



**EVALUATION OF RHEOLOGY OF HIGH DENSITY POLYPROPYLENE  
ASPHALT**

**BY  
HASSAN SULEIMAN OTUOZE**

**DEPARTMENT OF CIVIL ENGINEERING  
AHMADU BELLO UNIVERSITY, ZARIA  
NIGERIA**

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ASPHALT**

**BY**

**Hassan Suleiman OTUOZE**

**[M.Sc (2010); B.Eng (2005); R.COREN; MNSE]**

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FACULTY OF ENGINEERING  
AHMADU BELLO UNIVERSITY,  
ZARIA, NIGERIA**

**JANUARY, 2016**

## **DECLARATION**

I hereby declare that the work in this thesis entitled “EVALUATION OF RHEOLOGY OF HIGH DENSITY POLYPROPYLENE ASPHALT” has been carried out by me in the Department of Civil Engineering of Ahmadu Bello University, Zaria. The information derived from the literature has been duly acknowledged in the text and a list of references provided. No part of this thesis was previously presented for another degree in other institution.

Hassan Suleiman OTUOZE

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Name of Student

---

Signature

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Date

## CERTIFICATION

This thesis entitled “EVALUATION OF RHEOLOGY OF HIGH DENSITY POLYPROPYLENE ASPHALT” by Hassan Suleiman OTUOZE, meets the requirement governing the award of the degree of Doctor of Philosophy (PhD) in Civil Engineering of the Department of Civil Engineering, Ahmadu Bello University, Zaria and is approved for its contribution to knowledge and literary presentations.

**Professor S. P. Ejeh**

_____	_____	_____
Chairman, Supervisory Committee	Signature	Date

**Dr. Y. D. Amartey**

_____	_____	_____
Member, Supervisory Committee	Signature	Date

**Dr. M. Joel**

_____	_____	_____
Member, Supervisory Committee	Signature	Date

**Dr. Y. D. Amartey**

_____	_____	_____
Head of Department	Signature	Date

**Professor Kabir Bala**

_____	_____	_____
Dean, School of Postgraduate Studies	Signature	Date

## **DEDICATION**

This work is dedicated to the loving memory of my late father, Alhaji Suleiman Aniwe Otuoze Ege whose ethos and exemplary life I treadle with inspiration. May your soul rest in perfect peace (Amen).

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## ABSTRACT

Conventional flexible pavement road surfacings are failing due to increasing automobile traffic and heavy axle loading. In order to overcome the challenge, researchers and road authorities have used various materials as modifiers or reinforcements to improve on the properties. This research presents the rheological properties of High Density Polypropylene (HDPP) waste in wet and dry hot mixed asphalt (HMA). Asphalt mixes were constituted using 0, 0.5, 1.0, 1.5, 2.0, 2.5 and 3.0% HDPP by total weight of mix. Following Asphalt Institute heavy traffic requirements, Marshall tests were conducted on specimens to characterize the stability, flow and void requirement specified in codes. Simple Performance Tests (SPT), ageing process and morphological assessments were carried out, using dynamic modulus, indirect tensile strength, fatigue rutting, creep strain, Scan Electron Micrograph (SEM) and thermal resistivity tests to assess field performance and durability of mixes. The results of Marshall tests showed optimum improvements at 2% HDPP for wet process and 0.5% HDPP for dry process over the control (0% HDPP). The wet and dry processes dynamic moduli increased by 50% and 14% respectively. Indirect Tensile Strength (ITS) improved by about 14% for wet process and 8% for dry process. All the rutting test results for wet process were largely below the maximum threshold of 8mm specified by National Cooperative Highway Research Programme (NCHRP) and 12.5mm by American Association of State Highway Transport and Officials (AASHTO); while for the dry process, only 0.5% HDPP satisfied the recommendations of the two standards. Creep strain reduction of about 20% was obtained for wet process and about 10% reduction for dry process. The temperature profile conducted using Thermo-Gravimetric Analysis (TGA) and Differential Thermal Analysis (DTA) and Scan Electron Micrograph (SEM) for wet process showed increased performance and better enhancement at optimum value of 2.0% HDPP than the control. In terms of strength, SPT, morphology, ageing process and temperature resistivity, 2.0% HDPP asphalt mixes were optimally better than the control while the general performances of 0.5% HDPP dry mixes were equally enhanced and slightly better than control. The study observed that polypropylene modified specimens under heavy traffic conditions could improve physical, mechanical and rheological properties of HMA. Optimal modifications of mixes at 2.0% and 0.5% HDPP were recommended for wet and dry processes respectively. These optimal contents of HDPP could mitigate failures in both structural and serviceability conditions thereby impacting on durability and cost effectiveness of pavement surfacing.

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## ABBREVIATIONS AND SYMBOLS

AASHTO	American Association of State Highway and Transportation Officials
AI	Asphalt Institute
ASTM	American Standard of Testing Materials
aPP	atactic Polypropylene
BS	British Standard
BC	Bitumen Content
BS	British Standard
CDM	Compacted Density of the Mix
DoD	Department of Defence
DSIR	Department of Scientific and Industrial Research
DTA	Differential Thermal Analysis
ESAL	Equivalent Standard Axle Limit
FMW&H	Federal Ministry of Works and Housing
HDPP	High Density Polypropylene
HMA	Hot Mixed Asphalt
ITS	Indirect Tensile Strength
iPP	isotactic Polypropylene
LVDT's	Linearly Variable Differential Transducers
NCHRP	National Cooperative Highway Research Programme
PP	Polypropylene
SEM	Scan Electron Micrograph
SGM	Specific Gravity of the Mix
SGMA	Specific Gravity of the Mixed Aggregates

sPP	syndiotactic Polypropylene
TFOT	Thin Film Oven test
VFB	Voids Filled with Bitumen
VIM	Voids in the Mix
VMA	Voids in Mineral Aggregates
SHRP	Strategic Highway Research Program
SARA's	Saturates Aromatics Resins Asphaltenes
SPT	Simple performance tests
TGA	Thermo-Gravimetric Analysis
TTS	Time Temperature Superposition
$\epsilon_0$	Initial tensile strain
$\epsilon$	Accumulated strain
$\phi$	Phase angle
$N_f$	Number of cycles or repetitions to failure
S	Mixture Stiffness
$\eta$	Viscosity
$\rho$	Density
$\phi$	Fluidity
$\tau$	Shear stress
$\gamma$	Shear strain
$\eta_a$	Apparent viscosity
$G^*$	Complex modulus
$G'$	Storage modulus

$G''$	Loss (or Viscous) modulus
$\eta^*$	Dynamic Modulus
$\omega$	Angular Frequency
$G_{sb}$	Bulk specific gravity
$G_{mm}$	Maximum specific gravity
$f_r$	Reduced frequency (Hz)
$t_r$	Reduced loading time (s)
$\delta$	Minimum modulus value
$\alpha$	Span of modulus value
$\beta, \gamma$	Shape parameters
$\alpha(T)$	Shift factor at temperature, T
R	Ideal gas constant
$\Delta E_a$	Activation energy

## CHAPTER ONE

### INTRODUCTION

#### 1.1 Preamble

Transportation in all history has remained the cornerstone of civilization and an index of economic growth and development of every nation (Ayo and Ikechukwu, 2013). Highway transport mode in Nigeria, like many developing countries, accounts for 95% of all transport movements and this is not without the inevitable consequences of pavement failures resulting from excessive traffic loading and environmental factors (Owolabi, 1996; Road Sector Development Team, 2010; and Pison-REIPPS, 2013).

It is noteworthy that about ₦1.4 trillion had been appropriated for investment in the road sector infrastructures between 1999 and 2012 by the Federal Government of Nigeria (Ayo and Ikechukwu, 2013), however, distresses like raveling, cracking, rutting, creep, stripping and pot-holing are some of the major service problems of the roads leading to high rates of road crashes (Ogundipe, 2008).

Pavement is the structural material laid down on an area intended to sustain vehicular or pedestrian traffic, such as a road or walkway; and its structure normally consists of a few layered materials arranged from the topmost (surfacing) in the order of strength to ensure adequate stability under traffic loads (Hijab *et al.*, 2012). A number of factors such as poor materials selection and quality, design and construction lapses, climatic factors, maintenance negligence and excessive traffic loading are responsible for pavement failures (Abiola *et al.*, 2010; Ogundipe, 2008). Increasing vehicular traffic volume on roads and inefficient stake of government on public transport has increased the malady and lack of return on investment leading to pavement failure to reach the design life (Otuoze, *et al.*, 2014). Also, constant overload by trucks has made design projection life meaningless

(Jimoh, *et al.*, 2011) and efforts to strengthen and increase pavement life have become enormous challenges to road agencies and researchers. Increasing traffic loading in terms of both volume and axle loads is a world-wide phenomenon. This has resulted in traffic loading on some of the major highways which is in excess of the current design classes (Rust *et al.*, 1992).

Asphalt pavement either as wearing or binder courses in pavement structure help to distribute stresses caused by traffic loading and protect underlying unbound layers from disintegrating (Baha and Necati, 2007). The mixture must be able to withstand the effects of ambient temperature and water; resistance to permanent deformation and cracking caused by traffic loading and the environmental factors to adequately perform both of these functions over the pavement design life (Ashok, *et al.*, 2012). It is important to use mixes flexible enough at low service temperature to reduce excessive deformation like pavement cracking and to be stiff enough at high service temperature in order to prevent creep and rutting, which are time and temperature dependent (Kumar *et al.*, 2004). Pavement distresses and poor performance of bituminous mixtures under increased traffic volume and heavy axle loading have led to the increased use and development of bitumen modifier (Hamzah *et al.*, 2006) and fibre reinforcement (Abdelaziz *et al.*, 2005) for bituminous mixtures.

A number of polymers and fibre materials such as thermoplastics – ethylene vinyl acetate (EVA), low density polyethylene (LDPE), high density polyethylene (HDPE) and ethylene-propylene-diene (EPDM) have been used in asphalt mixes. Elastomers like styrene-butadiene-styrene (SBS), styrene-butadiene random copolymers (SBR) and styrene-isoprene-styrene (SIS) and poly-butadiene-base materials have also been used. Others are asbestos, glass, carbon and cellulose fibres which also impact on the desired properties of

pavement and suffice to mitigate the distresses. The materials have been used either as bitumen modifier called wet process (Airey *et al.*, 2003; Shen and Amirkhanian, 2005) or fibre reinforcement called dry process (Brown *et al.*, 1990). The ultimate goal of introducing the materials into asphalt mixes is to increase the service life of the pavement by preventing creep and fatigue failures and reflective cracks (Yasreen *et al.*, 2011).

Rheologically, it is possible to characterize asphalt as a viscoelastic material having high temperature sensitivity (Punith *et al.*, 2005 and Giovanni *et al.*, 2003) whose viscoelastic properties are dependent on their chemical composition (Yetkin, 2007). Rheological properties of polymer bitumen show drastic improvement in temperature resilience. It is affected by phase structure or polymer morphology in the binders (Soenen *et al.*, 2008). Fine dispersion of polymers and a continuous polymer phase in the modified binders are most desirable to achieve an optimal rheological improvement (Lu *et al.*, 2010).

Polypropylene (PP) is known to have good heat and chemical resistance, resistance to deformation at elevated temperatures, high stiffness, surface hardness and toughness at normal temperature (Mohammed and Patil, 2013). The differences between HDPP and LDPP are the densities and crystalline or amorphous structure depending on the desired phase.

The focus of this research is to evaluate the potentials of HDPP waste which is more abundant and readily available than LDPP in mitigating pavement distresses.

## **1.2 Statement of the Problem**

There is an increasing need to strengthen and extend pavement service life because of daily increasing traffic and associated loads on highways. Road as the most used means of transportation in developing countries like Nigeria are subjected to heavier loads than the

designed axle loading due to increasing use of Automobile; resulting in constant pavement failure. Additionally, ageing and deterioration may be induced by climatic and environmental factors which include moisture, temperature, irradiation and chemical attack which negatively imparts on physical and chemical properties of asphalt. Pavement distresses usually lead to loss of structural and functional performances (Jitendra *et al.*, 2013). It could manifest as ravelling, cracking, rutting and stripping which are related to the rheological and strength properties of asphalt (Vaitkus *et al.*, 2009). Pavement reinforcement and modification has received increased attention in the last decade just to improve the rheological properties of Asphalt.

Specific industrial materials used for pavement strengthening are expensive (US-EPA, 2009). Approximately thirty million (30,000,000) tons of HDPP was consumed worldwide in 2001 and the products generate monumental waste disposal and environmental problems after their use (Sperling, 2006). According to Yue *et al.*, (2007), the use of recycled secondary solid waste which has posed landfill and environmental problems presents opportunity for converting waste to wealth. Thus, the use of HDPP in asphalt mix is cheap and environmentally sustainable.

### **1.3 Justification of Study**

Although, long term cost puts rigid pavement at comparative cost advantage, flexible pavement still remain the choice of low income countries because of its lower initial cost of construction (Jain *et al.*, 2009; Embacher and Snyder, 2001). However, higher maintenance cost over the design life as a result of problems which include ravelling, creeping, rutting, stripping and potholing are common. Others are related to high traffic loading, quality of mix, soil type and environment conditions. The need, therefore, has arisen to source for

cheaper and durable additives in asphalt pavement where waste could be harnessed for pavement strengthening. The use of HDPP waste provides opportunity to improve upon asphalt pavement on highways. Approximately 30,000,000 tons of HDPP was consumed worldwide in 2001 (Sperling, 2006) and the products generate monumental waste after their useful service life. The utilization of HDPP waste in asphalt mixes is an effective strategy to reduce maintenance cost, while achieving the same or better performance of asphalt pavement. Also, it will help to reduce the problem of disposal of unwanted wastes of HDPP which is of environmental and health concerns.

#### **1.4 Aim and Objectives Of Research**

The aim of this work is to investigate possible improvements on desirable physical, mechanical and rheology properties of HDPP asphalt mixes over the conventional asphalt mixes. Pursuant to the aim, the following objectives were set to:

- (i) Determine optimum improvement in Marshall test properties such as stability, flow and void analysis for HDPP asphalt concrete in wet and dry mixes over conventional plain asphalt.
- (ii) Investigate the changes in consistency of HDPP modified bitumen over unmodified bitumen at low, intermediate and high temperatures.
- (iii) Evaluate optimal field performance characteristics which include dynamic modulus, creep, Indirect Tensile Strength and rutting for HDPP asphalt as they impact upon mechanical properties of HDPP asphalt over conventional hot mixed asphalt (HMA).
- (iv) Examine changes in morphology of HDPP modified bitumen and the effects on strength and durability of asphalt.

- (v) Assess improvement on temperature profiles of HDPP modified bitumen over conventional bitumen and its improvement on temperature susceptibility, rheology and pavement life cycle.

### **1.5 Scope of Research**

Preliminary tests were conducted on bitumen, coarse and fine aggregates and filler to assess their specification and quality. Marshall tests were used to evaluate optimal bitumen and HDPP contents for both fibre reinforced and polymer modified asphalt mixes. These optimal parameters were used to prepare specimens for further testing that include creep, Indirect Tensile Strength (ITS) and resilient modulus of asphalt mix. Scan Electron Micrograph (SEM) was conducted to study the morphology of bitumen-HDPP matrix. Also, Thermo-Gravimetric Analysis (TGA) and Differential Temperature Analysis (DTA) to determine the temperature profiles was conducted.

### **1.6 Limitation of Study**

This study is limited to conventional and HDPP asphalt mixes produced in wet and dry processes. The tests evaluated are Marshall properties, Simple Performance Tests (SPT's), temperature resistivity and morphology of the mix structure.

## CHAPTER TWO

### LITERATURE REVIEW

#### 2.1 Background

The work of Zube (1956) set the pace on strengthening and extending pavement life using wire mesh reinforcement in bituminous mixtures to improve on resistance to pavement deformation and reflective cracking. Maupin (1991) noticed similar improvement additive modified asphalt. Large interests apparently evolved in mitigating failures using different materials and different methods. Polymer Modified Asphalt according to Lu and Isaacsson, (2001) and Navarro *et al.* (2004) improves resistance of asphalt binder against rutting and thermal cracking (fracture of the pavement due to the lack of flexibility at low temperatures).

Roberts *et al.* (1996) observed that fatigue and rutting resistance of asphalt mixture could be enhanced considerably by utilization of different types of additives such as fibres that can increase the amount of strain energy absorbed during fatigue and fracture process of the mix in the resulting composite. Some specific technical reasons for using additives and modifiers in Hot Mixed Asphalt (HMA) as presented by Roberts *et al.* (1996) are:

- (i) Obtaining stiffer mixtures at high service temperatures to minimize rutting.
- (ii) Obtain softer mixtures at low service temperatures to minimize thermal cracking.
- (iii) Improve fatigue resistance of HMA mixtures.
- (iv) Improve asphalt-aggregate bonding to reduce stripping or moisture susceptibility.
- (v) Improve resistance to aging or oxidation; rejuvenate aged asphalt binders.
- (vi) Permit thicker asphalt films on aggregate for increased mix durability.
- (vii) Improve abrasion resistance of mixture to reduce ravelling.
- (viii) Reduce flushing or bleeding; reduce structural thickness of pavement layers.

- (ix) Reduce life cycle costs and improve overall performance of HMA pavements.

Against this background, researchers (Mohammad *et al.*, 2003; Kaloush *et al.*, 2006; and Ye and Wu, 2009) have reported the use of additives such as different types of polymer modifiers and fibre in asphalt to mitigate deterioration of pavement and enhanced performance. Arabani *et al.* (2010) suggested two principal ways of constructing a more durable pavement; by either applying a thicker asphalt pavement which will increase the construction cost or making an asphalt mixture with modified characteristics.

According to Taher *et al.* (2011), fibres and polymers in asphalt mixture can help improve resistance to high temperature rutting, medium temperature fatigue and low temperature cracking thereby increasing the durability of pavement structure. He observed that additives such as fibres absorb the amount of distresses imposed by repetitive heavy traffic loading during pavement life. There exists direct relationship between tensile strength of additives and engineering properties of asphalt concrete. In other words additives such as fibres can increase the amount of strain energy absorbed during fatigue and fracture process of the mix in the resulting composite (Mahrez *et.al.*, 2005). Also, fibres and polymers provide three-dimensional networking effect in asphalt concrete and stabilise the binder on surface of aggregate particles and prevent from any movement at higher temperature (Ahmedzade *et al.*, 2007). Yildirim (2007) and McDaniel and Shah (2003), reported the successful use of polymer modified binders at locations of high stress, such as at intersections of busy streets, airports, vehicle weight stations, and race tracks because of their achievement in mitigating rutting and cracking for HMA.

Louisiana State University researchers investigated the mixtures from Styrene-Butadiene-Styrene (SBS) modified binders extracted from aged pavements, and virgin SBS polymer

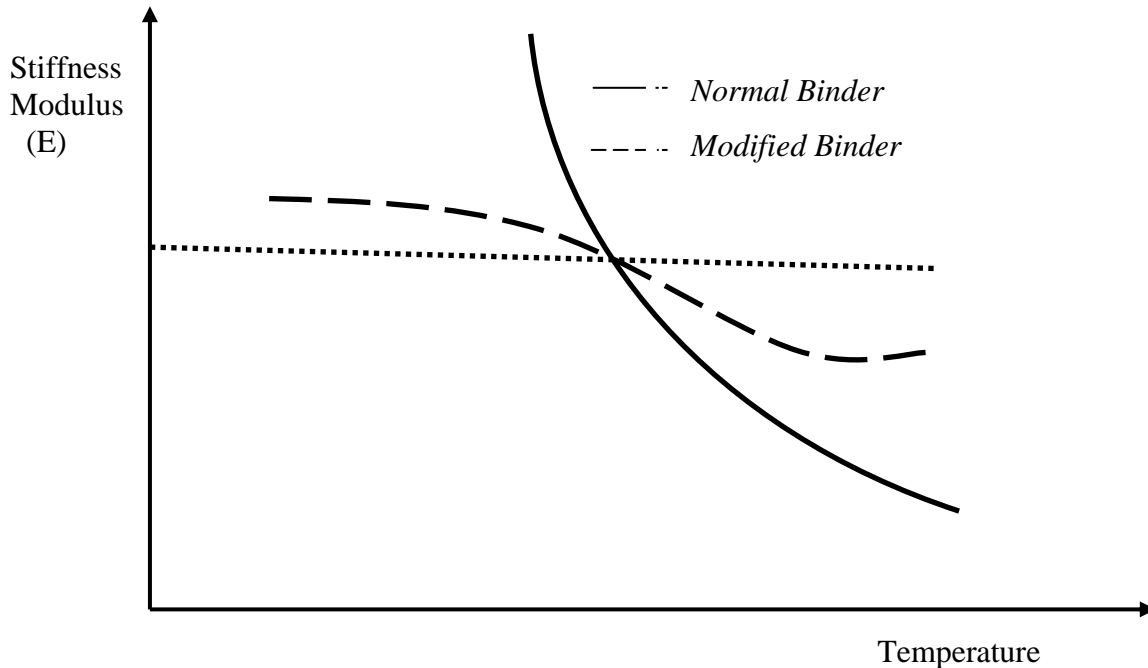
modified binder. The study observed that increasing the percentages of recycled polymer modified binder in mixtures would increase the rutting resistance but decrease the fatigue resistance of the recycled polymer modified asphalt concrete mixtures, as observed from laboratory tests (Mohammad *et al.*, 2003).

Kaloush *et al.* (2006) studied the behavior of mixtures of polypropylene and Aramid fibres to evaluate the performance characteristics of a modified asphalt mixture. Results showed that the fibres improved the mixture's performance in several unique ways against the anticipated major pavement distresses: permanent deformation, fatigue cracking, and thermal cracking. Mashaan *et al.* (2011) noted that the addition of crumb rubber to bitumen binder enhance the physical properties of rubberised bitumen binder as indicated by the reduction in penetration value and ductility, and an increase in the elastic recovery, thus enhancing rubberized binder elasticity and increase its ability to resist rutting deformation.

In the test on Stone Mastic Asphalt (SMA) and glass fibre, Mahrez and Karim (2010) observed though, decreasing stability and increasing voids trends and the mix with more than 0.2% fibre content exhibiting lower resistance to permanent deformation; the fibre has the potential to resist structural distress that occur in road pavement as a result of increased traffic loading, thus improving fatigue life by increasing the resistance to cracking and permanent deformation, especially at higher stress level.

Ye and Wu (2009) studied Cellulose, Polyester and Mineral fibres asphalt mixes and reported that with 0.5% fibre in the mix, viscosity and complex shear modulus of fibre reinforced Asphalt binder increased with the latter, thus, signifying improvement in stiffness. Also, the researchers reported a great deal of enhancement on the elastic part of visco-elasticity which reduced the phase angle between stress and strain due to reduction in total and permanent strains that revealed resistance to permanent deformation in the fibre

reinforced binders. Figure 2.1 shows the improvement on stiffness modulus performance of modified asphalt over plain one at higher temperature.



**Figure 2.1: Stiffness/Temperature Performances of Plain and Modified AC**  
*Source: Ye and Wu (2009)*

## 2.2 Major Types of Flexible Pavement Distresses

Akoto (2009) classified flexible pavements distresses into four main categories according to the structural failures:

- (a) **Surface defects:** These include fatty surfaces, smooth surfaces, streaking, hungry surfaces, etc. These are associated with the surfacing layers and may be due to excessive or deficient quantities of bitumen in the layers. Other defects like segregation, surface unevenness and roughness may occur at the time of construction and are therefore not influenced by either traffic or the environment.

- (b) **Cracks:** There are different types of cracks including alligator cracking which are interconnected cracks caused by fatigue due to repeated loads; block cracking are also interconnected cracks caused by shrinkage of the asphalt and daily temperature cycling; longitudinal cracks which are parallel to the centerline of the pavement and are attributed to construction defects or traffic loading. Other types of cracks are hair-line cracks, transverse cracks, edge cracks, shrinkage cracks and reflection cracks.
- (c) **Deformation:** Any change in the shape of the pavement from its original shape is a deformation. Deformation includes creep, slippage, rutting, corrugations, shallow depressions and settlements, upheavals, etc.
- (d) **Disintegration:** This includes stripping, loss of aggregates, raveling, formation of potholes, edge breaking, etc. The details of various types of defects by Akoto (2009) are summarized in Table 2.1.

**Table 2.1: Types of Distresses, Symptoms and Possible Causes**

<b>Types of Distress</b>	<b>Symptoms</b>	<b>Possible Causes</b>
<b>A. Surface Defects</b>		
Excess bitumen (Bleeding/ Flushing or Fatty Surface) Smooth Surface	Collection of a thin film of binder on the surface that creates smooth, shiny, greasy and reflective surface. Slippery surface	Mixture problem (excessive binder in the mix), loss of cover aggregates or paving over excess bitumen.  Polishing of aggregates under traffic, or excessive binder, too fine grading.
Streaking	Presence of alternating layers of lean and heavy bitumen.	Non-uniform application of bitumen or low temperatures.
Hungry Surface	Less aggregate or presence of fine cracks.	Use of less bitumen or use absorptive aggregates.
Segregation	Separation of coarse aggregate in the mix from the rest of the mass.	Manner in which the mix is loaded into trucks, hopper not being kept at least half full.
<b>B. Cracks</b>		
Hair-line Cracks	Short and fine cracks at close intervals on the surface.	Insufficient bitumen, excessive filler or improper compaction.
Edge Crack	Cracks near and parallel to the pavement edge.	Lack of support from shoulder, poor drainage or inadequate pavement width.
Block/Random Cracks	Block cracks are interconnected cracks ranging from 0.3 m to 3 m. They divide pavement into rough, approximately rectangular pieces. Generally, occurs over a wide area of the pavement surface.	They are caused mainly by shrinkage of the asphalt due to temperature cycling
Alligator/Fatigue Cracks	Inter-connected cracks in the asphalt layer forming a series of small blocks which resemble an alligator hide or chicken wire	Weak pavement, unstable conditions of sub grade or lower layers, excessive overloads or brittleness of binder.

**Table 2.1 continued: Types of Distresses, symptoms and Possible Causes**

<b>Types of Distress</b>	<b>Symptoms</b>	<b>Possible Causes</b>
Longitudinal Cracks	Cracks in a straight line along the road.	Poor drainage, shoulder settlement, weak joints between adjoining spreads of pavement layers.
Transverse Cracks	Cracks running transverse to the centerline of the pavement.	Temperature variations and hardening of asphalt, swelling or shrinkage of the sub-grade or settlement (trench backfill).
Reflection Cracks	Sympathetic cracks over joints and cracks in the pavement underneath.	Due to joints and cracks in the pavement layer underneath.
Shrinkage Crack	Cracks in transverse direction or interconnected cracks forming a series of large blocks.	Shrinkage of bituminous layer with age.
Slippage	Formation of crescent shaped cracks pointing in the direction of the thrust of wheels.	Unusual thrust of wheels in a direction, lack or failure of bond between surface and lower pavement courses.
<b><i>C Deformation</i></b>		
Rutting	Longitudinal depression in the wheel tracks.	Heavy and channeled traffic, inadequate compaction of pavement layers, poor stability of pavement material or heavy bullock-cart traffic.
Corrugations	Formation of regular undulations.	Lack of stability in the mix, oscillations set up by vehicle springs, or faulty laying of surface course.
Shallow Depression	Localized shallow depressions.	Presence of inadequately compacted pockets.
Shoving	Localized bulging of the pavement surface along with crescent-shaped cracks.	Unstable mix, Lack of bond between layers, heavy start-stop type movements and movements involving negotiations of curves and gradients.
Settlement and Upheaval	Large deformation of pavement.	Poor compaction of fills, poor drainage, inadequate pavement.

**Table 2.1 continued: Types of Distresses, symptoms and Possible Causes**

<b>Types of Distress</b>	<b>Symptoms</b>	<b>Possible Causes</b>
<b><i>D. Disintegration</i></b>		
Stripping	Separation of bitumen from aggregate in the presence of moisture.	Use of hydrophilic aggregate, inadequate mix composition, continuous contact with water, poor bond between binder and aggregate, dust on aggregate, poor compaction.
Edge-breaking	Irregular breakage of pavement edges.	Water infiltration, poor shoulder lateral support, inadequate strength of pavement edges.
Loss of Aggregate	Rough surface with loss of aggregates in some portions.	Ageing and hardening of binder, stripping, scuffing and dragging by screed plate wrongly set, Poor bond between binder and aggregate, insufficient binder, brittleness of binder, etc.
Raveling	Failure of binder to hold the aggregate shown up by pockmarks or eroded areas on the surface.	Poor compaction, poor bondage between binder and aggregate, insufficient binder brittleness of binder, etc.
Pot-hole	Appearance of bowl shaped holes, usually after rain.	Ingress of water in to the pavement, lack of bondage between the surfacing and WBM base, insufficient bitumen content, etc.

*Source: Akoto (2009)*

### **2.2.1 Pavement surface failure**

Smooth and comfortable riding surface throughout the design life of a pavement is the best service condition for road to perform excellently. Pavement failure is caused by the deterioration of the pavement quality, arising from the effect of traffic load and the disintegration of the road materials arising from climate and weather conditions (Kodrat *et*

*al.*, 2007). Pavement defects such as cracks and ruts, are the result of three main distress mechanisms; namely fatigue, thermal shrinkage and deformation. These modes of distress influence each other, while environmental and vehicle loading conditions determine the type and extent of pavement distress (Seibi *et al.*, 2001). Low temperatures cause the pavement to shrink, resulting in the build-up of thermal stresses that can cause the pavement to crack but at high temperatures heavy vehicles cause plastic deformation in the pavement, which causes rutting in the wheel paths (Smith, 2000). Thermal and load associated fatigue cracking occurs over a range of temperatures (Mashaan *et al.*, 2011).

### **2.2.2 Factors affecting rutting and creep of asphalt**

Creep (sometimes called cold flow, in some material parlance) is the tendency of a solid material to move slowly or deform permanently under the influence of mechanical stresses (Sperling, 2006). It may be defined as time-dependent strain per unit stress. The rate of deformation is a function of the material properties, exposure time, exposure temperature and the applied structural load (Turner, 2001). Depending on the magnitude of the applied stress and its duration, the deformation may become so large that a component can no longer perform its function (Ogunkoy *et al.*, 2011). Rutting or shearing uplift could be an incipient failure forms at the stage because of large deformation as it causes surface depression along the wheel path (Tashman *et al.*, 2005)

In asphalt pavement, creep and rutting are distress mechanisms in flexible pavements mainly dependent on asphalt mix compositions, that is, gradation, binder type, air void content, and the degree of both construction compaction and traffic densification under service conditions (Adlinge and Gupta 2013). It also depends on pavement temperature, characteristics of traffic loading, that is, number of load repetitions, axle weight, and

properties of pavement layers, for example, layer thicknesses, materials quality as well as subgrade support conditions (Zhou *et al.*, 2003; Kandhal and Cooley, 2003). Prominent among the factors are explained below:

**(a) Traffic loading**

It must be emphasized that the states of stress and strain caused by traffic loading significantly influence pavement rutting (Al-Qadi *et al.*, 2014). Normally a mixture is designed for a given intensity and distribution of traffic based on past correlations with field performance. Changes in the distribution of traffic, especially increases in the proportion of heavy trucks, may increase the rate of rutting, even if the pavement was properly designed and constructed originally. Eisenmann and Hilmer (1987) found that increasing the contact pressure of a twin tire from 0.6 MPa (87 psi) to 0.9 Mpa (130 psi) increases the regression coefficient (rut depth) by a factor of about 3, indicating a significant increase in rut depth.

**(b) Pavement thickness**

Researchers have found that the deformation through the asphalt-concrete layer was greatest near the loaded surface and gradually decreased at lower levels (Hofstra and Klomp, 1972; Gokhale *et al.*, 2005; and Zhou *et al.*, 2008). This is because rutting is caused by plastic flow; hence, such a distribution of rutting with depth is reasonable: more resistance to plastic flow is encountered at greater depths and shear stresses are smaller there as well (Sousa *et al.*, 1991). Thicker pavement does not exhibit additional rutting. Highway Research Board (1962) put the limiting value of surface rut depth in asphalt concrete to approximately 10mm. The value suggests that for reasonably stiff supporting materials, most pavement rutting is confined to the asphalt pavement layer.

**(c) Aggregate**

Evidences available from different research works have shown that dense aggregates could offer rutting resistance in asphalt concrete. Brown and Bell (1977) observed mixes with properly compacted, dense or continuous aggregate gradations have fewer voids and more contact points between particles than open or gap-graded mixtures.

Uge and Van de Loo (1974) reported that mixtures made from angular aggregates (obtained by crushing) deformed to a minor extent; and were more stable and stiffer than mixtures having the same composition and grading, but made from rounded aggregates (river gravel). Furthermore, Davis (1988) reported that asphalt pavements constructed with large maximum aggregate size of 38mm (1½ in.) or larger exhibited good rutting resistance of mixtures subjected to high tire pressures.

**(d) Bitumen**

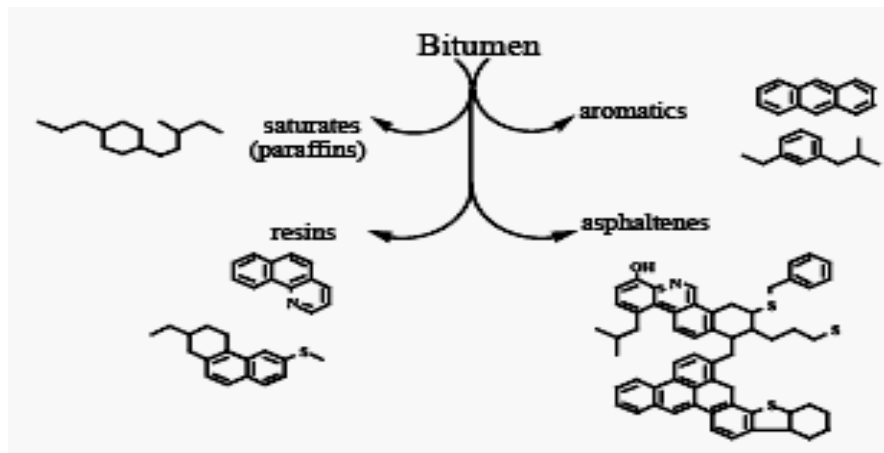
Bitumen is a mixture of hydrocarbons and compounds of a predominantly hydrocarbon character, varying both chemically and in molecular size; approximately 90 to 95 percent bitumen, while the remaining portion consists of hetero-atoms such as nitrogen, oxygen, and sulphur and metals atoms, such as vanadium, nickel, and iron, are present in trace quantities, usually far less than one percent (Smith, 2000). The hydrocarbon constituents according to Pfeiffer (1950) listed that bitumen consists of three basic forms (a) saturated aliphatic groups or paraffin, (b) naphthenic groups or cyclo-paraffin, and (c) groups composed of aromatic rings. Variations in the origin, formation of bitumen and composition have considerable influences on the character of the material.

There are two main groups of compounds that have been identified; asphaltenes and maltenes. Asphaltenes are insoluble in these organic solvents and are precipitated in the form of a solid, dark brown or black substance. They consist of aromatic hydrocarbons

containing paraffin chars to a varying extent (Pfeiffer, 1950). The fraction that remains dissolved during the separation of asphaltenes is called maltenes (sometimes referred to as petrolenes). This is dark brown oil containing varying amounts of aliphatics, naphthenics, and aromatics. Larger part of bitumen consists of carbon and hydrogen, which form nonpolar molecules with weak (dispersion) forces of attraction (Zhao *et al.*, 2003). However, the presence of highly electronegative atoms such as nitrogen, sulphur, and oxygen result in polar molecules with strong attractive forces. The polar and non-polar molecules exist together in a homogeneous mixture in which the polar molecules form a network or structure, and the non-polar molecules form a body of material around the network (Strangl *et al.*, 2006 and Oyekunle, 2006). The chemical bonds holding the molecules together are relatively weak and can easily be broken by heat or shear stress. This results in the viscoelastic nature of asphalt. The polar molecules give asphalt its elastic properties, while the non-polar molecules contribute to the viscous properties of the asphalt (Oyekule, 2007). Due to their similar physical properties and viscoelastic nature, asphalts are often classified with polymeric, or macro-molecular substances, although, their chemical composition is significantly different (Pfeiffer, 1950).

Pfeiffer and Saal (1940) depicted bitumen as “colloidal system”, with asphaltenes forming the centers of micelle and having a more pronounced aromatic nature, the asphaltenes were assumed to be surrounded by lighter constituents of less aromatic nature, and there were no distinct inter-phases between the micelles and the medium surrounding it”. Micelles may be visualized in terms of surfactants or compounds that lower surface or interfacial tension, and an aggregation of colloids or surfactant molecules coming into being as a result of association of several unit cells (Rosen and Kunjappu, 2012).

Basically, three constituent mixtures were largely identified in bitumen – asphaltenes, asphaltic resins and oily constituents (aromatics and saturates); the acronym (SARA's) was conveniently used by Raki *et al.*, (2000) and Carbognani *et al.* (2007) to depict the presence of saturates (S), aromatics (A), resins (R), and asphaltenes (As) in bitumen with proportion varying from deposit to deposit of petroleum or tar sands. Masson *et al.* (2001), depicted the representative structures for the four bitumen fractions shown in Figure 2.2.



**Figure 2.2: Structures of the four bitumen fractions**  
*Source: Masson et al. (2001)*

The peculiar properties and impacts of the three key components according to Masson *et al* (2001) are:

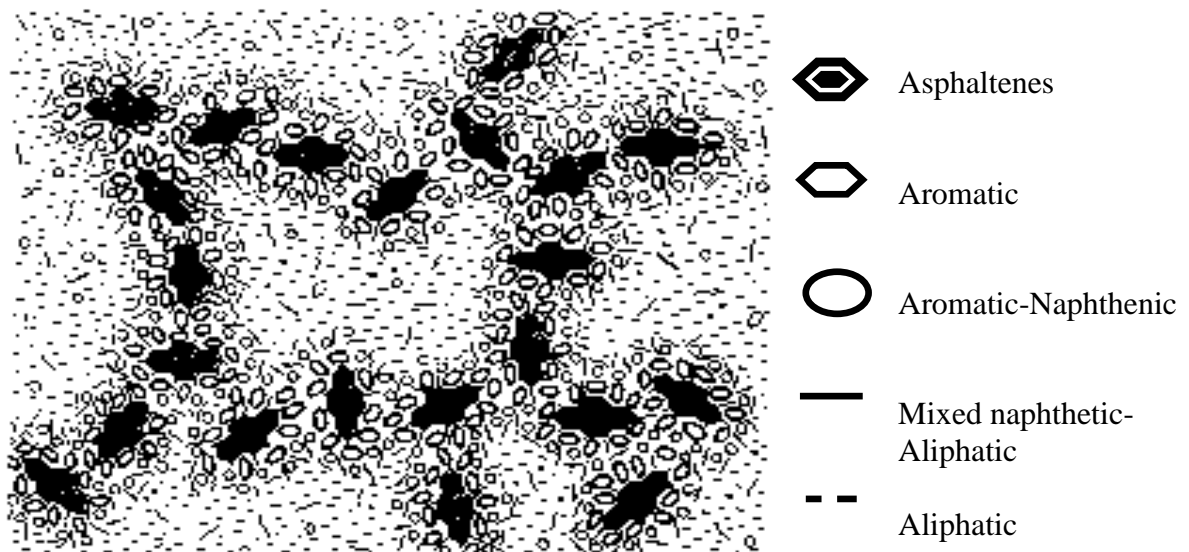
- (a) Asphaltenes which impart hardness and high melting point on the mixture to withstand both service loading and durability needs of the bonded material.
- (b) Asphaltic resins which gives ductility and tensile strength thereby acting as stabiliser between bitumen and mineral aggregates.
- (c) Oily constituents (aromatics and saturates) serving as dispersing fluid phase.

Bitumen stability depends upon micelles – oily medium interaction and the result of combined effects of cohesive and adhesive properties of bitumen which causes materials to

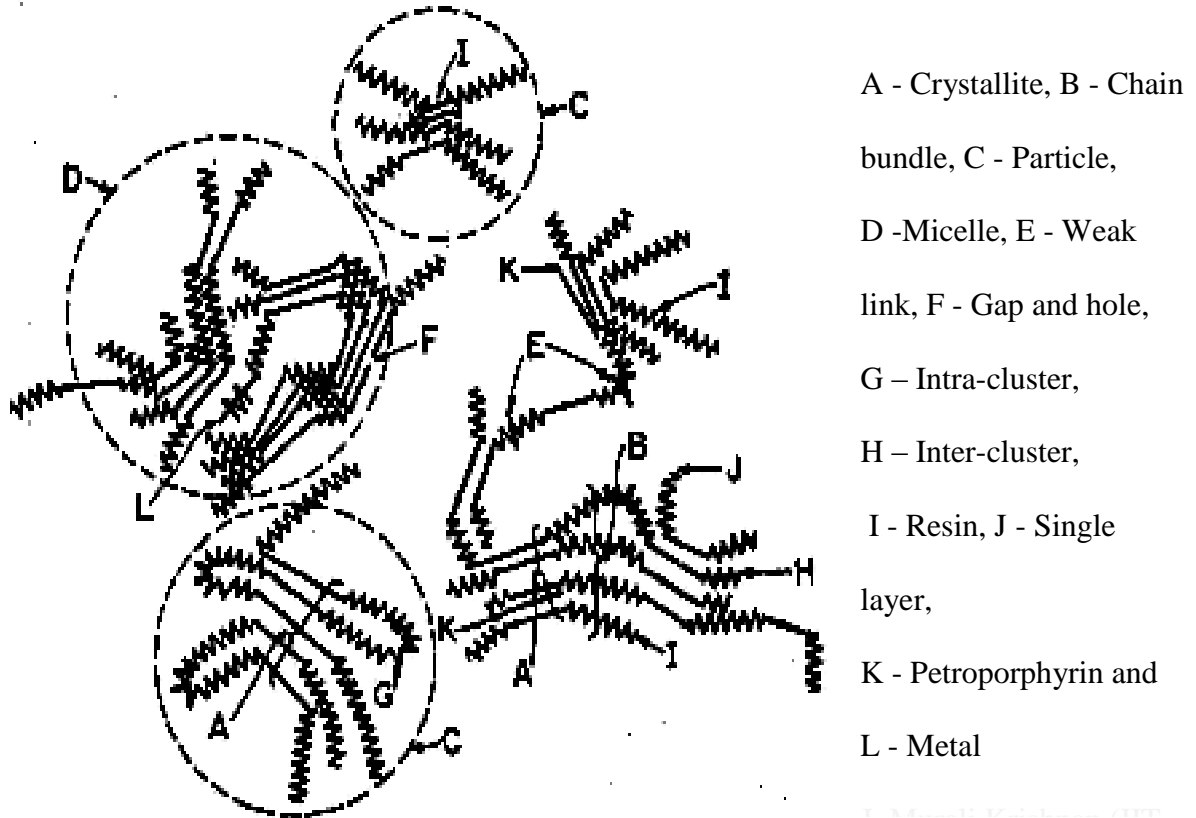
bond or solidify (Raki *et al.*, 2000). There is a marked increase in viscosity or crystallization as solidification continues. The agglomeration of micelles in bitumen structure largely affects the rheology and morphology of the bitumen material.

Blends of polymer and bitumen usually have tendency of phase separation as high molecular weight polymers are not readily miscible with low molecular weight bitumen (Stangl *et al.*, 2007). Dispersion of polymer in bitumen may not signify complete digestion of the material as digestion of the polymer occurs when it absorbs lighter resins and swells.

Polypropylene (PP) thermoplastic shows low tendency of dispersion with enhanced viscosity in bitumen because of its easy digestion and low molecular weight (Cleven, 2000). But higher proportion of PP may lead to phase inversion with flocculation of agglomerated structure and thus, could destabilise because of phase separation that has set in (Kamaruddin, *et al.*, 2012). Figures 2.3 and 2.4 respectively show the main constituent structure of bitumen and micelles unit cells formations.



**Figure 2.3: Gel Structure of Bitumen**  
*Source: Pfeiffer and Saal (1940)*



**Figure 2.4: Micelles unit cells formations**  
*Source: Pfeiffer and Saal, 1940*

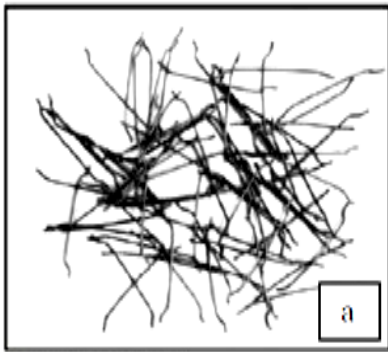
Observations have been made that harder or more viscous bitumen reduces rutting or deformation propensity of asphalt pavement especially in thicker pavements and hotter climates (Mahboub and Little, 1988; Monismith *et al.*, 1985) on the basis of uniaxial creep testing. Several researchers (Ould-Henia and Dumont, 2006; Lu and Redelius, 2006) have tried to improve rutting performance by using modifiers (polymers, micro-fillers, etc) intended to increase the viscosity of the asphalt binder at high temperatures without adverse effect at low temperatures .

Various researchers (Airey *et al.*, 2003; Shen and Amirkhanian, 2005; and Hamzah *et al.*, 2006) have observed positive influences on various distress modes when modifiers or fibres

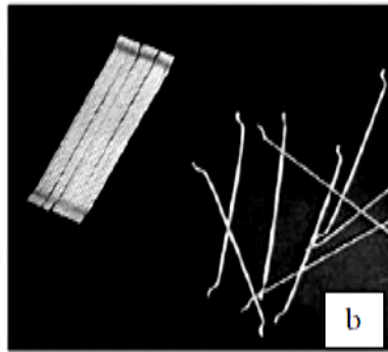
were introduced into HMA pavement. The list of some of the known modifiers and fibre materials used by some researchers ((Roberts *et al.*, 1996) is as presented by generic classification in Table 2.2. Plates I and II show the pictures of some construction fibre materials for HMA mixes. Plates IIIa and IIIb show collected HDPP waste materials.



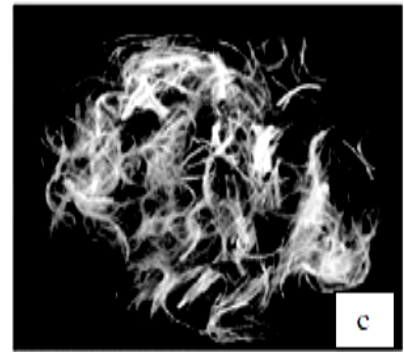
**Plate I: Woven fibre reinforcements**  
*Source: Bruck and Pietro, (2014)*



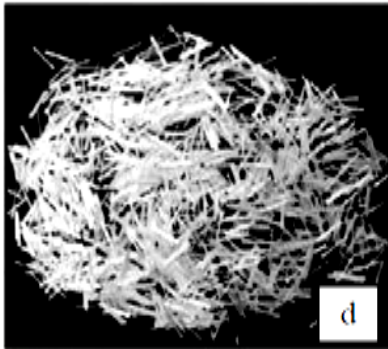
*Loos steel macrofibers.*



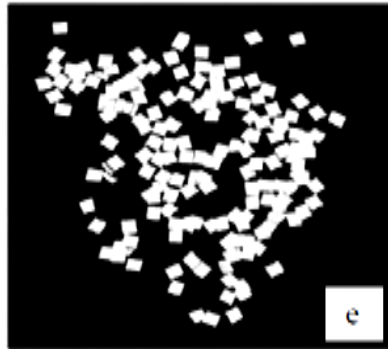
*Gludes steel macrofibers.*



*Polypropylene fibers.*



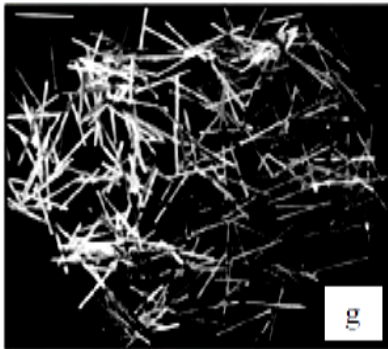
*Glass fibers.*



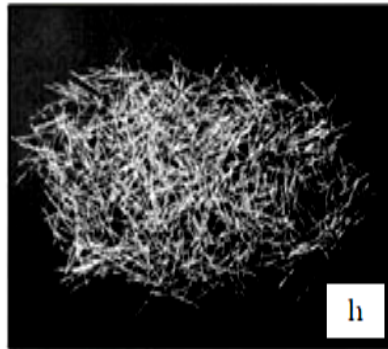
*Cellulose fibers.*



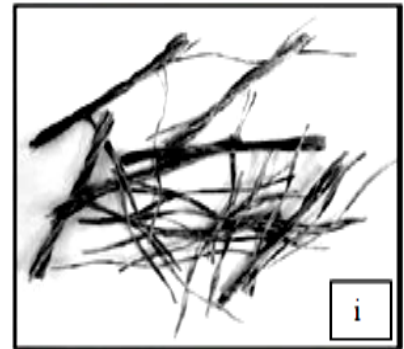
*Melt extract steel macrofibers.*



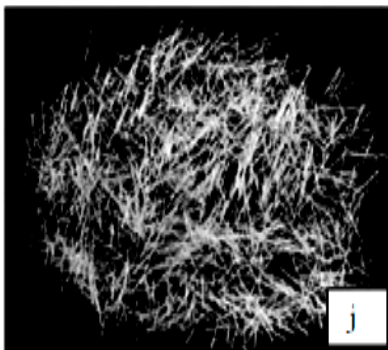
*Sheet steel macrofibers.*



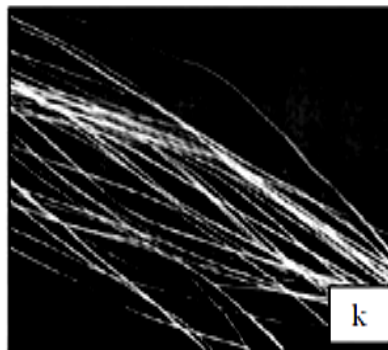
*Steel microfibers.*



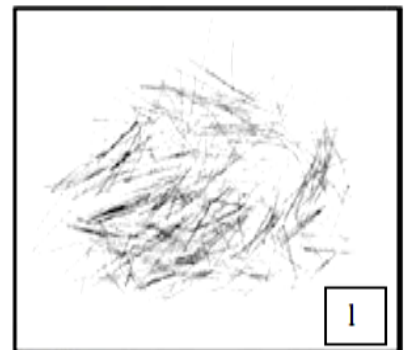
*Macrofibers of high tenacity polypropylene.*



*Polypropylene microfibers.*



*Synthetic macrofibers.*



*Steel fibers.*

**Plate II: Asphalt fibre materials**  
*Source: Bruck and Pietro, (2014)*



**Plate IIIa: High Density Polypropylene (HDDP) woven bag**  
*Source: Weir & Carmichael Ltd: <http://www.weirbags.co.uk>; assessed 5/03/2015*



**Plate IIIb: High Density Polypropylene (HDDP) Fibres**  
*Source: Sanblue Enterprises Pvt. Ltd: <http://www.fibre2fashion.com>, assessed 5/03/2015*

**Table 2.2: Generic classification of asphalt additives and modifiers**

<b>Type</b>	<b>Generic Examples</b>	
1. Filler	Mineral Filler: Crusher fines Lime Portland cement Fly ash Carbon black	
2. Extender	Sulfur Lignin	
3. Polymers	Rubber: a. Natural latex b. Synthetic latex c. Block copolymer d. Reclaimed rubber  Plastic  Combination	Natural rubber Styrene-butadiene or SBR Polychloroprene latex Styrene-butadiene-styrene (SBS) Styrene-isoprene-styrene (SIS) Crumb rubber modifier Polyethylene/Polypropylene Ethylene acrylate copolymer Ethyl-vinyl-acetate (EVA) Polyvinyl chloride (PVC) Ethylene propylene or EPDM  Blends of polymers above
4. Fiber	Natural: Asbestos Rock wool Man-made: Polypropylene Polyester Fiberglass Mineral Cellulose	
5. Oxidant	Manganese salts	
6. Antioxidant	Lead compounds Carbon Calcium salts	
7. Hydrocarbon	Recycling and rejuvenating oils Hard and natural asphalts	
8. Anti-stripping Agent	Amines Lime	
9. Waste Materials	Roofing shingles Recycled tires Glass	
10. Miscellaneous	Silicones Deicing calcium chloride granules	

*Source: Roberts et al. (1996)*

The uses of low density Polypropylene (LDPP) in HMA were reported by researchers (Cleven, 2000; Chen and Lin, 2005; and Serkan *et al.*, 2009), but little is known of the use of HDPP in HMA, hence, the need for such study. The background knowledge includes the work of Serkan *et al.* (2009) who observed a 20% increase in Marshall Stability and 50% decrease in repeated creep tests at optimum of 0.5% PP fibre reinforcement and 5% bitumen in HMA. The works of Chen and Lin (2005) and Cleven (2000) on mixtures containing polypropylene fibers were found to have higher tensile strengths, high temperature resilient modulus, resistance to thermal and reflective cracking and the fracture energy increased by 50 to 100 percent, implying increased toughness.

### **2.3 Development of Creep and Rutting**

Deformation in paving materials develops gradually into distresses called creep and rutting with increasing numbers of load applications. These usually appear as longitudinal depressions in the wheel paths accompanied by small upheavals to the sides. It is caused by a combination of densification (decrease in volume and, hence, increase in density) and shear deformation and can occur in any one or more of the pavement layers as well as in the subgrade (Kong Kam Wa and Muthen, 1997).

#### **2.3.1 Rutting in asphalt pavement**

Rutting, according to Sousa *et al.* (1991), is an excessive permanent deformation in heavy duty asphalt-concrete pavements resulting from frequent repetitions of heavy axle loads, many of which are operating with radial tires having pressures ranging from 0.138 to 0.172 N/mm<sup>2</sup> (20 to 25 psi) higher than the bias-ply tires which they have replaced, that is, 0.724 against 0.552 N/mm<sup>2</sup> (105 psi against 80 psi). Rutting gradually develops with increasing

numbers of load applications and appears as longitudinal depressions in the wheel paths (Uzan, 2004).

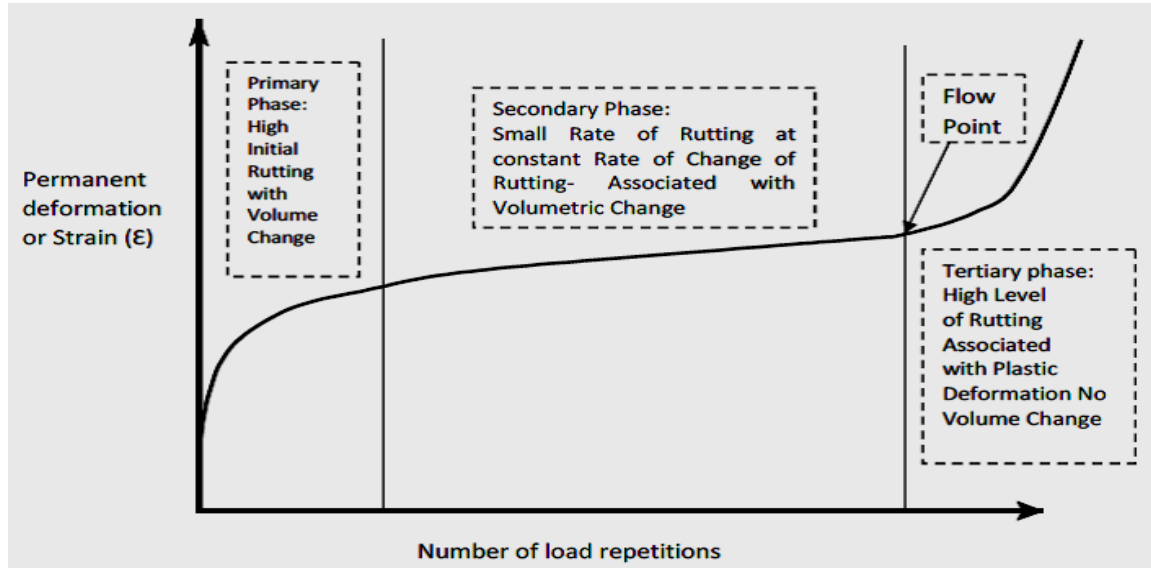
One cause of rutting can originate from the permanent deformation of either a weak or overstressed subgrade or from an asphalt layer of insufficient stability or shear resistance. The latter problem forms the focus of this investigation because it can be related to poor mix design, while the former is considered to be more of a structural problem (Tutumluer *et al.*, 2003).

Brown (1990) indicated that HMA pavements are generally constructed with approximately 7-8 percent air voids, then densified under traffic loads. During the densification process, the mix stability increases; this in turn resists further densification (Hishop and Coree, 2000a). In the case of a mix which performs well, equilibrium is reached at approximately 4 per cent air voids, where the mix is able to support the traffic loads without significant additional densification (Hishop and Coree, 2000b). Pavements that tend to rut typically do not stabilise at the design air voids, but continue to undergo densification under traffic loads to a lower air-void level (Kandhal *et al.*, 1998). As the asphalt mix approaches saturation, the binder begins to carry more of the load and the mix begins to flow, resulting in ruts (Touahamia *et al.*, 2002). Bouldin *et al.* (1994) and Carpenter (1993) identified three phases shown in Figure 2.5 and Plate IV which show the development of permanent deformation in asphalt. The three stages of permanent deformation are:

- (a) **Primary Phase:** During the initial load cycles the rate of accumulation of permanent deformation is high but decreases rapidly. The initial high rate of deformation may be attributed to the densification of the mix without significant shear deformation. The high deformation rate is probably intensified by the high stress concentrations induced as a result of the load surface irregularities of the sample. The decrease in the rate of

deformation may be due to the strain hardening as a result of aggregate reorientation. The resulting increase in aggregate interaction increases the internal friction component of the mix and hence increases the resistance to deformation.

- (b) **Secondary Phase:** The rate of deformation is nearly constant. In this phase stable shear occurs and the slope has sometimes been used to evaluate the deformation resistance of the mix.
- (c) **Tertiary Phase:** This stage is associated with an increase in the rate of deformation and is sometimes referred to as catastrophic range. It has been claimed that this large scale aggregate movement occurs when the air voids drop below a certain limit (typically 2 to 3 per cent) after which the binder start to acts as a lubricant between the aggregates and tends to push aggregates apart.



**Figure 2.5: Typical Formation of Permanent Deformation in Asphalt**

**Subjected to Repetitive Loading**

*Source: Carpenter, (1993)*



**Plate IV: Permanent Deformation in Asphalt Pavement**  
*Source: Taher (2011)*

### **2.3.2 Fatigue failure of road pavement**

Repeated traffic loads cause tensile and shear stresses to develop in the asphalt layer (Ali, 2005). In Plate IV what appears initially as minor cracks due to excessive loading was further propagated by moisture into a major failure called ‘alligator crack’ in Plate V. Fatigue cracking initiates at locations of critical strain and stress. These locations mainly depend on the stiffness of the asphalt layer and the load configuration (Nukunya *et al.*, 2001). Two types of fatigue cracks initiate in the asphalt layer (Uzan, 2004). The first is the commonly known fatigue cracking that initiates due to bending action, which results in flexural stresses at the bottom of the asphalt layer. The second type, which propagates from the surface to the bottom, is believed to be due to critical tensile and or shear stresses developed at the surface due to high contact pressures at the tire edges-pavement interface. Highly aged thin asphalt layers facilitate the initiation of fatigue related cracks of type two (Ali, 2005).

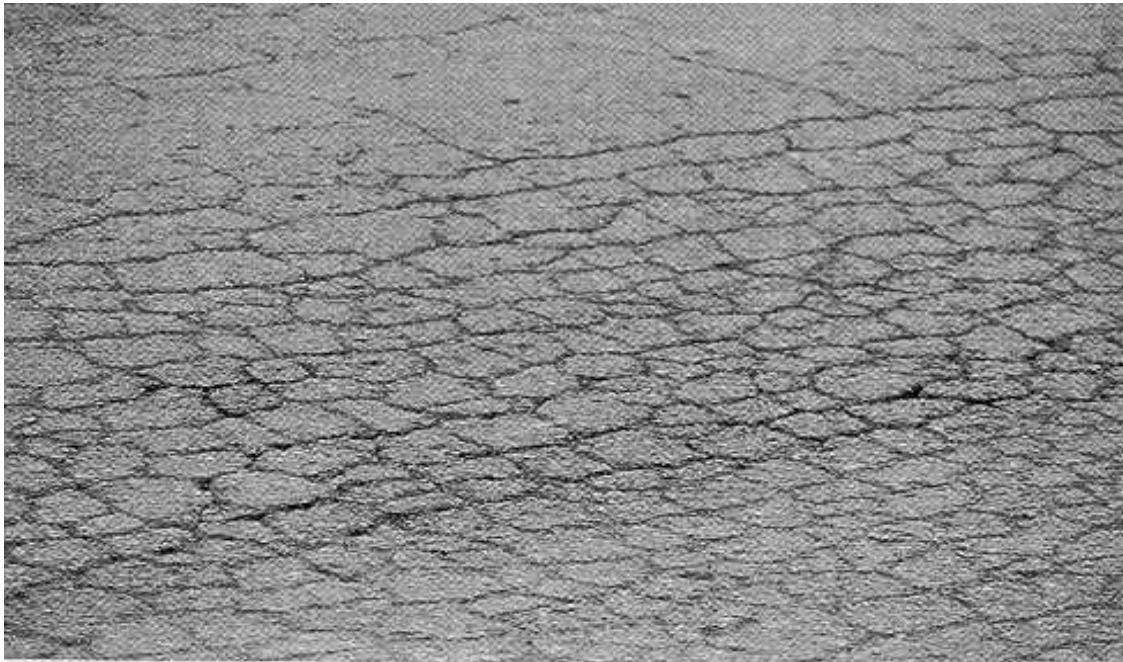
Polymer-modified bitumen bonding to the surface of the filler particles strengthens the interface and results in improved resistance to crack propagation. Smith (2000) discovered that polymer modified or grafted bitumen in mastic concrete has fatigue life approximately 40% greater than the unmodified or ungrafted mastics. The chemical bond between the polymer-modified bitumen and filler particles increases the ability of the composite to pin cracks by reducing particle-matrix debonding ahead of the crack tip and increase shear yielding around filler particles which toughens the asphalt thereby improving the fatigue performance of pavement (Smith, 2000).

Currently, the Department of Defense (DoD), (2001) criterion for fatigue life of HMA surfaces estimated allowable strain repetition cycle,  $r$ , as:

$$\log_{10} r = 2.68 - 5 * \log_{10}(S_A) - 2.665 * \log_{10}(E) \quad (2.1)$$

where  $S_A$  = tensile strain at the bottom of HMA pavement

$E$  = elastic modulus of HMA, psi



**Plate V: Fatigue crack (Alligator crack)**  
*Source: Taher (2011)*

### **2.3.3 Thermal cracking**

Thermal fracture analysis is based on the visco-elastic properties of the asphalt mixture. The procedure requires the characterization of the HMA mix in an indirect tensile mode to measure the creep compliance at one or three temperatures depending on the level of analysis (Mehta *et al.*, 1998; Bahia *et al.*, 2000). When ambient temperature, cooling exerts temperature gradient and the thermal stress increases to the point of equilibrium to tensile strength of the mix, thereby leading to fracture of the asphalt specimen (Chehab *et al.*, 2002; Daniel and Kim, 2002).

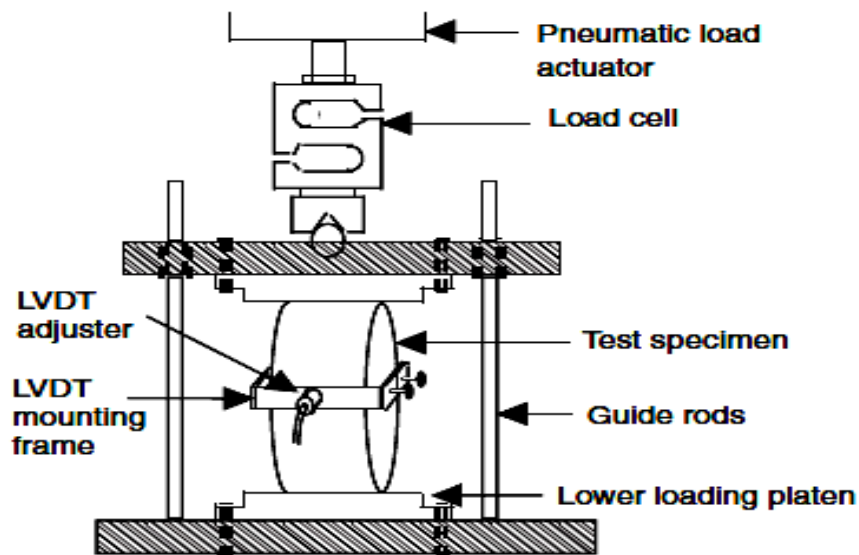
### **2.3.4 Asphalt creep effects and evaluation**

Creep is a deformation due to sustained loading with time or time-dependent strain per unit stress. It is used to evaluate the relative quality and performance of bituminous mix and a measure of deformation and mechanical failure resilience low temperature or thermal cracking (AASHTO T 322-07 or ASTM D 6931-07). According to Wang *et al.* (2012), shear deformations occurring as a result of high shear stresses in the top portion of a bituminous layer may be considered to be the primary cause of rutting in flexible pavements, but a gradual creep is observed at low stress level (O'Flaherty, 2007). The creep test may be determined either in the static or dynamic mode of loading. The method of ASTM D 6931-07 is usually preferred because of its simplicity (khattak and Baladi, 2001).

In the static mode, a loading plate is usually mounted on top of each test specimen and the linearly variable differential transducers (LVDT's) are mounted against the loading plate at points intervals equal to 1/3 of the circumference of the plate for recording values and averaging them (Guddatti *et al.*, 2002). At low stress level, a correction factor from

repeated creep test multiplied by creep test strain gives the expected rut depth (Static loading and unloading may be applied to each specimen in all duration of two hours (one hour loading and one hour unloading) for either four or six inch diameter. Stresses for the four inch (101.6mm) and six inch (152.4mm) diameter specimens are  $0.3565 \text{ N/mm}^2$  (51.7 psi) and  $0.3806 \text{ N/mm}^2$  (55.2 psi) respectively. Creep tests may be performed at temperatures ranging from 40 to  $60^\circ\text{C}$  (Hamzah *et al.*, 2006). While conducting the test, axial deformation is continuously observed with respect to time (Kumar and Veeraragavan, 2011). If the initial height of the specimen and the axial strain,  $\varepsilon$ , are known; the stiffness modulus  $S_{mix}$ , can be determined at any loading time as shown in Figure 2.6 using Equation 2.2 below:

$$S_{mix} = \gamma / \varepsilon \quad (2.2)$$



**Figure 2.6: Creep and Indirect tensile stiffness set-up**  
*Source: Cabrera and Dixon, (1994)*

### 2.3.5 Indirect Tensile Test (ITS)

Indirect Tensile Test (ITS) is a measure of the porosity and mechanical strength of asphalt mix and is relevant in the evaluation of moisture susceptibility of the mix (Lu, 2003). Many test standards may be used; the most prominent are ASTM D 4128-82, ASTM D 6931-07 and AASHTO T 322-07. It is conducted at a temperature of 77 °F (25°C) and at a standard load rate of 50.8mm (2 inches) per minute. Three specimens are usually prepared for testing. A choice between 101.6mm (four inch) and 152.4mm (six inch) diameter specimen is made before compaction but the results of the two ITS test should compare identically for 101.6mm (4 inch) and 152.4mm (6 inch) samples if all factors including homogeneity, isotropy, elasticity, and test condition are maintained (Garba, 2002). ASTM D 6931-07 method is used because its simplicity and adaptation to testing other parameters.

ITS is estimated by loading a cylindrical specimen with a single or repeated compressive load which acts parallel to and along the vertical diametric plane (Hanaa, 2012). The loading pattern is develops a relatively uniform tensile stress perpendicular to the direction of the applied load and along the vertical diametric plane, which ultimately causes the specimen to fail by splitting along the vertical diameter (Sulaiman *et al.*, 2004). Based upon the maximum load carried by a specimen at failure, the indirect tensile strength is calculated as:

$$ITS = \frac{2 * P_{max}}{\pi * t * d} \quad (2.3)$$

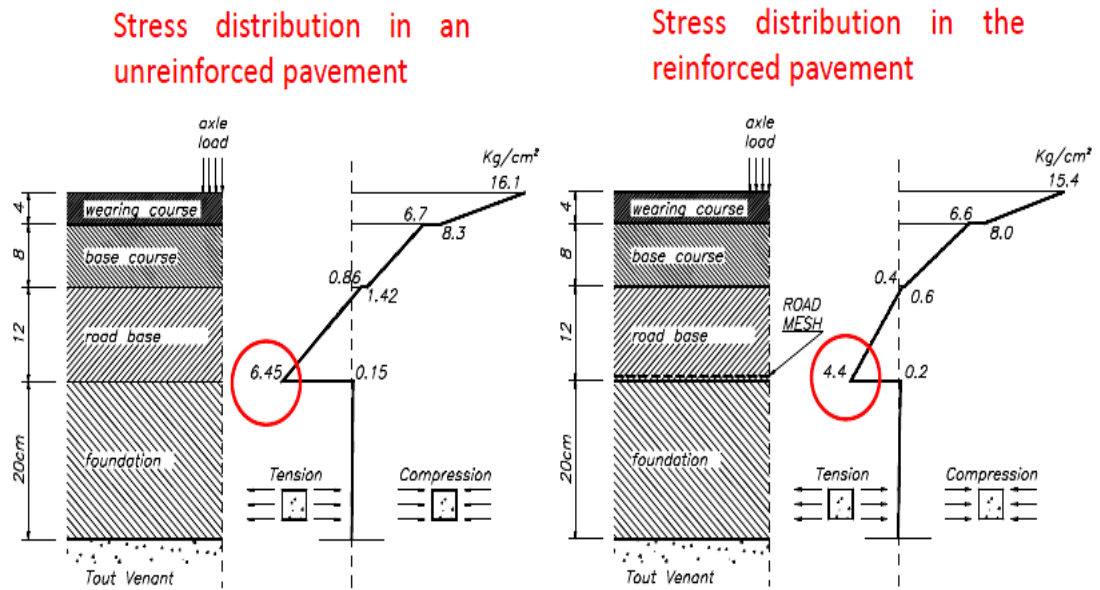
where  $ITS$  = indirect tensile strength (N/mm<sup>2</sup>),

$P_{max}$  = maximum applied load (N),

$t$  = average height of the test specimen (mm),

and  $d$  = diameter of the specimen (mm)

Pavement reinforcement acts as pressure relief, the most pronounced occurring at the base and sub-base, it dampens the stress effects of tyre contact pressure suffered by the surfacing. These tensile stresses in the asphalt are lower due to the presence of the reinforcement as demonstrated by Bruck and Pietro (2014) in Figure 2.7.



**Figure 2.7: Tensile stresses reduction in reinforced asphalt**  
*Source: Bruck and Pietro, (2014)*

### 2.3.6 Resilient modulus

Asphalt Resilient modulus is another mechanical property of asphalt and could be measured by indirect tensile mode (ASTM D 4123, 2005). As the most popular form of stress-strain measurement is used to determine elastic behaviour and for evaluating resilience to properties which include fatigue failure, stripping, and low temperature cracking (Brown and Foo, 1991). It has been discovered by researchers that stiffer pavements has greater resistance to permanent deformation and low response to temperature cracking, while mixes of low stiffness will tolerate low temperature cracking and fail due to excessive creep deformation (Ashekrana, 2004); a careful selection of

materials and quality of method to attain an optimum stiffness which gives excellent performances become necessary (Tahirou, 2009).

The resilient modulus tests are usually conducted on three specimens for each gradation using two load levels at three different temperatures for each load using Horizontal and Vertical (H & V) resilient modulus device which is a pneumatic pulse generator (Chehab *et al.*, 2000).

In the tropics, the ambient temperatures usually simulated are 77°F (25 °C), 104 °f (40 °C) and 140 °f (60°C). The two load levels were 10 percent and 15 percent of the indirect tensile strength at 77 °f. Load cell and Linear Variable Differential Transformers (LVDTs) are connected through alternating current carrier preamplifiers to a two-channel Oscillographic strip-chart recorder (Briggs and Lukanen, 2000). According to the requirements of ASTM D 4123 (1982), the value for Poisson's ratio used in calculating the test results was assumed to be 0.35 for all temperature. The load pulse duration was 0.10 sec. and the frequency was 1 pulse per second.

## **2.4 Field Test Conditions**

Temperature has been found to have a significant effect on rutting. Researchers (Diab *et al.*, 2013; Al-Omari *et al.*, 2002 and De Angelo *et al.*; 2001) have recognized the need to conduct laboratory tests at temperatures within the high-temperature range of those encountered in the field. Bonnot (1986) selected a test temperature of 60°C for wearing-course asphalt concrete and 50°C for base courses. Hofstra and Klomp (1972) and Diab *et al.* (2013) observed that rutting increased by a factor of 250 to 350 with a temperature increase from 68°F to 140 °F (20°C to 60°C).

Just as it must be emphasized that the states of stress and strain caused by traffic loading also significantly influence pavement rutting, all the above factors must be put into consideration to mitigate rutting propensity of asphalt concrete (Mohammad *et al.*, 2004). Table 2.3 summarises the characteristics of asphalt mixtures and test or field conditions, which affects rutting of asphalt concrete pavements as presented by Sousa *et al.* (1991).

**Table 2.3: Factors affecting rutting of asphalt-concrete mixtures**

Material	Factor	Change in Factor	Effect of Change in Factors on Rutting Resistance
Aggregate	Surface texture	Smooth to rough	Increase
	Gradation	Gap to continuous	Increase
	Shape	Rounded to angular	Increase
	Size	Increase in maximum size	Increase
Binder Mixture	Stiffness <sup>a</sup>	Increase	Increase
	Binder content	Increase	Decrease
	Air void content <sup>b</sup>	Increase	Decrease
	VMA Method of compaction <sup>d</sup>	Increase	Decrease <sup>c</sup>
Test field Conditions	Temperature	Increase	Decrease
	State of stress/strain	Increase in tire contact pressure	Decrease
	Load repetitions	Increase	Decrease
	Water	Dry to wet	Decrease if mix is water sensitive

<sup>a</sup>Refers to stiffness at temperature at which rutting propensity is being determined. Modifiers may be utilized to increase stiffness at critical temperatures, thereby reducing rutting potential.

<sup>b</sup>When air void contents are less than about 3 percent, the rutting potential of mixes increases.

<sup>c</sup>It is argued that very low VMA (e.g., less than 10 percent) should be avoided.

<sup>d</sup>The method of compaction, either laboratory or field, may influence the structure of the system and therefore the propensity for rutting.

**Source: Sousa et al.(1991).**

## 2.5 Approaches to Fatigue Failure of Pavement

Pavement fatigue is the process of cumulative damage resulting from repeated traffic loading; fatigue cracking is its resultant distress (Zhang *et al.*, 2000; Carpenter and Shen, 2005). According to Arabzadeh (2015), Two approaches to characterize the fatigue

behavior of asphalt concrete; one is a phenomenological approach and the other a mechanistic approach.

### **2.5.1 Phenomenological Approach**

Adedimila and Kennedy (1976) used the repeated load indirect tensile test to evaluate the fatigue behavior of a mixture which may be defined by the relationship between the number of cycles to failure and initial tensile strain, as follows:

$$N_f = a * (1/\varepsilon)^b \quad (2.4)$$

where  $N_f$  = number of cycles to failure,

$\varepsilon_0$  = initial tensile strain, and

a, b = material-dependent coefficients.

Monismith et al. (1985) improved equation (2.4) to account for the effects of mixture stiffness for asphalt concrete as:

$$N_f = a * (1/\varepsilon)^b S^c \quad (2.5)$$

where S is mixture stiffness and c is the material-dependent coefficient.

### **2.5.2 Mechanistic-Empirical Approach**

The above relationships are applicable only to specific environments, materials, and traffic conditions, and are empirical. It is required, however, that the parameter obtained from a simple performance test should be linked to the advanced material characterization methods needed for detailed distress prediction models (Arabzadeh, 2015).

### 2.5.3 Rheology and asphalt pavement structure

Rheology according to Sperling (2006) is the scientific study of deformation and flow properties of matter. This science is concerned with the response of materials to stress application. That response may be irreversible viscous flow, reversible elastic deformation, or a combination of the two. Deformation is the relative displacement of parts or points of a body. It can be divided into flow and elasticity. Flow is irreversible deformation whereby when the stress is removed; the material never reverts to its original form. It implies that work is converted to heat (Lafi, 2010).

Elasticity is recoverable deformation. The deformed body recovers to its original shape or form and the applied work is largely recoverable too. Viscoelastic materials exhibit both flow and elasticity (Sperling, 2006).

### 2.5.4 Influence of viscosity on bitumen

Sperling (2006) defined viscosity as the ratio of the stress to the shear rate; and this constant is equal to the slope of the flow curve,  $\eta = d\tau / d\dot{\gamma}$ . The relationship  $\tau / \dot{\gamma}$  is the viscosity,  $\eta$ , for a Newtonian liquid and the apparent viscosity,  $\eta_a$ , for a non-Newtonian liquid. The kinematic viscosity is the ratio of viscosity coefficient and the density, that is,  $\nu = \eta / \rho$ . The fluidity is the reciprocal or inverse of viscosity,  $\phi = 1/\eta$ . The units of viscosity, which is grams per centimetre second (g/cm·s) is called poise.

According to Dracapa *et al.* (2007), It is customary to represent the deformation behavior of metals and other solids by a model called the linear or Hookean [elastic solid](#) (displaying the property known as [elasticity](#)) and that of fluids by the model of the linear [viscous](#) or [Newtonian fluid](#) (displaying the property known as [viscosity](#)). These classical models are, however, inadequate to depict certain [nonlinear](#) and time-dependent deformation behaviour

that is sometimes observed (Ferguson and Kemblowski, 1991). Rheological behaviour is particularly readily observed in materials containing typically thousands of atoms per molecule, examples of which are asphalt, [Paint](#), [Plastics processing](#), [Polymer](#), [Rubber](#) and [Surface coating](#) (Sperling, 2006). A Hookean (purely elastic) solid under a constant stress deforms immediately to a constant strain, then recovers instantly and completely when the stress is removed (for example, a steel spring). According to Barnes (2000), Hooke's law describes the behaviour of a solid, relating the applied strain to the resultant stress through proportionality constant called the modulus,  $G$ , expressed as in Equation 2.6.:

$$\tau = G\gamma \quad (2.6)$$

Where,  $\tau$  is the shear stress and  $\gamma$  is the shear strain.

The modulus is a measure of the material's stiffness, that is, ability to resist deformation (Bell and Claxton, 2000). On the other hand, a Newtonian fluid (purely viscous) deforms continuously, while the stress is applied but does not recover when the stress is relieved (Smith, 2000).

Newton's law relates the applied stress to the shear rate through a proportionality factor,  $\eta$ ,

$$\tau = \eta * \frac{\partial \gamma}{\partial t} \quad (2.7)$$

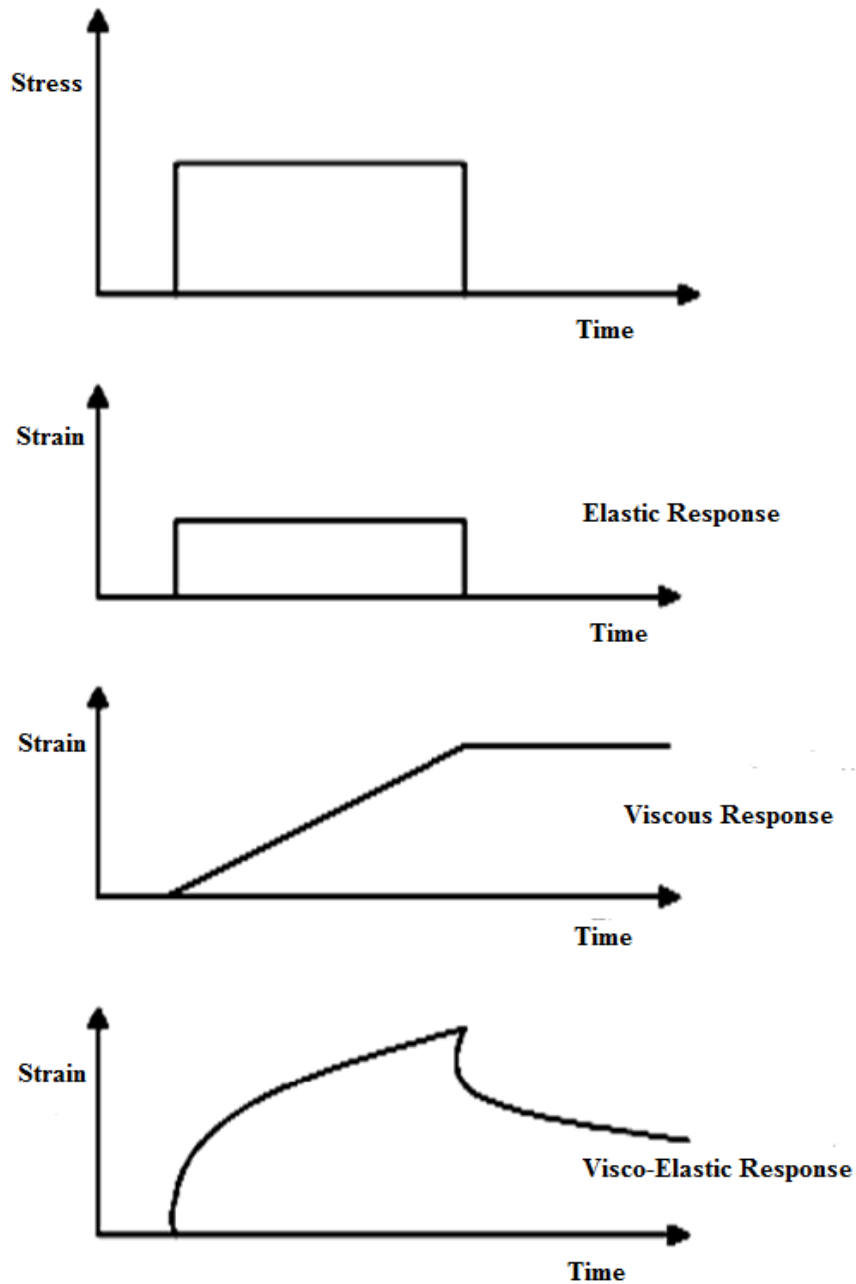
where,  $\tau$  is the coefficient of viscosity.

A fluid is Newtonian, if the viscosity is independent of shear rate. Viscoelastic materials like bitumen responds to an applied force or displacement through a combination of elastic and viscous behaviour (Hookean and Newtonian behaviour), but with incomplete recovery (Ould-Henia and Dumont, 2006). Thus, these materials exhibit the related phenomena of creep and stress relaxation (Rheometrics Asphalt Analyzer (RAA), 1992; ).

Since the behaviour of asphalt also depends significantly on the temperature, at high temperatures and under sustained loads, bitumen acts like a viscous Newtonian fluid. Viscous liquids like hot bitumen are sometimes called plastic because once they start flowing, they do not return to their original position (Zhao *et al.*, 2006; Rodríguez-Valverde *et al.*, 2008). As a result, in hot weather, some hot mix asphalt pavements flow under repeated loading and ruts forms (Rahman, 2004). At low temperatures, or under rapidly applied loads, bitumen behaves like an elastic solid. When loaded, the material deforms, and when unloaded, it returns to its original shape. However, when the material is stressed, through an applied load, it may become too brittle and crack. For this reason, low-temperature cracking may occur in asphalt pavement during cold weather. Most environmental conditions result in the asphalt material being subject to intermediate temperatures. In these climates, asphalt exhibits the characteristics of both viscous liquids and elastic solids (Kumnuantip and Sombatsompop, 2003). A portion of the material's response to an applied load is elastic or viscoelastic (recoverable with time), while some of the response is plastic and non-recoverable. The elastic response or recoverable part, viscoelastic response and plastic response or non-recoverable, which appears in the form of permanent deformation have been illustrated in Figure 2.8 (Gibb, 1996).

In a dynamic mechanical test, an oscillatory strain is applied to a sample and the resulting stress developed in the sample is measured (Kumnuantip and Sombatsompop, 2003). For solids that behave ideally (Hookean), the resulting stress is proportional to the strain amplitude, and the stress and strain signals are in phase (Hesp *et al.*, 2009; Bouldin *et al.*, 1990). If the sample is an ideal liquid (Newtonian), then the stress is proportional to the strain rate. In this case, the stress signal is out of phase with the strain signal, leading the strain signal by  $90^{\circ}$ . The stress signal generated by a viscoelastic material is out of phase

with the strain signal by an angle,  $\delta$ , called the phase angle. This can be separated into two components: an elastic stress  $t'$  which is in phase with the strain, and a viscous stress  $t''$  which is  $90^\circ$  out of phase with the strain, that is, in phase with the strain rate (Binard *et al.*, 2004).



**Figure 2.8: Stress Strain Behaviors of Bituminous Material**  
*Source: Gibbs (1996)*

The elastic stress is a measure of the degree to which a material behaves as an elastic solid; the viscous stress, the degree to which a material behaves as an ideal fluid (Malkin and Isayev, 2006; and Mohammed *et al.*, 2003). The ratio of the elastic stress to strain is the storage (or elastic) modulus  $G'$  and the ratio of the viscous stress to strain is the loss (or viscous) modulus  $G''$ . The complex modulus  $G^*$ , a measure of a material's overall resistance to deformation (that is, stiffness), is given by:

$$G^* = G' + iG'' \quad (2.8)$$

where  $i$  is the imaginary unit

Since temperature is very critical to viscosity, viscoelastic melts is associated with  $G'$  with the ability of a melt to recover from a deformation (Becker *et al.*, 2003). Unfortunately, this may not hold in some cases and stiffness of the melt may be preferred.

Also, viscoelastic material also possesses a complex dynamic viscosity as can be expressed in Equation 2.9,

$$\eta^* = \eta + i\eta'' = G/i\omega; \quad (2.9)$$

where  $\eta' = G'/\omega$ ;

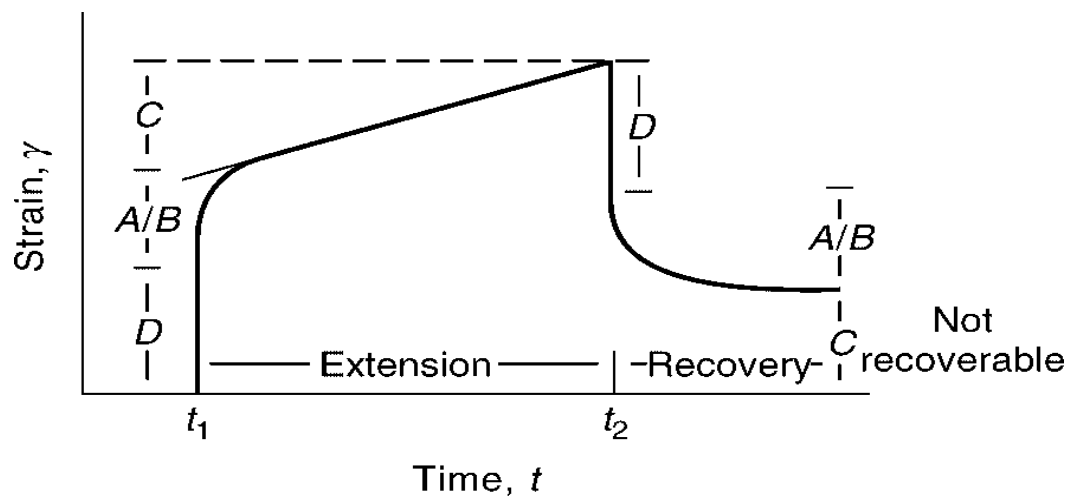
and  $\eta'' = G''/\omega$ ,

$\omega = 1/t$  is the angular frequency.

The parameter  $\eta^*$  is important for many viscoelastic fluids in that a plot of the absolute value of the complex  $\eta^*$  versus angular frequency ( $\omega$ ) is similar to a plot of shear viscosity  $\eta$  versus shear rate. The correspondence is called Cox–Merz empirical relationship (Geissle and Hochstein, 2003). The parameter  $\eta'$  is called the dynamic viscosity and has relationship with  $G'$ , the loss modulus;  $\eta''$  is a measure of elasticity.

Apart from depending on parameters like shear stress, shear rate, and time, viscosity is largely sensitive to changes in temperature (Osman *et al.*, 2004). As temperature increases, viscosity decreases. This dependence is logarithmic and can be substantial, up to 10% change/ $^{\circ}\text{C}$  (Seidel, 2008). This can lead the material to deform and flow and the measurement of flow with time after application of stress, which is called creep may be one of the consequences of viscosity (Le Meins *et al.*, 2002).

Figure 2.9 below shows a typical creep curve for a viscoelastic material. Stress is applied at time  $t_1$  and is removed at  $t_2$ . The linear portion of the slope of the creep curve gives the shear rate, and the viscosity is the applied stress divided by the slope. Steep slope shows a low viscosity, and a gentle slope indicates high viscosity. Up to maximum deformation at  $t_2$ , the strain is in visco-elastic strain and beyond it, is boundary for rebound recovery, which is transition to permanent deformation where the strain is not recoverable (Gibb, 1996).



**Figure 2.9: A typical creep curve for a viscoelastic material**  
*Source: Gibb (1996)*

## **2.6 Asphalt Durability**

Bitumen ageing affects pavement durability and the process occurs naturally when the binder is reduced to thin film around aggregates particles having consequences of increasing the mix stiffness, lowering the resilience to cracking strain and rendering the pavement surface susceptible to damage (Robison, 2004). Bitumen ages by oxidation, volatilization, exudation of oils and steric or physical means of molecular reorientation and steady wax crystallization (Read and Whiteoak, 2003). Zoorob *et al.* (2002) predicated field ageing on loss of volatiles and adapted it to laboratory tests. The nature, rate and amount of ageing depend on factors like temperature, exposure to oxygen, ultra-violet light, structural chemistry of bitumen (Tahirou, 2009).

In order to have an idea of durability, short term Thin Film Oven Test (TFOT) and the Pressure Ageing Vessel (PAV) test were developed (Bell *et al.*, 1994). The findings influence the performance of the bituminous pavement and are representative of field experience (Hagos, 2003; 2008). Age hardening of bitumen occurs in three stages: - storage, construction process and service life of pavement. Hardening or ageing of bitumen may increase resistance to deformation, at intermediate and low pavement temperatures, however, harder asphalts are susceptible to fatigue cracking (Bahia and Anderson, 1995)

## **2.7 Asphalt Mixture**

Marshall or Hveem method is generally selected as a preliminary design tool in the determination of adequate asphalt content because binder content also affects the mixture's ability to resist permanent deformation (Sousa *et al.*, 1991). Reducing air voids to point 4%, increases the resistance of the mixture to rutting. In the field, low air-void content is generally achieved with higher harsh or low workability mix and compactive energy

(Sadasivam, 2004). Such a combination according to Uge and van de Loo (1974) should result in an improved arrangement of the mineral skeleton and thereby an increase in internal friction and will be very resistant to permanent deformation.

However, Cooper *et al.* (1985) observed that good resistance to permanent deformation requires low voids in the Mineral Aggregate (VMA), they cautioned that the lowest theoretical VMA could be undesirable as it may not allow sufficient voids in the aggregate for enough binder to ensure satisfactory compaction without the mixture becoming overfilled. In the same vein, Mahboub and Little (1988) indicated that higher bitumen contents producing lower air voids, increases rutting potential. They suggested that the reduction in air voids as a result of increased asphalt content indicates that void space is becoming filled with asphalt. As a result, the increase in asphalt content is equivalent to the introduction of lubricants between aggregate particles otherwise separated by a very tight network of air voids. This phenomenon causes the mixture having the higher bitumen content to be more susceptible to permanent deformation.

## **2.8 Asphalt Constituent Materials and Characterizations**

### **2.8.1 Characterisation of polypropylene material**

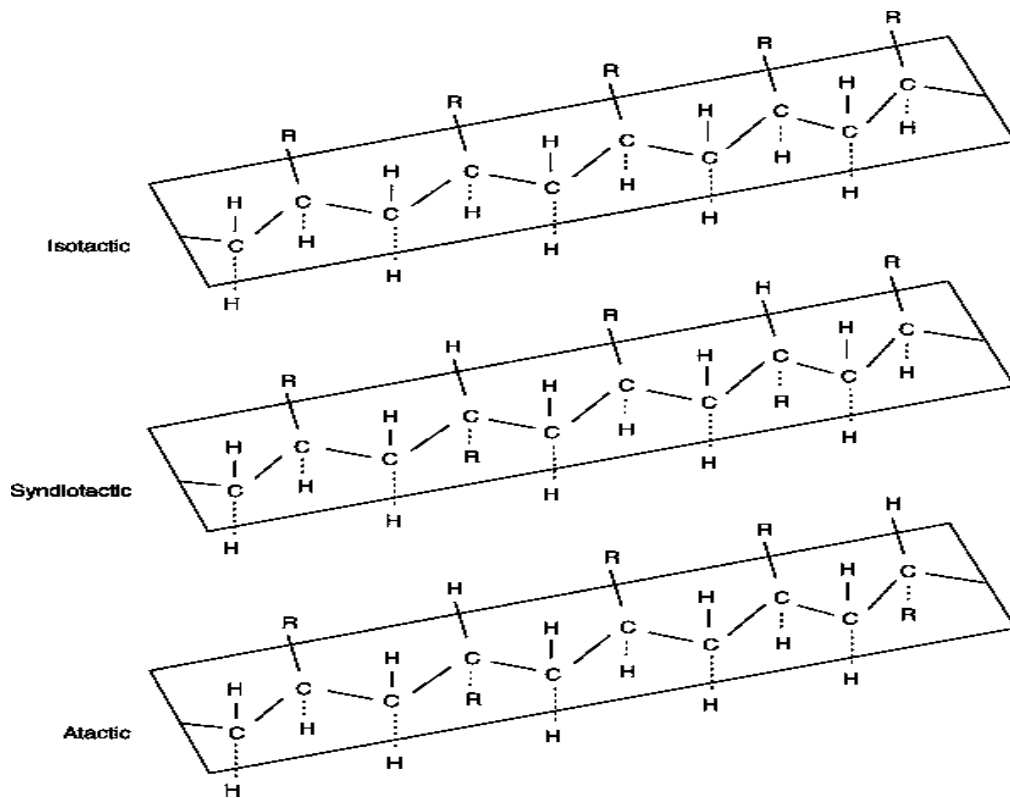
Polypropylene being one of the most widely used polymers in the world because of the widespread availability and low manufacturing cost. The reactivity of propylene is a result of the olefinic double bond in  $H_2C=CHCH_2$ , which gives rise to addition reactions (Gambarotta, 2003).

PP was invented in 1955 by an Italian Scientist F.J. Natta by addition reaction of propylene gas with a stereospecific catalyst titanium trichloride (Chung, 2013). Polypropylene is a hydrocarbon polymer, having molecular formula  $(C_3H_6)_n$  where n is the number of

molecules involved in the chain. **Polypropylene is an economical material that offers a combination of outstanding physical, chemical, mechanical, thermal and electrical properties not found in many other thermoplastics (Ghasem *et al.*, 2009). Compared to low or high density polyethylene, it has a lower impact strength, but superior working temperature and tensile strength. Polypropylene possesses excellent resistance to organic solvents, degreasing agents and electrolytic attack (Shiga, 2010). It has lower impact strength, but its working temperatures and tensile strength are superior to low or high density polyethylene (Janicek *et al.*, 2011 and Piel *et al.*, 2006). It is light in weight, resistant to staining, and has a low moisture absorption rate. This is a tough, heat-resistant, semi-rigid material, has excellent resistance to acids and alkali, but poor aromatic, aliphatic and chlorinated solvent resistance (Haque, 2014).**

#### **(A) Structures of the PP Molecules**

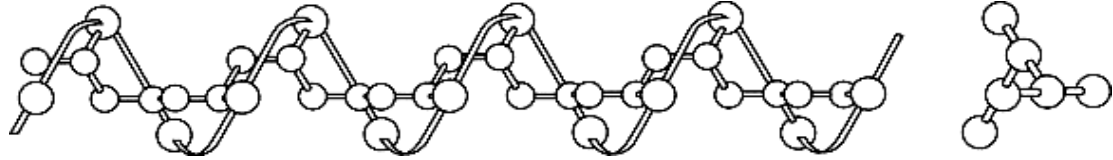
PP is basically methyl-ethylene polymer (Chen *et al.*, 2010). With respect to the regularity of the methyl group placement relative to the other methyl groups along the chain backbone otherwise called the stereospecificity of the polymerization, there are three limiting classifications of stereospecificity in PP which are: isotactic polypropylene (iPP), syndiotactic polypropylene (sPP) and atactic polypropylene (aPP) (Meyer, 2015). **Figure 2.10 shows the molecular structures of the three classifications of PP.**



**Figure 2.10: Structures of (iPP), (sPP) and (aPP)**  
*Source: Myer (2015)*

The isotactic polypropylene (iPP) has all of the methyl groups having the same configuration with respect to the polymer backbone, while syndiotactic polypropylene (sPP) has the methyl groups which have alternating configurations. The atactic polypropylene (aPP) has a random configuration (Meyer, 2015). iPP is overwhelmingly the most commercially significant form of PP for most products (Haylock, *et al.*, 2000; Galli *et al.*, 1995). Figure 2.11 shows the chain conformation of isotactic polypropylene (iPP) and **Table 2.4a and 2.4b summarizes the properties.** The density ( $\rho$ ) of iPP in the  $\alpha$ - form shows variation between the limit of 100% amorphous ( $\rho_a = 0.850$  to  $0.855$  g/cm<sup>3</sup>) and 100% crystalline ( $\rho_c = 0.936$  to  $0.946$  g/cm<sup>3</sup>) (Busico *et al.*, 1994). In this way, the

measured mass density,  $\rho$ , gives a measure of the crystallinity. Density gradient technique (ASTM D1505-10) is mostly used to measure  $\rho$ -values.



**Figure 2.11: Conformation chain of iPP**  
*Source: Haylock, et al (2000)*

**Table 2.4a: Physical Properties of PP Fibers**

<b>Characteristic</b>	<b>Value</b>	<b>Standard</b>
Water absorption, %	0	ASTM D570
Fibre length, mm	(12mm) $10 \pm 2$	-
Tensile strength (minimum), MPa	276	ASTM D-638
Specific gravity, kg/m <sup>3</sup>	$0.91 \pm 4$	ASTM D-792
Melting temperature, °C	160	-

*Source: (Abtahi et al., 2011)*

**Table 2.4b: Typical properties of polypropylene**

<b>Characteristic</b>	<b>Value</b>	<b>Standard</b>
Homogeneity,%	100%	-
Color	Transparent	-
Length, mm	3-50	-
Melting point, °C	160	-
Specific gravity, g/cm <sup>3</sup>	0.91	ASTM D-792
Specific volume (in. <sup>3</sup> /lb)	30.8-30.4	ASTM D-792
Fire point, °C	590	-
Glass transition temperature, °C	-18	-
Alkali resistance as % of strength	99.5	-
Retained after treatment in 40% NaOH solution at 20°C for 1000 h water absorption, (%)	0.01-0.02	ASTM D-570
Moisture retention, at 20 °C, 65% relative humidity and 24hrs (%)	<0.1%	ASTM D570
Rupture resistance, MPa	31-41	ASTM D-638
Elongation, %	10-20	ASTM D-638
Elongation at rupture,%	100-600	ASTM D-638
Tensile strength, MPa	31-37	ASTM D-638
Compressive strength, Mpa	37-55	ASTM D-695
Bending strength, MPa	41-55	ASTM D-790
Tensile modulus, MPa	1137-1551	ASTM D-638
Bending modulus,73°F, MPa	1172-1723	ASTM D-79
Hardness, Rockwell	R80-R110	ASTM D-785
Thermal conductivity (10 <sup>4</sup> cal-cm/sec-cm <sup>2</sup> -°C)	2.8	ASTM C-177
Thermal expansion, linear, m/m/°C	0.031-0.039	ASTM D-696

*Source: (Tapkin, 2007)*

### **(B) Properties/Merits of Polypropylene in Asphalt**

The properties inherent and merits of using PP materials as outlined by (Haque, 2014) are:

- (i) It has low cost advantage.
- (ii) The flexural strength is measurably excellent.
- (iii) Almost all thermoplastic equipment can process it.
- (iv) It has low coefficient of friction.
- (v) The electrical insulating property is excellent.
- (vi) It has good fatigue resistance.
- (vii) It is a moisture resisting material.

- (viii) The polymer has high heat resistance.
- (ix) PP has very good chemical resistance.
- (x) It has lightweight.
- (xi) The polymer has high tensile strength.
- (xii) It is impact resistant and has high compressive strength.
- (xiii) Most alkalis and acids are resisted by PP.
- (xiv) It can resist stress cracking, retains stiffness and flexibility.
- (xv) It is not toxic.

### 2.8.2 Grading of aggregates

The strength of asphalt is derived from friction between the aggregate particles, the viscosity of the bitumen under operating conditions and the cohesion within the mass resulting from the bitumen itself and the adhesion between the bitumen and aggregate. Typical standard proportions of asphalt mixtures used in compliance with (Asphalt Institute, 1997) for combined wearing course mixtures is shown in Table 2.5.

**Table 2.5: Aggregate grading for asphalt concrete wearing course**

Sieve Size (mm)	Percentage Passing Sieve Size for Nominal Size Aggregates		
	19mm size	12.5mm size	9.5mm size
25.00	100	-	-
19.00	90 – 100	100	-
12.50	-	90 – 100	100
9.50	56 – 80	-	90 – 100
4.75	35 – 65	44 – 74	55 – 85
2.36	23- 49	28 – 58	32 – 67
1.18	-	-	-
0.60	-	-	-
0.30	5 – 19	5 – 21	7 – 23
0.075	2 – 8	2 – 10	2 – 10

*(Asphalt Institute, 1997)*

Nominal size of aggregates determines the minimum VMA required and a compacted blend of crushed aggregates will approach a maximum density if the particle size distribution follows the Fuller curve (Roberts *et al.*, 1996; Zhou *et al.*, 2003; Kandhal and Cooley, 2003). Overseas Road Note 19 (TRRL, 2002) suggested that particle size distributions which pass below the restricted zone will normally provide the most effective material for roads carrying very heavy traffic and for severe situations, but it is possible to adjust the proportions of larger sized aggregates and produce an equivalent increase in VMA which has the propensity of adjusting the particle size distribution to pass outside the restricted zone (Roberts *et al.*, 1996). Grading which closely approach Fuller 0.45 power straight line grading charts must be adjusted within acceptable limits away from 0.45 power straight line to increase the VMA values as explained by Federal Highway Administration (Roberts *et al.*, 1996) in Figures 2.12 and 2.13. Finer aggregates impact on pavement by increasing both VMA and VIM, while coarser aggregates give the contrast.

Pavement mix should be designed not to exceed the 'refusal density' which occurs when the pavement attains its maximum density under any form of compaction. This allows enough bitumen to be used to achieve maximum durability of the asphalt without the mixture flushing (Arand, 1985 and Ter Huerne, 2000). At the point of refusal density, void in the mix (VIM) is compared to the lower threshold level of 3% and this is indicative to the propensity of the mix for rutting, corrugation, shoving, segregation and other distress forms within pavement service life (Cooper *et al.*, 1985). FHWA 0.45 Power Chart is a good tool for the gradation curve for HMA mix; however, it is always better to keep the gradation curve away from the maximum density line for the following reasons (Kandhal and Cooley, 2001):

- (a) Maximum surface area is unavoidable when aggregate gradation follows the maximum density line.
- (b) Due to excessive maximum surface area the demand for bitumen will increase and the design becomes uneconomical.
- (c) Percent air voids (VIM) and Voids in mineral aggregate (VMA) will inherently be on lesser side.
- (d) Due to higher bitumen content the mix will have the propensity to fail by plastic deformation and rutting.

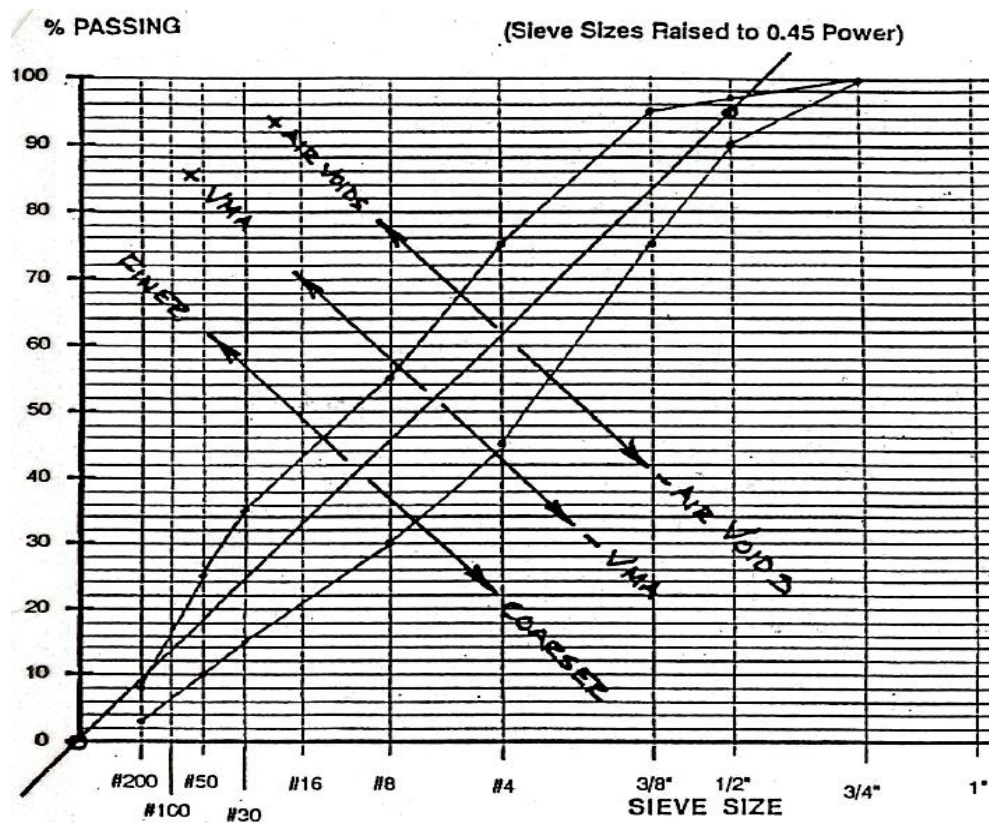
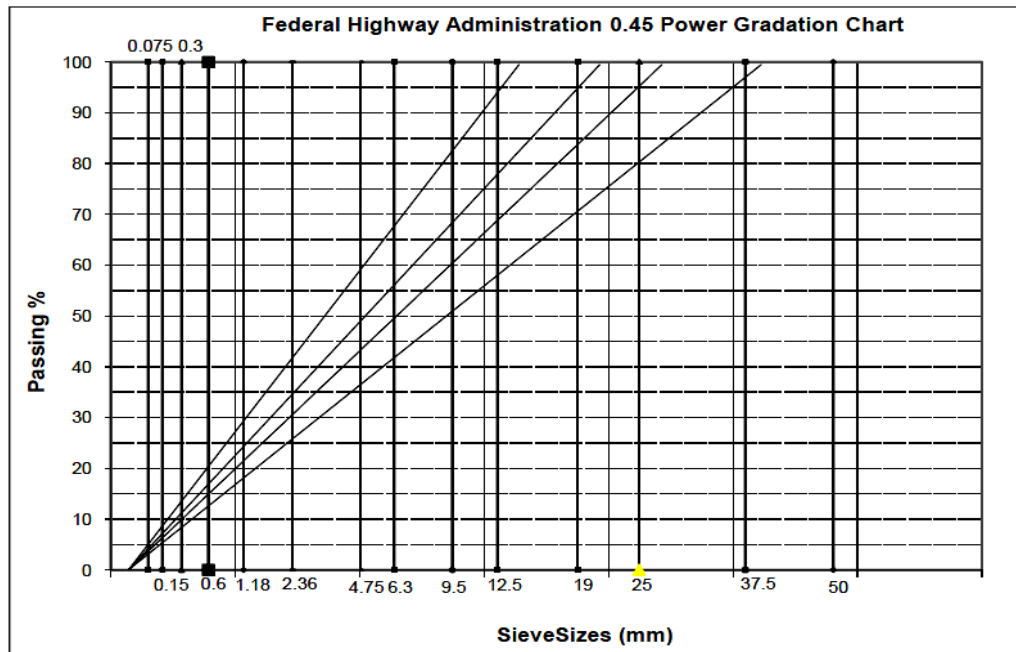


Figure 2.12: Grading maximum density line  
 Source: Roberts et al. (1996)



**Figure 2.13: Federal Highway Administration 0.45 Power Gradation Chart**  
*Source: Roberts et al. (1996)*

## 2.9 Desirable Aggregate Test Qualities

Aggregate forms the major component of asphalt and the quality and physical properties of this material has a large influence on mix performance both during preparation and in service. Studies by researchers (Philip *et al.*, 1994; Masahiko, 2006) have shown that aggregate interlocking and densification when they park the matrix of the structure contributes to greater stability through resistance it offers to deformation and plastic flow (Kumar *et al.*, 2007). This shear resistance and interlocking can improve the mix strength which is measure of load bearing capacity and resistance to rutting and shoving of asphalt pavement (Moore and Welke, 1979; Adedimila and Olutaiwo, 2008).

Basically, rounded aggregates have weak interlocking capability and can easily “slide by” each other when subjected to loading shear stresses. Increasing the proportion of crushed

coarse and fine aggregates in an asphalt mixture can significantly increase the propensity of the mix to resist plastic deformation (Putman and Amirkhanian, 2004).

Typically, the qualities required of aggregates are in terms of shape, hardness and strength, durability, cleanliness, bitumen affinity and porosity (Bose *et al.*, 2006). A good aggregate should have the following qualities:

- (a) It should be clean and free of clay and organic materials.
- (b) It should be cubical and not excessively flaky, to provide good mechanical interlock.
- (c) It is expected to be strong enough to resist crushing and impact.
- (d) It should be resistant to abrasion and polishing when exposed to traffic.
- (e) It should not be absorptive as highly absorptive aggregates are wasteful of bitumen and also give rise to problems in mix design.
- (f) It should have good affinity with bitumen to enhance the bond.

### **2.9.1 Aggregate flakiness index**

Flakiness index of aggregates represents the percentage of the coarse aggregate having least dimension less than 0.6 times the mean dimension (Baghini et al, 2015). It determines that bituminous mixtures should have a satisfactory shape and that a large proportion of the material should be cubical and not flaky as flaky aggregates packs poorly and has low strength. It is determined by passing a material through 63mm sieve and retained on a 6.3mm sieve. BS 812:105.1 (1990) recommends a value less than 35.

### **2.9.2 Strength and toughness**

Aggregates provide most of the load carrying capacity of the asphalt mixtures and should be sufficiently strong and tough to resist the applied wheel loads (Behnood, 2012). The Los

Angeles (L.A.) abrasion test, aggregate crushing and aggregate impact tests are commonly used to control the desired strength and toughness of mineral aggregates. ASTM C131 (2006) recommends values less than 25 to be adequate for both impact and crushing values. Some of the desirable aggregates properties according to standard recommendations are summarized in Table 2.6.

**Table 2.6: Recommended Aggregate Properties**

<b>Tests</b>	<b>Standards</b>	<b>Limits</b>
1. Sampling	ASTM D75:2014	-
2. Los Angeles Abrasion Loss Base Course & Binder Course Wearing Course	ASTM C131:2006	30% Max. 25% Max.
3. Aggregate Crushing Value Base Course & Binder Course Wearing Course	BS 812 Part 110:1990	25% Max. 20% Max.
4. Soundness (MgSO <sub>3</sub> )	ASTM C88:2005	10% Max
5. Acid Soluble Chloride	BS 812 Part 117:1988	0.1% Max.
6. Acid Soluble Sulphate	BS 812 Part 118:1988	0.5% Max.
7. Flakiness Index Base Course & Binder Course Wearing Course	BS 812 Part 105.1:1989	30% Max. 25% Max.
Elongation Index Base Course & Binder Course Wearing Course	BS 812 Part 105.2:1990	30% Max. 25% Max.
Water Absorption Coarse Aggregates Fine Aggregates	ASTM C127-88:2001 ASTM C128: 2015	2% Max. 2% Max.

## 2.10 Bitumen

Bitumen as a binder of mineral aggregates gives a stable pavement matrix. Generally, various tests used to select or assess the quality and ease of handling of bitumen are viscosity, penetration, ductility, specific gravity, solubility, flash and fire point and softening point (Read and Whiteoak, 2003).

### **2.10.1 Viscosity test (ASTM D88:2007)**

Viscosity is an inverse of fluidity which is resistance to flow. It is another means of grading bitumen apart from conventional penetration units. The degree of fluidity of the binder at the application temperature greatly influences the strength characteristics of the resulting paving mixes (Baghini *et al.*, 2013). There is an optimum value of viscosity for each aggregate gradation and bitumen grade, high or low viscosity during mixing or compaction has been observed to result in lower stability value. The viscosity is determined by measuring the time taken by 50ml of bitumen to flow from a cup through a specified orifice under standard test conditions and specified temperature.

### **2.10.2 Effect of temperature and binder viscosity**

The dependence of the flow properties of bituminous mixtures on temperature is due to the changes in the rheological properties of the binder, the dominating factor being the great dependence of viscosity on temperature. Thus, the resistance to deformation of asphaltic materials decreases rapidly with increase in temperature (Mashaan *et al.*, 2014). Therefore, the maximum temperature experienced in a road surfacing should be taken into account when giving requirements for stability. Also, various preliminary tests have been developed for control of quality of bitumen and assessing the suitability of bitumen for specific applications (Tahirou, 2009). Basically, these tests should measure consistency, which is vital to mixing and placing; durability, which makes it remain effective in hostile environments; rate of hardening, which is vital to construction operations and workability, and finally, safety test (John and Davis, 2003). The procedural details of the above tests should be carried out according to the relevant sections of ASTM standard to show the practical significance of the binder.

### **2.10.3 Ductility test (ASTM D113:2007)**

Ductility is a property of bitumen that is usually linked with adhesion and temperature susceptibility (Adriescu and Hesp, 2009). A bitumen sample having high ductility has high adhesion and susceptibility. Susceptibility is a measure of change in consistency for change in temperature (Binard *et al.*, 2003). Ductility is the distance in centimeters that a standard briquette of bitumen cement will stretch before breaking. The minimum cross section of the briquette is  $1\text{cm}^2$ ; the test is made at a temperature of  $25^\circ\text{C}$  and the rate of pulling of the two ends of briquette is  $5\text{cm}/\text{min}$ .

### **2.10.4 Penetration (ASTM D5:2005)**

This test is very vital to empirical measurement of consistency of bitumen (Toth, 2001). The test, a measure of hardness, is performed by allowing a needle of specified dimensions, which is loaded to  $100\text{g}$  to penetrate into the material for a period of 5 seconds. Penetration is measured by the distance the needle sinks into the bitumen, in units of  $1/100\text{cm}$ . When the penetration of bitumen is high, it becomes softer. Bitumen with penetration grade of 60-70 or 60/70 means that the penetration of the needle in bitumen is in the range of 60 to 70 at standard test condition.

### **2.10.5 Softening point (ASTM D36:2009)**

This is a measure of temperature susceptibility and a measure of flow of bitumen in service. It is a form of consistency test that determines the melting point of bitumen and is used for control in refining operation (Patekar and Ranadive, 2014). The test for determining softening is called ball-and-ring softening point. In the test, asphalt is placed in a small ring and chilled. The ring is then immersed in water or glycerin; a small steel ball is placed on

the asphalt and heat is applied to the liquid. As the temperature is raised, a point where bitumen softens and the ball sinks to the bottom of container is the temperature corresponding to softening point. At softening point temperature, bitumen is observed to have penetration value of approximately 800 or an absolute viscosity of 13,000poises.

The softening point temperature combined with the penetration of bitumen can be used to determine the temperature susceptibility and penetration index; both are indicators of temperature responses of bitumen (Yousefi *et al.*, 2000).

Temperature susceptibility of bitumen is often described by the expression (Whiteoak, 1990):

$$M = [\log \text{pen at } 25^{\circ}\text{c}] - \log 800 / [25 - \text{softening point temperature}] \quad (2.10)$$

Penetration Index (PI) is expressed as:

$$PI = \frac{20 - 500M}{1 + 50M} \quad (2.11)$$

The value of PI for normal bitumen is usually between +2 and -2. A PI of more than +2 means bitumen is of low temperature susceptibility, while bitumen having PI of less than -2 are of excessively high temperature susceptibility.

#### **2.10.6 Flash and fire point (ASTM D92:2001)**

The flash point of bituminous product is the temperature at which, during heating, its vapor will temporarily ignite, when a small flame is brought into contact with them (Zhang and Yu, 2009). While not specified, the fire point is the higher temperature at which the bitumen will support combustion. Flash and fire point is safety related which implies that suitable caution should be taken to eliminate fire hazards during heating and manipulation of bitumen.

### 2.10.7 Solubility test (ASTM D4-86:2010)

It is desirable that bitumen contain a negligible amount of salt, dirt, carbon and minerals. Such matters are insoluble in solvents like carbon disulphide, carbon tetrachloride and trichloroethylene (Patekar and Randive, 2014). The test is performed by simply dissolving a measure portion of the bitumen in the solvent and then filtered. The solution is dried and weighed to determine the amount of impurity present. The minimum proportion of bitumen soluble in carbon disulphide is 99%.

### 2.10.8 Specific gravity test (ASTM D70:2003)

This is the ratio of the mass of a given volume of bitumen to the same of an equal volume of water (Zumrawi, 2013). It is usually determined either by using pycnometer or by preparing a cube shape specimen of solid state and weighing in air and water. Generally, the specific gravity of pure bitumen is in the range of 0.97 to 1.02, but that of cutback may be lower depending on the type. Tar, one of the products of destructive distillation of coal, has specific gravity range of 1.10 to 1.25. Table 2.7 summarises consistency, purity and safety requirements of both 50/60 and 85/100 penetration grades bitumen.

**Table 2.7: Requirements for 60/70 and 85/100 Penetration Grades Bitumen**

Test	60/70		85/100	
	Min	Max	Min	Max
Penetration at 25 °C, 0.1mm	60	70	85	100
Flash point (Cleveland open cup), °C	232	-	232	-
Ductility at 25°C, cm	100	-	100	-
Solubility in trichloroethylene,%	99	-	99	-
Retained penetration after TFOT,%	52	-	47	-
Ductility at 25°C after TFOT, cm	50	-	75	-

*Source: ASTM D 946 (1994)*

### **2.10.9 Asphalt mix design bitumen content**

According to Overseas Road Note 19 (2002), the aggregate samples gradations are used to prepare triplicate asphalt specimens cubes at the estimated optimum bitumen content and at two increments of 0.5 per cent above and below this optimum. Though, the minimum bitumen content is usually 4.5%, the expected Design Bitumen Content (DBC) is estimated from the following formula (Asphalt Institute, 1994):

$$DBC = 0.035a + 0.04b + Kc + F \quad (2.12)$$

where DBC = approximate Design Bitumen Content (per cent by total weight of mix)

a = percent of mineral aggregate retained on the 2.36mm sieve

b = percent of aggregate passing 2.36mm sieve and retained on 0.075mm sieve

c = per cent of mineral aggregate passing the 0.075mm sieve

K = 0.15 for 11-15% passing the 0.075mm sieve; 0.18 for 6-10% passing the 0.075mm sieve; 0.20 for 5% or less passing the 0.075mm sieve;

F = 0-2%; based on absorption of bitumen. In the absence of other data, a value of 0.7 is suggested.

### **2.11 Cement Filler**

Filler plays an important role on voids content, strength and the stiffness of the bitumen-fines matrix. The classical fillers additives recommended are Portland cement complying with EN 197-1(2001), or hydrated lime complying with BS 890 (1995). At least 75% shall pass the 75µm BS sieve. Others are fly ash, stone dust; industrial and agricultural waste ashes with variable performances on the desired pavement qualities. A mineral filler proportion of 2 to 6% is usually used in dense-grade HMA mixtures depending on mix design.

## 2.12 Aggregate Proportioning and Blending

Proportioning of aggregates can be done using any of purely trial and error procedures, graphical methods, empirical or fineness modulus method and analytical/mathematical methods. Element of trial and error may be required if any of the methods does not meet the demand of the design (Adedimila and Olutaiwo, 2008).

Bailey Method (Vavrik *et al.*, 2001) for instance is empirical as it depends on Nominal Maximum Size of Aggregate (NMSA) and aggregate packing. Coarse and fine aggregates and fillers depend upon the nominal maximum size of aggregate. The sieve size that separates the coarse and fine aggregates is known as Primary Control Sieve (PCS) which is defined by (Vavrik *et al.*, 2001):

$$PCS = NMPS * 0.22 \quad (2.13)$$

where packing constant is 0.22

It means that when NMPS is 25mm, PCS is  $25 * 0.22$  that is, 5.5. Since the nearest sieve to 5.5mm is equal to 4.75mm, then, PCS for NMPS of 25mm is 4.75mm sieve and coarse aggregate is equivalent to particles retained on 4.75mm sieve size.

The Secondary Control Sieve (SCS) is the region of fine aggregate and is represented by the materials retained on sieve No. 200 (0.075mm) while the materials passing sieve no. 200 (0.075mm) is the Tertiary Control Sieve (TCS) (Vavrik *et al.*, 2001):

$$SCS = PCS * 0.22 \quad (2.14)$$

$$TCS = SCS * 0.22 \quad (2.15)$$

## 2.13 Marshall Test Evaluations

Marshall Mix design, a standard laboratory method, gained worldwide adoption for determining and reporting the strength and flow characteristics of bituminous paving mixes

because of its simplicity and lower cost. The method could be used to determine the mix Optimum Binder Content (OBC) and to evaluate various Marshall parameters such as Marshall stability, flow value, unit weight, air voids etc. Specimen may be prepared according to the method described in ASTM D 1559 (2004) and the test criteria of Asphalt Institute (1997) are in Tables 2.8 and 2.9.

**Table 2.8: Marshall Mix Design Criteria**

Mix criteria	Light traffic		Medium traffic		Heavy traffic	
	Surface and Base		Surface and Base		Surface and Base	
	EAL: $\square 10^4$		EAL: $10^4$ to $10^6$		EAL: $\square 10^6$	
	Min	Max	Min	Max	Min	Max
Compaction, No. of blows on each end of specimen	35		50		75	
Stability (N)	3333	-	5333	-	8000	-
Flow (0.25mm)	8	18	8	16	8	14
Percent air voids (VIM)	3	5	3	5	3	5
Percent Voids Filled with Bitumen (VFB)	70	80	65	78	65	75

*Source: Asphalt Institute (1997)*

**Table 2.9: Marshall Mix Design Criteria on VMA**

Nominal Maximum Aggregate Size	Minimum Required VMA,%		
	Design Air Voids,%		
	3.0	4.0	5.0
No. 8 (2.36 mm)	19.0	20.0	21.0
No. 4 (4.75 mm)	16.0	17.0	18.0
3/8 in. (9.5 mm)	14.0	15.0	16.0
1/2 in. (12.5 mm)	13.0	14.0	15.0
3/4 in. (19.0 mm)	12.0	13.0	14.0
1 in. (25.0 mm)	11.0	12.0	13.0
1.5 in. (37.5 mm)	10.0	11.0	12.0
2 in. (50 mm)	9.5	10.5	11.5
2.5 in. (63 mm)	9.0	10.0	11.0

*Source: Asphalt Institute (1997)*

### **2.13.1 Mix design by Marshall method**

Bruce Marshall, formerly a bituminous Engineer with the Mississippi State Highway Department first conceived the method and was subsequently improved by American Army Corps of Engineers to evolve the mix design criteria (Manders, 2011). This method has remained the method of choice because of simplicity, lower cost and time saving nature of the method in many parts of the world. Also, the method may be used for both laboratory design and field control of asphalt paving mixtures. Marshall method does not require climatic data which is vital to Superpave method (Hudson *et al.* 2002).

Asphalt Institute (1997) outlined a specimen of 1200g total weight mixture of aggregates, filler and bitumen; compacted in a 101.6-mm (4-in.) diameter and approximately 63.5 mm (2.5 in.) in height cylindrical mould by a Marshall compaction hammer, which is 6.5 kg (10 lb) in weight and dropped from a height of 457 mm (18 in.) for a specified number of blows per side of the specimen. The bitumen is equally preheated to a temperature of 121-145°C and mixed with the aggregate fraction which is heated to a temperature of 154 to 160°C. Compaction of the mixtures is usually done at a temperature when the bitumen kinematic viscosity is 280±20 centistokes. The standard Marshall specimen of 101.6-mm (4-in.) diameter and approximately 63.5±3 mm (2.5 ±0.12in.) in height may not necessarily be exact in some cases. It is necessary to correct the error or variation by multiplying the Marshall stability by the appropriate correction factor in Table 2.10. The Marshall Quotient (MQ) which is otherwise known as rigidity ratio is the ratio of stability to flow and average height of specimen and the value is a measure of stiffness of bituminous mixture.

Three to five specimens are usually prepared for trial bitumen content. The specimens are tested for bulk specific gravity in accordance with ASTM D2726 (2010). The specimens

are kept immersed in water in a thermostatically controlled water bath at 60°C for 30 to 40 minutes and then transferred within 30seconds to the Marshall Test head and tested for both Marshall stability and flow in accordance with ASTM D1559 (2004).

Stability is the maximum load in kg or kN before failure. Dial gauge reading is usually interpolated between the Marshall Machine calibration marks to obtain Marshall stability. The configuration of the Marshall stability is close to that of the indirect tensile strength, but for the confinement of the specimen imposed by the Marshall testing head. Thus, the Marshall stability is related to the tensile strength of the asphalt mixture.

Flow Value which is the deformation of the specimen in 0.25mm up to the maximum load. The Marshall flow can provide some indication of the resistance of the asphalt mixture to plastic deformation. Mixtures with low flow numbers are stiff and are more resistant to rutting than those with high flow numbers. These mixtures usually fail due to cracking without marginal stress resilience, a moderate combination of both stiffness and flow produces pavement that are stress and strain resistant and may not easily fail due to fatigue rutting, creep and cracking.

**Table 2.10: Marshall Stability Correction Factors**

<b>Volume of Specimen (cm<sup>3</sup>)</b>	<b>Thickness of Specimen (mm)</b>	<b>Correction factors</b>
354-367	44.4	1.92
368-379	46.0	1.79
380-392	47.6	1.67
393-405	49.2	1.56
406-420	50.8	1.47
421-431	52.4	1.39
432-443	54.0	1.32
444-456	55.6	1.25
457-470	57.2	1.19
471-482	58.7	1.14
483-495	60.3	1.09
496-508	61.9	1.04
509-522	63.5	1.00
523-535	64.0	0.96
536-546	65.1	0.93
547-559	66.7	0.89
560-573	68.3	0.86
574-585	71.4	0.83
586-598	73.0	0.81
599-610	74.6	0.78
611-625	76.2	0.76

*Source: Asphalt Institute (1993)*

### **2.13.2 Marshall Specimen Compaction**

According to the Asphalt Institute (1997), specimen compaction is a function of design traffic category. Application of 35, 50 and 75 blows respectively on each side of specimen depicts light ( $\square 10^4$  EAL), medium ( $10^4$  to  $10^6$  EAL) and heavy traffic ( $\square 10^6$  EAL) categories. VIM in the specimen correlates with the degree of compaction. Though, a VIM range of 3 to 5% which supports use of additives is suggested by AASHTO T312 (2005), a moderate VIM of 4% shows that excessive cracking supported by upper limit (5% VIM) and plastic flow and bleeding supported by the lower limit (3% VIM) can be mitigated.

Overseas Road Note 19 TRRL (2002) recommends 5% VIM for 75 blows if traffic is in excess of  $5 \times 10^6$  EAL.

### 2.13.3 Marshall Specimen Analysis and Computations

The Marshall mix design method as recommended by the Asphalt Institute uses five mix design criteria - a minimum Marshall stability, moderate flow, a range of acceptable air Voids in the Mix (VIM), percent Voids Filled with Bitumen (VFB) and a minimum amount of Void in Mineral Aggregates (VMA). The five criteria and bulk density of the mix (CDM) are plotted against bitumen content to facilitate the selection of optimum asphalt content. From the plot of VIM against bitumen content, a 4% VIM is selected and traced through VFB, VMA and CDM. The percentage Bitumen Contents (BC) for Marshall stability, Marshall flow, VMA and VFB based on this 4% VIM are estimated and an average is taken for optimum BC.

#### (A) Bulk specific gravity of Total Aggregates

Bulk Specific Gravity ( $G_{sb}$ ) of the total aggregates used - individual coarse aggregate fractions, the fine aggregate and mineral filler fractions is computed in accordance with the test procedure described in ASTM D2726 (2010). It is estimated using:

$$G_{sb} = \frac{P_c + P_f + P_{mf}}{\frac{P_c}{G_c} + \frac{P_f}{G_f} + \frac{P_{mf}}{G_{mf}}} \quad (2.16)$$

Where  $P_c$ ,  $P_f$  and  $P_{mf}$  are individual percentages by weight of coarse aggregate fractions, the fine aggregate and mineral filler fraction respectively.

$G_c$ ,  $G_f$  and  $G_{mf}$  are bulk specific gravities of individual coarse aggregate fractions, the fine aggregate and mineral filler fraction respectively.

### **(B) Maximum specific gravity of loose paving mixture ( $G_{mm}$ )**

The maximum specific gravity of loose paving mixture,  $G_{mm}$ , could be carried out following the recommendation of ASTM D2041 (2011).

$$G_{mm} = \frac{A}{B + D - E} \quad (2.17)$$

Where A= Mass of dry sample in air

B = Final surface dried mass of sample

D = Mass of flask filled with water at 25°C

E = Mass of flask filled with water and sample at 25°C

### **(C) Void in the Mineral Aggregates (VMA)**

It is the volume of air between the coated aggregate particles and the volume of effective bitumen and is usually expressed as per cent by weight of total mix using ASTM D2041 (2011):

$$VMA = 100 - \frac{G_{mb}P_s}{G_{sb}} \quad (2.18)$$

where  $P_s$  is the aggregate content, percent by total weight of mix

### **(D) Percent Air Voids in a Compacted Mix or Voids in the Mix (VIM)**

The percent air voids or Voids in the Mix (VIM) is the volume of air between the coated mineral aggregate particles. It is tested in accordance with the ASTM D2041 (2011) and relevant parameters for computation are the maximum specific gravity ( $G_{mm}$ ) and the bulk specific gravity of the mixture ( $G_{mb}$ ). Its estimate is as follows:

$$VIM = 100 * \left( \frac{G_{mm} - G_{mb}}{G_{mm}} \right) \quad (2.19)$$

$G_{mm}$  is the maximum specific gravity of the asphalt mixture

$G_{mb}$  is the bulk specific gravity of the mixture

### **(E) Voids filled with bitumen (VFB) in a compacted mix**

The percent voids filled with asphalt (VFA) is the percentage of VMA that is filled with bitumen and can be computed as follows:

$$VFB = 100 \left( \frac{VMA - VIM}{VMA} \right) \quad (2.20)$$

### **(F) Bulk Specific Gravity of Marshall Specimens ( $G_{mb}$ )**

The bulk specific gravity of saturated, but surface dried specimens could be conducted according to ASTM D2726 (2010). Specimens are usually weighed in air, and then weighed in water immersed for four (4) minutes and finally weighed saturated surface-dried after immersion.

Bulk specific gravity,  $G_{mb}$ , is given by

$$G_{mb} = \frac{A}{(B - C)} \quad (2.21)$$

A = Mass of dry sample in air, g,

B = Mass of saturated surface-dry sample in air g,

C = Mass of sample in water, g,

## **2.14 Simple Performance Test of Field Conditions**

Conventional asphalt pavements lack the mechanical strength, service requirements and longevity to withstand heavy traffic loading, varying regimes of temperature loading and distresses induced climatic and environmental conditions (Epps, *et al.*, 2000). It is important to assess the performance of pavement modifiers or reinforcing materials to withstand mechanical failures. Some of the prominent SPT are permanent deformation,

fatigue cracking, thermal cracking, moisture susceptibility and friction properties (Brown *et al.*, 2001).

Asphalt used as flexible pavement surfacing materials is susceptible to temperature and loading variation because of its viscoelastic and possibly plastic properties (Rais, *et al.*, 2013). Mechanical properties are used to characterize hot mix asphalt (HMA) in flexible pavement systems. Simple performance tests (SPT) have been recommended by National Cooperative Highway Research Program (NCHRP 9-19:2003) to evaluate the properties and parameters that measure the performance and durability (Bonaquist, *et al.*, 2003).

SPT are used to simulate actual field performance using laboratory mix design as information provided by Hveem (stabilometer) and Marshall (stability and flow) are not adequate to help understand the behavior of HMA owing to poor correlation, between stability and flow otherwise called Marshall Quotient (Whiteoak and Read, 2003). Since none of the methods is capable of predicting rutting in HMA under traffic service (Abdelhaq, 2015), the effort in performance testing of HMA is to evaluate physical tests that can satisfactorily characterize key performance parameters and how these parameters change throughout the life of a pavement. NCHRP 1-37a (2004), showed that the Simple Performance Tests (SPT) are accurate and reliable and could measure mixture response or characteristic that are correlated with the occurrence of pavement distress by traffic and climatic conditions (Witczak, *et al.*, 2002). Other performance tests are mix stiffness or dynamic modulus, resilient modulus, creep, wheel tracking and shear tests (ORN19, 2002).

#### **2.14.1 Permanent deformation (rutting and creep)**

Some of the tests usually used to predict HMA pavement deformation (rutting and creep) can be broadly categorized (Roberts *et al.*, 1996) as follows:

### **A. Static creep tests**

Static load is applied to a sample and the recovery as the load is removed, is determined.

The tests measure a specimen's permanent deformation; test results generally may not correlate well to actual in-service rutting experience but a large amount of permanent deformation would correlate to higher rutting potential (Brown *et al.*, 2001). Various types of static creep tests for evaluating permanent deformation are:

- (i) **Unconfined static creep tests:** The most popular static creep test, inexpensive and relatively easy to conduct. It is otherwise called simple creep test or uni-axial creep test. The test consists of a static axial stress of 100 kPa (14.5 psi) being applied to a specimen lasting a duration of 1 hour at a temperature of 40°C (104°F), which guarantees that the specimen would not fail prematurely. The applied pressure does not usually exceed 206.9 kPa (Brown *et al.*, 2001), but pavements field tyre pressures are up to 828 kPa (120 psi) and temperatures in excess of 60°C (140°F).
- (ii) **Confined static creep tests:** It is also known as triaxial creep test and the procedure is similar to unconfined static creep test but for use of confining pressure of about 138 kPa (20 psi). The pressure allows test conditions to be more closely simulative of field situation than unconfined static creep tests (Roberts *et al.*, 1996).
- (iii) **Diametral Static Creep Test:** It uses a typical HMA test specimen but loaded along its diametral plane. An example is AASHTO TP 9 (2002) usually used to evaluate creep compliance and strength of HMA using the indirect tensile test device.

### 2.14.2 Repeated load tests

The test is carried out by applying a repeated load cycles (usually in excess of 1,000) with many repetitions at a constant frequency to a test specimen. The specimen's recoverable strain and permanent deformation are determined. The specimen's resilient modulus can be calculated from horizontal deformation and an assumed Poisson's ratio. Cumulative permanent deformation as a function of the number of load cycles is recorded. This can be correlated to creep and rutting potentials. The tests can be run at varying temperatures and different loads. The load varies is applied in an alternation of short pulse to be followed by a rest period. The results usually correlate with in-service conditions than static creep test results (Brown *et al.*, 2001).

Some standard repeated load tests are AASHTO TP 7 (1994) which determines the permanent deformation and fatigue cracking characteristics of HMA using the Superpave Shear Tester (SST). Others are AASHTO TP 31 (1996) used for evaluating Resilient Modulus of Bituminous Mixtures by Indirect Tension and ASTM D4123 (2009) for Indirect Tension Test determining Resilient Modulus of Bituminous Mixtures. The methods are:

- (a) **Unconfined Repeated Load Test:** It is easier to run than the confined test because it does not involve any confining pressure or associated equipment. However, the allowable test loads are significantly lower than field experience.
- (b) **Confined Repeated Load Test:** This test is more complex than the former due to the required confining pressure and equipment required. The pressure allows test loads to be applied more accurately to reflect field situation experienced by in-place pavements.

(c) **Diametral Repeated Load Test:** The loading pattern is along its diametral plane.

According to Brown *et al.*, (2001), two critical draw-backs hinders its ability to determine permanent deformation characteristics:

- (i) The ideal uniform stress is attainable and its specimen shape dependency. Higher temperature and load causes permanent deformation that changes geometry of shape of specimen and could drastically affect both the measurements and state of stress test.
- (ii) Tensional stress along vertical diameter is uniform, others are not uniform.

(d) **Shear Repeated Load Test:** Superpave developed Superpave shear tester (SST) which can evaluate in shear the repeated load test. The test otherwise called repeated shear at constant height (RSCH) applies a repeated haversine (inverted cosine offset by half its amplitude). A continuous haversine wave resembles a sine wave whose negative peak lies at zero. An axially loaded specimen produces shear stress which records axial shear deformation including axial and shear load to high degree of viability (Brown *et al.*, 2001).

### 2.14.3 Dynamic modulus test

This test is carried out by applying load sinusoidally to specimen over a range of different temperature and different frequencies (Shenoy, 2002). Applying a repeated load at varying frequencies to a test specimen over a relatively short period of time, specimen's recoverable strain and permanent deformation were determined.

The repeated loads are axial cyclic load of fixed magnitude and cycle duration and are carried out at different temperatures and three different frequencies of loading (usually 1, 4 and 16 Hz) to the specimen. The applied load varies and its application is usually in a

haversine wave. Some dynamic modulus tests are also able to measure the lag between the peak applied stress and the peak resultant strain, which provides insight into a material's viscous properties (Brown *et al.*, 2001). Test results correlate reasonably well with in-service pavement rutting measurements but the test is somewhat involved and difficult to run.

The dynamic modulus test additionally determines lag between peak stress and peak recoverable strain called phase angle ( $\varphi$ ). The viscous properties of the material called complex modulus,  $E^*$ , is aggregation of the storage or elastic modulus component and the loss or viscous modulus. Figure 2.14 is a depiction of typical dynamic modulus. For visco-elastic tendencies of materials, according to Witczak *et al.*, (2002), "complex modulus" ( $E^*$ ) is related by:

$$E^* = |E^*| \cos \varphi + i|E^*| \sin \varphi \quad (2.22)$$

where:  $E^*$  = complex modulus

$|E^*|$  = dynamic modulus

$\varphi$  = phase angle – the angle by which  $\varepsilon_0$  lags behind  $\sigma_0$ . For a pure elastic material,  $\varphi = 0$ , and the complex modulus ( $E^*$ ) is equal to the absolute value, or dynamic modulus. For pure viscous materials,  $\varphi = 90^\circ$ .

$i$  = imaginary number.

The dynamic modulus which is the absolute value of the complex modulus,  $|E^*|$ , shown in Figure 2.1 is calculated by the relationship according to (Witczak *et al.*, 2002) as follows:

$$|E^*| = \delta_0 / \varepsilon_0 \quad (2.22)$$

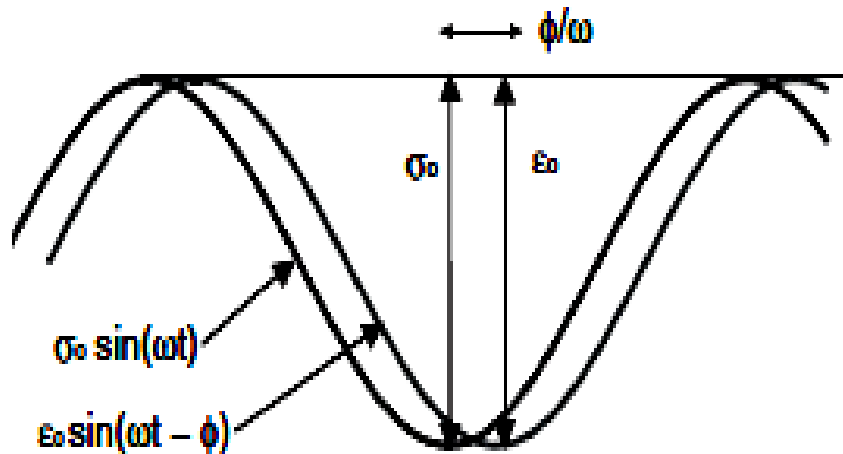
Where:  $|E^*|$  = dynamic modulus

$\delta_0$  = peak stress amplitude (applied load / sample cross sectional area)

$\epsilon_0$  = peak amplitude of recoverable axial strain =  $\Delta L/L$ . It is either measured directly with strain gauges or calculated from displacements measured with linear variable displacement transducers (LVDTs).

$\Delta L$  = the recoverable portion of the change in sample length due to the applied load

$L$  = original gauge length over which the sample deformation is measured



**Figure 2.14: Dynamic (complex) modulus test**  
*Source: Rowe et al. (2009)*

The various types of dynamic modulus test are unconfined, confined and shear dynamic modulus tests. Recommendations for carrying out dynamic modulus test by NCHRP 1-37a: (2004) (Draft Test Method for Dynamic Modulus Test) in simple performance test (SPT) are frequencies of 25, 10, 5, 1, 0.5, and 0.1Hz while temperature range are -10, 4, 20, 38 and 54.4<sup>0</sup>C.

The test is conducted in the order of lowest to highest temperature and from highest to lowest frequency of loading at each temperature to ensure that specimen damage is assuaged. To truly assess interdependency between physical quantities and change of viscoelastic properties of asphalt, time – temperature superposition (TTS) is necessary to fit

a master curve (Soleimani, 2009). A reference frequency or time at which data is taken is chosen and data at other frequencies or time are applied with shift factor to fit a smooth curve called Sigmoidal curve fitting (Pellinen, 2002). The Sigmoidal curve allows for interpolation of dynamic modulus and phase angle at an expanded frequencies and temperatures beyond the collected data sets (Rowe and Sharrock, 2011). This relationship is given by:

$$\log(|E^*|) = \delta + \frac{\alpha}{(1 + e^{(\beta - \gamma \log f_r)})} \quad (2.23)$$

Alternatively,

$$\log(|E^*|) = \delta + \frac{\alpha}{(1 + e^{(\beta + \gamma \log t_r)})} \quad (2.24)$$

Where:  $|E^*|$  = dynamic modulus or stiffness (Pa or Psi)

$f_r$  = Reduced frequency (Hz)

$t_r$  = reduced loading time (s)

$\delta$  = Minimum modulus value

$\alpha$  = Span of modulus value

$\beta, \gamma$  = Shape parameters

According to Witczak (2005), instead of laboratory dynamic test result for Mechanistic-Empirical (M-E) model for predicting the test, if all inputs of data need is aggregated the dynamic modulus is given:

$$\begin{aligned} \log|E^*| = & -1.249937 + 0.02932P_{200} - 0.001767(P_{200})^2 - 0.002841P_4 - 0.058097V_a \\ & - 0.8802208 \left( \frac{V_{eff}}{V_{eff} + V_a} \right) \\ & + \frac{(3.871977 - 0.0021P_4 + 0.003958P_{38} - 0.000017(P_{38})^2 + 0.005470P_{34})}{1 + e^{(-0.603313 - 0.313351 \log(f) - 0.393532 \log(\eta))}} \end{aligned} \quad (2.25)$$

Where  $|E^*|$  = Dynamic Modulus, psi

$\eta$  = Bitumen viscosity at the temperature of interest,  $10^6$  poise

$f$  = Loading frequency, Hz

$V_a$  = Air Voids Content, %

$V_{eff}$  = Effective bitumen content, % by volume

$P_{34}$  = Cumulative % retained on the 3/4-in (19mm) sieve

$P_{38}$  = Cumulative % retained on the 3/8-in (9.5mm) sieve

$P_4$  = Cumulative % retained on the #4 (4.76mm) sieve

$P_{200}$  = % passing the #200 (0.075mm) sieve

Witczak (2005) compared Equations 2.24 and 2.25 and concluded on the predictive equation having:

$$\log(|E^*|) = \delta + \frac{\alpha}{(1 + e^{(\beta + \gamma \log t_r)})} \quad (2.26)$$

Where:

$|E^*|$  = dynamic modulus, 105 psi

$\delta$  = minimum value of  $E^*$

$\delta + \alpha$  = maximum value of  $E^*$

$$\delta = -1.249937 + 0.02923P_{200} - 0.001767(P_{200})^2 - 0.002841P_4 - 0.058097V_a - 0.802208 \left( \frac{V_{beff}}{V_{beff} + V_a} \right)$$

$$\beta = -0.603313 - 0.393532 \log \eta_{Tr}$$

$$\gamma = 0.313351$$

$\eta_{Tr}$  = binder RTFOT viscosity at the reference temperature,  $10^6$  Poise

$V_a$  = air void content, %

$V_{\text{beff}}$  = effective binder content, % by volume

The master curve shift factor is a time-temperature superposition and could be adapted from Arrhenius shift factor law given by:

$$\log[\alpha(T)] = \log \frac{\Delta E_a}{2.303R} \left( \frac{1}{T} - \frac{1}{T_r} \right) = C \left( \frac{1}{T} - \frac{1}{T_r} \right) \quad (2.27)$$

Where  $\alpha(T)$  = Shift factor at temperature, T

R = Ideal gas constant = 8.314J/mol.K

$T_r$  = Reference temperature in Kelvin, usually taken at 21<sup>0</sup>C or 294.15K (Dougan *et al.*, 2003)

C = constant whose values were reported by different researchers to be 10920, 13060 and 7680K (Medani and Huurman, 2003)

$\Delta E_a$  = Activation energy.

$\Delta E_a$  describes the minimum energy required to enhance intermolecular movement and is usually treated as a parametric fitting. The value of  $\Delta E_a$  below reference temperature ( $T_r$ ) may be taken as 261kJ/mol (Abdelhaq, 2015).

Finally, the reduced frequency,  $f_r$ , is given by (Witczak, 2005):

$$f_r = f * \alpha_T \quad (2.28)$$

$$\log f_r = \log f + C \left( \frac{1}{T} - \frac{1}{T_r} \right) \quad (2.29)$$

Where f = reference temperature's loading frequency.

## **2.15 Empirical Tests**

Traditional Hveem and Marshall mix design tests are empirical tests used to quantify the potentiality of HMA for permanent deformation. While Hveem evaluates stability and cohesion, Marshall determines stability and flow, all of which impart upon deformation. The test results can correlate well with in-service pavement rutting measurements but these tests do not measure other fundamental material parameters (ORN 19, 2002).

## **2.16 Simulative Tests (Laboratory Wheel-Tracking Devices)**

Permanent deformation in asphalt layers manifests on pavement surface called rutting represents significant distress of asphalt pavements depending on the level and effects on safety of traffic, comfortability of ride, and overall pavement service life-cycle (Miljković and Radenberg, 2011). NCHRP Report 508 (2003) and as noted by Cooley *et al.*, (2000) recommended a maximum rut depth of 8mm at 8, 000 cycles of load repetitions while AASHTO T324 (2004) recommended value of 12.5mm at 10,000 cycles of load repetitions. The test results do correlate well with in-service pavement rutting measurements and has been used to successfully predict HMA permanent deformation characteristics and moisture susceptibility of asphalt (Cooley *et al.*, 2000). The methods usually adopted in evaluating asphalt mix rutting resistance are:

- (i) The Asphalt Mixture Performance Tester (AMPT), formerly known as the Simple Performance Tester (SPT).
- (ii) The Hamburg Wheel Tracking Device (HWTD).
- (iii) The Asphalt Pavement Analyzer (APA), formerly known as the Georgia Loaded-Wheel Tester.

## **2.17 Fatigue Life of Pavement**

It is an important performance based parameter that depends largely on structural design and subgrade support other than mix design alone. Evaluation of fatigue properties in HMA are relevant because one of the principal modes of HMA pavement distress is related fatigue cracking. Therefore, an accurate simulation of fatigue in HMA would be useful in predicting overall pavement service life.

### **2.17.1 Fatigue testing using flexural test method**

HMA fatigue properties using flexural test relates load cycles to failure to the magnitude of applied stress or strain. The test evaluates the fatigue life of a small HMA beam specimen measuring 380mm long x 50mm thick x 63mm wide, being subjected to repeated flexural bending to failure. The beam is either sawn from laboratory or field compacted HMA.

An example of standard fatigue test is AASHTO TP 8 which determines the fatigue life of compacted HMA subjected to repeated flexural bending.

## **2.18 Tensile strength**

Tensile strength could be related to low temperature cracking of HMA and a good cracking potential evaluator. The result of high tensile strain until failure shows that a particular HMA can tolerate higher strains before failing. This means that higher propensity to resist cracking than HMA with low tensile strain to failure. Additionally, determining tensile strength before and after water conditioning could elucidate on moisture susceptibility of HMA (Solaimanian and Kennedy, 2000). If the result of water-conditioned tensile strength is higher than the dry tensile strength, then it can be assumed reasonably that the HMA is

moisture resistant (Stuart and Youtcheff, 2001). Pavement interactive outlined two tests generally used to determine tensile strength of HMA - Indirect tensile test and Thermal cracking test (Over Sea Road Note 19, 2002).

**(a) Indirect tensile test**

The test employs the same testing equipment as the diametral repeated load test and applies a constant rate of vertical load deformation till failure occurs. It is similar to the splitting tension test of Portland Cement Concrete (PCC). AASHTO TP 9 (1996) used for determining the creep compliance and strength of HMA using the indirect tensile test device is a standard example.

**(b) Thermal cracking test**

The test determines the tensile strength and temperature at fracture of an HMA sample by determining the specimen tensile load cooled at constant rate, while being restrained from contraction. The test is terminated when the sample fails by cracking. AASHTO TP 10 (1993) method for thermal stress restrained specimen tensile strength is an example of thermal cracking test.

## **2.19 Moisture Susceptibility**

HMA moisture damage susceptibility can be evaluated through deformation resistance and tensile strength tests. Several tests can determine the comparison between wet and dry sample tests, modified Lottman Test presents the most current and near appropriate test results. The test basically compares dry sample and a sample exposed to water/freezing/thawing using indirect tensile strength test. AASHTO T283-89 (1993) recommends conditioning the specimen to vacuum saturation and an optional freeze cycle and a warm-water cycle before it is tested for indirect tensile strength. Test results are

ratios of average conditioned sample tensile strength to average dry sample tensile to strength as (Epps *et al.*, 2000):

$$TSR = S_2/S_1 \quad (2.30)$$

Where TSR = tensile strength ratio

$S_1$  = average dry sample tensile strength

$S_2$  = average conditioned sample tensile strength

Modified Lottman test recommends a minimum TSR of 0.70 and should be applied to field-produced rather than laboratory-produced samples (Roberts *et al.*, 1996). Other methods are AASHTO TP 4 (2000) (Method for Preparing and Determining the Density of HMA Specimens by Means of the Superpave Gyratory Compactor), AASHTO MP 2 (1999) (Specification for Superpave Volumetric Mix Design) and Hamburg Wheel Tracking Device (HWTDD).

## **2.20 Asphalt Ageing Process and Morphological Changes**

Bitumen ageing causes physical, chemical, structural and rheological change of properties of the material and often times, leads to loss of serviceability and functionality of pavement structures (Mouazen *et al.*, 2013). Asphalt creep is both a simple performance test (SPT) criterion and an ageing process leading to morphological changes and loss of durability (Xu and Sun, 2013). Ageing sensitivity is due to changes in physical quantities or phenomenon which includes loading stress (Du, 2006), temperature (Vargas *et al.*, 2008), frequency of exposure or time (Perez-Lepe *et al.*, 2003; Mouazen *et al.*, 2011), ultra violet irradiation (Mouillet *et al.*, 2008; Wu *et al.*; 2008) and gamma and microwave rays (Valcke, 2009). Also, Robertson (2000) studied negative effects of free acids and monovalent salts on

asphalt. Kanitpong and Bahia (2002) and Lu and Harvey (2005) concluded that the presence of moisture on asphalt reduces both chemical and mechanical bonds in asphalt matrix and could lead to disintegration. Oxidation of asphaltenes and resins alters consistency properties like viscosity, penetration index, and softening point (Mastrofini and Scarsella, 2000) and viscoelastic properties (Mouazen *et al.*, 2011; Doh *et al.*, 2008; Larsen *et al.*, 2009).

Asphalt creep is a gradual, but time, traffic and environment dependent degradation. Polymer admixed bitumen exhibits self healing during shear, produces best rheological properties and ageing resistance (Diego *et al.*, 2009; Kumar and Veeraragavan, 2011). It has been found to effectively increase rutting resistance at high temperatures and fatigue resistance at intermediate temperatures (Nasser *et al.*, 2012).

### **2.20.1 Morphological changes**

Morphology and mechanical properties studies could be used to assess bitumen ageing. Relevant tests are creep test, Scan electron microscopy (SEM), differential scanning calorimetry (DSC), dynamic mechanical spectroscopy (DMS) (Little, 1992) and likely physico-chemical transformation and change of phase as a result of physical and environmental factors may be assessed using thermo-gravimetric analysis (TGA), Differential Thermal Analysis (DTA) and Fourier transform infrared spectroscopy (FTIR) (Yao *et al.*, 2013).

Asphalt ageing induces hardening and increase in viscosity of bitumen which suffers natural degradation by environmental effects (Burnay, 1987). Morphology, shape and property changes shed light on structural and chemical changes in asphalt as a result of factors of ageing, fatigue failure and deformation (Adedeji *et al.*, 1996). In admixed or

modified materials, it is necessary to understand microstructure including the pore sizes, fibre formations and its length distribution as well as physical dispersion of the admixed materials (Yao *et al.*, 2013).

### **2.20.2 Creep Deformation**

Creep is defined as the slow time dependent deformation of a material measured under a constant stress (Xiao *et al.*, 2013). The rate of creep deformation is a function of properties of material, exposure time, exposure temperature and the structural load applied (Turner, 2001).

In asphalt creep test, a fixed shear stress is applied to the sample and the resultant strain is monitored for a predetermined amount of time. This gives an idea of the permanent deformation that the binder will undergo (Ayman and Hassan, 2006). After a predetermined period of time, the stress is removed, and the strain is further monitored. This allows the material to recover for a longer duration of time.

The creep test could be used to evaluate linear viscoelastic behaviour of HMA materials. This phenomenon of increase in deformation under a sustained stress loading is time and temperature dependent. The ratio of instantaneous strain over the applied stress is called creep compliance (Wen and Kim, 2002; Kim, *et al.*, 2002).

The mechanical response of asphalt is viscous at high temperatures and glassy or brittle at low-temperatures (Mouazen *et al.*, 2011). Physical aging can affect the volume, entropy and enthalpy of the aging material (Yut *et al.*, 2014; Qin *et al.*, 2014). In terms of rheological properties, it follows that material becomes stiffer, more elastic, and more brittle during physical ageing (Dongré, 2000; Soenen *et al.*, 2004 and Kriz *et al.*, 2008). Volumetric properties could determine the responses of asphalt to mechanical loading and

climatic durability of the field state conditions (Harman *et al.*, 2002; Hand *et al.*, 2003). The nature and type of response of the mix to volume change determines the type and rate of deformation. The physical mix volumetric properties directly relate to the field performance of asphalt pavements and changes as the loading and environmental conditions prevail (Dongre *et al.*, 2005). Witczak *et al.*, (2002), in NCHRP Report 465 recommended dynamic modulus, repeated load test and static creep test for evaluating mixes susceptible to permanent deformation. Hence, performance of asphalt-based road pavements is strongly associated with the rheology of asphalt (Wang *et al.*, 2014).

It is used to evaluate the relative quality and performance of bituminous mix and a measure of deformation and mechanical failure resilience low temperature or thermal cracking. The test may be done according to any of the standards - BS 598:111 (1995), AASHTO T 322 (2007) or ASTM D 6931 (2007). Shear deformations occurring as a result of high shear stresses in the top portion of a bituminous layer may be considered to be the primary cause of rutting in flexible pavements, but a gradual creep is observed at low stress level (O'flaherty, 2007). The creep test may be determined either in the static or dynamic mode of loading.

In static creep, a loading plate is usually mounted on top of each test specimen and the linearly variable differential transducers (LVDT's) are mounted against the loading plate at points intervals equal to 1/3 the circumference of the plate for recording values and averaging them. At low stress levels, a correction factor from repeated creep test multiplied by creep test strain gives the expected rut depth (Static loading and unloading may be applied to each specimen in all duration of two hours (one hour loading and one hour unloading) for either 101.6mm (4 inches) or 152.4mm (6 inches) diameter. The applied pressure does not usually exceed 206.9 kPa (Brown *et al.*, 2001), but Serkan *et al.* (2009)

argued that these stresses are not adequate to simulate tertiary creep. Serkan *et al.* (2009) suggested a stress value of 500kPa, while Little et al (1993) adopted 414kPa for standard stress values of tyre pressures of a loaded truck.

Creep tests may be performed at temperatures ranging from 40 °C to 60°C. While conducting the test, axial deformation is continuously observed with respect to time. If the initial height of the specimen and the axial strain, are known; the stiffness modulus  $S_{mix}$ , can be determined at any loading time using Equations (3.31) to (3.32) below (Al-Qadi *et al.*, 2014):

$$S_{mix} = \gamma / \varepsilon \quad (3.31)$$

$$\varepsilon = h / H_0 \quad (3.32)$$

Where  $\gamma$  = Applied stress (N/mm<sup>2</sup>)

$\varepsilon$  = Accumulated strain

h = Axial deformation (mm)

$H_0$  = Initial specimen height (mm)

### 2.20.3 Thermal Analysis

TGA is weight loss of a test material as a result of degradation under temperature with time (Flynn, 1988; 1989). Apart from degradation mechanism studies, it is used in prediction measurement of service lifetime of materials. DTA monitors the temperature differences existing between a sample and a reference material as a function of time and/or temperature assuming that both samples and references are subjected to the same environment at a selected heating or cooling rate (Ramachandran *et al.*, 2002). The plot of  $\Delta T$  as a function of temperature is termed a DTA curve and endothermic transitions are plotted downward on the y-axis, while temperature (or time) is plotted on the x-axis.

The minimal and maximal temperatures of accelerated aging are within range of chemical stability where no chemical changes are detected of a material (Budrugaec and Segal, 1998).

At the accelerated temperatures, changes in mechanical properties (elongation at break, traction resistance) and thermal response are exhibited by a first order phase change (melting or softening), followed by exothermic change and thermo-oxidative degradation (Perez-Lepe *et al.*, 2005; Naskar *et al.*, 2010). The oxidative ageing of bitumen gives rise to functional groups including carboxylic acids, ketones, sulfoxides and anhydrides (Lucena *et al.*, 2004).

The range of chemical stability, as well as the temperatures corresponding to the phase transitions can be determined by thermal analysis methods- Thermogravimetric Analysis (TGA), Differential Thermal Gravimetry or Analysis (DTG/DTA) and Differential Scan Calorimetry (DSC) (Fuentes-Auden *et al.*, 2008).

Weight loss from TG/DTA machine recorded by computer is plotted as a function of time for isothermal studies and as a function of temperature for experiments at constant heating rate. According to Ramachandran *et al.* (2002), degradation or service lifetime prediction is calculated by Arrhenius rate equation given by:

$$\frac{dx}{dt} = A * e^{\left(\frac{-E_a}{RT}\right)} * (1 - x)^n \quad (2.33)$$

where  $x$  = degree of conversion

$t$  = time

$dx/dt$  = reaction rate

$n$  = reaction order

$A$  = pre-exponential factor

$Ea$  = activation energy

$R$  = gas constant

$T$  = temperature (K)

The above expression in Equation 2.33 in log form gives:

$$\ln\left[\frac{1}{(1-x)^n}\right] = \left(\frac{-E_a}{R}\right)\left(\frac{1}{T}\right) + \ln A - \ln\left(\frac{dx}{dt}\right) \quad (2.34)$$

For a degree of conversion,  $x_i$ , under temperature,  $T_i$ ,

$$\ln\left[\frac{1}{(1-x_i)^n}\right] = \left(\frac{-E_a}{R}\right)\left(\frac{1}{T_i}\right) + \ln A - \ln\left(\frac{dx_i}{dt}\right) \quad (2.35)$$

Since the reaction rate is constant, then,

$$\ln A - \ln\left(\frac{dx_i}{dt}\right) = \beta \text{ (constant or intercept)} \quad (2.36)$$

Therefore, a plot of the logarithm of the heating rate  $\ln\left[\frac{1}{(1-x_i)^n}\right]$  versus reciprocal temperature  $\left(\frac{1}{T_i}\right)$  gives a straight line with a slope equal to  $\frac{-E_a}{R}$  and an intercept equal to  $\beta$ , assuming a first order reaction. TGA/DTA of materials may be determined under conditions that accelerate its rate and the resulting parameters extrapolate to predict a service lifetime for useful commercial importance.

Following the review of literature, marked interest have developed in pavement construction which requires inclusion of materials as wet process modifications, geo-stabilization and fibre reinforcement. The use of HDPP in HMA could impact outstanding long-term performance benefits and are less expensive compare to many industrial alternatives. The methodology employed with the background from aforementioned theories, tests and specifications were discussed in Chapter Three.

## CHAPTER THREE

### MATERIALS AND METHODS

#### 3.1 Tests on Materials

Tests were conducted on bitumen coarse and fine aggregates and filler to ascertain whether they meet standard specifications.

##### (a) Bitumen

Bitumen 60/70 penetration grade is the most widely used grade for road construction, surface dressing and other industrial purposes because of its superior properties. Grade 60/70 penetration bitumen was obtained from Triacta Construction Company in Kano State, Nigeria. Various tests were carried out on bitumen to assess that the bitumen satisfied the code's requirements before using it. These tests were conducted in accordance with relevant codes. The results are presented in Table 3.1. These show that the bitumen was of grade 60/70 penetration as shown in the Table.

Assessment of bitumen ageing and durability was carried out when it was heated at 163°C for 5hours. The test is otherwise called Thin film Oven Test (TFOT) and the results are shown in Table 3.2.

Other tests were carried out on coarse and fine aggregates and cement fillers as shown in Tables 3.3 to 3.5.

**Table 3.1: Test parameters of Unmodified bitumen with 0% HDPP**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Penetration at 25 °C, 0.1mm	ASTM D5-97	67.7
Penetration Index (PI)	ASTM D5-97	-0.338
Softening point (°C)	ASTM D36-95	50.5
Flash point (Cleveland open cup), °C	ASTM D92-02	295.2
Fire point Cleveland cup), °C	ASTM D92-02	306.5
Ductility at 25°c, cm	ASTM D113-07	122.4
Specific gravity at 25°C, (g/cc)	ASTM D70-03	1.022
Solubility in trichloroethylene,%	ASTM D2042-97	99.02

**Table 3.2: Thin film oven test (TFOT) result of unmodified bitumen (0% HDPP)**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Ductility at 25°c, cm	ASTM D113-07	59.2
Retained penetration (% of original)	ASTM D5-05	78.3
Loss on heating (% by mass)	ASTM D6-95	0.23

**(b) Test on Mineral Aggregates**

Strength characterization, shape, moisture absorption and gravity tests were conducted on aggregates (coarse and fine aggregates) to assess their quality. Table 3.3 shows the result of the tests with their respective codes.

**Table 3.3: Result of Preliminary Test on Aggregate materials**

<b>Test Conducted</b>	<b>Code Used</b>	<b>Test Result</b>
Aggregate Crushing Value (%)	BS 812:112-90	22.8
Aggregate Impact Value (%)	BS 812:111-90	16.3
Aggregate Los Angeles Abrasion Value (%)	ASTM C131-06	18.9
Specific Gravity (Coarse Aggregate) ( $G_c$ ) g/cc	ASTM C127-01	2.70
Aggregate Moisture Absorption (%)	BS 812:105.2-90	1.4
Coarse Aggregate Flakiness Index	BS812:105.1-89	26
Specific Gravity (Fine Aggregate) ( $G_f$ ) g/cc	ASTM C128-15	2.63
Bulk Specific Gravity of Total Aggregate ( $G_{sb}$ ), g/cc	ASTM C127-01	2.71

### (c) Aggregate Particle Size Distribution

Aggregate materials were sampled according to the recommendation of BS EN932-1 (2003) and particle size distribution was conducted according to BS EN 933-1 (2003). The result of PSD for Coarse and fine aggregates and cement filler are shown in Tables 3.4 to 3.5 respectively. The details are in Appendices A1 and A2.

**Table 3.4: Coarse Aggregate Sieve Analysis**

<b>BS Sieve Size</b>	<b>Cumulative Passing (%)</b>
1" (25.4mm)	100.0
¾" (19mm)	95.1
½" (12.7mm)	77.6
3/8" (9mm)	61.0
¼" (6mm)	38.1
No. 4 (4.75mm)	19.5
No.7 (2.36mm)	1.4
No.14 (1.18mm)	-

**Table 3.5: Fine Aggregate Sieve Analysis**

<b>BS Sieve Size</b>	<b>Cumulative Passing (%)</b>
No. 4 (4.75mm)	100
No.7 (2.36mm)	97.9
No.14 (1.18mm)	68.4
No.25 (0.60mm)	42.1
No.52 (0.30mm)	22.1
No.100 (0.15mm)	6.6
No.200 (0.075mm)	3.5
Pan	(0.1loss)

### (d) Consistency Tests on Cement Filler

Tests conducted on cement filler include specific gravity, setting time and soundness. Table 3.6a shows the results of specific gravity, setting time and soundness and Table 3.6b shows the grading of cement fillers and the details is contained in Appendix A3.

**Table 3.6a: Result of Test on Cement Filler**

<b>Test Conducted</b>	<b>Code Used</b>	<b>Result Obtained</b>
Specific gravity	ASTM C188-15	3.15
Initial Setting time (minutes)	BS EN 196 Part 3-05	98
Final Setting time (minutes)	BS EN 196 Part 3-05	230
Soundness (mm)	BS EN 196 Part 3-05	3.5

**Table 3.6b: Filler Sieve Analysis (Cement)**

<b>BS Sieve Size</b>	<b>Cumulative Passing (%)</b>
No.25 (0.60mm)	100
No.52 (0.30mm)	98.1
No.100 (0.15mm)	93.3
No.200 (0.075mm)	80.4
Pan	-

**(e) Proportioning and Blending**

Bailey Method's empirical approach (Vavrik *et al.*, 2001) was used as a guide and a further trial and error refinement to estimate the proportions of Coarse and fine aggregates and filler respectively. The proportions of aggregates and fillers are shown in Table 3.7.

**Table 3.7: Estimate of Aggregates and Filler Proportions**

<b>Barley Parameters</b>	<b>Estimated Values</b>
Nominal maximum Passing size (NMPS)	25.4
Parking constant	0.22
Primary Control Sieve Value (PCS)	5.5 (Sieve 4.75mm)
Secondary Control Sieve Value (SCS)	1.21 (Sieve 1.18mm)
Tertiary Control Sieve Value (TCS)	0.27 (Sieve 0.075mm)
<b>Aggregate Materials</b>	<b>Percent Material</b>
Coarse Aggregate	55.5
Fine Aggregate	38.0
Filler (Cement)	6.5

Based on the estimated proportions of materials - coarse aggregate (55.5%), fine aggregate (38.0%) and filler (6.5%), the mix mineral materials were combined for blending as in Table 3.8 and the summary is in Table 3.9.

**Table 3.8: Mix Material Combining and Blending**

<b>BS Sieve Size</b>	<b>CA= %Retained x (55.5%)</b>	<b>FA= %Retained x (38%)</b>	<b>CF= %Retained x (6.5%)</b>	<b>Sum of Retained (%)</b>	<b>Cum. Retained (%)</b>
P25.4 – R19	4.9*0.555 = 2.7	-	-	2.7	2.7
P19 – R12.7	17.5*0.555 = 9.7	-	-	9.7	12.4
P12.7 – R9.5	16.6*0.555 = 9.2	-	-	9.2	21.6
P9.5 – R6	22.9*0.555 = 12.7	-	-	12.7	34.3
P6 – R4.75	18.6*0.555 = 10.3	-	-	10.3	44.6
P4.75 – R2.36	18.2*0.555 = 10.1	2.1*0.38 = 0.8	-	10.9	55.5
P2.36 – R1.18	1.4*0.555 = 0.8	29.5*0.38 = 11.2	-	12.0	67.5
P1.18 – R0.60	-	26.3*0.38 = 10.0	-	10.0	77.5
P0.60 – R0.3	-	20.0*0.38 = 7.6	1.9*0.065 = 0.1	7.7	85.2
P0.30 – R0.15	-	15.5*0.38 = 5.9	4.8*0.065 = 0.3	6.2	91.4
P0.15 – R0.075	-	3.1*0.38 = 1.2	13.0*0.065 = 0.9	2.1	93.5
P0.075 – Pan	-	3.4*0.38 = 1.3	80.4*0.065 = 5.2	6.5	100

*P= passing sieve designation; R = retained on sieve designation; CA = coarse aggregate; FA = fine aggregate; CF = cement fillers*

**Table 3.9: Combined Material Mix and Range of Specification Requirements**

<b>Sieve (mm)</b>	<b>Size</b>	<b>Percentage Retained</b>	<b>Cumulative Percentage Retained</b>	<b>Cumulative Percentage Passing</b>
25.00		-	-	100
19.00		2.7	2.7	97.3
12.50		9.7	12.4	87.6
9.50		9.2	21.6	78.4
6.30		12.7	34.3	65.7
4.75		10.3	44.6	55.4
2.36		10.9	55.5	44.5
1.18		12.0	67.5	32.5
0.60		10.0	77.5	22.5
0.30		7.7	85.2	14.8
0.15		6.2	91.4	8.6
0.075		2.1	93.5	6.5
Pan		6.5	100	-

**(f) Design of Bitumen Content**

Asphalt Institute (1994) was used to estimate the expected Design Bitumen Content (DBC)

given by earlier Equation 2.12 as:

$$DBC = 0.035a + 0.04b + Kc + F$$

where,

a = % of mineral aggregate retained on the 2.36mm sieve = 55.5%

b = % of aggregate passing 2.36mm sieve and retained on 0.075mm sieve = 38%

c = % of mineral aggregate passing the 0.075mm sieve = 6.5%

K = 0.18 for 6-10% passing the 0.075mm sieve = 0.18

F = 0-2% for absorption of bitumen = 0.7

Therefore,

$$DBC = 0.035 (55.5) + 0.04(38) + 0.18(6.5) + 0.7 = 1 = 5.332$$

$$= 5.5\% \text{ (approximately)}$$

Asphalt Institute (1994) requires two other points at 0.5% above and below the optimum DBC which serves as the optimum. Thus, the ranges of bitumen content to be adopted in the mix are 4.5, 5.0, 5.5, 6.0 and 6.5%.

### **3.2 Test on HDPP Modified Bitumen**

Two mix processes were adopted, wet and dry processes. Following the recommendations of Iwanski *et al.* (2015); Yu *et al.*, (2013), HDPP modified bitumen samples otherwise known as ‘wet process’ were prepared using 0, 0.5, 1.0, 1.5, 2.0, 2.5 and 3.0% HDPP contents respectively by total weight of 1200g standard Marshall specimen weight. HDPP were heated and mixed together with various bitumen contents to temperature of about 170°C according to the recommendations of BS EN12591 (2009) to avoid vaporization and loss of volatiles (Habib *et al.*, 2010). The HDPP modified bitumen samples were tested and used to prepare Marshall samples.

Tests were conducted to determine the consistency and safety of HDPP modified bitumen. The tests conducted were according to the recommendations of the relevant code standards for the 60/70 penetration bitumen used. The various tests conducted are ductility, penetration, softening point, specific gravity, solubility and flash and fire point tests. Also, thin film oven test (TFOT) was included in the characterization of HDPP bitumen to evaluate the short term ageing, which affects the durability of the mix. The results are shown in Tables 3.10 to 3.21.

Secondly, HDPP was chopped to small sizes of about 12mm fibre length and used in the reinforced asphalt mixing called dry process before mixing and blending. The fibre length followed the recommendations of Abhati, *et al.*, (2011) and Tapkin (2006). In this dry

process, the same range of 0.5, 1.0, 1.5, 2.0, 2.5 and 3.0% HDPP contents by total weight of Marshall specimens were used.

**Table 3.10: Test parameters of bitumen modified with 0.5% HDPP**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Penetration at 25 °C, 0.1mm	ASTM D5-97	62.9
Penetration Index (PI)	ASTM D5-97	+0.239
Softening point (°C)	ASTM D36-95	53.6
Flash point (Cleveland open cup), °C	ASTM D92-02	311.4
Fire point Cleveland cup), °C	ASTM D92-02	317.2
Ductility at 25°c, cm	ASTM D113-07	107.2
Specific gravity at 25°C (g/cc)	ASTM D70-03	1.015

**Table 3.11: Test parameters of bitumen modified with 1.0% HDPP**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Penetration at 25 °C, 0.1mm	ASTM D5-97	53.8
Penetration Index (PI)	ASTM D5-97	+0.222
Softening point (°C)	ASTM D36-95	55.3
Flash point (Cleveland open cup), °C	ASTM D92-02	317.0
Fire point Cleveland cup), °C	ASTM D92-02	326.8
Ductility at 25°c, cm	ASTM D113-07	88.6
Specific gravity at 25°C (g/cc)	ASTM D70-03	1.011

**Table 3.12: Test parameters of bitumen modified with 1.5% HDPP**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Penetration at 25 °C, 0.1mm	ASTM D5-97	45.2
Penetration Index (PI)	ASTM D5-97	0.526
Softening point (°C)	ASTM D36-95	58.7
Flash point (Cleveland open cup), °C	ASTM D92-02	322.6
Fire point Cleveland cup), °C	ASTM D92-02	334.7
Ductility at 25°c, cm	ASTM D113-07	72.4
Specific gravity at 25°C (g/cc)	ASTM D70-03	1.008

**Table 3.13: Test parameters of bitumen modified with 2.0% HDPP**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Penetration at 25 °C, 0.1mm	ASTM D5-97	38.5
Penetration Index (PI)	ASTM D5-97	+1.516
Softening point (°C)	ASTM D36-95	66.0
Flash point (Cleveland open cup), °C	ASTM D92-02	330.9
Fire point Cleveland cup), °C	ASTM D92-02	341.5
Ductility at 25°c, cm	ASTM D113-07	54.3
Specific gravity at 25°C (g/cc)	ASTM D70-03	1.005

**Table 3.14: Test parameters of bitumen modified with 2.5% HDPP**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Penetration at 25 °C, 0.1mm	ASTM D5-97	30.9
Penetration Index (PI)	ASTM D5-97	+1.881
Softening point (°C)	ASTM D36-95	71.3
Flash point (Cleveland open cup), °C	ASTM D92-02	335.6
Fire point Cleveland cup), °C	ASTM D92-02	349.6
Ductility at 25°c, cm	ASTM D113-07	42.8
Specific gravity at 25°C (g/cc)	ASTM D70-03	1.003

**Table 3.15: Test parameters of bitumen modified with 3.0% HDPP**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Penetration at 25 °C, 0.1mm	ASTM D5-97	22.5
Penetration Index (PI)	ASTM D5-97	2.146
Softening point (°C)	ASTM D36-95	77.8
Flash point (Cleveland open cup), °C	ASTM D92-02	342.5
Fire point Cleveland cup), °C	ASTM D92-02	358.4
Ductility at 25°c, cm	ASTM D113-07	22.6
Specific gravity at 25°C (g/cc)	ASTM D70-03	1.001

**Table 3.16: Thin film oven test (TFOT) result of bitumen modified with 0.5% HDPP**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Ductility at 25°c, cm	ASTM D113-07	56.4
Retained penetration (% of original)	ASTM D5-97	72.6
Loss on heating (% by mass)	ASTM D6-95	0.25

**Table 3.17: Thin film oven test (TFOT) result of bitumen modified with 1.0% HDPP**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Ductility at 25°c, cm	ASTM D113-07	54.0
Retained penetration (% of original)	ASTM D5-97	65.9
Loss on heating (% by mass)	ASTM D6-95	0.38

**Table 3.18: Thin film oven test (TFOT) result of bitumen modified with 1.5% HDPP**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Ductility at 25°c, cm	ASTM D113-07	52.5
Retained penetration (% of original)	ASTM D5-97	59.4
Loss on heating (% by mass)	ASTM D6-95	0.41

**Table 3.19: Thin film oven test (TFOT) result of bitumen modified with 2.0% HDPP**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Ductility at 25°c, cm	ASTM D113-07	50.5
Retained penetration (% of original)	ASTM D5-97	56.0
Loss on heating (% by mass)	ASTM D6-95	0.42

**Table 3.20: Thin film oven test (TFOT) result of bitumen modified with 2.5% HDPP**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Ductility at 25°c, cm	ASTM D113-07	48.7
Retained penetration (% of original)	ASTM D5-97	50.2
Loss on heating (% by mass)	ASTM D6-95	0.46

**Table 3.21: Thin film oven test (TFOT) result of bitumen modified with 3.0% HDPP**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>
Ductility at 25°c, cm	ASTM D113-07	46.5
Retained penetration (% of original)	ASTM D5-97	44.8
Loss on heating (% by mass)	ASTM D6-95	0.59

### **3.3 Marshall Test Experimental Plan and Specimen Preparation**

All the specimens having standard weight of 1200g were compacted with 75 hammer blows on each side in the mould whose standard dimensions are 101.5mm diameter and 63.5mm height to simulate heavy traffic situation of greater than 10<sup>6</sup> ESALs. The specimens were tested for bulk specific gravity in accordance with ASTM D2726 (2001). The specimens were kept immersed in water in a thermostatically controlled water bath at 60°C for 30 to 40 minutes and then transferred within 30seconds to the Marshall test machine loading head and tested for both Marshall stability and flow in accordance with ASTM D1559 (2001).

Stability values were determined and were corrected by applying corrections ratios (as shown in Table 2.10), volumetric tests and stability and flow tests were determined. The volumetric tests carried out were CDM, VMA, VIM and VFB and for Marshall tests were Stability and Flow.

**(a) Wet Process**

The wet process is the mix prepared from liquid mixtures of bitumen and HDPP. Heating temperature was kept at about 170°C to avoid vaporization of bitumen (EN12591-09 and Habib *et al.*, 2011). HDPP were mixed with bitumen in the range of 0, 0.1, 1.0, 1.5, 2.0, 2.5 and 3.0% respectively, with 0% being the control and the rest percentages are for HDPP contents. Triplicate specimens were prepared for each bitumen contents. Thus, HDPP was mixed with bitumen for asphalt mix preparation and mould into specimens of 1200g weight, 101.5mm diameter and 63.5mm height. The experimental plan is as shown in Table 3.22. MA1 means specimen mix identification A and number 1 and MC1 means mix C number 1 to MG5 meaning mix G, number 5.

**Table 3.22: Marshall test plan for HDPP modified specimens (wet process)**

<b>Specimen Identification</b>	<b>% HDPP as Proportion of Total Mix Weight</b>	<b>Bitumen Content (% of Total Mix Weight)</b>	<b>Number of Samples</b>
MA1	0% HDPP	4.5	3
MA2	0% HDPP	5.0	3
MA3	0% HDPP	5.5	3
MA4	0% HDPP	6.0	3
MA5	0% HDPP	6.5	3
MB1	0.5% HDPP	4.5	3
MB2	0.5% HDPP	5.0	3
MB3	0.5% HDPP	5.5	3
MB4	0.5% HDPP	6.0	3
MB5	0.5% HDPP	6.5	3
MC1	1.0% HDPP	4.5	3
MC2	1.0% HDPP	5.0	3
MC3	1.0% HDPP	5.5	3
MC4	1.0% HDPP	6.0	3
MC5	1.0% HDPP	6.5	3
MD1	1.5% HDPP	4.5	3
MD2	1.5% HDPP	5.0	3
MD3	1.5% HDPP	5.5	3
MD4	1.5% HDPP	6.0	3
MD5	1.5% HDPP	6.5	3
ME1	2.0% HDPP	4.5	3
ME2	2.0% HDPP	5.0	3
ME3	2.0% HDPP	5.5	3
ME4	2.0% HDPP	6.0	3
ME5	2.0% HDPP	6.5	3
MF1	2.5% HDPP	4.5	3
MF2	2.5% HDPP	5.0	3
MF3	2.5% HDPP	5.5	3
MF4	2.5% HDPP	6.0	3
MF5	2.5% HDPP	6.5	3
MG1	3.0% HDPP	4.5	3
MG2	3.0% HDPP	5.0	3
MG3	3.0% HDPP	5.5	3
MG4	3.0% HDPP	6.0	3
MG5	3.0% HDPP	6.5	3
<b>TOTAL</b>			<b>105</b>

**(b) Dry Process**

Similarly, the percentages of HDPP and bitumen contents were the same as the wet process and the same three (3) Marshall specimens were prepared for each of the bitumen content. In the dry process, HDPP were chopped to 12mm (Abhati *et al.*, 2011; Tapkin, 2006) and blended with premixed asphalt before compacting. The experimental plan is as shown in Table 3.23 below. Specimen identities FA1, FB2 ..... to FF5 mean Fibre mix A number 1, Fibre mix B number 2 and Fibre mix F number 5 as in the Table 3.23. The weight, diameter and height of specimens were the same as wet process.

**Table 3.23: Marshall test plan for HDPP fibre specimens (dry process)**

<b>Specimen Identification</b>	<b>% HDPP as Proportion of Total Mix Weight</b>	<b>Bitumen Content (% of Total Mix Weight)</b>	<b>Number of Samples</b>
FA1	0.5% HDPP	4.5	3
FA2	0.5% HDPP	5.0	3
FA3	0.5% HDPP	5.5	3
FA4	0.5% HDPP	6.0	3
FA5	0.5% HDPP	6.5	3
FB1	1.0% HDPP	4.5	3
FB2	1.0% HDPP	5.0	3
FB3	1.0% HDPP	5.5	3
FB4	1.0% HDPP	6.0	3
FB5	1.0% HDPP	6.5	3
FC1	1.5% HDPP	4.5	3
FC2	1.5% HDPP	5.0	3
FC3	1.5% HDPP	5.5	3
FC4	1.5% HDPP	6.0	3
FC5	1.5% HDPP	6.5	3
FD1	2.0% HDPP	4.5	3
FD2	2.0% HDPP	5.0	3
FD3	2.0% HDPP	5.5	3
FD4	2.0% HDPP	6.0	3
FD5	2.0% HDPP	6.5	3
FE1	2.5% HDPP	4.5	3
FE2	2.5% HDPP	5.0	3
FE3	2.5% HDPP	5.5	3
FE4	2.5% HDPP	6.0	3
FE5	2.5% HDPP	6.5	3
FF1	3.0% HDPP	4.5	3
FF2	3.0% HDPP	5.0	3
FF3	3.0% HDPP	5.5	3
FF4	3.0% HDPP	6.0	3
FF5	3.0% HDPP	6.5	3
<b>TOTAL</b>			<b>90</b>

**(c) Result of Volumetric and Marshall Tests**

The Bulk Specific Gravity or Compacted Density of the Mix (CDM) was determined using ASTM D1188-07 (2015) (see details in Appendices B1 and B2). Theoretical Maximum Specific Gravity of the Mix ( $G_{mm}$ ) were determined using ASTM D 2041(2011) (see details in Appendices B3 and B4). ASTM D3203 (2011) was used to carry out voids analysis (See

details in Appendices B5, B6, B7 and B8). ASTM D1559 (2004) was used to determine the stability and flow of specimens. The summary of results of volumetric and Marshall tests for polymer modified HDPP ('wet' process) concretes are presented in Tables 3.24 to 3.30, while that of fibre reinforced HDPP ('dry' process) concretes are presented in Tables 3.31 to 3.36. Details of Marshall stability and flow are in Appendices B11, B12, B13 and B14.

**Table 3.24: Volumetric & Marshall tests for 0% HDPP Content (Control)**

<b>Bitumen Content (%)</b>	<b>Stability (kN)</b>	<b>Flow (mm)</b>	<b>CDM (g/cm<sup>3</sup>)</b>	<b>VIM (%)</b>	<b>VMA (%)</b>	<b>VFB (%)</b>
4.5	6.44	2.66	2.387	6.32	15.9	60.21
5.0	8.57	2.73	2.421	5.06	15.1	66.57
5.5	14.52	2.92	2.489	3.81	13.2	71.15
6.0	12.06	3.15	2.466	3.33	14.5	76.98
6.5	8.07	3.38	2.445	3.01	15.6	80.76

**Table 3.25: Volumetric & Marshall tests for 0.5% HDPP Wet process**

<b>Bitumen Content (%)</b>	<b>Stability (kN)</b>	<b>Flow (mm)</b>	<b>CDM (g/cm<sup>3</sup>)</b>	<b>VIM (%)</b>	<b>VMA (%)</b>	<b>VFB (%)</b>
4.5	8.01	2.51	2.410	6.13	16.3	62.35
5.0	13.08	2.63	2.443	4.91	16.0	69.23
5.5	17.42	2.76	2.464	3.70	15.3	75.92
6.0	14.67	3.09	2.451	3.23	18.0	82.03
6.5	8.28	3.31	2.414	2.92	19.3	84.88

**Table 3.26: Volumetric & Marshall tests for 1.0% HDPP Wet process**

<b>Bitumen Content (%)</b>	<b>Stability (kN)</b>	<b>Flow (mm)</b>	<b>CDM (g/cm<sup>3</sup>)</b>	<b>VIM (%)</b>	<b>VMA (%)</b>	<b>VFB (%)</b>
4.5	10.18	2.37	2.406	6.00	16.6	63.77
5.0	15.60	2.53	2.438	4.81	16.2	70.40
5.5	19.03	2.71	2.459	3.62	15.6	76.85
6.0	15.28	3.02	2.436	3.16	17.9	82.34
6.5	11.90	3.24	2.389	2.86	19.6	85.40

**Table 3.27: Volumetric & Marshall tests for 1.5% HDPP Wet process**

<b>Bitumen Content (%)</b>	<b>Stability (kN)</b>	<b>Flow (mm)</b>	<b>CDM (g/cm<sup>3</sup>)</b>	<b>VIM (%)</b>	<b>VMA (%)</b>	<b>VFB (%)</b>
4.5	16.26	2.22	2.401	5.75	16.9	65.89
5.0	20.86	2.38	2.424	4.60	16.5	72.14
5.5	25.78	2.65	2.454	3.47	15.9	78.24
6.0	20.59	2.86	2.421	3.03	17.9	83.03
6.5	19.11	3.18	2.375	2.74	19.9	86.22

**Table 3.28: Volumetric & Marshall tests for 2.0% HDPP Wet process**

<b>Bitumen Content (%)</b>	<b>Stability (kN)</b>	<b>Flow (mm)</b>	<b>CDM (g/cm<sup>3</sup>)</b>	<b>VIM (%)</b>	<b>VMA (%)</b>	<b>VFB (%)</b>
4.5	18.43	2.21	2.397	5.62	17.2	67.20
5.0	21.37	2.33	2.419	4.50	16.8	73.23
5.5	27.68	2.54	2.450	3.39	16.2	79.10
6.0	24.20	2.80	2.407	2.96	17.8	83.35
6.5	20.52	3.01	2.360	2.68	19.8	86.48

**Table 3.29: Volumetric & Marshall tests for 2.5% HDPP Wet process**

<b>Bitumen Content (%)</b>	<b>Stability (kN)</b>	<b>Flow (mm)</b>	<b>CDM (g/cm<sup>3</sup>)</b>	<b>VIM (%)</b>	<b>VMA (%)</b>	<b>VFB (%)</b>
4.5	15.51	2.15	2.393	5.44	17.4	68.84
5.0	20.13	2.28	2.415	4.35	16.8	74.06
5.5	24.14	2.44	2.445	3.28	16.5	80.17
6.0	22.51	2.64	2.392	2.86	17.8	83.87
6.5	18.33	2.85	2.356	2.59	19.8	86.90

**Table 3.30: Volumetric & Marshall tests for 3.0% HDPP Wet process**

<b>Bitumen Content (%)</b>	<b>Stability (kN)</b>	<b>Flow (mm)</b>	<b>CDM (g/cm<sup>3</sup>)</b>	<b>VIM (%)</b>	<b>VMA (%)</b>	<b>VFB (%)</b>
4.5	15.34	2.08	2.388	5.25	17.7	70.43
5.0	17.11	2.23	2.410	4.20	17.1	75.40
5.5	23.23	2.38	2.441	3.16	16.8	81.20
6.0	18.90	2.57	2.378	2.76	17.7	84.40
6.5	17.31	2.67	2.351	2.50	19.4	87.11

**Table 3.31: Volumetric & Marshall tests for 0.5% HDPP Dry Process**

<b>Bitumen Content (%)</b>	<b>Stability (kN)</b>	<b>Flow (mm)</b>	<b>CDM (g/cm<sup>3</sup>)</b>	<b>VIM (%)</b>	<b>VMA (%)</b>	<b>VFB (%)</b>
4.5	4.92	2.75	2.229	9.22	21.9	57.86
5.0	7.83	2.91	2.296	6.58	19.9	67.00
5.5	15.97	3.24	2.357	4.95	18.2	72.84
6.0	9.37	3.42	2.248	4.03	22.4	82.04
6.5	5.28	3.89	2.218	3.32	23.9	86.13

**Table 3.32: Volumetric & Marshall tests for 1.0% HDPP Dry process**

<b>Bitumen Content (%)</b>	<b>Stability (kN)</b>	<b>Flow (mm)</b>	<b>CDM (g/cm<sup>3</sup>)</b>	<b>VIM (%)</b>	<b>VMA (%)</b>	<b>VFB (%)</b>
4.5	4.55	3.07	2.078	10.11	27.6	63.30
5.0	5.84	3.29	2.179	8.10	24.4	66.83
5.5	7.90	3.67	2.258	6.10	22.1	72.41
6.0	4.57	4.10	2.115	5.63	27.4	79.47
6.5	3.22	4.39	2.090	4.08	28.7	85.77

**Table 3.33: Volumetric & Marshall tests for 1.5% HDPP Dry process**

<b>Bitumen Content (%)</b>	<b>Stability (kN)</b>	<b>Flow (mm)</b>	<b>CDM (g/cm<sup>3</sup>)</b>	<b>VIM (%)</b>	<b>VMA (%)</b>	<b>VFB (%)</b>
4.5	3.04	3.42	1.930	11.51	33.1	65.18
5.0	3.82	3.67	1.986	10.53	31.5	66.56
5.5	4.67	4.09	2.161	8.04	25.8	68.90
6.0	4.06	4.57	1.984	6.33	32.3	80.41
6.5	3.38	4.90	1.903	4.85	35.4	86.31

**Table 3.34: Volumetric & Marshall tests for 2.0% HDPP Dry process**

<b>Bitumen Content (%)</b>	<b>Stability (kN)</b>	<b>Flow (mm)</b>	<b>CDM (g/cm<sup>3</sup>)</b>	<b>VIM (%)</b>	<b>VMA (%)</b>	<b>VFB (%)</b>
4.5	2.17	3.78	1.786	13.90	38.4	63.76
5.0	2.51	4.05	1.925	11.13	33.9	67.19
5.5	2.90	4.51	1.970	8.38	32.8	74.41
6.0	2.21	4.84	1.854	7.33	37.1	80.24
6.5	1.81	5.11	1.738	5.61	41.3	86.42

**Table 3.35: Volumetric & Marshall tests for 2.5% HDPP Dry process**

<b>Bitumen Content (%)</b>	<b>Stability (kN)</b>	<b>Flow (mm)</b>	<b>CDM (g/cm<sup>3</sup>)</b>	<b>VIM (%)</b>	<b>VMA (%)</b>	<b>VFB (%)</b>
4.5	1.44	4.13	1.647	14.80	43.5	65.97
5.0	1.84	4.43	1.688	12.65	42.4	70.15
5.5	2.27	4.94	1.835	9.53	37.7	74.75
6.0	2.02	5.51	1.726	8.33	41.7	80.05
6.5	1.25	5.92	1.614	6.38	45.8	86.08

**Table 3.36: Volumetric & Marshall tests for 3.0% HDPP Dry process**

<b>Bitumen Content (%)</b>	<b>Stability (kN)</b>	<b>Flow (mm)</b>	<b>CDM (g/cm<sup>3</sup>)</b>	<b>VIM (%)</b>	<b>VMA (%)</b>	<b>VFB (%)</b>
4.5	0.31	4.48	1.510	17.70	48.4	63.47
5.0	0.73	4.81	1.554	14.17	47.3	70.02
5.5	1.06	5.36	1.672	10.67	43.6	75.51
6.0	0.90	5.99	1.599	8.72	46.3	81.16
6.5	0.46	6.42	1.571	8.14	47.5	82.88

### 3.4 Evaluation of Simple Performance Tests

The mechanical tests performed by simple performance test (SPT) evaluation of asphalt are dynamic modulus, indirect tensile strength and rutting resistance. Creep and morphological studies were evaluated under asphalt ageing.

All the tests conducted under simple performance test (SPT) and ageing process were done using 0%, 1.0%, 2.0% and 3.0% HDPP contents for wet process, and 0%, 0.5%, 1.0% and 1.5% HDPP contents for dry process. These are because of the optimum Marshall parameters which were attained at 2.0% and 0.5% HDPP contents respectively for wet and dry processes.

#### (a) Dynamic Modulus

The test was carried out using Simple Performance Tester (SPT) or Asphalt Mixture Performance Tester (AMPT) of International Process Control (IPC). It is a global test machine otherwise called Universal Test Machine and a load of 25kN pneumatic (UTM-

25P) was applied in accordance with AASHTO TP62 (2003) method. It delivered a constant pressure of up to 2800 kPa (2800kN/mm<sup>2</sup>) in an environmental chamber. It was conditioned to testing temperatures of 4.4, 21.1, 25, 45 and 60<sup>0</sup>C to simulate ambient environmental temperature in pavement. Test was done by using gluing gauge plugs onto the specimen sides and attached to Linear Variable Differential Transducer (LVDT) to determine the displacement. A haversine dynamic loading (*Pd*) was adjusted such that axial strains lie between 75 and 125 microstrain ( $\mu\text{m/m}$ ) without necessarily impacting in a cyclic manner. For each of polymer and fibre asphalt mixes, three (3) specimen replicates were prepared. Following the recommendations of AASHTO TP62 (2003) test loading frequencies considered were 0.1, 0.5, 1, 5, 10 and 25 Hz respectively. The recommended percentage air void (VIM) is  $7\pm 1.0\%$  (AASHTO T312-03: 2005).

From Marshall tests, optimum HDPP content for wet process is 2.0% while optimum for dry process is 0.5%. Thus SPT's and ageing process evaluations were carried out using 0, 1, 2, and 3% HDPP for wet process. The same tests were carried out using 0, 0.5, 1.0 and 1.5% HDPP for dry process being close to the optimum.

The result of shift factor, reduced frequency, dynamic modulus, phase angle and the recoverable strain are shown in Tables 3.37 to 3.43.

**Table 3.37: Results of  $E^*$ ,  $\phi$  and  $\varepsilon$  for 0% HDPP Wet Process Asphalt**

Temperature (°C)	Frequency (Hz)	Shift factor	Reduced frequency	Dynamic Modulus (MPa)	Phase Angle (deg)	Recoverable Strain (mm/mm)
4.4	25	1.5616	911.045377	1.66E+04	15.41	7.45E-05
	10	1.5616	364.418151	1.46E+04	16.89	9.62E-05
	5	1.5616	182.209075	1.39E+04	20.02	2.43E-04
	1	1.5616	36.441815	1.13E+04	20.95	3.35E-04
	0.5	1.5616	18.220908	1.05E+04	19.01	5.42E-04
	0.1	1.5616	3.644182	0.88E+04	16.63	7.98E-04
21.1	25	-0.0089	24.49288	1.29E+04	17.02	4.82E-05
	10	-0.0089	9.797155	1.05E+04	18.88	7.43E-05
	5	-0.0089	4.898577	0.72E+04	19.63	9.54E-05
	1	-0.0089	0.979715	0.66E+04	21.47	1.22E-04
	0.5	-0.0089	0.489858	0.58E+04	19.25	3.88E-04
	0.1	-0.0089	0.097972	0.49E+04	16.94	5.27 E-04
25	25	-0.3503	11.159379	1.01E+04	17.09	2.58E-05
	10	-0.3503	4.463751	8.62E+03	19.53	3.64E-05
	5	-0.3503	2.231876	7.48E+03	20.70	4.34E-05
	1	-0.3503	0.446375	5.19E+03	22.24	6.61E-05
	0.5	-0.3503	0.223188	4.62E+03	17.98	7.49E-05
	0.1	-0.3503	0.044638	3.38E+03	17.23	1.03E-04
45	25	-1.9696	0.268127	3.97E+03	21.14	1.09E-05
	10	-1.9696	0.107251	3.48E+03	22.45	2.04E-05
	5	-1.9696	0.053625	2.86E+03	24.16	3.00E-05
	1	-1.9696	0.010725	1.92E+03	22.59	5.46E-05
	0.5	-1.9696	0.005363	1.72E+03	18.14	6.24E-05
	0.1	-1.9696	0.001073	1.45E+03	16.03	4.05E-06
60	25	-3.0564	0.021955	1.92E+03	21.07	5.19E-06
	10	-3.0564	0.008782	1.70E+03	23.56	7.00E-06
	5	-3.0564	0.004391	1.36E+03	26.49	1.39E-05
	1	-3.0564	0.000878	8.43E+02	24.74	3.59E-05
	0.5	-3.0564	0.000439	7.56E+02	23.80	4.31E-05
	0.1	-3.0564	0.000088	6.70E+02	18.36	6.04E-05

E means scientific is exponent of the values i.e  $1.66E+04 = 1.66 \times 10^4$

**Table 3.38: Results of  $E^*$ ,  $\phi$  and  $\varepsilon$  for 1% HDPP Wet Process Asphalt**

Temperature (°C)	Frequency (Hz)	Shift factor	Reduced frequency	Dynamic Modulus (MPa)	Phase Angle (deg)	Recoverable Strain (mm/mm)
4.4	25	1.5616	911.045377	1.93E+04	14.64	1.19E-04
	10	1.5616	364.418151	1.64E+04	16.05	1.54E-04
	5	1.5616	182.209075	1.46E+04	19.62	3.89E-04
	1	1.5616	36.441815	1.39E+04	19.90	5.36E-04
	0.5	1.5616	18.220908	1.15E+04	18.06	8.67E-04
	0.1	1.5616	3.644182	9.80E+03	15.80	1.28E-03
21.1	25	-0.0089	24.49288	1.72E+04	16.17	7.71E-05
	10	-0.0089	9.797155	1.57E+04	17.94	1.19E-04
	5	-0.0089	4.898577	1.24E+04	18.65	1.53E-04
	1	-0.0089	0.979715	8.54E+03	20.40	1.95E-04
	0.5	-0.0089	0.489858	6.77E+03	19.06	6.21E-04
	0.1	-0.0089	0.097972	5.06E+03	16.09	8.43E-04
25	25	-0.3503	11.159379	1.14E+04	16.24	4.13E-05
	10	-0.3503	4.463751	9.05E+03	18.55	5.82E-05
	5	-0.3503	2.231876	7.46E+03	20.08	6.94E-05
	1	-0.3503	0.446375	5.95E+03	21.13	1.06E-04
	0.5	-0.3503	0.223188	5.13E+03	19.78	1.20E-04
	0.1	-0.3503	0.044638	4.36E+03	16.37	1.65E-04
45	25	-1.9696	0.268127	4.87E+03	20.08	1.74E-05
	10	-1.9696	0.107251	4.42E+03	21.33	3.26E-05
	5	-1.9696	0.053625	3.91E+03	22.95	4.80E-05
	1	-1.9696	0.010725	2.69E+03	21.46	8.74E-05
	0.5	-1.9696	0.005363	2.45E+03	17.60	9.98E-05
	0.1	-1.9696	0.001073	2.27E+03	15.23	6.48E-06
60	25	-3.0564	0.021955	3.64E+03	20.02	8.30E-06
	10	-3.0564	0.008782	2.10E+03	22.38	1.12E-05
	5	-3.0564	0.004391	1.51E+03	25.17	2.22E-05
	1	-3.0564	0.000878	9.26E+02	23.50	5.74E-05
	0.5	-3.0564	0.000439	9.01E+02	22.61	6.90E-05
	0.1	-3.0564	0.000088	6.74E+02	17.44	9.66E-05

**Table 3.39: Results of  $E^*$ ,  $\phi$  and  $\varepsilon$  for 2% HDPP Wet Process Asphalt**

Temperature ( $^{\circ}\text{C}$ )	Frequency (Hz)	Shift factor	Reduced frequency	Dynamic Modulus (MPa)	Phase Angle (deg)	Recoverable Strain (mm/mm)
4.4	25	1.5616	911.045377	2.59E+04	13.87	1.48E-04
	10	1.5616	364.418151	2.20E+04	15.20	1.90E-04
	5	1.5616	182.209075	1.96E+04	18.02	4.81E-04
	1	1.5616	36.441815	1.86E+04	18.44	6.63E-04
	0.5	1.5616	18.220908	1.54E+04	15.21	1.07E-03
	0.1	1.5616	3.644182	1.31E+04	14.97	1.58E-03
21.1	25	-0.0089	24.49288	2.30E+04	15.32	9.54E-05
	10	-0.0089	9.797155	2.10E+04	16.99	1.47E-04
	5	-0.0089	4.898577	1.66E+04	18.45	1.89E-04
	1	-0.0089	0.979715	1.14E+04	19.32	2.42E-04
	0.5	-0.0089	0.489858	9.07E+03	16.94	7.68E-04
	0.1	-0.0089	0.097972	6.78E+03	14.91	1.04E-03
25	25	-0.3503	11.159379	1.53E+04	15.38	5.11E-05
	10	-0.3503	4.463751	1.21E+04	17.58	7.21E-05
	5	-0.3503	2.231876	1.00E+04	19.46	8.59E-05
	1	-0.3503	0.446375	7.97E+03	20.02	1.31E-04
	0.5	-0.3503	0.223188	6.87E+03	16.18	1.48E-04
	0.1	-0.3503	0.044638	5.84E+03	14.65	2.04E-04
45	25	-1.9696	0.268127	6.53E+03	19.03	2.16E-05
	10	-1.9696	0.107251	5.92E+03	20.21	4.04E-05
	5	-1.9696	0.053625	5.24E+03	21.74	5.94E-05
	1	-1.9696	0.010725	3.60E+03	20.33	1.08E-04
	0.5	-1.9696	0.005363	3.28E+03	19.95	1.24E-04
	0.1	-1.9696	0.001073	3.04E+03	14.91	8.02E-06
60	25	-3.0564	0.021955	4.88E+03	18.96	1.03E-05
	10	-3.0564	0.008782	2.81E+03	21.20	1.39E-05
	5	-3.0564	0.004391	2.02E+03	23.84	2.75E-05
	1	-3.0564	0.000878	1.24E+03	22.27	7.11E-05
	0.5	-3.0564	0.000439	1.21E+03	21.42	8.53E-05
	0.1	-3.0564	0.000088	9.03E+02	16.52	1.20E-04

**Table 3.40: Results of  $E^*$ ,  $\phi$  and  $\varepsilon$  for 3% HDPP Wet Process Asphalt**

Temperature ( $^{\circ}$ C)	Frequency (Hz)	Shift factor	Reduced frequency	Dynamic Modulus (MPa)	Phase Angle (deg)	Recoverable Strain (mm/mm)
4.4	25	1.5616	911.045377	2.03E+04	11.10	1.79E-04
	10	1.5616	364.418151	1.72E+04	12.16	2.31E-04
	5	1.5616	182.209075	1.53E+04	15.02	5.83E-04
	1	1.5616	36.441815	1.46E+04	15.08	8.04E-04
	0.5	1.5616	18.220908	1.21E+04	15.21	1.30E-03
	0.1	1.5616	3.644182	1.03E+04	11.97	1.92E-03
21.1	25	-0.0089	24.49288	2.15E+04	12.25	1.16E-04
	10	-0.0089	9.797155	1.96E+04	13.59	1.78E-04
	5	-0.0089	4.898577	1.55E+04	14.13	2.29E-04
	1	-0.0089	0.979715	1.07E+04	15.46	2.93E-04
	0.5	-0.0089	0.489858	8.46E+03	14.44	9.31E-04
	0.1	-0.0089	0.097972	6.33E+03	12.20	1.26E-03
25	25	-0.3503	11.159379	1.48E+04	12.30	6.19E-05
	10	-0.3503	4.463751	1.18E+04	13.87	8.74E-05
	5	-0.3503	2.231876	9.70E+03	14.90	1.04E-04
	1	-0.3503	0.446375	7.74E+03	16.01	1.59E-04
	0.5	-0.3503	0.223188	6.67E+03	14.38	1.80E-04
	0.1	-0.3503	0.044638	5.67E+03	12.41	2.47E-04
45	25	-1.9696	0.268127	7.55E+03	15.22	2.62E-05
	10	-1.9696	0.107251	6.85E+03	16.16	4.90E-05
	5	-1.9696	0.053625	6.06E+03	17.40	7.20E-05
	1	-1.9696	0.010725	4.17E+03	16.26	1.31E-04
	0.5	-1.9696	0.005363	3.80E+03	15.78	1.50E-04
	0.1	-1.9696	0.001073	3.52E+03	11.54	9.72E-06
60	25	-3.0564	0.021955	5.82E+03	15.17	1.25E-05
	10	-3.0564	0.008782	3.36E+03	16.96	1.68E-05
	5	-3.0564	0.004391	2.42E+03	18.54	3.34E-05
	1	-3.0564	0.000878	1.48E+03	20.78	8.62E-05
	0.5	-3.0564	0.000439	1.44E+03	17.14	1.03E-04
	0.1	-3.0564	0.000088	1.08E+03	13.22	1.45E-04

**Table 3.41: Results of  $E^*$ ,  $\phi$  and  $\varepsilon$  for 0.5% HDPP Dry Process Asphalt**

Temperature (°C)	Frequency (Hz)	Shift factor	Reduced frequency	Dynamic Modulus (MPa)	Phase Angle (deg)	Recoverable Strain (mm/mm)
4.4	25	1.5616	911.045377	1.89E+04	20.50	8.57E-05
	10	1.5616	364.418151	1.66E+04	22.46	1.11E-04
	5	1.5616	182.209075	1.58E+04	23.97	2.79E-04
	1	1.5616	36.441815	1.29E+04	25.86	3.85E-04
	0.5	1.5616	18.220908	1.20E+04	25.28	6.23E-04
	0.1	1.5616	3.644182	1.00E+04	22.12	9.18E-04
21.1	25	-0.0089	24.49288	1.39E+04	22.64	5.54E-05
	10	-0.0089	9.797155	1.13E+04	25.11	8.54E-05
	5	-0.0089	4.898577	7.78E+03	26.11	1.10E-04
	1	-0.0089	0.979715	7.13E+03	27.56	1.40E-04
	0.5	-0.0089	0.489858	6.26E+03	25.27	4.46E-04
	0.1	-0.0089	0.097972	5.29E+03	22.53	6.06E-04
25	25	-0.3503	11.159379	1.06E+04	22.73	2.97E-05
	10	-0.3503	4.463751	9.05E+03	25.97	4.19E-05
	5	-0.3503	2.231876	7.85E+03	27.53	4.99E-05
	1	-0.3503	0.446375	5.45E+03	30.58	7.60E-05
	0.5	-0.3503	0.223188	4.85E+03	26.91	8.61E-05
	0.1	-0.3503	0.044638	3.55E+03	22.92	1.18E-04
45	25	-1.9696	0.268127	4.09E+03	28.12	1.25E-05
	10	-1.9696	0.107251	3.58E+03	29.86	2.35E-05
	5	-1.9696	0.053625	2.95E+03	30.80	3.45E-05
	1	-1.9696	0.010725	1.98E+03	32.70	6.28E-05
	0.5	-1.9696	0.005363	1.77E+03	29.13	7.18E-05
	0.1	-1.9696	0.001073	1.49E+03	21.32	4.66E-06
60	25	-3.0564	0.021955	1.95E+03	26.02	5.97E-06
	10	-3.0564	0.008782	1.73E+03	33.99	8.05E-06
	5	-3.0564	0.004391	1.38E+03	35.23	1.60E-05
	1	-3.0564	0.000878	8.57E+02	30.24	4.13E-05
	0.5	-3.0564	0.000439	7.69E+02	25.00	4.96E-05
	0.1	-3.0564	0.000088	6.81E+02	23.76	6.95E-05

**Table 3.42: Results of  $E^*$ ,  $\phi$  and  $\varepsilon$  for 1.0% HDPP Dry Process Asphalt**

Temperature (°C)	Frequency (Hz)	Shift factor	Reduced frequency	Dynamic Modulus (MPa)	Phase Angle (deg)	Recoverable Strain (mm/mm)
4.4	25	1.5616	911.045377	9.13E+03	29.12	6.85E-05
	10	1.5616	364.418151	8.03E+03	31.92	8.85E-05
	5	1.5616	182.209075	7.65E+03	32.06	2.24E-04
	1	1.5616	36.441815	6.22E+03	39.60	3.08E-04
	0.5	1.5616	18.220908	5.78E+03	35.93	4.99E-04
	0.1	1.5616	3.644182	4.84E+03	31.43	7.34E-04
21.1	25	-0.0089	24.49288	6.84E+03	32.17	4.43E-05
	10	-0.0089	9.797155	5.57E+03	35.68	6.84E-05
	5	-0.0089	4.898577	3.82E+03	37.10	8.78E-05
	1	-0.0089	0.979715	3.50E+03	40.58	1.12E-04
	0.5	-0.0089	0.489858	3.07E+03	36.49	3.57E-04
	0.1	-0.0089	0.097972	2.60E+03	32.02	4.85E-04
25	25	-0.3503	11.159379	5.35E+03	32.30	2.37E-05
	10	-0.3503	4.463751	4.57E+03	36.91	3.35E-05
	5	-0.3503	2.231876	3.96E+03	39.12	3.99E-05
	1	-0.3503	0.446375	2.75E+03	44.03	6.08E-05
	0.5	-0.3503	0.223188	2.45E+03	39.98	6.89E-05
	0.1	-0.3503	0.044638	1.79E+03	32.56	9.48E-05
45	25	-1.9696	0.268127	2.02E+03	39.95	1.00E-05
	10	-1.9696	0.107251	1.77E+03	42.43	1.88E-05
	5	-1.9696	0.053625	1.46E+03	48.77	2.76E-05
	1	-1.9696	0.010725	9.79E+02	46.48	5.02E-05
	0.5	-1.9696	0.005363	8.77E+02	34.28	5.74E-05
	0.1	-1.9696	0.001073	7.40E+02	30.30	3.73E-06
60	25	-3.0564	0.021955	9.79E+02	39.82	4.77E-06
	10	-3.0564	0.008782	8.67E+02	48.31	6.44E-06
	5	-3.0564	0.004391	6.94E+02	61.07	1.28E-05
	1	-3.0564	0.000878	4.30E+02	52.98	3.30E-05
	0.5	-3.0564	0.000439	3.86E+02	40.53	3.97E-05
	0.1	-3.0564	0.000088	3.42E+02	33.92	5.56E-05

**Table 3.43: Results of  $E^*$ ,  $\phi$  and  $\varepsilon$  for 1.5% HDPP Dry Process Asphalt**

Temperature (°C)	Frequency (Hz)	Shift factor	Reduced frequency	Dynamic Modulus (MPa)	Phase Angle (deg)	Recoverable Strain (mm/mm)
4.4	25	1.5616	911.045377	2.99E+03	41.61	6.33E-05
	10	1.5616	364.418151	2.63E+03	45.60	8.18E-05
	5	1.5616	182.209075	2.50E+03	48.65	2.07E-04
	1	1.5616	36.441815	2.03E+03	53.57	2.85E-04
	0.5	1.5616	18.220908	1.89E+03	50.33	4.61E-04
	0.1	1.5616	3.644182	1.58E+03	42.90	6.78E-04
21.1	25	-0.0089	24.49288	2.19E+03	45.95	4.10E-05
	10	-0.0089	9.797155	1.79E+03	53.98	6.32E-05
	5	-0.0089	4.898577	1.22E+03	56.00	8.11E-05
	1	-0.0089	0.979715	1.12E+03	57.97	1.04E-04
	0.5	-0.0089	0.489858	9.86E+02	50.28	3.30E-04
	0.1	-0.0089	0.097972	8.33E+02	44.74	4.48E-04
25	25	-0.3503	11.159379	1.62E+03	46.14	2.19E-05
	10	-0.3503	4.463751	1.38E+03	52.73	3.09E-05
	5	-0.3503	2.231876	1.20E+03	55.89	3.69E-05
	1	-0.3503	0.446375	8.30E+02	62.05	5.62E-05
	0.5	-0.3503	0.223188	7.39E+02	54.55	6.37E-05
	0.1	-0.3503	0.044638	5.41E+02	46.52	8.76E-05
45	25	-1.9696	0.268127	5.96E+02	57.08	9.27E-06
	10	-1.9696	0.107251	5.22E+02	60.62	1.73E-05
	5	-1.9696	0.053625	4.29E+02	64.53	2.55E-05
	1	-1.9696	0.010725	2.88E+02	66.39	4.64E-05
	0.5	-1.9696	0.005363	2.58E+02	58.98	5.30E-05
	0.1	-1.9696	0.001073	2.18E+02	46.28	3.44E-06
60	25	-3.0564	0.021955	3.00E+02	60.89	4.41E-06
	10	-3.0564	0.008782	2.65E+02	69.01	5.95E-06
	5	-3.0564	0.004391	2.12E+02	71.52	1.18E-05
	1	-3.0564	0.000878	1.32E+02	65.40	3.05E-05
	0.5	-3.0564	0.000439	1.18E+02	57.76	3.66E-05
	0.1	-3.0564	0.000088	1.05E+02	50.17	5.13E-05

### 3.5 Indirect Tensile Strength (ITS) Test

The test comprises measurement of compressive load, vertical and radial displacements. It is used to determine the loading values for resilient modulus, fracture energy and fatigue resistance tests. It was carried out following the recommendations of ASTM D4123 (2005a) and ASTM D6931 (2012) at a temperature of 25<sup>0</sup>C. The results of both wet and

dry processes for ITS in HDPP asphalt concrete are shown in Table 3.44 and 3.45 respectively.

**Table 3.44: ITS result of polymer wet process asphalt**

Asphalt type	Specimen Identity	Specimen diameter (mm)	Specimen height (mm)	Peak load at failure (N)	ITS (kN/m <sup>2</sup> )	Avg. ITS (kN/m <sup>2</sup> )
0% HDPP	1	101.6	62.8	13,455	1341.91	1339.78
	2	101.6	63.4	13,391	1322.89	
	3	101.6	64.4	13,928	1354.55	
1.0% HDPP	1	101.6	63.1	14,467	1435.94	1439.24
	2	101.6	64.5	14,972	1453.82	
	3	101.6	63.1	14,386	1427.96	
2.0% HDPP	1	101.6	64.0	15560	1522.73	1525.74
	2	101.6	63.8	15548	1526.32	
	3	101.6	64.1	15640	1528.17	
3.0% HDPP	1	101.6	63.1	14466.5	1435.91	1447.45
	2	101.6	62.9	14906.5	1484.28	
	3	101.6	63.8	14487	1422.16	

**Table 3.45: ITS result of polymer dry process and reinforced asphalt**

Asphalt type	Specimen Identity	Specimen diameter (mm)	Specimen height (mm)	Peak load at failure (N)	ITS (kN/m <sup>2</sup> )	Avg. ITS (kN/m <sup>2</sup> )
0% HDPP	1	101.6	62.8	13,455	1341.91	1339.78
	2	101.6	63.4	13,391	1322.89	
	3	101.6	64.4	13,928	1354.55	
0.5% HDPP	1	101.6	64.0	14494.5	1418.45	1441.79
	2	101.6	64.1	14935.5	1459.33	
	3	101.6	64.2	14838.5	1447.59	
1.0% HDPP	1	101.6	63.4	6282.5	620.63	595.75
	2	101.6	64.1	6363	621.72	
	3	101.6	64.8	5637.5	544.88	
1.5% HDPP	1	101.6	65.4	2288	219.11	230.92
	2	101.6	66.0	2595	246.26	
	3	101.6	65.8	2389	227.40	

**(b) Rutting of Asphalt**

Rutting resistance of the asphalt mixes was determined using Asphalt Pavement Analyzer (APA) and load repetitions of 0 to 10,000 cycles to permanent deformation and at 50°C following the recommendations of NCHRP Report 508:2003 (Cooley *et al.*, 2000) and AASHTO T324 (2015). The results of rutting test for wet and dry processes are shown in Tables 3.46 and 3.47 respectively (details in Appendix C5).

**Table 3.46: Asphalt rutting evaluation for wet process (mm)**

<b>% HDPP</b>	<b>0%</b>	<b>1%</b>	<b>2%</b>	<b>3%</b>
<b>No. of Cycles</b>				
0	0.00	0.00	0.00	0.00
25	0.22	0.21	0.18	0.17
50	0.48	0.45	0.44	0.36
75	0.81	0.77	0.68	0.61
100	0.91	0.86	0.73	0.69
125	0.95	0.90	0.84	0.72
150	1.00	0.95	0.90	0.76
175	1.05	0.99	0.97	0.80
200	1.22	1.16	1.06	0.92
225	1.27	1.17	1.10	0.98
250	1.47	1.19	1.12	1.00
275	1.60	1.22	1.15	1.02
300	1.69	1.26	1.19	1.04
325	1.70	1.26	1.19	1.06
350	1.81	1.27	1.20	1.07
375	1.92	1.30	1.23	1.09
400	2.03	1.31	1.24	1.10
425	1.76	1.41	1.39	1.06
475	2.15	1.72	1.38	1.20
500	2.36	1.89	1.40	1.22
550	2.48	1.98	1.44	1.25
575	2.55	2.04	1.51	1.31
600	2.75	2.20	1.45	1.26
1000	3.35	2.68	1.61	1.40
2000	4.62	3.25	1.86	1.16
3000	5.00	3.37	1.93	1.25
4000	5.95	3.47	1.98	1.49
6000	6.87	3.64	2.08	1.72
7000	6.96	3.77	2.16	1.74
7250	7.10	3.72	2.12	1.78
7500	7.20	3.73	2.13	1.80
7750	7.24	3.74	2.14	1.81
8000	7.33	3.75	2.15	1.83
8250	6.92	4.43	2.33	1.86
8500	7.26	4.65	2.37	1.88
8750	7.49	4.64	2.59	1.95
9000	6.29	4.72	2.73	2.03
9250	7.26	4.79	2.89	2.08
9500	7.23	4.99	3.03	2.19
9750	7.73	4.95	2.21	2.28
10000	7.65	5.36	3.55	2.30
<b>Max rut depth</b>	<b>7.73</b>	<b>5.35</b>	<b>3.553</b>	<b>2.30</b>
<b>Average</b>	<b>3.75</b>	<b>2.47</b>	<b>1.6</b>	<b>1.3</b>

**Table 3.47: Asphalt rutting evaluation for dry process (mm)**

<b>% HDPP</b>	<b>0%</b>	<b>0.5%</b>	<b>1.0%</b>	<b>1.5%</b>
<b>No. of Cycles</b>				
0	0.00	0.00	0.00	0.00
25	0.22	0.45	0.52	1.21
50	0.48	0.71	1.14	2.64
75	0.81	0.90	1.92	4.46
100	0.91	0.99	2.15	5.01
125	0.95	1.16	2.25	5.23
150	1.00	1.26	2.37	5.50
175	1.05	1.42	2.49	5.78
200	1.22	1.56	2.89	6.71
225	1.27	1.59	3.43	6.68
250	1.47	1.72	3.49	7.22
275	1.60	1.74	3.59	7.31
300	1.69	1.96	3.71	8.23
325	1.70	2.05	3.71	8.61
350	1.81	1.95	3.74	8.19
375	1.92	2.15	3.84	9.03
400	2.03	2.17	3.87	9.11
425	1.76	2.30	4.75	11.04
475	2.15	2.33	5.38	11.18
500	2.36	2.47	5.48	11.86
550	2.48	2.61	6.89	12.53
575	2.55	2.91	7.87	13.97
600	2.75	3.50	8.21	16.80
1000	3.35	3.92	9.08	18.82
2000	4.62	5.19	10.16	21.28
3000	5.00	6.11	11.45	25.05
4000	5.95	6.63	12.47	30.50
6000	6.87	7.38	13.10	30.26
7000	6.96	6.73	14.34	31.63
7250	7.10	7.58	15.23	34.11
7500	7.20	7.59	16.06	34.16
7750	7.24	7.58	17.21	34.19
8000	7.33	7.44	17.83	34.22
8250	6.92	7.85	18.60	35.22
8500	7.26	7.84	19.51	36.66
8750	7.49	7.83	20.43	36.70
9000	6.29	7.74	21.23	37.11
9250	7.26	7.72	21.56	37.03
9500	7.23	7.89	22.95	37.23
9750	7.73	8.30	23.25	39.35
10000	7.65	8.96	23.56	39.63
<b>Max rut depth</b>	<b>7.73</b>	<b>8.96</b>	<b>23.56</b>	<b>39.63</b>
<b>Average</b>	<b>3.75</b>	<b>4.15</b>	<b>9.55</b>	<b>18.82</b>

### 3.6 Asphalt Creep Deformation

#### (a) Short-term loading (uniaxial static creep test)

The test was conducted according to the recommendations of BS 598-111(1995). Marshall specimens were pre-conditioned to testing temperature; and a static load of 414 kN/mm<sup>2</sup> (60 psi) was applied; being the realistic stress state which correlates with field conditions of loading (Little et al, 1993). The load was applied for 1 hour and the sample allowed to recover for another hour. Triplicate samples were tested at 10, 25, 40 and 60°C and the cumulative strains were determined 0.1, 0.25, 0.5, 1, 2, 4, 8, 15, 30, 45, 60mins. Tables 3.48 and 3.49 show maximum creep deformation, creep recovery or rebound and permanent creep deformation for both wet and dry mix processes respectively (details are in Appendices C1 and C2).

**Table 3.48: Summary of short term creep deformation test for wet process**

Temperature (°C)	HDPP Content (%)	Maximum Creep *10 <sup>-3</sup> (mm/mm)	Permanent Creep *10 <sup>-3</sup> (mm/mm)	Creep Recovery *10 <sup>-3</sup> (mm/mm)
10	0	1.8223	1.512	0.310
	1	1.6881	1.350	0.338
	2	1.5600	1.240	0.320
	3	1.3995	1.117	0.283
25	0	4.5143	3.625	0.889
	1	4.1820	3.120	1.062
	2	3.8642	2.906	0.958
	3	3.4669	2.696	0.771
40	0	9.7913	7.299	2.492
	1	8.7218	6.501	2.221
	2	8.0590	6.007	2.052
	3	7.2305	5.390	1.841
60	0	11.6543	9.509	2.145
	1	10.2730	8.470	1.803
	2	9.2371	7.827	1.410
	3	8.2875	7.022	1.266

**Table 3.49: Summary of short term creep deformation test for dry process**

Temperature (°C)	HDPP Content (%)	Maximum Creep *10 <sup>-3</sup> (mm/mm)	Permanent Creep *10 <sup>-3</sup> (mm/mm)	Creep Recovery *10 <sup>-3</sup> (mm/mm)
10	0	1.8223	1.513	0.310
	0.5	1.6400	1.325	0.315
	1.0	2.1867	1.758	0.429
	1.5	2.5512	2.107	0.444
25	0	4.5143	3.625	0.889
	0.5	4.0628	3.214	0.849
	1.0	5.4171	4.296	1.121
	1.5	6.3200	5.138	1.182
40	0	9.7913	7.295	2.497
	0.5	8.8122	6.530	2.282
	1.0	11.7500	8.660	3.090
	1.5	13.7079	10.226	3.482
60	0	11.6543	9.510	2.144
	0.5	10.4890	8.643	1.846
	1.0	13.9851	11.636	2.349
	1.5	16.3160	13.885	2.431

**(b) Long-term loading (uni-axial static creep test)**

The test followed recommendations of BS 598-111 (1995). It was conducted at 25°C at the same loading stress of 414kpa (little *et al.*, 1993) and it lasted for 200days to assess tertiary creep that gives rise to rutting. The results are presented in Tables 3.50 and 3.51 for wet and dry processes respectively (details are in Appendices C3 and C4).

**Table 3.50: Accumulated Permanent Strain at 25°C for wet process \*10<sup>-3</sup> (mm/mm)**

<b>Time (day)</b>	<b>% HDPP</b>	<b>0%</b>	<b>1.0%</b>	<b>2.0%</b>	<b>3.0%</b>
0.000069		0.35728	0.349777	0.302565	0.283527
0.000174		0.65072	0.637055	0.551067	0.516393
0.000347		0.82348	0.806187	0.69737	0.65349
0.000694		1.13372	1.109912	0.960099	0.899688
0.001389		1.48428	1.45311	1.256973	1.177882
0.002778		1.7339	1.697488	1.468366	1.375974
0.005556		2.48528	2.259297	2.104677	1.972247
0.010417		3.1192	2.835576	2.641517	2.475308
0.020833		3.61875	3.202652	3.060478	2.806992
0.03125		4.14425	3.732643	3.504908	3.214612
0.041667		4.51425	4.094179	3.817828	3.501613
0.041736		4.847583	4.391143	4.107641	3.784413
0.04184		5.180917	4.688106	4.397454	4.053747
0.042014		5.51425	4.985069	4.687268	4.32308
0.042361		5.847583	5.282033	4.977081	4.592413
0.043056		6.180917	5.578996	5.266894	4.861747
0.044444		6.51425	5.875959	5.556708	5.13108
0.047222		6.847583	6.172923	5.846521	5.400413
0.052083		7.180917	6.469886	6.136334	5.669747
0.0625		7.51425	6.766849	6.426148	5.93908
0.072917		7.847583	7.063813	6.715961	6.208413
0.083333		8.180917	7.360776	7.005774	6.477747
0.125		8.51425	7.657739	7.295588	6.74708
0.167		8.847583	7.954703	7.585401	7.016413
0.208		9.180917	8.251666	7.875214	7.285747
0.25		9.51425	8.548629	8.165028	7.55508
0.5		9.847583	8.845593	8.454841	7.824413
0.75		10.18092	9.142556	8.744654	8.093747

**Table 3.50 contd: Accumulated Permanent Strain at 25°C for wet process**

<b>Time (day)</b>	<b>% HDPP</b>	<b>0%</b>	<b>1.0%</b>	<b>2.0%</b>	<b>3.0%</b>
1		10.51425	9.439519	9.034468	8.36308
2		10.84758	9.736483	9.324281	8.632413
3		11.18092	10.03345	9.614094	8.901747
4		11.51425	10.33041	9.903908	9.17108
5		11.84758	10.62737	10.19372	9.440413
6		12.18092	10.92434	10.48353	9.709747
7		12.51425	11.2213	10.77335	9.97908
14		12.84758	11.51826	10.97358	10.24841
21		13.18092	11.81523	11.26105	10.51775
28		13.51425	12.11219	11.54851	10.78708
56		14.40149	13.18235	11.83598	11.05641
84		14.74815	13.49781	12.8657	11.66216
112		15.09482	13.81328	13.17077	11.93949
140		15.44149	14.12875	13.47583	12.21682
168		15.78815	14.44421	13.7809	12.49416
192		16.13482	14.75968	14.08597	12.77149

**Table 3.51: Accumulated Permanent Strain at 25°C for dry process \*10<sup>-3</sup> (mm/mm)**

<b>Time (day)</b>	<b>% HDPP</b>	<b>0%</b>	<b>0.5%</b>	<b>1.0%</b>	<b>1.5%</b>
0.000069		0.35728	0.365517	0.408463	0.510349
0.000174		0.65072	0.665722	0.74394	0.929507
0.000347		0.82348	0.842465	0.941449	1.176282
0.000694		1.13372	1.159858	1.296133	1.619438
0.001389		1.48428	1.5185	1.696914	2.120188
0.002778		1.7339	1.773875	1.982294	2.476752
0.005556		2.48528	2.360965	2.841314	3.550045
0.010417		3.1192	2.963177	3.566048	4.455554
0.020833		3.61875	3.346771	4.131646	5.052586
0.03125		4.14425	3.900612	4.731626	5.786301
0.041667		4.51425	4.278417	5.154067	6.302904
0.041736		4.847583	4.588744	5.545315	6.811944
0.04184		5.180917	4.899071	5.936563	7.296744
0.042014		5.51425	5.209397	6.327811	7.781544
0.042361		5.847583	5.519724	6.719059	8.266344
0.043056		6.180917	5.830051	7.110307	8.751144
0.044444		6.51425	6.140377	7.501555	9.235944
0.047222		6.847583	6.450704	7.892803	9.720744
0.052083		7.180917	6.761031	8.284051	10.20554
0.0625		7.51425	7.071357	8.675299	10.69034
0.072917		7.847583	7.381684	9.066547	11.17514
0.083333		8.180917	7.692011	9.457795	11.65994
0.125		8.51425	8.002337	9.849043	14.16887
0.167		8.847583	8.312664	10.24029	14.38365
0.208		9.180917	8.622991	10.63154	14.93578
0.25		9.51425	8.933317	11.02279	15.48791
0.5		9.847583	9.243644	11.41404	16.04005
0.75		10.18092	9.188269	11.80528	16.59218

**Table 3.51 contd: Accumulated Permanent Strain at 25°C for dry process**

<b>Time (day)</b>	<b>% HDPP</b>	<b>0%</b>	<b>0.5%</b>	<b>1.0%</b>	<b>1.5%</b>
1		10.51425	9.486717	12.19653	17.14431
2		10.84758	9.785165	12.58778	17.69645
3		11.18092	10.08361	12.97903	18.24858
4		11.51425	10.38206	13.37028	18.80071
5		11.84758	10.62737	13.76152	19.35285
6		12.18092	10.92434	14.15277	19.90498
7		12.51425	11.2213	14.54402	20.45711
14		12.84758	11.51826	17.00905	21.00925
21		13.18092	11.81523	17.45462	21.56138
28		13.51425	12.11219	17.9002	22.11351
56		14.40149	13.18235	18.34577	22.66565
84		14.74815	13.49781	19.94183	23.90742
112		15.09482	13.81328	20.41469	25.07293
140		15.44149	14.12875	20.88754	28.09869
168		15.78815	15.02198	22.73848	31.23539
192		16.13482	15.35007	24.65044	33.84444

### 3.7 Thermal Analysis

The TGA curves and its differential (DTG) were carried out in a Shimadzu TGA-50 thermo-gravimetric analyzer. Ozawa method (Castro-Das *et al.*, 2010) was used to evaluate the apparent activation energy which is a function of degree of decomposition in air and nitrogen gas. Thermal decomposition was determined in about 30 mg bitumen samples in aluminum holder under a nitrogen or air flow ( $50 \text{ cm}^3/\text{min}$ ), heated from 25 to 630 °C at varying heating rates of 5, 10, 20 and 40 °C/min. The graphs were presented in Figures 4.41 and 4.42.

### 3.8 Morphological Study by Scan Electron Micrograph

Surface morphology and analysis of microstructure characteristics of both pure and modified bitumen samples were assessed using Hitachi S-4700 Field Emission Scan

Electron Microscope (FE-SEM). The Hitachi S-4700 FE-SEM produces cold field emission and high-resolution micrographic pictures. Bitumen specimens were flash frozen using liquid nitrogen at a temperature of  $-26^{\circ}\text{C}$  and at 30Pa pressure. Micrographic images were taken using cryogenic stage 15-kV electron beam at 2000x desired magnifications. These are shown Plates VI and VII.

## CHAPTER FOUR

### RESULT AND DISCUSSION

#### 4.1 Tests on Mix Materials

##### (a) Bitumen test

The test conducted on unmodified 60/70 penetration bitumen used satisfied various codes requirements for ductility, penetration, softening point, specific gravity, solubility, flash and fire point tests. The tests conducted on unmodified bitumen and HDPP modified 60/70 penetration bitumen used satisfied various codes requirements for ductility, penetration, softening point, specific gravity, solubility and flash and fire point tests. The various code limits are indicated in Table 4.1 showing that the quality and grade of the 60/70 penetration grade bitumen used is good and adequate.

**Table 4.1: Test parameters and Code Limit of Unmodified bitumen (Control)**

Test Conducted	ASTM Code	Test Result	Code Limit
Penetration at 25 °C, 0.1mm	ASTM D5-97	67.7	60-70
Penetration Index (PI)	ASTM D5-97	-0.338	-2 to +2
Softening point (°C)	ASTM D36-95	50.5	46-56
Flash point (Cleveland open cup), °C	ASTM D92-02	295.2	Min. 232
Fire point Cleveland cup), °C	ASTM D92-02	306.5	Min. 232
Ductility at 25°c, cm	ASTM D113-07	122.4	Min. 50
Specific gravity at 25°C, (g/cc)	ASTM D70-03	1.022	0.97 -1.02
Solubility in trichloroethylene,%	ASTM D2042-97	99.02	Min. 99

The factors that can affect ageing of bitumen are temperature, exposure to oxygen, UV light, chemical components and bitumen structure (Tahirou, 2009). As the consequences of ageing, volatilization followed by oxidation and minor cases of polymerization impart on short and long term ageing of the bitumen binder during its service. Table 4.2 shows that TFOT are within limits specified by the various codes and could be adjudged good to stand the test of ageing and durability.

**Table 4.2: Ageing TFOT result and Code Limit of unmodified bitumen (0% HDPP)**

<b>Test Conducted</b>	<b>ASTM Code</b>	<b>Test Result</b>	<b>Code Limit</b>
Ductility at 25°c, cm	ASTM D113-07	59.2	Min. 50
Retained penetration (% of original)	ASTM D5-97	78.3	Min. 54
Loss on heating (% by mass)	ASTM D6-95	0.23	Max. 0.5

**(b) Aggregates test**

Strength and specific tests carried on the mineral aggregates showed tough, dense and abrasion resistant materials that can stand the test serviceability and durability requirements of asphalt mixtures within the designed life. For instance, aggregate crushing value result in Table 4.3 is 22.8% while code limits the value to maximum of 25%. The various test results and their comparisons to codes, indicates that the materials meet specification requirements for quality aggregate materials.

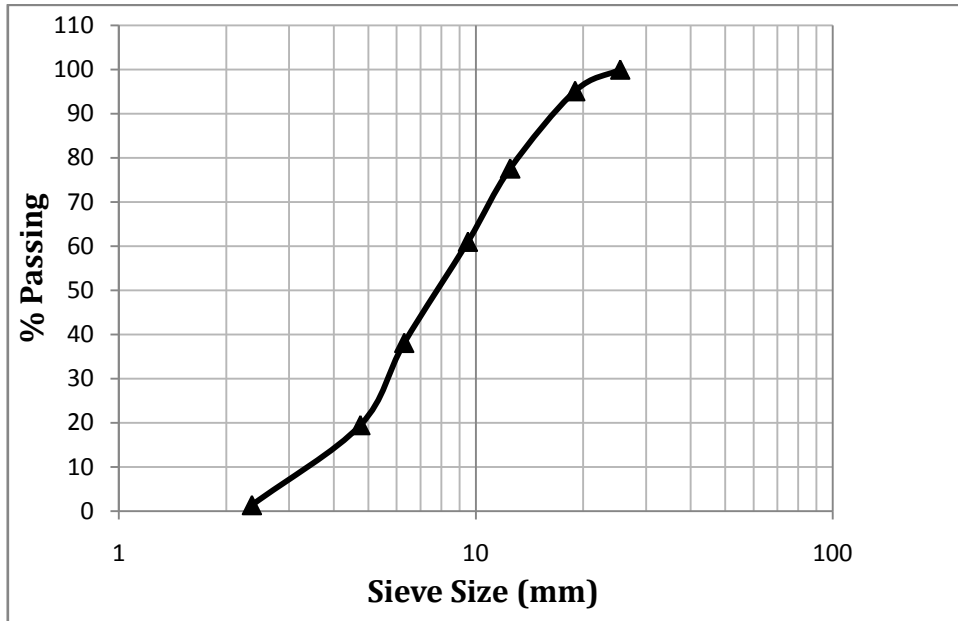
**Table 4.3: Result of Test on Aggregate materials**

<b>Test Conducted</b>	<b>Code Used</b>	<b>Test Result</b>	<b>Code Limit</b>
Aggregate Crushing Value (%)	BS 812:112-90	22.8	Max. 25
Aggregate Impact Value (%)	BS 812:111-90	16.3	Max. 25
Aggregate Los Angeles Abrasion Value (%)	ASTM C131-06	18.9	Max. 30
Specific Gravity (Coarse Aggregate) ( $G_c$ ) g/cc	ASTM C12701	2.70	2.55 – 2.75
Aggregate Moisture Absorption (%)	BS 812:105.2-90	1.4	Max. 2
Coarse Aggregate Flakiness Index	BS 812:105.1-90	26	□ 35
Specific Gravity (Fine Aggregate) ( $G_f$ ) g/cc	ASTM C128-15	2.63	2.55 – 2.75
Bulk Specific Gravity of Total Aggregate ( $G_{sb}$ ), g/cc	ASTM C127-01	2.71	2.6 - 2.7

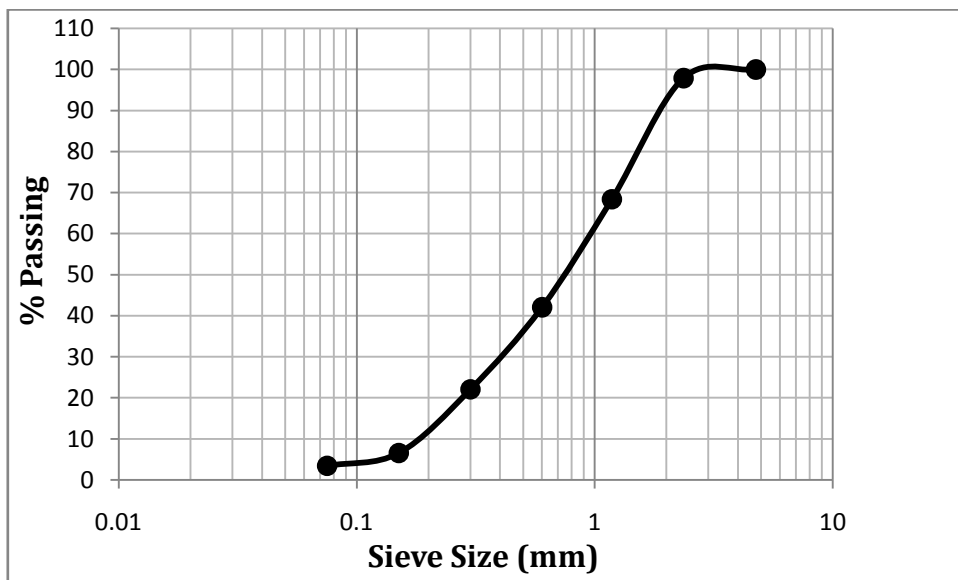
**(c) Aggregate Particle Size Distribution**

Good aggregate grading imparts on its density of mix packing and aggregate interlocking. The results of coarse and fine aggregates grading are shown in Figures 4.1 and 4.2 respectively. Depending on the nature of the grading envelopes formed after coarse and fine aggregates and fillers are combined, a combined grading curve embedded in the middle of

grading specification range (Figure 4.4) is adjudged a good mix. The details of aggregates grading are shown in Appendices A1 and A2.



**Figure 4.1: Coarse aggregate particle size distribution**



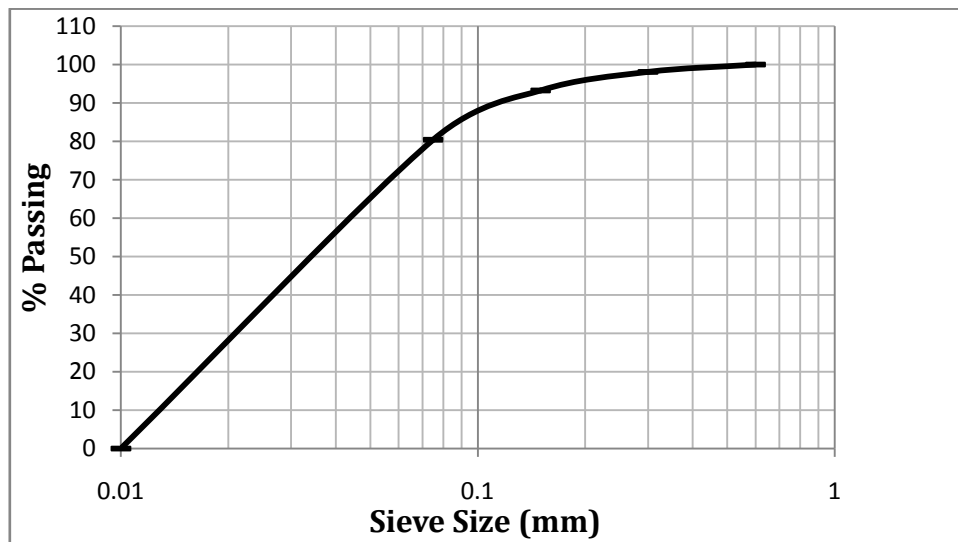
**Figure 4.2: Fine aggregate particle size distribution**

#### (d) Cement Fillers

The specific gravity, setting time and soundness tests conducted on cement fillers show that it meets code requirements with good pozzolanic properties. The setting times and soundness within the code limits are indication of good strength and the specific gravity imparted on the density of the mix. Table 4.4 shows the test results and code limits. Figure 4.3 shows the grading of the cement fillers and the details is contained in Appendix A3.

**Table 4.4: Result of Test on Cement Filler**

Test Conducted	Code Used	Test Result	Code Limit
Specific gravity	ASTM C188-15	3.15	Min 3.15
Initial Setting time (minutes)	BS EN 196:3-05	98	Min. 45
Final Setting time (minutes)	BS EN 196:3-05	230	Max 375
Soundness (mm)	BS EN 196:3-05	3.5	Max. 10



**Figure 4.3: Cement fillers particle size distribution**

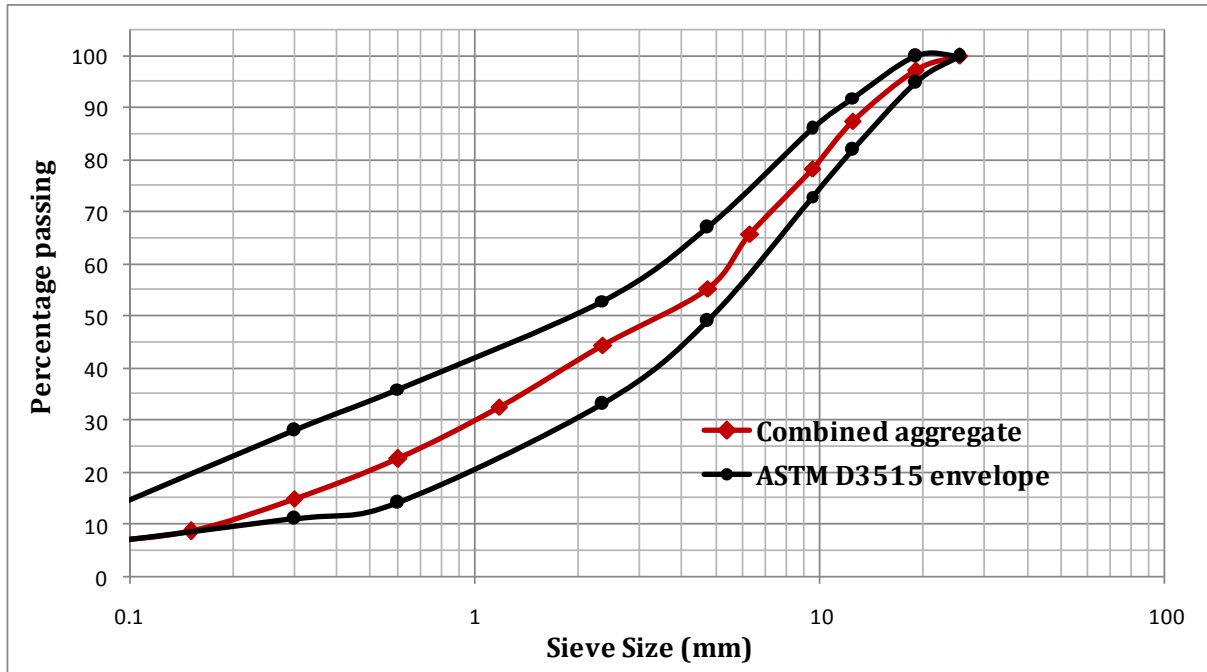
#### (e) Mix Proportioning and Blending

Proportioned coarse and fine aggregates and fillers were blended and compared to code requirements for good grading as shown in Table 4.5. The plot in Figure 4.4 shows the

combined aggregate envelope and the distribution of various materials. It shows the blending of the materials which has been sandwiched between the middle of the envelope and range of passing sieves sizes specified by ASTM D3515 (2001). Grading following the middle course exhibits better voids and density; the upper and upper limits of the grading envelop may affect void and density of the resulting mix (ASTM D3515:2001).

**Table 4.5: Combined Material Mix and Range of Specification Requirements**

<b>Sieve (mm)</b>	<b>Size</b>	<b>Percentage Retained</b>	<b>Cumulative Percentage Retained</b>	<b>Cumulative Percentage Passing</b>	<b>Range of Percentage Passing (ASTM D3515-01)</b>
25.00		-	-	100	100
19.00		2.7	2.7	97.3	95 – 100
12.50		9.7	12.4	87.6	82 – 92
9.50		9.2	21.6	78.4	73 – 86
6.30		12.7	34.3	65.7	-
4.75		10.3	44.6	55.4	49 – 67
2.36		10.9	55.5	44.5	33 – 53
1.18		12.0	67.5	32.5	-
0.60		10.0	77.5	22.5	14 – 36
0.30		7.7	85.2	14.8	11 – 28
0.15		6.2	91.4	8.6	-
0.075		2.1	93.5	6.5	6 – 11
Pan		6.5	100	-	-



**Figure 4.4: Combined aggregates particle size distribution**

## 4.2 Modified Bitumen

The result of ductility, penetration, softening point, specific gravity, solubility and flash and fire point tests conducted on HDPP modified 60/70 penetration bitumen met the various codes requirements for good control of quality of mix. Tables 4.6 and 4.7 are the results showing code limits and TFOT ageing assessment respectively.

As the HDPP content increased from 0.5-3%, penetration decrease due to increasing stiffness, conversely, elasticity is reduced thereby reducing ductility. With increasing HDPP content the softening point increased because of temperature resistivity of polymer interaction in the matrix. Flash and fire point is increased because of temperature resistivity of HDPP, but the densities are fairly the same. These findings agree with Giuliani *et al.* (2009) that polymer increases temperature resistivity of asphalt.

With respect to ageing, increasing HDPP content reduced retained ductility and penetration because of reduction in viscosity and increase in solid elasticity. From 0.5% to 3.0%, loss on heating increased because increased carbonation of increasing HDPP content.

From TFOT conducted to simulate short term ageing, 0.5 to 2.5% HDPP modified bitumen did not have considerable loss of volatiles which agrees with Zoorob, *et al.* (2002), but above 2.5% HDPP content led to changes in physical and rheological characteristics exhibited by loss of ductility, elasticity and retained penetration. Considering the recommendations of penetration grades bitumen (Oversea Road Note 19, 2002), an optimum HDPP content of 2.0% meets TFOT requirements for retained penetration, ductility and loss in mass.

**Table 4.6: Consistency and Safety Tests of Modified Bitumen**

<b>% HDPP</b>	<b>0.5%</b>	<b>1.0%</b>	<b>1.5%</b>	<b>2.0%</b>	<b>2.5%</b>	<b>3.0%</b>	<b>ASTM Code</b>	<b>Code Limit</b>
<b>Test</b>								
Penetration at 25 °C, 0.1mm	62.9	53.8	45.2	38.5	30.9	22.5	ASTM D5-97	60-70
Penetration Index (PI)	+0.24	+0.22	0.53	+1.52	+1.88	2.15	ASTM D5-97	-2 to +2
Softening point (°C)	53.6	55.3	58.7	66.0	71.3	77.8	ASTM D36-95	46-56
Flash point (Cleveland cup), °C	311.4	317.0	322.6	330.9	335.6	342.5	ASTM D92-02	Min. 232
Fire point (Cleveland cup), °C	317.2	326.8	334.7	341.5	349.6	358.4	ASTM D92-02	Min. 232
Ductility at 25°C, cm	107.2	88.6	72.4	54.3	42.8	22.6	ASTM D113	Min. 50
Specific gravity at 25°C (g/cc)	1.015	1.011	1.008	1.005	1.003	1.001	ASTM D70	0.97 - 1.02

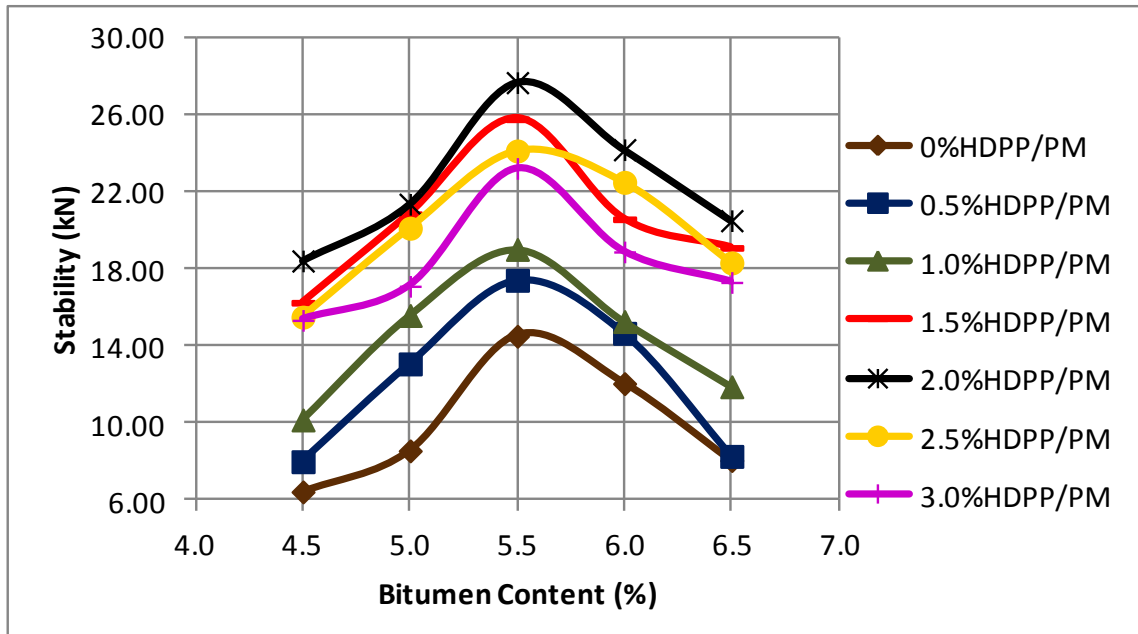
**Table 4.7: Properties of residue for TFOT Ageing Test on Modified Bitumen**

<b>Test</b>	<b>% HDPP</b>	<b>0.5%</b>	<b>1.0%</b>	<b>1.5%</b>	<b>2.0%</b>	<b>2.5%</b>	<b>3.0%</b>	<b>ASTM Code</b>	<b>Code Limit</b>
Retained Ductility at 25°c, cm		56.4	54.0	52.5	50.5	48.7	46.5	ASTM D113	Min. 50
Retained penetration (% of original)		72.6	65.9	59.4	56.0	50.2	44.8	ASTM D5	Min. 54
Loss on heating (% by mass)		0.25	0.38	0.41	0.42	0.46	0.59	ASTM D6-95	Max. 0.5

### 4.3 Marshall Test for HDPP Asphalt Mixes

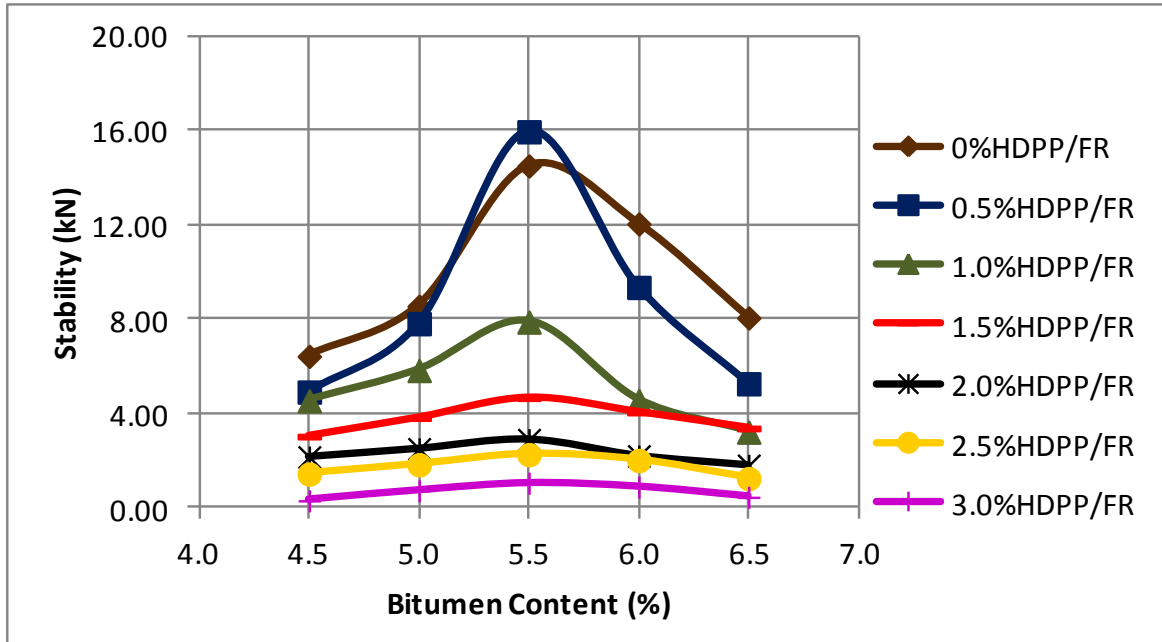
The summary of observations made for stability, flow and volumetric analysis (VIM, VMA and VFB) test evaluated in both wet process (polymer modified bitumen) and dry process (fibre reinforced) asphalt mixes (details are in Appendices B1 to B14). The following results were obtained:

- (a) The values of stability shown in Figure 4.5 steadily increased reaching the optimum at 2.0% HDPP content. The optimum stability for wet process is 27.68kN and this occurred at 2.0% HDPP content. The increment accounted for an improvement of 90.63% in strength compared to 0% HDPP (control), agrees with the work of Serkan *et al.* (2005). The initial increase in stability was as a result of increase in surface tension which coagulates the loosely bonded molecular structure of bitumen. Increased cohesion leads to increased stability, stiffness and strength (Aljassar *et al.*, 2004; Grabowski and Wilanowicz, 2008). Beyond the optimum, the polymer bitumen becomes more plastic than viscoelastic. The optimum bitumen content is 5.5%.



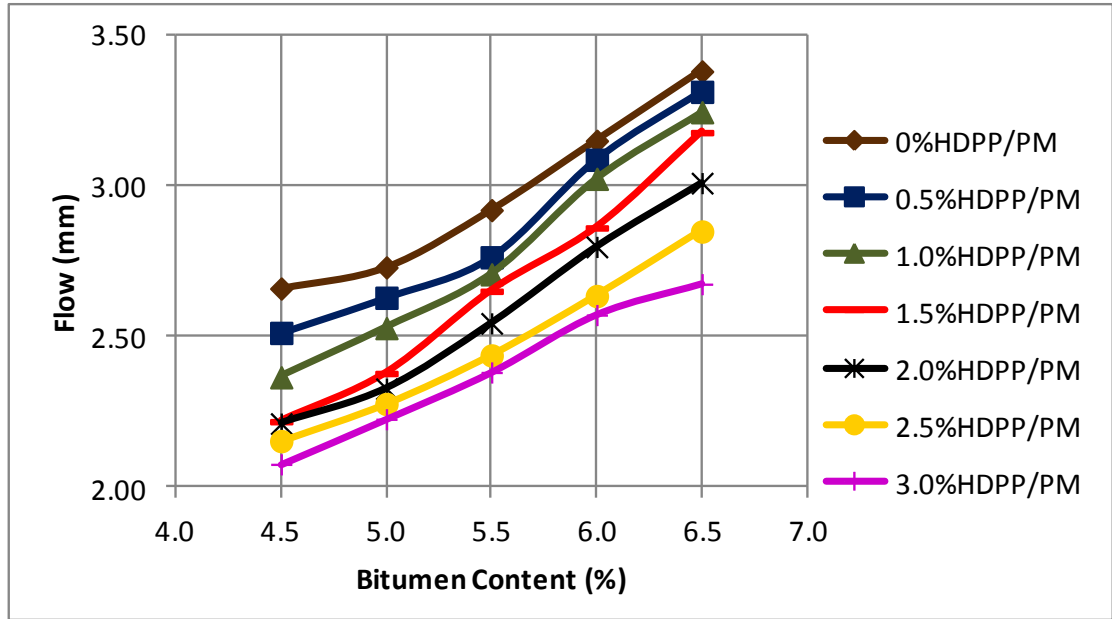
**Figures 4.5: Relationship between Stability and Bitumen Content in HMA Wet Process**

For dry process, stability increased from 0 to 0.5% HDPP contents, the optimum being 15.97kN at 0.5% HDPP as shown in Figure 4.6. The value accounts for 9.99% increased in strength which was compensated by tensile strain suffered when asphalt specimens were loaded. Increasing HDPP content beyond the optimum threshold drastically increased the void and deformation and reduces the overall strength of the mix. The findings agreed with Rahman and Wahab (2009); Cleven (2000); and Chen and Lin (2005) who observed increase in strength and higher fracture energy of PP fibre reinforced mix.



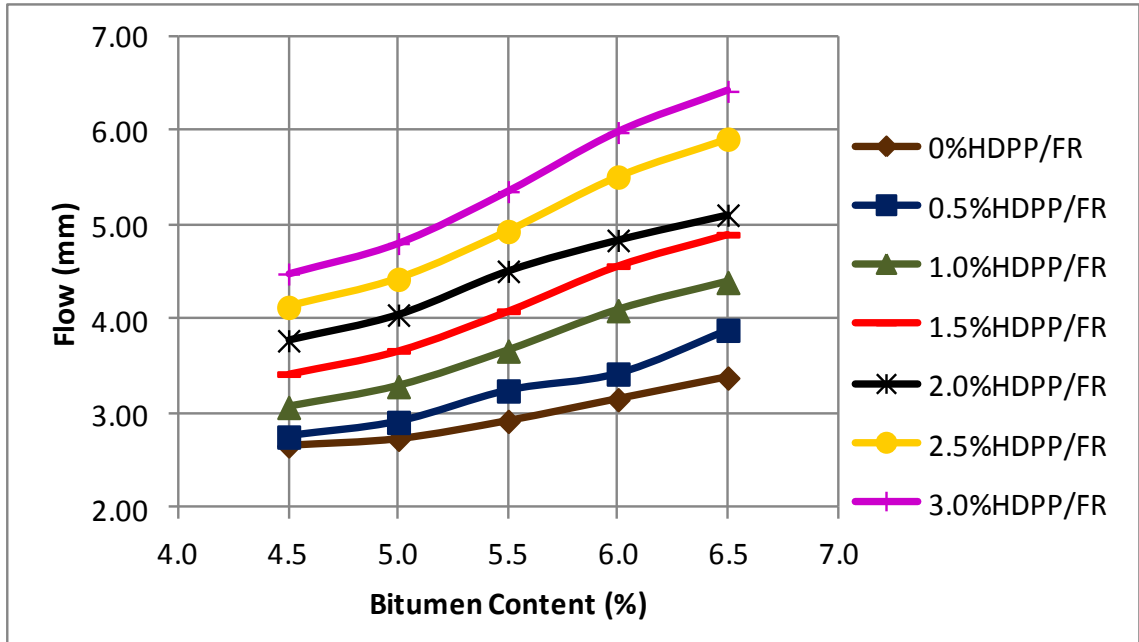
**Figures 4.6: Relationship between Stability and Bitumen Content in HMA Dry Process**

(b) The trends of flow deformations for wet process are shown in Figure 4.7. The value of flow at optimum bitumen content of 5.5% decreased from 2.8mm to 2.3mm respectively for increasing HDPP contents of 0 to 3.0%. This may be because of increasing interfacial tension created by surface tension and dispersion of bitumen and polymer colloids that leads to increasing unit cells formation called micelles (Rosen and Kunjappu, 2012). This trend lowers deformation as micelle units galvanises the strength of the intermolecular bonds in the asphalt matrix, even at increasing temperature it defines its deformation resilience that lowers creep and rutting (Lesueur, 2009). When HDPP is high, the mixture becomes phase inverted leading to flocculation and destabilisation (Kamaruddin, *et al.*, 2012). At this stage, there could be phase separation and transition from visco-elastic phase to plastic phase because of higher glass transition that causes premature cracking.



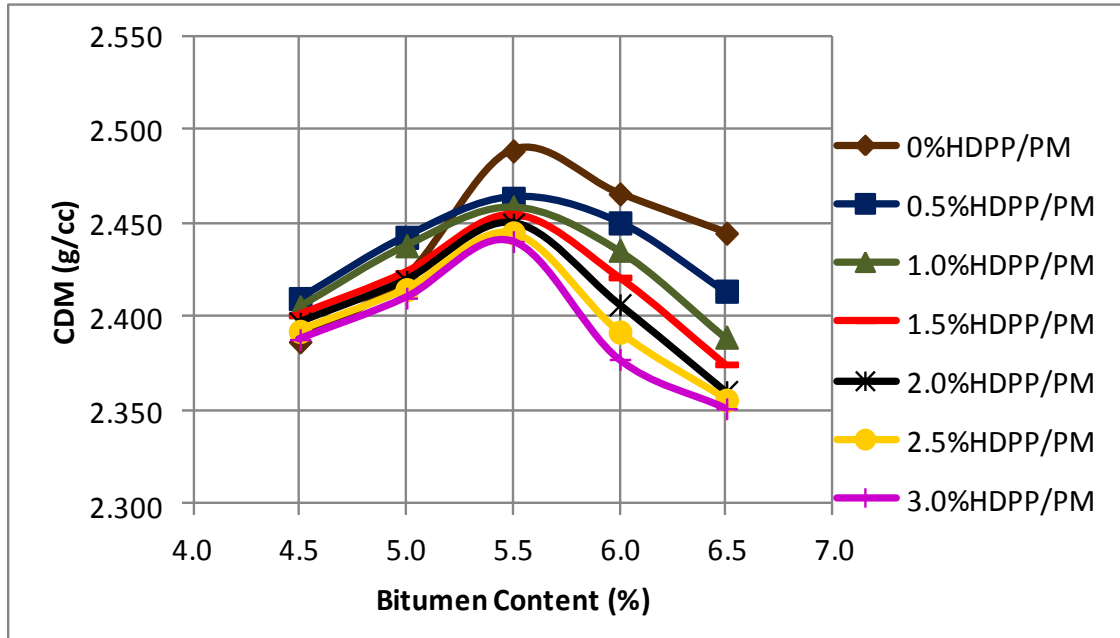
**Figures 4.7: Relationship between Flow and Bitumen Content in HMA Wet Process**

The flow values for dry mix process generally increased as HDPP increased 0 to 3.0% because of asphalt- HDPP composite formation. At the optimum bitumen content of 5.5%, the flow values for 0, 0.5 and 1.0% HDPP are 2.9mm, 3.3mm and 3.7mm respectively as shown in Figure 4.8. These values meet Marshall recommendation of 2mm to 4mm for stable mix (Asphalt Institute, 1997). For HDPP contents higher than 2.0%, flow values went out of Marshall range because of low deformation resistance and excessive rutting.



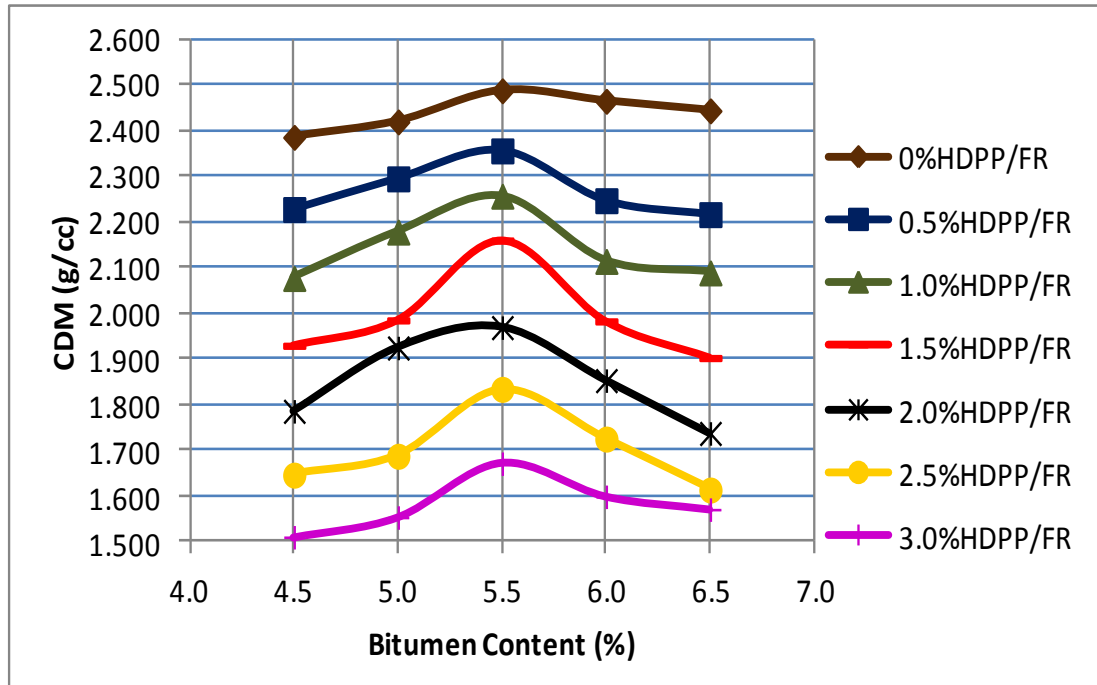
**Figures 4.8: Relationship between Flow and Bitumen Content in HMA Dry Process**

(c) The compacted density of the mix (CDM) or bulk specific gravity (BSG) for wet process decreased as HDPP increased as shown in Figure 4.9. This is because incorporating HDPP which is less dense compared to the other components of the mix lowers the unit weight of the compacted asphalt mix irrespective of the percentage voids attained. The optimum bitumen content is 5.5%.



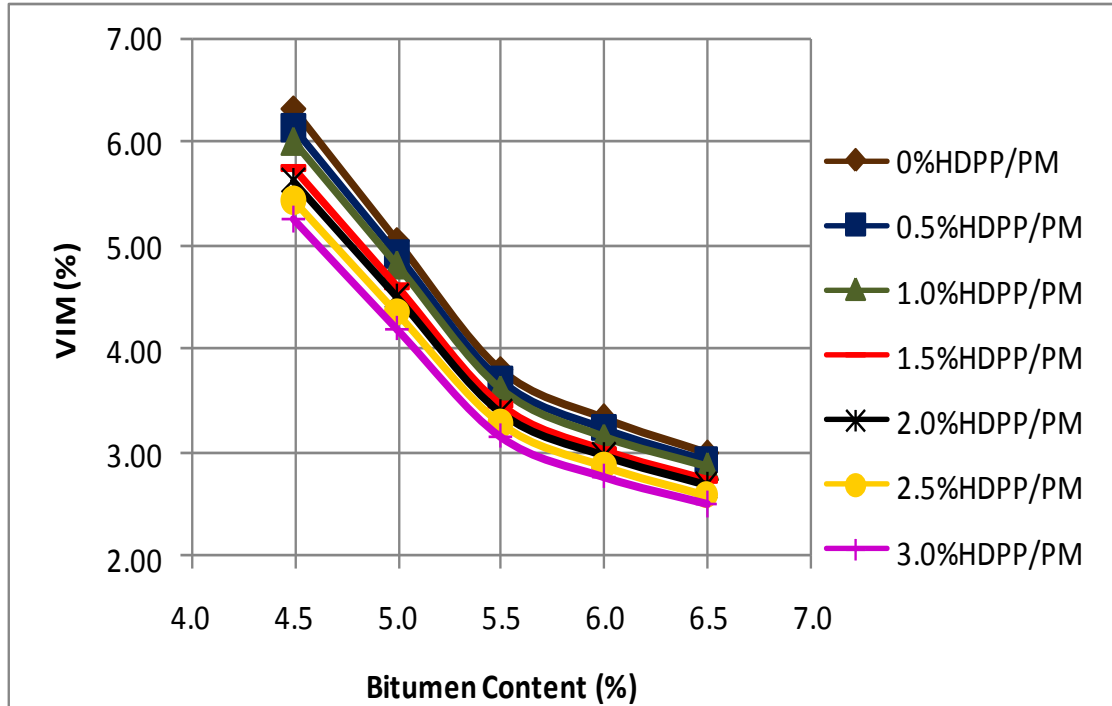
**Figures 4.9: Relationship between CDM and Bitumen Content in HMA Wet Process**

Generally, CDM of dry mix process increased to the optimum as bitumen content increased and then, dropped because of increasing bitumen content as shown in Figure 4.10. There is a decreasing trend in CDM as HDPP increased from 0 to 3.0% because of inadequate aggregates surface contact to HDPP, thereby entraining voids in the mix. HDPP has lower specific gravity than mineral aggregates and therefore, lowers the density of the compacted asphalt mix. The optimum bitumen content is 5.5%.



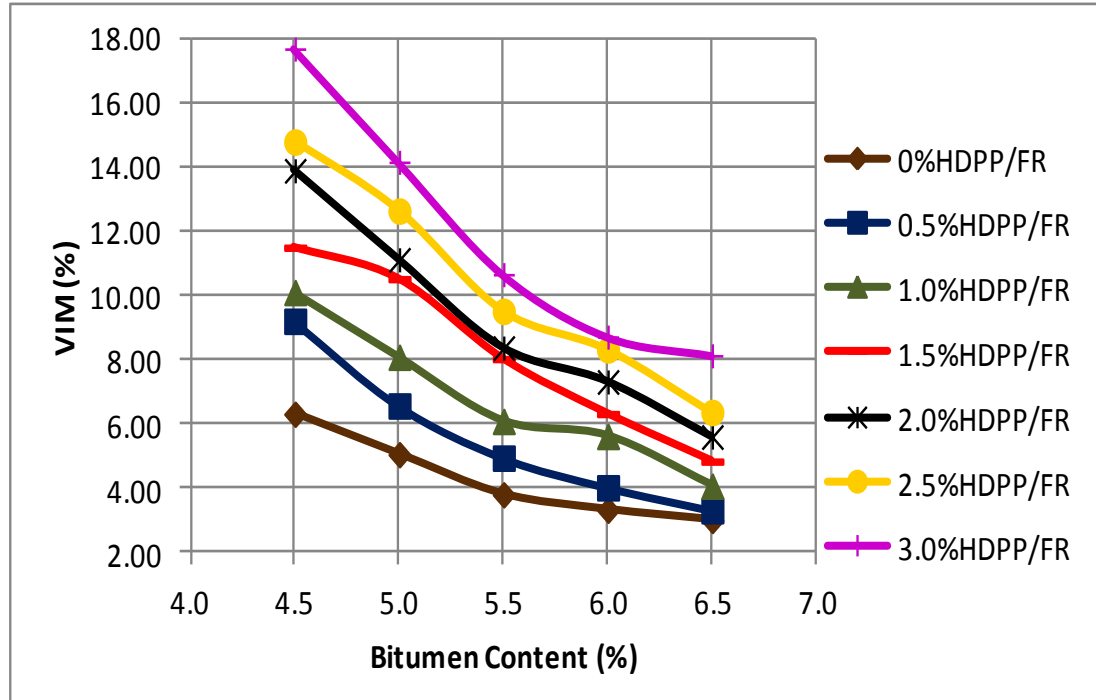
**Figures 4.10: Relationship between CDM and Bitumen Content in HMA Dry Process**

(d) The range of specification requirement of Asphalt Institute (1997) for VIM is 3 – 5%; the average (that is, 4.0% VIM) is usually preferred. Figure 4.11 shows the plots of VIM for wet process. The lower the bitumen content, the higher the voids entrainment leading to higher VIM. Moderate VIM leads to increased toughness, high fracture energy and low risk of voids entrained in the mix. Higher VIM values beyond the upper range 5% causes oxidation of bitumen and results to quick ageing and loss of durability, while lower than 3.0% VIM causes fatigue cracking (Oversea Road Note 19, 2002).



**Figures 4.11: Relationship between VIM and Bitumen Content in HMA Wet Process**

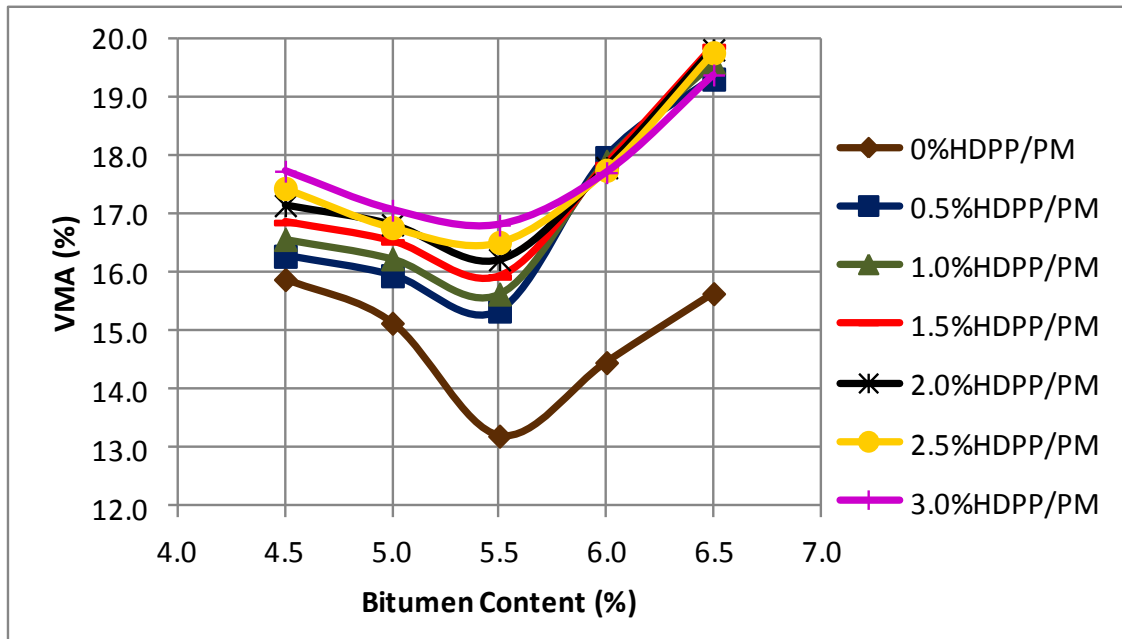
For dry process, the design VIM of 5.0% was used because of increasing voids in the compacted HDPP reinforced asphalt composite and that gave optimum bitumen content of 5.5%. From Figure 4.12, as HDPP content increased from 0 to 3.0%, 0 to 0.5% HDPP were accommodated in the range of 3.0 to 5.0% VIM content recommended by Asphalt Institute (1997) for a stable mix. VIM excessively increased for values of HDPP above 0.5% because of entrainment of voids, otherwise, the bitumen content has to be increased higher than the optimum of 5.5%.



**Figures 4.12: Relationship between VIM and Bitumen Content in HMA Dry Process**

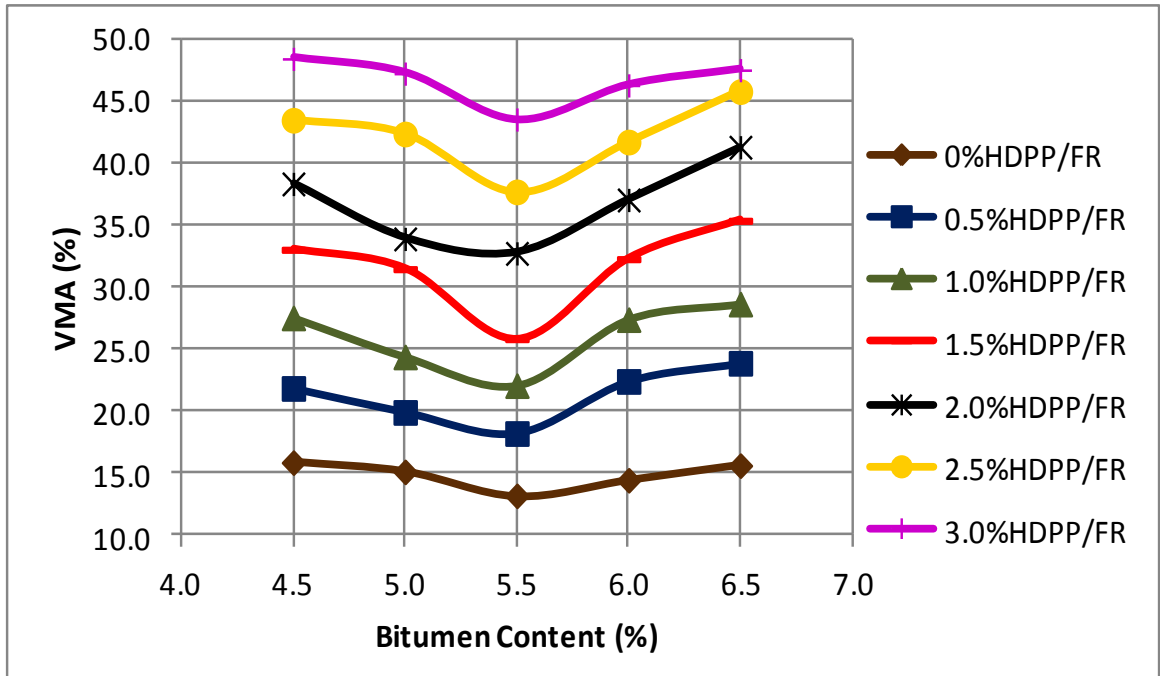
(e) Figure 4.13 shows the plot of VMA for wet process. Initially, as bitumen content increases, the value of VMA generally decreases down to a plot when increased filling of void might not translate to increased density. This is otherwise called ‘refusal density’. The optimum VMA occurs at the minimum point around the optimum bitumen content of 5.5%. Beyond this ‘refusal density’, a further increase in bitumen content increases VMA as a consequent of aggregate structure becoming overfilled with bitumen leading to deformation (ORN 19, 2002). A minimum VMA of 12.0% at an average of 4.0% VIM is recommended by Marshall criteria for nominal aggregate size of 1” (25mm) (Asphalt Institute, 1997). For all HDPP contents of 0 to 3.0%, optimum VMA increased. These values meet the recommendation of Asphalt Institute (1997). VMA increased because the film thickness of coated aggregates and effective bitumen

content that does not include absorbed bitumen increased. The optimum bitumen content is 5.5% at 2.0%HDPP content.



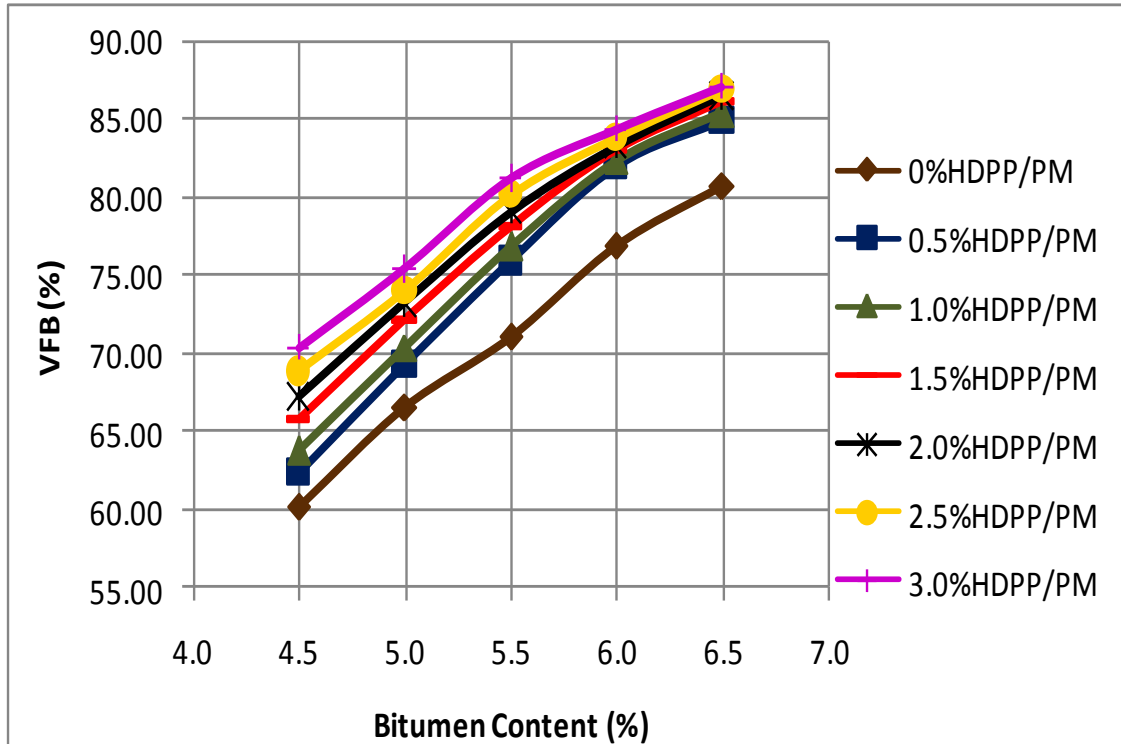
**Figures 4.13: Relationship between VMA and Bitumen Content in HMA Wet Process**

Figure 4.14 shows the plot of VMA for dry mix process. Using the optimum bitumen content of 5.5%, VMA for 0 to 3% HDPP fibre concrete meets the minimum requirement of 12.0% VMA recommended by Asphalt Institute (1997) assuaging problem of quick traffic densification that may lead to fatigue failure and pavement distress.



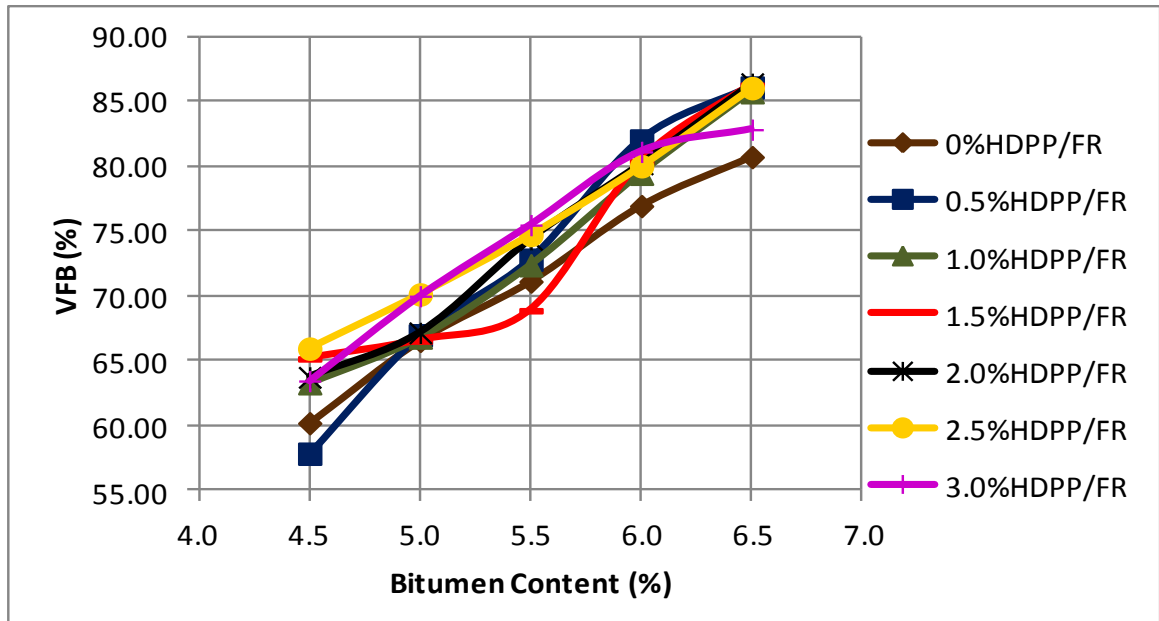
**Figures 4.14: Relationship between VMA and Bitumen Content in HMA Dry Process**

(f) Marshall recommendations for range of acceptable VFB for heavy traffic 65% to 75% (Asphalt Institute, 1997). Figure 4.15 shows the result VFB for wet process. At optimum bitumen content of 5.5%, the VFB for 0, 0.5, 1.0, 1.5 and 2.0% HDPP are respectively 67.5, 71.0, 72.0, 74.0 and 75.0%; and are within the range recommendation of 65% to 75% VFB (Asphalt Institute, 1997). The values of VFB above 2.0% HDPP are outside the range.



**Figures 4.15: Relationship between VFB and Bitumen Content in HMA Wet Process**

Finally, the VFB for 0 to 3.0% HDPP dry process mixes are plotted in Figure 4.16 below. Considering optimum bitumen content of 5.5%, the VFB values for 0 to 3% HDPP are all within the range of 65% – 75% recommended by Marshall criteria for heavy traffic axle loading of  $10^6$  ESAL.



**Figures 4.16: Relationship between VFB and Bitumen Content in HMA Dry Process**

#### 4.4 Simple Performance Tests

##### (a) Dynamic Modulus Test

##### (i) Absolute dynamic modulus, $|E^*|$

This quantifies the stiffness response of asphalt mixture by applying cyclic loading of a known magnitude on a specimen and measuring its deformation (Rais *et al.*, 2013). Length of loading pulse usually taken as time in seconds varies according to vehicular speed. The slowest speed of 40.2336 km/hr (25mph) has the longest pulse and highest magnitude of strain. AASHTO TP79 (2013) recommended loading pulse of 10Hz as highway speed equivalent.

Master curves were constructed using time-temperature superposition (TTS) following Arrhenius shift factor law to assess the relationships between the physical quantities and mechanical properties - dynamic modulus and phase angle (AASHTO PP61:13-2013).

Asphalt mixtures exhibit more stiff and increased elastic behaviour at low temperature and high loading frequency. The contrast of high temperature and low frequency of strain exhibits prevalent viscous property. As the relationship between stress and strain becomes linear due to effects of temperature and frequency or time, the viscoelastic property of asphalt becomes almost linear. These parameters, according to Brown (2000), concluded acts as indicators of failure since dynamic modulus is time, temperature and trafficking frequency rate are dependent.

The onset of serious deterioration begins at a point called ‘critical condition’ of the pavement where 50% of initial stiffness is lost and critical point might correspond to 10mm rut depth whereby strengthening of the pavement becomes necessary (Mahan, 2013).

Figures 4.17 to 4.20 show wet process TTS master curves for dynamic modulus,  $|E^*|$ . The dynamic modulus,  $|E^*|$  values increased from 0 to 2% HDPP content for the wet process because of increasing stiffness of mix for all conditions of temperature and frequency used. It shows that dynamic modulus is sensitive to viscosity or penetration of bitumen which reduced with increasing polymer content. At this range of 0 to 2% HDPP wet process;  $|E^*|$  results meet the recommendable minimum of 1,500MPa by Hasim *et al.* (1994) but only at a temperature of 60<sup>0</sup>C and lower frequencies of 1 to 0.1Hz, values of  $|E^*|$  fell below 1,500MPa minimum and could perform poorly because of increasing temperature. Generally, good correlations were observed between the laboratory results and field performances that were replicated (Pellinen and Witczak, 2002; Kim *et al.*, 2004; Garcia and Thompson, 2007).

At 3.0 % HDPP wet process (Figure 4.20), the values of  $|E^*|$  decreased because more polymer phases in the bitumen matrix initiates more elastic response than viscous property

at low temperature ranging from 4.4 to 21.1<sup>0</sup>C, but increases in temperature from 25 to 60<sup>0</sup>C increased viscoelastic and rheological performance of the mix. This viscoelastic relationship agrees with the findings of Lu, *et al.* (2011).

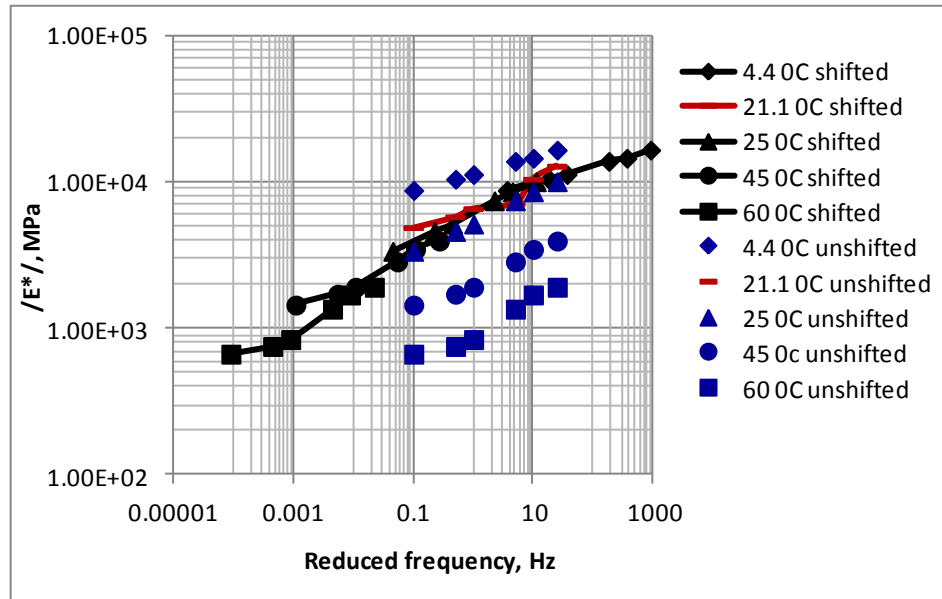


Figure 4.17: Dynamic modulus master curve for 0% HDPP wet mix process

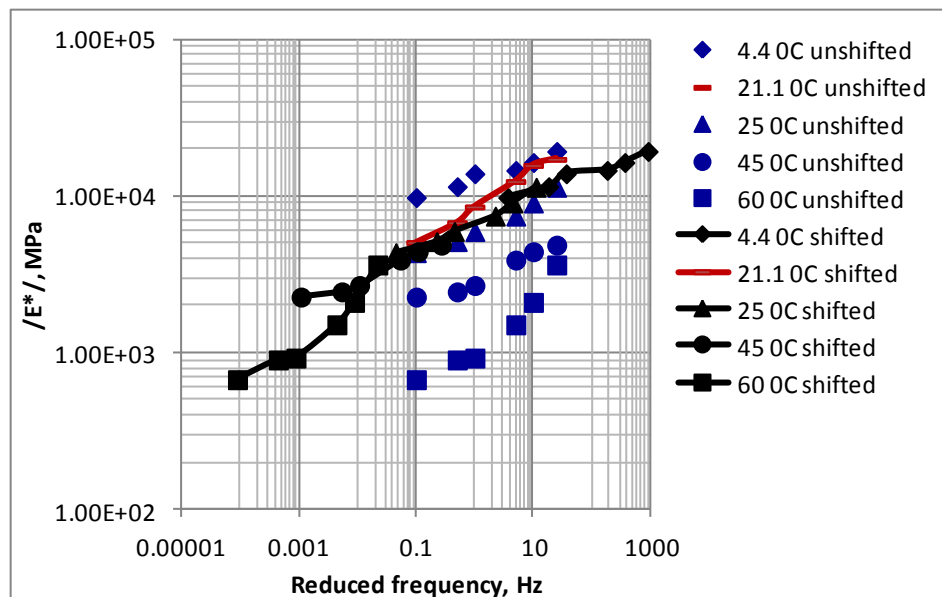


Figure 4.18: Dynamic modulus master curve for 1% HDPP wet mix process

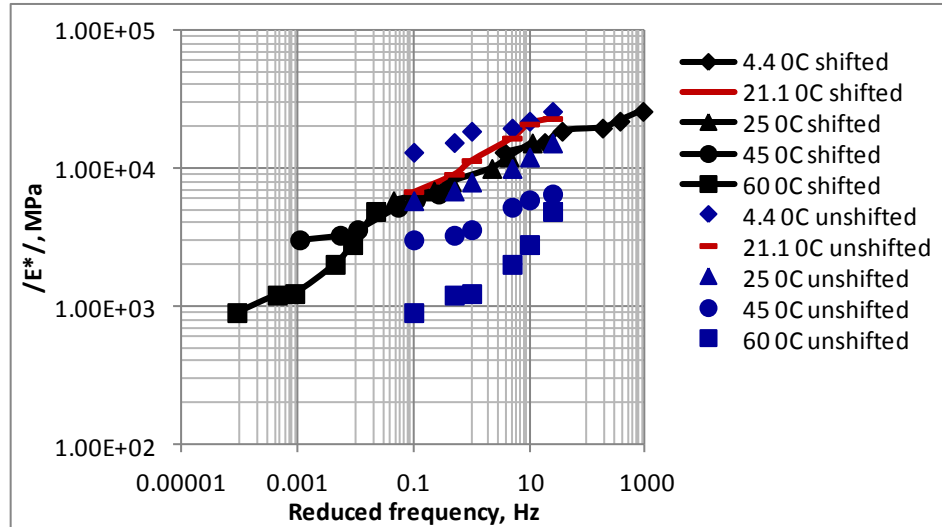


Figure 4.19: Dynamic modulus master curve for 2% HDPP wet mix process

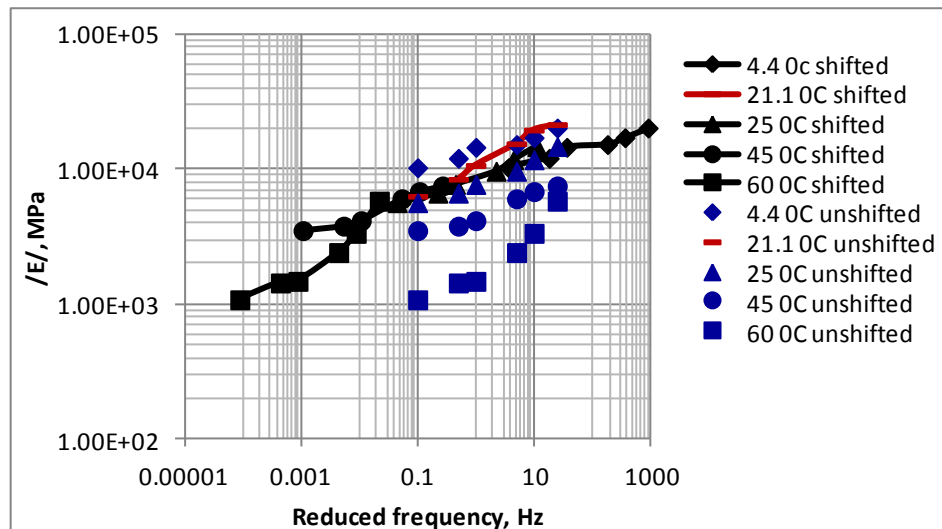


Figure 4.20: Dynamic modulus master curve for 0% HDPP wet mix process

For the  $|E^*|$  of dry process in Figure 4.21 to 4.23, the value of  $|E^*|$  increased from 0 to 0.5% HDPP because of bearable in void and improvement in the tensile property offered by

fibre reinforcement. From 1.0 to 1.5% HDPP, the values of  $|E^*|$  continue to drop as a result of increased void in the mix.

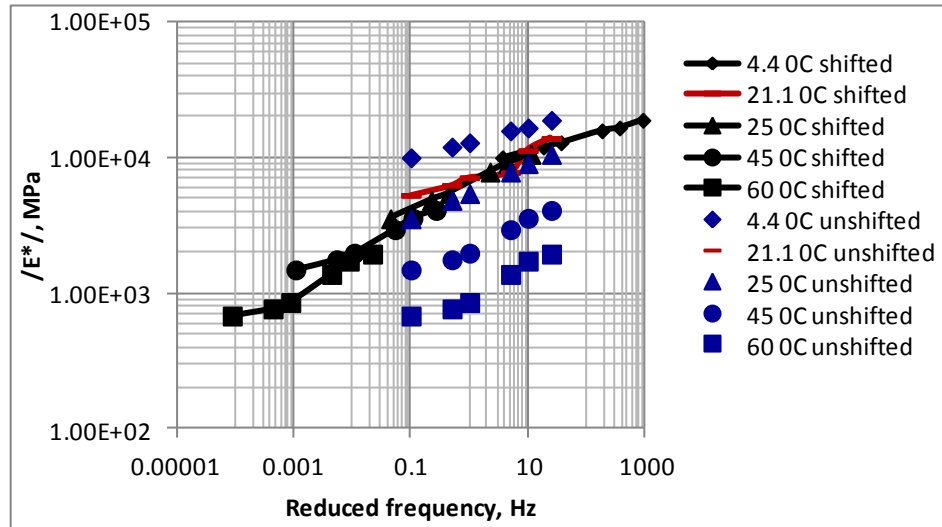


Figure 4.21: Dynamic modulus master curve for 0.5% HDPP dry mix process

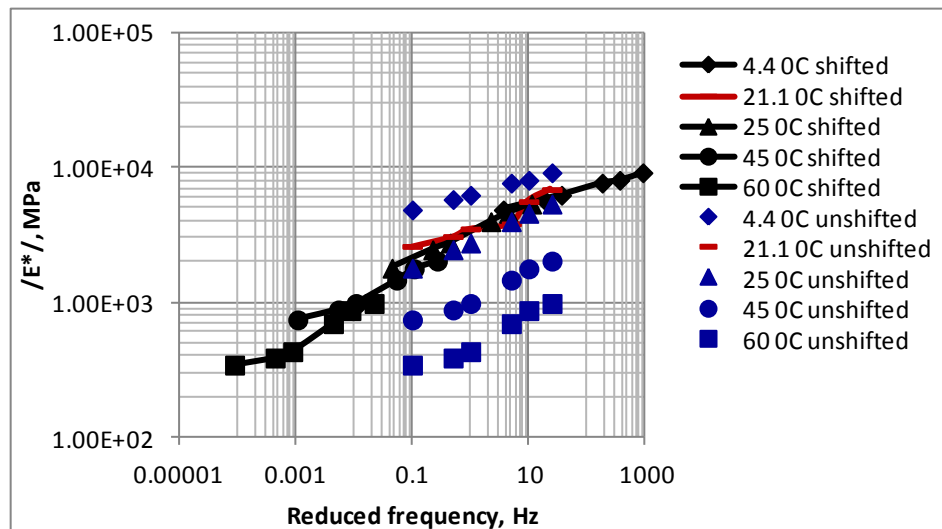
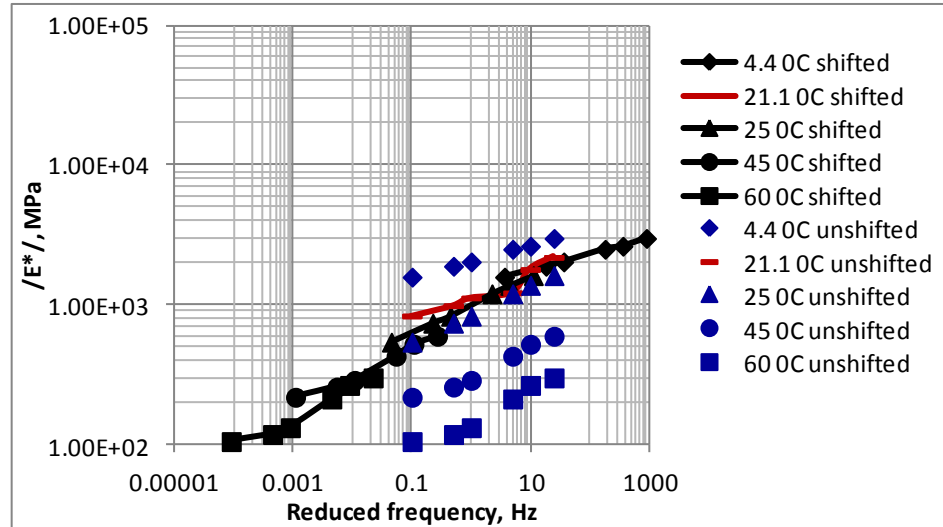


Figure 4.22: Dynamic modulus master curve for 1.0% HDPP dry mix process

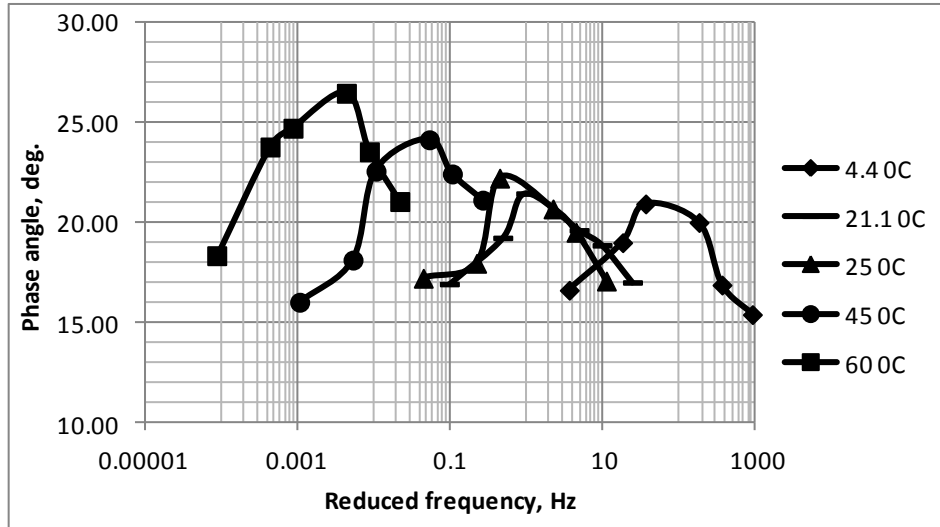


**Figure 4.23: Dynamic modulus master curve for 1.5% HDPP dry mix process**

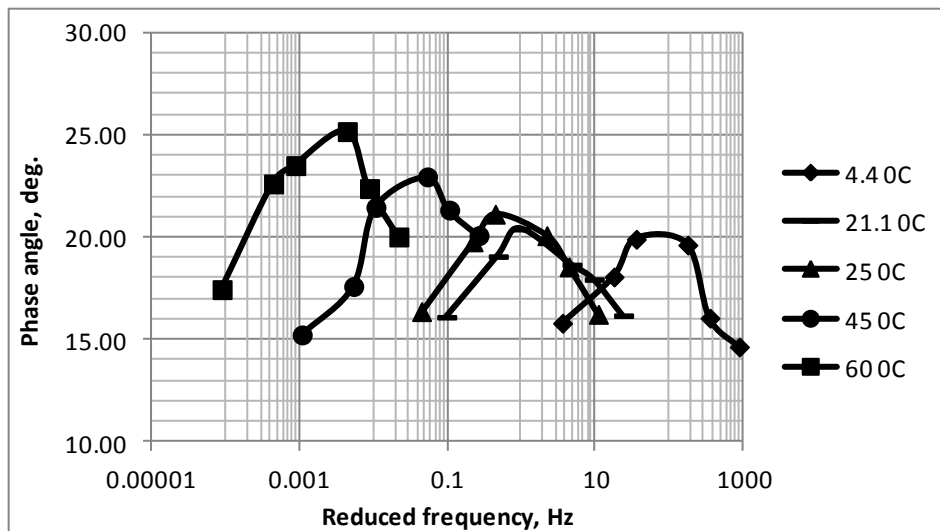
**(ii) Phase angle,  $\phi$**

Phase angle, theoretically, is the lag between strain and stress. It is a sign of the magnitude of loss modulus value and the degree of peak angle is indicative of pavement distress (Rais *et al.*, 2013). Figures 4.24 to 4.27 show the phase angle for wet process. From 0 to 2% HDPP, the phase angle,  $\phi$ , presents downward trends as a result of increasing elastic properties than viscous properties and reduction in deformation with respect to applied stress. The general trend of the result of  $\phi$  increases with rise in temperature. But the values of  $\phi$  also increases to a peak value before declining as frequencies reduced from 25 to 0.1Hz.

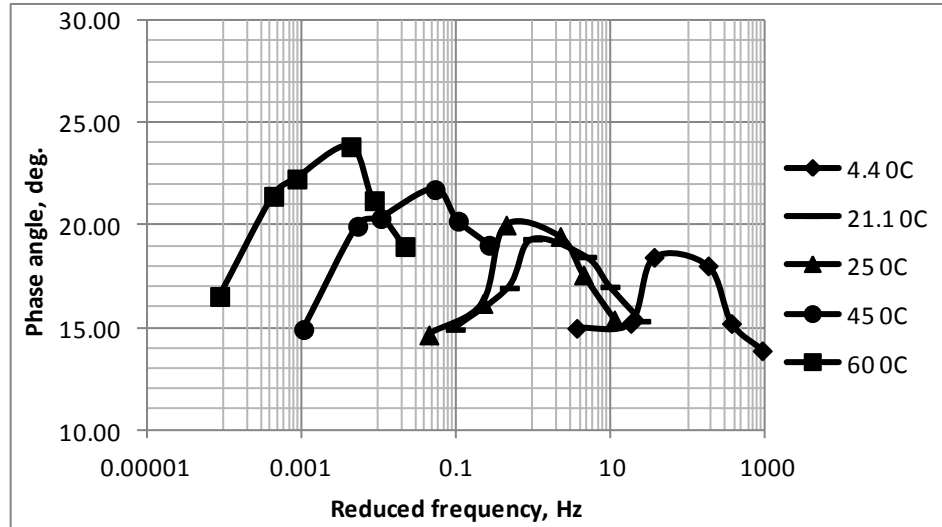
The downward trends of  $\phi$  for 3.0 HDPP polymer asphalt were results of reduction in deformation initiated by plastic nature of more HDPP in bitumen matrix which do not translate into strength of the mix.



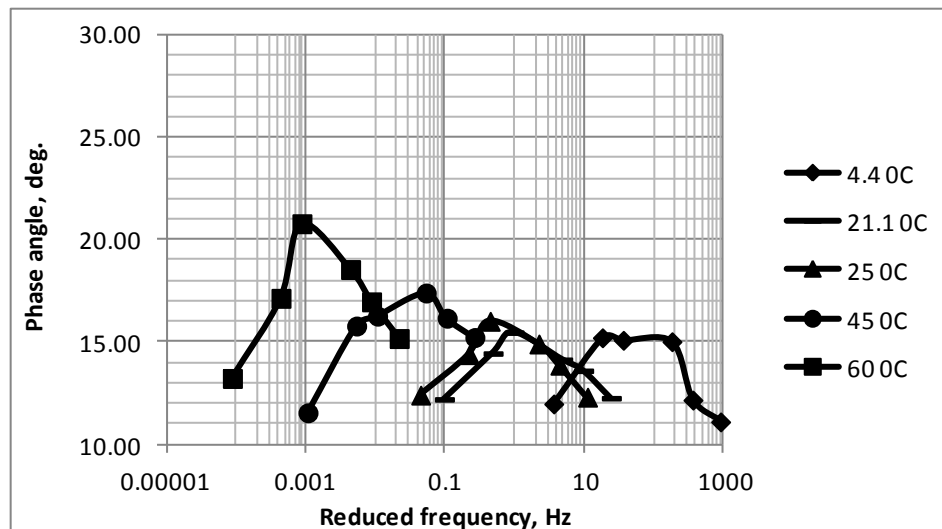
**Figure 4.24: Wet process phase angle for 0% HDPP asphalt**



**Figure 4.25: Wet process phase angle for 1.0% HDPP asphalt**



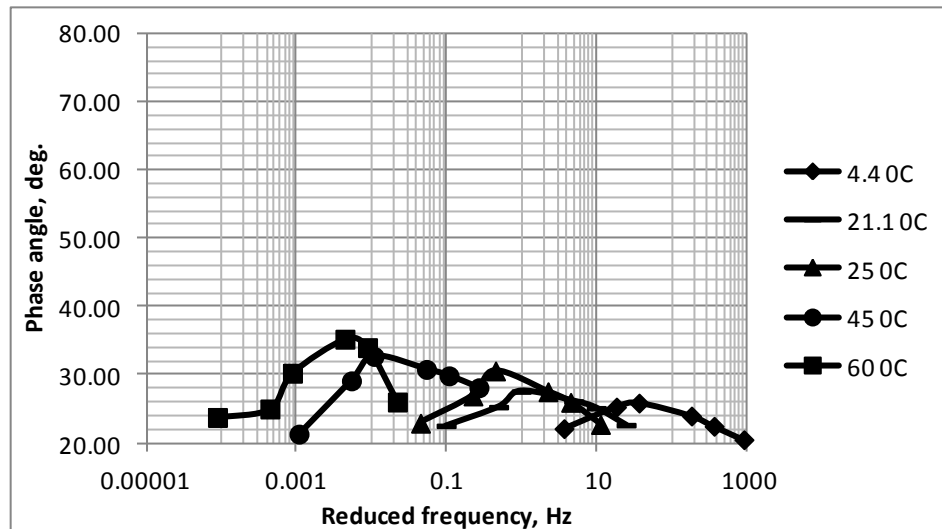
**Figure 4.26: Wet process phase angle for 2.0% HDPP asphalt**



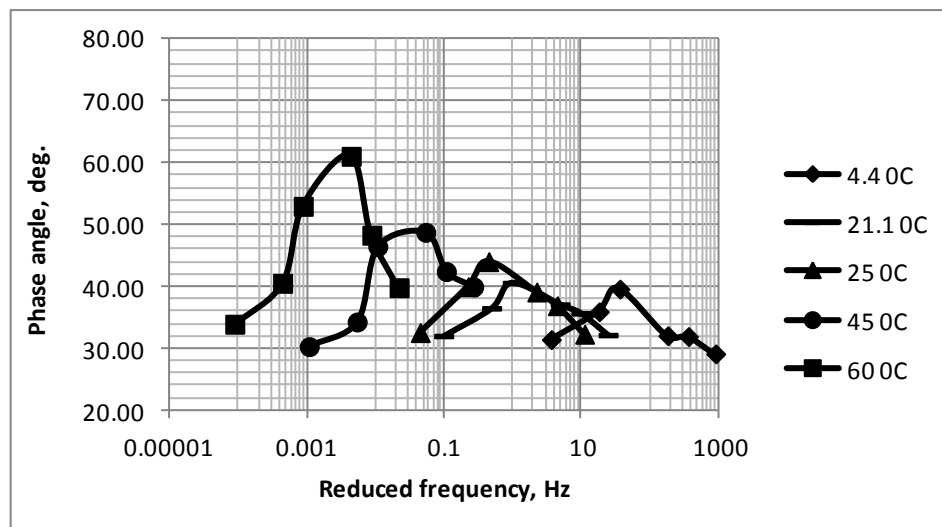
**Figure 4.27: Wet process phase angle for 3.0% HDPP asphalt**

Figures 4.28 to 4.30 show the trends of dry process phase angle,  $\phi$ . There was steady increase in  $\phi$  as a result of increasing voids and decreasing stiffness from 0 to 1.5% HDPP fibre asphalt in Figures 4.22 to 4.28. Higher  $\phi$  results in poor mixes decrease in durability because of increased oxidation and tendency of permanent deformation failures easily

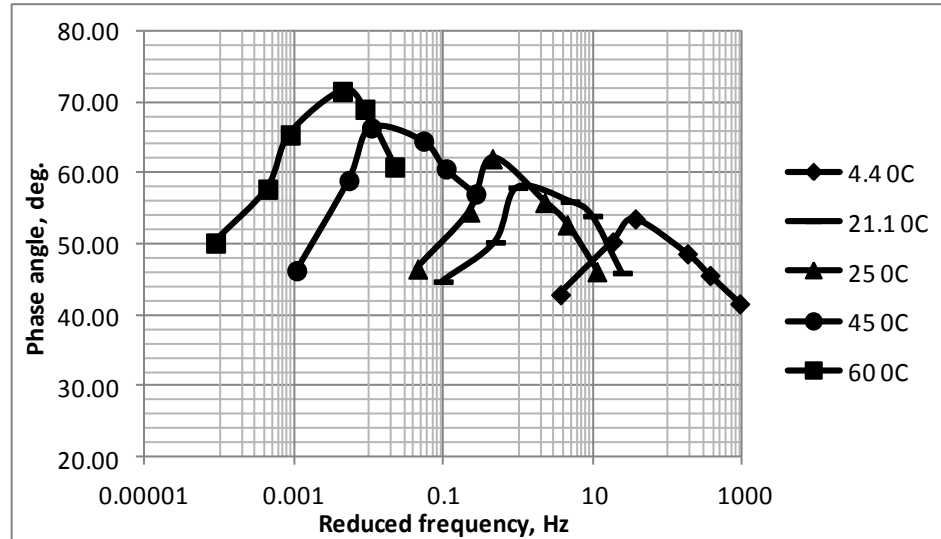
occurring. The general viscoelastic behaviour of asphalt indicates that the mixture is the most viscous as  $\phi$  approaches  $90^\circ$  and as  $\phi$  approaches  $0^\circ$  or infinity, the asphalt mixture behaves as an elastic solid at very low or very high frequencies. The observation agrees with the findings of Christensen (2003).



**Figure 4.28: Dry process phase angle for 0.5% HDPP asphalt**



**Figure 4.29: Dry process phase angle for 1.0% HDPP asphalt**



**Figure 4.30: Dry process phase angle for 1.5% HDPP asphalt**

**(iii) Recoverable strain,  $\epsilon$**

Recoverable strain decreased with increasing temperature and decreasing load cycles for both wet and dry processes. For wet processes, recoverable strain increased with increasing HDPP content because of increasing stiffness. In the dry process, recoverable strain decreased with increasing HDPP content in the mix because of void entrainment.

The wet and dry mixes experienced both recoverable strains,  $\epsilon$ , which is a relief or resilience when cycles of stresses goes away whereas irrecoverable strains are plastic and could lead to rutting or permanent deformation (Dawson, 2000). Both strains are slightly different from one cycle to another cycle due to the slight particle rearrangement and inter-particle motion, which may exude binder film in some aggressive situations and under a low dynamic modulus, which progresses as temperature and trafficking increases (Thom and Airey, 2006).

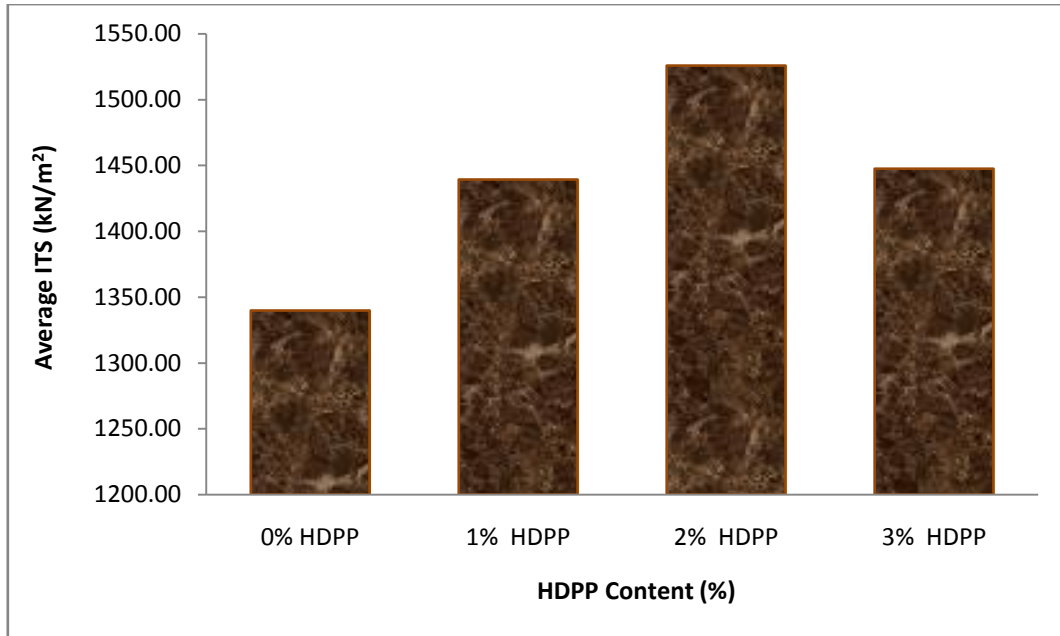
The recoverable strains of wet process increased from 0 to 3% HDPP content and decreased with increasing temperature from 4.4 °C to 60°C and increasing frequency from 0.1Hz to

25Hz because of increased flow deformation. While the recoverable strains of dry process are fairly close to the control, the gain in tensile strength is counteracted by the void entrainment of the two phase asphalt and reinforcing HDPP composite.

**(b) Indirect Tensile Strength (ITS)**

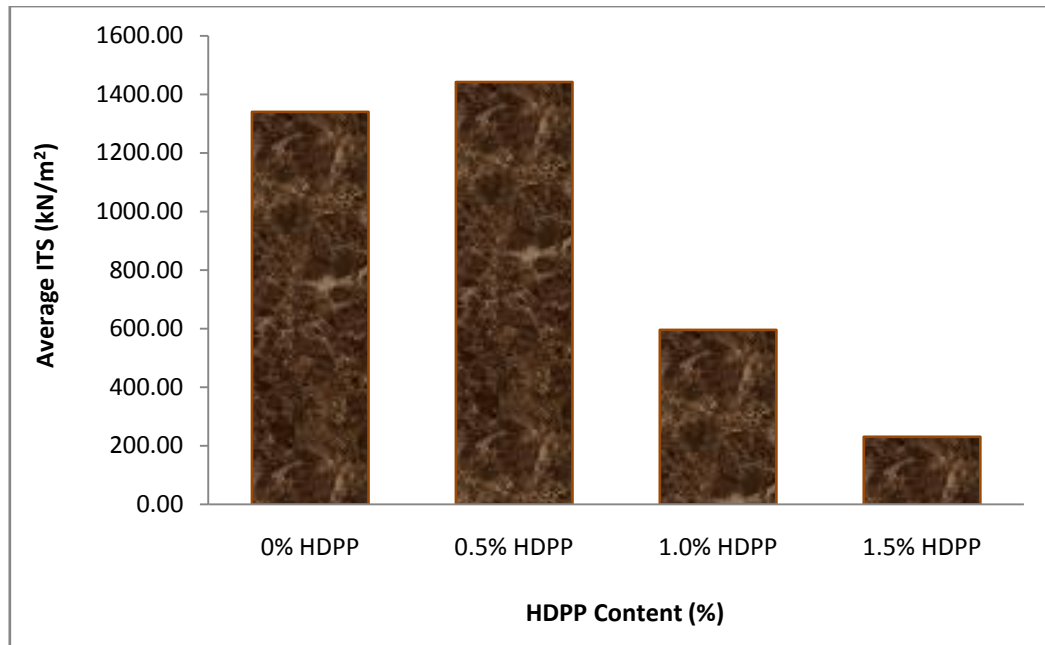
The test provides insight into the strength of the mix and is influenced by bitumen properties. Being an indicator of mix cohesion, it could be used to assess mix stability and is related to rutting resistance, durability and stripping potential (ASTM D4123: 2005a).

The average ITS for 0 (control), 1, 2 and 3 % HDPP wet mix are 1339.78, 1439.239, 1525.737 and 1447.45 kN/m<sup>2</sup> respectively as shown in Figure 4.31. The values of ITS for 0 to 3% HDPP wet process meets the minimum average ITS requirement of 1,100kPa (1,100 kN/m<sup>2</sup>) recommended by ASTM D4123 (2005a) and ASTM D6931 (2012); with 2.0% HDPP being the optimum. None of the values of ITS for polymer modified bitumen between the range of 0 to 3.0% is above maximum of 1,700kPa (1,700 kN/m<sup>2</sup>) which is indicative of brittle failure and low flexibility of the mix. At optimum of 2% HDPP, ITS increased by about 14%.



**Figure 4.31: Relationship between Average ITS and HDPP Content for Wet Process**

For the dry process, the ITS in Figure 4.32 for 0.5, 1.0 and 1.5% HDPP are respectively 1441.793, 595.746 and 230.9216kN/m<sup>2</sup>. Only 0.5% meets the recommendations of ASTM D6931 (2012) and ASTM D4123 (2005a). At this optimum of 0.5% HDPP for dry process, ITS increased by about 8%. The values of ITS for 1.0 to 1.5% HDPP dry process are below minimum requirement and could lead to poor rutting resistance especially at higher ambient temperature.

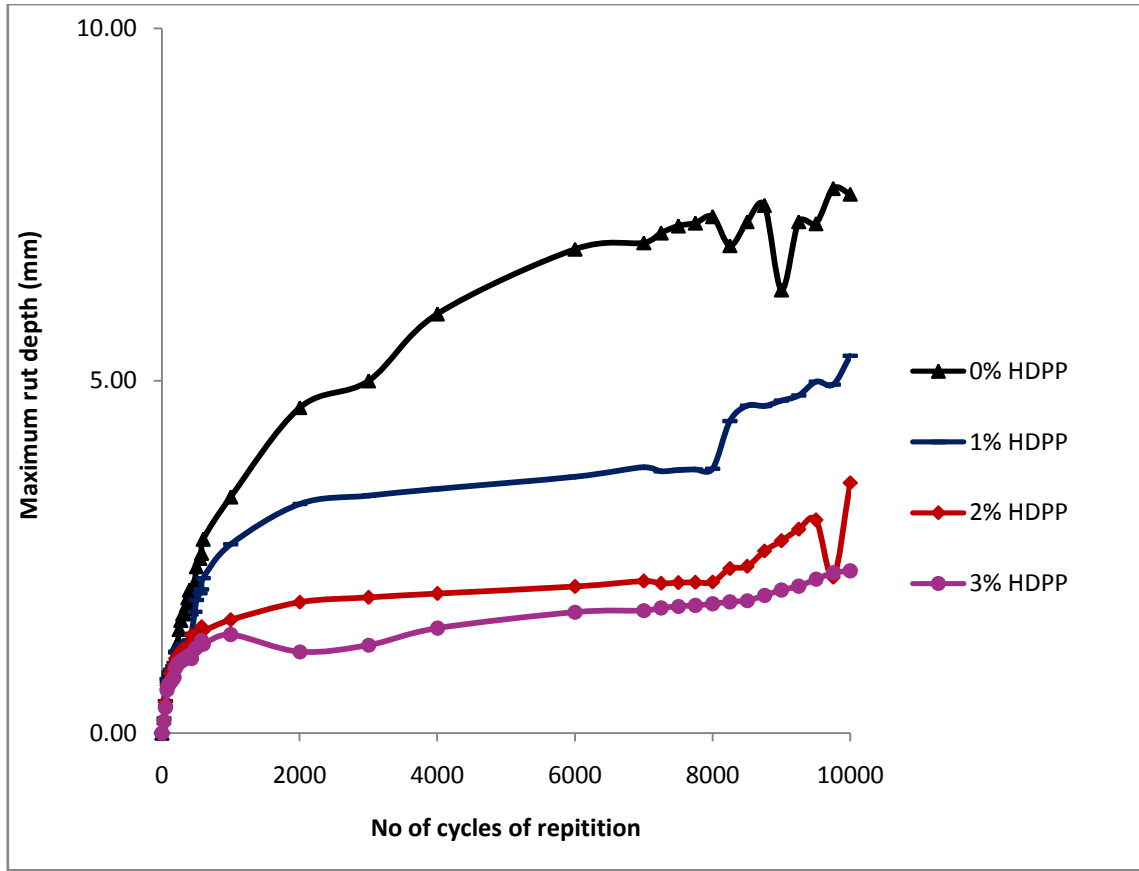


**Figure 4.32: Relationship between Average ITS and HDPP Content for Dry Process**

**(c) Permanent Deformation (Rutting)**

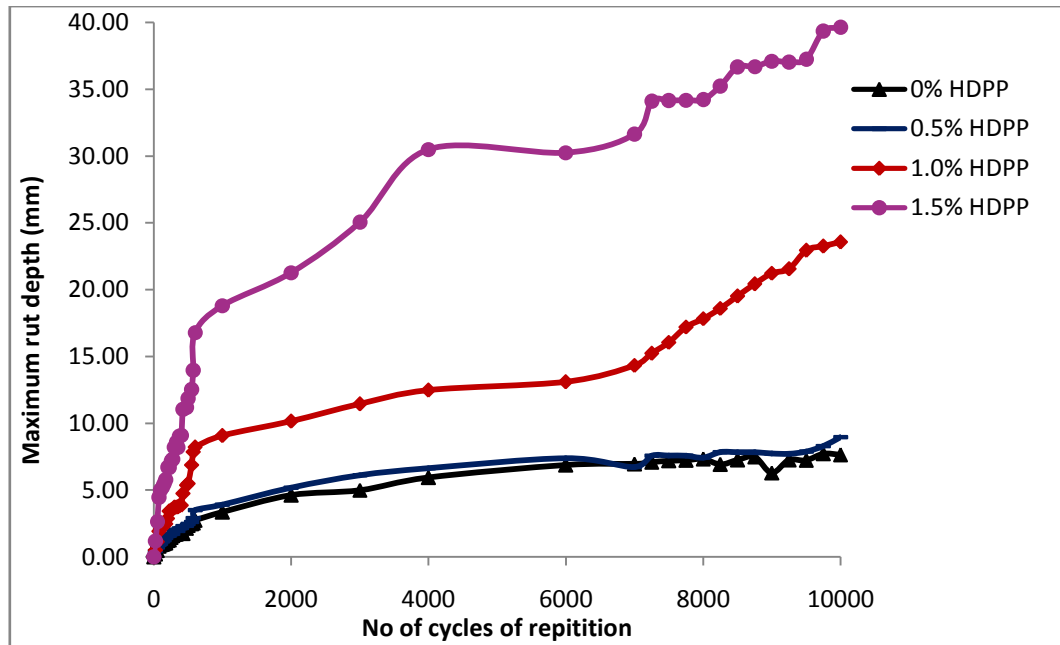
Rutting is a surface distortion as a result of inelastic or permanent deformation from wheel loads. At this point, the deformation is not recoverable.

Figure 4.33 shows plot of rut depth against number of cycles of load repetitions for wet process (details in Appendix C5). For wet process, the rut depths were decreasing from 0 to 3% HDPP showing an increasing trend of rutting resistance. The decreasing trends may be as a result of increasing stiffness (Moghaddam *et al.*, 2011; Tayfur *et al.*, 2007). Based on result of test presented in Figure 4.32, rut resistance increases with decreasing rut depth. The rut values of 0 to 3% HDPP wet process are significantly below 8 mm recommended for 8, 000 load cycles of NCHRP Report 508 (2003) (Cooley *et al.*, 2000) and 12.5mm at 10,000 cycles of load repetitions for AASHTO T324 (2015) recommendation.



**Figure 4.33: Relationship between Maximum Rut Depth and Load Cycles for Wet Process**

Figure 4.34 shows plot of rut depth against number of cycles of load repetitions for dry process. In dry mix process, only 0.5% HDPP content meets the recommendations of both NCHRP Report 508 (2003) (Cooley *et al.*, 2000) and AASHTO T324 (2015) to be adjudged a good mix. The rut depth of 1.0 – 1.5% HDPP dry process was increasing because of increasing voids and could cause high oxidation of bitumen, high moisture susceptibility and low deformation resistance failures.



**Figure 4.33: Relationship between Maximum Rut Depth and Load Cycles for Dry Process**

#### 4.5 Asphalt Creep as Ageing Process

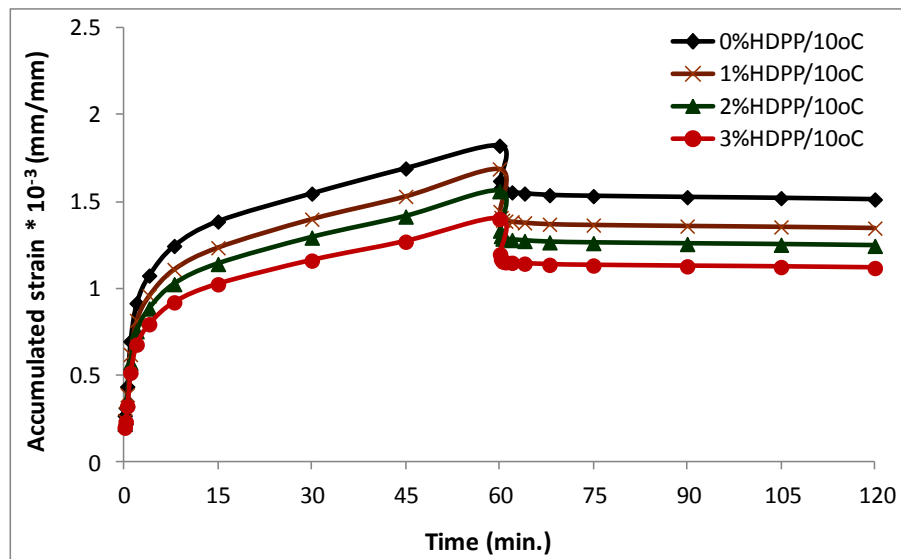
Asphalt ageing may lead to extreme situations of brittle glassy transition or stage wise creep or permanent deformation process.

##### (a) Short-term Loading (Uniaxial Static Creep Test)

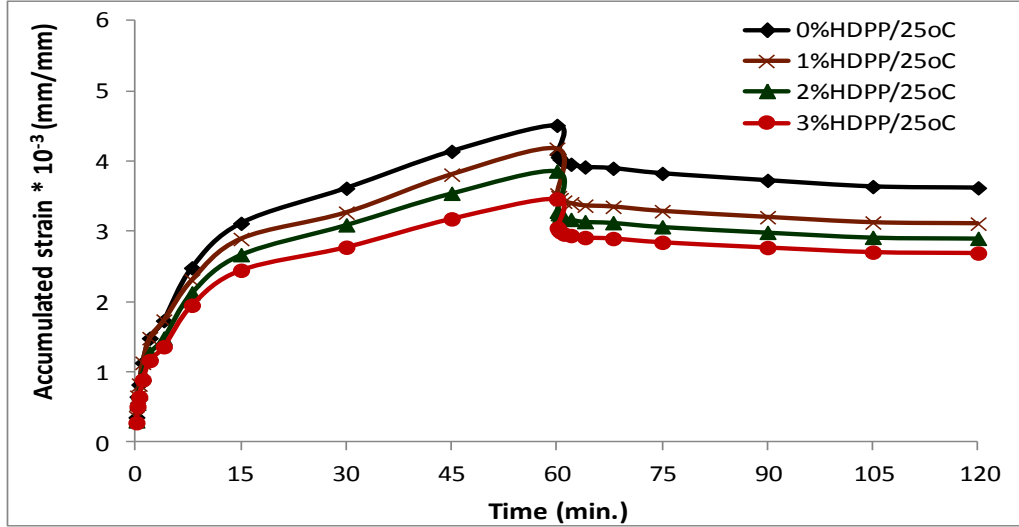
Static creep evaluation was conducted to assess the short term deformation of asphalt to static loading and unloading with time under a particular temperature. Creep strain is temperature dependent, that is, the higher the temperature the higher the strain. The creep strain consists of an instantaneous part and a time dependent part and the deformation is partially recovered. At high loading and high surrounding temperatures and time or frequency, creep deformation becomes plastic and can increase several times than instantaneous deformation, thereby leading to premature failure of road structure (Răcănel, 2004).

The deformation responses of wet process of asphalt mixes (0, 1, 2 and 3% HDPP) are shown in Figures 4.35 to 4.38, while dry process (0, 0.5, 1.0 and 1.5% HDPP) are shown in Figures 4.39 to 4.42 (details are in Appendices C1 and C2).

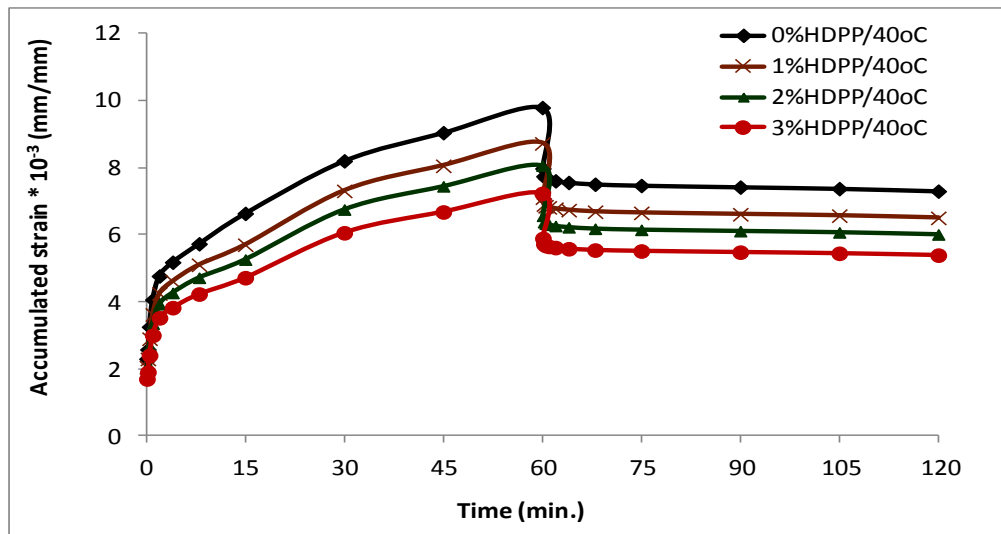
For the wet process, increasing HDPP content from 0 to 3% minimized creep strains even at the higher test temperature because of temperature resistivity of the polymer. Creep strains in Figures 4.35 to 4.38 decreased as HDPP increased from 0 to 3% and for increasing 10°C to 60°C temperature range considered. Higher molecular weight in HDPP and aromatic rings in bitumen add to thermal stability, increasing the creep resistance of a polymer-bitumen mix in wet process (Meyers and Chawla, 1999). As observed by Taherkhani and Javanmard (2011), shear deformation due to viscous behaviour that renders asphalt mix susceptible to rutting is lowered at both lower and higher temperature with increasing HDPP content.



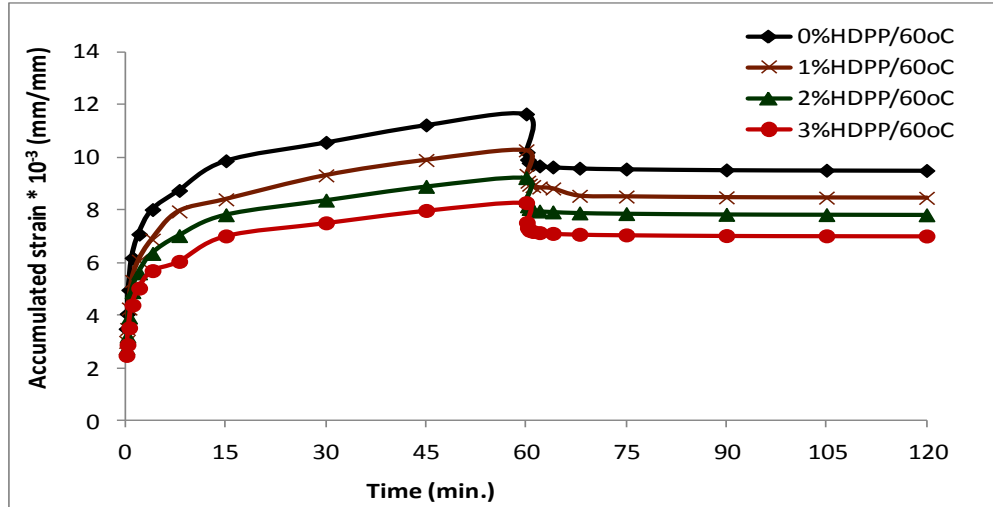
**Figure 4.35: Relationship between Accumulated Static Creep and Time for Specimens at 10°C in Wet Process**



**Figure 4.36: Relationship between Accumulated Static Creep and Time for Specimens at 25°C in Wet Process**

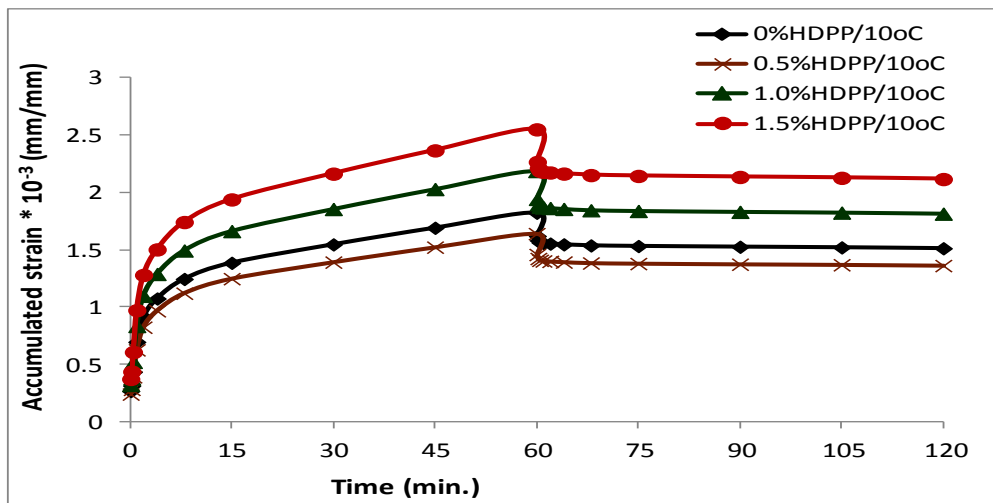


**Figure 4.37: Relationship between Accumulated Static Creep and Time for Specimens at 40°C in Wet Process**



**Figure 4.38: Relationship between Accumulated Static Creep and Time for Specimens at 60°C in Wet Process**

For the dry process in Figures 4.39 to 4.42, optimum creep strain which is the lowest attained is at 0.5% HDPP for the 10°C to 60°C range of temperature. This could be as a result of increase in tensile strength and stiffness, but beyond 0.5% HDPP, void content is increased and creep deformation and rutting are rapidly progressed. The observation agreed with the findings of Cleven (2000) and Ye *et al.* (2009).



**Figure 4.39: Relationship between Accumulated Static Creep and Time for Specimens at 10°C in Dry Process**

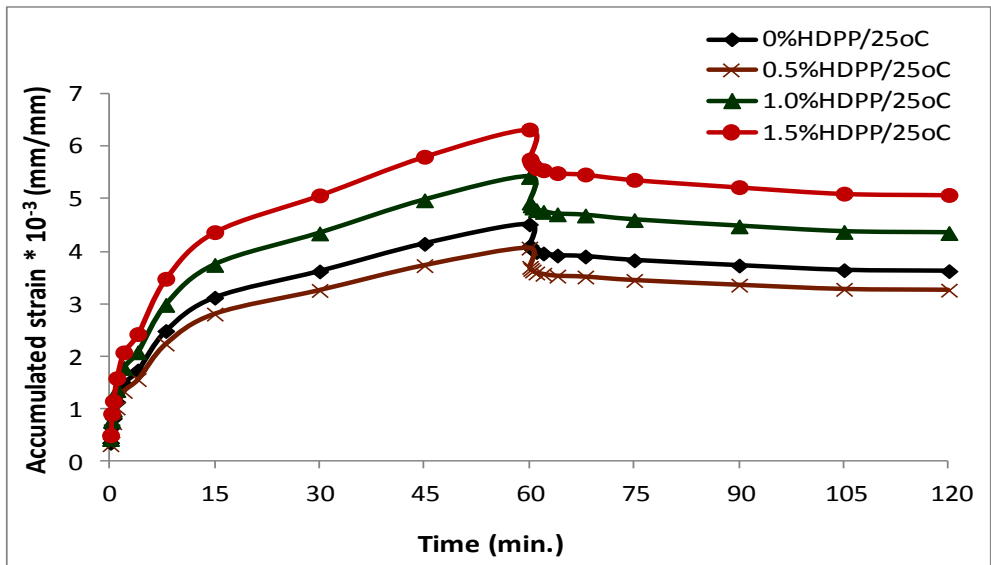


Figure 4.40: Relationship between Accumulated Static Creep and Time for Specimens at 25°C in Dry Process

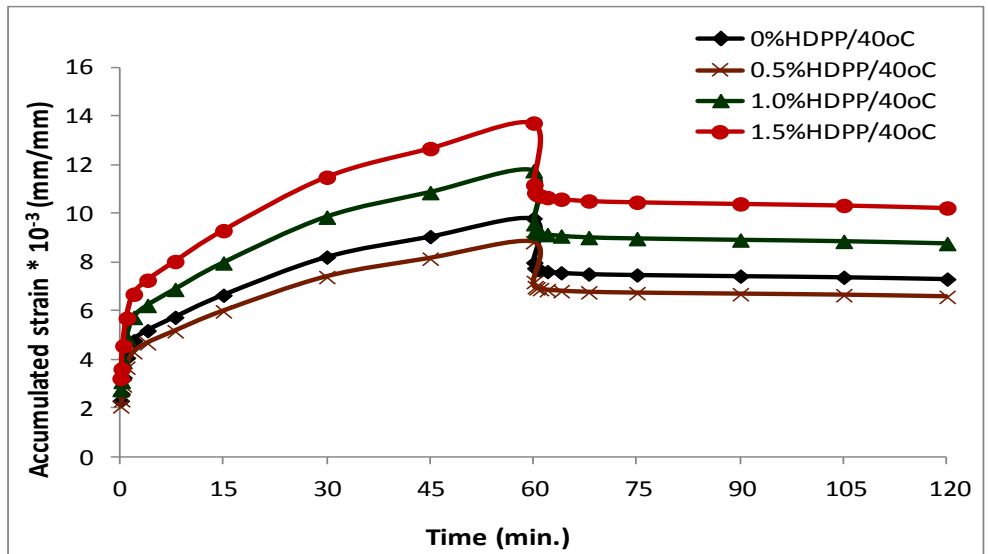
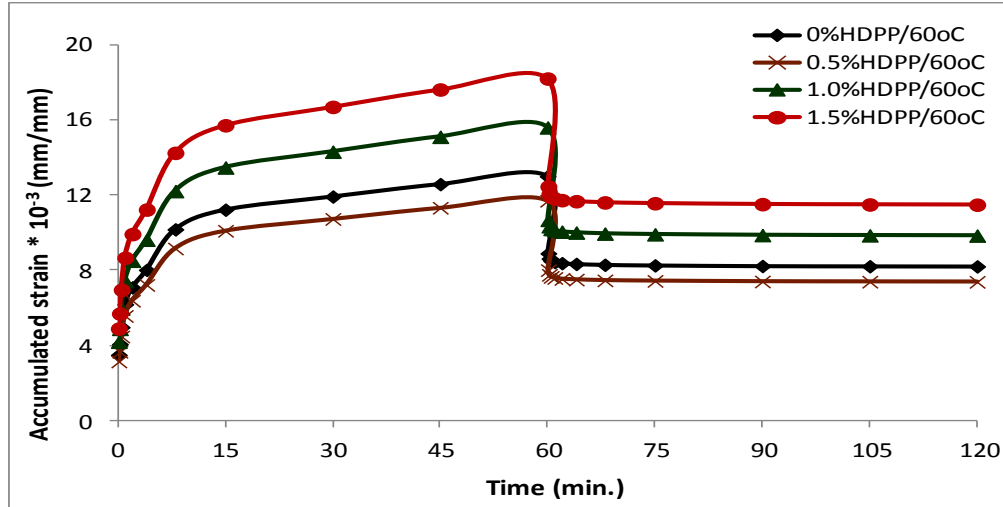


Figure 4.41: Relationship between Accumulated Static Creep and Time for Specimens at 40°C in Dry Process



**Figure 4.42: Relationship between Accumulated Static Creep and Time for Specimens at 60°C in Dry Process**

#### (b) Creep Recovery

Creep recovery of asphalt mix has effects on viscoelasticity and life cycle (Geber *et al.*, 2014). While recovery strain are elastic or delayed elastic, irrecoverable strains are permanent deformations, viscoplastic and lead to rutting (Elnasri *et al.*, 2013; 2014). Shenoy (2002) concluded that manifestation of rutting are because of decrease in volume or densification and shear displacement that leads to shift of the material without volume change. Tables 4.8 and 4.9 show the percentages of creep recovery at different temperatures. The plots of the recovery values are in Figures 4.43 and 4.44 for wet and dry processes respectively.

The creep recovery for wet and dry processes increased with increasing temperature from 10 °C to 40°C since the rate of stress absorption, stress transfer and viscoelastic property increased when temperature increased (Yao *et al.*, 2013; Mokhtari and Moghadas-Nejad 2012; and Khodaii and Mehrara, 2009). But beyond the optimum temperature of 40°C, the deformation flow of the mix is increased causing it to undergo plastic deformation.

Optimum creep recoveries for wet and dry processes were therefore attained at 40°C with marginal level of tolerance to sustain performances of the HDPP mixes up to 60°C even as the deformation increased.

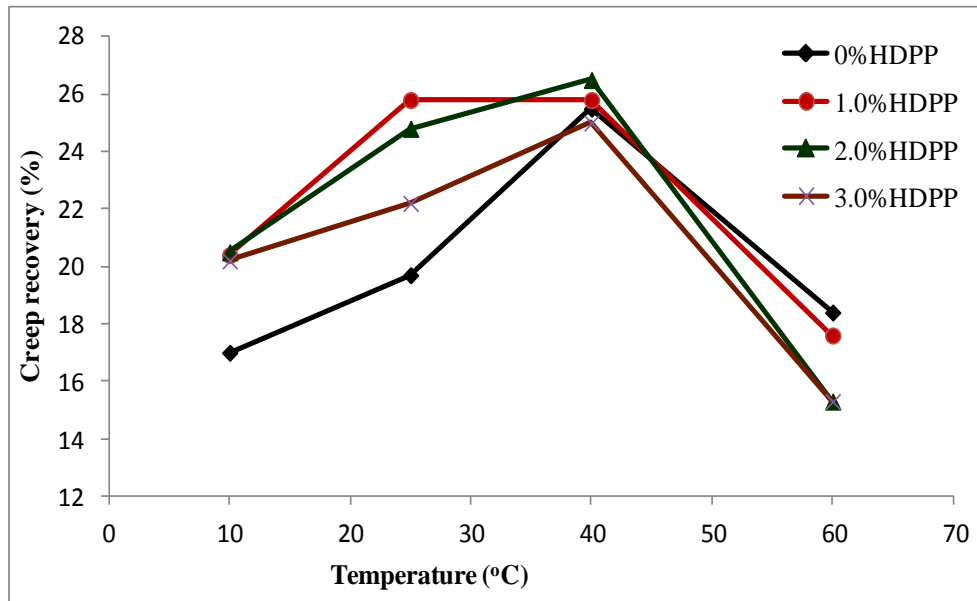
From Figure 4.43, the wet process optimum percentage recovery is 26.5% corresponding to 2.0% HDPP at optimum temperature of 40°C. For the dry process in Figure 4.44, optimum percentage recovery is 26.3%, which corresponds to 1.0% HDPP at optimum temperature of 40°C. Researchers like Elnasri *et al.* (2013) and Moghaddam *et al.*, (2014) have observed the same positive trends of good creep recovery and improve pavement performance at temperature of 40°C.

**Table 4.8: Creep recovery and permanent creep strains for wet process**

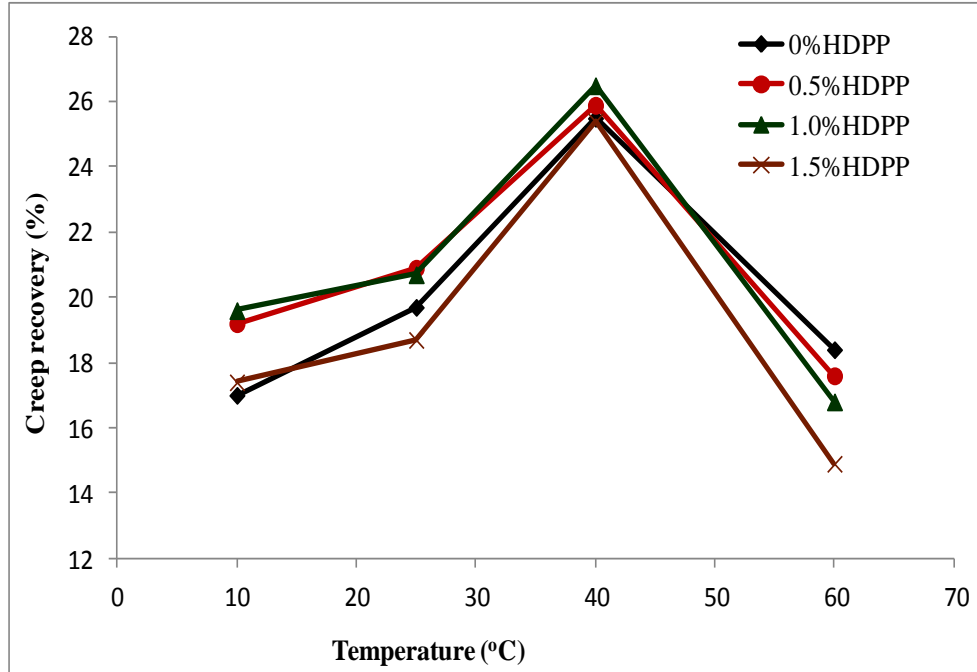
Temperature (°C)	HDPP Content (%)	Permanent Creep *10 <sup>-3</sup> (mm/mm)	Creep Recovery *10 <sup>-3</sup> (mm/mm)	Permanent Creep (%)	Creep Recovery (%)
10	0	1.513	0.310	83	17.0
	1	1.344	0.344	79.6	20.4
	2	1.240	0.320	79.5	20.5
	3	1.117	0.283	79.8	20.2
25	0	3.625	0.889	80.3	19.7
	1	3.103	1.079	74.2	25.8
	2	2.906	0.958	75.2	24.8
	3	2.697	0.770	77.8	22.2
40	0	7.295	2.497	74.5	25.5
	1	6.472	2.250	74.2	25.8
	2	5.923	2.136	73.5	26.5
	3	5.423	1.808	75	25.0
60	0	9.510	2.144	81.6	18.4
	1	8.465	1.808	82.4	17.6
	2	7.824	1.413	84.7	15.3
	3	7.020	1.268	84.7	15.3

**Table 4.9: Creep recovery and permanent creep strains for dry process**

Temperature (°C)	HDPP Content (%)	Permanent Creep *10 <sup>-3</sup> (mm/mm)	Creep Recovery *10 <sup>-3</sup> (mm/mm)	Permanent Creep (%)	Creep Recovery (%)
10	0	1.513	0.310	83	17.0
	0.5	1.325	0.315	80.8	19.2
	1.0	1.758	0.429	80.4	19.6
	1.5	2.107	0.444	82.6	17.4
25	0	3.625	0.889	80.3	19.7
	0.5	3.214	0.849	79.1	20.9
	1.0	4.296	1.121	79.3	20.7
	1.5	5.138	1.182	81.3	18.7
40	0	7.295	2.497	74.5	25.5
	0.5	6.530	2.282	74.1	25.9
	1.0	8.660	3.090	73.7	26.3
	1.5	10.226	3.482	74.6	25.4
60	0	9.510	2.144	81.6	18.4
	0.5	8.643	1.846	82.4	17.6
	1.0	11.636	2.349	83.2	16.8
	1.5	13.885	2.431	85.1	14.9



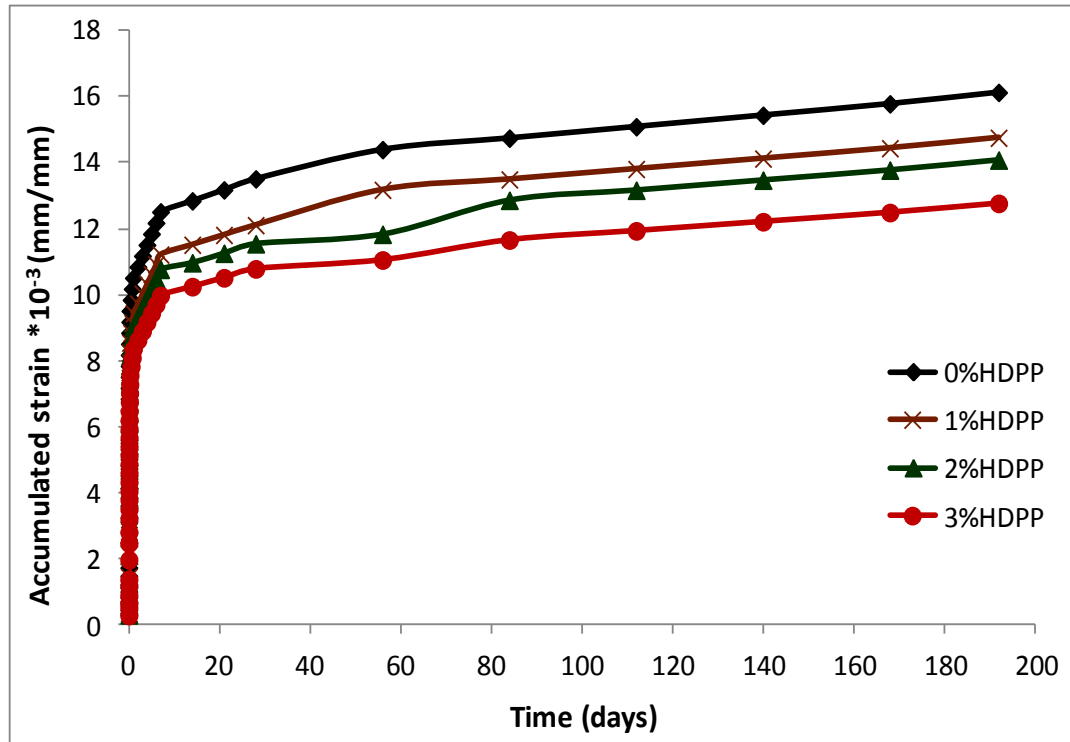
**Figures 4.43: Percentage Creep Recovery with Temperature for Wet Process**



**Figures 4.44: Percentage Creep Recovery with Temperature for Dry Process**

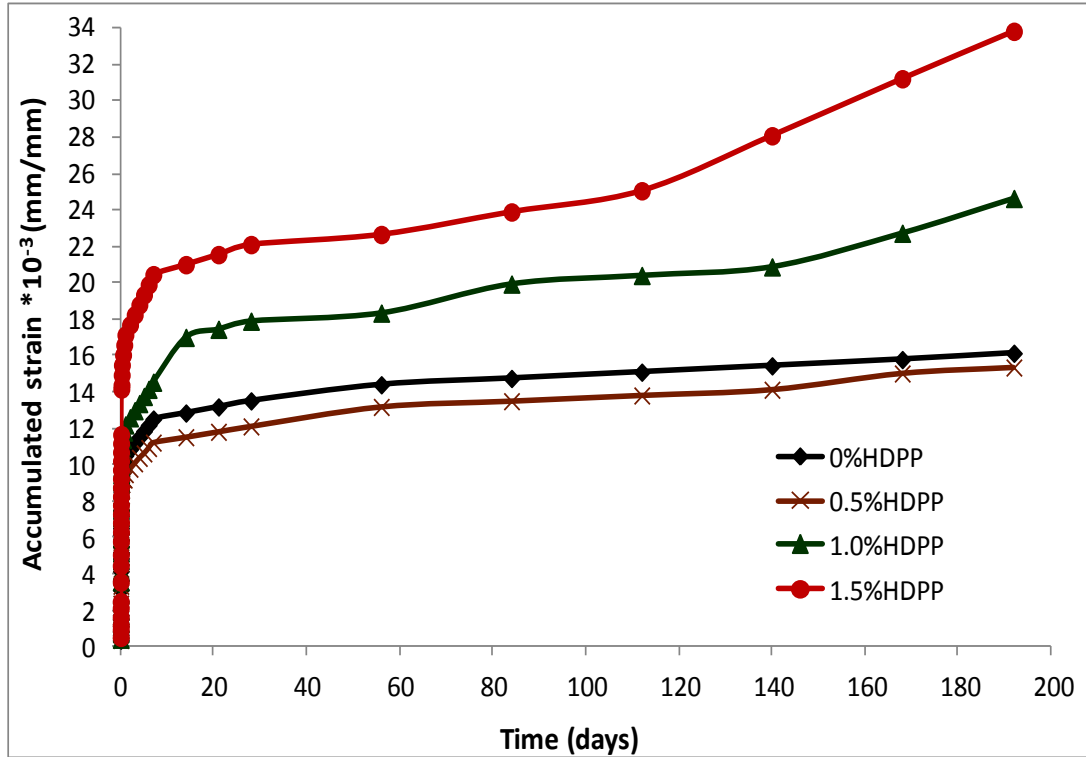
**(c) Long-term Loading (Uniaxial Static Creep Test)**

The result of long term loading for wet process is as shown in Figures 4.45 (details in Appendices C3 and C4). The deformation induced strains in the first phases of wet is instantaneous and could be associated with volume change, thus, compaction of asphalt concrete. This view is supported by Zhou *et al.*, (2004). For the wet process the deformation only ended at secondary phase showing constant slow rate of increase in rutting with increase in shear stress and did not progress to tertiary phase for 0 to 3% HDPP and for the duration of testing.



**Figure 4.45: Relationship between Long Term Loading Accumulated Static Creep and Time for specimens at 25°C in Wet process**

Figure 4.46 shows the plot of accumulated creep strains for dry process. The deformation induced strains in the first phases here is equally instantaneous and leads to compaction of asphalt concrete (Zhou *et al.*, 2004). There was no progression to tertiary rutting phases for 0 to 0.5% HDPP. But onset of tertiary rutting phases appeared at about 120days of testing for mixes having 1% and 1.5% HDPP contents in the dry process. This is because the dry mix entrained more voids and have tendency of increase in deformation. Researchers observed that that tertiary stage exhibits high level of rutting related to plastic deformation with flow under no volume change (Zhou and Scullion, 2002; Zhang, 2008; Xu and Sun, 2013).



**Figure 4.46: Relationship between Long Term Loading Accumulated Static Creep and Time for specimens at 25°C in Dry process**

#### 4.6 Thermal Analysis

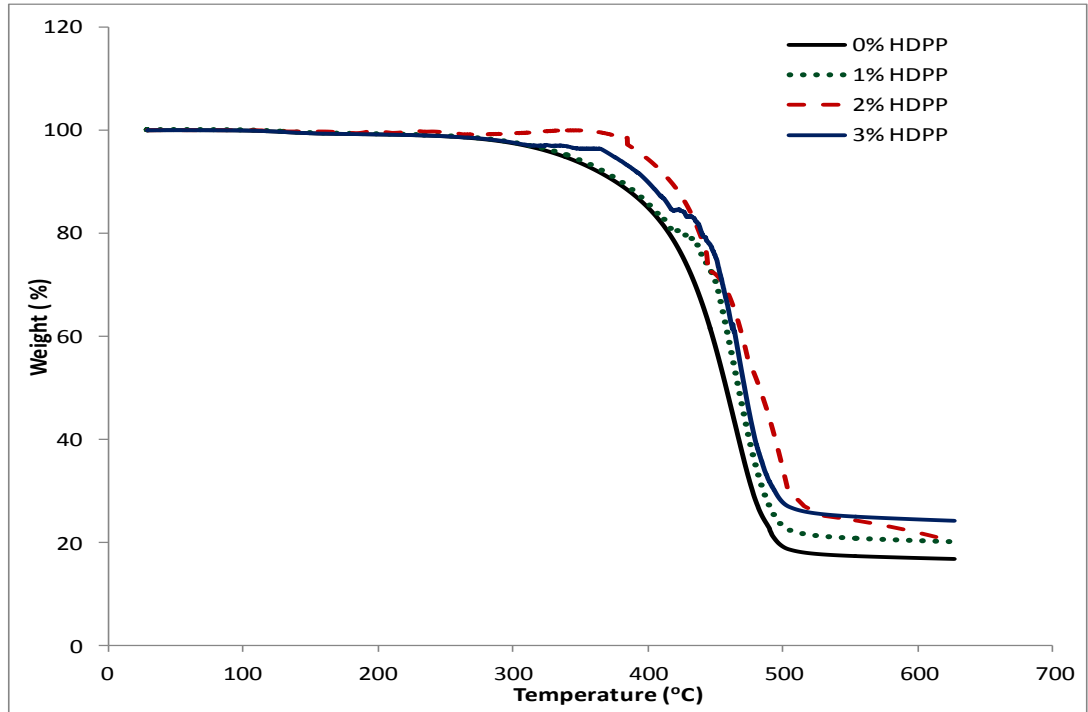
The thermo-gravimetric analysis (TGA) and differential thermal analysis (DTA) were shown in Figures 4.47 and 4.48 respectively. These evaluations show the trends of degradation and phase transition of unmodified and HDPP modified bitumen. ASTM D4124 (200 separated bitumen constituents according to molar mass, solubility and polarity of fractionates called SARA (S-Saturates, A-Aromatics, R-Resins, and A-Asphaltenes) (Lesueur, 2009). The first three together forms light molecular weight component called Maltene which mostly leads to ageing as they volatilized. The decomposition or oxidation of heavy molecular weight component called asphaltene further worsens ageing and deformation (Forbes *et al.*, 2001). According to Sekar *et al.* (2015), polymer-bitumen mix

forms chemical bonds between the asphaltene compounds and strong bonding of the diene molecules from the polymer and increases thermal stability.

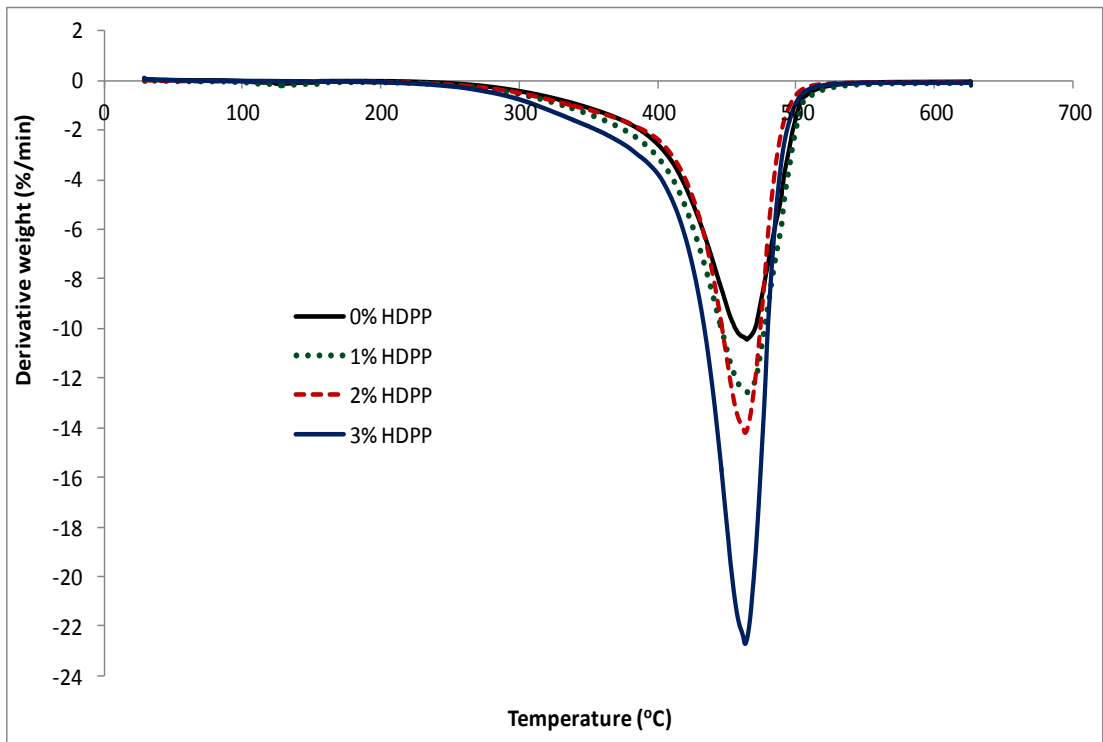
The result in Figure 4.47 indicates that at 450°C, for instance, the TGA weight losses of 42.2%, 29.6%, 27.9% and 24.5% respectively for 0, 1, 2 and 3% HDPP. Although, BS EN12591 (2009) recommends that the maximum temperature of bitumen at any stage of mix preparation to be 180 °C for 50/70 penetration grade, active degradation occurs between temperature of 250°C to 550°C where components such as saturates and aromatics are volatilized and asphaltene decomposed (Feng *et al.*, 2011; Nasser *et al.*, 2012; Zhang *et al.*, 2011). The mass losses between 200 °C and 350 °C are largely due to emission of H<sub>2</sub>, CH<sub>4</sub>, CO and CO<sub>2</sub> gasses (Cataldo *et al.*, 2004; Mouazen *et al.*, 2013). Addition of HDPP leads to reduced gasification, increase in the solid phase than fluid components (Navarro *et al.*, 2010). This induces more temperature resistivity as the ability to lose light components decreased (Lucena *et al.*, 2004) and thus, increasing the melting temperature of the blend containing HDPP polymer (Giuliani *et al.*, 2009).

The residual mass at the end of 650°C DTA/TGA were 16.8%, 20.7%, 20.9% and 24.1% respectively for 0%, 1.0%, 2.0% and 3.0%. The values show hierarchy of performance of modified bitumen in the order of HDPP content and increasing degree of cross-linking between aromatic rings which imparted upon the performance (Mouazen *et al.*, 2013).

The trends showed improved performance as HDPP bitumen has lower weight loss, lower rate of degradation and volatilization and more temperature resilience; thus, could improve the rheology and longer lifespan better than pure bitumen (Zhang *et al.*, 2009). The results agree with the conclusion of Xu and Huang (2010) that the main combustion and phase transition of bitumen lies in the exothermic reaction in second phase at temperature ranging from 405°C to 490°C, where the main weight loss occurred.



**Figures 4.47: TGA result of HDPP bitumen**

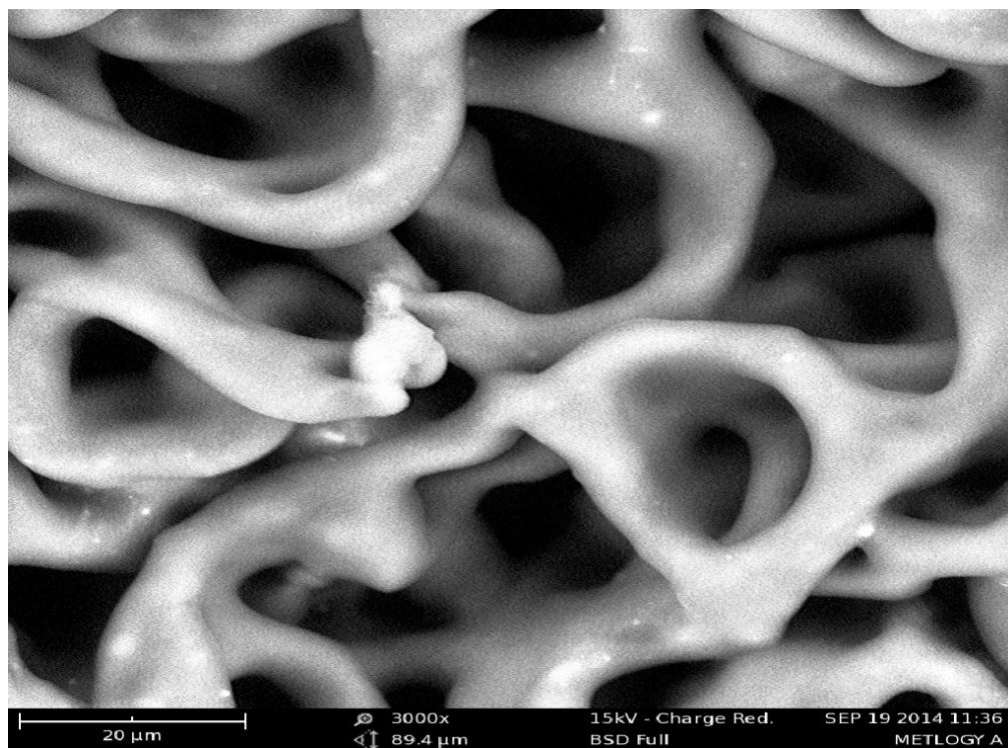


**Figures 4.48: DTA result of HDPP bitumen**

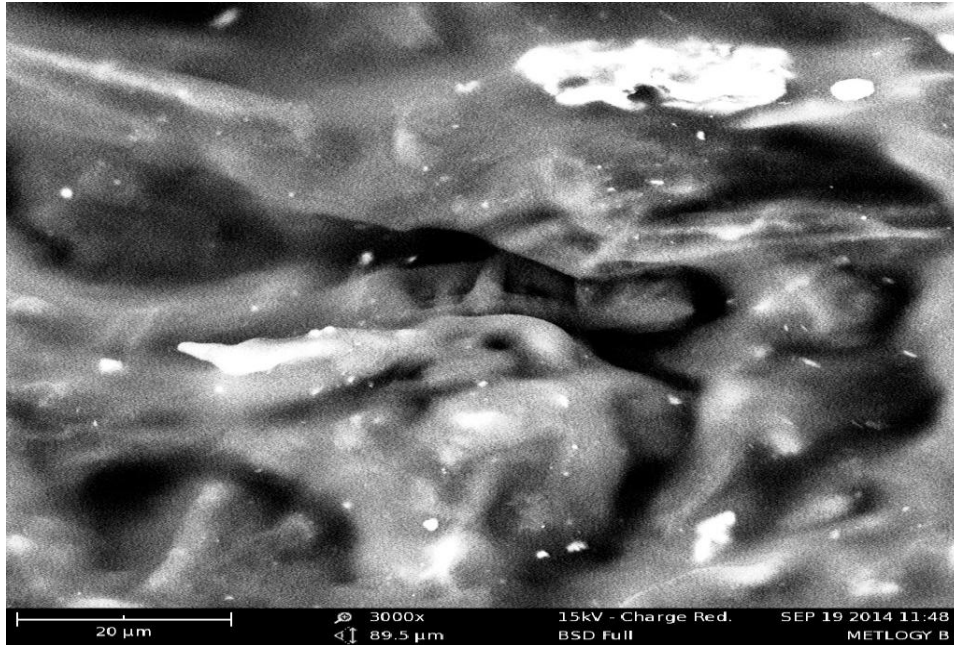
#### 4.7 Morphological Study using Scan Electron Micrograph

Compatibility between polymer and bitumen are critical to the morphological, thermal properties and mechanical performance of asphalt mixtures (Hailong *et al.*, 2003). Scan electron micrograph are helpful in understanding of polymer micro size reduction, pore microstructure and physical dispersion of polymer particles in bitumen (Yao *et al.*, 2013).

The micrograph images of 0% HDPP (control) and 2% HDPP modified bitumen being optimum for wet mix are shown in Plates VI and VII respectively. Plates VIII and IX are the pore histograms and Plates X and XI are the fibre histograms of the two samples. The micrographs were taken at the same resolutions and magnifications.

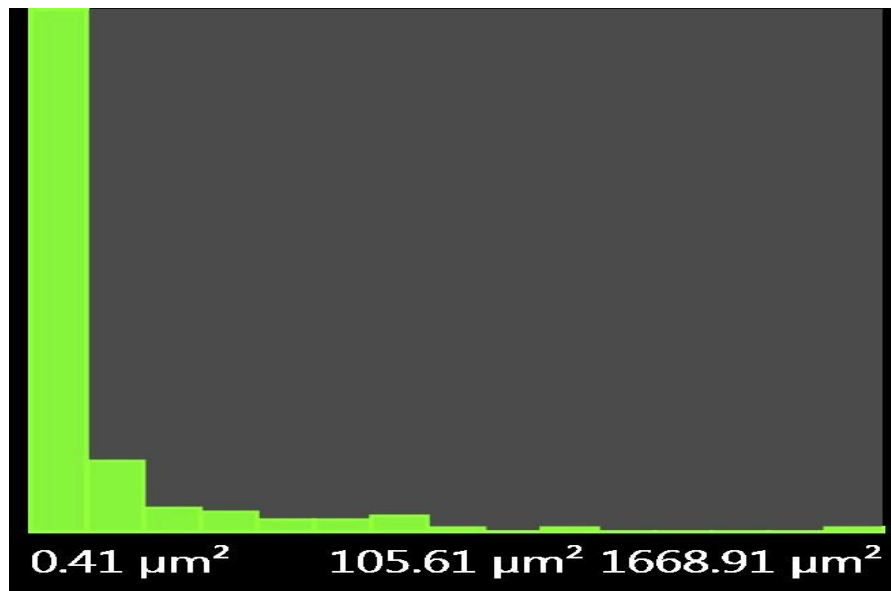


**Plate VI: Micrograph of 0% HDPP bitumen**

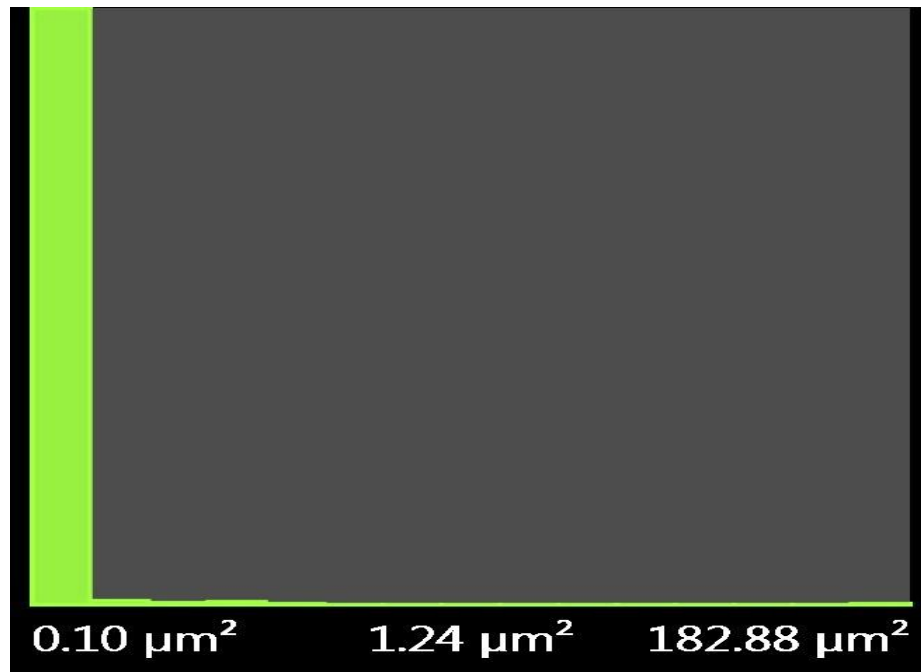


**Plate VII: Micrograph of 2% HDPP bitumen**

The specimen of 0% HDPP bitumen has larger pore areas ranging from 0.41- 1668.91 $\mu\text{m}^2$  (pore histogram in Plate VIII). Comparatively, 2% HDPP bitumen has smaller pores of range 0.1-182.88  $\mu\text{m}^2$  (pore histogram in Plate IX). It means that smaller pores translate into more contact surfaces, stronger bond and strength of the overall mix.

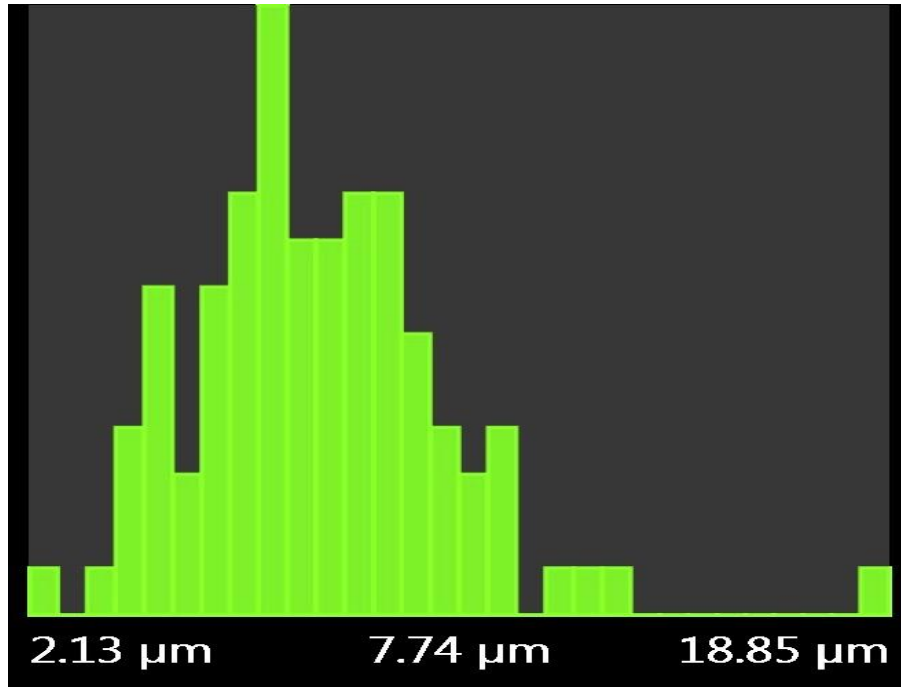


**Plate VIII: Pore histogram of 0% HDPP bitumen**

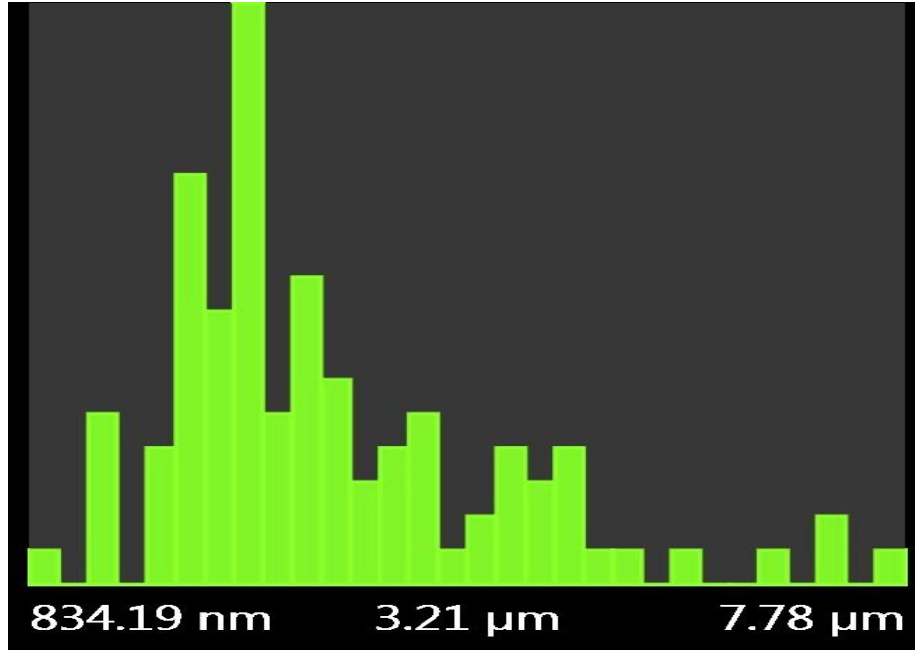


**Plate IX: Pore histogram of 2% HDPP bitumen**

Also, the histogram of the fibre nano diameter for 0% HDPP bitumen is between 2.13 to  $18.85 \mu\text{m}$  (Plate X), while 2% HDPP bitumen ranges from 834.19nm to  $7.78 \mu\text{m}$  (Plate XI). It showed that 2% HDPP bitumen has more reaction surface due to surface area than the control (0% HDPP bitumen). Elasticity and strength of unmodified bitumen may not be sufficient to resist the stresses that traffic places on the pavement for long duration. The dynamic interaction between polymer dispersed bitumen matrix coalesces the structure and reinforces the strength (Loeber *et al.*, 1996; Reena *et al.*, 2012).



**Plate X: Fibre histogram of 0% HDPP bitumen**



**Plate XI: Fibre histogram of 2% HDPP bitumen**

## CHAPTER FIVE

### CONCLUSION AND RECOMMENDATIONS

#### 5.1 Conclusion

Following the tests in Chapter three and the discussions in Chapter four, the conclusion drawn are:

- (1). For the wet process for HDPP polymer asphalt, 2.0% HDPP content gave improved stability, flow resilience and enhanced void properties (VMA, VIM and VFB).
- (2). Results of Marshall Parameters obtained for this wet mix performed better than 0% HDPP (control) and met the Marshall specifications (Asphalt Institute, 1997) for heavy traffic situation of more than  $10^6$  ESAL considered and hence, could mitigate pavement distresses.
- (3). Also, for the Marshall properties of dry process for fibre reinforced asphalt concrete, 0.5% HDPP content gave optimally better results than the control (0% HDPP) for the heavy traffic situation simulated.
- (4). The optimum dynamic modulus ( $E^*$ ) for wet process lies at 2.0% HDPP with about 50% increase in stiffness compared to the control (0% HDPP) for all temperature range of  $4.4^\circ\text{C} - 60^\circ\text{C}$  and frequencies of cycles of loading ranging between 25 – 0.1Hz.
- (5). However, optimum dynamic modulus ( $E^*$ ) for dry process lies at 0.5% HDPP content, have about 14% increase above the control mix (0% HDPP).
- (6). Also, the optimum 2.0% HDPP wet process reduces strain rate and lowers the value of phase angle,  $\phi$ , within viscoelastic boundary to be able to withstand more cycles of loading.
- (7). At optimum of 2% HDPP, ITS increased by 14% for wet process.

- (8). The enhancement in ITS at the optimum of 2.0% HDPP wet mix could mitigate low temperature cracking and moisture susceptibility of the asphalt mix.
- (9). For the dry process, the value of ITS is better than the control at 0.5% but failed as HDPP content increased beyond 0.5%.
- (10). At optimum of 0.5% HDPP dry process, ITS increased by 8%.
- (11). The decrease in rutting potentials from 0 to 3.0% HDPP wet process improves the resistance to permanent deformation, thereby increasing the life span of the pavement surfacing.
- (12). While for fibre reinforced mixes, only 0.5% meets the requirement of rutting resistance.
- (13). For all SPT conducted -  $\rho$ ,  $\epsilon$ , ITS and rutting, 2.0% HDPP wet process performed optimally for heavy traffic situation simulated and could stand the test of serviceability and durability requirements.
- (14). SPT parameters are only adequate at 0.5% HDPP content dry process, thereby meeting the mechanical requirements of heavy traffic.
- (15). Static and dynamic properties are improved by 2.0% HDPP wet process, thereby reducing brittle and low impact failure and increase in tensile strength.
- (16). For dry mix, 0.5% HDPP has superior crack resistance, enhanced ductility and distinctive post crack resilient behaviour that can delay its propagation than the control or other dry mixes.
- (17). Permanent creep strains for wet process decreased as HDPP content increased from 0-3%, but generally, there are increasing strain trend with increasing temperature from 10°C to 60 °C since flow is increased by higher temperature.

- (18). For the short term creep loading wet mix, the optimum creep resistance lies at 3.0% HDPP and value is  $1.3995 \times 10^{-3}$  (mm/mm) for low temperature of  $10^{\circ}\text{C}$  and  $8.2875 \times 10^{-3}$  (mm/mm) for  $60^{\circ}\text{C}$  field temperature, values better than the control.
- (19). The maximum creep at optimum 3.0% HDPP wet mix account for 23.2% strain reduction at  $10^{\circ}\text{C}$  and 28.9% strain reduction at  $60^{\circ}\text{C}$  and as such impart better creep resistivity on the wet mix than the control.
- (20). The maximum creep strains for 0% HDPP (control) are  $1.8223 \times 10^{-3}$  (mm/mm) and  $11.6543 \times 10^{-3}$  (mm/mm) respectively for  $10^{\circ}\text{C}$  and  $60^{\circ}\text{C}$ .
- (21). Permanent creep strains for the dry process reached an optimum at 0.5% for the range 0 -1.5% HDPP content, however, there is an increasing trend as temperature increased from  $10^{\circ}\text{C}$  to  $60^{\circ}\text{C}$ , which triggered increase in deformation.
- (22). For the short term loading dry mix, the optimum creep resistance lies at 0.5% HDPP with values of  $1.6 \times 10^{-3}$  (mm/mm) at  $10^{\circ}\text{C}$  and  $10.4889 \times 10^{-3}$  (mm/mm) at  $60^{\circ}\text{C}$ .
- (23). The values for 0.5% HDPP dry mix static creep account for 12.2% creep strain reduction at  $10^{\circ}\text{C}$  and 10.0% creep strain reduction at  $60^{\circ}\text{C}$  compared to the control.
- (24). In the long term loading wet mix process for the period of 192days, accumulated creep strain for 0% HDPP (control) is  $16.1348 \times 10^{-3}$  (mm/mm).
- (25). The optimum value for long term loading wet mix lies at 3% HDPP corresponding to  $12.7715 \times 10^{-3}$  (mm/mm) and accounting for 20.9% creep strain reduction.
- (26). For the period of 192days in the long term dry mix, accumulated creep strain for 0.5% HDPP is the optimum and having a value of  $15.35007 \times 10^{-3}$  (mm/mm), which accounts for 4.9% creep strain reduction.
- (27). For TGA/DTA test, the critical degradation temperature range from  $250^{\circ}\text{C}$  to  $550^{\circ}\text{C}$  and 2% HDPP modified bitumen has better resilience than 0, 1 and 3% HDPP.

- (28). The residual weight at the end of TGA/DTA test shows greater temperature resilience in the order of 3%, 2%, 1% and 0% HDPP.
- (29). The shape and dispersed structure of 2% HDPP modified bitumen has better morphology than the control.
- (30). The wet mix gave better morphological, temperature resistivity and creep deformation resistivity than dry process.
- (31). Considering the various test conducted, optimum HDPP content of 2.0% gave better performance of rheological and mechanical properties than the control mix (0% HDPP).
- (32). The rheological and mechanical properties of 0.5% HDPP content of the dry process is better enhanced than the control mix (0% HDPP).

## **5.2 Recommendations**

Based on the study, the following recommendations are made:

- (a) The wet mix having 2.0% HDPP produced optimum hot mixed asphalt with improved Marshall characteristics, simple performance tests (SPTs), ageing resistance and morphological properties that are better enhanced and correlated with field performance than the conventional mix. In all the parameters evaluated in the wet process above, asphalt mix of 2.0%HDPP can support heavy traffic of more than  $1*10^6$  equivalent standard axle (ESAL) recommended by Asphalt Institute (1997) and has potential of resisting fatigue failures and cracking which largely contribute to pavement distress. The morphology of the structure of 2.0%HDPP modified bitumen as well as the thermal profile would enhance the durability of the hot mixed asphalt.
- (b) The Marshall characteristics, simple performance tests (SPTs), ageing resistance and morphological properties of 0.5% HDPP optimum dry mix are better than the control

and could impact upon rheology, strength and durability of asphalt. The strength characterization and SPT criteria of 0.5% HDPP asphalt mix could withstand heavy traffic situation ( $1 \times 10^6$  ESAL).

### **5.3 Suggested Further Research**

The following studies are suggested for further knowledge and contribution to this research:

- (a) Longer duration of performance testing as well as actual field data collection will validate the perceived quality assurance of HDPP asphalt mix as it might correlates with laboratory tests on rheology, strength and durability.

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## **APPENDICES**

**APPENDIX A1: COARSE AGGREGATE SIEVE ANALYSIS**

BS Sieve Size	Material Retained (g)	Retained Percent	Cumulative Retained	Cumulative Passing (%)
1" (25.4mm)	-	-	0	100.0
¾" (19mm)	49.0	4.9	4.9	95.1
½" (12.7mm)	175.0	17.5	22.4	77.6
3/8" (9mm)	166.0	16.6	39.0	61.0
¼" (6mm)	229.0	22.9	61.9	38.1
No. 4 (4.75mm)	186.0	18.6	80.5	19.5
No.7 (2.36mm)	182.0	18.2	98.7	1.4
No.14 (1.18mm)	14.0	1.4	100	-
	$\Sigma = 1000$			

**APPENDIX A2: FINE AGGREGATE SIEVE ANALYSIS**

BS Sieve Size	Material Retained (g)	Retained (%)	Cumulative Retained (%)	Cumulative Passing (%)
No. 4 (4.75mm)	-	-	-	100
No.7 (2.36mm)	10.5	2.1	2.1	97.9
No.14 (1.18mm)	147.5	29.5	31.6	68.4
No.25 (0.60mm)	131.5	26.3	57.9	42.1
No.52 (0.30mm)	100.0	20.0	77.9	22.1
No.100 (0.15mm)	77.5	15.5	93.4	6.6
No.200 (0.075mm)	15.5	3.1	96.5	3.5
Pan	17.0 (0.2 loss)	3.4	99.9 +(0.1loss)	(0.1loss)
	$\Sigma = 499.8$			

**APPENDIX A3: FILLER SIEVE ANALYSIS (CEMENT)**

BS Sieve Size	Material Retained (g)	Retained Percent	Cumulative Retained	Total Percent passing
No.25 (0.60mm)	-	-	0	100
No.52 (0.30mm)	9.5	1.9	1.9	98.1
No.100 (0.15mm)	24.0	4.8	6.7	93.3
No.200 (0.075mm)	65.0	13.0	19.7	80.4
Pan	402.0	80.4	100	-
	$\Sigma = 500$			

## APPENDIX B1: BULK SPECIFIC GRAVITY OF WET MIX

SN	BC %	SAMPLE A				SAMPLE B				SAMPLE C				Average BSG, g/cm <sup>3</sup>
		wt in air (A), g	wt in air SSD (B), g	wt in H2O {C}, g	BSG {1} g/cm <sup>3</sup>	wt in air (A), g	wt in air SSD (B), g	wt in H2O {C}, g	BSG {2}, g/cm <sup>3</sup>	wt in air (A), g	wt in air SSD (B), g	wt in H2O {C}, g	BSG {3}, g/cm <sup>3</sup>	
1	4.5	1188.4	1192.9	528.8	2.393	1191.3	1203.4	530.7	2.385	1196.7	1200.5	527.7	2.383	2.387
2	5	1191.6	1192.8	525.5	2.427	1187.2	1189.6	531.6	2.418	1166.9	1171	515.3	2.418	2.421
3	5.5	1198.3	1185.2	521.1	2.495	1159	1169.5	525.8	2.486	1155.2	1161.1	518.8	2.486	2.489
4	6	1175.7	1177.7	522.2	2.472	1190.2	1209.7	545.4	2.463	1170.1	1170.9	509.9	2.463	2.466
5	6.5	1168.3	1178.9	516.3	2.451	1151.4	1152.7	504.3	2.442	1157	1157.7	506.3	2.442	2.445
6	4.5	1167.9	1180.8	545	2.416	1158	1171.9	544.1	2.407	1154.9	1166.1	530.2	2.407	2.410
7	5	1191.9	1187.2	548.6	2.449	1178.7	1191.5	556.5	2.440	1163.7	1178.4	542.5	2.440	2.443
8	5.5	1181.8	1180.5	548.9	2.470	1163.5	1172.1	547.6	2.461	1153.4	1160.3	537.2	2.461	2.464
9	6	1199.8	1222.3	566.6	2.457	1155.2	1162.7	533.7	2.448	1198.5	1210.6	564.2	2.448	2.451
10	6.5	1173.7	1180.2	530.7	2.420	1185.7	1192.2	549.2	2.411	1189.9	1195	542.7	2.411	2.414
11	4.5	1180.9	1208.4	569.9	2.412	1171.7	1195.8	565.1	2.403	1192.8	1207.6	576.2	2.403	2.406
12	5	1192.4	1213.1	573.3	2.444	1186.2	1205.1	572.5	2.435	1184.6	1204	569.5	2.435	2.438
13	5.5	1197.8	1211.4	585.5	2.465	1190.7	1208.7	579.3	2.456	1197	1214.8	578.5	2.456	2.459
14	6	1181.5	1194.8	568.4	2.442	1190.9	1204.8	572.4	2.433	1188.5	1209.3	576	2.433	2.436
15	6.5	1185.1	1206.7	568.6	2.395	1186.3	1198	565.6	2.386	1174.8	1192.8	562.8	2.386	2.389
16	4.5	1173.8	1190.8	569.6	2.407	1167.9	1183.7	560.8	2.398	1188.9	1204.5	569.5	2.398	2.401
17	5	1176.3	1193.8	574.1	2.430	1191	1209.9	580.6	2.421	1187.7	1197.7	564.1	2.421	2.424
18	5.5	1187.6	1195.6	573.7	2.460	1158.1	1185.7	575.7	2.451	1177.3	1180.1	577.2	2.451	2.454
19	6	1161.7	1179.1	565.4	2.427	1171.1	1201.2	583.5	2.418	1174.1	1190.2	570.7	2.418	2.421
20	6.5	1190.3	1198.4	565.9	2.381	1184.5	1193.5	562.7	2.372	1199.5	1211.3	572	2.372	2.375
21	4.5	1165	1170.6	540.9	2.403	1182.7	1195.5	557.9	2.394	1167.6	1189.9	572.2	2.394	2.397
22	5	1185.7	1205.8	574.9	2.425	1162.3	1176.3	560.9	2.416	1179.3	1195.2	578.1	2.416	2.419
23	5.5	1193.3	1204.7	570.9	2.456	1141.2	1176.5	589.3	2.447	1195.8	1202.9	589.5	2.447	2.450
24	6	1162.9	1173.8	543.1	2.413	1171.9	1186.3	566.1	2.404	1167.4	1203.8	585.2	2.404	2.407
25	6.5	1192.6	1198.5	549.1	2.366	1165.4	1185.4	552	2.357	1178.5	1211.7	587.3	2.357	2.360
26	4.5	1133.8	1142.5	505.4	2.399	1139.2	1144.5	484.6	2.390	1148.8	1152.2	523.6	2.390	2.393
27	5	1148.6	1158.5	536.1	2.421	1129.3	1156.9	587	2.412	1129.4	1174	578	2.412	2.415
28	5.5	1159.6	1196.2	606.8	2.451	1149.6	1168.7	597	2.442	1144.4	1185.7	604.6	2.442	2.445
29	6	1142.8	1189.9	601.4	2.398	1111.9	1146.1	571.2	2.389	1150.3	1193.1	604.8	2.389	2.392
30	6.5	1160.8	1199.9	600.4	2.362	1197.4	1236.3	623.7	2.353	1152.1	1191.1	601.2	2.353	2.356
31	4.5	1122.6	1171	595.9	2.394	1153	1190.4	604.8	2.385	1178.1	1232.9	605.1	2.385	2.388
32	5	1113	1162.7	597.4	2.416	1186.1	1234.4	644.7	2.407	1150	1185.9	586.3	2.407	2.410
33	5.5	1164.7	1228.1	649.8	2.447	1142.8	1186.6	634.2	2.438	1148.6	1166.1	560.5	2.438	2.441
34	6	1149.6	1202.1	602.2	2.384	1116.1	1157.6	612.6	2.375	1145.6	1159.1	557.1	2.375	2.378
35	6.5	1153.8	1190.7	581.5	2.357	1143.4	1172	586.5	2.348	1167.7	1188	590.1	2.348	2.351

## APPENDIX B2: BULK SPECIFIC GRAVITY OF DRY MIX

SN	BC %	SAMPLE A				SAMPLE B				SAMPLE C				Average BSG, g/cm <sup>3</sup>
		wt in air (A), g	wt in air SSD (B), g	wt in H2O {C}, g	BSG {1} g/cm <sup>3</sup>	wt in air (A), g	wt in air SSD (B), g	wt in H2O {C}, g	BSG {2}, g/cm <sup>3</sup>	wt in air (A), g	wt in air SSD (B), g	wt in H2O {C}, g	BSG {3}, g/cm <sup>3</sup>	
1	4.5	1188.4	1192.9	528.8	2.390	1191.3	1203.4	530.7	2.385	1196.7	1200.5	527.7	2.386	2.387
2	5	1191.6	1192.8	529.5	2.424	1187.2	1189.6	531.6	2.419	1166.9	1171	515.3	2.420	2.421
3	5.5	1198.3	1185.2	521.1	2.492	1159	1169.5	525.8	2.487	1155.2	1161.1	518.8	2.488	2.489
4	6	1175.7	1177.7	522.2	2.469	1190.2	1209.7	545.4	2.464	1170.1	1170.9	509.9	2.465	2.466
5	6.5	1168.3	1178.9	516.3	2.448	1151.4	1152.7	504.3	2.443	1157	1157.7	506.3	2.444	2.445
6	4.5	1159.2	1165	510.1	2.232	1164.8	1168.6	518.1	2.227	1168.3	1173.4	512.2	2.228	2.229
7	5	1172.7	1175.5	515.6	2.299	1175.5	1178.7	524.9	2.294	1179.5	1186.5	527.8	2.295	2.296
8	5.5	1178.7	1180.2	522.7	2.360	1170.2	1173.6	527.5	2.355	1180.2	1181.9	519.2	2.356	2.357
9	6	1169.7	1171.4	507.8	2.251	1170.4	1171.7	511.3	2.246	1169.5	1171.1	511.8	2.247	2.248
10	6.5	1175.5	1177	509.7	2.221	1191.8	1192	518.7	2.216	1176	1178.2	516.6	2.217	2.218
11	4.5	1179.5	1182.9	513.3	2.081	1179.2	1183.7	526.5	2.076	1166.9	1171.9	518.7	2.077	2.078
12	5	1168	1171	518.2	2.182	1169.8	1171.9	508	2.177	1181.4	1182.9	520.2	2.178	2.179
13	5.5	1169.7	1171.3	516.5	2.261	1149.7	1151.6	508.1	2.256	1173.1	1176.5	524.6	2.257	2.258
14	6	1180.5	1189.7	528.5	2.118	1167.6	1168.7	513.1	2.113	1186.8	1188.4	521.2	2.114	2.115
15	6.5	1184.6	1184.6	517.4	2.093	1180.3	1180.8	513.6	2.088	1203	1204.2	532	2.089	2.090
16	4.5	1154.5	1158.4	504.2	1.933	1192.7	1195.6	523.4	1.928	1174.2	1177.8	515.7	1.929	1.930
17	5	1166.8	1169.5	515.7	1.989	1168.1	1171.1	518.4	1.984	1157.7	1160.8	512.2	1.985	1.986
18	5.5	1162.5	1165.9	514.8	2.164	1186.5	1188.2	530.4	2.159	1152	1152.8	506.6	2.160	2.161
19	6	1164.2	1165.7	510.6	1.987	1170.9	1172.9	519.9	1.982	1168.4	1170	522.3	1.983	1.984
20	6.5	1176.7	1177.6	514	1.906	1169.4	1170.2	509.1	1.901	1161.3	1162.2	506	1.902	1.903
21	4.5	1177	1179.7	515.7	1.789	1162.1	1167.5	514	1.784	1156.9	1161.3	514	1.785	1.786
22	5	1186.1	1189.4	522.8	1.928	1179.1	1180.4	520.7	1.923	1171.9	1172.9	512.4	1.924	1.925
23	5.5	1182.9	1185.7	521.7	1.973	1167.5	1168.7	521.6	1.968	1188.4	1189.9	520.4	1.969	1.970
24	6	1184.5	1185.4	518.6	1.857	1160.2	1161.7	516.3	1.852	1195.8	1196.8	523.5	1.853	1.854
25	6.5	1177.7	1178.9	513.3	1.741	1158	1159.6	510.4	1.736	1189.9	1191.6	528.4	1.737	1.738
26	4.5	1171.4	1177.4	522.7	1.650	1180.1	1189.4	528.4	1.645	1194.7	1201.9	525.4	1.646	1.647
27	5	1181.3	1188.2	531.5	1.691	1195.7	1204.4	537.8	1.686	1183.8	1188.9	519.6	1.687	1.688
28	5.5	1187.9	1188.5	530.6	1.838	1176.9	1182.9	526.9	1.833	1180.9	1182.8	511.6	1.834	1.835
29	6	1183.8	1185.4	527.3	1.729	1186.7	1188.1	524	1.724	1191.1	1195.4	523.9	1.725	1.726
30	6.5	1164.9	1165.9	515.7	1.617	1200.6	1201.8	530.8	1.612	1175	1176.1	512	1.613	1.614
31	4.5	1171	1175.7	519.8	1.513	1163.9	1168.1	520.4	1.508	1180.8	1182.9	508.3	1.509	1.510
32	5	1139.2	1141.8	505.4	1.557	1180.1	1182.6	528.6	1.552	1141.6	1164.4	504.2	1.553	1.554
33	5.5	1167.1	1170	530.6	1.675	1171.3	1175.1	515.6	1.670	1163.3	1174.6	506.7	1.671	1.672
34	6	1197.1	1209.5	595.6	1.602	1293.2	1305.9	601.8	1.597	1308	1314.4	503.4	1.598	1.599
35	6.5	1151.8	1153.4	510.9	1.574	1150.4	1152.2	501.2	1.569	1161.9	1164	501.8	1.570	1.571

**APPENDIX B3: MAXIMUM SPECIFIC GRAVITY OF WET MIX**

S/ N	BC, %	SAMPLE A					SAMPLE B					SAMPLE C					Avg. G <sub>mm</sub> , g/cm <sup>3</sup>
		wt in air (B), g	wt in air SSD (A), g	wt of Flask+ H2O {D}, g	wt of Flask+ H2O +sampl {E}, g	G <sub>mm</sub> {1}, g/cm <sup>3</sup>	wt in air (B), g	wt in air SSD (A), g	wt of Flask+ H2O {D}, g	wt of Flask+ H2O +sampl {E}, g	G <sub>mm</sub> {2}, g/cm <sup>3</sup>	wt in air (B), g	wt in air SSD (A), g	wt of Flask+ H2O {D}, g	wt of Flask+ H2O +sampl {E}, g	G <sub>mm</sub> {3}, g/cm <sup>3</sup>	
1	4.5	8935.9	7394.1	2501.1	8599.7	2.606	8935.7	7394.0	2501.1	8610.4	2.616	8935.8	7394.0	2501.1	8602.2	2.608	2.611
2	5	8935.3	7394.1	2501.3	8597.0	2.604	8935.1	7393.9	2501.2	8607.7	2.614	8935.2	7394.0	2501.2	8599.6	2.606	2.609
3	5.5	8927.4	7394.1	2501.4	8565.8	2.583	8927.2	7393.9	2501.3	8576.6	2.593	8927.3	7394.0	2501.3	8568.3	2.585	2.588
4	6	8913.6	7394.0	2501.5	8510.8	2.546	8913.4	7393.9	2501.4	8522.0	2.556	8913.5	7394.0	2501.5	8513.6	2.548	2.551
5	6.5	8901.9	7394.0	2501.6	8464.6	2.516	8901.8	7393.9	2501.6	8476.1	2.526	8901.8	7393.9	2501.6	8467.4	2.518	2.521
6	4.5	8919.7	7394.0	2501.8	8535.0	2.562	8919.5	7393.8	2501.7	8546.0	2.572	8919.6	7393.9	2501.7	8537.6	2.564	2.567
7	5	8919.5	7393.9	2501.8	8534.2	2.561	8919.3	7393.7	2501.8	8545.3	2.571	8919.4	7393.8	2501.8	8536.9	2.563	2.566
8	5.5	8912.4	7393.8	2501.9	8506.0	2.542	8912.2	7393.7	2501.9	8517.2	2.552	8912.3	7393.7	2501.9	8508.7	2.545	2.547
9	6	8880.7	7393.7	2502.0	8380.7	2.463	8880.5	7393.6	2501.9	8392.6	2.473	8880.6	7393.7	2502.0	8383.7	2.465	2.468
10	6.5	8867.8	7393.7	2502.1	8329.6	2.432	8867.6	7393.5	2502.0	8341.9	2.442	8867.7	7393.6	2502.1	8332.7	2.435	2.437
11	4.5	8928.6	7393.6	2502.6	8569.7	2.584	8928.4	7393.4	2502.5	8580.5	2.594	8928.5	7393.5	2502.5	8572.3	2.586	2.589
12	5	8921.1	7393.4	2502.6	8540.4	2.564	8920.9	7393.3	2502.6	8551.4	2.574	8921.0	7393.4	2502.6	8543.1	2.567	2.569
13	5.5	8918.4	7393.3	2502.7	8530.1	2.557	8918.2	7393.2	2502.6	8541.1	2.567	8918.3	7393.2	2502.7	8532.7	2.560	2.562
14	6	8890.6	7393.2	2502.8	8420.2	2.487	8890.4	7393.0	2502.7	8432.0	2.497	8890.5	7393.1	2502.7	8423.1	2.489	2.492
15	6.5	8872.4	7393.1	2502.8	8348.7	2.443	8872.3	7392.9	2502.8	8360.9	2.453	8872.3	7393.0	2502.8	8351.8	2.445	2.448
16	4.5	8936.3	7392.9	2502.6	8602.5	2.606	8936.1	7392.8	2502.5	8613.2	2.616	8936.2	7392.8	2502.5	8605.0	2.609	2.611
17	5	8929.0	7392.7	2503.5	8572.2	2.585	8928.8	7392.6	2503.4	8583.1	2.595	8928.9	7392.7	2503.5	8574.8	2.587	2.590
18	5.5	8924.3	7392.6	2503.6	8554.1	2.572	8924.1	7392.4	2503.5	8565.0	2.582	8924.2	7392.5	2503.5	8556.7	2.575	2.577
19	6	8900.2	7392.4	2503.6	8459.4	2.511	8900.0	7392.2	2503.5	8470.9	2.521	8900.1	7392.3	2503.6	8462.2	2.513	2.516
20	6.5	8877.1	7392.2	2503.7	8368.5	2.454	8876.9	7392.1	2503.6	8380.6	2.464	8877.0	7392.1	2503.6	8371.5	2.457	2.459
21	4.5	8945.6	7392.0	2504.4	8638.6	2.629	8945.4	7391.9	2504.3	8649.1	2.639	8945.5	7391.9	2504.3	8641.1	2.632	2.634
22	5	8936.0	7391.8	2504.4	8601.3	2.604	8935.8	7391.6	2504.4	8612.0	2.614	8935.9	7391.7	2504.4	8603.9	2.606	2.609
23	5.5	8936.1	7391.6	2504.5	8602.2	2.604	8935.9	7391.4	2504.4	8612.9	2.614	8936.0	7391.5	2504.4	8604.7	2.607	2.609
24	6	8909.5	7391.3	2504.5	8498.0	2.535	8909.4	7391.2	2504.4	8509.3	2.545	8909.4	7391.3	2504.5	8500.7	2.537	2.540
25	6.5	8886.9	7391.1	2504.5	8409.3	2.478	8886.7	7391.0	2504.5	8421.1	2.488	8886.8	7391.0	2504.5	8412.2	2.481	2.483
26	4.5	8953.7	7390.9	2505.3	8672.8	2.653	8953.5	7390.7	2505.3	8683.1	2.663	8953.6	7390.8	2505.3	8675.1	2.655	2.658
27	5	8946.7	7390.6	2505.4	8645.7	2.634	8946.5	7390.4	2505.3	8656.2	2.644	8946.6	7390.5	2505.3	8648.1	2.636	2.639
28	5.5	8938.4	7390.3	2505.4	8613.7	2.611	8938.2	7390.2	2505.3	8624.4	2.621	8938.3	7390.2	2505.4	8616.2	2.614	2.616
29	6	8921.6	7390.0	2505.4	8548.3	2.567	8921.4	7389.9	2505.4	8559.3	2.577	8921.5	7390.0	2505.4	8550.9	2.570	2.572
30	6.5	8899.9	7389.8	2505.5	8463.1	2.512	8899.7	7389.6	2505.4	8474.6	2.522	8899.8	7389.7	2505.4	8466.0	2.514	2.517
31	4.5	8961.6	7389.5	2506.3	8706.6	2.676	8961.4	7389.3	2506.2	8716.7	2.686	8961.5	7389.4	2506.3	8708.9	2.678	2.681
32	5	8959.3	7389.1	2506.3	8698.6	2.670	8959.1	7389.0	2506.3	8708.7	2.680	8959.2	7389.1	2506.3	8700.9	2.673	2.675
33	5.5	8953.7	7388.8	2506.3	8677.5	2.655	8953.5	7388.7	2506.3	8687.7	2.665	8953.6	7388.7	2506.3	8679.8	2.658	2.660
34	6	8937.4	7388.5	2506.4	8614.1	2.611	8937.2	7388.3	2506.3	8624.7	2.621	8937.3	7388.4	2506.3	8616.6	2.613	2.616
35	6.5	8912.7	7388.2	2506.4	8517.6	2.546	8912.5	7388.0	2506.4	8528.8	2.556	8912.6	7388.1	2506.4	8520.3	2.549	2.551

**APPENDIX B4: MAXIMUM SPECIFIC GRAVITY OF DRY MIX**

S/ N	BC, %	SAMPLE A					SAMPLE B					SAMPLE C					Avg. G <sub>mm</sub> , g/cm <sup>3</sup>
		wt in air (B), g	wt in air SSD (A), g	wt of Flask+ H2O {D}, g	wt of Flask+ H2O +saml {E}, g	G <sub>mm</sub> {1}, g/cm <sup>3</sup>	wt in air (B), g	wt in air SSD (A), g	wt of Flask+ H2O {D}, g	wt of Flask+ H2O +saml {E}, g	G <sub>mm</sub> {2}, g/cm <sup>3</sup>	wt in air (B), g	wt in air SSD (A), g	wt of Flask+ H2O {D}, g	wt of Flask+ H2O +saml {E}, g	G <sub>mm</sub> {3}, g/cm <sup>3</sup>	
1	4.5	8939.6	7394.1	2501.1	8614.2	2.616	8939.4	7394.0	2501.1	8603.2	2.611	8939.5	7394.0	2501.1	8605.9	2.608	2.611
2	5	8939.0	7394.1	2501.3	8611.6	2.614	8938.8	7393.9	2501.2	8600.5	2.609	8938.9	7394.0	2501.2	8603.2	2.606	2.609
3	5.5	8931.2	7394.1	2501.4	8580.5	2.593	8931.0	7393.9	2501.3	8569.3	2.588	8931.1	7394.0	2501.3	8572.1	2.585	2.588
4	6	8917.4	7394.0	2501.5	8526.1	2.556	8917.3	7393.9	2501.4	8514.5	2.551	8917.3	7394.0	2501.5	8517.4	2.548	2.551
5	6.5	8905.9	7394.0	2501.6	8480.2	2.526	8905.7	7393.9	2501.6	8468.4	2.521	8905.8	7393.9	2501.6	8471.4	2.518	2.521
6	4.5	8879.5	7394.0	2501.8	8375.6	2.460	8879.5	7394.0	2501.8	8363.3	2.455	8879.4	7393.9	2501.7	8366.4	2.453	2.455
7	5	8880.6	7393.9	2501.8	8380.1	2.463	8880.6	7393.9	2501.8	8367.8	2.458	8880.5	7393.8	2501.8	8370.9	2.455	2.458
8	5.5	8889.8	7393.8	2501.9	8416.5	2.485	8889.8	7393.8	2501.9	8404.5	2.480	8889.7	7393.8	2501.9	8407.4	2.478	2.480
9	6	8830.8	7393.8	2502.0	8183.3	2.348	8830.8	7393.8	2502.0	8169.8	2.343	8830.7	7393.7	2502.0	8173.3	2.340	2.343
10	6.5	8808.2	7393.7	2502.1	8093.7	2.299	8808.2	7393.7	2502.1	8079.7	2.294	8808.1	7393.6	2502.1	8083.4	2.291	2.294
11	4.5	8816.9	7393.6	2502.6	8127.4	2.316	8816.9	7393.6	2502.6	8113.6	2.311	8816.8	7393.5	2502.5	8117.2	2.309	2.311
12	5	8844.1	7393.5	2502.6	8235.4	2.376	8844.1	7393.5	2502.6	8222.3	2.371	8844.0	7393.4	2502.6	8225.6	2.369	2.371
13	5.5	8858.5	7393.4	2502.7	8292.9	2.410	8858.6	7393.4	2502.7	8280.2	2.405	8858.5	7393.3	2502.7	8283.4	2.402	2.405
14	6	8783.0	7393.3	2502.8	7994.3	2.246	8783.0	7393.3	2502.8	7979.6	2.241	8782.9	7393.2	2502.7	7983.6	2.239	2.241
15	6.5	8751.1	7393.1	2502.8	7868.4	2.184	8751.1	7393.1	2502.8	7852.8	2.179	8751.0	7393.1	2502.8	7857.1	2.177	2.179
16	4.5	8752.0	7393.0	2502.6	7872.8	2.186	8752.0	7393.0	2502.6	7857.3	2.181	8751.9	7392.9	2502.5	7861.6	2.179	2.181
17	5	8772.5	7392.8	2503.5	7952.6	2.224	8772.5	7392.8	2503.5	7937.6	2.219	8772.4	7392.8	2503.5	7941.7	2.217	2.219
18	5.5	8834.5	7392.7	2503.6	8198.5	2.355	8834.5	7392.7	2503.6	8185.1	2.350	8834.4	7392.6	2503.5	8188.6	2.347	2.350
19	6	8718.3	7392.5	2503.6	7739.1	2.123	8718.3	7392.5	2503.6	7722.7	2.118	8718.2	7392.4	2503.6	7727.3	2.115	2.118
20	6.5	8649.2	7392.3	2503.7	7466.3	2.005	8649.2	7392.3	2503.7	7447.8	2.000	8649.1	7392.2	2503.6	7453.3	1.998	2.000
21	4.5	8694.6	7392.1	2504.4	7645.0	2.080	8694.6	7392.1	2504.4	7627.9	2.075	8694.5	7392.0	2504.4	7632.8	2.073	2.075
22	5	8745.1	7391.9	2504.4	7845.4	2.171	8745.1	7391.9	2504.4	7829.7	2.166	8745.0	7391.8	2504.4	7834.0	2.164	2.166
23	5.5	8736.2	7391.7	2504.5	7810.9	2.155	8736.2	7391.7	2504.5	7794.9	2.150	8736.1	7391.6	2504.4	7799.4	2.148	2.150
24	6	8649.3	7391.5	2504.5	7467.9	2.005	8649.3	7391.5	2504.5	7449.5	2.000	8649.2	7391.4	2504.5	7454.9	1.998	2.000
25	6.5	8541.7	7391.2	2504.5	7042.9	1.846	8541.7	7391.3	2504.5	7021.1	1.841	8541.6	7391.2	2504.5	7028.0	1.839	1.841
26	4.5	8606.1	7391.0	2505.3	7296.9	1.938	8606.1	7391.0	2505.3	7277.2	1.933	8606.0	7390.9	2505.3	7283.2	1.931	1.933
27	5	8605.7	7390.8	2505.4	7296.3	1.937	8605.7	7390.8	2505.4	7276.5	1.932	8605.6	7390.7	2505.3	7282.5	1.930	1.932
28	5.5	8666.0	7390.5	2505.4	7535.7	2.033	8666.0	7390.5	2505.4	7517.8	2.028	8665.9	7390.4	2505.4	7523.1	2.026	2.028
29	6	8571.0	7390.2	2505.4	7160.7	1.887	8571.0	7390.2	2505.4	7139.9	1.882	8570.9	7390.1	2505.4	7146.3	1.880	1.882
30	6.5	8449.3	7389.9	2505.5	6680.1	1.729	8449.3	7389.9	2505.5	6655.3	1.724	8449.2	7389.9	2505.4	6663.4	1.722	1.724
31	4.5	8537.4	7389.6	2506.3	7028.1	1.840	8537.4	7389.6	2506.3	7006.1	1.835	8537.3	7389.6	2506.3	7013.0	1.833	1.835
32	5	8518.4	7389.3	2506.3	6953.8	1.815	8518.4	7389.3	2506.3	6931.2	1.810	8518.3	7389.3	2506.3	6938.4	1.808	1.810
33	5.5	8563.0	7389.0	2506.3	7131.5	1.876	8563.0	7389.0	2506.3	7110.4	1.871	8562.9	7389.0	2506.3	7116.9	1.870	1.871
34	6	8472.1	7388.7	2506.4	6773.1	1.757	8472.2	7388.7	2506.4	6749.0	1.752	8472.1	7388.6	2506.4	6756.8	1.750	1.752
35	6.5	8437.3	7388.4	2506.4	6636.2	1.715	8437.3	7388.4	2506.4	6611.0	1.710	8437.2	7388.3	2506.4	6619.2	1.709	1.710

**APPENDIX B5: VMA OF WET MIX**

VMA WET MIX %				
SN	BC, %	% Ps	Avg. Gmb, g/cm <sup>3</sup>	VMA
1	4.5	95.5	2.387	15.9
2	5	95	2.421	15.1
3	5.5	94.5	2.489	13.2
4	6	94	2.466	14.5
5	6.5	93.5	2.445	15.6
6	4.5	95	2.388	16.3
7	5	94.5	2.410	16.0
8	5.5	94	2.441	15.3
9	6	93.5	2.378	18.0
10	6.5	93	2.351	19.3
11	4.5	94.5	2.393	16.6
12	5	94	2.415	16.2
13	5.5	93.5	2.445	15.6
14	6	93	2.392	17.9
15	6.5	92.5	2.356	19.6
16	4.5	94	2.397	16.9
17	5	93.5	2.419	16.5
18	5.5	93	2.450	15.9
19	6	92.5	2.407	17.9
20	6.5	92	2.360	19.9
21	4.5	93.5	2.401	17.2
22	5	93	2.424	16.8
23	5.5	92.5	2.454	16.2
24	6	92	2.421	17.8
25	6.5	91.5	2.375	19.8
26	4.5	93	2.406	17.4
27	5	92.5	2.438	16.8
28	5.5	92	2.459	16.5
29	6	91.5	2.436	17.8
30	6.5	91	2.389	19.8
31	4.5	92.5	2.410	17.7
32	5	92	2.443	17.1
33	5.5	91.5	2.464	16.8
34	6	91	2.451	17.7
35	6.5	90.5	2.414	19.4

**APPENDIX B6: VMA OF DRY MIX**

VMA DRY MIX %				
SN	BC, %	% Ps	avg. Gmb, g/cm <sup>3</sup>	VMA, %
1	4.5	95.5	2.387	15.9
2	5	95	2.421	15.1
3	5.5	94.5	2.489	13.2
4	6	94	2.466	14.5
5	6.5	93.5	2.445	15.6
6	4.5	95	2.229	21.9
7	5	94.5	2.296	19.9
8	5.5	94	2.357	18.2
9	6	93.5	2.248	22.4
10	6.5	93	2.218	23.9
11	4.5	94.5	2.078	27.6
12	5	94	2.179	24.4
13	5.5	93.5	2.258	22.1
14	6	93	2.115	27.4
15	6.5	92.5	2.090	28.7
16	4.5	94	1.930	33.1
17	5	93.5	1.986	31.5
18	5.5	93	2.161	25.8
19	6	92.5	1.984	32.3
20	6.5	92	1.903	35.4
21	4.5	93.5	1.786	38.4
22	5	93	1.925	33.9
23	5.5	92.5	1.970	32.8
24	6	92	1.854	37.1
25	6.5	91.5	1.738	41.3
26	4.5	93	1.647	43.5
27	5	92.5	1.688	42.4
28	5.5	92	1.835	37.7
29	6	91.5	1.726	41.7
30	6.5	91	1.614	45.8
31	4.5	92.5	1.510	48.4
32	5	92	1.554	47.3
33	5.5	91.5	1.672	43.6
34	6	91	1.599	46.3
35	6.5	90.5	1.571	47.5

**APPENDIX B7: VIM OF WET MIX**

VIM WET MIX				
SN	BC, %	% Ps	Avg. Gmb, g/cm <sup>3</sup>	VMA
1	4.5	2.611	2.446	6.32
2	5	2.609	2.477	5.06
3	5.5	2.588	2.489	3.81
4	6	2.551	2.466	3.33
5	6.5	2.521	2.445	3.01
6	4.5	2.544	2.388	6.13
7	5	2.535	2.410	4.91
8	5.5	2.534	2.441	3.70
9	6	2.457	2.378	3.23
10	6.5	2.422	2.351	2.92
11	4.5	2.545	2.393	6.00
12	5	2.537	2.415	4.81
13	5.5	2.537	2.445	3.62
14	6	2.470	2.392	3.16
15	6.5	2.425	2.356	2.86
16	4.5	2.543	2.397	5.75
17	5	2.536	2.419	4.60
18	5.5	2.538	2.450	3.47
19	6	2.482	2.407	3.03
20	6.5	2.427	2.360	2.74
21	4.5	2.544	2.401	5.62
22	5	2.538	2.424	4.50
23	5.5	2.541	2.454	3.39
24	6	2.495	2.421	2.96
25	6.5	2.440	2.375	2.68
26	4.5	2.544	2.406	5.44
27	5	2.549	2.438	4.35
28	5.5	2.542	2.459	3.28
29	6	2.508	2.436	2.86
30	6.5	2.453	2.389	2.59
31	4.5	2.543	2.410	5.25
32	5	2.550	2.443	4.20
33	5.5	2.544	2.464	3.16
34	6	2.520	2.451	2.76
35	6.5	2.476	2.414	2.50

**APPENDIX B8: VIM OF DRY MIX**

VIM DRY MIX				
SN	BC, %	% Ps	avg. Gmb, g/cm <sup>3</sup>	VMA, %
1	4.5	2.611	2.446	6.32
2	5	2.609	2.477	5.06
3	5.5	2.588	2.489	3.81
4	6	2.551	2.466	3.33
5	6.5	2.521	2.445	3.01
6	4.5	2.455	2.229	9.22
7	5	2.458	2.296	6.58
8	5.5	2.480	2.357	4.95
9	6	2.343	2.248	4.03
10	6.5	2.294	2.218	3.32
11	4.5	2.311	2.078	10.11
12	5	2.371	2.179	8.10
13	5.5	2.405	2.258	6.10
14	6	2.241	2.115	5.63
15	6.5	2.179	2.090	4.08
16	4.5	2.181	1.930	11.51
17	5	2.219	1.986	10.53
18	5.5	2.350	2.161	8.04
19	6	2.118	1.984	6.33
20	6.5	2.000	1.903	4.85
21	4.5	2.075	1.786	13.90
22	5	2.166	1.925	11.13
23	5.5	2.150	1.970	8.38
24	6	2.000	1.854	7.33
25	6.5	1.841	1.738	5.61
26	4.5	1.933	1.647	14.80
27	5	1.932	1.688	12.65
28	5.5	2.028	1.835	9.53
29	6	1.882	1.726	8.33
30	6.5	1.724	1.614	6.38
31	4.5	1.835	1.510	17.70
32	5	1.810	1.554	14.17
33	5.5	1.871	1.672	10.67
34	6	1.752	1.599	8.72
35	6.5	1.710	1.571	8.14

**APPENDIX B9: VFB OF WET MIX**

VFB WET MIX %				
SN	BC,%	VMA, %	VIM, %	VFB, %
1	4.5	15.88495	6.32	60.21
2	5	14.69786	5.06	66.57
3	5.5	15.33229	3.81	71.15
4	6	14.46347	3.33	76.98
5	6.5	14.17014	3.01	80.76
6	4.5	16.28362	6.13	62.35
7	5	15.95065	4.91	69.23
8	5.5	15.34668	3.70	75.92
9	6	17.97128	3.23	82.03
10	6.5	19.31422	2.92	84.88
11	4.5	16.5712	6.00	63.77
12	5	16.23929	4.81	70.40
13	5.5	15.63742	3.62	76.85
14	6	17.90888	3.16	82.34
15	6.5	19.59204	2.86	85.40
16	4.5	16.86045	5.75	65.89
17	5	16.52964	4.60	72.14
18	5.5	15.9299	3.47	78.24
19	6	17.85189	3.03	83.03
20	6.5	19.87157	2.74	86.22
21	4.5	17.15137	5.62	67.20
22	5	16.82168	4.50	73.23
23	5.5	16.22412	3.39	79.10
24	6	17.80031	2.96	83.35
25	6.5	19.81515	2.68	86.48
26	4.5	17.44396	5.44	68.84
27	5	16.7741	4.35	74.06
28	5.5	16.52008	3.28	80.17
29	6	17.75414	2.86	83.87
30	6.5	19.76413	2.59	86.90
31	4.5	17.73821	5.25	70.43
32	5	17.07139	4.20	75.40
33	5.5	16.81778	3.16	81.20
34	6	17.71338	2.76	84.40
35	6.5	19.38455	2.50	87.11

**APPENDIX B10: VFB OF DRY MIX**

VFB DRY MIX %				
SN	BC, %	VMA, %	VIM, %	VFB, %
1	4.5	15.88495	6.32	60.21
2	5	14.69786	5.06	66.57
3	5.5	15.33229	3.81	71.15
4	6	14.46347	3.33	76.98
5	6.5	14.17014	3.01	80.76
6	4.5	21.86963	9.22	57.86
7	5	19.93563	6.58	67.00
8	5.5	18.23395	4.95	72.84
9	6	22.43386	4.03	82.04
10	6.5	23.89788	3.32	86.13
11	4.5	27.55305	10.11	63.30
12	5	24.40623	8.10	66.83
13	5.5	22.09451	6.10	72.41
14	6	27.4163	5.63	79.47
15	6.5	28.66813	4.08	85.77
16	4.5	33.05007	11.51	65.18
17	5	31.48684	10.53	66.56
18	5.5	25.84794	8.04	68.90
19	6	32.29358	6.33	80.41
20	6.5	35.38645	4.85	86.31
21	4.5	38.36278	13.90	63.76
22	5	33.92824	11.13	67.19
23	5.5	32.75989	8.38	74.41
24	6	37.06663	7.33	80.24
25	6.5	41.31918	5.61	86.42
26	4.5	43.49325	14.80	65.97
27	5	42.3853	12.65	70.15
28	5.5	37.71776	9.53	74.75
29	6	41.73634	8.33	80.05
30	6.5	45.80651	6.38	86.08
31	4.5	48.44359	17.70	63.47
32	5	47.25456	14.17	70.02
33	5.5	43.55549	10.67	75.51
34	6	46.30362	8.72	81.16
35	6.5	47.53545	8.14	82.88

**APPENDIX B11: STABILITY OF WET MIX**

SN	B C, %	Ht1, mm	Ht2, mm	Ht3, mm	Corr factor 1	Corr factor 2	Corr factor 3	Stab1, kN	Stab 2, kN	Stab 3, kN	corr Stab 1	corr Stab 2	corr Stab 3	avg stab. kN
1	4.5	63.1	62.7	63.3	1.04	1.04	1.04	5.94	5.82	6.19	6.18	6.05	7.08	6.44
2	5	62.9	63.2	62.9	1.04	1.04	1.04	7.91	7.75	8.24	8.23	8.06	9.43	8.57
3	5.5	63.3	63.4	63.3	1.04	1.04	1.04	13.40	13.12	13.96	13.94	13.65	15.97	14.52
4	6	63.4	62.9	63.1	1.04	1.04	1.04	11.13	10.90	11.60	11.58	11.34	13.27	12.06
5	6.5	63	62.7	62.8	1.04	1.04	1.04	7.45	7.29	7.76	7.75	7.59	8.88	8.07
6	4.5	62.8	63.1	63.5	1.04	1.04	1	7.39	7.24	8.01	7.69	7.53	8.81	8.01
7	5	63.4	63.5	63.4	1.04	1	1.04	12.08	12.30	12.58	12.56	12.30	14.39	13.08
8	5.5	62.9	63.3	62.7	1.04	1.04	1.04	16.08	15.75	16.75	16.73	16.38	19.17	17.42
9	6	62.7	63.5	63.3	1.04	1.04	1.04	13.54	13.26	14.11	14.09	13.79	16.14	14.67
10	6.5	63.4	62.9	62.6	1.04	1.04	1.04	7.65	7.49	7.97	7.95	7.79	9.11	8.28
11	4.5	63.1	63.4	62.4	1.04	1.04	1.04	9.39	9.20	9.78	9.77	9.57	11.19	10.18
12	5	63.4	63.5	62.5	1.04	1	1.04	14.40	14.66	15.00	14.97	14.66	17.16	15.60
13	5.5	62.6	62.7	63.4	1.04	1.04	1.04	17.56	17.20	18.30	18.27	17.89	20.93	19.03
14	6	63	63.4	63.3	1.04	1.04	1.04	14.11	13.81	14.70	14.67	14.37	16.81	15.28
15	6.5	62.8	63	62.8	1.04	1.04	1.04	10.98	10.75	11.44	11.42	11.18	13.09	11.90
16	4.5	62.5	62.8	63.1	1.04	1.04	1.04	15.01	14.70	15.63	15.61	15.28	17.89	16.26
17	5	63.4	63.4	62.6	1.04	1.04	1.04	19.25	18.85	20.05	20.02	19.60	22.94	20.86
18	5.5	63	62.7	63.3	1.04	1.04	1.04	23.80	23.30	24.79	24.75	24.23	28.36	25.78
19	6	63.1	63.5	62.8	1.04	1.04	1.04	19.01	18.61	19.80	19.77	19.35	22.65	20.59
20	6.5	62.7	62.8	62.8	1.04	1.04	1.04	17.64	17.27	18.37	18.34	17.96	21.02	19.11
21	4.5	62.9	63.1	63.5	1.04	1.04	1	17.01	16.66	18.43	17.69	17.32	20.27	18.43
22	5	63.1	63.5	63	1.04	1	1.04	19.73	20.09	20.55	20.51	20.09	23.51	21.37
23	5.5	62.7	62.9	62.9	1.04	1.04	1.04	25.55	25.02	26.62	26.58	26.02	30.45	27.68
24	6	63	63	62.8	1.04	1.04	1.04	22.34	21.87	23.27	23.23	22.75	26.62	24.20
25	6.5	63.4	62.7	63.4	1.04	1.04	1.04	18.94	18.55	19.73	19.70	19.29	22.57	20.52
26	4.5	63.5	62.8	63.1	1	1.04	1.04	14.89	14.02	14.92	14.89	14.58	17.06	15.51
27	5	62.8	63.5	63.4	1.04	1	1.04	18.58	18.92	19.35	19.32	18.92	22.14	20.13
28	5.5	63.4	62.9	63	1.04	1.04	1.04	22.28	21.82	23.21	23.17	22.69	26.55	24.14
29	6	62.7	63.1	63.3	1.04	1.04	1.04	20.78	20.34	21.64	21.61	21.16	24.76	22.51
30	6.5	62.8	63.4	62.6	1.04	1.04	1.04	16.92	16.56	17.62	17.59	17.23	20.16	18.33
31	4.5	63.5	62.9	62.5	1	1.04	1.04	14.73	13.87	14.75	14.73	14.42	16.88	15.34
32	5	63.1	63.4	63.3	1.04	1.04	1.04	15.80	15.47	16.45	16.43	16.09	18.82	17.11
33	5.5	63.4	62.7	62.7	1.04	1.04	1.04	21.44	21.00	22.34	22.30	21.84	25.56	23.23
34	6	62.9	62.8	63.1	1.04	1.04	1.04	17.44	17.08	18.17	18.14	17.76	20.79	18.90
35	6.5	62.7	63.3	62.8	1.04	1.04	1.04	15.98	15.65	16.65	16.62	16.27	19.04	17.31

**APPENDIX B12: STABILITY OF DRY MIX**

S N	B C, %	Ht1, mm	Ht2, mm	Ht3, mm	Corr factor 1	Corr factor 2	Corr factor 3	Stab1, kN	Stab2, kN	Stab3, kN	corr Stab 1	corr Stab 2	corr Stab 3	avg stab. kN
1	4.5	63.1	62.7	63.3	1.04	1.04	1.04	5.94	5.82	6.19	6.18	6.05	7.08	6.44
2	5	62.9	63.2	62.9	1.04	1.04	1.04	7.91	7.75	8.24	8.23	8.06	9.43	8.57
3	5.5	63.3	63.4	63.3	1.04	1.04	1.04	13.40	13.12	13.96	13.94	13.65	15.97	14.52
4	6	63.4	62.9	63.1	1.04	1.04	1.04	11.13	10.90	11.60	11.58	11.34	13.27	12.06
5	6.5	63.0	62.7	62.8	1.04	1.04	1.04	7.45	7.29	7.76	7.75	7.59	8.88	8.07
6	4.5	64.7	64.3	64.9	0.96	1	0.96	4.82	5.42	5.13	4.63	5.42	4.73	4.92
7	5	64.5	64.8	64.5	0.96	0.96	0.96	6.03	10.60	8.15	5.79	10.18	7.51	7.83
8	5.5	64.9	65	64.9	0.96	0.96	0.96	12.31	21.63	16.64	11.82	20.76	15.33	15.97
9	6	65.0	64.5	64.7	0.96	0.96	0.96	7.22	12.68	9.76	6.93	12.18	8.99	9.37
10	6.5	64.6	64.3	64.4	0.96	1	1	4.07	6.86	5.28	3.90	6.86	5.07	5.28
11	4.5	66.3	65.8	66.5	0.93	0.96	0.93	3.62	6.16	4.89	3.37	5.92	4.37	4.55
12	5	66.1	66.4	66.1	0.93	0.93	0.93	4.65	8.16	6.28	4.32	7.59	5.61	5.84
13	5.5	66.5	66.6	66.5	0.93	0.93	0.93	6.28	11.04	8.49	5.84	10.26	7.58	7.90
14	6	66.6	66.1	66.3	0.93	0.93	0.93	3.64	6.39	4.92	3.38	5.94	4.39	4.57
15	6.5	66.2	65.8	65.9	0.93	0.96	0.96	2.57	4.37	3.36	2.39	4.19	3.10	3.22
16	4.5	67.8	67.4	68.1	0.89	0.89	0.89	2.52	4.43	3.41	2.25	3.95	2.91	3.04
17	5	67.6	67.9	67.6	0.89	0.89	0.89	3.18	5.58	4.29	2.83	4.97	3.67	3.82
18	5.5	68.1	68.2	68.1	0.89	0.89	0.89	3.88	6.81	5.24	3.45	6.07	4.48	4.67
19	6	68.2	67.6	67.8	0.89	0.89	0.89	3.37	5.93	4.56	3.00	5.27	3.89	4.06
20	6.5	67.7	67.4	67.5	0.89	0.89	0.89	2.81	4.94	3.80	2.50	4.39	3.24	3.38
21	4.5	69.4	69.0	69.6	0.86	0.89	0.86	1.87	3.17	2.52	1.60	2.82	2.08	2.17
22	5	69.2	69.5	69.2	0.86	0.86	0.86	2.16	3.80	2.92	1.86	3.27	2.41	2.51
23	5.5	69.6	69.7	69.6	0.86	0.86	0.86	2.50	4.39	3.38	2.15	3.78	2.79	2.90
24	6	69.7	69.2	69.4	0.86	0.86	0.86	1.90	3.34	2.57	1.64	2.88	2.12	2.21
25	6.5	69.3	69.0	69.1	0.86	0.89	0.86	1.56	2.65	2.11	1.34	2.36	1.74	1.81
26	4.5	71.0	70.5	71.2	0.83	0.86	0.83	1.29	2.18	1.74	1.07	1.88	1.39	1.44
27	5	70.8	71.1	70.8	0.83	0.83	0.83	1.64	2.88	2.21	1.36	2.39	1.76	1.84
28	5.5	71.2	71.3	71.2	0.83	0.83	0.83	2.02	3.55	2.73	1.68	2.95	2.18	2.27
29	6	71.3	70.8	71.0	0.83	0.83	0.83	1.80	3.16	2.43	1.49	2.63	1.94	2.02
30	6.5	70.9	70.5	70.7	0.83	0.86	0.86	1.12	1.89	1.46	0.93	1.63	1.20	1.25
31	4.5	72.8	72.1	72.8	0.81	0.83	0.81	0.28	0.48	0.38	0.23	0.40	0.30	0.31
32	5	72.3	72.7	72.3	0.81	0.81	0.81	0.67	1.18	0.91	0.54	0.95	0.70	0.73
33	5.5	72.8	72.9	72.8	0.81	0.81	0.81	0.97	1.70	1.31	0.78	1.38	1.02	1.06
34	6	72.9	72.3 35	72.6	0.81	0.81	0.81	0.82	1.44	1.11	0.66	1.17	0.86	0.90
35	6.5	72.5	72.1	72.2	0.81	0.83	0.81	0.42	0.72	0.56	0.34	0.59	0.44	0.46

**APPENDIX B13: FLOW OF WET MIX**

SN	BC, %	F1, mm	F2, mm	F3, mm	avg. Flow mm
1	4.5	3.28	2.31	2.39	2.66
2	5	3.22	2.49	2.48	2.73
3	5.5	3.03	2.81	2.92	2.92
4	6	3.26	3.02	3.17	3.15
5	6.5	3.51	3.22	3.41	3.38
6	4.5	2.84	2.34	2.36	2.51
7	5	2.97	2.47	2.45	2.63
8	5.5	2.72	2.88	2.69	2.76
9	6	3.09	3.11	3.06	3.09
10	6.5	3.30	3.31	3.33	3.31
11	4.5	2.47	2.31	2.32	2.37
12	5	2.74	2.41	2.44	2.53
13	5.5	2.63	2.68	2.81	2.71
14	6	2.97	3.12	2.98	3.02
15	6.5	3.12	3.34	3.27	3.24
16	4.5	2.18	2.25	2.23	2.22
17	5	2.47	2.35	2.31	2.38
18	5.5	2.61	2.62	2.72	2.65
19	6	2.54	2.98	3.06	2.86
20	6.5	3.17	3.15	3.21	3.18
21	4.5	2.27	2.22	2.15	2.21
22	5	2.27	2.41	2.30	2.33
23	5.5	2.62	2.52	2.49	2.54
24	6	2.59	2.88	2.92	2.80
25	6.5	2.78	3.21	3.04	3.01
26	4.5	2.23	2.05	2.18	2.15
27	5	2.26	2.26	2.31	2.28
28	5.5	2.27	2.51	2.53	2.44
29	6	2.17	2.88	2.86	2.64
30	6.5	2.45	3.09	3.01	2.85
31	4.5	2.06	2.06	2.11	2.08
32	5	2.25	2.22	2.21	2.23
33	5.5	2.26	2.52	2.36	2.38
34	6	2.12	2.81	2.79	2.57
35	6.5	2.07	2.94	3.01	2.67

**APPENDIX B14: FLOW OF DRY MIX**

SN	BC, %	F1, mm	F2, mm	F3, mm	avg. Flow mm
1	4.5	3.28	2.31	2.39	2.66
2	5	3.22	2.49	2.48	2.73
3	5.5	3.03	2.81	2.92	2.92
4	6	3.26	3.02	3.17	3.15
5	6.5	3.51	3.22	3.41	3.38
6	4.5	2.82	2.66	2.78	2.75
7	5	3.04	2.87	2.82	2.91
8	5.5	3.20	3.31	3.22	3.24
9	6	3.04	3.59	3.64	3.42
10	6.5	3.99	3.82	3.85	3.89
11	4.5	2.98	3.01	3.21	3.07
12	5	3.32	3.41	3.14	3.29
13	5.5	3.73	3.56	3.71	3.67
14	6	4.05	4.16	4.08	4.10
15	6.5	4.55	4.22	4.41	4.39
16	4.5	3.35	3.45	3.47	3.42
17	5	3.74	3.59	3.68	3.67
18	5.5	4.09	4.13	4.05	4.09
19	6	4.51	4.64	4.55	4.57
20	6.5	4.89	4.95	4.86	4.90
21	4.5	3.88	3.66	3.79	3.78
22	5	3.98	4.12	4.04	4.05
23	5.5	4.54	4.44	4.56	4.51
24	6	4.57	4.96	4.99	4.84
25	6.5	4.53	5.46	5.33	5.11
26	4.5	4.08	4.23	4.08	4.13
27	5	4.44	4.48	4.36	4.43
28	5.5	4.80	4.99	5.02	4.94
29	6	5.61	5.54	5.39	5.51
30	6.5	5.82	5.89	6.04	5.92
31	4.5	4.46	4.51	4.48	4.48
32	5	4.79	4.91	4.72	4.81
33	5.5	5.24	5.41	5.42	5.36
34	6	5.94	6.02	6.00	5.99
35	6.5	6.49	6.33	6.45	6.42

## APPENDIX C1: SHORT TERM CREEP OF WET MIX

Time mins	Accumulated Creep Strain of 0% HDPP (µs)				Time mins	Accumulated Creep Strain of 1% HDPP (µs)				Time mins	Accumulated Creep Strain of 2% HDPP (µs)				Time mins	Accumulated Creep Strain of 3% HDPP (µs)			
	10°C	25°C	40°C	60°C		10°C	25°C	40°C	60°C		10°C	25°C	40°C	60°C		10°C	25°C	40°C	60°C
0.1	0.264	0.357	2.299	3.500	0.1	0.236	0.357	2.048	3.002	0.1	0.218	0.306	1.892	2.774	0.1	0.195	0.281	1.698	2.489
0.25	0.310	0.651	2.578	4.071	0.25	0.276	0.651	2.297	3.492	0.25	0.255	0.558	2.122	3.227	0.25	0.229	0.511	1.904	2.895
0.5	0.432	0.824	3.254	4.975	0.5	0.385	0.823	2.899	4.268	0.5	0.356	0.706	2.678	3.943	0.5	0.319	0.647	2.403	3.538
1	0.694	1.134	4.064	6.192	1	0.618	1.134	3.62	5.311	1	0.571	0.972	3.345	4.908	1	0.512	0.891	3.001	4.403
2	0.914	1.484	4.769	7.092	2	0.814	1.484	4.248	6.083	2	0.752	1.272	3.925	5.621	2	0.675	1.166	3.521	5.043
4	1.073	1.734	5.180	8.025	4	0.956	1.734	4.614	6.883	4	0.883	1.486	4.263	6.360	4	0.792	1.362	3.825	5.706
8	1.244	2.485	5.729	8.750	8	1.108	2.308	5.103	7.954	8	1.024	2.130	4.715	7.037	8	0.918	1.953	4.23	6.058
15	1.385	3.119	6.634	9.877	15	1.234	2.896	5.701	8.414	15	1.14	2.674	5.258	7.829	15	1.023	2.451	4.718	7.024
30	1.546	3.619	8.201	10.58	30	1.398	3.271	7.305	9.323	30	1.292	3.098	6.75	8.383	30	1.159	2.779	6.056	7.521
45	1.691	4.144	9.042	11.24	45	1.53	3.813	8.054	9.906	45	1.414	3.548	7.442	8.907	45	1.268	3.183	6.677	7.991
60	1.822	4.514	9.791	11.65	60	1.688	4.182	8.722	10.27	60	1.56	3.864	8.059	9.237	60	1.400	3.467	7.231	8.288
60.1	1.619	4.107	7.976	10.20	60.1	1.442	3.53	7.104	9.339	60.1	1.333	3.292	6.565	8.397	60.1	1.196	3.055	5.89	7.534
60.3	1.580	4.063	7.744	9.922	60.25	1.408	3.492	6.898	9.083	60.25	1.301	3.257	6.374	8.167	60.25	1.167	3.022	5.718	7.327
60.5	1.563	4.029	7.688	9.799	60.5	1.392	3.463	6.848	8.970	60.5	1.286	3.230	6.328	8.066	60.5	1.154	2.997	5.677	7.236
61	1.557	3.988	7.651	9.740	61	1.387	3.428	6.815	8.916	61	1.281	3.197	6.297	8.017	61	1.150	2.966	5.65	7.193
62	1.553	3.960	7.605	9.682	62	1.383	3.404	6.774	8.863	62	1.278	3.175	6.259	7.969	62	1.147	2.946	5.616	7.150
64	1.546	3.922	7.557	9.641	64	1.377	3.371	6.732	8.826	64	1.273	3.144	6.22	7.936	64	1.142	2.917	5.581	7.119 74
68	1.537	3.903	7.504	9.595	68	1.369	3.355	6.684	8.547	68	1.265	3.129	6.176	7.898	68	1.135	2.903	5.542	7.086
75	1.532	3.830	7.466	9.564	75	1.364	3.292	6.651	8.519	75	1.261	3.071	6.145	7.872	75	1.131	2.849	5.513	7.062
90	1.525	3.731	7.419	9.530	90	1.359	3.207	6.608	8.489	90	1.255	2.991	6.106	7.844	90	1.126	2.775	5.479	7.037
105	1.520	3.643	7.373	9.517	105	1.354	3.131	6.568	8.478	105	1.251	2.920	6.069	7.834	105	1.122	2.709	5.445	7.028
120	1.512	3.625	7.299	9.509	120	1.347	3.115	6.501	8.470	120	1.245	2.906	6.007	7.827	120	1.117	2.696	5.39	7.021

## APPENDIX C2: SHORT TERM CREEP OF DRY MIX

Time mins	Accumulated Creep Strain of 0% HDPP (µs)				Time mins	Accumulated Creep Strain of 0.5% HDPP (µs)				Time mins	Accumulated Creep Strain of 1.0% HDPP (µs)				Time mins	Accumulated Creep Strain of 1.5% HDPP (µs)			
	10°C	25°C	40°C	60°C		10°C	25°C	40°C	60°C		10°C	25°C	40°C	60°C		10°C	25°C	40°C	60°C
0.1	0.264	0.357	2.299	3.500	0.1	0.238	0.322	2.069	3.15	0.1	0.317	0.429	2.76	4.20	0.1	0.370	0.500	3.22	4.90
0.25	0.310	0.651	2.578	4.071	0.25	0.279	0.586	2.320	3.66	0.25	0.372	0.781	3.09	4.88	0.25	0.434	0.911	3.61	5.70
0.5	0.432	0.824	3.254	4.975	0.5	0.389	0.741	2.929	4.48	0.5	0.519	0.988	3.90	5.97	0.5	0.605	1.153	4.56	6.97
1	0.694	1.134	4.064	6.192	1	0.624	1.020	3.658	5.57	1	0.832	1.360	4.88	7.43	1	0.971	1.587	5.69	8.67
2	0.914	1.484	4.769	7.092	2	0.822	1.336	4.292	6.38	2	1.096	1.781	5.72	8.51	2	1.279	2.078	6.68	9.93
4	1.073	1.734	5.180	8.025	4	0.966	1.561	4.662	7.22	4	1.287	2.081	6.22	9.63	4	1.502	2.427	7.25	11.23
8	1.244	2.485	5.729	8.750	8	1.119	2.237	5.156	7.87	8	1.492	2.982	6.87	10.50	8	1.741	3.479	8.02	12.25
15	1.385	3.119	6.634	9.877	15	1.246	2.807	5.971	8.89	15	1.662	3.743	7.96	11.85	15	1.939	4.367	9.29	13.83
30	1.546	3.619	8.201	10.58	30	1.391	3.257	7.381	9.52	30	1.855	4.343	9.84	12.69	30	2.164	5.066	11.48	14.81
45	1.691	4.144	9.042	11.24	45	1.522	3.730	8.138	10.11	45	2.029	4.973	10.85	13.49	45	2.367	5.802	12.66	15.73
60	1.822	4.514	9.791	11.65	60	1.640	4.063	8.812	10.49	60	2.187	5.417	11.75	13.99	60	2.551	6.320	13.71	16.32
60.1	1.619	4.107	7.976	10.20	60.1	1.457	3.696	7.178	9.18	60.1	1.943	4.928	9.57	12.24	60.1	2.267	5.750	11.17	14.28
60.3	1.580	4.063	7.744	9.922	60.25	1.423	3.657	6.969	8.93	60.25	1.897	4.876	9.29	11.91	60.25	2.213	5.688	10.84	13.89
60.5	1.563	4.029	7.688	9.799	60.5	1.407	3.626	6.919	8.82	60.5	1.876	4.835	9.23	11.76	60.5	2.188	5.641	10.76	13.72
61	1.557	3.988	7.651	9.740	61	1.401	3.589	6.886	8.77	61	1.868	4.786	9.18	11.69	61	2.180	5.584	10.71	13.64
62	1.553	3.960	7.605	9.682	62	1.398	3.564	6.844	8.71	62	1.863	4.753	9.13	11.62	62	2.174	5.545	10.65	13.55
64	1.546	3.922	7.557	9.641	64	1.392	3.530	6.802	8.68	64	1.855	4.706	9.07	11.57	64	2.165	5.490	10.58	13.50
68	1.537	3.903	7.504	9.595	68	1.383	3.513	6.754	8.64	68	1.844	4.684	9.00	11.51	68	2.152	5.464	10.51	13.43
75	1.532	3.830	7.466	9.564	75	1.378	3.447	6.720	8.61	75	1.838	4.596	8.96	11.48	75	2.144	5.362	10.45	13.39
90	1.525	3.731	7.419	9.530	90	1.373	3.358	6.677	8.58	90	1.830	4.477	8.90	11.44	90	2.135	5.223	10.39	13.34
105	1.520	3.643	7.373	9.517	105	1.368	3.278	6.636	8.57	105	1.824	4.371	8.85	11.42	105	2.128	5.099	10.32	13.32
120	1.512	3.625	7.299	9.509	120	1.361	3.262	6.569	8.56	120	1.814	4.350	8.76	11.41	120	2.117	5.075	10.22	13.31

### APPENDIX C3: LONG TERM CREEP OF WET MIX

Time (days)	Accumulated Creep Strain at 25°C (μs)			
	0% HDPP	1% HDPP	2% HDPP	3% HDPP
0.000069	0.357	0.350	0.303	0.284
0.000174	0.651	0.637	0.551	0.516
0.000347	0.823	0.806	0.697	0.653
0.000694	1.134	1.110	0.960	0.900
0.001389	1.484	1.453	1.257	1.178
0.002778	1.734	1.697	1.468	1.376
0.005556	2.485	2.259	2.105	1.972
0.010417	3.119	2.836	2.642	2.475
0.020833	3.619	3.203	3.060	2.807
0.031250	4.144	3.733	3.505	3.215
0.041667	4.514	4.094	3.818	3.502
0.041736	4.848	4.391	4.108	3.784
0.041840	5.181	4.688	4.397	4.054
0.042014	5.514	4.985	4.687	4.323
0.042361	5.848	5.282	4.977	4.592
0.043056	6.181	5.579	5.267	4.862
0.044444	6.514	5.876	5.557	5.131
0.047222	6.848	6.173	5.847	5.400
0.052083	7.181	6.470	6.136	5.670
0.062500	7.514	6.767	6.426	5.939
0.072917	7.848	7.064	6.716	6.208
0.083333	8.181	7.361	7.006	6.478
0.125000	8.514	7.658	7.296	6.747
0.167000	8.848	7.955	7.585	7.016
0.208000	9.181	8.252	7.875	7.286
0.250000	9.514	8.549	8.165	7.555
0.500000	9.848	8.846	8.455	7.824
0.750000	10.181	9.143	8.745	8.094
1.000000	10.514	9.440	9.034	8.363
2.000000	10.848	9.736	9.324	8.632
3.000000	11.181	10.033	9.614	8.902
4.000000	11.514	10.330	9.904	9.171
5.000000	11.848	10.627	10.194	9.440
6.000000	12.181	10.924	10.484	9.710
7.000000	12.514	11.221	10.773	9.979
14.000000	12.848	11.518	10.974	10.248
21.000000	13.181	11.815	11.261	10.518
28.000000	13.514	12.112	11.549	10.787
56.000000	14.401	13.182	11.836	11.056
84.000000	14.748	13.498	12.866	11.662
112.000000	15.095	13.813	13.171	11.939
140.000000	15.441	14.129	13.476	12.217
168.000000	15.788	14.444	13.781	12.494
192.000000	16.135	14.760	14.086	12.771

## APPENDIX C4: LONG TERM CREEP OF DRY MIX

Time (days)	Accumulated Creep Strain at 25°C (μs)			
	0% HDPP	1% HDPP	2% HDPP	3% HDPP
0.000069	0.357	0.366	0.408	0.510
0.000174	0.651	0.666	0.744	0.930
0.000347	0.823	0.842	0.941	1.176
0.000694	1.134	1.160	1.296	1.619
0.001389	1.484	1.519	1.697	2.120
0.002778	1.734	1.774	1.982	2.477
0.005556	2.485	2.361	2.841	3.550
0.010417	3.119	2.963	3.566	4.456
0.020833	3.619	3.347	4.132	5.053
0.031250	4.144	3.901	4.732	5.786
0.041667	4.514	4.278	5.154	6.303
0.041736	4.848	4.589	5.545	6.812
0.041840	5.181	4.899	5.937	7.297
0.042014	5.514	5.209	6.328	7.782
0.042361	5.848	5.520	6.719	8.266
0.043056	6.181	5.830	7.110	8.751
0.044444	6.514	6.140	7.502	9.236
0.047222	6.848	6.451	7.893	9.721
0.052083	7.181	6.761	8.284	10.206
0.062500	7.514	7.071	8.675	10.690
0.072917	7.848	7.382	9.067	11.175
0.083333	8.181	7.692	9.458	11.660
0.125000	8.514	8.002	9.849	14.169
0.167000	8.848	8.313	10.240	14.384
0.208000	9.181	8.623	10.632	14.936
0.250000	9.514	8.933	11.023	15.488
0.500000	9.848	9.244	11.414	16.040
0.750000	10.181	9.188	11.805	16.592
1.000000	10.514	9.487	12.197	17.144
2.000000	10.848	9.785	12.588	17.696
3.000000	11.181	10.084	12.979	18.249
4.000000	11.514	10.382	13.370	18.801
5.000000	11.848	10.627	13.762	19.353
6.000000	12.181	10.924	14.153	19.905
7.000000	12.514	11.221	14.544	20.457
14.000000	12.848	11.518	17.009	21.009
21.000000	13.181	11.815	17.455	21.561
28.000000	13.514	12.112	17.900	22.114
56.000000	14.401	13.182	18.346	22.666
84.000000	14.748	13.498	19.942	23.907
112.000000	15.095	13.813	20.415	25.073
140.000000	15.441	14.129	20.888	28.099
168.000000	15.788	15.022	22.738	31.235
192.000000	16.135	15.350	24.650	33.844

**APPENDIX C5: WET MIX WHEEL RUT TEST (PERMANENT DEFORMATION)**

Cycles	0%HDPP	1%HDPP	2%HDPP	3%HDPP
0	0.00	0.00	0.00	0.00
25	0.22	0.21	0.18	0.17
50	0.48	0.45	0.44	0.36
75	0.81	0.77	0.68	0.61
100	0.91	0.86	0.73	0.69
125	0.95	0.90	0.84	0.72
150	1.00	0.95	0.90	0.76
175	1.05	0.99	0.97	0.80
200	1.22	1.16	1.06	0.92
225	1.27	1.17	1.10	0.98
250	1.47	1.19	1.12	1.00
275	1.60	1.22	1.15	1.02
300	1.69	1.26	1.19	1.04
325	1.70	1.26	1.19	1.06
350	1.81	1.27	1.20	1.07
375	1.92	1.30	1.23	1.09
400	2.03	1.31	1.24	1.10
425	1.76	1.41	1.39	1.06
475	2.15	1.72	1.38	1.20
500	2.36	1.89	1.40	1.22
550	2.48	1.98	1.44	1.25
575	2.55	2.04	1.51	1.31
600	2.75	2.20	1.45	1.26
1000	3.35	2.68	1.61	1.40
2000	4.62	3.25	1.86	1.16
3000	5.00	3.37	1.93	1.25
4000	5.95	3.47	1.98	1.49
6000	6.87	3.64	2.08	1.72
7000	6.96	3.77	2.16	1.74
7250	7.10	3.72	2.12	1.78
7500	7.20	3.73	2.13	1.80
7750	7.24	3.74	2.14	1.81
8000	7.33	3.75	2.15	1.83
8250	6.92	4.43	2.33	1.86
8500	7.26	4.65	2.37	1.88
8750	7.49	4.64	2.59	1.95
9000	6.29	4.72	2.73	2.03
9250	7.26	4.79	2.89	2.08
9500	7.23	4.99	3.03	2.19
9750	7.73	4.95	2.21	2.28
10000	7.65	5.36	3.55	2.30
Slope	0.000208928	0.00034176	0.000199961	0.000132962
Intercept	5.4202745	1.54389692	0.823513364	0.847276224
X1	0	0	0	0
X2	10000	10000	10000	10000
Y1	5.4202745	1.54389692	0.823513364	0.847276224
Y2	7.509559355	4.96147337	2.823123284	2.176896493
Max rut depth	7.73	5.355	3.553	2.30265
Average	3.75	2.47	1.6	1.3

**APPENDIX C6: DRY MIX WHEEL RUT TEST (PERMANENT DEFORMATION)**

Cycles	0%HDPP	0.5%HDPP	1.0%HDPP	1.5%HDPP
0	0.00	0.00	0.00	0.00
25	0.22	0.45	0.52	1.21
50	0.48	0.71	1.14	2.64
75	0.81	0.90	1.92	4.46
100	0.91	0.99	2.15	5.01
125	0.95	1.16	2.25	5.23
150	1.00	1.26	2.37	5.50
175	1.05	1.42	2.49	5.78
200	1.22	1.56	2.89	6.71
225	1.27	1.59	3.43	6.68
250	1.47	1.72	3.49	7.22
275	1.60	1.74	3.59	7.31
300	1.69	1.96	3.71	8.23
325	1.70	2.05	3.71	8.61
350	1.81	1.95	3.74	8.19
375	1.92	2.15	3.84	9.03
400	2.03	2.17	3.87	9.11
425	1.76	2.30	4.75	11.04
475	2.15	2.33	5.38	11.18
500	2.36	2.47	5.48	11.86
550	2.48	2.61	6.89	12.53
575	2.55	2.91	7.87	13.97
600	2.75	3.50	8.21	16.80
1000	3.35	3.92	9.08	18.82
2000	4.62	5.19	10.16	21.28
3000	5.00	6.11	11.45	25.05
4000	5.95	6.63	12.47	30.50
6000	6.87	7.38	13.10	30.26
7000	6.96	6.73	14.34	31.63
7250	7.10	7.58	15.23	34.11
7500	7.20	7.59	16.06	34.16
7750	7.24	7.58	17.21	34.19
8000	7.33	7.44	17.83	34.22
8250	6.92	7.85	18.60	35.22
8500	7.26	7.84	19.51	36.66
8750	7.49	7.83	20.43	36.70
9000	6.29	7.74	21.23	37.11
9250	7.26	7.72	21.56	37.03
9500	7.23	7.89	22.95	37.23
9750	7.73	8.30	23.25	39.35
10000	7.65	8.96	23.56	39.63
Slope	0.000208928	0.000294862	0.002216825	0.001706871
Intercept	5.4202745	5.301945582	0.681008322	21.48765526
X1	0	0	0	0
X2	10000	10000	10000	10000
Y1	5.4202745	5.301945582	0.681008322	21.48765526
Y2	7.509559355	8.25056104	22.84926	38.55636655
Max rut depth	7.73	8.96	23.562	39.627
Average	3.75	4.15	9.55	18.82