PERFORMANCE OF SURFACE LAYER WITH ASPHALTIC CONCRETE (AC) AS WEARING COURSE AND HOT-ROLLED ASPHALT (HRA) AS BINDER COURSE

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TITLE PAGE

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Performance of surface layer with Asphaltic Concrete (AC) as wearing course and

Hot-Rolled Asphalt (HRA) as binder course

By

Shuhaila Binti Shamsuddin (©)

A THESIS

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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.

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ABSTRACT

Rutting and fatigue cracking are two apparent failure modes in a flexible pavement. They do not only shorten the performance life of a pavement but also require a lot of money and energy on repair works. This study presents a laboratory investigation of combining asphaltic concrete bituminous mix (AC) as wearing course and hot-rolled asphalt (HRA) binder course in road surfacing structure to overcome the problems. Four types of mixtures namely AC with 80 PEN bitumen grade (AC(80)), AC with 50 PEN bitumen grade (AC(50)), HRA with 50 PEN bitumen grade (HRA(50)) and HRA with 80 PEN bitumen grade (HRA(80)) were made as control specimens to compare the performance of the combination of AC wearing course and HRA binder course, (AC(80)+HRA(50)). Results from dynamic creep test and wheel tracking test confirm that AC is more superior in reducing rutting. The combined mix provides promising findings when it surpasses rutting performance when compared to normal AC. Meanwhile HRA mix shows a longer fatigue life compared to AC. The combined mix (AC(80)+HRA(50)) shows a better resistance towards fatigue compared to AC(80). At low strain level, the former achieved a fatigue life of 1E+8 cycles while a fatigue life for the latter mix is only 9.98E+5 cycles. Thus, the usage of combined mix has potential to improve the road surfacing performance.

KEYWORDS

asphaltic concrete bituminous mix (AC), hot-rolled asphalt (HRA), combined mix, surfacing structure, rutting, fatigue cracking

ABSTRAK

Ubah bentuk kekal dan Keretakan lesu adalah dua mod kerosakan dalam turapan boleh lentur. Kerosakan tersebut bukan sahaja memendekkan jangka hayat turapan malah memerlukan wang dan tenaga yang tinggi dalam kerja pembaikan. Kajian ini terdiri daripada beberapa ujian makmal yang menggunakan gabungan konkrit berasfalt (AC) sebagai lapisan permukaan dan tergelek panas asfalt (HRA) sebagai lapisan pengikat dalam struktur permukaan jalan raya bagi mengatasi kerosakan tersebut. Empat jenis campuran biasa terdiri daripada AC bersama 80 PEN bitumen (AC(80)), AC bersama 50 PEN bitumen (50), HRA bersama 50 PEN bitumen (50) dan HRA bersama 80 PEN bitumen (80) digunakan sebagai spesimen kawalan untuk dibandingkan dengan prestasi gabungan bancuhan (AC(80)+HRA(50)). Hasil daripada ujian Rayapan Dinamik dan ujian Penjejakan Roda menunjukkan bahawa AC mempunyai lebih rintangan terhadap ubah bentuk kekal. Gabungan bancuhan boleh mengatasi kerosakan ubah bentuk kekal jika dibandingkan dengan AC biasa. Sementara itu, bancuhan HRA telah menunjukkan hayat lesu yang panjang jika dibandingkan dengan AC. Begitu juga, gabungan bancuhan (AC(80)+HRA(50)) menunjukkan rintangan yang lebih baik terhadap lesu berbanding AC(80). Pada aras keterikan yang rendah, gabungan bancuhan telah mencapai hayat lesu pada kitaran 1E+8 berbanding AC(80) yang cuma mencapai 9.98E+5 kitaran. Oleh yang demikian, penggunaan gabungan bancuhan berpontensi untuk meningkatkan prestasi permukaan jalan raya.

KATA KUNCI

konkrit berasfalt (AC) , tergelek panas asfalt (HRA), gabungan bancuhan, struktur permukaan, ubah bentuk kekal, keretakan lesu

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List of Abbreviations and Symbols

1. σ = applied stress, MPa
2. S_{bit} = Bitumen Stiffness, N/m^2
3. S_{mix} = Mix Stiffness, MPa
4. ε = strain, mm/mm
5. AAPT = Association of Asphalt Paving Technologist
6. AC = Asphaltic Concrete
7. BS = British Standard
8. C = Celcius
9. E = Elastic modulus
10. GTM = Gyratory Testing Machine
11. HRA = Hot-Rolled Asphalt
12. in. = inch
13. kN = kilo Newton
14. kPa = kilo Pascal
15. mm = millimeter
16. MPa = Mega Pascal
17. N = Newton
18. obc = optimum bitumen content
19. PEN = penetration, 0.1mm
20. PI = Penetration Index
21. SHRP = Strategies Highway Research Program
22. T = Temperature
23. t = time
24. μm = micrometer

CHAPTER ONE
INTRODUCTION

1.0 Background

Throughout the years, rutting (permanent deformation) and fatigue cracking are the two major problems often faced by engineer on a surfacing structure of a flexible pavement after opening to traffic for certain time.

Rutting is a surface deformation in the wheel path due to insufficient strength of asphalt mixtures whereas fatigue cracking occurs when temperature or traffic generated stresses and strains exceed the tensile strength of a compacted asphalt mixture. In a flexible pavement, wearing course and binder course contribute as part of a surfacing structure as shown in Figure 1.

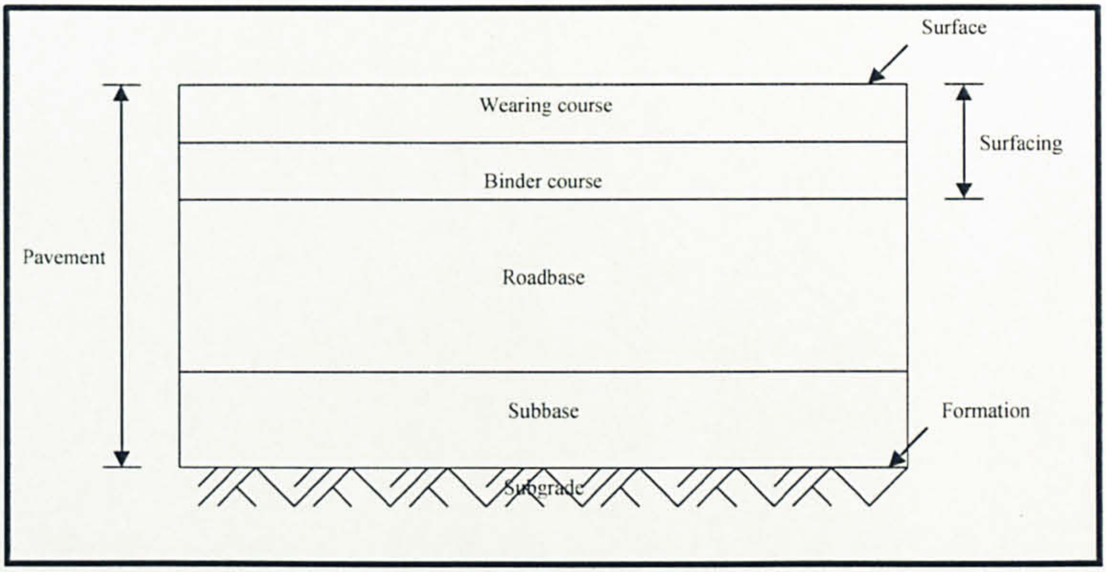

Figure 1: Layer component of a flexible pavement

In many continental countries including United States of America, asphaltic concrete bituminous mix (AC) is used as wearing course due to its ability to withstand rutting meanwhile hot–rolled asphalt (HRA), which is popular in United Kingdom, is commonly used because it can reduce fatigue cracking in the pavement.

Thus, this research presents laboratory investigations to determine the performance of road surfacing structure based on combining AC as wearing course and HRA as binder course.

1.1 Problem Statement

Two major types of failure in road surfacing structures are rutting and fatigue cracking. Over the years, rutting has been a major problem to Malaysia's surfacing structure. Rapid rutting can be seen throughout Peninsular Malaysia especially at areas like climbing lanes and junction (Hizam Harun et al., 1992). Cracking also has been reported to be a problem when the signs of small cracking appeared on the surface of road made with Asphaltic Concrete, which is a common material used to make the surfacing structure (Morosiuk et al., 1999).

Therefore, this research is to evaluate the performance of road surfacing structure by combining AC and HRA because AC is good for reducing rutting whereas HRA is good in reducing fatigue cracking.

1.2 Objective

The objective of this study to determine the road surfacing performances by combining both AC as wearing course and HRA as binder course.

1.3 Scope

The scope of this research is to evaluate the rutting and fatigue cracking performances of the surfacing layer in a flexible pavement. The research is laboratory based where all the tests are conducted at the Highway Material Laboratory at Universiti Teknologi Petronas (UTP). The optimum bitumen content (obc) for AC is based on the JKR standard JKR/SPJ/1988 whereas the obc for HRA is in accordance to BS 594:1992. Dynamic creep and wheel tracking tests were used to measure the rutting performance whereas beam fatigue test was used to calculate the fatigue cracking performance. In this research, five types of mixtures were prepared, namely AC with 80 PEN (AC(80)) , AC with 50 PEN (AC(50)), HRA with 80 PEN (HRA(80)), HRA with 50 PEN (HRA(50)) and the combined mixture of AC(80)+HRA(50). A total of 100 samples were fabricated for the whole tests. The Marshall design method and the dynamic creep test required a circular type of sample whereas the wheel tracking test needed a square shape sample. The beam fatigue test needed a rectangular shape sample.

CHAPTER TWO
LITERATURE REVIEW

2.0 Asphaltic Concrete Bituminous Mix (AC)

2.0.1 Introduction

A dense graded asphalt mix has a continuous distribution of aggregate particle size and filler (i.e. evenly distributed from coarse to fine) which produces a smooth curve as shown in Figure 2 and a low design air void content, generally in the range of 3 to 7%. Dense graded mixes are also often referred to as AC and represent the most widely used form of asphalt mix.

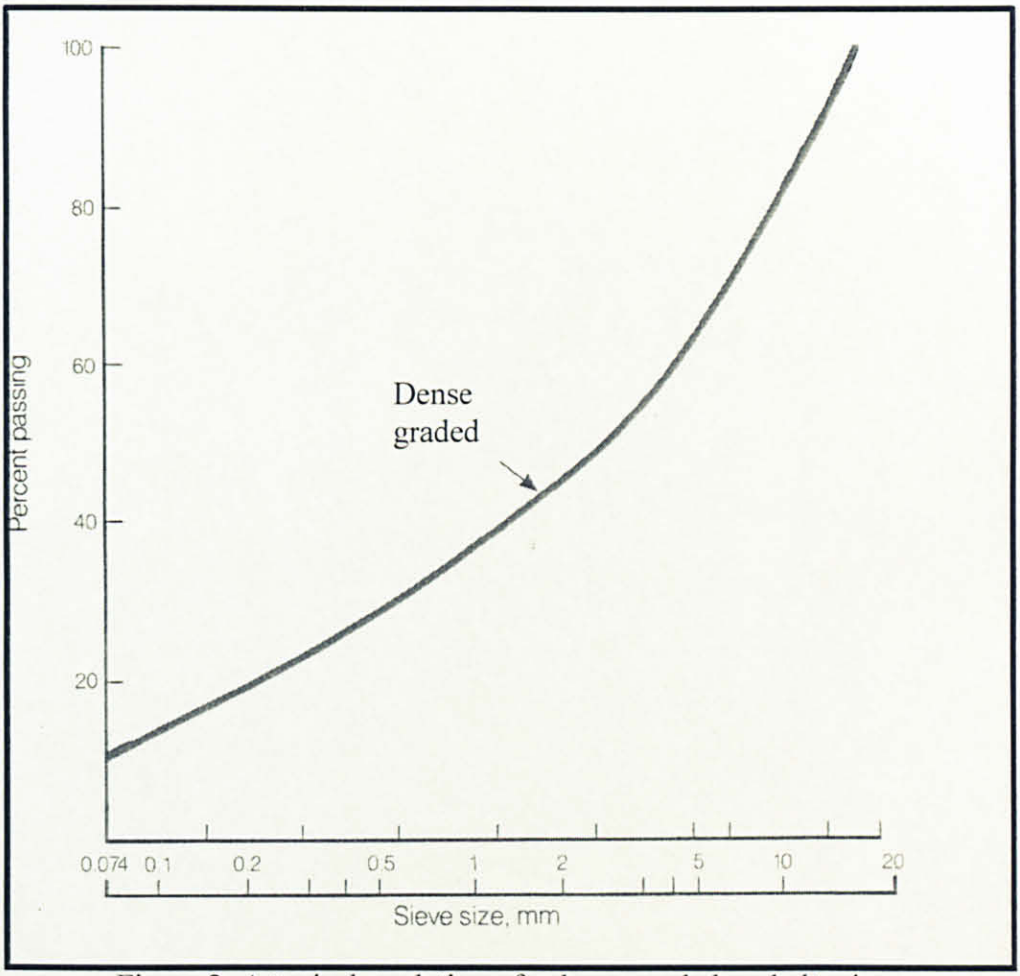

Figure 2: A typical gradation of a dense graded asphalt mix

Figure 3 shows the main characteristic of AC mixture. AC is a continuously graded mixture of mineral aggregate, filler and bitumen. These elements form an interlocking structure. This is very important because it contributes to the strength of the mixture, thus helps to withstand deformation.

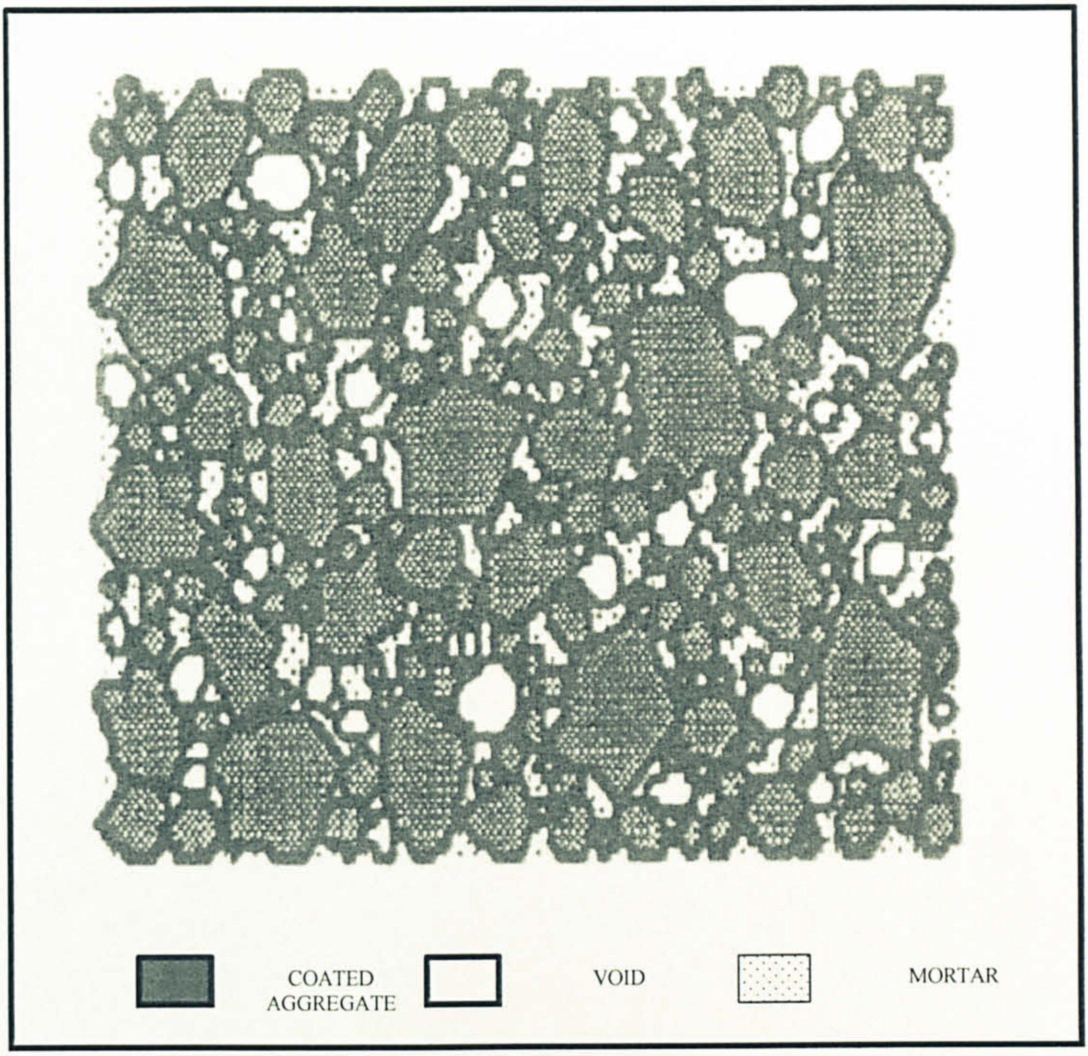

Figure 3: The characteristic of AC mixture (O'Flaherty, 2002)

AC mixtures are those that have a particle size distribution that in general follows the Fuller Curve Equation (O'Flaherty, 2002) as stated below:

$$P = 100 \times \left(\frac{d}{D}\right)^{n} \qquad ....1$$

Where P = total percentage by mass sieve d,

D = maximum size of aggregate particle in the sample

n = typically 0.5 (without binder) to 0.45 (with binder)

d = size of sieve opening

2.0.2 Material

2.0.2.1 Coarse Aggregate

Any aggregate retained on 3.35mm sieve is called coarse aggregate. In AC mixture, it tends to have higher coarse aggregate content due to the aggregate friction and interlock that generally controls the rate of material deformation. In other words, the strength and stability of AC is primarily derived from particle-to-particle contact, and inter-particle friction between the aggregate.

Therefore, it is important that the aggregate particles to be sufficiently strong to ensure that they do not tend to break down under rollers when they are being compacted, or under the subsequent repeated impact and crushing actions of moving vehicles.

2.0.2.2 Fine Aggregate

Any aggregate passing the 3.35mm sieve, but retained on the 75μm sieve is considered as fine aggregate. The type of fine aggregate is either; (Atkins, 2003)

- Natural bank, river dune or pit sand
- Crushed rock fines; or
- Blends of sand and crushed rock fines.

2.0.2.3 Filler

The permitted types of filler in AC are wide and allow the usage of any of the fines from the permitted aggregates, Portland cement or limestone as long as the material has a minimum of 75% passes the 75μm sieve (Atkins, 2003).

2.0.2.4 Bitumen

Bitumen is a black to dark brown sticky material, composed principally of high molecular – weight hydrocarbons. In some countries, it can be found as a component of natural rocks asphalt, but most bitumen is derived from the distillation of crude oil. The types or grade of bitumen are much similar for all AC surface course materials. They cover 60 to 450 penetration grade of petroleum bitumen. The function of bitumen besides lubricating the aggregate particles during compaction, it also acts as a bonding and waterproofing agent when the pavement is in service.

2.0.3 The Concept of Stiffness

2.0.3.1 Time-Temperature Equivalence

For the viscoelastic material, the stress–strain characteristics generally are temperature-dependent as well as time-dependent. In order to define the viscoelastic properties, Van der Poel introduced the concept of stiffness modulus as a fundamental parameter to describe the mechanical properties of bitumen by analogy with the elastic modulus of solids.

For such a material, at constant temperature, if a tensile strain ε_o is applied at zero time, an initial stress that developed will decay with time in a manner as illustrated schematically in Figure 4.

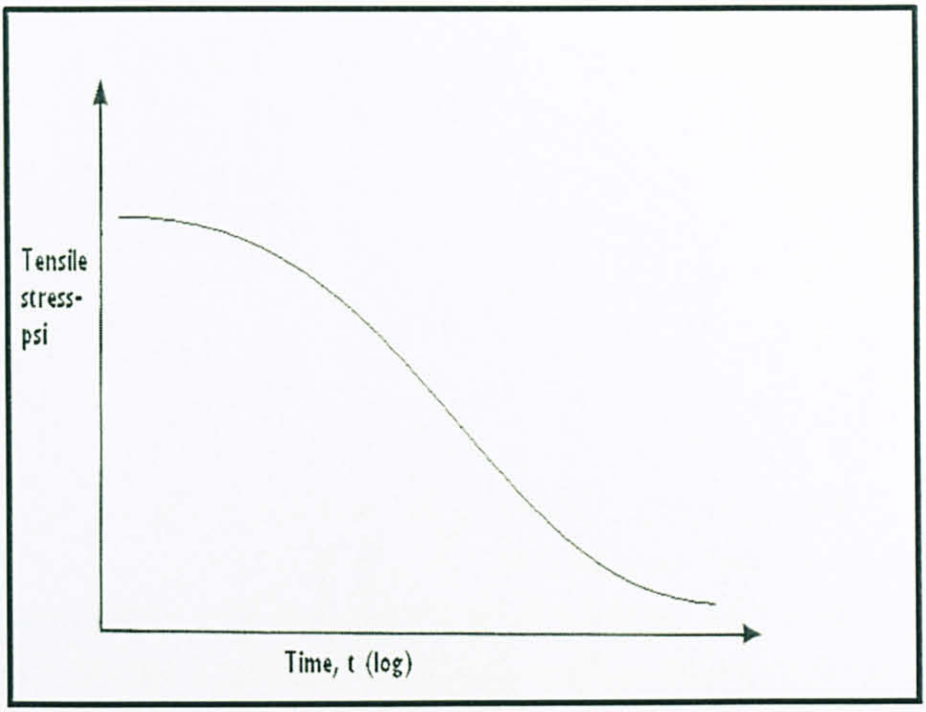

Figure 4: Response of a viscoelastic material at constant temperature to a constant strain applied at zero time

The elastic modulus, E for this material is defined as

$$E(t) = \frac{\sigma(t)}{\varepsilon_o} \qquad \ldots\ldots 2$$

In the case of viscoelastic materials such as bitumen, a tensile stress, σ, applied at loading time, t = o, causes a strain, ε, that increases, but not proportionately, with loading time. The stiffness modulus, S_t, at loading time t, is defined as the ratio between the applied stress and the resulting strain at loading time t:

$$S_t = \frac{\sigma}{\varepsilon_t} \qquad \ldots\ldots 3$$

Similarly, the stiffness modulus also depends on the temperature, T. Therefore, it is necessary to state both temperature and time of loading of any stiffness measurement.

$$S_{t,T} = \frac{\sigma}{\varepsilon_{t,T}} \qquad \text{.........4}$$

In finding the analytical design of the bituminous mixtures, two aspects of the material properties will have to be examined. Firstly is the stress- strain characteristic, which determines the critical stresses and strain in the structure. Secondly is the performance characteristic of the material themselves, which are rutting (permanent deformation) and fatigue cracking.

2.0.4 Pavement Performance

2.0.4.1 Distress

Distress is an important factor of pavement design. However, many of the distresses are caused by the deficiencies in construction, material, and maintenance and not related to design.

Table 1 shows all the possible types of distress in asphalt pavements. Table 1 also indicates whether the distresses are structural or functional failures and load–associated or non–load associated. Structural failure is related with the ability of the pavement to carry the design load meanwhile functional failure is associated with ride quality and safety. When structural failure increases in severity, it always results in functional failure as well. The non–load–associated distress is caused by climates, materials or construction. A non–load–associated distress may be increased in severity by traffic loads. Rutting is categorized as functional failure because when the rut becomes large, it can cause hydroplaning and affects the driving manoeuvre. Fatigue cracking is considered as structural failure because the cracks usually occur when the repeated application of stress is less than the tensile strength of the material. But both problems are load-associated failure because it starts at the underside of the wheel.

Rutting which is considered as the most common distress, develops somewhat rapidly during the first few years and then levels off to a much slower rate. Fatigue cracking meanwhile, does not normally occur until after considerable loadings and then increases rapidly as the pavement weakens.

Table 1: Distress in Asphalt Pavement (Huang, 2004)

Types of Distress	Structural	Functional	Load-associated	Non-load-associated
Alligator or fatigue cracking	x		x	
Bleeding		x		x
Block cracking	x			x
Corrugation		x		x
Depression		x		x
Joint reflection cracking	x			x
Lane/shoulder dropoff or heave		x		x
Lane/shoulder separation		x		x
Longitudinal and transverse cracking	x			x
Patch deterioration	x	x	x[a]	
Polished aggregate		x	x	
Potholes	x	x	x	
Pumping and water bleeding	x	x		x
Raveling and weathering		x		x
Rutting		x	x	
Slippage cracking	x		x	
Swell	x	x		x

Note : [a] = Tire abrasion.

2.0.4.2 Rutting Characteristic of Bituminous Mixes

2.0.4.2.1 Description

There are two types of surface deformation namely rutting and shoving. Rutting is a longitudinal surface depression in the wheel path. It occurs as the result of both vertical and horizontal movement of the material in the pavement layers. Shoving is a longitudinal displacement of a localized area of the pavement surface. Vertical displacement is the cause of the shoving.

Figure 5 shows the occurrence rutting, where it originates from the surface of the asphalt layer which then causes the road to depress to a certain depth called rut depth denoted as the black shaded area.

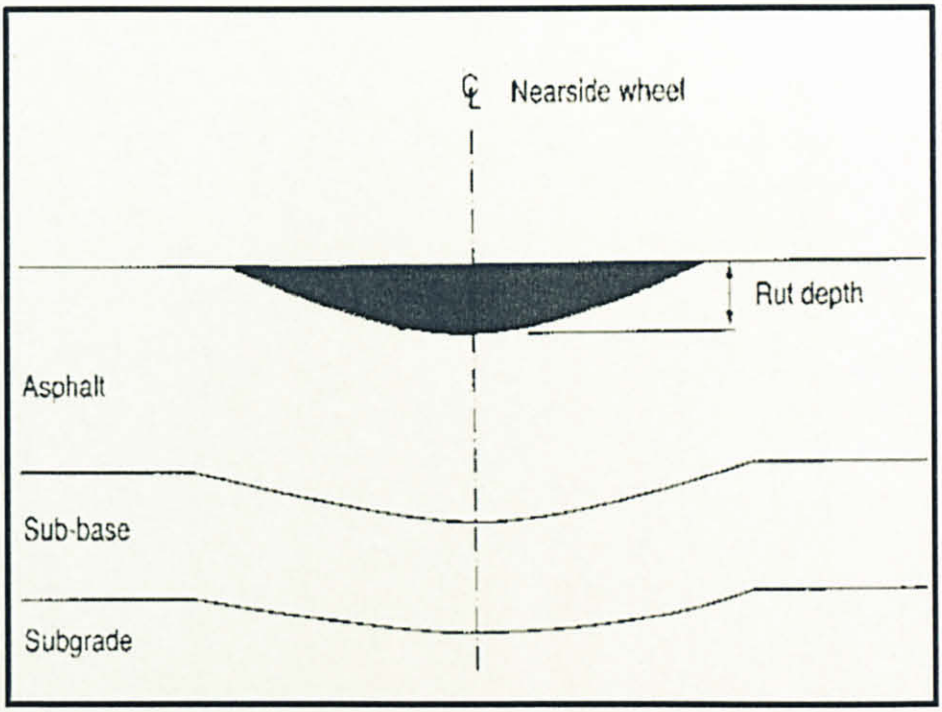

Figure 5: The cross-section of a pavement where rutting occurs (Whiteoak and Read,2003)

Below are some photographs showing example of rutting. Mix rutting (as shown in Figure 6) occurs when the subgrade still has not rut but the pavement surface already exhibits wheelpath depressions as a result of compaction/mix design problems. The picture in Figure 6 was taken at a traffic light area where many vehicles including cars, buses, motorcycles and lorries stop for a certain time. This is an example of the relationship between loading and strain as shown in Figure 8(b), which shows the response to a load pulse induced in an element of asphalt due to moving traffic loads. Although Figure 8(b) shows a small permanent strain for a single pulse load but when millions of load applications are applied to a pavement, a large accumulation will develop. It is this component that results in surface deformation.

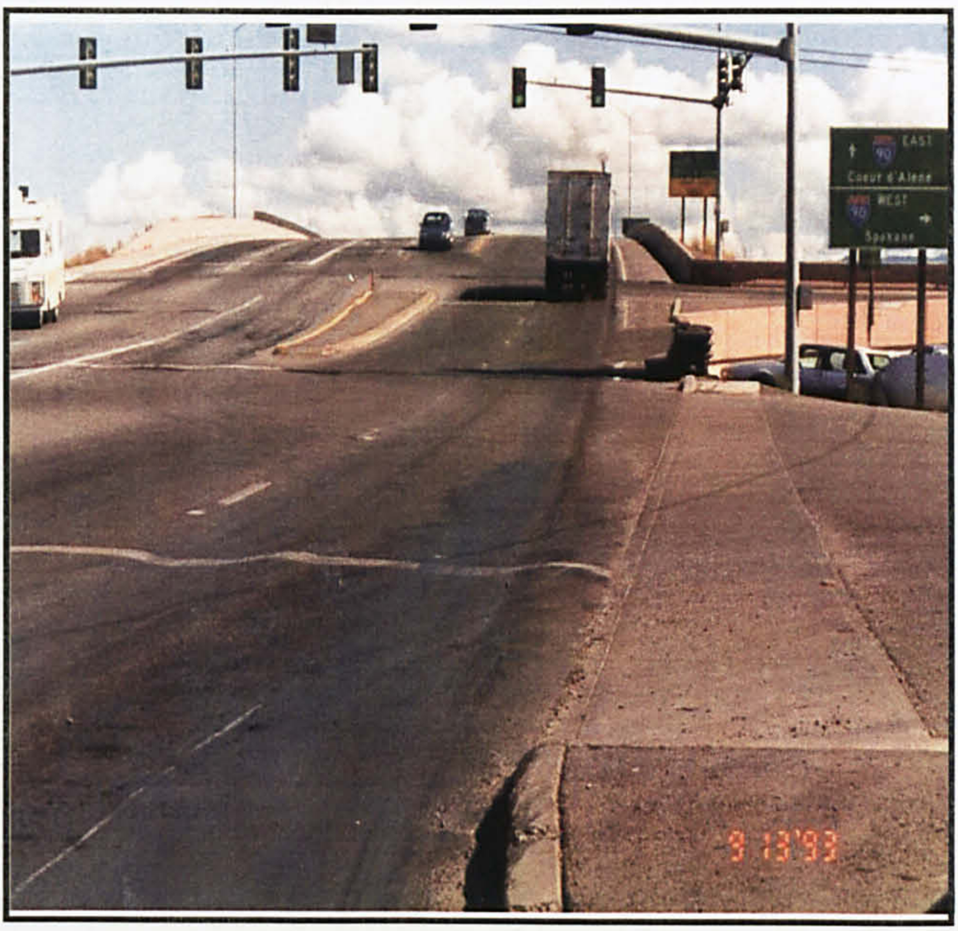

Figure 6: Mix Rutting

Figure 7 shows an example of rutting caused from mix instability. The possible causes of failure could be insufficient compaction, subgrade rutting, poor mix design or studded tyre wear.

Figure 7: Rutting from mix instability

2.0.4.2.2 Severity Levels

Table 2: Tolerances For Surface Irregularities (Jabatan Kerja Raya (JKR) standard JKR/SPJ/1988)

Class of Surface Regularity	Longitudinal Direction				Transverse Direction
	Maximum Permissible Number of Surface Irregularites				Maximum Permissible Depth of Transverse Irregularities
	Depth Exceeding 4 mm		Depth Exceeding 7 mm		
	Over Traverse Length of 300 mm	over traverse length of 75 mm	over traverse length of 300 mm	over traverse length of 75 mm	
Class SR 1	20	9	2	1	4 mm
Class SR 2	40	18	4	2	8 mm
Class SR 3	60	27	6	3	12 mm

The implication of rutting is not always apparent. But, when rut depths progress beyond a certain depth-often as little as 13mm-two major problems occur. Firstly, a serious safety hazard results from the collection of water or ice in temperate countries, which can cause hydroplaning or skidding. Besides that, the rut depth can make lane changing a dangerous manoeuvre. Secondly, cracking along the middle and sides of the wheel paths will accelerate, thus reducing the shelf life for the pavement structure. (Meyer et al., 1974)

2.0.4.2.3 The Theory on Rutting

Asphalt mix is made of aggregate and viscous matrix (bitumen). Because of the viscous matrix, asphalt mix behaves like viscoelastic material. A viscous material is semi-fluid in nature. When stressed, it will deform or tend to deform and any deformation is unrecovered when the loading is removed. Elastic materials also deform or tend to deform when stressed. However, when the loading is removed, any deformation will be fully recovered. Both temperature and loading time govern the degree to which the behaviour will be either viscous or elastic or both. At high temperatures or long loading time, they behave as viscous liquids whereas at very low temperatures or short loading time, they act as elastic (brittle) solids. The intermediate range of temperature and loading times, more typical of conditions in service, results in viscoelastic behaviour. As a result, the stress-strain response depends on the loading rate and applied temperature.

In general, the proportion of any induced strain in asphalt that is attributable to viscous flow i.e. non–recoverable, increases with both loading time and temperature. The effect of this is illustrated in Figures 8(a) and 8(b). Figure 8(a) shows the response of asphalt sample in a simple creep test. The strain resulting from the applied loading shows an immediate elastic response followed by a gradual increase in strain with increase in time until the load is removed. The change in strain with time is caused by the viscous behaviour of the material. With the removal of the load, the elastic strain is recovered immediately and some additional recovery occurs with time. This is known as 'delayed elasticity'. In the end, a permanent residual strain remains, which is irrecoverable and is directly caused by viscous behaviour.

Figure 8(b) shows the response to a load pulse induced in an element of asphalt due to moving traffic loads. Eventhough it is not possible to differentiate between the two components of elastic response but the small permanent strain and larger elastic strain do exist as shown. Due to that, it is clear why more deformation occurs at high temperatures and especially when traffic is slow moving or stationary.

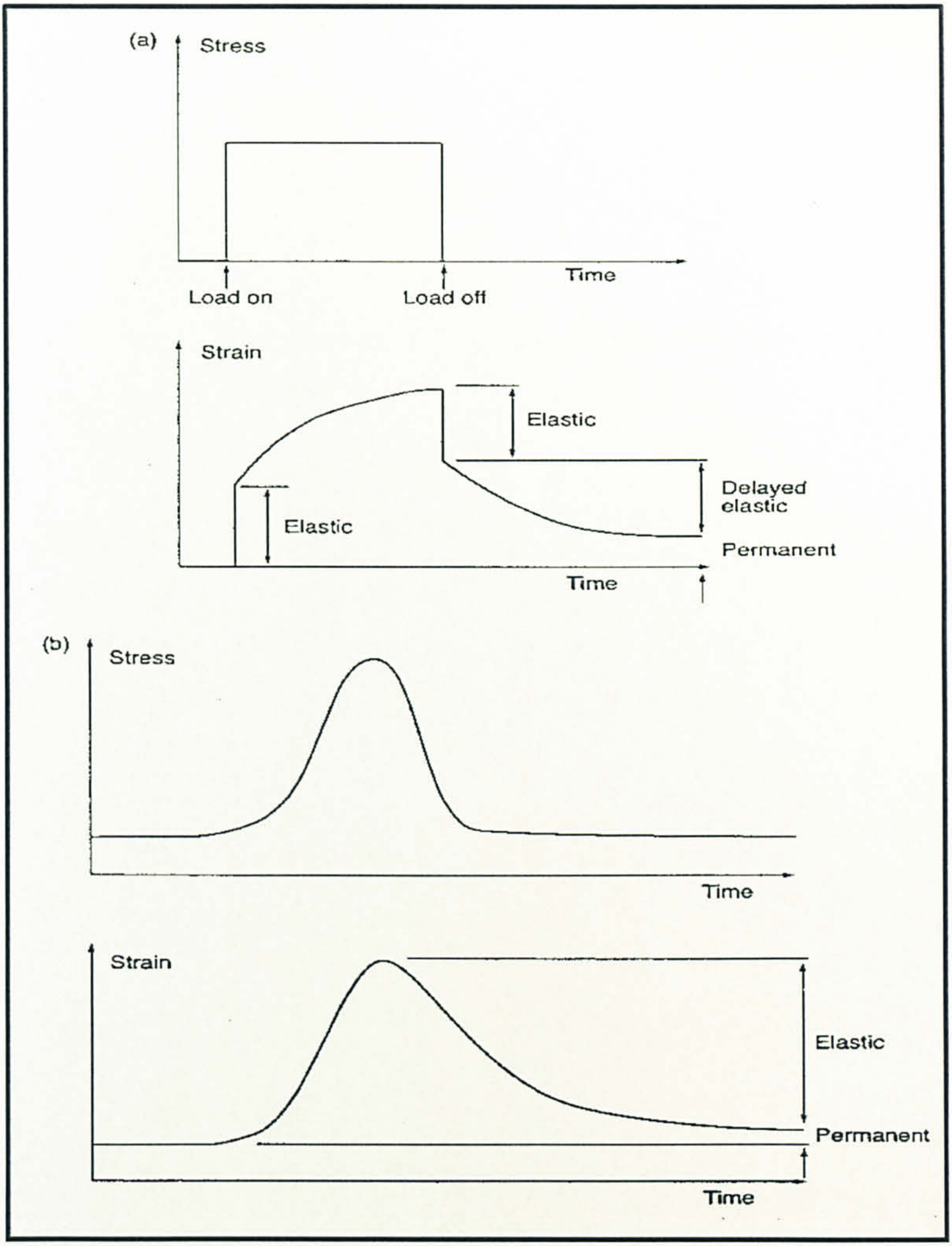

Figure 8: Viscoelastic responses of asphalt under (a) static load and (b) moving wheel load (Whiteoak and Read, 2003)

2.0.4.2.4 Repair

A heavily rutted pavement should be investigated to determine the root cause of failure. Slight ruts (< 9mm deep) can generally be left untreated. Pavement with deeper ruts should be leveled or overlaid.

2.0.5 The Performance of Asphaltic Concrete Bituminous Mix (AC) on Rutting

Shook and Kallas (1969) studied the factors that influenced the dynamic modulus of AC. In their study, continuously graded mixture that resembled Dense Bitumen Macadams was used. The mixture was mixed with two different kinds of bitumen-100 PEN and 50 PEN. At 4.5°C, harder bitumen (50 PEN) produced significantly higher modulus of 19000 MN/m^2 than 100 PEN that only produced 13000 MN/m^2. Besides that, increasing the bitumen content causes the modulus to decrease.

Molenaar et al. did a study to determine the rutting resistance of an AC wearing course mix with 60/70 PEN and the combination of the 60/70 PEN and Refined Buton Asphalt (Retona). Retona 60 is a kind of bitumen extracted from Asbuton with 60% bitumen and 40% natural filler. AC wearing Course type 1 (ACWC-1) with a maximum nominal aggregate size 12.5 mm was used based on Superpave specification. The AC mixture was mixed with four different bitumen namely (60/70 PEN), (10% bitumen Retona 60 + 90% 60/70 PEN), (20% bitumen Retona 60 + 80% 60/70 PEN) and (30% bitumen Retona 60 + 70% 60/70 PEN). From the Repeated Load uni-axial Compression test, a modified asphalt mixture with Retona 60 gives a stronger improvement in the rutting resistance (http://www.vbk.tudelft.nl).

O'Flaherty (2002) concluded that continuously graded mixture such as AC has a very good load-spreading properties and high resistance to deformation due to the aggregate to aggregate contact that governs the mixture strength.

2.1 Hot-Rolled Asphalt (HRA)

2.1.1 Introduction

Hot-Rolled Asphalt (HRA) or also known as Rolled Asphalt is a gap graded mixture where it is much similar to dense graded mixture, except that significant portions of the intermediate sieve (300 μm to 2.38 mm sizes) are missing. Thus resulting in a 'stepped' distribution curve as shown in Figure 9.

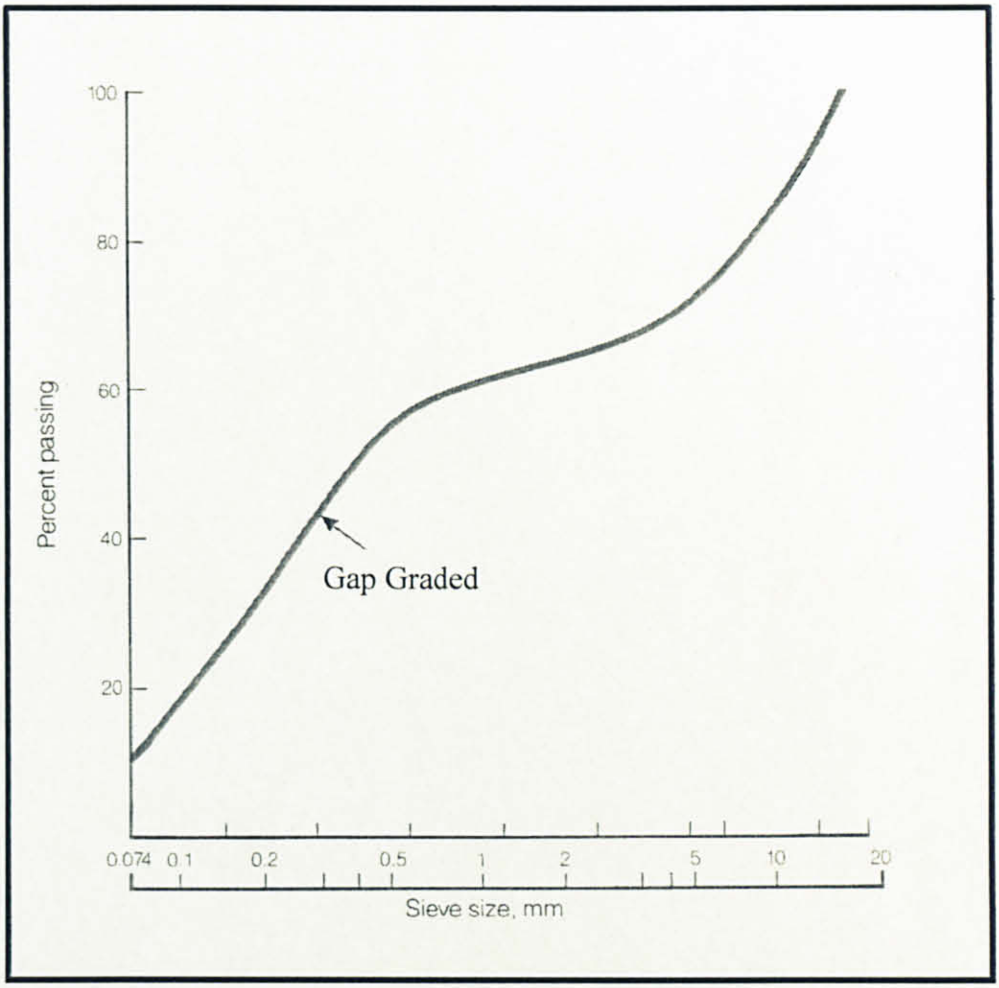

Figure 9: A typical gradation of a gap graded asphalt

Figure 10 shows that gap graded mixes contain a relatively small amount of coarse aggregate and relatively high content of hard bitumen. The stability of gap graded mixes depends on the stiffness of the fine aggregate / filler / bitumen mixture, which form a mortar. When used in residential streets and other lightly traffic applications, they provide a fine textured surface and a workable mix that is more readily compacted to produce low insitu air voids.

The combination of low air voids (3 -8%) and relatively high binder contents, provides an extremely durable surface as well as good fatigue resistance.

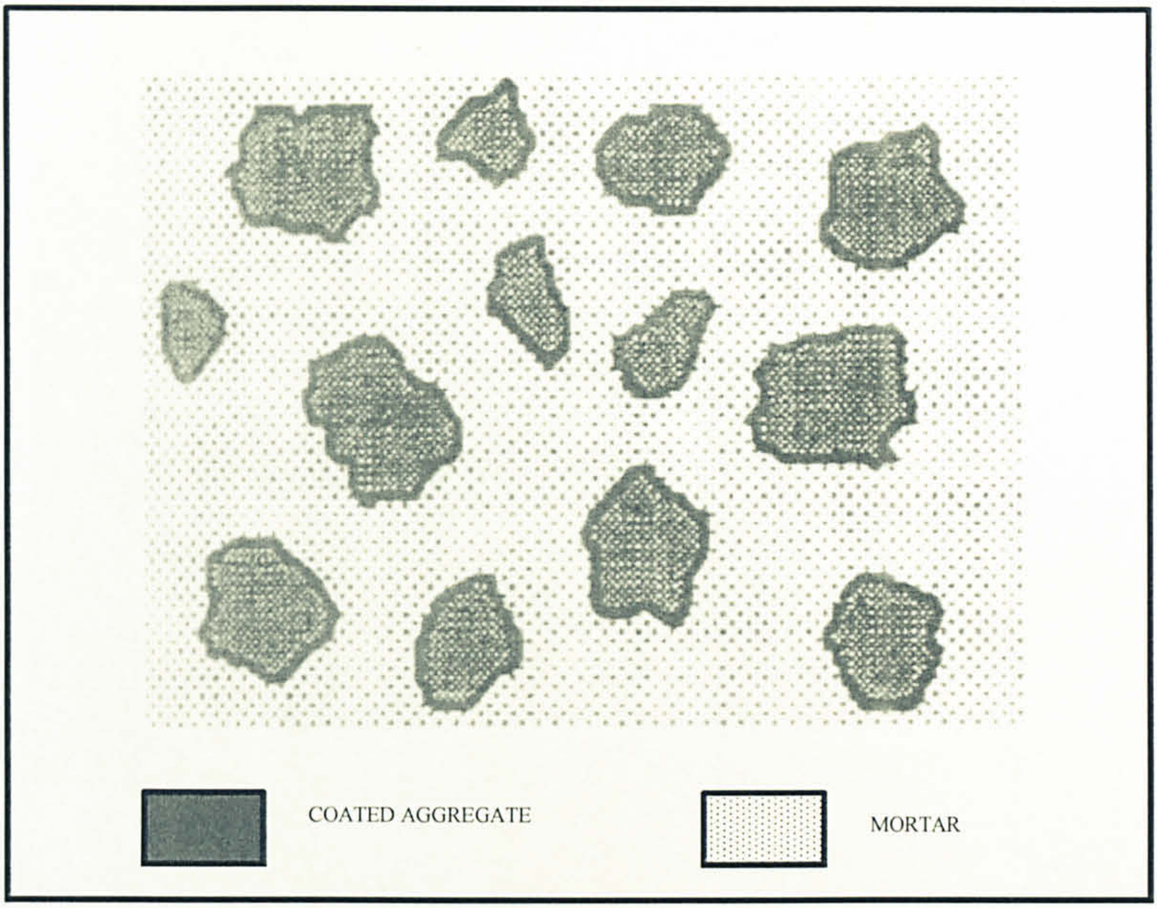

Figure 10: The characteristic of HRA mixture (O'Flaherty, 2002)

2.1.2 Material

2.1.2.1 Coarse Aggregate

BS594: Part 1(BSI, 1992) defines coarse aggregate as the materials that retain on 2.36 mm test sieve and shall consist of the following:

- crushed rock
- gravel
- blast-furnace slag; and
- steel lag

Grading is of secondary importance in BS594 mixture as the mortar primarily governs the mechanical stability. However, it is essential that correct proportion of coarse aggregate is used or otherwise bitumen drainage will occur, resulting in a variation in bitumen content of the finished surface course and hence reduced durability, leaving excess bitumen at the surface (Atkins, 2003).

2.1.2.2 Fine Aggregate

BS594: Part 1(BSI, 1992) defines fine aggregate as the material that passes a 2.36 mm test sieve and shall consist of the following:

- sand
- fines produced by crushing material that would be acceptable as a course aggregate (generally referred to as crushed rock fines); and
- a mixture of sand and crushed rock fines (generally referred to as a blend).

The characteristics of the fine aggregate, which passes a 2.36 mm but retained on a 75 μm BS sieve are very important because it forms a large element of the mortar. Natural sands or sand blends have been traditionally used as fine aggregate in rolled asphalt surface course because it consists of fairly rounded particles that improve workability on site and also permits the use of high bitumen content for a dense and impervious surface.

2.1.2.3 Filler

Filler is considered as any material that passes a 75 μm BS sieve. In BS594 : Part 1 (BSI, 1992), it is stated that added filler shall be limestone or Portland cement, and shall have a bulk density in toluene of not less than 0.5 g/ml and not more than 0.9 g/ml.

As with fine aggregate, high filler content will result in a large surface area of aggregate hence increased bitumen demand. The filler can modify the grading of the fine aggregate to give a denser mixture with greater aggregate contact. Besides that, filler helps bitumen to form the 'binder', which lubricates and binds the fine aggregate to form the mortar.

2.1.2.4 Bitumen

Commonly in HRA, 50 PEN is used for pavement surfaces. It is considered the most important factor in determining the resistance of rolled asphalt to the effect of deformation under traffic.

2.1.3 Fatigue Characteristic of Bituminous Mixes

2.1.3.1 Description

"Fatigue Cracking occurs in areas subjected to repeated traffic loadings (wheel path). It can be a series of interconnected cracks in early stages of development. It will develop into many–sided, sharp-angled pieces usually less than 0.3m (1ft) on the longest side, characteristically with a chicken wire/alligator pattern, in later stages" (SHRP, 1993).

2.1.3.2 Severity Levels

The severity of fatigue cracking can be divided into three categories:

a. Low

Low level is an area of cracks with no or only a few connecting cracks; cracks are not spalled or sealed; pumping is not evident.

b. Moderate

Moderate level is defined as an area of interconnected cracks forming a complete pattern; cracks may be slightly spalled; cracks may be sealed; pumping is not evident.

c. High

High level is an area of moderately or severely spalled interconnected cracks forming a complete pattern; pieces may move when subjected to traffic; cracks may be sealed; pumping may be evident (SHRP, 1993).

Figure 11 shows a drawn picture of the three types of severity levels. Figure 12 shows an example of fatigue/alligator cracking that occur on the road.

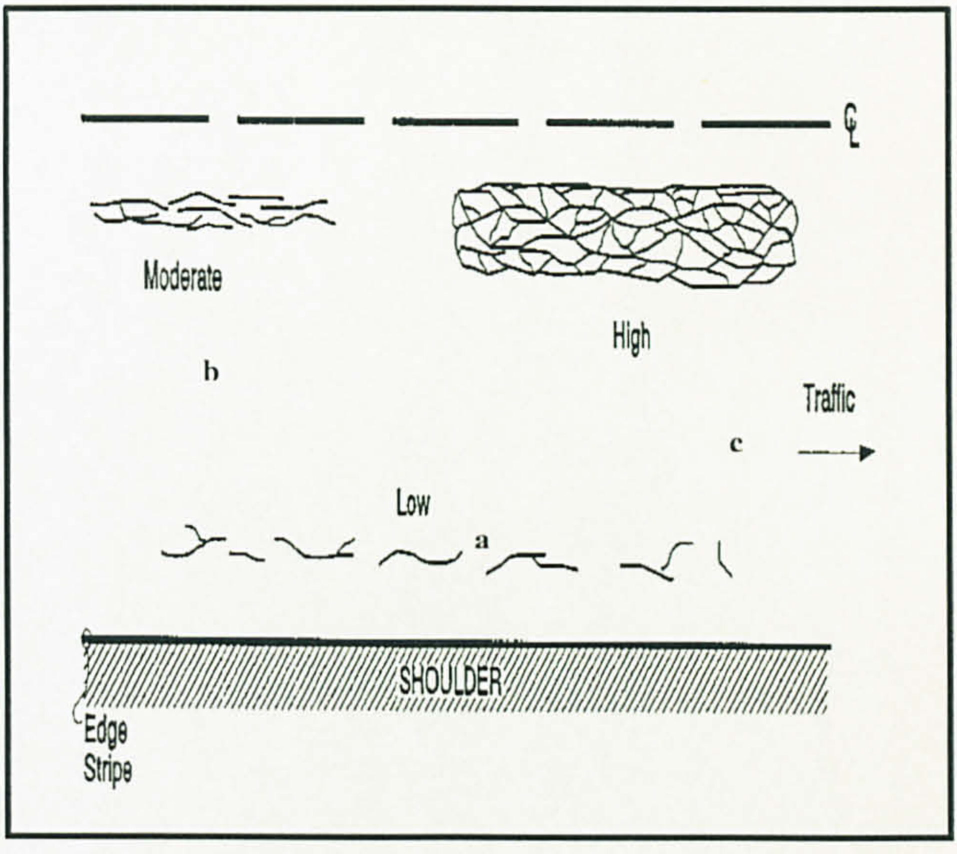

Figure 11: Fatigue cracking (SHRP, 1993)

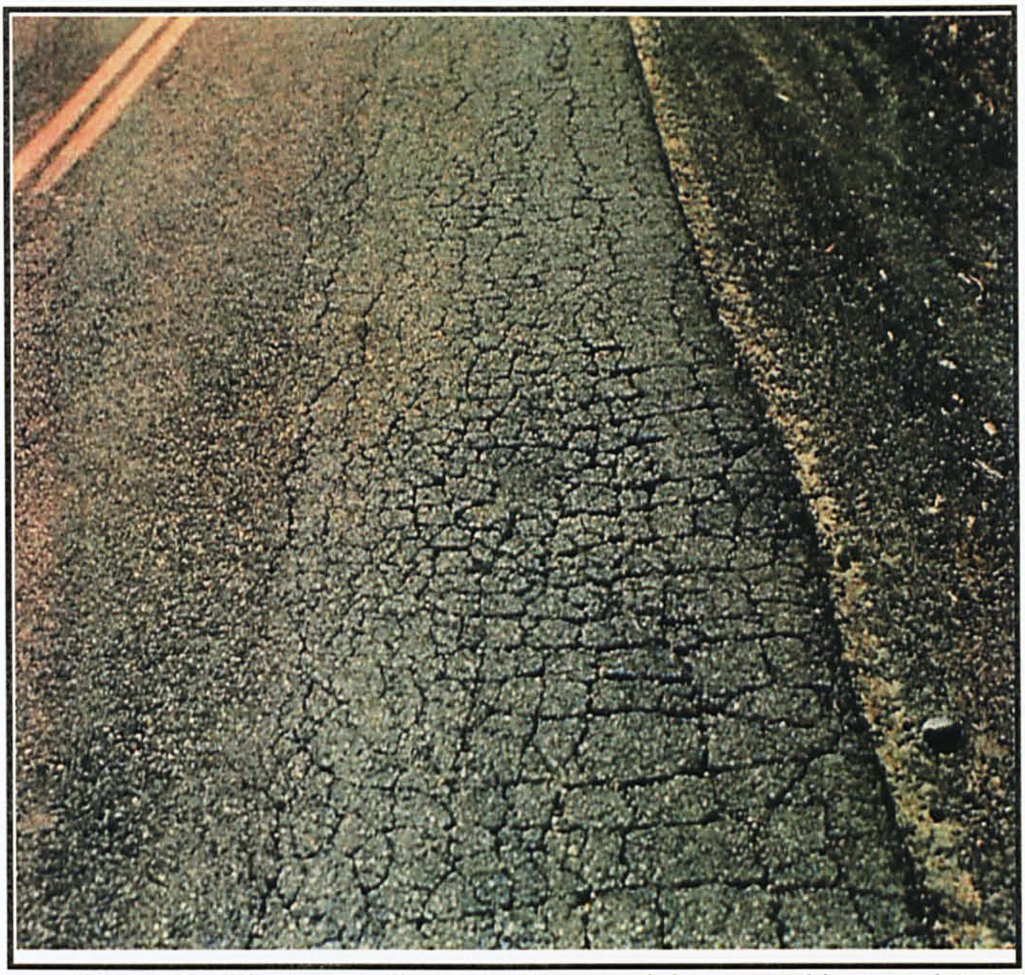

Figure 12: Alligator cracking (http://www.training.ce.washington.edu)

2.1.3.3 The Theory on Fatigue Cracking

Fatigue is considered the main distress mode in pavements associated with repeated tensile strains. It manifests itself in the form of cracking due to traffic loading. In thin pavements, cracking initiates at the bottom of HMA layer where the tensile strain is the highest then propagates to the surface as one or more longtitudinal cracks. This is commonly referred to as "bottom-up" or "classical" fatigue cracking. In thick pavements, the cracks most likely initiate from the top in areas of high localized tensile stresses resulting from tyre-pavement interaction and asphalt binder aging (top-down cracking) (http://www.training.ce.washington.edu).

In the design procedures, which are commonly in use, the maximum tensile strain is calculated at the bottom of the bituminous layer (as shown in Figure 13) (http://www.vbk.citg.tudelft.nl).

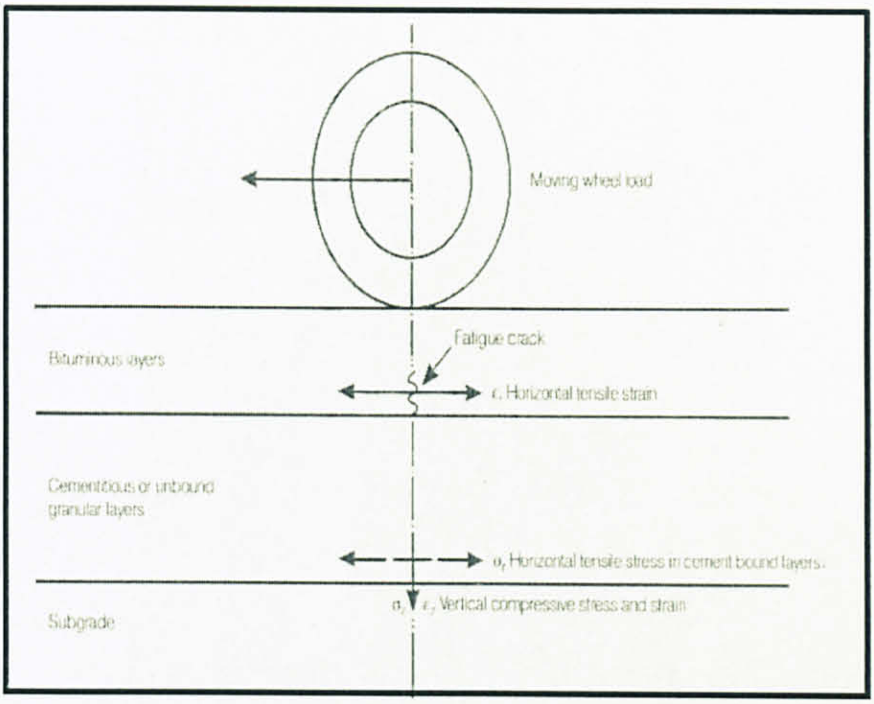

Figure 13: The cross-section of the pavement where fatigue cracking occurs (Whiteoak and Read,2003)

This implies that fatigue cracking initiates at the bottom of the asphalt and propagates upwards. It is very difficult to verify this hypothesis in practice. However, strain measurements in sections tested at the Delft University accelerated pavement testing device, LINTRACK, indicated the presence and growth of cracks at the bottom of the asphalt layer. Furthermore, laboratory produced slabs tested by means of a small scale wheel tracking device also showed that the hypothesis was true (http://www.vbk.citg.tudelft.nl).

2.1.3.4 Repair

A fatigue cracked pavement should be investigated to decide the main cause of the problem. Any investigation should consist of digging a pit or coring the pavement to determine the pavement's structural makeup as well as influential factor as to whether or not subsurface moisture is the contributing factor. Once the characteristic fatigue cracking pattern is apparent, repair by crack sealing is generally ineffective. Therefore, repair is classified into two categories:

- Small, localized fatigue cracking indicative of a loss of subgrade support. For this type of fatigue cracking, the surface of the cracked pavement is first removed. Then the area is dug out and the poor subgrade is replaced. If necessary the drainage of the area is also improved. Finally the repaired subgrade is patched over.
- Large fatigue cracked areas indicative of a general structural failure. The repair work is carried out by placing an HMA overlay over the entire pavement surface. This overlay must be structurally strong enough to carry the anticipated loading because the underlying fatigue cracking pavement most likely contributes no strength at all.

2.1.4 The Performance of Hot-Rolled Asphalt (HRA) on Fatigue Cracking

O' Flaherty (2002) concluded that HRA good resistance to fatigue cracking was due to the quality of fines-filler-bitumen mortar that mainly controlled the HRA's mechanical stability and stress distribution quality.

Read and Brown (1994) performed a study on the fatigue characterization using Indirect Tensile Fatigue test. In their study , they used four types of mixes comprising 30% of 14 mm HRA, Styrene Butadiene Styrene (SBS) polymer modified with 30% of 14 mm HRA, a 20 mm Dense Bitumen Macadam (DBM) and 28 mm DBM 50 PEN. The polymer modified HRA performed a higher initial strain of 200 microstrain, somewhat better than the unmodified HRA that produced 195 microstrain at 2E+5 cycle to failure. The two DBM mixes (20 mm DBM and 28 mm DBM 50PEN) performed less satisfactorily than the HRA with 112 microstrain and 110 microstrain respectively at 2E+5 cycle.

Besides that, Hartman and Gilchrist (2004), who studied fatigue properties on two standard Irish asphalt mixes namely a 30% of 14 mm HRA mix and a 20 mm Dense Base Course (DBC) macadam mix, concluded that the fatigue properties of DBC mix was inferior to those of HRA mix. Unlike Read and Brown who used Indirect Tensile Fatigue test, Hartman and Gilchrist used Beam Fatigue test. Their result, showed that HRA produced a longer fatigue life of 3,638,829 cycles compared to DBC that only produced 169,458 cycles at 100 microstrain.

2.2 Performance Test

2.2.1 Wheel Tracking Test

Wheel tracking devices measure rutting by rolling a small loaded wheel device repeatedly across a prepared HMA sample. Some of these devices are relatively new and some have been used for more than fifteen years like the

Laboratoire Central des et Chausées (LCPC) wheel tracker - also known as the French Rutting Tester (FRT) (http://training.ce.washington.edu).

2.2.1.1 Advantages of Wheel Tracking Test

Cooley et al. (2000) reviewed U.S. loaded wheel testers and found out that results obtained from the wheel tracking devices correlated reasonably well to actual field performance when the in-service loading and environmental conditions of that location were considered. Besides that, wheel tracking devices can reasonably differentiate between bitumen performance grades.

Furthermore, wheel tracking test has been used by many researchers in United Kingdom, America and European countries because it can be carried out in many shapes like Georgia Loaded Wheel Test (GLWT) and Asphalt Pavement Analyzer (APA) where the sample can be fabricated in a shape of a beam or cylindrical. Sample can also be taken from a core extracted from a road structure. Therefore, the laboratory results can be compared with the actual performance of the road structure.

2.2.1.2 Previous Work using Wheel Tracking Test

2.2.1.2.1 Type of Bitumen and Hardness

Uge and Van De Loo (1974) reported that rutting problem can best be tackled by the use of hard bitumen since it has low temperature susceptibility.

Corte et al. (1994) reported that the type of bitumen does affect the improvement of rutting resistance of an unstable mixture. The aggregate used was densely graded Cusset asphalt concrete with 32% ground sand including 1.5% added fines. This was proven when two modified bitumen such as SBS-polymer-modified and Shell Multigrade bitumen produced a lower rut depth compared to the conventional 50/70 PEN bitumen.

Rockliff et al. (1997) did a study with Tilcon (North), Tilcon (South) and Shell Bitumen to evaluate HRA wearing course made with two modified bitumen namely Multiphalte 35/50 and Cariphalte DM. The comparison was made by using conventional 50 PEN bitumen. One of the prime roles of bitumen modifier is to increase the resistance of asphalt to overcome permanent deformation at high road temperatures, without adversely affecting the properties of the bitumen or asphalt at other temperatures. The results showed that the modified bitumen had a lower rut depth compared to the conventional 50 PEN bitumen. Multiphalte 35/50 had a rut depth of 1.2 mm which was half of the 50 PEN bitumen. Furthermore, when the test results were compared with the performance on the road, Multiphalte 35/50 and Cariphalte DM performed substantially better than the conventional 50 PEN bitumen.

2.2.1.2.2 Bitumen Content

Choyce et al. (1984) carried out a study of rutting on HRA. They found that in general, as bitumen content increased, the wheel tracking rate also increased (resistance to deformation decreases). At low bitumen content (6-9%), the wheel tracking rate did not vary markedly (around 2 mm/hr). However, above certain bitumen content, there was a marked increase in wheel tracking rate. The bitumen content at which this change occurred was been found to correspond closely with that at which the aggregate structure was most densely packed. At bitumen contents above this, the voids in the aggregate structure started to become overfilled with bitumen. The excess bitumen acted as a lubricant, destroying internal friction, so drastically reducing resistance to deformation.

Nunn et al. (2000) did a study on ranking four mixtures based on their performance towards rutting resistance. In their study, four kinds of mixtures were used such as Porous Asphalt (PA), HRA, Dense Bitumen Macadam with 4.5% bitumen content (DBM+4.5%) and Dense Bitumen Macadam with 6.5% bitumen content (DBM+6.5%). The test was conducted at a temperature of 40°C. The results showed that (DBM+ 4.5%) ranked the lowest rut rate measurement followed by HRA, PA and lastly (DBM+6.5%). (DBM+6.5%) ranked the worst because the mixture was overfilled with bitumen to the extent that aggregate to aggregate contact was seriously reduced; thus leading to material instability and very low rutting resistance.

2.2.1.2.3 Aggregate Characteristic

Previous work by Brown and Gibbs (1996) demonstrated the importance of good aggregate grading in providing long term deformation resistance.

Soelistijo (1995) did a study on rutting characteristic between HRA and LAC (Leeds Asphaltic Concrete). For HRA, he used conventional bitumen of 50 PEN bitumen whereas for LAC, he used 100 PEN bitumen. LAC is the name of the place where he obtained the aggregate which is from Leeds. From the results obtained, it can be concluded that LAC demonstrated higher resistance to rutting compare to HRA. This was because the continuous aggregate grading of the LAC contributed to build higher resistance to rutting through its aggregate interlocking and it contains less mortar than HRA.

2.2.2 Creep Test

Creep test was first used in order to develop a mathematical model for the behaviour of bitumen and bituminous mixtures under creep loading (Cabrera and Nikolaides, 1988). Samples up to 200 mm long are normally tested with axial stresses of up to 500 kPa on a 100 mm diameter specimen. An optional pre-test conditioning period (0 to 60 minutes) is followed by a test period of up to 10000 seconds. Test data are stored automatically and can be printed out if required (Whiteoak and Read, 2003).

2.2.2.1 Advantages of Creep test

Corte et al. (1994) showed evidence that dynamic creep test to be more representative of what happen in the field. This was proven when the results from the dynamic creep test had the same pattern with the rut depth results obtain from the Laboratoire Central des Ponts et Chaussèes (LCPC) circular test track that was carried out on a pavement. Thus, it was also considered that the dynamic creep test showed a good test for predicting the rutting performance of the types of the mixes that was used in the study.

This test gained wide acceptance due to the ease of specimen preparation, the simplicity of the test procedure and the low cost of test equipment (Whiteoak and Read, 2003).

2.2.2.2 Common Typical Rutting Representations

Hills in Cabrera et al. (1988) published his theoretical model for the rutting of bituminous mixtures under constant uniaxial static load. It was based on the hypothesis that any deformation in the mixture is the result of sliding displacement between adjacent mineral particles, separated by a thin film of bitumen. The rate of strain is dependent on the magnitude of the stresses, the thickness of the bitumen film, and the properties of the bitumen. He found however, that by plotting the creep stiffness of the mixture (S_{mix}) against the stiffness of the bitumen (S_{bit}), the effect of the temperature, the applied stress level, the type of bitumen and the size of specimen on the rate of strain were completely eliminated. The shape of this creep curve depends only on the composition of the mixture and its internal structure. Hills (1973) said that S_{mix} is defined as $\frac{\sigma}{\varepsilon_{mix}}$, σ being the applied stress and the strain in the mix (ε_{mix}) being defined as $\frac{\delta H}{H_o}$, where H_o is the initial height of the specimen and δH the change in height. Using S_{bit} as the independent variable, took account of the type and grade of bitumen in the mix and the time of loading and temperature for the creep test. Furthermore, in 1974, Hills et al. proposed a formula that calculates the rut depth of a pavement from laboratory creep test results after a good correlation was obtained between creep results and rutting. The accuracy of this formula was improved further by Van de Loo. The modified version, which is also used by Shell, reads as follows:

$$RutDepth = C_m \times h \times \frac{\sigma_{ave}}{S_{mix}} \qquad \text{......5}$$

Where C_m = correlation factor for dynamic effect, varies between 1.0 and 2.0

h = layer thickness, mm

σ_{ave} = average stress in the pavement, related to wheel loading and stress distribution

S_{mix} = stiffness of the design mixture from creep test at a certain value of stiffness (S_{bit}), which is related to the viscosity of the bitumen, the duration of the wheel loading and the number of standard axles.

2.2.2.2.1 Prediction of Bitumen Stiffness Modulus (S_{bit})

Van der Poel nomograph (as shown in Figure 14) can predict the stiffness modulus if direct measurement is not feasible. This can be achieved by using only penetration and softening point (Whiteoak and Read, 2003).

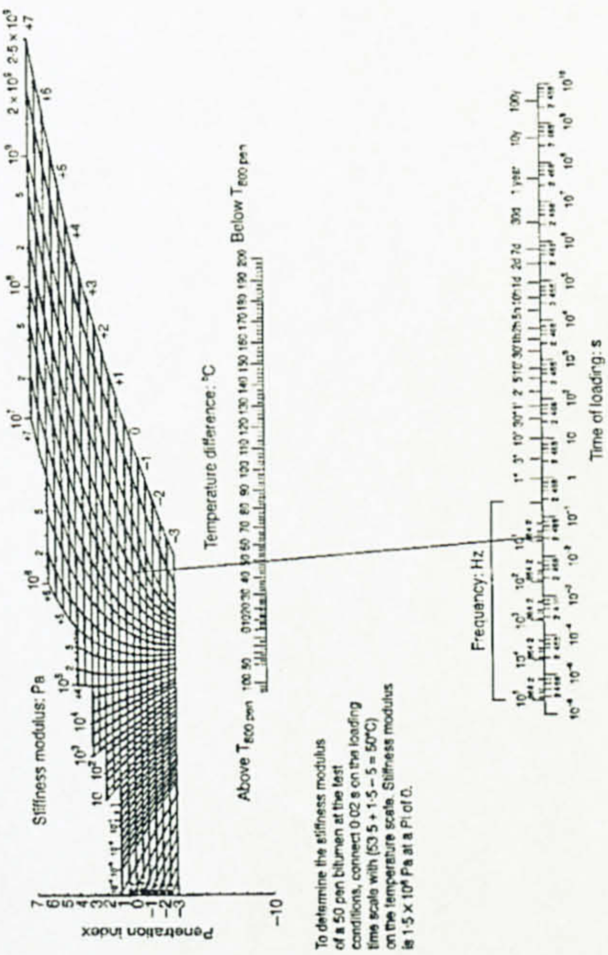

Figure 14: Nomograph for determining the stiffness modulus of bitumen

2.2.2.3 Previous Works using Creep Test

2.2.2.3.1 Type of Bitumen & Hardness

Cabrera et al. (1988) evaluated the rutting performance of cold dense bituminous mixtures by using Canik U L creep machine developed by Leeds University, which was suitable to test cold bituminous emulsion mixtures as well as hot bituminous mixtures. The selected mixtures were Cold Rolled Asphalt (CRA) bitumen emulsion mixtures and Cold Dense Macadam (CDMc) bitumen emulsion mixture.

In this study, the bituminous emulsion used was a slow setting cationic emulsion, called NH-10, specially formulated for dense mixtures and particularly for rolled asphalt mixtures. The examination of creep performance of the two bituminous emulsion mixtures revealed that bitumen content significantly affected the creep stiffness and the rate of change of creep stiffness only for the CRA mixtures.

In general, the gap graded cold bituminous emulsion mixtures (CRA) are more sensitive to the factors affecting the permanent deformation compared to continuously graded cold macadam. Additionally, CRA mixtures are expected to rut slightly more than CDMc mixtures. It is therefore very important, especially when CRA mixtures are used, to determine accurately the optimum bitumen content and added water in order to avoid excessive rutting.

They also compared the rut depth results with the hot conventional mixtures. It was suggested that emulsion treated mixtures can be used as base course material without undergoing any greater deformation than conventional hot mixtures.

Napiah (1993), who also used the Canik U L creep machine in determining the rutting performance of polymer modified

HRA, found out that the effect of polymer additives can be seen when the addition of Ethylene-Vinyl-Acetate (EVA) and Styrene-Butadiene-Styrene (SBS) to the base penetration 50 PEN considerably reduced the creep of mixtures compared to the conventional mixture (base 50 PEN bitumen) alone. This maybe due to the addition of the stiffening effect of the polymer on the existing bituminous mixture that helped improved better creep performance.

2.2.2.3.2 Bitumen Content

Napiah (1993) also discovered the effect of the bitumen content on the creep properties of the HRA mix. Bitumen content of the mixes containing polymer modified bitumen was varied by 0.5% below and above the optimum bitumen content. The result indicated that at 40°C, the vertical strain of the mixes containing 0.5% bitumen below and above the optimum did not differ significantly from the strain experienced by the mixes at optimum bitumen content. However, at 60°C the mixes with 0.5% more and 0.5% less bitumen experienced higher strains compared to the mixes with optimum bitumen content at 40°C. He also found out that at 60°C, the correct amount of bitumen was an important factor for achieving a mix with good creep properties.

2.2.2.3.3 Aggregate Characteristic

Uge and Van De Loo (1974) stated that mixes made from angular aggregate (obtained for instance by crushing materials) deformed to a minor extent and are more stable than mixes having the same composition and grading curve but made from round shaped aggregate (river gravels). This is proven when at the same void content, a mix composed of angular aggregate have a better resistance to flow than a rounded material composition.

2.2.3 Beam Fatigue Test

Researchers have developed a number of tests to measure the fatigue characteristics of bituminous mixes which are shown diagrammatically in Figure 15. Bending tests using beams or cantilevers involving repeated applications of load have been used to determine fatigue resistance. Fatigue tests are carried out by applying a load to a sample in the form of an alternating stress or strain and determining the number of load applications required to induce ' failure' to the sample.

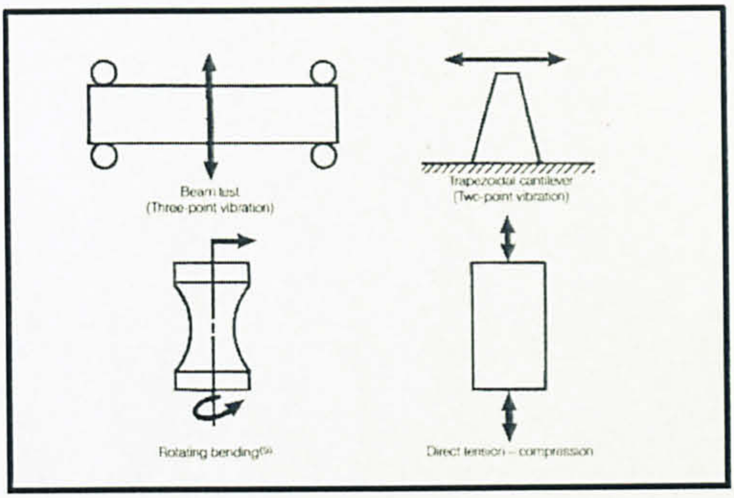

Figure 15: Different configurations for measuring the fatigue life of bituminous sample (Whiteoak and Read,2003)

2.2.3.1 Advantages of Beam Fatigue Test

Huang (2004) says that any weak spot due to nonuniform material properties will show up in the test result due to the existence of a constant bending movement over the middle third of the sample.

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 bitumen type and aggregate grading.

2.2.3.2 Common Typical Fatigue Representations

Fatigue life (N_f) is defined as the number of load cycles to fail the asphalt concrete at strain (stress) occurring at the bottom of asphalt layer (Ghuzlan and Carpenter, 2002).

Failure criterion used to interpret the fatigue test results is very important to establish fatigue life. Huang (2004) wrote in his book that there are two types of loading that can be applied for testing beam fatigue test namely constant stress and constant strain, as shown in Figure 16. In a constant stress test, the stress remains constant but the strain increases with the number of repetitions. In a constant strain test, the strain is kept constant, and the load or stress is decreased with the number of repetitions.

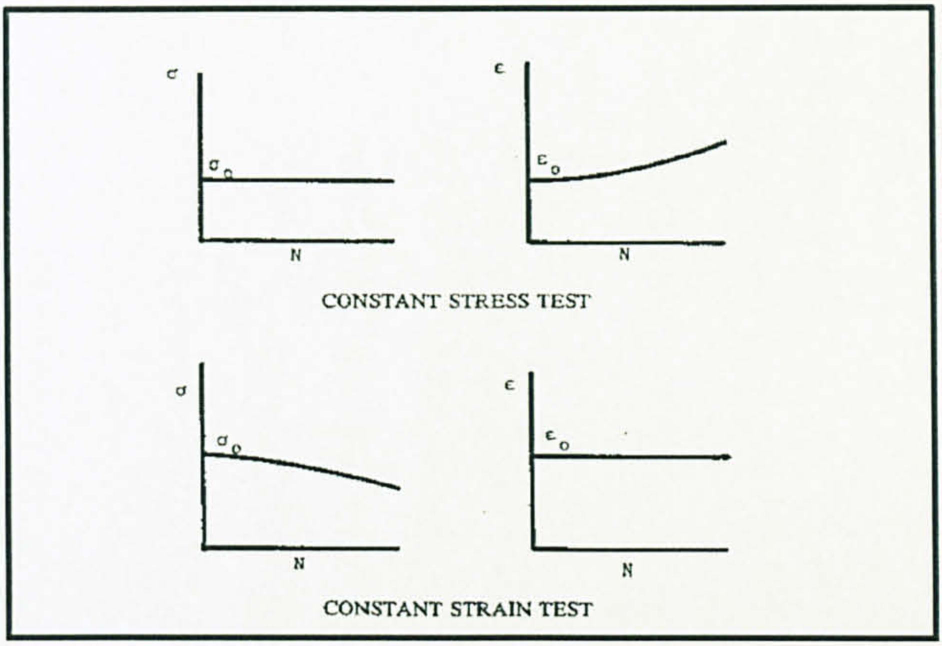

Figure 16: Two types of constant loading for fatigue testing (Huang, 2004)

The constant stress type of loading is applicable to thicker pavements, wherein the HMA is more than 152 mm thick and is the main load–

carrying components. As the HMA becomes weaker under repeated loads, the strain should increase with the number of repetitions. The constant strain type of loading is applicable to thin pavements with HMA less than 51 mm thick because the strain in asphalt layer is governed by the underlying layers and is not affected by the decrease in stiffness of HMA.

In a constant strain test the sample is usually deemed to have 'failed' when the load required to maintain that level of strain has fallen to 50% of its initial value. Whereas in a constant stress the failure is well defined since the specimens are cracked through at the end of the test (Tayebali et al., 1992).

When a sample is subjected to a loading cycles in the laboratory, it can be subjected to either a constant stress or a constant strain level. The results are shown in Figure 17, where the number of cycles of different stress or strain levels required to produce failure are shown as a function of the stress or strain level.

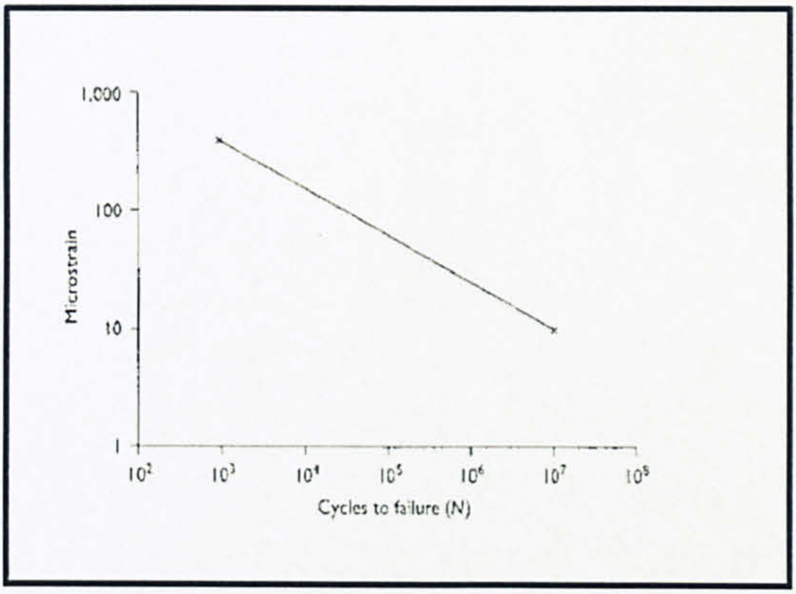

Figure 17: Fatigue laws for pavement materials (Robinson and Thagesen, 2003)

The fatigue life (N_f) of the bituminous mixes that utilizes the flexural bending beam fatigue test (third-point loading) and considers bottom-up cracking can be expressed in the form of an equation (equation 6) (Walubita, 2006).

$$N_f = K_1\left(\frac{1}{\varepsilon}\right)^{K_2} \qquad \text{......6}$$

Ghuzlan and Carpenter (2002) suggested that the value of K_2 signify the slope and usually the range of values is between 3 until 6 whereas K_1 may vary by several magnitudes. In some models, the strain is replaced by stress.

$$N_f = K_1\left(\frac{1}{\sigma}\right)^{K_2} \qquad \text{......7}$$

Where N_f = Number of load cycles to fatigue failure

ε = strain.

σ = stress

K_1 *and* K_2 = constant for any mix and depends on the mix composition.

In fatigue testing, it is normal to test several samples at each stress or strain level and the results are plotted as stress or strain vs. cycles to failure on a log–log graph as shown in Figure 18 (Whiteoak and Read, 2003).

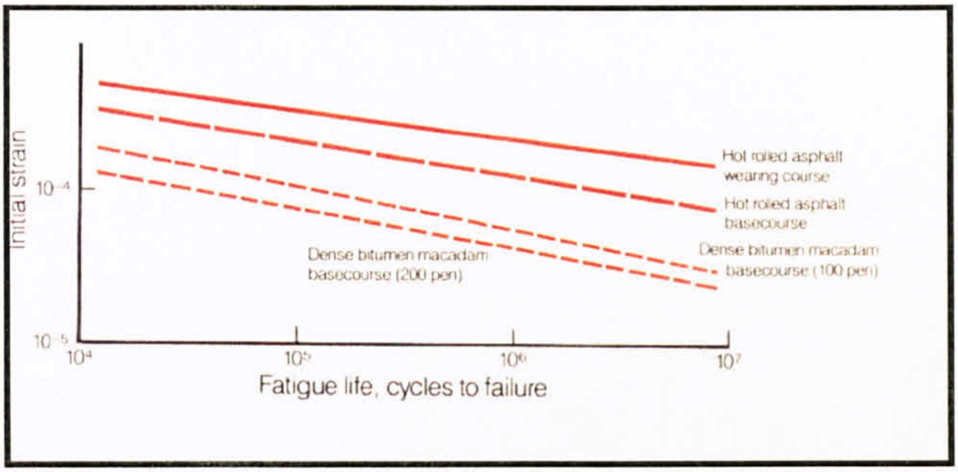

Figure 18: Fatigue properties of different mixes (constant stress) (Whiteoak and Read, 2003)

Pais et al. did a study on the number of strain levels and number of test repetitions required to obtain fatigue characteristics of bituminous mixture. In their study, eight dense graded bituminous mixtures were combined with three types of bitumen, which were 10/20 PEN, 35/50 PEN and 50/70 PEN, modified with SBS polymer. The flexural fatigue test was conducted as a constant strain test and the stress (stiffness) decreased as the sample failed. Three strains (high, medium and low) 700E-6, 300E-6 and 100E-6 were chosen respectively. They found out that the use of three strain levels produced more accurate results (fewer variables) on the estimation of fatigue life. The variability in the estimated fatigue life will reflect uncertainties in fatigue testing. Furthermore, the fatigue life shows great variation if less than three test replicates for each strain level were performed. Therefore, the fatigue life should be obtained with three strain levels and at least three tests replicates for each strain level (http://www.civil.uminho.pt).

2.2.3.3 Previous Works using Beam Fatigue Test

2.2.3.3.1 Type of Bitumen and Hardness

Artamendi and Khalid did an investigation on the fatigue characteristic of two typical bituminous materials namely Dense Bitumen Macadam (DBM) and Stone Mastic Asphalt (SMA). In this study two modes of loading were used which were constant strain and constant stress. All tests were conducted at a temperature of 10°C and at a frequency of 10 Hertz. For constant strain testing, the strain amplitude levels selected varied between 125 and 200 μm/m whereas for constant stress testing, the stress amplitude levels selected were 1.25 and 2 MPa. The result showed that the fatigue life for SMA material was longer than that for DBM material. The result also suggested that the fatigue lives from constant stress tests were shorter than those from constant strain tests for both materials. Longer lives for the SMA material are attributed to volumetric composition and type of bitumen. SMA material contained 50 PEN bitumen meanwhile DBM contained 100 PEN bitumen (http://www.hallf.kth.se).

Dijk et al. (1972) demonstrated that strain has a major influence on fatigue life. This is shown when fatigue life decreases as the strain increases. For the same stress values with smaller strain, increasing the stiffness modulus of the bitumen produces longer life. Under the same conditions of temperature and frequency, a longer life will be obtained with harder bitumen.

2.2.3.3.2 Bitumen Content

Dijk (1972) reported that strong compaction and high content of hard bitumen have beneficial effect on the fatigue life of bituminous mixes.

Napiah (1993) studied the fatigue performance of the polymer modified HRA and found out that bitumen content did affect fatigue properties. In his study, a constant stress mode of loading was used and all testings were conducted at a temperature of $23 \pm 1^{\circ}$C. The results showed that beam containing both polymer modified, Styrene-Butadiene-Styrene (SBS) and Ethylene-Vinyl-Acetate (EVA) had a superior fatigue properties compared to control beam made of base 50 PEN bitumen. This was proven when 7.3% of 5EVA had a longer fatigue life of 3000 cycles compared to the 7% of B50 that had a fatigue life of 210 cycles at the same mixing and compaction temperature of 160/140. The same comparison was also made between 7.7% of 7SBS and 7% of B50. Again the polymer modified mix had a longer fatigue life of 4300 cycles. The results also indicated that varying the bitumen content had an effect on the fatigue properties. The study showed that an increase in bitumen content has a significant increase of fatigue life.

Hajj et al. (2005) did a study on the fatigue characteristics of hot mix asphalt (HMA) mixtures designed using the Superpave volumetric method and Nevada‘s Hveem method. The mode of loading chosen for this test was constant strain and the test temperature used was 22°C. The results indicated that regardless of the design method, higher asphalt bitumen content could generate a better laboratory fatigue resistance. This is supported where the Superpave mixtures having higher percent of bitumen content (0.6-1.6% difference) compared to the Nevada's Hveem mixtures.

2.2.3.3.3 Aggregate Characteristic

Hartman and Gilchrist (2004) did a study on fatigue characteristic between two mixtures, which were HRA and dense base course (DBC) macadam mixture. The test was conducted at a temperature of 20°C and at a frequency of 4 Hertz. The outcome showed that the fatigue properties of DBC mixture were inferior to those of the HRA mixture. This was due to the consequence of the larger aggregate content and maximum aggregate size.

2.3 Summary

Previous works by many researchers have shown that wheel tracking and dynamic creep test are considered as a reliable testing method to determine rutting or permanent deformation performance. Both testing will be conducted at a temperature of 40°C. The gradation plays an important part in resisting rutting especially in AC where the gradation is a continuous gradation that contributes to the strength of the mixture.

In this research, a formula used by Hills will be used to calculate the rut depth because this formula produced a good correlation between creep results and rutting.

$$RutDepth = C_m \times h \times \frac{\sigma_{ave}}{S_{mix}}$$

Where C_m = correlation factor for dynamic effect, varies between 1.0 and 2.0

h = layer thickness, mm

σ_{ave} = average stress in the pavement, related to wheel loading and stress distribution

S_{mix} = stiffness of the design mixture from creep test at a certain value of stiffness (S_{bit}), which is related to the viscosity of the bitumen, the duration of the wheel loading and the number of standard axles.

Meanwhile, the beam fatigue test will be conducted in a constant strain mode of loading where the failure is measured when the stiffness is reduced to 50%

of the initial stiffness. This mode is chosen because the thickness of the pavement is less than 152 mm.

The fatigue life the will be calculated using formula as shown below,

$$N_{ff} = K_1 \left(\frac{1}{\varepsilon} \right)^{K_2}$$

Where N_f = Number of load cycles to fatigue failure

ε = strain.

K_1 *and* K_2 = constant for any mix and depends on the mix composition.

Besides that, three strain levels will be used to produce an accurate fatigue life. The strain levels will be varied from the low strain to the high strain. The testing would be conducted at a room temperature of 20°C where the bitumen would be stiffer. Here the fatigue resistance would be more pronounced.

CHAPTER THREE

METHODOLOGY

3.1 Experimental Programme

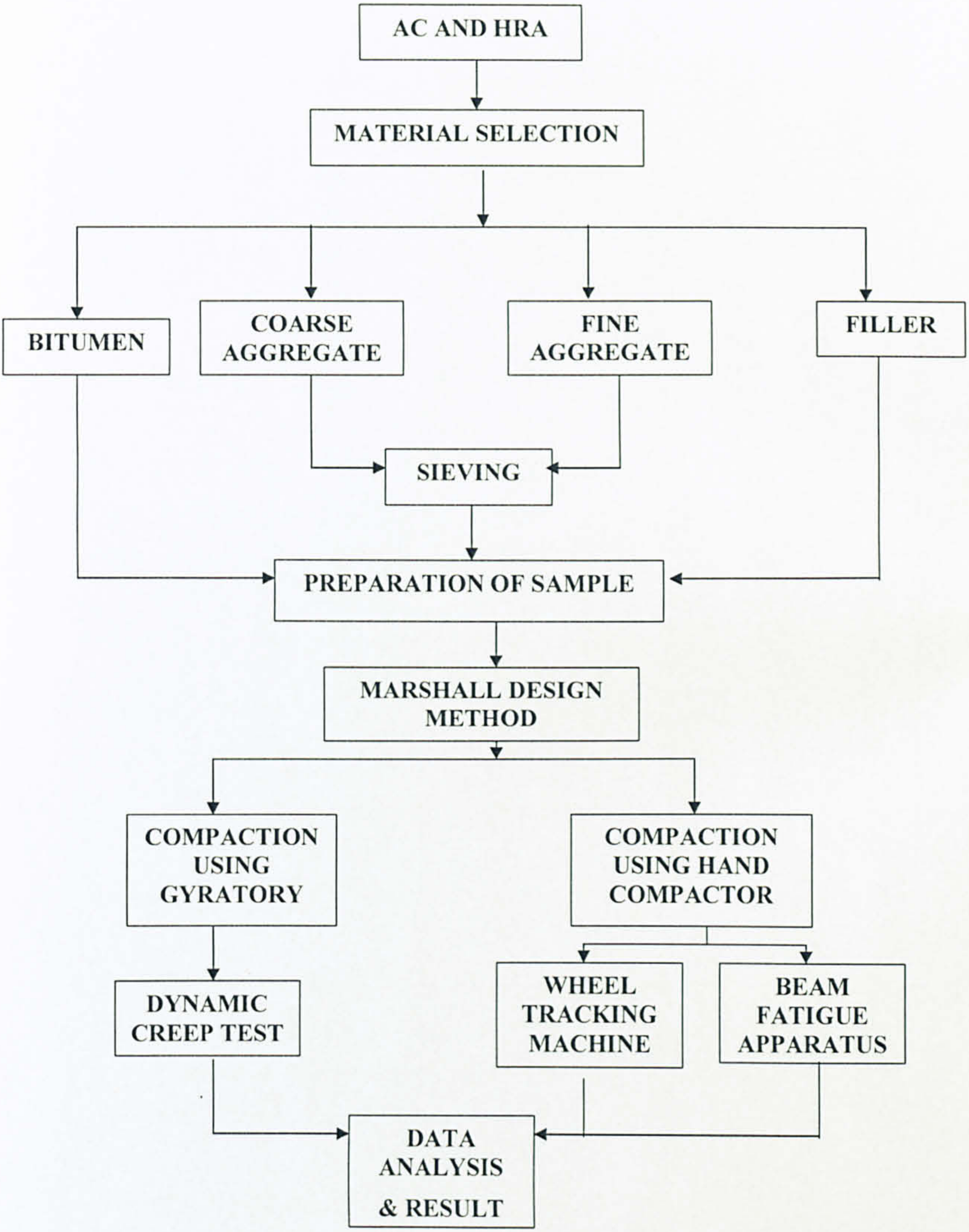

Figure 19: Flow Chart of Research Methodology

Figure 19 is the flowchart of the research methodology for this study. The AC was prepared according to Jabatan Kerja Raya (JKR) standard JKR/SPJ/1988 whereas HRA was prepared according to the British Standard BS594: Part 1: 1992.

The optimum bitumen content was determined using Marshall Design method. At least three samples were fabricated for each mix. Dynamic creep test (testing method was in accordance to BS DD226) and wheel tracking test (testing method was in accordance to BS 598) were used to determine the permanent deformation whereas beam fatigue test was conducted to study the fatigue cracking performance.

3.2 Material Properties

The specific gravity for the material used in this research is as listed below in Table 3. Granite was used as coarse aggregate whereas sand river and Portland cement were fine aggregate and filler respectively.

Table 3: The specific gravity of the material

Material	Specific Gravity
Granite	2.695
River Sand	2.652
Portland Cement	3.150
80 PEN bitumen	1.020
50 PEN bitumen	1.025

Besides the specific gravity of the material, two other parameters such as the penetration and softening point values were also determined. These values were important for determining the stiffness modulus of the bitumen in the dynamic creep test.

Penetration is an empirical test method. A sample of the bitumen in a small container is stabilized in a water bath at standard temperature (normally 25°C).

A prescribed needle, weighing about 100g, is allowed to bear on the surface of the bitumen for 5 seconds. The penetration is defined as a distance, measured in tenth of millimetre (decimillimetre, dmm) that the needle penetrates into the bitumen. The test is as illustrated in Figure 20. The penetration value for 80 PEN bitumen was 8 mm whereas for 50 PEN bitumen was 5 mm.

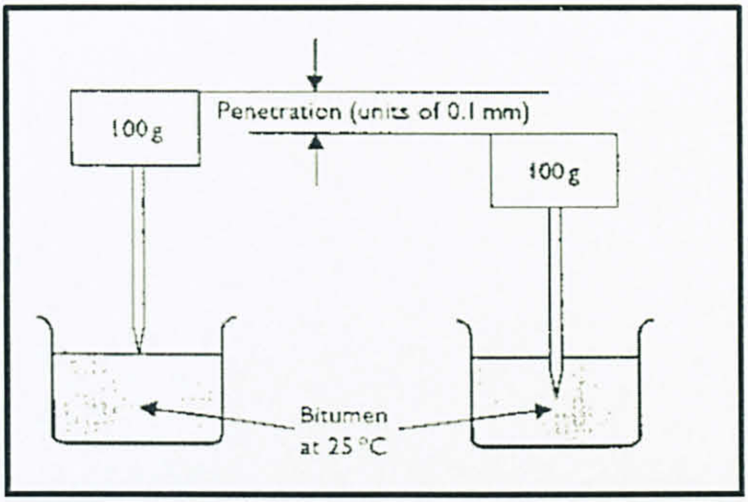

Figure 20: Penetration test (Robinson and Thagesen, 2003)

The softening point test is also an empirical test. As shown in Figure 21, a steel ball weighing about 3.5 g is placed on a bitumen sample contained in a brass ring, which is then suspended in a water or glycerine bath at a temperature of 5°C. Water is used for bitumen with a softening point of 80°C or below and glycerine is used for softening point greater than 80°C. The water is heated at a rate of 5°C per minute. The bitumen starts to soften and eventually will sag under the weight of the ball. The temperature at which the bitumen and the steel ball fall over a distance of 25.4 mm, is the softening point. The softening point for 80 PEN bitumen was 44°C and for 50 PEN bitumen was 53.5°C.

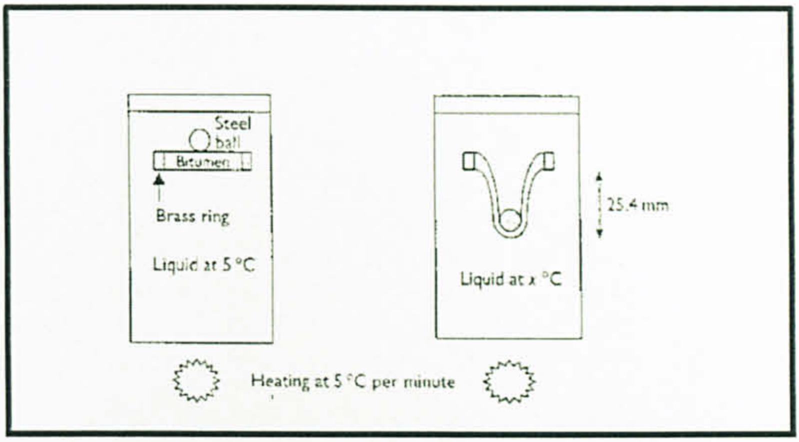

Figure 21: Softening point test (Robinson and Thagesen, 2003)

3.3 Preparation of Sample

3.3.1 Material Gradation for AC

Gradation test was used for quality control purposes and to check the compliance with the specification. It provided the quantities, expressed in percentages by mass, of the various aggregate particle sizes for a mixture.

The gradation used in this research was based on the criteria specified by the JKR/SPJ/1988. Table 4 shows the gradation limit specified by this standard. Sieve size from 28 mm until 5 mm is considered as coarse aggregate, 3.35 mm until 150 μm are fine aggregate and 0.075 mm (75 μm) is filler.

In this research, the wearing course ACW 20 gradation limit was used as a point reference. The sieve size starts at 28 mm and ends at 75 μm.

Table 4: Gradation limits for AC specified in Jabatan Kerja Raya (JKR) standard JKR/SPJ/1988

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

From a trial and error method, the amount of each element was calculated. Coarse aggregate used was 42% whereas fine used was 52% and filler used was 6% of the total weight of the sample. Table 5 shows the combination of aggregate for ACW 20.

Table 5: The combination of sieve analysis following Jabatan Kerja Raya (JKR) standard JKR/SPJ/1988

Sieve Size (mm)	Asphaltic Concrete Bituminous Mix (AC) JKR Standard ACW 20			Total Sample that Passing			Mix Gradation
				Coarse Aggregate (42%)	Fine Aggregate (52%)	Filler (6%)	
28	100	-	100	100	100	100	100.00
20	76	-	100	98.97	100	100	99.57
14	64	-	89	66.61	100	100	85.98
10	56	-	81	28.58	100	100	70.00
5	46	-	71	1.83	97.95	100	57.70
3.35	32	-	58	0	93.40	100	54.57
1.18	20	-	42	0	63.62	100	39.08
0.425	12	-	28	0	21.24	100	17.04
0.15	6	-	16	0	0.89	100	6.47
0.075	4	-	8	0	0	100	6.00

As shown in Figure 22, the sample, which is presented as blue line in the graph, lies within the upper limit pink line and the lower limit of JKR requirement yellow line.

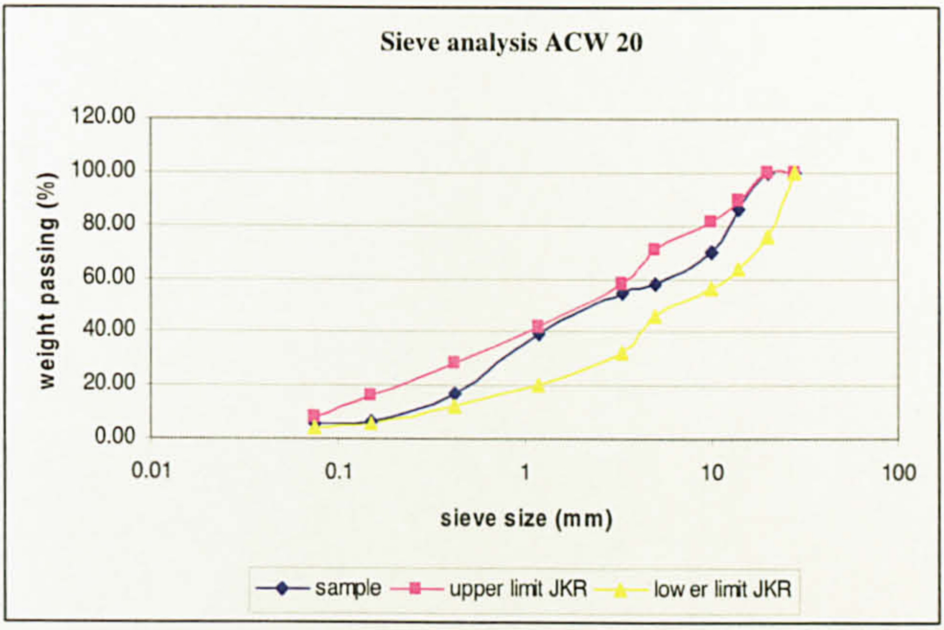

Figure 22: The gradation analysis for ACW 20

3.3.2 Material Gradation for HRA

Table 6 shows the gradation limit specified by the British Standard BS594: Part 1:1992. In this research, the wearing course Type F 30/14 gradation limit was used as a point reference. The sieve size starts at 20 mm and ends at 75 μm.

Table 6: Gradation limits for HRA BS594: Part 1: 1992

Usage	Base course/Roadbase		Wearing Course*
Designation	**50/20**	**60/20**	**30/14**
Grading:			
% by mass passing BS sieve			
28 mm	100	100	
20 mm	90-100	90-100	100
14 mm	65-100	30-65	85-100
10 mm	35-75	-	60-90
6.3 mm	-	-	-
2.36 mm	35-55	30-44	60-72
0.6 mm	15-55	10-44	45-72
0.212 mm	5-30	3-25	15-50
0.075 mm	2-9	2-8	8-12
% aggregate passing 2.36 mm			
and retained on 0.6 mm			
Maximum	-	-	14
Bitumen content:			
% by mass of total mix±0.6%			
Crushed rock or steel slag	6.5	5.7	7.8
Gravel	6.3	5.5	7.5
Blastfurnace slag of bulk density:			
1440 kg/m^3	6.6	5.7	7.9
1280 kg/m^3	6.8	6	8.1
1200 kg/m^3	6.9	6.1	8.2
1120 kg/m^3	7.1	6.3	8.3
Layer depth (mm):			
Thickness	45-80	45-80	40

Note: * = Type F mix

Table 7 shows the gradation used in this research based on the criteria approved by the British Standard BS594: Part 1:1992. The sieve size starts at 20 mm until 0.075 mm (75 μm). From a trial and error method, the amount of each element was calculated. Coarse aggregate used was 34% fine 56% and filler 10% of the total weight of the sample. A graphical representation of the gradation is illustrated in Figure 23.

Table 7: The combination of sieve analysis following BS 594:Part 1:1992

Sieve Size (mm)	Hot-Rolled Asphalt (HRA) BS 594 : Part 1 : 1992 50/14 composition of base course mixture			Total Sample that Passing			Mix Gradation
				Coarse Aggregate (34%)	Fine Aggregate (56%)	Filler (10%)	
20	100	-	100	100.00	100	100	100.00
14	85	-	100	86.00	100	100	92.58
10	60	-	90	66.49	100	100	82.24
2.36	60	-	72	0	97.15	100	45.86
0.6	45	-	72	0	90.35	100	43.14
0.212	15	-	50	0	36.35	100	21.54
0.075	8	-	12	0	3.20	100	8.28

As shown in Figure 23, the sample which is presented in blue line lies between the upper and lower limit of the British Standard BS594: Part 1: 1992, which were marked as pink and yellow line respectively. The curve of the blue line is almost the same as in Figure 9, which shows a 'stepped' distribution curve.

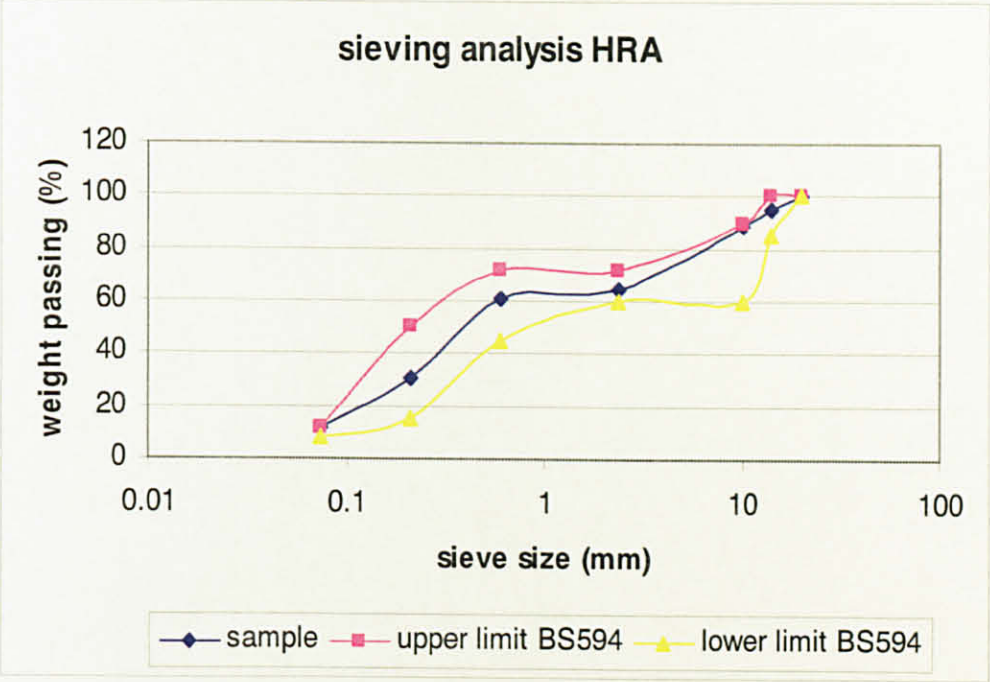

Figure 23: The gradation analysis for HRA

3.3.3 Drying & Mixing process

After identifying the amount of aggregate and filler needed for the research, they were placed together in a container. The bitumen was heated in a different container. All the materials were dried at least 24 hours at a temperature of 150°C. The materials were then mixed in a mixer. A simple calculation was used to determine the amount of bitumen (in gram) that needs to be mixed with the aggregate and filler.

$$Bitumen\ content\ (\%) = \frac{B}{B + mass\ of\ total\ aggregate} \times 100 \qquad \ldots 8$$

Then, the mixture was mixed together using a spatula for at least one minute at a temperature of 160°C to ensure that the aggregate was evenly coated.

There were five types of mixtures used as shown in Table 8. The four mixtures were the normal samples that have been fabricated using only one type of bitumen while the last mixture was the combination of two types of bitumen which were 80 PEN bitumen and 50 PEN bitumen. In this research, the normal samples were taken as control samples.

Table 8: Type of mixes used in the research

Type of mix	Description
AC(80)	Asphaltic concrete with 80 PEN bitumen.
HRA(50)	Hot–rolled asphalt with 50 PEN bitumen.
AC(50)	Asphaltic concrete with 50 PEN bitumen.
HRA(80)	Hot –rolled asphalt with 80 PEN bitumen.
AC(80)+HRA(50)	Combined mix of asphaltic concrete with 80 PEN and Hot–rolled asphalt with 50 PEN.

Table 9 shows the number of samples for each mixture used for each test. All tests required three samples of each mixture except beam fatigue test where nine samples were required for each mixture. This was because in the beam fatigue test, three strain levels were chosen to determine the fatigue life. Therefore, at least three samples were used at one strain level. The other two tests only required one level of parameter.

Table 9: Number of samples prepared for testing

Test name	No of sample for each mixture
1. Dynamic creep test	3
2. Wheel tracking test	3
3. Beam fatigue test	9

3.3.4 Compaction

A circular shape sample was needed for Marshall design method and dynamic creep test. Therefore, the samples were compacted using gyratory testing machine (GTM). The samples were compacted in a 100 mm diameter steel moulds which were kept at 150°C – 160°C. The sample needs to be evenly distributed inside the mould. This was done by tamping the sample by using a steel rod about fifteen times around the edges and five times in the centre. The compaction energy used in this research consisted of applying 0.7 MPa axial pressure and 30 revolutions at an angle of gyration of 1 degree. This energy is equivalent to a compactive effort of 50 blows using Marshall Hammer. For the combined mixture, as for example AC(80)+HRA(50), the sample was mixed separately and compacted layer by layer starting with the bottom layer HRA(50) followed by the upper layer AC(80). As shown in Figure 24, the main characteristic of a gyratory compactor is that it facilitates the application of an axial static pressure at the same time as it subjects a sample to dynamic shear 'kneading motion'. This shearing force is similar to the force encountered under the action of a roller during field compaction (O'Flaherty, 2002).

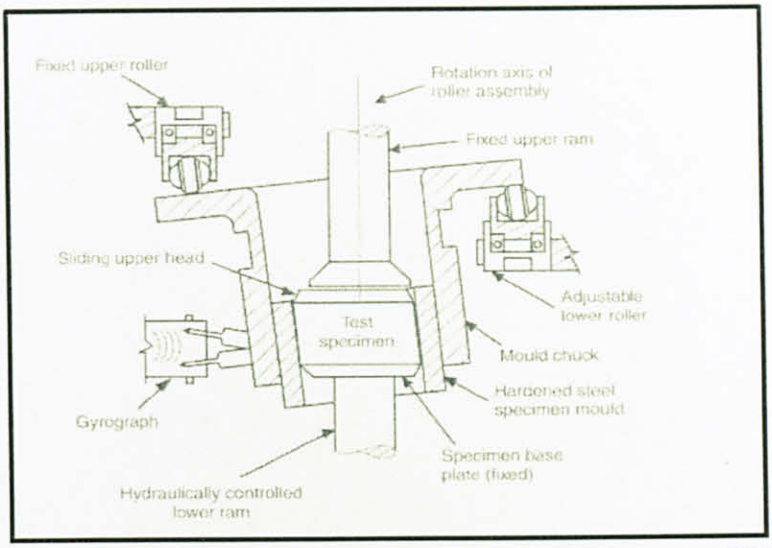

Figure 24: Cross section of a Gyratory compactor (O'Flaherty, 2002)

Meanwhile for wheel tracking and beam fatigue test, the samples were compacted using a hand compactor. The inner surface of the mould for each test was applied with a thin layer of grease to avoid the hot mix sticking to the mould. The same procedure for the combined mixture was also executed for these tests. Since the combined mixture had to be compacted layer by layer, all the normal samples were compacted twice to achieve the same density as the combined mixture. Furthermore, a standard timing about five to ten minutes of compaction was recorded in order to achieve the same density for the entire sample. After compaction, the sample was left in its mould to cool to the ambient temperature before extrusion.

Lastly, the samples were weighed in air and in water to determine the density. The sample dimensions were also measured by recording the height and the width at three locations of the samples. The dimensions were then used in the dynamic creep test and beam fatigue test.

3.3.5 Marshall Design Method

The usage of Marshall design method was to determine the optimum bitumen content for both AC and HRA mixture. For AC(80) mixture, the design of the bitumen content ranging from 4.5% until 6.5% were used. This was in accordance to the Jabatan Kerja Raya (JKR) standard JKR/SPJ/1988. Whereas for HRA(50), the design of the bitumen content from 6.5% until 8% was in compliance to BS 594: Part :1 1992. At least three samples were fabricated for each bitumen content. Before the Marshall test, the samples were immersed in a water bath at a temperature of 60°C for 30 minutes. The testing was done by applying load until the sample failed through cylindrical testing heads at a constant rate of vertical strain of 51 mm per minute. Figure 25 shows the Marshall testing equipment.

Figure 25: Marshall testing equipment

3.3.5.1 Determination of optimum bitumen content

The result between stability and bitumen content is presented in Figure 26. The plotted line is an average of three samples for each bitumen content. Overall (as shown in Figure 26) the stability started to increase as the bitumen content increased until the stability reached a maximum level of 8 kN at the bitumen content of 6%, eventhough there was a slight drop in stability at 5.5% of bitumen content. After that, it regained its strength until it achieved the maximum value of stability. After which the line will start to decrease gradually. This is because the voids in the aggregate become overfilled with bitumen, therefore the excess bitumen acts as lubricant destroying internal friction, thus in the end reducing the mixture stability.

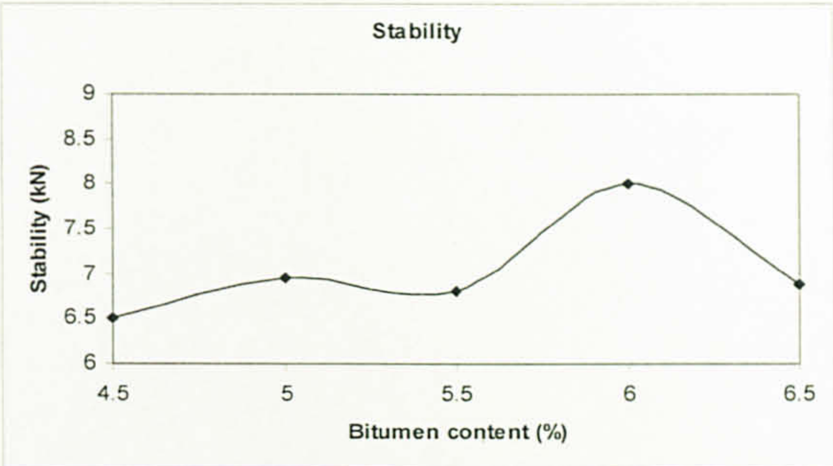

Figure 26: The relationship between stability and bitumen content for AC(80)

The graph in Figure 27 shows a steady density increase from 4.5% until 6% of bitumen content. The maximum mix density was recorded at 6% of bitumen content with a value of 2.36 g/cm^3.

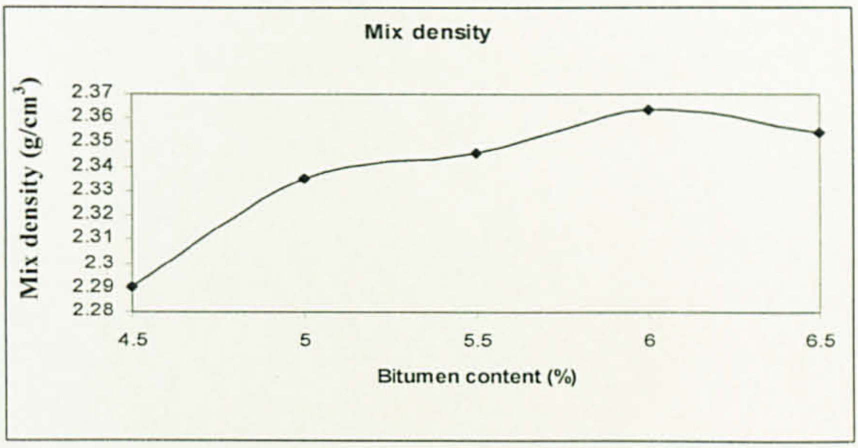

Figure 27: The relationship between mix density and bitumen content for AC(80)

Figure 28 shows the relationship between air void and bitumen content for AC(80). The graph shows that the air void will gradually decrease as the bitumen content started to increase. This is because the voids in the asphalt mixtures had been filled with bitumen. Therefore, the mixture became less susceptible to any air or water intrusion. The mean air void taken was 4%, so 5% of the bitumen content was taken. To acquire the optimum bitumen content, all the three bitumen contents that match the maximum stability, maximum mix density and mean air void respectively were recorded and averaged. Therefore, the optimum bitumen content for AC was 5.7%.

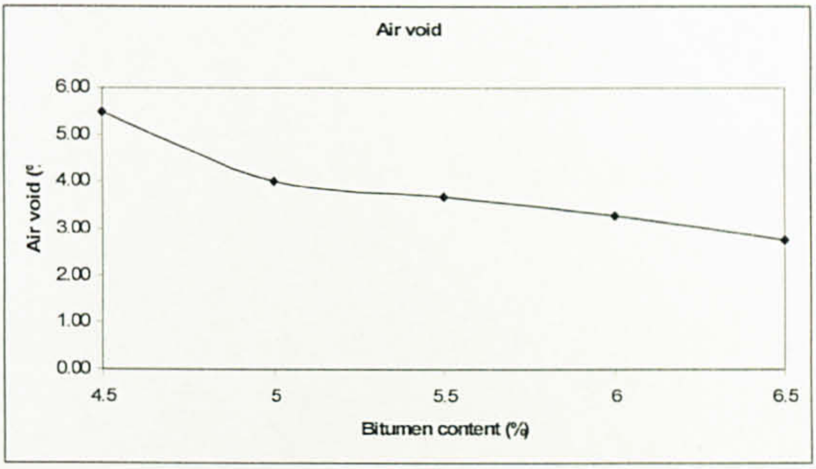

Figure 28: The relationship between Air void and Bitumen content for AC(80)

Since AC(50) is using the same gradation as AC(80), therefore the value of 5.7% was used for all AC mixtures as the optimum bitumen content.

Figure 29 shows an initial gradual increase of stability from 6.5% to 7% of bitumen content followed by a gradual decrease. HRA(50) produced a maximum stability value of 6.7 kN at 7% of bitumen content. The reduced stability is due to bitumen overfilling the voids in the aggregate. Again the excess bitumen acts as lubricant destroying internal friction thus reducing the mixture stability.

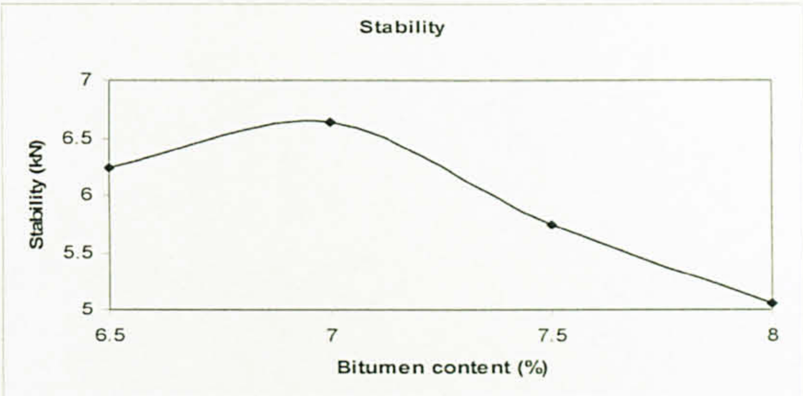

Figure 29: The relationship between stability and bitumen content for HRA(50)

In Figure 30, the line started to increase at a rapid rate from 6.5 % to 7% followed by a gradual increase from 7% until 8% of bitumen content. The maximum mix density for HRA(50) recorded 2.33 g/cm^3 at 8% bitumen content.

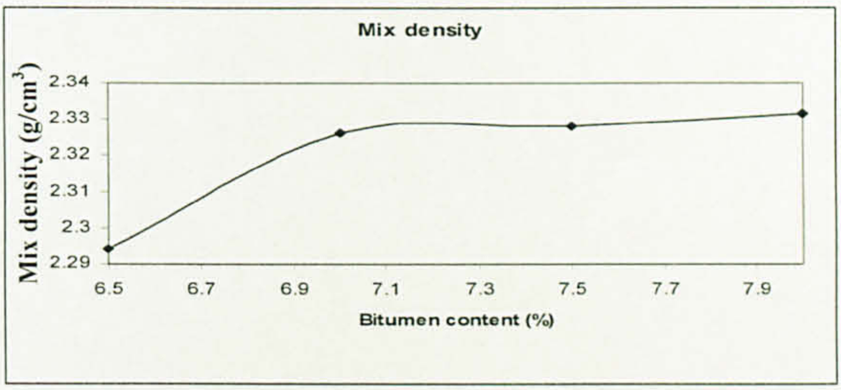

Figure 30: The relationship between mix density and bitumen content for HRA(50)

Figure 31 shows the relationship between compacted aggregate density and bitumen content for HRA(50). The maximum compacted aggregate density 2.163 g/cm^3 was noticed at 7.05% of bitumen content. To acquire the optimum bitumen content, all the three bitumen contents that match the maximum stability, maximum mix density and maximum compacted aggregate density respectively were recorded and averaged. Therefore, the optimum bitumen content for HRA(50) was 7.3%.

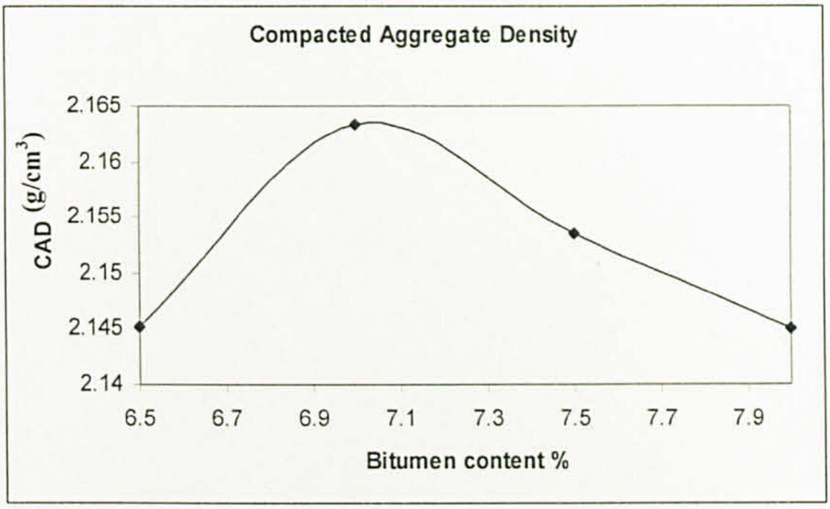

Figure 31: The relationship between compacted aggregate density and bitumen content for HRA(50)

Since the same gradation as HRA(50) was used for HRA(80), therefore the value of 7.3% was used for all HRA mixtures as the optimum bitumen content.

3.4 Test Procedure

3.4.1 Dynamic creep test

It is believed that creep test provides invaluable insight into the rut susceptibility of hot mix. The creep test is a simple test that can satisfactorily predict hot mix asphalt (Brown et al., 1994). This test applies a repeated axial cyclic load of a fixed magnitude and cycle duration to the sample.

Figure 32 shows the machine used for dynamic creep test. This machine is called Universal testing machine (UTM) which was produced by Industrial Process Control Ltd. (IPC), Melbourne, Australia. The sample was placed into the pre-heated chamber for at least one hour to reach a uniform temperature of 40°C. Then, the sample was loaded with a low pressure of 0.01 MPa for 2 minutes. After that, the high–pressure of 0.1 MPa was subjected and the readings were taken at intervals of 1 hour (10, 20, 40, 60, 120, 300, 600, 1200, 2400 and 3600 seconds). The creep stiffness readings can be obtained directly from the test.

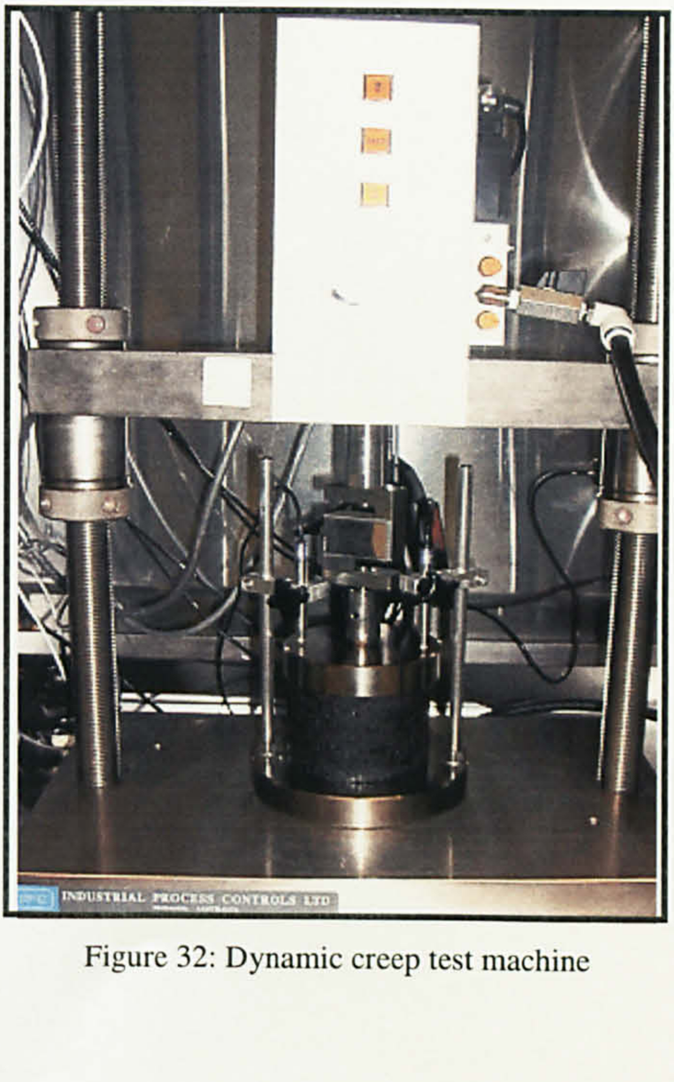

Figure 32: Dynamic creep test machine

Figure 33 shows some examples of the dynamic creep samples. The shape of these samples is similar to the shape of the Marshall design method. Since three samples were fabricated for each mixture, a total of fifteen samples were made. All of the samples were compacted using gyratory compactor that has been described earlier.

Figure 33: Dynamic creep samples

Figure 34 display diagrams of five examples of the creep samples. Each sample measures a 100 mm in diameter and 65 mm in height. The four samples at the top of the diagram are the normal creep samples, whereas the bottom one is the combined sample where the lower portion is made of HRA(50) and the upper is AC(80).

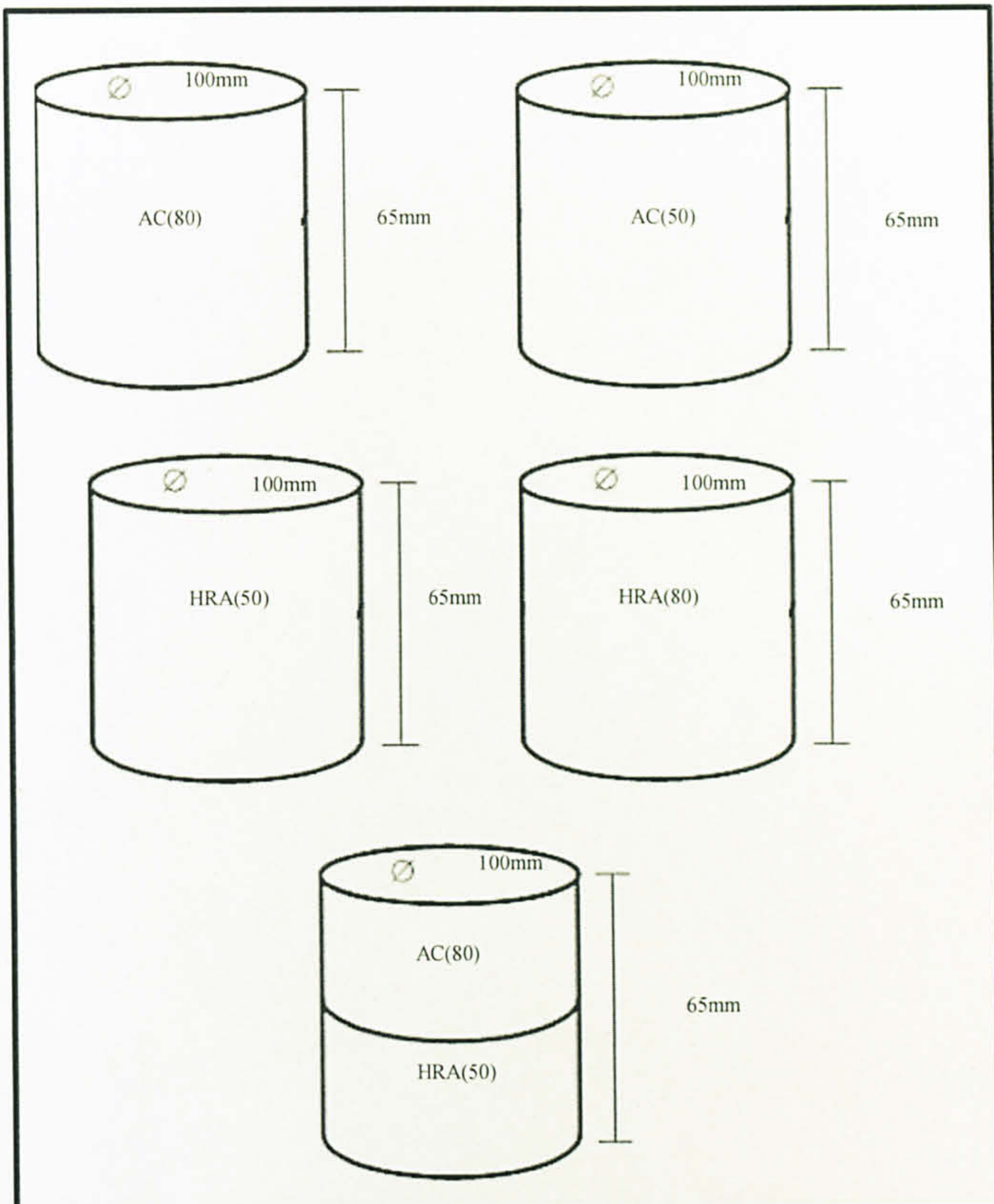

Figure 34: Diagram dynamic creep samples (drawing not to scale)

3.4.2 Wheel tracking test

This device functions by rolling a small loaded wheel repeatedly across a sample. Figure 35 shows the wheel tracking devices. The wheel tracking machine was set at least for one hour so that it reached a constant temperature of 40°C. The sample was also placed in an oven for one hour to achieve the same temperature as the machine. The test will run for 1946 cycles with two passes, and the testing duration for a complete cycle for each sample was approximately 45 minutes. The total rut depth value can be observed from the computer connected to the wheel tracking machine.

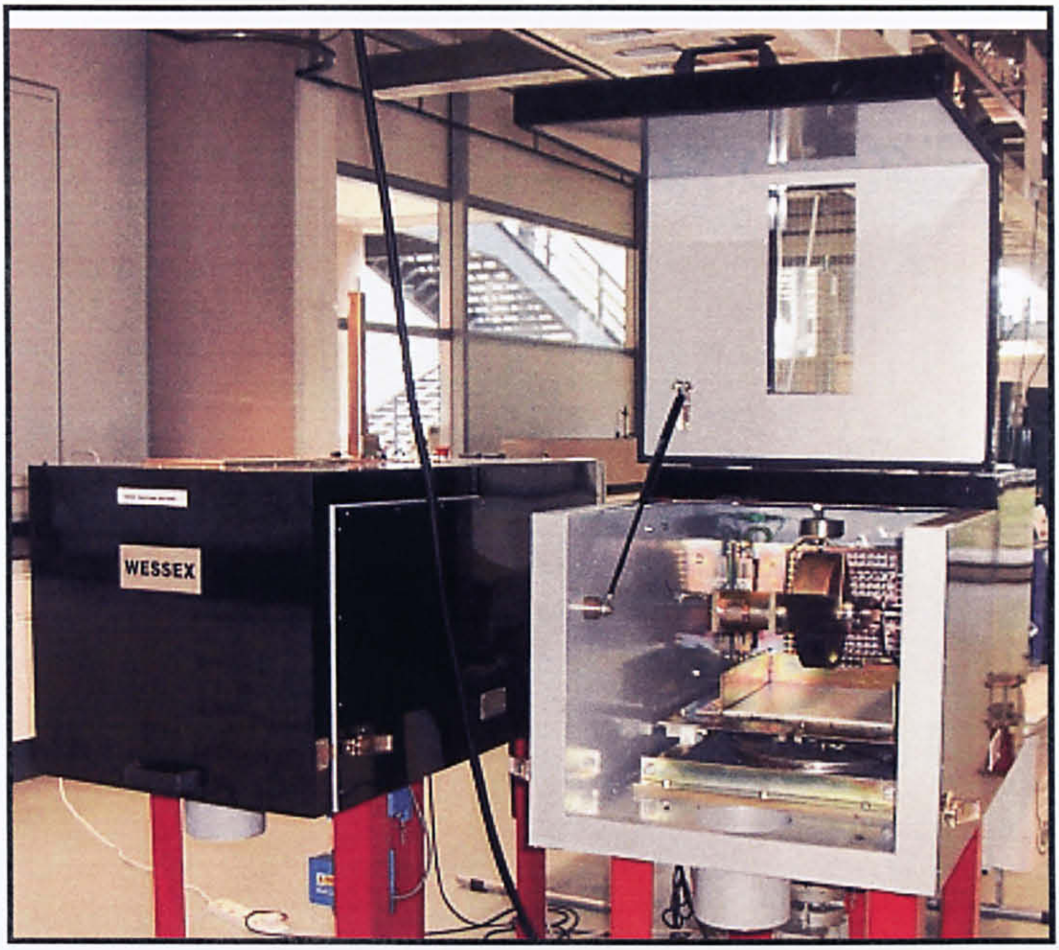

Figure 35: Wheel tracking devices

Figure 36 shows some samples for the wheel tracking test. Since three samples were fabricated for each mixture, a total of fifteen samples were made. All of the samples were compacted using a hand compactor.

Figure 36: Wheel tracking samples

Figure 37 displays diagrams of five examples of the wheel tracking samples. Each sample was a 305 x 305 mm square shape with a height of 50 mm. The first four samples namely AC(80), AC(50), HRA(50) and HRA(80) are the normal creep samples; whereas the last sample is a combined sample where the lower portion is made of HRA(50) and the upper is AC(80).

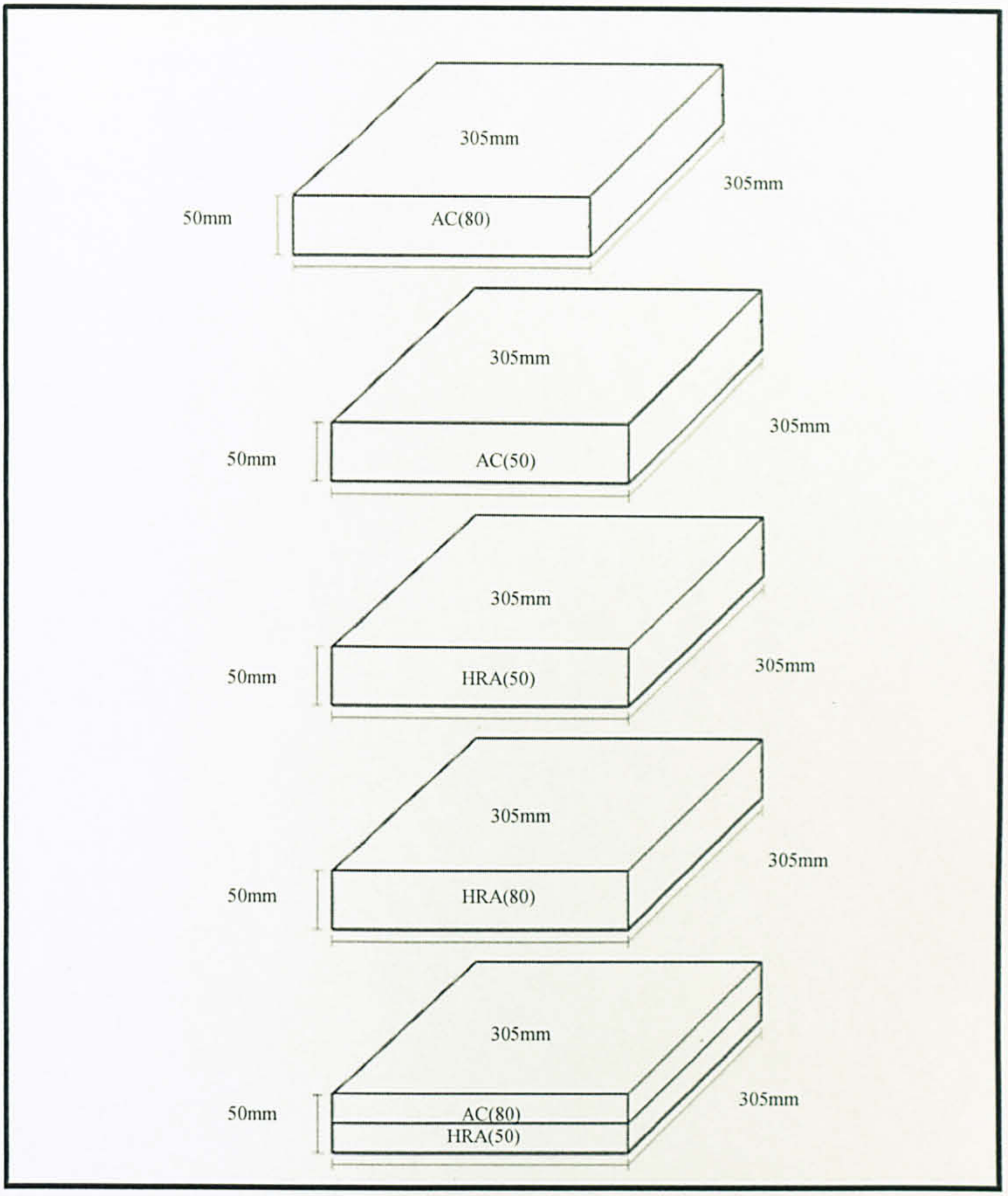

Figure 37: Diagram wheel tracking samples (drawing not to scale)

3.4.3 Beam fatigue test

Over the years, many studies have been conducted in finding an appropriate laboratory test procedures that best simulate field conditions. (Hajj et al., 2005). As a result, the repeated flexural beam (4 point loading) fatigue test method under the constant strain mode of loading was selected for fatigue response prediction.

In this work, the beam fatigue test was performed on a Universal testing machine (UTM) produced by Industrial Process Control Ltd. (IPC), Melbourne, Australia. Figures 38 and 39 show the Beam Fatigue testing device.

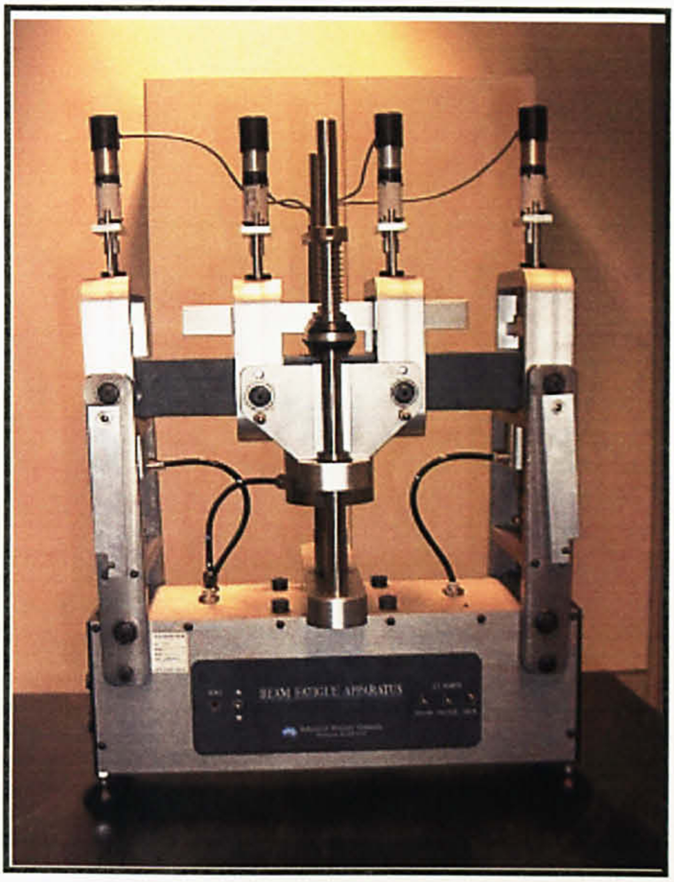

Figure 38: Beam fatigue device

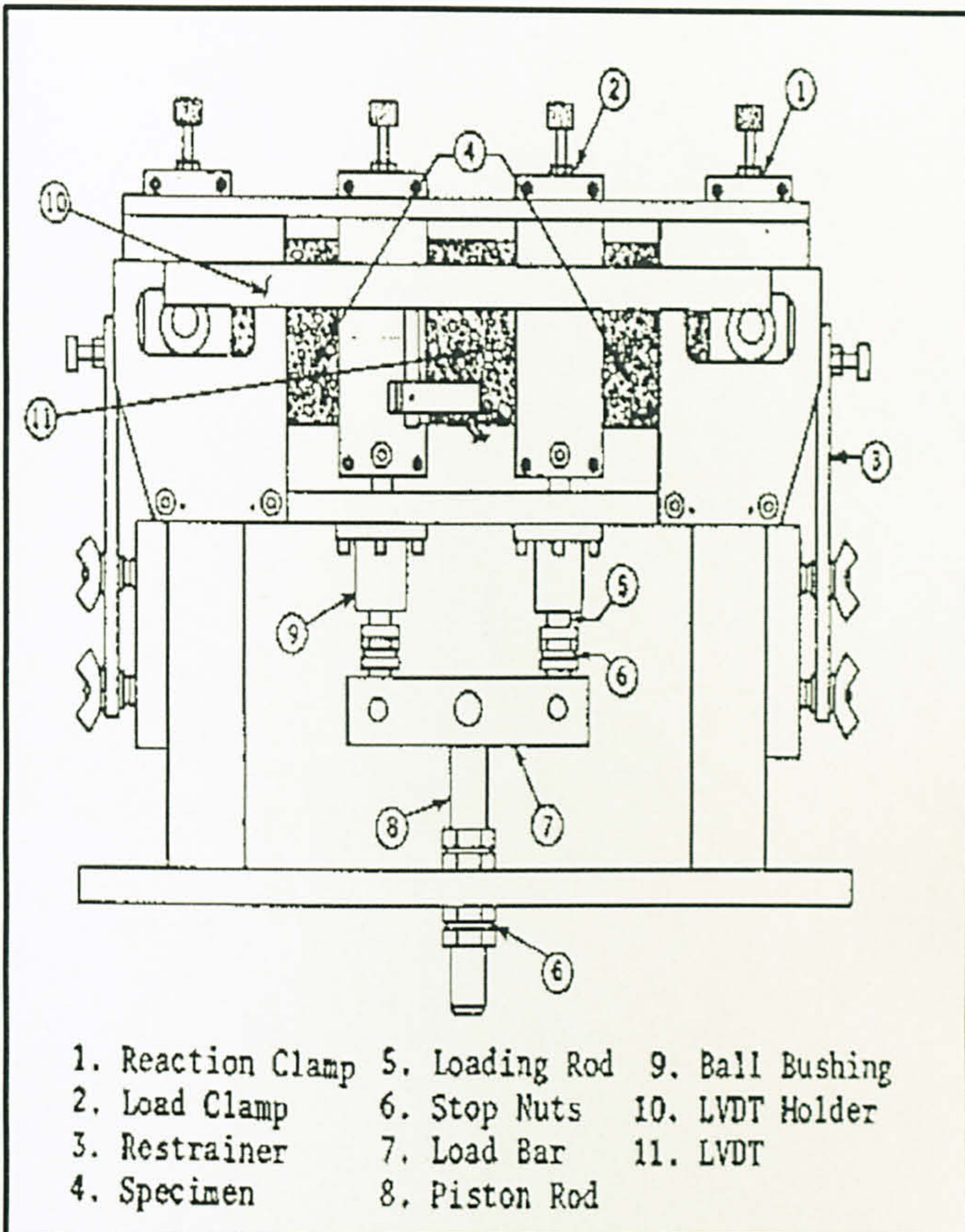

Figure 39: Schematic diagram of fatigue device (Huang, 2004)

Figure 40 shows the beam fatigue sample. At least nine replicates were made for each mix. Before testing, the samples were placed in the temperature control chamber for at least one hour at the desired testing temperature. The parameters input were haversine with the frequency of 5 Hertz and no rest period. Three strain values (low, medium and high) were selected–100E-6, 150E-6 and 200E-6 respectively. The test temperature used was room temperature of 20°C. The failure of the sample was considered when the beam reached 50% of the initial stiffness.

Figure 40: Beam fatigue sample

Figure 41 displays five diagrams of the beam fatigue samples. Each sample was 380 mm long, 64 mm wide and 51 mm deep. The first four samples are the normal creep samples and the last is the combined sample of HRA(50) and AC(80). All samples were compacted using a hand compactor. The samples were left inside the mould for at least 24 hours before being extruded.

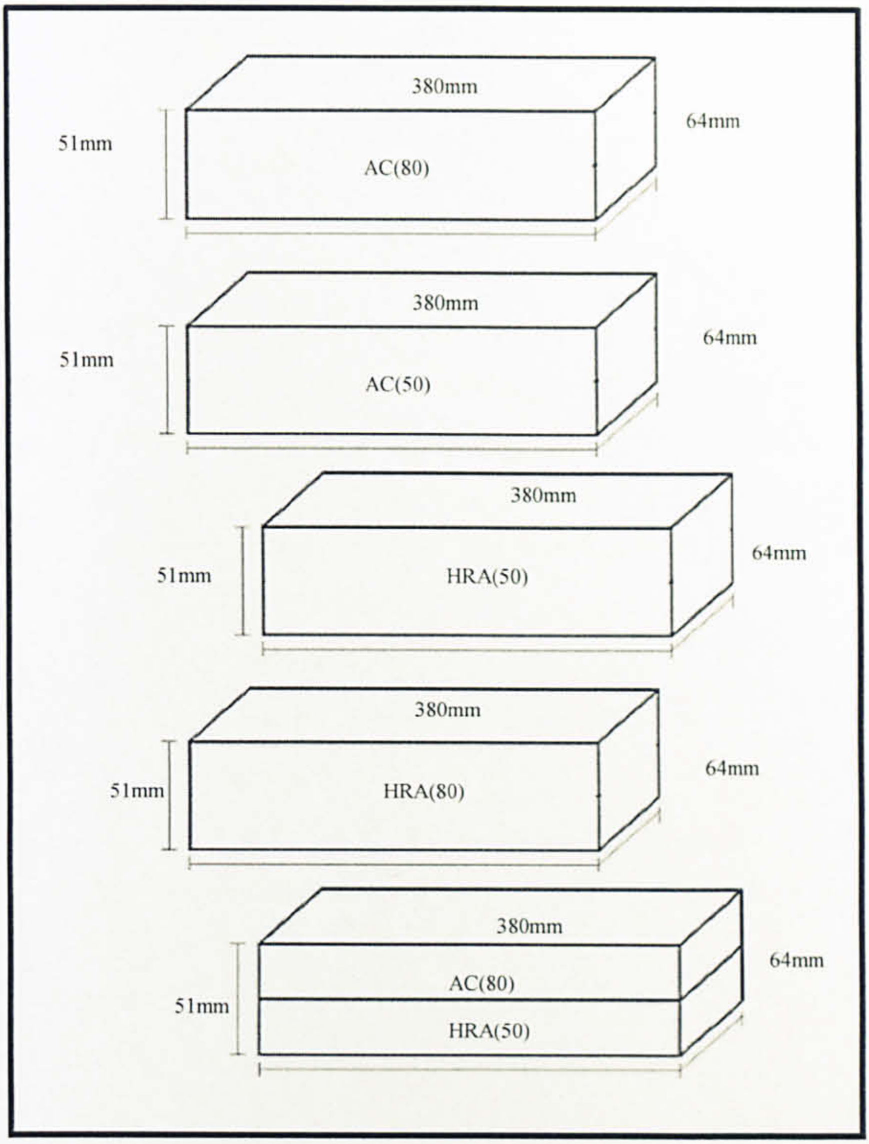

Figure 41: Diagram beam fatigue samples (drawing not to scale)

CHAPTER FOUR
RESULTS AND DISCUSSION

4.1 Dynamic Creep Test results

The result obtained from the Dynamic Creep Test is the creep stiffness of the mixture (S_{mix}). The stiffness of the bitumen (S_{bit}) was calculated from the Van der poel monograph (as shown in Figure 42) based on the loading time equal to creep loading and temperature equal to test temperature of 40°C. From the measured S_{mix} and the calculated S_{bit}, a double- logarithmic graph of S_{mix} vs. S_{bit} was plotted.. The equation of the lines takes the following form:

$$Log(Y) = Log(a) + bLog(x) \qquad \text{........9}$$

or

$$Y = ax^b \qquad \text{......10}$$

Where Y = stiffness of mix (MPa)

x = stiffness of bitumen (MPa)

a = the intercept at Y – axis

b = the slope of the line

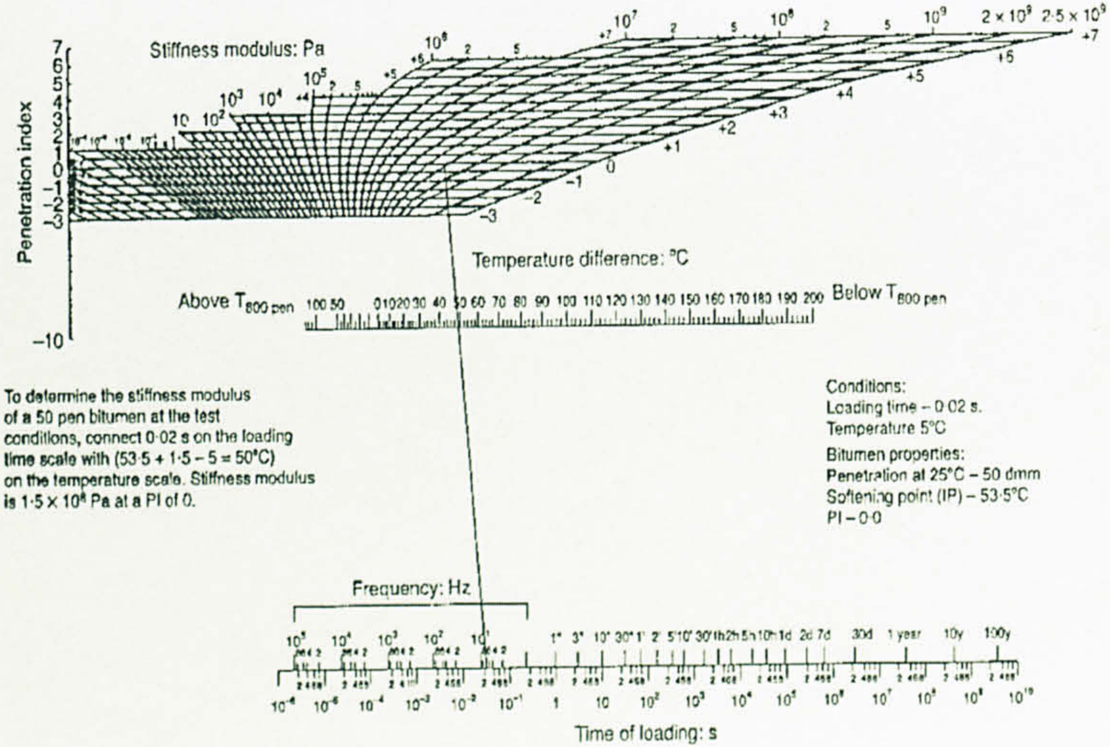

Figure 42: Example of Nomograph for determining the stiffness modulus of 50 graded penetration bitumen (Whiteoak and Read, 2003)

Since the reaction of bitumen (stiffness modulus) also depends on temperature and time of loading, hence the permanent deformation depends on the viscous component of the modulus $(S_{bit})^v$. Hills et al. in Napiah, 1993 recommended the following equation to measure the stiffness modulus of the bitumen corresponding to its viscous part:

$$(S_{bit})^v = \frac{3\eta}{NT_w} \qquad \text{......11}$$

Where $(S_{bit})^v$ = 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.

N = the number of wheel passes in standard axles

T_w = the time of loading for one wheel pass.

Figure 43 shows a graph was used to determine the viscosity value of the bitumen, η. The value of η can be obtained if the two parameters which are the Penetration Index, PI and softening point or also known as ring and ball test temperature are identified.

PI = 0; Softening point for 80 pen graded bitumen = 44^{0}C; Softening point for 50 pen graded bitumen = 53.5^{0}C

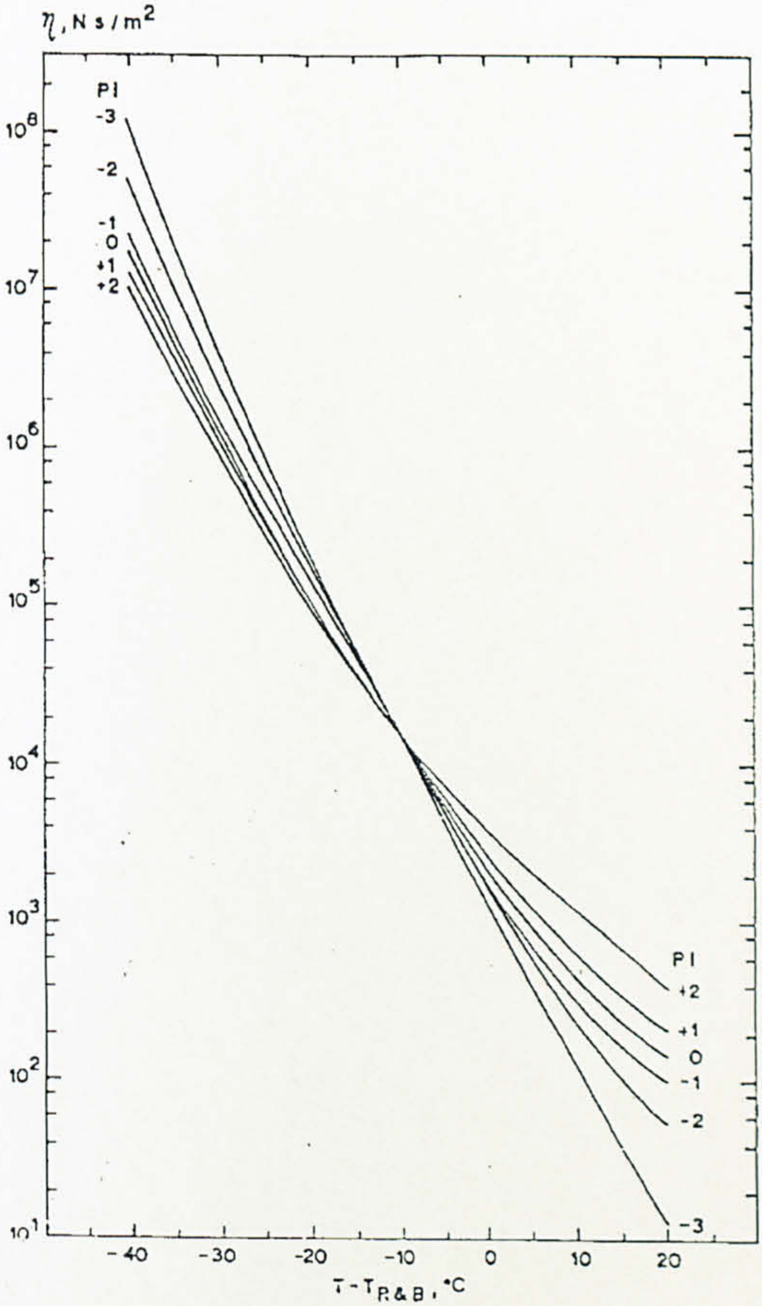

Figure 43: Viscosity of bitumen of $(T - T_{R\&B})$ and PI

From the values of S_{mix} obtained from the Dynamic Creep Test, the rut depth was calculated using Van der Loo's equation (equation 5) with the following numerical assumptions:

$$T_w = 0.02\ second;\ C_m = 1.5;\ H = 70mm;\ \sigma_{av} = 0.25MPa$$

Figure 44 shows the rut depth of HRA(50) and HRA(80). HRA(80) is represented by the dark blue line whereas HRA(50) is represented by the pink line. Both mixtures were mixed with the same gradation of HRA but with different types of bitumen. At ten million standard axle, HRA(50) exhibited a lower rut depth of 5.87 mm compared to HRA(80) that had a higher rut of 7.20 mm. This is due to the harder type of bitumen used in HRA(50), which is 50 PEN bitumen, therefore resulting in lower rut depth.

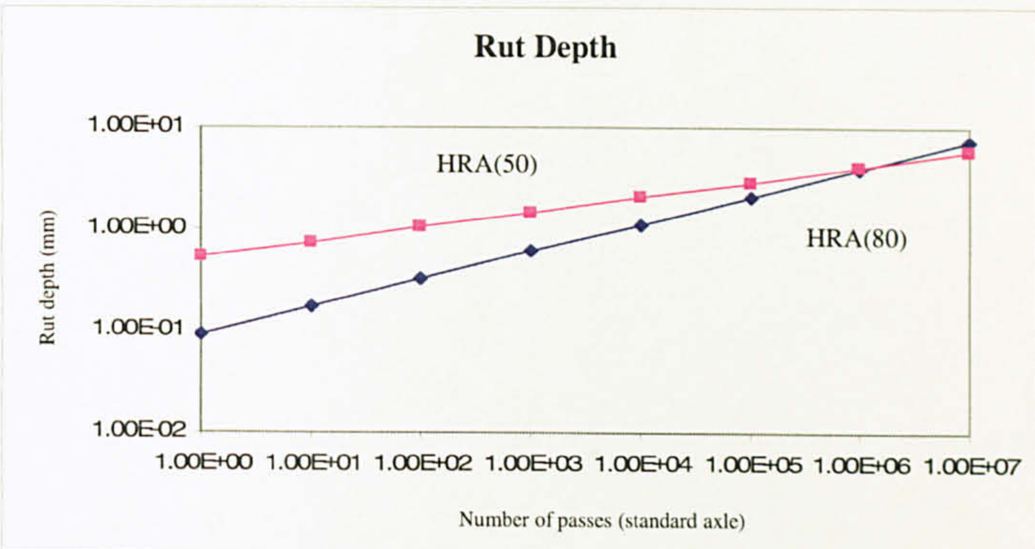

Figure 44: Graph of rut depth between HRA(50) and HRA(80)

Figure 45 shows a rut depth comparison between AC(50) and AC(80). The violet line represents AC(50) whereas the dark red line represents AC(80). Both mixtures are of the same aggregate gradation which is AC mixture but were mixed with two different kinds of bitumen. As shown in this graph, AC(50) exhibited a lower rut depth of 3.53 mm at ten million standard axle whereas AC(80) had a rut depth of 10.6 mm. At the beginning of one standard axle to ten standard axle, AC(80) experienced a lower rut depth until both lines reached one thousand standard axle where AC(50) had a lower rut depth of 0.58mm compared to AC(80) which had 0.65 mm. After that, AC(80) experienced higher rut depth until ten million standard axle. This observation is attributed to the harder type of bitumen 50 PEN used in AC(50).

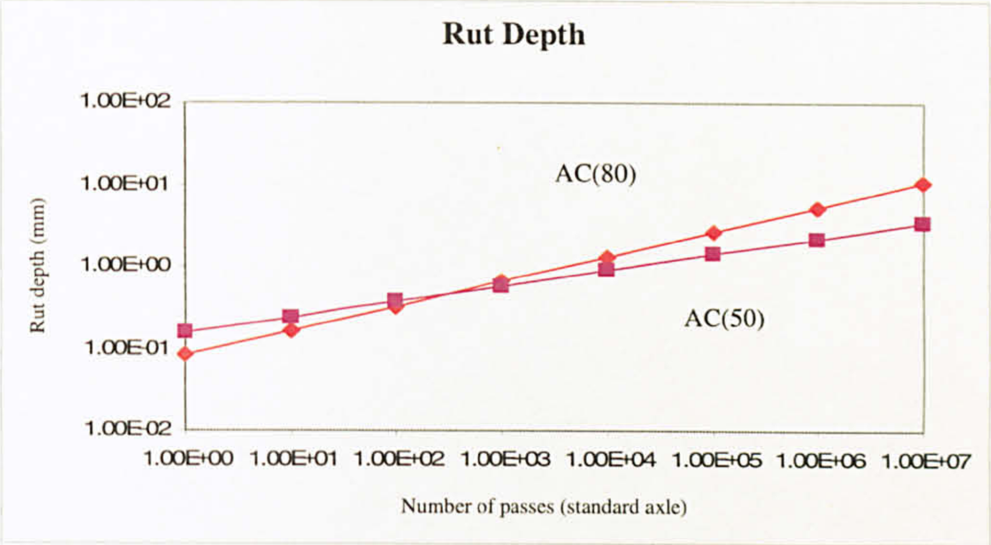

Figure 45: Graph of rut depth between AC(50) and AC(80)

Figure 46 shows a rut depth between normal AC(80) and the combined AC(80) +HRA(50). AC(80) is the light orange line whereas AC(80)+HRA(50) is the green line. At ten million standard axle, AC(80) had a lower rut depth of 10.4 mm whereas the combined AC(80)+HRA(50) had a 14.3 mm rut depth.

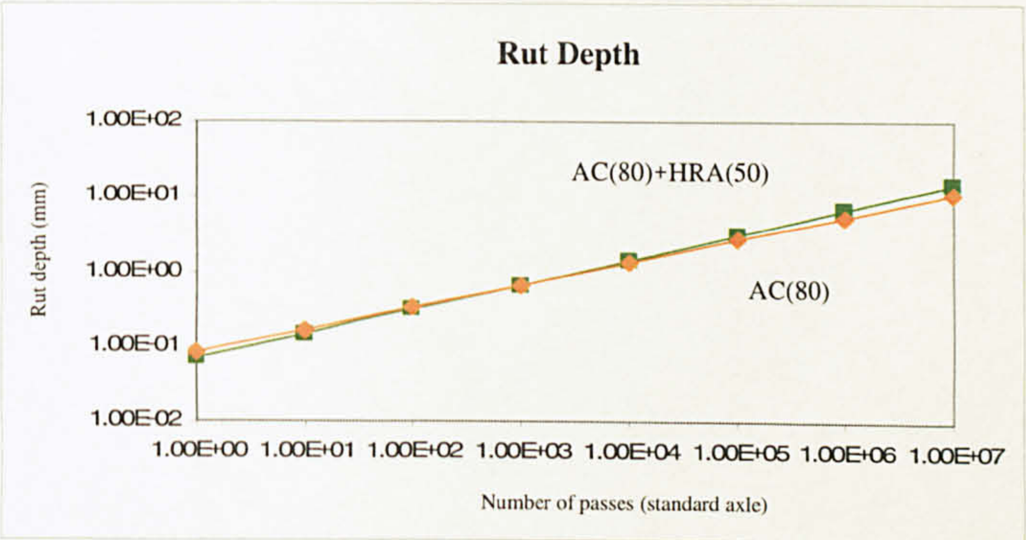

Figure 46: Graph of rut depth between AC(80)+HRA(50) and AC(80)

4.2 Wheel tracking test results

The test was conducted in accordance to BS 598, which have been programmed into the Wessex machine. The data for HRA(80) and HRA(50) were not included in the table because all the samples failed before the end of the 45 minutes test period. Table 10 shows that AC(50) had a lower rate of deformation of 0.066 mm/min compared to AC(80) that showed a higher rate of 0.08 mm/min. Both mixtures are of the same aggregate gradation which is asphaltic concrete but were mixed with different types of bitumen. AC(50) was mixed with 50 PEN bitumen, a harder bitumen than 80 PEN bitumen which was mixed in AC(80). The harder bitumen provided a better load resistance in AC(50), hence less deformation was observed in AC(50).

Table 10: Comparison of rut depth rate between AC(80) and AC(50)

Type	Rate of deformation (mm/min)
AC(80)	0.08
AC(50)	0.066

However, the rate of deformation was better on the combined sample of AC(80)+HRA(50).

As shown in Table 11, the rate of deformation for the combined sample is 0.037 mm/min, which is approximately half of the value for AC(80). This indicates that the combination of AC(80)+HRA(50) might help reduce rutting potential.

Table 11: Comparison of rut depth between AC(80) and AC(80) + HRA(50)

Type	Rate of deformation (mm/min)
AC(80)	0.08
AC(80)+HRA(50)	0.037

4.3 Beam fatigue test result

In the Beam fatigue test, a total of forty five samples were tested. For each mixture, three strain levels were chosen namely high stain level (200E-6), medium strain level (150E-6) and low strain level (100E-6). All samples were tested at room temperature of 20°C.

The following figures (Figure 47, 48 and 49) illustrate the results of the beam fatigue test for AC(50), AC(80) and AC(80)+HRA(50) performed at low strain level. In the constant strain test, the stress (and stiffness) decreased as the sample failed as shown in the yellow scatter data. The evolution of the stiffness in the constant strain was used to define the failure of the sample.

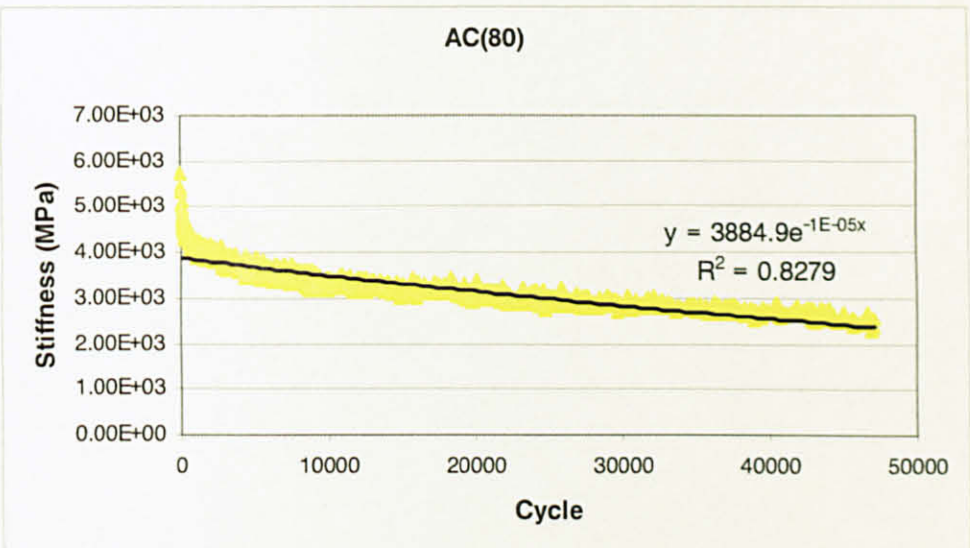

Figure 47: Stiffness evolution during constant strain test for AC(80)

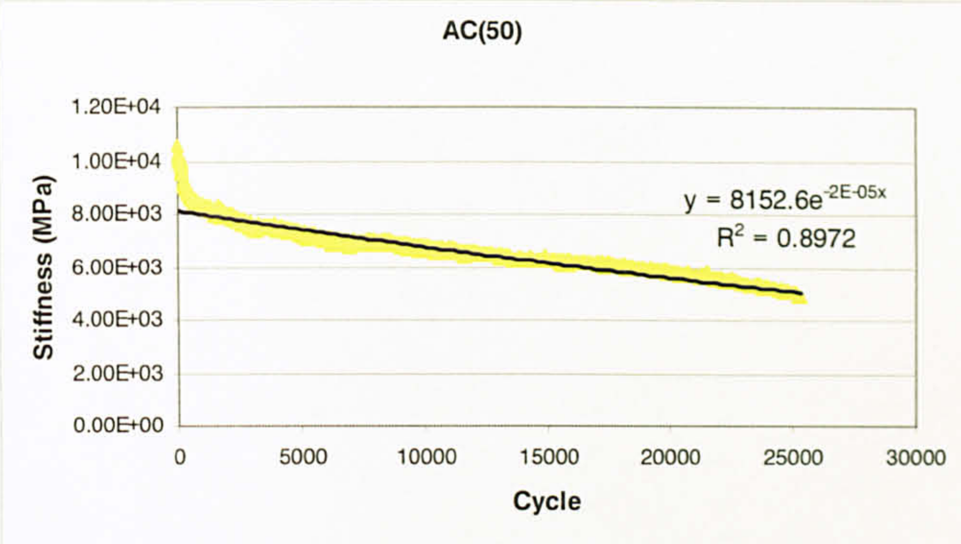

Figure 48: Stiffness evolution during constant strain test for AC(50)

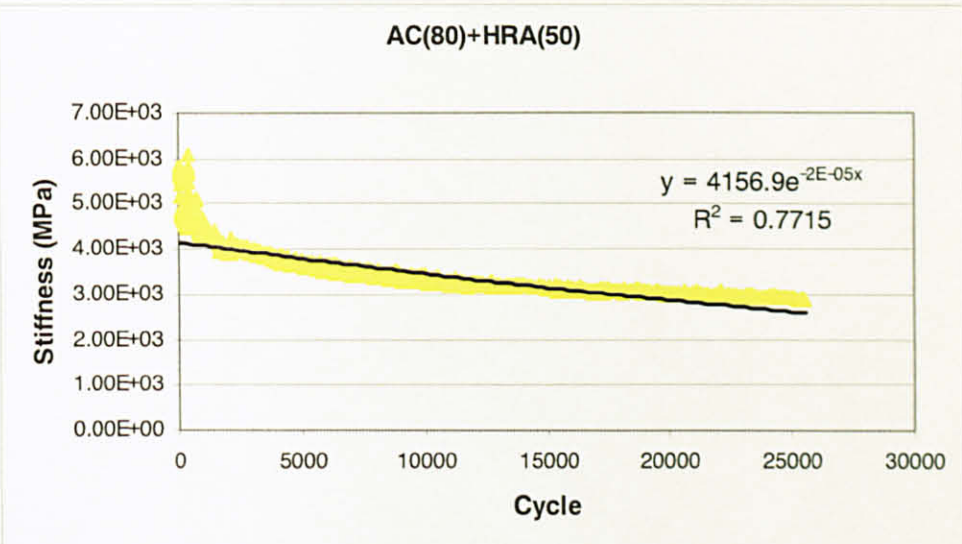

Figure 49: Stiffness evolution during constant strain test for AC(80)+HRA(50)

4.3.1 Fatigue life results

The results of the Beam Fatigue Test for HRA(50) and HRA(80) are illustrated graphically in Figure 50. Both samples show a similar trend of increasing cycles to failure with decreasing strain level. This trend is similar to the observation by Robinson and Thagesen as shown in Figure 17. However, HRA(50) shows a longer fatigue life of 5.05E+7 compared to HRA(80), which a maximum obtained 1.9E+7 cycles.

This is due to the type of bitumen. 50 PEN bitumen is considered to be hard bitumen whereas 80 PEN bitumen is soft bitumen. The stiffness of the bitumen is usually associated with the type of the bitumen. The harder the bitumen, the stiffer the mixture will become. Eventhough both mixtures were mixed using HRA, the stiffness of the bitumen that has been incorporated into the mixture has made the mixture to excel in a longer fatigue life. The fatigue equations for both mixtures are also presented in the graph.

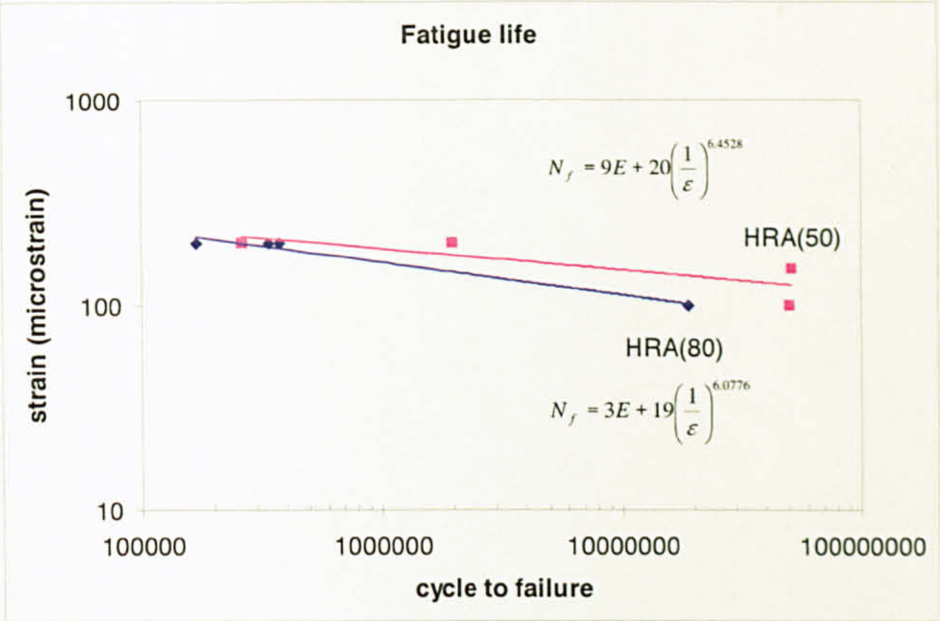

Figure 50: Graph of fatigue life, N_f of HRA(50) and HRA(80)

Figure 51 shows the fatigue life of AC(50) and AC(80). AC(80) is represented by the orange line meanwhile AC(50) is represented by the green line. AC(50) produced a longer fatigue life of 2.16E+6 than AC(80) that only achieved 9.98E+5 number of cycles to failure. Again, this is due to the type of bitumen. The tensile properties such as tensile strength, tensile strain and stiffness depend on the type of bitumen. Soelistijo., 1995 stated that for the same mixture, the stiffer the bitumen the lower the mixture tensile strain at failure, and the stiffer the bitumen the higher the mixture tensile stiffness at failure.

Nevertheless, all bitumen types would have approximately the same tensile stresses at failure.

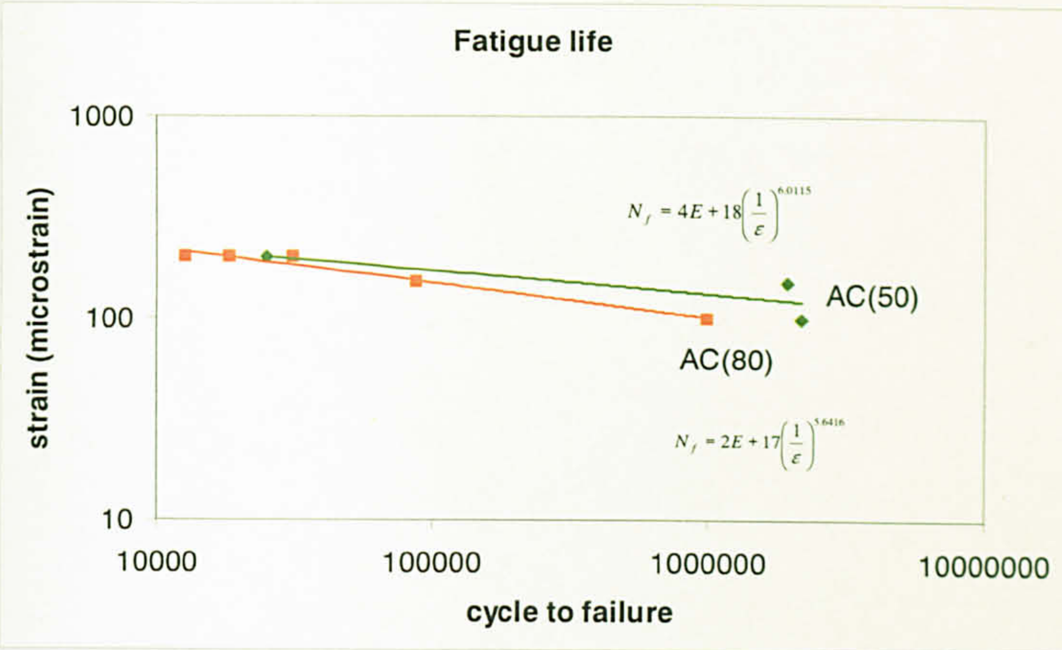

Figure 51: Graph of fatigue life, N_f of AC(50) and AC(80)

Figure 52 shows the fatigue life of AC(50) and HRA(50). AC(50) is represented by the turquoise line meanwhile HRA(50) by the brown line. As predicted, HRA(50) has a longer fatigue life of 5.04E+7 cycles compared to AC(50) with 2.16E+6 number of cycles. Since the stability of the HRA is governed by the amount of sand/filler and bitumen, the excessive amount of coarse aggregate could experience aggregate breakage, hence make AC(50) less durable.

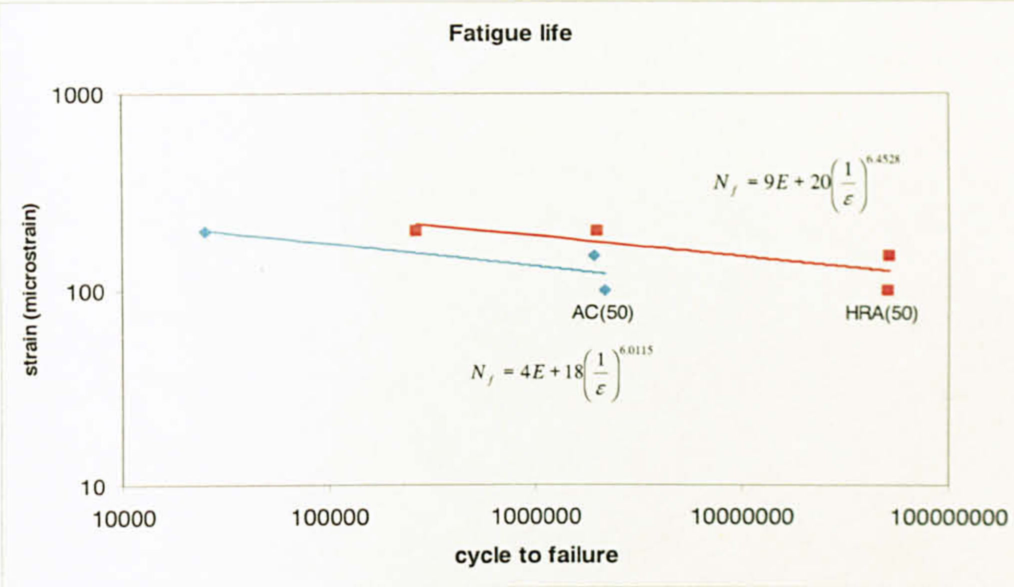

Figure 52: Graph of fatigue life, N_f of AC(50) and HRA(50)

Figure 53 compares the fatigue life of the normal AC(80) and combined AC(80)+HRA(50). The purple line represents AC(80) and the black line represent AC(80)+HRA(50). AC(80)+HRA(50) has a longer fatigue life of 1E+8 number of cycles than AC(80) that achieved a maximum life of 9.98E+5 at the lower strain of 100E-6. Therefore, it can be concluded that the combination AC(80)+HRA(50) proved to be more resistance to fatigue cracking.

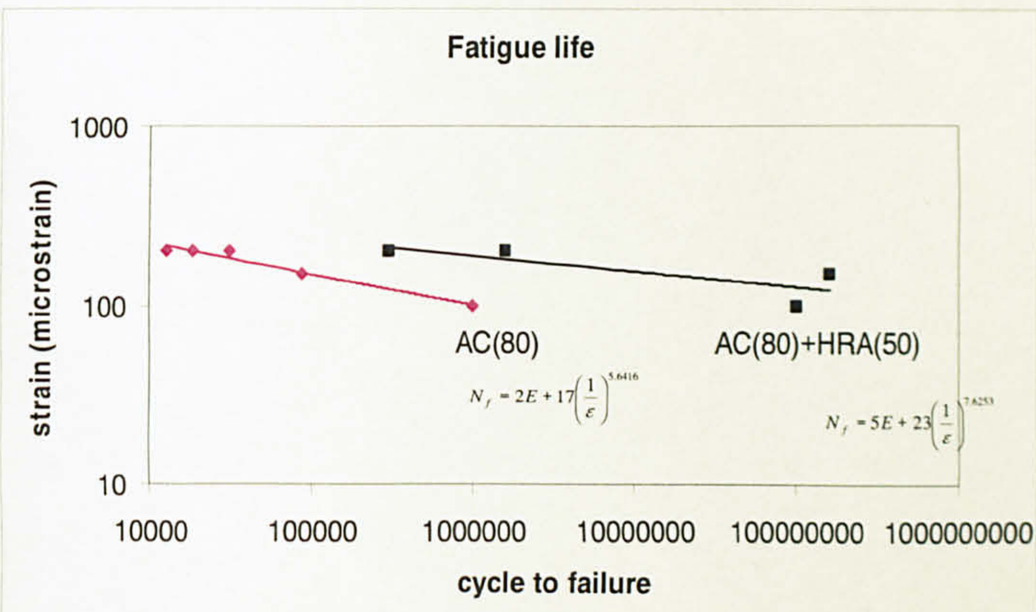

Figure 53: Graph of fatigue life, N_f of AC(80) and AC(80)+HRA(50)

CHAPTER FIVE
CONCLUSIONS AND RECOMMENDATION

The research on the combination of Asphaltic Concrete Bituminous mixture (AC) as wearing course and Hot–Rolled asphalt (HRA) as binder course through the experimental works suggested the following conclusions:

1. The combination of AC(80)+HRA(50) proves to be a more stable mixture in reducing rutting by 54% when compared to AC(80) in the wheel tracking test.
2. Furthermore, when the combination of AC(80)+HRA(50) was compared to AC(80) alone, the combination improved the fatigue life when it lasted up to 1E+8 number of cycles to failure compared to AC(80) that lasted up to 9.98E+5 number of cycles.

3. The results obtained from the dynamic creep test, wheel tracking test and beam fatigue test strongly suggest that a combination of AC as wearing course and HRA as binder course can reduce both rutting and fatigue cracking.

4. The dynamic creep test was found to be effective in testing the different bituminous mixture.

5. HRA is a superior material in reducing fatigue cracking. HRA(50) shows a longer fatigue life of 5.05E+7 compared to HRA(80) which only obtained 1.9E+7 number of cycles to failure. This is due to the type of bitumen. The stiffer the bitumen, lower the mixture tensile strain at failure and the stiffer the bitumen the higher the mixture tensile stiffness to failure.

6. HRA(50) produced a longer fatigue life of 5.04E+7 than AC(50) that only achieved 2.16E+6 number of cycles to failure. This result confirmed the theory that HRA does reduce fatigue cracking.

Looking from the success of the research, I would like to recommend to the relevant authority that the combination of AC(80)+HRA(50) could be adopted as one of the option in building a strong surfacing structure in the near future.

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Marshall test result AC(80)

obc (%)	Height			Mass of Specimen		Air void (%)	Flow (mm)	Stability (kN)		
	1	2	3	Wa	Ww			Measured	C.F.	Corrected
4.5	70.05	70.04	69.90	1244.50	704.80	6.11	2.33	7.96	0.89	7.08
4.5	71.00	71.02	70.83	1248.90	704.20	5.25	3.15	7.57	0.89	6.74
4.5	71.74	72.19	71.94	1258.40	704.80	5.10	2.65	6.64	0.86	5.71
Average						5.49	2.71			6.51
5	69.76	70.06	69.74	1248.60	714.20	4.55	3.35	7.64	0.89	6.80
5	69.31	69.66	69.60	1251.00	713.50	4.05	0.00	7.62	0.93	7.09
5	68.72	69.08	68.81	1243.20	712.20	3.44	3.99	7.54	0.93	7.01
Average						4.01	2.45			6.97
5.5	70.41	70.48	70.53	1270.80	728.60	3.69	3.72	6.73	0.89	5.99
5.5	69.38	69.59	69.37	1257.50	722.50	3.79	3.96	8.48	0.93	7.89
5.5	69.01	69.27	69.04	1250.10	716.60	3.55	2.74	7.04	0.93	6.55
Average						3.68	3.47			6.81
6	68.83	68.64	68.65	1257.70	726.00	3.55	3.40	8.35	0.93	7.77
6	69.59	69.83	69.58	1274.50	734.90	3.11	3.28	9.66	0.89	8.60
6	69.54	69.49	69.49	1267.70	731.10	3.20	3.22	8.61	0.89	7.66
Average						3.29	3.30			8.01
6.5	69.55	69.51	69.61	1273.00	735.90	2.59	4.67	7.67	0.93	7.13
6.5	70.81	70.58	70.89	1281.90	735.60	3.01	4.20	7.54	0.89	6.71
6.5	69.70	69.88	69.83	1267.60	727.50	2.69	4.32	7.67	0.89	6.83
Average						2.76	4.40			6.89

Marshall Test result for HRA(50)

obc (%)	Height 1	Height 2	Height 3	Mass of Specimen Wa	Mass of Specimen Ww	Air void (%)	Flow (mm)	Stability (kN) Measured	C.F.	Corrected
6.5	71.72	71.95	71.81	1274.10	721.60	5.71	3.33	8.23	0.86	7.08
6.5	72.37	72.37	72.35	1273.40	717.40	6.36	2.75	6.80	0.86	5.85
6.5	72.25	72.22	72.24	1268.40	713.70	6.51	2.81	6.75	0.86	5.81
Average						6.19	2.96			6.24
7	69.65	70.36	69.7	1265.90	725.60	3.50	3.81	9.23	0.89	8.21
7	72.05	71.73	71.77	1285.20	731.30	4.43	3.37	7.31	0.86	6.29
7	71.78	70.56	71.8	1257.80	714.60	4.63	2.78	6.31	0.86	5.43
Average						4.18	3.32			6.64
7.5	72.62	72.93	72.56	1304.70	744.20	3.42	3.70	6.94	0.86	5.97
7.5	71.27	71.3	71.27	1282.20	731.70	3.36	3.34	6.90	0.89	6.14
7.5	70.63	70.47	70.64	1275.10	727.30	3.42	4.00	5.75	0.89	5.12
Average						3.40	3.68			5.74
8	71.14	70.74	70.91	1275.50	725.30	3.11	3.52	5.31	0.89	4.73
8	68.22	69.34	69.24	1250.00	714.40	2.46	5.04	5.45	0.93	5.07
8	68.39	68.73	68.51	1247.90	715.20	2.09	4.83	5.79	0.93	5.38
Average						2.55	4.46			5.06

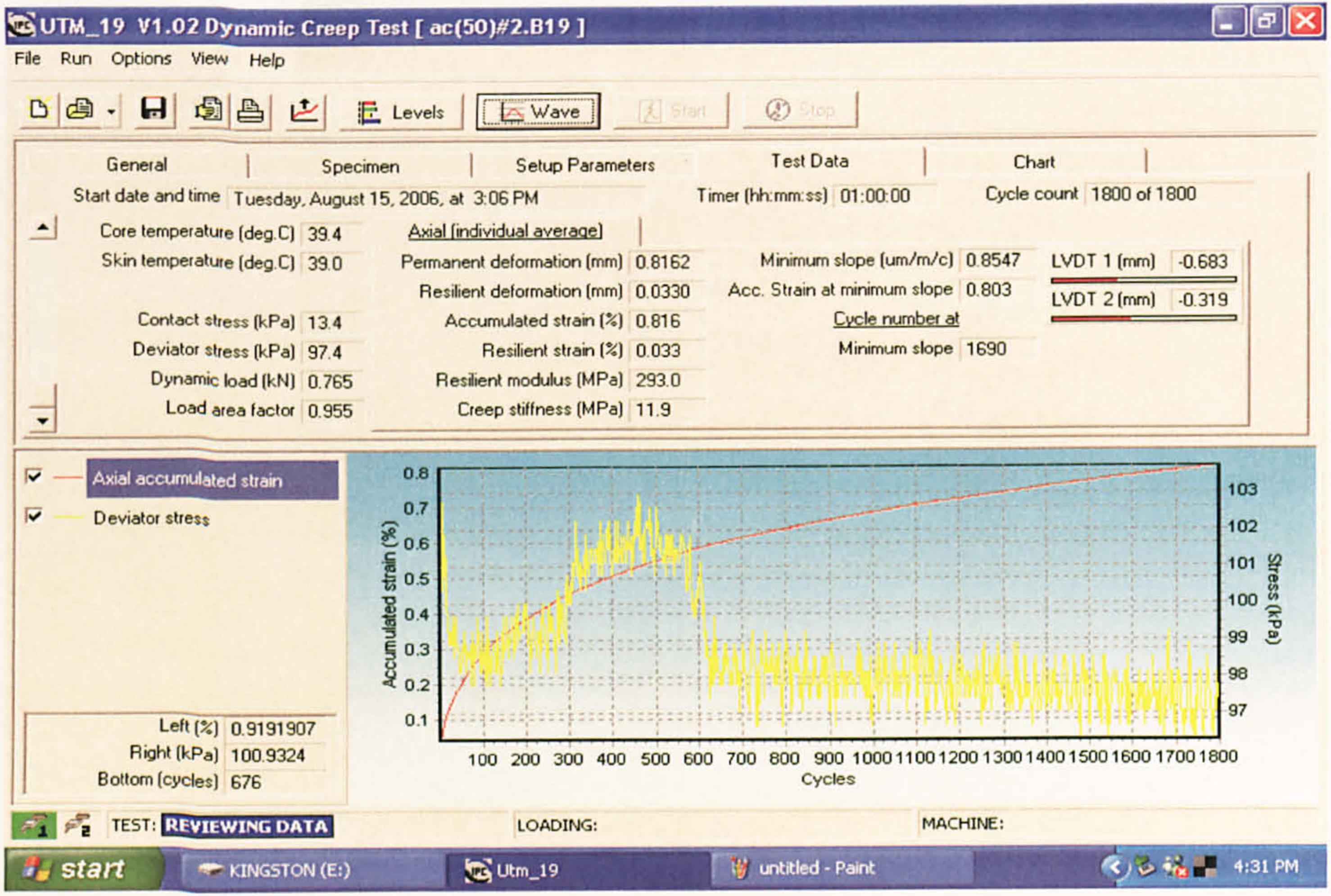
Density and porosity for dynamic creep, wheel tracking and beam fatigue

Type of mix	No	Dynamic creep		Wheel tracking	
AC(80)		Density	Porosity	Density	Porosity
	AC(80)#1	2.398	2.52	2.291	6.88
	AC(80)#2	2.406	2.22	2.245	8.74
	AC(80)#3	2.383	3.15	2.339	4.92
	Average	**2.396**	**2.63**	**2.292**	**6.85**
AC(50)					
	AC(50)#1	2.404	2.91	2.318	3.59
	AC(50)#2	2.405	2.89	2.314	3.79
	AC(50)#3	2.432	1.78	2.330	3.13
	Average	**2.414**	**2.53**	**2.321**	**3.50**
HRA(50)					
	HRA(50)#1	2.330	3.71	2.303	4.82
	HRA(50)#2	2.332	3.61	2.999	4.98
	HRA(50)#3	2.322	4.04	2.339	4.92
	Average	**2.328**	**3.79**	**2.547**	**4.91**
HRA(80)					
	HRA(80)#1	2.346	2.46	2.318	3.59
	HRA(80)#2	2.357	1.99	2.314	3.79
	HRA(80)#3	2.370	1.47	2.330	3.13
	Average	**2.358**	**1.97**	**2.321**	**3.50**
AC(80)+HRA(50)					
	AC(80)+HRA(50)#1	2.349	3.72	2.288	6.23
	AC(80)+HRA(50)#2	2.340	4.07	2.283	6.41
	AC(80)+HRA(50)#3	2.339	4.14	2.295	5.95
	Average	**2.343**	**3.98**	**2.289**	**6.20**

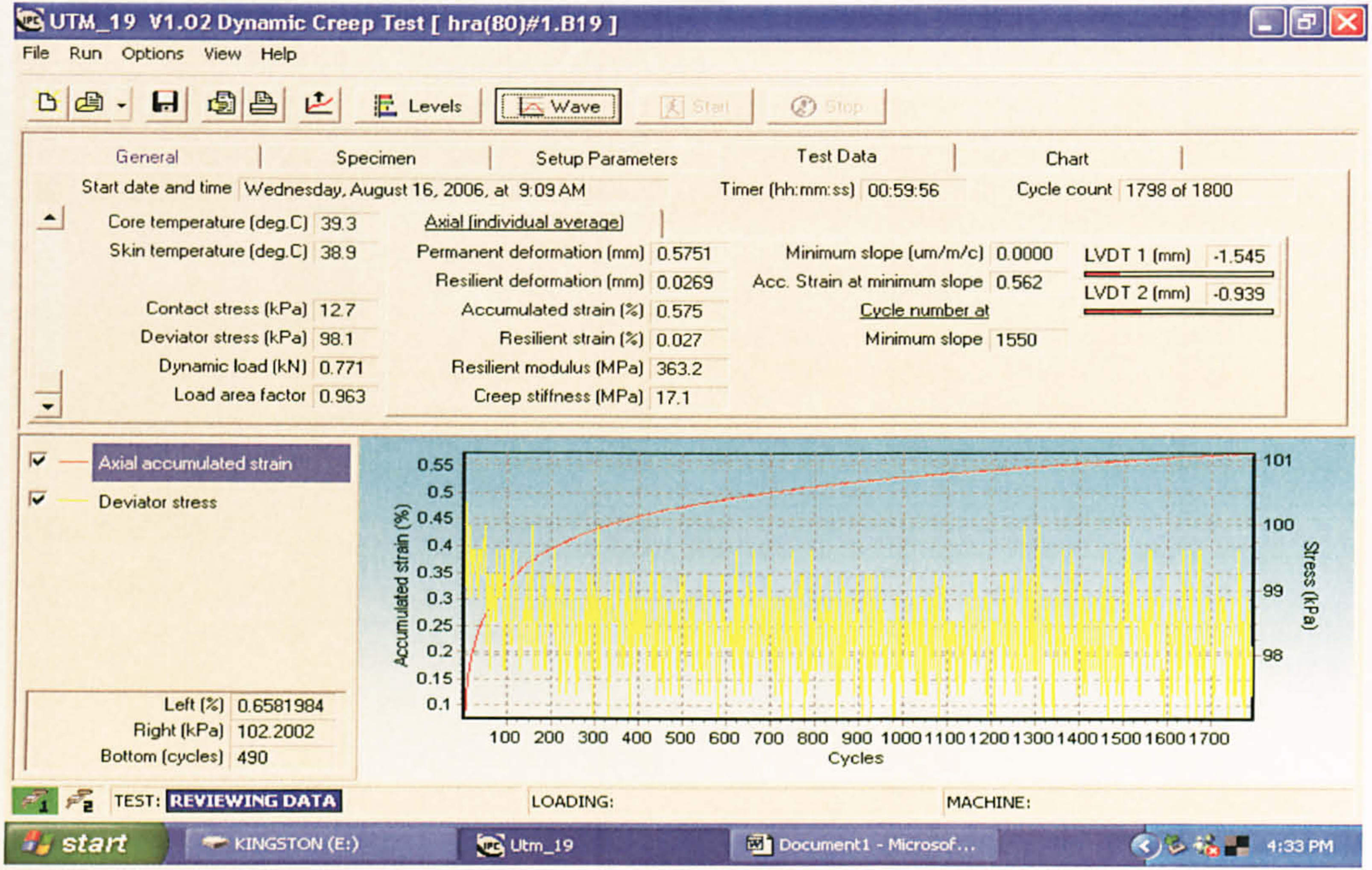
Type of mix	No	Beam fatigue	
AC(80)		Density	Porosity
	AC(80)#1	2.296	6.69
	AC(80)#2	2.294	6.77
	AC(80)#3	2.285	7.11
	AC(80)#4	2.272	7.67
	AC(80)#5	2.276	7.49
	AC(80)#6	2.283	7.23
	AC(80)#7	2.281	7.30
	AC(80)#8	2.248	8.61
	AC(80)#9	2.263	8.04
	Average	**2.278**	**7.43**
AC(50)	AC(50)#1	2.262	5.90
	AC(50)#2	2.272	5.53
	AC(50)#3	2.273	5.50
	AC(50)#4	2.296	4.55
	AC(50)#5	2.284	5.02
	AC(50)#6	2.281	5.13
	AC(50)#7	2.314	5.17
	AC(50)#8	2.313	5.20
	AC(50)#9	2.331	4.45
	Average	**2.292**	**5.16**
HRA(50)	HRA(50)#1	2.335	3.52
	HRA(50)#2	2.335	3.50
	HRA(50)#3	2.319	4.18
	HRA(50)#4	2.327	3.84
	HRA(50)#5	2.313	4.39
	HRA(50)#6	2.336	3.44
	HRA(50)#7	2.337	3.41
	HRA(50)#8	2.343	3.15
	HRA(50)#9	2.346	3.04
	Average	**2.332**	**3.61**
HRA(80)			
	HRA(80)#1	2.284	7.76
	HRA(80)#2	2.283	7.78
	HRA(80)#3	2.306	6.89
	HRA(80)#4	2.307	6.83
	HRA(80)#5	2.324	6.15
	HRA(80)#6	2.263	7.27
	HRA(80)#7	2.293	6.00
	HRA(80)#8	2.270	6.96
	HRA(80)#9	2.305	6.89
	Average	**2.293**	**6.95**
AC(80)+HRA(50)			
	AC(80)+HRA(50)#1	2.307	5.44
	AC(80)+HRA(50)#2	2.306	5.48
	AC(80)+HRA(50)#3	2.302	5.64
	AC(80)+HRA(50)#4	2.293	6.00
	AC(80)+HRA(50)#5	2.316	5.08

	AC(80)+HRA(50)#6	2.296	5.89
	AC(80)+HRA(50)#7	2.270	6.96
	AC(80)+HRA(50)#8	2.263	7.27
	AC(80)+HRA(50)#9	2.296	5.88
	Average	**2.294**	**5.96**

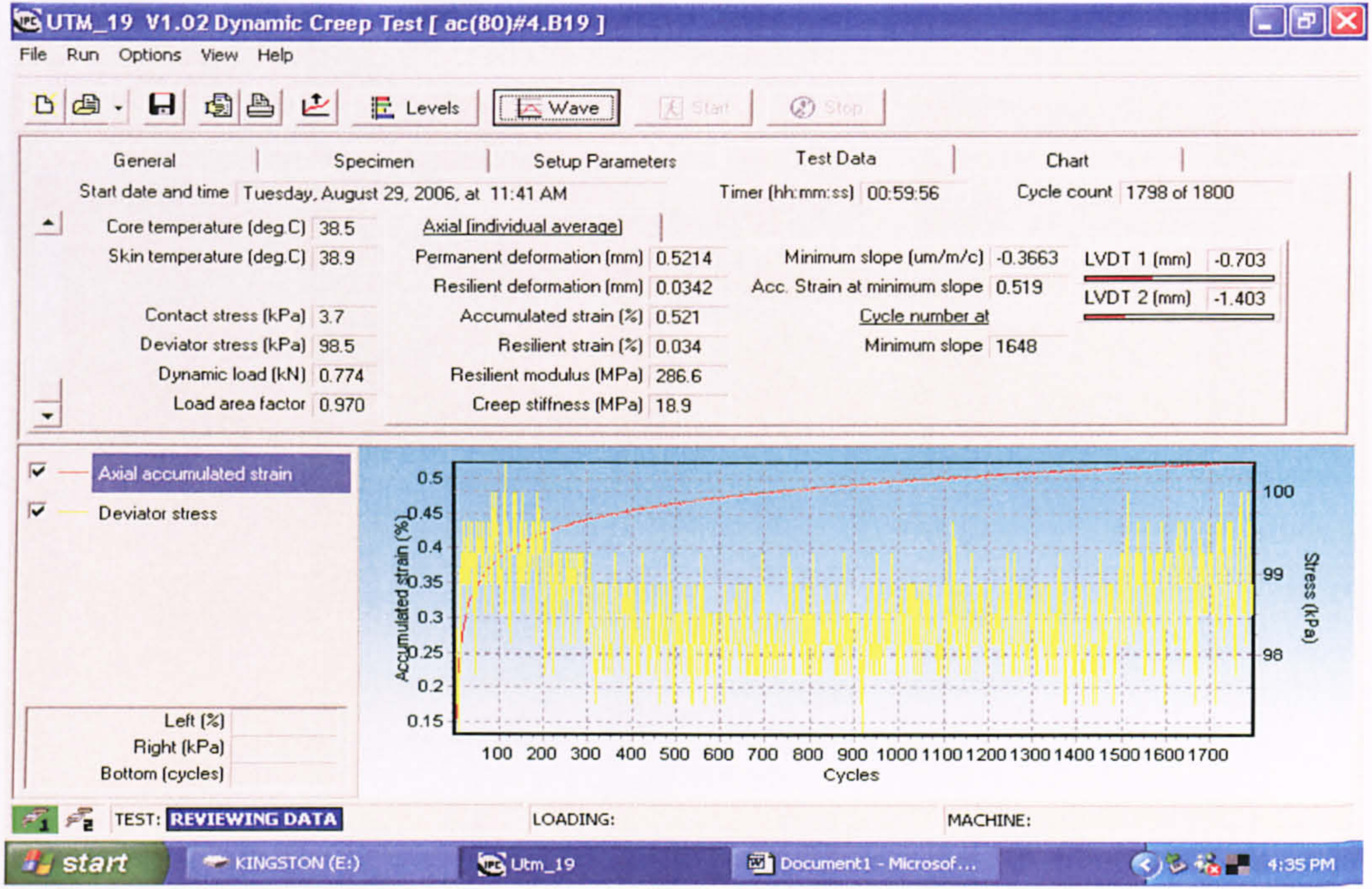
Dynamic Creep Test-AC(50)



Dynamic Creep Test-HRA(80)



Dynamic Creep Test-AC(80)



Wheel tracking result AC(50)

AC(50)#1		AC(50)#2		AC(50)#3	
time	depth	time	depth	time	depth
0	0	0	0	0	0
1	0.3	1	0.6	1	0.5
2	0.4	2	0.7	2	0.6
3	0.5	3	0.7	3	0.7
4	0.6	4	0.8	4	0.8
5	0.6	5	0.9	5	0.9
6	0.7	6	1	6	1
7	0.7	7	1.1	7	1
8	0.8	8	1.2	8	1.2
9	0.8	9	1.2	9	1.2
10	0.8	10	1.3	10	1.3
11	0.8	11	1.4	11	1.3
12	0.9	12	1.4	12	1.4
13	0.9	13	1.4	13	1.5
14	0.9	14	1.5	14	1.6
15	1.1	15	1.5	15	1.6
16	1.1	16	1.6	16	1.6
17	1.1	17	1.6	17	1.6
18	1.1	18	1.7	18	1.6
19	1.1	19	1.7	19	1.7
20	1.1	20	1.7	20	1.8
21	1.2	21	1.8	21	1.8
22	1.2	22	1.8	22	1.9
23	1.2	23	1.9	23	1.9
24	1.2	24	1.9	24	1.9
25	1.3	25	1.9	25	2
26	1.4	26	2	26	2
27	1.4	27	2	27	2.1
28	1.5	28	2	28	2.2
29	1.6	29	2.1	29	2.2
30	1.6	30	2.1	30	2.2
31	1.7	31	2.1	31	2.3
32	1.8	32	2.2	32	2.3
33	1.9	33	2.2	33	2.5
34	1.9	34	2.2	34	2.5
35	2	35	2.3	35	2.6
36	2.1	36	2.3	36	2.6
37	2.1	37	2.4	37	2.7
38	2.2	38	2.4	38	2.7
39	2.3	39	2.4	39	2.8
40	2.4	40	2.4	40	2.9
41	2.5	41	2.5	41	3
42	2.6	42	2.5	42	3.1
43	2.7	43	2.5	43	3.1
44	2.8	44	2.6	44	3.2
45	3	45	2.6	45	3.3
46	3	46	2.6	46	3.4

Wheel tracking result for AC(80)+HRA(50)

AC(80)+HRA(50)#1		AC(80)+HRA(50)#2		AC(80)+HRA(50)#3	
time	**depth**	**time**	**depth**	**time**	**depth**
0	0	0	0	0	0
1	0	1	0	1	0.1
2	0.2	2	0.1	2	0.2
3	0.3	3	0.2	3	0.2
4	0.3	4	0.2	4	0.2
5	0.4	5	0.3	5	0.3
6	0.5	6	0.3	6	0.3
7	0.5	7	0.4	7	0.3
8	0.6	8	0.4	8	0.3
9	0.6	9	0.5	9	0.3
10	0.7	10	0.5	10	0.4
11	0.7	11	0.6	11	0.4
12	0.7	12	0.7	12	0.4
13	0.7	13	0.7	13	0.4
14	0.8	14	0.7	14	0.4
15	0.8	15	0.8	15	0.4
16	0.9	16	0.8	16	0.4
17	0.8	17	0.9	17	0.4
18	0.9	18	0.9	18	0.4
19	0.9	19	1	19	0.4
20	1	20	1	20	0.4
21	1	21	1.1	21	0.4
22	1	22	1.1	22	0.3
23	1	23	1.2	23	0.3
24	1	24	1.2	24	0.4
25	1.1	25	1.3	25	0.4
26	1.2	26	1.4	26	0.4
27	1.2	27	1.3	27	0.4
28	1.2	28	1.4	28	0.4
29	1.2	29	1.4	29	0.4
30	1.2	30	1.5	30	0.4
31	1.3	31	1.5	31	0.5
32	1.3	32	1.6	32	0.6
33	1.3	33	1.6	33	0.6
34	1.3	34	1.7	34	0.6
35	1.3	35	1.8	35	0.6
36	1.3	36	1.8	36	0.6
37	1.4	37	1.9	37	0.6
38	1.4	38	2	38	0.7
39	1.5	39	2.1	39	0.7
40	1.5	40	2.1	40	0.8
41	1.5	41	2.2	41	0.7
42	1.5	42	2.2	42	0.7
43	1.5	43	2.3	43	0.7
44	1.5	44	2.4	44	0.9
45	1.6	45	2.5	45	0.9
46	1.6	46	2.5	46	0.9