THE PERFORMANCE OF ASPHALTIC CONCRETE USING VARIOUS FILLERS

NOOR SABRINA ZAHARUDIN

CIVIL ENGINEERING
UNIVERSITI TEKNOLOGI PETRONAS
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Noor Sabrina binti Zaharudin

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Universiti Teknologi PETRONAS
Bandar Seri Iskandar
31750 Tronoh
Perak Darul Ridzuan

CERTIFICATION OF APPROVAL

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(Assoc. Prof. Dr. Madzlan Napiah)

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CERTIFICATION OF ORIGINALITY

This is so to certify that I am responsible for the work submitted in this project, that the original work is my own concept as specified in the references and acknowledgements. And that the original work contained herein have not been undertaken or done by unspecified sources or persons

(NOOR SABRINA BINTI ZAHARUDIN)

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ABSTRACT

Asphalt concretes are made of asphalt binders and aggregates. Although asphalt cement is predominantly considered the binder holding the aggregates together, the actual product used to connect larger-size aggregate particles is the asphalt mineral filler mastics. It improves the resistance to permanent deformation in asphalt concrete mixtures by improvement of rheological properties of asphalt binders through a filler effect, and by acting as a microcrack arrester as well as improving the bonding interaction between asphalt binder and aggregates. Samples having different types of filler were prepared and optimum binder content was determined by Marshall Test procedure. Optimum filler content was determined considering the filler/bitumen ratio and filler ratio. Creep test, was carried out to determine the mixture properties and performance. Utilization of waste material as filler material shall reduce cost and contributes to the conservation of the environment without compromising the performance of the asphaltic concrete.

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ABBREVIATION AND NOMENCLATURES

OPC – Ordinary Portland Cement

PFA – Pulverized Fuel Ash

RHA – Rice Husk Ash

CHAPTER 1

INTRODUCTION

1. 1 Background of Study

In recent years, increased traffic levels, larger and heavier trucks, new axle designs, and increased tyre pressures (due to radial tyre design) have added into the already severe demands of loads and environment on the highway system. Most specifications and design methods for asphalt and mixtures are empirically based and over half century old.

In Malaysia, hot-mixed bituminous mixtures are used for binder and wearing course. Asphaltic pavements take precedence over concrete pavements due to its ease of construction, material availability and most importantly, low costs. The compositions are designed based on the Standard Marshall Test procedure and consists of well graded mixture of coarse aggregates, fine aggregates and filler, bound together with bitumen. Their stability derives both from the interlocking of well-graded aggregates and from the cohesion provided by the bitumen binder. Thus, care must be taken in the selection of materials, gradation and bitumen content so as to obtain a mix with desirable stability, durability and sufficient skid resistance.

Clause 6.2.3 of the JKR Manual on Pavement Design states that mineral filler shall be Portland cement which fulfils the specified grading requirements. Mineral fillers have traditionally been used in asphalt mixtures to fill the voids between larger aggregate particles. Generally the aggregate passing the No. 200 sieve has been called filler. The amount of filler material is specified as a percentage of the weight of the mix, and becomes part of the mixture design. The motivation for using filler in asphaltic mixtures is based upon the following concerns of the user agencies (JKR Manual 5/85):

- Reducing initial costs
- Stiffening asphalt mixtures
- Improving pavement performance

One way to evaluate the performance capabilities of asphalt pavements is by conducting laboratory tests on the samples, either cored from existing roads or mixed in the lab. In this study, a destructive test, which is the creep test, will be carried on samples mixed in the laboratory.

1.2 Problem Statement

The most commonly used filler in asphalt mixture is Ordinary Portland Cement (OPC). The use of OPC is expensive; therefore, there is a need to find alternative filler material, most preferably cheaper waste materials which can replace OPC filler without compromising the performance of the asphalt concrete.

1.3 Objective and Scope of Study

This study will focus on the feasibility of potential alternative materials from waste materials and by-products which can be used as filler in asphalt concrete mixes to replace Ordinary Portland cement. The effects of adding these by-products on the permanent deformation will be evaluated.

1.3.1 Scope of Study

i. **Literature Review**
   This is a continuous process throughout the duration of the project. Research of relevant information regarding the project is obtained from journals, magazine articles and reference books. This is used as a guideline in further understanding the properties and scope of the project.

ii. **Laboratory tests**
   Marshall samples were prepared to determine the optimum binder content of the mixes. Three samples ranging from 4% to 7% were prepared using different fillers, namely Ordinary Portland Cement,

Pulverized Fuel Ash and Rice Husk Ash. The optimum binder content mixes are then tested using Creep Test.

iii. **Data Analysis**

Using the optimum binder content samples, the performance of mixes using various fillers is tested using the Creep Test to analyze the performance. The rut depth for every cycle, N is obtained. The mix type that can withstand the longest cycle at 15mm yields the best performance.

1.3.2 Feasibility of the Project within the Scope and Time Frame

The project takes 2 semesters to be completed. Within this time frame given, the project needs to evaluate the performance of asphalt concrete mixes with various types of fillers and mixtures. The first semester will focus mainly on research and literature review while the actual laboratory testing will be carried out during the second semester. By the end of this project, the suitability of filler materials and their best composition that affects the performance of the asphalt concrete will be known.

1.4 Assumptions

This study focuses mainly on the performance of asphaltic concrete by using Ordinary Portland Cement, Pulverized Fuel Ash and Rice Husk Ash filler in terms of rutting by conducting the creep test. The mix design is used to determine the optimum binder/aggregate content used for testing purposes. All other factors, including mix temperature, material, binder and aggregate types used will be disregarded. This research will also confined to the asphalt layer (wearing course) of the pavement, with total disregard to the base and subbase layers of the pavement.

CHAPTER 2

LITERATURE REVIEW AND THEORY

In this chapter, the general concept of asphaltic concrete pavements and its composition materials, namely mineral fillers, aggregates and binder along with reviews on the use of Ordinary Portland Cement, Pulverized Fuel Ash and Rice Husk Ash as fillers in asphalt pavements.

2.1 Aggregates

The proper selection of materials is one of the most important tasks in developing an asphalt mixture that shows improved resistance to permanent deformation. Results of previous investigations to determine the type of aggregates that provide better resistance to permanent deformation, show that angular aggregates play a major role in contributing to greater stability (resistance to deformation and plastic flow) of hot mix asphalt concrete. These studies show that angular aggregates, through interlocking and shear resistance, can improve mixture shear strength that is a measure of loading bearing capacity and resistance to rutting and shoving (horizontal displacement of an asphalt mixture)

Marshall stability of mixtures increased consistently with an increase in the amount of crushed coarse aggregate (Figure 1), whether the crushed aggregate particles were limestone or river gravel. A significant influence (according to a paired t-test) of the crushed aggregate on Marshall stability values was observed. In this research, both the long term static and cyclic creep test (unconfined compression) tests were sensitive to changes in coarse aggregate surface characteristics. Both showed a decrease in creep and permanent deformation and an increase in the amount of crushed coarse aggregates in asphalt mixtures of the same gradation (Zollinger et al. 1996)

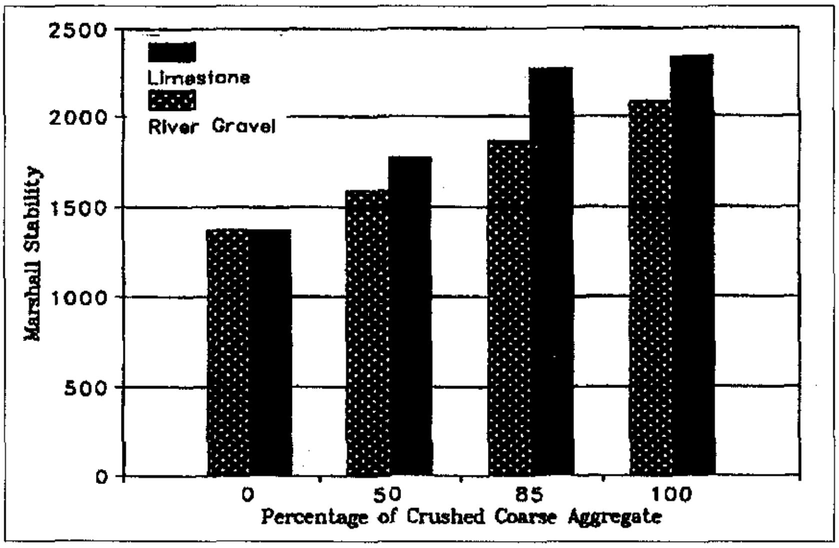

Figure 1: Marshall Stability versus Percentage of Crushed Aggregates

Crushed gravel, which is the product of crusher run, can be made of many different types of mineral particles; limestone, sandstone and granite, of which granite is the preferred choice for construction purposes due to its durability, strength and hydrophobic characteristics. Coarse aggregates function to provide stability due to its interlocking behaviour and acts to withstand most of the traffic loads. The shape and texture affect the stability of any mix. Therefore, good aggregates that are hard, round shaped with an overall angular shaped and rough surface texture.

Fine aggregates enhance the stability of the mix by filling up the voids left out by the composition of coarse aggregates. Fine aggregates should be of good gradation between 2.36mm to 0.075mm sieve sizes. Smaller size fine aggregates increase the surface area and this enables the aggregate mix to contain higher content of bitumen, thus enhancing the binding force of the mix.

2.2 Mineral Filler

Fillers are generally added into asphalt pavement mixes to improve the stiffness and load carrying capabilities. Fillers, for most part, are inert but their physical properties influence the performance of asphalt mixtures. These properties include surface area, particle shape, particle size, packing arrangement and void volume. Void volume is decidedly a predominant factor in the design of filler modified asphalt concrete mixes.

The size distribution of the particles passing the 75μm can influence the stiffness of the asphalt binder in the mixture. If the majority of the mineral filler is smaller than 20μm, the asphalt binder portion of the mixture will become stiffer. Size distribution larger than 20μm does not by itself have stiffening effect on the asphalt binder (Lavin et al. 2003)

When mineral fillers are mixed with asphalt cements, the resulting asphalt-mineral filler mastics are a *particulate-filled composite*. The asphalt cement is the matrix and the mineral filler is the particulate phase. Researches have analyzed various particulate-filled composites under various conditions. Two general limits can be summarized for the tensile strength of the particulate-filled composites as shown in Figure 2. The upper bound response represents strong adhesion between matrix and filler, while the lower bound response indicates weak or no adhesion between these two phases. In this study, the addition of mineral fillers caused an increase in tensile strength at all three test temperatures, -10°C, -15 °C and -20 °C. The increase in tensile strength with increasing amount of filler implies that there is good adhesion between asphalt cements and mineral fillers. With this good adhesion, asphalt binders are able to hold mineral filler particles together during loading. As a result, the tensile strength of the whole system increases (Figure 2). It has been shown that if there is good adhesion in the particulate-filled composite, mineral fillers carry parts of tensile loads. When the asphalt-mineral filler mastics are tested under the direct tension, the stress is transmitted from the matrix to the filler. Parts of the tensile stress can be held by the filler. More filler can share more tensile stresses with the matrix; therefore, the tensile strength increases with increasing filler volume concentration. It appears that the mechanical bonding between mineral fillers and asphalt binders play an important role in increasing the tensile strength of asphalt-mineral filler mastics. (Chen et al. 1998)

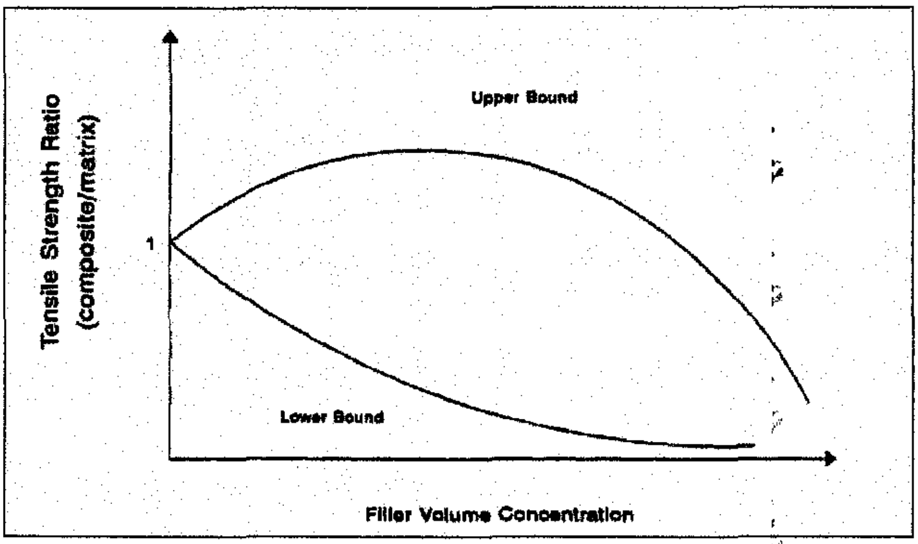

Figure 2: Effects of fillers on tensile properties on particulate-filled composites

2.2.1 Gradation

The mineral filler shall meet the following gradation requirements:

Total passing No. 30 [600 µm] sieve.. 100%

Total passing No. 80 [180 µm] sieve........................ 95% (minimum)

Total passing No. 200 [75 µm] sieve........................ 65% (minimum)

2.2.2 Ordinary Portland Cement

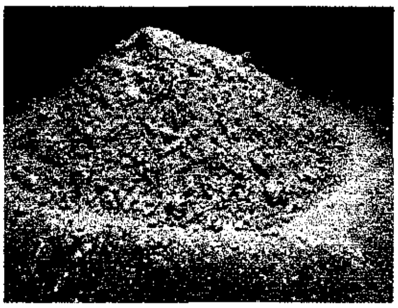

Figure 3: Ordinary Portland Cement

In this project, OPC (Figure 3) is used as control to evaluate the performance of other types of filler. Generally, OPC is characterized by high CaO, K_2O, Na_2O, and Cl_2 contents. The use of OPC as filler material is common and possesses no environmental risks. OPC is also added to the combined aggregate for asphaltic concrete to serve as adhesion and anti-stripping agent. The typical chemical composition of Ordinary Portland Cement is presented in Table 1.

Table 1. Typical Chemial Composition of OPC

Component	*Percentage by Weight (%)*
CaO	64.40
SiO_2	22.60
Al_2O_3	4.30
Fe_2O_3	2.40
MgO	2.10
Na_2O	0.60
K_2O	0.60
SO_3	2.30

2.3 Pulverized Fuel Ash (PFA)

Figure 4: Pulverized Fuel Ash

PFA (Figure 4), also known as fly ash is a well known industrial waste material produced from the combustion of coal. It mainly contains the inorganic part of the coal that is fused during the combustion phase and subsequently solidified and collected by electrostatic precipitation. Its particles are spherical and generally of greater fineness than cement particles. Physically, PFA is a fine powder which bears a close resemblance to Portland cement in general fineness and usually also in color.PFA is composed mainly of oxides of silicon, aluminium and iron which combine to form complex amorphous and crystalline compounds. It is the silica that facilitates the pozzolanic reactions. The typical chemical composition of PFA is presented in Table 2

Table 2 Typical Chemial Composition of PFA

Component	***Percentage by Weight (%)***
CaO	2.3
SiO_2	50
Al_2O_3	28
Fe_2O_3	10.5
MgO	1.6
Na_2O	1.2
K_2O	3.6
TiO_2	1.0
SO_3	0.7
Cl	0.08

Research conducted over many years has determined that fly ash is a suitable mineral filler material. The earliest study of this application dates back to 1931, when the Detroit Edison Company compared the physical properties of fly ash with those of limestone dust. Fly ash was shown to have comparable physical properties to limestone dust, to possess good void filling characteristics, and to be hydrophobic, meaning it sheds water easily, thus reducing the potential for asphalt stripping.

The Federal Highway Administration, FHWA, compared the retained strength of asphalt mixes containing various mineral fillers by means of the immersion-compression test. This test is used as an indicator to evaluate resistance to stripping. Four sources of fly ash were evaluated, along with silica dust, limestone dust, mica dust, and traprock dust. Similarly, North Dakota State University compared lignite fly ash as mineral filler with hydrated lime and crusher dust. In both investigations, mixes containing the fly ash fillers had higher retained strengths than the other filler sources tested, indicating that fly ash fillers can be expected to provide excellent resistance to stripping.

Further confirmation of the beneficial anti-stripping characteristics of fly ash mineral fillers was provided from an investigation of two western coal fly ashes (one Class C and one Class F) in combination with, or as a replacement for, Portland cement or hydrated lime. All mixes which contained fly ash showed comparable or improved retained strengths in the immersion-compression test using two different sources of aggregate. A study of Texas lignite fly ash indicated that the use of these fly ashes as mineral filler retards the rate of age hardening of asphalt cement. The high lime content of these fly ashes also appears to be particularly beneficial as an anti-stripping agent for polish-susceptible aggregates.

As quoted from the research done by Ali et al. 1996, the use of PFA as mineral filler is not a new concept. It is found that Class F PFA provided superior results in retained compressive strength for asphalt concrete specimens immersed in water. The addition of 4% Class C PFA produced the highest stability and flow, while specimens containing PFA produced lower air voids. It is also reported that PFA improved the stability after immersion in water. PFA when compared to other fillers such as crushed dust and kaolin clay provided the highest stability at 2% filler content. The highest retained strengths after immersion were produced by mixes with 2% PFA and 5% asphalt, and 6% PFA and 4% asphalt.

The use of PFA was proposed to make a stiffer mixture, one less susceptible to moisture damage. It was found that the addition of Class C PFA increased permeability, stiffness and compressive strength values. Test sections of recycled mixtures containing PFA are presently performing well with only minor rutting and cracking problems.

PFA, when used as a mineral was beneficial in terms of improved strength and stripping resistance. Mechanical properties and moisture damage results indicated that the use of 2% PFA improved the resilient modulus of the mix at high and low temperatures (Figure 5). The results also indicated that stripping resistance of the mix was increased with the addition of PFA. There was no indication that the addition of PFA in asphalt concrete mix reduced pavement distress and improved field performance of asphalt pavement. VESYS performance prediction results showed that for fatigue cracking index, the use of PFA in asphalt concrete mixture did not significantly reduce field performance in terms of rut depth and present serviceability index. However, pavement constructed with PFA asphalt concrete will experience moderate and severe cracking after 10 years of service compared to light cracking for conventional asphalt concrete pavement. (Ali et al. 1996)

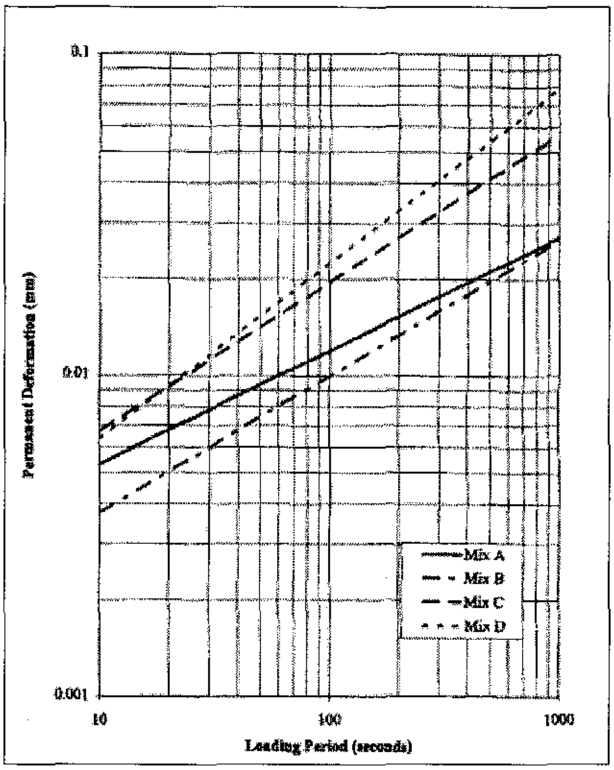

Figure 5: Permanent Deformation at 40°C

2.2.4 Rice Husk Ash (RHA)

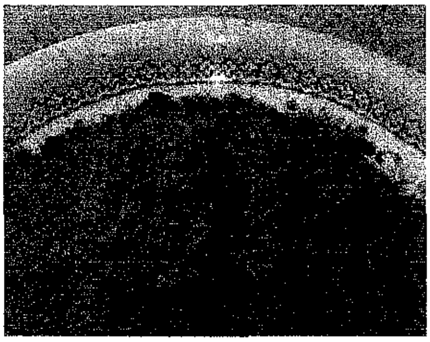

Figure 6: Rice Husk Ash

Rice husk ash (RHA) (Figure 6) was obtained by burning RH in a furnace with a controlled temperature in order to establish the optimum burning temperature and burning time. Grinding of RHA aims to achieve the best specific surface area. It was found that the most convenient and economical temperature required for conversion of the RH into ash was 600°C for 3 hours. The RHA that was used had a specific surface area of 5.6 x 10^6 mm2 /g, and the unite weight was 2.06 x 10^3 kg/m3. The chemical composition of the RHA was 87.0% SiO_2, 1.75% Al_2O_3, 2.5% Fe_2O_3, 2.5% CaO, 2.3% MgO, and 2.5% K_2O. The silica content of the ash was derived from the amorphous silica present in the cellular structure of the husks. X-ray diffraction of the RHA showed that the RHA contained mainly amorphous materials with a very small amount of crystallized quartz (Sakr et al. 2006)

When burnt under controlled conditions, the RHA is highly pozzolanic and suitable for use in lime-pozzolana mixes and for Portland cement replacement. When burnt in an uncontrolled manner, the ash, which is essentially silica, is converted to crystalline forms and is less reactive. Table 2 shows the typical chemical content of RHA.

RHA has been widely used in the concrete industry as cement replacement material. Their characteristic which resembles OPC in fineness (passing No. 200 sieve) makes it a suitable candidate to be used as mineral filler in asphalt concrete mixtures. Advantages of application of PFA in road engineering are well known for its environmental benefits, since there are no or less deposit areas for stockpiling are necessary, technical, due to hydraulic properties of the material, economic, due to low cost of the by-product and practical, especially in countries where aggregates are rare and cement is expensive.

Table 3. Typical Chemical Composition of RHA

Component	***Percentage by weight (%)***
CaO	0.41
SiO_2	92.15
Al_2O_3	0.41
Fe_2O_3	0.21
MgO	0.45
Na_2O	0.08
K_2O	2.31

2.3 Pavement Performance Factors

From the research done by Zhiming et al. 2002, in investigating effects of inorganic and polymer filler on tertiary damage development in asphalt mixtures, one of the mechanisms for permanent deformation in the asphalt mix is growth of microcracks. Besides plastic flow, the initiation and growth of microcracks in the asphalt mix under repeated loading is a cause of permanent deformation in pavements. When load is applied to the asphalt mixture, it can experience consolidation and strain hardening. If no microcracking or microdamage occurs, the plot of the logarithmic rate of change of permanent strain versus the logarithm number of loading cycles should be a straight line. However, many asphalt mixes do not follow this predicted pattern. They deviate upward from the straight line with an increasing number of loading cycles. This upward departure from the straight line indicates that more damage than predicted. The number of loading applications at which the departure occurs is a sign that damage has been done to the material due to microcracking.

In other investigation of evaluating the use of marble waste dust in the mixture of asphaltic concrete by Karashin and Terzi et al. 2005, it states that different filler materials may have different mechanical properties in the asphalt mixture. Dukatz and Anderson have investigated eight different filler materials to investigate the mechanical properties of asphalt and they found that different filler materials have different effects on stiffness and had almost no effect Marshall Stability and void ratio. Puzinauskas has investigated that mixture of filler-asphalt. However, Mogawer and Stuart investigated eight different filler materials which were known in Europe and they found that good quality fillers and poor quality fillers did not affect the performance of mixtures. Many tests were carried out on asphalt mixtures to investigate the filler behavior.

Thus it can be expected that the results of this project, i.e. to evaluate the performance of asphaltic concrete by using various fillers will yield similar results to the above literature reviews. Laboratory tests that relate to field performance will be used. These procedures and properties will consider two basic modes of distress: rutting and fatigue.

2.3.1 Rutting

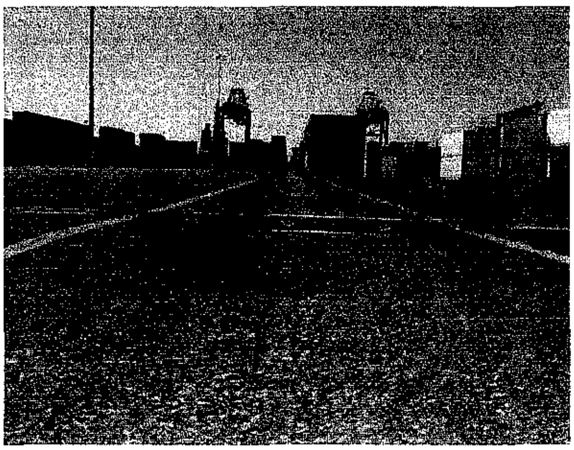

Figure 7: Rutting in asphalt pavement caused by channelized loading

Rutting (Figure 7) is a distress characterized by an accumulation of small amounts of deformation that occurs during each loading cycle. Rutting of asphalt pavements has a major impact on pavement performance. It reduces the useful service life of the pavement and poses a potential safety hazard because the ruts can trap enough water to cause hydroplaning and ice accumulation. Repeated loading of pavement layers at higher pavement temperatures is one of the reasons for the accumulation of permanent strain. At high pavement temperatures the hot mix asphalt (HMA) becomes softer due to reduction in viscosity. The softening of the asphalt binder increases the rutting potential of the pavement. Using harder asphalt binder can increase the permanent deformation resistance of the mix, but this creates problems that are related to lack of flexibility and cracking at lower pavement temperatures. (Zhiming et al. 2002)

The three constituents of HMA are aggregate, binder, and air. All three can have an effect on rutting of an HMA pavement. Aggregate makes up about 90 percent of a dense-graded HMA. The shape and texture of the aggregate can influence the performance of the mixture. In general, a rough-textured cubical-shaped aggregate performs better than a smooth, rounded aggregate. The rougher texture and cubical shape aid in providing aggregate interlock. This aggregate interlock reduces the potential for rutting as movement of the aggregate under loading is reduced by the interlocking mechanism. The binder is also an important factor in rutting. At higher temperatures, the asphalt binder becomes less viscous. This lower viscosity produces a less stiff pavement that can be susceptible to lateral movement attributable to traffic loads. Compaction during construction is a vital part of producing a more durable pavement. The final constituent is air. If a mixture has a high air content, it can be susceptible to rutting in the sense that it will compact more under traffic loading. However, if the air content is too low, there is probably too much binder in the mixture. Too much binder produces a less stiff pavement and increases the probability of rutting

Other factors that influence rutting in HMA pavements include truck speed, contact pressure, HMA layer thickness, and truck wheel wander. As truck speeds are decreased on an HMA pavement, the stresses are increased because of longer pavement contact times. These higher stresses increase the probability of rutting. The contact pressure also influences the performance of the pavement. Higher tire pressures create higher stresses in the pavement. A thicker HMA layer is better able to resist rutting in the sense that the layer is usually stiffer. Finally, truck wheel wander can influence rutting. The increase in wheel wander can increase the amount and distance of lateral movement in the pavement. Excessive wheel wander has the potential to create wider and possibly deeper ruts in an HMA pavement (Maupin et al. 2006) (Figure 1 & 2 –Appendix)

2.3.2 Fatigue

Fatigue cracking is caused by repeated loading of asphaltic layers in the pavement and has been the subject of detailed research over the years. It is manifested as a network of cracks in the wheel tracks. The interaction between fatigue characteristics and elastic stiffness of the mixture is a crucial one. It is clear that the asphalt cement has a dominant influence on both properties. The fatigue life resistance of a bituminous mixture is defined as its ability to respond to repeated traffic loading under the prevailing environmental conditions without significant cracking or premature failure being induced. Damage in asphalt pavements, due to repetitive stresses and strains caused by both traffic loading and environmental factors, can manifest itself as fatigue cracking which is considered as a primary distress mechanism in asphalt pavements. The fatigue characteristics of asphalt are, therefore, an important structural pavement design parameter.

A typical fatigue process for asphalt mixtures can be characterized by three distinct phases denoted Phase I, II and III, respectively. The first phase is characterized by a rapid increase in sample temperature. During this phase, the stiffness of the sample decreases due to both fatigue damage and temperature increase. The effect of heating is very difficult to separate from the fatigue damage during Phase I and therefore difficult to analyze. Phase II is characterized by a quasi-linear decrease in stiffness. At the beginning of the Phase III, the sample starts to collapse, often due to increased non-uniformity in strain field. The behaviour during such a three-step evolution of the stiffness can be very different for different temperatures and binder stiffness used.

2.3.3 Creep Test

Type of test that can potentially be used to predict performance is the uniaxial test. The four types of test that were considered were creep, repeated load permanent deformation, dynamic modulus, and strength test. One of the biggest problems with this type of test is its questionable ability to predict performance because of the amount of load and temperature that can be used for testing. It is believed that the temperature and stress applied in the laboratory should be similar to that which the mixes are actually subjected to in the field. The load and/or temperature must be decreased significantly from that expected in the field; otherwise these tests cannot be conducted without immediate failure of the samples. The test is simple and inexpensive to conduct when using static loads, however, the complexity and cost increase considerably when dynamic loads are required. There is little information available for these tests that correlate test results to performance. Due to the lack of performance information, none of these tests are recommended for immediate adoption to predict permanent deformation; however some of these tests are being studied and may prove to be acceptable when this study is completed.

Another type of test that was considered is the triaxial test. The difference between this series of tests and the uniaxial tests discussed above is that the triaxial tests include confining pressure. Applying a confining pressure allows one to more closely duplicate the in-place pressure and temperature without prematurely failing the test sample. There is some rutting information available for the confined creep and repeated load tests. There is less information available for the dynamic modulus and strength tests. These traxial tests are complicated somewhat by the requirement for a triaxial cell but this does not preclude the use of this test. The confined creep and repeated load tests have been used and do have some potential in predicting rutting. The confined creep test is simple and easy, but the correlation with rutting is not very good. It has been recognized widely that the confined repeated load deformation test is better correlated with performance but more difficult to conduct. At this time these tests are not recommended for immediate adoption. At the conclusion of NCHRP 9-19, sufficient data will be available to adopt one or more of these tests if appropriate and to provide details concerning test procedures. (Brown et al. 2001) *(refer Appendix - Table 1)*

CHAPTER 3

METHODOLOGY & PROJECT WORK

There several steps adopted in completing this project starting from research, pre-laboratory work, sample preparation, testing and data collection and analysis.

3.1 Research

The first part of this project is to conduct a literature review on the materials that will be used as potential filler, laboratory procedures involved and testing. This step involves and in-depth research from various published journals, books and websites.

3.2 Pre-laboratory Works

Before the actual laboratory work is carried out, sample preparation is important to obtain reliable results. In this project, a sieve analysis is carried out to obtain the best aggregate – binder composition and to ensure that it complies with the gradation limit requirements.

The materials, which include coarse aggregates, fine aggregates, filler and bitumen, are prepared before it can be used for sample preparation. Coarse and fine aggregates are sieved and thoroughly washed and dried for 24 hours before it can be used. The filler materials, OPC, PFA and RHA are sieved passing 0.075μm sieve to ensure that it meets the filler requirements and oven dried for 24 hours. Bitumen and moulds to be used are heated to 150°C.

3.2.1 Materials Requirements

Materials that will be used in this study are mixture components of asphaltic concrete: bitumen, coarse aggregates, fine aggregates and proposed fillers which will include Ordinary Portland Cement (OPC) as control, Pulverized Fuel Ash (PFA) and Rice Husk Ash (RHA).

Specimens shall be designed according to Standard Marshall Test procedure and complies with the JKR Manual on Pavement Design specifications

3.2.2 Bitumen

Bitumen shall be from straight-run bitumen (petroleum bitumen) and shall be of penetration grade 80-100 grade conforming to MS 124. However, harder grade bitumen of 60-80 is recommended to be used under heavy traffic roads in order to achieve higher stability of mixture and to lessen the possibility of bitumen bleeding of flushing at high temperatures.

3.2.3 Coarse Aggregates

Coarse aggregates shall be material substantially retained on 2.4 mm sieve opening and shall be crushed rock or crushed gravel, angular in shape and free from dust, clay, vegetative and other organic matter, and other deleterious substances.

3.2.4 Fine Aggregates

Fine aggregates shall be material passing a 2.4mm sieve opening. It shall be clean natural sand or screenings or a mixture thereof. It shall be clean, hard, durable and free from clay, mud and other foreign materials. The minus 0.425mm sieve fraction shall be non plastic when tested in accordance with British Standard B.S 1377:1975. Mining sand shall be thoroughly washed before use. Fine aggregates shall be non-plastic and free from clay, loam, aggregation of material, vegetative and other organic matter, and other deleterious substances.

3.2.5 Mineral Filler

Mineral filler shall consist of finely divided material matter such as rock dust, slag dust, hydrated lime, hydraulic cement, fly ash, loess, or other suitable mineral matter. At the time of use, it shall be sufficiently dry to flow freely and essentially free from agglomerations. It shall be essentially free from organic impurities and have a plasticity index not greater than 4 (ASTM D242-04, 2005)

3.3 Mixture Requirements

The materials of the mixture shall meet the following gradation requirements as stated in the manual. The mixture shall be designed in accordance to the Standard Marshall Test method and shall conform to the specified requirements of the JKR Standards (Table 2 & 3– Appendix).

3.4 Sample Preparation

Bituminous mixes is prepared by mixing the aggregates with 80/100 penetration grade bitumen and fillers. The dry blending method is used in which the hot aggregate and the filler blended before the binder was added. The filler content is 4% - 7% by weight of mix. Samples based on several trial gradations within the limits set in the JKR standards (JKR/SPJ/1988) is prepared and tested to attain the optimum binder content.

Specimens were prepared using a Marshall Compactor machine (Figure 8). The number of compaction was 75 blows for top and bottom side of the specimens as specified by the Malaysian standard for heavily trafficked roads. The temperatures for mixing and compaction were designated at 150°C.

A number of 21 samples were prepared for each type of filler mixes which sums up to 63 Marshall Samples. Then, the optimum binder content is determined for OPC, PFA and RHA samples. 3 samples of the optimum binder samples are produced for Creep testing purposes.

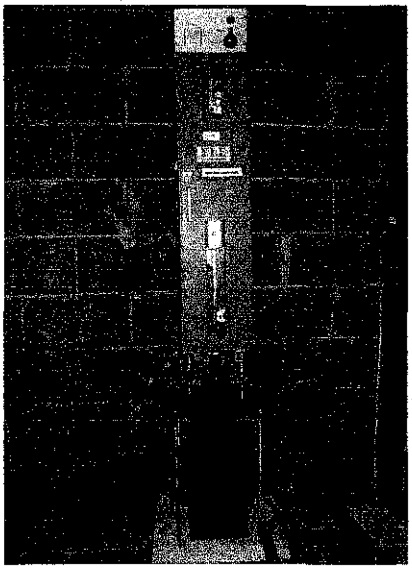

Figure 8: Marshall Compactor (left) and mixer (right)

3.5 Marshall Test

The completed samples are measured using the digital caliper to obtain the dimensions and using the buoyancy balance, the samples' weight in air and in water is known. The samples are then soaked in the water bath for 30 minutes at 60°C before tested on the Marshall Testing Rig (Figure 9 and 10)

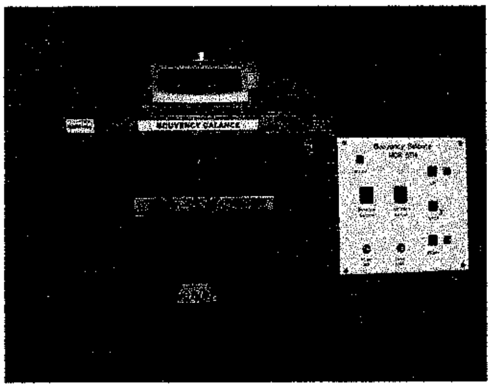

Figure 9: Bouyancy Balance (left) and Water Bath (right)

A sample of each trial mix (i.e. each combination of trial gradation and bitumen content) shall be subject to a comprehensive Marshall Method Test and analysis as follows:

i. Preparation of specimens for the standard stability and flow test in accordance with AASHTO Test Method T 245 using 75 blows / face compaction standard.

ii. Determination of the bulk specific gravity of the specimens in accordance with AASHTO Test Method T 166

iii. Determination of stability and flow values in accordance with AASHTO Test Method T 245

iv. Analysis of the density and voids parameters to determine the percentage of voids in the compacted aggregate filled with bitumen, and hence the percentage of air voids in the compacted mix.

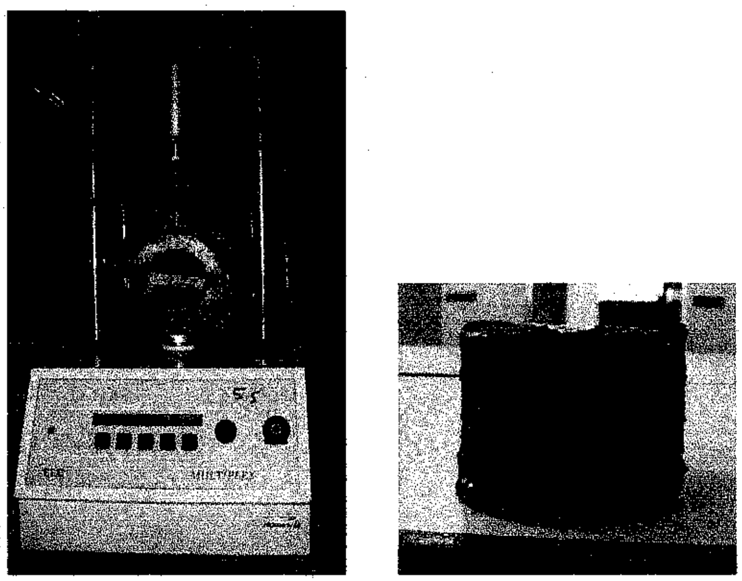

Figure 10: Marshall Testing Rig (left) and tested sample (right)

The following relationships were developed for each mixture as part of the Marshall Mix design method:

1. Unit Weight versus bitumen content,
2. Marshall Stability versus bitumen content,
3. Flow versus bitumen content,
4. Voids in total mix - VTM versus bitumen content,
5. Voids in mineral aggregate - VMA versus bitumen content

3.6 Creep Test

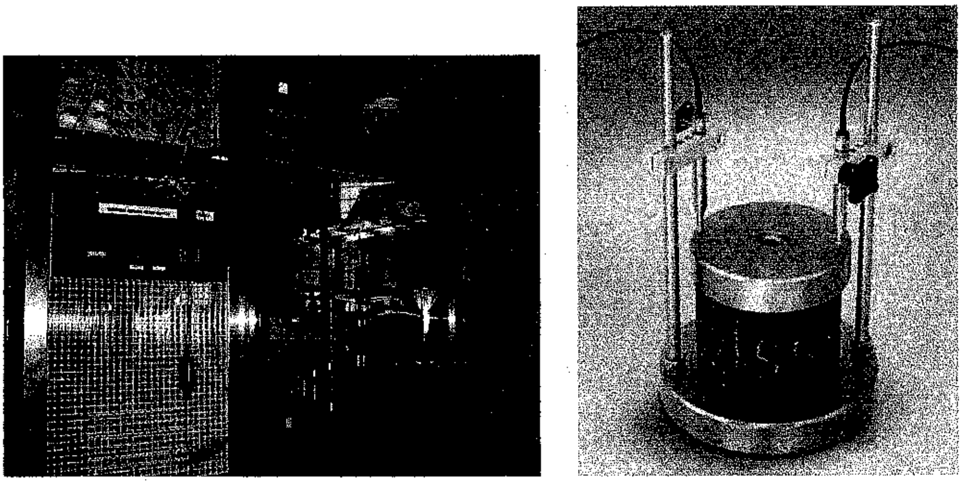

Figure 11: Universal Testing Machine (left) and Creep Testing Jig (right)

The confined dynamic creep test will be used to evaluate and control permanent deformation. This test involves cylindrical specimens (100mm diameter; 60-70 mm height) that are subjected to a vertical axial stress and to a repeated shear stress. The contact stress is applied for 1800 cycles, and the accumulation of permanent strain is measured. This test simulates a heavy vehicle moving on a pavement specimen and to determine permanent deformation due to temperature and load. The output for this test is flow time, which is the length of time the pavement can withstand the steady pressure until flow occurs.

The creep test was conducted using British Standard BS DD226 specification. The tools required are the loading press, temperature control system with confined environment, dynamic creep test jig complete with Linear Variable Differential Transducers (LVDT) and Windows based software for dynamic creep test (Figure 11).

CHAPTER 4

RESULTS AND DISCUSSION

This chapter will discuss results obtained from the sieve analysis carried out prior to laboratory works, Marshall Test results to determine the optimum binder content of the mixes and Creep test results to evaluate the performance of the samples. Samples from the same batch of materials were used and comparison is made between samples using OPC, PFA and RHA fillers. Complete set of calculations can be referred to in the Appendix.

4.1 Sieve Analysis Results

Sieve analysis was carried out to according to BS 812: Part 103: 1985 to determine the aggregate gradation of coarse and fine aggregates. For coarse aggregates, the sieve analysis was carried out using 2000g of sample and for fine aggregates; 500g of sample was used. Three trials were carried out for accuracy and the weight and percentage passing for each sieve is calculated. The results for the sieve analysis are as in Table 4 & 5 -Appendix.

The average passing is calculated for each sieve and the percentage passing is determined. The results are then tabulated according to their respective sieve sizes. The percentage of passing for filler is taken as 100% (Table 4)

Table 4. Summary of Percentage Passing of Aggregates

Sieve Size	Percentage Passing (%)			JKR Standard (%)	
(mm)	Coarse Agg (A)	Fine Agg (B)	Filler (C)	Min	Max
28	100.00	100.00	100.00	100	100
20	98.40	100.00	100.00	76	100
14	51.28	100.00	100.00	64	89
10	12.82	100.00	100.00	56	81
5	0.32	100.00	100.00	46	71
3.350	0.00	92.60	100.00	32	58
1.180	0.00	66.87	100.00	20	42
0.425	0.00	33.47	100.00	12	28
0.150	0.00	7.73	100.00	6	16
0.075	0.00	1.53	100.00	4	8

From the results above, a set of equations can be generated based on the general equation below to calculate trial mixes.

$$P = aA + bB + cC \quad (1)$$

$$a + b + c = 1 \quad (2)$$

The resulting equations are shown below in Table 5

Table 5. Trial Mix Equations

Sieve size (mm)	Equation
3.350	92.60b + 100c = 45
1.180	66.87b + 100c = 31
0.425	33.47b + 100c = 20
0.150	7.73b + 100c = 11
0.075	1.53b + 100c = 6

Thus, from the equations above, 11 trial mixes were calculated and the results are as follows:

Table 6. Trial Mix Composition Percentage

Trial mix	Coarse Agg (%)	Fine Agg (%)	Filler (%)
1	41	54	5
2	52	42	6
3	52	41	8
4	54	41	5
5	67	19	14
6	58	34	8
7	57	38	5
8	52	42	6
9	57	35	8
10	51	44	5
11	14	81	5

From the proportions obtained from the results above, the percentages of different aggregate sizes can be determined. The results are then compared to the minimum and maximum range as specified by the JKR Standard. From the calculations, it is found that Trial Mix 1, Trial Mix 2 and Trial Mix 4 meet the JKR Standard Specifications. Full set of trial mix gradations can be refereed to in Table 6-16 in Appendix.

Table 7. Trial Mix 2 (Coarse Aggregates: 52%, Fine Aggregates: 42%, Filler: 6%)

Sieve Size	Percent by Weight			Total	JKR Standard (%)	
(mm)	Coarse	Fine	Filler	Aggregate	Min	Max
28	52.00	42.00	6.00	100.00	100	100
20	51.17	42.00	6.00	99.17	76	100
14	26.67	42.00	6.00	74.67	64	89
10	6.67	42.00	6.00	54.67	56	81
5	0.17	42.00	6.00	48.17	46	71
3.350	0.00	38.89	6.00	44.89	32	58
1.180	0.00	28.09	6.00	34.09	20	42
0.425	0.00	14.06	6.00	20.06	12	28
0.150	0.00	3.25	6.00	9.25	6	16
0.075	0.00	0.64	6.00	6.64	4	8

Thus, trial mix 6 with 52% of Coarse Aggregates, 42% Fine Aggregates and 6% filler is adopted in this study (Table 7). The total percentage (given by the aggregates gradation curve) is plotted in a semi-logarithmic graph and compared to the ACW20 envelope. The graph shows that the assumption of 52% coarse aggregate, 42% fine aggregate and 6% filler is sufficient to meet the ACW20 specification as the line stayed within the maximum and minimum gradation range (Figure 12).

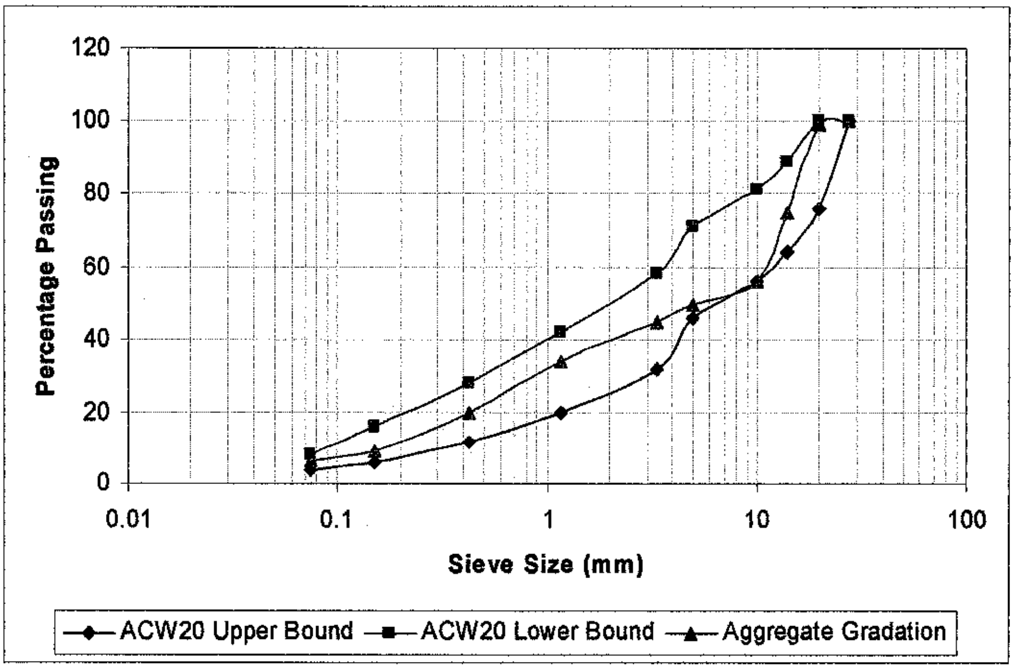

Figure 12: Aggregate Gradation Curve

Finally, ratio of 52:42:6 is used to determine the amount of coarse aggregate, fine aggregate and filler needed, based on 1200g mixture. The calculations have yielded the following required amounts:

- Coarse Aggregates : 624 grams
- Fine Aggregates : 504 grams
- Filler : 72 grams

Sample calculations regarding sieve analysis and results can be seen in Appendix.

4.2 Marshall Test Design Results and Discussion

The following are the results for the Marshall specimens using Ordinary Portland Cement and Rice Husk Ash as filler. 3 specimens with the same bitumen content ranging from 4% to 7% were prepared for accuracy and tested using the Marshall Testing Rig. The first step in analysis of the results is the determination of the average bulk specific gravity for all test specimens having the same bitumen content. The average density of each mixture is then obtained by multiplying its average specific gravity by the density of water, γ_w (1g/cm^3). The bulk specific gravity, G_{bcm} of the sample, i.e the compacted mixture is given as:

$$G_{bcm} = \frac{Wa}{Wa - Ww} \tag{3}$$

Where

W_a = weight of sample in air (g)

W_w = weight of sample in water (g)

The *bulk specific gravity* is defined as the weight in air of a unit volume (including all normal voids) of a permeable material at a selected temperature, divided by weight of air of the same density of gas-free distilled water at the same temperature. Since the aggregate mixture consists of different fractions of coarse aggregates, fine aggregates and mineral fillers with different specific gravities, the bulk specific gravity of the total aggregate in the paving mixture is given as

$$G_{bam} = \frac{P_{ca} + P_{fa} + P_{mf}}{\frac{P_{ca}}{G_{bca}} + \frac{P_{fa}}{G_{bfa}} + \frac{P_{mf}}{G_{bmf}}} \qquad (4)$$

Where

G_{bam} = bulk specific gravity of aggregates in paving mixture

P_{ca}, P_{fa}, P_{mf} = percent of weight of coarse aggregates, fine aggregates, and mineral Filler, respectively in paving mixture

G_{bca}, G_{bfa}, G_{bmf} = bulk specific gravities of coarse aggregates, fine aggregates and Mineral filler, respectively

In order to compute the percent air voids in total mix and percent air voids in mineral aggregates, it is first necessary to calculate the maximum specific gravity of the paving mixture, G_{mp}. G_{mp} assumes that there no voids in the asphalt concrete. Although the G_{mp} can be determined in the laboratory by conducting the standard test (ASTM D2041), the best accuracy is attained at mixtures near the optimum bitumen content. The maximum specific gravity of the paving mixtures with different bitumen contents using equation (2)

$$G_{mp} = \frac{100}{(P_{ta} / G_{ea}) + (P_{ac} / G_{ac})} \qquad (5)$$

Where

G_{mp} = maximum specific gravity of paving mixture

P_{ta} = percent by weight of aggregates in paving mixtures

P_{ac} = percent by weight of asphalt in paving mixtures

G_{ea} = effective specific gravity of the aggregates (assumed to be constant for different asphalt cement contents)

G_{ac} = specific gravity of asphalt

The percentage of air voids in mineral aggregates or VMA is the percentage of voids spaces between the granular particles in the compacted paving mixtures, including the air voids and volume occupied by the effective bitumen content. It is given as

$$VMA = 100 - \frac{G_{bcm} P_{ta}}{G_{bcm}} \qquad (6)$$

The percentage of air voids in compacted mixture is a ratio between the volume of small air voids between the coated particles and the total volume of the mixture. It can be obtained from

$$P_{av} = 100 \frac{G_{mp} - G_{bcm}}{G_{mp}} \qquad (7)$$

Where

P_{av} = percent air voids in compacted paving mixtures

G_{mp} = maximum specific gravity of the compacted paving mixtures

G_{bcm} = bulk specific gravity of the compacted paving mixture

For stability calculations, the obtained stability values are corrected (in order to take into account the dimensions of the samples) by the appropriate coefficient (Table 7).

Table 8 Coefficient Factor (C.F) for Adjusting Stability Values

Volume of specimen (cm3)	Approx. thickness of specimen (cm)	Correction Coefficient
536 – 546	6.67	0.93
547 – 559	6.83	0.89
560 – 573	6.99	0.86

The following relationships were developed for each mixture

1. Unit Weight versus bitumen content (Figure 13)
2. Marshall Stability versus bitumen content (Figure 14)
3. Voids in total mix - VTM versus bitumen content (Figure 15)
4. Voids in mineral aggregate - VMA versus bitumen content (Figure 16)
5. Flow versus bitumen content (Figure 17)

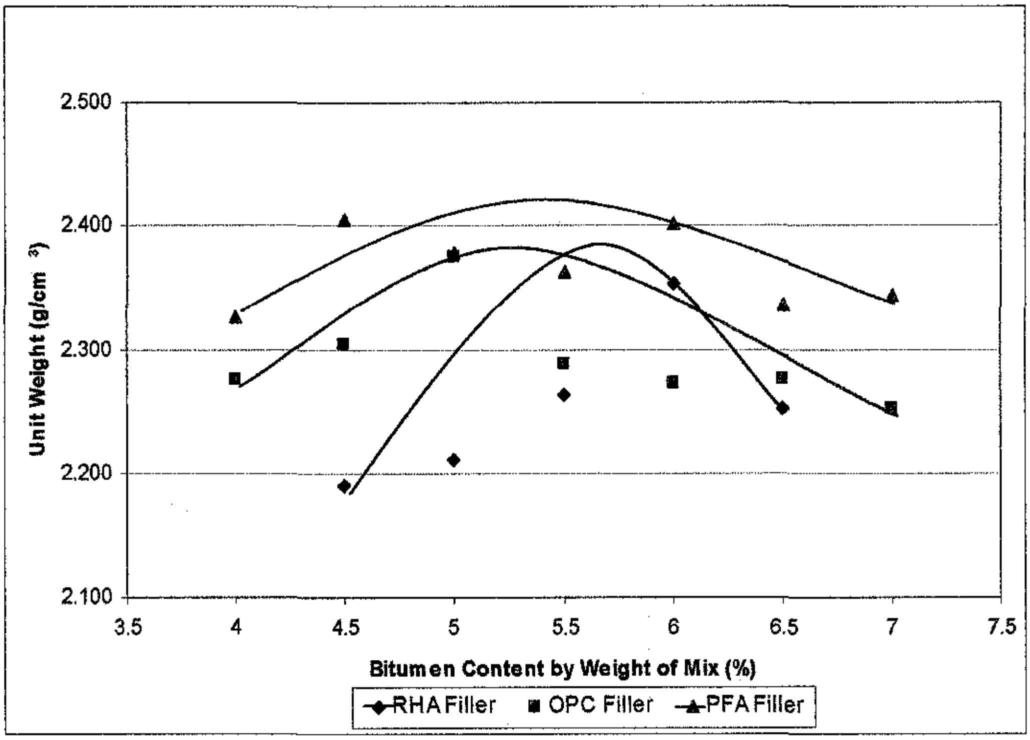

Figure 13. Unit Weight versus Bitumen Content by Mass of Mix

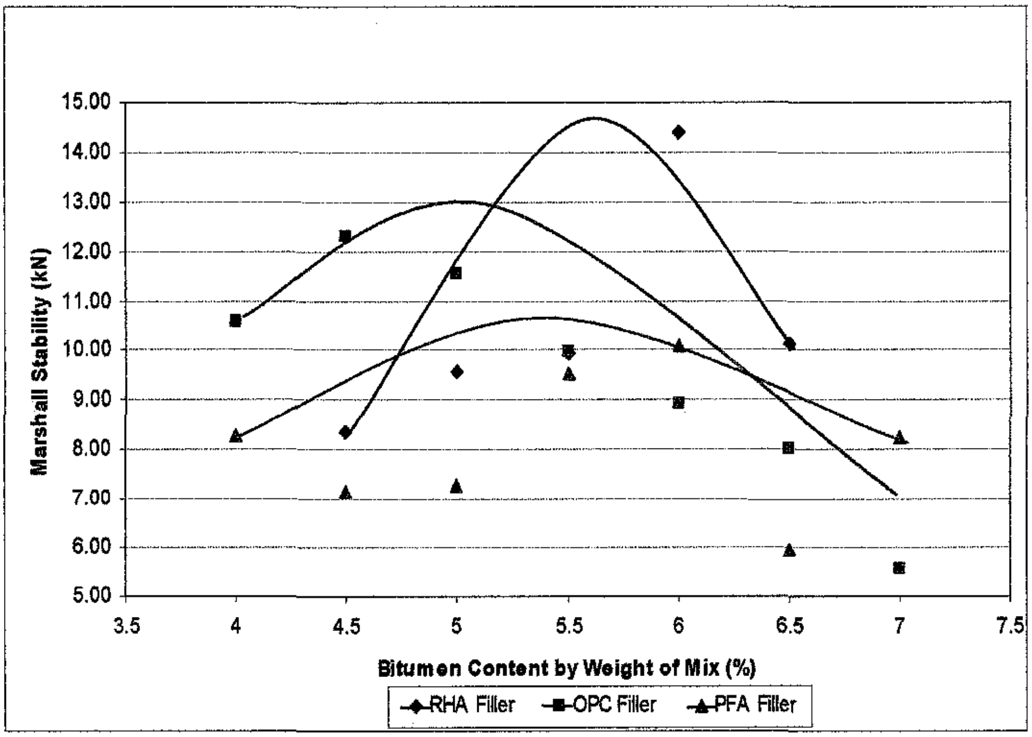

Figure 14. Marshall Stability versus Bitumen Content by Mass of Mix

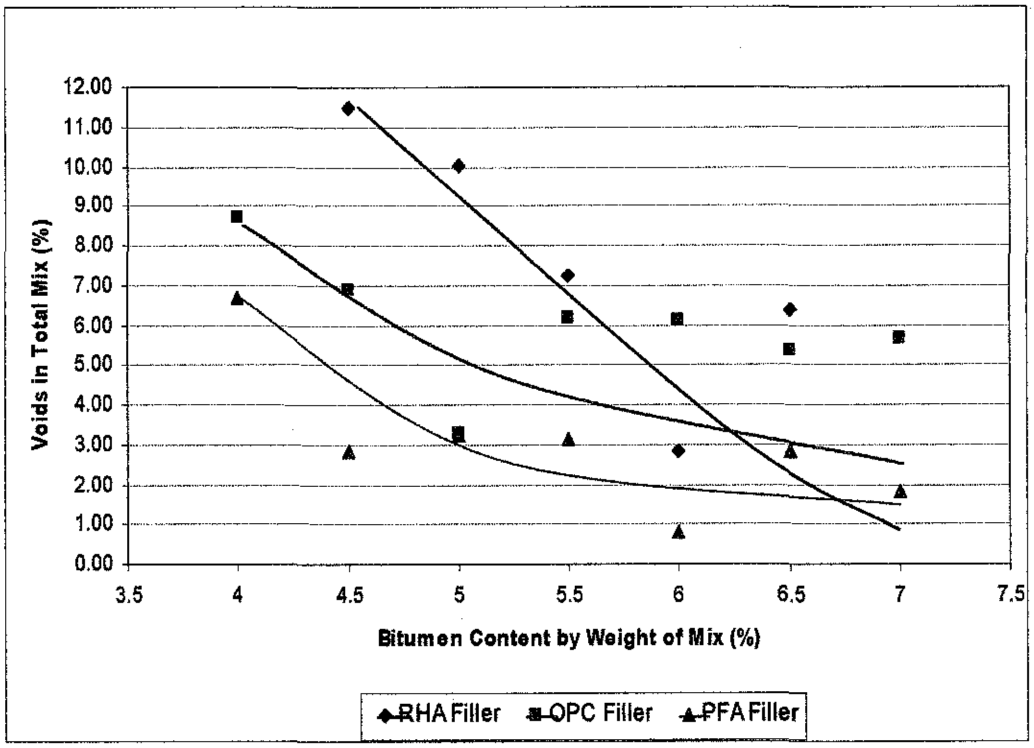

Figure 15. Voids in Total Mix versus Bitumen Content by Mass of Mix

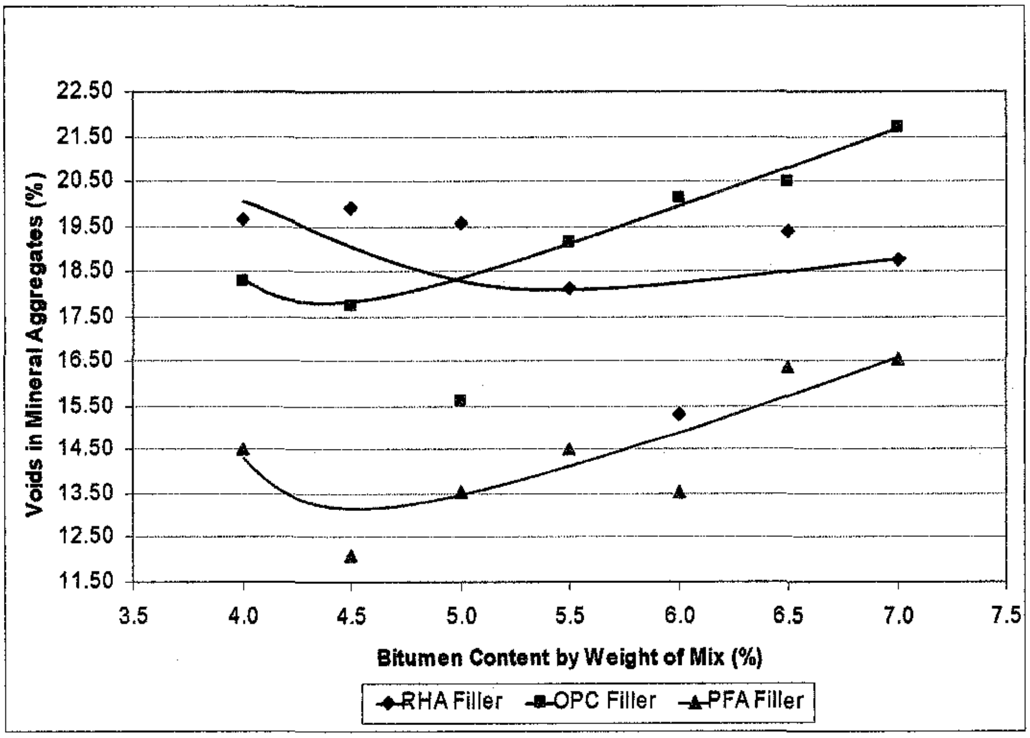

Figure 16. Voids in Mineral Aggregate versus Bitumen Content by Mass of Mix

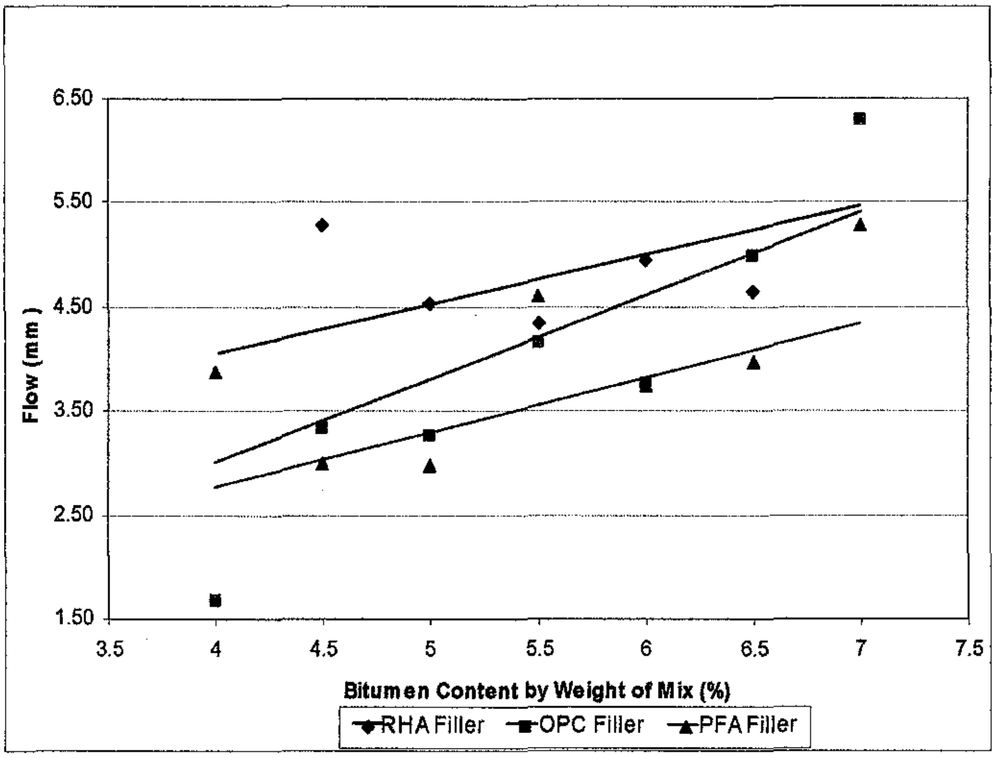

Figure 17. Flow versus Bitumen Content by Mass of Mix

The bitumen content having the maximum value of unit weight, stability and voids in total mix are selected from each of the respective plots. For voids in total mix (VTM) and voids in mineral aggregates (VMA), the mid points of the average of the upper and lower limits are selected.

4.2.1 Ordinary Portland Cement Filler Mix

a) Maximum unit weight = 5.25% (Figure 13)
b) Maximum stability = 5% (Figure 14)
c) Percent of VTM using mean of limits
[i.e (8.7+5.2)/2=6.95] = 5.6% (Figure 15)

The optimum bitumen content is determined as the average.

Therefore, the optimum bitumen content is

$$\frac{5.25+5+5.6}{3}=5.28\%$$

The properties of the paving mixture containing optimum bitumen content can now be determined from Figure 13, 14, 15, 16 and 17. The values for this mixture are

Unit Weight = 2.383 g/cm^3

Stability = 12.80 kN

Percent Voids in Total Mix = 5.1%

Percent Voids in Mineral Aggregate = 18.75%

Flow = 3.95mm

4.2.2 Pulverized Fuel Ash Filler Mix

a) Maximum unit weight = 5.4% (Figure 13)

b) Maximum stability = 5.4% (Figure 14)

c) Percent of VTM using mean of limits
[i.e (6.70+1.81)/2=4.255] = 4.7% (Figure 15)

The optimum bitumen content is determined as the average.

Therefore, the optimum bitumen content is

$$\frac{5.4 + 5.4 + 4.7}{3} = 5.17\%$$

The properties of the paving mixture containing optimum bitumen content can now be determined from Figure 13, 14, 15, 16 and 17. The values for this mixture are

Unit Weight = 2.414 g/cm^3

Stability = 10.52 kN

Percent Voids in Total Mix = 2.8%

Percent Voids in Mineral Aggregate = 13.80%

Flow = 3.3 mm

4.2.3 Rice Husk Ash Filler Mix

d) Maximum unit weight = 5.6% (Figure 13)
e) Maximum stability = 5.65% (Figure 14)
f) Percent of VTM using mean of limits
[i.e (12.33+6.37)/2=9.35] = 6.25% (Figure 15)

The optimum bitumen content is determined as the average.

Therefore, the optimum bitumen content is

$$\frac{5.6+5.65+6.6}{3}=5.95\%$$

The properties of the paving mixture containing optimum bitumen content can now be determined from Figure 9, 10, 11, 12 and 13. The values for this mixture are

Unit Weight = 2.352 g/cm^3
Stability = 13.5 kN
Percent Voids in Total Mix = 5 %
Percent Voids in Mineral Aggregate = 18.18%
Flow = 4.95 mm

Both OPC and RHA Filler mix displays high voids with satisfactory stability. When voids are high, it is likely that the permeability of the pavement will be high, which will allow water and air to circulate through the pavement, resulting in premature hardening of asphalt. High voids should be reduced to acceptable limits, even though stability is satisfactory. This can be achieved by adding amount of mineral filler in the mix.

On the other hand, PFA Filler mix yields low voids with satisfactory stability. This mix can cause reorientation of particles and additional compaction of the pavement with time and continued traffic load is imposed on the pavement. This may lead to instability or flushing or pavement. Mixes with low voids should be altered by adding more aggregates. Complete data and calculation on optimum binder content can be referred to in Table 17-25 and Calculation 2 in Appendix.

4.3 Creep Test

For this test, 9 samples were tested; 3 samples for each type of mixes using OPC, PFA and RHA Fillers. The creep modulus results are used to determine the mix stiffness. The stiffness mix is then plotted against stiffness of bitumen derived from the nomograph in Figure 3 - Appendix. Complete results of the creep test can be seen in the Appendix. The average values of each range were plotted in Figure 18.

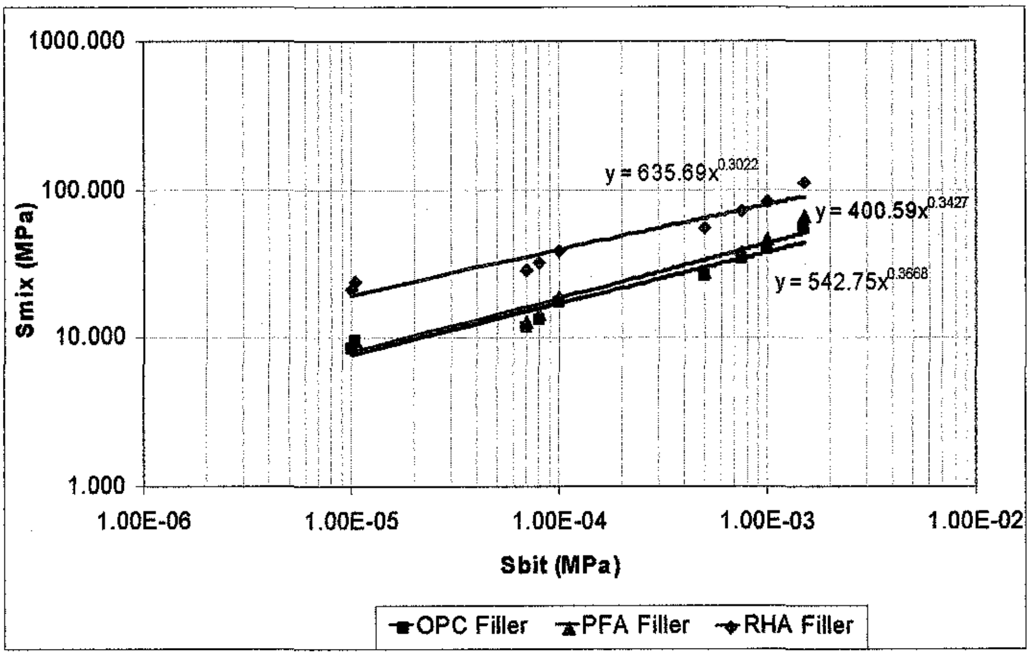

Figure 18: Relationship between mix stiffness and bitumen stiffness for optimum OPC, PFA and RHA Filler mixes

From the figure above, the relationship between mix stiffness and bitumen stiffness can be obtained

OPC Filler, Smix = $y = 400.59x^{0.3427}$

PFA Filler, Smix = $y = 542.75x^{0.3668}$

RHA Filer, Smix = $y = 635.69x^{0.3022}$

Bitumen stiffness which is calculated using the equation (8) below

$$(Sbit)v = \frac{3\eta}{NT_w} \qquad (8)$$

(Sbit)v = the viscous component of the stiffness modulus of the bitumen

η = the viscosity of the bitumen as a function of PI and ring and ball temperature from Figure 4 – Appendix (5 x 10^{-3} MPa)

N = the number of wheel passes in million ESAL

T_w = the time loading for one wheel pass, taken as 0.02s

The rut depth is then calculated using the stiffness linear relationship obtained from Figure 14. The equation (9) below is used to calculate the rut depth

$$R_d = C_m \times H \times \left(\frac{\sigma_{av}}{S_{mix}}\right) \qquad (9)$$

R_d = calculated rut depth of the pavement in mm

C_m = correlation factor for dynamic effect, varying from 1.0 to 2.0

H = pavement layer thickness, assumed 65mm

σ_{av} = average stress in the pavement, related to wheel loading and stresss, taken as 2.5 MPa

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

From the calculations using the above equations, a relationship between rut depths and cycles to standard axial loading can be established as in Figure 19.

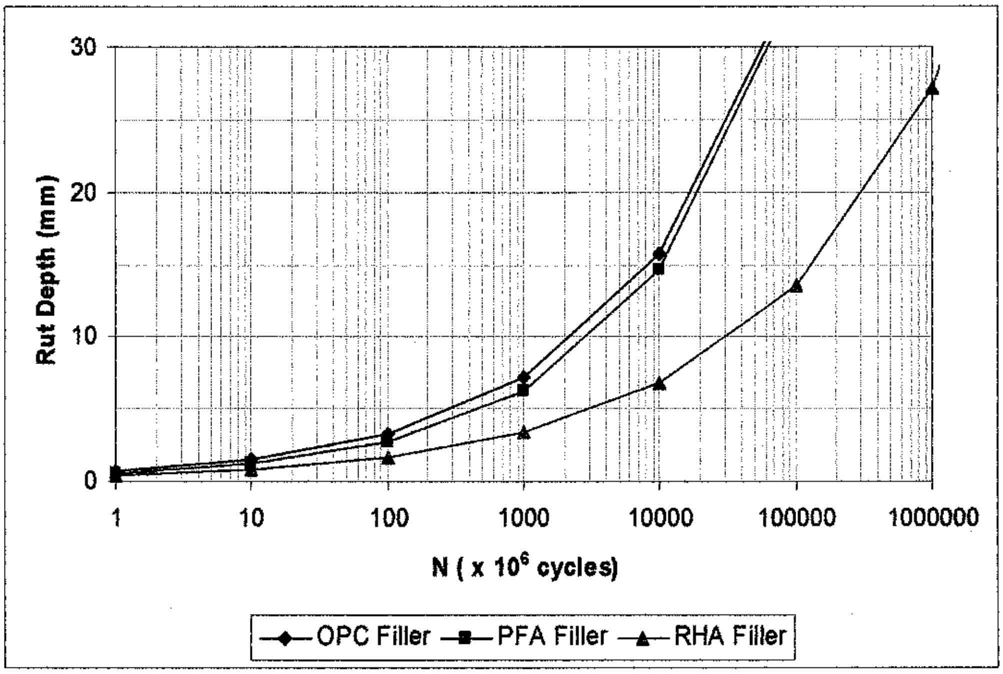

Figure 19: Estimated rut depth of road pavement for OPC, PFA and RHA Filler mixes
Complete calculations and tables can be seen in Calculation 4 - Appendix.

The value of Equivalent Standard Axle (ESAL) corresponding to a defined level of critical rutting is then determined for any particular level of statistical reliability. The 90th percentile is recommended with a critical rut depth of 10mm for roads with asphalt surfacing and 15mm for those with thin bituminous seals.

Taking a maximum rut depth of 15mm before rehabilitation and maintenance works on the pavement, samples with RHA as filler can withstand longer cycles of 140 Giga cycles while samples using OPC and PFA as filler displays almost similar results, they could withstand loading at 15mm up to 7 Giga cycles and 10 Giga cycles respectively. At 25mm, where the pavement is subjected to failure, again, both samples using OPC and PFA fillers displays relatively similar results, and could withstand loading up to 12 Giga cycles and 12.5 Giga cycles. RHA filler samples lasted until 600 Giga cycles before failing.

mix at higher temperature would be able to withstand rutting resistance better. Thus, it is proven in this study, that samples using RHA as filler, which has the highest stiffness could withstand longer cycles at 15mm. Samples using PFA and OPC as filler displays almost similar results with OPC filler yielding only slightly higher resistance to rutting. This might be caused by the filler properties of both OPC and PFA which are almost similar in particles size and chemical composition.

CHAPTER 5

CONCLUSION & RECOMMENDATIONS

Results from this study have shown that samples mixed using Rice Husk Ash as fillers yielded a higher resistance at 15mm rut depth and were able to withstand up to 140 Giga ESAL. Samples mixed using Ordinary Portland Cement and Pulverized Fuel Ash Fillers lasted for 7 Giga ESAL and 10 Giga ESAL before rutting occurs. Therefore, as rutting is concerned, samples with Rice Husk Ash filler yields the best performance. Asphalt demand is reduced by fine filler, and thus the cost of asphalt mixture is decreased. In addition, mineral fillers can be used to improve pavement performance. Adding mineral fillers into asphaltic mixtures enhances the pavement resistance to rutting at high temperatures. The stiffer the mix, the higher resistance it has to rutting. Permanent deformation of asphalt concrete is influenced by the nature and amount of fillers in the mix. Utilization of waste material and by product further reduces the cost and contributes to the conservation of the environment

As part of future work that can be incorporated to discover the true potential of filler materials used in this study, the chemical and binding properties of the filler when mixed with bitumen can be studied. The chemical compatibility and adhesion between the binder and filler helps in binding the aggregates and thus, increasing the stability and strength.

Additional performance testing such as beam fatigue test, wheel tracking test, tensile strength test and static creep test can be performed to investigate the performance of the pavements with regards to other parameters which includes surface cracking, moisture damage, fatigue and tensile strength and determine the feasibility of the proposed fillers. In real life investigation, samples cored from existing roads can be tested and the performance can be evaluated for needs of maintenance and rehabilitation works.

Through in depth research on the feasibility of these proposed fillers, costs of road construction projects can be reduced significantly. A detailed cost analysis on the pavement lifespan and life-cost cycle models can be established. Actual costs and figures of road construction projects can be obtained from the Public Works Department, Malaysia (JKR) and current material costs from various suppliers. The use of filler material that yields better pavement performance results in immense cost savings and provides longer pavement lifespan.

REFERENCES

1. AASHTO Test Method T166 – Standard Method of Test for Bulk Specific Gravity of Compacted Bituminous Mixture Using Saturated Surface Dry Specimens

2. AASHTO Test Method T245 – Standard Method of Test for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus

3. ASTM D242-04, "Standard Specification for Mineral Filler for Bituminous Paving Mixtures", Annual Book of ASTM Standard 2005, Volume 04-03, Road and Paving Materials; Vehicle – Pavement Systems

4. Ali N., Chan J.S, Simms S., Bushman R. and Bergen A.T (1996), "Mechanistic Evaluation of Fly Ash Asphalt Concrete Mixtures", *Journal of Materials In Civil Engineering*, Vol.8, No.1

5. Brown E.R, Kandhal P.S, Zhang J. (2001). "Performance Testing for Hot Mix Asphalt." *National Centre for Asphalt Technology (NCAT) Report.* November 2001

6. Chen, J.S and Peng, C.H (1998)."Analyses of Tensile Failure Properties of Asphalt – Mineral Filler Mastrics." *Journal of Materials in Civil Engineering.* 10(4), 256-262

7. JKR Manual, Arahan Teknik (Jalan) 5/85

8. JKR Manual on Pavement Design, JKR/SPJ/1988

9. Karasahin, M. and Terzi, S. (2004). "Use of Marble Waste Dust In The Mixture of Asphaltic Concrete as Mineral Filler." *Euro Bitume, Viyana*

10. Lavin. P (2003). *Asphalt Pavements: A Practical Guude to Design, Production and Maintenance for Engineers and Architects.* Spon Press. London and New York

11. Maupin, G.W Jr., Mokarem D.W (2006). "Investigion of Proposed AASHTO Rut Test Procedure Using the Asphalt Pavement Analyzer." *Virginia Transportation Research Council.* October 2006

12. Sakr, K. (2006). "Effects of Silica Fume and RHA on the Properties of Heavy Weight Concrete". *Journal of Materials in Civil Engineering,* 18(3). 367-376.

13. U.S Department of Transportation , Federal Highway Administration Website, <http://www.tfhrc.gov/hnr20/recycle/waste/cfa52.htm>

14. Zhiming Si, D.N Little and R.L Lytton (2002). "Effects of Inorganic and Polymer Filler on Tertiary Damage Development in Asphalt Mixtures." *Journal of Materials in Civil Engineering.* 14(2). 164-172

15. Zollinger D.G, Button J.W, Yeggoni M. (1996). "Fractals of Aggregates Correlated with Creep in Asphalt Concrete." *Journal of Transportation Engineering.* 122 (1)

LIST OF APPENDIX

TABLES

FIGURES

CALCULATIONS

Table 1. Comparative Assessment of Test Methods

	Test Method	Sample Dimension	Advantages	Disadvantages
Fundamental: Diametral Tests	Diametral Static (creep)	4 in. diameter × 2.5 in. height	• Test is easy to perform • Equipment is generally available in most labs • Specimen is easy to fabricate	• State of stress is nonuniform and strongly dependent on the shape of the specimen • Maybe inappropriate for estimating permanent deformation • High temperature (load) changes in the specimen shape affect the state of stress and the test measurement significantly • Were found to overestimate rutting • For the dynamic test, the equipment is complex
	Diametral Repeated Load	4 in. diameter × 2.5 in. height	• Test is easy to perform • Specimen is easy to fabricate	
	Diametral Dynamic Modulus	4 in. diameter × 2.5 in. height	• Specimen is easy to fabricate • Non destructive test	
	Diametral Strength Test	4 in. diameter × 2.5 in. height	• Test is easy to perform • Equipment is generally available in most labs • Specimen is easy to fabricate • Minimum test time	
Fundamental: Uniaxial Tests	Uniaxial Static (Creep)	4 in. diameter × 8 in. height & others	• Easy to perform • Test equipment is simple and generally available • Wide spread, well known • More technical information	• Ability to predict performance is questionable • Restricted test temperature and load levels does not simulate field conditions • Does not simulate field dynamic phenomena • Difficult to obtain 2:1 ratio specimens in lab
	Uniaxial repeated Load	4 in. diameter × 8 in. height & others	• Better simulates traffic conditions	• Equipment is more complex • Restricted test temperature and load levels does not simulate field conditions • Difficult to obtain 2:1 ratio specimens in lab
	Uniaxial Dynamic Modulus	4 in. diameter × 8 in. height & others	• Non destructive tests	• Equipment is more complex • Difficult to obtain 2:1 ratio specimens in lab
	Uniaxial Strength Test	4 in. diameter × 8 in. height & others	• Easy to perform • Test equipment is simple and generally available • Minimum test time	• Questionable ability to predict permanent deformation
Fundamental: Triaxial Tests	Triaxial Static (creep confined)	4 in. diameter × 8 in. height & others	• Relatively simple test and equipment • Test temperature and load levels better simulate field conditions than unconfined • Potentially inexpensive	• Requires a triaxial chamber • Confinement increases complexity of the test
	Triaxial Repeated Load	4 in. diameter × 8 in. height & others	• Test temperature and load levels better simulate field conditions than unconfined • Better expresses traffic conditions • Can accommodate varied specimen sizes • Criteria available	• Equipment is relatively complex and expensive • Requires a triaxial chamber
	Triaxial Dynamic Modulus	4 in. diameter × 8 in. height & others	• Provides necessary input for structural analysis • Non destructive test	• At high temperature it is a complex test system (small deformation measurement sensitivity is needed at high temperature) • Some possible minor problem due to stud, LVDT arrangement. • Equipment is more complex and expensive • Requires a triaxial chamber
	Triaxial Strength	4 or 6 in. diameter × 8 in. height & others	• Relative simple test and equipment • Minimum test time	• Ability to predict permanent deformation is questionable • Requires a triaxial chamber

(continued) **Table 1. Comparative Assessment of Test Methods**

	Test Method	Sample Dimension	Advantages	Disadvantages
Fundamental: Shear Tests	SST Frequency Sweep Test – Shear Dynamic Modulus	6 in. diameter × 2 in. height	• The applied shear strain simulate the effect of road traffic • AASHTO standardized procedure available • Specimen is prepared with SGC samples • Master curve could be drawn from different temperatures and frequencies • Non destructive test	• Equipment is extremely expensive and rarely available • Test is complex and difficult to run, usually need special training • SGC samples need to be cut and glued before testing
	SST Repeated Shear at Constant Height	6 in. diameter × 2 in. height	• The applied shear strains simulate the effect of road traffic • AASHTO procedure available • Specimen available from SGC samples	• Equipment is extremely expensive and rarely available • Test is complex and difficult to run, usually need special training • SGC samples need to be cut and glued before testing • High COV of test results • More than three replicates are needed
	Triaxial Shear Strength Test	6 in. diameter × 2 in. height	Short test time	• Much less used • Confined specimen requirements add complexity
Empirical Tests	Marshall Test	4 in. diameter × 2.5 in. height or 6 in. diameter × 3.75 in. height	• Wide spread, well known, standardized for mix design • Test procedure standardized • Easiest to implement and short test time • Equipment available in all labs.	• Not able to correctly rank mixes for permanent deformation • Little data to indicate it is related to performance
	Hveem Test	4 in. diameter × 2.5 in. height	• Developed with a good basic philosophy • Short test time • Triaxial load applied	• Not used as widely as Marshall in the past • California kneading compacter needed • Not able to correctly rank mixes for permanent deformation
	GTM	Loose HMA	• Simulate the action of rollers during construction • Parameters are generated during compaction • Criteria available	• Equipment not widely available • Not able to correctly rank mixes for permanent deformation
	Lateral Pressure Indicator	Loose HMA	• Test during compaction	• Problems to interpret test results • Not much data available

Table 2 Gradation Limits for Asphaltic Concrete

Mix Type	Wearing Course	Binder Course
Mix Designation	ACW 20	ACB 28
B.S Sieve	% Passing by Weight	
37.5 mm		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

Table 3 Design Bitumen Contents

ACW 20 – Wearing Course	4.5 – 6.5 %
ACW 28 – Binder Course	4.0 – 6.0 %

Table 4 Coarse Aggregate Gradation

Sieve Size (mm)	Weight Passing (g)			Average passing (g)	Percentage Passing (%)
	Sample 1	Sample 2	Sample 3		
28	2000.000	2000	2000	2000	100.00
20	1992	1958	1956	1968	98.40
14	1098	915	1064	1026	51.28
10	304	213	252	256	12.82
5	8	6	5	6	0.32

Sample size: 2000 g

Table 5 Fine Aggregate Gradation

Sieve Size (mm)	Weight Passing (g)			Average passing (g)	Percentage Passing (%)
	Sample 1	Sample 2	Sample 3		
3.350	464	459	466	463	92.60
1.180	342	327	334	334	66.87
0.425	186	176	140	167	33.47
0.150	48	44	24	39	7.73
0.075	10	9	4	8	1.53

Sample size: 500 g

Table 9 Trial Mix 4 (Coarse = 52%, Fine = 41%, Filler = 5%)

Sieve Size	Percent by Weight			Total	JKR Standard (%)	
(mm)	Coarse	Fine	Filler	Aggregate	Min	Max
28	52.00	43.00	5.00	100.00	100	100
20	51.17	43.00	5.00	99.17	76	100
14	26.67	43.00	5.00	74.67	64	89
10	6.67	43.00	5.00	54.67	56	81
5	0.17	43.00	5.00	48.17	46	71
3.350	0.00	37.97	5.00	42.97	32	58
1.180	0.00	27.42	5.00	32.42	20	42
0.425	0.00	13.72	5.00	18.72	12	28
0.150	0.00	3.17	5.00	8.17	6	16
0.075	0.00	0.63	5.00	5.63	4	8

Table 10 Trial Mix 5 (Coarse = 58%, Fine = 33%, Filler = 9%)

Sieve Size	Percent by Weight			Total	JKR Standard (%)	
(mm)	Coarse	Fine	Filler	Aggregate	Min	Max
28	58.00	33.00	9.00	100.00	100	100
20	57.07	33.00	9.00	99.07	76	100
14	29.74	33.00	9.00	71.74	64	89
10	7.44	33.00	9.00	49.44	56	81
5	0.19	33.00	9.00	42.19	46	71
3.350	0.00	30.56	9.00	39.56	32	58
1.180	0.00	22.07	9.00	31.07	20	42
0.425	0.00	11.05	9.00	20.05	12	28
0.150	0.00	2.55	9.00	11.55	6	16
0.075	0.00	0.50	9.00	9.50	4	8

Table 11 Trial Mix 6 (Coarse = 58%, Fine = 34%, Filler = 8%)

Sieve Size	Percent by Weight			Total	JKR Standard (%)	
(mm)	Coarse	Fine	Filler	Aggregate	Min	Max
28	58.00	34.00	8.00	100.00	100	100
20	57.07	34.00	8.00	99.07	76	100
14	29.74	34.00	8.00	71.74	64	89
10	7.44	34.00	8.00	49.44	56	81
5	0.19	34.00	8.00	42.19	46	71
3.350	0.00	31.48	8.00	39.48	32	58
1.180	0.00	22.74	8.00	30.74	20	42
0.425	0.00	11.38	8.00	19.38	12	28
0.150	0.00	2.63	8.00	10.63	6	16
0.075	0.00	0.52	8.00	8.52	4	8

Table 12 Trial Mix 7 (Coarse = 57%, Fine = 38%, Filler = 5%)

Sieve Size	Percent by Weight			Total	JKR Standard (%)	
(mm)	Coarse	Fine	Filler	Aggregate	Min	Max
28	57.00	38.00	5.00	100.00	100	100
20	56.09	38.00	5.00	99.09	76	100
14	29.23	38.00	5.00	72.23	64	89
10	7.31	38.00	5.00	50.31	56	81
5	0.18	38.00	5.00	43.18	46	71
3.350	0.00	35.19	5.00	40.19	32	58
1.180	0.00	25.41	5.00	30.41	20	42
0.425	0.00	12.72	5.00	17.72	12	28
0.150	0.00	2.94	5.00	7.94	6	16
0.075	0.00	0.58	5.00	5.58	4	8

Table 13 Trial Mix 8 (Coarse = 52%, Fine = 42%, Filler = 6%)

Sieve Size	Percent by Weight			Total	JKR Standard (%)	
(mm)	Coarse	Fine	Filler	Aggregate	Min	Max
28	52.00	42.00	6.00	100.00	100	100
20	51.17	42.00	6.00	99.17	76	100
14	26.67	42.00	6.00	74.67	64	89
10	6.67	42.00	6.00	54.67	56	81
5	0.17	42.00	6.00	48.17	46	71
3.350	0.00	38.89	6.00	44.89	32	58
1.180	0.00	28.09	6.00	34.09	20	42
0.425	0.00	14.06	6.00	20.06	12	28
0.150	0.00	3.25	6.00	9.25	6	16
0.075	0.00	0.64	6.00	6.64	4	8

Table 14 Trial Mix 9 (Coarse = 57%, Fine = 35%, Filler = 8%)

Sieve Size	Percent by Weight			Total	JKR Standard (%)	
(mm)	Coarse	Fine	Filler	Aggregate	Min	Max
28	57.00	35.00	8.00	100.00	100	100
20	56.09	35.00	8.00	99.09	76	100
14	29.23	35.00	8.00	72.23	64	89
10	7.31	35.00	8.00	50.31	56	81
5	0.18	35.00	8.00	43.18	46	71
3.350	0.00	32.41	8.00	40.41	32	58
1.180	0.00	23.40	8.00	31.40	20	42
0.425	0.00	11.71	8.00	19.71	12	28
0.150	0.00	2.71	8.00	10.71	6	16
0.075	0.00	0.54	8.00	8.54	4	8

Table 15 Trial Mix 10 (Coarse = 51%, Fine = 44%, Filler = 5%)

Sieve Size	Percent by Weight			Total	JKR Standard (%)	
(mm)	Coarse	Fine	Filler	Aggregate	Min	Max
28	51.00	44.00	5.00	100.00	100	100
20	50.18	44.00	5.00	99.18	76	100
14	26.15	44.00	5.00	75.15	64	89
10	6.54	44.00	5.00	55.54	56	81
5	0.16	44.00	5.00	49.16	46	71
3.350	0.00	40.74	5.00	45.74	32	58
1.180	0.00	29.42	5.00	34.42	20	42
0.425	0.00	14.73	5.00	19.73	12	28
0.150	0.00	3.40	5.00	8.40	6	16
0.075	0.00	0.67	5.00	5.67	4	8

Table 16 Trial Mix 11 (Coarse = 14%, Fine = 81%, Filler = 5%)

Sieve Size	Percent by Weight			Total	JKR Standard (%)	
(mm)	Coarse	Fine	Filler	Aggregate	Min	Max
28	14.00	81.00	5.00	100.00	100	100
20	13.78	81.00	5.00	99.78	76	100
14	7.18	81.00	5.00	93.18	64	89
10	1.79	81.00	5.00	87.79	56	81
5	0.04	81.00	5.00	86.04	46	71
3.350	0.00	75.01	5.00	80.01	32	58
1.180	0.00	54.16	5.00	59.16	20	42
0.425	0.00	27.11	5.00	32.11	12	28
0.150	0.00	6.26	5.00	11.26	6	16
0.075	0.00	1.25	5.00	6.25	4	8

Marshall Mix Design Method BS598:1985

Ordinary Portland Cement Filler Mix

Density of water	=	1	g/cm^3
SG BITUMEN	=	1.03	
SG FINE AGG	=	2.65	
SG COARSE AGG	=	2.65	
SG OPC	=	3.15	

Table 17 OPC Filler Marshall Mix Design Results 1

B.C (%)	G*bcm*	Density (g/cm^3)	Gbam	Gmp
4.0	2.276	2.276	2.675	2.493
4.5	2.304	2.304	2.675	2.475
5.0	2.376	2.376	2.675	2.457
5.5	2.289	2.289	2.675	2.439
6.0	2.273	2.273	2.675	2.421
6.5	2.275	2.275	2.675	2.404
7.0	2.252	2.252	2.675	2.387

Table 21 RHA Filler Marshall Mix Design Results 2

Bitumen Grade = 80				SG Bitumen = 1.03							Density of water = 1g/cm3		
Coarse Agg = 52%						Fine Agg = 42%					Filler = 6%		
B.C (%)	Stability (kN) (Measured)				C.F	Corrected	Flow (mm)				Air voids (%)		Density
	Sample 1	Sample 2	Sample 3	Average			Sample 1	Sample 2	Sample 3	Average	VMA	VTM	(g/cm³)
4.0	8.73	13.3	6.52	9.52	0.89	8.47	5.64	2.18	3.45	3.76	19.66	12.33	2.186
4.5	7.02	11.46	10.72	9.73	0.86	8.37	5.32	5.42	5.10	5.28	19.92	11.51	2.190
5.0	13.18	10.53	9.75	11.15	0.86	9.59	4.55	4.56	4.50	4.54	19.59	10.01	2.211
5.5	11.10	10.53	11.87	11.17	0.89	9.94	4.31	4.37	4.36	4.35	18.13	7.23	2.263
6.0	19.91	12.24	14.3	15.48	0.93	14.40	4.25	4.67	5.94	4.95	15.33	2.84	2.353
6.5	10.18	11.65	13.56	11.80	0.86	10.15	5.33	6.13	2.49	4.65	19.42	6.37	2.251
7.0	17.15	18.58	15.52	17.08	0.89	15.20	6.66	5.34	5.32	5.77	18.74	4.40	2.282

Table 22 RHA Filler Marshall Mix Design Results 3

Bitumen Grade = 80				SG Bitumen = 1.03							Density of water = 1g/cm3		
Coarse Agg = 52%						Fine Agg = 42%					Filler = 6%		
B.C (%)	Mass in air (g)				Mass in water (g)				Height (mm)				Volume
	Sample 1	Sample 2	Sample 3	Average	Sample 1	Sample 2	Sample 3	Average	Sample 1	Sample 2	Sample 3	Average	(cm³)
4.0	1212.0	1219.0	1217.0	1216.0	656.5	662.0	660.5	659.7	69.13	71.78	71.72	70.88	556.33
4.5	1231.0	1246.5	1240.0	1239.2	663.5	681.5	675.0	673.3	69.34	72.4	70.32	70.69	565.83
5.0	1241.5	1236.5	1238.5	1238.8	681.0	676.0	678.5	678.5	69.35	69.26	69.33	69.31	560.33
5.5	1246.5	1257.0	1252.5	1252.0	698.5	698.5	699.0	698.7	70.50	69.55	70.50	70.18	553.31
6.0	1279.0	1252.5	1236.0	1255.8	709.0	699.0	751.5	719.8	72.41	69.73	74.45	72.20	533.81
6.5	1252.5	1228.0	1308.0	1262.8	711.0	666.0	728.0	701.7	69.74	70.42	74.36	71.51	560.99
7.0	1278.5	1269.0	1229.5	1259.0	719.0	712.0	691.0	707.3	69.17	69.43	69.7	69.43	551.66

Table 24 PFA Filler Marshall Mix Design Results 2

Bitumen Grade = 80				SG Bitumen = 1.03							Density of water = 1g/cm3		
Coarse Agg = 52%						Fine Agg = 42%					Filler = 6%		
B.C (%)	Stability (kN) (Measured)				C.F	Corrected	Flow (mm)				Air voids (%)		Density
	Sample 1	Sample 2	Sample 3	Average			Sample 1	Sample 2	Sample 3	Average	VMA	VTM	(g/cm³)
4.0	16.70	7.33	1.90	8.64	0.96	8.30	3.52	5.51	2.61	3.88	14.50	6.70	2.326
4.5	8.84	9.18	2.64	6.89	1.04	7.16	4.37	3.34	1.27	2.99	12.08	2.84	2.405
5.0	8.92	10.62	2.38	7.31	1.00	7.31	3.6	2.84	2.48	2.97	13.52	3.22	2.378
5.5	13.73	8.21	7.92	9.95	0.96	9.56	2.84	3.52	7.49	4.62	14.51	3.13	2.363
6.0	15.55	6.90	7.82	10.09	1.00	10.09	2.73	4.32	4.20	3.75	13.54	0.79	2.402
6.5	6.42	6.35	5.84	6.20	0.96	5.96	4.06	4.12	3.76	3.98	16.39	2.85	2.336
7.0	11.63	7.96	7.03	8.87	0.93	8.25	4.45	5.16	6.24	5.28	16.54	1.81	2.344

Table 25 PFA Filler Marshall Mix Design Results 3

Bitumen Grade = 80				SG Bitumen = 1.03							Density of water = 1g/cm3		
Coarse Agg = 52%						Fine Agg = 42%					Filler = 6%		
B.C (%)	Mass in air (g)				Mass in water (g)				Height (mm)				Volume
	Sample 1	Sample 2	Sample 3	Average	Sample 1	Sample 2	Sample 3	Average	Sample 1	Sample 2	Sample 3	Average	(cm³)
4.0	1228.0	1191.0	1197.0	1205.3	700.5	680.0	681.0	687.2	66.56	67.28	65.54	66.46	518.17
4.5	1221.5	1233.5	1201.5	1218.8	705.0	747.5	681.5	711.3	63.45	67.28	64.42	65.05	506.89
5.0	1220.0	1222.0	1219.0	1220.3	680.0	732.5	695.0	702.5	65.90	65.74	65.48	65.71	516.94
5.5	1222.5	1235.0	1250.5	1236.0	706.5	716.0	716.0	712.8	65.38	62.18	67.05	64.87	523.11
6.0	1234.5	1231.0	1215.5	1227.0	716.5	716.0	715.5	716.0	66.40	61.94	63.23	63.86	510.93
6.5	1231.0	1231.5	1230.0	1230.8	705.0	709.5	697.5	704.0	63.64	63.49	66.77	64.63	526.80
7.0	1268.5	1303.5	1251.0	1274.3	729.5	745.5	717.0	730.7	68.8	70.67	68.48	69.32	543.65

Table 26 Bitumen Stiffness vs Stiffness Mix

Sbit	Smix (Mpa)		
	OPC	PFA	RHA
1.50E-03	53.660	65.256	111.540
1.00E-03	40.062	45.495	83.382
7.50E-04	33.771	37.340	71.306
5.00E-04	25.836	27.932	55.337
1.00E-04	17.275	19.054	38.622
8.00E-05	13.173	14.451	31.743
7.00E-05	11.799	12.822	28.238
1.05E-05	9.532	9.471	24.029
1.00E-05	8.467	8.880	21.057

Table 27 Creep Calculation Results for OPC, RHA and PFA Filler Mix

N (x 10^6)	Sbit vics (MPa)	Smix (MPa)			Rd (mm)		
		OPC	PFA	RHA	OPC	PFA	RHA
1	0.75	362.981	488.396	582.759	0.672	0.499	0.418
10	0.075	164.886	209.881	290.596	1.478	1.161	0.839
100	0.0075	74.900	90.193	144.907	3.254	2.703	1.682
1000	0.00075	34.024	38.759	72.258	7.164	6.289	3.373
10000	0.000075	15.456	16.656	36.032	15.771	14.634	6.765
100000	0.0000075	7.021	7.158	17.968	34.719	34.054	13.566
1000000	0.00000075	3.189	3.076	8.960	76.430	79.245	27.205
10000000	0.000000075	1.449	1.322	4.468	168.252	184.405	54.558
100000000	0.0000000075	0.658	0.568	2.228	370.392	429.114	109.410

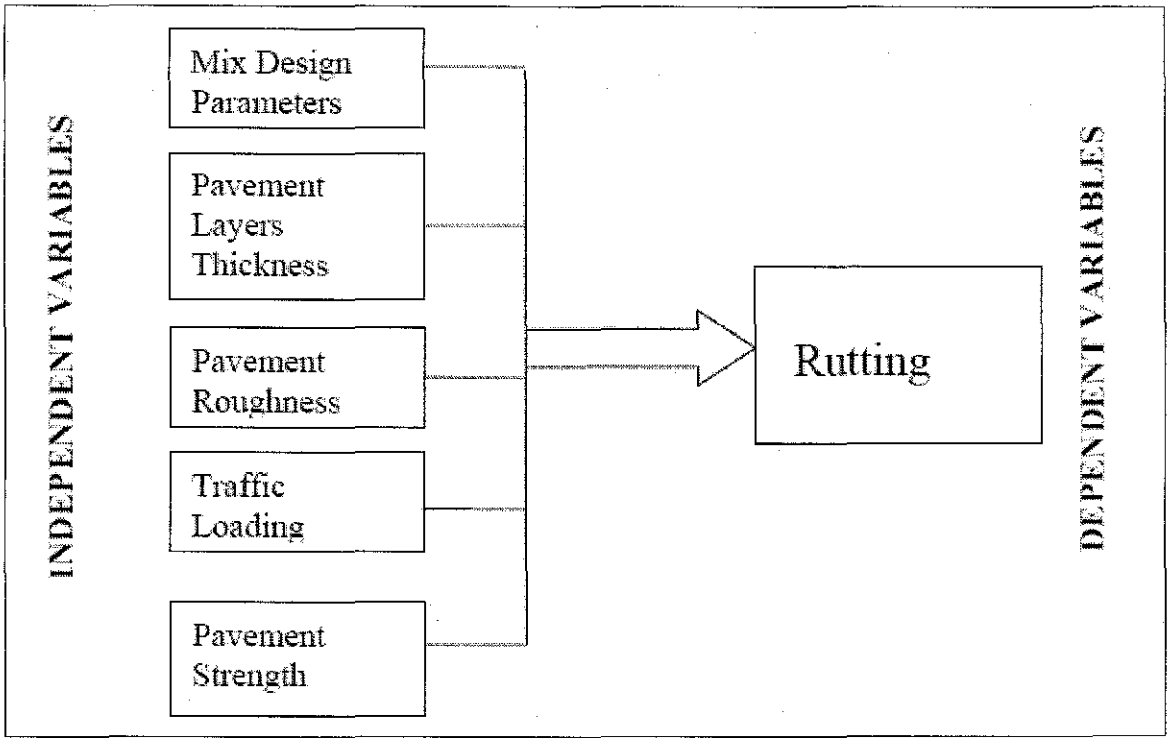

Figure 1 Among Factors Influencing Pavement Rutting in Relation to Modelling

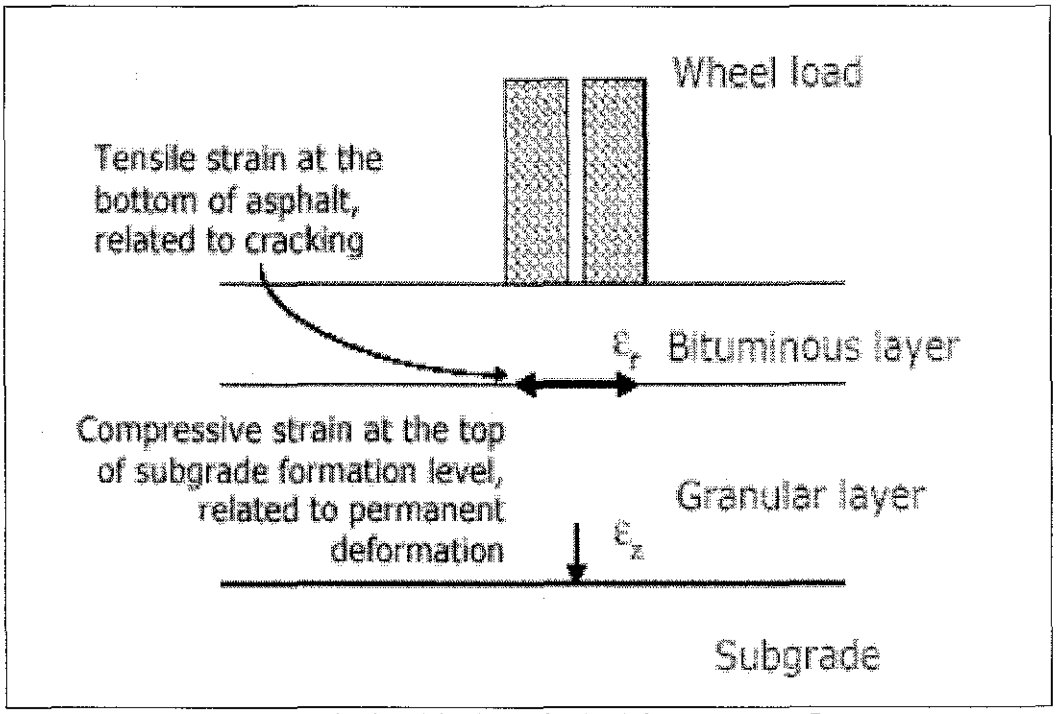

Figure 2 Vertical Critical Strain (εz) in Pavement Layers

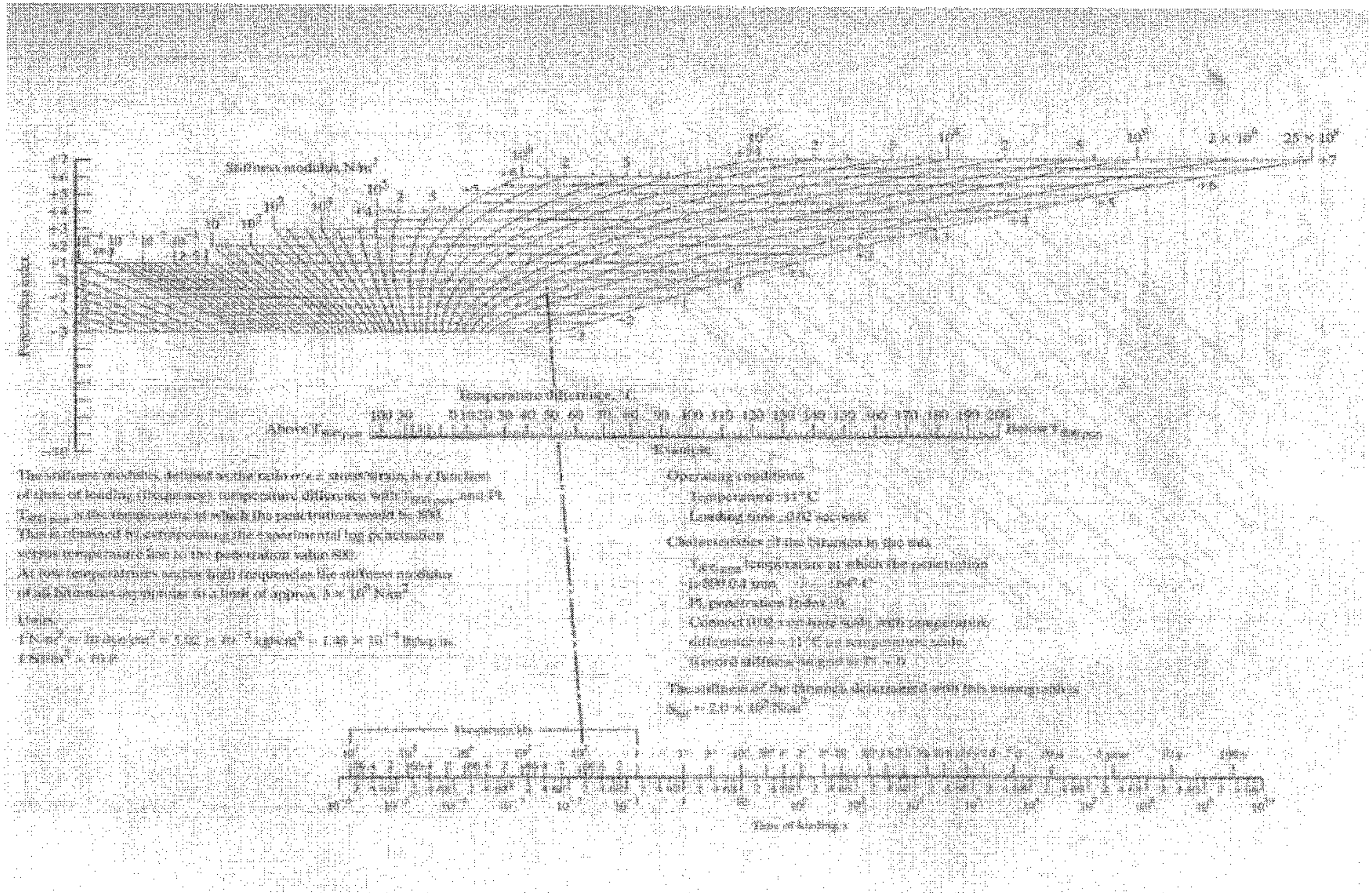

Figure 3 Nomograph for determining the Stiffness Modulus of Bitumens

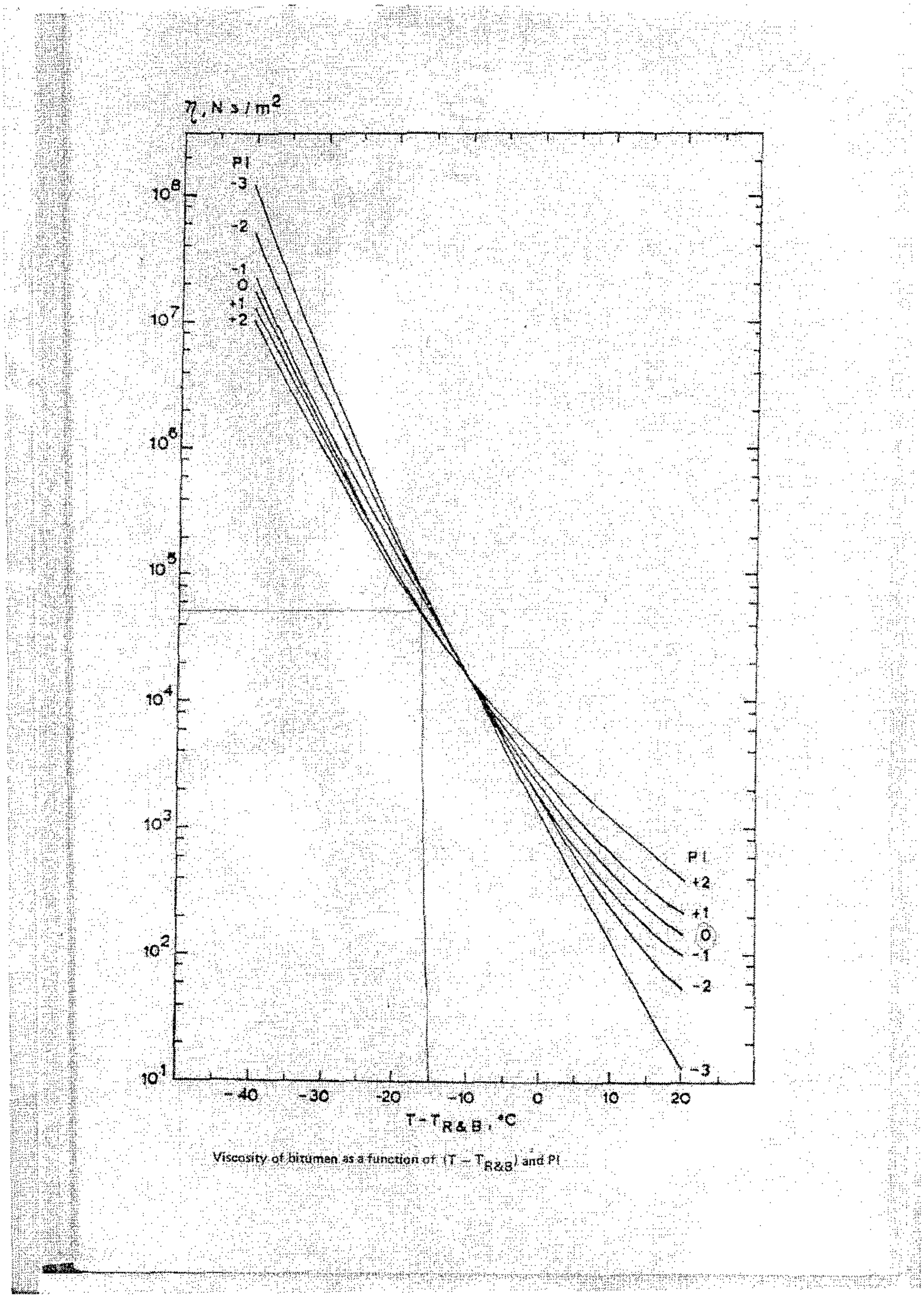

Figure 4 Viscosity Of Bitumen As A Function Of $(T - T_{r\&B})$ And PI

Calculation 1 – Trial Mix Calculations

Trial Mix 1

Using sieve size 3.350mm and 1.180mm,

$$92.60b + 100c = 45$$
$$66.87b + 100c = 31$$

solving the equations
b = 0.54
c = 0.05

solving for a, a = 0.41

Thus, a = 41%, b = 54%, c = 5%

Trial Mix 2

Using sieve size 3.350mm and 0.425mm,

$$92.60b + 100c = 45$$
$$33.47b + 100c = 20$$

solving the equations
b = 0.42
c = 0.06

solving for a, a = 0.52

Thus, a = 52%, b = 42%, c = 6%

Trial Mix 3

Using sieve size 3.350mm and 0.150mm,

$$92.60b + 100c = 45$$
$$7.73b + 100c = 11$$

solving the equations
b = 0.40
c = 0.08

solving for a, a = 0.52

Thus, a = 52%, b = 40%, c = 8%

Trial Mix 4

Using sieve size 3.350mm and 0.075mm,

$$92.60b + 100c = 45$$
$$1.53b + 100c = 6$$

solving the equations
b = 0.43
c = 0.05

solving for a, a = 52

Thus, a = 52%, b = 43%, c = 5%

Trial Mix 5

Using sieve size 1.180mm and 0.425mm,

$$66.87b + 100c = 31$$
$$33.47b + 100c = 20$$

solving the equations
b = 0.33
c = 0.09

solving for a, a = 0.58

Thus, a = 58%, b = 33%, c = 9%

Trial Mix 6

Using sieve size 1.180mm and 0.150mm,

$$66.87b + 100c = 31$$
$$7.73b + 100c = 11$$

solving the equations
b = 0.34
c = 0.08

solving for a, a = 0.58

Thus, a = 58%, b = 34%, c = 8%

Trial Mix 7

Using sieve size 1.180mm and 0.075mm,

$$66.87b + 100c = 31$$
$$1.53b + 100c = 6$$

solving the equations

$b = 0.38$
$c = 0.05$

solving for a, $a = 0.57$

Thus, a = 57%, b = 38%, c = 5%

Trial Mix 8

Using sieve size 0.425mm and 3.350mm,

$$92.60b + 100c = 45$$
$$33.47b + 100c = 20$$

solving the equations

$b = 0.42$
$c = 0.06$

solving for a, $a = 0.52$

Thus, a = 52%, b = 42%, c = 6%

Trial Mix 9

Using sieve size 0.425mm and 0.150mm,

$$33.47b + 100c = 20$$
$$7.73b + 100c = 11$$

solving the equations

$b = 0.35$
$c = 0.08$

solving for a, $a = 0.57$

Thus, a = 57%, b = 35%, c = 8%

Trial Mix 10

Using sieve size 0.425mm and 0.075mm,

$$33.47b + 100c = 20$$
$$1.53b + 100c = 6$$

solving the equations

$b = 0.44$
$c = 0.05$

solving for a, $a = 0.51$

Thus, a = 51%, b = 44%, c = 5%

Trial Mix 11

Using sieve size 0.150m and 0.075mm,

$$7.73b + 100c = 11$$
$$1.53b + 100c = 6$$

solving the equations

$b = 0.81$
$c = 0.05$

solving for a, $a = 0.14$

Thus, a = 14%, b = 81, c = 5%

Calculation 2 – Sample Marshall Calculations

* All sample calculations are using Ordinary Portland Cement Filler

The bulk specific gravity of the mix using each bitumen content is determined by calculating the average value for the specimens with the same bitumen content using equation

$$G_{bcm} = \frac{Wa}{Wa - Ww} \tag{1}$$

For 5% bitumen content, the average bulk specific gravity is given as

$$Gbcm = \frac{1}{3}\left(\frac{1239.5}{1239.5-710.0} + \frac{1244.0}{1244.0-722.0} + \frac{1255.5}{1255.5-733.0}\right)$$

$$= \frac{1}{3}(2.34 + 2.38 + 2.40)$$

$$= 2.37$$

The average density of each mixture is obtained by multiplying its average specific gravity by the density of water, γ_w (1g/cm^3).

Therefore the average density is $2.37 \times 1 = 2.37 g/cm^3$

The bulk specific gravity of aggregates with different bitumen contents is obtained using equation

$$G_{bam} = \frac{P_{ca} + P_{fa} + P_{mf}}{\dfrac{P_{ca}}{G_{bca}} + \dfrac{P_{fa}}{G_{bfa}} + \dfrac{P_{mf}}{G_{bmf}}} \tag{2}$$

For 5% bitumen content, the bulk specific gravity of aggregates is given as

Pca = 0.52 x 95 = 49.4

Pfa = 0.42 x 95 = 40.11

Pmf = 0.06 x 95 = 5.73

Gbc_a & Gbfa = 2.65

Gbmf = 3.15 (OPC)

2.13 (RHA)

2.40 (PFA)

$G_{ac} = 1.03$

$P_{ac} = 5$

Using equation (2) G_{bam}

$$G_{bam} = \frac{49.4 + 40.11 + 5.73}{\frac{49.4}{2.65} + \frac{40.11}{2.65} + \frac{5.73}{1.03}}$$
$$= 2.675$$

The maximum specific gravity of the paving mixture is calculated using equation (3)

$$G_{mp} = \frac{100}{(P_{ta} / G_{ea}) + (P_{ac} / G_{ac})}$$

For 5% bitumen content, the G_{mp} is given as

$$Gmp = \left(\frac{100}{(95/2.65) + (5/1.03)} \right)$$
$$= 2.457$$

The percentage of voids in compacted mineral aggregates can be determined from equation

$$VMA = 100 - \frac{G_{bcm} P_{ta}}{G_{bcm}} \qquad (4)$$

For 5% bitumen content,

$$VMA = 100 - \frac{2.37 * 95}{2.675}$$

$$= 2.63$$

The percentage of air voids in compacted mixture can be obtained from (5)

$$P_{av} = 100 \frac{G_{mp} - G_{bcm}}{G_{mp}} \tag{5}$$

For 5% bitumen content,

$G_{mp} = 2.457$

$G_{bcm} = 2.376$

Hence

$$Pav = 100 \frac{2.457 - 2.376}{2.457}$$

$$= 3.30$$

Calculation 3 – Weight of bitumen in a sample mix

In this study, binder range of 4% - 7% is used. The amount of bitumen is determined from the sample calculation below

For 4% bitumen content,

$$0.04 = \frac{B}{B+1200}$$

Solving for B,

B = 50 g

Thus in a 1200g sample, 50g of bitumen will be added to the mix for 4% bitumen content

Calculation 4 – Creep Test Calculations

To determine the bitumen stiffness viscosity, the following equation is adopted

$$(Sbit)v = \frac{3\eta}{NT_w}$$

(Sbit)v = the viscous component of the stiffness modulus of the bitumen

η = the viscosity of the bitumen as a function of PI and ring and ball temperature from Figure 4 - Appendix

N = the number of wheel passes in standard axles

T_w = the time loading for one wheel pass, taken as 0.02s

Sample calculation:

For N = 10 cycles

η = 5 x 10^3 at -4°C

T_w = 0.02s

Therefore,

$$(Sbit)v = \frac{3 \times (5 \times 10E-3)}{10 \times 0.02}$$

= 0.075 MPa

From Figure 14, 3 sets of linear equations were obtained

For OPC Filler: Smix = y = $400.59x^{0.3427}$

For PFA Filler: Smix = y = $542.75x^{0.3668}$

For RHA Filler: Smix = y = $635.69x^{0.3022}$

Substituting values of x in the equations with N, Smix can be determined.

N (x 10^6)	Sbit vics (MPa)	Smix (MPa)			Rd (mm)		
		OPC	PFA	RHA	OPC	PFA	RHA
1	0.75	362.981	488.396	582.759	0.672	0.499	0.418
10	0.075	164.886	209.881	290.596	1.478	1.161	0.839
100	0.0075	74.900	90.193	144.907	3.254	2.703	1.682
1000	0.00075	34.024	38.759	72.258	7.164	6.289	3.373
10000	0.000075	15.456	16.656	36.032	15.771	14.634	6.765
100000	0.0000075	7.021	7.158	17.968	34.719	34.054	13.566
1000000	0.00000075	3.189	3.076	8.960	76.430	79.245	27.205
10000000	0.000000075	1.449	1.322	4.468	168.252	184.405	54.558
00000000	0.0000000075	0.658	0.568	2.228	370.392	429.114	109.410

To determine the rut depth, the following equation is adopted

$$R_d = C_m \times H \times \left(\frac{\sigma_{av}}{S_{mix}} \right)$$

R_d = calculated rut depth of the pavement in mm

C_m = correlation factor for dynamic effect, varying from 1.0 to 2.0

H = pavement layer thickness, assumed 65mm

σ_{av} = average stress in the pavement, related to wheel loading and stresss, taken as 2.5 MPa

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

Therefore, for rut depth at S_{mix} (OPC) = 362.981 MPa

$$R_d = 1.5 \times 65 \times \left(\frac{2.5}{362.981} \right)$$

$= 0.67$ mm