THE USE OF RECYCLED VEHICLE TYRE

ON ROAD PAVING IN HONG KONG

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CERTRIFICATE OF ORIGINALITY

I hereby declare that this thesis is my own work and that, to the best of my knowledge and belief, The Work undertaken, the results obtained and the conclusions drawn are the results of my own efforts, except where otherwise stated by reference.

Wong Cheuk Ching

Abstract of thesis entitled “The Use of Recycled Vehicle Tyre on Road Paving In Hong Kong” submitted by Wong Cheuk Ching for the degree of Master of Philosophy at The Hong Kong Polytechnic University in January 2006.

ABSTRACT

In Hong Kong, more than half of the highways are paved with asphalt, some of them are requiring resurfacing and inlaying due to the end of service life. With the ever-increasing traffic engineers have been trying to develop a new mix with a better and durable performance. The main research effort, so far, has been in introducing proprietary modifiers, additives and fibers to asphalt cement. However, disposal of waste tires (a valuable resource for pavement construction in many other countries) carries on and now has been becoming a serious environmental concern in Hong Kong.

Over 2 million wasted tires are discarded in Hong Kong each year. The Government’s latest budget plan stated that a sales tax on tires would be introduced and effective in the middle of 2006. The money from the sales tax of tires would be used as a fund for conducting research in waste tire recycling.

This study aimed to conduct a comparative study of crumb rubber modifier on high temperature susceptibility of wearing course mixture is conducted in the hope to gain basic information on using crumb rubber modified mixtures in Hong Kong.

This study investigates the effects of different sizes of crumb rubber modifier

(CRM) on the high temperature susceptibility of three gradations (AC-10, AC-20 and PA) of wearing course mixtures. A wet process and 10% CRM by total weight of binders were used in these studies and the control variables for these studies included three CRMs of sizes 0.15mm, 0.30mm and 0.60mm. The evaluations were twofold. Firstly, a comparison of the properties of those modified and unmodified binders at a wide range of testing temperatures and ageing conditions was conducted. Secondary, a comparison of the rutting resistance of the CRM and conventional mixtures was made.

The results show that all the CRMs have overall contributed to better performance of both binders and mixtures at high temperatures. In addition, among these three CRM sizes, mixtures modified with 0.15mm CRM exhibited the best effect on the dense-graded mixture (AC-10 and AC-20) whereas mixtures modified with 0.60mm CRM exhibited the best effect on the open-graded mixture of porous asphalt (PA).

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CHAPTER 1

INTRODUCTION

1.1 Background

The use of crumb rubber modified (CRM) asphalts has been increasing significantly in recent years over the world, especially in the U.S. The purpose of using crumb rubber modified binders is to improve the hot mix asphalt (HMA) mixture's resistance to cracking and rutting failures under loading and severe environmental condition. Until now, two major processes have been developed; the wet process defines any method that adds crumb rubber modifier to asphalt cement before incorporating the resultant binder into aggregates, and the dry process defines any method of pre-blending crumb rubber modifier with heated aggregates before charging the mix with asphalt cement. The wet process is the most commonly used method and it is found to exhibit consistent performance. Past experiences have shown that dry process applications exhibited widely varying performance, ranging from acceptable to disastrous. The main reasons for the fluctuated performance of dry process are claimed to be due to the poor control in meshing the gradation of the aggregate and crumb rubber and the lack of understanding of the volume changes taking place due to swelling of the crumb rubber during the mix process and handling.

In the U.S, a great deal of tones of asphalt rubber modifier mix was placed throughout the country since the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA). Most of the roads have been reported in good conditions after several years in service when compared with the conventional design. Some States like California, Florida, and Arizona, have reported cost-effective application while other states have reported inferior.

Another main reason for the increasing use of the crumb rubber modifier is due to the environmental concern as million tones of used tyres have been disposed throughout the world every year that create the environment waste treatment problem.

The performances of crumb rubber modified mixtures are influenced by many factors, for instance, mixing process (wet or dry process), asphalt types, rubber contents and aggregate gradations. Hence, the design in a place might be unsuitable for another. Under these circumstances, a comparative study of crumb rubber modifier on high temperature susceptibility of wearing course mixture is conducted in the hope to gain basic information on using crumb rubber modified mixtures in Hong Kong.

1.2 Objectives

The main objectives of this study include the following:

1) To evaluate the basic rheological properties of crumb rubber modified binders at a wide range of testing temperatures and ageing conditions by using the new testing techniques (ring and ball, viscosity and dynamic shear rehometer test) and ageing procedures developed by the Strategic Highway Research Program (SHRP).
2) To compare the high temperature susceptibility and moisture susceptibility of CRM mixtures with those of the unmodified mixture as well as to develop a procedure for mix design to select the most appropriate proportioning of binder and aggregate gradation for wearing course design.

1.3 Organisation of thesis

The organizations of this thesis are as follows:

1) Chapter 1 introduces the general background and objective of the study.
2) Chapter 2 is the literature review. It provides the basic concept on CRM paving materials through reviewing a great deal of historical papers and thesis.
3) Chapter 3 is the methodology that describes the test equipments and procedures.
4) Chapter 4 compares the performance of crumb rubber modified binder and that of conventional binder.
5) Chapter 5 compares the high temperature and moisture susceptibility between CRM

mixtures (AC-10 and AC-20) and unmodified mixtures.

6) Chapter 6 studies the rutting behaviour of crumb rubber modified porous asphalt.

7) Chapter 7 is the summary of the conclusions. Further recommendations are also presented.

CHAPTER 2

LITERATURE REVIEW

2.1 Definition of Crumb Rubber Modifier and Asphalt Rubber

Over the past few decades, engineers have been trying their best to blend natural rubber (latex) and synthetic rubber (polymers) in asphalt cements to enhance the elastic properties of the binder and tyre rubber, a compound of natural and synthetic rubber is regarded as an available raw material (Heitzman, 1992). Crumb Rubber modifier is a general term for scrap tyre rubber that is reduced in size and is used as modifier in asphalt paving materials. Asphalt rubber is a mixture of crumb rubber, asphalt cement and in some cases extender oil, which is reacted at elevated temperatures prior to being mixed with aggregate (Hicks et al, 1995).

2.1.1 Methods of Processing Scrap Tyre Rubber into CRM

In most specifications the crumb rubber particles must be free of metal, fabric and moisture, also, research has indicated that crumb rubber particles with greater surface area produced in ambient systems provide greater reactivity with liquid asphalt. Normally, a scrap tyre weighing approximately 9kg will produced 4.5 to 5.5kg of crumb rubber modifiers as the steel belting and fiber reinforcing are separated and removed from the rubber during the processing of producing the crumb rubber modifiers. Currently, there are three methods currently used to process scrap tyre rubber into CRM.

They are the crackermill process, the granulator process and the micromill process. They are as follows:

- **The crackermill process**: this is the most common method, it tears apart scrap tyre rubber, reducing the size of the rubber by passing the materials between rotating corrugated steel drums. This process produces an irregularly shaped torn particle with a large surface area with a range of sizes from 4.75mm to 0.425mm. These particles are commonly described as a ground CRM.
- **The granulator process**: it shears apart the scrap tyre rubber, cutting the rubber with revolving steel plates that pass at close tolerance. This process produces a cubical, uniformly shaped cut particle with a low surface area and the particles sizes typically range from 9.5mm down to 2.00mm. The particles are usually called granulated CRMs.
- **The micro-mill process**: this process further reduces a crumb rubber to a very fine ground particle with particles ranges from 0.425mm down to 0.075mm.

2.2 Terminologies

Publications during the last 30 years used a variety of terms to define different processes and products as the technology evolved. Conflicting terminology has made it difficult for many user agencies to understand it. But In general, CRM technology can be divided into two categories. These categories define the basic process used to add the crumb rubber to an asphalt paving material. They are the wet process and the dry process (see Figure 2-1).

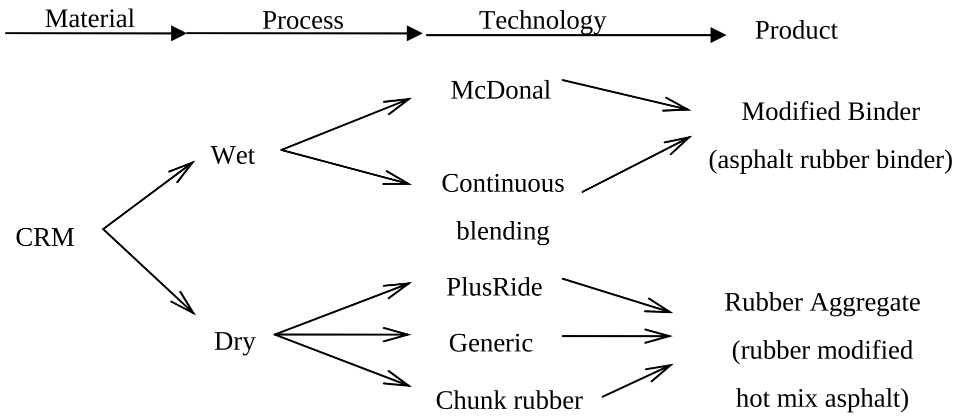


Figure 2-1. Relationship of Crumb Rubber Modifier Terminology.

2.2.1 The Wet Process:

The term wet process defines any method that blends the crumb rubber with the asphalt cement before incorporating the binder into the mixture, the product of this process is usually entitled Asphalt-Rubber Concrete. The wet process has been applied to crack sealants, surface treatments, and HMA mixtures (Heitzman, 1992).

Normally, for the wet process, 5 to 20% (by totally weight of binder) tyre rubber is reacted with bitumen at an elevated temperature (170ºC to 190ºC) for about one hour under the high-speed shear machine to produce a crumb rubber modified binder in HMA construction. The mixing and compaction temperatures used for CRM asphalt concrete are relatively higher than those used for conventional asphalt concrete. The primary reasons for such high temperatures are due to the very high viscosity of the CRM asphalt cement binder at temperature defined for the Marshall method, and due to the difficult of wetting the aggregate surface with CRM asphalt cement at normal temperatures (Veizer, 1993).

2.2.2 The Dry Process

The term dry process defines those methods that mix the crumb rubber with the aggregate before the mixture is charged with asphalt binder and it is usually regarded as Crumb Rubber Modified Asphalt Concrete. The dry process is limited to HMA applications (Heitzman, 1992).

Normally, for the dry process, tyre rubber amounting to about 2 to 3% of the aggregate weight is added to the aggregate as a replacement of the aggregate before the asphalt is introduced and mixing occurs. The method of adding rubber to asphalt mixes in a dry process was originally developed in the late 1960’s and patented under the trade names of “PlusRide” in the U.S. and “Rubit” in Sweden.

2.2.3 The Principle Difference between the Dry and Wet Process

The principle difference between these two processes includes the sizes of rubber (the rubber particle used in the dry process is much coarser than that in the wet process), the amount of rubber (the tyre rubber used in the dry process is 2 to 4 times as much as in the wet process), function of the rubber (in the dry process, the rubber acts more like an aggregate but in the wet process it acts more like the binder), and ease of incorporation into the mix (in the dry process, no special equipment is required while in the wet process special mixing chambers, reaction and blending tanks, and oversized pumps are required) (Freddy et al, 1989).

2.3 Experiences in the United States

2.3.1 The First Introduction of Rubber in Hot Mix Asphalt

The first introduction of using rubber as a modified binder can be traced back to 1960s, Charles H. Macdonald, a pavement engineer from the U.S. Bureau of Public Roads, first thought of asphalt-rubber while travelling across the country inspecting highway material sources for the Bureau of Roads. His mobile trailer's roof cracked and he used asphalt as a quick patching material. However, after frequent moves and long exposure to the sunlight, the asphalt oxidized and became brittle. The roof cracked through to the surface of each successive asphalt patch. He then thought he could solve the cracking problem if he incorporated rubber in his next round of patching (Winter, 1989). So the first attempt on using rubber in asphalt was made by Charles H. Macdonald.

2.3.2 The Development of CRM in the U.S

In the early 1960s, Charles Mcdonald, materials engineer for the city of Phoenix, began working with a local asphalt company, Sahuaro Petroleum, to develop a highly elastic maintenance surface patching using CRM. Since the 1960s, researchers and engineers have used shredded automobile tyres in hot-mix asphalt (HMA) mixtures for pavements. In 1968, the Arizona Department of Transportation (ADOT) placed its first stress absorbing membrane (SAM), a surface treatment using an asphalt rubber binder. ADOT placed their first stress-absorbing membrane interlayer (SAMI) in 1972 and used the asphalt rubber binder in HMA open-graded friction course in 1975. The dry process was developed in the late 1960s in Sweden. The European trade name for this HMA mixture with CRM as a rubber aggregate was Rubit. The Swedish technology was patented for use in the United States in 1978 under the trade name PlusRide (Heitzman, 1992).

The use of Crumb Rubber Modifiers (CRM) in asphalt mixtures has been increasing significantly in recent years over the world, especially in the U.S. Some of the roads have been reported in good conditions after several years in service by comparison with the conventional design.

In U.S, States like California, Florida and Arizona have reported cost-effective application while other states have reported inferior (Maupin, 1996) (Hicks et al, 2001)

(Ruth, 1992). As mentioned before, until present, two major processes have been developed; the wet process defines any method that adds the CRM to the bitumen and well blended before incorporating the modified binder into the mix aggregates. The dry process defines any method of adding CRM directly into the HMA mix process. Past experiences have shown that dry process applications exhibited unstable performance, ranging from acceptable to disastrous. The main reason for the fluctuated performance of dry process is claimed to be owing to the poor control in meshing the gradation of the aggregate and crumb rubber and lack of understanding of the volume changes taking place due to swelling of the crumb rubber during the mixing process and handling (Lougheed and Papagiannakis, 1996).

The development of the CRM technology has been passing through three stages. The first stages was the patent stage which was mainly between the years of 1965 to 1991, in this early stage, the application of CRM technology on road pavement was uncommon because of the high initial construction price due to the patent of those technologies. Also at that time, those technologies were basically on the basis of application experience and there was lack of research works. The second stage was the after-patent stage; most of the patents were expired in the early 1990. At that time, engineers and researchers have put great emphasis on the use of crumb rubber modified pavement in order to become one of the effective solutions to solve this solid waste problem as well as attempting to improve pavement performance. Many cases were cited in literature where rubber modified asphalt has shown superior performance to conventional asphalt cement. In 1992, the United Stated Congress passed the Intermodal Surface Transportation Efficiency Act (ISTEA) to mandate the use of CRM asphalt concrete beginning on January 1, 1994. According to the ISTEA, all states were required to use increasing percentage of rubber in the construction of asphalt concrete pavements (5, 10, 15 and 20% by weight) over the next five years (starting in 1994). But after a few years of implementing the ISTEA, some states claimed that this federal law was a bit severe as some states did not even have enough experiences for using CRM technology for national standards on mix design procedures and performance criteria are lacking, also, some engineers questioned about the consistent of the advantages of this technology. The third stage is in the last decade, more and more researchers and engineering have put great emphasis on this kind of modified materials, not only for solving the environment problems, but also for the enhancement of the paving materials.

2.3.3 Experiences in Arizona

Arizona is considered as the pioneer to the asphalt rubber industry. And Mcdonald is regarded as the pioneer to incorporate crumb rubber modifiers in the paving construction. He patented what is currently called the Mcdonald Process or Wet Process for making asphalt rubber in the early 1960's. From 1968 to 1972, the Arizona Department of

Transportation (ADOT) placed six projects with AR in a seal coat type applications. In 1968, the Arizona Department of Transportation (ADOT) places its first stress absorbing membrane (SAM), a surface treatment using an asphalt rubber binder. ADOT placed their first stress-absorbing membrane interlayer (SAMI) in 1972. From 1974 until 1989, approximately 660 miles of state highways were built using a SAM or SAMI application of AR and both the SAM and SAMI applications showed great promise in reducing reflective cracking (Way, 1989). Nowadays, the asphalt rubber mixes in Arizona are either open-graded or gap-graded and from half inch to one inch or one inch to two inches in thickness respectively.

- **The Use of Stress Absorbing Membrane (SAM) as Surface Treatments**

  A surface treatment using an asphalt rubber spray application is called a Stress Absorbing Membrane (SAM) (T. S. Shuler, et al). The majority function of SMA is to resist and delay the development of reflective cracks when the cracks are generally inactive (see Figure 2-2).

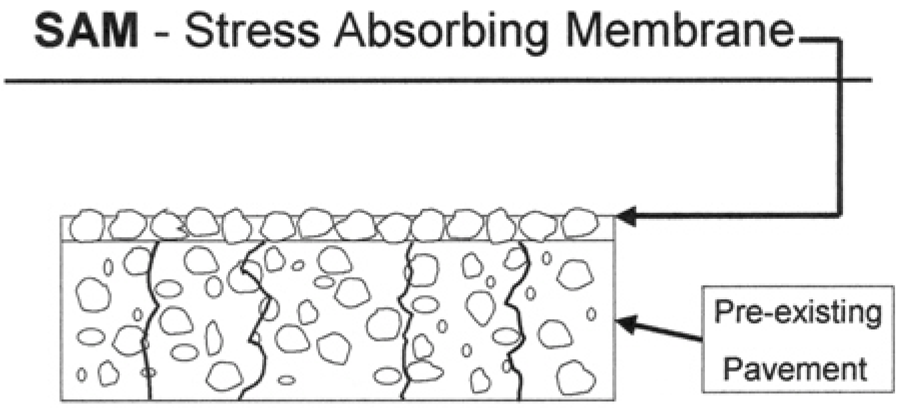


Figure 2-2. The Stress Absorbing Membrane

- **The Use of Stress Absorbing Membrane Interlayer (SAMI)**

The Stress Absorbing Membrane Interlayer (SAMI) is composite designs which place a membrane beneath the overlay that can resist the stress-strain of reflective cracks and delay the propagation of the crack through the new overlay. Generally, there are two composite design systems; there are a two-layer SAMI (see Figure 2-3) and a three-layer SAMI (see Figure 2-4). For the two layer system, the SAMI is placed on the existing pavement and overlays the SAMI with a 25mm to 75mm thick HMA. For the three-layer system, the existing pavement is first placed with a HMA levelling course, which provides an acceptable uniform surface for placing the SAMI, then the SAMI is followed by an additional of 25 to 75mm HMA. Usually, the three-layer system is used when there is deterioration of the existing pavement cracks and joints. The main benefit for using SAMI is to delay the reflective cracking.

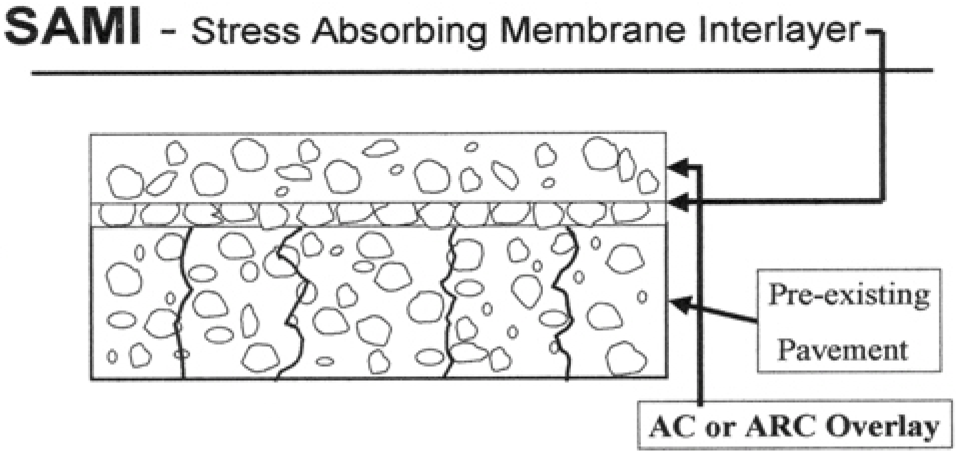


Figure 2-3 A Two Layer System of SAMI

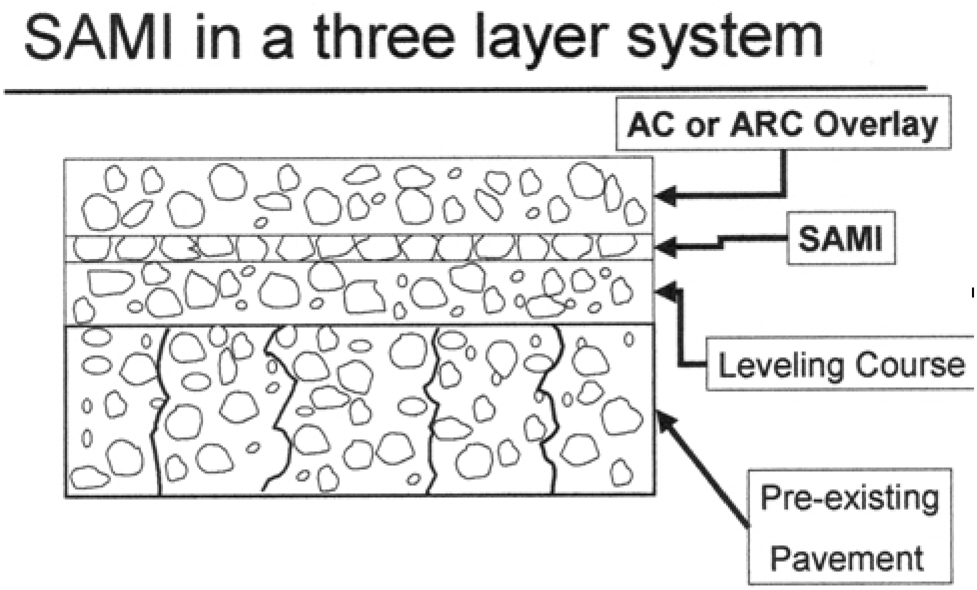


Figure 2-4 A Three Layer System of SAMI

- **The history of OGFC in Arizona**

The Open Graded Friction Course (OGFC) was first used by the Arizona Department of Transportation (ADOT) in 1954 (Morris and Scott, 1973). The primary reason for using OGFC was to provide a surface with good skid resistance, good readability and appearance. It was until in 1998, a one inch layer of an open-graded asphalt rubber asphalt concrete friction course commonly referred to as AR-ACFC was placed on several miles of Interstate 19, South of Tucson and there was no cracks reflected through until 1998, where only a few transverse cracks appeared over the concrete joints. In 1998 District Maintenance reviewed this project and concluded that no maintenance was needed. Following that project, several other projects, with the purposes of reducing reflective cracks with a thin layer of very elastic materials, have been built with asphalt rubber as the binder. And up to now, all those projects have performed well (George and Way, 1998). Nowadays, AR-ACFC is generally used as the final wearing surface for both rigid and flexible pavement. About all, it can be used to improve smoothes, reduce cracking and rutting, provide adequate skid resistance and reduce noise.

2.3.4 Experiences in Florida

In the late of 1970s, the Florida Department of Transportation (FDOT) started to use asphalt-rubber as a stress absorbing membrane interlayer and as moisture barrier and the results were encourageing. So in 1988, the Florida legislature passed a bill (Senate Bill 1192) directing the Florida Department of Transportation (FDOT) to conduct an investigation on the feasibility and potential use of ground tyre rubber in asphalt concrete mixtures. To address the legislative mandate, the FDOT considered its different mixture types that would potentially benefit from the addition of rubber. It was found that a more promising use of rubber would be in friction course mixes. The addition of GTR would increase the overall strength or rutting resistance of the dense-graded friction courses (Ruth et al, 1989) and would also improve the durability of the open-graded friction course (Ruth, 1989). On the other hand, the mixtures which contain reclaimed asphalt pavement (RAP) would not be suited for the addition of rubber since the latter would interfere with the rejuvenation of the older binder that occurs during the recycling process (Choubane et al, 1988). The experiences in Florida also indicated that all wet process asphalt-rubber sections performed significantly better than dry-mixed asphalt-rubber test sections. In 1994, the FDOT initiated the implementation of specifications requiring the use of ground tyre rubber in all asphalt surface mixes. Since the implementation of these specifications in 1994, over 2700000 metric tones of rubberized asphalt surface mixtures have been placed throughout the state by the year of 1998 and it’s expected that about one-fifth of this quantity will be used annually in hot mix applications.

2.3.5 Experience in Texas

Like Arizona and Florida, Texas has a long history of utilizing asphalt rubber in construction and rehabilitation of pavements. The first project of using asphalt rubber in Texas can be traced back to 1976. In 1982, an extensive research project conducted by Texas A&M University in 1982, the researchers evaluated performance of nearly 800 miles of seal coat and under seal constructed throughout Texas from 1976 and 1981 and it was concluded that asphalt rubber binders can be effectively used in seal coat construction to reduce alligator cracks and raveling when compared to conventional seal coat performance (Schuler et al, 1982).

In Texas, asphalt rubber has been used in four different types of applications; there are chip seal coat (SAM), underseal (SAMI), hot mix asphalt and porous friction Course. Based on the conclusions from a studies conducted by PaveTex Engineering and Testing, Inc, most of the projects containing asphalt rubber were in better condition by compare with the conventional design.

2.3.6 Experience in Other States

In 1991, the Intermodal Surface Transportation Efficiency Act (ISTEA) specified that all asphalt pavement project funded by federal agencies must use certain percentages of scrap tyres. Although this mandate was later suspended from the ISTEA legislation, it has greatly encouraged the research and application of CRM asphalt in HMA pavement.

Since then, many state agencies have conducted numerous field studies for the feasibility of using recycled rubber tyre products in HMA Pavements. The National Cooperative Highway Research Programs (NCHRP) “Synthesis of Highway Practice 198 – Uses of Recycled Rubber tyres in Highways” provides comprehensive review of the use of recycled rubber tyres in highways based on a review of nearly 500 references and on information recorded from state highway agencies’ responses to a 1991 survey of current practices (Epps, 1994).

Virginia DOT constructed pavements containing CRM asphalt in 1990, Virginia Senate Bill No. 287, which encouraged demonstration projects using ground rubber from used tyres in road surfacing, was passed. Since then, the Virginia DOT constructed pavements containing CRM asphalt mixtures produced by two wet processes, McDonald and Rouse compared the pavement performance, and the result showed that the mixes containing asphalt rubber performed at least as well as conventional mixes (Maupin, 1996).

2.3.7 Cost Analysis (Initial Cost vs Lift Cycle Cost)

Generally speaking, due to the blending process of crumb rubber modifier and asphalt cement, the cost of asphalt rubber binder is more than twice that of conventional asphalt cement and can be slightly more than polymer modified asphalt, as a result of that the initial cost for asphalt rubber mixes can be 30 to 80% higher than that of the conventional mixes. These higher initial costs have retarded some paving agencies to expend their business into this area. But this is obviously foolish to just consider the initial cost and another picture can be seen if we look into the life cycle cost. It can be easily proved from the experience in states like California and Arizona which have extensive experience with asphalt rubber has been shown to be a very cost-effective binder for pavement maintenance and rehabilitation strategies when properly produced and constructed (Kirk et al, 2000). The main reasons for the overall cost of the asphalt rubber project is less than the conventional project are the reduction in thickness and the lower maintenance costs in the long run. Over the past two decades, researches compared asphalt rubber concrete to conventional asphalt concrete (AC) in field evaluations. It was determined through these field evaluations that the asphalt rubber pavements could be significantly reduced in thickness and provides the same service life as thicker conventional AC pavements. The thickness can be reduced up to 50%. It was substantiated by research in South Africa in 1994 in field installations using the Heavy Vehicle Simulator (Rust et al, 1993), by the University of California, Berkeley in 1994 in the laboratory (Kirk et al, 2000) and by the University of Alaska, Fairbanks in 1995 (Raad et al, 1995) in the laboratory.

A significant cost can be saved when multi-layer asphalt rubber is used especially when reconstruction is the recommended solution for the conventional design, many cities in the U.S have realized saving of 1/2 to 2/3 the cost of reconstruction. Here is one example, a project in Hemet, California where reconstruction was the recommended strategy on the project. Table 2-1 below shows the cost estimate for the alternatives recommended by the design engineer for the project, which was based on the pavement deflections.

Alternative 1	135mm conventional AC overlay	Cost $363,000
Alternative 2	90mm conventional AC over 330mm class 2 aggregate base	Cost $646,000
Alternative 3	39mm ARHM-GG over 48mm conventional AC	Cost $264,000
	Savings (Alt. 3 over Alt. 1)	$124,000
	Savings (Alt. 3 over Alt. 1)	$382,000

Table 2-1 The Cost Estimate for a Project in Hemet, California

2.4 Experience In Other Countries

2.4.1 In Canada

- Stone Mastic Asphalt (SMA) with CRM binder

  Stone Mastic Asphalt (SMA) was invented in Germany about 35 years ago to better resist the wear of studded tyres. SMA is now widely used around the world. Stone Mastic Asphalt (SMA) is a gap-graded, dense, hot-mix asphalt with a large proportion of course aggregates which is limited to about 20 percent and rich asphalt cement/filler mastic. SMA has a coarse, aggregate skeleton, like Porous Asphalt (PA), but the voids are filled with a fine aggregate/filler/bitumen mortar. In 1993, Robert Veizer from the Carleton University studied the effect of crumb rubber modifier on the Stone Mastic Asphalt. It was found that the crumb rubber modified asphalt SMA mixes showed better performance by comparing with the conventional SMA Mix. The results of the performance related tests were very promising. The benefits are not only to achieve an excellent resistant to cracking but also increase the rutting resistance.

2.4.2 In Taiwan

- CRM mixtures have been used in Taiwan for over a few years; several Roads including the one in Hsin-chu County have been paved with CRM asphalt. All those design was using wet process. The initial field data show that CRM mixtures perform better than the dense-graded control mixture. According to a research by Taiwan Construction Research Institute, Using wet process to produce asphalt mixtures is applicable in Taiwan.

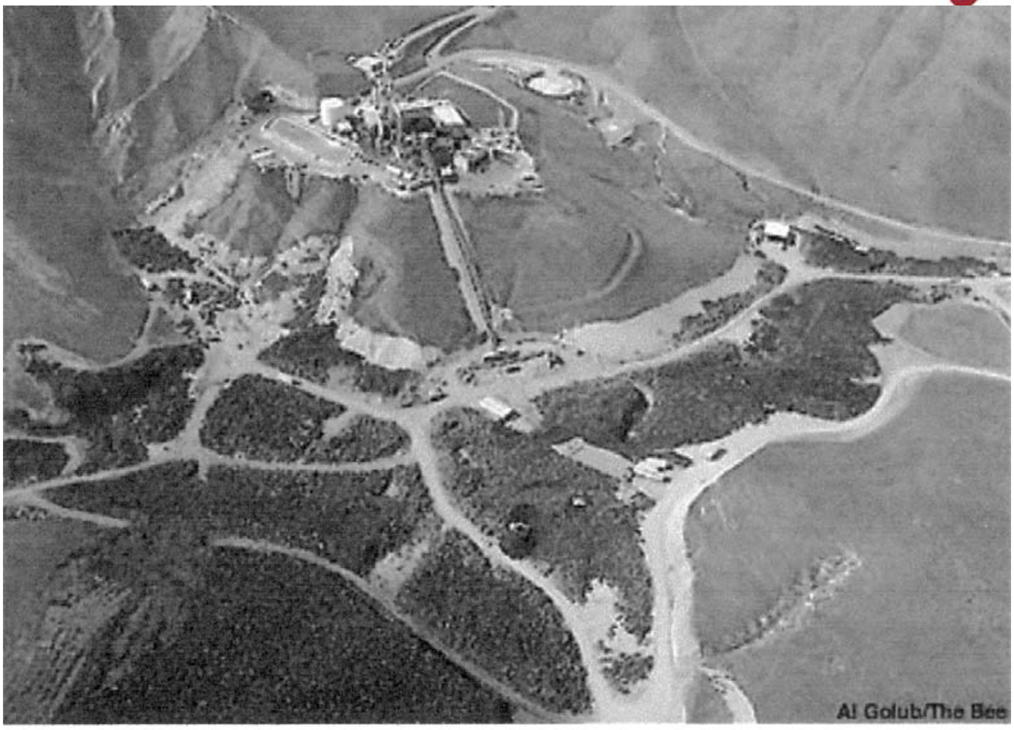
- **2.4.3 In China**

CRM mixture have been using in China for nearly 10 years, this materials was first introduced in the area with cold-climate for the purpose of reduce cracking. Now more and more engineers and scholars put emphasis on the studies of CRM mixture. Also, more and more local paving producer is involving in this market and some of them have done a very good job of that.

2.5 Environmental Concern

It is estimated that in most developed countries approximately one tyre is discarded per adult each year. Discarding of rubber tyre is becoming a worldwide environmental concern. In the U.S. alone, approximately 250 million scrap tyres are generated each year and nearly 200 million tyres are added to stockpiles, landfills and illegal dumps across the country annually (see Pictures 2-1 and 2-2).

Picture 2-1 Discarding of Rubber Tyre



Picture 2-2 Tyres are added to Stockpiles, Landfills and Illegal Dumps

An even larger environmental problem has been from fires of tyre pile, which can be continued for days giving thick acrid black smoke that can spread to a large area. The plume of the burnt tyres contains air pollutants and toxic chemicals. In the U.S. the last tyre fire came in 1999, when a 140-acre tyre dumping site in Ohio caught fire. A column of toxic smoke were seen some 60 miles away releasing toxic which killed thousands of birds and fish to the immediate ecosystem.

Discarded rubber tyres can threaten public health and safety. Dumps of tyres are excellent breeding grounds for health threatened inserts (such as mosquitoes etc.). It elevates incidents of mosquito-borne disease, like “West Nile Virus” in Canada, “Aedes albopictus” of the Asian Tiger Mosquito in the US and “Dengue Fever” in Hong Kong.

A potential use for a significant number of tyres is to expend the use of Asphalt-Rubber in HMA construction and highway maintenance activities. According to the Rubber Pavements Association, a two-inch thick overlay of asphalt-Rubber Hot Mix will use about 2000 tyres per lane per mile.

Most states in the US initiated legislation to control the flow of waste tyres in the 1990s in response to heightened environmental concerns. For example, in 1989 only five states regulated the flow of waste tyres. By 1991, thirty-five states had adopted tyre legislation. By 1998, forty-eight states had implemented scrap tyre legislation or regulations. Thirty-five states banned the practice of tyre disposal in landfills. Fees related to tyre sales or vehicle legislations were established to provide funding for pile clean up efforts and to simulate waste market development to divert the flow of scrap tyres from the piles. Funding is typically made available to tyre processors through grants or direct reimbursement to product output. The following are the tyre program management in the States of Arizona, California and Florida.

In Arizona, according to the Arizona Department of Environmental Quality, approximately 4 million scrap tyres are generated annually, of these, three million are diverted to a crumbing facility near Phoenix. Approximately 2.6 million are used in paving applications and the remainder are used in molded products, gardening products or safety material (see Table 2-2)

Cement kilns / energy recovery	0
Exported or landfill cover	1 million
Reuse/retread or other recycled products	0.4 million
Crumb for paving applications	2.6 million

Table 2-2 Scrap Tyre uses in Arizona, 1998

In California, according to the California Integrated waste Management Board (CIWMB), approximately 30 million waste tyres are generated each year and approximately 50% of annual waste tyre flow is diverted to alternative end uses. Of these, approximately 2.7million are used in paving (see table 2-3)

Cement kilns	4.9 million
Energy recovery	3.5 million
Reuse/retread or other recycled products	4.3 million
Crumb for paving applications	2.7 million

Table 2-3 Scrap Tyre uses in California, 1998

In Florida, the Florida tyre program was established in 1988. At that time, the estimated 18 million tyres were stockpiled around the state. By the year 1999, less than 3 million tyres remained. According to the Florida Department of Environmental Protection (FDEP), approximately 20 million waste tyres are generated annually. Of these, approximately 3 million are used in paving, other end uses of waste tyres are depicted in Table 2-4.

Cement kilns/energy recovery	9.1 million
Reuse/retread or other recycled products	6.3 million
Crumb for paving applications	3.0 million

Table 2-4. Scrap Tyre uses in Florida, 1997-98

2.6 Properties Considered In Mix Design

Rubberized Asphalt Concrete（RAC）does not function well until it's properly design. Certain desirable properties should be aware during the design process, they are as follows:

- **Stability:** Stability in a RAC pavement is its ability to resist shoving and ruffling under traffic loads. Stability standards should be designed for the type of traffic. High stability design produces a pavement that is too stiff and therefore less durable. Low stability results in ruts, ripples, and other signs of movement. Stability is the result of friction between the aggregate particles and the cohesion provided by the asphalt rubber binder. Lowered stability is caused by excessive binder, excessive medium sand, or excessive rounded aggregates.
- **Durability:** Durability is the ability to resist changes in the asphalt such as polymerization or oxidation, disintegration of the aggregate, and stripping of the asphalt rubber binder from the aggregate. Durability is enhanced by providing the

maximum amount of binder and by compacting the mixture to reduce the air voids to between three percent and five percent. Poor durability is the result of too little binder, improper compaction, or water susceptible aggregates.

- **Impermeability:** Impermeability is the resistance of a RAC pavement to allow water to pass through. A gap graded RAC pavement is more permeable than a dense graded pavement mix. However, the additional binder in the gap graded mix compensates for the stripping effect of a permeable mix and provides a highly skid resistance surface that seems to absorb rainwater without the splash effect of a conventional dense graded mix.

- **Workability:** workability is the ease with which a paving mixture can be placed and compacted. The workability of a RAC mix is somewhat less than a conventional mix and requires special consideration during construction. First, RAC can not be easily raked; therefore, it's necessary that the screed height be set so that the compacted mat is even with the adjacent pavement. Second, the breakdown rolling must be completed before the mat reaches a temperature of 290°F. Two breakdown rollers are strongly suggested. RAC cannot be compacted at cooler temperature as well as conventional asphalt concrete. Poor workability is the result of excess coarse aggregate, low temperatures, excess medium sand, or improper mineral filler content.

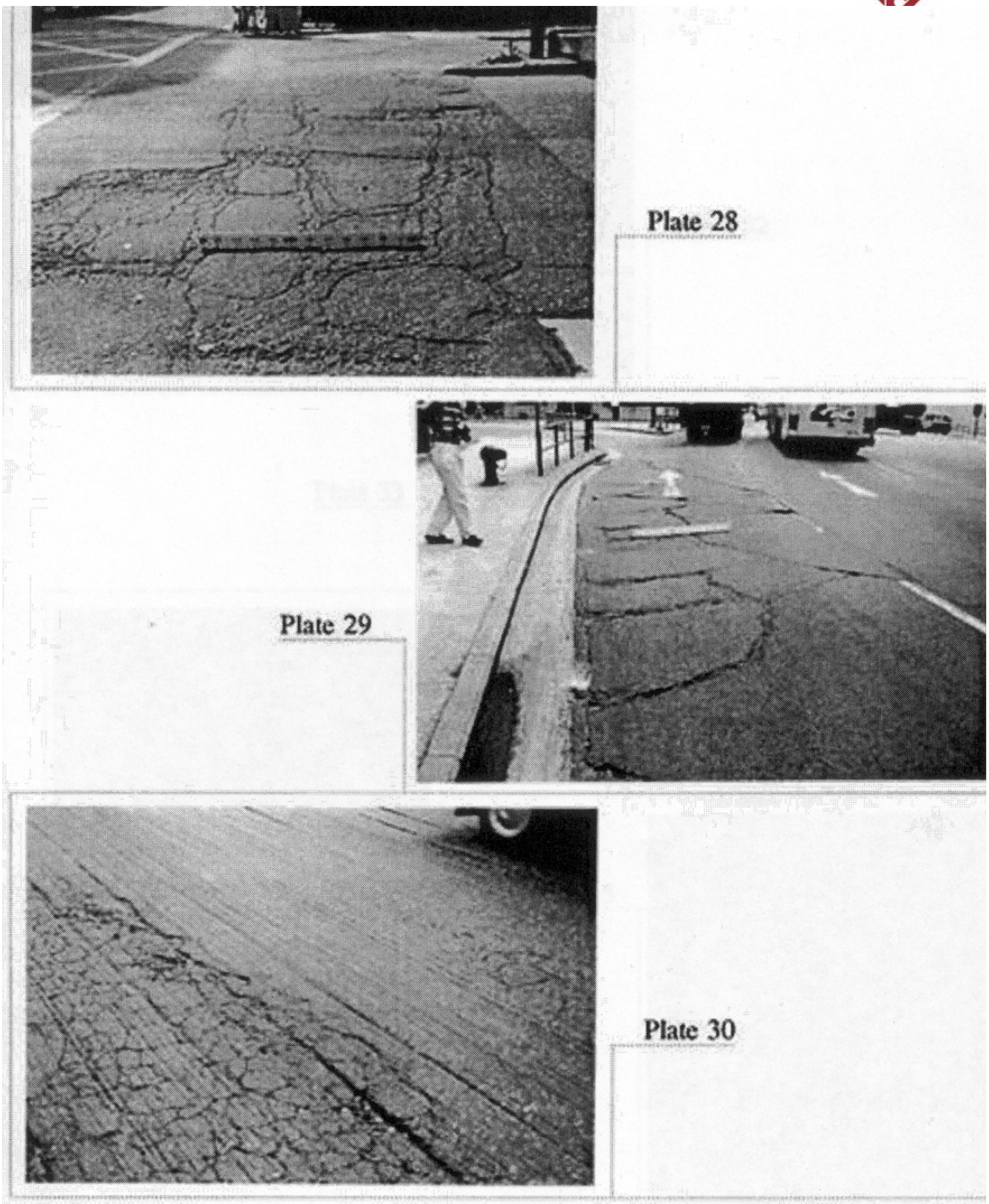
- **Flexibility:** Flexibility is the ability of a pavement to adjust to gradual settlements and movement without cracking. A gap graded or open graded mix is more flexible than a dense graded mix. This is one of the reason RAC has a high resistance to reflective cracking.

- **Fatigue Resistance:** Fatigue resistance is the pavement's resistance to repeated bending under wheel loads. Air voids and binder viscosity has a significant influence on fatigue resistance. Poor resistance is caused by low asphalt or asphalt rubber content, high air voids, inadequate pavement thickness. Recent studies have shown that a reduced thickness of RAC (one half) resist fatigue cracking 1.5 times longer than a full depth thickness of a dense graded conventional asphalt concrete. A minimum thickness of 1-1/2 inches of RAC is recommended.
- **Skid Resistance:** Skid resistance is the ability of a pavement surface to minimize skidding or slipping of vehicle tyres by reducing the effects of hydroplaning. A gap graded RAC pavement has a high skid resistance value due to the open nature of the surface and its ability to resist bleeding or flushing. Poor skid resistance is caused by excess asphalt rubber binder (flushing), improperly graded aggregates, or soft aggregates resulting in polishing of the aggregate at the surface.

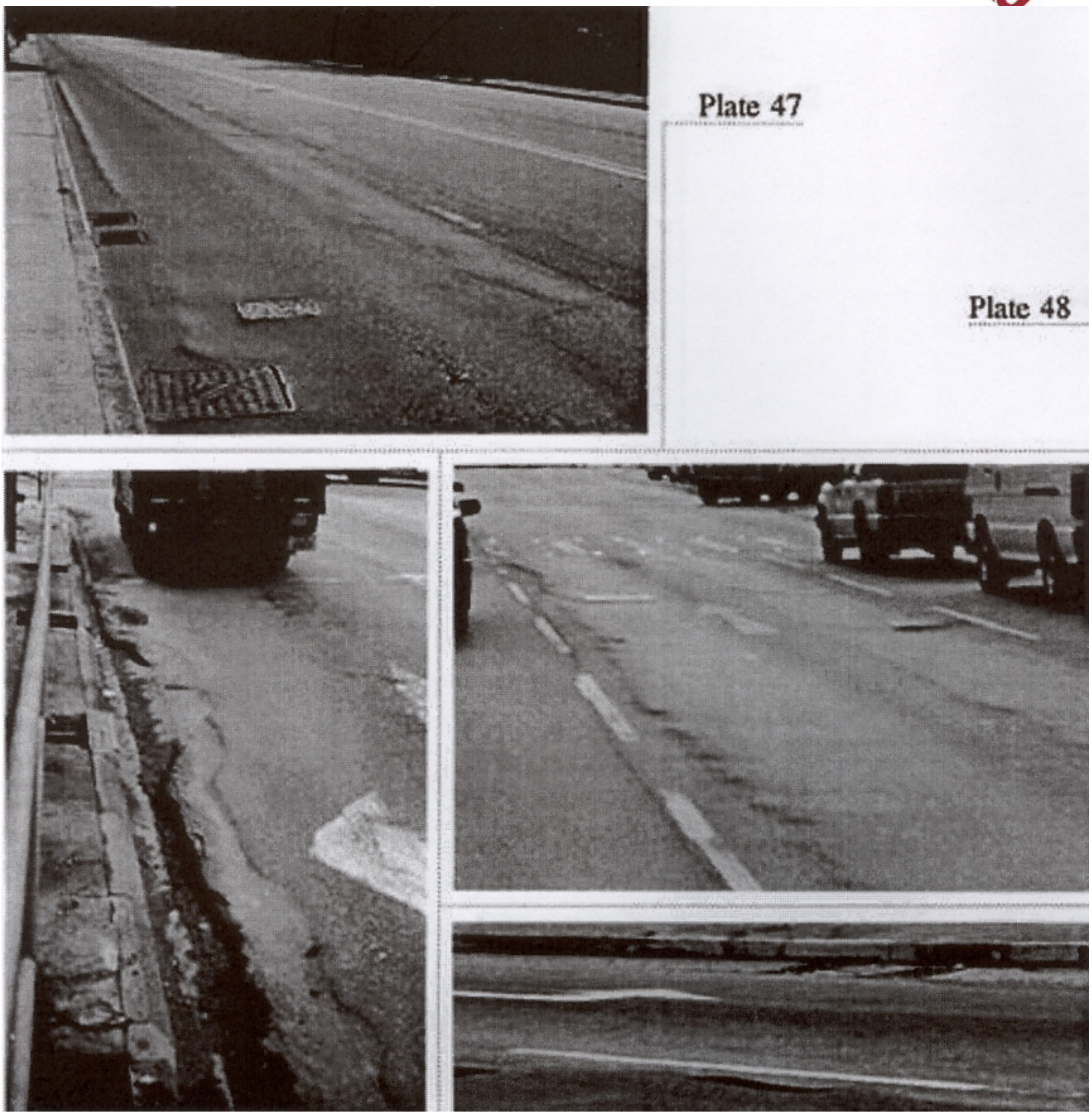
2.7 The Distress of Road Pavement

In most countries, particularly in developing countries, the number of private cars and trucks are increasing dramatically so traffic volume is increasing in most traffic roads, thus shortening the service life of the roads. Some of the roads have been severely damaged within the design service life. The distress of cracking, rutting and ageing are the common distress among those damages (Bolk, 1982) (Charania et al, 1991).

Repeated applications of traffic road cause structural damage to asphalt pavements in the form of fatigue cracking (see Picture 2.3) of asphalt bound layer and rutting along wheel tracks. While fatigue failure is the result of flexural cracking of asphalt bound layer (Harvey and Monismith, 1993). Rutting (see Picture 2.4) is the manifestation of permanent deformation in different layers of the pavement. The asphalt layer itself may display a significant amount of permanent deformation in hot climatic conditions. Asphalt pavements can also be damaged by climate factors such as temperature and moisture, so oxidative ageing (see Picture 2-3) of asphalt layers is another major cause for concern (Harvey and Tsai, 1997).



Picture 2-3 Cracking along the Road



Picture 2.4 Rutting along The Wheel Path

Picture 2.5 Ravelling due to Ageing and Moisture Effect

Rutting (permanent deformation) along the wheel tracks is one of the main pavement defects due to structural damage by repeating application of the traffic road, especially under the hot climatic conditions. Rutting is the manifestation of permanent deformation in different layers of the pavement (Palit et al, 2004). In order to improve the resistance of the permanent deformations, more and more scholars and pavement engineers have been placing emphasis on the development of the modified binder mixture. Amid those modified methods, use of waste vehicle tyre in pavement was one of the steps taken in this direction over the past several decades. However, the performances of mixtures modified by CRM are influenced by many factors, for instance, mixing process (wet or dry process), binder types, rubber contents and aggregate gradations. Furthermore, there is inadequate information or a insufficient guideline for achieving good CRM mixtures and the design in a place might be unsuitable for another. Under these circumstances, a comparative study of CRM on high temperature susceptibility of wearing course mixture is conducted in the hope of gaining basic information on using CRM in asphalt mixtures in Hong Kong.

2.8 Situations in Hong Kong

In Hong Kong, more than half of the highways are paved with asphalt. and in order to reduce the traffic noise, some of the roads in Hon g Kong were paved with special design like Porous asphalt; but this material have been reported to have a relative shorter service life. Besides, with the ever-increasing traffic Engineers and road expert in Hong Kong is try to introduce and develop a new mix with a better and durable performance. The main research effort, so far, has been in introducing proprietary modifiers, additives and fibers to asphalt cement.

Over 2 million wasted tyres are discarded in Hong Kong each year. The Government's latest budget plan stated that a sales tax on tyres would be introduced and effective in the middle of 2006. The money from the sales tax of tyres would be used as a fund for conducting research in waste tyre recycling.

In the year of 2002, the first pavement research laboratory in Hong Kong, **The Hong Kong Road Research laboratory (HKRRL)**, was set up. The researchers of the HKRRL recognized the urgency of starting research on the use of recycled rubber for pavement construction in Hong Kong. So this project is based on this concept which a two year research project on the feasibility of incorporating crumb rubber modifier in pavement was conducted. It examined the effect of crumb rubber modifier on the permanent deformation performance of the friction course and wearing course mixtures. The results of these preliminary study show that both the CRM modified asphalt cement and the CRM modified asphalt mixtures perform better than the conventional materials. Due to the encouraging results, the researchers suggest that CRM asphalt concrete design and specification should be developed in Hong Kong in the near future . The use of crumb rubber from waste tyres in road pavement has an eightfold benefit for Hong Kong:

1) Eliminating disposal problem of scrap tyre;
2) Releasing space of dumping site;
3) Preventing the health and safety hazard;
4) Beatifying the landscape and enhancing the image of the society;
5) Releasing valuable land;
6) Conserving natural materials (less asphalt cement and aggregate needed);

7) Enhancing the performance of asphalt concrete pavement (a more durable pavement);

8) Reducing cost of construction and maintenance.

CHAPTER 3

The Preparation of Raw Materials and Test Samples

3.1 Materials

The materials used in this study are aggregates, mineral filler, virgin binder, and crumb rubber modifiers. The aggregates were crushed granite and the aggregate grading included two dense-graded wearing courses, AC-10 and AC-20, and an open-graded porous asphalt (PA) (Figure 3-1). The aggregate grading is given in Tables 3-1, 3-2 and 3-3. The mineral filler used was Portland cement. The crumb rubber for modifying the binder was generated from recycled used vehicle tyre provided by a local plant. The virgin binder used in this study was 60/70 penetration grade bitumen provided by a local supplier. The properties of the aggregates, the CRM and the main technical specifications of the conventional pen-grade 60/70(PG60/70) bitumen are shown in Table 3-4.

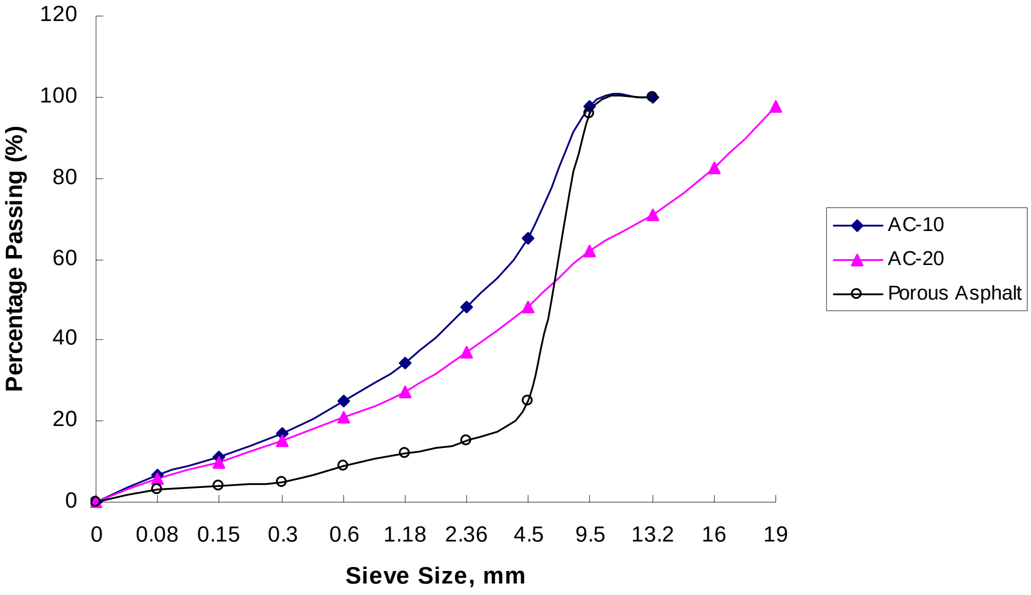


Figure. 3-1. Gradation of Aggregates for Friction Course

Wearing Course AC-10	
Sieve Sive (mm)	Percent Passing (%)
13.2	100
9.5	97.5
6.3	65
2.36	48
1.18	34.5
0.6	25
0.3	17
0.15	11
0.075	6.5

Table 3-1.The Target Aggregate Gradation of AC-10

Porous Asphalt (PA)	
Sieve Sive (mm)	Percent Passing (%)
26.5	100
19	97.5
16	82.1
13.2	71
9.5	62
6.3	48
2.36	37
1.18	27
0.6	21
0.3	15
0.15	10
0.075	6

Table 3-2. The Target Aggregate Gradation of AC-20

Porous Asphalt (PA)	
Sieve Sive (mm)	Percent Passing (%)
14	100
10	96
5	25
2.36	15
1.18	--
0.6	--
0.3	--
0.15	--
0.075	3.3

Table 3-3.The Target Aggregate Gradation of Porous Asphalt (PA)

Material	Parameter Measured	Value
Coarse aggregate	Apparent Specific Gravity	2.617
	Bulk Specific Gravity	2.566
Fine aggregate	Apparent Specific Gravity	2.605
PG60/70	Penetration (25ºC, 5s, 100g)	66.7
	Softening Point (Ring and Ball)	49.5
	Ductility (15ºC 50mm/min.)	>100
	Specific Gravity	1.029
Crumb Rubber	Specific Gravity	1.026

Table 3.4. Properties of Aggregates, Conventional PG60/70 and Crumb Ruber Modifier

3.2 Aggregate Properties

The bulk specific gravity and the apparent specific gravity were tested for the coarse aggregates; only the apparent specific gravity was tested for the fine aggregates. The test of specific gravity for the coarse aggregates was complied with ASTM C127-01 and the test of apparent specific gravity for the fine aggregates was complied with ASTM C128-01(ASTM C127-01 & ASTM C-128-01). The test results of bulk specific gravity and the apparent specific gravity of the coarse aggregates and the apparent specific gravity of the fine aggregates are given in Table3-5。

Sieve Size (mm)	Test No.	Specific Gravity (g/cm^3)			
		Apparent Specific Gravity（ASG）	Average ASG	Bulk Specific Gravity （BSG）	Average BSG
19	1	2.619	2.617	2.574	2.573
	2	2.616		2.572	
13.2	1	2.617	2.617	2.565	2.566
	2	2.617		2.567	
9.5	1	2.612	2.610	2.562	2.560
	2	2.608		2.559	
6.3	1	2.620	2.618	2.536	2.525
	2	2.615		2.513	
2.36	1	2.598	2.601	2.500	2.504
	2	2.619		2.518	
	3	2.587		2.495	
1.18	1	2.620	2.617	--	--
	2	2.613		--	
0.6	1	2.618	2.620	--	--
	2	2.623		--	
0.3	1	2.610	2.616	--	--
	2	2.622		--	
0.15	1	2.614	2.618	--	--
	2	2.621		--	
0.075	1	2.564	2.539	--	--
	2	2.513		--	

Table 3-5. Specific Gravity of Aggregates

3.3 The Mixing between CRM and Virgin Binder

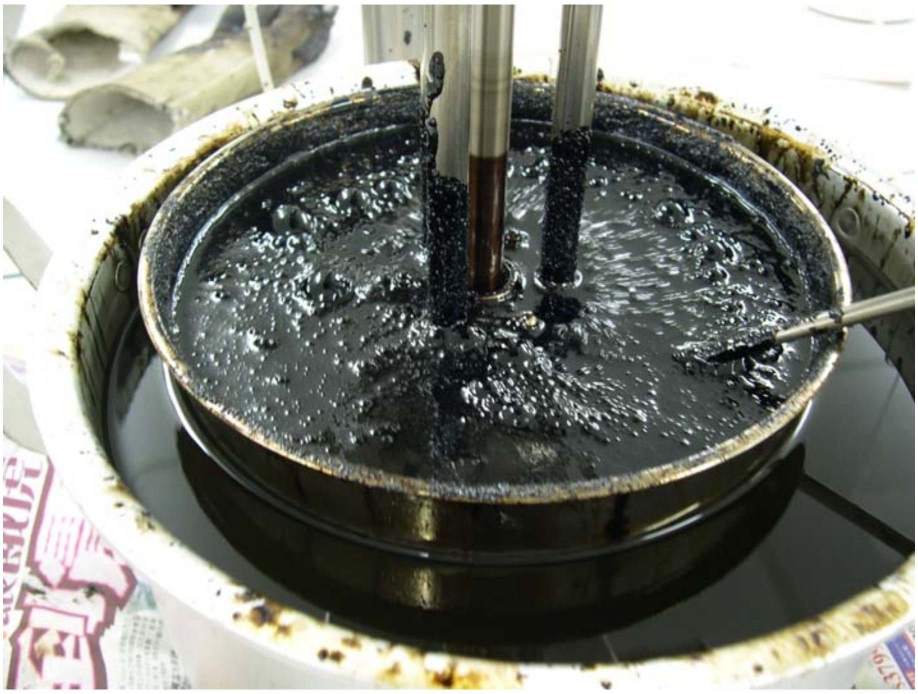
Currently, there are two kinds of mixing methods to produce polymer modified asphalt mixture in asphalt industry, namely, wet process and dry process. In this study, wet process, the widely used method, was used. The wet process defines any method that CRM to add the bitumen which is then well-blended before incorporating the modified binder into the mix. The mixing procedures are as follows:

Firstly, the PG60/70 bitumen and crumb rubber modifier (CRM) were heated to 150ºC separately and then the bitumen was further hit up to around 170ºC. In order to avoid ageing problem of the binder, a certain portion of CRM was added into the bitumen after the bitumen temperature reached 150ºC and the bitumen should be kept stirring by a rod while adding the CRM into it.

Secondly, the remaining CRM was added into the bitumen and mixing was done by using a high-speed laboratory mixer equipped with a heat mantle. The mixer consisted of a two-blade blender with controlled motor [see Picture 3-1]. At the beginning stage, the blend was stirred at a low speed for about five minutes. Then the blend was heated up to 175ºC to 185ºC and agitated vigorously for about 45 minutes to one hour by using a mechanical stirrer operated at 2000 rpm (Chehovits et al, 1993), compare with the conventional binder which as high temperature as 175ºC will cause ageing of the binder, but for CRM binder, the binder will not likely to age with the introduction of the CRM which can be easily seen from the subsequent performance test results.

Picture 3-1. The High-Speed Laboratory Mixer

Picture 3-2. Side-View of the High-Speed Laboratory Mixer



Picture 3-3. Temperature was Control in a Oil Tank During the Mix

3.4 The Preparation of Ageing Binders

According to the performance-based asphalt binder specification from Superpave, for the preparation of ageing binders, the rolling thin film oven test was used to achieve short-term ageing, and long-term ageing was achieved by additional ageing of the binder using the pressure ageing vessel (PAV) (SHRP-A-410).

3.4.1. Short-term Ageing

Short-term ageing, which simulates field ageing during the construction phase, is achieved using the rolling thin film oven test (RTFOT) ((Picture 3-4). In this test, a specified amount of asphalt binder is poured into a specially designed bottle. The bottle is placed in a rack that rotates around a horizontal axis. The rotating bottle causes fresh films of asphalt binder to be continuously exposed. Heated air is blown into the bottle to purge vapours from the bottle once each rotation. The test is conducted at 163ºC with a test time of 85 minutes (ASTM D2872). The primary reasons for the inclusion of the RTFOT in the Superpave system are that the temperature control is more precise, the RTFOT can accommodate more samples and the test time is significantly shorter than the traditional and more widely used than thin film oven test (TFOT).

Picture 3-4. The Rolling Thin Film Oven

3.4.2. Long-term ageing

Long-term ageing, which simulates field ageing during the first 5 to 10 years of service, is achieved by additional ageing of the binder using the pressure ageing vessel (PAV) (Harrigan et al. 1994). The asphalt binder from the RTFOT is placed in pans which are loaded into the PAV (Picture 3-5). The depth of the binder is 3.2mm±0.1mm (approximately 50g). The binder is then subjected to an air pressure of 2100±10kPa for 20 hours at a temperature of 100ºC.

Picture 3-5. The Pressure Air Verssel (PAV)

3.5 The Preparation of Mixtures

The mix designs are shown in Tables 3-6 and 3-7. And the materials used are shown in Table 3-8. The crumb rubber modified binder (wet product) was heated up to 165ºC and stirred gently to prevent segregation before being introduced and mixed with aggregates in the mixer.

AC-20	Passing	Percentage (%)	Percentage in Mix (%)	Dosage
26.5	100	0	0.0	0.0
19	97.5	2.5	2.4	73.2
16	82.5	15	14.3	439.4
13.2	71	11.5	11.0	336.9
9.5	62	9	8.6	263.6
6.3	48	14	13.4	410.1
2.36	37	11	10.5	322.2
1.18	27	10	9.6	292.9
0.6	21	6	5.7	175.8
0.3	15	6	5.7	175.8
0.15	10	5	4.8	146.5
0.075	6	4	3.8	117.2
<0.075	0	6	5.7	175.8
Binder content =4.5％， Mass of binder=138g Theoretical maximum specific density of mixture =2.417g/cm^3. Density of cement=3.0 g/cm^3， Density of binder= 1.029 g/cm^3, Air void=4.2%. The volume of each specimen is 1324.7cm^3 and total mass of each specimen is 3067.3g				

Table 3-6. The Gradation and the Design of Wearing Course AC-20

AC-10	Passing	Percentage (%)	Percentage in Mix (%)	Dosage
13.2	100	0	0.0	0.0
9.5	97.5	2.5	2.4	72.0
6.3	65	32.5	30.7	935.8
2.36	48	17	16.1	489.5
1.18	34.5	13.5	12.8	388.7
0.6	25	9.5	9.0	273.5
0.3	17	8	7.6	230.3
0.15	11	6	5.7	172.8
0.075	6.5	4.5	4.3	129.6
<0.075	0	6.5	6.1	187.2
Binder content =4.5％，Mass of binder=138g Theoretical maximum specific density of mixture =2.417g/cm^3. Density of cement=3.0 g/cm^3，Density of binder= 1.029 g/cm^3, Air void=4.2%. The volume of each specimen is 1324.7cm^3 and total mass of each specimen is 3067.3g				

Table 3-7. The Gradation and the Design of Wearing Course AC-10

Materials	Supplier	Type
Pen60/70 bitumen	Anderson Asphalt Limited	Shell 60/70
Coarse aggregates	Anderson Asphalt Limited	Granite
Fine aggregates	Anderson Asphalt Limited	Granite

Table 3-8 Sources of Constituent Materials

3.5.1 The Preparation of Aggregates

- Aggregates were washed over a 75μm sieve and dried to a constant weight at 105^{o}C±5^{o}C in an oven. The dried aggregates were separated into the desired coarse and fine fractions by dry sieving, using the same sieve sizes as those given in the grading specification.
- The aggregates for each test specimen were prepared and weighed in accordance with the mix design.

3.5.2 The Preparation of Binders

The crumb rubber modified binder (wet product) was heated up to 165ºC and stirred gently to prevent segregation before being introduced and mixed with aggregates in the mixer.

3.5.3 The Preparation of Cylindrical Samples

For prepare the specimens of the indirect tensile strength (ITS) test, indirect tensile stiffness modulus (ITSM) and dynamic creep test, compaction was performed using a Superpave gyratory compactor (see Picture 3-6). Compaction is the process of compressing a given volume of mix into a small volume. It is accomplished by pressing the asphalt rubber coated aggregate particles closer together by eliminating excessive air voids. During the compaction process, density and air void was achieved by controlling the volume of the specimen. As the diameter is fixed, so the height of the specimen is controlled during the compaction process.

The entyre mix design system, including field control, was based on the use of the Superpave gyratory compactor. The compactor is capable of quickly molding specimens with minimal specimen-to-specimen variation. The performance properties of the compacted specimen simulate the performance properties of cores from pavements constructed with the same asphalt aggregate combination. The compactor also allows the compatibility of the mix to be evaluated, including an estimate of the final air voids content under traffic (the probability of the mix becoming plastic under traffic) and a measure of the structuring of the aggregate in the mix.

The Superpave gyratory compaction is a method of laboratory compaction which is intended to simulate the in-situ compaction applied to asphaltic paving material. Materials were compacted in a cylindrical mould. The cylindrical samples are usually with cross section sizes of 150mm diameter and 75mm diameter and the height of the sample usually ranges from 30mm to 75mm. The gyratory compactor was used to prepare specimens for the current study. The size of the specimen used for preparing ITS test, is Ø100 X 60mm height, for preparing dynamic creep test, is Ø150 X 60mm height. Asphalt mixture specimens were compacted at a constant pressure of 0.6MPa.

The Superpave gyratory compactor has the following characteristics:

- An angle of gyration of 1.25±0.02^{o}C;
- A rate of 30 gyrations per minute;
- A vertical pressure during gyration of 600 kPa; and
- The capacity of producing 150×150 mm specimens.

Picture 3-6. The Gyratory Compactor

3.5.3.1 Mixing of Binder and Aggregates for Slab Samples

- The aggregate blend was placed in an oven and heated to a temperature of 180^{o}C which was approximately 28^{o}C above the mixing temperature in accordance with MS-2 of Asphalt Institute for 8 hours.
- The binder was preheated in an oven, for Shell 60/70 bitumen, the binder was preheated in an oven to the mixing temperature 135 ^{o}C; for crumb rubber modified binders, the binder was preheated in an oven to 155 ^{o}C.
- The cylindrical mould with two covers and a compaction pedestal were preheated to a temperature of 100^{o}C-135^{o}C.
- The mass of the binder to be added to each aggregate blend was calculated to the nearest 0.1g.
- The aggregate blend was transferred from the oven to the mixer bowl.
- The binder was stirred in its container and the required amount of binder was weighed into the mix. The aggregates and binder were rapidly mixed with a mechanical mixer until the aggregates were coated uniformly with the binder.
- A cylindrical mould (150mm or 100mm in diameter) was taken from the oven. A plain paper was placed in the bottom of the mould. All the mixed materials were transferred into the mould. The top of the mixture was evened with a heated spatula, another plan paper and the upper cover were placed on the top of the

mixed materials and the mould should be put in the gyratory compactor immediately.

3.5.3.2 Compaction

For preparating the specimens of the indirect tensile strength (ITS) test, indirect tensile stiffness modulus (ITSM) and dynamic creep test, compaction was performed using a Superpave gyratory compactor with volume control during the compaction process.

- The computer control unit of the gyratory compactor (the compaction sequence was preset by the gyratory compactor manufacturer and complied with the SHRP) was being set before the cylindrical mould putting on. In this study, the volume of the compacted materials was achieved by controlling the high of specimen.
- The cone-shaped mould was removed from the compactor and put on the extract unit. When the specimen was extracted, the paper was removed from the top of the specimen while the specimen was still warm to avoid excessive sticking.

3.5.4 The Preparation of Slab Sample

The slab samples were prepared for the wheel tracking tests, the sizes are standardizing in 305mm×305mm×50mm. The following procedures were adopted for the preparation of the wearing course specimens.

Preparation of aggregates

- Aggregates were washed over a 75μm sieve and dried to a constant weight at 105^{o}C±5^{o}C in an oven. The dried aggregates were separated into the desired coarse and fine fractions by dry sieving, using the same sieve sizes as those given in the grading specification.
- The aggregates for each test specimen were prepared and weighed in accordance with the mix design.

Mixing of binder and aggregates for slab sample

- The aggregate blend was placed in an oven and heated to a temperature of 180^{o}C which was approximately 28^{o}C above the mixing temperature in accordance with MS-2 of Asphalt Institute for 8 hours.
- The binder was preheated in an oven, for Shell 60/70 bitumen, the binder was preheated in an oven to the mixing temperature 135 ^{o}C; for crumb rubber modified binders, the binder was preheated in an oven to 155 ^{o}C.

- Moulds, extension collars, roller head and compaction pedestal were preheated to a temperature of 100°C-135°C.
- The mass of the binder to be added to each aggregate blend was calculated to the nearest 0.1g.
- The aggregate blend was transferred from the oven to the mixer bowl.
- The binder was stirred in its container and the required amount of binder was weighed into the mix. The aggregates and binder were rapidly mixed with a mechanical mixer until the aggregates were coated uniformly with binder.
- A mould assembly was taken from the oven. A plain paper was placed in the bottom of the mould. All the mixed materials were transferred into the mould. The top of the mixture was evened with a heated spatula.

Compaction

For specimens of the wheel tracking test, compaction was performed using a roller compactor (see Picture 3-7).

- When the temperature of the bituminous mixture lied at the compaction temperature ±2°C, another plain paper was placed on top of the material. The mould assembly was transferred to the roller compactor and located in the mould holder

- The mixture was compacted with the roller compactor (the compaction sequence was preset by the wheel roller manufacturer).
- The mould was removed from the compactor. The extension collar was removed. The paper was removed from the top of the specimen while the specimen was still warm to avoid excessive sticking.

Picture 3-7 Roller Compactor

CHAPTER 4

TESTING ON CRM MODIFIED BINDERS

Nowadays, wet process are commonly used in most CRM projects as the performance of the mixtures produced by wet process are more excellently and consistent as compared with dry process. So researchers have been putting more emphasis on the performance of crumb rubber modified binders. Also, as previously mentioned, the size of crumb rubber modifiers might influence the performance of mixtures as different particle sizes, which have different surface area, can have different interaction with virgin binder. Generally, fine particles provide stiffer binder with more elastic component than that of the coarse material. However, for economic reasons, coarse materials are preferable to be used on the premise that it has satisfactory performance. Under these circumstances, wet process and 10% CRMs with three CRMs of sizes 0.15mm, 0.30mm and 0.60mm were used in this study. The penetration test, softening point test, viscosity test and dynamic shear rheometer (DSR) test were used to study the properties of CRM binders.

4.1 Materials Used

The aggregates used in this study were crushed granite. The aggregate grading included two dense-graded wearing courses, AC-10 and AC-20, and an open-graded porous asphalt (PA) (see Figure 3-1). The crumb rubber used for modifying the binder was generated from recycled used vehicle tyres provided by a local plant. The properties of the aggregates, the CRM and the main technical specifications of the conventional pen-grade 60/70 (PG60/70) bitumen are shown in Table 3-4.

4.2 The Idea of SHRP Research Program

The Strategic Highway Research Program (SHRP) Asphalt Research Program was developed in the early 1980s because of an increasing number of premature asphalt pavement failures. In recognition of this problem, the U.S initiated the development of a coordinated, well-funded, national research effort to develop improved specifications for asphalt and, ultimately, asphalt mixtures. This project consisted of two tasks: to develop an asphalt specification and to develop an asphalt-aggregate mixture analysis system.

4.2.1 The Performance-Based Asphalt Binder Specification

The performance-based asphalt binder specification was a principal product of the SHRP asphalt research program. A major objective of the asphalt research program was to identify and validate engineering properties that could be directly linked to the performance of asphalt binders. Superpave (Superior Performing Asphalt Pavements) was the final product of the Strategic Highway Research Program (SHRP) Asphalt program, which

- Superpave is a comprehensive system for the design of paving mixes that are tailored to the unique performance requirements dictated by the traffic, environment (climate), and structural section at a pavement site. It enhances pavement performance through the selection and combination of the most suitable asphalt binder, aggregate, and, where necessary, modifier from all the possible choices;
- Superpave represents the integration of more than twenty-five products of the SHRP asphalt research program into a single system for the design and analysis of paving mixes. It encompasses new material specifications, test methods and equipment, a mixture design method, and a software system in a comprehensive, coordinated package;
- Superpave was devised to replace the diverse materials specifications and mixture design methods used by the fifty states with a single system that can provide results tailored to the distinct environment and traffic conditions found at any given pavement location in the United States and Canada;
- Superpave is applicable to virgin and recycled, dense-graded, hot-mix asphalt, with or without modification, for use in new construction and overlays.

It was developed to address and minimize permanent deformation, fatigue cracking, low temperature cracking, and it considers how the effects of ageing and moisture damage contribute to the development of three distresses.

The asphalt binder specification requires tests on unaged, short-term aged and long-term aged asphalt binder. The specification is based on pavement temperature and performance-based engineering properties. Instead of changing engineering property requirements for different grades, the specification calls for the same engineering properties for all grades but specifies different temperatures at which the properties must be met. The performance-related requirements are as follows:

- A minimum stiffness (1.0 kPa) is specified on the unaged binder to guard against mixture tenderness.
- A minimum stiffness (2.2 kPa) is specified on the short-term aged binder to ensure adequate permanent deformation resistance immediately after construction.
- A maximum stiffness (5,000 kPa) is specified on long-term aged material to guard against fatigue cracking caused by excessively stiff asphalt binders.
- A maximum stiffness (300 MPa) and minimum slope (0.30) of the creep deformation are specified to limited excessive stiffness at low temperature.
- A higher creep stiffness is allowed by the specification if a minimum tensile strain (1 percent) at failure is achieved. (It is anticipated that this will apply primary for modified binders which have high stiffness at low temperature, but exhibit large failure strains).

4.3 Testing Methods on Binders

The testing methods on binders were mainly complied with the performance-based binder specification from Superpave, They consisted of the penetration test, softening point test, viscosity test and dynamic shear rheometer (DSR) test. In order to evaluate the effect of different sizes of crumb rubber modifiers on the performance of conventional binder, three CRMs of sizes 0.15mm, 0.30mm and 0.60mm are used in this study.

The condition of asphalt binder changes with temperature, time and loading. Four testing methods, penetration test, softening point test, viscosity test and dynamic shear rheometer (DSR) test, were performed on each condition of the four types of binders. The whole test procedure is summarized as shown in Figure 4-1.

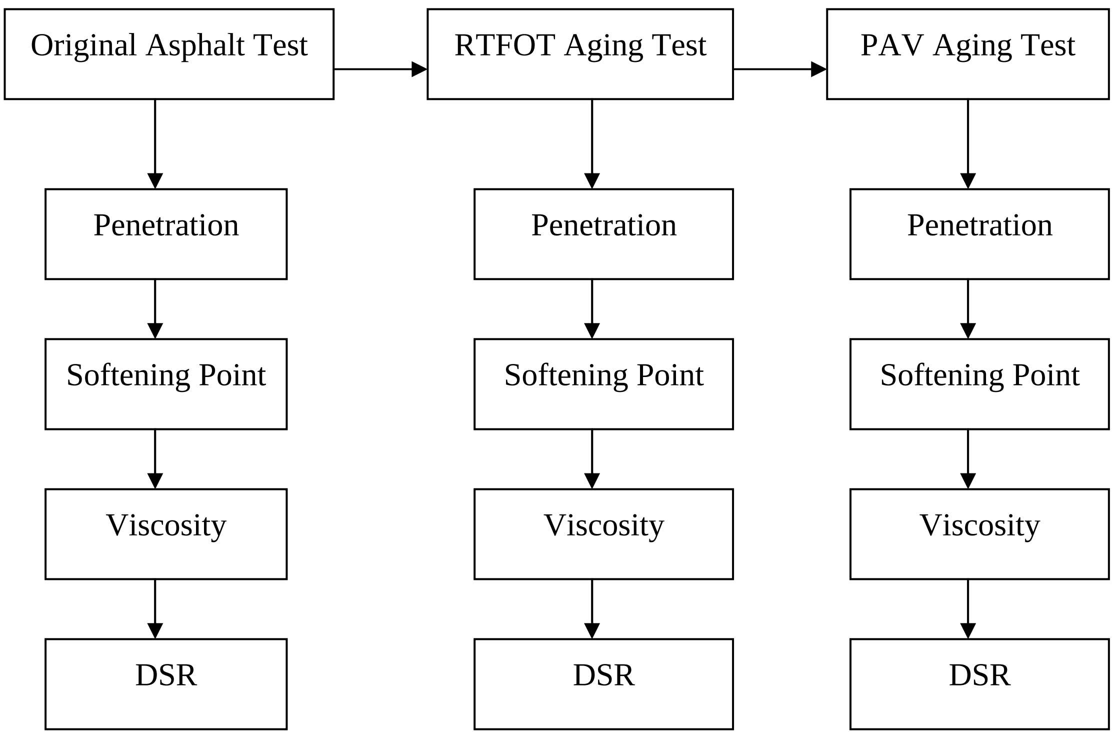


Figure 4-1 Test Procedure on the Four Types of Asphalt Binders

The pivotal technological points of each test are briefly described as follows:

4.3.1 RTFO Ageing Test

Short-term ageing, which simulates construction ageing, is achieved using the rolling thin film oven test (ASTM D2872-97). A specified amount of asphalt binder is poured into a specially designed bottle. The bottle is placed in a rack that rotates around a horizontal axis. The rotating bottle causes fresh films of asphalt binder to be continuously exposed. Heated air is blown into the bottle to purge vapors from the bottle once each rotation. The test is conducted at 163°C with a test time of 85minutes.

4.3.2 PAV Ageing Testing

Long-term ageing, which simulates field ageing during the first 5 to 10 years of service life, is achieved by additional ageing of the binder using the pressure ageing vessel (PAV, SHRP A-367). The asphalt binder from the RTFOT is placed in pans which are loaded into the PAV. The depth of the binder is 3.2 mm ± 0.1mm (approximately 50 g). The binder is then subjected to an air pressure of 2100 ± 100kPa for 20 hours at a temperature of 100°C.

4.3.3 Penetration Test

The penetration test is used as a measure of consistency of bituminous material. Higher values of penetration indicate softer consistency. The penetration is measured with a penetrometer by means of which a standard needle is vertically applied to the sample under known conditions of loading, time and temperature. In this project the test conditions of temperature, load and time were 25°C, 100g and 5 second respectively.

4.3.4 Softening Point Test

This test method covers the determination of softening point of asphalt binders in the range from 30 to 157°C. The softening point is a conventional index in the classification of asphalt binders and is indicative of the tendency of material to flow at elevated temperatures encountered in service.

4.3.5 Rotational Viscosity Test

The rotational viscometer (Figure 4-2) described in ASTM D4402, "Measurement of Asphalt Viscosity Using a Rotational Viscometer," measures the high temperature viscosity of asphalt binders. The test is conducted by rotating a spindle immersed in the asphalt binder at a specified temperature. To ensure pumpability, the specification stipulates that the binder must have a maximum viscosity of 3Pa.s at a test temperature of 135°C.

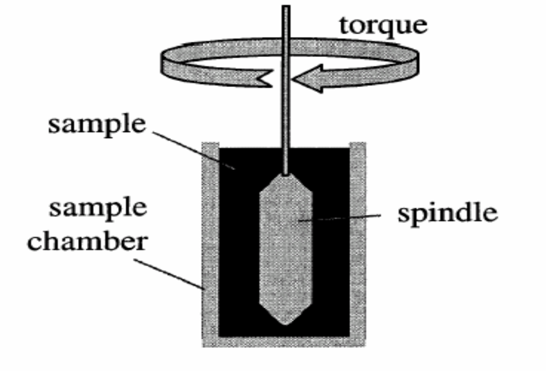


Figure 4-2 Principle of Operation of the Rotational Viscometer

4.3.6 Dynamic Shear Rheometer (DSR)

The dynamic shear rheometer is used to determine complex shear modulus ,G^* and phase angle δ, by measuring the shear strain response of a specimen to a fixed torque as shown in Figure 4-3. A 1 to 2mm thick sample of asphalt is placed between two parallel circular plates (8 or 25 mm in diameter). The actual thickness depends on the stiffness of the binder. The bottom plate is fixed and the top plate is oscillated by a computer-controlled electronic motor. For specification purposes, the frequency is 10 radians per second which has been related to a traffic speed of 100 km/hr. The temperature of the sample must be within + 0.1 °C of the specified test temperature. Rutting factor, $G^*/\sin\delta$, and fatigue factor, $G^*\sin\delta$, were measured.

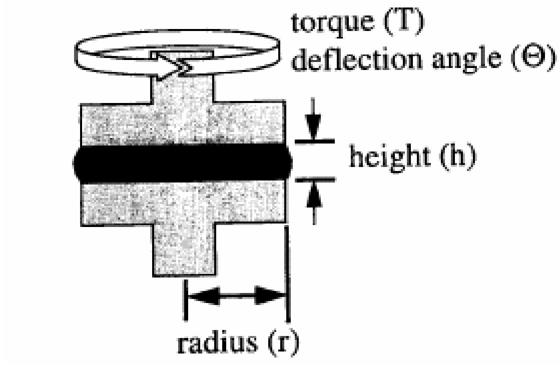


Figure 4-3 Principle of Operation of the Dynamic Shear Rheometer

The stiffness value, $G^*/\sin\delta$, of the binder after the RTFOT must be greater than 2.2 kPa at the maximum 7-day average pavement design temperature. To control possible tenderness, if ageing does not occur during construction, the stiffness value $G^*/\sin\delta$, of the original unaged asphalt must be greater than 1 kPa at the same pavement temperature. This parameter is correlated to that portion of the accumulated, non-recoverable deformation occurring in a pavement that is attributable to the asphalt binder. Comparison of the value of $G^*/\sin\delta$for asphalt binders to this specification limit will indicate how well they will perform with respect to permanent deformation.

The stiffness value of the binder after PAV ageing must be less than 5,000 kPa at the approximate average (termed "intermediate") pavement temperature. This parameter is related to the contribution of the asphalt binder to the dissipation of energy in a pavement during each loading cycle. Comparison of the measured value of $G^*\sin\delta$ for asphalt binders with this specification limit will indicate how well they will perform with respect to fatigue cracking.

4.4 Testing Results and Analysis

A summary of data and results of analysis of the four asphalt binders (including the original grade 60/70 binder) are presented in the following sections.

4.4.1 The Result of Penetration Test

The penetration test is conducted by penetrometer and is defined in 0.1mm that a standard needle penetrates into the bitumen under a load of 100g applied for 5 seconds at 4°C and 25°C, these two temperatures are recommended by the standard of ASTM. So in this study, 4°C is also used for understanding the low-temperature behaviour of a modified binder except for the evaluation of the high temperature susceptibility which this study focused on.

The penetration test results of the four asphalt binders (shell 60/70, 0.6mm CRM, 0.3mm CRM and 0.15mm CRM) at 25°C are shown in Table 4-2 and Table 4-3. The penetration test results of the four asphalt binders at 4°C are shown in Table 4-4. Also, the penetration ratio (%) between the virgin binder and the RTFOT, 4°C and 25°C, are presented in the Table 4-5.

	Shell 60/70	0.60mm CRM	0.30mm CRM	0.15mm CRM
Data 1	66.00	46.50	45.50	43.00
Data 2	68.00	47.00	42.00	41.00
Data 3	64.50	44.50	41.50	40.50
Data 4	68.50	47.00	44.00	43.00
Data 5	66.00	45.00	42.00	42.00
Data 6	67.00	48.00	44.00	39.00
Average	**66.67**	**46.33**	**43.17**	**41.42**
This is done by penetrometer and is defined in 0.1mm that a standard needle will penetrate into the bitumen under a load of 100g applied for 5 seconds at 25°C				

Table 4-2. Penetration Test On The Binder Before Ageing At 25°

	Shell 60/70	0.60mm CRM	0.30mm CRM	0.15mm CRM
Data 1	38.00	27.50	28.00	28.50
Data 2	34.00	29.00	31.00	30.50
Data 3	37.00	27.00	30.00	26.00
Data 4	36.00	29.00	28.50	29.00
Data 5	34.00	28.00	32.00	28.00
Data 6	34.00	30.50	28.00	30.00
Average	**35.50**	**28.50**	**29.58**	**28.67**
This is done by penetrometer and is defined in 0.1mm that a standard needle will penetrate into the bitumen under a load of 100g applied for 5 seconds at 25°C				

Table 4-3. Penetration Test On The Binder After RTFO At 25°C

	Shell 60/70	0.60mm CRM	0.30mm CRM	0.15mm CRM
Data 1	12.00	13.50	16.00	11.00
Data 2	11.00	12.00	14.00	12.30
Data 3	12.00	12.50	13.00	11.00
Data 4	9.50	14.00	13.50	13.50
Data 5	11.50	13.50	15.50	13.00
Data 6	9.00	12.50	12.00	11.00
Average	**11**	**13**	**14**	**12**
This is done by penetrometer and is defined in 0.1mm that a standard needle will penetrate into the bitumen under a load of 100g applied for 5 seconds at 4°C				

Table 4-4. Penetration Test On The Binder Before Ageing At 4°C

Penetration Ratio		
	At 25°C	**Before Ageing**
	RTFOT(25°C) / Before RTFOT	**4°C / 25°C**
Shell 60/70	53%	16%
0.60mm CRM	62%	30%
0.30mm CRM	69%	35%
0.15mm CRM	69%	36%

Table 4-5. Penetration Ratio of Ageing and Temperature Conditions

Note:

Shell 60/70 – denotes the conventional 60/70 penetration grade bitumen

0.60mm CRM – denotes the binder modified with 10%, 0.6mm crumb rubber modifier

0.30mm CRM – denotes the binder modified with 10%, 0.3mm crumb rubber modifier

0.15mm CRM –denotes the binder modified with 10%, 0.15mm crumb rubber modifier

From the Penetration test data above, the conclusions can be drawn as follows:

1) As shown in the Table 4-2, penetration values of all the four binders at aged condition are small than at unaged condition. It indicates that binders tend to harden with binder ageing going on.
2) As shown in Table 4-2, penetration values of crumb rubber modified binders are small than original 60/70 binder, but there is no big difference among those CRM binders, it indicates that crumb rubber modified binders can provide better resistance against deformation than the conventional binder in the high temperature areas.
3) From Table 4-4, penetrations of the crumb rubber modified binders at 4°**C** were found to be much higher than that of the original 60/70 binder, whereas at 25°C, original 60/70 binder had higher penetration value. The crumb rubber modified binders, therefore, are expected to maintain flexibility at lower temperatures without being soft at higher temperatures. Also, according to Table 4-5, among those three crumb rubber modified binders, penetration ratio (4°**C**/25°**C)** of 0.60mm CRM is less

than those of 0.30mm and 0.15mm CRM, but the differences in the penetration ratio (4°C/25°C) of 0.30mm CRM binder and 0.15mm CRM binder are not very significant.

4.4.2 The Result of Softening Point Test

The softening point test results of the four asphalt binders (including the original grade 60/70 binder) are in Table 4-6. The unit is **°C.**

Test	**Data**	**Shell 60-70**	**0.6mm CRM**	**0.3mm CRM**	**0.15mm CRM**
1	Data 1	49.50	55.30	52.80	55.10
	Data 2	49.10	56.30	53.60	55.30
2	Data 1	49.70	53.10	53.40	55.20
	Data 2	49.50	55.00	53.90	55.30
	Average	**49.45**	**54.93**	**53.43**	**55.23**
The softening point is reported as the mean of the temperature at which the soften enough to allow each ball, enveloped in bitumen, to fall a distance of 25mm.					

Table 4-6. Ring & Ball Test On The Binder Before Ageing

Test	Data	Shell 60-70	0.6mm CRM	0.3mm CRM	0.15mm CRM
1	Data 1	53.80	63.90	61.80	59.10
	Data 2	54.70	64.00	61.90	58.90
2	Data 1	53.90	62.80	62.40	59.50
	Data 2	54.20	64.10	62.60	54.30
	Average	**54.15**	**63.70**	**62.18**	**57.95**

The softening point is reported as the mean of the temperature at which the soften enough to allow each ball, enveloped in bitumen, to fall a distance of 25mm.

Table 4-7. Ring & Ball Test On The Binder After RTFOT

Test	Data	Shell 60-70	0.6mm CRM	0.3mm CRM	0.15mm CRM
1	Data 1	60.90	65.00	67.80	69.80
	Data 2	60.00	65.70	67.90	69.30
2	Data 1	61.00	65.00	67.30	69.50
	Data 2	60.00	65.30	67.60	70.00
	Average	**60.48**	**65.25**	**67.65**	**69.65**

The softening point is reported as the mean of the temperature at which the soften enough to allow each ball, enveloped in bitumen, to fall a distance of 25mm.

Table 4-8. Ring & Ball Test On The Binder After PAV

	Increased Percentage of Softening Point after Ageing	
	(RTFOT-Unageing)/Unageing	(PAV-Unageing)/Unageing
Shell 60/70	9.5%	22%
0.60mm CRM	16%	19%
0.30mm CRM	16%	27%
0.15mm CRM	5%	21%

Table 4-9. Softening Point Ratio Analysis Results

From the data above, the conclusions can be drawn as follows:

1) As shown in the Table 4-2, the softening point values of all the four binders at unaged are smaller than those at aged conditions, it indicates that binders tend to harden with binder ageing going on.

2) At aged condition three crumb rubber modified binders have much higher softening point than the conventional shell 60/70 binder. The softening point of 0.15mm CRM binder is the highest among the four binders. It indicates that the temperature susceptibility of those three crumb rubber modified binders is less than that of the conventional shell 60/70 binder.

3) Besides, increased percentages of softening point with binder ageing are shown in Table 4-9. Results show that 0.15mm CRM binder exhibits the lowest increased percentage among the four studied binders after short-term ageing, also, there is no

difference between 0.30mm CRM and 0.60mm, as for the long-term ageing, 0.60mm CRM exhibit the lowest increased percentage among the four studied binders. The rank is followed by 0.15mm CRM, 0.30mm CRM and the conventional 60/70 binder. In a word, crumb rubber modified binders will have a better ability of ageing resistance.

Statistical analysis of two way variance (ANOVA) and means comparison

Statistical analysis of two way variance (ANOVA) (α=0.05) and means comparison (Douglas, 1997) (Minitab User's Guide 2, 2002) was conducted with the softening point data. In the ANOVA, the softening point is the response variable and the binder types and ageing conditions are the influence Factor A and Factor B respectively.

The test results analysis by two way variance (ANOVA) (α=0.05) are shown in Table 4-10. As measured by the P-values, it is observed that the P values of the influence Factors A and B are equal to 0.000 which is less than the assumed value 0.05. Also, the P value (7.065E-12) of interaction of Factor A and Factor B is less than the assumed value 0.05. Hence, the statistical analysis indicates that, from the results of the ring and ball test, binder types (Factor A) and ageing conditions (Factor B) and their interaction have a significant effect on the softening point of binders.

Analysis results by means comparison using the Tukey Test are shown in Tables 4-11 and 4-12. The Tukey test was used in the comparison of means with an α-level set at 0.05.

The null hypothesis is that the mean of the softening point between the CRM modified binder (aged or unaged) and PG60/70 (age or unaged) are equal and the alternative hypothesis is that the means are different. The results are shown in Table 4-11 and 4-12 respectively.

Source	Degree of Freedom	Sum of Squares	Mean Square	F Value	P Value
Factor A	3	370.986	123.662	159.236	0.000
Factor B	2	1250.002	625.001	804.794	0.000
Factor A*B(1)	6	124.585	20.764	26.737	0.000
Error	36	27.958	0.777		
Total	47	1773.530			

Note (1): Factor A*B means the interaction of Factor A and Factor B

Table 4-10. Effect of CRM Binders on Softening Point analysis by Two-WayANOVA

Level	Mean (ºC)		Simultaneous Confidence Intervals		
0.15mm CRM	60.942		Lower Limit	Upper Limit	
0.30mm CRM	61.083	-0.142	-1.111	0.827	No
0.60mm CRM	61.292	-0.350	-1.319	0.619	No
Shell 60/70	54.692	6.250	5.281	7.219	Yes
0.30mm CRM	61.083				
0.60mm CRM	61.292	-0.208	-1.177	0.761	No
Shell 60/70	54.692	6.392	5.423	7.361	Yes
0.60mm CRM	61.292				
Shell 60/70	54.692	6.600	5.631	7.569	Yes

Table 4-11. Mean Comparison of Softening Points of CRM Binders with Factor A by using the Tukey Test

Level	Mean (ºC)		Simultaneous Confidence Intervals		
OB	53.256		Lower Limit	Upper Limit	
PAV	65.756	-12.500	-13.262	-11.738	Yes
RTFOT	59.494	-6.238	-6.999	-5.476	Yes
PAV	65.756				
RTFOT	59.494	6.263	5.501	7.024	Yes

Table 4-12. Mean Comparation on Softening Points of CRM Binders with Factor B by using Tukey Test

According to the analysis data from Table 4-11, there is only a slight difference between the means of softening points of the three CRM binders. However, the difference between the softening points of the CRM modified binders and PG60/70 is significant. The analysis data from Table 4-12 indicates that mean difference between ageing conditions (OB, RTFOT and PAV) are significant.

Conclusions can be drawn as follows: A certain level of modified effect was obtained by CRM modified binders, but only a slight effect was seen among these three CRM modified binders. It shows that binders prepared with three different sizes (0.15mm, 0.30mm and 0.60mm) under the same mixing process (wet process) with the same CRM content (10%) all show better performance for resistance of high temperature susceptibility than the unmodified binder.

4.4.3 The Result of Viscosity Test

Measurement of the high-temperature properties of bitumen gives an indication of its ease of handling at a coating plant. The SHRP used a rotational viscometer to measure the viscosity of bitumen at elevated temperatures. ASTM D4402 describes the use of a Brookfield viscometer. Under SHRP, the determination of viscosity is carried out using a Brookfield viscometer and thermoses at 135°C. In order to make sure the workability during the construction phase, the maximum allowable viscosity is 3Pas. In practice, it is desirable to measure the viscosity over a range of temperatures and shear rates so that an indication of binder behaviour during mixing and compaction can be obtained.

The viscosity test results of the four asphalt binders (including the original grade 60/70 binder) are in Table 4-13.

	Binder Condition (135°C)		
	Unageing (10^{-3}Pa s)	**RTFOT (10^{-3}Pa s)**	**PAV (10^{-3}Pa s)**
Shell60/70	475	614.2	445
0.60mm CRM	1553	2251	602
0.30mm CRM	1417	2438	533
0.15mm CRM	1256	2112	426
The apparent viscosity (the ratio between the applied shear stress and the rate of shear) is carried out using a Brookfield viscometer and thermoses at 135°C. This is a measure of the resistance to flow of the liquid and the SI unit of viscosity is Pa.s			

Table 4-13 Viscosity Testing Results

Major findings are presented as follows:

1) Table 4-13 shows that at unaged condition the viscosities of 0.60mm and 0.30mm CRM binders are the highest among the four studied binders and are about three times higher than that of the conventional shell 60/70 binder. All of the three crumb rubber modified binders have lower temperature susceptibility than the conventional shell 60/70 binder.

2) According to the requirement of SHRP performance-based binder specification,

SHRP A410, in order to warrant binders to have enough workability during construction phase, viscosity value of any binder at 135°C should be less than 3Pa.s. This requirement is satisfied by the four studied binders.

3) In addition, the ageing degree will incur the change of binder viscosity value, Viscosity values of the three studied binders at RTFOT ageing condition are higher than those at unageing condition; however contrary to PAV condition. The main reason for this is that RTFOT ageing causes binder hardening with no change in component ingredients, but PAV ageing changes binder ingredients and reduces binder viscosity sharply.

4.4.4 The Result of Dynamic Shear Rheometer (DSR) Test

The DSR testing results of the four studied binders are shown in Table 4-14, Table 4-15, Table 4-16 and Table 4-17, respectively. Also, to illustrate the temperature effect on rutting factor, $G^*/\sin\delta$, and fatigue factor, $G^* \sin\delta$, the testing result are plotted in the Figure 4-4, Figure 4-5, and Figure 4-6 respectively. Major findings are described as follows:

1) The critical high temperature of binder is defined as the temperature, at which the stiffness value, $G^*/\sin\delta$, of the binder just exceed 1kPa and 2.2kPa on the original condition and RTFOT condition respectively. The critical high temperatures of the four studied binders are 70°C for conventional shell 60/70 binder, 76°C for the three

crumb rubber modified binders respectively. The values of the three crumb rubber modified binders are comparable and are much higher than that of conventional shell 60/70 binder. It means that the modified binders have the higher resistance to permanent deformation under the condition of high pavement temperature when compared with the conventional shell 60/70 binder.

	Binder Condition			
	Before Ageing			
Shell 60/70	Temperature (°C)	Complex Modulus (G*) (kPa)	Phase Angle(δ) (Degree)	G*/sinδ (kPa)
	52.1	15.621	84.2	15.70116
	57.8	7.0618	85.9	7.07957
	63.7	3.321	87.2	3.324946
	69.8	1.9454	87.8	1.946815
	75.7	1.1648	88.4	1.165259
	81.6	0.53628	88.8	0.536402
	RTFOT ageing			
	Temperature (°C)	Complex Modulus (G*) (kPa)	Phase Angle(δ) (Degree)	G*/sinδ (kPa)
	52	36.762	78.9	37.4656
	58.1	11.45	82.7	11.54341
	63.8	5.4676	84.7	5.491135
	69.8	2.4197	86.3	2.424716
	75.8	1.2209	87.5	1.222114
	PAV ageing			
	Temperature (°C)	Complex Modulus (G*) (MPa)	Phase Angle(δ) (Degree)	G*sinδ (kPa)
	7.2	32.851	32.8	17817.33
	10	26.728	34.5	15122.99
	12.8	21.637	36.2	12767.46
	15.8	15.183	39	9564.517
	18.8	11.348	41.4	7510.266
	21.8	8.1082	44.2	5647.788
	24.8	5.6364	46.7	4103.639

Table 4-14 DSR Testing Results of Shell 60/70 Binder

	Binder Condition			
	Before Ageing			
0.60mm CRM	Temperature (°C)	Complex Modulus (G*) (kPa)	Phase Angle(δ) (Degree)	G*/sinδ (kPa)
	64.3	3.931	82.4	3.966
	69.6	2.072	83.8	2.084
	75.7	1.045	84.7	1.050
	81.9	0.541	84.9	0.543
	RTFOT Ageing			
	Temperature (°C)	Complex Modulus (G*) (kPa)	Phase Angle(δ) (Degree)	G*/sinδ (kPa)
	64.3	10.104	74.8	10.471
	69.6	5.274	76.0	5.436
	75.7	3.045	76.8	3.128
	81.9	1.711	77.3	1.754
	PAV Ageing			
	Temperature (°C)	Complex Modulus (G*) (MPa)	Phase Angle(δ) (Degree)	G*/sinδ (kPa)
	21.3	6.630	43.7	4584.59
	18.2	9.449	41.17	6220.44

Note: 0.60mm CRM –denotes the binder modified with 10%, 0.60mm crumb rubber modifier

Table 4-15 DSR Test Results of 0.60mm CRM Binder

	Binder Condition			
	Before Ageing			
0.30mm CRM	Temperature (°C)	Complex Modulus (G*) (kPa)	Phase Angle(δ) (Dgree)	G*/sinδ (kPa)
	64.3	4.056	78.6	4.138
	69.7	2.240	80.4	2.272
	75.6	1.187	82.3	1.198
	81.8	0.651	83.8	0.655
	RTFOT Ageing			
	Temperature (°C)	Complex Modulus (G*) (kPa)	Phase Angle(δ) (Dgree)	G*/sinδ (kPa)
	46.0	67.583	65.6843	74.162
	51.7	35.130	67.0014	38.164
	57.6	16.974	68.6702	18.222
	63.6	8.621	70.6623	9.137
	69.6	4.525	72.8005	4.737
	75.6	2.420	74.8602	2.507
	81.6	1.350	76.7755	1.387
	PAV Ageing			
	Temperature (°C)	Complex Modulus (G*) (MPa)	Phase Angle(δ) (Dgree)	G*/sinδ (kPa)
	8.4	10.732	38.6	6690.08
	9.9	19.494	34.4	11018.40
	12.8	14.532	36.5	8636.87
	15.9	10.301	39.0	6488.01
	19.0	7.118	41.7	4738.75

Note: 0.30mm CRM –denotes the binder which modified with 10%, 0.30mm crumb rubber modifier

Table 4-16 DSR Test Results of 0.30mm CRM Binder

	Binder Condition			
	Before Ageing			
0.15mm CRM	Temperature (°C)	Complex Modulus (G*) (kPa)	Phase Angle(δ) (Dgree)	G*/sinδ (kPa)
	46.0	38.283	71.0	40.496
	51.6	19.691	73.7	20.512
	57.6	9.006	76.9	9.245
	64.1	4.435	79.9	4.505
	69.7	2.355	82.2	2.377
	75.7	1.238	83.9	1.245
	81.6	0.653	85.4	0.655
	RTFOT Ageing			
	Temperature (°C)	Complex Modulus (G*) (kPa)	Phase Angle(δ) (Dgree)	G*/sinδ (kPa)
	46.0	53.938	65.2	59.424
	51.7	27.678	66.5	30.178
	57.6	14.061	68.2	15.143
	63.6	7.212	70.3	7.662
	69.6	3.932	72.5	4.124
	75.6	2.160	74.7	2.240
	81.6	1.223	76.8	1.257
	PAV Ageing			
	Temperature (°C)	Complex Modulus (G*) (MPa)	Phase Angle(δ) (Dgree)	G*/sinδ (kPa)
	4.1	34.302	32.8	18595.500
	7.1	27.677	34.4	15651.400
	9.8	20.888	36.6	12455.100
	12.8	15.306	39.0	9635.250
	15.8	11.012	41.7	7318.760
	18.8	7.457	44.6	5234.710
	21.8	5.199	47.2	3813.540

Note: 0.15mm CRM –denotes the binder which modified with 10%, 0.15mm crumb rubber modifier

Table 4-17 DSR Test Results of 0.15mm CRM Binder

2) The high temperature performance grades of the four studied binders are listed in the Table 4-18. According to the performance-based binder specification, SHRP A410, the crumb rubber modified binders have the same performance grade, PG76 and are one grade higher than the conventional shell 60/70 binder, PG70. It means that the three crumb rubber modified binders have less temperature susceptibility and may be used in the relative hot climate areas. Results shown in Table 4-18 can further explain the benefit of introducing the crumb rubber modifiers in the pavement materials for improving pavement performance at high temperature.

	Before Ageing ($G^*/\sin\delta \geqq 1$kPa)	After RTFOT ($G^*/\sin\delta \geqq 2.2$kPa)	Performance Grade (PG)
Shell 60/70	76	70	PG 70
0.60mm CRM	76	76	PG 76
0.30mm CRM	76	76	PG 76
0.15mm CRM	76	76	PG 76

Table 4-18 High Temperature Performance Grade

In order to explicitly explicate the temperature effect on the change regularity of rutting factor and fatigue factor of four binders on the three different conditions, Figure 4-4 and Figure 4-5 shows the results for rutting factor, $G^*/\sin\delta$, of the four binders on unaged and RTFOT aged conditions.

3) As shown in Figure 4-4 and Figure 4-5, the general trend of $G^*/\sin\delta$ changes over

temperature is similar for the three binders, i.e., G*/sinδ drops with the increase in temperature. Generally, when the temperature is low, G*/sinδ drops rapidly but the rate of drop decreases with the temperature increase. Also, the rutting factor (G*/sinδ) of the three crumb rubber modified binders are higher than that of the conventional shell 60/70 binder particularly when the testing temperature is below 70°C. The difference is narrowed down with increasing temperature.

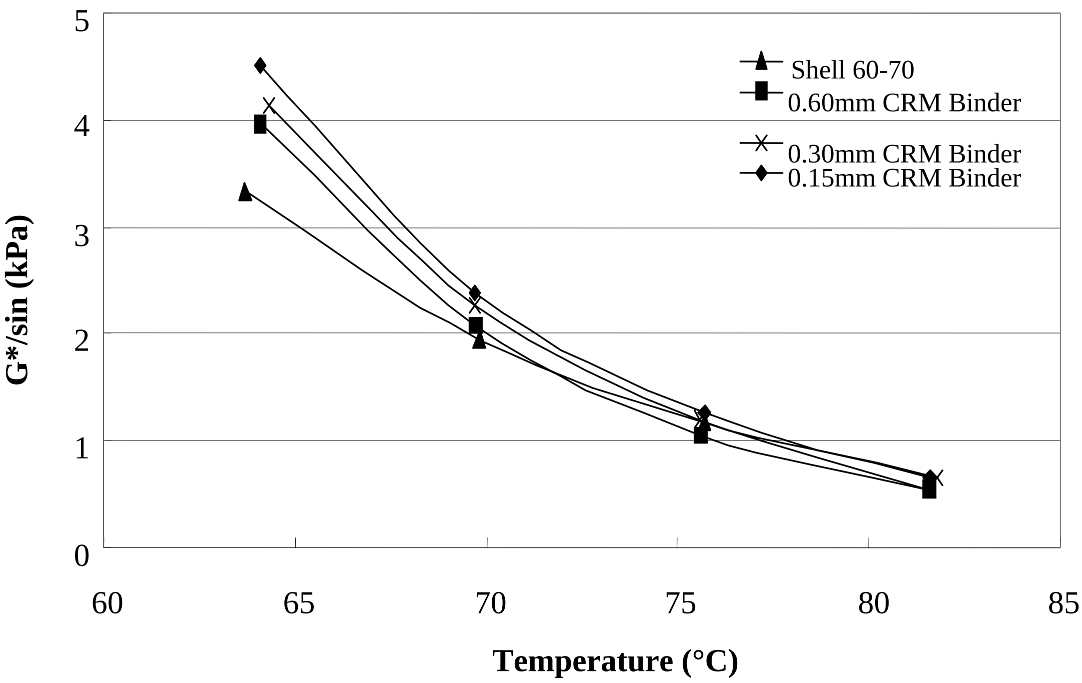


Figure 4-4. G*/sin(δ) Verse Temperature At Unaged State

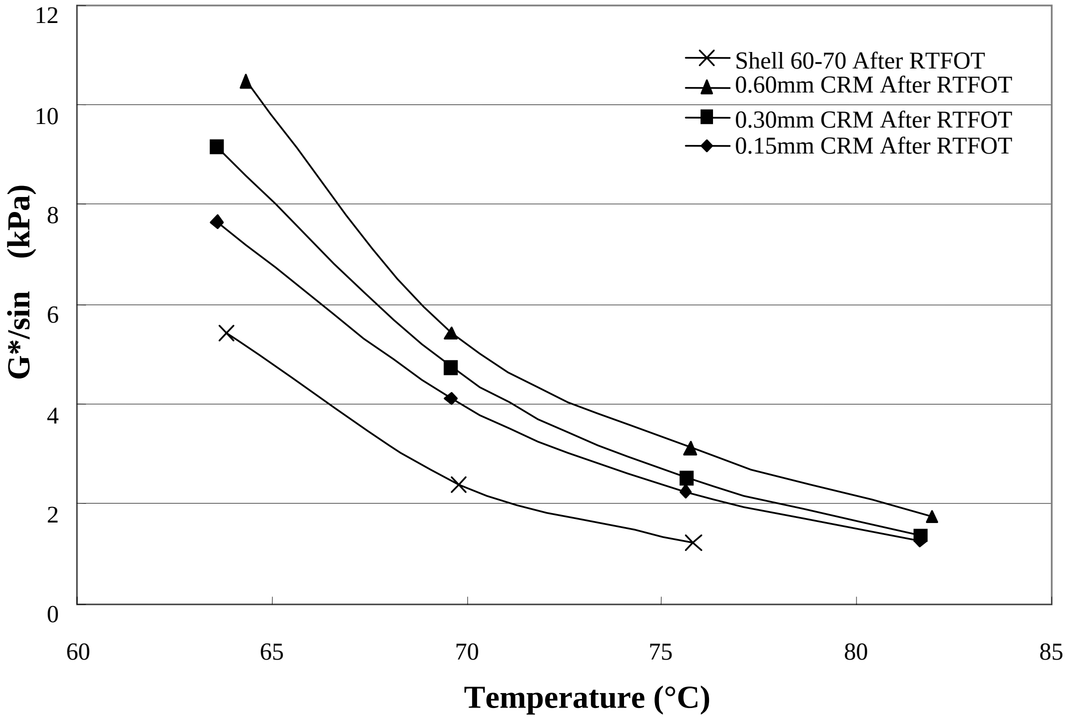


Figure 4-5. G*/sin(δ) Verse Temperature At RTFO Aged State

Figure 4-6 shows the change tendency of the fatigue factor, G*sinδ, of the four binders on PAV ageing condition. SHRP binder specification requires that G*sinδ of binder after PAV ageing must be less than 5000kPa.

4) Figure 4-6 presents the development tendency of fatigue factor, G*sinδ, of the four studied binders on PAV aged condition. The linear reducing tendency of G*sinδ can be observed from Figure 4-6. All three crumb rubber modified binders have smaller G*sinδ than the conventional shell 60/70 binder. And among those four binders, 0.30mm crumb rubber modified binder has the lowest G*sinδ. It indicates that crumb

rubber modified binders, especially 0.30mm crumb rubber modified binder, performs well with respect to fatigue resistance at the approximate average (termed "intermediate") pavement temperature.

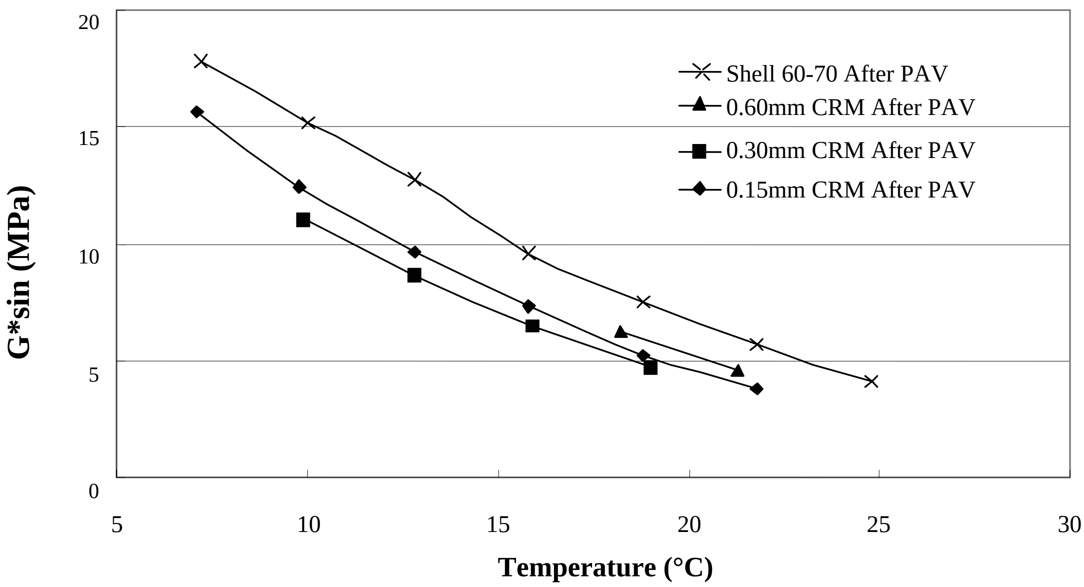


Figure 4-6. G*sin(δ) Verse Temperature At PAV Aged State

Statistical analysis of two way variance (ANOVA) and means comparison

Same as for softening point analysis, statistical analysis of two way variance (ANOVA) (α=0.05) and means comparison was conducted with the DSR data. In the ANOVA, the softening point is the response variable and the binder types and ageing conditions are the influence Factor A and Factor B respectively.

The test results analysis by two way variance (ANOVA) (α=0.05) are shown in Table 4-19. As measured by the P-values, it is observed that the P values of influence factor A and B are equal to 0.000 which is less than the assumed value 0.05. Also, the P value (7.065E-12) of interaction of Factor A and Factor B is less than the assumed value 0.05. Hence, the statistical analysis indicates that, from the results of the ring and ball test, binder types (Factor A) and ageing conditions (Factor B) and their interaction have a significant effect on the softening point of binders.

Analysis results by means comparison using the Tukey Test are shown in Table 4 and 5. The Tukey test was used in the comparison of means with an α-level set at 0.05. The null hypothesis is that the mean of the softening point between the CRM modified binder (aged or unaged) and PG60/70 (age or unaged) are equal and the alternative hypothesis is that the means are different. The results are shown in Table 4-20 and 4-21 respectively.

Source	**Degree of Freedom**	**Sum of Squares**	**Mean Square**	**F Value**	**P Value**
Factor A	3	2.834E+14	9.447E+13	0.95302	0.422
Factor B	2	6.077E+15	3.039E+15	30.6517	1.428E-09
Factor A*B	6	5.049E+14	8.414E+13	0.84881	0.538
Error	53	5.254E+15	9.913E+13		
Total	64	1.210E+16			

Factor A*B means the interaction of Factor A and Factor B

Table 4-19. Effect of CRM Binders on G/sinδ analysis by Two-Way ANOVE

Level	Mean (ºC)		Simultaneous Confidence Intervals		
			Lower Limit	Upper Limit	
0.15mm CRM	9.81E+06				
0.30mm CRM	6.46E+06	3.35E+06	5.41E+06	1.21E+07	No
0.60mm CRM	2.40E+06	7.42E+06	2.73E+06	1.76E+07	No
Shell 60/70	1.14E+06	1.59E+06	1.01E+06	6.90E+06	No
0.30mm CRM	6.46E+06				
0.60mm CRM	2.40E+06	4.06E+06	6.58E+06	1.47E+07	No
Shell 60/70	1.14E+06	4.94E+06	1.40E+06	4.14E+06	No
0.60mm CRM	2.40E+06				
Shell 60/70	1.14E+06	9.00E+06	1.94E+06	1.41E+06	No

Table 4-20. Mean Comparation on G/sinδ of CRM Binders with Factor A by using Tukey Test

Level	Mean (ºC)		Simultaneous Confidence Intervals		
			Lower Limit	Upper Limit	
OB	5.94E+03				
PAV	2.56E+07	2.562E+07	3.303E+07	1.821E+07	Yes
RTFOT	1.51E+04	9.161E+03	7.255E+06	7.237E+06	Yes
PAV	2.56E+07				
RTFOT	1.51E+04	2.56E+07	1.836E+07	3.286E+07	Yes

Table 4-21. Mean Comparation on G/sin(δ) of CRM Binders with Factor B by using Tukey Test

The results of the two way variance (ANOVA) (α=0.05) analysis are shown in Table 4-19. As measured by the P-values, it is observed that the P value of influence Factor A (0.42177) is larger than the assumed value (α=0.05), whereas the P values of influence Factor B and the interaction of Factors A and B are 1.42803E-9 and 0.5383 respectively, both values are less than the assumed one (α=0.05). It indicates that the ageing condition has significantly influenced on the rutting factor G/sin(δ) whereas there is no significant influence on G/sin(δ) among the three CRM modified binders is detected.

The results of comparison using Tukey Test are shown in Tables 4-20 and 4-21. From Table 4-20, there is only a slight difference between the means of rutting factor G/sin(δ) for these four types of binders (three CRM modified and one PG60/70 binders). From Table 4-21, different levels of influence on the rutting factor (G/sin(δ)) under the three ageing conditions (OB, RTFOT, PAV) are shown. The means of rutting factors (G/sin(δ)) between original and after RTFO binder are slightly different whereas the means of the rutting factor (G/sin(δ)) between OB and PAV, RTFOT and PAV binders are obviously different.

CHAPTER 5

PERFORMANCE OF CRUMB RUBBER MODIFIED WEARING COURSE MIXTURES

According to the test results from Chapter 4, the binder modified by crumb rubber modifier has shown better performance, particularly in high temperature susceptibility. In this chapter, the performances of crumb rubber modified mixtures are presented.

5.1 Testing methods on Modified Wearing Courses

A series of laboratory tests were designed to assess the fundamental mechanical properties and durability performance of the modified mixtures of AC-10 and AC-20. The bitumen used included the conventional Shell 60/70 bitumen and the other three crumb rubber modified binders which were modified by 10% 0.60mm, 0.30mm and 0.15mm crumb rubber modifiers. The testing methods are briefly described as follows.

5.1.1 Testing on Volumetric Property

The test on volumetric property contained the determination of the theoretical maximum specific density, bulk relative density and air voids for the four wearing course mixtures. Maximum specific density was a vital factor and used as follows:

(a) To calculate the air voids in the compacted bituminous paving mixture,

(b) To calculate the amount of bitumen absorbed by the aggregates; and

(c) To provide target values for the compaction of paving mixtures.

As specified in ASTM D 3203-94, for wearing course mixtures, the bulk density of a regularly shaped specimen of compacted mixture was determined from its dry mass and volume. The height of the specimen was obtained by Test Method ASTM D 3549-93a. After measuring the diameter of the specimen at four locations and averageing, the volume of the specimen was calculated based on the average height and diameter measurement. The density was converted to bulk specific gravity by dividing it with 0.99707 g/cm^3 or 997 kg/ m^3, the density of water at 25°C. The percent air voids in a compacted bituminous paving mixture is calculated as follows:

$$\text{Percent Air Voids} = 100 \times \left(1 - \frac{\text{Gsb}}{\text{Gmm}}\right)$$

where:

Gsb=Bulk specified gravity of mixture; and

Gmm=Theoretical maximum specified gravity of mixture.

A total of 8 cylindrical specimens (150mm × 75mm) were required because each type of the two wearing course mixtures required four specimens. Details for test procedure can be found in reference of ASTM D2041-00 and ASTM D3203-94.

5.1.2 Indirect Tensile Stiffness Modulus Test

Indirect tensile stiffness modulus (ITSM) test (see Picture 5-1) was conducted by using the NU-10 (see Picture 5-2) in accordance with BS DD213:1993. A total of 32 cylindrical specimens (150mm × 75mm) were required because each type of the eight different wearing course mixtures required 4 specimens. The specified test temperature was 30°C. The stiffness modulus of porous asphalts is calculated as follows:

$$S_m = \frac{L}{(D \times t)} \times (v + 0.27)$$

where:

L =the peak value of the applied vertical load (in N);

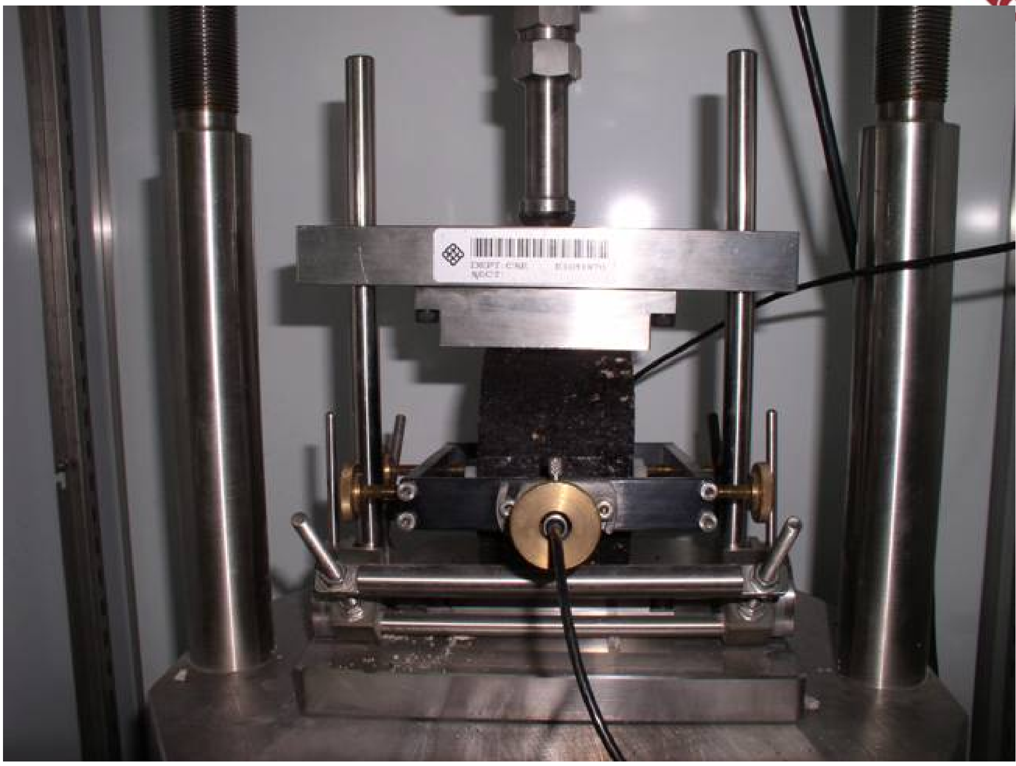
D =the peak horizontal diametral deformation resulting from the applied load (in mm);

t =the mean thickness of the test specimen (in mm); and

v =the value of Poisson's ratio for the bituminous mixture at the temperature of test. (According to the specification, v =0.35 for 20ºC, 0.45 for 30 ºC)

.

Picture 5-1 Laboratory Equipment of NU-10



Picture 5-2 The Apparatus for ITSM Test

The major problem with most stiffness modulus testing systems is their inability to determine a reasonable Poisson ratio, which is one of the most important parameters influencing the accuracy of the stiffness modulus; however the NU-10 measurement system used in this testing item provides a promising measurement method for the determination of consistent and reasonable Poisson ratios.

5.1.3 Wheel Tracking Test

Laboratory wheel tracking was shown to have an acceptable correlation with asphalt rutting. Rutting is the result of accumulation of permanent strains. Permanent strain has a higher value on the condition of a higher temperature and a low traffic speed (longer loading time), which can be simulated most closely by the wheel tracking test.

The wheel tacking machine used in the study was based on a U.K. Transport Research Laboratory design, modified so that the samples could be loaded using a pneumatic ram and tracking data could be acquired using a desktop computer. It complied with British Standard 598 Part 110 and with the draft European CEN standard on wheel tracking. The machine consisted of a loaded 200mm diameter, 50mm wide rubber tyred wheel which rested on a 305mm square specimen held on a moving table. The pressure acting on the sample is 0.425MPa. The standard test lasted for 45 minutes or until the rut depth exceeded 15mm. The rut depth at the centre of the specimen was measured and recorded every minute. At the end of the test the rutting rate in mm per hour was calculated from the slope of the rut depth versus time relationship over the last third of the test (see Figures 5-1 & 5-2).

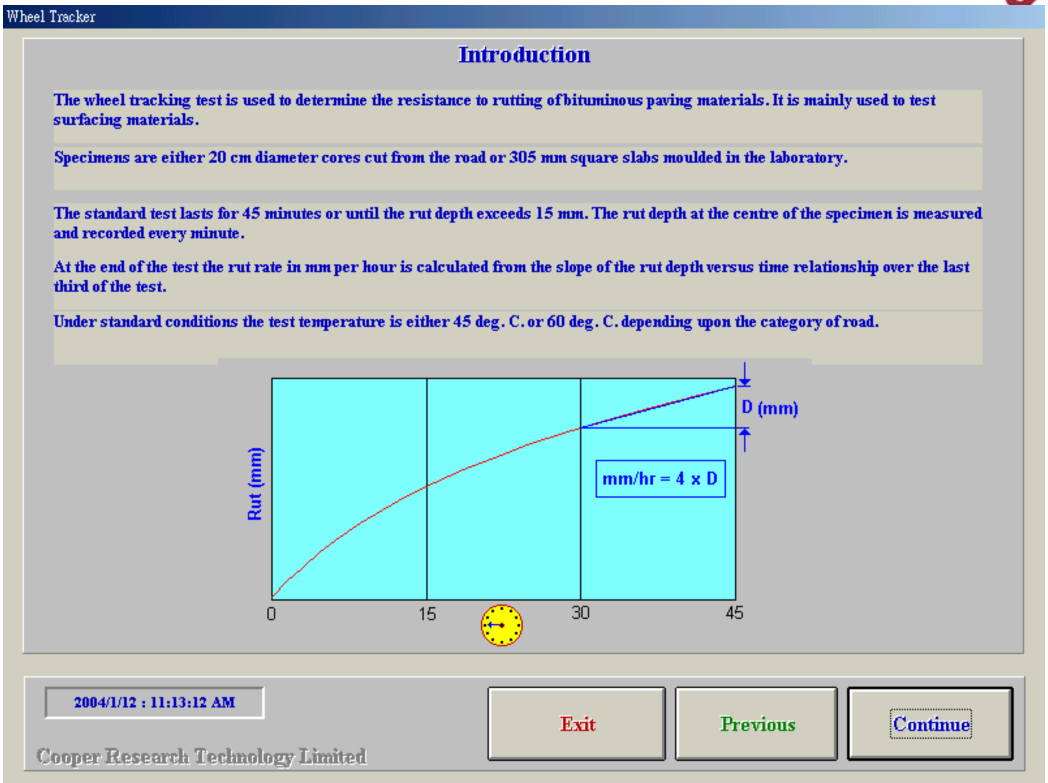


Figure 5-1 The Introduction of The Wheel Trackign Test

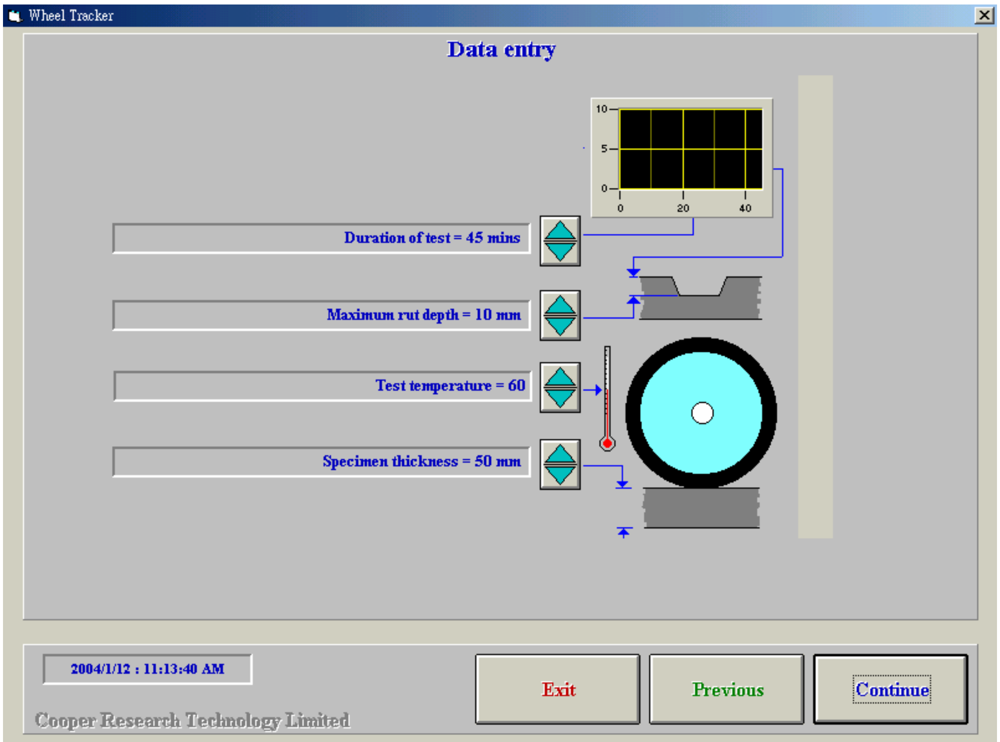


Figure 5-2 The Conditions of The Wheel Trackign Test

Three specimens were required for each of the eight types of wearing course mixtures, including four mixtures of AC-10 and four mixtures of AC-20. A total of 24 slab specimens (305mm x 305mm x 50mm) were prepared. The test temperature was 60°C [BS 598 -110].

Rut depth and rutting rate, are used to evaluate the rutting susceptibility of the three porous asphalts. The rut depth of the specimen was measured and recorded every minute (see Figure 5-1). At the end of the test the rutting rate in mm per hour was deduced from the slope of the rutting depth versus time relationship over the last third of the testing period. The following function was used to calculate the rutting rate.

$$T_R = 3.6\{r_n - r_{(n-3)}\} + 1.2\{r_{(n-1)} - r_{(n-2)}\}$$

where：

n is the total number of readings taken at 5 min intervals for up to 45 minutes, excluding the

initial reading; and

r_n is the vertical displacement measured at nth reading, in millimeters (mm).

5.1.4 Dynamic Creep and Indentation Creep Test

The repeated load axial test (RLAT) included two testing models, dynamic creep test (see Picture 5-3) and indentation creep test (see Picture 5-4), which were used to rank bituminous mixtures in terms of resistance to permanent deformations.

Picture 5-3 The Apparatus for Dynamic Creep Test

Picture 5-4 The Apparatus for Indentation Creep Test

A total of 36 cylinder specimens (150mm × 75mm) were used to perform the dynamic creep and indentation creep tests with NU-10. 18 dynamic creep tests and 18 indention creep tests were performed.

Dynamic creep test is an unconfined test, which is considered to be more simulative of traffic loading than static creep. In reality, the material has lateral support from the surrounding material, so the indentation creep test was put forward to supplement the dynamic creep test. The dynamic creep testing was conducted in accordance with BS DD226. The testing conditions of the indention and dynamic creep tests are shown in Table5-1.

	Dynamic Creep	Indention Creep
Testing Temp (^{0}C)	30±0.5	30±0.5
Axial Stress (kPa)	100	100
Load Application Period (S)	1	1
Rest Period (S)	1	1
Test Duration (S)	3600	3600
Upper Loading Diameter (mm)	150	150
Condition Stress (kPa)	10	10

Table5-1 Test Condition for Dynamic Creep and Indention Creep Test

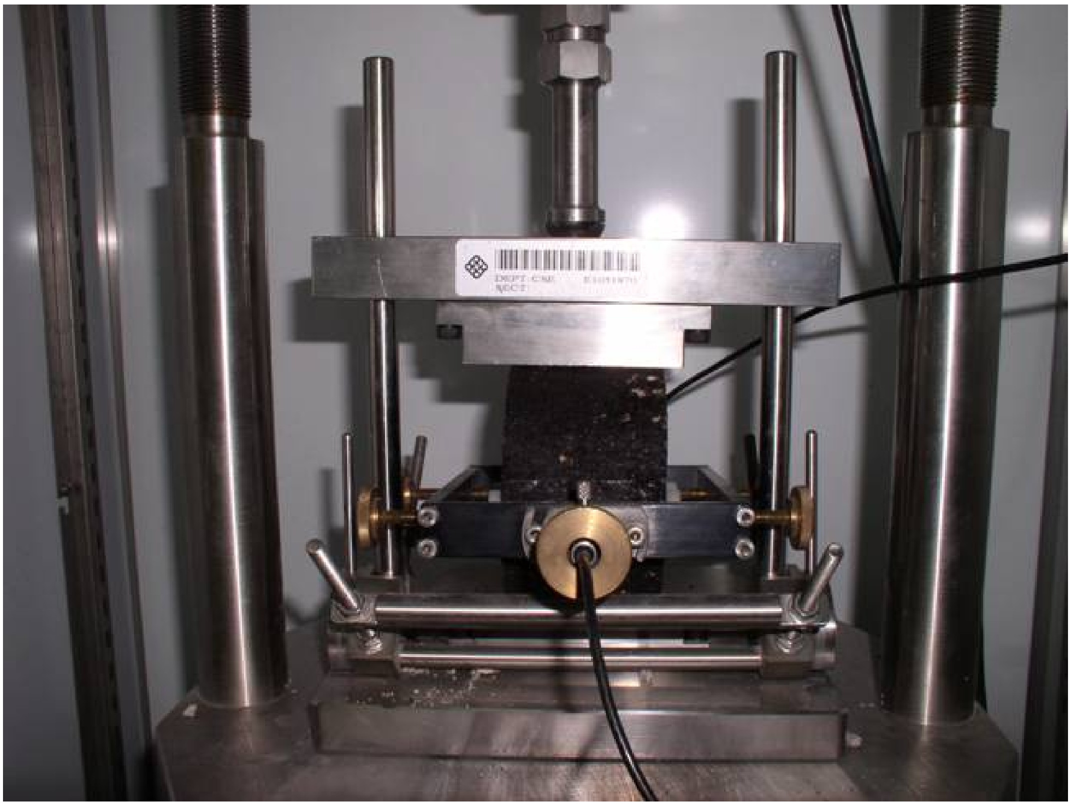
5.1.5 Moisture Damage Test

Moisture is a key element in the deterioration of wearing course materials. There are two main mechanisms by which moisture can degrade the integrity of surfacing asphalts:

(1) Loss of cohesion and stiffness of binder film; and

(2) Failure of the adhesion between the aggregate and binder.

When the aggregate tends to have a preference for absorbing water, the binder is stripped away. Stripping leads to premature pavement distress and then ultimate failure of the pavement. The adhesion properties of normal and modified binders with aggregates were tested by using indirect tensile strength (ITS) test and indirect tensile stiffness modulus (ITSM) test in accordance with AASHTO T283 (see Picture 5-5 & 5-6). The results helped to evaluate the water sensitivity and predicted long-term ravelling susceptibility of the four wearing course mixtures.



Picture 5-5 The Apparatus for ITSM Test

Picture 5-6 The Apparatus for ITS Test

A total of 48 (24 for each AC-10 and AC-20) cylinder specimens (100mm×60mm) were used to compete the indirect tensile stiffness modulus ratio test. The tensile strength (ITS) before and after soaking was obtained.

The indirect tensile strength (ITS) is defined as the maximum stress from a diametrical vertical force that a sample can withstand. It is determined using the following equation:

$$S_t = \frac{2000 \times P}{\pi \times t \times D}$$

where:

St =indirect tensile strength (kPa),

P = maximum load (N),

t = specimen height (mm), and

D = specimen diameter (mm).

The indirect tensile strength test was performed at 25°C. A loading rate of 50 mm per minute until the maximum load was reached.

From the measured indirect tensile strengths, ITSR was calculated for each sample as follows:

$$ITSR = \frac{S_{tm}}{S_{td}}$$

where:

S_{tm} =the average tensile strength of the moisture-conditioned subset; and

S_{td} =the average tensile strength of the dry (unconditioned) subset.

The measuring method of indirect tensile stiffness modulus (ITSM) was based on BS DD213. The testing condition of indirect tensile stiffness modulus ratio (ITSMR) satisfied the requirement of ASTM D4867. ITSMR was calculated for each sample as follows:

$$ITSMR = \frac{S_{mm}}{S_{md}}$$

where:

S_{mm} =the average stiffness modulus of the moisture-conditioned subset, and

S_{md} =the average stiffness modulus of the dry (unconditioned) subset.

It is worthwhile to mention that ASTM D4867 and ASSHTO T283 differ in their treatment of samples before they are loaded. ASTM states that “Adjust the temperature of the dry subset by soaking in a water bath for 20 min at 77±1.8°F (25±1°C).” AASHTO T283 states that “The dry subset will be stored at room temperature until test. The specimens shall be wrapped with plastic or placed in a heavy-duty leak proof plastic bag. The specimens shall then be placed in a 77°F (25°C) water bath for a minimum of 2 hours” The ASTM procedure allows compacted samples to be immersed in water for 20 min, whereas the AASHTO procedure attempts to keep the samples dry while they are submerged.

5.2 Results and Analysis

5.2.1 Result of Volumetric Property Test

The results of theoretical maximum specific density, bulk relative density and air voids of the unmodified and modified mixtures (0.15mm CRM was used in this test) are listed in Table 5-2. The testing results show that of the volumetric properties meet the requirement of wearing course mixture design specification of Hong Kong. The air void of the crumb rubber modified mixtures is greater than those of the unmodified mixtures. It may be due to the elastic property of the rubber particle, when the sample was just finished compaction, the rubber particle was seem to be well compacted at the high temperature, but this sample is not 100% stable and as the time goes by the temperature of the sample would dropped and the rubber particle expend a certain scale.

Mixture Type	Spec. No.	Mass In Air (g)	Specimen Avg. Height (mm)	Specimen Avg. Diameter (mm)	G_{sb} (g/cm^3)	G_{mm} (g/cm^3)	Air Voids (%)
Shell 60/70	1#	3067.3	75.07	150.17	2.307	2.417	4.55
	2#	3060.1	75.10	150.11	2.302	2.412	4.54
	3#	3055.9	74.97	150.09	2.304	2.407	4.28
	4#	3058.4	74.91	150.12	2.307	2.409	4.25
	Avg.	-	-	-	2.305	2.411	4.41
CRM	1#	3061.5	75.29	150.22	2.272	2.410	5.71
	2#	3055.3	75.10	150.11	2.295	2.401	4.41
	3#	3062.6	75.24	150.23	2.293	2.415	5.07
	4#	3059.7	74.18	150.15	2.326	2.404	3.26
	Avg.	-	-	-	2.296	2.411	4.61

Notes: Shell 60/70 – denotes conventional wearing course asphalt
CRM –denotes crumb rubber modified wearing course asphalt

Table 5-2 Volumetric Property Testing Results

5.2.2 Result of Indirect Tensile Stiffness Modulus Test

Stiffness modulus of a mix is considered to be related to its rutting resistance. Poor stiffness modulus of a material normally would result larger rutting. The testing results of indirect tensile stiffness modulus (ITSM) of the specimens of the tested wearing course mixtures (AC-10 & AC-20) are shown in Table 5-3 and Table 5-4 respectively. Major findings are described as follows:

1) For AC-20, all of the three crumb rubber modified mixtures are shown to have greater stiffness modulus than unmodified mixture and among those three modified binders the one modified with 0.60mm crumb rubber modifier has the lowest stiffness modulus. The performance of the 0.30mm CRM mixtures and the 0.15mm CRM mixtures are similar and have larger stiffness modulus. From Table 5-5 and Table 5-6, the stiffness modulus value of the 0.15mm CRM mixture is about 40 percent higher than that of the Shell 60/70 mixture, the stiffness modulus value of 0.30mm CRM mixture is about 36 percent higher than the Shell 60/70 mixture and the stiffness modulus value of 0.60mm CRM mixture is about 12 percent higher than the Shell 60/70 mixture.

2) However, for the AC-10, the stiffness modulus of crumb rubber modified binders slightly larger than that of the unmodified mixture. The reason for the drop of AC-10 might be caused by their higher binder content (5.7%) compared with the AC-20 (4.5%), it indicates that the CRM binder is more likely benefits the dense-graded mixture with coarser mixture. From the wheel tracking test results, both the modified AC-10 and AC-20 exhibit stronger resistance against rutting than the unmodified mixture.

Table 5-3. Testing Results of ITSM for AC-20

Spec. No.		Average Diameter (mm)	Average Thickness (mm)	Test Temperature (°C)	Testing Stiffness (MPa)	Adjusted Stiffness (MPa)	Average Adjusted Stiffness (MPa)
Shell 60/70	1#	100	60	30	714	725	703
	2#	100	60.5	30	682	689	
	3#	100	59.5	30	681	692	
0.15mm CRM	1#	100	60.5	30	853	877	982
	2#	100	60	30	993	1006	
	3#	100	60	30	856	860	
0.30mm CRM	1#	100	60	30	981	996	957
	2#	100	59.5	30	938	949	
	3#	100	60	30	921	928	
0.60mm CRM	1#	100	60.5	30	772	781	790
	2#	100	60	30	763	778	
	3#	100	60.5	30	798	811	

Notes: Shell 60/70 – denotes conventional porous asphalt

0.60mm CRM – denotes modified mixture modified by 0.60mm crumb rubber modifier

0.30mm CRM – denotes modified mixture modified by 0.30mm crumb rubber modifier

0.15mm CRM – denotes modified mixture modified by 0.15mm crumb rubber modifier

Table 5-4. Testing Results of ITSM for AC-10

Spec. No.		Average Diameter (mm)	Average Thickness (mm)	Test Temperature (°C)	Testing Stiffness (MPa)	Adjusted Stiffness (MPa)	Average Adjusted Stiffness (MPa)
Shell 60/70	1#	100	60	30	650	659	639
	2#	100	60	30	619	626	
	3#	100	59.5	30	618	629	
0.15mm CRM	1#	100	60.5	30	549	566	679
	2#	100	60	30	690	707	
	3#	100	60.5	30	553	561	
0.30mm CRM	1#	100	60	30	683	695	660
	2#	100	60.5	30	641	650	
	3#	100	59.5	30	623	634	
0.60mm CRM	1#	100	60.5	30	626	635	644
	2#	100	60	30	618	632	
	3#	100	59.5	30	653	665	

Notes: Shell 60/70 – denotes conventional porous asphalt

0.60mm CRM – denotes modified mixture modified by 0.60mm crumb rubber modifier

0.30mm CRM – denotes modified mixture modified by 0.30mm crumb rubber modifier

0.15mm CRM – denotes modified mixture modified by 0.15mm crumb rubber modifier

	Stiffness of AC-20 (MPa)	Stiffness of AC-10 (MPa)
Shell 60/70	**703**	**639**
0.15mm CRM	**982**	**679**
0.30mm CRM	**957**	**660**
0.60mm CRM	**790**	**644**

Table 5-5. Summary of the ITSM Test

	AC-20 (%)	AC-10 (%)
(0.15mm CRM-Shell 60/70) / Shell 60/70	40	6
(0.30mm CRM-Shell 60/70) / Shell 60/70	36	3
(0.60mm CRM-Shell 60/70) / Shell 60/70	12	2

Table 5-6 The Comparison Between Modified Mixtures and Unmodified Mixture

5.2.3 Result of Moisture Damage Test

The testing results of the AC-10 and AC-20 for conditioned and unconditioned specimens are given in Tables 5-7, Table 5-8 and Figure 5-1, respectively.

Mixture Type	Average Tensile Strength(MPa)		ITS Ratio (%)
	Unconditioned	Conditioned	
Shell 60/70	802	944	117.6%
0.15mm CRM	1085	1085	100.0%
0.30mm CRM	1062	1038	97.7%
0.60mm CRM	897	1084	120.9%

Table 5-7. Testing Results of ITSR For AC-20

Mixture Type	Average Tensile Strength(MPa)		ITS Ratio (%)
	Unconditioned	Conditioned	
Shell 60/70	755	897	118.8%
0.15mm CRM	779	849	109.0%
0.30mm CRM	707	802	113.5%
0.60mm CRM	637	802	125.9%

Table 5-8. Testing Results of ITSR For AC-10

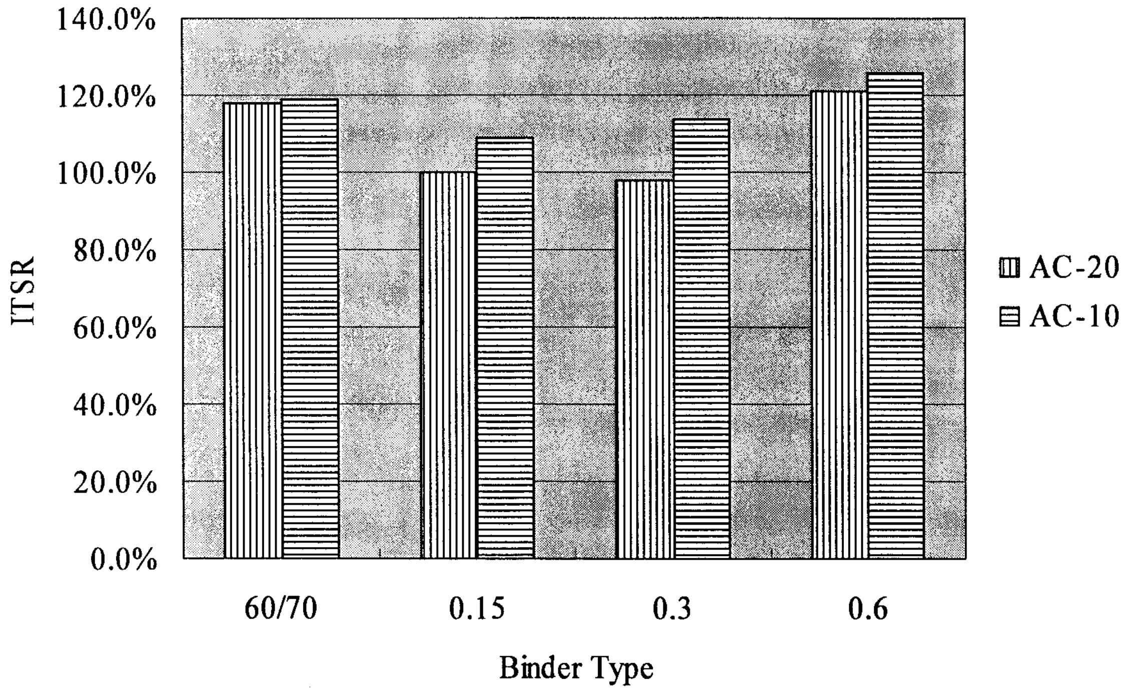


Figure 5-3. Effect of Gradation and Binder on ITSR

Conclusion:

The modified mixtures would exhibit less susceptibility to moisture damage than that of the conventional porous asphalts due to the relative lower reduction in indirect tensile strength (ITSR). It might also imply the mechanism of a tighter cohesion between modified binders and aggregates [S.D. Williamson]. The laboratory testing results of ITSR for the four types of AC-10 and AC-20 require to be considered together in order to give a reasonable evaluation for their resistance to water damage. Major findings from the testing results are presented as follows：

For AC-20:

1) From Table 5-7, the indirect tensile strengths of the modified mixtures are all greater than that of the unmodified mixture both before and after water treatment. Furthermore, the mixture with 0.15mm CRM has the greatest value among the four mixtures. It indicates that crumb rubber modifier strengthens the stiffness of the mixtures, particularly with 0.15mm CRM.

2) For the values of the mixture with 0.60mm CRM has the greatest strength among the four mixtures, but the ITSR of 0.30mm CRM and 0.15mm CRM mixtures are a slightly smaller than those of the unmodified mixtures; Same for the AC-10, the indirect tensile strength (ITSR) of all the four mixtures are over 100%, it indicates that CRM mixtures should exhibit less susceptibility to moisture damage.

For AC-10:

1) From Table 5-8, only the indirect Tensile Strength of 0.15mm CRM mixtures is greater than the unmodified mixture both before and after water treatment. Both the other two modified mixtures show lower value than the unmodified mixture. It indicates that crumb rubber modified only increase the stiffness of the 0.15mm CRM

mixture.

2) For the indirect tensile strength (ITSR), the mixture with 0.60mm CRM is the highest among the four mixtures, Furthermore; the ITSR's of all those three modified mixtures are over 100%. It indicates that CRM mixtures should exhibit less susceptibility to moisture damage when compared with unmodified mixtures..

From Figure 5-3, those values of ITSR were over 100%, the reason for that might be due to that when the loading was act quickly (loading rate of 50 mm per minute) on the dense grade sample, there is not enough time for the water to escape thus creating a large water pressure inside the mixture.

5.2.4 Result of Wheel Tracking Test

Wheel tracking test was selected to study the effect of temperature on the permanent deformation behaviour of wearing course material used in Hong Kong. Details of the testing results of the wheel tracking are presented from Table 5-9 to Table 5-16 and a summary is presented the Table 5-17, Figure 5-2 and Figure 5-3.

minute	def(mm)	minute	def(mm)
1	1.08	24	4.72
2	1.53	25	4.92
3	1.93	26	4.85
4	2.11	27	4.95
5	2.35	28	5.03
6	2.54	29	5.17
7	2.75	30	5.18
8	3.02	31	5.27
9	3.16	32	5.38
10	3.33	33	5.40
11	3.48	34	5.52
12	3.65	35	5.57
13	3.61	36	5.54
14	3.77	37	5.6
15	4.01	38	5.66
16	3.97	39	5.76
17	4.11	40	5.8
18	4.19	41	5.89
19	4.28	42	5.92
20	4.39	43	6.1
21	4.52	44	6.04
22	4.56	45	6.11
23	4.74		

Table 5-9 Wheel Tracking Test Result of AC-20 (Shell 60/70)

minute	def(mm)	minute	def(mm)
1	0.53	24	1.72
2	0.69	25	1.8
3	0.81	26	1.81
4	0.84	27	1.85
5	0.97	28	1.9
6	0.97	29	1.8
7	1.15	30	1.92
8	1.12	31	1.85
9	1.17	32	1.95
10	1.31	33	1.87
11	1.33	34	1.88
12	1.45	35	2.02
13	1.39	36	1.95
14	1.42	37	2.07
15	1.52	38	2.07
16	1.48	39	2.14
17	1.57	40	2.11
18	1.65	41	2.12
19	1.66	42	2.13
20	1.65	43	2.12
21	1.63	44	2.07
22	1.78	45	2.11
23	1.68		

Table 5-10 Wheel Tracking Test Result of AC-20 (0.15mm CRM)

minute	def(mm)	minute	def(mm)
1	0.61	24	2.31
2	0.86	25	2.43
3	1.09	26	2.44
4	1.22	27	2.42
5	1.23	28	2.49
6	1.38	29	2.34
7	1.41	30	2.53
8	1.54	31	2.61
9	1.70	32	2.43
10	1.77	33	2.64
11	1.74	34	2.60
12	1.87	35	2.70
13	1.86	36	2.54
14	1.90	37	2.82
15	1.83	38	2.62
16	2.03	39	2.82
17	2.03	40	2.92
18	2.06	41	2.9
19	2.05	42	2.88
20	2.16	43	2.91
21	2.24	44	2.74
22	2.24	45	2.97
23	2.19		

Table 5-11 Wheel Tracking Test Result of AC-20 (0.30mm CRM)

minute	def(mm)	minute	def(mm)
1	0.84	24	2.78
2	1.11	25	3.19
3	1.34	26	2.82
4	1.41	27	2.89
5	1.58	28	2.89
6	1.67	29	2.93
7	1.73	30	2.98
8	1.93	31	3.02
9	1.95	32	3.04
10	2.03	33	3.11
11	2.12	34	3.17
12	2.19	35	3.48
13	2.25	36	3.20
14	2.27	37	3.17
15	2.30	38	3.26
16	2.39	39	3.19
17	2.43	40	3.22
18	2.48	41	3.31
19	2.55	42	3.36
20	2.54	43	3.32
21	2.62	44	3.79
22	2.73	45	3.47
23	2.64		

Table 5-12 Wheel Tracking Test Result of AC-20 (0.60 CRM)

minute	def(mm)	minute	def(mm)
1	1.05	24	3.66
2	1.33	25	3.74
3	1.62	26	3.79
4	1.75	27	3.97
5	1.94	28	4.01
6	2.13	29	4.06
7	2.19	30	4.14
8	2.31	31	4.12
9	2.43	32	4.26
10	2.58	33	4.30
11	2.68	34	4.30
12	2.78	35	4.43
13	2.91	36	4.52
14	2.97	37	4.48
15	2.99	38	4.58
16	3.07	39	4.70
17	3.21	40	4.74
18	3.22	41	4.82
19	3.29	42	4.77
20	3.46	43	4.93
21	3.44	44	4.99
22	3.60	45	4.94
23	3.67		

Table 5-13 Wheel Tracking Test Result of AC-10 (shell 60/70)

minute	def(mm)	minute	def(mm)
1	0.15	24	1.07
2	0.20	25	1.09
3	0.34	26	1.10
4	0.36	27	1.12
5	0.37	28	1.13
6	0.49	29	1.16
7	0.55	30	1.17
8	0.57	31	1.18
9	0.60	32	1.21
10	0.66	33	1.22
11	0.66	34	1.25
12	0.74	35	1.25
13	0.77	36	1.27
14	0.78	37	1.32
15	0.82	38	1.31
16	0.86	39	1.33
17	0.9	40	1.36
18	0.93	41	1.36
19	0.95	42	1.40
20	0.97	43	1.40
21	0.99	44	1.43
22	1.00	45	1.43
23	1.02		

Table 5-14 Wheel Tracking Test Result of AC-10 (0.15mm CRM)

minute	def(mm)	minute	def(mm)
1	0.79	24	2.27
2	0.98	25	2.23
3	1.12	26	2.28
4	1.21	27	2.30
5	1.44	28	2.45
6	1.52	29	2.36
7	1.62	30	2.39
8	1.59	31	2.49
9	1.65	32	2.47
10	1.68	33	2.52
11	1.75	34	2.46
12	1.76	35	2.49
13	1.80	36	2.62
14	1.88	37	2.53
15	1.87	38	2.69
16	1.96	39	2.60
17	1.97	40	2.61
18	2.02	41	2.64
19	2.08	42	2.65
20	2.08	43	2.67
21	2.14	44	2.70
22	2.14	45	2.8
23	2.17		

Table 5-15 Wheel Tracking Test Result of AC-10 (0.30mm CRM)

minute	def(mm)	minute	def(mm)
1	2.27	24	1.75
2	2.23	25	1.77
3	2.28	26	1.80
4	2.30	27	1.84
5	2.45	28	1.90
6	2.36	29	1.87
7	2.39	30	1.92
8	2.49	31	1.92
9	2.47	32	1.92
10	2.52	33	1.92
11	2.46	34	2.00
12	2.49	35	1.97
13	2.62	36	1.98
14	2.53	37	2.02
15	2.69	38	2.03
16	2.6	39	2.05
17	2.61	40	2.06
18	2.64	41	2.17
19	2.65	42	2.08
20	2.67	43	2.17
21	2.70	44	2.15
22	2.80	45	2.12
23	2.27		

Table 5-16 Wheel Tracking Test Result of AC-10 (0.60 CRM)

Specimen Reference		Test Temperature (^0C)	Rutting Rate (mm/hr)	Rut Depth (mm)
AC-20	Shell 60/70	60	4.32	6.11
	0.15mm CRM	60	1.61	2.11
	0.30mm CRM	60	2.07	2.97
	0.60mm CRM	60	2.54	3.47
AC-10	Shell 60/70	60	3.16	4.94
	0.15mm CRM	60	1.22	1.43
	0.30mm CRM	60	1.95	2.8
	0.60mm CRM	60	1.73	2.15

Table 5-17 Testing Results of Rutting Depth

Notes: Shell 60/70 – denotes conventional porous asphalt

0.15mm CRM – denotes modified mixture modified by 0.60mm crumb rubber modifier

0.30mm CRM – denotes modified mixture modified by 0.30mm crumb rubber modifier

0.60mm CRM – denotes modified mixture modified by 0.15mm crumb rubber modifier

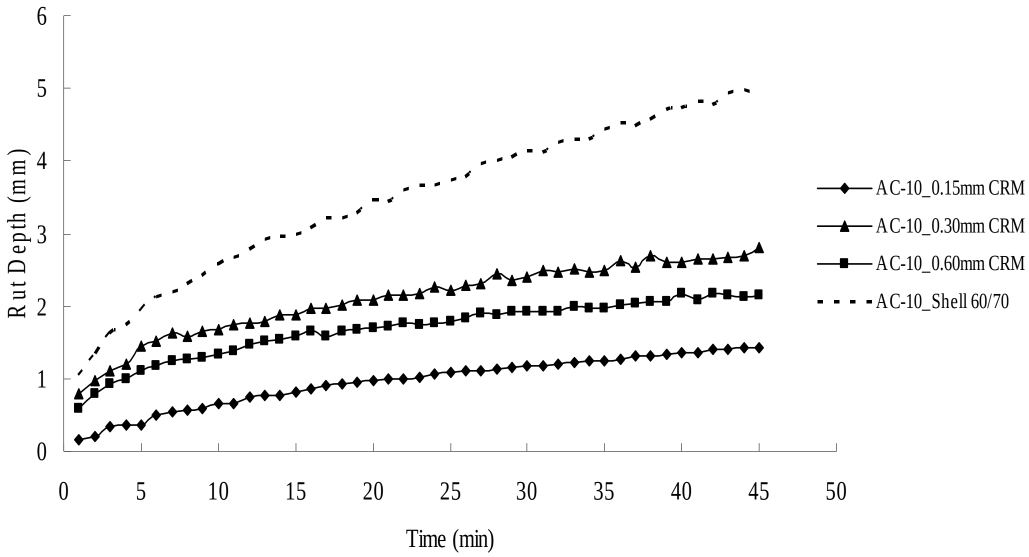


Figure 5-4 Graph of Rutting Depth Verse Time for AC-10

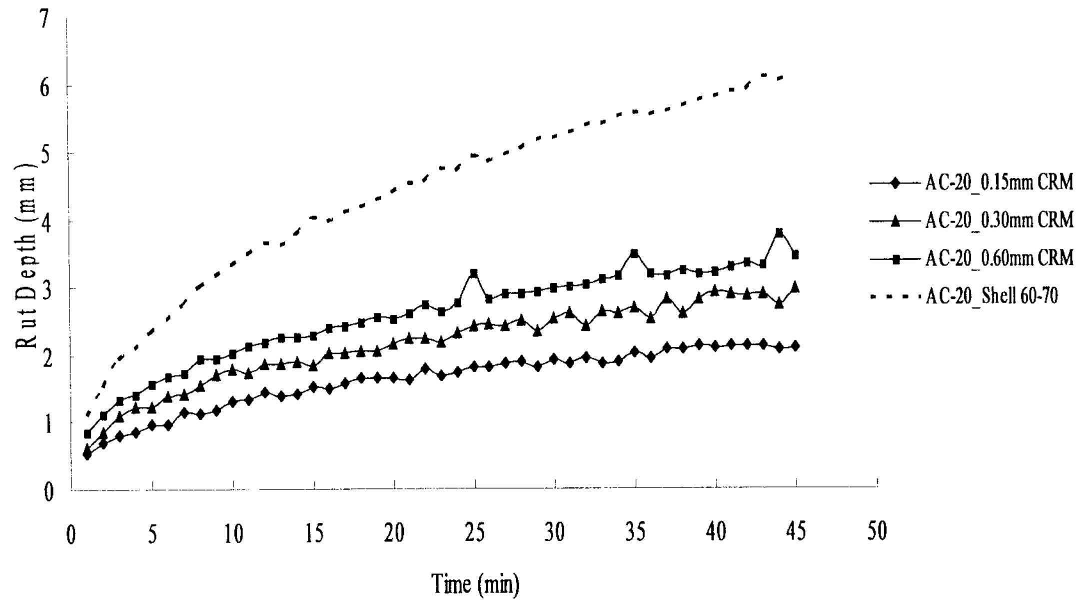


Figure 5-5 Graph of Rutting Depth Verse Time for AC-20

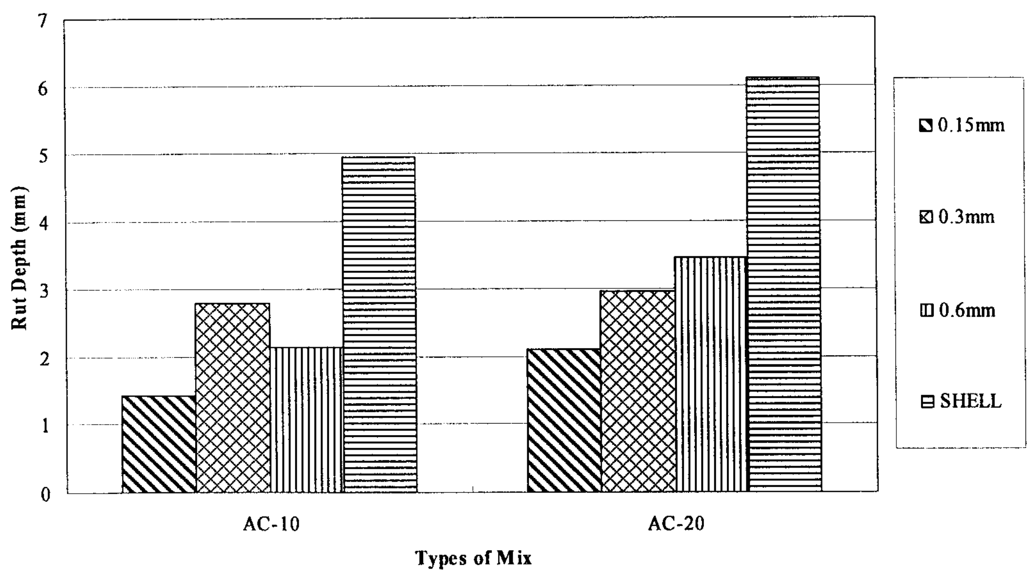


Figure 5-6 Graph of Rutting Depth Verse Time for Porous Asphalts

Major findings from the testing results are presented as follows:

1) From Figure 5-2 to Figure 5-3, the mean rut depths of the three crumb rubber

modified mixtures are less than that of the conventional mixture. It suggests that the modified mixtures tested in this study have been significantly improved the rut resistance, when compared with the unmodified mixture.

2) The Figures also indicate that specimens deform rapidly at the beginning of the test, steady at the middle and slowly at the end of the test.

3) Figure 5-2 shows that the dense-graded CRM modified mixtures of both AC-10 and AC-20 exhibit great improvement in rutting resistance compared with the unmodified mixtures when the same binder content (4.5% for AC-10 and 5.4% for AC-20) is used. In addition, AC-10 displays better rutting resistance performance than AC-20. The mixture prepared with 0.15mm CRM modified binder produces the smallest rut depth, exhibiting the highest rutting resistance.

4) The rutting rate results are shown in Table 5-17. Rutting rate for AC-20 are 4.32mm/h for the Shell 60/70 mixture, 1.61mm/h for 0.15 CRM mixture, 2.07mm/h for 0.30mm CRM mixture and 2.54mm/h for 0.60mm CRM mixture; Rutting rate for AC-10 are 3.16mm/h for the Shell 60/70 mixture, 1.22mm/h for 0.15 CRM mixture, 1.95mm/h for 0.30mm CRM mixture and 1.73mm/h for 0.60mm CRM mixture. The least rutting rate of 0.15mm CRM mixture further exhibits that it has the lowest accumulation of permanent strain and has the highest ability of rutting resistance among all the specimens.

CHAPTER 6

THE DEVELOPMENT AND THE EFFECT OF CRM ON POROUS ASPHALTS

Crumb rubber from ground tyres has been used since the late 1960's. It is primary used to reduce reflective cracking. In 1988, ADOT started to use crumb rubber mixed with hot asphalt, commonly referred to as asphalt rubber (AR) as a binder in hot mix asphalt(HMA). Typically, this mixes are either open-graded or gap-graded and from half inch to one inch or one inch to two inches in thickness, respectively. To date, many of the trial road have obtained good performance in service, particularly for the open-graded, like porous asphalt (PA). Porous asphalt is commonly used as friction course in the world as well as in Hong Kong. So in this study, crumb rubber modified binder was introduced in the local design.

6.1 The History of Porous Asphalts

6.1.1 Definition of Porous Asphalt

Porous Asphalt (PA) was originally known as "Friction Course" material when it was developed to avoid aquaplaning on military airfields; however, for use on highways, it was subsequently termed "Pervious Macadam" in the United Kingdom, "Drain Asphalt" in France, "Whispering Asphalt" in Germany and "Popcorn Mix" in the United States. Up to 1992, the term "Porous Asphalt" was widely adopted in Europe, following agreement to include this term in the draft CEN terminology for highways materials which gives porous asphalt an accurate definition that porous asphalts are bituminous bound mixtures with carefully selected grading so as to have about 20% interconnecting voids contents when fully compacted, or more with some mixtures.

6.1.2 Advantages and Disadvantages of Porous Asphalt

Up to now porous asphalt can be considered as the safest wearing course available to enhance the driving environment, reduce the risk of aquaplaning as friction course material on high speed roads and reduce noise generated on both high speed and local roads. However, there are other benefits with the principal advantages of using porous asphalt as the wearing course [Fabb, T R J] being the reduction of:

- Spray thrown up by vehicle tyres in wet conditions;
- Reflected glare at night in wet conditions; and
- Rolling resistance leading to improved fuel economy.

In addition, safety is enhanced by there being less water on the road surface (which allows better tyre/road grip) and the capacity of roads in wet conditions may be increased by improved driver visibility. Safety may also be improved in the dry because of the high texture depth of the surface. Nevertheless, porous asphalt also has disadvantages as follows:

- Increased costs;
- Possibly low structural strength, due to its high void content;
- Possibly shorter service life;

- Complications to winter maintenance procedures; and
- Maintenance patching difficulties.

Also, being relatively weak in shear, the material may be vulnerable at high stress sites, although recent modified porous asphalts should be less susceptible. Furthermore, careful consideration needs to be given to minimizing the drainage path length to allow water passing through the layer to enter the drainage system.

6.1.3 Historical Perspectives and Current Applications of Porous Asphalts

A challenge for a porous asphalt mixture is to develop a way to keep the surface drainable for a longer period of time. The durability of these open-graded mixtures depends on many important factors such as aggregate gradation, binder type, hydraulic conductivity and climatic conditions. The search for a durable superior performing porous asphalt mixture has become a major goal for the asphalt industry, particularly after some premature failures were found. Regarding historical perspectives and recent applications of porous asphalt in other parts of the world, the experience and applications of PA in the U.S, the United Kingdom, and Europe, and summarized below:

(1) USA

Open-graded friction course (OGFC) mixtures, an earlier variety of porous asphalt have been used in the United States of America since at least 1944 [Morris, G.R.]. Initially, the main benefit contributed by the material was to increase surface friction in wet weather. Later on, further benefits of reducing water spray and noise were also detected.

Unfortunately their use across the country was limited for several reasons. These mixtures require a different mix design method and special construction considerations. They also have special maintenance issues and tend to fail suddenly at the end of their design life.

Unsatisfactory experience is associated mostly with the length of service life and the failure mechanism at the end of design life. Pavement life has been reported to be 7 to 13 years, somewhat less than typical dense-graded mixtures. Functionally, the mixtures fail when voids become clogged. Once the voids are clogged, an open-graded mixture performs as a dense-graded mixture with relatively low permeability. Structurally, OGFC mixtures fail by raveling. The pavement begins to ravel and deteriorates very rapidly, often in a matter of months. In 1998, the TRB Committee on Characteristics of Bituminous-Aggregate Combinations to Meet Surface Requirements, A2D03, did conduct a survey on the application of porous asphalt in the U.S. Forty-two out of the fifty states responded. Nineteen indicated that they used OGFC mixtures. Some agencies construct more than a thousand lane-km per year, others only a few. Seven agencies constructed more than 300 lane-km per year. Another ten agencies routinely constructed some open-graded mixtures each year. The remaining agencies had either discontinued use or had never used open-graded mixtures.

Florida used only one gradation for porous asphalt. A modified binder used was composed of an AC30 bitumen with 12 % (by weight of binder) ground tyre rubber.

Several aggregates are allowed including crushed granite, blast furnace slag, crushed oolitic limestone (high friction limestone) and lightweight aggregate.

Arizona DOT specified two different porous asphalt mixtures, one for unmodified binder and the other for rubber modified binder. The bitumen for the unmodified mixture is PG 64-16. For the modified mixture, the base bitumen was PG 64-16 except in colder locations (at high altitude) where PG 58-22 was used. The binder was bitumen modified by the addition of 20 % (by weight of binder) of ground tyre rubber.

Arizona used several criteria to specify acceptable aggregate including proportion of carbonate, crushed faces, flakiness index, Los Angeles abrasion, sand equivalent, water absorption and combined bulk specific gravity. Binder content was determined by a formula that depends upon aggregate water absorption, aggregate specific gravity and proportion passing the 2.36 mm sieve. A binder drainage test was not required for the rubber-modified porous asphalt mixture because the binder is very resistant to binder drainage.

Currently in the US, changes are occurring to open-graded mixtures. Agencies are using modified binders that are less susceptible to binder drainage during construction. Modified binders are also more durable, reducing ageing and the potential for raveling. A new generation of mix designs that use larger aggregates, 12.5 mm and 16.0 mm nominal maximum sizes instead of 9.5 mm that was used previously. The larger aggregate size generates larger voids that are less susceptible to clogging.

In general the main conclusions from the U.S. experience and current application with porous asphalt are:

- Open-graded mixtures have been in use for more than 50 years, which were developed as an alternative to chip seals. The benefits from porous asphalt were the increasing in wet weather friction and the reduction of water spray and noise.
- The short life and the tendency of rapid failure have hampered the widespread use of historical open-graded mixtures.
- Modified binders, like Crumb rubber modified binder are being used to increase the life of the historical open-graded mixtures.
- European mixtures, having larger aggregate size, higher air voids contents and modified binders, are being adopted by the U.S.

(2) United Kingdom

Transport & Road Research laboratory (TRRL), on behalf of the Highways Agency in the United Kingdom, has been monitoring the performance of various trial sites since 1967. This experience has been used to assess several factors in the design and maintenance of porous asphalt [Nicholls, J C]. However, experience with porous asphalt has also taken place elsewhere in the world and the conclusions from the UK is relatively general and can be usefully supplemented with experience elsewhere.

From 1967 to 1991 there were records of 28 road trials with a total of 83 sections of different variations of porous asphalt in the U.K. The trials have included a range of constituent materials including:

- 20 mm, 14 mm and 10 mm maximum nominal aggregate sizes;
- Various grades of unmodified bitumen;
- Binder contents between 3.2 % and 5.7 %; and
- Different polymer, fiber and other modifiers.

Since the durability of porous asphalt can be expressed in terms of the time for which the material effectively reduces spray during wet weather and noise during all weather conditions and also enhances the structural integrity, wide variation has been observed on the UK road trials in both noise reducing and the ultimate life of porous asphalt. The following equations have been developed from multiple regression analyses for data from the UK trial sections for which there are estimates of the ultimate and noise-reducing life.

$$L_U = 0.6B + M + 1.4\times10^{-3}P + 0.4A + 0.5\times10^{-4}T - 3.3 \qquad (R_{adj}^2 = 0{,}34)$$

$$L_N = -2.3B + 1.5M + 3\times10^{-3}P + 0.2A - 0.6\times10^{-4}T + 11 \qquad (R_{adj}^2 = 0{,}38)$$

where: LU = ultimate life (years)

LN = noise-reducing life (years)

T = traffic intensity (cv/d)

A = nominal size of aggregate (mm)

P = penetration of the base bitumen (mm/10)

M = modified (1) or unmodified (0) binder

B = proportion of binder in the mixture by mass (%)

Radj = correlation coefficient adjusted for the degrees of freedom.

These equations show that, in terms of the potential life of porous asphalt:

- Traffic has only a small effect (beneficial for the ultimate life and detrimental for the spray-reducing life);
- The use of larger nominal sizes of aggregate and of softer grades of bitumen are both beneficial;
- The inclusion of modifiers appears to be beneficial over and above the increased binder content they permit; and
- An increase in binder content extends the ultimate life but reduces the spray-reducing life.

Aggregate

All the aggregate types used in the UK road trials are strong and have a low flakiness index. Such aggregates are selected because in porous asphalt:

- Tyre-induced stresses are applied to relatively few point-to-point contact areas between the essentially single-sized coarse aggregate skeleton; and
- Relatively cuboid aggregates will provide good drainage and enhance the potential spray-reducing life.

The UK Specification for Highway Works [Y. Swyddfagymreig] requires the coarse aggregate in porous asphalt to have:

- A minimum 10 percent fines value of 180 kN;
- A maximum aggregate abrasion value of 12;
- A maximum flakiness index of 25;
- A minimum polished stone value dependent on the design traffic intensity;
- A maximum aggregate impact value of 30 %; and
- A minimum magnesium sulphate soundness value of 75.

The aggregates in porous asphalt are gap-graded such that they consist of coarse aggregate bound with a fine mortar. The proportions of aggregate in the smaller fractions are restricted to avoid choking the porous asphalt. For 20 mm porous asphalt, the coarse aggregate is in the 20 mm to 14 mm fraction and the gap is in the 10 mm to 6.3 mm fraction. With a limit on the proportion passing the 6.3 mm sieve of 25 ± 5 % by mass and limiting the proportion passing the 10 mm sieve to 35 ± 5 %, it ensures a gap because only about 10 % (maximum of 20 %) of the aggregate can be in that fraction.

An analysis of all the trials shows that:

- The average ultimate life of porous asphalt was:

  -5 years (with a range of 1 to 12 years) for 10 mm aggregate mixtures; and

  -8 years (with a range 0 to 15 years) for 20 mm aggregate mixtures;

- The spray-reducing life of porous asphalt was:

-4 years (with a range 2½ to 6½ years) for 10 mm aggregate mixtures; and

-6½ years (with a range 2 to 8 years) for 20 mm aggregate mixtures.

Hence, the trials showed that the 20 mm porous asphalt grading, particularly when using modified binders at higher binder content, is a compromise that increases durability at the expense of some loss of hydraulic conductivity, and hence spray-reducing life.

Binder

High binder contents improve the durability of trial porous asphalt by providing a thicker binder film, but they reduce the hydraulic conductivity by filling the pores. The maximum binder content is limited by the tendency of binder draining from the mixture, which can result in areas of the finished mat either binder-rich or lean and lacking in fines; the binder-rich areas will have inadequate hydraulic conductivity while the binder-lean areas may be prone to premature fretting. The binder drainage test, used to identify the maximum quantity of a particular binder that can be mixed with a particular aggregate to the required grading without significant drainage, was developed to deal with this problem [Daines, M E]. The test was first used on a series of trials in 1987 that showed that the most satisfactory target binder content, around 4.5 % for 20 mm nominal size aggregates, was difficult to achieve with unmodified bitumen without drainage. The experience from the UK shows that the use of high penetration bitumen enhances durability at the expense of earlier closing-up of the surfacing. The extra durability is related to the time taken for the binder to harden to the critical condition, when it can no longer accommodate the traffic-induced strains at low temperatures.

Modifier

According to the results of the laboratory study of TRL in the UK, Binder modifiers may be used to increase the binder content that can be incorporated into the mixture without binder drainage occurring. These modifiers include both polymer-modifiers, which alter the rheology of the bitumen, and fibres, which increase the surface area over which the bitumen spreads. Polymer-modifiers can be regarded as part of the binder. Polymer modifiers that have been tried such as natural rubber, styrene-butadienestyrene (SBS) block copolymer, ethylene vinyl acetate (EVA), epoxy resin and hydrated lime.

Although it has been a long time that extensive study of porous asphalt had been conducted, current use of porous asphalt in the UK is not satisfactory. The UK has taken a considerable time to gain assurance on the durability of porous asphalt before its application was accepted for routine use on the trunk road network. However, at about the same time that the material became acceptable, thin surfacing materials were introduced into the country and were accepted for general use. Since thin surfacing provides the same advantages as porous asphalt, even if not quite to the same extent, but at a lower cost and with better durability. Thus, there are not many schemes using porous asphalt. Furthermore, two prominent schemes that did use porous asphalt had problems with durability and the early replacement of the surfacing. This had caused a further impact on the general use of porous asphalt.

(3) Europe

European agencies use modified binder exclusively that is less susceptible to binder drainage during construction. Binder drainage, a separation of the binder/fine aggregate mastic from the coarse skeleton, can occur in a mixture storage silo or in the truck during transport. Mixtures that have suffered from binder drainage produce binder rich areas on the road that have a flushed surface and no voids. However, other areas with little binder and high voids may result in quicker raveling. Modified binders are also less susceptible to binder drainage during service. Although not documented in research, field experience indicates that the thick films of unmodified bitumen tend to drain downward with time in hot summer weather. The remaining thin films on the surface particles age and become brittle more rapidly. When the binder becomes sufficiently brittle, aggregate particles are dislodged by traffic and the layer ravels. Modified binders retain film thickness, thereby reducing ageing and stone loss.

Porous European Mix (PEM) mixtures have higher air voids than American OGFC mixtures. Continued benefit from an open graded mixture is dependent on the void structure remaining open. Clogging from road debris and winter sanding nullifies the mixture’s benefits. Increasing the voids to 20 % or more provides more resistance to clogging. Larger voids tend to be cleaned by hydraulic action of traffic during rainfall, particularly on high speed pavements.

6.2 Porous Asphalts Design

6.2.1 Porous Asphalts Gradation

The same gradation of Anderson's mix design specification [Anderson, Hong Kong] was used to prepare different porous asphalts of the three binders including conventional Shell 60/70 binder, conventional polymer modified binder and pre-blend polymer modified binder presented in Table 6-1.

Sieve Size (mm)	14.0	10.0	5.0	2.36	1.18	0.6	0.3	0.15	0.075
Percent Passing (%)	100	96	25	15	-	-	-	-	3.3

Table 6-1 Mixture Proposed Aggregate Gradation

At the same time as entered on the Anderson's mix design specification, weight ratio of 5mm to sand was approximately 7 to 3, so the aggregates less than 5mm size were partially replaced by the sand.

The mineral filler used in the four porous asphalts was lime and aggregate fines. The respective corresponding ratio to the total aggregate mass was 1.5 percent and 1.8 percent. Although the same aggregate gradation was used in the four porous asphalts, the optimum binder contents and air voids were different for them, which were listed in Table6-2.

Mixture Type	Shell60/70	**CRM**
Optimum Binder Content (%)	4.7	5.1
Air voids (%)	19.1	18.6

Note: Shell60/70 – denotes porous asphalt made of conventional 60/70 binder

CRM–denotes porous asphalt made of CRM modified binder

Table 6-2 Values of Optimum Binder Content

In order to validate the optimum binder content, another design was set by increasing the binder content from 5.1% to 5.7% (.5.7% was used here as take reference to the SBR mixture design from a local producer). It was used as the control and was listed in Table 6-3.

Mixture Type	Shell60/70	**CRM**
Optimum Binder Content (%)	4.7	5.7
Air voids (%)	19.1	18.6

Note: Shell60/70 – denotes porous asphalt made of conventional 60/70 binder

CRM–denotes porous asphalt made of CRM modified binder

Table 6-3 Values of Optimum Binder Content

6.2.2 Aggregate Properties

The specific gravity of aggregates was tested in accordance to the needed sizes with reference to the gradation of three porous asphalts. The bulk specific gravity and the apparent specific gravity were tested for the coarse aggregates; only the apparent specific gravity was tested for the fine aggregates. The testing results of the bulk specific gravity and the apparent specific gravity of the coarse aggregates and the apparent specific gravity of the fine aggregates are given in Table6-4.

Sieve Size(mm)	Apparent Specific Gravity (g/cm^3)	Bulk Specific Gravity (g/cm^3)
10	2.624	2.565
5	2.616	2.552
2.36	2.600	2.517
1.18	2.603	
0.6	2.599	
0.3	2.590	
0.15	2.613	
0.075	2.641	
<0.075	2.658	

Table 6-4 Specific Gravity of Porous Asphalt Aggregate

According to data provided by Anderson [Test Procedure A AL/LAB/07], the apparent specific gravity of the lime and the bulk specific gravity of the sand were 2.63 g/cm^3 and 2.58 g/cm^3 respectively.

6.3 Specimens Preparation

Laboratory batched specimens with three different binders, including the conventional Shell60/70 binder, and the three crumb rubber modified binders, were prepared according to the specified grading. Superpave Gyratory Compactor (SGC) was used to fabricate testing specimens in accordance with ASTM D 3387-83 [ASTM D 3387]. The SHRP gyratory compactor effectively simulates the compaction process of the field mixtures to ensure that engineering properties of laboratory compacted specimens are equivalent to those of in-place paving mix. It permits real-time determination of bulk specific gravity and air voids content during compaction. The gyratory compactor can produce cylindrical testing specimens of paving mix, diameter 150 mm or 100mm, through a combination of vertical consolidation pressure and gyratory kneading effort. The unit operates at an angle of gyration of 1.25°, a speed of 30 gyrations per minute, and a vertical ram pressure of 0.6 MPa. In this project the controlled mode of bulk specific gravity of porous asphalt was applied during the course of compacting.

6.4 Testing result

The results of the wheel tracking test for the porous asphalt of the four binders (the conventional shell 60/70, and the three crumb rubber modified binder) are tabulated in Tables 6-5, 6-6, 6-7 and 6-8.

mins	def(mm)	mins	def(mm)
1	0.71	24	4.59
2	1.02	25	4.74
3	1.45	26	4.79
4	1.76	27	4.94
5	2.14	28	5.21
6	2.4	29	5.25
7	2.41	30	5.36
8	2.61	31	5.5
9	2.85	32	5.7
10	2.8	33	5.79
11	2.96	34	5.92
12	3.05	35	6.08
13	3.18	36	6.21
14	3.34	37	6.3
15	3.45	38	6.37
16	3.65	39	6.5
17	3.7	40	6.61
18	3.89	41	6.72
19	4.03	42	6.79
20	4.12	43	6.92
21	4.43	44	6.97
22	4.3	45	7.05
23	4.61		

Table 6-5. Wheel Tracking Test Result of Porous Asphalt (4.7%_shell 60/70)

mins	def(mm)	mins	def(mm)
1	0.86	24	2.98
2	1.05	25	2.99
3	1.21	26	3.11
4	1.35	27	3.08
5	1.51	28	3.21
6	1.63	29	3.24
7	1.71	30	3.23
8	1.88	31	3.33
9	1.87	32	3.3
10	2.03	33	3.44
11	2.16	34	3.48
12	2.19	35	3.45
13	2.21	36	3.57
14	2.34	37	3.56
15	2.36	38	3.58
16	2.45	39	3.66
17	2.52	40	3.7
18	2.58	41	3.82
19	2.69	42	3.92
20	2.75	43	3.93
21	2.8	44	3.95
22	2.87	45	4
23	2.93		

Table 6-6. Wheel Tracking Test Result of Porous Asphalt (5.1%_0.60mm CRM)

mins	def(mm)	mins	def(mm)
1	0.94	24	3.36
2	1.17	25	3.42
3	1.38	26	3.5
4	1.54	27	3.57
5	1.74	28	3.65
6	1.83	29	3.71
7	1.95	30	3.77
8	2.09	31	3.82
9	2.2	32	3.91
10	2.29	33	3.97
11	2.39	34	4.05
12	2.45	35	4.11
13	2.54	36	4.16
14	2.66	37	4.21
15	2.74	38	4.32
16	2.79	39	4.35
17	2.87	40	4.46
18	2.94	41	4.51
19	3.02	42	4.59
20	3.09	43	4.62
21	3.17	44	4.61
22	3.23	45	4.71
23	3.29		

Table 6-7. Wheel Tracking Test Result of Porous Asphalt (5.1%_0.30mm CRM)

mins	def(mm)	mins	def(mm)
1	0.93	24	0.93
2	1.29	25	1.29
3	1.48	26	1.48
4	1.66	27	1.66
5	1.77	28	1.77
6	1.8	29	1.8
7	2.02	30	2.02
8	2.19	31	2.19
9	2.19	32	2.19
10	2.46	33	2.46
11	2.66	34	2.66
12	2.58	35	2.58
13	2.82	36	2.82
14	2.78	37	2.78
15	2.97	38	2.97
16	3.1	39	3.1
17	3.04	40	3.04
18	3.23	41	3.23
19	3.32	42	3.32
20	3.44	43	3.44
21	3.43	44	3.43
22	3.53	45	3.53
23	3.6		

Table 6-8. Wheel Tracking Test Result of Porous Asphalt (5.1%_0.15mm CRM)

mins	def(mm)	mins	def(mm)
1	0.86	24	2.98
2	1.05	25	2.99
3	1.21	26	3.11
4	1.35	27	3.08
5	1.51	28	3.21
6	1.63	29	3.24
7	1.71	30	3.23
8	1.88	31	3.33
9	1.87	32	3.3
10	2.03	33	3.44
11	2.16	34	3.48
12	2.19	35	3.45
13	2.21	36	3.57
14	2.34	37	3.56
15	2.36	38	3.58
16	2.45	39	3.66
17	2.52	40	3.7
18	2.58	41	3.82
19	2.69	42	3.92
20	2.75	43	3.93
21	2.8	44	3.95
22	2.87	45	4
23	2.93		

Table 6-9. Wheel Tracking Test Result of Porous Asphalt (5.7%_0.60mm CRM)

mins	def(mm)	mins	def(mm)
1	0.94	24	3.36
2	1.17	25	3.42
3	1.38	26	3.5
4	1.54	27	3.57
5	1.74	28	3.65
6	1.83	29	3.71
7	1.95	30	3.77
8	2.09	31	3.82
9	2.2	32	3.91
10	2.29	33	3.97
11	2.39	34	4.05
12	2.45	35	4.11
13	2.54	36	4.16
14	2.66	37	4.21
15	2.74	38	4.32
16	2.79	39	4.35
17	2.87	40	4.46
18	2.94	41	4.51
19	3.02	42	4.59
20	3.09	43	4.62
21	3.17	44	4.61
22	3.23	45	4.71
23	3.29		

Table 6-10. Wheel Tracking Test Result of Porous Asphalt (5.7%_0.30mm CRM)

mins	def(mm)	mins	def(mm)
1	1.22	24	4.05
2	1.61	25	4.23
3	1.82	26	4.21
4	2.1	27	4.36
5	2.25	28	4.41
6	2.43	29	4.44
7	2.55	30	4.58
8	2.67	31	4.62
9	2.77	32	4.71
10	2.86	33	4.81
11	2.91	34	4.8
12	3.06	35	4.94
13	3.18	36	4.98
14	3.14	37	5.08
15	3.3	38	5.16
16	3.38	39	5.14
17	3.42	40	5.3
18	3.52	41	5.39
19	3.65	42	5.46
20	3.71	43	5.48
21	3.84	44	5.58
22	3.98	45	5.61
23	4.06		

Table 6-11. Wheel Tracking Test Result of Porous Asphalt (5.7%_0.15mm CRM)

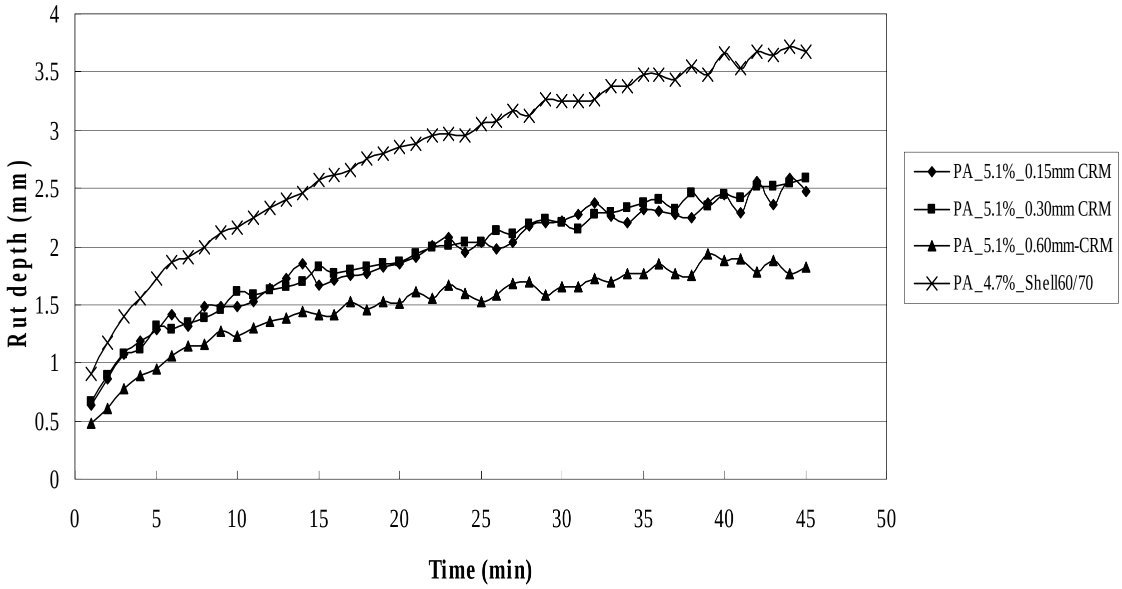


Figure 6.1. Rut depth verse Time Of Wheel Tracking Test

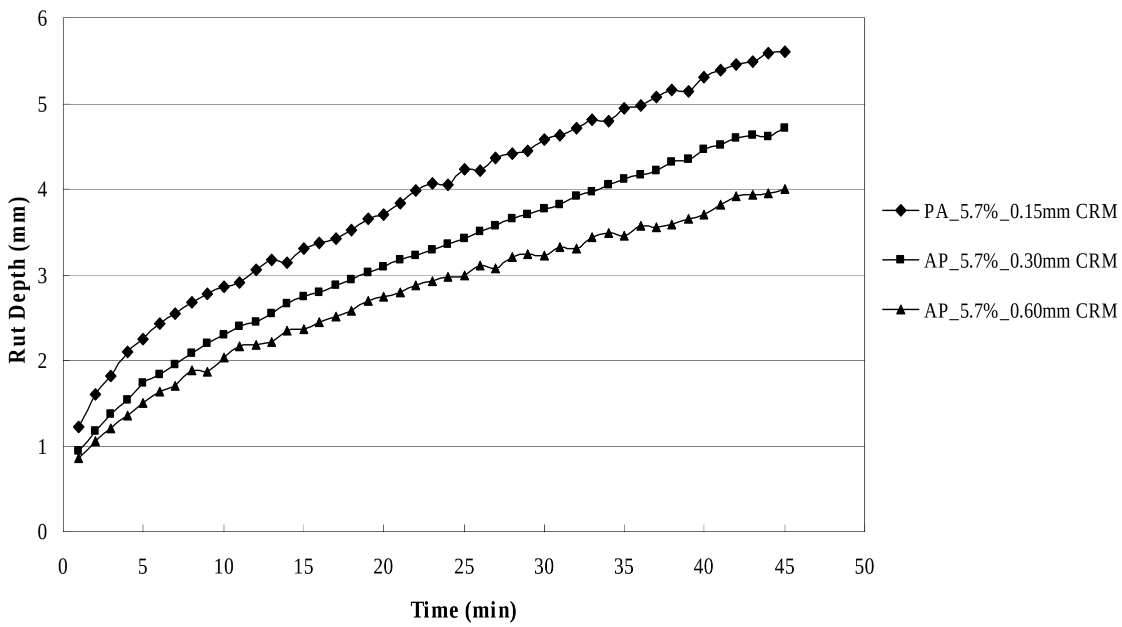


Figure 6.2. Rut Depth Verse Time of Wheel Tracking Test

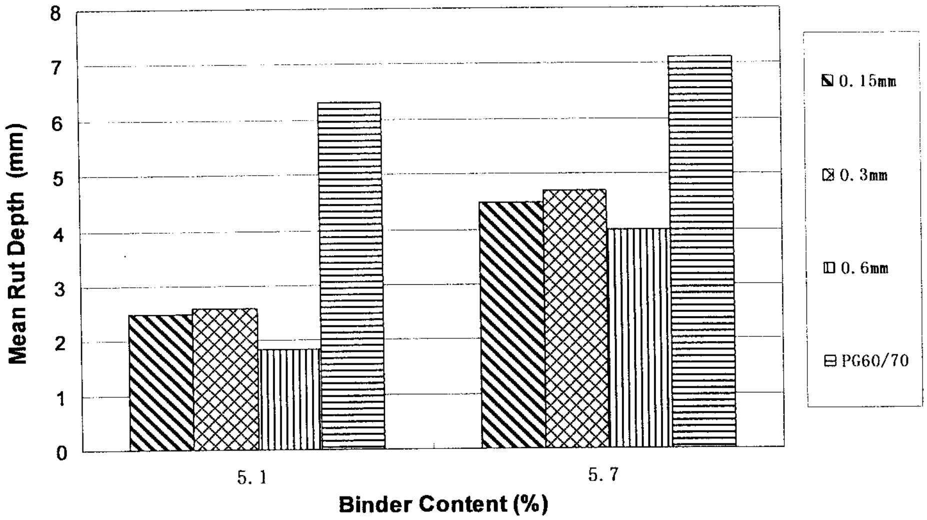


Figure 6.3. The Effect Of PA Mixtures with Different Modified Binders On Mean Rut Depth

Specimen Reference		Test Temperature (^{0}C)	Rutting Rate (mm/hr)	Rut Depth (mm)
Porous Asphalt	Shell 60/70	60	6.72	7.05
	0.15mm CRM	60	5.81	4.71
	0.30mm CRM	60	3.81	4
	0.60mm CRM	60	3.07	3.53

Table 6.5 Testing Results of Rutting Rate and Rutting Depth of PA

Conclusion

Based on the data above, the following findings can be drawn:

1) The mean rut depth of the three crumb rubber modified mixture is less than that of the conventional mixture. Results indicate that the modified mixtures used in this study have significant improvement on the resistance against rutting

2) The mixture modified by 0.60mm CRM has shown to have the smallest rut depth and thus the performance is considered to be the best among all the mixtures, while the performance of the other two modified mixtures which modified by 0.30mm and 0.15mm CRM has shown nearly the same performance.

3) The rut depth of the control design with 5.7% binder content is much large than the optimum design.

Conclusion can be drawn that for a mixture of PA, the performance of high temperature susceptibility can be significantly improved by incorporating 5.1% CRM modified binder (with 10% CRM by weight of total binder) into the mixture especially for the binder modified by 0.60mm CRM.

CHAPTER7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Overall Conclusion

7.1.1 Literature Review

The literature review provides an overview of the terminology, process, products, and applications of crumb rubber modifier (CRM) technology. Specifically, it reviews the historical perspectives and the current applications in the United States. The technology includes the use of scrap tyre rubber asphalt paving materials. The following conclusions are drawn:

(a) There are three methods currently used to process scrap tyre rubber into CRM. There are the crackermill process, the granulator process and the micromill process. The crackermill process is the most common method used.

(b) In general, CRM technology can be divided into two categories, the wet process and the dry process. The term wet process defines any method that blends crumb rubber modifier with asphalt cement before incorporating the resultant binder into aggregates, the term dry process defines those methods that mix the crumb rubber with the aggregates before the mixture is charged with asphalt cement. Past experiences have shown that dry process applications exhibited unstable performance, ranging from acceptable to disastrous. The main reason for the fluctuated performance of dry process is claimed to be owing to the poor control in meshing the gradation of the aggregate and crumb rubber and the lack of understanding of the volume changes taking place due to swelling of the crumb rubber during the mixing process and handling.

(c) In 1968, the Arizona Department of Transportation (ADOT) placed its first stress absorbing membrane (SAM), a surface treatment using an asphalt rubber binder. ADOT placed their first stress-absorbing membrane interlayer (SAMI) in 1972 and used the asphalt rubber binder in HMA open-graded friction course in 1975. The dry process was developed in the late 1960s in Sweden. The European trade name for this HMA mixture with CRM as a rubber aggregate was Rubit. The Swedish technology was patented for use in the United States in 1978 under the trade name PlusRide.

(d) In 1992, the United Stated Congress passed the Intermodal Surface Transportation Efficiency Act (ISTEA) to mandate the use of CRM asphalt concrete beginning on January 1, 1994 for any state require funds, According to the ISTEA, all states were required to use increasing percentage of rubber in the construction of asphalt concrete pavements (5, 10, 15 and 20% by weight) over the next following years (starting from 1994). But after a few years of implementing the ISTEA, some states claimed that this federal law was a bit severe as some states didn't even have enough experiences for using CRM technology for national standards on mix design procedures and performance criteria were lacking, also, some engineers questioned about the consistent of the advantages of this technology.

(e) The Open Graded Friction Course (OGFC) was first used by the Arizona Department of Transportation (ADOT) in 1954. Most OGFC projects which used modified crumb rubber modified binder have been reported performing well. Nowadays, crumb rubber modified open graded friction course is generally used as the final wearing surface for both rigid and flexible pavement to improve smoothness, reduce cracking and rutting, enhance skid resistance and reduce noise.

(f) In general, the initial cost for asphalt rubber mixes can be 30 to 80% higher than that of the conventional mixes. Extensive experience with asphalt rubber has been shown to be a very cost-effective binder for pavement maintenance and rehabilitation strategies when properly produced and constructed. The main reasons for the overall cost of the asphalt rubber project is less than that of the conventional project are the reduction in thickness and the lowering of the maintenance cost in long run.

(g) Discarding of rubber tyre has becoming a worldwide environmental concern. Significant number of wasted tyres can be reduced by using Asphalt-Rubber in HMA construction and highway maintenance activities. According to the Rubber Pavements Association, a two-inch thick overlay of asphalt-Rubber Hot Mix will use about 2000 tyres per lane per mile.

(h) Despite the past successful experiences, it seems that further research is still needed in the field of crumb rubber modified mixture design and a comparative study involving different crumb rubber modified contents, different sizes of the crumb rubber modifiers and different kinds of recycled tyres by evaluating the mechanical properties and durability performance of crumb rubber modified asphalts in terms of rutting, stiffness modulus and moisture damage, etc. Designs taking into account of the climate and traffic conditions such as the Superpave Performance Grade bitumen system are required.

(i) Crumb rubber modified asphalt is regarded as one of the superior paving material for both friction course and wearing course materials. Due to its outstanding properties on high temperature susceptibility, water susceptibility and noise reduction, as well as solving the environmental problem created by disposal of waste tyres, therefore, effort towards its future development is worthwhile.

7.1.2 Test of CRM Binder

Four evaluation methods, penetration test, softening point test, viscosity test and DSR test, were performed on each condition of the four types of binders both before ageing and after ageing. Particular points of the test results are noted as follows:

7.1.2.1 Result of Penetration Test

- Penetration values of crumb rubber modified binders at 25°C are small than original 60/70 binder, but there is no significant difference among the CRM binders.
- Penetration of the crumb rubber modified binders at 4°C were found to be much

higher than that of the original 60/70 binder whereas at 25°C, original 60/70 binder had higher penetration value. The crumb rubber modified binders, therefore, are expected to maintain flexibility at low temperatures without being soft at higher temperatures. Also, among those three crumb rubber modified binders, penetration ratio (4°C /25°C) of 0.60mm CRM is less than those of 0.30mm and 0.15mm CRM, but the differences in the penetration ratio (4°C/25°C) of 0.30mm CRM binder and 0.15mm CRM binder are not very significant.

7.1.2.2 Result of Softening Point test

- For all the four binder's softening point values, the unaged condition are smaller than those after the aged. It indicates that binders tend to harden with binder ageing going on.
- At unaged condition the three crumb rubber modified binders have much higher softening point than that of the conventional Shell 60/70 binder. The softening point of 0.60mm CRM binder is the highest among the four binders.
- Results show that 0.15mm CRM binder exhibits the lowest increased percentage among the four studied binders after short-term ageing, as for the long-term ageing, 0.60mm CRM exhibits the lowest increased percentage among the four studied binders. The rank is followed by 0.15mm CRM, 0.30mm CRM and the conventional 60/70 binder. Therefore, crumb rubber modified binders have a better ability of

ageing resistance.

7.1.2.3 Result of Viscosity Test

- All of the three crumb rubber modified binders have lower temperature susceptibility than the conventional Shell 60/70 binder.
- According to the requirement of SHRP performance-based binder specification, SHRP A410, in order to warrant binders to have enough workability during construction phase, viscosity value of any binder at 135°C should be less than 3Pa.s. The requirement is satisfied by the four studied binders.
- Viscosity values of the three studied binders at RTFOT ageing condition are higher than those at unaged condition; however, it is the contrary in PAV condition. The main reason for this is that RTFOT ageing causes binder hardening with no change in component ingredients, but PAV ageing changes binder ingredients and reduces binder viscosity sharply.

7.1.2.4 Result of DSR Test:

- It indicates that the ageing condition has significantly influence on the rutting factor G/sin(δ) whereas no significant influence on G/sin(δ) among these three CRM modified binders is detected.
- The crumb rubber modified binders have the same performance grade, PG76 and are one grade higher than the conventional Shell 60/70 binder, PG70. It means that the three crumb rubber modified binders are less temperature susceptible and may be used

in a relative hot climate area.

- All of the three crumb rubber modified binders have smaller G*sinδ than the conventional Shell 60/70 binder. It indicates that crumb rubber modified binders, especially 0.30mm crumb rubber modified binder, performs well with respect to fatigue resistance.

7.1.3 Test Results of CRM Mixtures

The performance of the three types of porous asphalts is judged by the value of stiffness modulus and their ability to resist water damage, permanent deformation distresses and rutting. The following key conclusions are drawn:

7.1.3.1 Result of Moisture Damage Test

- The results of water damage of the four mixtures indicate that the 0.15mm crumb rubber modified mixture has the best ability of resistance to water damage shown by the higher values in ITSR and ITSMR before and after the water treatment.
- The mixture with 0.15mm CRM has the greatest stiffness value among the four mixtures. It indicates that crumb rubber modifier can strengthen the stiffnesses of mixtures, particularly for 0.15mm CRM mixture.
- The indirect tensile strength (ITSR) of all the three CRM mixtures are over 100%, it indicated that CRM mixtures should exhibit less susceptibility to moisture damage.

7.1.3.2 Result of Wheel Tracking Test

- For porous asphalt (PA) mix, the performance of high temperature susceptibility can be significantly improved by incorporating 5.1% CRM modified binder (with 10% CRM by weight of total binder) into the mixture especially for the binder modified by 0.60mm CRM.
- Both crumb rubber modified AC-10 and AC-20 exhibit great improvement in rutting resistance compared with the unmodified mixtures when the same binder content (4.5% for AC-10 and 5.4% for AC-20) is used. In addition, AC-10 displays better rutting resistance performance than AC-20. The mixture prepared with 0.15mm CRM modified binder produces the smallest rut depth and it exhibits the best high temperature resistance.
- The least rutting rate of 0.15mm CRM mixture further exhibits that it has the lowest accumulation of permanent strain and has the highest ability of rutting resistance with loading time going on among all the specimens.

7.2 Recommendation

The current findings indicates that crumb rubber modified binder and mixtures prepared by using the same mixing process (wet process), the same content (10%) and the three different sizes (0.15mm, 0.30mm and 0.60mm) have shown to have improved performance for resistance of high temperature susceptibility and moisture susceptibility. As only crumb rubber particle sizes has concerned in this study. Further studies are needed to be conducted in order to get a better understanding of this modified material. The recommendations are as follows:

- The influence on high temperature resistance of incorporating higher percentages (15%, 20% and 25%) of crumb rubber into wearing course mixes.
- The influence of different sources of waste tyres on performance of the crumb rubber modified binder.
- The mechanisms of chemical aspects of CRM binder production are considered particularly on the depolymerization and devulcanization activities. As the performance of the modified binder is hinged on the fusion between crumb rubber modifier and the asphalt cement.
- It is prudent that study should be carried out to evaluate fatigue behaviour and moisture susceptibility performance on CRM mixtures.
- The influence on low temperature resistance after incorporation the crumb rubber modifier into wearing course mixtures is recommended to be studied.
- Effects of CRM on the other mix type of wearing course, like SMA.
- Trial road section is recommended to be paved in Hong Kong to obtain real performance results on site.
- To compare its performance against polymer modified asphalt and see whether there is a clear advantage in CRM asphalt over polymer modified.
- The economics and logistics of locally producing and supplying crumb rubber

modifier and CRM asphalt need to be thoroughly evaluated to establish the commercial viability.

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