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UNIVERSITI TEKNOLOGI PETRONAS

The Performance of Conventional and Polymer Modified Bituminous Mixture Containing Different Types of Sand as Fine Aggregate

By

Yasreen Gasm Elkhalig

A THESIS

SUBMITTED TO THE POSTGRADUATE STUDIES PROGRAMME

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DEGREE OF MASTER OF SCIENCE IN CIVIL ENGINEERING

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BANDAR SERI ISKANDAR

PERAK

JULY-2009

DECLARATION

I hereby declare that the thesis is based on my original work except for quotations and citations which have been duly acknowledged. I also declare that it has not been previously or concurrently submitted for any other degree at UTP or other institutions.

Signature: ____________________

Name : Yasreen Gasm Elkhalig Suliman

Date : ____________________

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DEDICATION

To My Cousin`s Soul

(Gah Allah)

ABSTRACT

Roads are the heart of any nation's economic and social integration but due to different distresses on it like fatigue and rutting, a number of research have been carried out on modifying the bituminous mixtures to bring real benefits to highway performance in terms of better and longer lasting roads and savings in vehicle operating cost (VOC). Material properties play an important role in determining the final characteristics of the mixture and its performance. This study looks at the incorporation of different types of fine aggregate into bituminous mixtures to predict the performance of the bituminous mixture that related to fatigue and rutting, where both conventional bitumen penetration 50/60 and 80/100, and polymer modified bitumen PM1_82 and PM1_76 and PM2_82 and PM2_76 were used. PM1 is consisting of styrene butadiene styrene (SBS), while PM2 is consist of one of the plastomers polymer. Physical, chemical and mechanical tests were performed on the different types of sand to determine their effect when incorporated with a bituminous mixture. A series of extensive laboratory test programs were carried out. The tests conducted include; the Marshall Test, the creep test, wheel tracking test and beam fatigue test. Results from the Marshall Test showed that fine aggregate characteristics influence the optimum bitumen contents, workability and other engineering properties such as stability, density and stiffness. The results of the performance tests indicated that the resistances of the mixtures with quarry sand against rutting and fatigue damage were superior to those of the other sand mixtures. This was followed by mixture containing river, mining and marine sand respectively. This may be due to the physical, chemical and mechanical properties of the sand, as quarry sand exhibited greater angularity, rougher and it has bigger particles and higher shear strength and higher content of alumina (Al_2O_3) and hematite (Fe_2O_3). Polymer modified bitumen mixtures reveal more resistance to rutting and fatigue than the conventional mixtures. Polymer modified mixtures PM1 was found to offer the highest resistance in rutting followed by the polymer modified mixtures PM2, 50/60 and 80/100 penetration bitumen mixtures respectively. While in fatigue resistance polymer modified mixtures PM1 also exhibit the best fatigue performance followed by PM2, 80/100 and 50/60 penetration bitumen mixtures respectively. This may be due to the PMB having better viscosity property than that of the conventional binder.

ABSTRAK

Jalanraya merupakan nadi kepada ekonomi dan integrasi sosial sesebuah negara. Namun begitu, disebabkan oleh pelbagai masalah yang dihadapi seperti keretakan atau kerosakan jalan dan lelubang akibat daripada kesan tayar kenderaan, beberapa kajian telah dijalankan untuk mengubahsuai campuran bitumen supaya dapat memperbaiki dan memanjangkan hayat jalanraya dan juga menjimatkan kos operasi kenderaan (VOC). Sifat-sifat bahan memainkan peranan yang penting dalam menentukan ciri akhir dan prestasi campuran tersebut. Kajian ini memberi tumpuan kepada penggabungan jenis campuran batu halus yang berbeza ke dalam campuran bitumen untuk menentukan prestasi campuran tersebut yang berkaitan dengan masalah keretakan dan lelubang diatas, dimana kedua-dua penetrasi bitumen dasar adalah 50/60 dan 80/100. Selain itu, bitumen ubahsuai polimer PM1_82 dan PM1_76, dan PM2_82 dan PM2_76 juga telah digunakan. PM1 meliputi styrene butadiene styrene (SBS), manakala PM2 meliputi satu daripada polimer plastomer. Ujian-ujian fizikal, kimia dan mekanikal telah dijalankan ke atas jenis-jenis pasir yang berbeza untuk menentukan kesannya apabila digabungkan dengan campuran bitumen. Satu siri program ujian makmal yang ekstensif telah dijalankan. Ujian-ujian yang telah dijalankan adalah termasuk Ujian Marshall, Ujian Cengkaman, Ujian Kesan Tayar dan Ujian Hentaman Keretakan. Hasil Ujian Marshall menunjukkan bahawa ciri-ciri campuran batu halus mempengaruhi kandungan bitumin yang optima, kebolehkerjaan, dan lain-lain ciri-ciri kejuruteraan seperti kestabilan, kepadatan dan kekerasan. Hasil ujian prestasi menunjukkan bahawa ketahanan campuran dengan pasir kuari terhadap kerosakan iaitu keretakan dan lelubang pada jalan adalah melebihi daripada keputusan ujian campuran yang melibatkan jenis pasir yang lain. Ini diikuti dengan campuran yang mengandungi pasir sungai, pasir lombong dan pasir pantai. Ini kemungkinan disebabkan oleh ciri-ciri fizikal, kimia dan mekanikal pasir-pasir tersebut yang mana pasir kuari mempunyai bentuk dan permukaan yang lebih besar, lebih kasar, mempunyai partikel-partikel yang lebih besar, kekuatan riceh yang lebih tinggi dan tinggi kandungan alumina (Al_2O_3) dan hematite (Fe_2O_3). Campuran bitumin ubahsuai polimer mempamerkan lebih rintangan terhadap keretakan dan lelubang jalan berbanding campuran konvensional. Campuran ubahsuai polimer (PM1) didapati memberikan rintangan tertinggi terhadap keretakan diikuti PM2, masing-masing 50/60 dan 80/100 penetrasi campuran bitumin. Bagi masalah keretakan jalan, prestasi PM1 juga

menunjukkan prestasi penentang keretakan terbaik diikuti dengan PM2, masing-masing 80/100 dan 50/60 penetrasi campuran bitumin. Ini kemungkinan disebabkan oleh PMB yang mempunyai ciri kepekatan yang lebih baik berbanding pengikat konvensional.

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ABBREVIATION

µ m	Micronmetter
AC	Asphalt Concrete
ACW	Asphalt Concrete Wearing Coarse
ARD	Apparent relative density
ASTM	American Society for Testing and Materials
AV	Air Voids
BS	British Standard
FAA	Fine Aggregate Angularty
g	gram
HMA	Hot Mixture Asphalt
JKR	Jabatan Kerja Raya
kg	kilogram
kN	Kilonewton
kPa	Kilopascal
LVDTs	Linear Variable Displacement Transducers
M S	Marshall Stability
min	minute
ml	milliliter
mm	millimeter
MPa	Mega Pascal
Ø	Shear strength or shear resistance
OBC	Optimum Bitumen Content
OPC	Ordinary Portland Cement
PEN	Penetration, 0.1mm
PI	Penetration Index
PMB	Polymer Modified Bitumen
RDD	Relative density on an oven-dried basis
RDS	Relative density on a saturated and surface-dried basis
S_{bit}	Bitumen Stiffness
sec	Second
SEM	Scanning Electron Microscopy
S_{mix}	Mix Stiffness
T	Temperature
t	time
UTM	Universal Testing Machine
VFB	Voids Filled with Bitumen
VMA	Voids in Mineral Aggregate
WA	Water absorption (% of dry mass)
XRF	X-Ray Fluorescence

CHAPTER 1

1. INTRODUCTION

1.1 Background

Roads in the olden days were not able to cope with heavy traffic because shear failures would occur in the wheel path in most soil and also ruts would be formed if the vehicles were to travel on the natural soil itself. It became clear that alternatives to improve the roads had to be found. Hot mixture asphalt (HMA) pavement has been found as an alternative, to prevent premature failure. However the use of HMA mixture in pavement construction has been associated with some performance problems which become the main focus of present day research.

The frequent problems associated with road pavements like rutting, abrasion, fatigue cracking, thermal cracking, aging and stripping cause the roads to wear away or fail (Navarro *et al*., 2002; Lu and Isacsson, 1997). Among these road related problems, rutting and fatigue are considered to be the main problems in highway pavements (Lu *et al*., 1998).

Rutting and fatigue cracking have been related to heavy traffic loads due to increase of axle load and tyre pressure. The exposure of roads in service to such heavy traffic loading proved that stress is the reason behind pavement failure problems. Both the magnitudes and numbers of traffic load repetitions have been found to contribute to damages in flexible pavement (Chavez-Valencia *et al*., 2007; Abo Qudais and Shatnawi, 2007; Tayfur *et al.*, 2007). In addition, the structural integrity of the pavement can also contribute to pavement distress.

Structural factors included sub-grade condition and pavement layer strength and composition of the layer material (Chavez-Valencia *et al*., 2007). Environmental effects such as moisture and position of the water table can also contribute to the damage of highway pavement. Temperature also has a massive influence on the properties and performance of bituminous materials (Navarro *et al*., 2002).

In order to ensure a strong and long lasting pavement, a better understanding of rutting and fatigue cracking phenomena of asphalt concrete mixture is needed because rutting and fatigue have been considered as the leading distress modes in asphalt pavement.

1.2 Pavement Distresses

Permanent deformation (rutting) and fatigue cracking continue to be the main challenges in improving the performance of bituminous mixture pavements. As bitumen is a complex material with a complex response to stress depending on the temperature and loading time, it is a viscoelastic material at room temperature whereas the use of bitumen at high temperature make its viscosity so low, that it can deform easily even under light traffic loads, while the use of bitumen at low temperature causes stiffening, making it brittle (Champion *et al.*, 2001; Whiteoak, 1990; Garcıa-Morales *et al*., 2004). For these reasons when planning for a new road the effects of traffic loads and environmental impact must be taken into consideration.

Rutting occurs only on flexible pavements as indicated by the permanent deformation or rut depth along the wheel paths. Earlier studies have identified that rutting occurs as a result of accumulated plastic deformation due to high traffic loads and high temperatures (Navarro *et al.,* 2007). Robinson and Thagesen (2004) considered rutting as a longitudinal subsidence localized in the wheel tracks caused by vehicles loads as shown in Figure 1-1. It occurs when road does not have sufficient stability of the asphalt material at the surface, insufficient compaction of the pavement and insufficient pavement strength. Rutting also can occur at low stiffness condition for the pavement mixture, namely at high temperature and long durations of loading, when the mixture is approaching its viscous condition (Pell, 1979).

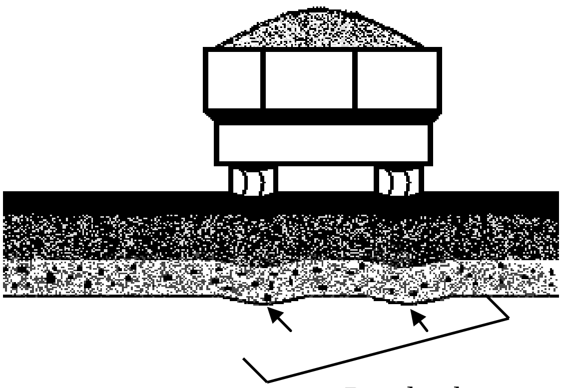


Figure 1-1: Rutting under the wheel loads

Materials proportion also has a great effect on pavement deformation. High bitumen content gives rise to higher plastic flow susceptibility which can then lead to permanent deformation. This is because high bitumen content in the mix can cause loss of internal friction between aggregate particles, this causes the loads to be carried by the bitumen instead of the aggregate structure (Tayfur *et al*., 2007). It is believed that the rate of permanent deformation is influenced by the magnitude of stress, the thickness of the bitumen film, and the properties of binder (Cabrera and Nikolaides, 1988).

The second form of pavement distress is cracking, it is considered as one of the primary reasons that can lead to failure of the structural components of the pavement. There are two types of cracking, thermal cracking and fatigue cracking. Thermal cracking are of two types, low temperature cracking which is usually associated with flexible pavement temperature falling bellow (-23^{0}C), and thermal cracking which occur in much milder regions if an excessively hard asphalt is used or the asphalt becomes hardened by aging (Huang, 2004).

In low temperature cracking the pavement will crack when the computed thermal stress is greater than fracture strength of the materials, while the thermal fatigue cracking is similar to the fatigue cracking caused by repeated loads. It caused by the tensile strain in the asphalt layer that is due to daily temperature cycle (Huang, 2004).

When a bituminous pavement is loaded tensile stresses and strains are induced at the underside of the bitumen bound layer as shown in Figure 1-2. If the structure is inadequate for imposed loading or if the characteristics of the sub grade change through ingress of water, the tensile strength of the material will be exceeded and crack at the bottom of the bituminous layer will result. Repeated loads inducing tensile stresses above the tensile strength of the mix will cause the crack to propagate upwards towards the road surface as shown in Figure 1-3 (Peattie, 1979; Whiteoak, 1990). This occurs because there is a progressive weakening of these layers which in turn increases the level of stress transmitted to the lower layers and sub grade to level that bring about excessive deformation and as the transmitted stress increases, the development of deformation is accumulated. Under traffic loading bituminous pavement materials are subjected to repeated stresses and the possibility of damage by fatigue cracking continually increases. Therefore the strain has a major influence on the pavement life, because the fatigue life decrease as the strain increases.

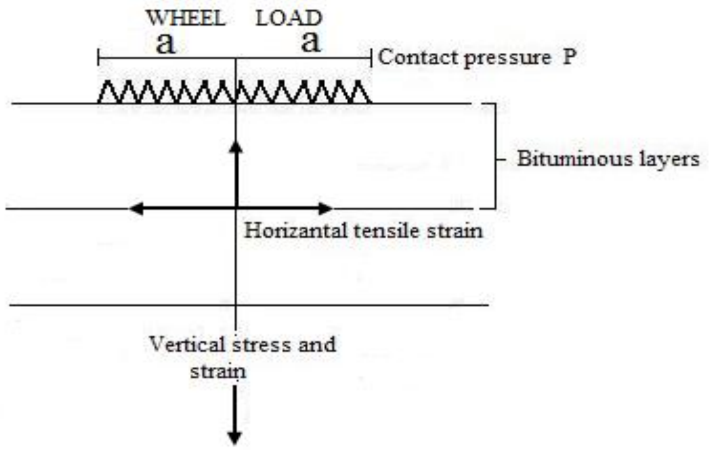


Figure 1-2: Location of stress and strain in the pavement layers
(Peattie, 1979)

Figure 1-3: Fatigue cracking. (Photo taken from Pangkor. Malaysia, Nov 2008)

1.3 Factors Affecting the Characteristics of Bituminous Mixture

Bituminous mixture is made of several components namely coarse aggregate, fine aggregate, filler and binder. There are many factors that can influence the properties and performance of bituminous mixture such as the properties of binder, the type of aggregate used in the bituminous mixture and the properties of the aggregate. The proportion of material, such as composition of aggregate and binder can also influence the properties and performance of bituminous mixture. Therefore a good quality properly blended mix can reduce or eliminate the rutting and fatigue failure in the pavement.

Modifying the physical properties of the binder by using additives is one possible solution that may improve the rutting and fatigue resistance. Polymer modification offers one solution to overcome the deficiencies of bitumen and thereby improves the performance of asphalt mixtures. Many studies have found that the addition of polymer can decrease the penetration, increases the softening point, and also increases the viscosity of the bitumen. The increase in viscosity may increase stiffness of the polymer modified bitumen, which improves Marshall Stability. These produce polymer modified bitumen mixture with improved resistance to permanent deformation and fatigue cracking (Ahmedzade *et al*., 2007; Ahmedzade and Yilmaz, 2007; Hamid *et al.*, 2008; Awwad and Shbeeb, 2007; Kamaruddin, 1998).

Fine aggregate is a primary constituent in asphalt mixtures. For that reason, the properties of fine aggregates namely its physical, chemical and mechanical properties played a significant role in determining the characteristics of the resulting bituminous mixtures.

The physical characteristics of fine aggregate (shape and surface texture) have been found to affect the workability and optimum bitumen content of the mixture. They also affect the asphalt mixture properties and its performance (Topal and Sengoz, 2005; Eyad *et al*., 2001; Chapuis and Legare, 1992). It had been found that grading, shape and surface texture of mineral aggregate affect stiffness of the mixture. The angular particle provides better interlocking property than rounded particles and rough surface of aggregate provides a greater bonding strength with asphalt cement and gives better frictional resistance between particles. This resulted in greater mechanical stability which reflects on the better rutting resistance (Choyce; Shen *et al.*, 2007).

Many researchers investigated the effect of fine aggregate angularity in relation with the resistance to rutting of hot mix asphalt. Park and Lee (2002) and Topal and Sengoz (2005) used natural and crushed fine aggregates in their mixtures. Rutting test was performed on HMA specimens with fine aggregate which had different angularity values. Their results indicated that higher fine aggregate angularity values increased resistance to rutting in hot mix asphalt mixtures. Thus, fine aggregate angularity can be used as one of the parameters to evaluate the performance of hot mix asphalt, other fine aggregate properties such as chemical and mechanical properties must be taken into account for evaluating the mixture performance.

Abo Qudais and Al Shweily (2007) studied the effects of aggregate physical and chemical properties on the creep and stripping behavior of hot mix asphalt. They used two types of aggregates which were limestone and basalt with two types of bitumen PEN 60/70 and 80/100. The mixture prepared using basalt aggregate has better creep resistance than those prepared using limestone aggregate, while the limestone mixture showed better resistance to stripping than the basalt mixture. They recommended further evaluation on the effect of aggregate type or other types of binder on hot mix asphalt characteristics to be carried out. The effect of aggregate chemical composition can be observed in the degree of water sensitivity, this mainly affects the bonding between binder and aggregate particles (Abo Qudais and Al Shweily, 2007; Atkins, 2003). The amount of chemical component of aggregate was found to effect on the bonding strength and on the adhesion performance between aggregate and binder and also on the hardness of the bituminous mixture. Higher silica (SiO_2) content can cause stripping of HMA pavements because silica reduces the bond strength between the aggregate and binder, while higher Alumina (Al_2O_3) content tends to increase the hardness of bituminous mixtures (Abo Qudais and Al Shweily, 2007; Wu *et al.*, 2007).

Other factors that can influence the performance of HMA mixture are the mechanical properties of the fine aggregate. Aggregate’s shear strength was found to have a significant effect on the rutting resistance. Fine aggregate with higher shear strength presents better rutting resistance than fine aggregate with lower shear strength (Topal and Sengoz, 2006; Das, 1998; Park and Lee, 2002).

1.4 Objective of Study

Most of the research works have been concentrated on varying or modifying the properties of the material to produce new mixtures that will be able to perform better as a highway building material. Several researches on bitumen modification have been carried out with the objective of improving the properties of HMA and hence to improve its resistance to different distress modes. Some of the studies have looked into material characteristics and how these characteristics affect the mixture properties and performance. One of the materials that have been the focus of many studies was fine aggregate because the characteristics of fine aggregate have been found to improve the mixture properties and performance (Park and Lee, 2002; Shen *et al.*, 2005; Topal and Sengoz, 2005, 2006; Abo Qudais and Al Shweily, 2007).

Recognizing the significant impact of fine aggregate on pavement performance, it is the interest of this research to investigate the fine aggregate types and properties in determining bituminous mixture properties and its performance. Fine aggregate properties namely physical, chemical and mechanical properties were studied. Different types of fine aggregate were used with conventional bitumen and polymer modified bitumen. The properties of fine aggregate were considered with modified bitumen as it can vary the resulting mixtures properties and its performance.

The main objectives of this study are:

i. To investigate the effects of fine aggregate physical, chemical and mechanical properties on HMA characteristics.
ii. To assess the performance of conventional and polymer modified bitumen mixtures containing different types of fine aggregate (sand).

1.5 Scope of Study

The scope of this work is to study the effect of fine aggregate characteristics on the properties and performance of hot mix asphalt. Two types of conventional bitumen PEN 50/60 and 80/100 were used for the conventional mix while four types of polymer modified bitumen (PM1_76, PM1_82) and (PM2_76, PM2_82) were used for the modified mixture. PM1 is consisting of styrene butadiene styrene (SBS), while PM2 is consist of one of the plastomers polymer. SBS elastomer polymer is mixed with two penetration bitumen, 82 pen and 76 pen. PM2 plastomer polymer is mixed with the same penetration bitumen, 82 pen and 76 pen. Four types of sand namely quarry sand, river sand, mining sand, and marine sand were used with the binders to study their engineering properties and performance of the mixtures in terms of rutting and fatigue characteristics. The asphaltic concrete (AC) mixture used in this study was designed based on the standard by Jabatan Kerja Raya (JKR) Malaysia. The best mixture combination was evaluated based on optimized engineering properties and mixture performance.

1.6 Thesis Outline

This thesis consists of five chapters including introduction, literature review, methodology, results and discussion, and finally conclusions and recommendations.
Descriptions of the background of this study, objectives, scope of study and thesis outline are presented in Chapter 1.

Chapter 2 describes a comprehensive literature review of existing knowledge and past research results on the rutting and fatigue mechanisms of HMA, experimental methods and results, and the effect of fine aggregate on the behavior of bituminous mixtures. Brief information of other materials such as binder and aggregate, and the recent test track research activities are also included.

The laboratory work carried out in this research is presented in Chapter 3. The materials used for preparing the HMA are described, followed by a brief summary of HMA mix design for various types of mixtures including both conventional and polymer modified bitumen mixes. The laboratory test programme is described to cover a wide range of testing conditions. The laboratory tests included; the physical properties of conventional bitumen and polymer modified bitumen, the physical properties of aggregate, chemical and mechanical properties of fine aggregates, the determined of the optimum bitumen content and engineering properties from the Marshall Test, the creep test, the wheel tracking test, and the beam fatigue test. Pertinent experimental testing procedures are successively summarized in this chapter.

The tests results are presented and discussed in Chapter 4, which includes the properties of binders and coarse aggregate, fine aggregate characteristics, mixture properties i.e. density, voids in mineral aggregate (VMA), voids filled with bitumen (VFB), air voids, stability, flow and stiffness, and mixture performance i.e. permanent deformation and fatigue characteristic.

The conclusions of this study were based on the experimental results, and also recommendations for further research are presented in Chapter 5.

CHAPTER 2

2. LITERATURE REVIEW

2.1 Introduction

This research was conducted to investigate the fine aggregate properties namely physical, chemical and mechanical properties on the properties and performance of bituminous mixture. In reviewing the research, the following sequence was adopted. Firstly for the better understanding of fine aggregates characteristics, physical, chemical and mechanical properties of fine aggregate were analyzed. Secondly the effect of fine aggregate properties on the properties and performance of asphaltic concrete mixture were discussed. Thirdly, both conventional and polymer modified bitumen mixture design were undertaken, not only its materials properties but also its effect on the performance characteristics. The mixture properties and performance tests were also talk about. In the end the properties and the performance which include the rutting resistance and fatigue resistance characteristic of asphalt concrete mixtures were discussed too.

2.2 Background of Fine Aggregate

One of the primary constituents that can be used as fine aggregate in bituminous mix is sand. Sand is defined as granular material that passes through different sizes of sieves. Fine aggregate is defined in the JKR standard as material passing 5mm and retained on 0.075mm (JKR Standard, 1988). The purpose of using fine aggregates (sand) into bituminous mixture is to enhance the stability of the mix with its interlocking characteristics and at the same time to fill up the voids left out by the composition of the coarse aggregates.

There are two types of sand; natural and manufactured. The natural sand comes from beaches, rivers and ponds while the manufactured one comes from the parent material consisting of dolomite, limestone and glacial gravel (Eyad *et al.,* 2001). Many materials have been used as fine aggregate in bituminous mixture such as limestone (Topal and

Sengoz, 2006; Abo Qudais and Al Shweily, 2007; Ahmedzade *et al.,* 2007; Cao, 2007), crushed basalt and granite (Fernandes and Gouveia), crushed granite and limestone (Park and Lee, 2002). The basalt and andesite after being crushed show more angularity shape than the limestone thus can be used as fine aggregate (Topal and Sengoz, 2005). Calcareous from the rock that has originated from the calcium deposit also can be used as fine aggregate (Tayfur *et al.*, 2007). Each type of fine aggregate, depending on their source (quarries, rivers, ponds or beaches) has specific characteristic and effect that is expected to influence the bituminous mixture properties and performance. Therefore, the selection of the type of sand is dependent upon the specific goal or desired characteristic of the resulting mixture.

2.3 Fine Aggregate Properties

Fine aggregate properties such as physical, chemical and mechanical properties that have been found to improve the bituminous mixture properties and performance are discussed in the following section. Among these properties, physical properties have been most influential on the properties and performance of hot mix asphalt pavements.

2.3.1 Physical Properties

The physical properties of fine aggregate refer to the physical structure of the particles that make up the sand. The physical properties include particle shape, surface texture, particle size and distribution, and colour of the sand.

2.3.1.1 Particle shape

Particle shape of fine aggregate it’s the one of the most important factors affecting mixture stability and the capability to resist permanent deformation (Kandhal *et al.*, 1991; Lee *et al.*, 1999). The shape of fine aggregate varies depending upon the source and can be described as elongated and angular (Abo Qudais and Al Shweily, 2007), cubic, flat and thin (Topal and Sengoz, 2006). A classification of the shape used in USA is as follows; well-rounded, rounded, sub-rounded, sub-angular and angular (Topal and Sengoz, 2005). Particle shape has an effect on the strength of the aggregate particles, on the bond with cementing materials, and on the resistance to sliding of one particle over another. Atkins (2003) found that flat particles, thin particles and needle shaped particles break more easily than cubical particles. Angular particles with rough fractured face allow a better bond with cements than do rounded and smooth gravel particles. Rounded particles provide better workability during compaction but tend to continue to compact under traffic loading due to lack of interlocking property. While angular particles give the asphalt mix a harder consistency making it more difficult to handle and compact. On the other hand it provides a better interlocking than rounded particles (Topal and Sengoz, 2006). As was cited by Kandhal *et al.* (1991) for gradation, the closer the gradation was the fuller curve for maximum density, the higher was the stability. Rounded sands of relatively uniform size were reported to result in lower stability, while manufactured sands with a highly angular particle shape produce mixture with higher stability. An excessive amount of rounded sand contributed to a loss of rut resistance of HMA. Therefore increase the rutting as the amount of rounded sand was increased (Park and Lee, 2002).

One investigation carried by Janoo *et al.* (2004) found that angularity shape is important not only on the surface layer but they also have significant effect on the base course layer. Another study by Topal and Sengoz (2005) found that aggregate shape has effects on the bituminous mixture workability and performance. It was also found that particle shape has an effect on the air voids content in the mixture.

2.3.1.2 Surface texture

Surface texture is the relative roughness or smoothness of the aggregate particle. It plays a big role in improving the bond between an aggregate and asphalt binder. A rougher surface produces a strong bond thus creating a strong mixture. The rougher surface also affects the workability and asphalt requirements of hot mixture asphalt (Topal and Sengoz, 2005, 2006). The crushed aggregate that comes from crushed gravel have a rougher texture that could provide greater bonding strength with asphalt cement and better frictional resistance between particles, which contribute to higher rutting resistance.

Atkins (2003) found that soft and lightweight particles are scratched easily, and may be unsuitable where they may be exposed to abrasion. Light weight particles might be weak or porous and result in poor surfaces or pavements. Some aggregate may initially have a good surface texture but may polish smooth later under traffic, these aggregate are unacceptable for final wearing surfaces. Abo Qudais and Al Shweily (2007) investigated the effect of aggregate surface texture on the stripping resistance. They found that rougher surface texture gives better adhesion. Therefore porous aggregate usually shows better adhesion to asphalt due to better mechanical interlock. This property can affect stripping resistance. In bituminous mixture, the finer fraction of the sand has the highest surface area. Surface area related physico-chemical properties are known to largely influence the performance of asphalt mixture (Chapuis and Legare, 1992).

2.3.1.3 Particles size and distribution

The size of the grains always important, it's possible to get the same sample however the particles size looks is different. The particles size of fine aggregate in the bituminous mixture meets the passing through the sieve size 5mm. The JKR standard specifies that fine aggregate used in the bituminous mixture passes 5mm and retained on 0.075mm sieve size (JKR Standard, 1988).

The fine aggregate is in the size range of 5mm to 1.18mm provides a rough surface on the pavement where it functions to give a frictional resistance to the surface of the pavement. While fine aggregate from sieve sizes of 600μm to 75μm are important of a mix to increase the surface area of the aggregates, which will enable the mix to absorb a high content of bitumen and hence enhancing the binding force of the mix. Thus, it can be concluded that the gradation of fine materials is very important and a balance mixture of coarse aggregates and fine aggregates is needed in order to provide required frictional effects and optimum binder content (Anderson). Aggregate particle size and its distribution or gradation is normally expressed in percentage of the total weight. The gradation should allow the larger particles to be in contact with each other, because the gradations with an excessive amount of finer particles are not effective in distributing load (Atkins, 2003).

2.3.1.4 Colour

The colour of sand is related to the composition of the individual particles. High content of quartz will produce icing white, while high feldspar content will make a more orange coloured. Common black minerals in sand are mica and horn blend. In general, the colour of fine aggregate does not have any effect on the mixture properties and its performance.

2.3.2 Chemical Properties

The chemical properties of aggregates are determined by the mineral composition in the aggregate particles. The chemical composition of aggregate is significant in differentiating the types of aggregate. The X-ray fluorescence (XRF) apparatus can be used to predict the chemical composition of fine aggregate.

The chemical composition of aggregate particles that determines the chemical stability, can affect the mixture performance. The effect of aggregate chemical composition can be observed in the degree of water sensitivity, which affects the bonding strength between the binder and aggregate. Aggregates that fall in the hydrophilic category can cause stripping, which leads to disintegration of the asphalt surfaces (Atkins, 2003; Abo Qudais and Al Shweily, 2007). The amount of silica also was found to affect the pavement performance. A large amount of silica (SiO_2) can cause stripping of HMA pavements because silica reduces the bond strength between the aggregate and binder, while the high amount of alkali found to improve the adhesion performance between aggregate and bitumen (Abo Qudais and Al Shweily, 2007; Wu *et al*., 2007).

2.3.3 Mechanical Properties

The mechanical properties of aggregate can be defined by the shear strength property. Strength is a measure of the ability of an aggregate particle to withstand pulling or crushing force. High strength is desirable in aggregate base and surface courses. This quality minimizes the rate of disintegration and maximizes the stability of the compacted material. Crushed aggregate has higher shear strength compared to the natural aggregates (Topal and Sengoz, 2006). Atkins (2003) found that the strength of layer or base course materials is very important to the load-carrying capacity. The mechanical properties (shear resistance) of fine aggregate have a significant effect on mixture resistance to the permanent deformation. Therefore a high shear resistance is an indicator of resistance to mixture deformation (Das, 1998; Topal and Sengoz, 2006). It was found that HMA mixture containing river gravel fines with lower friction angle shows higher rut depth, while the HMA mixture containing granite fines with higher friction angle shows lowest rut depth (Park and Lee, 2002).

2.4 Effect of Fine Aggregate Properties on the Properties and Performance of Asphaltic Concrete Mixture

Several studies were conducted to investigate the effect of fine aggregate characteristic in bituminous mixture properties and performance. Some of the researchers found that shape and surface texture of fine aggregate can affect the workability and optimum asphalt cement content of the mixture, as well as the asphalt mixture properties. These include stability, air voids in the mixture, and durability (Choyce; Topal and Sengoz, 2005, 2006).

It was cited by Shen *et al.* (2007) that angular shaped particles which are preferred in HMA exhibit greater interlocking and internal friction, thus result in greater mechanical stability than do rounded particles. As was also cited by Lee *et al.* (1999) that fine aggregate angularity and mixture gradation are the two critical factors affecting mixture stability, the more angular fine aggregate, the higher the mixture stability.

Park and Lee were found that HMA mixtures containing river gravel fines and natural sand fines shows higher rut depth, while the mixture containing granite and limestone fines shows lower rut depth (Park and Lee, 2002). Therefore fine aggregate surface texture plays an important role in HMA rutting resistance. The advantage of using crushed rock as fine aggregate in HMA wearing course results in produce mixture with higher resistance to deformation, compared to the most natural sand fine aggregate (Choyce).

Another study conducted by Eyad *et al*. (2001) investigated the relationship between fine aggregate shape and hot mix asphalt performance. They expressed aggregate shape as three independent properties; form, angularity and texture. They used twenty two aggregate samples to measure hot mix asphalt rutting resistance. Their results showed that among the three aggregate shape properties, texture had the strongest correlation with rutting resistance as shown by wheel tracking test results. They concluded that resistance to rutting increased with increase rougher fine aggregate texture.

It was also found by Park and Lee (2002) there is a good correlation between frictional angle and rut depth. Therefore HMA mixtures containing river fines with a measured friction angle in direct shear test of 40.3^0 has the highest rut depth compared with mixtures containing granite fines with a friction angle of 45.2^0. Another study cited by Fernandes and Gouveia investigated the effect of crushed fine aggregate. They found that the replacement of the rounded aggregate by crush fine aggregates improved mixture properties such as stability, rutting and water resistance.

Stakston and Bahia (2003) conducted a study that aimed at gaining a better understanding of the influence of fine aggregates angularity, asphalt content and performance grade of asphalt on hot mixture asphalt (HMA). By using fine aggregate from four different sources, they found that the effect of fine aggregate angularity were highly dependent on the source of aggregate and their gradation. The results also indicated that varying the performance grade of asphalt had an important influence on the critical properties of HMA mixture. The effect of asphalt content was found to be highly dependent on the source of the fine aggregate also.

Lee *et al*. (1999) was studied the fine aggregate angularity and their effect on the asphalt mixture rutting performance. They designed a total of 18 mixtures and their results indicated that mixtures with higher fine aggregate angularity values exhibited better rutting performance.

2.5 Mixture Design

2.5.1 Materials

Bituminous mixture consists of aggregate, filler and finally binder that can bind all of this material together and also to give the mixture its durability. The properties of each material and their function in the bituminous mixture are described in the following sections.

2.5.1.1 Aggregate

Aggregates are granular mineral particles, it is account for 90–95% of asphaltic mixture by weight and 75-85% of asphaltic mixture by volume (Topal and Sengoz, 2006). In highway construction, aggregates are used in a number of different ways. In all cases the aggregate used should be strong, tough, durable, and has the ability to be crushed into bulky particles without many flaky particles. In addition to gradation requirements, the aggregate are also required to possess the strength to carry and transmit the applied loads.

The aggregate gradation specification for highway bases, concrete and asphalt mixture requires a grain size distribution that will provide a dense, strong mixture. There are four types of aggregate gradation namely, well-graded, gap graded, open graded and uniform graded (Atkins, 2003). Aggregate gradation was found to be one of the most important factors to resist pavement distress (Shen *et al*., 2005). Abo Qudas and Al Shweily (2007) found that aggregate gradation have a significant influence on the creep behavior of HMA, aggregate gradation influences air voids (AV) and voids in mineral aggregate (VMA), and it affects the creep behavior. It can also affect stripping resistance and the fatigue life behavior of pavements (Abo Qudais and Shatnawi, 2007; Abo Qudas and Al Shweily, 2007).

Some typical terms are used in describing the aggregates depending on their sizes. Coarse aggregate (gravel size) is the aggregate particles mainly larger than 4.75mm. Fine aggregate is defined for aggregate particles between 4.75mm and 0.075mm while filler is used to describe particles that are smaller than 0.075mm (Atkins, 2003). The aggregate percentage and sieves sizes commonly used in wearing course construction in highways are indicated in Table 2-1 as represented in Jabatan Kerja Raya (JKR) Malaysian Standard.

Table 2-1: Percentage of aggregate gradation of JKR standard 1988

Mix type	**Wearing course**
Mix designation	ACW 20
B.S Sieve	% passing by weight
37.5 mm	-
28.0 mm	100
20.0 mm	76-100
14.0 mm	64-100
10.0 mm	56-81
5.0 mm	46-71
3.35 mm	32-58
1.18 mm	20-42
0.425 mm	12-28
0.150 mm	6-16
0.075 mm	4-8

The aggregate size is based on the mass retained and passing through each sieve, for example fine aggregate has a maximum size of 3.35mm in well-graded mixtures however in gap-graded mixture the maximum size of the fine aggregate is taken as 2.36mm (Atkins, 2003).

The function of coarse aggregate in the mix is to provide stability to the pavement due to the interlocking behavior between the coarse particles. One of major requirements for coarse aggregates used in bituminous mix is the gradation of the material. Good distribution for aggregate could give a strong mixture that reflects on better fatigue resistance (Asi, 2006).

2.5.1.2 Filler

Filler in the mix basically fill up the voids left in the aggregates, namely the coarse and fine aggregates. At least 75 % of filler shall pass 75 micron test sieve. One of the criteria's that will affect the suitability of a filler to be used is its fineness. The loads are transmitted mainly by the cementing agent in asphalt mixture (Atkins, 2003).

2.5.1.3 Bitumen

Bitumen is a black coloured hydrocarbon substance that is soluble in carbon disulphate. It can be derived from native asphalt, rock asphalt, tar asphalt and petroleum asphalt. The last resource is more important because it is used for pavement and can be obtained by distillation of crude oil.

Bitumen is the most suitable material as a binder of mineral aggregate in paving applications and has been widely used as an adhesive material in pavement mixtures, surface dressing, bridge deck waterproofing, overlays and the protection of buildings. These applications of bitumen are owed to its many interesting characteristics, such as its strength, readily adhesive, highly waterproof, durable, elastic, impermeable (Navarro *et al*., 2002; Garcia-Morales *et al*., 2006).

An analysis of bitumen obtained from a variety of crude oils shows that most bitumen contain carbon (82-88%), hydrogen (8-11%), sulphur (0-6 %), oxygen (0-1.5 %) and nitrogen (0-1 %) (Whiteoak, 1990). Bitumen has two broad chemical groups called asphaltenes and maltenes; the maltenes can be further subdivided into saturates, aromatics and resins (Whiteoak, 1990; Garcıa-Morales *et al*., 2004; Navarro *et al*., 2007). Bitumen can be produced in various grades by modifying its basic properties using flux oils. Typically there are four types of bitumen namely penetration grade bitumen, oxidized bitumen, hard bitumen and cut back bitumen (Whiteoak, 1990).

The rheology of bitumen at a given temperature is determined by its chemical constituents (chemical composition) and structure (physical arrangement); any changes in either constituents or structure, or both will result in a change in the rheology or viscoelastic properties (Whiteoak, 1990; Perez-Lepe, 2003). Bitumen is a viscoelastic material at room temperature, i.e. any changes in temperature will change its flow properties. Therefore a low thermal susceptibility is required for the use of the bituminous materials (Champion *et al.*, 2001; Garcıa-Morales *et al.*, 2004).

The viscoelastic properties of bitumen, and consequently its performance as a road paving binder, are dramatically influenced by the ratio between the asphaltene and maltene fractions (Navarro *et al.*, 2002). Because the rheological properties of bitumen depend strongly on the asphaltene content, by holding the asphaltene content constant and varying the concentration of the other three fractions the viscosity of bitumen can be affected. A constant ratio of resins to aromatics and increasing the saturate content will soften the bitumen. While addition of resins hardens the bitumen, and results in reducing the penetration index, but increases its viscosity (Whiteoak, 1990; Garcıa-Morales *et al.*, 2004).

To minimize the deterioration in flexible pavement, the bituminous layers should be improved with regard to performance related properties such as resistance to permanent deformation, low temperature cracking, load associated fatigue. One way of increasing the quality of a flexible material layer is by enhancing the properties of existing asphalt material. This can be achieved by modifying the bitumen using different additives to increase the overall performance of the binder (Ahmedzade *et al*., 2007). Modified bitumen materials can bring real benefits to highway maintenance and construction in terms of better and longer lasting roads and savings in vehicle operating cost (VOC). Most additives frequently used for the modification and performance improvement of petroleum bitumen are fillers, fibres, rubber and polymers (Giavarini *et al*., 1996).

2.5.1.4 Polymer and families of polymer

Polymer is one of the additives that can be used to improve the bitumen properties, is used as a modifier in bituminous mixture to enhance the mixture characteristics. Polymer has two main families' namely thermoplastic crystalline polymers and thermoplastic rubbers. Thermoplastic crystalline polymers (plastomers) include many materials such as polyethylene (PE), polypropylene, polyvinylchloride (PVC), polystyrene (PS), ethylene vinyl acetate (EVA) and ethylene methyl acrylate (EMA). While the thermoplastic rubbers (elastomers) include such materials as natural rubber, styrene butadiene rubber (SBR), styrene butadiene styrene (SBS), styrene isoprene styrene (SIS), polybutadiene (PBD) and polyisoprene (Nicholls, 1998; Airey, 2002). The difference between plastomers and elastomers is that plastomers are tough, which can improve rigidity and reduce deformations under load, while the elastomers gives better elastic properties to resist deformation by stretching to recover their initial shapes (Airey, 2002; Navarro *et al*., 2007). Ahmedzade and Yilmaz (2007) classified polymer into four broad categories; plastomers, elastomers, fibers and additives/coating. Napiah (1993) also divided the polymers into four major groups as follows: thermoplastic materials, thermoplastic rubber, thermoplastic resins and rubbers.

The most polymers widely used for bitumen modification is ethylene vinyl acetate (EVA), which is a thermoplastic polymer and has been used for long in bitumen modification (Murphy *et al*., 2000). The (EVA) polymer modifier is used for more than 20 years in order to improve both the workability (mixing, laying and compaction) of the asphalt during the construction and its deformation resistance in service (Airey, 2002). The second polymer that also most widely used is styrene butadiene styrene (SBS), which is a thermoplastic rubber or styrene block copolymer. SBS is a very strong and elastic polymer and it consists of hard polystyrene end blocks and rubbery midblock. The hard polystyrene end blocks give high tensile strength and flow resistance at high temperature, whereas the rubbery midblocks are responsible for the elasticity, fatigue resistance and flexibility at low temperature (Murphy *et al*., 2000; Ahmedzade *et al*., 2007).

The main function of polymer is to change or improve the physical nature of bitumen. Polymer additive does not change the chemical nature of the bitumen being modified, but rather the physical nature of bitumen that is related to physical properties. Polymer modification increased the softening point and viscosity, elastic recovery or ductility while reduced the penetration (Wekumbura *et al*., 2007). The addition of polymer into bitumen has been found to increase the stiffness of the bitumen and improves its temperature susceptibility. Polymer modified binders also show improved adhesion and cohesion properties (Awwad and Shbeeb, 2007). Polymer modified bitumen to be effective it must be stable physically and chemically during storage, application and service. Many researchers used polymer as modifier material among the other additives because polymer is easily available and can be obtained from recycled tyres and waste polymer (Garcia-Morales et al., 2006; Cao, 2007).

Several studies have been conducted on modifying the bitumen to improve its physical properties to resist stresses. A study by Champion *et al*. (2001) found that an increase in toughness resulted from adding polymer ethylene vinyl acetate, ethylene methyl acrylate and ethylene butyl acrylate (EVA, EMA, EBA) to bitumen70/100 penetration grade however the improvement was higher with styrene butadiene copolymer (SBS). While Murphy *et al*. (2000) added SBS, EVA, crumb rubber, rubber flour, polyethylene, polypropylene, polyether polymers and polyurethane waste to bitumen penetration grade 200 to improve the physical properties of the bitumen. The results show increased the softening point and viscosity and decreased the penetration. A study by Navarro, *et al*. (2007) found that polymer modified bitumen can improve the viscosity at high temperature and can achieve better performance at low temperature also. Garcıa-Morales *et al*. (2004) found that the use of recycled ethylene vinyl acetate (EVA) as modifier can improve the viscous properties of bitumen at high temperature. Another study by Sirin, *et al*. (2006) found that the high viscosity of SBS modified binder was the main reason for the high rutting resistance.

The effects of polymer modified bitumen on the asphalt concrete mixtures properties and their performances are well documented. One of the mixture properties that are affected when the polymer modified bitumen (PMB) is used is the Marshall Stability. Tayfur *et al*. (2007), Awwad and Shbeeb (2007) and Hamid *et al*. (2008) investigated Marshall Stability of bituminous mixtures through two types of mixture; conventional mixture and polymer modified mixture. Their results indicated that generally modified mixtures have higher stability than the control mixture. In addition, the voids filled with binder and the voids in the mineral aggregate and the air voids in the modified mixture were increased.

Some researchers investigated the possibility to overcome the rutting phenomena by using different types of polymers. Tayfur *et al*. (2007) used five types of polymer to modify asphalt mixture namely amorphous polyalphaolefin, cellulose fibers, polyolefin, bituminous cellulose fibers and styrene butadiene styrene; while Ahmedzade and Yilmaz (2007) used polyester resin (PR), and Chiu and Lu (2007) used ground tyre rubber (GTR). All of these polymers were found to increase the resistance to permanent deformation of asphalt concrete mixture. Chavez-Valencia *et al*. (2007) investigated the resistance to rutting and fatigue. They used polyacetate emulsion (PVAC-E) to modify the asphalt mixture. Their results indicated that modified mixture has better resistant to rutting and fatigue than the conventional one. Another study by Hamid *et al*. (2008) investigated the rutting and fatigue cracking resistance when styrene butadiene styrene (SBS) and ethylene vinyl acetate (EVA) were used. They observed that the modified mixture has better resistance to rutting and fatigue cracking compared to the conventional mixture. In conclusion, all of the earlier research works discovered that polymer modified bitumen regardless of the modified type or state could improve the mixture properties and its performance.

2.5.2 Asphaltic Concrete Design Mixture

Asphalt concrete mixture is designed to have stiff and strong pavement to carry the heavy loads and high tyre pressures. The composition of asphaltic concrete mixture includes aggregate which is the main structure contributor in asphaltic concrete mixtures, while the bituminous binder plays a minor role, hence the percentage of bituminous binder is relatively low. The design of a bituminous mix involves the aggregate type, aggregate grading, bitumen grade and the determination of bitumen content. In this study the Jabatan Kerja Raya (JKR, 1988) Malaysian standard will be used to recommend the grading and to design the bitumen content as shown in Table 2-2.

Table 2-2: JKR Gradation Limits and Design Bitumen Content for Asphaltic Concrete Mixture (JKR, 1988)

Mix type	**Wearing course**
Mix designation	ACW 20
B.S Sieve	% passing by weight
37.5mm	-
28.0mm	100
20.0mm	76-100
14.0mm	64-100
10.0mm	56-81
5.0mm	46-71
3.35mm	32-58
1.18 mm	20-42
425 mm	12-28
150 mm	6-16
0.075 mm	4-8
Design bitumen content	3.5-7%

2.6 Mixture Properties and Performance Tests

The mixture properties test includes stability and flow which can be determined by the Marshall test. The other mixture properties such as density, voids in mineral aggregate, voids filled with bitumen and air voids which can be determined by weight in air and weight in water for the Marshall specimen. The performance tests include rutting test which can be assessed by dynamic creep test and wheel tracking test. The other performance test is fatigue test which can be assessed by beam fatigue test.

2.6.1 Marshall Test

Marshall Test is an empirical test used to measure the stability and flow when cylindrical compacted specimens is loaded to failure using Marshall apparatus along a diameter of specimen at constant rate of compression of 51mm/min as shown in Figure 2-1. Marshall Stability value (in kN) is the maximum force recorded during compression whilst the flow (in mm) is the deformation recorded at maximum force (Ahmedzade *et al*., 2007; Ahmedzade and Yilmaz, 2007).

For determining the optimum bitumen content (OBC) both of the stability and flow can be used beside other parameters such as density, voids in mineral aggregate, voids filled with bitumen and air voids. The OBC can be determined as the average of maximum stability, maximum density, minimum voids in mineral aggregate, recommended value of flow, voids filled with asphalt and air voids.

Voids in mineral aggregate (VMA) is the space between the aggregate particles of bituminous mixture. It is expressed as the percentage of volume voids to the total volume of mix. VMA is important as it provides sufficient space between the aggregates that can be filled by bitumen in order to obtain maximum strength of the design mixture. Void filled with bitumen (VFB) represent the percentage of voids filled by bitumen, while air voids (AV) is the percentage of air volume to the total volume of compacted bituminous mixtures. Marshall Stiffness is a parameter to measure the stiffness of bituminous mixture; it obtained by dividing the stability over flow as shown in Equation 2-1.

$$\textit{Marshall Stiffness} = \frac{\textit{Stability}\,(kN)}{\textit{Flow}(mm)} \qquad 2\text{-}1$$

Figure 2-1: Curved steel loading plates used in the Marshall Test set-up

2.6.2 Dynamic Creep Test

Dynamic creep test is used to assess the resistance of hot mix asphalt to permanent deformation (rutting). The test is conducted by applying repeated pulsed uniaxial load onto the bituminous mixture specimen and measuring the resulting deformation using Linear Variable Displacement Transducers (LVDTs) (Asi, 2007). The test conditions were; 40 ^{0}C is the test temperature, 0.01 MPa is the conditioning preloading for 2 min and 0.1 MPa is constant loading stress during the test for 1 hr (Ahmedzade and Yilmaz, 2007; Cabrera and Nikolaides, 1988).

The results are plotted as creep modulus vs. no. of cycles or log creep modulus vs. log no. of cycles and determining the deformation by the slope which obtained from the formula. Fewer slopes means less sensitive to deformation. Another way of presenting the creep test results is the plotting the stiffness of mix (S_{mix}) versus stiffness of binder (S_{bit}). S_{bit} is parameter determined using Van der Pool Nomograph as shown in Figure 2-2 depend on the viscosity of the bitumen. The viscosity of bitumen is a function of PI and ring and ball temperature, the number of wheel passes in standard axles and the time of loading for one wheel pass. $S_{mix,}$ is the stiffness of the design mixture derived from creep test at certain value of stiffness which is related to viscous part of the bitumen. Both S_{mix} and S_{bit} are independent of temperature, time of loading and stress levels. The Equation 2-2 is used to calculate the rut depth of a pavement from laboratory creep test results (Cabrera and Nikolaides, 1988).

$$R_d = C_m \times H \times \frac{\sigma_{av}}{S_{mix.creep}} \qquad 2\text{-}2$$

Equation 2-2 which is used to calculate the rut depth of the pavement from laboratory creep test results was initially proposed by Hills *et al.* (1974).

where: R_d - calculated rut depth of the pavement.

C_m- correlation factor for dynamic effect, varying between 1.0 and 2.0

H- pavement layer thickness.

σ_{av}- average stress in the pavement related to wheel loading and stress distribution.

S_{mix}- stiffness of the design mixture derived from creep test at a certain value of stiffness which is related to the viscous part of the bitumen.

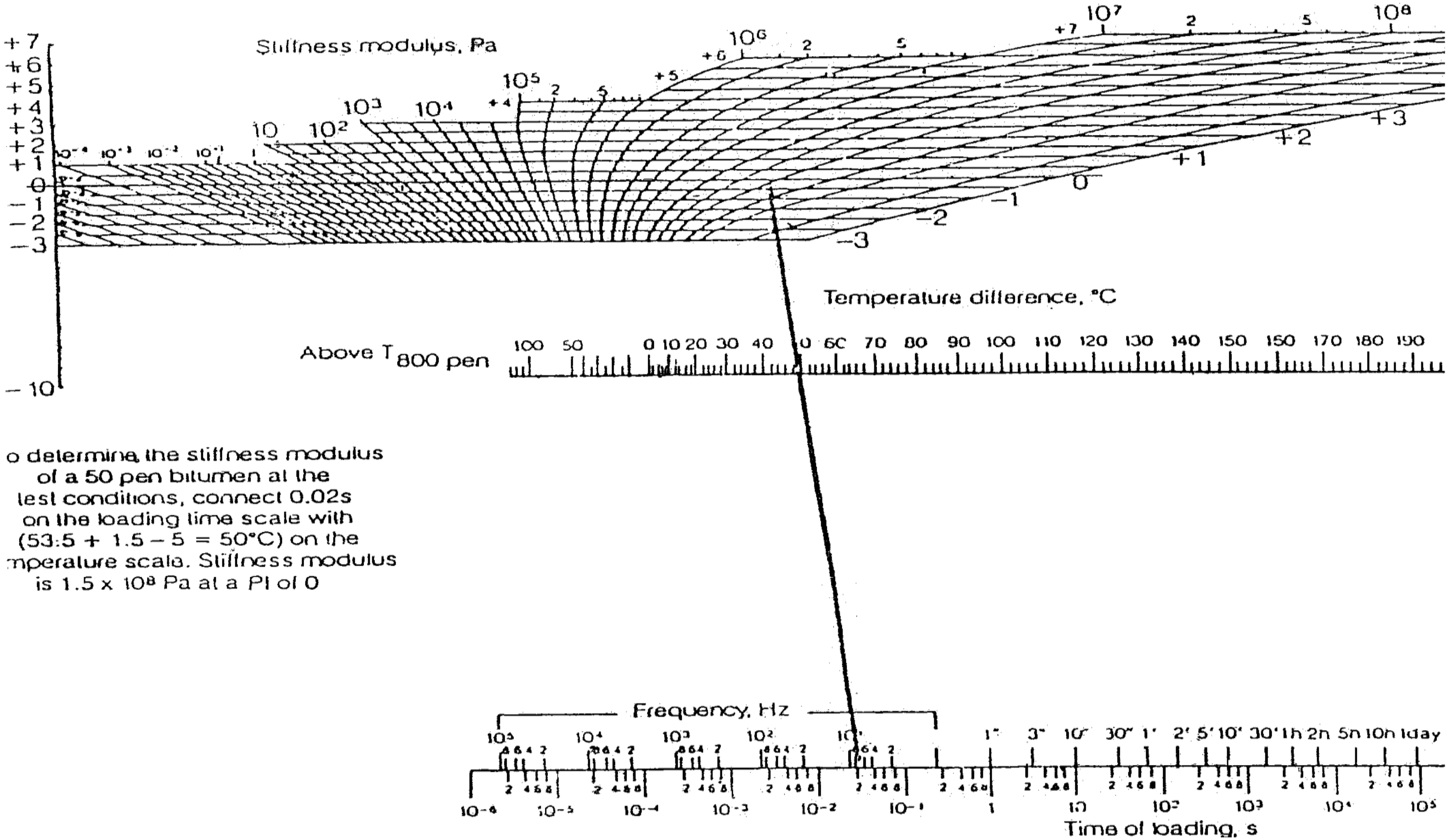


Figure 2-2: Van der Pool nomograph for *Sbit* determination (Pell, 1979)

2.6.3 Wheel Tracking Test

Wheel tracking test is used to measure the permanent deformation (rutting) of hot mixture asphalt at high temperature. Optimum bitumen content is used to prepare the square specimen 300mm×300mm, 50mm depth. The test is conducted by applied wheel load on the specimen, the speed of wheel (N) passing over the center of the specimen was 42 cycles per minute for 45 minute (Cao, 2007). Equation 2-3 can be used to calculate the dynamic stability (DS).

$$DS = \frac{t_2 - t_1}{d_2 - d_1} \qquad 2\text{-}3$$

where: DS - dynamic stability
t_2 - temperature at 60 ^{0}C
t_1 - temperature at 45 ^{0}C
d_2 - rut depth at 60 ^{0}C
d_1 - rut depth at 45 ^{0}C.

Higher dynamic stability (DS) of asphalt mixtures shows better rutting resistance.

2.6.4 Beam Fatigue Test

Fatigue properties can be obtained by carried out beam fatigue test. There are two types of mode load which can be used in this test; stress controlling mode and strain controlling mode as shown in Figure 2-3. In stress controlling mode the strain is increased with cycle while in strain controlling mode the stress is decreased with the cycle. For thinner asphalt layer less than 50 mm constant strain can be applied while for thicker pavement layer 100 mm and over, constant stress can be applied (Pell, 1979). Failure occurs quicker with constant stress, because both stress and strain are normally larger for constant stress than constant strain, and the failure is easy to define using constant stress. For arbitrary failure criterion, for example stress is equal to 50% from the initial stress, constant strain is used. In this test, it is necessary to select the range of sufficient stress that could make the specimens fail in the range of 1,000 to 1,000,000 repetitions. The test temperature is 20 ^{0}C. The shape of the loading wave is sinusoidal without rest periods and the fatigue has been reached and the test completed when the applied load is half of the initial load or stress (Castro and Sanchez, 2007). The deformation of the specimen was monitored through linear variable differential transducers (LVDTs).

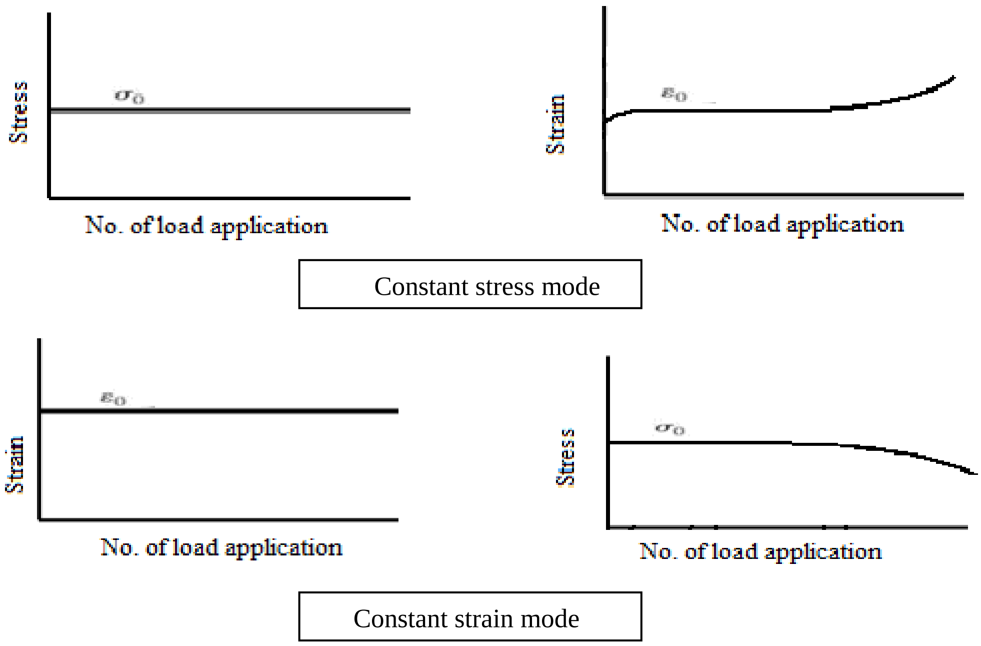


Figure 2-3: Types of controlled loading modes for the beam fatigue test.

The result of this test is plotted as the normal linear relationship between the logarithm of applied initial tensile strain and the logarithm of fatigue life (number of applied load repetitions until failure). The fatigue data were analyzed by running a regression analysis to determine the fatigue relationship parameters in the Equation 2-4 (Asi, 2006).

$$\varepsilon_t = I * (N_f)^s \qquad 2\text{-}4$$

where: ε_t - initial tensile strain.

N_f - number of load repetition to failure.

I - inti-log of the intercept of the logarithmic relationship, and s is the slope of the logarithmic relationship.

The general equation 2-5 for the plot of log of applied stress versus log of cycles to failure also can be used (Xue *et al.*, 2006):

$$N_f = k\left(\frac{1}{\sigma}\right)^n \qquad 2\text{-}5$$

where: N_f - number of cycles to failure.

k, n- regression constants.

σ - initial stress.

2.7 Properties and Performance of Asphaltic Concrete Mixtures

2.7.1 Properties of Asphaltic Concrete Mixture

Many researchers have been conducted study on the effect of fine aggregate characteristics on the asphalt concrete mixture properties and its performance. The correlation between fine aggregate characteristics and the mixture properties has been established by several researchers. The geometric irregularity of the fine aggregate particles has a major effect on the physical properties and mechanical behavior of bituminous paving mixture. It was cited by Park and Lee (2002) that an increase in angularity of fine aggregate increases the Marshall stability values at optimum bitumen content and also increased the voids content at optimum bitumen content while Choyce found that optimum binder contents are much lower for mixtures containing crushed rock fine aggregate than those obtained for mixtures containing natural sand fines.

Topal and Sengoz (2005) and Choyce investigated the effect of fine aggregate shape and surface texture on hot mix asphalt characteristics. They found that angular shaped particle gives better interlocking between particles than rounded particles, such that after mixing and compaction the air void is decreased. They also found that shape and surface texture of aggregate particles affect asphalt demands of mix bonding, workability, density, durability and stability of the asphaltic concrete mixture. Eyad *et al.* (2001) found that fine aggregate with a high degree of angularity (e.g. broken faces) will have a higher uncompacted void contents as compared to fine aggregate with lower degree of angularity (e.g. natural sand). The dense aggregate gradation having maximum density provides increased stability through the increase in interparticle contact and reduces VMA (Abo Qudais and Al Shweily, 2007). The effect of asphalt content is found to be highly dependent on the source of aggregate (Stakston and Bahia, 2003).

2.7.2 Permanent Deformation Resistance of Asphalt Concrete Mixture

The susceptibility of hot bituminous mixture to permanent deformation (rutting) can lead to premature failure of the pavement. Rutting in bituminous mixture can be caused by exceedingly heavy axle loads. It is not only decreases the useful service life of the pavement, but also creates a safety hazard for the traveling public; therefore the resistance to permanent deformation has been studied by many researchers. It was found that physical, chemical and mechanical properties of fine aggregate have played a significant role in the rutting resistance of HMA (Wu et al., 2007; Qudais and Al Shweily, 2007; Topal and Sengoz, 2005).

Shen *et al.* (2005) found that large-size, angular and rough textured aggregates can contribute to rutting resistance and minimize plastic flow, while Qudais and Al Shweily (2007) found that rougher aggregates gave higher resistance to creep, due to high bonding strength between aggregate and binder. A number of researchers have found in their work that fine crush aggregate which is rougher in texture and more angular in shape produced mixtures with higher resistance to deformation, compared to natural sand fine aggregate.

Qudais and Al Shweily (2007) found that aggregate gradation have a significant influence on the creep behavior of hot bituminous mixture. Therefore the dense aggregate gradation having maximum density provides increased stability through the increase in inter-particle contact. Even the HMA containing blended fine aggregate shows lower rut depth than 100% natural sand, because the higher rougher texture contribute lower rut depths of the mixture containing granite and lime stone fines as compared to the mixture containing river gravel and natural sand fines (Park and Lee, 2002).

2.7.3 Fatigue Resistance of Asphaltic Concrete Mixture

One of the major problems affecting the performance of hot mix asphalt is fatigue. Fatigue can be introduced by cyclic loading of traffic, inhomogeneous distribution of asphalt binder, aggregate and voids which makes the stiffness modulus on the pavement to vary, resulting in inhomogeneous induced stress concentration and strain localization. Fatigue cracking decreases pavement performance which leads to increased maintenance as well as user cost; therefore measure should be taken for resisting the fatigue in asphalt concrete mixture (Abo Qudais and Shatnawi, 2007).

Shen *et al*. (2005) found that aggregate gradation (distribution of particle sizes) is one of the most important factors to resist pavement distress. In another study by Asi (2006) the effects of aggregate interlocking on the fatigue life was investigated. He assessed the fatigue life by using control mixture and stone matrix asphalt. The results from the study show that stone matrix asphalt mixture have lower fatigue life than control mixtures. This is referred to the lack of mechanical locking of the aggregate because stone matrix asphalt mixture is a gap graded asphalt mixture. Abo Qudais and Al Shweily (2007) found that chemical composition of aggregate has a significant effect on the stripping behavior, indirectly it effects on the cracking because one of the distresses that might be caused by stripping is cracking.

2.8 Summary

A summary of the earlier studies on fine aggregate and polymer modified bitumen characteristic have been highlighted. These studies have confirmed that physical, chemical and mechanical characteristics of fine aggregate have significant effects on the properties and performance of HMA pavements. Polymer modified bitumen showed highly enhanced properties and performance mixture at both low and high temperature ranges. The characteristic of fine aggregate have been found to increase the stability, density, and affects the optimum bitumen content (OBC), and it changes the workability of the bituminous mixture readiness of the type and properties of fine aggregate. Several studies by many researchers showed that fine aggregate properties largely influence the asphalt mixture properties and its performance, depending mostly on their mineralogy and finer size particles. Therefore angular, rougher and higher shear strength fine aggregates can increase the resistance to permanent deformation (rutting), and also rough, angular, high strength and good distribution for particles (gradation) also could give a strong mixture, that can reflect in better fatigue resistance. Finally from the literature review, it can be concluded that aggregate characteristics have strong effect on the mixture properties and performance.

CHAPTER 3

3. RESEARCH METHODOLOGY

3.1 Introduction

This chapter describes the experimental program of the study. The program can be divided into three parts: preparation of the raw materials, preparation of the bituminous mixtures specimens and performance tests of the bituminous mixtures.

The first part deals with raw material preparation that consists of material selection and determines its characteristics. The material selection consist of aggregate which includes granite as coarse aggregate, quarry sand, river sand, mining sand and marine sand as fine aggregate. The binder's were two conventional bitumen types and four polymer modified bitumen. Characteristics of these materials were determined through several tests. For binder characterization penetration test, softening point test, ductility test and specific gravity test were conducted, while for aggregates and filler sieve analysis test and specific gravity test were performed. For fine aggregate addition tests were used which were fine aggregate angularity test, physical appearance test, X-ray fluorescence (XRF) and shear strength test. These materials which have different characteristics were mixed to get the mixture engineering properties using Marshall Mix design method.

Marshall Test was carried out to get the optimum bitumen content which was used for preparation of the performance tests specimens. Performance tests carried out included the permanent deformation and fatigue tests which were performed in the final part of the experimental program. Two types of tests namely the wheel tracking test and dynamic creep test were used to assess the permanent deformation of the bituminous mixture, while beam fatigue test was used to assess the fatigue properties of the bituminous mixture. The mixture properties from Marshall Test and mixture performance from permanent deformation and fatigue tests were evaluated and discussion. The conclusions and recommendation were found. The research methodology summarised through a flow chart as shown in Figure 3-1.

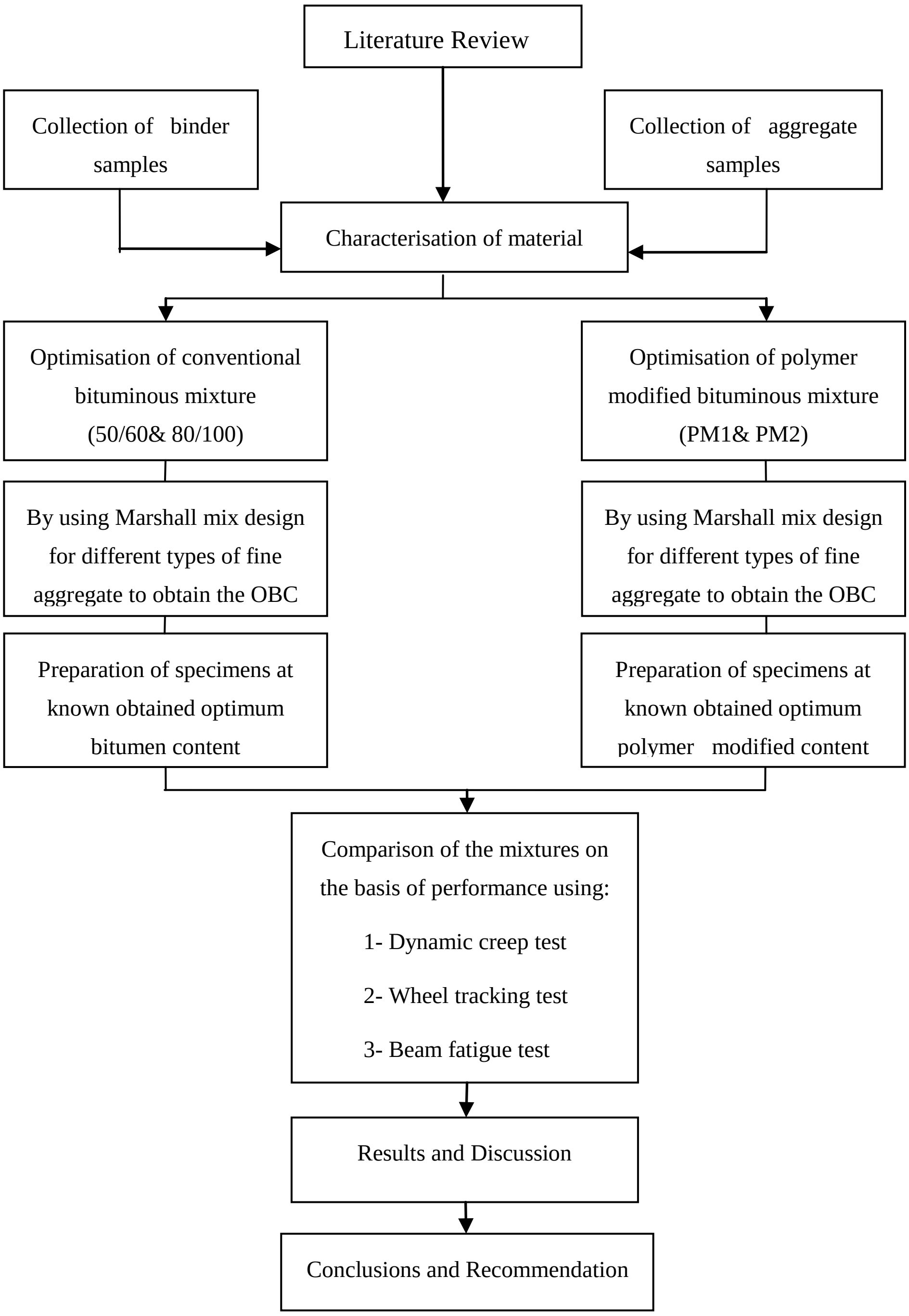


Figure 3-1: Flow Chart of Research Methodology

3.2 Material Preparation

Material preparation includes material selection and determining its characteristics as discussed in the following sections.

3.2.1 Materials Selection

The two main constituents of bituminous mix were binders and aggregates. The selections of material and mixture design were referred to the JKR standard (1988). In this study, asphaltic concrete (AC) mixture designated as ACW 20 was used. The maximum aggregate size used was that retained on sieve size 20 mm and the bitumen content ranged from 3.5% to 7% by weight of the total mix. The materials selections which include binder, coarse aggregate, fine aggregate and filler were discussed as following.

3.2.1.1 Binder

A total of six binders i.e. two conventional bitumen and four polymers modified bitumen were used in this study. The conventional bitumen were PEN 50/60 and PEN 80/100, while the polymer modified bitumen were designated as PM1 (PM1_82& PM1_76) and PM2 (PM2_82& PM2_76). PM1 modified bitumen is made up by addition styrene butadiene styrene (SBS) to bitumen penetration 82 and bitumen penetration 76 to come up with PM1_82 and PM1_76. PM2 modified bitumen is made up by addition one of the plastomer polymer to bitumen penetration 82 and bitumen penetration 76 to come up with PM2_82 and PM2_76. The SBS which is considering one of the elastomer polymers consists of two elements: hard polystyrene end blocks and rubbery midblock. The hard polystyrene end blocks give high tensile strength and flow resistance at high temperature, whereas the rubbery midblocks are responsible for the elasticity, fatigue resistance and flexibility at low temperature (Airey, 2003; Ahmedzade *et al.*, 2007). Both of the conventional bitumen are manufactured from refined crude oil and were obtained from Bellamy Precision and Berne Science respectively. Polymer modified bitumen was manufactured by adding polymers to the bitumen and were obtained from PPMS Technologies supplier in Malaysia.

3.2.1.2 Coarse aggregate

For the purpose of the study, granite has been chosen as the coarse aggregate. The granite coarse aggregate is material retained on 5 mm sieve size. A well-graded aggregate gradation with a maximum aggregate size equal to 20 mm was used according to the JKR standard. The granite was collected from a quarry around Ipoh.

3.2.1.3 Fine aggregate

Fine aggregate should pass through 5 mm sieve size for well graded bituminous mixture, the fine aggregate used in study was mainly sand. Four different types of sand namely quarry sand, river sand, mining sand and marine sand were collected from four different locations in Malaysia. River sand, mining sand and marine sand are natural sand while quarry sand is crushed granite.

3.2.1.4 Filler

Filler in the mix is used to fill up the voids left between the aggregate and to provide cohesion to the mixture. The filler used in this study was the Portland cement, at least 75% of it shall pass 0.075 mm sieve size. The Portland cement was obtained from YTL Cement Berhad.

3.2.2 Raw Material Characterization

3.2.2.1 Binder

To determine the specification for binders, several physical properties of the conventional bitumen and polymer modified bitumen were obtained by penetration test, softening point test, ductility test and specific gravity test.

3.2.2.1.1 Penetration test

The penetration test is a test used to measure the consistency of semisolid and solid asphaltic materials. The specified penetration grades for bitumen as per ASTM D946 are 40/50 Pen, 60/70 Pen, 85/100 Pen, 120/150 Pen, and 200/300 Pen. High penetration grade bitumen is used in colder weather to minimize low temperature cracking, while low penetration grade bitumen are preferred for use in hot climate to minimize high temperature rutting of the asphalt pavement (Brown *et al.*, 1996). Bituminous binder for asphaltic concrete shall be penetration graded bitumen of 80/100 grade (JKR, 1988), also there is no specific specification for bitumen pen 50/60 in ASTM there for 40/50 pen and 60/70 pen are taken as arrange for the pen 50/60 and 85/100 for pen 80/100. The test was carried out according to the British Standard test procedure B.S. 2000: Part 49: 1983. The test consists of determining how far a standard steel needle will penetrate into the bituminous material under standard conditions which is maintain at a temperature of 25 ^{0}C and under the load of 100g for 5s. The results are expressed in points of penetration (0.1 mm). The bitumen is tested in a container which is kept in a water bath maintained at the standard temperature of 25 ^{0}C during the test. The penetration test apparatus is shown in Figure 3-2.

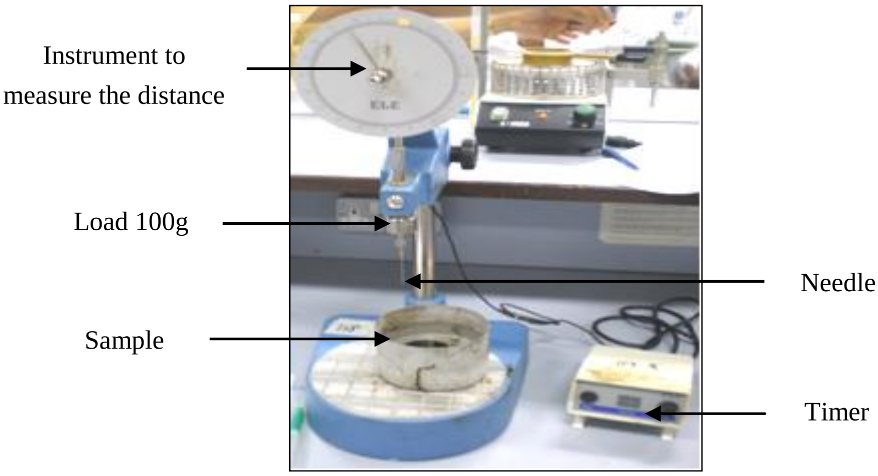


Figure 3-2: Penetrometer apparatus

3.2.2.1.2 Ductility test

Low temperature ductility was used to characterize the cohesive strength of penetration grade bitumen. In the standard ductility test a briquette of semisolid asphalt with a minimum central cross section of 100mm^2 is stretched to a thread at the rate of pull of 50 mm/min in a water bath at a temperature of 25 ^{0}C. The maximum elongation of the thread at failure measured in centimeters is the ductility value. Three dumbbell shape samples of bitumen were tested. The ductility test was carried out in accordance to the ASTM D 113. The Ductilometer apparatus test is shown in Figure 3-3.

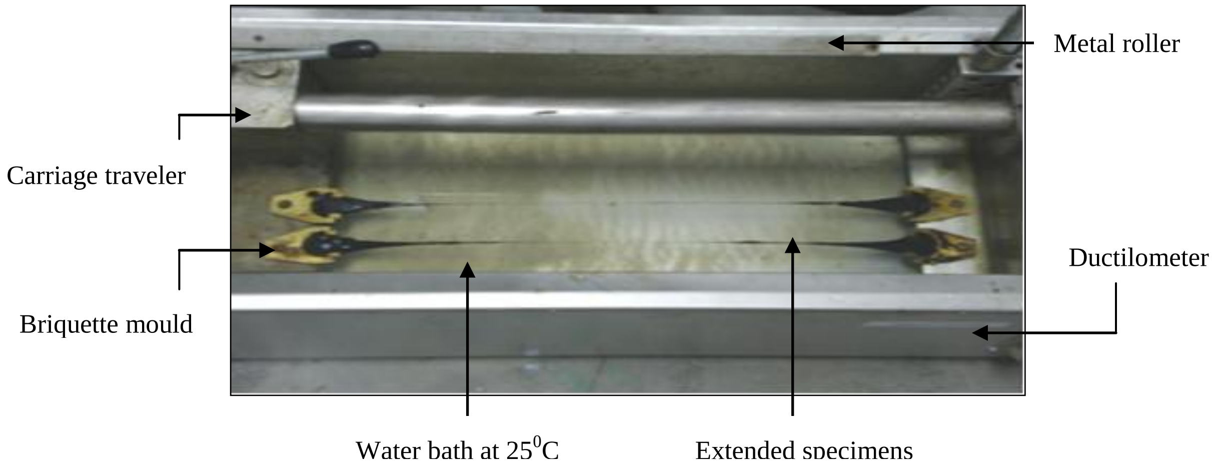


Figure 3-3: Ductilometer apparatus

3.2.2.1.3 Ring and ball test (Softening Point)

The purpose of the test is to determine the temperature at which a phase change occurs in the bitumen. A steel ball of 3.5g weight is placed on a disk of sample contained within a horizontal, vertically supported, metal ring of specified dimensions. The assembly is heated in water bath at a uniform, prescribed rate until the binder started to deform due to decreasing viscosity as the temperature was increasing. In time, the steel ball will drop as the bitumen deformed, the temperature at which it hit the plate was recorded as the softening point temperature. This test was done in accordance to B.S 2000: Part 58: 1983. The softening point apparatus is shown in Figure 3-4.

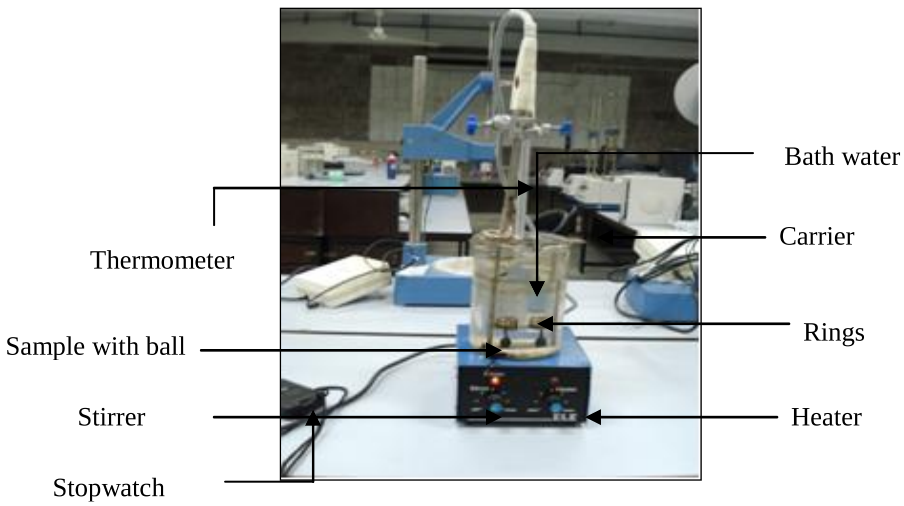


Figure 3-4: Softening point apparatus

3.2.2.1.4 Specific gravity test

The specific gravity test was conducted accordance to ASTM D3289 test method. This test involves placing a small amount of bituminous material into a pycnometer and measuring the weight of the sample compared to that for an equal volume of water. The specific gravity is the ratio of the weight of a material compared to the weight of an equal volume of water. This is typically conducted at 25^{0}C. Figure 3-5 shows the pycnometers for the specific gravity determination.

$$S.G = \frac{(C-A)}{[(B-A)-(D-C)]} \qquad 3\text{-}1$$

where: *S.G*- specific gravity.

A- mass of empty pycnometer (g).

B- mass of pycnometer filled with water at 25 ^{0}C (g).

C- mass of pycnometer with sample at 25 ^{0}C (g).

D- mass of pycnometer with sample and water at 25 ^{0}C (g).

Dry and clean pycnometer Pycnometer with sample Pycnometer+sample+water

Figure 3-5: The procedure in measuring the specific gravity of binder

3.2.2.2 Aggregate

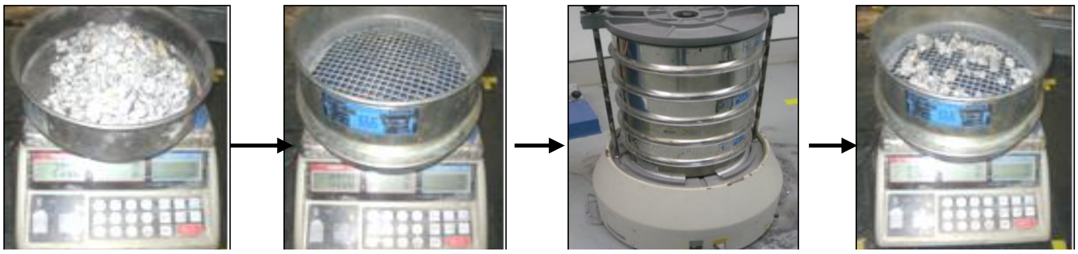
The type of coarse aggregates used is granite while the types of fine aggregates are quarry sand, river sand, mining sand and marine sand. Sieve analysis and specific gravity tests were performed for aggregate and filler, while additional tests i.e. physical, chemical and mechanical tests were performed for fine aggregates because the fine aggregate is the main part in this study.

3.2.2.2.1 Sieve analysis test

This test was conducted on all aggregate types to determine the gradation and proportions of the aggregate to obtain the required gradations to be used for Marshall Design Test. The proportions are determined for coarse, fine aggregate and filler. The aggregates were sieved according to JKR (1988) specification using a series of sieves with different sizes as shown in Table 3-1.

To prepare the materials for sieve analysis test coarse and fine aggregate were washed with tap water and then the clean samples were dried in the oven for 24 hours at 100 ^{0}C. Three samples of each aggregate type were prepared.

The sieve analysis test for coarse aggregate was conducted using two kg of coarse aggregate, the sample was then passed through a set of five sieves with sizes 28mm, 20mm, 14mm, 10mm, and 5mm respectively. The set of sieves were shaken by a machine shaker for 10 minutes. The remaining aggregate on each sieve was then weighed together with the sieve as shown in Figure 3-6. Three samples were tested and the actual coarse aggregate percentage passing was calculated.



Weighting sample | Weighting the sieve | Sieving process | Weighting sieve+ sample

Figure 3-6: The procedure in conducting sieve analysis test

The sieve analysis test for fine aggregate is conducted by the same procedure as for coarse aggregate was followed. 0.5 kg of fine aggregate was sieved using different sieves sizes: 5mm, 3.35mm, 1.18mm, 0.425mm, 0.150mm, 0.075mm. The percentages of material retained on the sieves were calculated. Three samples were tested and the actual fine aggregate percentage passing was calculated.

The gradation of fine material is very important because a balanced mixture of coarse aggregates and fine aggregates is needed in order to provide the required friction on the pavement and to obtain optimum binder content (Anderson). Four types of sand which were quarry sand, river sand, mining sand and marine sand were sieved.

The materials gradation for asphaltic concrete (AC) mixture specified in the JKR standard (1988) is shown in Table 3-1. The wearing course ACW 20 gradation limit was used as reference in this study.

Table 3-1: Gradation limits for AC as in JKR Specification for Road Works

Mix Type	**Wearing Course**	**Binder Course**
Mix Designation	ACW 20	ACB 28
B.S Sieve	% Passing by weight	
37.5 mm	100	100
28.0 mm	100	80-100
20.0 mm	76-100	72-93
14.0 mm	64-89	58-82
10.0 mm	56-81	50-75
5.0 mm	46-71	36-58
3.35 mm	32-58	30-52
1.18 mm	20-42	18-38
425 µm	12-28	11-25
150 µm	6-16	5-14
75 µm	4-8	3-8

3.2.2.2.2 Specific gravity test

Specific gravity (particle density) is an important property especially in asphalt mixture design. Considering the voids in the aggregate, three relative densities can be measured, namely the apparent relative density, relative density on a saturated and surface and relative density on an oven dry. The test also enhances the absorption properties of aggregate tested.

The specific gravity test for coarse aggregate was conducted using 1kg of coarse aggregate. The coarse aggregate sample is washed to remove finer particles and then is immersed in water in the glass vessel at 25^0C for 24 hours. The vessel is overfill by adding water and weighted as mass B. Aggregate is dried using cloth until all visible films of water are removed and weighted as mass A. The pycnometer is refilled with water only to the top of cone and the weight is determined as mass C. The aggregate is then dried on the oven for 24 hours and the weight is determined as mass D. This test was conducted accordance to BS 812: Part 107, 1990. The specific gravity and absorption are determined using the following equations:

$$RDD = \frac{D}{A-(B-C)} \tag{3-2}$$

$$RDS = \frac{A}{A-(B-C)} \tag{3-3}$$

$$ARD = \frac{D}{D-(B-C)} \tag{3-4}$$

$$WA = \frac{100 \times (A-D)}{D} \tag{3- 5}$$

where: *RDD*- relative density on an oven-dried basis.

RDS- relative density on a saturated and surface-dried basis.

ARD- apparent relative density.

WA- water absorption (% of dry mass).

To determine the specific gravity of fine aggregate, a sample of fine aggregate is washed to remove all materials finer than 75µm and then immersed in water at 25 ^{0}C for 24 hours. This followed by exposing the sample to warm air to evaporate surface moisture and stir it at frequent intervals to ensure uniform drying. Then the saturated surface dry is weighted as mass A. 500 g of the saturated surface dry material is placed in the pycnometer, water is added to the top of pycnometer`s cone and the weight is determined again as mass B. The pycnometer is refilled with water only to the top of cone and the weight is determined as mass C. Finally, the sample is dried on the oven for 24 hours and the weight is determined as mass D. This test was conducted accordance to BS 812: Part 107, 1990. The specific gravity and absorption are determined using the following equations:

$$RDD = \frac{D}{A-(B-C)} \tag{3-6}$$

$$RDS = \frac{A}{A-(B-C)} \qquad 3\text{-}7$$

$$ARD = \frac{D}{D-(B-C)} \qquad 3\text{-}8$$

$$WA = \frac{100 \times (A-D)}{D} \qquad 3\text{-}9$$

where: *RDD*- relative density on an oven-dried basis

RDS- relative density on a saturated and surface-dried basis.

ARD- apparent relative density.

WA- water absorption (% of dry mass).

Ultra pycnometer 1000 equipment was used to measure the specific gravity of filler. It allows the volume of small amounts of sample to be accurately measured with enhanced reliability to within a fraction of a microliter. Three samples were used to obtain the average specific gravity. The procedure in measurement is shown in Figure 3-7.

Weighting the sample   Sample in the ultra pycnometer   Running the test

Figure 3-7: The procedure in measuring specific gravity of filler a hip the pycnometer

3.2.2.2.3 Fine aggregate angularity test

The fine aggregate angularity (FAA) test is used to measure the angularity and surface texture of fine aggregates. In this test a sample of fine aggregate is poured into a small cylinder by flowing it through a funnel. The voids in the uncompacted fine aggregate in the cylinder, is expressed as a percentage, and are used to gage fine aggregate angularity. The higher the amount of void content, the more angular and the rougher the surface texture of fine aggregate. FAA test method is based on the ASTM C 1252 specification. The following equation can be used to calculate the percentage of voids:

$$(V-(W/G_{SB}))/V) \times 100 \qquad 3\text{-}10$$

where: V- volume of cylinder (mm^3)

W- weight of loose fine aggregate to fill the cylinder (g).

G_{SB}- bulk specific gravity of fine aggregate.

3.2.2.2.4 Physical appearance and chemical properties of fine aggregate

The physical appearance of fine aggregate was determined by using the scanning electron microscopy (SEM) (Wu *et al*., 2007; Xue *et al*., 2006). In this work the SEM was also employed at different magnifications, while the chemical composition was determined by the X-ray fluorescence (XRF) technique.

3.2.2.2.5 Mechanical property of fine aggregate

Resistance to shear of a cohesionless soil or fine aggregate is derived from friction between grains and the interlocking of the grains (Park and Lee, 2002). The shear strength of a soil mass is the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it (Das, 1998). The shear strength parameters of a soil can be determined in the laboratory by direct shear test. Typical values of angle of internal friction (Ø) for some granular soils are given in Table 3-2.

Table 3-2: Typical angle values of internal friction for sands and silts (Das, 1998)

Soil type	Ø (deg)
Sand- Rounded grains	
Loose	27-30
Medium	30-35
Dense	35-38
Sand- Angular grains	
Loose	30-35
Medium	35-40
Dense	40-45
Gravel with some sand	34-48
Silts	26-35

The direct shear test was conducted on a metal shear box (100mm×100mm) by using direct shear box apparatus with different load as shown in Figure 3-8 and 3-9, in which the soil specimen was placed. The box was split horizontally into two halves, and normal force on the specimen was applied from the top of the shear box. Shear force was applied by moving one half of the box relative to the other to cause failure in the soil specimen. The readings from three gauges (vertical, horizontal and force gauge) were taken after every 2 minutes, until they decreased or became constant for the last three readings.

In loose sand the resisting shear stress increases with shear displacement until a failure shear stress is reached. After that the shear resistance remains approximately constant.

In dense sand, the resisting shear stress increases with shear displacement until it reaches a failure stress which is the peak shear strength. After the failure stress was attained, the shear stress resistance gradually decreases as shear displacement increases, until it finally reaches a constant value called the ultimate shear strength. This procedure was repeated three times with loads of 10 kg, 20 kg, and 30 kg. A shear stress versus horizontal displacement for each loading condition was plotted and the maximum shear stress (τ) at each load was determined. Then the three shears stress values were plotted against the mass, as shown in Figure 3-10, to determine the angle of internal friction (Ø). This test was conducted according to BS 1377: Part 7: 1990 using the direct shear box apparatus.

Figure 3-8: Equipment for the test

Figure 3-9: Direct shear box apparatus with different load

The friction angle, Ø is defined as follows;

$$Ø = \tan^{-1}(\tau_f/\sigma) \quad \text{3-10}$$

where: Ø- angle of internal friction.

τ_f- shear stress (peak shear strength).

σ- normal stress.

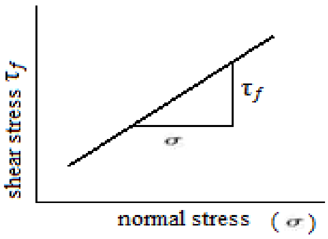


Figure 3-10: Determination of the angle of internal friction (Ø) (Das, 1998)

3.3 Mixtures

A total of 24 different mixtures were designed and prepared for the Marshall Mix design test and performance tests. The performance tests conducted in this study were the dynamic creep test and wheel tracking test (to determine the deformation characteristics of the mix) and the beam fatigue test (to determine the fatigue or cracking properties of the mix).

3.3.1 Mixture Design

Six types of binder were used to prepare different bituminous mixtures. The binders were four types of polymer modified bitumen PM1_82, PM1_76, PM2_82 and PM2_76 and two types of conventional bitumen PEN 50/60 and PEN 80/100. Four types of sand (quarry sand, river sand, mining sand, and marine sand) were used also in the preparation of the bituminous mixtures. Each sand type was blended with different type of binder, resulting in a total of 24 bituminous mixtures. Each mixture was assigned a designated code based on the type of binder and type of sand as listed on Table 3-3.

Table 3-3: Mixture design variations using different type of sand and binder

Type of binder	**A type of mixture depends on type of fine aggregate (sand) and binder.**			
	Quarry sand	**River sand**	**Mining sand**	**Marine sand**
PM1_82 (SBS)	(Q/PM1_82)	(R/PM1_82)	(M/PM1_82)	(MR/PM1_82)
PM1_76 (SBS)	(Q/PM1_76)	(R/PM1_76)	(M/PM1_76)	(MR/PM1_76)
PM2_829 (Plastomer)	(Q/PM2_82)	(R/PM2_82)	(M/PM2_82)	(MR/PM2_82)
PM2_76 (Plastomer)	(Q/PM2_76)	(R/PM2_76)	(M/PM2_76)	(MR/PM2_76)
PEN 50/60	(Q/PEN 50/60)	(R/PEN 50/60)	(M/PEN 50/60)	(MR/PEN 50/60)
PEN 80/100	(Q/PEN 80/100)	(R/PEN 80/100)	(M/PEN 80/100)	(MR/PEN 80/100)

3.3.2 Mixture Specimens Preparation

Specimen's preparation for the Marshall test was conducted to get the optimum bitumen content. This optimum bitumen content will be used to prepare the specimens for performance tests. More descriptions are given in the following sections.

3.4 Marshall Test

Marshall Test method was used to determine the optimum bitumen content. Marshall Test includes the specimen's preparation and Marshall Stability and flow tests.

The aggregates in the mix consisting of coarse aggregate, fine aggregate and filler were heated to 160 ^{0}C and mixed with heated bitumen at the same temperature until all particles are coated with bitumen. The mix material was compacted in 100 mm diameter steel moulds which was also kept at 160 ^{0}C. To ensure that all the materials will be compacted evenly in the mould, the mix was tamped using a steel rod for 25 times at the periphery and 5 times at the centre. The Marshall Compactor was used to compact the sample at 75 blows per face. This is in accordance to the requirement of compaction for heavy traffic as specified by Asphalt Institute Manual MS 2. The bitumen used ranged from 3.5 % to 7 %, three specimens of each bitumen contents was prepared to conduct Marshall Test. The sample preparation procedures are shown in Figure 3-11.

Figure 3-11: The procedure involved in preparing the specimens for the Marshall test

The Marshall Stability and flow tests were carried out on compacted specimens at various binder contents according to BS 598 1985. Before performing the Marshall test, the height of specimen was determined. The samples were then weighted in air and water to determine their properties relative to density and porosity. The Marshall Test is an empirical test in which cylindrical compacted specimen, 100 mm in diameter was immersed in water at 60 ^{0}C for 30 min. The specimen is then loaded on the Marshall Test apparatus. The apparatus consist of curved steel loading plates along a diameter of specimen. The test involves loading the specimen at a constant rate of strain 50.8 mm/min. The Marshall Stability value (in kN) is the maximum force that the specimen could withstand until it fails, while the flow (in mm) is the deformation at that force. The stability value obtained was then corrected by the appropriate coefficient Appendix A. Marshall Stability divided by flow is used to assess stiffness of the bituminous mixture. The Marshall Test apparatus for stability and flow determination is shown in Figure 3-12.

Figure 3-12: Marshall Test apparatus

From the measurements of weight and height of the specimen, mixture density, voids in mineral aggregate (VMA), voids filled with bitumen (VFB) and air voids (AV) for the mixture can be calculated. Mixture stiffness was calculated from the stability and flow values determined from Marshall Test.

$$D = \frac{W_{air}}{W_{air} - W_{water}} \qquad 3\text{-}11$$

$$VMA = (V_A + V_B) \times 100\% \qquad 3\text{-}12$$

$$VFB = \left(\frac{V_B}{(V_B + V_A)} \right) \times 100\% \qquad 3\text{-}13$$

$$AV = (1 - (V_G + V_B)) \times 100\% \qquad 3\text{-}14$$

$$M\,S = \frac{Stability\,(kN)}{Flow(mm)} \tag{3-15}$$

where: D- density of bituminous mixture

W_{air} - weight of sample in air (gm).

W_{water} - weight of the sample in water (gm).

VMA- voids in mineral aggregate.

VFB- voids fill with bitumen.

AV- air voids.

V_G- volume of aggregate (m^3).

V_B- volume of bitumen (m^3) and V_A is the volume of air (m^3).

$M\,S$- Marshall Stiffness (kN/mm)

The values of stability, flow, air voids, density, voids filled with bitumen and voids in mineral aggregate for each binder and sand type were obtained by averaging three calculated values. All of these parameters were plotted against the binder content. The optimum binder content was calculated as the average of asphalt contents that meet the maximum stability, maximum density, minimum voids in mineral aggregate and 5 % air voids.

3.5 Performance Tests

To assess the mechanical characteristic of bituminous mixtures for purpose of pavement design and performance prediction, three main tests were used. Dynamic creep test and wheel tracking test were used to evaluate the rutting performance of the mix while the beam fatigue test was used to evaluate the fatigue properties of the mix.

3.5.1 Dynamic Creep Test

The creep test is a simple test that can satisfactorily predict hot mix asphalt performance in deformation. It measures the rut susceptibility of hot asphalt mixtures (Ahmedzade and Yilmaz, 2007). After the optimum bitumen content (OBC) was obtained from the Marshall test, similar specimens as in the Marshall test were prepared at optimum binder content. Three specimens were prepared for each mixture variation (type of binder vs. type of sand).

Figure 3-13: Specimen for Creep test

Creep test is a simple test carried out on specimens of actual bituminous mixes for known temperatures at a specified frequency of loading. This test applies a repeated pulsed uniaxial load to a mixture specimen and measures the resulting deformations in the same axis using Linear Variable Displacement Transducers (LVDTs). The data of the creep test were plotted to show the relationship between permanent deformations (mm) and number of cycles. The set-up for the dynamic creep test is shown in Figure 3-14. The specimen was placed inside the pre-heated chamber for 2 hours to reach a uniform temperature of 40 ^{0}C. This test was conducted in accordance to British Standard DD226 with the following specifications:

1. Preload Option
    - Stress: 12 kPa
    - Holding Time: 120 s
2. Loading Options
    - Wave shape: square pulse
    - Pulse width: 1,000 ms
    - Rest Period: 1,000 ms
    - Contact Stress: 2 kPa
    - Deviator Stress: 100 kPa
3. Termination Option

- Axial load reaches 30,000 micro-strains or 0% (if strain displayed as a percentage)
- 1,800 loading cycles.

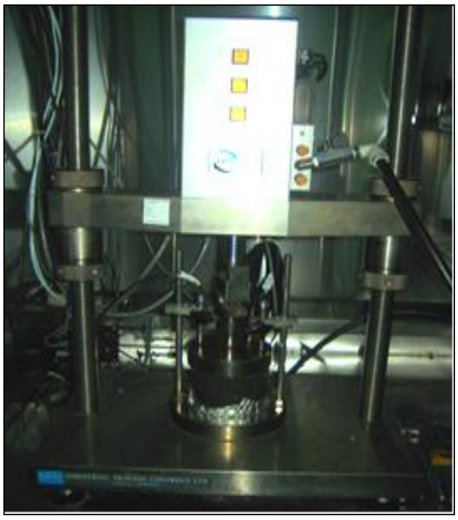


Figure 3-14: Creep test apparatus

3.5.2 Wheel Tracking Test

The wheel tracking test was used to measure the mixture’s resistance to permanent deformation under repeated loading (Shen *et al.*, 2005; Tayfur *et al.*, 2007). It was performed using wheel tracking device machine (Topal and Sengoz, 2006).

Square specimens were prepared for the wheel tracking test, each specimen weighting 10 kg having dimensions 300mm×300mm×50mm depth. All the materials used consisting coarse aggregate, fine aggregate and filler were heated to 160 ^{0}C and mixed with the binder at optimum bitumen content at same temperature (160^{0}C) using the electric mixer shown in Figure 3-15. Grease was applied on the inside of the square metal mould before the mixture was poured, so that the specimen can be removed easily from the mould. The mixture was then compacted in two layers using electric hand compacter. Two samples were prepared foreach mixture variation (type of binder vs. type of sand).

Figure 3-15: Electric mixer

The wheel tracking test was conducted at 40 ^{0}C. An actual wheel of 200 mm diameter and 50 mm width with a total wheel load of 520 N was applied on the square specimen. The wheel was run backward and forward across the center of the bituminous specimen at the frequency of 42 times per minute for 45 minutes loading. The total rut depth was determined and recorded by the Wessex software, the software that came together with the testing machine. The apparatus for wheel tracking is shown in Figure 3-16. Figure 3-17 shows the rut depth on the specimens after the test. The testing procedures conform to the specification of BS 598 110: 1998. The wheel tracking test results were correlated to the dynamic creep test results to determine the effect of fine aggregate properties on the rutting resistance of the bituminous and polymer modified bituminous mixtures.

Figure 3-16: Wessex wheel trackers

Figure 3-17: Wheel tracking specimen with different rut depth

3.5.3 Beam Fatigue Test

The name “fatigue” is based on the concept that a material becomes “tired” and fails at a stress level below the nominal strength of the material. Dynamic bending is the test used to assess the fatigue life on bituminous mix specimens (Whiteoak, 1990). Rectangular specimens were prepared using rectangular metal mould. The mass of material used in the mixture is approximately 7700 g with a dimension of 100 mm x 100 mm x 500 mm. The materials used consisted of coarse aggregate, fine aggregate and filler were heated and mixed with bitumen at the optimum binder content at 160 ^{0}C using the electric mixer. Grease was applied on the inside of the rectangular metal mould before the mixture was poured, for easy removal of the solidified specimen. The specimen was then compacted in two layers using the electric hand compactor shown in Figure 3-18. An electric cutter was used to cut the sample into two specimens with the dimension of approximately 50 mm x 65 mm x 380 mm. Two specimens were prepared for each mixture variation (type of binder vs. type of sand).

Figure 3-18: Hand compactor

Fatigue test was performed using the MATTA apparatus. All the specimens were kept at 20^0C for 2 hours before testing. The test temperature 20^0C was chosen since the effect of air void content on fatigue life is more pronounced than at lower temperatures (Kim *et al*., 1991). The beam fatigue test was conducted by applying a repeated flexural bending to a bituminous beam specimen in control strain mode. A thin pavement with thickness of less than 60 mm is suggested for use in the control strain mode because failure will be more noticeable in this mode (Doan, 1997). The applied force and the resulting beam deflection were measured using an on-specimen Linear Variable Displacement Transducers (LVDTs). The number of load repetitions at which the current stiffness decreases to 50% of the initial value is defined as the fatigue life of the specimen. Figure 3-19 shows the beam fatigue apparatus and Figure 3-20 shows the different cracks on the bituminous specimens after the test performed.

Figure 3-19: Beam fatigue apparatus

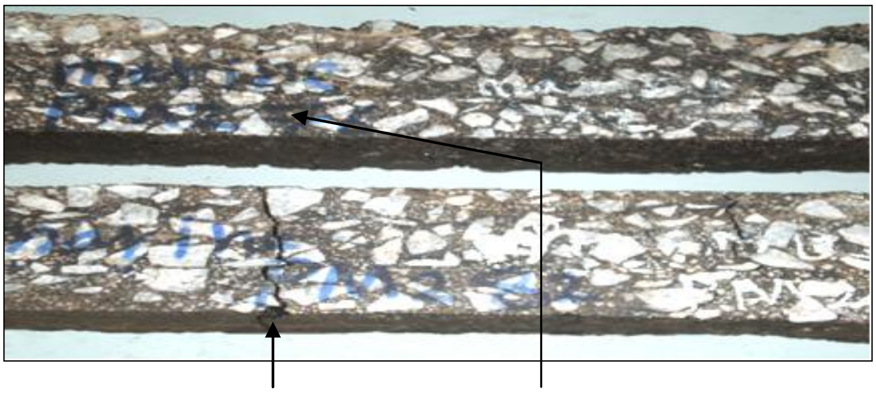


Figure 3-20: Different cracks on different tested specimens

The parameters used in beam fatigue test were as follow:

1. Default Poisson ratio: 0.4
2. Loading conditions:
   - Control mode: sinusoidal strain
   - Pulse width: 200 ms
   - Frequency: 5 Hz
   - Peak to peak: 100 micro strain
   - Conditioning: 50 cycles
3. Termination conditions:
   - Termination stiffness: 50% of the initial stiffness or
   - Stop test after 1,000,000 cycles

The tabulated test data of the load and deformation were updated every 10th cycle by the corresponding UTM 4-21 software of the apparatus.

3.6 Mixture Optimization

The bituminous mixtures were evaluated based on their properties and performance to determine the optimum bituminous mixture. The optimum bituminous mixture should have the highest density, stability and stiffness and low values of VMA. The mixtures performance was given a ranking from the most optimum performance to the least optimum performance. The evaluation and ranking for each different mixture is summarized in the next chapter.

CHAPTER 4

4. RESULTS AND DISCUSSIONS

4.1 Introduction

This chapter presents and discusses the results of material characterisation, Marshall Test and performance tests of bituminous mixtures. The results highlighted the effect of fine aggregate properties on the bituminous mixture properties and performance.

4.2 Materials Properties

Material properties include binders and aggregates properties which were used in either calculation or to verify the finding from the other tests results.

4.2.1 Binder Properties

Before actual stress tests were performed on the bituminous mixtures, the mechanical properties of the binders were determined through various tests which include penetration test, softening point test, ductility test and specific gravity test. These tests were performed in order to know the consistency of binders. The results of the consistency tests are shown in Table 4-1. The penetration and softening point values of conventional binder were obtained to confirm the bitumen specifications provided by suppliers. The results show that the bitumen used, meets the requirement of the specification as out fined in the various ASTM standard as shown in the table. The consistency tests proved that bitumen PEN 50/60 has higher consistency than bitumen PEN 80/100. The ductility results indicate that bitumen PEN 80/100 is more flexible than bitumen PEN 50/60, because bitumen PEN 80/100 has higher value of ductility than bitumen PEN 50/60. The softening point results show that bitumen PEN 50/60 needs higher temperature to soften than bitumen PEN 80/100. The results proved that bitumen PEN 50/60 is harder than bitumen PEN 80/100.

Table 4-1: Physical characteristics of binders

Type of bitumen	Penetration@ 25^{0}C		Softening point (^{0}C)		Ductility (cm)		Specific gravity@ 25 ^{0}C	
	Value obtained	Guaranteed level	Value obtained	Guaranteed level	Value obtained	Guaranteed level	Value obtained	Guaranteed level
PEN 50/60	49	50 - 60	52	52.5	36	-	1.032	1.0 - 1.04
PEN 80/100	90	80 - 100	49	45 - 52	125.6	100 and above	1.030	1.0 - 1.04
PM1_82 (SBS)	39	35 - 50	70	> 70	59.1	-	1.028	1.0 - 1.04
PM1_76 (SBS)	47	35 - 50	67	> 60	56.4	-	1.027	1.0 - 1.04
PM2_82 (Plastomer)	48	35 - 50	65	> 60	35.5	-	1.032	1.0 - 1.04
PM2_76 (Plastomer)	48	40 - 60	53	55	29.6	-	1.035	1.0 - 1.04

Tables 4-2 and 4-3 show the comparison between conventional binders and polymer modified binders. The results show that polymer modified bitumen has lower penetration and higher softening point compared to the conventional bitumen. This means the stiffness for polymer modified bitumen is higher than the conventional bitumen, thus it may exhibit better resistance to rutting.

Table 4-2: Comparison between PMB and bitumen PEN 50-60

Type of test	PEN 50/60	PM1_82		PM1_76		PM2_82		PM2_76	
		Obtained value	Decrease or increase from the base value	Obtained value	Decrease or increase from the base value	Obtained value	Decrease or increase from the base value	Obtained value	Decrease or increase from the base value
Penetration (mm)	49	39	(-20.4%)	47	(-4.1 %)	48	(-2 %)	48	(-2 %)
Softening Point (^{O}C)	52	70	(+34.6%)	67	(+28.8%)	65	(+25%)	53	(+1.9%)
Ductility (cm)	36	59	(+63.9%)	56	(+55.6%)	35	(-2.8%)	29	(-19.4%)

Table 4-3: Comparison between PMB and bitumen PEN 80-100

Type of test	PEN 80/100	PM1_82		PM1_76		PM2_82		PM2_76	
		Obtained value	**Decrease or increase from the base value**	**Obtained value**	**Decrease or increase from the base value**	**Obtained value**	**Decrease or increase from the base value**	**Obtained value**	**Decrease or increase from the base value**
Penetration (mm)	90	39	(-56.6%)	47	(- 47.8%)	48	(- 46.7%)	48	(- 46.7%)
Softening Point (^{O}C)	49	70	(+42.9%)	67	(+36.7%)	65	(+32.7%)	53	(+8.2 %)
Ductility (cm)	125	59	(-52.8%)	56	(-55.2%)	35	(- 72%)	29	(- 76.8%)

The density of binders was determined using calibrated pycnometer, 25 ml volume. It is noted that the density of bitumen PEN 50/60 is higher than that of bitumen PEN 80/100. This is because bitumen PEN 50/60 is harder than bitumen PEN 80/100. PM2 polymer modified has the highest density than PM1 polymer modified and conventional bitumen. These results agree with the ductility results where PM2 polymer modified is less ductile compared to PM1 polymer modified and conventional binders. This means PM1 polymer modified has high tensile strength, even though the high flow resistance makes the PM1 polymer modified hard and able to resist deformations at both high and low temperature better than PM2 polymer modified. Because the PM1 polymer modififed bitumen consists of SBS polymer, the SBS consists of two elements: hard polystyrene end blocks and rubbery midblock. The hard polystyrene end blocks give high tensile strength and flow resistance at high temperature, whereas the rubbery midblocks are responsible for the elasticity, fatigue resistance and flexibility at low temperature (Airey, 2003; Ahmedzade *et al*., 2007). The specific gravity results for all binders are tabulated in Table 4-1 and all the results are within the standard range (1.0-1.04) (Brown *et al*., 1996).

It can also be seen from the comparison in Tables 4-4 to 4-7 that PM1_82 and PM1_76 the polymer modified bitumen have lower penetration and higher softening point and ductility values as compared to the PM2_82 and PM2_76 polymer modified bitumen. This is because PM1 consist of SBS polymer modifier, which gives high tensile strength and flow resistance at high temperature and also is responsible for the elasticity, fatigue resistance and flexibility at low temperature. It can also be noticed that PM1_82 is harder than PM1_76 as shown by their consistency tests result. This is because of the SBS polymer modified can work well and it can give more hardness or stiffness when it mixed with softer binder (Napiah, 1993).

Table 4-4: Comparison between PM1_82 and other types of polymer

Type of test	PM1_82	PM1_76		PM2_82		PM2_76	
		Obtained value	Decrease or increase from the base value	Obtained value	Decrease or increase from the base value	Obtained value	Decrease or increase from the base value
Penetration (mm)	39	47	(+20.5%)	48	(+23.1%)	48	(+23.1%)
Softening Point (^{O}C)	70	67	(- 4.3%)	65	(- 7.1%)	53	(- 24.3%)
Ductility (cm)	59	56	(- 5.1%)	35	(- 40.7%)	29	(- 50.8%)

Table 4-5: Comparison between PM1_76 and other types of polymer

Type of test	PM1_76	PM1_82		PM2_82		PM2_76	
		Obtained value	Decrease or increase from the base value	Obtained value	Decrease or increase from the base value	Obtained value	Decrease or increase from the base value
Penetration (mm)	47	39	(- 17 %)	48	(+ 2.1%)	48	(+ 2.1%)
Softening Point (^{O}C)	67	70	(+ 4.5%)	65	(- 3%)	53	(- 20.9%)
Ductility (cm)	56	59	(+ 5.4%)	35	(- 37.5%)	29	(- 48.2%)

Table 4-6: Comparison between PM2_82 and other types of polymer

Type of test	PM2_82	PM1_82		PM1_76		PM2_76	
		Obtained value	Decrease or increase from the base value	Obtained value	Decrease or increase from the base value	Obtained value	Decrease or increase from the base value
Penetration (mm)	48	39	(-18.8%)	47	(- 2.1%)	48	-
Softening Point (^{O}C)	65	70	(+7.7%)	67	(+3.1%)	53	(-18.5%)
Ductility (cm)	35	59	(+68.6%)	56	(+60%)	29	(-17.1%)

Table 4-7: Comparison between PM2_76 and other types of polymer

Type of test	PM2_76	PM1_82		PM1_76		PM2_82	
		Obtained value	Decrease or increase from the base value	Obtained value	Decrease or increase from the base value	Obtained value	Decrease or increase from the base value
Penetration (mm)	48	39	(-18.8%)	47	(-2.1%)	48	-
Softening Point (^{O}C)	53	70	(+32.1%)	67	(+26.4%)	65	(+22.6%)
Ductility (cm)	29	59	(+103.4%)	56	(+93.1%)	35	(+20.7%)

4.2.2 Aggregate Properties

The aggregates were characterized in terms of their physical, chemical and mechanical properties.

4.2.2.1 Physical properties

4.2.2.1.1 Sieve Analysis

The percentage of coarse aggregate, fine aggregate and filler were estimated after sieve analysis. The estimated values should be within the range given for the material (JKR, 1988). The percentage retained of each material, determined earlier from the sieve analysis test was multiplied with the estimated percentage of each material. The total aggregate was then determined by adding the percentage determined for each sieve size. Aggregates in the range from 28 mm to 5 mm were considered as coarse aggregate, 5mm to 150 µm were classified as fine aggregate and finally less than 75µm were categorized as filler. The total percentage passing was calculated from the total aggregate determined using a trial and error method. The curve of total percentage passing versus sieve size was plotted. The JKR standard for design curve was also plotted in the same graph. The results of the sieve analysis are presented in Figures 4-1 to 4-4. The graph shows that all the results indicated by the green line lie within the upper and lower range of the JKR standard (1988). Since the results met the standard criteria of JKR, these percentages of coarse, fine, and filler were used for mixture design.

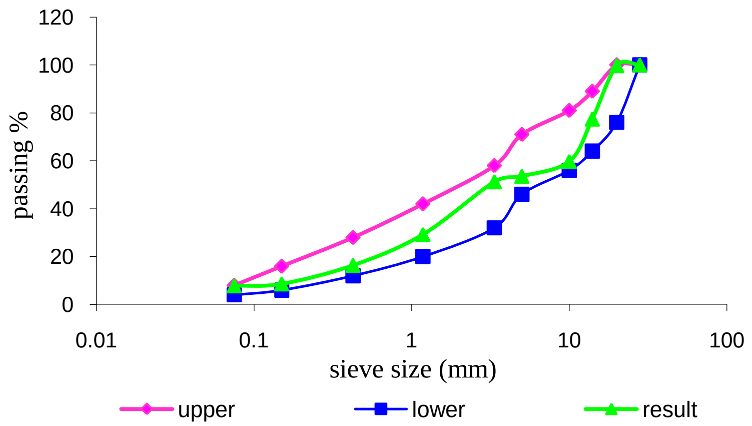


Figure 4-1: Gradation curve of the aggregate containing quarry sand

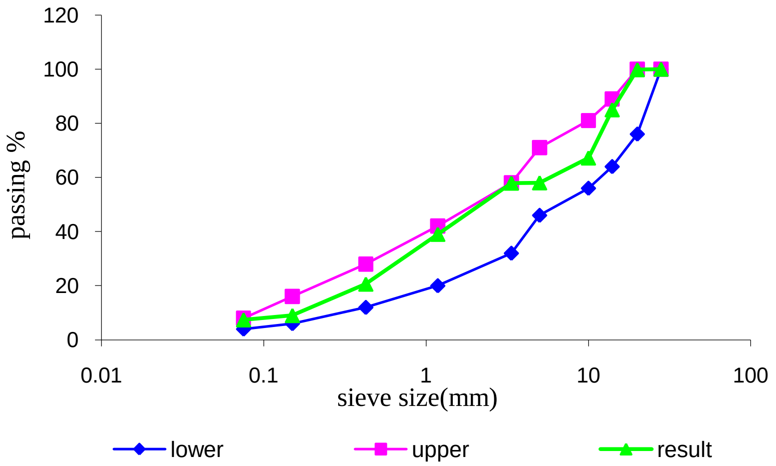


Figure 4-2: Gradation curve of the aggregate containing river sand

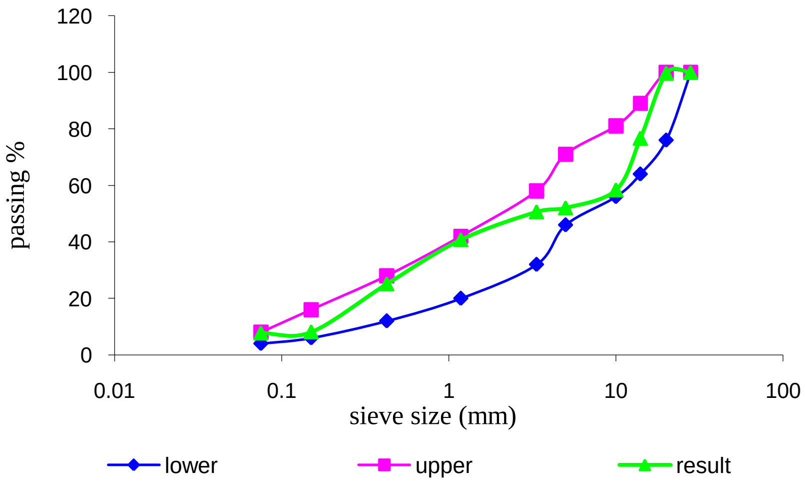


Figure 4-3: Gradation curve of the aggregate containing mining sand

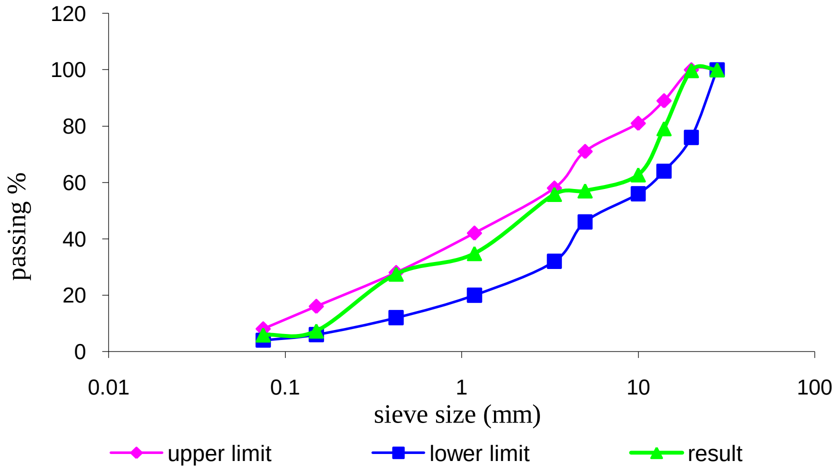


Figure 4-4: Gradation curve of the aggregate containing marine sand

4.2.2.1.2 Specific gravity

Three samples each were used to determine the specific gravity values. Table 4-8 shows the specific gravity results of the aggregate and filler. The results show that the four types of fine aggregate have a little different in their specific gravity values.

Table 4-8: Specific gravity of aggregate and filler used

Types of the aggregate	**Quarry/ Region**	**Specific gravity(g/cm^3)**
Coarse aggregate (Granite)	Quarry	2.655
Fine aggregate (River sand)	River	2.631
Fine aggregate (Mining sand)	Pond	2.695
Fine aggregate (Quarry sand)	Quarry	2.690
Fine aggregate (Marine sand)	Beach	2.710
Filler (Portland cement)	Factory	3.135

4.2.2.1.3 Fine aggregate angularity

Four different types of sands were selected and tested for the fine aggregate angularity (FAA) test. FAA is measured by determining the percentage of voids in the sand, the higher the percentage of voids the more angular the fine aggregate. The results of fine aggregate angularity is shown in Table 4-9, it can be noticed that quarry sand has more angularity compared with the other types of sand. This is followed by the river sand, mining sand and marine sand respectively. It was found by Fernandes and Gouveia, that fine aggregate with higher values of FAA produce more angular particles and greater rough surface texture, resulting in a larger interlock between the particles consequently resulting an a higher shear strength.

Table 4-9: Fine aggregate angularity (FAA) for the fine aggregate used

Types of the aggregate	**Quarry/ Region**	**Fine aggregate angularity (FAA) %**
Fine aggregate (Quarry sand)	Quarry	46.91
Fine aggregate (River sand)	River	43.05
Fine aggregate (Mining sand)	Pond	42.58
Fine aggregate (Marine sand)	Beach	40.32

4.2.2.1.4 Physical appearance of fine aggregate

The physical properties of fine aggregate which include particle shape, size and surface texture have been found to affect the properties and performance of bituminous paving mixtures (Topal and Sengoz, 2005, 2006). Park and Lee (2002) found that angular shape and rougher surface texture of fine aggregate are important factors that provide resistance to permanent deformation. Angular shape and rougher surface texture of fine aggregate also have been found to improve the mechanical properties of aggregate (shear strength) (Das, 1998). The shape, surface texture and particle size distribution can be determined by the scanning electron microscopy (SEM) (Xue *et al*., 2006). Visual comparative analyses can also be used for the analysis of fine aggregate shape, angularity and texture (Fernandes and Gouveia).

The physical appearance of quarry sand, river sand, mining sand, and marine sand at 14 times and 1500 times magnifications are shown in Figure 4-5 to Figure 4-12. The results indicate that quarry sand seems or looks angular in shape and has bigger particles size compared to the other types of sand. The angular shape will affect the particles interlocking property, the higher the angularity the better interlocking property. River sand and mining sand exhibited lower angularity in shape as shown in Figures 4-6 and 4-7. On the other hand, marine sand is more rounded in shape as shown in Figure 4-8. The rounded particles and smooth texture of marine sand is due to transfer and agitation of water as the sand particles was carried into the sea. The more angular and the rougher surface textured aggregate in the mix, the more rut resistance it can offer than mixtures with rounded and smooth textured aggregate as cited by Park and Lee (2002).

From the morphology, it is observed that quarry sand shows a different texture from the natural sand (mining sand, river sand and marine sand). The surface texture of quarry sand is rougher than the river sand and mining sand, whereas the surface texture of marine sand looks like smooth as shown in Figure 4-12. Surface roughness is a factor that affects the adhesion ability of sand with binder. Park and Lee (2002) found that rougher surface texture is an important property in offering rutting resistance.

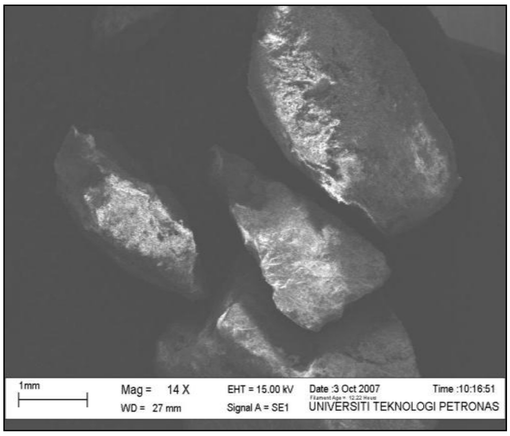


Figure 4-5: Physical Appearance of Quarry sand (Shape) (Magnified 14x)

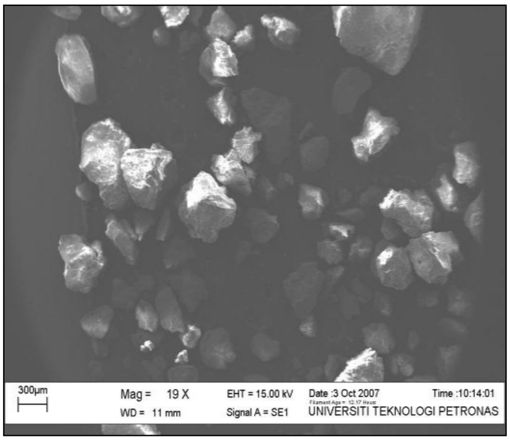


Figure 4-7: Physical Appearance of Mining sand (Shape) (Magnified 19x)

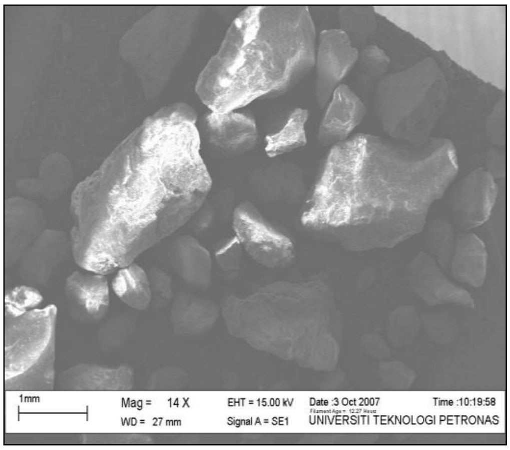


Figure 4-6: Physical Appearance of River sand (Shape) (Magnified 14x)

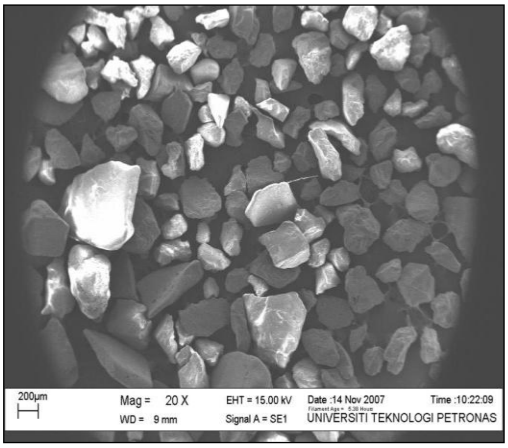


Figure 4-8: Physical Appearance of Marine sand (Shape) (Magnified 20x)

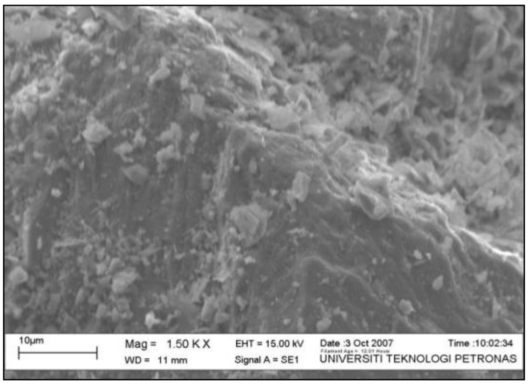


Figure 4-9: Physical Appearance of Quarry sand (Surface Texture) (Magnified1500x)

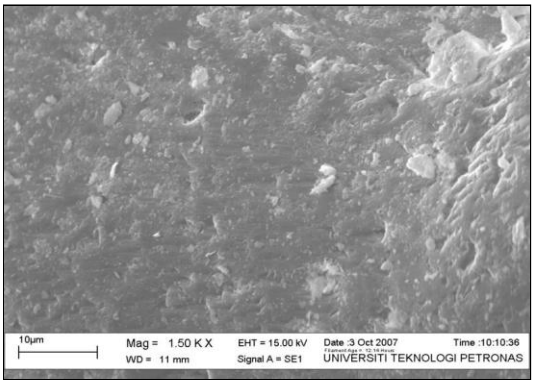


Figure 4-11: Physical Appearance of Mining sand (Surface Texture) (Magnified1500x)

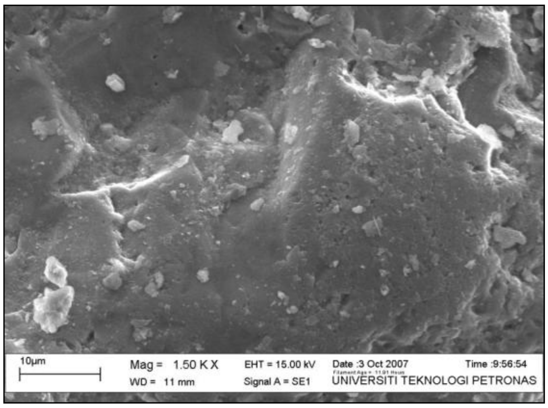


Figure 4-10: Physical Appearance of River sand (Surface Texture) (Magnified1500x)

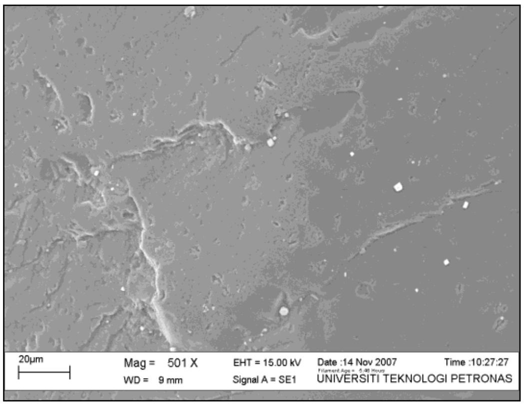


Figure 4-12: Physical Appearance of Marinesand (Surface Texture) (Magnified 501)

The results also show that crushed aggregate such as quarry sand is more cubical and angular than natural fine aggregates. However, there are also some sub angular natural fine aggregates such as river sand.

4.2.2.2 Chemical properties of fine aggregate

The chemical composition of aggregate has an effect on the properties and performance of bituminous mixtures (Abo Qudais and Al Shweily, 2007). The composition of sand is highly variable depending on the rock sources and conditions. Table 4-10 shows the chemical composition for quarry sand, river sand, mining sand, and marine sand obtained from the X-ray fluorescence (XRF) test. The composition contains of CaO, SiO_2, Al_2O_3, Fe_2O_3, K_2O and Na_2O in different proportions. SiO_2 constitute represent the highest chemical composition in all the sand types. The table also shows that other elements are also present in varying degree of chemical composition.

Each compound that make-up the sand has different hardness values as tabulated in Table 4-11 (Wypych, 1999). From the table it can be seen that Alumina (Al_2O_3) has the highest hardness value, followed by silica (SiO_2) and hematite (Fe_2O_3). The hard constituents tend to increase the hardness of bituminous mixtures, hence increasing the stiffness of the mixtures. From the data in Table 4-11 also, it can be observed that hematite (Fe_2O_3) has the smallest particle followed by alumina (Al_2O_3) and silica (SiO_2). Earlier studies have established that size of particles also has some effect on the performance of paving mixtures because smaller particles tend to decrease the void within the bituminous mixture, which consequently will increase the density and this will increase the resistance of bituminous mixture to permanent deformation.

Table 4-10: Chemical composition results for fine aggregate

Fine aggregate type	**CaO**	**SiO_2**	**Al_2O_3**	**Fe_2O_3**	**K_2O**	**Na_2O**
Quarry sand	7.19 %	66.1 %	12.6 %	5.30 %	10.5 %	1.95 %
River sand	0.4 %	76.8 %	8.6 %	2.7 %	9.3 %	0.5 %
Mining sand	3.6 %	80.0 %	8.98 %	2.2 %	2.2 %	0
Marine sand	33.5 %	57.5 %	1.67 %	3.20 %	1.71 %	0.31 %

Table 4-11: Characteristics of Compounds (Wypych, 1999)

Components	**Solubility in Water**	**Hardness (Mohr)**	**Particle Size (µm)**	**Oil Absorption (g/100g)**
CaO	Hydrophilic	-	-	-
SiO_2	Hydrophobic	7	2-19	17-20
Al_2O_3	Hydrophobic	9	0.8-10	25-225
Fe_2O_3	Hydrophobic	3.8-5.1	0.013-0.105	10-35
K_2O	Hydrophilic	-	-	-

Based on the results of fine aggregate chemical composition shown in Table 4-10, it can be noticed that the most common constituent of sand is silica (silicon dioxide, or SiO_2), usually in the form of quartz, which is known for its hardness. However the amount of silica affects pavement performance. A large amount of SiO_2 can cause stripping of HMA pavements because silica reduces the bond strength between the aggregate and binder (Abo Qudais and Al Shweily, 2007).

The XRF test results show that mining sand has the highest percentage of SiO_2 (80%), followed by river sand (76.8%), quarry sand (66.1%), and marine sand (57.5%). Other dominant compounds of quarry sand are Al_2O_3, K_2O, CaO, Fe_2O_3, and Na_2O, of river sand are K_2O, Al_2O_3, Fe_2O_3, Na_2O and CaO, of mining sand are Al_2O_3, CaO, Fe_2O_3, and K_2O, and of marine sand are CaO, Fe_2O_3, K_2O, Al_2O_3, and Na_2O.

Quarry sand has the highest total percentage of alumina (Al_2O_3) that is 12.6% compared with other types of sand, followed by mining sand which containing 8.98%, river sand containing 8.6% and marine sand containing 1.67% of the Al_2O_3 . The Al_2O_3 content is related to the hardness of the material, this can be seen from the Table 4-11.

Quarry sand also has the highest total percentage of hematite (Fe_2O_3) that is 5.30% compared with other types of sand, followed by marine sand which containing 3.20%, river sand containing 2.7% and mining sand containing 2.2% of the hematite. The Fe_2O_3 content which has the smallest particle size compared to the other elements as shown in the Table 4-11 is related to the density of the bituminous mixture.

The oil absorption property is needed to absorb the extensive oils in the bituminous mixture, this property could decrease the rutting behavior (Wu *et al.*, 2007). Al_2O_3 also has the highest oils absorption value (25-225 g/100g) followed by Fe_2O_3 and SiO_2 respectively. This indicates that quarry sand showed the highest ability to absorb the extensive oils in bituminous mixture, because it has the highest Al_2O_3, whilst marine sand exhibited the lowest ability to absorb the extensive oils in bituminous mixture.

4.2.2.3 Mechanical properties of fine aggregate

The angle of internal friction (Ø) is an indication of particle interlocking and hence particle shape and surface texture (Park and Lee, 2002). The direct shear test results in Figure 4-13 showed that quarry sand has the highest Ø value (45^0) followed by river sand (37.8^0), mining sand (34.9^0) and marine sand (33.7^0) respectively.

It was cited by Park and Lee (2002) that rounded smooth textured aggregate particles tend to slide past one another producing a HMA mixture with relatively low shear strength. These results explain why crushed aggregate like quarry sand has higher shear resistance compared to natural aggregate samples (river sand, mining sand and marine sand). Because natural sand have more rounded particles with smooth surface textured resulting in less interlock between particles (Fernandes and Gouveia). The high shear resistance is an indicator of resistance to mixture deformation (Park and Lee, 2002; Topal and Sengoz, 2006). It was confirmed by the studies carried out by Fernandes and Gouveia that the higher internal friction angle of fine aggregate indicates better interlocking mechanism results in a more resistant granular structure.

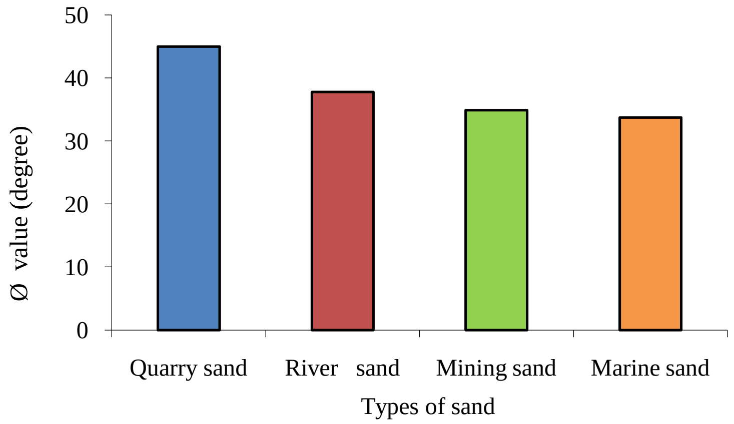


Figure 4-13: Ø values for different types of sand.

4.3 Mixture Properties (Marshall Test results)

The effect of physical characteristics of polymer modified bitumen, and physical, chemical and mechanical properties of sand on the properties of bituminous mixture are discussed in this section. The percentage of bitumen used in the mixtures ranges from 3.5% up to 7% by weight of the total mix. The mixture properties i.e. density, voids in mineral aggregate (VMA), voids filled with bitumen (VFB), air voids (AV), stability, flow and stiffness were analyzed at optimum bitumen content. The requirements for asphaltic concrete mixture properties as specified by Jabatan Kerja Raya (JKR, 1988) Malaysia standard are shown in Table 4-12.

Table 4-12: Asphaltic Concrete Mixture Requirements (JKR, 1988)

Parameter	**Wearing Course**
Stability (S)	> 8000 N
Flow (F)	2.0-4.0 mm
Stiffness (S/F)	> 2000 N/mm
Air void in mix	3-5 %
Void in aggregate filled with bitumen	70-80 %

However, the JKR standard does not provide the voids in mineral aggregate (VMA) requirements. Thus, the requirement of VMA in this study is referred to the Asphalt Institute design criteria (1990) as shown in Table 4-13, which is subjected to 5% air voids.

Table 4-13: Asphalt Institute Design Criteria for VMA subjected to 5% Air Voids

Maximum Size of Aggregate (mm)	Minimum Voids in Mineral Aggregate (%)
25	13
19	14
12.5	15
9.5	16
4.75	18
2.36	21
1.18	23.5

4.3.1 Optimum Binder Content

High bitumen content may lead to high permanent deformation, because the loads are carried by the asphalt cement rather than the aggregate structure. The high ratio of the asphalt cement in the mixture causes the loss of internal friction between aggregate particles, caused the higher plastic flow susceptibility (Tayfur *et al.*, 2007). By considering the above, the engineering properties can be optimized according to the desired behavior in service by determining the optimum bitumen content. Marshall Parameters namely stability, density, VMA, and air voids are used to determine the optimum bitumen content (OBC) in this study. Figures 4-14 to 4-17 showed the way to calculate the OBC for quarry sand mixture with bitumen pen 80/100. The OBC was obtained by calculate the average bitumen content that meets maximum stability, maximum density, minimum VMA and bitumen content that meets 5% AV as shown by Equation 4-1.

$$\text{OBC \%} = \frac{4.4 + 5.5 + 4.5 + 4.8}{4} = 4.80\% \qquad \text{4-1}$$

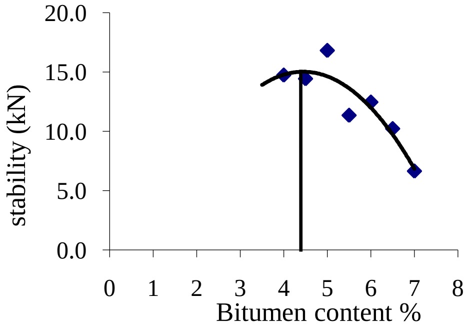


Figure 4- 14: Stability of quarry sand mixture with bitumen pen 80/100

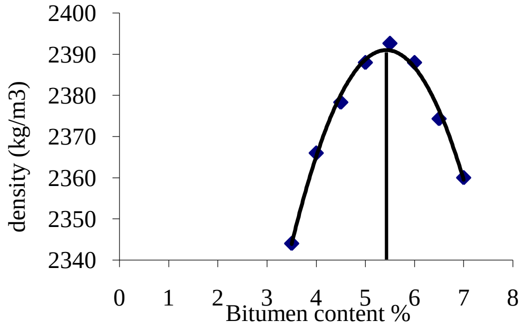


Figure 4- 15: Density of quarry sand mixture with bitumen pen 80/100

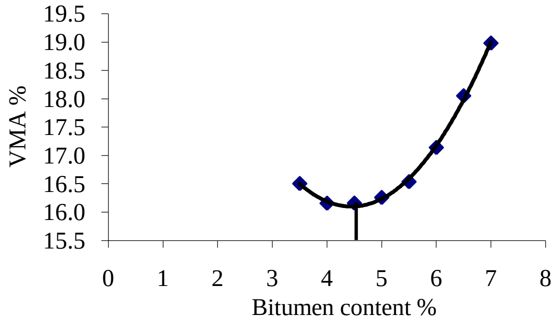


Figure 4- 16: VMA % of quarry sand mixture with bitumen pen 80/100

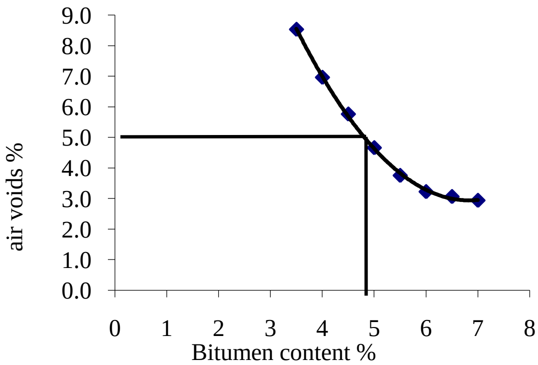


Figure 4- 17: AV % of quarry sand mixture with bitumen pen 80/100

All the OBC results are summarized in Table 4-14 and are also presented in the form of bar chart as shown in Figure 4-18. It can be noted that all the polymer modified bituminous mixtures have a higher OBC content than the conventional bituminous mixtures, because polymer modified has the higher stability, higher VMA and higher AV which consequently increase the OBC. This agrees with an earlier study made by Tayfur *et al.* (2007), except for quarry sand mixtures where the results do not have a consistent trend. Therefore quarry sand with PM2_76 showed higher OBC than PEN 50/60 and with PEN 80/100 showed higher OBC content than polymer modified mixtures.

PM1_76 and PM2_82 mixtures with marine sand have lower OBC content than bitumen PEN 50/60 mixtures. Bitumen PEN 50/60 mixtures have lower OBC content than bitumen PEN 80/100 mixtures, except in marine sand mixtures where the OBC content is higher than bitumen PEN 80/100 mixtures. This is because marine sand has finer particles which means higher surface area, there for it needs more amount of binder to cover all the particles.

The amount of design binder contents for hot rolled asphalt (HRA) mixtures containing crushed rock fine aggregate therefore reflect the fact that the overall aggregate grading is much coarser and more continuous than is the case for mixture containing natural sand fine (Choyce). As a result, binder contents are much lower for quarry sand incorporated bituminous mixtures than those obtained for mixtures containing natural sand like marine sand, river sand, and mining sand. This result is attributed to the shape, size, particles distribution and surface area of fine aggregate. These parameters have been shown to influence the density, stability, VMA and VFB results.

Finally when the OBC is compared to the four types of the sand, it can be observed that OBC is higher in the mixture having mining sand, followed by river sand, marine sand and quarry sand respectively, because mining sand has more fines (small particles). This means large or more surface area which require for more amount of binder to cover all the particles. The quarry sand has large or big particles, meaning the surface area is smaller thus a smaller amount of binder is required to cover the particles.

Table 4-14: OBC (%) for all types of binders & sands

Type of binder	Quarry sand (%)	River sand (%)	Mining sand (%)	Marine sand (%)
PM1_82	4.19	5.40	6.33	5.04
PM1_76	4.53	5.45	6.06	5.03
PM2_82	4.48	5.31	6.40	4.90
PM2_76	4.69	5.33	6.61	5.09
PEN 50/60	4.59	5.20	5.86	5.04
PEN 80/100	4.80	5.30	6.14	4.90

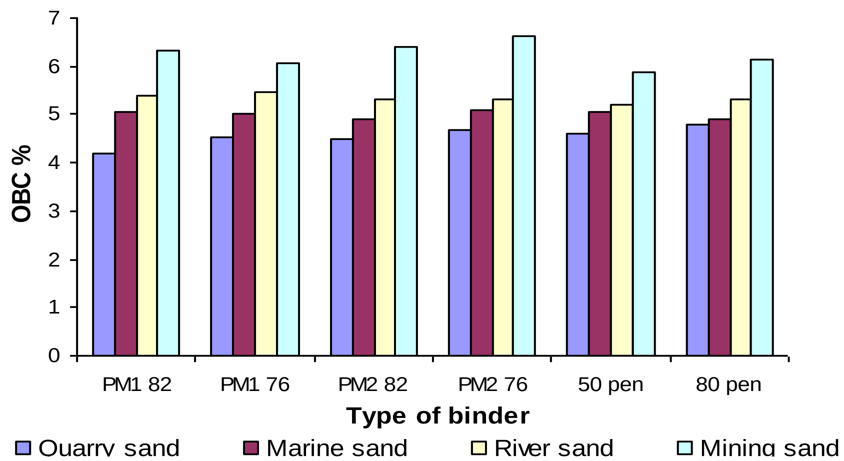


Figure 4-18: OBC (%) for all types of binders & sands

4.3.2 Density

The density of the asphaltic mixtures is presented in Figures 4-19 to 4-22. In all these figures, density is observed to increase with increasing binder content until a maximum value is reached, beyond that it starts to decrease at higher bitumen content. However, in mining sand mixture with PM1_82 and PM2_76 the density still increases at 7% bitumen content. The density at optimum bitumen contain for all mixture variations is summarized in Table 4-15. The results show that the density of the mixtures ranged from 2.307 to 2.394 g/cm^3.

The maximum density of the bituminous mixture with fine aggregate was observed to increase with rougher, angular fine aggregates particles and with the amount of hematite (Fe_2O_3) in the fine aggregate also, whereas with different types of binder no significant change in the density values was observed, this means when polymer modified bitumen was used the density does not follow consistent trend. The fact that polymer modified bituminous mixture sometime has a lower density compared with conventional mixture was also as certained in an earlier study by Awwad and Shbeeb (2007). At other times, the opposite results were observed. The density for conventional mixture bitumen PEN 50/60 is higher than bitumen PEN 80/100 mixtures in all types of the sand, this result agree with consistency results of the bitumen as shown in Table 4-1.

Angular and rougher fine aggregate influenced the demand for binder amount to reach maximum density. For example, quarry sand need less amount of binder (less than 5.5%) to reach the maximum density, but mining sand need higher amount of binder (more than 6%) to reach the maximum density, while river and marine sand lies in between. Density of mixture with different types of sand is shown in Table 4-15, it can be observed that quarry sand mixture has the highest density, followed by marine sand, river sand and mining sand mixtures respectively. High density is related to the roughness of the surface texture which provides a greater bonding strength with asphalt cement and frictional resistance between particles due to better mechanical interlock (Abo Qudais and Al Shweily, 2007). High density is also related to the smaller particles size of the hematite content as shown by the XRF results quarry sand has the highest hematite content followed by marine sand, river sand and mining sand respectively. Therefore the hematite has significant effect on the density and is play a big role in improving the density.

The rougher surface texture of the quarry sand can also be observed from the FAA results and from the scanning electron microscopy as shown in Figure 4-9. For the marine sand, smaller particles tend to fill the voids in the bituminous mixture, also the smooth rounded particles provides better workability during compaction, which enables for maximum compaction (Choyce; Topal and Sengoz, 2006). Thus, density is increased. But some researches show that easily compactable asphaltic mixtures can rut easily and quickly under traffic loads (Topal and Sengoz, 2005).

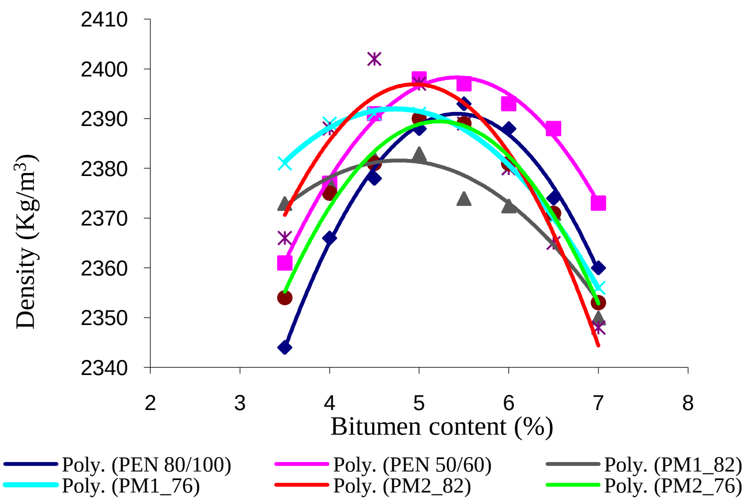


Figure 4-19: Density of bituminous mixtures containing quarry sand

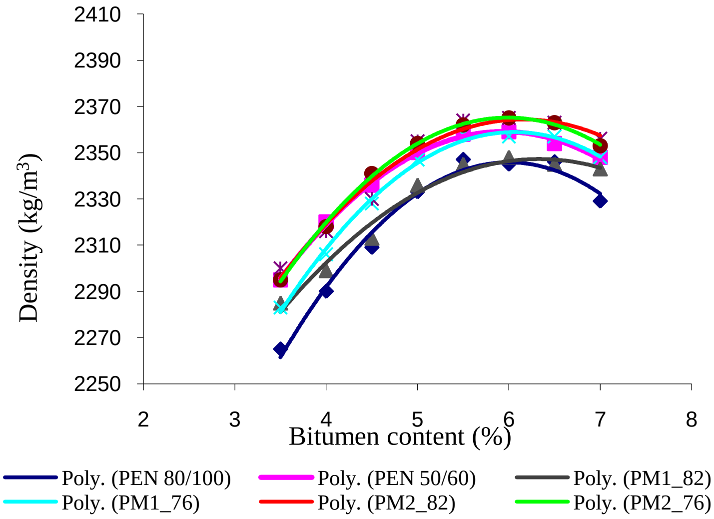


Figure 4-20: Density of bituminous mixtures containing river sand

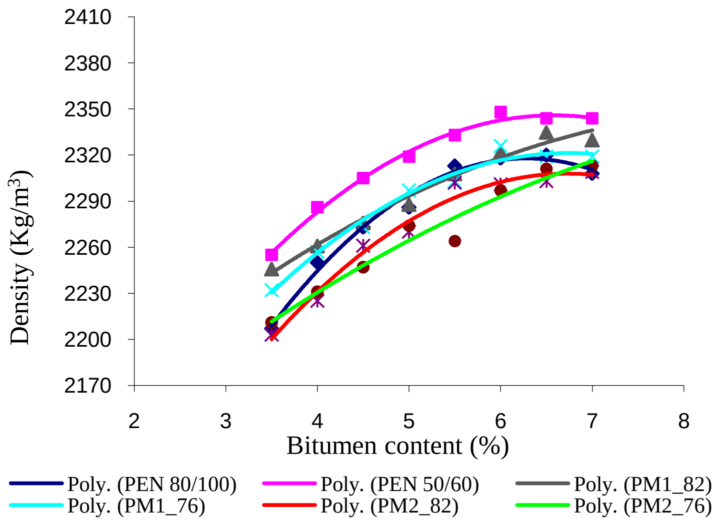


Figure 4-21: Density of bituminous mixtures containing mining sand

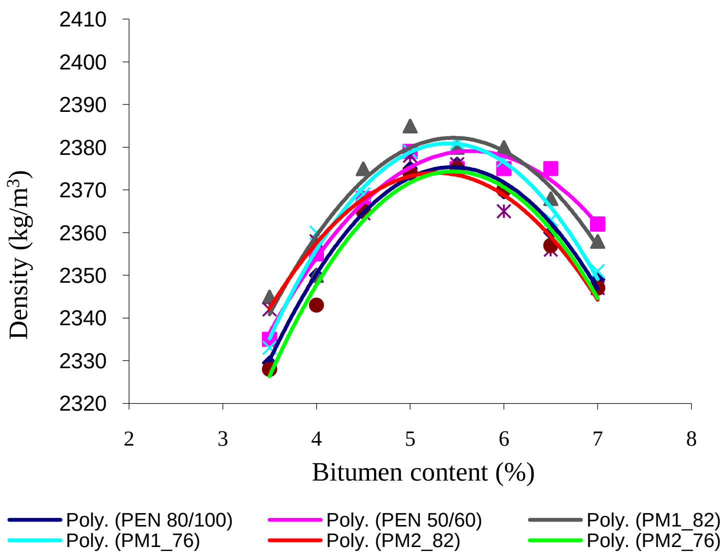


Figure 4-22: Density of bituminous mixtures containing marine sand

Table 4-15: Density (kg/m^3) at OBC for all types of binders & sands

Type of binder	Quarry sand	River sand	Mining sand	Marine sand
PM1_82	2386	2340	2324	2380
PM1_76	2391	2355	2318	2379
PM2_82	2380	2358	2307	2372
PM2_76	2391	2360	2308	2372
PEN 50/60	2394	2354	2342	2376
PEN 80/100	2386	2339	2317	2372

4.3.3 Voids in Mineral Aggregate

Voids in mineral aggregate (VMA) is the percentage of voids volume to the total volume of mix. It is important to provide sufficient space between aggregate that can be filled by bitumen in order to obtain maximum strength of the design mixture. According to the Asphalt Institute Manual (MS-2) (1990) standard, a minimum 13% of VMA is required for the ACW20 mix design that has 25 mm of maximum aggregate size. VMA results for all types of mixture are shown in Figures 4-23 to 4-26.

All figures show decreasing VMA with increasing bitumen content until minimum VMA value is reached, after that it continues to increase with increasing bitumen content. The minimum VMA at optimum bitumen content for all mixtures is presented in Table 4-16. It is observed that the VMA is slightly increased for polymer modified bitumen mixtures compared with conventional bituminous mixtures, this being in agreement with the study done by Awwad and Shbeeb (2007) and Hamid *et al*., (2008). However quarry sand with PM1 and PM2 have lower VMA than mixture containing bitumen PEN 80/100, in other cases, it exhibited higher VMA than mixture containing bitumen PEN 50/60 as observed in PM1_76 and PM2_76.

Table 4-16 showed that river sand mixtures with PM1_76 and PM2_76 and PM2_82 have less VMA than bitumen PEN 80/100 and PM1_76 and PM1_82 have more VMA than PEN 50/60. PM2_72 and PM2_82 have less VMA compared with bitumen PEN 50/60. For mining sand mixtures PM1_82 has less VMA than bitumen PEN 80/100 but PM1_76 and PM2 has more VMA than bitumen PEN 50/60 and 80/100. For marine sand mixtures PM1 has less VMA than bitumen PEN 50/60 and 80/100 however PM2 has more VMA than bitumen PEN 50/60 and less than PEN 80/100 for PM2_82. In general, it can be concluded that PMB affects VMA because polymer modified bitumen has higher viscosity than conventional bitumen, making it difficult to flow easily into the aggregate voids (Garcıa-Morales *et al*., 2004; Sirin *et al*., 2006; Navarro *et al*., 2007).

From Table 4-16 it can be noticed that among the different fine aggregates, quarry sand bituminous mixtures have lowest VMA compared with other fine aggregates. This is because the rough or porous surface allows the binder to pass through it to fill the voids inside the quarry particles which provides a greater bonding strength with asphalt cement and relatively the voids in it will be decreased and the density will be increased. River sand with PM2_82, PM2_76, and PEN 50/60 have lower VMA than marine and mining sand, however river sand with PM1 and bitumen PEN 80/100 is presents more VMA than marine sand. Marine sand has lower VMA than mining sand because the higher proportion of fine material is found to decrease the VMA in the mixture. The results show that in general, the bitumen content required to obtain minimum VMA decreases with increased the angular and rough of sand. Quarry sand bituminous mixtures indicate the lowest required bitumen content followed by river sand, marine sand, and mining sand. Angular and rougher particles are responsible for the reduction of required bitumen content. In general, a mixture with lower VMA exhibits less binder.

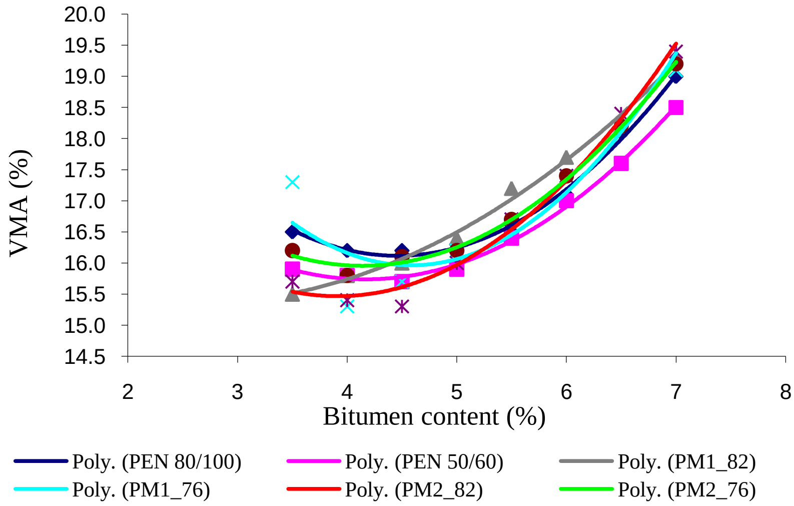


Figure 4-23: VMA of bituminous mixtures containing quarry sand

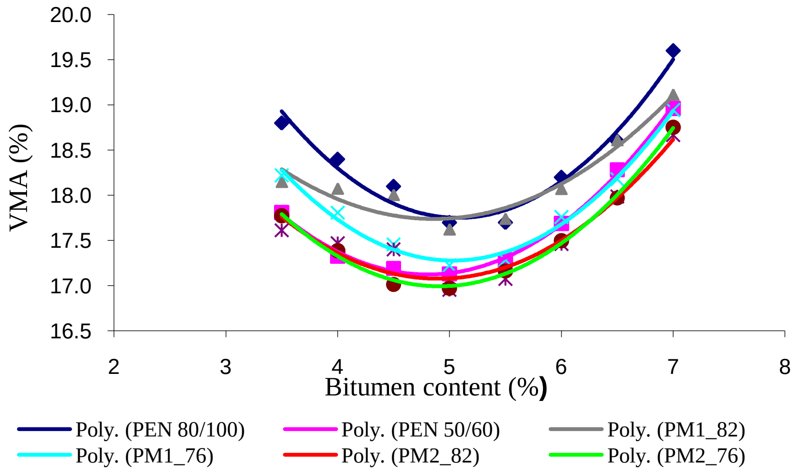


Figure 4-24: VMA of bituminous mixtures containing river sand

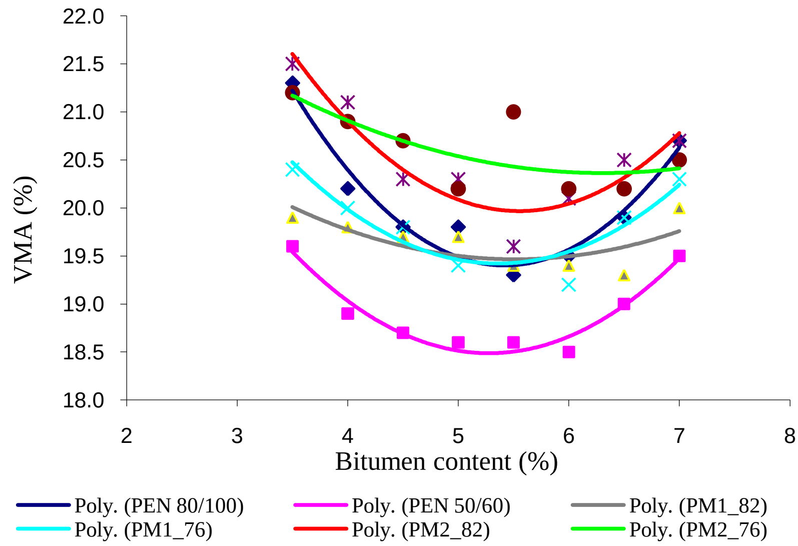


Figure 4-25: VMA of bituminous mixtures containing mining sand

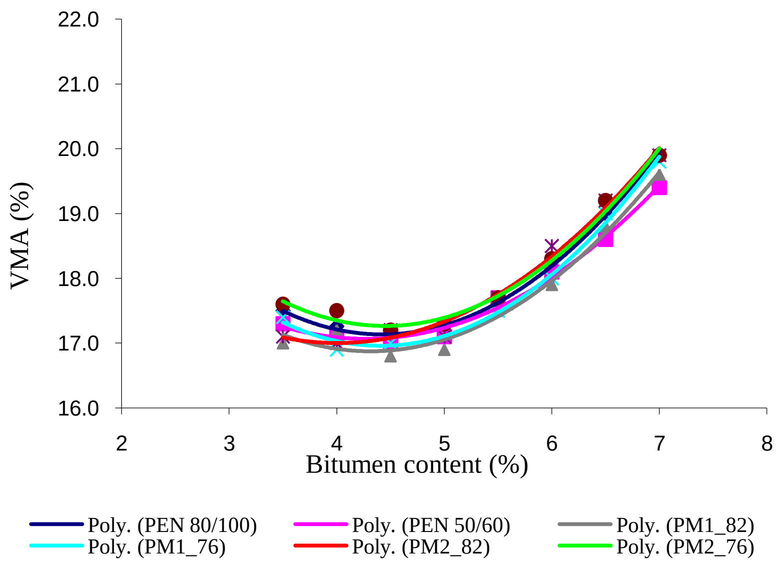


Figure 4-26: VMA of bituminous mixtures containing marine sand

Table 4-16: VMA (%) at OBC for all types of binders & sands

Type of binder	Quarry sand	River sand	Mining sand	Marine sand
PM1_82	15.9	17. 83	19. 54	17.05
PM1_76	16.0	17.35	19.60	17.10
PM2_82	15.1	17.13	20.24	17.25
PM2_76	16.0	17.06	20.38	17.40
PEN 50/60	15.8	17.18	18.61	17.22
PEN 80/100	16.2	17. 76	19. 56	17.28

4.3.4 Voids Filled With Bitumen

The percentage of voids filled with binder is called void filled with bitumen (VFB). In the JKR standard the requirement for void filled with bitumen is in the range of 70-80%. The VFB results for all mixtures are presented in Figures 4-27 to 4-30. It can be observed that VFB increases with increasing bitumen content for all mixtures.

In order to fulfill the requirement for VFB, different bitumen content is required for each mixture. In quarry sand mixture the required bitumen was 5%, while river and marine sand mixture have almost the same bitumen requirement (5%-5.5%), but for mining sand mixture the bitumen requirement is 6%, this due to the fact that mining sand has more fines (small particle) as shown in scanning electron microscopy. This means large or more surface area needs more amount of binder to coat all the particles. The quarry sand has large particles, which mean small surface area need less amount of binder to coat the particles. The results in Table 4-17 are based on VFB at OBC, from where it can be observed that in all types of sand the mixtures with bitumen PEN 50/60 have the higher VFB than mixtures with bitumen PEN 80/100, except for quarry sand mixture with bitumen PEN 50/60 which shows lower VFB than the quarry sand mixture with bitumen PEN 80/100. Quarry sand polymer modified bitumen mixtures have lower VFB compared to conventional mixtures, however different behaviors are observed in other types of sand where sometimes the VFB decreases and sometimes it increases. Generally among the four types of fine aggregate, quarry sand have the lowest VFB, followed by marine sand, river sand, and mining sand respectively. This is referring to the surface area of the fine aggregate particle, small surface area generally reduces the bitumen requirement for the mixture at the same percentage of VFB.

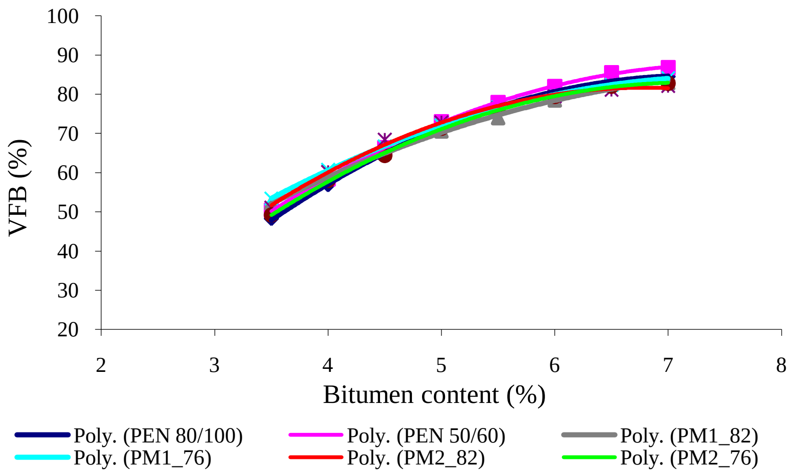


Figure 4-27: VFB results of bituminous mixtures containing quarry sand

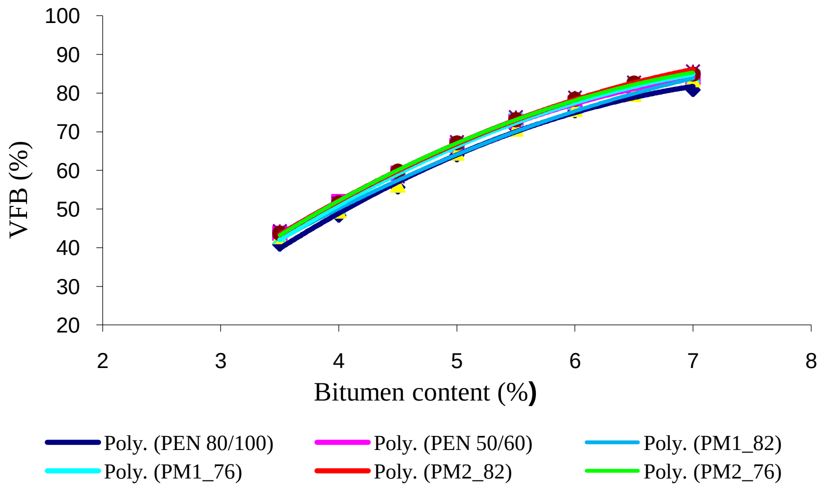


Figure 4-28: VFB results of bituminous mixtures containing river sand

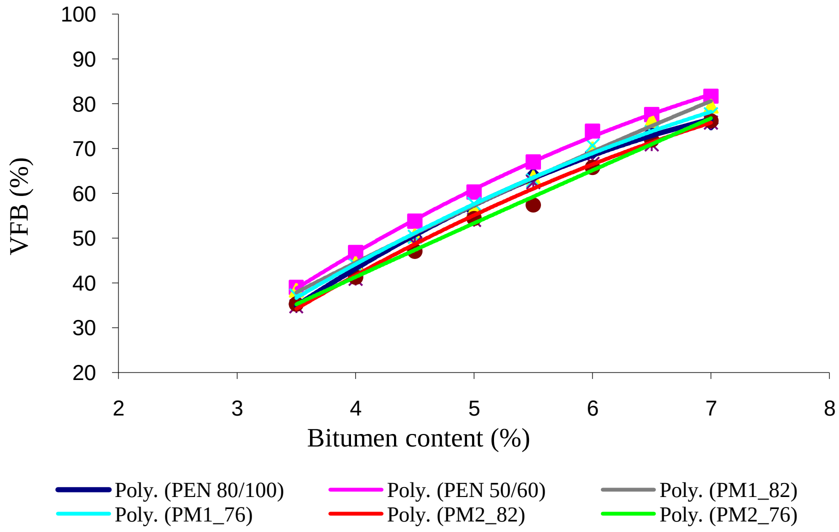


Figure 4-29: VFB results of bituminous mixtures containing mining sand

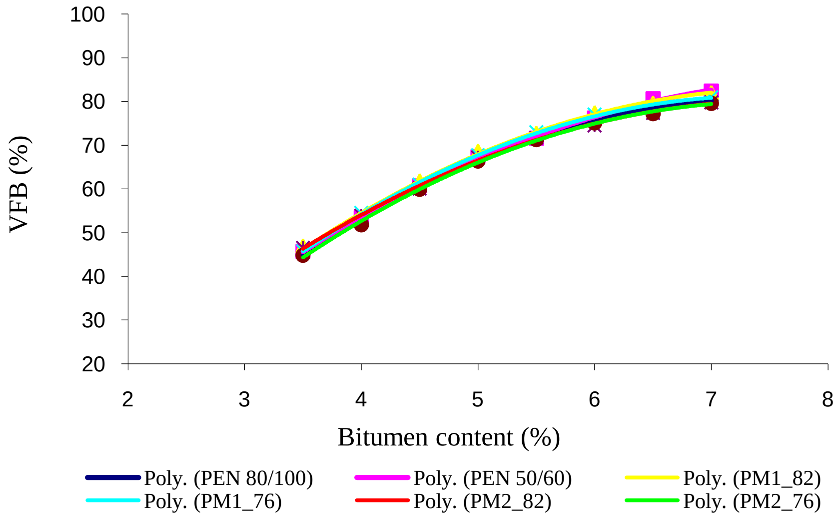


Figure 4-30: VFB results of bituminous mixtures containing marine sand

Table 4-17: VFB (%) at OBC for all types of binders & sands

Type of binder	**Quarry sand**	**River sand**	**Mining sand**	**Marine sand**
PM1_82	61. 5	69	73	68
PM1_76	67	72	70	67.7
PM2_82	67	71	70.5	65.8
PM2_76	67.5	71	72	67.0
PEN 50/60	68	69	71.5	67
PEN 80/100	69	68	68. 5	66

4.3.5 Air Voids

Air voids (AV) is the percentage of air volume to the total volume of compacted asphalt mixtures. The performance of asphalt mixtures decreases due to high air voids content. The required air voids as specified by JKR standard (adjusted) is 3% up to 5%. The air voids in all the design mixtures which were used in the OBC determination was 5%, because minimum air voids in most of the mixture types is 5%.

The air void results for all mixtures are shown in Figures 4-31 to 4-34. As can be observed from these graphs, the air voids decrease with increasing bitumen content. Bitumen PEN 80/100 mixtures were observed to have lower air voids than bitumen PEN 50/60 mixtures when it mixed with quarry sand. However, the opposite results were observed with river sand, mining sand and marine sand mixtures. It’s also been observed that air voids some time increase and some time decrease when polymer modified bitumen is used. It can be concluded from the results that polymer modified bitumen can slightly increase the percentage of air voids in the bituminous mixture and this agrees with studies made by Awwad and Shbeeb (2007) and Hamid *et al*., (2008). This is because polymer modified bitumen have higher viscosity, which significantly reduces the workability of the mixture and thus more difficult to compact to reduce the air voids content in the mixture.

River sand mixtures have lower air voids than quarry sand, mining sand, and marine sand when mixed with polymer modified bitumen. Quarry sand mixture has lower air voids than mining and marine sand except in PM1_82 where it has higher air voids than river sand, mining sand and marine sand mixtures. Mining sand and marine sand mixtures have varying air voids as shown by the results in Table 4-18. It can be concluded that there is no specific trend can be observed in these mixtures, sometimes the air voids decrease in mining sand and sometimes they increase when compared to the marine sand. The distribution of particles size and the particle shape affect the gradation, because the angular particles are locked together to a greater degree and are tightly bound together due to the interlocking effects of the smaller size particles with large size particles. The air voids are observed not only to depend on sand type but also on the binder type. Rounded particles of sand incorporated with harder bitumen PEN 50/60 produce lower air voids while the angular particles of sand needed softer bitumen to produce lower air voids. From the air void result for different mixtures it observed that most of mixtures have air voids more than 5%, this is because the minimum air void of mixtures is 5% and when the other criteria are includes it reduce the binder content in the mixture and this will give a higher air void.

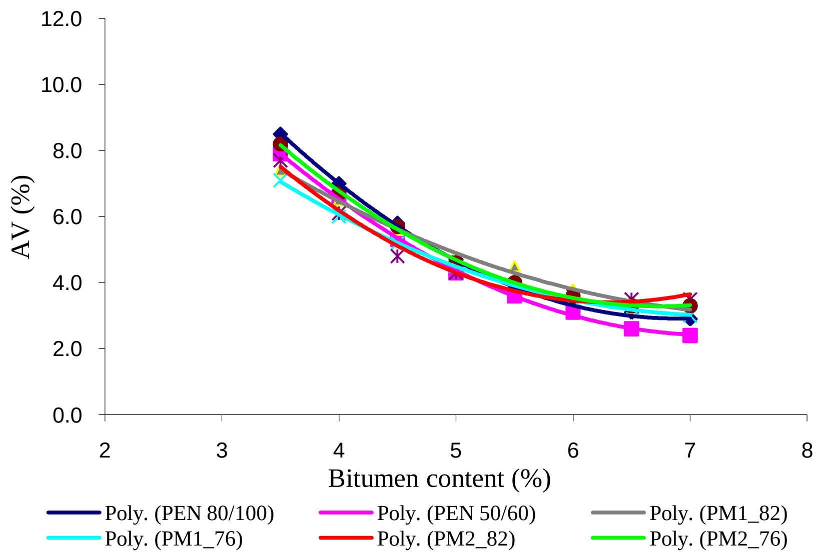


Figure 4-31: Air voids of bituminous mixtures containing quarry sand

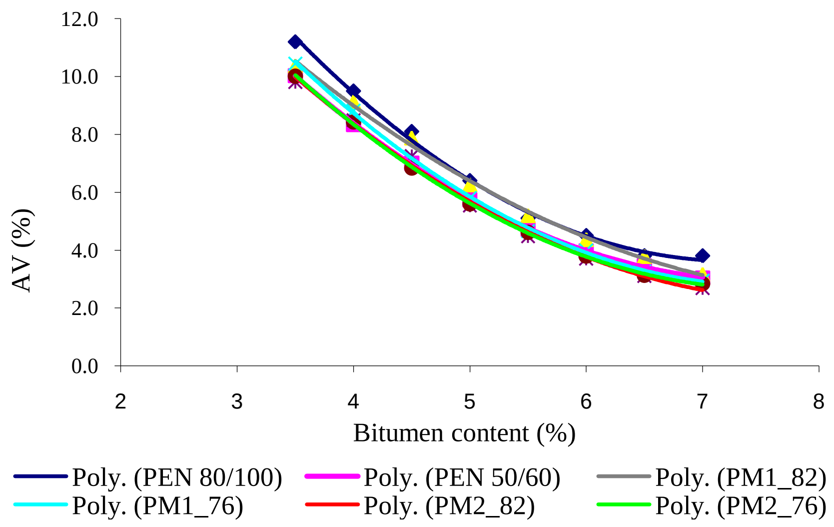


Figure 4-32: Air voids of bituminous mixtures containing river sand

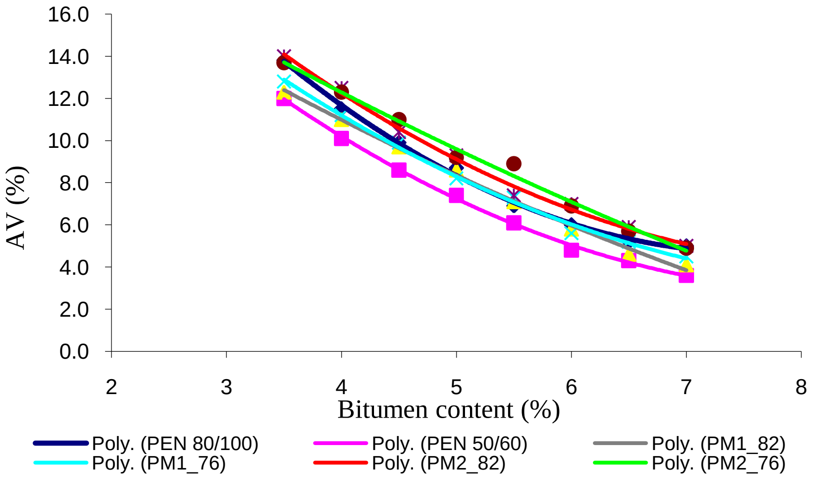


Figure 4-33: Air voids of bituminous mixtures containing mining sand

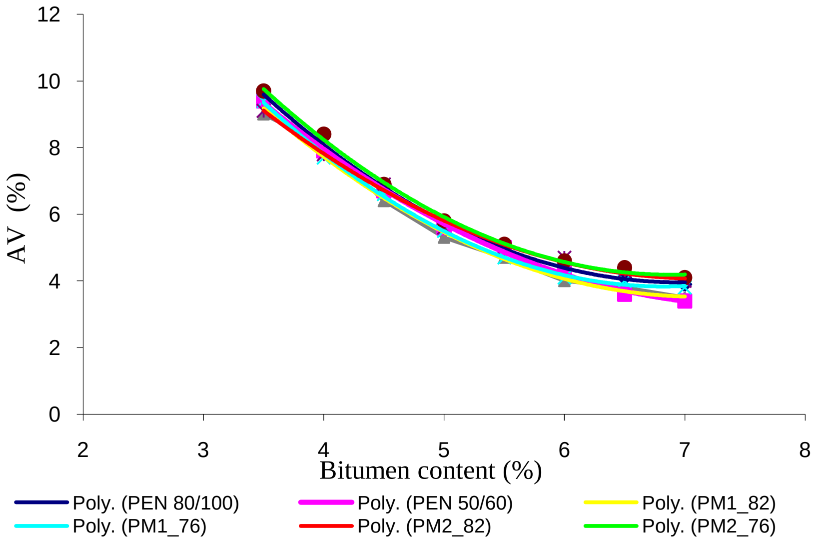


Figure 4-34: Air voids of bituminous mixtures containing marine sand

Table 4-18: AV (%) at OBC for all types of binders & sands

Type of binder	**Quarry sand (%)**	**River sand (%)**	**Mining sand (%)**	**Marine sand (%)**
PM1_82	6. 1	5. 45	5. 3	5.5
PM1_76	5.2	4.85	5.85	5.5
PM2_82	5.15	5.00	6	6
PM2_76	5.2	4.95	5.7	5.7
PEN 50/60	5.1	5.30	5.25	5.7
PEN 80/100	5	5. 75	6. 1	6

4.3.6 Stability

The maximum load that a specimen can withstand at a loading rate of 50.8 mm/minute is used to determine the stability value. The stability parameter is used for measuring the strength of compacted mixtures. Stability requirement for high traffic loading design according to specified JKR standard is higher than 8 kN. All the stability results are shown in Figures 4-35 to 4-38, it can be observed that with increasing bitumen content, the stability increases until it reaches a maximum value and then it started to decrease at much higher bitumen content. Table 4-19 shows the stability values obtained at OBC for all types of mixtures. It is observed that maximum stability values for all fine aggregate mixtures exhibit a consistent behavior with the properties of fine aggregate and binder.

Depending on the type of binder, bituminous mixtures for bitumen PEN 50/60 are observed to have higher maximum stability than the bituminous mixtures with the bitumen PEN 80/100, for all types of fine aggregate mixture. Polymer modified mixtures have higher stability values compared with conventional mixtures for some types of fine aggregate. The bituminous mixtures of PEN 50/60 with quarry sand, mining sand and marine sand have the highest stability value at 17.3 kN, 9.14 kN and 6.70 kN respictivily, which is higher than the polymer modified mixture. PM1 and PM2 polymer modified bitumen mixtures do not exhibit a consistent behavior on stability; sometimes PM1 have higher stability values than PM2, and sometimes the opposite result is observed. It can be concluded that the stability of the Polymer modified bituminous mixture regardless of the modified type is higher than the conventional bituminous mixture, this agrees with studies made by Awwad and Shbeeb (2007) and Hamid *et al*. (2008).

The physical properties of fine aggregate play an important role on the mixture stability. An angular particle provides a better interlocking property and a rough surface provides a greater bonding strength with asphalt cement and frictional resistance between particles to provide stability. This is in tandem with the results obtained from other researchers (Topal and Sengoz, 2005, 2006; Abo Qudais and Al Shweily, 2007). Parak and Lee (2002) found that an increase in angularity of fines increase the Marshall stability values at optimum binder content. Choyce found that Marshall Values for mixtures containing crushed fine aggregate are greater than those achieved by mixtures containing natural sand.

Based on the Marshall stability values results quarry sand has highest stability value 14.8-17.3 kN as shown in Table 4-19. This is due to much greater degree of aggregate interlock and inter-particle friction derived from the angular shape and rough surface texture of quarry sand particles and also the overall more continuous and therefore more interlocking nature of the aggregate gradation. This is followed by river sand, mining sand and marine sand mixture respectively. The range of stability value for river sand mixtures, mining sand mixtures, and marine sand mixtures are 7.8-10.9 kN, 7.1-9.1 kN and 5.3-8.2 kN respectively. From Table 4-19 also it can be noticed that there are six different mixtures have stability value less than 8 kN, this material can't be used in wearing course pavement because it doesn't meet the criteria of JKR.

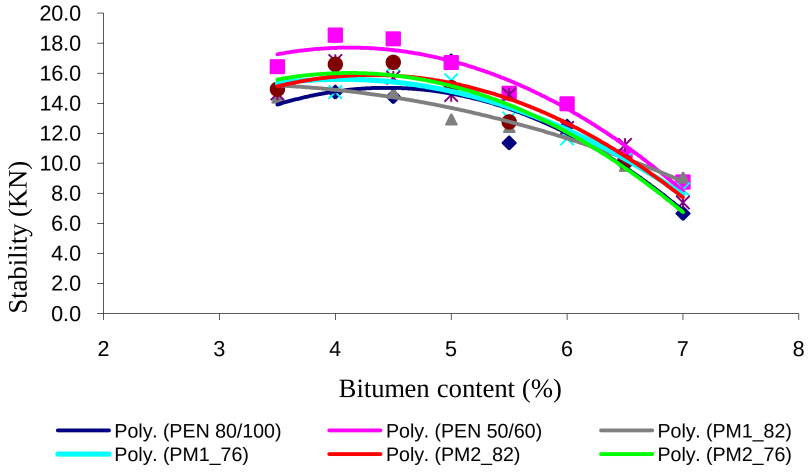


Figure 4-35: Stability of bituminous mixtures containing quarry sand

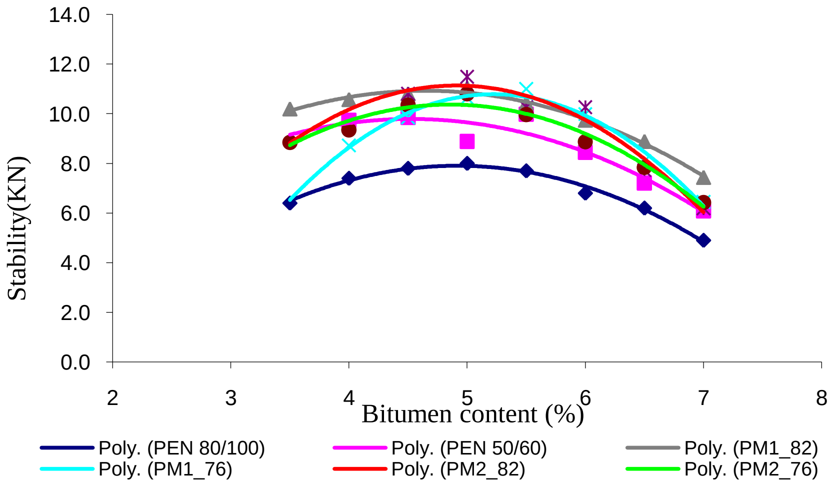


Figure 4-36: Stability of bituminous mixtures containing river sand

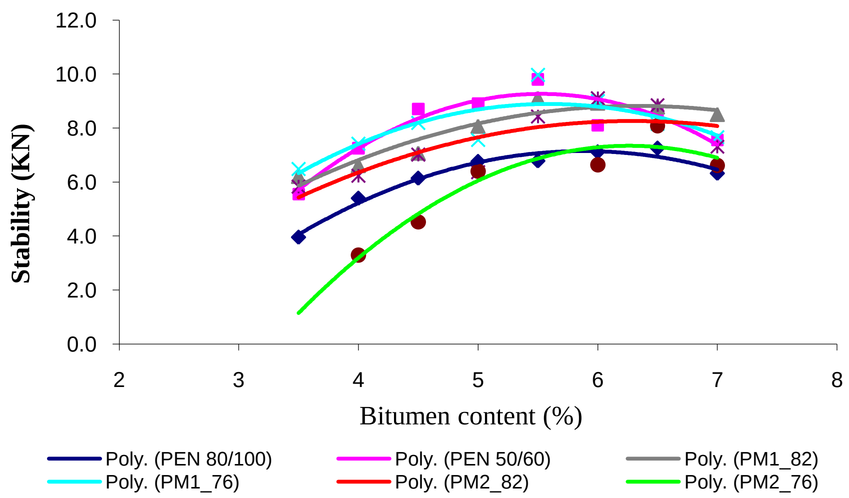


Figure 4-37: Stability of bituminous mixtures containing mining sand

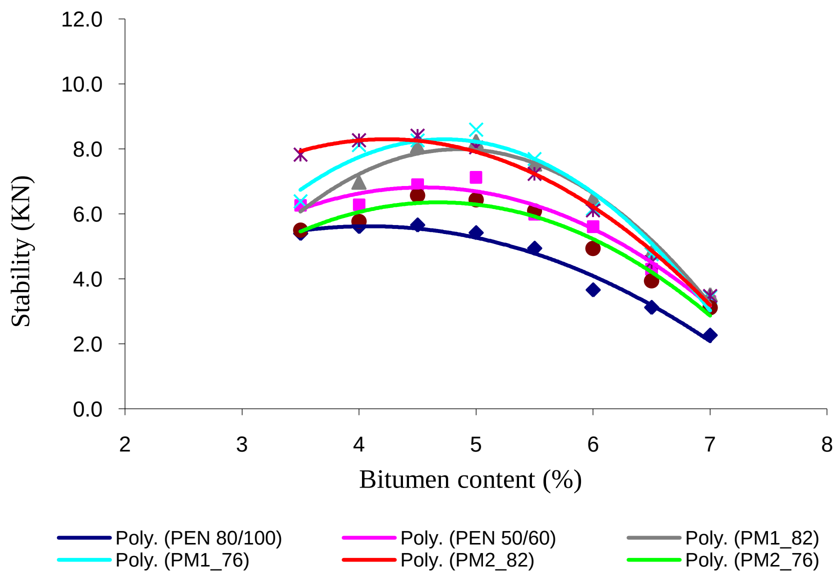


Figure 4-38: Stability of bituminous mixtures containing marine sand

Table 4-19: Stability (kN) at OBC for all types of binders & sands

Type of binder	**Quarry sand**	**River sand**	**Mining sand**	**Marine sand**
PM1_82	14. 800	10. 598	8. 800	8.000
PM1_76	15.400	10.700	8.750	8.200
PM2_82	15.800	10.950	8. 250	8.000
PM2_76	15.600	10.200	7.250	6.200
PEN 50/60	17.300	9.500	9.140	6.700
PEN 80/100	14.900	7.800	7. 125	5.325

4.3.7 Flow

Flow is the deformation that occurs at maximum load when the specimen is subjected to a loading rate of 50.8 mm/minute. The adjusted flow requirement as specified by JKR standard is 2 mm up to 4 mm. Flow does not represent the performance of permanent deformation since it does not perform the equivalent loading mechanism of permanent deformation. It is only used as one of the considered parameters to determine OBC of bituminous mixtures in general. The flow results for all mixtures are shown in Figures 4-39 to 4-42, as can be seen from the figures, the flow increases with increasing asphalt content. From Table 4-20 it can be noticed that the flow value at OBC for conventional mixtures is lower than for modified mixtures in all types of sand except in quarry sand mixture, therefore quarry sand polymer modified mixtures have lower flow values than conventional mixtures. PM1 and PM2 polymer modified bitumen mixtures do not exhibit a consistent behavior on flow; sometimes PM1 have higher flow values than PM2, and sometimes the opposite result is observed. It can be concluded that the flow from Polymer modified bituminous mixture regardless of the modified type is higher than the conventional bituminous mixture, this agrees with studies made by Awwad and Shbeeb (2007) and Hamid *et al*. (2008). Among the four types of fine aggregate mixtures, marine sand exhibited lowest flow values followed by river sand, mining sand and quarry sand mixtures. Except in PM1_82 and PM2_82, where quarry sand mixtures is exhibited lower flow than mining sand mixtures. Because marine sand being rounded and smooth texture that offer less bonding with asphalt and this relatively form thin bitumen film thickness which indicates less flow.

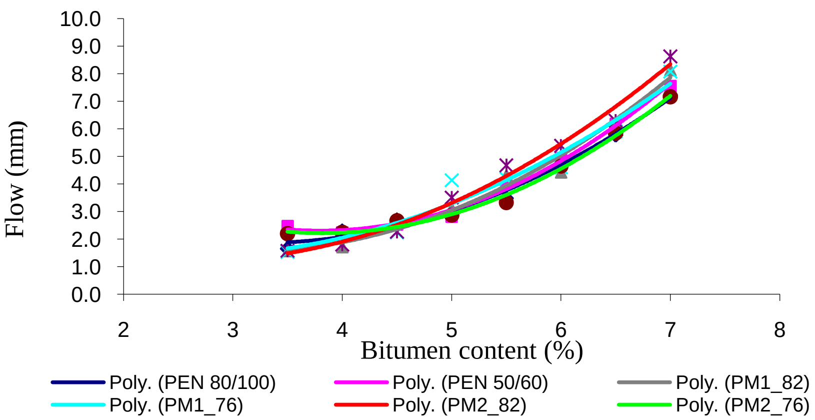


Figure 4-39: Flow of bituminous mixtures containing quarry sand

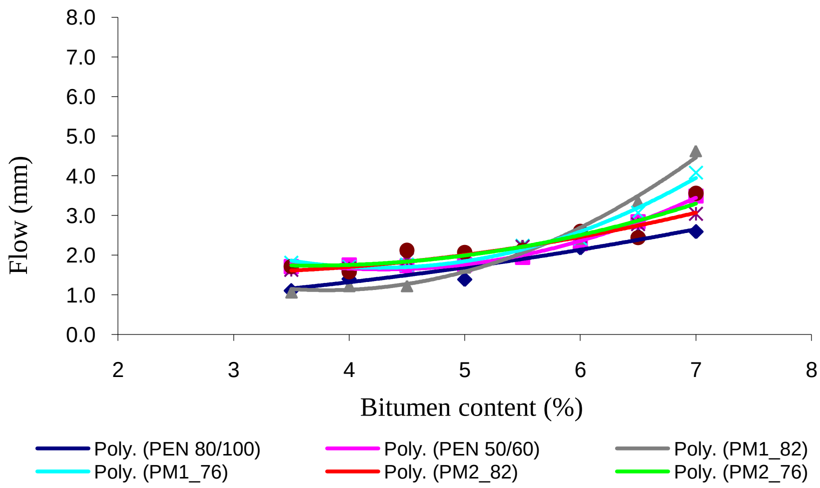


Figure 4-40: Flow of bituminous mixtures containing river sand

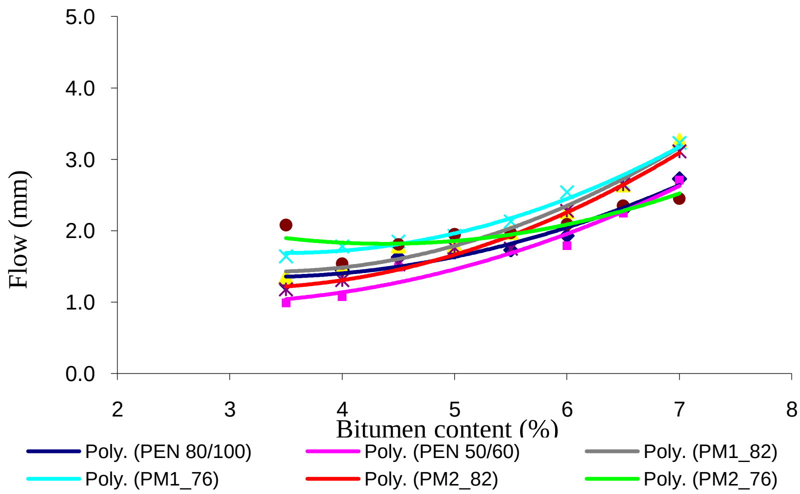


Figure 4-41: Flow of bituminous mixtures containing mining sand

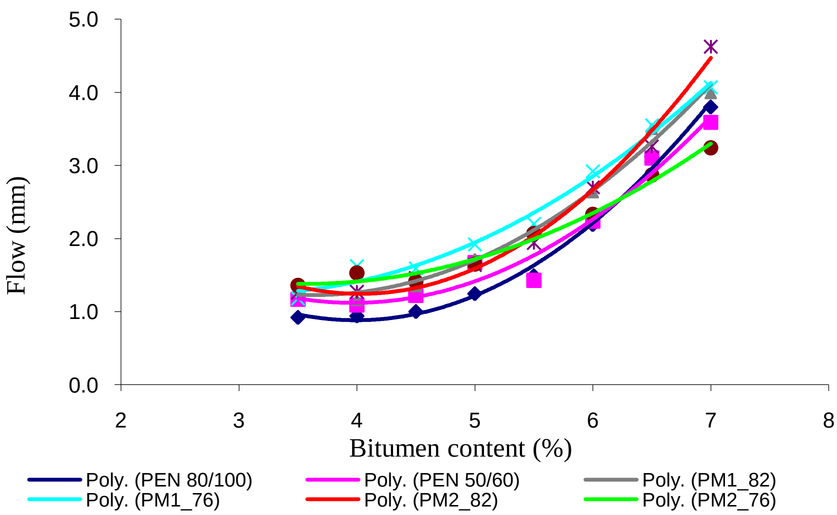


Figure 4-42: Flow of bituminous mixtures containing marine sand

Table 4-20: Flow (mm) at OBC for all types of binders & sands

Type of binder	Quarry sand	River sand	Mining sand	Marine sand
PM1_82	2. 01	1. 95	2. 56	1.72
PM1_76	2.55	2.10	2.50	1.90
PM2_82	2.45	2.08	2.56	1.52
PM2_76	2.60	2.10	2.33	1.75
PEN50/60	2.65	1.85	1.90	1.40
PEN80/100	2. 70	1. 81	2. 06	1.15

4.3.8 Stiffness

Besides stability, stiffness is also used to measure the strength of the mixtures. The stiffness values for all mixtures at OBC are tabulated in Table 4-21. It can be seen that stiffness follows two distinct trends: one it is based on the type of binder and the second is based on the type of fine aggregate in the mixtures.

Depending on the type of binder, bituminous mixtures of PEN 50/60 were observed to have higher stiffness values than bituminous mixtures of PEN 80/100 for all types of sand. This is due to the different in stiffness of both binders when analyzing the softening point and penetration properties it was found that bitumen PEN 50/60 has higher consistency than bitumen PEN 80/100 which means bitumen PEN 50/60 is harder compared to the bitumen PEN 80/100. Polymer modified mixtures have higher stiffness values than conventional mixtures, except in PM1_76 and PM2_76 where quarry sand and marine sand mixtures have lower stiffness than bituminous mixtures of PEN 50/60. River sand mixture with PM2_76 has lower stiffness than river sand mixture with bitumen PEN 50/60, at 4.75 kN/mm and 5.05 kN/mm respectively.

Depending on the type of sand quarry sand incorporating bituminous mixture exhibited highest stiffness values as obtained from the Marshall Stability test compared to the other sand types. This is because quarry sand particles being most angular and rougher will result in better interlocking between the particles in the mix and also result in better bond strength between the particles and the binder. A higher stiffness value of a mixture indicates that the mixture is likely to be more resistant to the permanent deformation.

Physical properties of fine aggregate such as angular shape and rough surface texture results in a much stiffer mix (Topal and Sengoz, 2005, 2006). The natural solid particle of bitumen influences the stiffness results also. Thus, it can be concluded that the stiffness of the mixtures is influenced not only fine particle shapes and surface texture but also on the type of binder (binder stiffness value).

Table 4-21: Stiffness (kN/mm) at OBC for all types of binders & sands

Type of binder	**Quarry sand**	**River sand**	**Mining sand**	**Marine sand**
PM1_82	7.1	5.6	3.55	4.6
PM1_76	5.9	5.05	3.6	4.2
PM2_82	6.4	5.13	3.35	4.88
PM2_76	5.85	4.75	3.13	3.55
PEN 50/60	6.28	5.05	4.85	4.6
PEN 80/100	4.7	4.4	3.53	4.4

4.4 Mixture Performance

The performance of bituminous pavement is affected by permanent deformation (rutting) and fatigue cracking. These distresses occur due to increasing traffic loading on road building materials besides environmental factors. Creep test and wheel tracking test can be used to assess the rutting resistance of bituminous pavements, while beam fatigue test was used to evaluate the fatigue cracking. In this study the effect of fine aggregate characteristics and polymer modified bitumen on the rutting and fatigue resistance of bituminous mixtures was determined.

4.4.1 Dynamic creep

Corte *et al*. (1994) had shown evidence that dynamic creep test is more representative of what happened in the field, this proved when results from dynamic creep test shown same pattern with the rut depth results obtained from the Laboratoire Central des Ponts et Chaussees (LCPC) circular test track that was carried out on a pavement. Thus, it is also considered that the dynamic creep test shows a good test for predicting the rutting performance of the types of the mixes that is used in the study. The results of creep test presented below are average values obtained from three specimens. These mixtures were made at optimum binder content in order to obtain better creep properties compared to other mixtures with binder content below or above the optimum binder contain (Napiah, 1993). To investigate the creep properties of the asphalt concrete mixture, two main variables were included. These variables are fine aggregate type and binder type (polymer modified bitumen versus conventional binder). Figures 4-43 to 4-46 show the results of the dynamic creep test in terms of creep modulus against number of cycles. The graphs indicate that creep modulus decreases with increasing loading cycles. Quarry sand has higher creep modules compared to the other types of sand, followed by river sand, mining sand and marine sand respectively. This is due to the physical properties of quarry sand particles, which has the highest angularity, rougher in surface, highest shear resistance and highest percentage of Al_2O_3 which is hard constituent tend to increase the hardness of bituminous mixtures, hence increasing the stiffness of the mixtures. All of these properties made quarry sand mixture as better than other types, for resistance to the creep deformation.

Polymer modified bitumen mixture shows higher creep modulus compared to the conventional bitumen mixture. PM1 has higher creep modulus compared to the PM2 since PM1 is stiffer than PM2 as proven by the physical properties test results. PM1_82 also has higher creep modulus than PM1_76. Bitumen PEN 50/60 mixture has demonstrated higher creep modulus than bitumen PEN 80/100 mixture because bitumen PEN 50/60 is stiffer than bitumen PEN 80/100 as proved by physical properties and the stiffness values obtained from Van Der Pool nomograph.

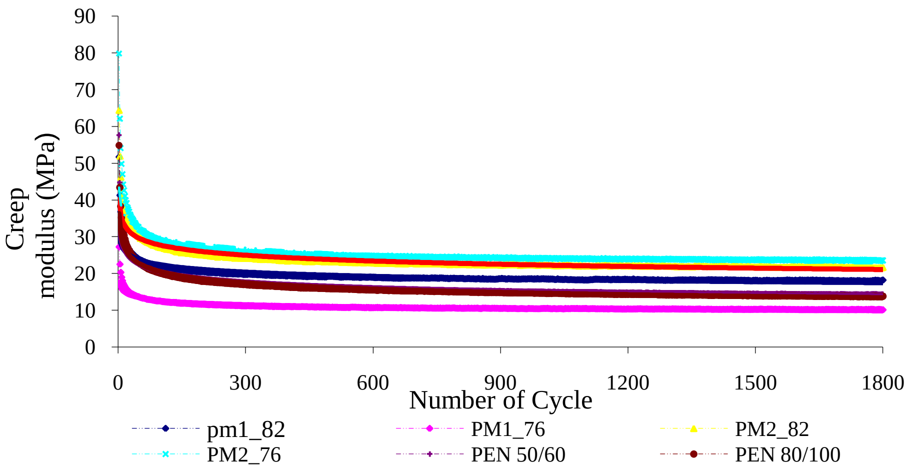


Figure 4-43: Creep stiffness of quarry sand mixtures

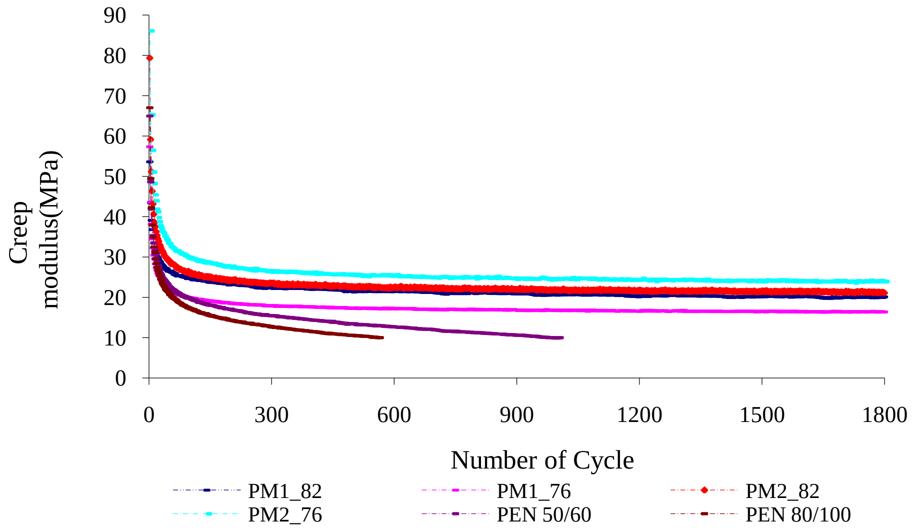


Figure 4-44: Creep stiffness of river sand mixtures

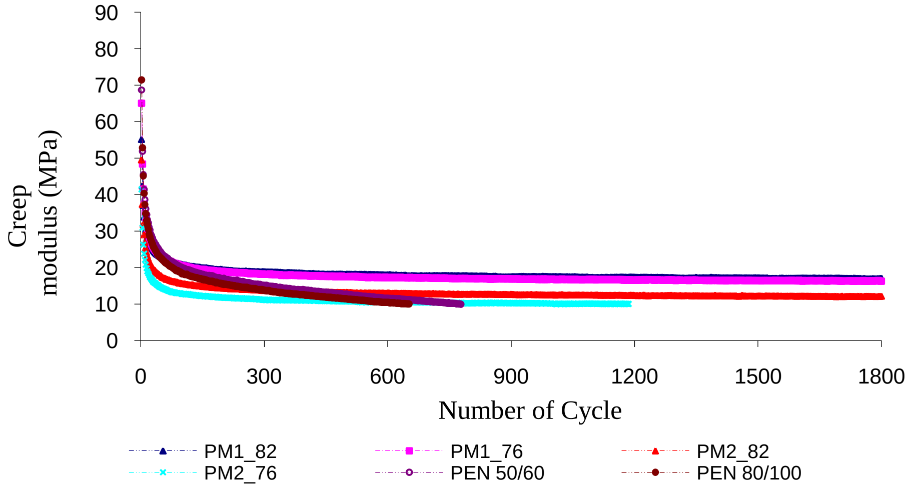


Figure 4-45: Creep stiffness of mining sand mixtures

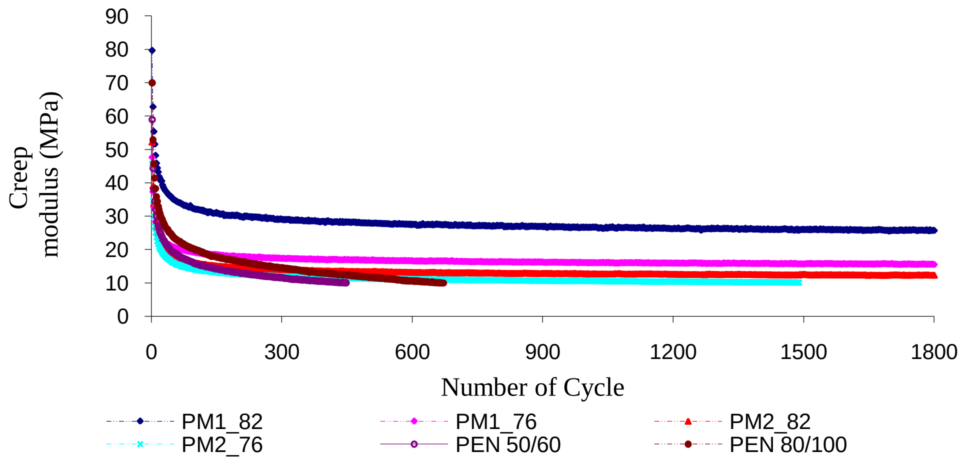


Figure 4-46: Creep stiffness of marine sand mixtures

Figures 4-47 to 4-50 show a graphical representation of the creep results in terms of creep modulus versus number of cycles in logarithmic scales. The comparison between the combinations was carried out based on the slope of the graph, which represents the sensitivity of the mixture to creep deformation. As can be seen from Table 4-22, quarry sand mixture has the lowest slope compared to the other types of sands, which means quarry sand is less susceptible to the creep deformation. This is because quarry sand is the most angular in shape, rougher in texture, and has higher shear resistance, and higher oil absorption values as compared to the others types of sand. The second type of sand that has less susceptible to the creep deformation is river sand followed by mining sand and marine sand mixture respectively. This is due to their physical, chemical and mechanical properties of sand. On the basis of above discussion it is concluded that the physical, chemical and mechanical properties of fine aggregate have positive effect on creep resistance. This is in agreement with studies made by many researchers (Lee *et al*., 1999; Eyad *et al.*, 2001; Topal and Sengoz, 2005, 2006; Abo Qudais and Al Shweily, 2007).

Polymer modified bitumen mixture has lesser slope compared to the conventional bitumen mixture, this is because polymer modified bitumen is more viscous, which increase the stiffness of the bitumen and thus deform less under traffic loading (Ahmedzade *et al.*, 2007), similarly PM1_82 is less susceptible to deformation followed by PM1_76, PM2_82, PM2_76, 50/60 PEN, and 80/100 PEN respectively, this is due to the effect of their physical properties which are stiffness and hardness.

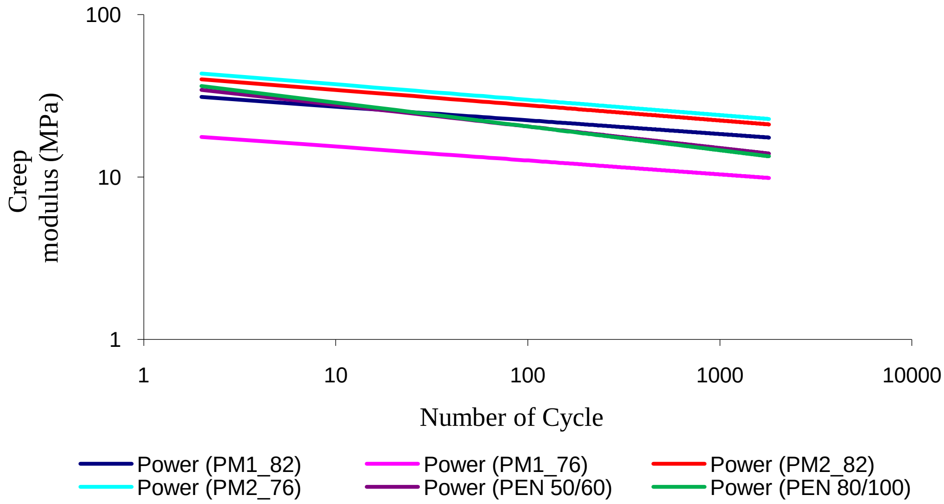


Figure 4-47: Creep stiffness vs. Cycles for quarry sand mixtures

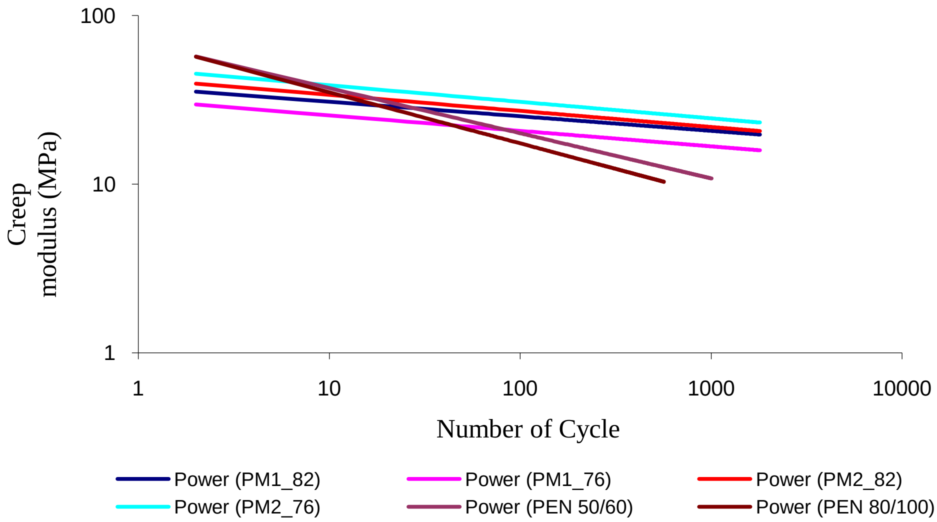


Figure 4-48: Creep stiffness vs. Cycles for river sand mixtures

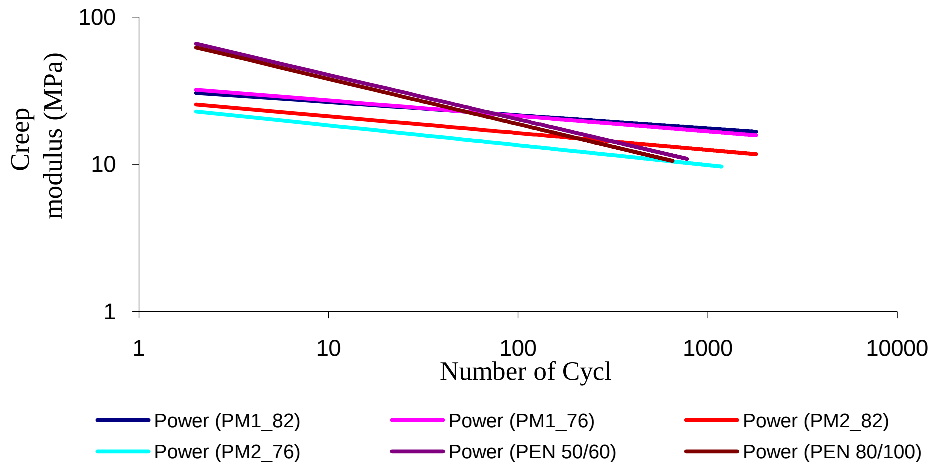


Figure 4-49: Creep stiffness vs. Cycles for mining sand mixtures

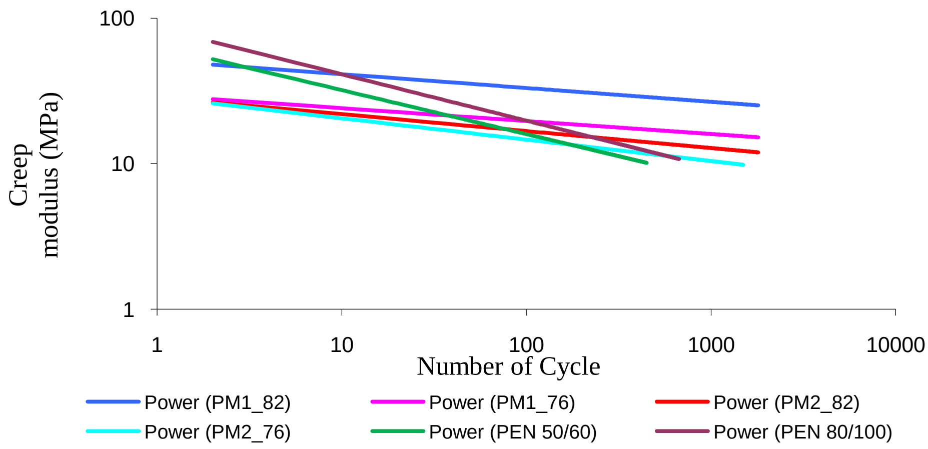


Figure 4-50: Creep stiffness vs. Cycles for marine sand mixtures

Table 4-22 and Figure 4-51 show the varying performance of different types of sand with different types of binder based on the slope. It can be concluded from the graph that having lower slope, means less susceptible to deformation or better resistance to the creep.

Table 4-22: Dynamic creep results- slope from Creep modulus vs. Cycle

Type of binder	quarry sand	river sand	mining sand	marine sand
PM1_82	0.0845	0.0861	0.0895	0.0950
PM1_76	0.0856	0.0919	0.105	0.0889
PM2_82	0.0942	0.0953	0.1145	0.1164
PM2_76	0.0946	0.0977	0.1351	0.1469
PEN 50/60	0.1328	0.2678	0.3024	0.3042
PEN 80/100	0.1465	0.3044	0.3086	0.3197

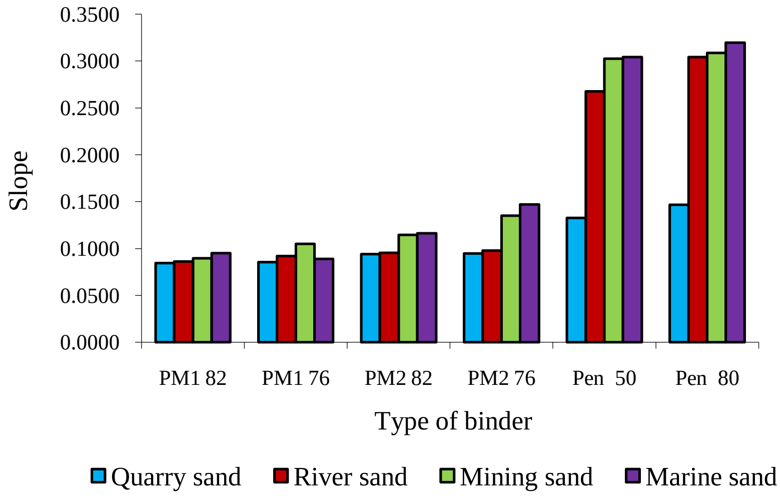


Figure 4-51: Sensitivity degree of fine aggregate mixtures to the creep deformation

Figure 4-52 to 4-57 show typical graphical plots of mixture stiffness (S_{mix}) versus bitumen stiffness (S_{bit}) in a double logarithmic graph. Mixture stiffness was measured by universal testing machine at 40^0C for 1 hour loading time period and the corresponding bitumen stiffness S_{bit} was calculated by using Van Der Poel's nomograph.

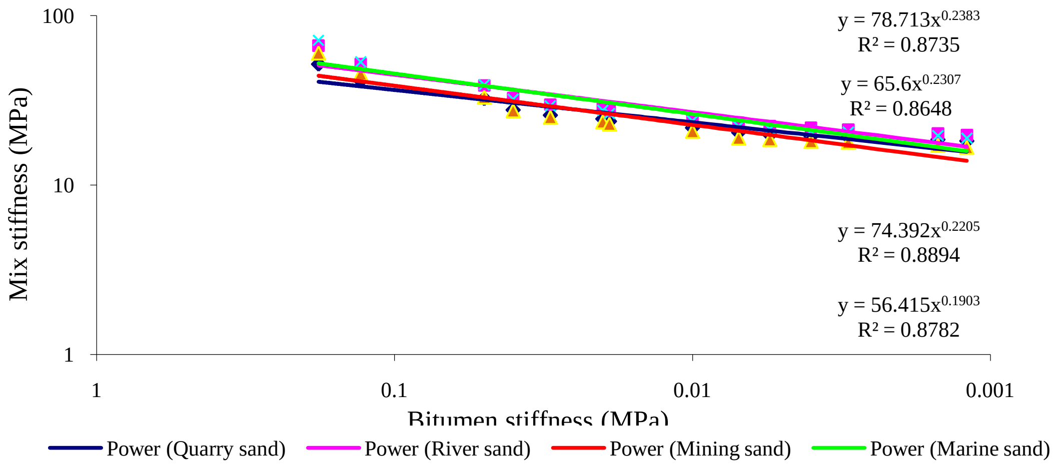


Figure 4-52: Mix stiffness vs. bitumen stiffness for PM1_82 with sand

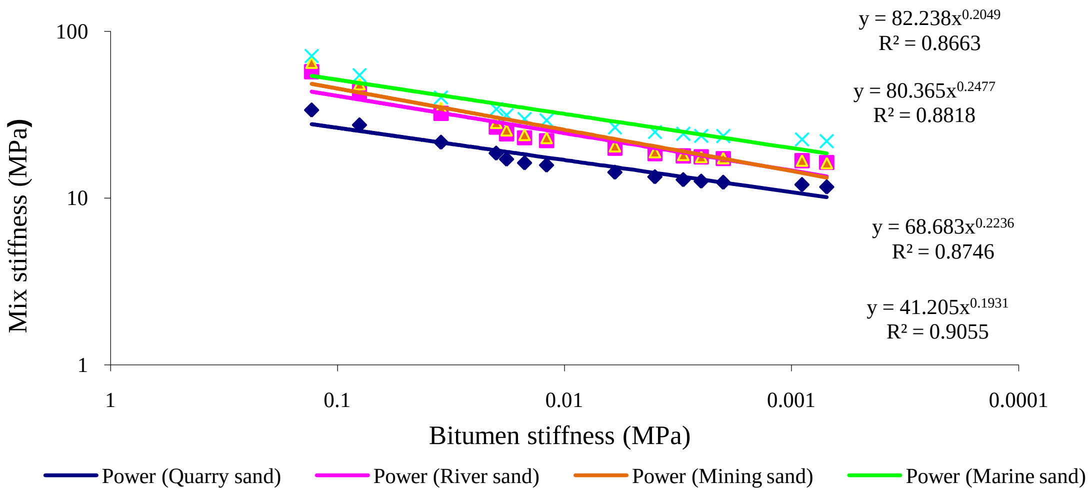


Figure 4-53: Mix stiffness vs. bitumen stiffness for PM1_76 with sand

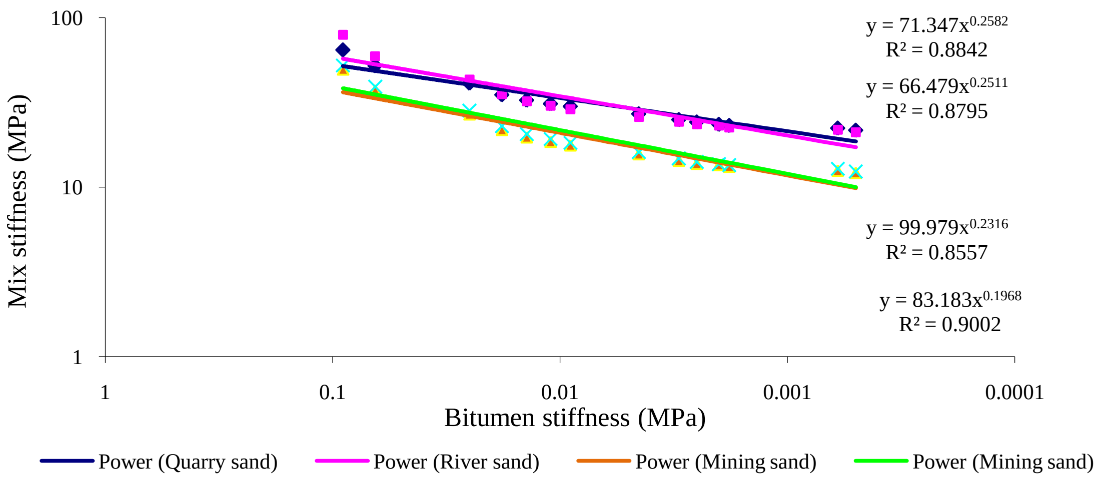


Figure 4-54: Mix stiffness vs. bitumen stiffness for PM2_82 with sand

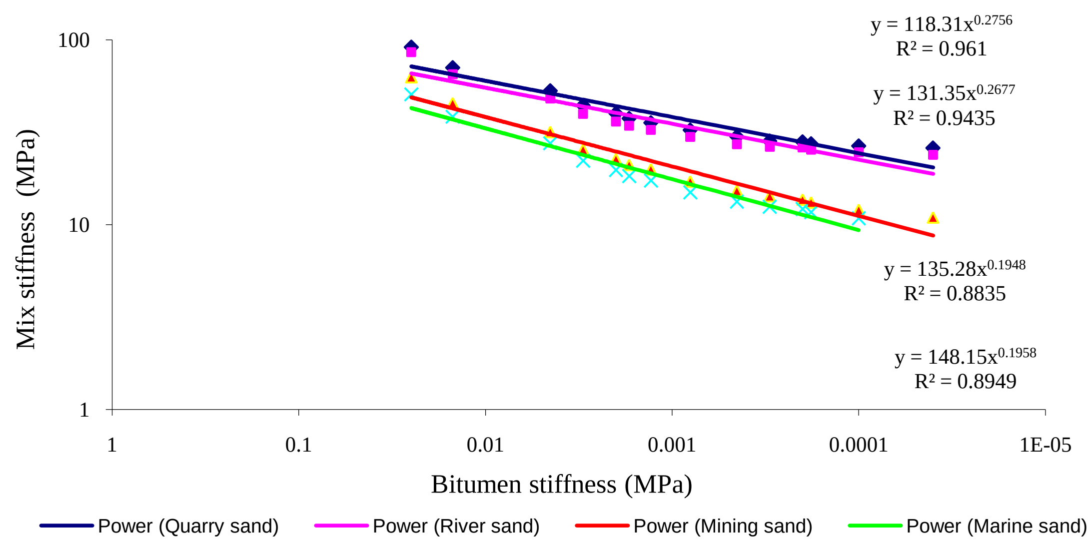


Figure 4-55: Mix stiffness vs. bitumen stiffness for PM2_76 with sand

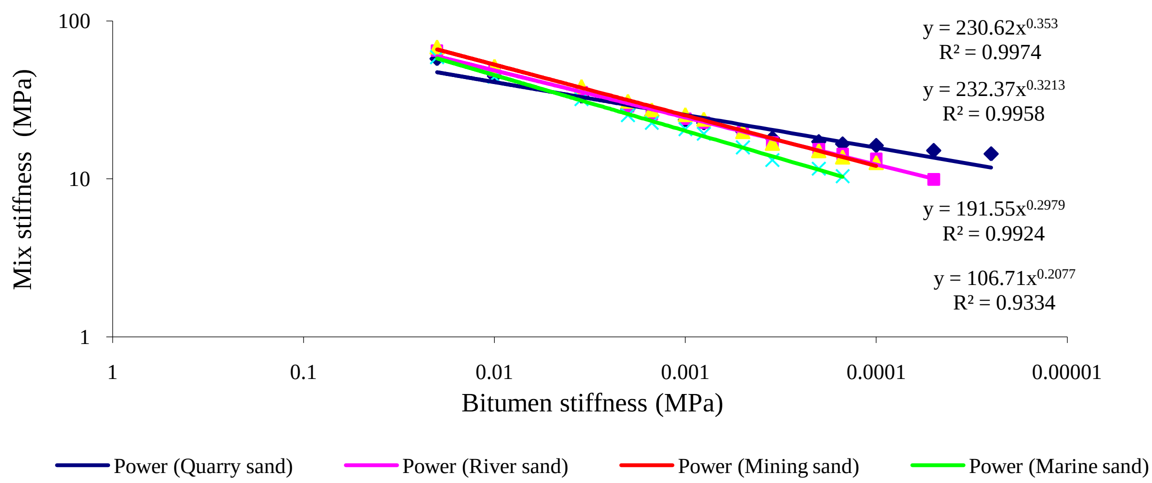


Figure 4-56: Mix stiffness vs. bitumen stiffness for PEN 50/60 with sand

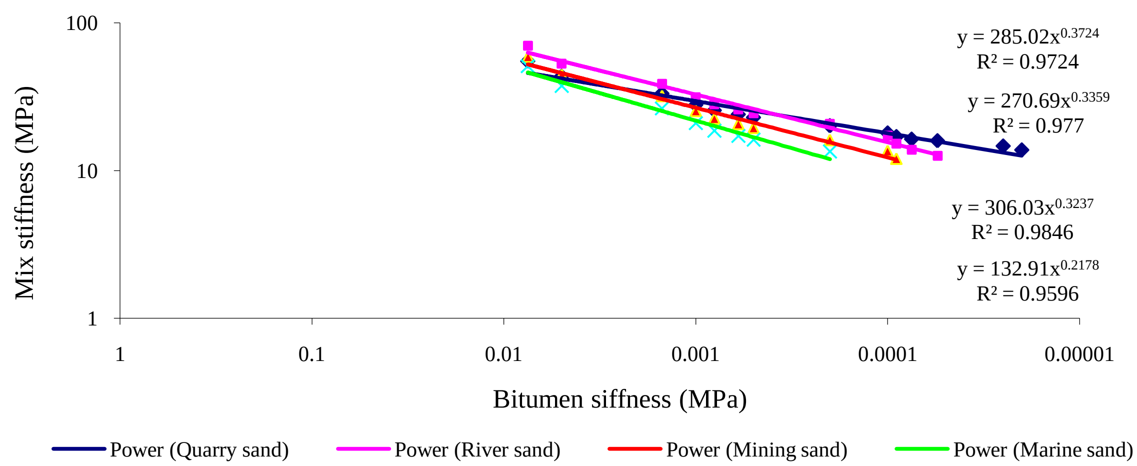


Figure 4-57: Mix stiffness vs. bitumen stiffness for PEN 80/100 with sand

The resistances to permanent deformation from the creep tests were determined using the slope from the log-log relationship of mixture stiffness versus binder stiffness. Mixture stiffness corresponds to a fixed loading time, or the time to reach a critical strain level. This manner of characterization in the mix is based on the fact that more resistant mixtures have stiffness that are greater and decrease less rapidly with increasing time. Stiffness of mixes containing conventional and polymer modified bitumen for all types of sand are shown in Figures 4-52 to 4-57 at any particular time of loading.

The stiffness of mixture containing polymer modified bitumen is higher than the stiffness of the mixture made of conventional bitumen. The slope of the S_{mix} versus S_{bit} relationship also indicates the mixture's susceptibility to time of loading that is smaller value of slope indicates less susceptibility to the creep deformation (better performance).

The results in Table 4-23 indicated that mixtures containing polymer modified bitumen were less susceptible to loading time in comparison to the conventional mixtures. Better results also obtained by PM1_82 compared to the PM1_76, and by PM2_ 82 compared to the PM2_76 mixtures. Mixtures containing bitumen PEN 50/60 show less susceptibility to loading time in comparison to the mixtures containing bitumen PEN 80/100, because bitumen PEN 50/60 is stiffer or harder compared to the bitumen PEN 80/100 as the consistency tests results proved.

Based on type of sand it can be noticed that quarry sand mixtures are less susceptible to the loading time in comparison to the other types of sand. This is due to the effect of the physical, chemical and mechanical properties of the quarry sand. The second sand which shows less susceptibility to the loading time is river sand followed by mining sand and marine sand mixture respectively.

Table 4-23: Creep result in term of S_{mix} vs. S_{bit}

Mixture	**Formula**	**(a) Value**	**(b) Slope**
Q/PM1_82	$y = 56.415x^{0.1903}$	56.415	0.1903
Q/PM1_76	$y = 41.205x^{0.1931}$	41.205	0.1931
Q/PM2_82	$y = 83.183x^{0.1968}$	83.183	0.1968
Q/PM2_76	$y = 148.15x^{0.1958}$	148.15	0.1958
Q/PEN 50/60	$y = 106.71x^{0.2077}$	106.71	0.2077
Q/PEN 80/100	$y = 132.91x^{0.2178}$	132.91	0.2178
R/PM1_82	$y = 74.392x^{0.2205}$	74.392	0.2205
R/PM1_76	$y = 68.683x^{0.2236}$	68.683	0.2236
R/PM2_82	$y = 99.979x^{0.2316}$	99.979	0.2316
R/PM2_76	$y = 135.28x^{0.1948}$	135.28	0.1948
R/PEN 50/60	$y = 191.55x^{0.2979}$	191.55	0.2979
R/PEN 80/100	$y = 306.03x^{0.3237}$	306.03	0.3237
M/PM1_82	$y = 65.600x^{0.2307}$	65.600	0.2307
M/PM1_76	$y = 80.365x^{0.2477}$	80.365	0.2477
M/PM2_82	$y = 66.479x^{0.2511}$	66.479	0.2511
M/PM2_76	$y = 131.35x^{0.2677}$	131.35	0.2677
M/PEN 50/60	$y = 232.37x^{0.3213}$	232.37	0.3213
M/PEN 80/100	$y = 270.69x^{0.3359}$	270.69	0.3359
MR/PM1_82	$y = 78.713x^{0.2383}$	78.713	0.2383
MR/PM1_76	$y = 82.238x^{0.2049}$	82.238	0.2049
MR/PM2_82	$y = 71.347x^{0.2582}$	71.347	0.2582
MR/PM2_76	$y = 118.31x^{0.2756}$	118.31	0.2756
MR/PEN 50/60	$y = 230.62x^{0.353}$	230.62	0.353
MR/PEN 80/100	$y = 285.02x^{0.3724}$	285.02	0.3724

The relationship between stiffness of mixture and stiffness of bitumen can be expressed in the following form of equation,

$$S_{mix} = a\,(S_{bit})^{b}$$

A good correlation between S_{mix} and S_{bit} can also be seen from the plots. The equation of the lines can be expressed as:

$$Log(Y) = Log\,(a) + b\,Log(X) \qquad \text{4-2}$$

or

$$Y = a\,X^{b} \qquad \text{4-3}$$

where: *a*- interception of the line with Y-axis.

b- slope of the line.

Y- stiffness of the mix in MPa.

X - stiffness of the binder in MPa.

In this equation, the coefficients “a” and “b” represent the mixture in terms of deformation performance. Mixture stiffness is indicated by the constant “a” and the slope “b” indicates the sensitivity of the mixture to loading time and hence the bitumen stiffness. Therefore, mixture with a high value of “a” and low slope “b” will exhibit good deformation performance (Cabrera and Nikolaides, 1988).

Figure 4-58 and Table 4-24 show the stiffness values of different types of binder, obtained from the Van Der Poel nomograph based on the time of loading, penetration index (PI), and temperature difference (temperature of the creep test and ring and ball temperature of each binder). The loading time represents creep loading time which is 1hr (3600 s) and the temperature is the creep test temperature 40^{0}C. The penetration index is a measure of the temperature-susceptibility of bitumen, based on the linear relationship between penetration and temperature, assuming that all bitumens have a penetration of approximately 800 at their softening point. The PI was developed by Pfeiffer and Van Doormaal (1936), and it can be determined using the following equation:

$$\frac{\log 800 - \log pen}{T_{R+B} - T} = \frac{20 - PI}{10 + PI} \times \frac{1}{50} \qquad \text{4-4}$$

where: *PI*- penetration index.

Pen- measured penetration at temperature T (normally 25^{0}C).

T_{R+B}- softening point temperature.

T- penetration test temperature (normally 25^{0}C).

As it can be seen polymer modified bitumen has higher stiffness compared to conventional bitumen. That means polymer modified bitumen is harder because it has higher softening point and lower penetration compared to the conventional binder. The order of the binders in decreasing stiffness is PM1_82, PM1_76, PM2_82, PM2_76, PEN 50/60, and PEN 80/100 respectively.

Table 4-24: Stiffness values of the binders

Loading time	Stiffness of binder (MPa)					
time/second	PM1_82	PM1_76	PM2_82	PM2_76	PEN 50/60	PEN 80/100
4	0.18	0.13	0.09	0.025	0.02	0.0075
8	0.13	0.08	0.065	0.015	0.01	0.005
20	0.05	0.035	0.025	0.0045	0.0035	0.0015
40	0.04	0.02	0.018	0.003	0.002	0.001
60	0.03	0.018	0.014	0.002	0.0015	0.0008
80	0.02	0.015	0.011	0.0017	0.001	0.0006
100	0.019	0.012	0.009	0.0013	0.0008	0.0005
200	0.01	0.006	0.0045	0.0008	0.0005	0.0002
400	0.007	0.004	0.003	0.00045	0.00035	0.0001
600	0.0055	0.003	0.0025	0.0003	0.0002	0.00009
800	0.004	0.0025	0.002	0.0002	0.00015	0.000075
1000	0.003	0.002	0.0018	0.00018	0.0001	0.000055
2000	0.0015	0.0009	0.0006	0.0001	0.00005	0.000025
3600	0.0012	0.0007	0.0005	0.00004	0.000025	0.00002

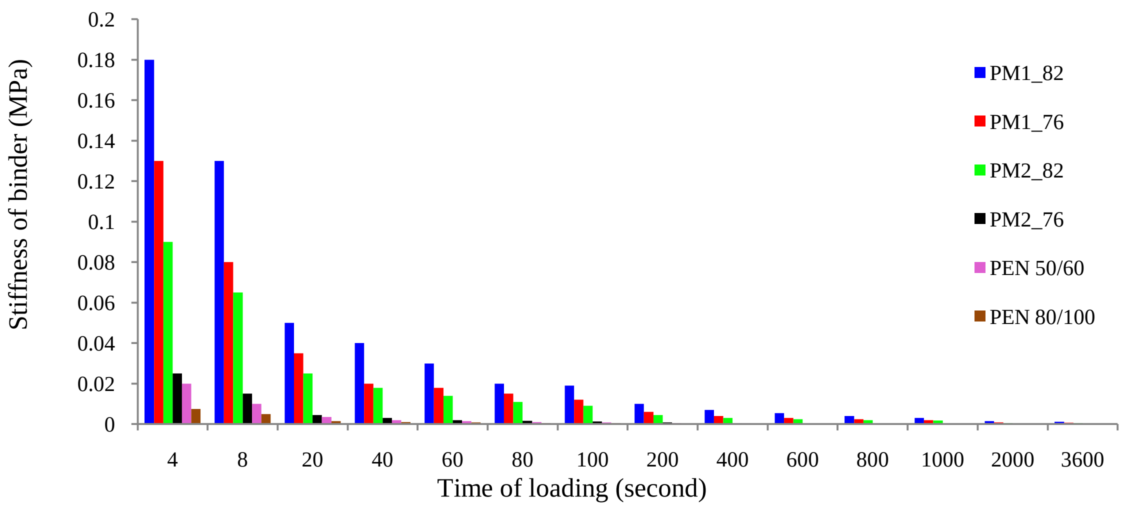


Figure 4-58: Stiffness comparison for types of binders

4.4.1.1 Estimation of rut depth:

Hills *et al*. (1974) suggested the following equation for determining the stiffness modulus of bitumen corresponding to its viscous part:

$$(S_{bit})^{v} = 3\eta/N.T_{W} \qquad 4\text{-}5$$

where: $(S_{bit})^{v}$- viscous component of the stiffness modulus of the bitumen.

η- viscosity of the binder as a function of PI, and ring and ball temperature and it was obtained from Figure 4-59.

N- number of wheel passes in standard axles.

T_w - time of loading for one wheel pass.

The formula which was used to calculate the rut depth of the pavement from laboratory creep test results was initially proposed by Hills *et al.* and Van Der Loo as in (Cabrera and Nikolaides, 1988).

$$R_d = C_m \times H \times \sigma_{av} / S_{mix.creep} \qquad 4\text{-}6$$

where: R_d- calculated rut depth of pavement.

C_m- correlation factor for dynamic effect varying between 1.0 and 2.0.

H- pavement layer thickness.

σ_{av}- average stress in pavement related to wheel loading and stress distribution.

$S_{mix\text{-}}$ stiffness of design mixture derived from creep test at a certain value of stiffness related to the viscous part of bitumen.

The results of estimated rut depth presented in this section were derived from Van der Loo`s equation with the following numerical assumption:

T_w = 0.02 sec, C_m = 1.5, H = 100 mm, and σ_{av} = 0.25 MPa.

The results of rut depth estimation are shown in Figures 4-60 to 4-63, and presented in relation to the number of standard axle repetitions. The results indicate that a high significant correlation exists between estimated rut depth and number of standard axle repetitions. The estimated rut depth of all mixtures containing polymer modified binder is lower than the estimated rut depth of conventional mixtures. Since the polymer modified bitumen is more viscous, this contribution leads to increasing stiffness of the bituminous mixture and thus consequently improves the resistance to permanent deformation, therefore pavements will deform less under traffic loading. This is in line with the research findings of others (Giavarini et al., 1996; Tayfur *et al*., 2007; Ahmedzade and Yilmaz, 2007; Chiu and Lu, 2007). However quarry sand mixtures with PEN 50/60 have a lesser or almost the same rut depth as quarry sand mixtures with PM1_76.

For resistance to permanent deformation, quarry sand mixtures exhibit the best performance among the four types of sand since they have the least estimated rut depth. This is followed by river sand, mining sand and marine sand as was shown in the Figures 4-60 to 4-63. These observations were also confirmed by the physical, chemical and mechanical properties of the sand. Quarry sand, which has the best resistance to rutting, also has the best physical chemical and mechanical properties. The order of rutting resistance for the four types of sand is also in accordance with the order of their physical chemical and mechanical properties. This observation agrees well with previous work by many researchers (Lee *et al*., 1999; Eyad *et al.*, 2001; Topal and Sengoz, 2005, 2006; Abo Qudais and Al Shweily, 2007). They found that the characteristic of fine aggregate (physical, chemical and mechanical) have important roles in resisting permanent deformation.

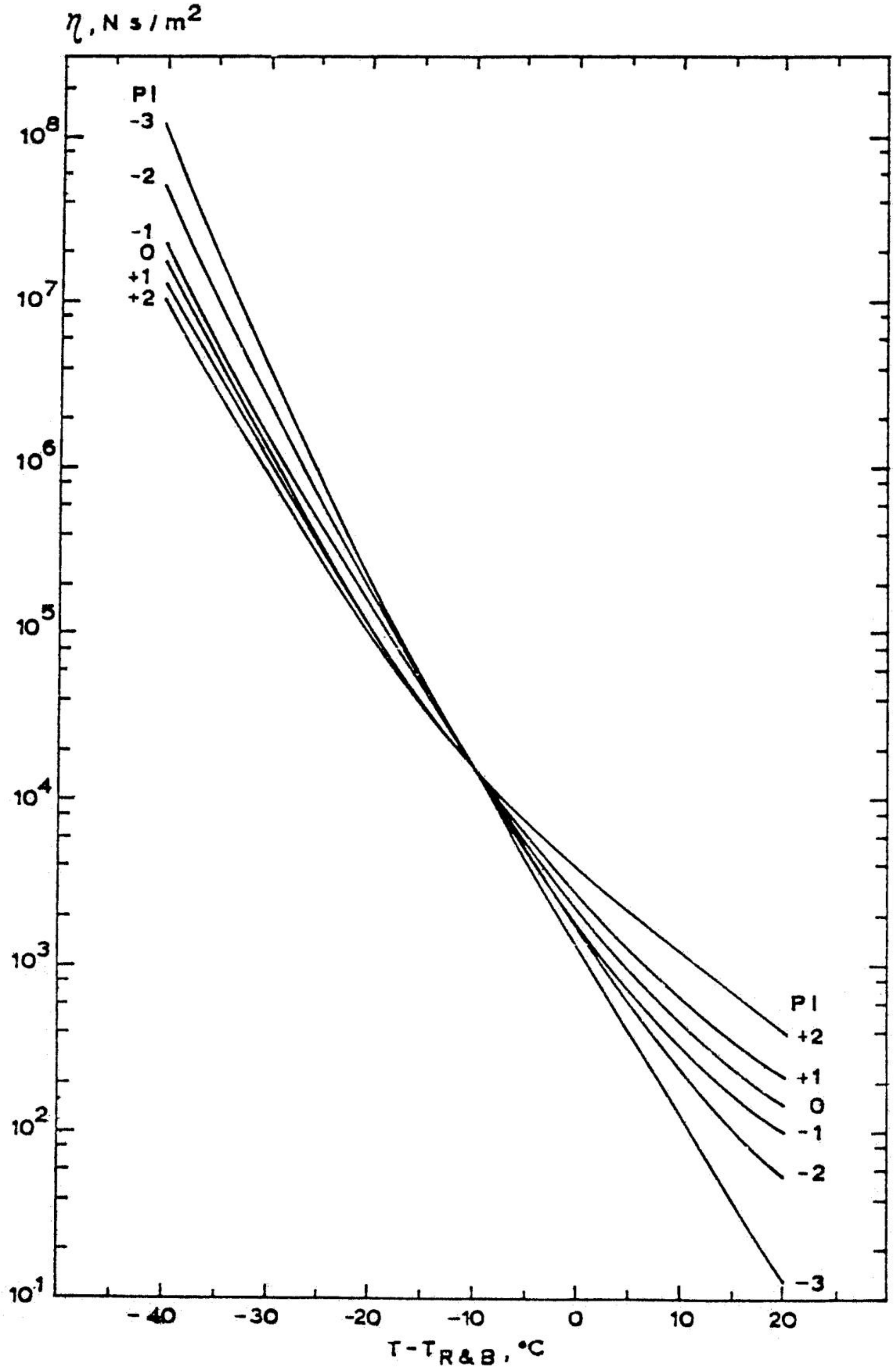


Figure 4-59: Viscosity of bitumen as a function of (T-$T_{R\&B}$) and PI

obtained from Van der pool's nomograph

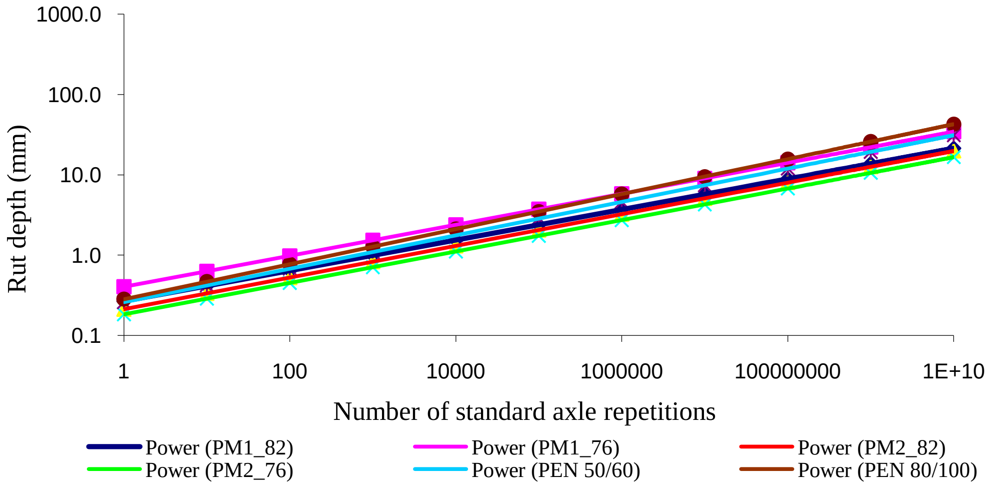


Figure 4-60: R_d estimation related to the number of standard axle repetitions for quarry sand mixtures

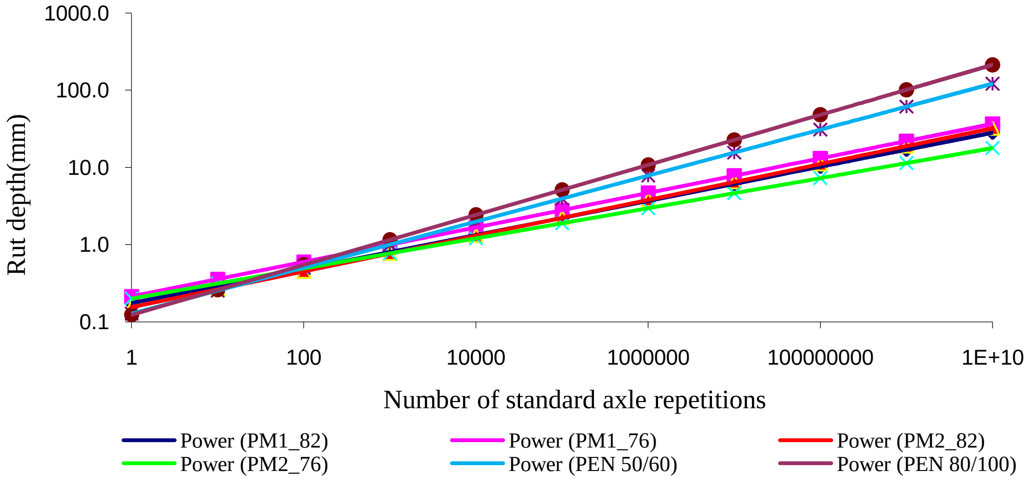


Figure 4-61: R_d estimation related to the number of standard axle repetitions for river sand mixtures.

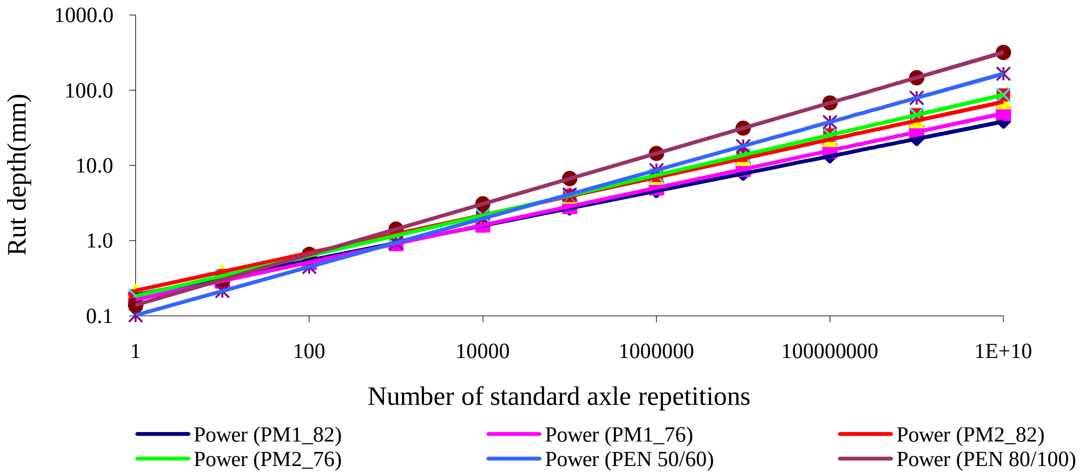


Figure 4-62: R_d estimation related to the number of standard axle repetitions for mining sand mixtures.

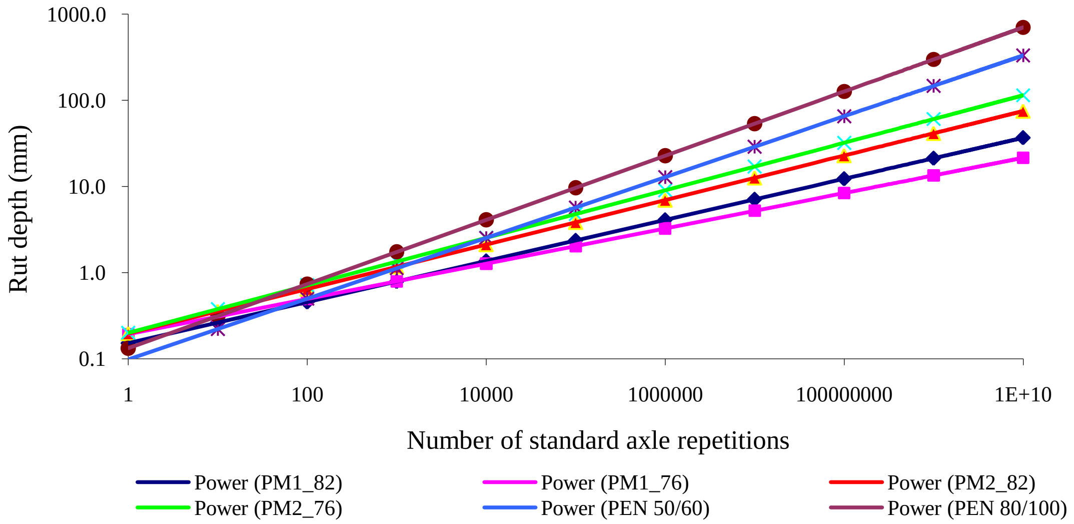


Figure 4-63: R_d estimation related to the number of standard axle repetitions for marine sand mixtures.

Table 4-25 presents the creep characteristics equations and shows the values of slope “b”, which represents deformation and the constant coefficient “a” which represents the magnitude of creep stiffness for all types of sand/binder mixtures. The values of “a” and “b” were obtained from the plot of rut depth versus number of standard axle repetitions, the best mixture is the one that has a higher value of “a” and lower slope value “b” (Cabrera and Nikolaides, 1988).

The table also show that polymer modified bituminous mixtures have smaller slope “b” compared to the conventional bituminous mixtures indicating that they are less susceptible to permanent deformation. In order of least susceptibility to permanent deformation PM1_82 exhibited the best characteristics followed by PM1_76, PM2_82, PM2_76, PEN 50/60, and PEN 80/100 respectively, as the consistency properties proved. However PM2_76 has a less steep slope than PM2_82 for quarry sand mixtures, and lesser value than any other type of binders for river sand mixtures. For marine sand mixture, PM1_76 has the least slope.

The value of “a” for the polymer modified bitumen is larger compared to the conventional bitumen for all types of sand, with the exception of quarry sand where the 50/60 mixture and 80/100 mixture have larger “a” values than PM2_82 and PM2_76. Generally PM1_82 and PM1_76 have highest resistance to permanent deformation, but PM2_ 82 and PM2_76 increases the resistance to permanent deformation and mix cohesion.

Among the four types of sand, quarry sand mixture exhibited least slope “b” and highest “a” value compared to the other types of sand mixtures. This means quarry sand mixture has least susceptibility to the permanent deformation, whilst marine sand mixture showed the highest susceptibility to the permanent deformation.

Table 4-25: Creep Characteristic Equations

mixture	Formula	“a” value	“b” Slope
Q/PM1_82	$y = 0.2673x^{0.1903}$	0.2673	0.1903
Q/PM1_76	$y = 0.4035x^{0.1931}$	0.4035	0.1931
Q/PM2_82	$y = 0.2131x^{0.1968}$	0.2131	0.1968
Q/PM2_76	$y = 0.1829x^{0.1958}$	0.1829	0.1958
Q/PEN 50/60	$y = 0.2608x^{0.2077}$	0.2608	0.2077
Q/PEN 80/100	$y = 0.2837x^{0.2178}$	0.2837	0.2178
R/PM1_82	$y = 0.1754x^{0.2205}$	0.1754	0.2205
R/PM1_76	$y = 0.2129x^{0.2236}$	0.2129	0.2236
R/PM2_82	$y = 0.1553x^{0.2316}$	0.1553	0.2316
R/PM2_76	$y = 0.2007x^{0.1948}$	0.2007	0.1948
R/PEN 50/60	$y = 0.1277x^{0.2979}$	0.1277	0.2979
R/PEN 80/100	$y = 0.1235x^{0.3237}$	0.1235	0.3237
M/PM1_82	$y = 0.1894x^{0.2307}$	0.1894	0.2307
M/PM1_76	$y = 0.1644x^{0.2477}$	0.1644	0.2477
M/PM2_82	$y = 0.2169x^{0.2511}$	0.2169	0.2511
M/PM2_76	$y = 0.1832x^{0.2677}$	0.1832	0.2677
M/PEN50/60	$y = 0.1018x^{0.3213}$	0.1018	0.3213
M/PEN80/100	$y = 0.1397x^{0.3359}$	0.1397	0.3359
MR/PM1_82	$y = 0.1522x^{0.2383}$	0.1522	0.2383
MR/PM1_76	$y = 0.1924x^{0.2049}$	0.1924	0.2049
MR/PM2_82	$y = 0.1967x^{0.2582}$	0.1967	0.2582
MR/PM2_76	$y = 0.2007x^{0.2756}$	0.2007	0.2756
MR/PEN 50/60	$y = 0.0980x^{0.353}$	0.098	0.353
MR/PEN 80/100	$y = 0.1328x^{0.3724}$	0.1328	0.3724

Table 4-26 and Figure 4-64 show the estimated number of cycle at maximum rut depth allowable in the pavement, which is 25mm. It can be seen that polymer modified bitumen mixtures are able to take more number of cycles than the conventional mixture. PM2_76 has highest number of cycle compared to the other binders, followed by PM2_82, PM1_82, PEN 50/60, PM1_76 and PEN 80/100 respectively. Quarry sand mixture has the ability to take maximum number of cycles compared to others sand mixtures, whilst marine sand mixture showed the ability to take minimum number of cycles. However marine sand has the ability to take maximum number of cycle when it mixed with PM1_76 compared to other sand types with the same type of binder. Based on the characteristic of fine aggregate as shown in the previous results, the mixture that was prepared with quarry sand showed the lowest rut depth compared to the others fine aggregate mixtures.

Table 4-26: Number of estimated cycles at maximum R_d allowable in the pavement

Binder type	**Quarry sand**	**River sand**	**Mining sand**	**Marine sand**
PM1_82	$22752{*}10^6$	$58654{*}10^5$	$15554{*}10^5$	$19824{*}10^5$
PM1_76	$19083{*}10^5$	$18052{*}10^5$	$64446{*}10^4$	$20698{*}10^6$
PM2_82	$32735{*}10^6$	$33756{*}10^5$	$16240{*}10^4$	$14100{*}10^4$
PM2_76	$80852{*}10^6$	$57099{*}10^6$	$94493{*}10^3$	$40088{*}10^3$
PEN 50/60	$34740{*}10^5$	$49319{*}10^3$	$27487{*}10^3$	$65748{*}10^2$
PEN 80/100	$85227{*}10^4$	$13326{*}10^3$	$50885{*}10^2$	$12833{*}10^2$

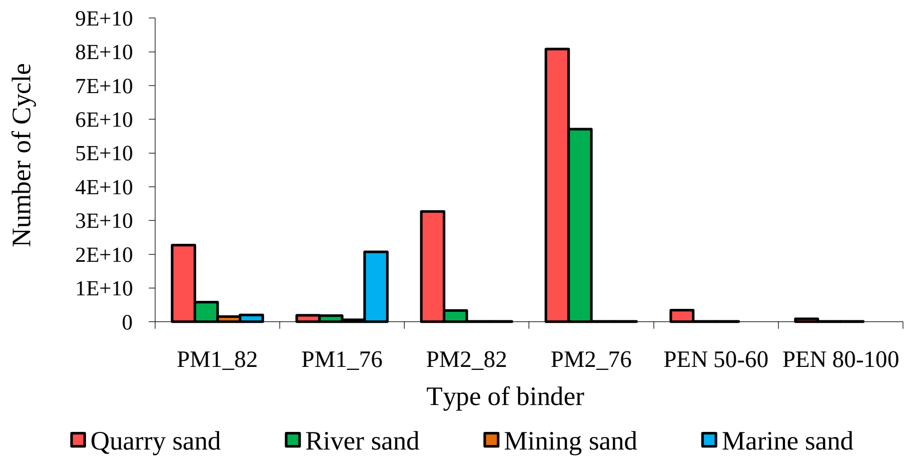


Figure 4-64: Number of cycles at maximum R_d allowable in the pavement

4.4.2 Wheel Tracking

Collop *et al.* (1995) found that wheel tracking test has been used by many researchers because it can be carried out in many shape like Georgia loaded wheel test (GLWT) and Asphalt Pavement Analyzer (APA) where the sample can also be fabricated in a shape of a beam or cylindrical. The choice of sample can also be taken from core extracted from road structure. Therefore the laboratory results can be compared with the actual performance of the road structure, because the tests were done in conditions similar to actual road conditions. The test machine was set to operate for 45 minutes at 42 cycles per minute. The wheel tracking test results are presented in Table 4-27, and the rut depth values at maximum cycle are shown in Figure 4-65.

Table 4-27: Wheel tracking results for the different sands

Type of binder	**Type of fine aggregate/ Rut depth (mm)**			
	quarry sand	**river sand**	**mining sand**	**marine sand**
PM1_82	0.04	1.4	1.8	1
PM1_76	0.6	1.9	2.5	2
PM2_82	0.3	2.2	2.8	3
PM2_76	0.5	2.5	2.7	3
PEN 50/60	2.1	3.4	3.7	3.9
PEN 80/100	3.4	4.2	4.6	4.7

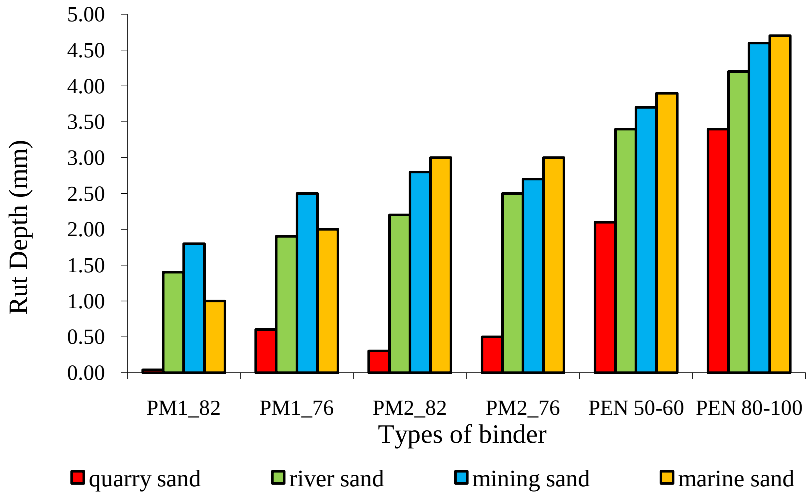


Figure 4-65: Wheel tracking results for the different sands

Fine aggregate is the primary constituent of asphaltic concrete mixture, the properties of fine aggregate are very important in influencing the rutting performance of HMA. Table 4-27 shows the results of the rate of deformation for all types of binder and all types of sand used in this study. From Table 4-27, it can be noted that quarry sand mixture exhibit higher resistance to rutting compared to the other sand types. This is because quarry sand mixture has the least rut depth as compared to the other sand mixtures. This behavior has similar trend as physical, chemical and mechanical properties of fine aggregate and as other researchers have discovered that fine aggregate could improve the rutting resistance (Lee *et al*., 1999; Park and Lee, 2002; Topal and Sengoz, 2005, 2006; Abo Qudais and Al Shweily, 2007). However, PM1_82 marine sand mixture is more resistant to rutting than PM1_82 river sand mixture and PM1_82 mining sand mixture. It is also noted that PM1_76 marine sand mixture gives more resistance than PM1_76 mining sand mixture.

From Table 4-27 it can be noticed that a mixtures containing bitumen PEN 80/100 exhibits higher rate of deformation compared to the mixture containing bitumen PEN 50/60 and polymer modified bitumen. This trend was also observed in all types of sand. Bitumen PEN 80/100 has lower viscosity and stiffness, therefore it can deform easily at long time of loading and high temperature.

It is also observed that all rut depth values are affected by the use of polymer modified bitumen, in contrast to conventional mixtures. The mixture containing polymer modified bitumen (PM1& PM2) has lower rate of deformation compared to the mixture containing conventional bitumen and this agrees with earlier study carried by Tayfur *et al.* (2007). This observation can also be explained by the viscosity of the PMB, which is higher than conventional binders. Earlier study carried by Sirin *et al*. (2006) found that more viscous binder has better rutting resistance. PM1 mixture has lower deformation rate compared to the PM2 mixture, because PM1 consist of SBS polymer modification which increases the binder elasticity at high temperatures which then lead to increase the resistance to asphalt rutting at high temperature, Lu and Isacsson (1997) obtained similar behavior of polymer modified bitumen in their work. This is due to a stiffer binder as shown by its physical properties and as indicated by the stiffness results from Van Der Poels nomograph. This however was not the case in quarry sand mixtures with PM1_76 which exhibited higher rut depth compared to PM2_76 and PM2_82. PM2_76 and PM2_82 in marine sand mixtures show the same rut depth of about 3 mm.

Improvement in the physical properties of polymer modified bitumen is required to increase binder stiffness at high pavement service temperature in order to reduce rutting. It can also be concluded that the use of polymer modified bitumen could improve properties of resistance to permanent deformation at 40^0C. As shown in Figure 4-61 quarry sand with modified binder shows highest resistance to the permanent deformation when compared with other sand types. This is followed by river sand, mining sand, and marine sand mixture respectively.

Table 4-28: Rutting resistance of 50/60 pen and 80/100 pen for different types of sand

Type of binder	Quarry sand	River sand	Mining sand	Marine sand
PEN 50/60	61.9 %	23.5 %	24.3 %	20.5 %
PEN 80/100	38 %	19 %	19.6 %	17 %

Table 4-29: Rutting resistance of 80/100 pen and PMB for different types of sand

Type of sand	**Rut depth (mm)**								
	Control 80/100	**PM1_82**		**PM1_76**		**PM2_82**		**PM2_76**	
Quarry sand	3.4	0.04	98.8%	0.6	82.4%	0.3	91.2%	0.5	85.3%
River sand	4.2	1.4	66.7%	1.9	54.8%	2.2	47.6%	2.5	40.5%
Mining sand	4.6	1.8	60.9%	2.5	45.7%	2.8	39.1%	2.7	41.3%
Marine sand	4.7	1.0	78.7%	2.0	57.4%	3.0	36.2%	3.0	36.2%

Table 4-30: Rutting resistance of 50/60 pen and PMB for different types of sand

Type of sand	**Rut depth (mm)**								
	Control 50/60	**PM1_82**		**PM1_76**		**PM2_82**		**PM2_76**	
Quarry sand	2.1	0.04	98.1%	0.6	71.4%	0.3	85.7%	0.5	76.2%
River sand	3.4	1.4	58.8%	1.9	44.1%	2.2	35.3%	2.5	26.5%
Mining sand	3.7	1.8	51.4%	2.5	32.4%	2.8	24.3%	2.7	27%
Marine sand	3.9	1.0	74.4%	2.0	48.7%	3.0	23.1%	3.0	23.1%

Comparisons of the data from Tables 4-28, 4-29 and 4-30 shown that bitumen PEN 50/60 mixtures have better resistance (61.9%) than bitumen PEN 80/100 mixtures (38%). It can also be observed that polymer modified bitumen mixtures PM1 and PM2 have the highest resistance (98.8%) than bitumen PEN 50/60 and bitumen PEN 80/100 mixtures (61.9%). PM1_82 has the highest rutting resistance, which is associated with its high hardness and stiffness as proven by the consistency test results. This is followed by PM1_76, PM2_82 and PM2_76 respectively.

Quarry sand mixture has the highest rutting resistance followed by river sand, mining sand and marine sand mixture. This observation can be related to the angular shape, rougher texture, bigger particle size, higher hardness constituent, higher oil absorption and higher shear strength of the sand.

Based on the results obtained from this study and the findings of others on rutting resistance of polymer modified mixtures containing different types of sand as fine aggregate, the following remarks are made:

- Bitumen PEN 50/60 mixture has higher rutting resistance than the Bitumen PEN 80/100 mixture. Polymer modified bitumen mixture has higher rutting resistance than the conventional mixture. PM1 has demonstrated better rutting resistance than PM2. Because PM1 being a SBS modified bitumen exhibited higher stiffness than that of PM2. This is due to verified through the consistency tests. PM1_82 has better rutting resistance than PM1_76 and PM2_82 has better rutting resistance than PM2_76. This is due to PM1_82 is harder than PM1_76, and PM2_82 harder than PM2_76 as the consistency tests proved.
- Fine aggregate that has more angular particles, rougher surface, higher shear strength and higher hardness and oil absorption value contribute to increased rutting resistance of hot mixture asphalt. A PMB mixture containing quarry sand exhibits highest rutting resistance (98.8%) followed by river sand, mining sand, and marine sand mixture respectively as compared to bitumen PEN 80/100 and PEN 50/60 mixtures.
- The wheel tracking results were founded to show similar trend with the results obtained from the dynamic creep test.

4.4.3 Beam Fatigue

Fatigue cracking is one of the primary damage mechanisms of structural components. It results from cyclic stresses that are below the ultimate tensile stress, or even the yield stress of the material. Huang (2004) said that any weak spot due to nonuniform materials properties will show up in the test results due to the existence of a constant bending movement over the middle third of the specimen. SHRP in Hartman and Gilchrist (2004) found out by using beam fatigue test the failure can initiate in an area of uniform stress between the two center loads. Besides that, this method of loading is also said to be more sensitive to mixture variables such as binder type and aggregate grading. A total of 48 beams with 24 different combinations of mixes were tested using the Universal testing machine with 3 points loading method, under the constant strain loading to determine the bituminous mixture fatigue properties. The results of the beam fatigue test are presented in Table 4-31, which are expressed as the numbers of loading cycles required to initiate a fatigue crack as a function of both the constant load applied to the beam and the maximum initial tensile bending stress. In a plot relating tensile stress and number of cycles, a linear part of the curve which represents an initial period of large stress and a part of it represents a constant rate of stress amplitude were extrapolated. The stress value corresponding to the intersection point (×) of these two extrapolations is defined as the initial stress. The same principle was applied in determining the number of cycles to failure. Figure 4-66 shows the procedure adopted to determine the initial stress and the number of cycles to failure on each beam.

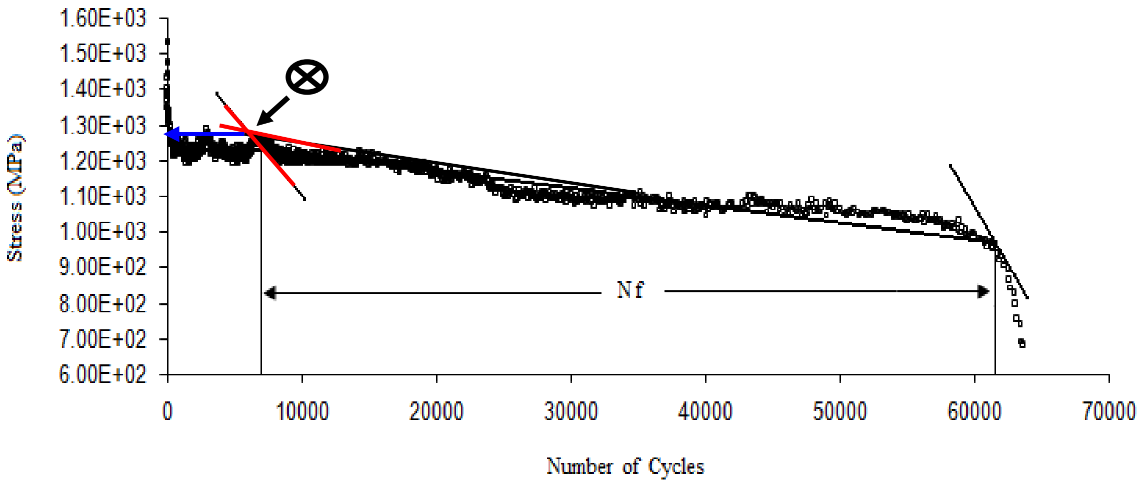


Figure 4-66: Determination of initial stress and number of cycles to failure

In this study, the beam fatigue test results were analyzed based on the number of cycles (fatigue life) for binder and fine aggregate mixtures. From Table 4-31, it can be noted that the control mixtures consisting of all types of sand with bitumen grade 80/100 are observed to have higher fatigue life (cycles) compared to the bitumen grade 50/60 mixtures. This demonstrates that bitumen grade 80/100 mixtures have a better fatigue resistance than bitumen grade 50/60 mixtures. The soft or low consistency bitumen (flexible binder) tends to absorb stress better than hard binder and relatively it shows much better fatigue resistant than the harder bitumen. This result is consistent with the physical properties results where the bitumen grade 80/100 has lower consistency (softer binder) than bitumen grade 50/60. For the modified mixtures, with almost all types of sand PM1 has higher fatigue life (cycles) compared to PM2. This means PM1 mixtures have better fatigue resistance than PM2 mixtures. The reason behind that is the use of SBS polymer in PM1 which improved the flexibility of the binder at low temperature which lead to an increase resistance to the asphalt cracking at low temperature (Lu and Isacsson, 1997). That means PM1 is more elastic and flexible than PM2, even though this result is not consistent with the physical properties results, where PM2 has lower consistency (softer) than PM1, but the presence of SBS in PM1 has given the binder more flexibility that contributes to improved resistance to fatigue cracking better than PM2 (Ahmedzade *et al*., 2007). PM1_76 mixtures can resist fatigue cracking better than PM1_82 mixtures, because the PM1_76 mixtures have higher fatigue life compared to PM1_82 mixtures for almost all types of sand. This result is consistent with the physical properties results where the PM1_76 has been found to have lower consistency (softer) than PM1_82. The same trend was observed for PM2_76 & PM2_82. This indicates that PM2_76 mixtures have longer fatigue life compared to PM2_82 mixtures. PM1_76 has the best fatigue resistance, followed by PM1_82, PM2_76, PM2_82, PEN 80/100 and PEN 50/60 respectively, except for quarry sand mixtures with PM2_76 and PM2_82 which showed lower fatigue resistance compared with other types of sand at the same binder PM2_76 & PM2_82. An analysis of the results show that polymer modified mixtures has relatively longer fatigue life when compared to unmodified mixtures.

Based on the types of sand; quarry sand mixtures have the highest fatigue life (cycle) followed by river sand, mining sand, and marine sand mixtures. This is due to the angularity of the quarry sand particles which provides a better interlocking property, as suggested by Asi (2006). The rougher surface provides a greater bonding strength between the aggregate and binder, this leads to improve the frictional resistance between particles. The distribution of large and small particles (gradation) varies from one type of sand to another. The relative size of large and small particles among the different types of sand decreases from quarry sand down to marine sand. The marine sand has very fine, small and rounded particle which improved the mixture’s workability and compaction. This is reflects positively on the mixture density and stiffness which can resist the fatigue cracking, however marine sand mixture showed lowest fatigue life because marine sand particles have lowest shear strength. Therefore the strength of material can be one of the possible ways to resist the cracking. This agrees with earlier study carried by Kim *et al.* (1999). The larger particles resist compressive stress better than the smaller particles. From the shear box test, marine sand which has smaller round particle show the lowest strength among the four types of sand and it also has the lowest fatigue resistance. The aggregate gradation has important effects on the distress resistance (Shen *et al.*, 2005). Therefore to get better resistance to fatigue deformation good distribution of particles is required. This is because inhomogeneous distribution reduces adhesion by fines leading to less fatigue resistant. These incompatible materials form poor bonding that results in low crack resistance (Abo Qudais and Al Shweily, 2007; Abo Qudais and Shatnawi, 2007). This study also investigates the gradation or size distribution of different types of sand. The good shape, size, strength, and distribution of the particles could give better fatigue resistance as found in quarry sand followed by river sand, mining sand, and marine sand mixture respectively.

Table 4-31: Fatigue life of binders and fine aggregate mixtures variations

Mixture Variations	**Initial Stress (kPa)**	**Fatigue Life (N_f)**
Q/PM1_76	0.308E+03	437610
Q/PM1_82	0.470E+03	328400
Q/PM2_76	0.880E+03	91580
Q/PM2_82	1.55E+03	73400
Q/PEN 80/100	1.26E+03	88000
Q/PEN 50/60	1.31E+03	51130
R/PM1_76	0.241E+03	241670
R/PM1_82	1.56E+03	209180
R/PM2_76	1.26E+03	146150
R/PM2_82	1.32E+03	129120
R/PEN 80/100	1.31E+03	75900
R/PEN 50/60	1.20E+03	47960
M/PM1_76	1.29E+03	122310
M/PM1_82	1.26E+03	111230
M/PM2_76	0.403E+03	104020
M/PM2_82	1.04E+03	100070
M/PEN 80/100	0.836E+03	38930
M/PEN 50/60	1.68E+03	11970
MR/PM1_76	0.203E+03	78220
MR/PM1_82	1.18E+03	75140
MR/PM2_76	1.04E+03	55900
MR/PM2_82	0.800E+03	32250
MR/PEN 80/100	1.10E+03	22960
MR/PEN 50/60	1.09E+03	10020

The results of the fatigue test are also presented in terms of the number of cycles to failure as a function of initial stress as shown in Figures 4-67 and 4-68. The results of the regression analysis, including the regression coefficients intercept (k) and (n) are given in Tables 4-32 and 4-33.

Figure 4-67 shows the fatigue curve for quarry sand, river sand, mining sand and marine sand mixtures. The results of different mixtures (same fine aggregate with different types of binder) were grouped together and a linear regression line was drawn for each group. The result indicates that mixtures containing quarry sand possess a superior fatigue properties compared to other sand types. The orders of the sand in decreasing fatigue properties are river sand, mining sand and marine sand.

The fatigue lines for beams mixtures containing different type of binders (same binder types with different fine aggregate types) are shown in Figure 4-68. The beams with polymer modified bitumen exhibit superior fatigue properties over the conventional binder’s beams. The results also show a significant increase in fatigue life for PM1 compared to PM2.

Table 4-32: Fatigue curve regression parameters of sand

Fine aggregate used	**Intercept K**	**n**	**1/n (sensitively)**
Quarry sand	9.2475E+11	2.2978	0.435
River sand	8.1575E+10	1.9414	0.515
Mining sand	9.9069E+8	1.5198	0.658
Marine sand	6.3921E+7	1.2051	0.830

Table 4-33: Fatigue curve regression parameters of binders

Binder used	**Intercept K**	**n**	**1/n (sensitively)**
PM1_76	3.6431E+21	6.341	0.158
PM1_82	3.6831E+11	2.125	0.471
PM2_76	3.0133E+10	1.855	0.539
PM2_82	6.4486E+11	2.256	0.443
PEN 80/100	1.2082E+07	0.885	1.130
PEN 50/60	1.1019E+06	0.638	1.567

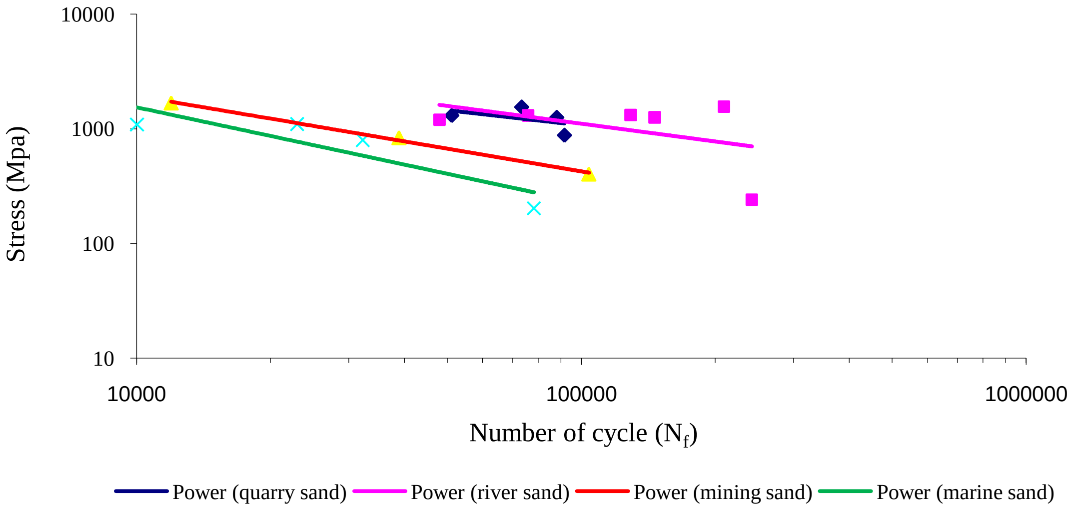


Figure 4-67: Fatigue line of types of fine aggregate used

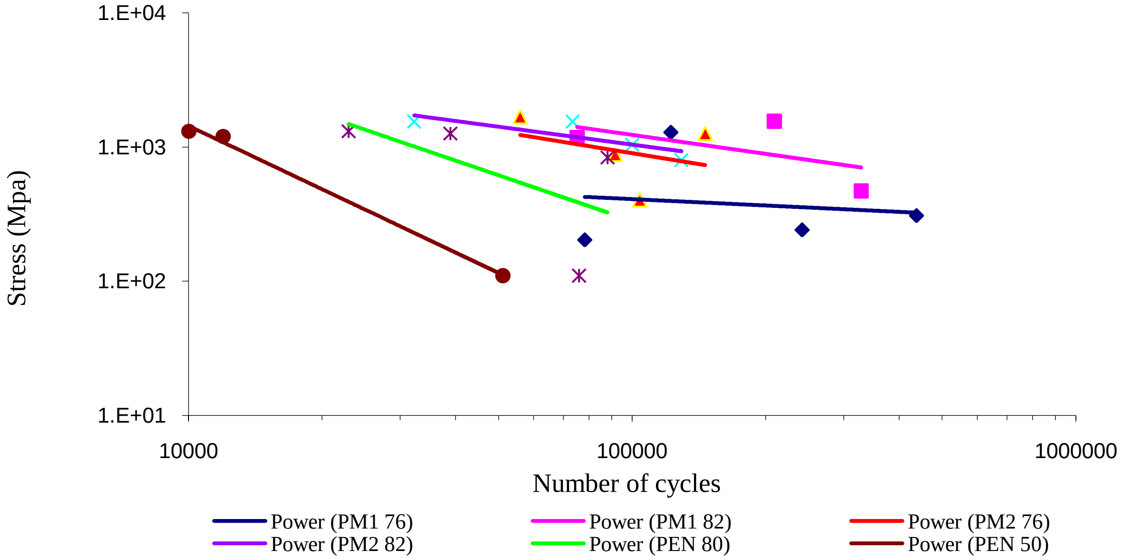


Figure 4-68: Fatigue line of types of binder used

A control strain mode was used in this study, thus the results of the beam fatigue test are expressed in the form of stress versus load cycles and are presented in Figure 4-67 and Figure 4-68. The resulting equations of stress and cycle for bituminous mixtures are shown below. The graph shows a logarithmic function and the general equation is expressed as:

$$Y = k.x^{-n} \quad (4\text{-}7)$$

$$x^{-n} = (1/k).y \quad (4\text{-}8)$$

$$x = (1/k)^{-1/n}.y^{-1/n} \quad (4\text{-}9)$$

$$x = K.\ (1/y)^n \quad \equiv \quad N_f = K.\ (1/\sigma)^n \qquad \text{4-10}$$

where: N_f - no. of cycles.

σ- stress.

K- constant.

n- equation gradient/slope factor.

The fatigue line equation for the plots of log initial stress versus log number of cycles to failure as shown in Figures 4-67 and 4-68 are as follows:

1. Quarry sand mixture $N_f = 9.2475E+11(1/\sigma)^{2.298}$ $R^2 = 0.2363$ 4-11
2. River sand mixture $N_f = 8.1575E+10(1/\sigma)^{1.941}$ $R^2 = 0.2039$ 4-12
3. Mining sand mixture $N_f = 9.9069E+8(1/\sigma)^{1.52}$ $R^2 = 0.9957$ 4-13
4. Marine sand mixture $N_f = 6.3921E+7(1/\sigma)^{1.205}$ $R^2 = 0.7703$ 4-14

1. PM1_76 $N_f = 3.6431E+21(1/\sigma)^{6.341}$ $R^2 = 0.02$ 4-15
2. PM1_82 $N_f = 3.6831E+11(1/\sigma)^{2.125}$ $R^2 = 0.3211$ 4-16
3. PM2_76 $N_f = 3.0133E+10(1/\sigma)^{1.855}$ $R^2 = 0.1209$ 4-17
4. PM2_82 $N_f = 6.4486E+11(1/\sigma)^{2.256}$ $R^2 = 0.6788$ 4-18

1. PEN 80/100 $N_f = 1.2082E+07(1/\sigma)^{0.885}$ $R^2 = 0.3579$ 4-19
2. PEN 50/60 $N_f = 1.1019E+06(1/\sigma)^{0.6376}$ $R^2 = 0.9953$ 4-20

Based on the results obtained from this study on fatigue performance of conventional and polymer modified mixtures containing different types of sand as fine aggregate, the following remarks can be made:

- The soft binder (flexible) would give mixtures that perform better fatigue resistance than the mixtures of harder binder, because softer binder is more elastic as compared to harder binder. The elastic or soft properties allow better stress absorption than harder binder. Therefore bitumen PEN 80/100 mixture has higher fatigue life compared to the bitumen PEN 50/60 mixture.
- The resistance to fatigue cracking in bituminous pavement can be enhanced by using polymer modified bitumen. Mixtures containing SBS polymer modified bitumen (PM1) exhibit better fatigue performance than the mixtures having plastomer modified bitumen (PM2).
- Strength, shape, texture, size, and good distribution of fine aggregate particles could also contribute to increase the mixture strength that is reflected in better

fatigue resistance. A mixture containing quarry sand exhibit better fatigue performance followed by river sand, mining sand and marine sand mixture.

4.5 Summary

The properties and performance of the bituminous mixtures were analyzed. The mixture properties exhibit varying response, for example a mixture that has the highest stability and lowest air voids is quarry sand mixture followed by river sand, mining sand and marine sand mixtures respectively. On the other hand, the mixture that has lowest OBC and highest density is quarry sand mixture followed by marine sand, river sand and mining sand mixtures. While a mixture that has the highest stiffness and lowest voids in mineral aggregate is quarry sand mixture followed by river sand, marine sand and mining sand mixtures.

From mixture performance, a mixture with the lowest permanent deformation obtained from the wheel tracking test and dynamic creep test, and the highest fatigue life or number of cycles to failure in the fatigue tests, is the quarry sand mixture, followed by river sand, mining sand and marine sand mixture.

Polymer modified bitumen has been successfully used to reduce permanent deformation at high temperatures 40^0C and simultaneously to resist fatigue cracking at lower temperature 20^0C. A hard binder is required to resist rutting at high temperature, whereas at low temperature this hard binder becomes brittle that can easily cause cracking. For that reason, by using polymer modified bitumen the binder elasticity at high temperature and the flexibility at low temperature can be improved. These improved properties lead to an increased resistance to asphalt mixture rutting and cracking at high and low temperature.

CHAPTER 5

5. CONCLUSION AND RECOMMENDATIONS

5.1 Conclusions

This research was conducted to determine the properties and characteristics of bituminous mixture using four different types of sand as fine aggregate. A comparison study was carried out to design the asphalt concrete mixture using both conventional and polymer modified binders. The performance of each mixture was evaluated through a series of laboratory tests. Based on the experimental results the following conclusions can be drawn:

1. The penetration and softening point results have demonstrated that the use of polymer modifier increased the stiffness of the binder at high pavement service temperature. This has the potential to improve the rutting resistance of the polymer modified bituminous mixture. At the same time, the stiffness of the binder at low pavement service temperatures is expected to decrease therefore reducing the mix potential to brittleness and cracking.

2. The scanning electron microscope (SEM) results give an indicator of fine aggregate angularity, size and texture. Quarry sand particles seem to be larger, angular in shape and rougher in texture compared to the other types of sand. Marine sand portrays the smallest particles, rounded in shape and smooth in texture.

3. The fine aggregate angularity (FAA) was measured using a simple laboratory test. The higher the values of fine aggregate angularity, the more angular are the particles with rougher surface texture. This will result in a better interlocking mechanism between the particles and thus offering better shear strength. Quarry sand was found to have the highest FAA values compared to the other sands.

4. The content of alumina (Al_2O_3) in fine aggregate affects the quality of bituminous pavement by increasing the hardness of the mix and hence the pavement becomes more resistant to permanent deformation. However, increase in hardness may cause also the pavement to become more brittle, promoting fatigue cracking to occur early in the pavement life. Also alumina has the highest ability to absorb the extensive oils in the bituminous mix. This property could decrease the rutting behavior in bituminous mixtures. From the results obtained, it can be concluded that quarry sand has the highest value of hardness and oil absorption property compared to the other types of sands, while marine sand showing the least.

5. It was also found that the content of hematite (Fe_2O_3) in fine aggregate affect the density of the bituminous mixture. This will subsequently increase the resistance to permanent deformation. Quarry sand has the highest content of hematite followed by the marine sand, river sand and mining sand respectively.

6. Quarry sand incorporated in bituminous mixtures exhibit highest density because quarry sand has the highest amount of hematite (Fe_2O_3). The hematite content has the smallest particle size compared to the other elements. Smaller particles tend to decrease the void within the bituminous mixture, which consequently will increase the density of the bituminous mixture as shown by the mixture containing quarry sand.

7. The high shear strength is a good indicator of bituminous mixture resistance to rutting. Quarry sand portrayed the highest shear strength value followed by river sand, mining sand and marine sand respectively. A good relation has been found between the angle of shear resistance (Ø) and rut depth. Therefore a fine aggregate mixtures containing sand with the highest shear resistance angle exhibited lowest rut depth.

8. The angular, rougher and high shear strength particles of fine aggregate increased stability and stiffness of the resulting mixtures. Quarry sand mixture exhibited the highest stability followed by river sand, mining sand and marine sand respectively. This is observed from fine aggregate particles shapes and surface texture and its influence on the properties of bituminous mixtures. Polymer

modified mixtures generally have higher stability results compared to the conventional mixtures, this was confirmed from the Marshall Test.

9. Among the four types of fine aggregate, quarry sand have the lowest voids filled with bitumen (VFB), followed by marine sand, river sand, and mining sand respectively. It is observed that more angular, rougher, and a bigger sized particle with fewer amounts of fines generally reduces the bitumen requirement for the mixture at the same percentage of VFB.

10. Quarry sand incorporating bituminous mixtures exhibited lowest voids in mineral aggregate (VMA) compared with the other sand type. This is because the rough or porous surface allows the binder to pass through it to fill up the voids inside the quarry particles which provides a greater bonding strength with asphalt cement and relatively the voids on it will be decreased. VMA is slightly increased for polymer modified bitumen mixtures compared to the conventional mixtures. This is because polymer modified bitumen has higher viscosity than conventional bitumen, making it difficult to flow easily into the voids. The higher viscosity of polymer can also reduce the workability of the mixture and thus become more difficult to compact which consequently leads to slightly increase the percentage of air voids in the mixture.

11. The optimum bitumen contents (OBC) for bituminous mixtures containing crushed rock fine aggregate reflect the fact that the overall aggregate grading is much coarser and more continuous than in the case for mixture containing natural sand fine aggregate. As a result, binder contents are much lower for quarry sand incorporated bituminous mixtures than those obtained for mixtures containing natural sand like marine sand, river sand, and mining sand. This is also because natural sand like mining sand has more fines (small particles). This means large or more surface area which required more amount of binder to cover all the particles. However the quarry sand has large or big particles, meaning smaller surface area which required smaller amount of binder to cover the particles. Polymer modified bituminous mixtures have a higher OBC content compared to the conventional mixtures.

12. Rutting results from the dynamic creep test and wheel tracking test showed that fine aggregate that has more angular particles, rougher surface texture, higher shear strength, higher oil absorption and higher hardness value contributes to increase the rutting resistance of hot mixture asphalt. A mixture containing quarry sand exhibits highest rutting resistance followed by river sand, mining sand, and marine sand respectively. The conventional mixtures have maximum deformation of about 4.7 mm while the modified mixtures have deformations of 3mm lesser than the conventional mixture, indicating that modified mixture is more resistant to permanent deformation.

13. Wheel tracking results were found to show similar trends with the dynamic creep results in term of rutting resistance. Wheel tracking test and dynamic creep test are both being dynamic tests. The results reinforce that both the dynamic creep and wheel tracking test are generally in agreement with each other in determining the pavement deformation characteristics of various mixtures.

14. High shear strength, angular shape, rougher texture, large size, and good distribution of particles could also contribute to the mixture strength that is reflected in better fatigue resistance. A mixture containing quarry sand exhibited the best fatigue performance as compared to mixtures incorporating the other sand types. Modification of bitumen with polymer highly increases the fatigue life of the pavement, because modified mixtures gives longer fatigue life as compared to the conventional mixture.

15. A PM1 mixture, which consists of styrene butadiene styrene (SBS) polymer as a modifier, have better resistance to permanent deformation (rutting) and fatigue cracking compared to the PM2 plastomer polymer modified mixtures. This is because SBS polymer is elastic in nature lead to increase the binder elasticity at high temperature and improves the flexibility at low temperature.

16. The bituminous mixtures using bitumen PEN 50/60 exhibited a better resistance to rutting because stiffer binder is better to resist the deformation, while the mixtures using bitumen PEN 80/100 has a better resistance to fatigue cracking, because soft binder is more elastic and is better for absorption the stress compared to the hard binder.

17. In general taking into account both the rutting and the fatigue characteristics of the mixtures, the quarry sand incorporated bituminous mixtures give the best performance. This may be due to the effect of sand properties such as angularity, surface texture, shear strength, particle size and distribution and the content of alumina and hematite.

18. The results obtained from the Marshall test, dynamic creep test, wheel tracking test and beam fatigue test provide an insight view that physical, chemical and mechanical properties of fine aggregate could improve the mixture properties and its performance.

5.2 Recommendations and Future Work

This research presents laboratory results of the influence of fine aggregate properties on the properties and performance of bituminous mixture. For better assessment of their influences on highway performance, the following recommendations are suggested:

- In this study one type of aggregate gradation was used (well graded). In order to assess the affectability of fine aggregate properties with different aggregate gradation, further tests can be carried out using various aggregate gradations.

- In this study one temperature was used for rutting test (40^0C) and fatigue test (20^0C). In order to have better understanding of the rutting and fatigue behavior, these tests can be carried out at various temperatures that are true representative of actual temperatures that the pavement may encounter during its design life.

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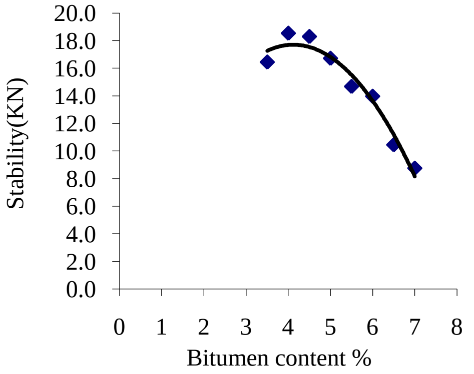
APPENDICES

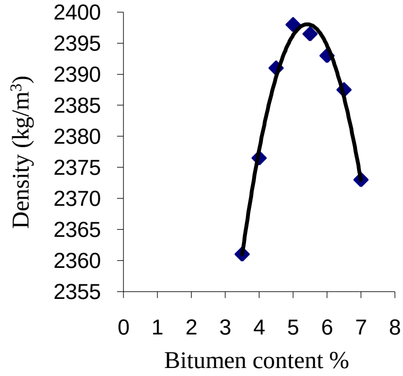
***Appendix A**: Coefficient Factor (C.F) for Adjusting Stability Values*

Volume of Specimen (cm3)	*Approximate Thickness of specimen (cm)*	*Correction Coefficient*
200-213	2.54	5.56
214-225	2.7	5
226-237	2.86	4.55
238-250	3.02	4.17
251-264	3.18	3.85
265-276	3.34	3.57
277-289	3.49	3.33
290-301	3.65	3.03
302-316	3.81	2.78
317-328	3.97	2.5
329-340	4.13	2.27
341-353	4.29	2.08
354-367	4.45	1.92
368-379	4.6	1.79
380-392	4.76	1.67
393-405	4.92	1.56
406-420	5.08	1.47
421-431	5.24	1.39
432-443	5.4	1.32
444-456	5.56	1.25
457-470	5.72	1.19
471-482	5.88	1.14
483-495	6.03	1.09
496-508	6.19	1.04
509-522	6.35	1
523-535	6.51	0.96
536-546	6.67	0.93
547-559	6.83	0.89
560-573	6.99	0.86
574-585	7.14	0.83
586-598	7.3	0.81
599-610	7.46	0.78
611-625	7.62	0.76

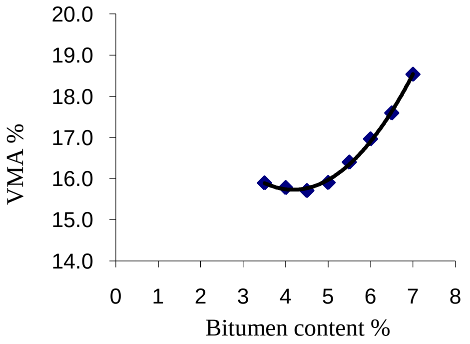
***Appendix B**: Marshall Parameters*



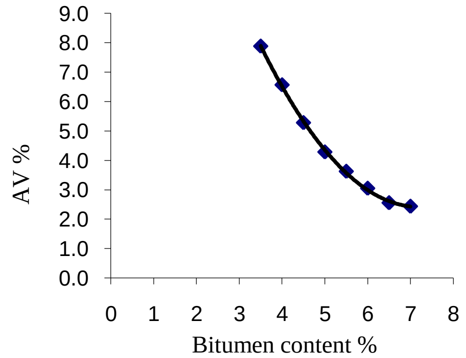
Stability of quarry sand mixture with bitumen pen 50/60



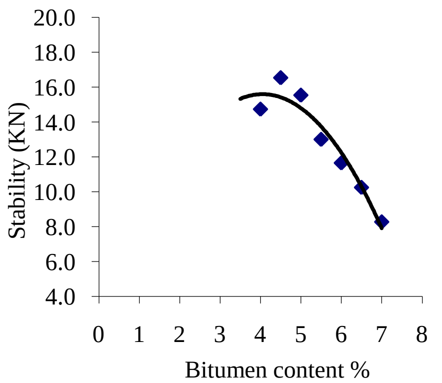
Density of quarry sand mixture with bitumen pen 50/60



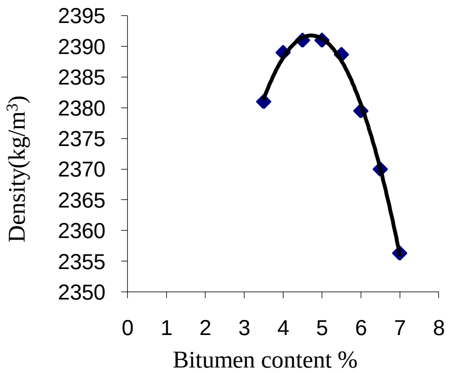
VMA of quarry sand mixture with bitumen pen 50/60



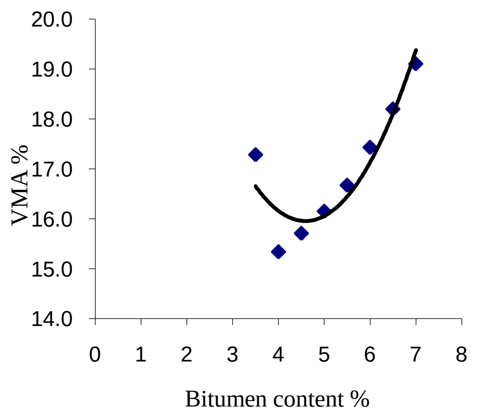
AV of quarry sand mixture with bitumen pen 50/60



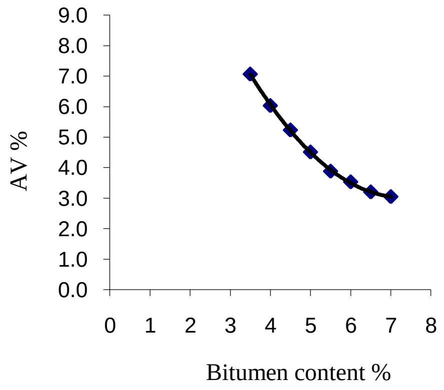
Stability of quarry sand mixture with Polymer modified bitumen PM1_76



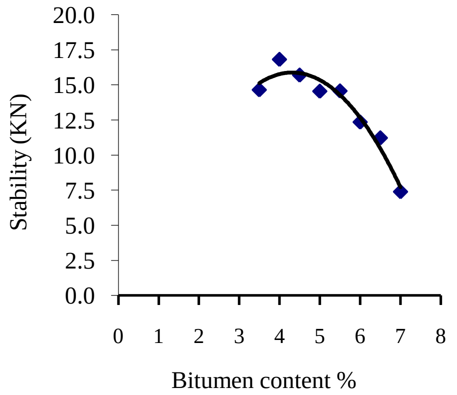
Density of quarry sand mixture with Polymer modified bitumen PM1_76



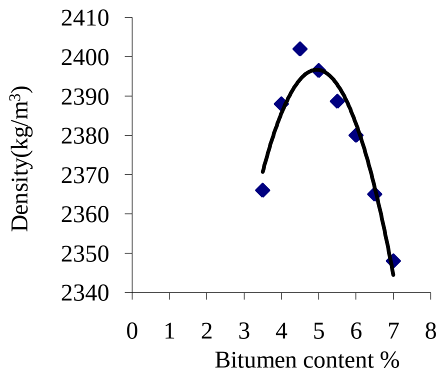
VMA of quarry sand mixture with
Polymer modified bitumen PM1_76



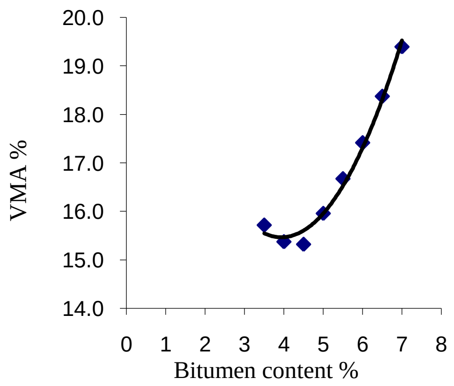
AV of quarry sand mixture with
Polymer modified bitumen PM1_76



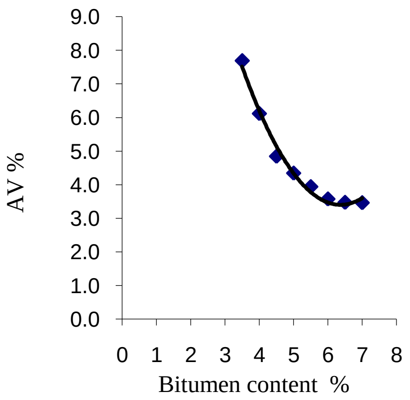
Stability of quarry sand mixture with
Polymer modified bitumen PM2_82



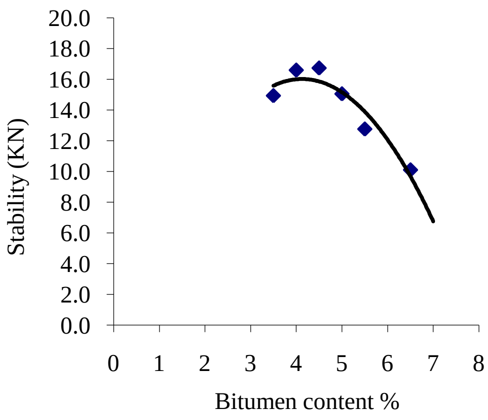
Density of quarry sand mixture with
Polymer modified bitumen PM2_82



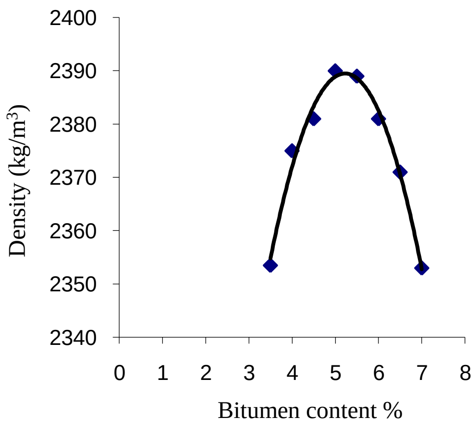
VMA of quarry sand mixture with
Polymer modified bitumen PM2_82



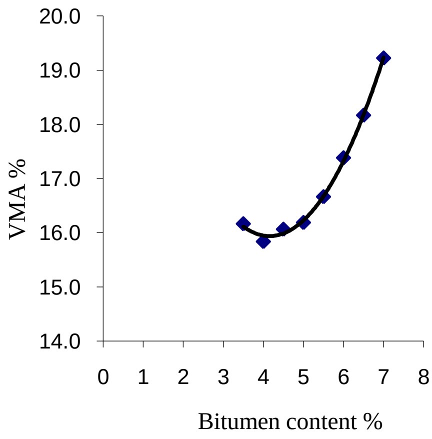
AV of quarry sand mixture with
Polymer modified bitumen PM2_82



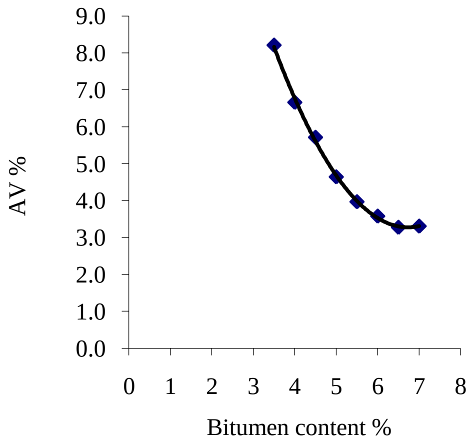
Stability of quarry sand mixture with Polymer modified bitumen PM2_76



Density of quarry sand mixture with Polymer modified bitumen PM2_76



VMA of quarry sand mixture with Polymer modified bitumen PM2_76



AV of quarry sand mixture with Polymer modified bitumen PM2_76

Appendix C: *Marshall Data for Bituminous Mixture Based on Type of Binder and Type of Fine Aggregate*

Type of sand	Quarry sand						River sand					
	PM1_82	***PM1_76***	***PM2_82***	***PM2_76***	***50-60***	***80-100***	***PM1_82***	***PM1_76***	***PM2_82***	***PM2_76***	***50-60***	***80-100***
Density (g/cm^3)	2. 380	2. 391	2. 394	2. 386	2. 391	2. 386	2. 340	2. 355	2. 358	2. 360	2. 354	2. 339
VMA %	15. 85	16	15. 1	16	15. 8	16. 16	17. 83	17. 35	17. 13	17. 06	17. 18	17. 76
VFA %	61. 5	67	67	67. 5	68	69	69	72	71	71	69	68
AV %	6. 1	5. 2	5. 15	5. 2	5. 1	5	5. 45	4. 85	5. 00	4. 95	5. 30	5. 75
Stability (KN)	14. 80	15. 4	15. 800	15. 600	17. 300	14. 900	10. 598	10. 70	10. 950	10. 200	9. 5	7. 8
Flow (mm)	2. 01	2. 55	2. 45	2.60	2. 65	2. 70	1. 950	2. 100	2.080	2. 100	1. 850	1. 810
Stiffness (KN/mm)	7.1	5.9	6.4	5.85	6.28	4.7	5.6	5.05	5.13	4.75	5.05	4.4
OBC %	4. 19	4. 53	4. 48	4.69	4. 59	4. 80	5. 40	5. 45	5. 31	5. 33	5. 20	5. 30

Marshall Data for Bituminous Mixture Based on Type of Binder and Type of Fine Aggregate

Type of sand	Mining sand						Marine sand					
	PM1_82	***PM1_76***	***PM2_82***	***PM2_76***	***50-60***	***80-100***	***PM1_82***	***PM1_76***	***PM2_82***	***PM2_76***	***50-60***	***80-100***
Density (g/cm^3)	2. 324	2. 318	2. 307	2. 308	2. 342	2. 317	2. 380	2. 379	2. 372	2. 372	2. 376	2. 372
VMA %	19. 54	19. 6	20. 24	20. 38	18.61	19.6	17. 1	17. 1	17. 3	17. 4	17. 2	17. 28
VFA %	73	70	70. 5	72	71. 5	68. 5	68	67.7	65.8	67.	67	66
AV %	5. 3	5. 85	6	5. 7	5. 25	6. 1	5. 5	5. 5	6	5.7	5. 7	6
Stability (KN)	8. 80	8. 750	8. 25	7. 250	9. 140	7. 13	8.00	8. 20	8.00	6.20	6.70	5. 325
Flow (mm)	2. 56	2. 5	2. 56	2. 33	1. 90	2. 06	1. 72	1. 90	1. 52	1.75	1. 40	1. 15
Stiffness (KN/mm)	3.55	3.6	3.35	3.13	4.85	3.53	4.6	4.2	4.88	3.55	4.6	4.4
OBC %	6. 33	6. 06	6. 40	6. 61	5. 86	6. 14	5. 04	5. 03	4.9	5. 09	5.04	4.9