UNIVERSITY OF NAIROBI

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FORMULATION OF ASPHALT CONCRETE USING CRUMB WASTE TYRES

BY

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DECLARATION

This thesis is my original work and has not been presented to any other university for examination. Where other people's work or my own work has been used, this has properly been acknowledged and referenced in accordance with the University of Nairobi's requirements.

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DEDICATION

I dedicate this work to my beloved wife and child Kethi for their moral support and inspiration while doing this study.

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ABSTRACT

Third world and developing countries are realizing tremendous growth in their economies. The factors that increase the growth of the economy of a country are transport infrastructural facilities, including roads, airports, railways, and water. Building reliable roads is the solution to developing a sustainable road transport network in a country. This is achieved by investing much in research on improving road construction materials' performance. The ability of roads to transport goods, people, and services within acceptable costs in terms of comfort, time, and safety is governed by road pavement life. Pavement permanent deformation in asphalt concrete mixes is among the most prevalent road pavement distress noticed on many roads in Kenya that affect the pavement life. These permanent deformations occur mainly in the wheel paths because of heavy traffic and high temperatures, resulting in depression on the pavement surface along where the wheels pass causing axial and lateral displacement, making it a risk for the motorist. The materials used in road construction are bitumen, cement, lime, aggregates, bitumen emulsions, and cutbacks. Bitumen is a critical component of road pavement; it's excellent performance increases pavement life. Other studies have shown that waste-crumbed rubber can be used to modify bitumen binders, enhance their characteristics and improve their performance. This study aimed to characterize the effects of modifying bitumen with waste-crumbed rubber to enhance the pavement life and determine the rubber content for asphalt concrete mix design. The penetration, softening point, viscosity, ductility, and aging characteristics of bitumen that are known to be improved positively by modification with rubber were determined. The samples of waste tyre rubber were obtained from the dump sites in Donholm and bitumen samples for penetration grade 80/100, aggregates of different single sizes ranging from 0/6, 6/10, 10/14, and 14/20 mm were also purchased from Colas East Africa and Aristocrats in Nairobi County. The results obtained in this study showed that penetration of bituminous binders reduced from 88 to 44 tenths of a mm at 20 % rubber content. It shows that the modified binders can withstand the elevated temperatures of asphalt pavement surfaces. Softening point, a candidate also crucial for pavement temperature susceptibility, increased from 48.8^{o}C to 62.3^{o}C at 20 % added rubber content. The flashpoint and specific gravity of the neat and rubber-modified binders did not significantly change. Viscosity at 135^{o}C increased from 304.3cSt to 330.7cSt at a rubber content of 12% and became unworkable when determined using the standard reverse flow viscometers method as more rubber was added. Ductility at 25^{o}C reduced from135.3 cm to 50.9 cm as 3% to 20% rubber was added, implying that the rubber makes bituminous binders stiffer. The determination of the suitable amount of rubber that can be used to modify bitumen a mix design was done as per the procedures given in Road Design Manual Part III of the Ministry of Transport and Infrastructure, Kenya. The Mix design for asphalt concrete was carried out using the Marshall Test procedure, where an asphalt concrete mix of 5.5 % binder content was obtained. This binder content was further refined at the refusal by the vibrating hammer method, where an optimum binder content of 5.1 % was considered optimum for the design used to deduce the suitable amount of bitumen and single-size aggregates in the mix. At the optimum bitumen content of 5.1 % different proportions of rubber content were used to deduce the suitable amount of rubber for the mix. 10.0 % rubber content was considered the optimum for improving the performance of asphalt and asphalt concrete formulations. The study also assessed how the varied intrinsic characteristics of the modified bitumen affected the modified asphalt concrete formulation. Characteristics of flow, stability, tensile strength, density, and resistance to rutting, moisture-induced damage at elevated temperatures of 60 ^oC for both the neat bitumen formulation and the rubber-modified bitumen formulation were analyzed. The results obtained shows that density, stability, tensile strength and elastic modulus of rubber modified bitumen formulation increased to a maximum of 2.262 g/cc, 15.4 kN, 842 kPa and 5236 MPa respectively as more rubber of 3 % to 20 % was added, and reduced to 2.205g/cc, 10.3 kN, 398 kPa and 2528 MPa respectively, implying that the formulations became stiffer with increased rubber content.

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LIST OF ABBREVIATIONS AND ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ASTM	American Society for Testing and Materials
ACV	Aggregate Crushing Value
BS	British Standard
CRBC	China Road and Bridge Corporation
CRMB	Crumbed Rubber Modified Bitumen
cSt	Centistoke
ESA	Equivalent standard Axle
EVA	Ethylene Vinyl Acetate
FI	Flakiness Index
FTIR	Fourier transform infrared spectroscopy
HMA	Hot Mix Asphalt
KeNHA	Kenya National Highways Authority
LAA	Loss Angelis Abrasion
MS-2	Manual Series-2
MOTI	Ministry of Transport and Infrastructure
NMR	Nuclear Magnetic Resonance
NR	Natural Rubber
OBC	Optimum Binder Content
ORC	Optimum Rubber Content
ORN	Overseas Road Note
PAHs	Polycyclic Aromatic Hydrocarbons
RTFOT	Rolling Thin Film Oven Test
SARA	Saturates Aromatic Resins Asphaltenes
SE 1	Surface Elastomer 1
SBR	Styrene Butadiene Rubber
SBS	Styrene Butadiene Styrene
SSS	Sodium Sulphate Soundness
UNEP	United Nations Environmental Program

UV	Ultra Violet
VFB	Voids Filled with Binder
VIM	Voids in Mix
VMA	Voids in Mineral Aggregates

CHAPTER ONE

1.0 INTRODUCTION

The performance of Hot Mix Asphalt in countries that lie in the tropics has not always satisfied the need of road users, where in some cases, road pavements have failed after a short time of construction and use as reported by Department for International Development (DID, 2002). High temperatures of 27°C characterize the climate in these countries on average, making the bitumen used softer than normal (DID, 2002). This calls for the need to develop a design of mix that can withstand an extensive range of temperature variations. Asphalt pavement defects include rutting, cracking, potholes, surface disintegration, and stripping. Water is another cause of asphalt concrete failure as it penetrates the base of the pavement and causes oxidation of the asphalt mix leading to stripping and disintegration of the pavement (DID, 1999).

1.1 Asphalt Pavement Failures

1.1.1 Pavement Rutting

According to data from the Ministry of Transport and Infrastructure, the figures below are some of the asphalt pavement failures noticed on Kenyan road A109 along Mombasa Road (MOTI, 2013). Several sections from Machakos turn-off (Figure 1.1) experienced several road pavements distresses. This was the situation of severe rutting and shoving on Mombasa Road at Machakos Turn off (Figure 1.1), Salama town (Figure 1.2), Sultan Hamud town (Figure 1.3), Emali town (Figure 1.4), and Makindu town (Figure 1.5) along the busy class-A highway in 2013 which have since been rehabilitated.

Figure 1.1: Rutting on Mombasa Road, Machakos Turn off

Figure 1.2: Rutting on Mombasa Road, Salama Town

Figure 1.3: Rutting on Mombasa Road, Sultan Hamud Town

Figure 1.4: Rutting on Mombasa Road, Emali Town

Figure 1.5: Rutting on Mombasa Road, Makindu Town

This research was undertaken to determine the possibility of using waste scrap rubber tyres in an asphalt concrete paving mixture to lengthen the time that asphalt pavement can last and reduce road traffic delays. Bitumen aging is one of the principal factors causing the loss of the good properties of bitumen binders, leading to the deterioration of bitumen pavements (Zhang et al., 2018). The effects of traffic loading on road pavements cause natural aging processes resulting in poor performance of bitumen binders (Chen et al., 2019). Despite the Kenyan government's effort to control traffic loading through legislation (The Traffic Act., 2012) and other mechanisms, more and more overloaded traffic have been noticed on the roads (Bonnet et al., 2018). According to the Kenya National Highways Authority (KeNHA), the maximum permissible loads are given in the weighbridge specification (KeNHA, 2021).

Table 1.1: Maximum Permissible Axle Loads in Kenya (Kg)

Axle	**Total Weight (Kgs)**
Single Steering Axle (2 wheels -single tires**)**	8,000
Single Steering Axle (4 wheels – dual tires)	10,000
Tandem Axle Group (8 wheels – dual tires)	16,000
Triple Axel Group (12 wheels – dual tires)	24,000
Single Steering Axle (2 wheels -single tires)	8,000
Single Steering Axle (4 wheels – dual tires)	10,000
Tandem Axle Group (8 wheels – dual tires)	16,000

Source: KeNHA, 2021

These exceeding loads have adverse effects on our roads. Other factors that have effects on the durability of asphalt concrete mixes are temperature, air, and water (Dzhabrailov et al., 2020). The quality of standard bitumen used in road construction is not good enough to completely prevent pavement distress (Majidifard et al., 2020). Because of this attribute, the bitumen used in road construction needs to be modified to increase its pavement performance. Bitumen that has not been blended with rubber is most affected by increased traffic and elevated temperature. According to Swetha and Rani (2014), the bitumen blended with rubber minimizes rutting with increased traffic and is not affected by diurnal temperature fluctuations. They found that rubber makes the bitumen more viscous at high temperatures of 180^{o}C and does not crack when the temperatures go low at 0 ^{o}C. It was noted that the addition of waste rubber resulted in improved overall performance in the bituminous concrete mixes (Swetha & Rani, 2014). With this in mind, it is prudent that waste tyres may help build more durable asphalt pavements in the future.

1.1.2 Pavement Cracking

Pavement cracking Figures 1.6 (a) and (b) are manifested by large interconnected rectangles that are not due to heavier loads but are a result of the rigidity of bitumen at high and low temperatures

(ORN 18, 1999). This occurs when a poor design was used, the mix was laid when it was too dry, or the aggregates used for the mix were too porous (Debnath & Sarkar, 2019). Okan Sirin et al. (2018) cite that asphalt is mainly composed of carbon (about 80–88 %) and hydrogen atoms (10–12 %), thereby has a hydrocarbon content of around 90 %. The rest of the components are made up of heteroatoms and metals.

Heteroatoms include nitrogen (0–2 %), oxygen (0– 2 %), and Sulphur (0–9 %). Metal atoms include vanadium, nickel, and iron, which are present in tiny trace quantities of less than 1 %. Hydrocarbons make the main overall structure of asphalt, whereas heteroatoms contribute to most of asphalt's special chemical and physical properties through interaction with other molecules (Okan Sirin et al., 2018). For instance, Sulphur reacts more readily with oxygen than carbon and hydrogen, which introduces oxygen into the asphalt structure leading to oxidative aging. Oxidation is the irreversible chemical reaction between oxygen molecules and the component species of bulk asphalt resulting in a change in the physical and/or mechanical properties of asphalt. Oxidative aging of asphalt is believed to be caused by the generation of oxygen-containing polar chemical functionalities on asphalt molecules (Okan Sirin et al., 2018).

Bitumen oxidation in a pavement is a complex process where the bitumen organic components undergo a reaction with atmospheric oxygen and ultraviolet (UV) radiation from the sun for a prolonged time and as a result, the pavement surface is hardened, which leads to pavement cracking at low temperatures under traffic loading as shown in Figures 1.6 (a) and (b) (Okan Sirin et al., 2018)

(a) **(b)**

Figures 1.6 (a) and (b): Pavement cracking of asphalt pavement

Kenya is a vast country lying in the tropics with varying climatic conditions, terrains, construction materials, and mixed traffic conditions of heavy and light road loads. The main transport corridor from the port of Mombasa through the capital city of Nairobi to the border towns traverses through different climatic conditions with different temperatures and rainfall (World Bank, 2020). Because of this, there is a need to use quality binders on the roads for satisfactory pavement performance. With the increasing heavier loads, high traffic volumes, and high tyre pressures at high temperatures, there is a need to build high-performance pavements. Most synthetic additives used to modify bitumen are expensive, while waste scrap tyres are readily and locally available and, to some extent, waste for disposal. The photos Donholm Dump site in Nairobi County Figures 1.7 (a) and (b) depict the challenge of waste tyre disposal in Kenya.


Figure 1.7 (a): Photo of waste tyres at Donholm in Nairobi County


Figure 1.7 (b): Photo of waste tyres at Donholm in Nairobi County

In recent times, it is becoming apparent that waste rubber and plastic disposal is becoming a challenge to environmentalists in Kenya Figures 1.7 (a) and (b). For instance, all vehicles running on Kenyan roads today use rubber tyres, and the question remains as to where they end up at the end of use (Figures 1.7 (a) and (b)). Kenya enacted the Environmental Management and Coordination Act (EMCA) in 1999 to help as a platform to deal with environmental issues. The relevant environmental laws are anchored in EMCA 1999, which gave birth to National Environmental Management Authority (NEMA). However, the most striking and missing link is the issue of specific disposal of waste tyres and the role of NEMA, which is tasked to regulate matters to do with the environment. At the current rate, the tyres presently produced in the world will undoubtedly increase in the future as automotive industries are growing exponentially (Yuen Fook & Balaraman, 2018). According to data from the National Environmental Management Authority (2013) it is estimated that 34,000 tons of waste tyres are produced yearly in Kenya. The National transport and Safety Authority had registered a total of 4,353891vehicles in Kenya by the year 2021 and this number is projected to increase therefore waste tyre availability will never be a challenge. The quantities of waste tyres needed in asphalt concrete mix formulations are minimal therefore it is expected that there will be enough waste tyres needed for bitumen modification.

During their manufacture and depending on the tyre's intended use, some compounds of copper, cadmium, and zinc end up in the final product (UNEP, 2011). The raw materials for the manufacture of tyres contain butadiene and styrene, which are hazardous to human health and carcinogens. Burning tyres produce styrene, a benzene derivative (Carman, 1997). Several products of decomposition are emitted during the burning of tyres, including carbon, zinc oxide, titanium dioxide, silicon dioxide, cadmium, lead, and other heavy metals (UNEP, 2011). According to UNEP (2011), sulphur compounds, polycyclic aromatic hydrocarbons (PAHs), aromatic oils, carbon and nitrogen oxides, particulates, and various light-end aromatic hydrocarbons (such as toluene, xylene, and benzene) are also emitted. UNEP (2011) points out that burning tyres (in the open air emit smoke that has carbon dioxide and carbon monoxide, which are part of the greenhouse gases and contribute to the greenhouse effect (UNEP, 2011). In order to reduce the maintenance of asphalt concrete pavement and increase its length of use, more efforts should be put into the distress resistance of asphalt concrete pavements.

1.2 Composition of Bitumen

According to the European Standard (EN 12597), bitumen is defined as "a nearly volatile, adhesive and water proof substance extracted from crude oil, or existing in natural asphalt, which can totally dissolve in toluene, and very thick or almost hard at favourable heat conditions." According to Kaseer et al. (2019), bitumen is one of the products that comes from the chemical distillation of crude petroleum at 350-380ºC and pressure considerably higher than atmospheric. The two sources of asphalt are natural deposits and petroleum residues (Rudyk, 2018). Chemical characterization of bitumen is complicated because bitumen is a compound combination of 300 to 2000 chemical elements (Michele Porto et al., 2019). Bitumen's chemical composition is highly complex (Ensley et al., 1984). According to the most accepted analysis by (Topal et al., 2012), bitumen can be classified into the following three main fractions asphaltenes, maltenes, and carbenes as described below.

1.2.1 Asphaltenes

Okan Sirin et al. (2018) note that asphalt is mainly composed of carbon (80–88 %) and hydrogen atoms (10–12 %), thereby has a hydrocarbon content of around 90 %, and the rest of the components are made up of heteroatoms and metals. Heteroatoms include nitrogen (0-2 %), oxygen (0-2 %), and Sulphur (0-9 %). Metal atoms include vanadium, nickel, and iron, which are in minimal trace quantities of less than 1 %. Hydrocarbons form the core composition of asphalt, whereas heteroatoms form many of asphalt's distinctive physical and chemical components by reacting with molecules.

Asphaltenes are not soluble in hydrocarbon solvents that are light, petroleum ether included. Asphaltenes contain oxygen, nitrogen, sulfur, metals vanadium, nickel and iron in the form of complexes such as metallo-porphyrins with long aliphatic chains (up to 30 carbon atoms), and pyrrolic and pyridinic rings (Porto et al., 2019).

1.2.2 Maltenes

Maltenes dissolve in hydrocarbon solvents and carbenes fraction cannot dissolve in carbon tetrachloride (Topal et al., 2012). Maltenes are composed of four elements: nitrogen base, first acidaffins, second acidaffins, and paraffins. The maltene portion can be further sub classified into oils and resins. Maltenes are then divided into saturate aromatic, and resin (Porto et al., 2019) which jointly with asphaltene, are known as the bitumen SARA (Saturate, Aromatic, Resin, Asphaltene) fraction, as shown in Figure (1.8)

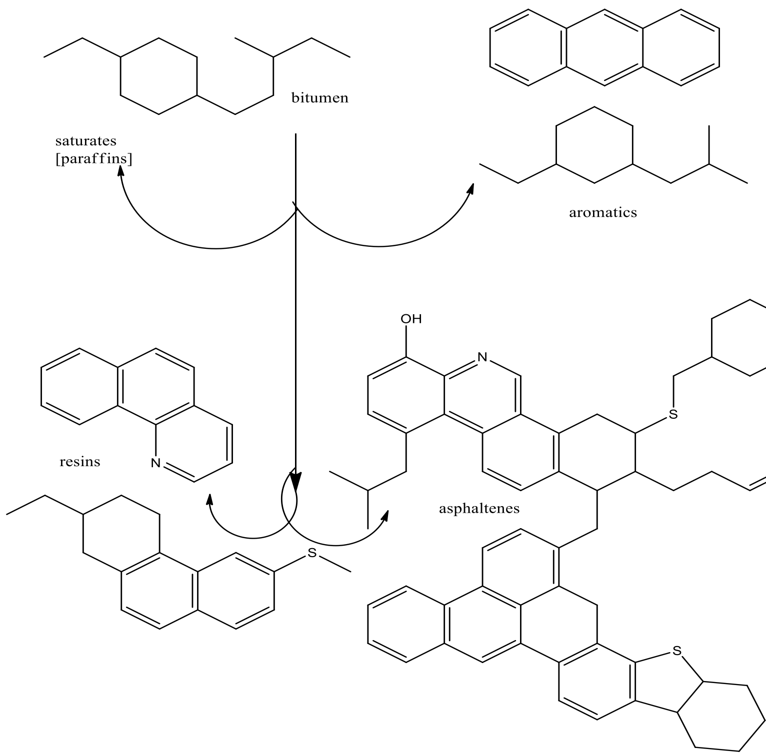
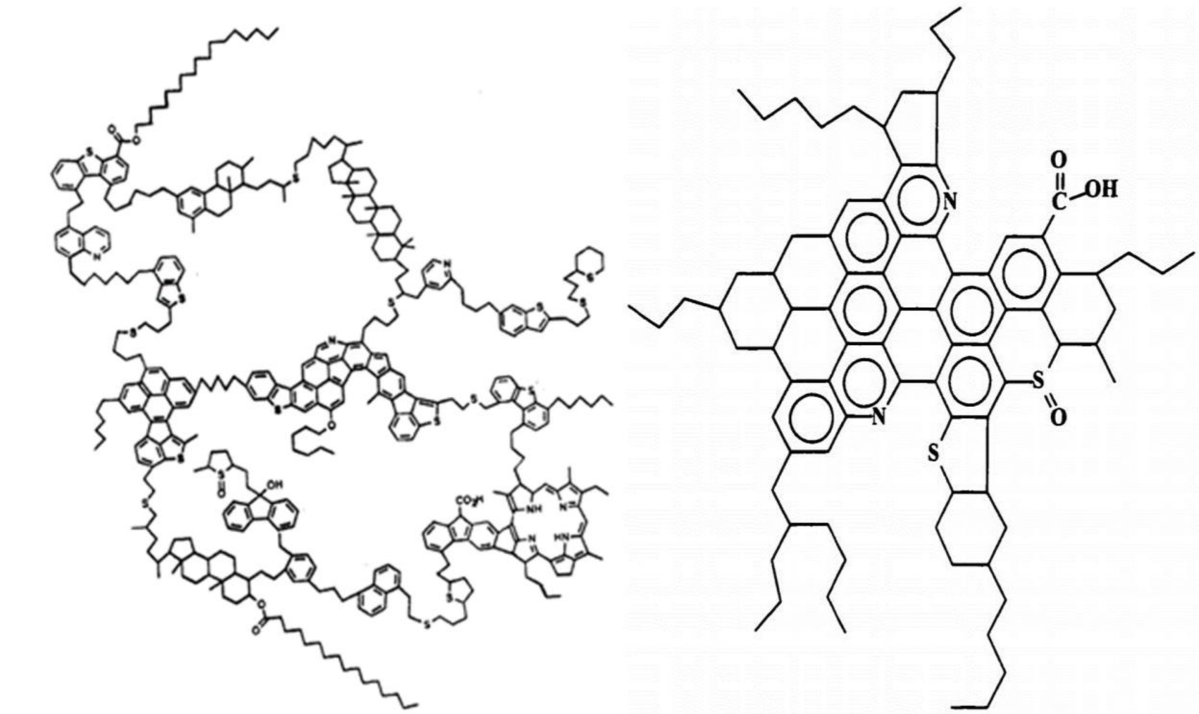


Figure 1.8 Bitumen SARA structure (Masson *et al*., 2001)

The asphaltenes correspond to a compound mixture of elements, according to Mullins (2008) and are defined by their solubility features, not by a specific chemical categorization. A typical physical definition of asphaltenes is that they can dissolve in toluene and not in n-heptane. Mullins (2008) states that other lower-carbon alkanes are at times used to separate asphaltenes.

The asphaltene portion and fraction of the maltene form the discrete phase. The lower molecular weight maltenes are said to form the unbroken phase (Topal et al., 2012). It is considered that Asphaltenes are tiny particles enclosed by a resin as the outside layer. The oil acts as a mode through which the asphaltene resin can survive. Asphaltenes make up the body, resins provide gumminess and ductility, and oils contribute to the thickness and flow (Topal et al., 2012). As demonstrated by UV-fluorescence spectroscopy), Fourier transform infrared spectroscopy (FTIR), and Nuclear Magnetic Resonance (NMR) spectroscopy (Scotti et al., 1998; Mullins, 1998), asphaltene molecules are made up of approximately 4-10 units of fused aromatic rings and some aliphatic chains as ring constituents, as shown in Figures 1.9 (a) and (b)



Archipelago structure (b) Continent structure

Figure 1.9 (a) Asphaltene hypothetical and Figure 1.9 (b) Continental structures as per (Mullins, 2008, Murgich, *et al*, 1999)

1.2.3 Carbenes

Carbenes cannot dissolve in carbon tetrachloride but can dissolve in carbon disulfide (Scotti, 1998). The carbenes are present in minimal amounts in asphalt.

1.3 South African Technical Guideline for most commonly used bitumen modifiers

The 2007 South African Technical Guideline has named the most frequently used bitumen modifiers including;

i. Styrene Butadiene Styrene (SBS)
ii. Ethylene Vinyl Acetate (EVA)
iii. Styrene Butadiene Rubber (SBR) Latex
iv. Natural hydrocarbon modifiers
v. Synthetic Waxes
vi. Rubber cramps

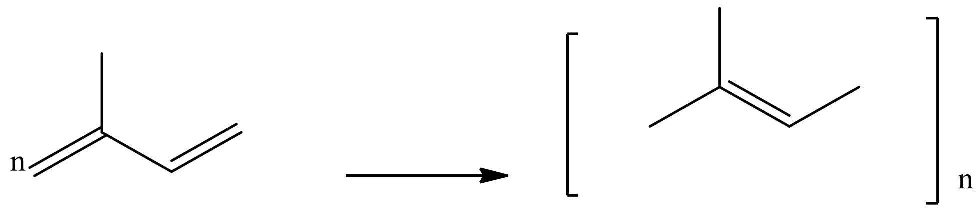
With these additives, rubber crumbs, are all readily available from waste tyres and, therefore, can be used to modify bitumen instead of burning them to release dangerous chemicals into the atmosphere (Markl & Lackner, 2020). To minimize the many waste tyres dumped in our environment, the health and well-being of the human population should not be put on trial (Carnman, 1997), and thus the question beckons; should we pay for this with our health? Natural rubber (NR) is an essential and strategic industrial raw material for making a variety of industrial commodities. They include medical devices, personal protective equipment, heat insulators, tiles, sound absorbers, waterproofing membranes, cushions, gaskets, and aircraft tyres (Khalid et al., 2017).

Even though Olmec, often referred to as the ‘rubber people' made rubber balls from NR as early as 1,600 BC, it was not until 1839 that rubber had its first known important application in the industry. It was when Charles Goodyear accidentally dropped rubber and sulfur on a hot stovetop, causing it to be resilient while retaining elasticity (Hobhouse, 2005; Hosler et al., 1999).

The most common important source of NR has been Hevea brasiliensis. Hevea brasiliensis (the Brazilian rubber tree) is the commercial source of NR and naturally, there are at least 2,500 different latex-producing plant species (Puskas et al., 2014). The chemical structure of natural rubber is cis-1, 4-polyisoprene. The polymerization makes it of isoprene (2 methyl-1, 3-butadiene) (Figures 1. 10 (a) and (b), which have a chemical formula (C_5H_8) (Puskas et al., 2014), and it is known as *cis*- 1, 4- polyisoprene made by loosely joining the monomers of isoprene (C_5H_8) in the form of a long-tangled chain (Figure 1.11).

Polymerization of Isoprene may follow either of the two pathways either cis-polymerization or trans-polymerization. The rubber formed from cis-polymerization is called cis-Polyisoprene or Natural Rubber (Figure 1.12), and the rubber formed from trans-polymerization is called Synthetic Rubber. Isoprene polymerizes to give Polyisoprene polymer, a simple alkene with each unit still containing one double bond (Puskas et al., 2014). Natural rubber has flexible properties, and it goes through long-range reversible extension even if a relatively less pressure is applied to it, thereby making it an Elastomer.

(a) (b)



2-Methyl-1,3-butadiene natural rubber

Figure 1.10 (a) and (b) Isoprene (2-Methyl -1, 3-butadiene) undergoes cis-polymerization to form natural rubber

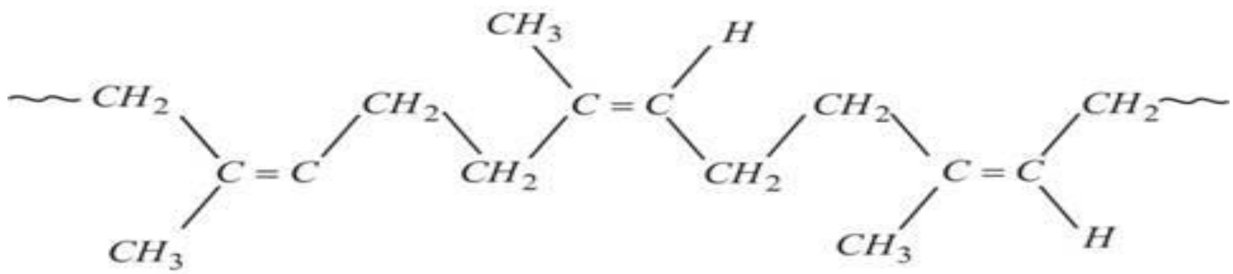


Figure 1.11 Natural rubber structure

To enhance the properties of natural rubber Sulphur is mixed with the rubber, and it is heated at a high temperature which ranges from 100^{o} C to 142^{o} C. This process of heating natural rubber added to Sulphur is known as vulcanization (Puskas et al., 2014). The Sulphur in this process is a cross-linking agent; after vulcanization, rubber gets cross-linked (Figure 1.12) and becomes hard

(Puskas et al., 2014).

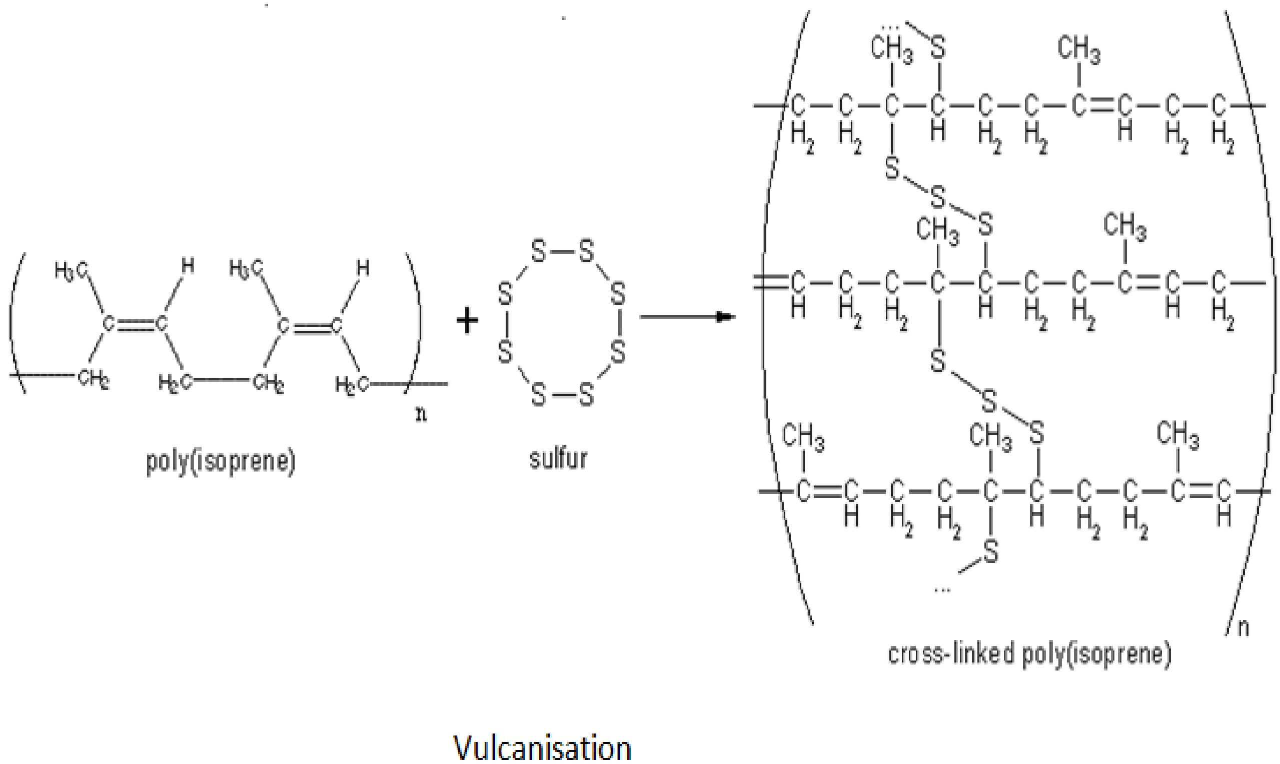


Figure 1.12 Natural Rubber vulcanization

Rubber crumbs combine with bitumen to form a complex which has both the elastic and stiffness of rubber and flow characteristics of bitumen (Carnman, 1997). At 180 ^{o}C the maltene fraction of bitumen is absorbed into the rubber, which swells and increases the viscosity of bitumen binders (Wang et al., 2019). Due to the varying climatic conditions in Kenya, it is necessary to save on costs of road construction at the same time enhancing road pavement life by use of waste tyre rubber. Besides this, waste rubber that would have been burned and released dangerous environmental pollutants should be safely reused (Bulei et al., 2018).

1.4 Statement of the problem

In the latest years ago, the third world and developing countries have witnessed tremendous economic growth (UNDP, 2013). Among the factors propelling the growth of a country's economy are transport infrastructure facilities, including roads, airports, water, and railways. To achieve a sustainable road transport network, a country needs to invest much in research on improving road construction materials' performance, including bitumen, emulsion, cutback, soil, and aggregates (Krishnapriya, 2015). The road's ability to transport goods and people within manageable costs in terms of comfortability, timing, and protection is dictated by the pavement condition and its life span. The most noticeable pavement defects are rutting, shoving, potholes, and cracking as reported by Department for International Development (ORN 19, 2002).

The ever-increasing traffic loads and volumes demand high-performance pavements (Rana & Magar, 2014). Rutting (Figures 1.1-1.5), the most common pavement distress in Kenya today, is associated with elevated pavement temperatures and increased traffic loading (ORN 18, 1999). With these two factors in mind, it is, therefore, necessary to construct high-performance pavements using improved bitumen from locally available materials such as waste rubber tyres, polyethylene wastes, lime, and cement (Ali et al., 2013). To realize this goal and increase the durability of asphalt pavement, it is necessary that the conventional bitumen used to be improved (Ali et al., 2013). Most synthetic additives used to modify bitumen are expensive, while waste scrap tyres are easily and locally available. This study aims to improve asphalt pavement performance using waste tyres with environmental disposal challenges rather than burning them to release dangerous chemicals (Wang et al., 2020).

1.5 Hypothesis

Below are the hypotheses used before the commencement of this study;

i. Most asphalt concrete pavement defects occur due to the failure of bitumen used while constructing roads.
ii. Pavement defects occur where unmodified bitumen is used in asphalt pavement construction.

1.6 Main Objective

The aim of this research was formulation of asphalt concrete using crumb waste tyres for improved pavement performance.

1.6.1: Specific Objectives

The specific objectives of the study included:

1. To assess the physical parameters of crumb waste tyres modified bitumen
2. To formulate asphalt concrete using crumb waste tyres
3. To evaluate the formulated asphalt concrete

1.7: Justification of the Study

Kenya is a vast country that lies in the tropics with varying climatic conditions (Nations Encyclopedia, 2021), she serves many landlocked countries, including Uganda, Rwanda, Burundi, Ethiopia, and South Sudan, in terms of access to her Indian Ocean port of Mombasa (Inon Africa, 2017). This translates to all these countries using the Kenya road networks to export and import their products. It calls for efficient rapid road transport networks. Apart from serving these other countries, Kenya is a vast country that requires a more elaborate road transport network system. From the port of Mombasa hinterland to the capital city of Nairobi, Western, Central, Eastern and North Eastern need to be connected with efficient sustainable road networks.

Polymer modification of bitumen has not been used much in Kenya because of the high cost of purchasing such synthetic polymers (Thakre et al., 2016). For instance, some section of the southern bypass in Nairobi was constructed using Styrene Butadiene Styrene, Surface Elastomer (SE1) synthetic polymer. However, the initiative could not be replicated in other road projects because of the high cost. This research is therefore important because it will bring into the utility of waste rubber tyre use in bitumen modification for sustainable roads and avoid the menace of waste rubber tyre disposal at a minimal cost. Besides this, the study will curb environmental pollution by finding a suitable alternative for waste rubber tyre disposal rather than burning them to release dangerous chemicals into the environment. It is also anticipated that the findings of this study will form a basic ground for decision-making by implementing authorities, private investors, and policy makers.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Asphalt concrete pavement deformation

Bitumen is produced as heavier residue from the chemical distillation of crude petroleum oil when less dense fractions of petrol, kerosene, and diesel are separated (Swetha & Raniet, 2014). The use of asphalt binder has some shortcomings because it can become brittle and hard in cold environments making it easy to crack and soft in hot environments leading to plastic deformation (Fakri & Rahim, 2016). Apart from this, the volume of road users has increased year by year, and this is among the reasons that cause road pavement to undergo plastic deformation by cracking (Fakri & Rahim, 2016).

High traffic and high loading weight also affect pavements' quality and performance, and failures occur in such cases. Hence, to reduce damage and defect, improved road pavement structures are necessary (Fakri & Rahim, 2016). Plastic deformation in asphalt concrete pavements is the major critical example of pavement defect because the altered material is irreparable and must be detached completely prior to the pavement rehabilitation (ORN 19, 2002). Apart from the type of binder leading to pavement failure, the quality of the asphalt mix laid is paramount. Underestimation of how much traffic and secondary compaction will occur in the field can also lead to plastic deformation as mixes of low air void content are prone to rutting under such conditions (ORN 19, 2002) also, mixes with high binder content undergo rutting in similar conditions. The air spaces in an asphalt concrete mix, also known as voids in the mix, are crucial in stress transfer in the mix. When the voids in the mix go below 3 %, plastic deformation occurs because stress conveyance, which was taking place through the contact between stones, changes to bitumen and fine components in the mix resulting in pavement failure (ORN 19, 2002).

2.2 Other studies

Modifying bitumen to enhance its characteristics has recently been an area of interest, where waste tyre rubber and plastics have been used for construction (Chirag & Damodariya, 2012). Because rubber is a non-biodegradable waste, it is hazardous to the environment. Rubber is among the main contributors to pollution of land because it is non-biodegradable besides plastic, glass, bottles, cans, and other examples of rubbish (Swetha & Raniet, 2014). Rubber releases many harmful substances that have at times caused severe skin diseases, drinking water pollution, and fish harm (Swetha & Raniet, 2014). Open air burning of rubber leads to the emission of chemicals into the surrounding air and environment and exhibits toxicity in the environment, which has adverse effects on life (Swetha & Raniet, 2014).

According to Papagiannakis and Lougheed (1995), the use of scrap rubber tyres has been going through the evolution process in the past twenty-five years. Using scrap tyre rubber in making asphalt pavements may improve performance features and enhance the rehabilitation of the environment and control of waste tyre stockpiles, as cited by Papagiannakis and Lougheed (1995). Kishore et al. (2015) opines that bitumen improved by rubber exhibits minimal permanent deformations defects because of heavy traffic experienced by the road and is not prone to fluctuating atmospheric temperature, and improves skid resistance. Rubber-modified bitumen has low flow at higher temperatures and improves the increased resistance to brittle fracture at very low temperatures. The incorporation of waste tyre rubber asphalt pavement construction helps as an alternative means of waste rubber disposal and reduces the cost of pavement construction (Chirag & Damodariya, 2012).

The commonly used conventional bitumen has shortcomings when used in asphalt pavements where the climatic conditions vary in temperature and rainfall (Chirag & Damodariya, 2012). The main reason for incorporating rubber in asphalt concrete pavements is that rubberized bitumen exhibits enhanced properties compared to conventional unmodified asphalt pavements (Abusharar et al., 2016). According to Juang and Amirkhanian (1992), dry and wet processes are two methods that can be used to design and construct an asphalt concrete rubber-modified mix. In the wet process, rubber is added to bitumen; while in the dry process, crumbed rubber is added to the dry aggregates. Juang and Amirkhanian (1992) assert that 5 % to 10 % of rubber can be used in the wet process, while 3 % of rubber is used in the dry process, where the rubber is added to the dry aggregates.

In their study, Singh and Kumar (2012) found that the penetration of rubberized bitumen was reduced, and viscosity increased with the addition of waste rubber. They found that the softening point increased when more of the modifier was added, while the elastic recovery was enhanced by adding rubber. Nabin (2014) found that penetration values and the softening point were improved significantly for the rubber-modified bitumen compared to the plain bitumen. In the study, 15% of rubber was used to modify the bitumen, once again proving that there has not been standard optimum rubber content for asphalt concrete mixtures. Mashaan and Abdelaziz (2014) used 5 % rubber as a modifier and found that the modified bitumen mix had more resistance to temperature variations and improved tensile strength properties. In their study, Abusharar and Al-Tayeb (2016) found out that Marshall Quotient decreases slightly with the addition of the rubber up to 5 % and then decreases as the percentage of rubber is further added up from 5 % to 15 %.

Mashaan and Karim (2013) evaluated crumb rubber content of 16 % and found that an asphalt concrete mix with 16 % rubber was stiffer and resisted permanent deformation more than a mix of conventional bitumen. Results showed that waste crumb rubber could increase the rutting resistance of hot mix asphalt mixture depending on the percentage used, again not concluding on a specified amount of rubber content. Plastic deformation is reduced as the crumb rubber content increases up to an optimum value then it reduces back (Mashaan & Karim, 2013). The main potential benefits of using scrap tyres in pavement construction include reducing asphalt binder aggregates and conserving the environment by alleviating waste tyre dumping (Juang & Amirkhanian, 1992). In their study, Juang and Amirkhanian (1992) used 3 % rubber content to show the benefits of using rubber in road construction as a bitumen modifier. They noted that the benefits of using rubber were that rubber eliminates potholes, absorbs stress, reduces noise, and stops reflective cracking for over 12 years, thus eliminating maintenance-related costs.

In a study, Roy (2015) used rubber content of 4 %, 8 %, 12 %, and 16 % to investigate the effect of rubber on an asphalt concrete mix. The results showed that the softening point increased with rubber content while ductility decreased. The bitumen became more viscous with increased rubber content as stability increased. The result showed that adding rubber made the bitumen harder, reducing plastic deformation. In other words, the bitumen's viscosity is increased, making it possible to achieve a stiffer bitumen asphalt (Roy, 2015). The results showed that increasing a high percentage of rubber led to reduced stability value, and the density of the mix increased.

The utilization of waste ground tyre rubber to modify bitumen used in asphalt concrete mixes enhances the adhesion of the combined particles in the asphalt concrete mix which increases the ability of the mix to reduce plastic deformation of road pavement under traffic loads (Roy, 2015). In their study, Ramezani et.al (2013) found out that the stability of rubberized asphalt mixes increased with addition of rubber of up to 10 %, and reduced with addition of rubber of higher percentages. They also found out that at small bitumen content asphalt concrete mixes had low stability compared to asphalt concrete mixes with 10 % rubber added. Their findings showed that the flow value increased with addition of more bitumen in the mix. They noted that the flow value of neat asphalt concrete was lower when compared to the asphalt with 5 %, and 20 % rubber but higher than 10 % rubber. The results of stability and flow showed that there was an improvement in stability at addition of 10 % rubber especially at bitumen percentage of 5 % and above when the results of flow and stability were compared with neat asphalt concrete mix.

The modification of bitumen using waste tyre rubber lowers the susceptibility of heat leading to more uniform fatigue characteristic of modified asphalt concrete under large working heat conditions. In their work, Papagiannakis and Lougheed, (1995) found that the reduced temperature susceptibility increases resistance to plastic deformation at high temperatures and fatigue cracking at low temperatures and the viscosity of the binder was increased considerably using 18 % crumb rubber. Besides failure of asphalt pavement due to poor qualities of bitumen aggregates used should be of high quality. The aggregate properties are very important to pavement performance (Wu *et. al*, 1998). They found out that pavement distress, including stripping and rutting are traceable directly to the aggregates used and therefore they concluded that proper aggregate selection is essential in order to attain desired good performance.

2.3 Properties of bitumen

2.3.1 Penetration

Penetration is a test that is used to evaluate the consistency of bitumen. Penetration is the length in a 10th of a millimeter which an average needle with a standard weight of 100 grams penetrates the sample at a standard temperature of 25 ^{o}C (BS EN 12591, 2000). Penetrometer equipment works by allowing the needle to drop vertically downwards into the sample for five seconds and gives the distance of penetration to the nearest 0.1mm. The penetration for 80/100 means that the needle penetrates through the sample 80/100 tenths of a millimeter. Penetration involves determination of penetration of solid and semi-solid bituminous materials.

2.3.2 Softening point

The heat level over which bitumen sample starts melting is known as the Softening point and is determined using Ring and Ball Apparatus. The apparatus consists of two rings and a 3.5 g standard ball and centering guides. The apparatus works by determining the temperature at which the sample falls off the rings immersed in a water bath. Softening point test is crucial in performance of asphalt concrete mixes. Softening point is important because it gives an indication of how asphalt concrete mixes are likely to be affected by high temperatures when used to pave roads and airport runways (Asphalt Institute MS 25, 2008)

2.3.3 Flash point

Flash point refers to the lowest heat conditions under which liquids forms an ignitable mixture in the air (AASHTO D 92, 2006). Flash point is arrived at using the Cleveland Open Cup as described in AASHTO D 92 (2006) where the temperature at which the bitumen sample could catch fire is determined. This test is important because it ensures that handling of samples is done at temperatures at which there is no risk of fire outbreak (ASTM D 92). It important to determine how rubber can affect and influence the flash point.

2.3.4 Ductility

Ductility is used to describe the ductile and tensile nature of bituminous binders and it reflects the homogeneity of the binder and its ability to flow (Asphalt Institute MS 25, 2008). Ductility is determined by the procedures described in AASHTO T 51-06 standard method using a ductility machine called Ductilometer (ASTM D113). Ductilometer machine pulls the sample in a standard size briquette at a specified speed of 5cm per minute and the distance at which the stretched sample breaks is the ductility of the sample. Ductility is the distance to which a briquette specimen of the binder can elongate at specific test temperature of 25^{o}C and speed of pull of 5cm per minute and the distance at which the stretched samples break is the ductility of the binder (Asphalt Institute MS 25, 2008).

2.3.5 Aging Test

Aging test indicates the aging of bitumen during hot mixing and laying at about 150 ^{o}C as indicated by viscosity and other rheological measurements. This test indicates resistance to hardening of asphalt binders when subjected to oxidative aging (BS EN 12591:2000). This test is performed as per AASHTO T 240-06 standard test method where bitumen is heated while placed in an oven for 5 hours and 30 minutes at 163 ^{0}C and the effects of air and heat is evaluated from the changes in physical tests values as measured before (Asphalt Institute MS 25, 2008).

2.3.6 Viscosity

Viscosity refers to the amount of resistance to flow of a fluid in m^2/sec or in centistokes. Viscosity is affected by temperature and is important because it helps in mixing and compaction of asphalt concrete mixes. Viscosity is determined as per AASHTO D 2170-01 a standard (ASTM D4402-02). The method embraces use of U-Tube modified reverse flow viscometer at 135^{o}C. The thermometers used are standardized at total immersion in oil bath controlled to 135^{o}C (ASTM D4402-02)

2.4 Asphalt concrete mix

2.4.1 Density

Density is the specific gravity of asphalt concrete mixes. Mixes of 1100 g and varying percentages of neat bitumen are prepared by proportioning duplicates batches of 1100 g combined aggregates and mixing them with dissimilar portions of bitumen from 4.5 %, 5.0 %, 5.5 %, and 6.0. The mixes are molded in molds of specified dimensions 100 mm diameter and 63.5 mm height when hot after mixing and left to cool overnight. The molding is done using a Marshal Compacter of standard free fall weight of 4.5 Kg and 75 blows (Ministry of Transport and Communication, 1987). The samples are weighed in air, in water and the saturated surface dry (SSD) weights taken. The difference in SSD weight and weight in water is the volume and therefore the density is the weight in air divided by volume.

2.4.2 Stability

The Marshall stability of the mixture is the greatest weight applied at a strain of 5 cm per minute that a standard Marshall specimen can withstand at a specific test heat condition of 60 °C. Stability is measured using the Marshall Tester Machine (Asphalt Institute., 2004).

2.4.3 Flow

Marshall Flow is the deformation in mm as per (Asphalt Institute., 2014) & (ASTM D2041) and is attained when a standard Marshall Specimen is subjected to a strain of 5 cm per minute at a specific test heat of 60°C. This is also measured alongside the stability of the asphalt concrete specimens (MS -2, 1994).

2.4.4 Tensile strength

The tensile strength of asphalt concrete mixtures is measured using the Marshall tester but using an ITS jig or ITS breaking head. The tensile strength is the resistance of asphalt concrete mixes to cracks and is measured in kPa (Asphalt Institute., 2014).

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CHAPTER THREE

3.0 METHODOLOGY

3.1 Study area

This study was carried out in Nairobi County, Kenya's capital city with a population of 4,397,073 according to the national population census (GOK, 2019). The metropolitan area, which includes Counties of Machakos, Kiambu and Kajiado has a population of 9,354,580 as per the national enumeration census (GOK, 2019). The study was undertaken in Nairobi County within the coordinates 1.2921° S, 36.8219° E (Figure 3.1). The bitumen samples were sourced from Colas East Africa, waste tyres from Donholm in Nairobi County and aggregates from Aristocrats in Katani Machakos County as shown in the map below.

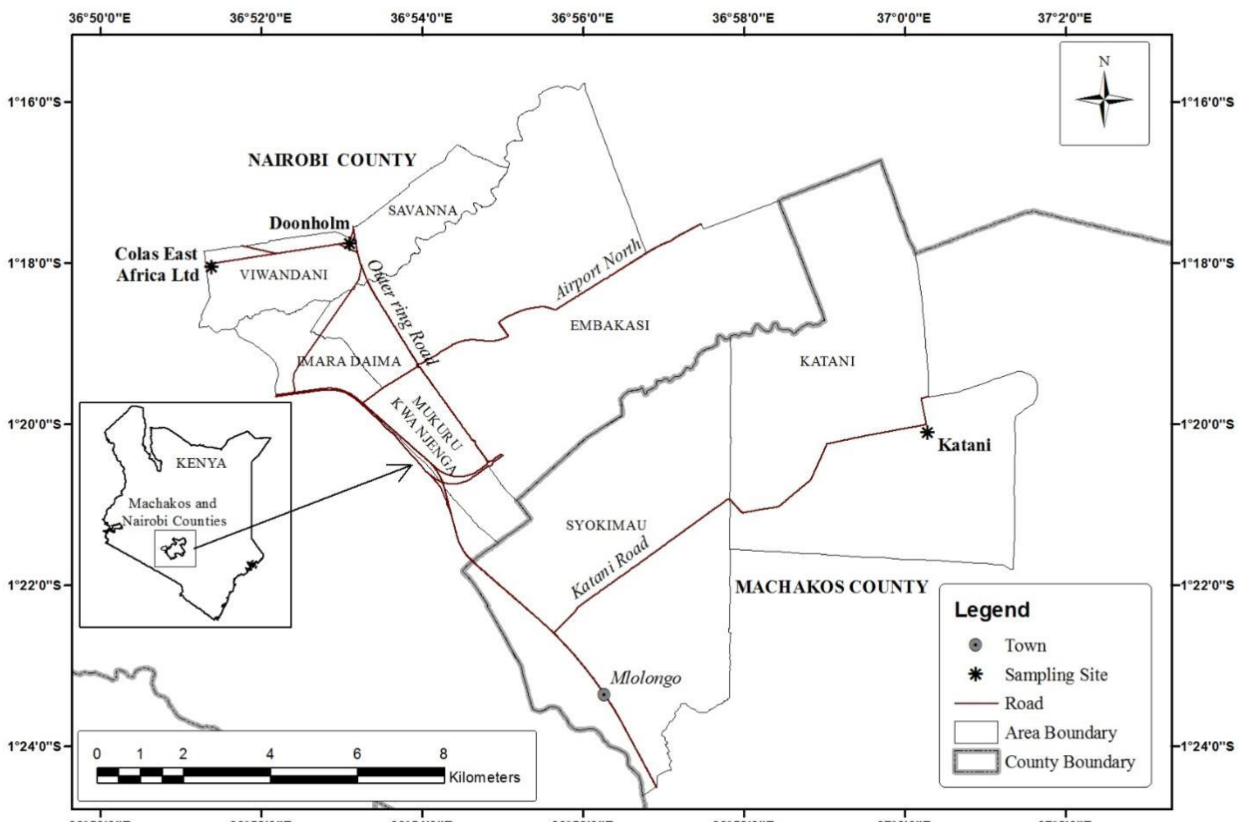


Figure 3.1: Map of Industrial Area and Donholm in Nairobi County

Table 3.1: The coordinates and human activities of the study area

Sampling Point	Longitude	Latitude	Observed Human Activities
Colas East Africa	36.856E	1.301S	-Industrial activities -Hotels -Solid waste dumping -Selling of berets for luster of heavy weights
Donholm	36.885E	1.297S	-Dumpsite for used tyres -Sorting and burning of plastics and other waste by small scale traders -Railway line -Informal settlement - Hotels and shops - Public vehicle pickup point
Katani	37.005E	1.335S	- Many surrounding quarries -Hotels and shops -Grazing fields/ Manyatas

3.2 Chemicals and reagents used

This study was done using several chemicals and reagents which served different purposes. The chemicals were sourced from laboratory chemicals and reagents dealers Chemoquip limited in Nairobi. Trichloroethylene was used to find out the solubility of asphalt binders. The

Trichloroethylene used was 99.9% pure while Methylene Chloride used for determining solubility of asphalt binders and also cleaning of the viscosity reverse flow viscometers was 99.8% pure. Kerosene was used to clean the moulds used for compaction and equipment. Glycerol in immersion chambers for determination of viscosity and flash point. Grease and oil as a lubricant on moving machine parts and lubrication of compaction moulds.

3.3 Instruments and apparatus used

The instruments used in this research included Penetrometer for penetration value for binders. The penetrometer used was manufactured by Normalab Analis of model number 40-873402. Ring-and Ball apparatus used for determining the softening point was manufactured by Matest and its model number was B070N. Cleveland Open Cup to determine Flash point for asphalt binders. Ductilometer used for determining ductility of asphalt binders was manufactured by Matest and was model number B054 while reverse flow viscometers were used to find out the viscosity of the asphalt binders. Other equipment was applied in the analysis of asphalt concrete mixtures like the Marshall compactor manufactured by ELE and of model number A-090526. Roller compactor which was used for compaction of wheel track slabs was manufactured by Controls and was of model number 77-B3602. Marshall Tester used to determine stability, flow and tensile strength was manufactured by Controls and was of model number 76-B0030/A. Wheel track Tester used to determine the rutting resistance of asphalt concrete mixtures was manufactured by Controls and was of model number 77-PV33B05. Electric stirrer used to mix rubber with bitumen was manufactured by Controls and was of model number 22-T006/1. Analytical balances were used for weighing and ovens were used for heating of samples.

3.4 Study design

In undertaking this study, several samples were needed. Fresh uncontaminated samples of straight run bitumen 80/100 conforming to standard requirements in the Standard Specification for Road and Bridge Construction were sourced from bitumen importers Colas East Africa where 40 liters was considered sufficient for this study. The samples of tyre rubber were obtained from the dump sites in Donholm Nairobi County (Figure 3.1). Aggregates were sourced from locally available aggregate dealers Aristocrats Limited located in Katani area in Machakos County (Figure 3.1). The coordinates and the human activities in the sampling sites are shown in Table 3.1. Aggregates were tested for conformity to toughness, durability, resistance to abrasion and flakiness as specified in the Road Design Manual Part III (Ministry of Transport and Communication, 1987).

The tests that were done on both the neat and rubber modified bitumen were Penetration, Softening point, Flash point, Ductility, Aging Test and Viscosity. These tests were done to investigate how modification of bitumen with rubber affects the properties of unmodified straight run bitumen. All these tests were done as per AASHTO, (2006) and their equivalent ASTM test methods (2006). Mix designs were done as per the procedures given in Manual Series-2 (Asphalt Institute MS-2, 2014) of the Asphalt Institute, Overseas Road Note 19 (2002) of the Department for International Development UK and the Road Design Manual Part III of the Ministry of Transport and Infrastructure Kenya (1987) Sampling of bitumen samples of straight run bitumen were sourced from Colas East Africa in Industrial area Nairobi.

3.5 Assessing the physical parameters of crumb waste tyres modified bitumen

Several tests were done to determine how modification of bitumen with rubber affects the properties of unmodified straight run bitumen.

3.5.1: Bitumen analysis

The tests that were done on both the neat and rubber modified bitumen to determine effect of rubber on the penetration, softening point, flash point, ductility, viscosity, specific gravity aging test and Solubility for bitumen samples.

3.5.1.1: Penetration

Penetration was done using a Penetrometer (Figure 3.2) as per AASHTO: T 49-06 Standard (2006). Eight clean tins, one for the neat and the others for varying percentages of rubber were used to analyze penetration through sample 80/100 tenths of a millimeter. The seven tins were each filled halfway with a known weight of bitumen samples. Ground rubber was added under heat to the seven of the tins in varying percentages of weight of rubber by weight of bitumen in the percentages of 3 %, 5 %, 8 %, 12 %, 15 %, 18 % and 20 % and thoroughly mixed well by stirring using an electric stirrer. Eight sets of penetration cans were also prepared and the preheated bitumen samples from the eight sets was poured into separate penetration cans to a minimum of 10 millimeter above the estimated penetration depth and analyzed using penetrometer equipment (Figure 3.2), which allows the needle to drop vertically downwards into the sample for five seconds and gives the distance of penetration to the nearest 0.1mm, the readings were recorded.

Figure 3.2: Penetrometer equipment

3.5.1.2: Softening Point

The heat level under which bitumen sample starts melting is known as the Softening point and was established using a Ring-and Ball Apparatus (Figure 3.3) as per AASHTO: T 53-06 Standard. The apparatus consists of two rings and a 3.5 g standard ball and centering guides. Eight heated rings were placed on a metal plate with a coating of glycerol and filled with the heated liquid bitumen samples and allowed to cool. The centering guides were placed on each of the rings followed by the standard balls. The cold samples were then each placed in a container of cold water and the softening point established using the Ring and Ball apparatus (Figure 3.3). The apparatus consists of a ring holder designed to support the two rings in a horizontal position in water or glycerol container. The temperature at which the samples melted and fell off the rings was measured automatically and recorded.

Figure 3.3: Ring-and Ball Apparatus

3.5.1.3: Flash Point

Flash point was determined using the Cleveland Open Cup (Figure 3.4) as described in AASHTO D 92 (2006) where the temperature at which the bitumen sample could catch fire was determined. This test is important because it ensures that handling of samples is done at temperatures at which there is no risk of fire outbreak (ASTM D 92). The bitumen samples were heated in an open cup to a high enough temperature that can make it catch fire in an open flame. The heat level under which it ignited was recorded as the flash point. The aim was determining how rubber can affect and influence the flash point Figure 3. 4 below shows Cleveland open cup.

Figure 3.4: Cleveland open cup

3.5.1.4: Ductility Determination

Ductility is used to describe the ductile and tensile nature of bituminous binders. Ductility was determined by the procedures described in AASHTO T 51-06 standard method using a ductility machine (Figure 3.5) called Ductilometer (ASTM D113). Ductilometer machine pulls the sample in a standard size briquette at a specified speed of 5cm per minute and the distance at which the stretched sample breaks is noted.

The elongation achieved in centimeters when the stretched sample breaks is the ductility of the binder. Ductility is the distance to which a briquette specimen of the binder can elongate at specific temperature and rate of pull. In doing this test sixteen ductility molds were filled separately with preheated bitumen samples of varying rubber content 3 %, 5 %, 8 %, 12 %, 15 %, 18 % and 20 %. At room temperatures the cold molds were placed in a Ductilometer and pulled at the standard speed of 5cm per minute and the distance at which the stretched samples broke was noted as its ductility in centimeters. The test was performed at ambient temperatures of 25^{o}C and is believed to reflect the homogeneity of the binder and its ability to flow.

Figure 3.5: Ductility Machine

3.5.1.5: Viscosity Determination

Viscosity is the measure of resistance to flow of a liquid in m^{2}/sec or in centistokes. Kinematic viscosities of the neat and modified binders were determined as per AASHTO D 2170-01a standard (ASTM D4402-02). The method embraces use of U-Tube modified reverse flow viscometer (Figure 3.6) at 135^{o}C. The thermometers used were standardized at total immersion in oil bath controlled to 135°C. 20 ml of test specimen of each of the eight samples were permitted to stream through the viscometer and the time measured to within 0.1sec the time it takes the leading edge of the meniscus to move from the first timing mark to the second was recorded. If a sample gave an efflux of less than 60 sec another viscometer of smaller capillary was used to repeat the test. A total of three readings were made for each of the samples tested. The kinematic viscosity is given by the calibration constant of the viscometer multiplied by the time taken in seconds. Several viscometers were used depending on their capillary size and the viscosity of the bitumen. Less viscous bitumen required use of viscometers with smaller capillary to achieve the minimum efflux time of 60 sec. Figure 3.5 shows viscometers used;

Figure 3.6: Viscometers

For standard viscometers the time of flow of affixed fluid volume is directly proportional to its kinematic viscosity. Kinematic viscosity = Ct, where; "C" is the calibration constant of the viscometer and "t" is the efflux time.

3.5.1.6: Aging Test (Rolling Thin-Film oven Test) Determination

This test was performed as per AASHTO T 240-06 standard test method where bitumen was heated while placed in an oven for 5 hours and 30 minutes at 163 ^{0}C and the effects of air and heat is evaluated from the changes in physical tests values as measured before. The method (ASTM D 5-06) indicates the aging of bitumen during hot mixing and laying at about 150 ^{0}C as indicated by viscosity and other rheological measurements.

The samples were placed in a ventilated oven with a plate rotating 5-6 times per minute and maintained at 163 ^{0}C for 5 hours 30minutes. The samples were placed when hot in pans approximately 55mm diameter and 35mm thickness. The aged samples were removed and previously done tests of penetration, softening point, flash point, ductility, and viscosity were repeated. The variation in properties of the aged bitumen to those of neat bitumen was then evaluated.

3.6 Formulation of asphalt concrete using crumb waste tyres

To determine the optimum amount of rubber ideal for asphalt concrete a formulation of the mix was done and involved several steps as follows;

3.6.1: Aggregate sampling

Aggregates were sourced from locally available aggregate dealers Aristocrats off Mombasa Road, Katani in Machakos County. Preliminary tests were done on the aggregates to ascertain their fitness to use in this study. Aggregates used were less than 25 % flaky as part of the requirements (Road Design Manual Part III, 1987).

3.6.2: Flakiness (FI) Determination

Flakiness was determined using flakiness gauges 20/14 mm, 14/20 mm and 10/6 mm where combined and flakiness of 20 % was achieved for the mix design. Flakiness was found by separating the flaky aggregates using flakiness gauges and their masses expressed as a percent of the total amount of the sample tested.

The sizes of the aggregates used ranged from 0/6, 6/10, 10/14 to 14/20 mm. Before further work was done the aggregate samples were subjected to mechanical tests to make sure they are mechanically fit to use (Ministry of Roads and Public Works, 1986). These tests included; Aggregate Crushing Value (ACV), Los Angelis Abrasion (LAA) and sodium sulphate soundness (SSS).

3.6.3: Aggregate Crushing Value (ACV) Determination

ACV is a test that gives the aggregate's resistance to crushing beneath a steadily applied compressive weight and was done as per procedures given in the British Standard (BS) 812 part 3. The material for the tests consisted of the aggregates passing 14.0 mm sieve and reserved on sieve 10.0 mm BS test sieves.

A compression testing machine was used to conduct the test. The compressor applies a force of 400 KN uniformly reached in 10 minutes. The crushed samples were then sieved using BS sieve 2.36 mm and the percentage of the sample running through sieve 2.36mm was taken as the aggregate crushing value for the samples.

3.6.4: Los Angelis Abrasion (LAA) Determination

LAA was done using Loss Angelis testing machine which produces abrasion by use of steel balls. The percentage of the material passing sieve 1.7mm was taken as the LAA for the aggregates Road Design Manual Part III, (1987).

3.6.5: Sodium Sulphate Soundness (SSS) Determination

Aggregate soundness was determined as per the procedures given in BS 812 part 121 (Road Design Manual Part III, 1987). In this method the aggregates were taken through repetitive cycles of immersion in saturated solution of Magnesium sulphate and then dried in an oven. The method subjects the sample to disruptive effect of repeated crystallization and rehydration of Magnesium sulphate heptahydrate $MgSo_4.7H_2O$ within the pores of the aggregates.

The extent of degradation was measured by the quantity of materials finer than 10 mm produced as a percent in which case 8% was reported as the result. Wuet al. (1998) reported that the aggregates must be strong and resistant to abrasion to avoid being crushed, degraded, and disintegrated on stockpiles passed through an asphalt plant, positioned with a paver, packed down with rollers and subjected to traffic loads.

In their research Saba Uthus and Aurstad (2012) found out that low quality aggregates exhibited the least resistance to permanent deformation indicating that use of polymer modified binder would not compensate for low quality aggregates. This is the reason why this research embraced testing of the mechanical properties of the aggregates before their use. The Road Design Manual Part III (1987) has specified the requirement for aggregates as per the Table 3.2.

Table 3.2: Aggregate mechanical tests requirements

Test		Maximum (%)
ACV	Aggregate Crushing Value	25
SSS	Sodium Sulphate Soundness	12
LAA	Loss Angelis Abrasion	30
FI	Flakiness Index	25

3.7: Mix Design

A mix design of asphalt concrete 0/20mm was done as per the specification given in Standard Specifications for Roads and Bridge Construction (1986). The specification for asphalt concrete is as shown in Table 3.3 below.

3.7.1: Selection of mix type

The selection of a mix type highly dictates the aggregate envelope within which the asphalt mix is to be designed. The Standard Specification for Roads and Bridge Construction, Road Design manual Part III (1987), and the ORN 19 (Department for International Development, 2002) have tabulated grading requirements for asphalt concrete of varying maximum aggregate sizes as shown in Table 3.3.

The purpose of evaluating the effect of rubber on bitumen in this study a binder course design of 0/20 mm Type I was chosen as it is the type of mix that is mostly used for most asphalt concrete binder course pavements (Department for International Development, 2002).

Table 3.3: Aggregate grading requirements for asphalt concrete

Sieve size (mm)	**% By Weight passing**							
	Type I						**Type II**	
	Wearing course			**Binder course**			**Wearing Course**	
	0/14	0/10	0/6	0/20	0/14	0/10	0/14	0/10
28	-	-	-	100	-		-	-
20	100			90-100	100		100	
14	90-100	100		75-95	90-100	100	90-100	100
10	70-90	90-100	100	60-82	70-90	90-100	70-95	90-100
6.3	55-75	60-82	90-100	47-68	52-75	60-82	55-85	62-90
4	45-63	47-67	75-95	37-57	40-60	45-65	46-75	50-80
2	33-48	33-50	50-70	25-43	30-45	30-47	35-60	35-65
1	23-38	23-38	33-50	18-32	20-35	20-35	25-45	25-50
0.425	14-25	14-25	20-33	11-22	12-24	12-24	14-32	14-33
0.300	12-22	12-22	16-28	9-17	10-20	10-20	11-27	11-27
0.150	8-16	8-16	10-20	5-12	6-14	6-14	6-17	6-17
0.075	5-10	5-10	6-12	3-7	4-8	4-8	3-8	3-8

Source: Standard Specifications for Roads and Bridge Construction (1986)

3.7.2: Selection of component materials

Aggregates used were first tested to meet the requirements for mechanical soundness of ACV, LAA, FI and SSS as stipulated in Road design manual part III, 1987.

3.7.3: Mixing and compaction of asphalt mix

Mixes of 1100 g and varying percentages of neat bitumen were prepared and then the determination of the optimal binder constituents for the design was done. This was done by proportioning in duplicates batches of 1100 g combined aggregates and mixing them with varying bitumen proportions from 4.5 %, 5.0 %, 5.5 %, and 6.0. The mixes were molded in molds of specified dimensions 100 mm diameter and 63.5 mm height when hot after mixing and left to cool overnight, this molding was done using a Marshal Compacter of standard fall weight of 4.5 Kg and 75 blows as per the Road Design Manual Part III (Ministry of Transport and Communications, 1987) illustrated in Figures 3.7 (a) compaction and (b) Marshall specimens.

Figure 3.7 (a) Marshall Compaction   Figure 3.7 (b): Marshall Specimens

Measurement of the volumetric properties of compacted mix the specimen samples were removed from the molds after 24 hours using an extruder and tests for density, stability, flow and computation of voids were done. The extruded samples were tested using a Marshall Tester (Figures 3.8 (a) and (b).) The Road Design Manual Part III (1987), ORN 19 (2002) and MS-2(2014) have specified the Marshall Design criteria as shown in Table 3.4 below

Table 3.4: Marshall Design criteria used for road design

Parameter	Results
Marshall Air Voids, %	4.0 – 7.0 %
Voids in Mineral Aggregate, %	Not Less than 14.0 %
Stability, kN	Not Less than 9.0 kN
Flow, mm	2.0 – 4.0 mm

Source: Road Design Manual Part III Materials and Pavement Design for New Roads (1987).

The Marshall stability of the mix is the greatest load applied at a strain of 5 cm per minute that a specimen can withstand at a standard test temperature of 60°C while the deformation in mm is the Marshall Flow (MS -2, 1994).

Figure 3.8 (a): Marshall Specimens Figure 3.8 (b) Taking the Marshall readings

The results were tabulated as density in gcm^{-3}, stability in kilo Newton, and flow in mm

The optimum bitumen content of the different maximum aggregate size mixes is also tabulated as shown in Table 3.5 in the Road Design Manual Part III, of 1987.

Table 3.5: Optimum binder Content for different mixes

Usual Bitumen Content (%)		
	Wearing Course	Binder Course
Type I (0/6)	6.0-8.0	-
Type I (0/10-0/14)	5.5-7.0	5.0-6.5
Type I (0/20)	-	4.5-6.0
Type II (0/10-0/14)	5.5-7.5	

Source: Road Design Manual Part III Materials and Pavement Design for New Roads (1987).

3.7.4: Specific Gravity Determination

The bulk specific gravity of each specimen briquettes was determined as described in ASTMD2726 method. Other parameters of voids in mineral aggregates (VIM) calculated using equation 3.1, voids in mix (VMA) calculated using equation 3.2 and voids filled with binder (VFB) were determined through computation from the maximum specific gravity from equation 3.3. The maximum specific gravity was determined by the methods described in ASTM D204 and MS-2 of 2014. To determine the maximum specific gravity, a pycnometer full of water was weighed and a loose sample of 300g was placed in the empty pycnometer and covered with water. The pycnometer was covered with a vacuum lid and placed on a vibratory shaker. A vacuum pump was used to suck all the air from the sample with agitation from the shaker. Agitation helps to remove all the air from the sample.

Maximum specific gravity (Gmm) = weight of sample / ((weight of sample + weight of pycnometer + water) - (weight of pycnometer + sample + water))

3.7.4.1: Calculation of Voids in Mix (VIM)

Voids in mix are given as:

Voids in Mix (VIM) = 100 (Gmm-Gmb)/Gmm)) -- equation 3.1

Where; Gmm = maximum specific gravity of mix

Gmb = bulk specific gravity of compacted mix

3.7.4.2: Calculation of Voids in Mineral Aggregates (VMA)

Voids in Mineral Aggregates are given as:

Voids in Mineral Aggregates (VMA) =100- ((Gmb*Ps)/Gsb) ------------------------- equation 3.2

Where; Gmb = bulk specific gravity of compacted mix,

Gsb= bulk specific gravity of total aggregates and

Ps = Percentage of aggregates by weight in the mix

3.7.4.3: Calculation of Voids Filled with Binder (VFB)

Voids Filled with Binder is given as (VFB) =100((VMA-VIM)/VMA) ---------------- equation 3.3

Where VMA = Voids in Mineral Aggregates and VIM = Voids in mix

3.7.5: Flow and stability of the mix determination

Stability and flow were read directly from the Marshall Tester equipment and the results recorded.

3.8: Marshall Method of mix design Determination

The Marshall method of mix design specified in Manual Series, MS -2(2014), (ORN 19, 2002) and (Road Design manual Part III, 1987) were used to produce the design mix. The aim of the method was to determine via a series of laboratory produced analytical trial mixes for an optimal combination of aggregates and bitumen to achieve a specific ideal criterion for design. After the determination of the different properties of the mixes, the data obtained was used to plot curves for the properties of the mixes and obtain the optimum design binder content as per the Marshall criteria. The Marshall Properties for asphalt mixes considered were;

3.8.1: Marshall Stability Determination

Marshall Stability of an asphalt concrete mix is the greatest load applied at a strain of 5 cm per minute that a Marshall specimen can withstand at a standard test heat level of 60°C (MS -2, 1994). The stability result of each Marshall Test specimen was determined using a Marshall tester (Figures 3.8 (b)). The Marshall Test specimens were prepared for testing through the immersion in a water bath at a test heat of 60 °C±1 °C for thirty minutes duration (Road Design manual Part III, 1987). The samples were then put in the Marshall Tester machine and loaded at a stable rate of deformation of 5 cm/minute until the maximum load was arrived at. The value of the load was recorded as the Marshall stability in kN.

3.8.2: Marshall Flow Determination

Marshall Flow is the deformation in mm which occurs when a maximum load is applied on a Marshall specimen at a strain of 5cm per minute (MS -2, 1994). Marshall Flow was determined from the same Marshall specimens when doing the stability measurement.

3.9: Asphalt Concrete Mix

3.9.1: Voids in mineral aggregate (VMA) Determination

Voids in mineral aggregates of an asphalt concrete mix is the air voids in the aggregates before addition of bitumen binder (ORN 19, 2002). The VMA has to be large enough to allow for the volume of effective binder in the mix apart from that which is absorbed by the aggregates and the air spaces in compacted mix (ORN 19, 2002). This ensured that there was sufficient volume within the mix to accommodate the aggregates and more so the fine aggregates and sufficient air voids after mixing and compaction to allow for expansion of binder when the pavement is subjected to its hot field conditions as described in MS-2, 2014. Voids in Mineral Aggregates is computed from bulk specific gravity of compacted mix,

Aggregates (VMA) $=100-((Gmb*Ps)/Gsb)$ -- equation 3.2

3.9.2: Voids Filled with Bitumen (VFB) Determination

VFB is the intergranular voids space between the aggregate particles that are filled with asphalt (MS-2, 2014) Voids Filled with Binder is given as (VFB) $=100((VMA-VIM)/VMA)$ ------- equation 3.3

3.9.3: Voids in Mix (VIM) Determination

Voids in mix are the air spaces left after the addition of binder (ORN 19, 2002). The important aspect of the mix design was to ensure that the volumetric properties were fit for purpose.

Voids in mix are given as:

Voids in Mix (VIM) = 100((Gmm-Gmb)/Gmm)) --------------------------------------- equation 3.1

3.9.4: Density Determination

Density is the specific gravity of asphalt concrete mixes or the mass per unit volume of asphalt concrete mixes. To determine density mixes of 1100 g and varying percentages of neat bitumen were prepared by proportioning in duplicates batches of 1100 g combined aggregates and mixing them with varying percentages of bitumen from 4.5%, 5.0%, 5.5%, and 6.0 %. The mixes were molded in standard Marshall Molds of dimensions 100 mm in diameter and 63.5 mm height when hot and were left to cool overnight. The molding was done using a Marshal Compacter of standard free fall weight of 4.5Kg and 75 blows (Ministry of Transport and Communication, 1987).

The samples were weighed in air, in water and the saturated surface dry (SSD) weights taken. The difference in SSD weight and weight in water is the volume and therefore the density is the weight in air divided by volume. The graphs of density, stability, VMA, VIM, VFB and flow respectively all against bitumen content were plotted. Considering the Marshall Design criteria, the optimum binder content was extrapolated from the marshal criteria graphs at;

i. Binder content at highest stability
ii. Binder content at highest density
iii. Binder content at 4% VIM
iv. Binder content at minimum VMA
v. Binder content at 70% VFB
vi. Binder content at median flow

The average of all these values at the optimum binder content of each property was taken as the optimum bitumen content (BC). The asphalt concrete was subjected to secondary compaction as it would happen in the field by compaction to refusal density using a vibrating hummer. This method simulates what would happen in real situations in the field when the road has been opened to traffic. Three batches of the mix samples were prepared with the same proportions as for the Marshall but this time round with a weight of 2200 g an amount of material that was capable of attaining a height of 60 mm as the conditions expected when laid on the road. The three had bitumen content of 4.0 %, 5.0 % and 5.5 %.

The three specimens were compacted to refusal using a vibrating hammer which consists of a tamping foot of diameter 102 mm and 146 mm, split molds and base plates. Heated samples were placed in the molds and compacted using the hammer. The hammer vibrates at frequency of 50 Hz. The samples were left to cool for 24 hours and demolded. Their densities were determined and their voids computed. The optimized binder content was deduced from the curves plotted of binder content against the refusal voids as done previously in the Marshal method. The optimal content of bitumen was then used in determining the suitable amount of rubber by weight of bitumen for a 0/20 mm asphalt concrete mix.

This was done by molding samples with varying percentage weight of rubber content by weight of neat bitumen and the rubber content with optimum design characteristics was deduced. Duplicates of mixes with varying percentage of rubber 3 %, 5 %, 8 %, 12 % 15 %, 18 % and 20 % were prepared and evaluated for stability, density, flow, VMA, VIM, VFB and elastic modulus to determine which percentage of rubber modified bitumen gives the best characteristic mix.

3.10 Evaluation the formulated asphalt concrete

The impacts of rubber modified bitumen on asphalt concrete mix properties of density, stability, flow and tensile strength were determined as follows. Mixes were produced in the laboratory by proportioning the various aggregates together, heating in an oven, adding rubberized bitumen and mixing the asphalt in a bench mixer or manually. The laboratory prepared mix was then tested for all properties specified below

3.10.1 Density

The effect of rubber on density of asphalt concrete mixes was evaluated by preparing triplicate batches of 1100 g combined aggregates and mixing them with varying percentages of rubberized bitumen from 3 % 5 % 8 % 12 % 15 % 18 % and 20 % rubber content at the optimal content of bitumen of 5.1 % determined for the mix design. The mixes were molded in standard Marshall Molds of 100 mm diameter and 63.5 mm height when hot and were left to cool overnight. The molding was done using a Marshal Compacter of standard free fall weight of 4.5 Kg and 75 blows (Ministry of Transport and Communication, 1987). The samples were measured in air, in water and the saturated surface dry (SSD) weights taken. The difference in SSD weight and weight in water is the volume and therefore the density is the weight in air divided by volume.

3.10.2 Stability Determination

After the Marshall densities were taken the Marshall specimens were further used to determine their Marshall stability for the whole range of rubber content from 3 %, 5 %, 8 %, 12 %, 15 %, 18 % and 20 % at optimum binder content of 5.1%. The stabilities of the rubberized asphalt concrete mixes were determined using a Marshall tester at an average heat of 60 °C (MS -2, 1994). The Marshall Test specimens were immersed in a water bath at a test temperature of 60 °C±1 °C for a time duration of thirty minutes (Road Design manual Part III, 1987). The samples were then placed in the Marshall Tester machine and loaded at a 5 cm/minute constant rate of deformation of until the maximum load was reached. The value of the load was recorded as the Marshall stability in kilo Newton (kN).

3.10.3 Flow Determination

Marshall Flow was determined from the same Marshall specimens when doing the stability measurements. Flow for the whole range of rubberized asphalt concrete mixes from 3 %, 5 %, 8 %, 12 %, 15 %, 18 % and 20 % was taken as the mixes Marshall stabilities were being measured. The deformation in mm is the flow of the mixes and is achieved when a maximum load is applied on the Marshall specimens (MS -2, 1994)

3.10.4 Tensile strength Determination

The Tensile strength was evaluated by preparing triplicate batches of 1100 g combined aggregates and mixing them with varying percentages of rubberized bitumen from 3 % 5 % 8 % 12 % 15 % 18 % and 20 % rubber content at the determined optimum bitumen content of 5.1% for the mix design. The mixes were molded in standard Marshall Molds of dimensions 100 mm in diameter and 63.5 mm height when hot and were left to cool overnight. The molding was done using a Marshal Compacter of standard free fall weight of 4.5 Kg and 75 blows (Ministry of Transport and Communication, 1987). The tensile strength of asphalt concrete mixes was determined using the Marshall tester but using an ITS jig or ITS breaking head and the tensile strength of asphalt concrete mixes was recorded in kPa

3.10.5: Performance testing Determination

Assessing the impacts of ground rubber modified bitumen on asphalt pavement rutting and Moisture induced damage performance were done as shown below.

Both the waste rubber modified bitumen mix and neat mix were further subjected to wheel track testing to determine which mix is more resistance to rutting at simulated field conditions. The test was done as per the method stipulated in AASHTO T 324, ORN 19 (2002) and MS-2 (2014). The wheel tracking test simulates the performance of asphalt concrete mix in the field. This test was done using wheel track test machine which simulates field condition of the mix under traffic.

This test was used to evaluate how susceptible an asphalt concrete mix is prone to plastic deformation at elevated temperatures and pressure equivalent to those conditions which may be the same as those that occur on the road. The susceptibility of bituminous mixes to plastic deformation is based on pass failure criteria developed continuously repeated passes and cycles of a loaded standard wheel. This method determines the rutting depth and number of passes to failures.

The test measures the plastic deformation of asphalt concrete mixture where a rubber or steel wheel of standard dimensions and load rolls over a mix slab or core sample held at simulated field temperatures. A sample of 13 kg was prepared from the proportions of aggregates got for the design and compacted when hot using a roller compacter (Figure 3.9) to target voids of 4.0 % by setting the machine (Figure 3.10) to a compaction force of 28 kN.

Figure 3.9: Preparing samples for roller compaction in the laboratory

The specimen was held in a water bath whose temperature was controlled at 60 ^{o}C to simulate experiences obtained on road in the field. The wheel track tester is an electronically powered and the wheel of 47 mm wide is capable of moving 203.2 mm diameter over the test specimen under load of 705 ± 5 N. The wheel reciprocates on the mix sample and makes 25 cycles per minute over the specimen with a maximum speed of 0.305 m/s attained at the center of the sample. The wheel track tester machine (Figure 3.10) is software operated and gives the rutting in mm over the cycles made on the sample Asphalt Institute (2014) Manual Series 2.

Figure 3.10: Performance testing using wheel track tester

The aim of the test was to investigate the anti-rutting property of neat bitumen mix and rubber modified bitumen mix. The neat asphalt concrete mix and rubber modified concrete mixes were placed under different rutting wheel to compare the two anti-rutting properties of the mixes as shown in Figure 3.10.

3.10.6: Moisture induced damage Determination

Using the neat and the optimum rubber modified bitumen, samples were molded to determine the Tensile Strength of the mixes after 24 hours and extended 7 days of curing to deduce the mix with the best properties.

3.11: Statistical data analysis

Data obtained was analyzed and represented by use of text, graphs and statistical tables to show the interrelationships of various variables from Microsoft Excel and Statistical Package for Social Sciences tools (SPSS) version 10.1.

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1 Assessment of the physical parameters of crumb waste tyres modified bitumen

In order to assess the physical parameters of waste tyres modified bitumen several tests were done. The Tests done to assess the physical parameters of crumb waste tyres modified bitumen gave the following results on analysis.

4.2: Test Results for Neat straight Run Bitumen

The neat bitumen was analyzed for conformity to the standard test given in the Standard Specification for Roads and Bridges (1986), which gave the results illustrated in Table 4.1. The bitumen used was 80/100 penetration grade because it is the most commonly used grade of bitumen in asphalt concrete pavements.

Table 4.1: Results for analysis of neat bitumen (80/100 grade)

Test Description	Neat Bitumen	Specification as Per STD Spec (1986)		
Penetration Grade 80/100		Minimum	Maximum	
Penetration 25 ^{0}C (100g, 5s) 0.1mm	88	80	100	
Penetration (after RTFOT) %	77	50		
Original Pen.				
Softening point (Ring and Ball) ^{0c}(NEAT)	48.8	45	52	
Softening Point (after RTFOT) ^{o}C	50.2	45		
Flash Point (Cleveland Open Cup) ^{0}c	250+	225		
Viscosity at 135 ^{0}C (Virgin) cSt	304.3			
Viscosity at 135^{0}C (AFTER RTFOT) cSt	315.1			
Ductility at 25^{o}C (cm)	135.5	100		
Ductility at 25^{o}c (AFTER RTFOT), cm	124.8	75		
Specific gravity at 25 ^{0}c, g/cc	1.00	1.00	1.05	
Thin Film Oven Test (TFOT) 5 hrs. at 163 0c) %	0.2		0.5	
Solubility in Trichloroethylene (%)	99.9	99		

Table 4.1 shows that the neat sample was a penetration grade 80/100 bitumen with penetration value of 88. The results show that all the other parameters of softening point, ductility, specific

gravity and solubility conform to the specifications for 80/100 penetration grade given in standard specification (Ministry of Roads and Public Works., 1986). The bitumen was therefore deemed fit for this study (ASTM D 5).

4.3: Test Results for Rubber Modified Bitumen

The modification of the neat bitumen with varying percentage ranges of rubber of 3 %, 5 %, 8 %, 10 %, 12 %, 15 %, 18 %, and 20 %, the following results were obtained for the different proportions (Table 4.2).

Table 4.2: Test results for the rubber modified bitumen 80/100 grade

Test description	Results									Specification for 80/100 as Per STD Spec	
Rubber Added %	0%	3 %	5 %	8 %	10%	12%	15%	18%	20%	**MIN**	**MAX**
Penetration 25^0C (100g, 5s) 0.1mm	88	81	79	72	66	61	55	44	41	**80**	**100**
Penetration (After RTFOT) % Original Pen.	77	71	65	60	56	47	42	38	33	**50**	
Softening point (Ring and Ball) ^{0}C	48.8	49.2	50.6	54.2	55.4	56.2	59.1	62.3	63.9	**45**	**52**
Softening Point (after RTFOT) ^{0}C	50.2	50.7	50.9	60.3	61.1	61.6	63.0	66.8	67.2	**45**	
Flash Point (Cleveland Open Cup) ^{0}C	250+	250+	250+	250+	250+	250+	250+	250+	250+	**225**	
Viscosity at 135 ^{0}C (VIRGIN) cSt	304.3	316.3	322.7	325.6	328.9	330.7	-	-	-		
Viscosity at 135^0c (AFTER RTFOT) cSt	315.1	330.4	331.9	333.8	-	-	-	-	-		
Ductility at 25^0C, cm	135.5	121.2	111.5	89.1	77.4	60.0	55.3	50.9	48.3	**100**	
Ductility at 25^oc (AFTER RTFOT), cm	124.8	111.2	103.1	79.3	68.6	51.6	44.2	39.1	32.6	**75**	
Specific gravity at 25^0C, g/cc	1.00	1.01	1.02	1.02	1.03	1.004	1.05	1.06	1.07	**1.00**	**1.05**
Thin Film Oven Test (TFOT), Aging Test	0.2	0.1	0.2	0.2	0.2	0.4	0.7	0.9	1.2		**0.5**
Solubility in Trichloroethylene %	99.9	99.6	99.6	99.5	99.3	99.1	99.0	98.8	98.3	**99**	

4.3.1: Penetration

The addition of varying percentages of rubber to neat bitumen 80/100 penetration grade altered the properties of the bitumen significantly (Table 4.2). The penetration reduced from 81 to 41 as the percentage of rubber was increased from 3% to 20% (Table 4.2). The aged samples gave penetration values slightly lower 71 to 33 as the percentage of rubber was increased from 3% to 20% (Table 4.2) notably that all the penetration values reduced as more rubber was added (Figure 4.1). The variation in penetration was because the aged samples became harder due to aging.

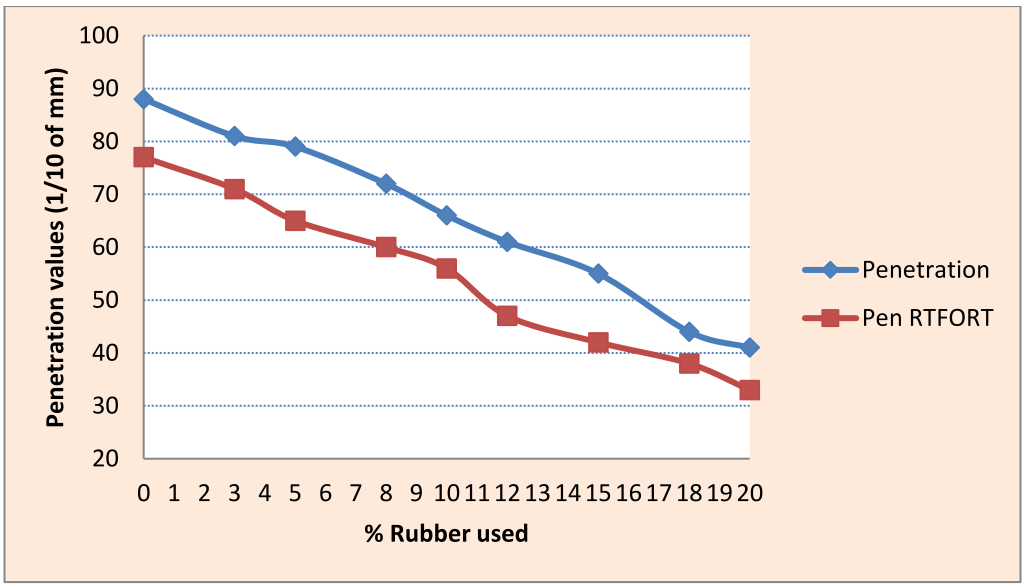


Figure 4.1: Penetration of bitumen sample against percentage rubber used

4.3.2: Softening Point

The softening point of both the neat and rubber modified bitumen increased as more rubber was added. Addition of 3% and 5% rubber gave 49.2 °C and 50.6 °C which did not have a significant effect on the softening point of bitumen (Table 4.2). Addition of 8% of rubber gave 54.2 °C as softening point (Figure 4.2). The softening point increased drastically for the aged modified bitumen to 67.2 °C when 20% of rubber was added and was 63.9 °C for the unaged bitumen when 20% of rubber was added. This implied that rubber modified bitumen was less susceptible to temperature induced deterioration (Swetha & Rani, 2014) and thus field conditions of high pavement surface temperatures will be a factor of less concern in rubber modified asphalt concrete mixes (Figure 4.2). This implied that rubber modified bitumen was less susceptible to temperature variations compared to unmodified bitumen.

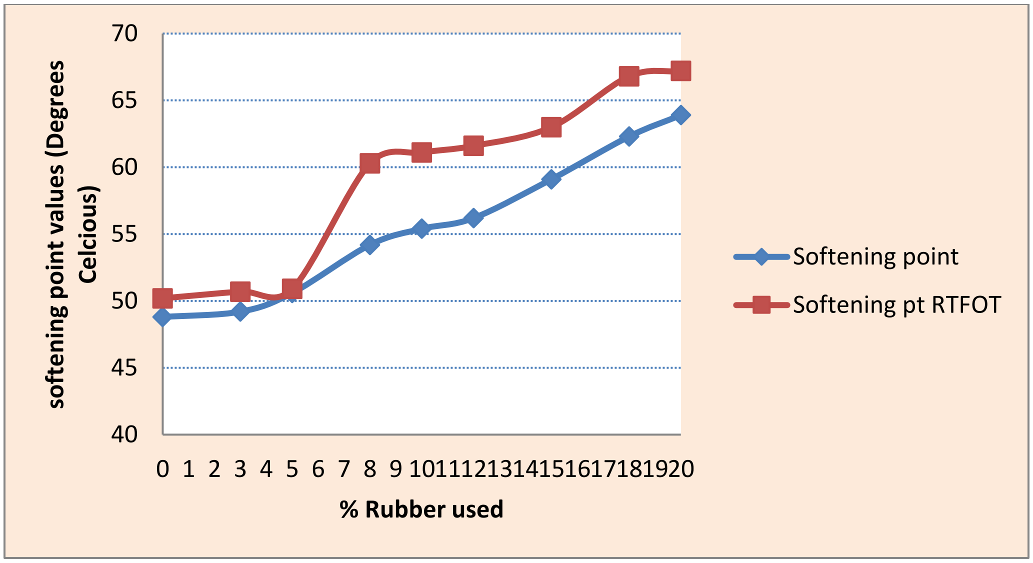


Figure 4.2: Softening point of bitumen sample against percentage rubber used

4.3.3: Flash Point

Flash point for both unmodified and rubber modified bitumen was all above the standard required minimum of 225^{0}C (Table 4.2) as per the Standard Specification for Roads and Bridges (1986). This implied that use of rubber in bitumen modification did not pose any risk of fire outbreak during preparation of the mixes at high temperatures.

4.3.4: Viscosity

From the results obtained it was noted that viscosity of unaged rubber modified bituminous binders increased as more rubber was added from 316.3 cSt to 322.7 cSt at 135 ^{o}C with 3 % to 5 % added rubber (Table 4.2), Addition of 12 % rubber gave the maximum value of viscosity of 330.7 cSt (Figure 4.3). It was noted that the viscosity at 135 ^{o}C of more than 12 % rubber added was undeterminable using standard reverse flow viscometers as the modified bitumen became very viscous to flow through the viscometers. This implied that percentages higher than 12 % of rubber modified bituminous binders could be a candidate for fatigue cracking in rubber modified asphalt concrete mixes at low pavement temperature (Kishore & Gottala, 2015).

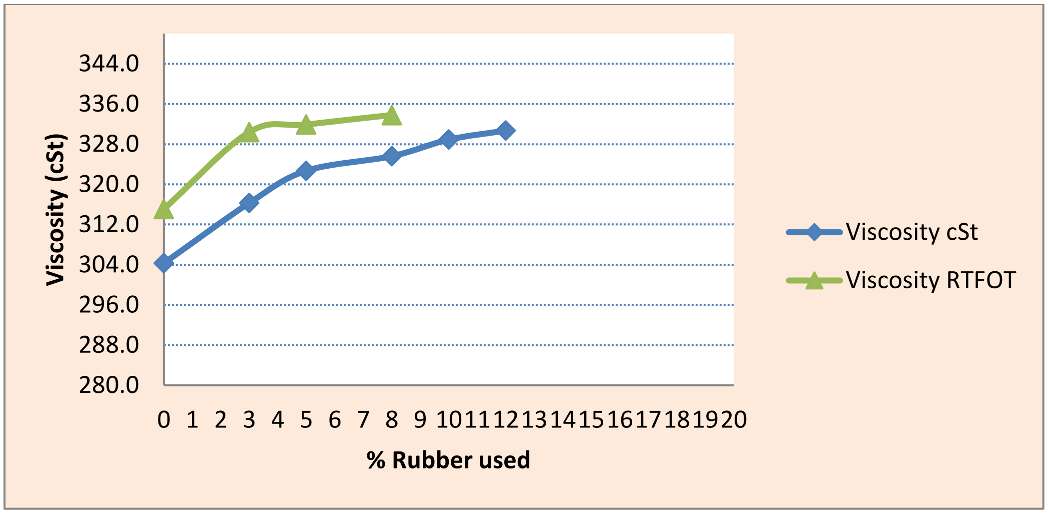


Figure 4.3: The viscosity of bitumen sample against percentage rubber used

4.3.5: Ductility

Ductility of both the unaged bitumen binder decreased from 121.2cm to 48.3cm at 25°C while the aged rubber modified bitumen decreased as more rubber was added from 111.2cm at 25°C to 32.6cm at 25°C as rubber was increased from 3 % to 20 % respectively (Table 4.2). It was further found that rubber addition to bituminous binders reduces the ductility of the binders by 10% at 3% rubber to 64% at 20% rubber added (Figure 4.4) thereby suggesting that the binders became stiffer with addition of more rubber (Swetha & Rani 2014).

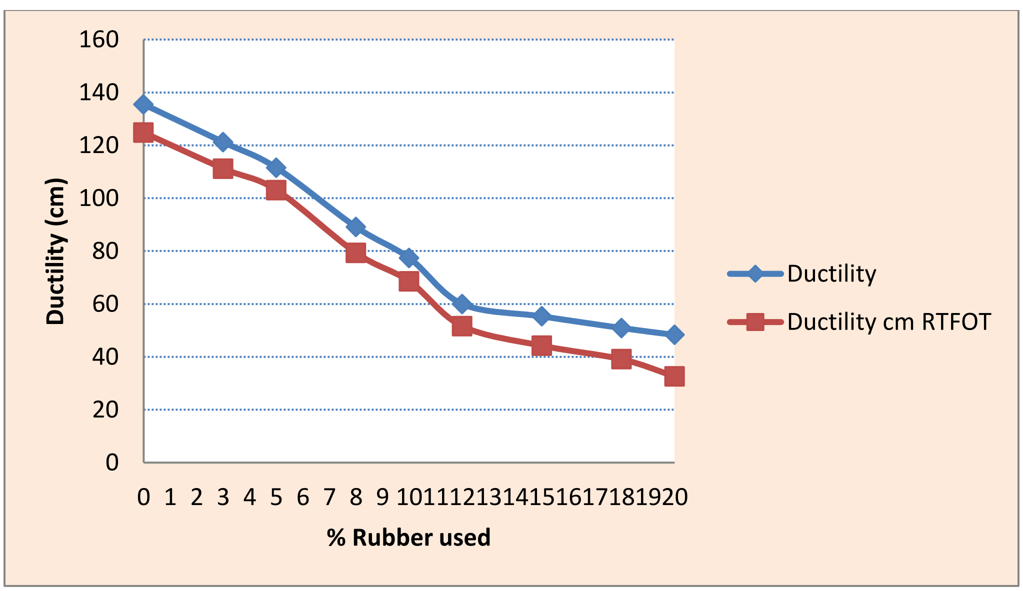


Figure 4.4: The ductility of bitumen sample against percentage rubber used

This factor is of concern in addressing the ever-increasing traffic and heavier loads on the Kenyan roads.

4.3.6: Specific Gravity

Addition of waste rubber in bituminous binders had little effect on the specific gravity of bituminous binders (Table 4.2) though the specific gravity increased as more rubber was added (Figure 4.5) hence rubber modified binders can withstand higher traffic loads (Swetha & Rani, 2014)

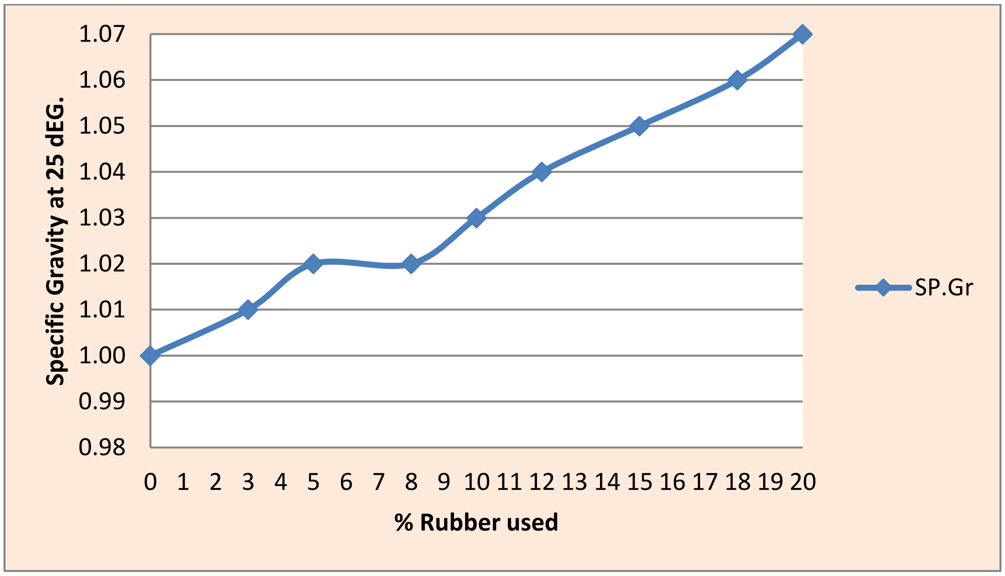


Figure 4.5: The specific gravity of bitumen sample against percentage rubber used

4.3.7: Loss on Heating

When the modified bitumen was aged, there was a tremendous loss on heating as more rubber was added (Figure 4.6). The addition of 15 % rubber and above was more prone to loss on heating which implied that addition of more rubber increased the quantity of volatiles by 0.7 % in the modified binders (Table 4.2). Therefore, the optimum amount of rubber ideal for the asphalt concrete was 10% as above this the minimum loss on heating of 0.5% given in standard specification (Ministry of Roads and Public Works., 1986) Table 4.1 was violated.

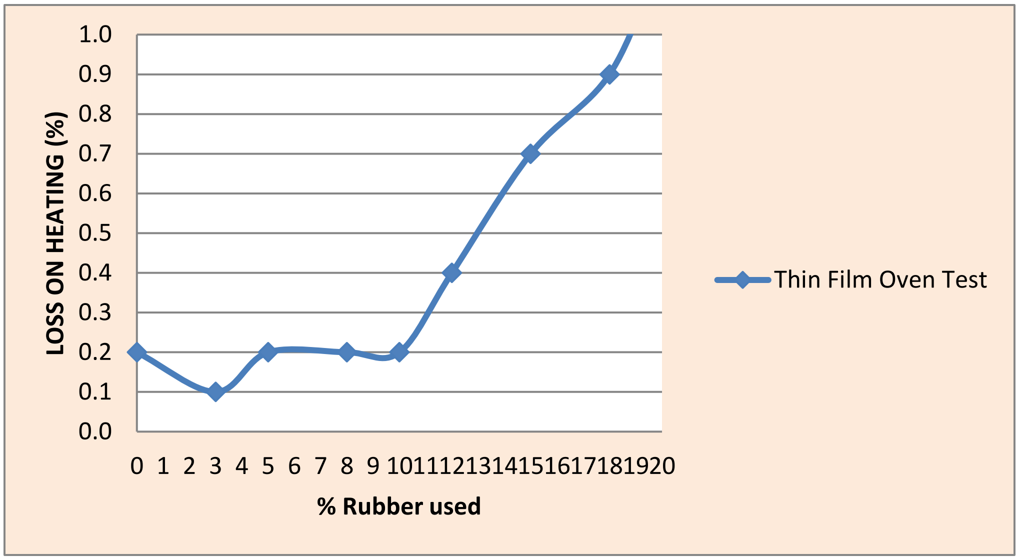


Figure 4.6: Loss on heating of bitumen sample against percentage rubber used

4.3.8 Solubility

Data obtained from this study showed that the solubility of rubber bituminous binders decreased from 99.6% as more rubber was added (Table 4.2) to 98.3 % at 20 % rubber added. The common practice is that bitumen binder solubility in trichloroethylene should not be way below 99 %. The study showed that 15% rubber content had 99.0% solubility thus higher percentages of rubber in bitumen modification could be prone to many impurities thereby compromising the quality of the binder (Figure 4.7).

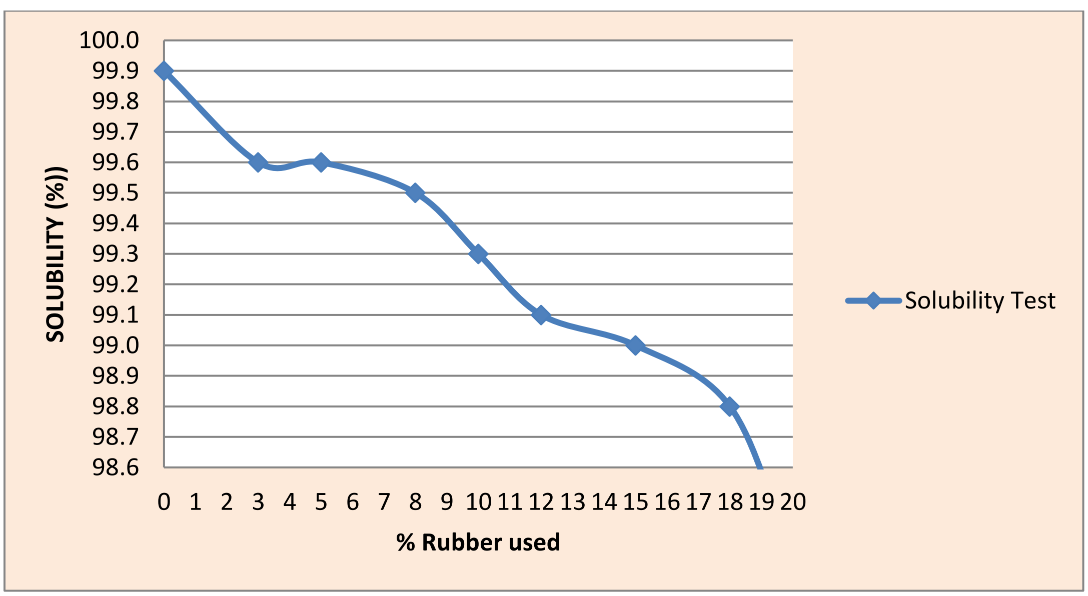


Figure 4.7: The solubility of bitumen sample against percentage rubber used

4.4 Formulation of asphalt concrete using crumb waste tyres

The asphalt concrete was designed by Marshall Method of mix design Mix design as per the procedures given in Manual Series-2, of the Asphalt Institute Overseas Road Note 19, 2002 from the Department for International Development, UK and the Road Design Manual Part III (1987) of the Ministry of Transport and Infrastructure Kenya.

4.4.1 Coarse aggregate samples used

The different single sizes of aggregates used for the mix design are as illustrated in the Table 4.3; the bitumen used was 80/100 penetration grade.

Table 4.3: Coarse aggregate samples used and bitumen

Material	Material Description
Coarse Aggregate	14~20mm
Coarse Aggregate	10~14mm
Coarse Aggregate	6~10mm
Fine Aggregate	0~6mm
Bitumen	80/100 Pen Grade

To ascertain the suitability of the aggregates for use in this study mechanical test were done on the aggregate samples in Table 4.3 and the outcome in Table 4.4 was obtained. This illustrated that the aggregates were mechanically sound for the study (Road Design Manual Part III, 1987). Aggregates used were 20 % flaky meaning that they complied the specification requiring that they be less than 25% flaky. Flaky aggregates are easier to break and therefore the aggregates were fit for the study (Road Design Manual Part III, 1987).

Table 4.4: Results for aggregate mechanical tests

TEST	Mechanical suitability results achieved (%)	Specification (Max %) RDM Part III-1987
ACV	18	25
SSS	8	12
LAA	16	30
FI	20	25

4.4.2: Aggregate Crushing Value (ACV)

ACV is a test that gives the resistance of an aggregate to crushing under a progressively applied compressive load and was done as per procedures given in BS 812 part 3. The aggregates used in this study complied with the specification that has a crushing value of 18% (Table 4.4) which is less than 25 % maximum requirement.

4.4.3: Loss Angelis Abrasion (LAA)

LAA is a test that produces abrasion by use of steel balls. This test gave the abrasion value of the aggregates in which case they complied with the specification requirements. Wu *et .al*, (1998) reported that the aggregates should be strong and resist scratch to avoid being crushed, degraded, and disintegrated on stockpiles, put into an asphalt plant, placed with a paver, compressed with steel and pneumatic rollers, and subjected to any extent of traffic loads. These tests were done to ascertain that these properties were all catered for. In their work, Saba et al. (2012) found out that low quality aggregates exhibited the least resistance to permanent deformation indicating that use of polymer modified binder would not compensate for low quality aggregates. The LAA of the aggregates used gave result value of 16% (Table 4.4) which is lower than the recommended maximum of 30%. This shows that the aggregates are resistant to abrasion and therefore fit for the study. This is the reason why this research embraced testing of the mechanical properties of the aggregates before their use.

4.4.4: Sodium Sulphate Soundness

Sodium Sulphate Soundness (SSS) a method used to subject the sample to disruptive effect of repeated crystallization and rehydration of magnesium sulphate heptahydrate ($MgSo_4.7H_2O$) within the pores of the aggregates was used to test the soundness of aggregates to chemical weathering in which case the aggregates complied with a result value of 8% (Table 4.4). These results show that the aggregates are mechanically sound and fit for the study.

Aggregates to be used in designing an asphalt concrete must meet some specification in terms of particle size distribution. These specifications (Table 4.5) are given in Road design manual part III of 1987 and the Standard Specification for Roads and Bridges of 1986 as shown below. The Table 4.5 is important because it gives the specification grading for different types of asphalt concrete. In undertaking this study, we chose asphalt concrete 0/20mm binder course. In the Table 4.5 below the ranges 0.075-28 signify the particle size distribution in the formulated asphalt concrete using British Standard sieves.

Table 4.5: Aggregate grading requirements

Sieve size (mm)	**% By Weight passing**							
	Type I						**Type II**	
	Wearing course			**Binder course**			**Wearing Course**	
	0/14	0/10	0/6	0/20	0/14	0/10	0/14	0/10
28	-	-	-	100	-		-	-
20	100			90-100	100		100	
14	90-100	100		75-95	90-100	100	90-100	100
10	70-90	90-100	100	60-82	70-90	90-100	70-95	90-100
6.3	55-75	60-82	90-100	47-68	52-75	60-82	55-85	62-90
4	45-63	47-67	75-95	37-57	40-60	45-65	46-75	50-80
2	33-48	33-50	50-70	25-43	30-45	30-47	35-60	35-65
1	23-38	23-38	33-50	18-32	20-35	20-35	25-45	25-50
0.425	14-25	14-25	20-33	11-22	12-24	12-24	14-32	14-33
0.300	12-22	12-22	16-28	9-17	10-20	10-20	11-27	11-27
0.150	8-16	8-16	10-20	5-12	6-14	6-14	6-17	6-17
0.075	5-10	5-10	6-12	3-7	4-8	4-8	3-8	3-8

Source: Standard Specifications for Roads and Bridge Construction (1986)

4.4.5 Grading envelope specification curve Determination

The aggregates should be able to meet target grading envelope specification (Table 4.5). The single size aggregates were grade using the British Standard (BS) sieves. The combined grading of the different single size aggregates was computed and a plot of the same was done on the grading envelop (Figure 4.8).

The grading curve should lie within the given specification envelope as illustrated in the Figure 4.8, where the upper and lower curves are the limits and the middle graph is the combined grading of all the single size aggregates constituting the mix, Standard Specification for Road and Bridge, (1986). The different sizes of aggregates were graded using the standard BS sieves in the Table 4.5 and a curve of the combined grading was plotted in the grading envelop for the chosen mix (Figure 4.8). If the curve lies within the envelop, then the aggregates blend is considered ok for the mix in which case the aggregates complied Standard Specification for Road and Bridge (1986).

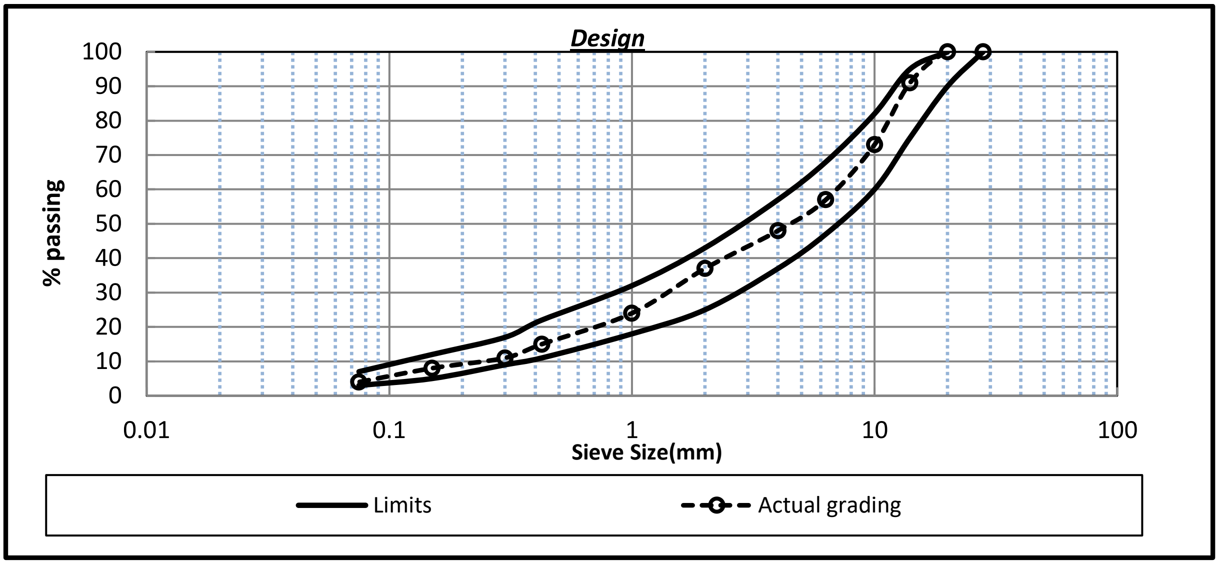


Figure 4.8: Grading envelope specification curves

4.4.5: Aggregate single size grading

Each of the single size aggregates was graded using British Standard (BS) sieves to determine the particle size distribution as shown in Table 4.6. The single size grading was used to determine the combined grading of all the aggregates. The following results were obtained for the single size grading using various single size grading sieves (Table 4.6).

Table 4.6: Aggregate single size grading

Single size aggregates (mm)	Sieve sizes (mm)												
	28	20	14	10	6.3	4	2	1	0.425	0.300	0.150	0.075	Flakiness Index (FI)
0/6(% passing)	100	100	100	100	99	83	57	41	26	22	17	11	-
6/10 (% passing)	100	100	100	84	23	0.4							22
10/14 (% passing)	100	100	95	7	0.1								19
14/20 (% passing)	100	89	22	0.4									17

The British Standard sieves used for the grading to determine the particle size distribution ranged from 0.075 – 28 mm in size. The grading above Table 4.6 shows the grading as the percentage passing each sieve from the smallest sieve 0.075mm to the largest sieve 28mm.

4.4.6: Aggregate single size and combined grading

The single sizes grading was proportioned (Table 4.7) and produced a theoretical blend which when plotted in the grading curve (Figure 4.9) was found to lie within the limits of the desired grading envelop of an AC 0/20 mm envelop as shown in Table 4.7

The formula used for proportioning is as shown in equation 4.1

P=Aa+Bb+Cc +Zz ……………………………………….. equation 4.1

Where capital letters A, B, C …Z was the percentage proportion chosen for each single aggregate size and small letters a, b, c…z was the percentage passing per each sieve for each single size.

The best combined grading was obtained using equation 4.1 where the proportions of single size aggregates were determined and plotted in the grading envelop (Figure 4.9).

Table 4.7: Combined single and composite size grading

Aggregate size	14/20mm	10/14mm	6/10mm	0/6mm	GRADING			
	PROPORTIONS				Theoretical grading	Composite grading	Standard specification	
Sieve	15	8	27	50			min	max
28	100	100	100	100	100	100	100	
20	89	100	100	100	98	98	90	100
14	22	95	100	100	88	87	75	95
10	0.4	7	84	100	73	75	60	82
6.3		0.1	23	99	56	52	47	68
4			0.4	83	42	41	37	57
2				57	29	28	25	43
1				41	21	20	18	32
0.425				26	13	14	11	22
0.3				22	11	11	9	17
0.15				17	9	9	5	12
0.075				11	6	6	3	7
Flakiness Index (FI)						18		

The grading mix proportions adopted for the mix in Table 4.7 were obtained as shown Table 4.8

Table 4.8: Aggregate size and proportions cluster values.

Aggregate size ((mm)	Proposition cluster (%)
0/6	50
6/10	27
10/14	8
14/20	15

To ascertain that these theoretical proportions were within the standard specification for the mix an actual composite curve was drawn by doing actual grading using the above proportions where a batch weight of 1200 grams was used Road Design manual Part III (1987). This was done by mixing the percentage weights of individual single sizes together as follows and doing the grading and combined Flakiness (FI)

Batch Weight (1200g) = (50/100 *1200) + (27/100*1200) + (8/100*1200) + (15/100*1200) The composite grading curve was drawn and was found to overlap the theoretical one implying that the proportions were within the standard specification in Road Design manual Part III (1987) and fit for the design (Figure 4.9).

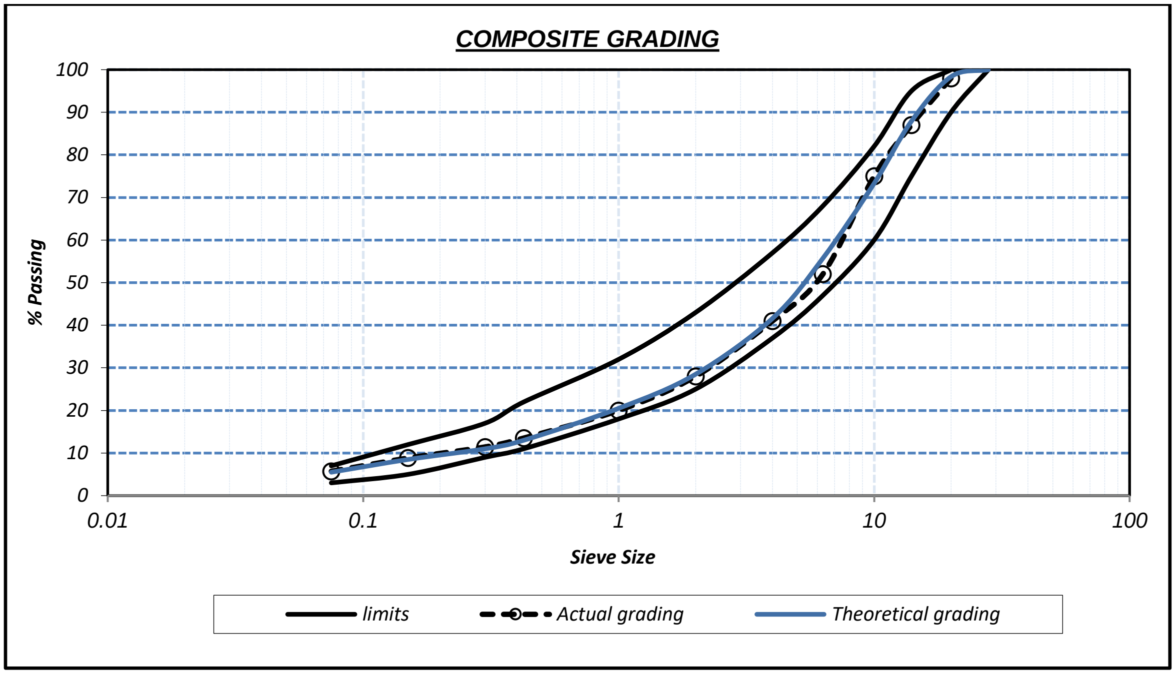


Figure 4.9: Theoretical and composite grading curves

Using the proportions in Table 4.6, 16 batches of 1100 g combined batch weights were prepared and molded and gave the results shown in Table 4.8 upon analysis. The anticipated optimum binder content for a selected blend of aggregates was determined by testing specimens prepared at bitumen contents, which have a range spanning through the expected optimum value for the design (ORN 19, 2002).

4.5: Volumetric Computation

4.5.1: Stability and Flow of asphalt concrete mix

The bulk specific gravity of each specimen briquettes was determined as described in ASTMD 2726 method and gave the results shown in Table 4.9. Stability and flow were recorded directly from the Marshall Tester equipment and the results were as shown in Table 4.9.

Other parameters of voids in mineral aggregates (VIM), voids in mix (VMA) and voids filled with binder (VFB) were determined through computation from the maximum specific gravity determined by the methods described in ASTM D2041 and MS-2 of 2014. The maximum specific gravity of the mixes was found to be as shown in the Table 4.9

4.5.2: Calculation of Voids in Mix (VIM)

Voids in mix are given as:

$$\text{Voids in Mix (VIM)} = \frac{100\,(Gmm-Gmb)}{(Gmm)} \quad \text{…………………} \text{ equation 4.2}$$

Where;

Gmm = maximum specific gravity of mix

Gmb = bulk specific gravity of compacted mix

The Voids in mix decreased from 9.2% as more bitumen was added to 5.8% (Table 4.9) implying that the voids in the mix decrease as more bitumen is added. The value for the voids in the mix should not be less than 3% at refusal density (ORN 19, 2002) as such mixes will be prone to plastic deformation.

4.5.3: Calculation of Voids in Mineral Aggregates (VMA)

Voids in Mineral Aggregates are given as:

$$\text{Voids in Mineral Aggregates (VMA)} = \frac{100-((Gmb*Ps)}{(Gsb)} \quad \text{…..……… …………. ….} \text{ equation 4.3}$$

Where;

Gmb = bulk specific gravity of compacted mix,

Gsb= bulk specific gravity of total aggregates and Ps= percentage of aggregates by weight in the mix. VMA is the total void space between aggregate particles in a compacted asphalt concrete mix Asphalt Institute (2014). The minimum VMA of the mix was 17.0% a value, which is sufficient to accommodate the binder after compaction. Sufficient VMA allows the Voids in the mix to remain above 4.0% after compaction (ORN 19, 2002).

4.5.4: Calculation of Voids Filled with Binder (VFB)

Voids Filled with Binder is given as (VFB) = 100((VMA-VIM)/VMA) ……………. equation 4.4

Where VMA = Voids in Mineral Aggregates and VIM = Voids in mix

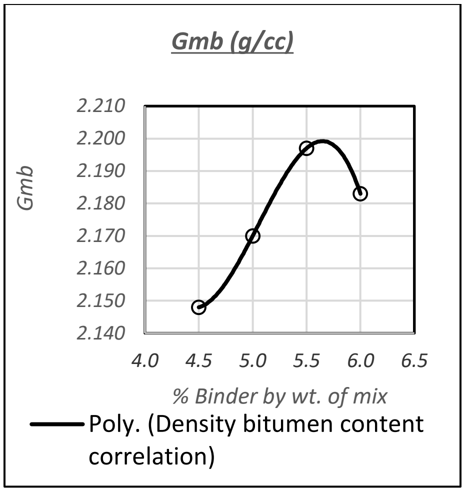
The Voids filled with binder had a range of 48.9 % to 67.7 % (Table 4.9) as more bitumen was added. This shows the required minimum of 65% Voids filled with binder was achieved.

Table 4.9: Volumetric analysis

Materials used	Specific. Gr.		Specimen Number					Specification	
	App	Bulk		1	2	3	4	min	max
14~20 mm	2.586	2.458	P1	14.3	14.3	14.2	14.1		
10~14 mm	2.631	2.494	P2	7.6	7.6	7.6	7.5		
6~10 mm	2.680	2.465	P3	25.8	25.7	25.5	25.4		
0~6 mm	2.740	2.536	P4	47.8	47.5	47.3	47.0		
Total Aggregate	2.690	2.501	P5	95.5	95	94.5	94		
Bitumen	1.02	N/A	P6	4.5	5.0	5.5	6.0		
Bulk Sp. Gr (mb), Compacted mix (ASTM D2726)				2.148	2.170	2.197	2.183		
Bulk Sp. Gr (Gsb), Total Aggregates				2.501	2.501	2.501	2.501		
Effective specific Gravity (Gse)				2.522	2.522	2.522	2.522		
Binder Absorption (Pba)				0.3	0.3	0.3	0.3		1.0
Max. Sp. Gr. (Gmm) Paving mixture				2.365	2.349	2.333	2.317		
Voids in Mineral Aggregates (V.M.A)				18.0	17.6	17.0	18.0		
Voids in mix (VIM)				9.2	7.6	5.8	5.8	3	7
Voids Filled with Binder (V.F.B)				48.9	56.6	65.7	67.7	65	70
Stability (KN)				11.5	11.8	12.2	11.1	9	18
Flow Value (mm)				2.6	2.7	2.9	3.0	2	4

4.6: Marshall Design Curves

The results were plotted in graphs to obtain smooth curves of density, stability, VMA, VIM, VFB and flow respectively (Figures 4.10: Marshall Design curves (a) - (f)) all against bitumen contents.



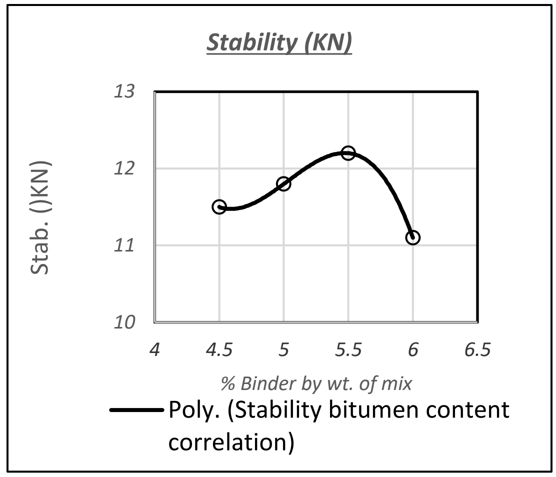
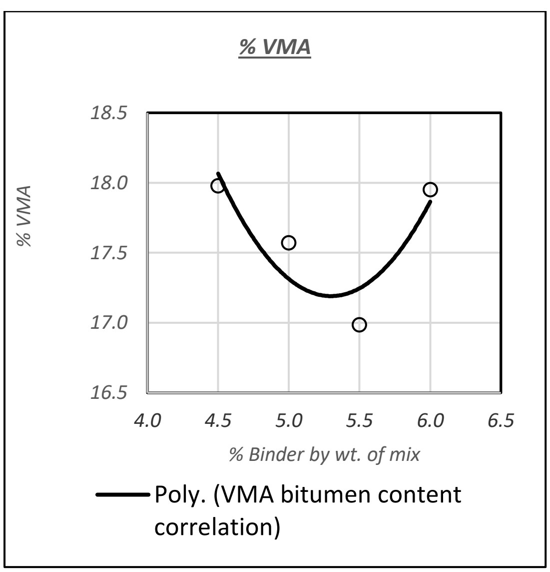


Figure 4.10 (a): Gmb of compacted mix. Figure 4.10 (b): Stability of asphalt concrete mix

The bulk density of the mixes increases as more binder is added up to a maximum value after 5.5 % binder by weight, it starts to fall (Figure 4.10 (a)). This trend is shown because at constant volume the increasing binder replaces air of low density until after maximum where at constant volume the volume of binder exceeds that of more dense aggregates thereby reducing the density (Asphalt Institute, 2014)

Stability increases to a maximum of 5.5 % of binder addition and reduces (Figure 4.10 (b)), therefore the binder content giving maximum stability was taken as the optimum bitumen content (Figure 4.10 (b))



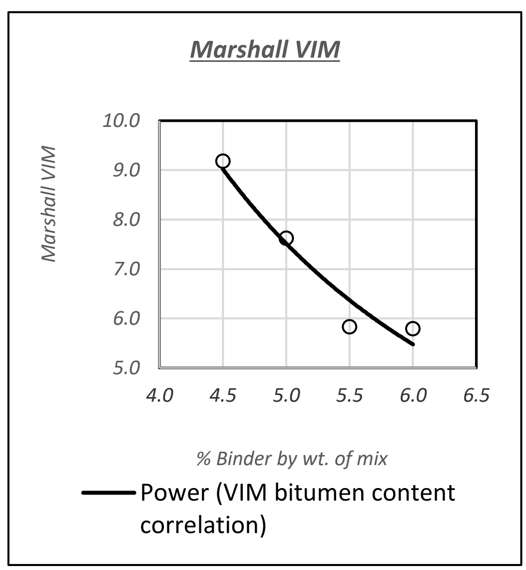
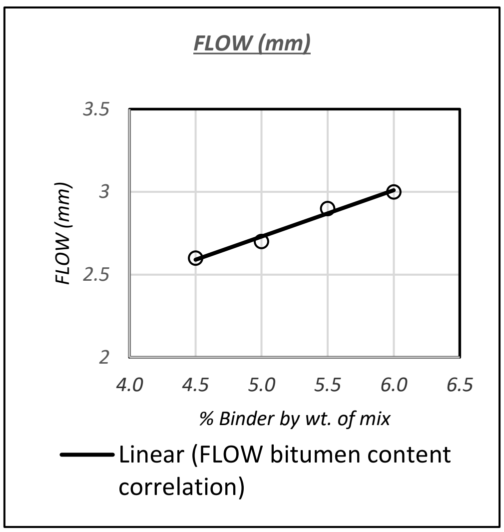


Figure 4.10 (c): Voids in Mineral Aggregates (VMA), Figure 4.10 (d): Voids in Mix for asphalt concrete

The VMA (Figure 4.10 (c)) reduces to a minimum of 17.3 % upon addition of more binder and increases again because less dense binder pushes the dense aggregates further apart thereby increasing the VMA (ORN 19, 2002). Asphalt content on the west side of the mix where the VMA starts to increase (Figure 4.10 (c)) are avoided because they can lead to rutting when applied on the pavement because they have less voids to accommodate compaction from traffic loads that's why the binder content on the dry side of the mix was used (ORN 19, 2002).

Estimation of optimum binder content for the mix by Flow (Figure 4.10 (e)) and VIM properties (Figure 4.10 (d)) has been tabulated in the Standard Specification for Roads and Bridges as the binder content corresponding to median flow 2.75 mm and 4-7% VIM, Ministry of Transport and Communications (1987).



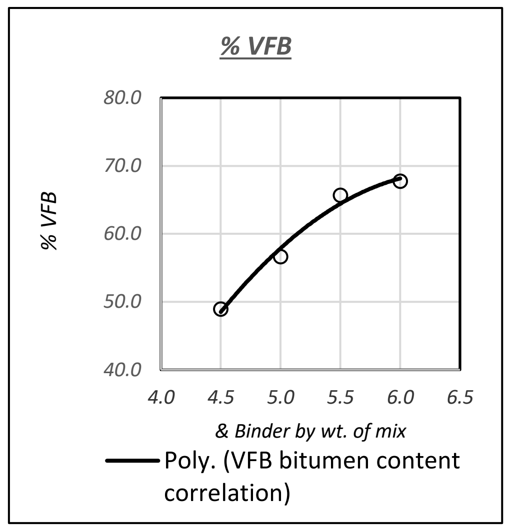


Figure 4.10 (e): Marshal Design curve for Flow Value, Figure 4.10 (f): Voids Filled with binder

4.6.1: Optimum Binder Content by Marshall Method

The optimum binder content (OBC) value for each parameter were read from the design curves (Figures 4.10 (a) – (f)) as shown in Table 4.10 below.

Table 4.10: Optimum Binder Content from Marshal Design curves

Parameter	BC
1. At maximum Density	5.7
2. At maximum Stability	5.5
3. At 6.0 % voids in mix (VIM)	5.6
4. At 2.75 mm Flow	5.2
5. At 17.2 % voids in mineral aggregates (VMA)	5.2
6. At 65 % voids filled with binder (VFB)	5.5
TOTAL	32.7

The optimum binder content from Marshal Design curves is the average 32.7/6 =5.5 %

The average of the results from Table 4.10 was taken as the optimum percentage (5.5%) of bitumen by weight of total aggregates needed for the asphalt concrete mix AC 0/20 mm. However, according to (ORN 19, 2002) the degree to which the aggregates interlock and shear friction between aggregate particles is an important factor of resistance of asphalt concrete mixes to shear failure (ORN 19, 2002).

This is contributed by that fact that rounded aggregates can meet the lowest VIM requirement when compacted to refusal but the same aggregates have little aggregate interlock, which can lead to plastic deformation under heavy traffic (ORN 19, 2002). This prompted the refining of the bitumen mix further by vibrating hummer method (Thakre et al., 2016) to take care of such worries and lay the sole cause of pavement failure to be as a result of the binder being used and its weaknesses. Considering this aspect, we banked on the idea that rubber modified bitumen was to be the solution to plastic deformation of properly designed asphalt concrete mixes.

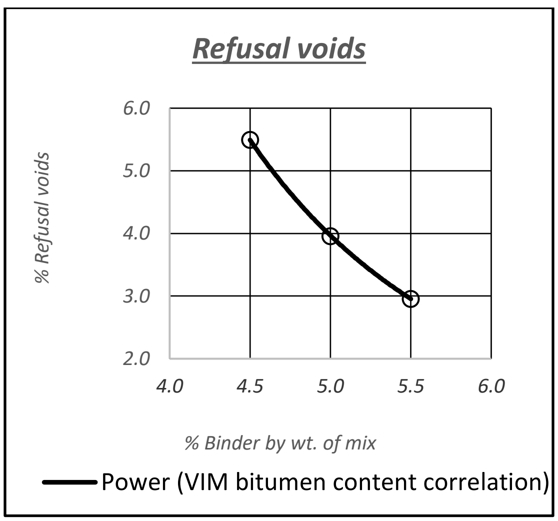
4.6.2: Refusal Density by Vibrating Hammer Method

The results in Table 4.11 were obtained after subjecting the mix to refusal density by vibrating hummer (ORN 19, 2002).

Table 4.11: Refusal density by vibrating hummer

Binder Content	4.5	5.0	5.5
Refusal Density	2.235	2.256	2.264
Maximum Specific Gravity	2.365	2.349	2.333
Refusal Voids in Mix	5.5	4.0	3.0
Voids in Mineral Aggregates	14.7	14.3	14.5
Voids Filled with Binder	62.5	72.3	79.5

It was noted that the Voids in the Mix at refusal density did not go below 3.0 % (Table 4.11) conforming to the requirements specified in Road Design Manual Part III, 1987. The Voids in mix decreased from 5.5 % as more bitumen was added to 3.0 % (Table 4.11) implying that the voids in the mix decrease as more bitumen is added. The value for the voids in the mix should not be less than 3.0 % at refusal density (ORN 19, 2002) as such mixes will be prone to plastic. The VIM, VMA, VFB were computed as per the Marshall criteria and their smooth design curves against bitumen content were plotted as shown in Figures 4.11 (a)-(c)



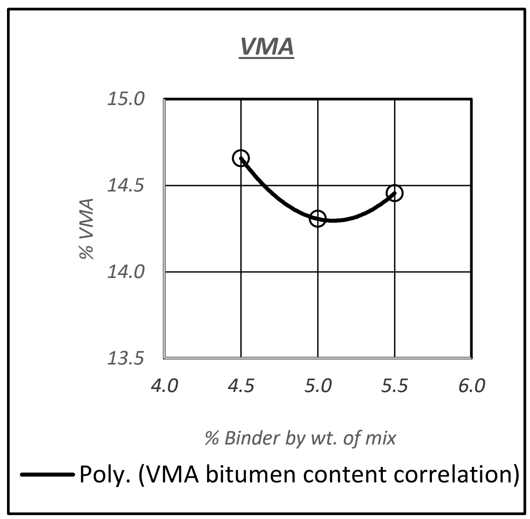


Figure 4.11: Vibrating hammer design curves

Figure 4.11: (a): Refusal density voids in mix (VIM), Figure 4.11 (b): Voids in mineral aggregates (VMA)

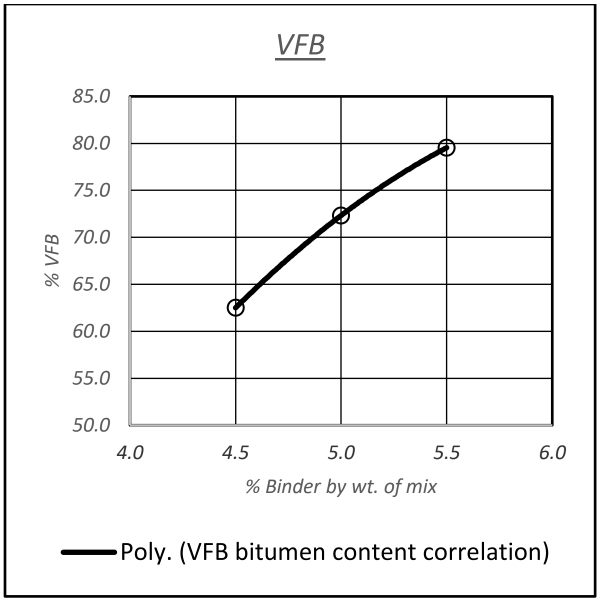


Figure 4.11 (c): Voids filled with Binder

The values for each parameter (Table 4.12) were read from the design curves in Figures 4.11 ((a)-(c))

Table 4.12: Vibrating hammer design criteria

Parameter	%BC
At 4.0% Refusal Voids	5.0
At 14.3% Voids in Mineral Aggregates	5.0
At 75% Voids Filled with Binder	5.2
TOTAL	15.2

The average design binder content from vibrating hammer test was 15.2/3 = 5.1 %

The average design binder content from vibrating hammer test was obtained as 5.1 %. This showed that the mix reduced its binder content requirement when the degree to which the aggregates interlock was at maximum of 5.1 % after secondary compaction. According to ORN 19 (2002), rounded aggregates can meet the lowest VIM of 3.0 % required when compacted to refusal but the same aggregates have little aggregate interlock which can lead to plastic deformation under heavy traffic (ORN 19, 2002). This is what prompted this study to refine the mix further by vibrating hammer method (Asphalt Institute, 2014).

4.7: Evaluation of formulated asphalt concrete

The asphalt concrete mix was analyzed for several parameters to determine the effect of waste crumb rubber on several parameters.

4.7.1: Marshall Properties values

The effect of rubber-modified bitumen on the density, stability, flow and tensile strengths of asphalt concrete mix are shown on Table 4.13.

Table 4.13: Results for properties of rubber modified bitumen mixes

Rubber waste content (%) dose	PROPERTIES									
	Bulk Density, g/cc	Marshall Stability, KN	Flow values, mm	Marshall Quotient	Max. Sp. Gravity, g/cc	Voids in mix, %	Voids in Mineral Aggregates, %	Voids Filled with Binder, %	ITS, N	Elastic Modulus, mPa
	A	B	C	D=B/C	E	F=100*(1-(A/E))				
0	2.242	10.66	4.5	2369	2.329	3.7	14.9	76.8	598	3748
3	2.249	11.98	4	2995	2.330	3.5	14.8	77.7	619	3876
5	2.252	10.92	3.6	3033	2.330	3.4	14.4	80.0	813	5059
8	2.262	12.45	3.2	3891	2.331	3.0	13.6	85.5	842	5236
10	2.260	14.15	3	4717	2.332	3.1	14.2	81.6	734	4577
12	2.251	15.41	2.8	5504	2.332	3.5	15.0	76.7	597	3742
15	2.226	11.26	2.6	4331	2.333	4.6	16.0	71.2	435	2754
18	2.205	10.25	2.3	4457	2.334	5.5	16.8	67.0	398	2528

From the data in Table 4.13, the percentage change in the parameters of stability, tensile strength and elastic modulus were calculated using the following equations.

$$\%\ \text{Increase in stability } (\%\ IS) = \frac{(\text{Maximum Stability} - \text{Minimum stability})}{\text{Maximum stability}} * 100 \quad \ldots\ldots \text{equation 4.5}$$

$$\%\ IS = \frac{(15.41-10.66)}{15.41} * 100 = 30.8\%$$

The minimum value of stability is taken as the stability obtained before addition of rubber and this was the stability for the neat asphalt concrete which was 10.66KN. The value 10.66KN represent the baseline value for the neat bitumen. The maximum stability was the stability maximum obtained with addition of rubber which was 15.41KN.

% Increase in Tensile strength (% ITS) = $\frac{(\text{Maximum strength- minimum strength)}}{\text{Maximum strength}}$ *100----- equation 4.6

$$(\%\ ITS) = \frac{(842-598)}{842} *100 = 29.0\ \%$$

% Increase in elastic modulus (% IEM) = $\frac{(\text{Maximum elasticity-minimum elasticity}}{\text{Maximum elasticity}}$ *100--- equation 4.7

$$\%\ IEM = \frac{(5236-3748)}{5236} *100 = 28.4\ \%$$

These calculations from equations 4.5-4.7 showed that rubber improved characteristics of bitumen almost in the same range as in Table 4.14 by Kishore and Gottala (2015)

Table 4.14: Improvement properties of rubber modified bitumen

Property	% Increase
Stability	30.8
Tensile strength	29.0
Elastic Modulus	28.4

4.7.2: Effects of rubber on the density of asphalt concrete mixes

When crumb waste tyre was used to determine its effect on the density of asphalt concrete mixes, the results showed that the density of rubber modified bitumen mixes increased to a maximum of 2.262g/cc from 2.242g/cc and reduced with addition of more rubber to 2.205g/cc (Figure 4.12). As shown in Figure 4.12 the rubber content which gave the maximum density of 2.262 g/cm^3 was 8.0 % and therefore this was taken as the optimum rubber content to produce the highest density of modified bitumen mixes (Swetha & Rani (2014).

At maximum density ORC = 8.0 %

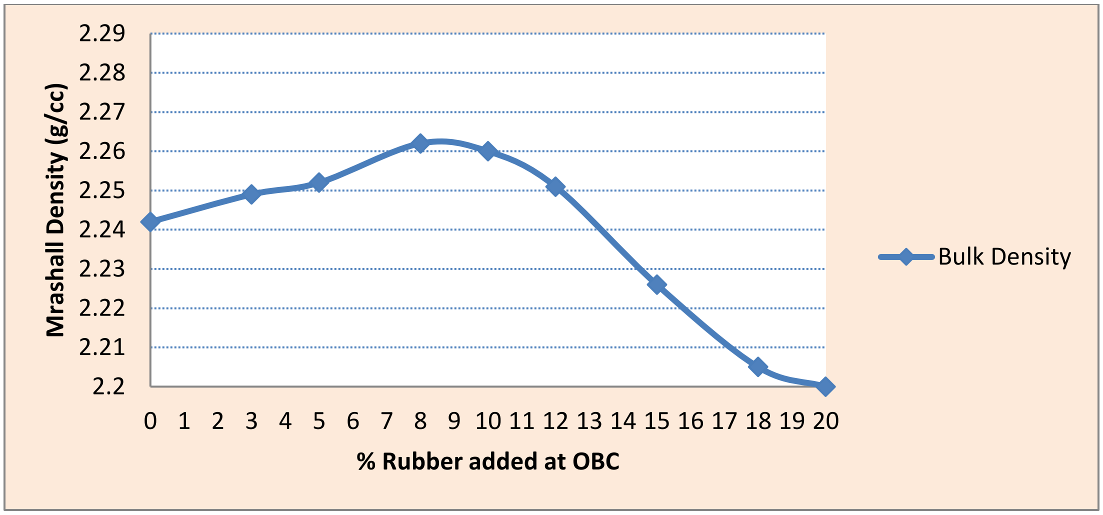


Figure 4.12: Bulk density of rubber modified bitumen mixes

4.7.3: Stability of rubber modified bitumen mixes

The stability of rubber modified bitumen mixes was maximum of 15.41 KN at optimum rubber content of 11.5% rubber (Figure 4.13) and therefore this was taken as the optimum rubber content to produce the maximum stability of the mix (Swetha & Rani, 2014).

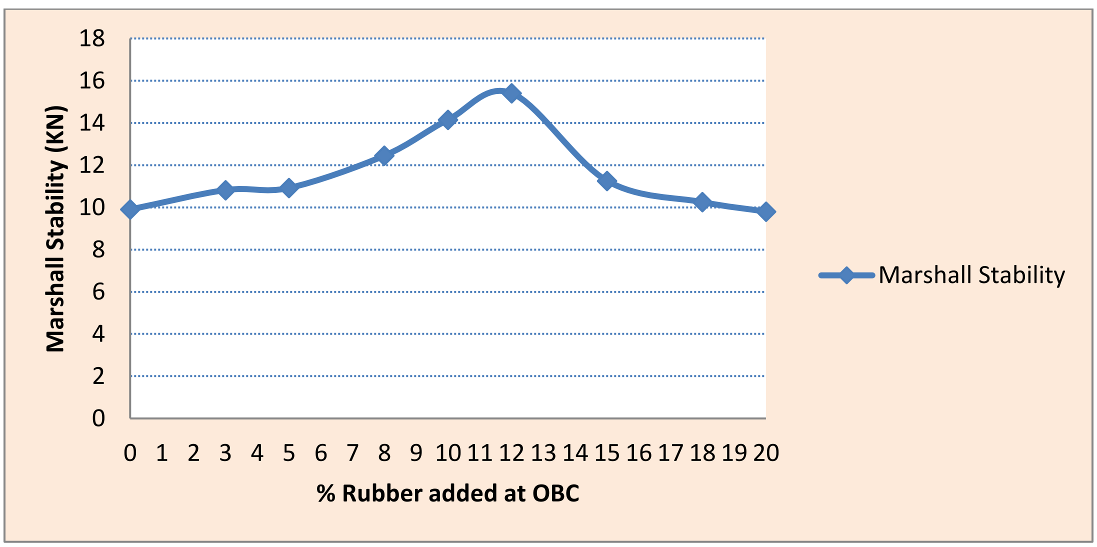


Figure 4.13: The stability of rubber modified bitumen mixes

4.7.4: Flow of rubber modified bitumen mixes

The flow of rubber modified bitumen mixes reduced from 4.5 mm as more rubber was added and to deduce the optimum rubber content of 8.0 % with median flow of 3.2 mm was taken as the one representing the optimum amount of rubber for the mix (Figure 4.14). The median flow for rubber modified asphalt concrete was used to determine the optimum amount of rubber ideal for the asphalt concrete. At median flow (3.2 mm) ORC = 8.0 %.

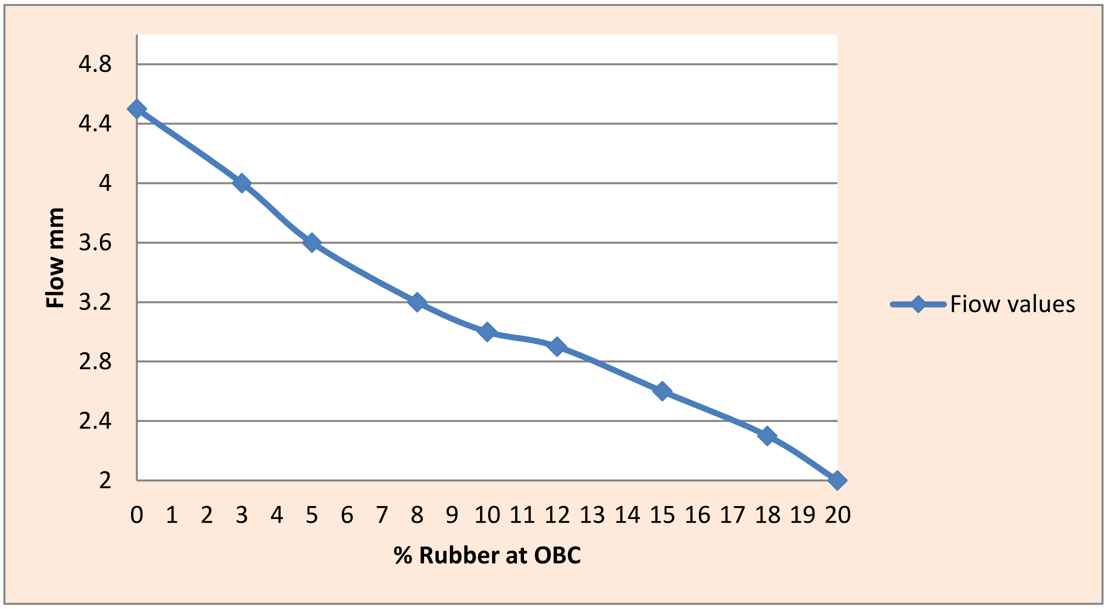


Figure 4.14: The flow of rubber modified bitumen mixes

4.7.5: Voids in Mix (VIM)

Rubber content that produced desired voids of 4.0 % was found to be 13.3 % and thus this was taken as the optimum amount of rubber for the rubber modified bitumen mix (Figure 4.15). 4.0 % voids in mix were considered because according to Standard Specification for Roads and Bridge Construction it is the stipulated minimum voids in a mix that will not undergo rutting when applied on a pavement (Asphalt Institute., 2014).

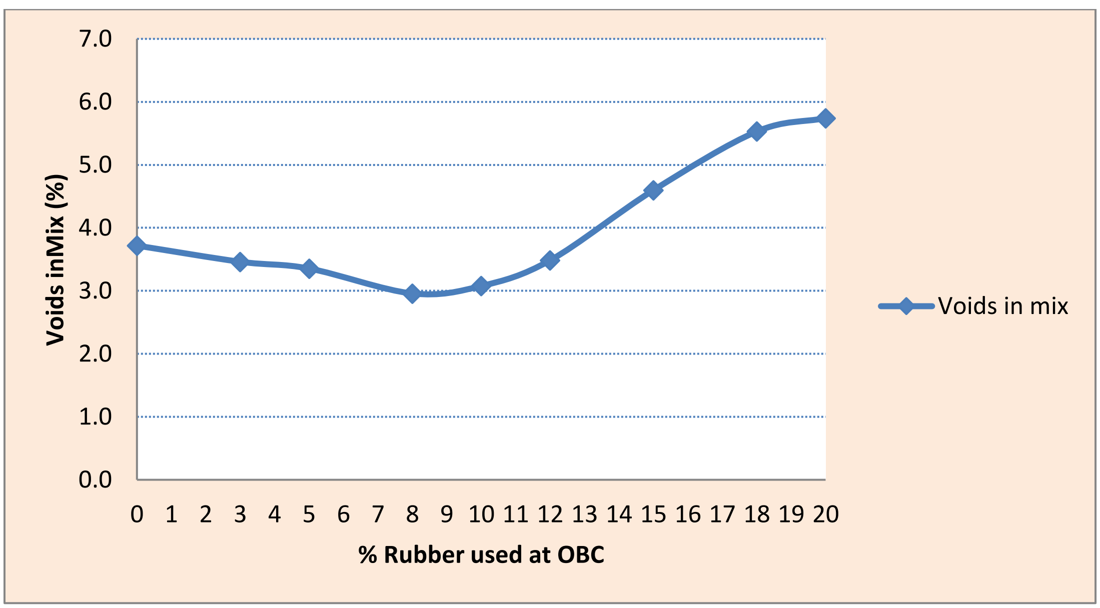


Figure 4.15: VIM of rubber modified bitumen mixes

It was found that at 4.0 % VIM ORC =13.3%

4.7.6: Voids in Mineral Aggregates (VMA)

The voids in mineral aggregates gave an optimum rubber content of 8.0 % at 14.5 % VMA when extrapolated to the dry side of the mix (Figure 4.16).

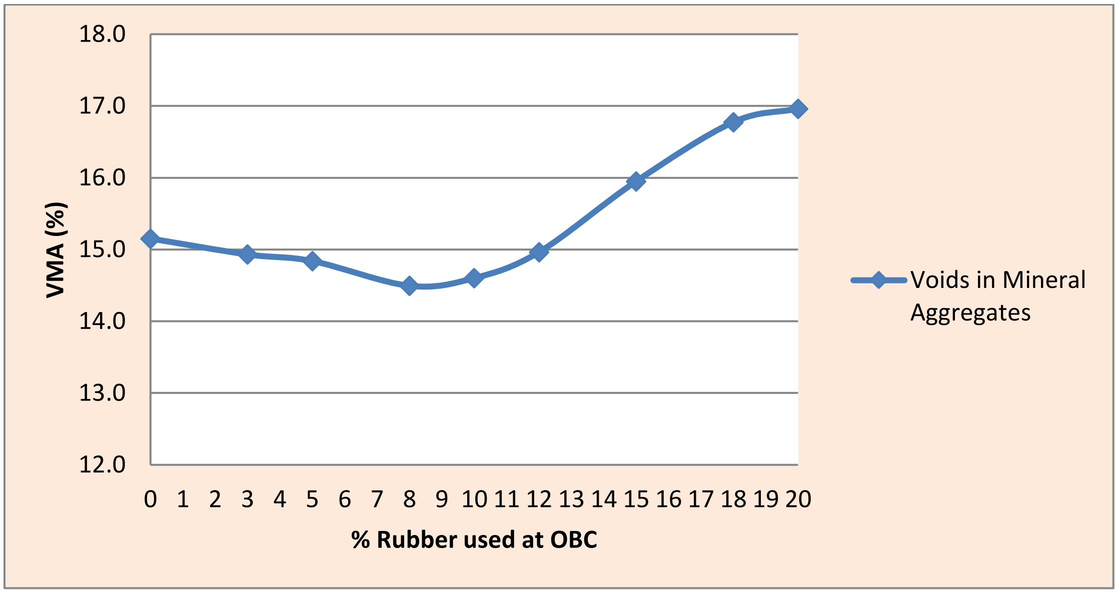


Figure 4.16: VMA rubber modified bitumen mixes

4.7.7: Voids Filled with Binder (VFB)

It was noted that voids filled with binder (Figure 4.17) gave the highest optimum of the amount of rubber needed for an asphalt concrete mix implying that more rubber was needed to fill up the voids, thus at 70% VFB ORC =15.5%.

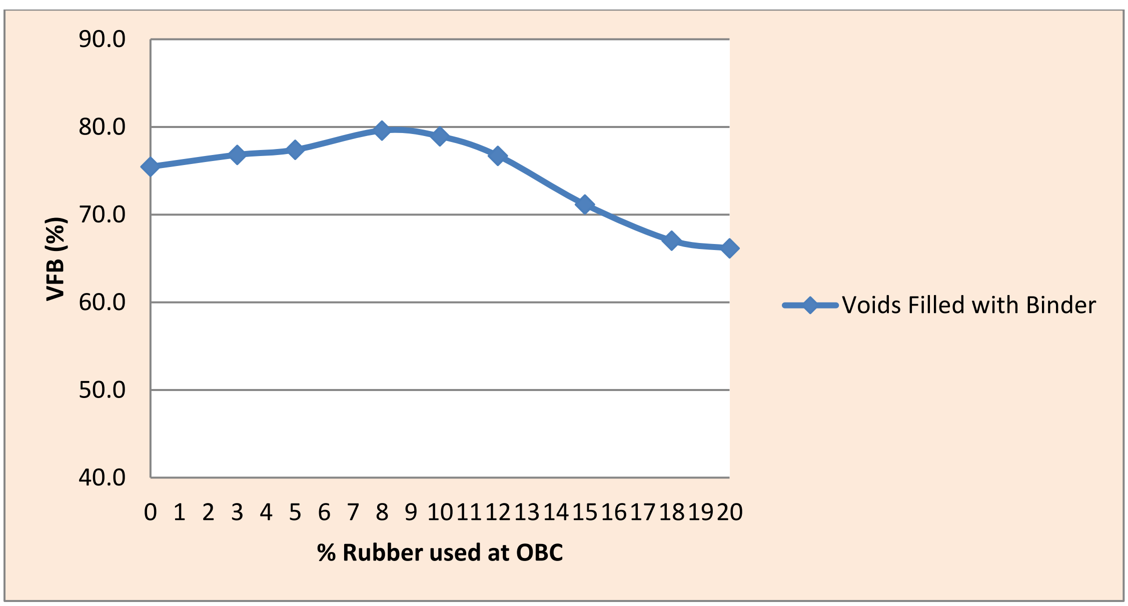


Figure 4.17: VFB rubber modified bitumen mixes

4.7.8: Indirect Tensile Strength

It was noted that the tensile strength of rubber modified binders increased from 600 kPa as more rubber was added but reached a maximum of 842 kPa before it dropped to 597 kPa (Figure 4.18). This characteristic implied that too much rubber could lead to loss in elasticity of the binder and eventually lead to poor adhesion of the binder to the aggregates leading to stripping and raveling of the pavement (Asphalt Institute., 2014). The maximum tensile strength of 842 kPa was achieved with a rubber content of 8.0 % hence this was taken as the optimum rubber content ideal for the mix.

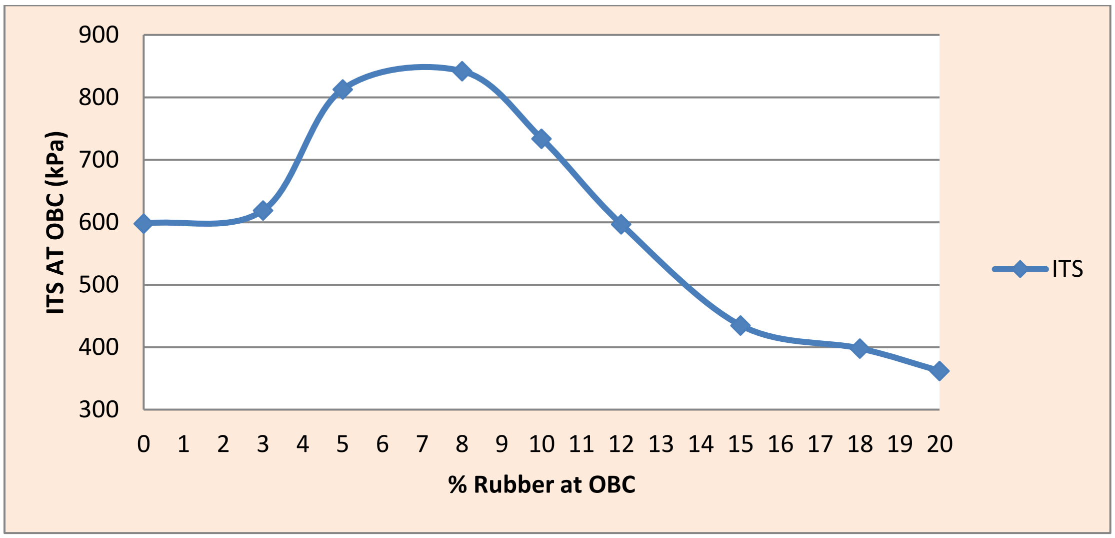


Figure 4.18: Indirect Tensile strength of rubber modified bitumen mixes

4.7.9: Elastic Modulus

A replica of tensile strength of rubber modified binders was found to be the elastic modulus of the binders at maximum of 5236 mPa where it was noted that at rubber content beyond 8.0 % the elasticity of the rubber modified bitumen mixes reduced drastically to 4577 mPa (Figure 4.19). The maximum elastic modulus of 5236 mPa gave an optimum rubber content of 8.0%

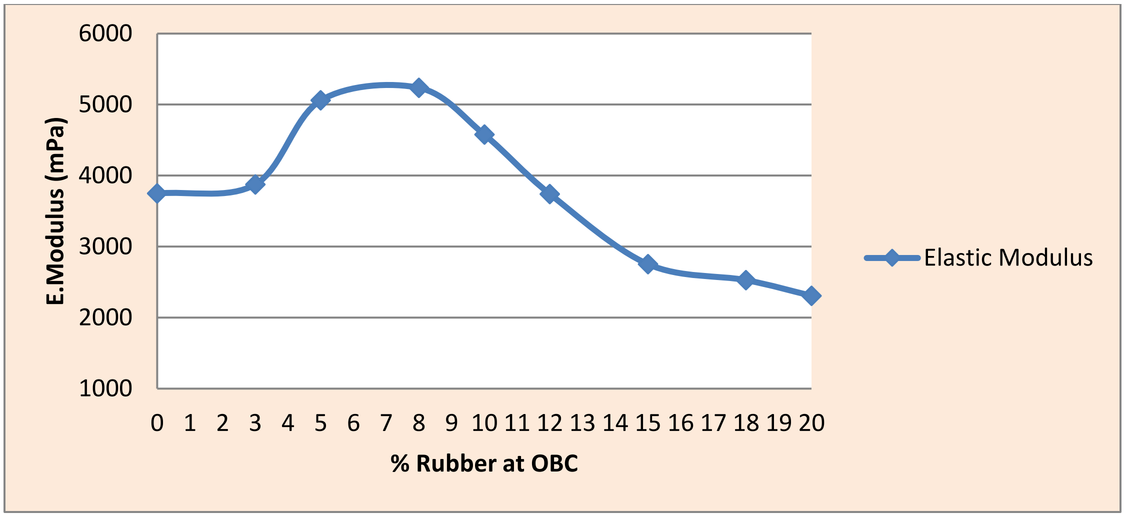


Figure 4.19: Elastic Modulus of rubber modified bitumen mixes

4.7.10: Design criteria for Optimum Rubber Content (ORC)

The optimum amount of rubber by weight of binder was deduced from the average of the graphs above. The average was taken as the optimum percentage of weight of rubber by weight of binder needed for an asphalt concrete mix AC 0/20 mm as summarized in Table 4.15.

Table 4.15: Optimum Rubber content

Property	% Optimum Rubber
At Maximum Density, g/cc	8.0
At Maximum Stability, kN	11.5
At Median Flow (3.2mm)	8.0
At 4.0% VIM	13.3
At 14.5% VMA, Dry side	8.0
At 70% VFB	15.5
At Maximum ITS, kPA	8.0
At Maximum Elastic Modulus, mPA	8.0

$$\text{Optimum Rubber Content (ORC)} = \frac{8.0+11.5+8.0+13.3+8.0+15.5+8.0+8.0}{8} = 10.04\ \%$$

The result is 10.04 % and is taken as the optimum amount of rubber needed for an ideal 0/20 mm asphalt concrete formulation. This result 10.04 % is the average of the parameters considered. It was noted that tensile strength and elastic modulus had a tremendous decrease which was attributed to decreased adhesion of the rubberized bitumen onto the aggregates and increased viscosity which reduced elasticity of the bitumen.

4.8 Performance testing

The determination of temperature and moisture susceptibility of neat and rubber modified bitumen using the Hamburg Wheel Track gave the following results (Table 4.16) at 60 ^{o}C test temperature. Plastic deformation also known as rutting of asphalt concrete pavement is the distortion of asphalt concrete pavement under stress that occurs at high pavement temperature under traffic loads (Asphalt Institute, 2014). At high temperatures asphalt concrete becomes softer and hence can undergo plastic deformation under heavy traffic.

Table 4.16: Rut depth measurement at 60 ^{o}C

Cycles (*10)	Neat Bitumen Rut Depth, mm	10 % Rubber bitumen Rut Depth, mm
0	0	0
100	1.3044	0.9672
200	1.8519	1.3067
300	2.1547	1.5671
400	2.4109	1.7159
500	2.7866	1.8879
600	3.0864	2.0460
700	3.4707	2.1716
800	3.6688	2.2785
900	4.1929	2.3901
1000	4.6006	2.5064

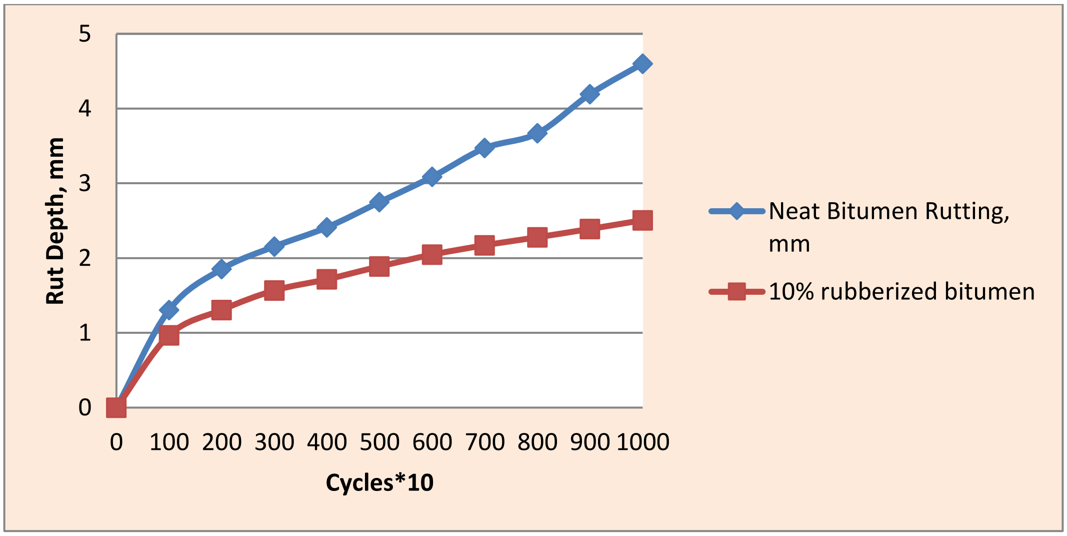


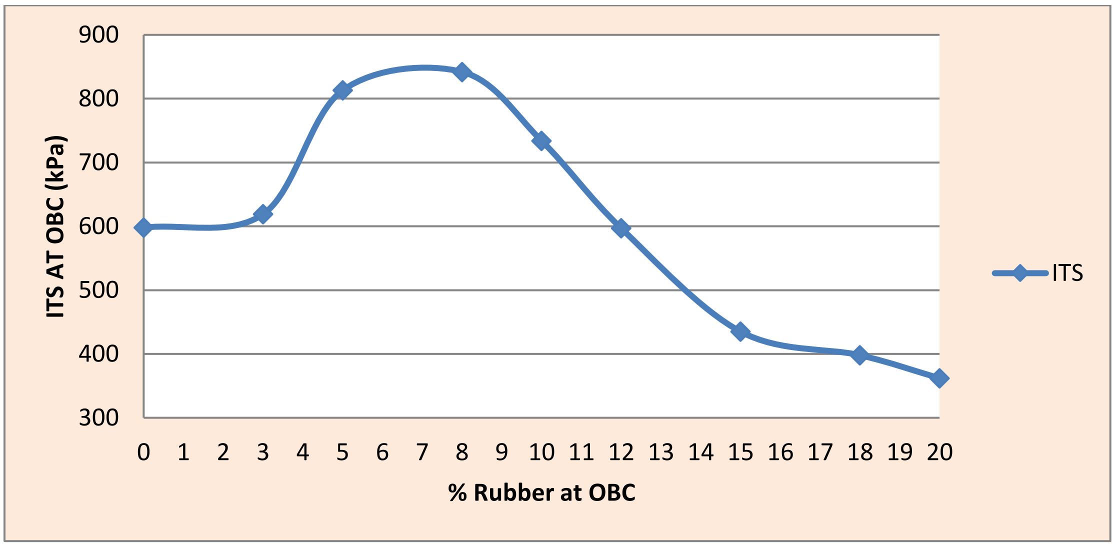
Figure 4.20: Rutting graph

On average the neat asphalt concrete mix gave 4.6 mm as the rutting value while the rubber modified bitumen gave 2.5 mm as the rutting value (Figure 4.20). This result showed that addition of 10.04 % rubber reduces the rutting of asphalt concrete mixes significantly by 46 %.

% Reduction in rutting = (4.6-2.5)/4.6*100= 46%

4.9 Indirect Tensile Strength

From the results in Table 4.13, the tensile strength of the neat bitumen increased from 598 kPa to 734 kPa (Figure 4.18) when 10 % of rubber was used to modify bitumen implying that rubber improves adhesion of bitumen onto aggregates reducing moisture damage thereby reducing raveling of asphalt concrete pavement.



(Figure 4.18) Indirect Tensile strength

$$\% \text{ Increase in Tensile strength } (\% \text{ ITS}) = \frac{\text{Strength at 10\% rubber- minimum strength}}{\text{Strength at 10\% rubber}} * 100 \quad \text{......equation 4.8}$$

$$(\% \text{ ITS}) = \frac{(734-598)}{734} * 100 = 18.5 \%$$

The tensile strength increased by a factor of 18.5% with addition of 10% rubber to the asphalt concrete. This showed that rubberized asphalt concrete mixes became stiffer with addition of rubber.

CHAPTER FIVE

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

This research culminated to a number of findings when waste rubber was used to modify bitumen for the purpose of achieving asphalt concrete mixes which are resistant to plastic deformations. In this study it was found that penetration of rubber modified bitumen depends on the amount of rubber added with the penetration value reducing from 88 to 41 as more rubber is added from 3 to 20 % at 25 ^{o}C. There was uniform reduction in penetration as more rubber was added on aged rubber modified bitumen. It was also noted that the softening point increased from 48.8 ^{o}C to 63.9 ^{o}C as rubber was added from 3 to 20 % implying that addition of rubber to bitumen makes it less susceptible to temperature induced damage. Viscosity increased from 304.3 to 330.7 cSt as more rubber was added from 3 to 12 %. However, loss on heating increased from 0.2 to 1.2 % as more rubber was added implying that too much rubber could be detrimental to improving the characteristic of bitumen. It was also found that solubility decreased from 0.1 to 1.7 % as more rubber from 3 to 20 % implying that the bitumen could become more contaminated with addition of too much rubber. Ductility reduced from 135.5 to 48.3 cm as more rubber was added from 3 to 20 %. The reduction in ductility can lead to fatigue cracking at extremely low temperatures because the mixes became more rigid.

In this study, it was also noted that addition of rubber to bitumen in asphalt concrete mixes had similar effects. The properties of density, stability and tensile strength increased to a maximum and reduced as more rubber was added in the mixes. The density, stability and tensile strength increased to a maximum of 2.262 g/cc, 15.41 kN and 842 Pa respectively and reduced as more rubber was added from 3 to 20 %. However, it was found that the flow of the mixes reduced from 4.5 to 2.3 mm as more rubber was added. Stability increased from 10.66 KN to a maximum of 15.41 KN at 12 % rubber added. Tensile strength increased from 598 N to a maximum of 842 Pa with addition of 3 to 8 % rubber. The mixes evaluated showed that 10.04 % of rubber added had the best properties aspired for an AC 0/20 mm mix. Tested for performance on plastic deformation at this optimum amount of rubber it was found that rubber modified bitumen reduces rutting by almost a half. The neat bitumen gave a rutting value of 4.6 mm while the rubber modified bitumen gave a value of 2.5 mm translating to 46 % reduction in rutting.

5.2 Recommendations

Kenya is aspiring to be a medium income country by the year 2030. To achieve this infrastructural development has been listed as one of the key sectors of the economy to be developed. The sector aims at boosting the infrastructure development and raising the efficiency and quality of infrastructural projects besides maintaining integrated safe and efficient transport network. To achieve this agenda Kenya has to embark on research to have quality transport network. It is therefore the responsively of the Kenya Government (GOK) and its implementing authorities to expedite the findings of this study. However more work needs to be done on areas where the study did not achieve hence, we recommend the following;

1. The Government of Kenya and its implementing authorities should commission and set up trial sections involving use of waste rubber modified bitumen awaiting approval for full implementation.
2. Further the authorities should enact suitable environmental laws on use and dumping or disposal of waste rubber tyres and plastics.
3. This study involved only an asphalt concrete 0/20 mm type of mix and therefore we recommend further work on effect of rubber on bitumen when used on other mixes like AC 0/10 mm, AC/14 mm and DBM 0/30 mm
4. Apart from the different types of formulations, this study considered one grade of bitumen 80/100 and therefore further work is therefore recommended to be done on other grades of bitumen.
5. On another note, this study considered performance of rubber modified bitumen concrete mixes at higher temperatures and therefore we recommend further work to be done on such formulations at very low temperatures as it would be in Polar Regions and mountainous areas.
6. Because the chemistry of reaction of rubber and bitumen has not been made elaborate further work needs to be done to establish the chemical reaction nature of the two.

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APPENDICES

Appendix I Test report

MINISTRY OF TRANSPORT & INFRASTRUCTURE, HOUSING & URBAN DEVELOPMENT

STATE DEPARTMENT OF INFRASTRUCTURE

MATERIALS AND RESEARCH DIVISION

Telegraphic Address: “MINWORKS”, Nairobi Materials Testing & Research Division

Telephone: Nairobi 554950/3/4 Machakos Road, Industrial Area

Fax: 554877

P.O. Box 11873 – 00400

E-mail: chief.engineer@materials.go.ke NAIROBI

Ref No. Date: 26 September, 2016

Laboratory Test Report

1. **Client`s Name Johnstone Maleve**
2. Box 11873-00400 Nairobi

   Project:
3. **Client’s mobile**: 0724832025
4. **Sample Description**: Aggregates
5. **Sample Submitted by**: Student
6. **Date of Sample receipt**: 23/9/2016

6. **Job Card No**:

7**. Date fee paid**:

8. **Gok MR No**.

9. **Date Analysis started**: 26/9/2016

STONE TYPE	**-**		SPECFICATION (%), RDM Part III-1987	
SAMPLE NO	466		**MIN**	**MAX**
NOMINAL SIZE (mm)	-			
A.C.V (%)	18			**25**
L.A.A (%)	16			**30**
SODIUM SULPHATE SOUNDNESS (%)	8			**12**

Eng Charles M. Muriuki

For: CHIEF ENGINEER (MATERIALS)