and SMA	Lower values indicate high resistance to crushing force
10% Fines Aggregate Crushing value, kN	400	Minimum of: 160 kN: HMA surfacings 210 kN: open-graded surfacings and SMA	Hard aggregate
Aggregate (Treton) Impact value, %	5.3	Maximum 25%	Lower values indicate high resistance to impact force
L.A Abrasion, %	10	10 %: very hard aggregate 60%: very soft aggregate	Very hard aggregate
Flakiness Index	See Section 4.2.1.5	HMA surfacings: 19 mm and 13.2 mm aggregate: 25 (grade 1*) or 30 (grade 2*) 9.5 mm and 6.7 mm aggregate: 30 (grade 1*) or 35 (grade 2*)	-
Fractured faces, %	99	HMA surfacings: >95%	Mechanically crushed aggregates
Sand equivalent, %	69.2	Minimum of: 50: total fines fraction 30: natural sand fraction to be mixed with aggregate	High sand equivalent value indicates low silt content
Density, g/cm^3 Coarse aggregates Fine aggregates	 2.673 2.619	Does not influence the suitability aggregate for HMA used but used in volumetric calculations.	Higher density materials are compact with less voids
Water absorption, % Coarse aggregates Fine aggregates	 0.44 0.7	 Maximum 1% Maximum 1.5%	Low absorption values indicate good quality aggregate
Sand Equivalent, %	69	Minimum 45%	

*As defined in TRH14 (currently under revision)

The mineral aggregate exceeds the minimum criteria for use in hot mix asphalt concrete mixture.

4.2.1.1 Mineral aggregate grading

The results of an aggregate sieve analysis in hot mix asphalt are usually presented as weight percent passing. The percent passing from the results of a sieve analysis is given in Table 4.2. This table shows the results of a sieve analysis of the aggregates, along with the calculations of percent retained on each sieve. The weight retained, as shown in Column 6, is the mass in grams of the aggregate separated onto each sieve. The total of these values is 1200 g (Commitee of Land Transport Officials (COLTO), 1986, p. 171).

Table 4.2: Grading of mineral aggregate used in this research

Sieve size (mm)	Specified Grading (SAPEM 2003 p. 4-26)	Adopted Gradation	Percentage		Mass per 1200g briquette
			Retained	Mass	
37.5	100	100	0	0	0
26.5	100	100	0	0	0
19	100	100	0	0	0
13.2	84-96	90	10	10	120
9.5	70-84	76	24	14	168
4.75	45-63	50	50	26	312
2.36	29-47	36	64	14	168
1.18	19-33	26	74	10	120
0.6	13-25	18	82	8	96
0.3	10-18	13	87	5	60
0.15	6-13	10	90	3	36
0.075	4-10	7	93	3	36
<0.075			100	7	84
					1200

Figure 4.1 shows the grading curve of the mineral aggregate used in this research with the upper and lower limits.

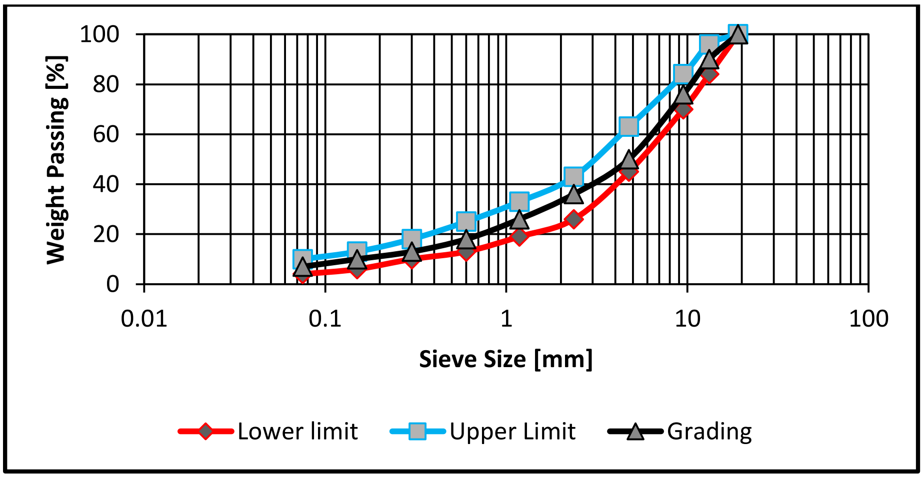

Figure 4.1: Mineral aggregate grading curve

The grading selected fits within the grading limit for continuously graded asphalt mix. Densely graded sand-skeleton mixes are the most commonly used mix type for low to high traffic volume designs (The South African National Roads Agency, 2001, p. 4-6). This design derives its stability from a sand skeleton which comprises of a densely graded mix with the spaces between the coarse aggregate particles filled with well-graded portions of finer aggregate.

4.2.1.2 Aggregate crushing value and 10% FACT

The ACV test is an assessment of the strength of aggregates to resist crushing under traffic wheel loads. A test result of 10% for ACV under 400 kN on dry material was obtained. This also doubles up as the ten percent FACT value. The 10% FACT value of 400 kN obtained is an indication of a high resistance of the aggregate material to crushing under a gradually applied load. The result shows that the aggregates are strong enough to resist crushing under traffic wheel loads.

4.2.1.3 Aggregate (Treton) Impact value (AIV)

The aggregate impact value (AIV) test is a relative measure of the resistance of an aggregate to sudden shock or impact. The test value of 5.3% indicates high resistance to impact of the aggregate as it is less than the maximum specification of 25%. This value shows that there was little disintegration which is an indication of a hard material.

4.2.1.4 Los Angeles Abrasion (LAA)

The aggregates LAA result of 10% is an indication of very hard aggregate. Hence this material is of good aggregate strength when subject to abrasive wear. This implies that the aggregate particles would not break during construction therefore ensuring that the asphalt concrete mix can be placed and compacted properly. Aggregate toughness and abrasion resistance have been shown to relate to the pavement performance. Aggregates with poor abrasion resistance polish under traffic action causing the pavement surface to lose skid resistance especially under wet conditions. The LAA test is not a standard test in South Africa because the correlation between LAA test results and aggregate performance in service appears to be poor. ASTM C131 recommends an LAA value of less than 30% for wearing course.

4.2.1.5 Flakiness Index

The flakiness index (FI) of a coarse aggregate is the mass of particles in the aggregate, expressed as a percentage of the total mass of the sample that will pass the slots of a specified width for the appropriate size fraction. The results shown in Table 4.3 indicate that aggregate sizes 37.5 mm, 26.5 mm, 19.0 mm, 13.2 mm and 6.5 mm are within the recommended limits of flakiness index. The 9.5 mm aggregate size exceeds the maximum FI value. This is an indication of the presence of aggregates with flat, thin and elongated particles. These aggregates may prevent proper compaction and may also have a tendency to break down during production and compaction of the asphalt concrete mix. This in turn reduces the durability of the asphalt pavement.

Table 4.3: Flakiness Index for various aggregate sizes

Flakiness Index [%]					
Sieve size (mm)	Width of slot (mm)	Mass passing (Kg)	Mass of sample (kg)	FI (%)	Maximum flakiness index (%)
37.5	19.0	0.707	5	14.1	30
26.5	13.2	0.511	4	12.8	30
19.0	9.5	0.828	3.5	23.7	30
13.2	6.7	0.839	0.5	8.2	30
9.5	4.8	0.390	1	39	35
6.7	3.4	0.041	0.5	8.2	35

4.2.1.6 Coarse aggregate fractured faces

The aggregates from AFRISAM crushing plant have over 95% of all particles having at least three fractured faces. Angular and rough textured aggregates are desirable within the asphalt mix as they resist permanent deformation and fatigue cracking. They also provide better interlock between aggregate particles which prevents plastic deformation (rutting). It also increases the pavement’s frictional properties which is an important safety consideration.

4.2.1.7 Density of coarse and fine mineral aggregates

The density of aggregates is used in asphalt mix design and analysis. The bitumen binder content, normally expressed as a percentage weight, is considered in proportion to the density of the aggregates. The density is used to calculate the void contents in asphalt mixes, a very important parameter in mix design and evaluation. The calculated densities of coarse and fine aggregates are 2.673 g/cm^3 and 2.619 g/cm^3 respectively.

4.2.1.8 Water absorption of coarse and fine mineral aggregates

The water absorption value is the amount of water absorbed into the permeable voids of the aggregate. The calculated absorption values of 0.44 % for coarse aggregates and 0.7% for fine aggregates is lower than the maximum limits of 1% and 1.5% respectively. This indicates that the mineral aggregate have a low water absorption capacity. The mineral aggregate was therefore not porous. Porous aggregates have high water absorption and this leads to stripping of the asphalt binder from the aggregate. Porous aggregate can continue to absorb binder long after construction which could lead to apparent binder hardening (SABITA, 2005, p. 26).

4.2.1.9 Sand equivalent test

The sand equivalent test shows the presence or absence of detrimental fines or clay-like materials in the aggregates. The test value of 69.2% for sand equivalent was obtained for the crusher sand material passing 4.75 mm test sieve. The high sand equivalent value is an indication that the aggregates are relatively free of dust and clay particles. Minimum values for sand equivalency are usually specified with higher values indicating the desirable state. Very fine clay-like particles can affect the adhesion between the asphalt binder and aggregate particles leading to moisture susceptibility.

4.2.2 Rubber aggregates

Ground rubber tyre of sieve size fraction passing the 4.75 mm and retained on the 2.36 mm test sieve was used for this experiment. Figure 4.2 shows the crumb rubber material used in this research experiment.

Figure 4.2: 2.36 mm crumb rubber

4.2.3 Bitumen characterisation

The bitumen characteristics used in this study are discussed below.

a) Density

The density of bitumen varies depending on the crude oil source. The density does not influence the suitability of bitumen for HMA but is used in volumetric calculations.

b) Penetration

The bitumen used in this research was penetration grade 60/70. This bitumen grade has a penetration range of 60-70 dmm. The sample tested gave a penetration value of 68 dmm which is within the specified range. High penetration grade is desirable in colder regions. Penetration below 20 will result in cracking. For lower penetration grades the bonding of the binder with the aggregate is difficult to achieve but once achieved, the bond lasts a long time.

c) Softening point

The softening point is an indication of the capacity of the bitumen to perform adequately at high in-service temperatures. The sample giving a value of 44.4 is within the standard specifications of this bitumen grade. Higher values indicate greater susceptibility to rutting while lower values greater susceptibility to fatigue cracking. Bitumen with higher softening point is used for warmer places.

d) Ductility

The ductility improves the physical interlocking of aggregate bituminous mixes. Binder material with low ductility would crack under traffic loading therefore becoming a pervious pavement surface.

e) Dynamic viscosity

Dynamic viscosity value is used to assess consistency at high in-service and application temperatures. This is used to determine the correct pumping, spraying, mixing and compaction temperatures. The dynamic viscosity value at 60 ˚C of 193.7 Pa.s (Pascal second) is within the specified range. Lower values indicate greater susceptibility to rutting of the mix during service of the road pavement.

f) Rolling Thin Film Oven Test Aging

RTFO test exposed the bitumen to simulated short term ageing and hardening due to the effect of heat and oxidation in the presence of air as in the hot mix asphalt manufacturing process. A change in mass of 0.23% which is less than the specified limit of 0.5% is an indication of acceptable loss of volatiles. The retained penetration value of 58 dmm is greater than the minimum specification for grade 60/70 pen grade bitumen. Lower penetration values indicate susceptibility to thermal cracking. The increase in softening point by 5.4 ˚C is below the maximum allowable value of 9 ˚C. Higher values indicate low durability of the binder. The change in dynamic viscosity at 60 ˚C is 51.45% of the original. This is lower than the specified maximum of 300%. This is an indication of a durable binder.

Table 4.4 shows the standard tests results and specifications for bituminous binder used in this study.

Table 4.4: Properties of 60/70 bitumen binder

Bitumen Test	Standard Test Method	Bitumen grade 60/70	Standard Specification (SABS 307-1972)	Interpretation
a) Density (g/cm^3)	ASTM D70-97	1.03	1.00-1.05	
b) Penetration at 25˚C, (dmm)	ASTM D5-86	68	60-70	High value indicates higher consistency and greater susceptibility to rutting of the mix
c) Softening point, R&B (˚C)	ASTM D36-70	44.4	46-56	This is the temperature at which the bitumen becomes more plastic
d) Ductility at 25˚C (cm)	ASTM D113-86	67	50-100	Measure of bitumen deformation under tensile stress
e) Dynamic Viscosity at 60˚C (Pa.s)	ASTM D4402-91	193.7	120-250	Indicates stability. Lower values indicate greater susceptibility to rutting of the mix
Properties after RTFOT				
i) Change in mass (% of original)	ASTM D 2872	0.23	0.5% (max)	Provides a quantitative measure of loss of volatiles when bitumen is exposed to elevated temperatures during manufacturing and placement of asphalt mixes.
ii) Retained penetration at 25˚C, (dmm)	ASTM D5-86	58	55 (min)	Indication of stability. Higher values may indicate greater susceptibility to rutting of the mix
iii) Increase in softening point, R&B (˚C)	ASTM D36-70	5.4	9 (max)	Values close to or higher than the upper may indicate low durability of the binder
iv) Ductility at 25˚C (cm)	ASTM D113-86	57	50 (min)	Lower values indicate susceptibility to fatigue cracking
v) Change in Dynamic Viscosity at 60 ˚C (% of original)	ASTM D4402-91	51.45	300 (max)	Values close to or higher than the upper may indicate low durability of the binder

4.2.4 LDPE modified bitumen

The following properties of LDPE modified bitumen are presented and discussed: penetration, softening point, ductility, dynamic viscosity and durability.

a) Penetration

Figure 4.3 shows the effect of addition of plastic waste in 60/70 bitumen. It is observed that the penetration value decreases with an increase in the amount of plastic in the bitumen. Modified binder with greater than 5% LDPE content have penetration values falling outside the allowable range of the 60/70 penetration grade bitumen. This can be attributed to the fact that LDPE has a higher melting point than bitumen. The LDPE tends to stiffen the bitumen therefore increasing its consistency. Al-Hadidy et al., (2009, p.1456) also found out that penetration at 25˚C will generally decrease as LDPE content increases which indicates an improved shear resistance in medium to high temperatures. Higher penetration values indicate greater susceptibility to rutting of the asphalt mix (CSIR & SABITA, 2001, p. 3-14). A regression analysis of this data as shown in Figure 4.3 gives a linear function with R^2 of 0.892 which indicates a good fit.

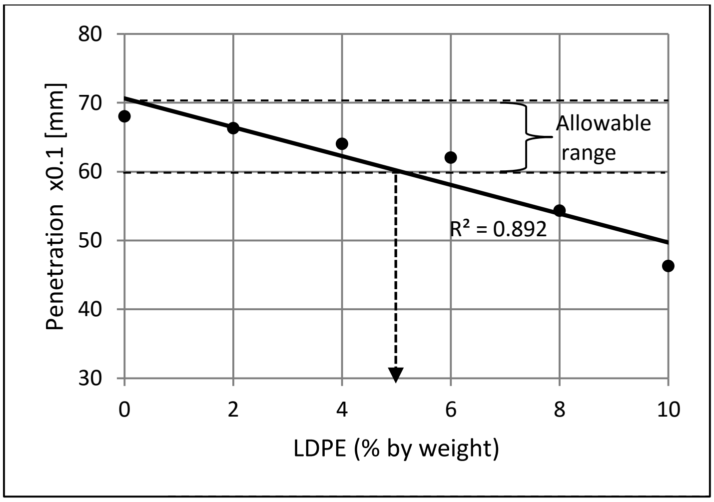

Figure 4.3: Influence of LDPE on penetration

b) Softening point

Figure 4.4 shows the influence of LDPE on the softening point of 60/70 bitumen. It is observed that the softening point of the bitumen increases with an increase in the plastic content. The softening point values increases with an increase in plastic content. It is evident that addition of 1.5 to 7% LDPE by weight will result to an acceptable range of penetration. Higher softening point values is an indication of improvement in resistance to deformation of the asphalt mix (Al-Hadidy, et al., 2009, p. 1463). Thus inclusion of LDPE improves the asphalt mixture's resistance to permanent deformation. Asphalt with high softening point make mixtures with better permanent deformation resistance (Fontes, et al., 2010, p. 1198). A regression analysis of this data as shown in Figure 4.4 gives a linear function with R^2 of 0.9397 which indicates a good fit.

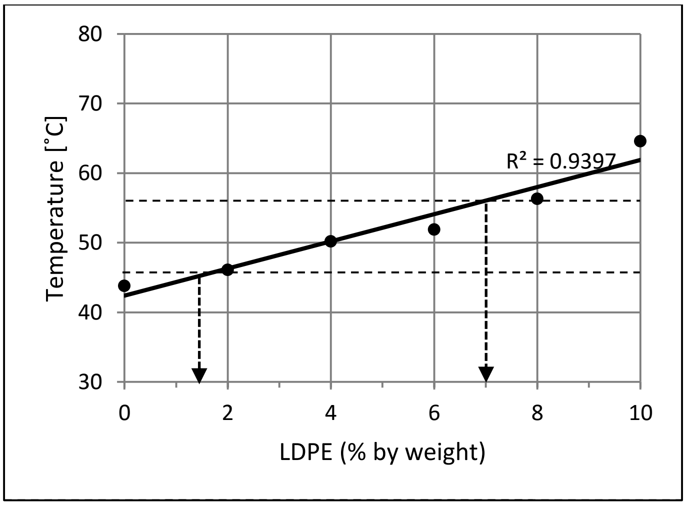

Figure 4.4: Influence of LDPE on softening point

c) Ductility

Figure 4.5 shows the influence of LDPE on ductility of 60/70 bitumen. It is observed that the ductility decreases with an increase in the plastic content in the bitumen. The results of all the LDPE modified binders tested were within the specification range of 50-100 cm. This test has limited use since it is empirical and conducted only at one temperature (25 ˚C). A regression analysis of this data as shown in Figure 4.5 gives a linear function with R^2 of 0.8906 which indicates a good fit.

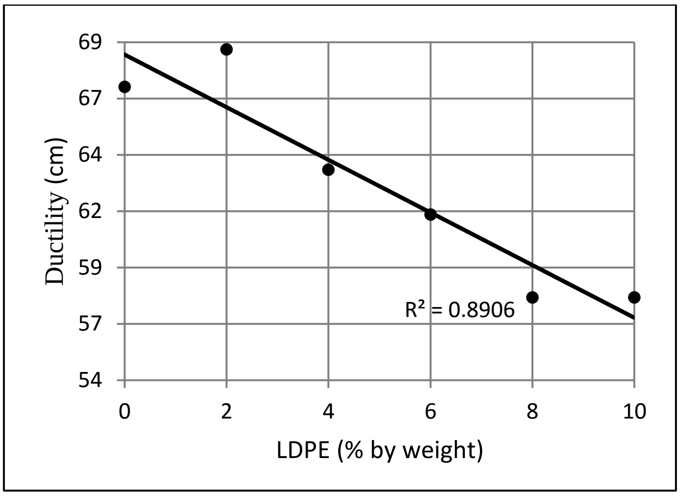

Figure 4.5: Influence of LDPE on ductility

d) Dynamic viscosity

Figure 4.6 shows the effect of the addition of LDPE on the dynamic viscosity of 60/70 bitumen. It is observed that dynamic viscosity of the bitumen increases with an increase in plastic content. The dynamic viscosity of the modified binder with up to 6% LDPE is within the specified range. Higher viscosity values is an indication of stability whereas lower values indicate greater susceptibility to rutting of the mix. A regression analysis of this data as shown in Figure 4.6 gives a linear function with R^2 of 0.9975 which indicates a good fit.

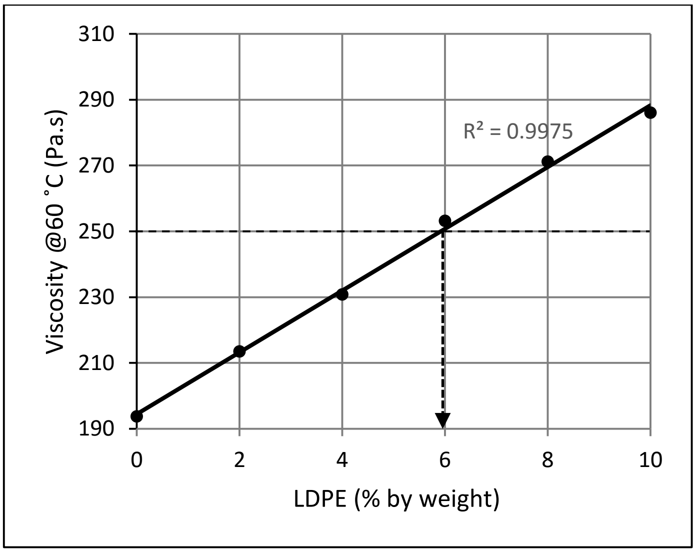

Figure 4.6: Influence of LDPE on dynamic viscosity at 60˚C

e) Durability test

Figure 4.7 shows the influence of LDPE on the short term ageing. It shows that there is a reduction in percentage of loss in mass in the modified binders with an increase in LDPE content in the bitumen (Al-Hadidy, et al., 2009, p. 1460). There is a reduction in loss of mass from 0.23% in conventional binder (0% LDPE) to 0.08% for 10% LDPE binder. This indicates improved durability of the binder due to decreased loss in volatiles during production and application of the asphalt concrete mix. The ageing of binders leads to increased stiffness through excessive loss of the lighter fraction of bitumen. Excessive age hardening of the binder during service life of the pavement contributes to several pavement distresses such as ravelling, top-down fatigue cracking, thermal cracking and moisture damage (NCHRP, 2011, p. 69)

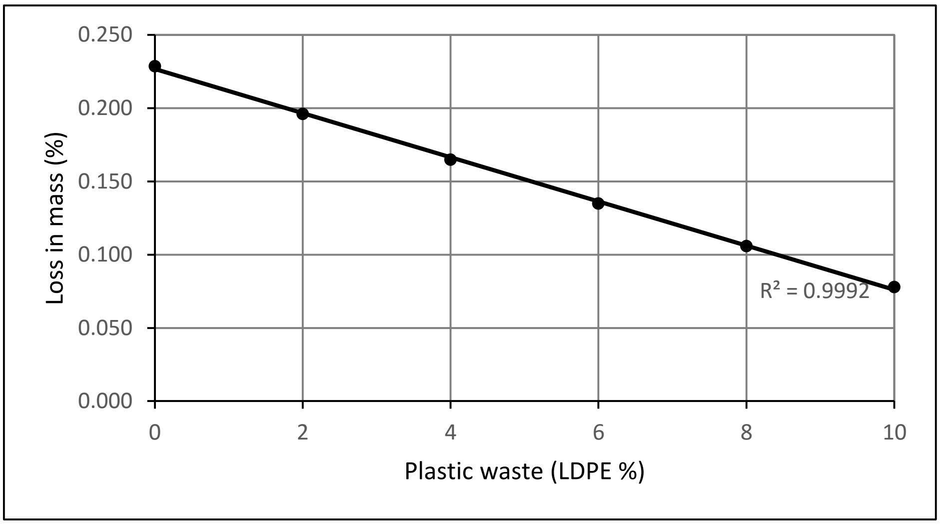

Figure 4.7: Influence of LDPE on bitumen mass loss

4.3 Determination of optimum binder content of control HMA

At the beginning of the experimentation Marshall briquettes of the control mix i.e. zero percent LDPE and crumb rubber, were made at 4.5, 5, 5.5, 6 and 6.5% bitumen content by weight of the total mix. Three samples at each bitumen content were tested for Marshall stability and flow. Density and void analysis were also done on the samples. The results are reported in Table 4.5.

Table 4.5: Summary of Volumetric and Marshall data for 60/70 penetration grade bitumen.

Binder content %	*Stability (kN)*	*Flow (0.25mm)*	*Bulk Density (g/cm^3)*	*Air voids %*	*VMA %*	*VFB %*
4.5	8.85	3.10	2.452	6.71	16.82	59.74
5.0	9.97	3.26	2.473	5.45	16.21	65.88
5.5	10.61	3.71	2.485	4.62	15.85	70.35
6.0	10.40	3.98	2.489	3.66	15.40	75.79
6.5	8.94	4.22	2.482	4.21	14.75	70.96

Figure 4.8 to 4.13 show the results of the Marshall mix design of the control HMA. A bitumen content giving 4 percent air voids was chosen as the design bitumen content. The optimum binder content was selected as the average binder content for maximum density, maximum stability, 2-4 mm flow and 4% air voids in total mix (COLTO, 1998). The air void content of 4% is adequate to permit small amount of compaction under traffic load without bleeding and loss of stability but also provides adequate mix stability to prevent unacceptable deformation under traffic loading (Department for International Development, 2002, p. 19). This design allowed the bitumen content parameter to be fixed as other asphalt concrete mix parameters were investigated in Section 4.4.

In Figure 4.8 density increases with increasing asphalt content, reaching a maximum of 2.489 g/cm^3 at 6% binder content, then starts to decrease. Peak density usually occurs at higher asphalt binder content than peak stability. In Figure 4.9 stability increases with increasing asphalt binder content, reaching a peak of 10.61 kN at 5.5% binder content, then decreases. In Figure 4.10 the voids in mineral aggregate decreases with an increase in the binder content. This can be attributed to the spaces between aggregate particles in the mix being filled with the binder. In Figure 4.11 flow increases with increasing asphalt binder content. In Figure 4.12 the percentage air voids should decrease with increasing asphalt binder content. A high air void content increases permeability to air and water resulting in significant moisture damage and rapid age hardening (NCHRP, 2011, p. 47). Low air void content may result to high asphalt binder content resulting in a mixture prone to rutting and shoving.

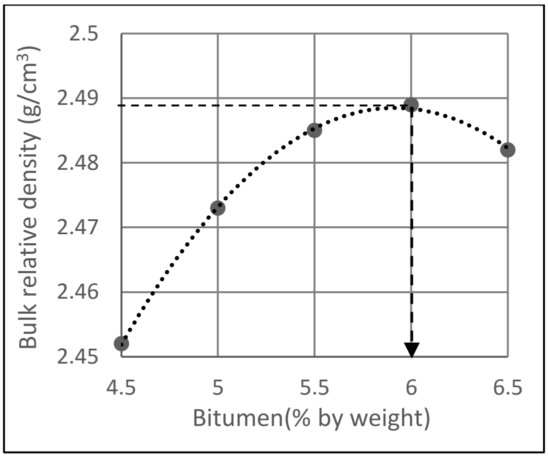

Figure 4.8: Bitumen against Bulk density

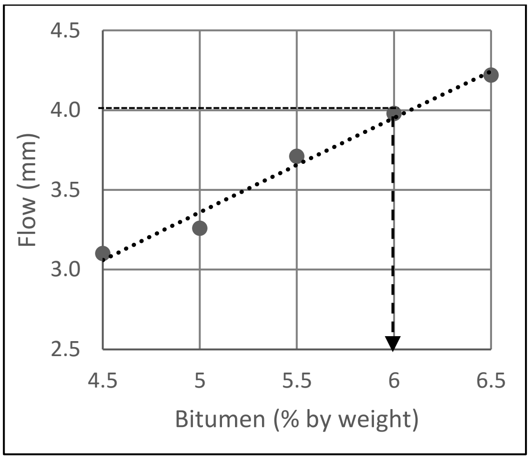

Figure 4.11: Bitumen against Flow

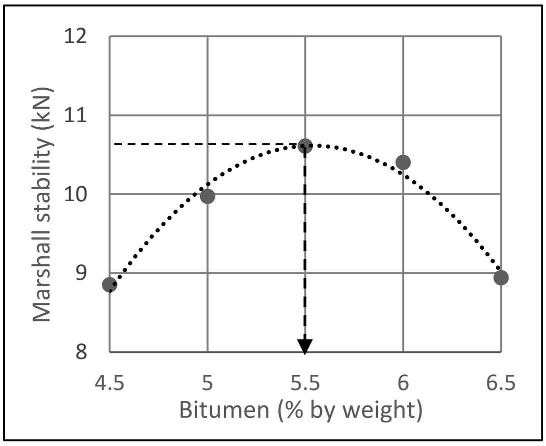

Figure 4.9: Bitumen against Marshall stability

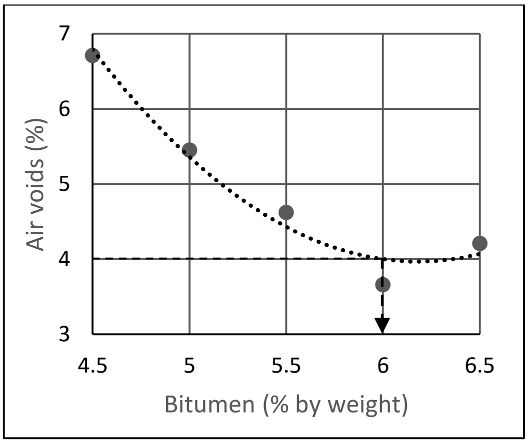

Figure 4.12: Bitumen against Air voids

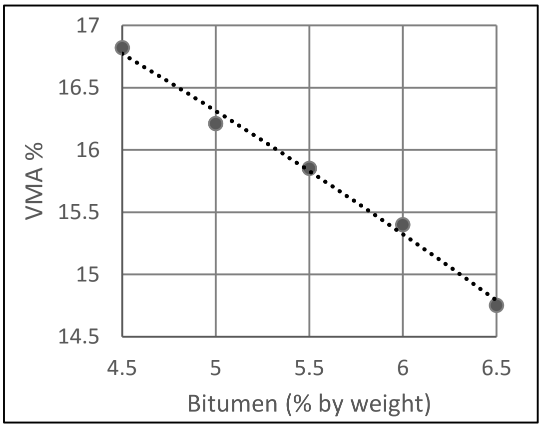

Figure 4.10: Bitumen against VMA

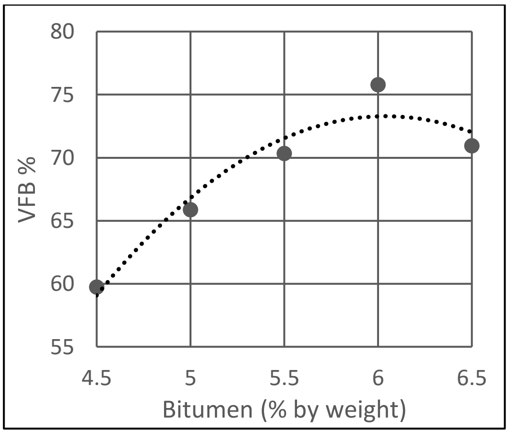

Figure 4.13: Bitumen against VFB

In Figure 4.10 percentage VMA decreases with increasing asphalt binder content. The minimum VMA required for 13.2 mm nominal maximum stone size at 4.0 % voids in mix is 11 % (Department for International Development, 2002, p. 19). This is to ensure that the compacted mineral aggregate mix has a void content large enough to contain sufficient bitumen. In Figure 4.13 the voids filled with binder (VFB) percentage increases with increasing asphalt binder content. VFB is one of the volumetric parameters with the strongest relationship to rutting performance. A void structure overfilled with binder tends to lubricate aggregates thereby reducing frictional resistance resulting in an increase in rutting potential.

The design binder content was calculated from the mean value of the binder contents for maximum stability, maximum density, the mean value for the specified range of void contents and the mean value for the specified range of flow values. Values of 6%, 5.5%, 6% and 6% bitumen are read off from Figures 4.8, 4.9, 4.11 and 4.12 respectively giving an average optimum binder content of 5.9%. This value was rounded up to 6% to cater for asphalt binder that may be absorbed by the more porous crumb rubber aggregates in modified asphalt concrete mixtures. This mix design ensures adequate amount of asphalt binder for durability of the pavement as well as workability during production and construction.

4.4 Optimization

Table 4.6 is a summary of the Marshall test results carried out on modified asphalt concrete mix briquettes. The table shows results of bulk relative density, Marshall stability, flow and Marshall quotient of the briquettes prepared for the study. These properties were used to determine the optimum proportions of LDPE and crumb rubber for the asphalt concrete mix. The optimal proportions were then used in preparation of the modified asphalt concrete specimen used in subsequent performance tests.

Table 4.6: Summary of the effect of crumb rubber and waste polyethylene on HMA

Crumb Rubber Content (%)	LDPE content (%)	Bulk Density (g/cm^3)	Marshall Stability (kN)	Flow (mm)	Marshall Quotient
0	0	2.564	6.87	3.26	2.107
	2	2.567	7.05	3.60	1.958
	4	2.557	7.75	4.20	1.844
	6	2.559	8.53	4.47	1.908
	8	2.551	8.06	4.96	1.625
	10	2.549	8.30	5.59	1.485
1	0	2.534	7.09	4.45	1.593
	2	2.543	7.92	4.55	1.741
	4	2.536	8.08	4.82	1.676
	6	2.539	7.84	4.98	1.574
	8	2.526	7.18	5.20	1.381
	10	2.525	7.67	5.74	1.336
2	0	2.527	7.30	5.64	1.294
	2	2.528	7.90	5.79	1.364
	4	2.518	8.76	6.26	1.399
	6	2.503	8.71	6.47	1.346
	8	2.486	7.99	6.60	1.211
	10	2.501	7.98	6.96	1.147
3	0	2.483	6.76	6.50	1.040
	2	2.493	7.41	6.87	1.079
	4	2.479	7.46	6.99	1.067
	6	2.468	7.58	7.09	1.069
	8	2.483	7.94	7.30	1.088
	10	2.489	8.17	7.46	1.095
4	0	2.446	5.94	7.25	0.819
	2	2.443	6.19	7.46	0.830
	4	2.449	6.96	7.58	0.918
	6	2.470	7.09	7.76	0.914
	8	2.454	7.09	7.94	0.893
	10	2.453	6.84	8.17	0.837
5	0	2.377	5.68	8.41	0.675
	2	2.427	5.79	8.53	0.679
	4	2.432	5.39	8.75	0.616
	6	2.468	6.30	8.92	0.706
	8	2.453	6.12	9.08	0.674
	10	2.446	6.26	9.30	0.673

4.4.1 Bulk relative density

Column 3 of Table 4.6 shows the effect of waste LDPE and crumb rubber on the bulk relative density of the Marshall briquette. Figure 4.14 indicates a decrease in the bulk relative density of the asphalt mix containing 0%, 1% and 2% crumb rubber with increasing percentages of waste LDPE in the mix. This can be attributed to the relatively low density of crumb rubber and waste plastic as compared to the bitumen (Siddique & Naik, 2004, p. 568; Ahmadinia, et al., 2011, p. 4847). The bulk relative density of the mixture remains relatively constant for the mixture with 3% crumb rubber even with an increase in LDPE content. There is an increase in the bulk relative density of the asphalt mixes containing 4% and 5% crumb rubber with an increase in LDPE content. Ahmadinia, et al (2011, p. 4847) notes that regardless of the waste plastic bottles (PET) content, the bulk relative density of the PET-mixture was lower than that of the control mix

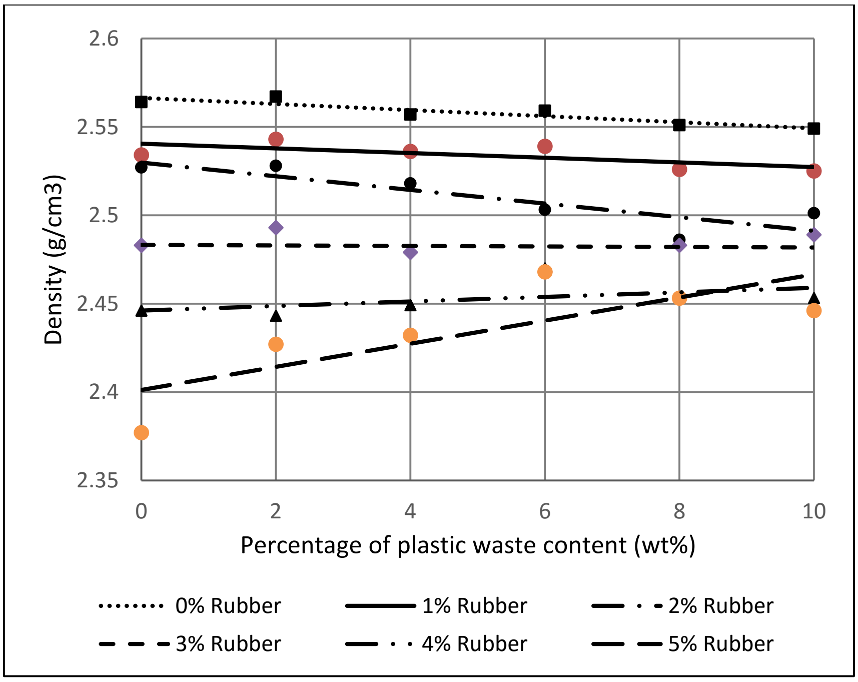

Figure 4.14: Influence of plastic waste (LDPE) on density of modified HMA

There is a reduction in density of the mixtures with increase in the crumb rubber content. This can be attributed to the springy effect of the rubber material therefore the briquette does not achieve the desired compaction with higher percentages of crumb rubber. Poor compaction results to higher voids in the surfacing pavement layer.

Figure 4.15 indicates the effects in the relative densities of the modified asphalt mixture with varying crumb rubber contents. It is evident that there is a decrease in the densities with an increase in the crumb rubber content in the mixtures for all LDPE modified samples. The density decreased for all LDPE content (Nuha, et al., 2013, p. 5).

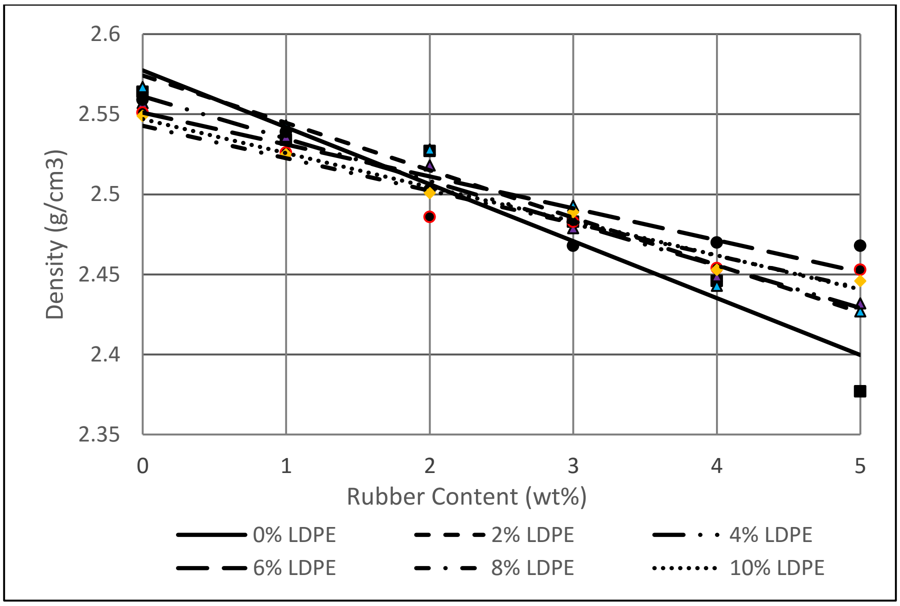

Figure 4.15: Influence of crumb rubber on density of modified HMA

4.4.2 Marshall stability

Column 4 of Table 4.6 shows the influence of LDPE on Marshall stability with variation crumb rubber content. The data shows an increase in the Marshall stability of the compacted briquette with an increase in percentage of polyethylene and rubber content in the mix. However there is a decrease in the stability values with specimens containing more than 2% crumb rubber. The optimal polyethylene content is at 6% giving a maximum Marshall stability value of 8.76 kN. The results show that there is impoverishment of quality of mixes with crumb rubber contents higher than 2%. This loss of quality occurred due to a decrease in mix density, and air voids. After reaching the optimum content there is a decline behaviour for both polymers (Rahman, et al., 2013). Rokade (2012) reported an increase in Marshall stability values of 10.5 kN, 11.2 kN and 11.85 kN for 3%, 6% and 9% of modifier (LDPE) respectively.

Figure 4.16 shows the influence of addition of LDPE and crumb rubber on Marshall stability. The mixture with no LDPE exhibits the lowest Marshall stability for all percentages of rubber in the mix. This shows that the LDPE in the HMA mixtures effectively improves the stability of the mixtures irrespective of the crumb rubber content. There is a significant improvement in stability up to a limit of 2% crumb rubber content beyond which the stability values begin to dip for all LDPE modified samples. Cross-linking agents such as sulphur present in the rubber is known to help improve the stability of polymer-bitumen compositions (Kalantar, et al., 2012, p. 59). All the Marshall stability values at 5% crumb rubber are lower than that of the initial mixtures i.e. 0% crumb rubber. The maximum stability value of 8.86 kN is obtained at 2% crumb rubber and 4% LDPE. Ahmadinia et al. (2011, p. 4847) records an increase in stability value until a maximum at 6% PET content, after which it decreased. Isacsson and Lu (1999) and Lu and Isacsson (2001) observed phase inversion in polymer concentration higher than 6%. Hinisliglu and Agar (2004) found that the optimum result for Marshall stability, Marshall quotient and flow happened in the binder with 4% HDPE. Higher stability values translates to higher HMA pavement performance.

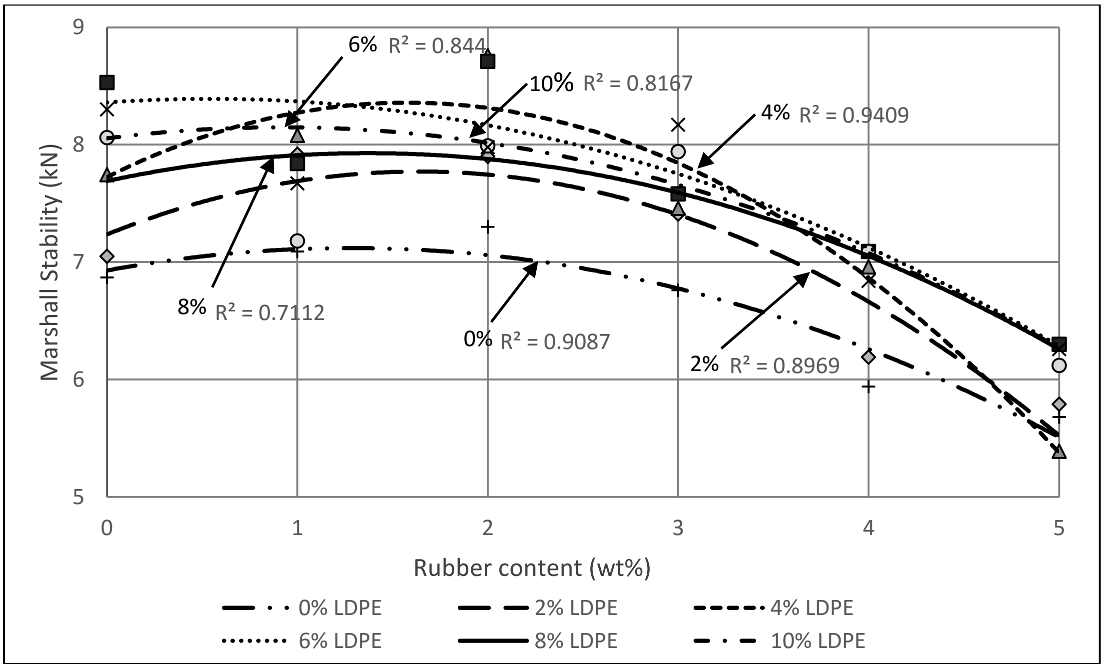

Figure 4.16: Influence of crumb rubber content on Marshall stability with variation in LDPE

From Figure 4.17 it can be noted that curves of mixtures modified with 1, 2 and 3% crumb rubber shows an improvement in the Marshall stability. However 4 and 5% crumb rubber content curves exhibit lower stability values than the control sample irrespective of the LDPE content in the mix. 5% rubber content has the lowest stability values. It can be attributed to the springy effect of the rubber material resulting to higher air voids in the sample. This results to water ingress into the briquette resulting in poor binding of the mixture. It is also important to note that all mixtures containing LDPE shows an increase in Marshall stability values irrespective of the crumb rubber content. This is an indication that LDPE has potential in improving the strength of the asphalt concrete mix

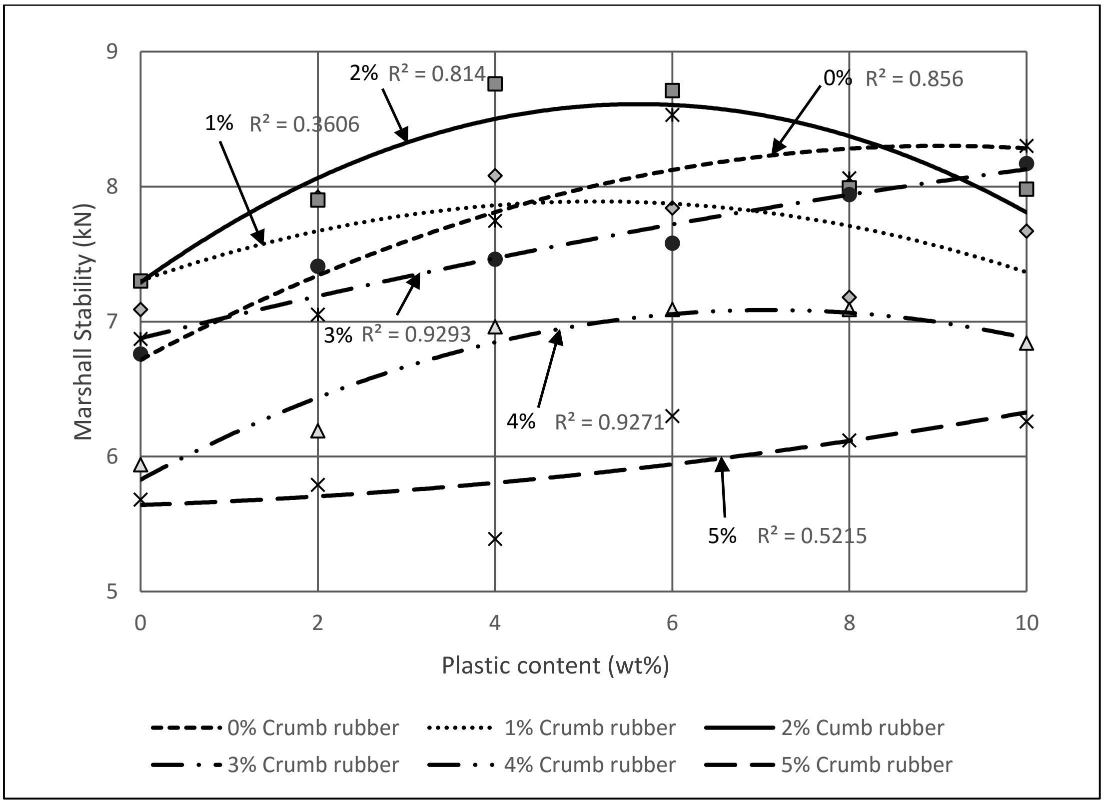

Figure 4.17: LDPE content against Marshall stability with variation in crumb rubber contents

4.4.3 Marshall flow

Figure 4.18 shows an increase in Marshall flow with an increase in the LDPE and crumb rubber. The linear regression lines are characterised by high correlation coefficient, R^2. The lines are almost parallel to each other showing that both LDPE and crumb rubber increases the flow of the mix proportionally. The higher flow values indicates high flexibility, i.e. the ability of an HMA pavement to adjust to gradual movements in the pavement layers without cracking (Hinislioglu & Agar, 2004). The highest flow values are exhibited by 5% crumb rubber and 10% LDPE content. This is an indication that both crumb rubber and waste LDPE increases the flow of the bituminous mix with a potential of reducing cracking in HMA pavements. This potentially increases chances of the pavement to rut under loading and increased temperature. Ahmadinia et al. (2011, p. 4847) reports an increase in the PET causes the flow value to decrease slightly until 4% of PET and then it changes to an upward movement.

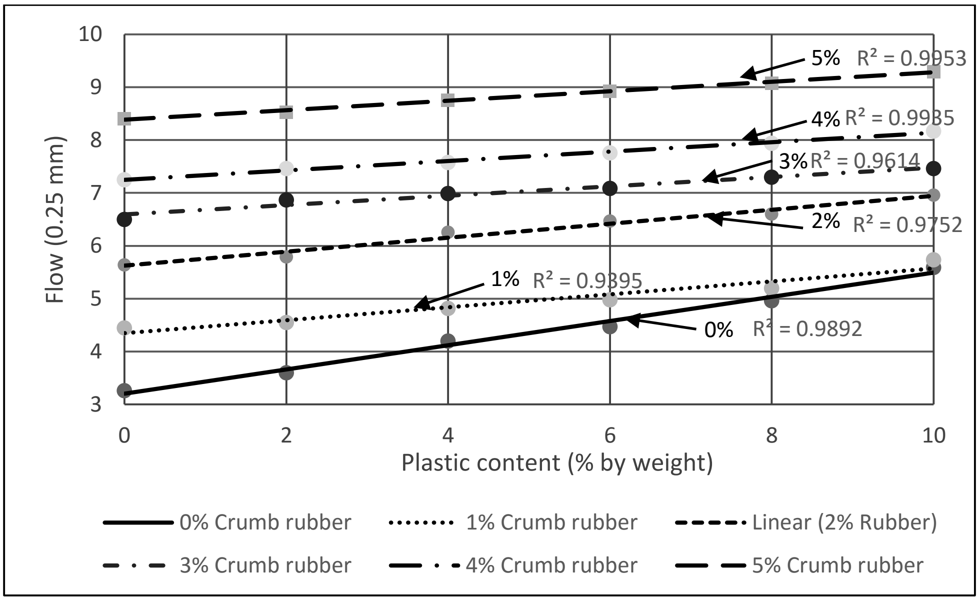

Figure 4.18: Influence of crumb rubber and LDPE on Marshall Flow

4.5 Modified HMA Performance

Performance tests were done on asphalt concrete mix samples containing the optimum proportions of crumb rubber and LDPE waste. The optimum plastic waste (LDPE) and crumb rubber contents were determined based on rheological properties and Marshall stability. The asphalt mixture specimen had 2% crumb rubber passing the 2.36 mm standard sieve as partial replacement of mineral aggregates and 4% waste polyethylene modified bitumen binder.

4.5.1 MMLS3 test

Table 4.7 is a summary of the MMLS3 test parameters carried out on densely graded asphalt mix specimen with 2% crumb rubber passing the 2.36 mm standard sieve as partial replacement of mineral aggregates and 4% waste polyethylene modified bitumen binder. Resistance to rutting depends on factors such as volumetric composition, aggregate and binder characteristics. The wheel tracking test is one of the most direct means of assessing this effect on the performance of mixes (Jain, et al., 2011, p. 237)

Table 4.7: MMLS3 Test parameters

Rubber and Plastic Modified Design	
Load per Wheel	2.7 kN
Tyre Pressure	690 kPa
Asphalt temperature at 25 mm depth	50ºC ±2ºC
Load repetitions per hour	7200
Wet/dry test	Dry
Material Type	Modified

A cross section of the surface rutting of the test sample is shown in Figure 4.19. The effect of the channelization of traffic can be seen. The maximum rut depth at the channelized section after 100,000 repetitions is 9.81 mm. Chen et al., (2003, p.598) points out that rut depth decreases with increasing polymer percentage. Sengul (2013, p. 782) concludes that Styrene-butadiene-styrene (SBS) polymer modified asphalt mixtures gives at least double resistance for 50,000 cycles with wheel tracking test.

Figure 4.19: Deformation of slab after wheel tracking test

The rut depth of a number of load repetitions is defined as the maximum vertical downwards deformations in the wheel track. The upward deformation next to the wheel track is defined as the heaving. It occurs when material from the path of the wheel is pushed outwards to the side of the wheel, either through creep or classical shear. The results showing the heaving of the control and modified samples heaving on the right and left sections are shown in Table 4.8 toTable ***4.11***. The deformation per width interval measured is shown in Figure 4.20.

Table 4.8: Control sample cumulative right heave

Right Heave (control sample)							
Position	0	2500	5000	10000	25000	50000	100000
100	0	0.43	0.65	0.32	0.63	0.55	0.69
200	0	0.45	0.45	1.25	1.12	2.47	2.87
300	0	0.48	0.84	0.67	1.87	3.19	0.57
400	0	0.36	0.60	0.39	0.65	0.85	0.98
500	0	0.42	1.02	0.98	2.37	0.75	5.09
600	0	0.42	0.92	0.78	1.65	0.54	2.99
700	0	0.48	0.17	1.46	1.76	0.74	0.73
800	0	0.23	0.36	0.54	0.86	0.60	0.65
900	0	0.82	0.41	1.12	0.61	0.71	0.65
Avg	0	0.45	0.60	0.83	1.28	1.16	1.69
Stdev	0	0.16	0.28	0.39	0.65	0.97	1.60

Table 4.9: Control sample cumulative left heave

Left Heave (control sample)							
Position	0	2500	5000	10000	25000	50000	100000
100	0	0.56	0.69	0.95	1.05	2.12	2.19
200	0	0.64	0.52	0.37	1.91	1.30	0.32
300	0	0.63	0.60	1.60	0.88	0.54	0.18
400	0	0.78	0.28	0.81	1.06	0.64	0.87
500	0	0.19	1.60	0.65	1.43	0.87	0.63
600	0	0.54	0.77	1.36	0.64	1.03	2.70
700	0	0.34	0.81	0.78	1.86	1.23	0.36
800	0	0.90	0.72	0.43	0.32	0.80	0.23
900	0	0.25	0.65	0.78	0.73	2.60	0.42
Avg	0	0.54	0.74	0.86	1.10	1.24	0.88
Stdev	0	0.24	0.36	0.40	0.54	0.69	0.92

Table 4.10: Modified sample cumulative right heave

Right Heave (Modified Sample)							
Position	0	2500	5000	10000	25000	50000	100000
100	0	0.42	0.41	0.57	0.59	0.44	0.59
200	0	0.41	0.42	0.61	2.19	2.47	3.34
300	0	0.47	1.11	1.35	2.38	0.40	0.15
400	0	0.15	0.50	0.59	0.24	0.60	0.50
500	0	0.28	1.09	1.50	3.09	0.72	4.33
600	0	0.67	0.75	0.87	1.97	0.69	3.10
700	0	0.48	0.17	1.59	0.76	0.65	0.04
800	0	-0.10	0.18	0.39	0.43	0.70	0.62
900	0	0.36	0.56	0.67	0.54	0.43	0.65
Avg	0	0.35	0.58	0.90	1.35	0.79	1.48
Stdev	0	0.22	0.35	0.45	1.05	0.64	1.63

Table 4.11: Modified sample cumulative left eave

Left Heave (Modified sample)							
Position	0	2500	5000	10000	25000	50000	100000
100	0	0.49	0.39	0.39	1.15	1.04	1.70
200	0	0.45	0.48	0.73	0.19	0.20	0.19
300	0	0.46	0.69	1.75	0.66	0.56	0.61
400	0	0.42	0.40	0.18	0.96	1.02	1.73
500	0	0.44	0.59	0.59	0.67	0.61	0.63
600	0	0.45	0.67	0.77	1.83	1.76	2.27
700	0	0.76	1.18	1.41	2.29	2.21	0.30
800	0	0.07	0.40	0.43	0.58	0.82	0.10
900	0	0.40	0.59	0.53	0.73	1.06	0.15
Avg	0	0.44	0.60	0.75	1.01	1.03	0.85
Stdev	0	0.17	0.25	0.51	0.66	0.62	0.82

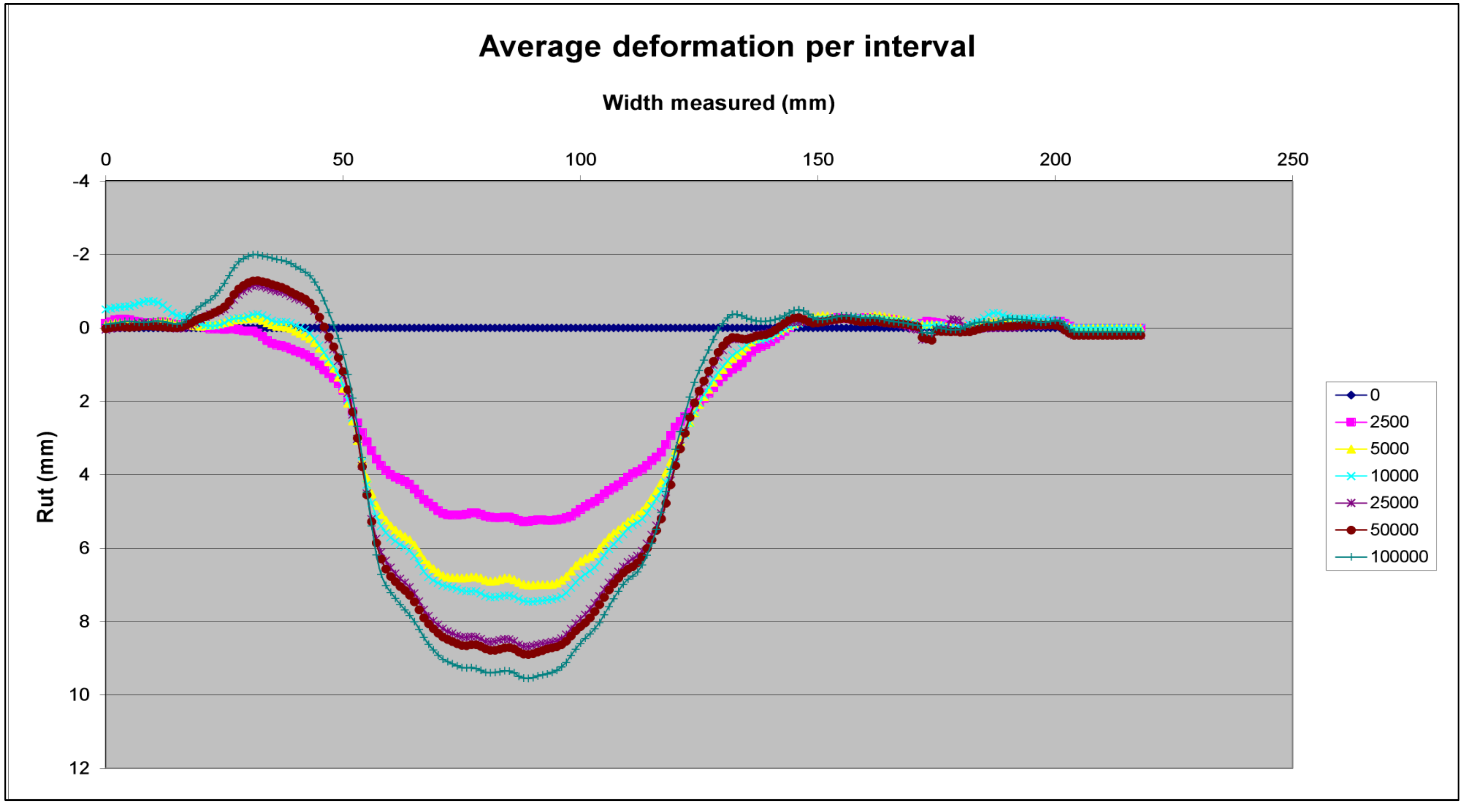

Figure 4.20: Cross-section of the pavement deformation

Figure 4.21**Error! Reference source not found.** shows the cumulative heave for both the control sample and the modified sample. It indicates that there is less heaving on the modified sample as compared to the control sample. This is an indication of reduced rutting due to the repeated load cycles. These results further indicate that the rutting took place mainly during the setting in phase after which the rut rate decreased. (Fontes, et al., 2010, p. 1197) A secondary settling in phase is normally observed when the temperature or wheel load is increased during the test (Rust, et al., 1994, p. 205). This is then usually followed by another flattening off of the rut curve.

Figure 4.22**Error! Reference source not found.** shows the average cumulative rut for the modified and control samples. The rutting behaviour observed in can be summed up in three phases (Rust, et al., 1994, p. 204):

Phase I: A setting in phase between 0-5000 wheel load repetitions during which a significant rutting takes place over a relatively short period of time.

Phase II: In this phase rutting takes place at a more constant rate between 5000-25000 wheel load repetitions.

Phase III: the final phase between 25000-100000 wheel load repetitions, less significant rutting takes place over long durations.

Table 4.12: Average cumulative heave for the control sample

Trial J (control sample)							
Position	0	2500	5000	10000	25000	50000	100000
100	0	5.25	9.85	9.64	11.54	11.80	13.10
200	0	6.74	7.25	11.36	10.65	10.28	12.60
300	0	7.02	8.64	10.03	10.56	11.03	12.25
400	0	6.50	11.35	11.18	9.56	10.76	12.68
500	0	7.10	8.54	8.36	7.10	10.01	11.69
600	0	5.36	4.98	7.65	8.00	8.69	9.65
700	0	3.80	6.54	5.65	8.65	9.60	10.23
800	0	6.54	7.36	6.13	7.10	8.15	8.01
900	0	7.00	6.02	4.53	7.60	5.68	7.59
Avg	0	6.15	7.84	8.28	8.97	9.56	**10.87**
Stdev	0	1.12	1.98	2.47	1.67	1.84	2.08

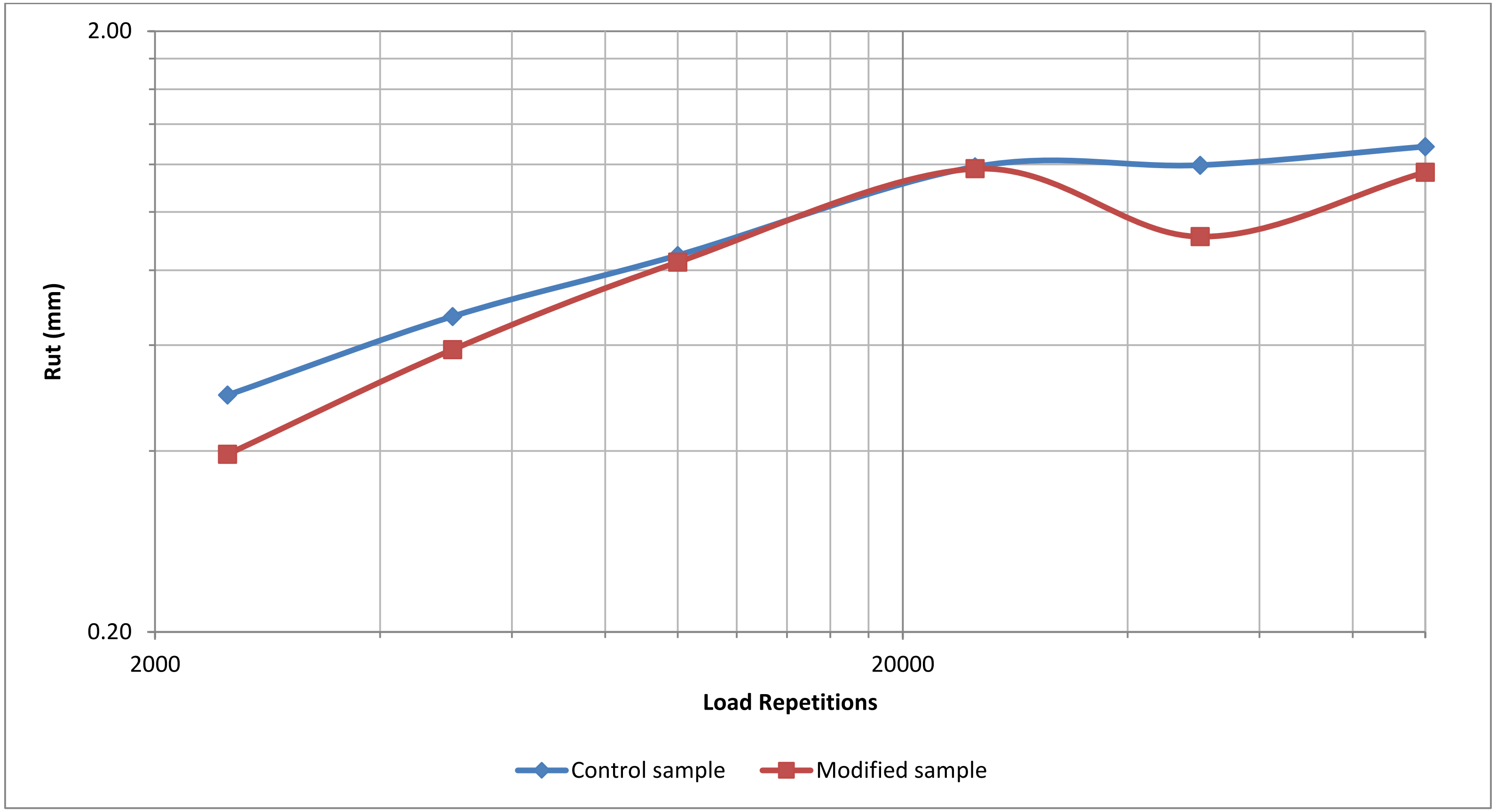

Figure 4.21: Cumulative Heave

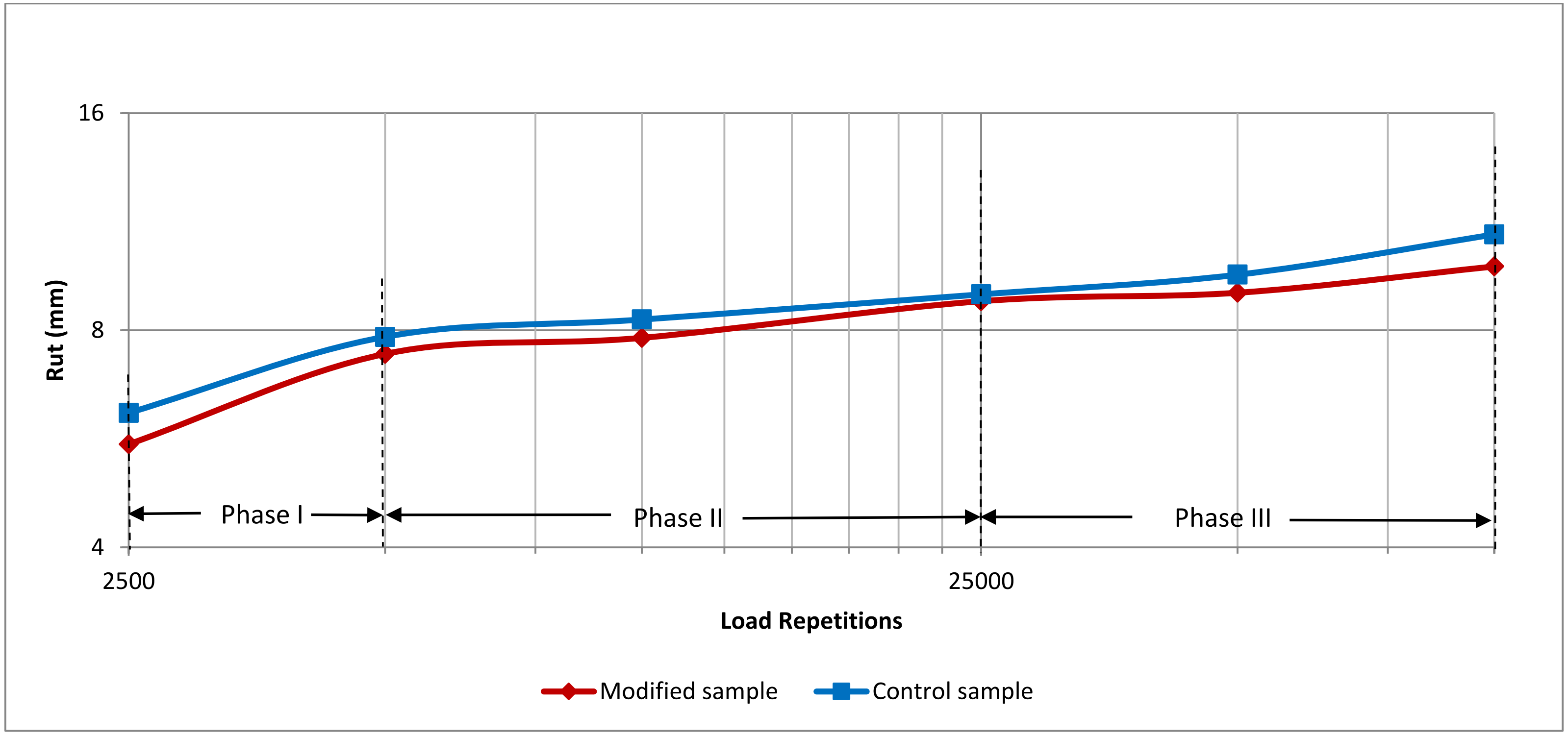

Figure 4.22: Average Cumulative rut

Table 4.13: Average cumulative rut for modified sample

Trial J (Modified sample)							
Position	0	2500	5000	10000	25000	50000	100000
100	0	8.44	10.29	10.69	11.47	11.67	12.13
200	0	6.96	8.83	9.32	10.25	10.48	11.43
300	0	6.50	8.35	8.58	9.68	10.00	11.08
400	0	7.76	10.01	10.99	11.75	11.92	13.07
500	0	5.10	7.19	7.62	8.86	9.05	9.97
600	0	4.13	5.94	6.37	7.60	7.83	8.71
700	0	2.58	4.36	4.63	5.30	5.77	6.58
800	0	4.15	5.51	5.69	6.53	6.75	7.33
900	0	4.40	6.26	6.38	7.60	7.73	7.99
Avg	0	5.56	7.42	7.81	8.78	9.02	**9.81**
Stdev	0	1.95	2.07	2.24	2.21	2.16	2.27

From Table 4.12 and Table 4.13 it can be concluded that the 9.81 mm rut depth obtained from the modified concrete mix is indicative of a good performance with respect to permanent deformation. The cumulative rut depth for the control was 10.87 mm. The reduced rutting can be attributed to the improved flexibility of the modified asphalt concrete mixture (Rust, et al., 1994, p. 204). Chen et al, (2003, p. 598) reported that rut depth decreases with increasing polymer percentages with the most significant reduction occurring at 6% polymer. Cao (2007, 1014) concludes that asphalt mixtures containing 3% tyre rubber has the best performance both at high temperature 60 ºC and low temperature (-10 ºC)

Table 4.14: Interim Guidelines for the interpretation of wheel tracking results *(CSIR & SABITA, 2001, p. 6-8)*

Repeatitions to 10 mm Rut Depth	Mix Classification
<2500	Poor
2500-5000	Medium
>5000	Good

Permanent deformation can lead to ponding of water in wheel tracks which is a hazard in wet weather. Water does not flow easily through properly constructed HMA pavements, but it will flow very slowly even through well compacted material (NCHRP, 2011, p. 9). Rutting also leads to poor riding quality leading to increased vehicle operating cost (CSIR & SABITA, 2001, p. 6-1).

4.5.2 *Indirect tensile strength*

Creep tests on asphalt mixes suggest that temperature is the most influential variable affecting rutting behaviour (CSIR & SABITA, 2001, p. 6-2). At temperatures above 45 ºC the binder softens considerably and the amount of cohesion that is still offered by the binder at these high temperatures is a function of the binder type. Polymer modified binders may still contribute considerably to the shear resistance at high temperatures. Table 4.15 gives a summary of the indirect tensile strength test results.

Table 4.15: Summary of Indirect Tensile Strength Test Results

Test Results	Control sample				Modified Sample			
	Conditioned		Unconditioned		Conditioned		Unconditioned	
Briquette No.	1	2	3	4	1	2	3	4
Bulk Relative Density (BRD)	2.374	2.389	2.381	2.383	2.341	2.347	2.349	2.353
Maximum Theoretical Relative Density (MTRD)	2.538				2.480			
Voids (%)	6.5	5.9	6.2	6.1	6.3	6.6	6.2	6.5
Voids filled with water (%)	62	72	-	-	66	76	-	-
I.T.S (kPa)	1141	1046	1069	1183	986	881	908	1000
I.T.S (ave.)	1093		1126		934		954	
Tensile Strength Ratio	0.971				0.979			

The average ITS value for the unconditioned modified asphalt concrete mix is 954 kPa while that of the control mix is 1126 kPa. The minimum ITS value in South Africa is 800 kPa (The South African National Roads Agency, 2001, p. 8-2). ITS values exceeding 1700 kPa may indicate a tendency to brittleness and low flexibility. Values below 900 kPa may be indicative of poor rutting or stripping performance. However, mixes manufactured with polymer modified bitumen binders may have low ITS values and still exhibit good performance (The South African National Roads Agency, 2001, p. 8-3). Pinheiro and Soares (2003, p.5) reported that mixtures with rubber presented lower ITS and resilient modulus compared to conventional mixtures.

Anti-stripping agents such as hydrated lime, cement and amines have been added to enhance adhesion of bitumen with aggregates. However for this research work no anti-stripping agent was added in order to utilize the desirable adhesion properties of polymers (Halstead, 1994). For routine mix design purposes, a minimum Tensile Strength Ratio (TSR) of 0.7 is usually specified. For mixes in high rainfall areas and high traffic applications, a minimum TSR of 0.8 is recommended (The South African National Roads Agency, 2001, p. 8-5). The modified asphalt mix had a tensile strength ratio of 0.979. This is an indication of low moisture sensitivity of the modified mix. Al-Hadidy and Yi-qui (2009, p.1461) reports that the tensile strength ratios for mixtures containing the LDPE were greater than 85% which indicates that this type of additive does not cause the mixture to weaken when exposed to moisture. This is an indication of compatibility of the crumb rubber and LDPE in the asphalt concrete mixture. Table 4.16 provides guidelines for the interpretation of ITS data measured at 25°C for fatigue evaluation of asphalt wearing course.

Table 4.16: Guidelines for the interpretation of ITS results for fatigue performance evaluation *(CSIR & SABITA, 2001, p. 7-2)*

Relative Fatigue Performance	ITS (kPa)	ITS strain at Maximum Stress
Good	< 1000	> 2.2
Medium	1000 to 1400	1.5 to 2.2
Poor	>1400	< 1.5

The average ITS value of 954 kPa which is less than 1000 kPa is indicative of a good relative fatigue performance of the modified asphalt mix.

These results show that it is possible to increase the capability of resistance pavements to permanent deformation and greater stability. There is an increase in Marshall stability of 8.76 kN for the modified asphalt concrete as compared to 6.87 kN for the conventional asphalt mixtures. This translates to a more durable pavement structure with a longer service life and minimal maintenance.

4.6 Economic assessment

Table 4.17 shows the present worth costs for each pavement type taking the construction and maintenance strategies from Table 24 of Draft TRH4: 1996. The sensitivity analysis shows that the discount rate can vary from 6% to 10% without having a significant influence on the present worth of costs. It is assumed that the pavements with modified asphalt mixtures and that made of conventional asphalt concrete have the same initial costs. This is because the modifiers used are waste material with insignificant additional costs to the pavement material costs. The equipment used for construction do not also require any modification for the production of the modified asphalt mixtures.

It was proposed that maintenance measures consist of single surface treatment (S1) at 11 years and 21 years for the conventional asphalt concrete pavement. The maintenance period for modified asphalt concrete pavement was varied by 30% due to the improved Marshall stability values and rutting resistance. Hence a maintenance period of 15 years and 27 years are selected for the modified asphalt concrete pavement. The present worth of cost at a discounted rate of 8% was R27.258/m^2 which is more than the calculated R25.62/m^2 for a similar pavement built of modified asphalt concrete mixture.

Table 4.17: Present worth of costs

Pavement structure	Initial costs/m²	Maintenance	Discounted maintenance costs/m²			Present worth of costs/m²		
			Discounted rate			Discounted rate		
			6%	**8%**	**10%**	**6%**	**8%**	**10%**
AC	10.5	S1 (12 yrs)	5.219	4.169	3.350			
	10.5	S1 (21 yrs)	3.087	2.090	1.418			
	21		**8.306**	**6.258**	**4.767**	**29.306**	**27.258**	**25.767**
AC (Modified)	10.5	S1 (15 yrs)	4.379	3.308	2.510			
	10.5	S1 (27 yrs)	2.174	1.313	0.798			
	21		**6.552**	**4.620**	**3.308**	**27.552**	**25.620**	**24.308**

5 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusions

The main objective of this research work was to determine the feasibility of the use of rubber tyre waste and plastic wastes in asphalt concrete pavements with the aim of improving the engineering properties of the asphalt concrete mix and safe disposal of these non-biodegradable wastes. The research showed that it is indeed feasible to use various proportions of these wastes to achieve greater performance of the pavement. The optimum values of 4% of the waste plastic and 2% of the crumb rubber used in the asphalt concrete mixture gave a significant improvement in the performance of the mixture.

The plastic waste blended with 60/70 penetration grade bitumen showed an improvement in the PMB's rheological properties. Addition of 4% LDPE reduced susceptibility of the asphalt binder to temperature by increasing its softening point from 43.8 °C to 50.2 °C and reducing its penetration value from 68 mm to 64 mm. This reduced temperature susceptibility is beneficial for use in high temperature areas. The addition of LDPE caused the bitumen to be stiffer making it susceptible to low temperature cracking and fatigue failure.

Addition of 4% LDPE and 2% crumb rubber reduced the susceptibility of the asphalt concrete mix to moisture and permanent deformation. There was a reduced rut depth from 10.87 mm for the control sample to 9.81 mm for the modified asphalt mixture which led to an improved service life of the pavement. The better Tensile Strength Ratio of 0.979 for the modified sample up from 0.971 for the control sample attests to the improved binding property of plastics in its molten state (wet process) which helped improve cohesion of the modified asphalt mix. This is an indication of good compatibility of the wastes in the asphalt concrete mix. This in effect reduced the susceptibility of the mixture to moisture therefore increasing the service life of the asphalt concrete pavement.

The extended service life in effect increased the time interval needed for road maintenance from 11 years and 21 years for the conventional asphalt to 15 years and 21 years for modified asphalt concrete. This reduced the present worth of cost value for the modified asphalt pavement therefore making it cost effective as an alternative design strategy.

5.2 Recommendations

Characterisation tests have been shown to be a good measurement of polymer contribution to binder performance. The conventional measurement techniques are inconsistent in ranking multiple-polymer modified HMA performance and may only measure whether or not a modifier is present in an asphalt specimen but not its contribution to the asphalts performance. Therefore this research work recommends the following:

a) There is need to for further research on performance tests and modelling of the outcome to ensure consistent response (output) for the input variables using the optimization process.
b) A study needs to be done to establish the interactions between the bitumen binder, LDPE and crumb rubber that influence the physical and rheological properties of the asphalt concrete mix during processing, storage and application of the modified asphalt concrete.
c) Polymer modified binders have proven successful in the laboratory. Efforts should be made to develop a correlation between results from laboratory tests and field performance by large scale modelling and testing.
d) This research findings need to be followed by technology implementation and monitoring to ensure the desired life cycle performance improvements are achieved.

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