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DESIGN OF A CRUMB RUBBER MODIFIED ASPHALT PAVEMENT USING THE STONE MASTIC ASPHALT CONCEPT

by

Robert Veizer

A thesis submitted to
the Faculty of Graduate Studies and Research
in partial fulfillment of the requirements
for the degree of

Master of Engineering*

Department of Civil and Environmental Engineering
Carleton University
Ottawa, Canada

© Robert Veizer, November, 1993

- The Master of Engineering in the Civil and Environmental Engineering Program is a joint program with the University of Ottawa, administered by the Ottawa-Carleton Institute for Civil Engineering.

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**'Design of a Crumb Rubber Modified Asphalt Pavement using
the Stone Mastic Asphalt Concept'**

Submitted by

ROBERT VEIZER, B.A.Sc.

**in partial fulfilment of the requirements for
the degree of Master of Engineering**

**Dr. A.G. Razaqpur, Department of Civil and Environmental
Engineering**

Dr. O.J. Svec, Thesis Supervisor

Dr. Ralph Haas, External Examiner

**Carleton University
Ottawa, Ontario, Canada
November 30, 1993**

Abstract

One serious environmental problem related to the transportation field is the stockpiling of old rubber tires. Several huge old tire fires have already occurred in Canada and the USA and have caused considerable air and possibly soil pollution. Therefore, it is of great importance to develop techniques to recycle this potentially valuable material, such as by incorporating it into asphalt concrete.

A detailed program of literature review, Marshall mix design, and performance testing was carried out in order to investigate the effectiveness of adding crumb tire rubber into asphalt pavements. The objective of this research was to develop an asphalt rubber pavement using the stone matrix asphalt concept.

The results of the literature review suggested that adding crumb tire rubber to asphalt cement is a very promising endeavour and that the resulting material may provide improved capability of resisting rutting and cracking failure.

A Marshall mix design program effectively selected the optimum mix of each SMA mix design. The experimental results of the performance testing clearly showed that the rubber modified SMA mixes are superior to the conventional SMA mixes. The large aggregate mix results were not as promising but can be improved with slight modifications.

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List of Symbols and Abbreviations

ARPG	:	Asphalt Rubber Producers Group
BRD	:	Bulk Relative Density
CRAC	:	Chunk Rubber Asphalt Concrete
CRREL	:	Cold Regions Research and Engineering Laboratory
CRM	:	Crumb Rubber Modified
CSTT/NRC	:	Centre for Surface Transportation Technology / National Research Council of Canada
EPA	:	Environmental Protection Agency
GTR	:	Ground Tire Rubber
ISTEA	:	Intermodal Surface Transportation Efficiency Act
ITS	:	Indirect Tensile Strength
K	:	Rate of Permanent Deformation for RUCT
LA	:	Large Aggregate
LAM	:	Large Aggregate Mix
LVDT	:	Linear Variable Displacement Transducer
HMA	:	Hot Mix Asphalt
M_R	:	Resilient Modulus

MTO	:	Ministry of Transportation of Ontario
MTS	:	Material Test System
NAPA	:	National Asphalt Pavement Association
N_f	:	Number of Cycles at Fatigue Failure
PARC	:	Porous Asphalt Rubber Concrete
PMA	:	Polymer Modified Asphalt
RUCT	:	Repetitive Uniaxial Compression Test
SAM	:	Stress Absorbing Membrane
SAMI	:	Stress Absorbing Membrane Interlayer
SHRP	:	Strategic Highway Research Program
SMA	:	Stone Matrix Asphalt, Spiittmastixasphalt
TDF	:	Tire Derived Fuel
VMA	:	Voids in Mineral Aggregate

Chapter 1

INTRODUCTION

1.1 The Scrap Tire Disposal Crisis

Every year the industrialized nations generate more than nine million tons of scrap tires. More than 200 million waste tires are currently found in stockpiles across Canada with this number increasing by more than 20 million tires per year. In Ontario, six million passenger tires plus one million truck tires are scrapped each year. In the United States, approximately 285 million tires are discarded. Of this figure, 33 million are retreaded and 22 million are reused. Forty two million additional tires are used in various alternative manners. The remaining 188 million scrap tires are discarded in stockpiles, landfills, or illegal dumps (Heitzman 1993). This procedure produces enormous strains on already overloaded landfills and threatens the environment with fire and health hazards. Used tire stockpiles are an excellent breeding ground for mosquitos and an ideal habitat for vermin. Rubber from old automobile tires, however, can become valuable material for other uses. Every effort should be made to explore possibilities in civil engineering and other fields. Numerous other projects are being carried out, seeking ways to reduce

this solid waste problem. Promising research into areas such as tire-derived fuel (TDF), lightweight embankments, slope stability and retaining walls, and rubber modified asphalt give hope to reducing discarded tires as a solid waste problem. The remoulding of crumb tire rubber into mud guards, floor mats, carpet padding, or other rubber products also helps in reducing tire stockpiles. Of the available disposal methods, only fuel for combustion and crumb rubber modifier (CRM) for asphalt paving, have shown the potential to use a significant number of scrap tires. This thesis addresses the solid waste problem by developing a new crumb rubber asphalt material.

1.2 Utilization Technologies for Scrap Tires

Most discarded truck and automobile tires go into landfills and since they do not disintegrate with time, they remain a continual problem. A report sponsored by the National Asphalt Pavement Association (Hughes 1993) outlines interesting new technologies for scrap tire usage as an energy resource in civil engineering applications, and as crumb rubber modified (CRM) asphalt pavement.

1.2.1 Tire-Derived Fuel (TDF)

Tire derived fuel has a heating value of 9.68 Kw-hrs per kg (15000 Btu per pound), which is higher than most types of coal. In a report prepared for the Environmental Protection Agency (EPA), the potential energy source of 2.05×10^7 Mw-hrs per year (0.07 quadrillion Btu per year) that would be furnished by the roughly 242 million waste tires per year "...is equivalent to 12 million barrels of crude oil..." (EPA,

1991). With proper emission controls, burning tires for fuel can be a clean and environmentally sound method of disposing of a difficult waste. Current uses of TDF are in cement kilns, electric utilities, tire-to-energy facilities, and pulp/paper mills. The report has assessed the current and five-year potential usage by industry as shown in Table 1.1. (Hughes 1993).

From an environmental standpoint, the burning of tires in cement kilns, boilers at electric power plants, dedicated tires-to-energy facilities, or pulp and paper mills, has little adverse affect on pollution and at worst is better than coal emissions.

Creating tire derived fuel has been shown to be economical with great potential cost savings. For example, in electric utilities the cost of the TDF ranges from zero to \$1.13/Mw-hr (\$0.33/million Btu's). The cost of coal is between \$6.16 and \$6.85/Mw-hr (\$1.80 and \$2.00/million Btu's).

1.2.2 Civil Engineering Usage of Scrap Tires

Civil engineering usages of scrap tires include lightweight embankments, drainage layers, slope stability and retaining walls. Hughes lists some advantages attributed to shredded and whole tires. He states that they:

1. are a lightweight material,
2. are replacements for more expensive or scarce aggregate,
3. provide a material with high permeability characteristics,
4. are resistant to ultraviolet radiation,
5. are non-biodegradable (important as a stable fill),

Table 1.1: Current and Potential 5 Year Usage of Scrap Tires

Industry	1992 Usage	Potential 1997
Cement Kilns	7*	60
Pulp and Paper	14	55
Electricity	21	75
Tires-to-Energy	15	40

Note: numbers are in millions of scrap tires

*** Permitted Capacity is 14 million tires**

(from Hughes 1993)

6. provide a material with improved thermal characteristics relative to frost penetration,
7. are cost effective, and
8. are technically superior to land filling.

Though these methods have proven to be sound in an engineering sense they still do not eliminate the scrap tires as solid waste.

The use of scrap tire rubber as a lightweight fill has been used by Oregon, Vermont, Minnesota, and Washington. Hughes states that shredded scrap tire rubber proved to be the safest, quickest, and most cost-effective solution available for lightweight fill design in Minnesota. The report also shows other interesting uses of this potentially valuable material.

The use of shredded scrap tires for subbase drainage layers in highways is attractive due to the substantial quantity of scrap tires which can be disposed of in a mile of highway. Coarsely shredded or chopped tires has permeability values comparable to the permeability values of 38.1mm x 19.1 mm (1 1/2" x 3/4") coarse aggregate. The low unit weight of the material allows the chips to be placed over marshy land where heavier materials tend to sink.

Whole tires are used for retaining walls to improve slope stability. The scrap tires are piled in such a way as to not alter their original shape when stacked. The tires should be used on lower slopes and in locations not visible to motorists. Engineering fabric is often used behind, under and over the tires to prevent soil erosion. The cost of such retaining walls is about half the cost of a gabion and a third the cost of a concrete crib wall.

1.2.3 Crumb Rubber Modifier (CRM) Technology

Crumb rubber modified asphalt has a potential to eliminate a large number of scrap tires as waste if it becomes widely accepted and used. CRM asphalt cement presently consumes 1 to 2 million tires/yr in the United States. The CRM technology incorporates from 2 to 6 tires per metric ton of hot-mix asphalt (HMA) paving material.

The use of rubber modified asphalt has been given much attention through many investigations as a means of utilizing waste tires, while at the same time attempting to improve pavement performance. Many cases are cited in literature where rubber modified asphalt has shown superior performance to conventional asphalt cement. Some examples are described by Charania et al. (1991), Larsen (1989), and Renshaw et al. (1989). Superior performance has been documented in many field tests of CRM pavements, while equal and sometimes inferior performance has been shown in others. Uncertainty over performance and recyclability as well as increased costs has persuaded some people to dismiss CRM asphalt as a viable paving material. Robert M. Thompson, immediate past chairman of the National Asphalt Pavement Association (NAPA), is trying to prevent the implementation of Section 1038 (b) of the Intermodal Surface Transportation Efficiency Act (ISTEA) until further definitive research is carried out on CRM Asphalt pavements (NAPA 1993). Section 1038 requires that a certain percentage of Hot Mix Asphalt (HMA) tonnage constructed with Federal-aid funds contain crumb rubber from scrap tires. These minimum utilization requirements are 5 percent of the Federal-aided HMA tonnage in 1994, 10 percent in 1995, 15 percent in 1996, and 20 percent in 1997, must contain crumb rubber modifier (CRM). Failure to comply with these minimum requirements will

result in the loss of Federal-aided highway funds. The cost of HMA increases with the addition of CRM anywhere from 20 to 100 percent. It is inconclusive whether improved performance (if any) offsets the additional cost of CRM asphalt cement pavements.

In general the application of crumb rubber modifier (CRM) technology can be divided into two categories: the wet process and the dry process. These categories are derived from the basic process used to add the crumb rubber to an asphalt paving mixture. The term wet process defines any method that blends the crumb tire rubber with the asphalt cement before introduction to the aggregate. The dry process mixes the crumb tire rubber, usually in larger chunks, with the aggregate before this mixture encounters the asphalt binder. When CRM is incorporated into an asphalt paving material, the CRM will modify the properties of the binder and is called CRM asphalt concrete. When CRM is used as a rubber aggregate, the HMA is called rubber-modified hot-mix asphalt (Heitzman 1993). Figure 1.1 presents a summary of CRM asphalt processes. The individual processes shown in this figure are describe in Chapter 2 - Literature Review.

1.3 Barriers to Implementation

There are several barriers to the increased use of scrap tires in any of the previous applications. General problems stated in the report prepared by Hughes are:

1. source of supply,
2. consistency of supply,
3. high capital or initial costs,
4. conservative nature of user industries,

Figure 1.1: Relationship of Crumb Rubber Modifier (after Heitzman 1993)

5. nonstandardization of air permit modifications,
6. lack of education of the public.

While these problems pertain to all the utilization technologies of scrap tire rubber specific problems which relate to CRM asphalt are as follows:

1. only 50 to 60 percent of the scrap tire is used,
2. production requires increased energy consumption,
3. the cost of an asphalt pavement is increased by more than 50 percent with as yet, unproven equivalent life performance benefits,
4. there is uncertainty over recyclability of the material,
5. the effect of fumes on workers is not known,
6. national standards on mix design procedures and performance criteria are lacking.

These problems are serious and must be addressed before widespread implementation of CRM asphalt pavements is possible.

1.4 Objectives of the Research

In this chapter, it has been indicated that there is a serious disposal problem involving discarded tires. One of the methods used in recycling scrap tires is to incorporate ground tire rubber crumbs with asphalt cement to produce CRM asphalt concrete. While there has been documented evidence of superior performance of CRM asphalt concrete, some researchers still remain unconvinced of its effectiveness under varying climatic conditions.

The major objective of this thesis is to develop a new CRM pavement system

based on the stone matrix asphalt (SMA) concept. The SMA concept has been selected due to unique gradation specifications which, in the authors opinion, are favourable for use with CRM pavement systems. A full depth pavement system will be designed using CRM pavement technology.

1.5 Scope - Organization of the Thesis

This section briefly outlines the scope of the investigation along with the remaining chapters of the thesis and their purpose.

Chapter 2 presents the major results of a literature review. Initially, only applications of CRM asphalt concrete that have been described in literature during the past five years are examined. Older reports that are referenced and of interest are also reviewed. A brief study of laboratory and full-scale field tests of both wet process and dry process pavements, and the economics of CRM asphalt are also presented. The literature review shows the promise of CRM asphalt concrete and demonstrates that further investigation is needed.

Chapter 3 contains the testing philosophy adopted for this research along with gradation selections for use with the CRM asphalt cement binder.

In Chapter 4, the concept of SMA is described and the design of the SMA mix selected for this project is documented. The details of the experimental program and the procedures used are described. The Marshall mix design was chosen to select the optimum asphalt content for the SMA mixes used for this study. The method of selection of materials, the manufacture of the mixes, as well as the testing procedures are outlined

in detail.

Chapter 5 contains the results of the mix design procedure describe in Chapter 4. The optimum contents are selected and presented along with mix comparisons. Preliminary results are drawn from the Marshall mix design study.

The specifications for the large aggregate mix are presented in Chapter 6. Aggregate gradation and the mix preparation conditions are detailed.

Chapter 7 describes the performance based testing program. A description of the equipment and testing methods used to evaluate and compare the various mixes, is presented.

Chapter 8 describes the results of the performance based testing program. A full evaluation of the effects of rubber on the stone mastic asphalt and large aggregate mixes is also documented.

Chapter 9 gives a complete analysis and discussion of the testing results. Both the Marshall program and performance test results will be discussed.

Finally, Chapter 10 presents the summary and the conclusions of this study and recommendations for future research.

Chapter 2

LITERATURE REVIEW

This chapter reviews the laboratory research, available in the literature, which was performed on crumb rubber modified (CRM) asphalt concrete and CRM applications on roadways.

All documents dealing with CRM asphalt concrete in the last five years and found on the Transportation Research Information Service (TRIS) database were reviewed. Among the other services contributing to TRIS are the Highway Research Information Service (HRIS), the Maritime Research Information Service (MRIS), the Railroad Information Service (RRIS), the Air Transportation Research Information Service (ATRIS), the Urban Mass Transportation Research Information Service (UMTRIS), the International Road Research Documentation (IRRD), Transportation Libraries (TLIB), and Highway Safety Literature (HSL). As well any references of interest found in the reports obtained from TRIS were also examined. The author's information was kept current by reviewing a bi-monthly update on any new CRM asphalt concrete publications.

Sections on modified asphalts, advantages and disadvantages of CRM asphalt concrete, and economics are also included.

Finally, the major conclusions of the literature review as they apply to the use of CRM asphalt concrete are presented.

2.1 Modified Asphalts

In an attempt to utilize waste and by-products to save the environment from further contamination, research into the recycling of materials has greatly increased. A potentially huge opportunity for utilizing various wastes and by-products is in highway construction. Studies on fly ash, bottom ash, incinerator residue, sulfate wastes, digested sewage sludge, coal mine refuse, glass, plastics, waste rubber, and cement manufacturing wastes have been completed.

A large-scale industrial waste utilization project was constructed in connection with Transpo 72, an International Transportation Exposition held at Dulles International Airport. The basic waste composition used consisted of fly ash, dolomitic lime, sulfate wastes, flue gas desulphurization sludge (FG) and fluorogyp (FLG). Brink (1973) indicates that satisfactory mixtures of the waste materials could be produced in a portable plant and that a suitable pavement could be constructed. However, localized failures indicated that further study was needed to develop optimum compaction conditions in the field.

Other wastes used in asphalt concrete have been summarized by Ormsby and Fohs (1990). The wastes reviewed in the report include:

- lime and cement kiln dusts
- cellulosic wastes

- wood lignins
- bottom ash
- incinerator residue
- sewage sludge
- mining wastes
- scrap rubber

Although many wastes have been investigated for use with asphalt pavements, only scrap tire rubber is of interest in this thesis.

2.2 Laboratory Studies of Crumb Rubber Modified Asphalt Pavements

2.2.1 Historical Development

The first patents issued for modification of bitumen with rubbery materials was in 1843 by a British patent. The following year a British patent was obtained by Cassell concerning the application of bitumen-rubber mixtures to road surfacing. In 1935 Russian literature carried a first reference on the work of D. Pospkhov and A. Fokin titled "Mixtures of Asphalt and Synthetic Rubber" (Allison 1967). However, it wasn't until the mid 1960's that CRM modified asphalt cement, which was pioneered by Charles H. McDonald of the City of Phoenix, has been more readily accepted (Charania et al.-undated). Earlier experiments of CRM asphalt concrete used a relatively small percentage of rubber to asphalt, usually as a dry additive. Virtually none of this work showed significant benefits to justify the additional costs of construction. Charles McDonald

discovered that a time-temperature related gel-like material resulted from the pre-reaction of mixing vulcanized crumb rubber with hot paving grade asphalt. This new material is considerably more viscous than regular asphalt. This type of CRM asphalt concrete is produced by the "wet process". The first uses of a CRM asphalt concrete chip seal showed good engineering characteristics; however chip loss, which caused damage to cars, drew public opposition to the project. A special CRM asphalt concrete hot mix was developed to solve this problem. There are two primary applications of "wet-process" CRM asphalt concrete today. The first is a Stress Absorbing Membrane (SAM) where a hot asphalt rubber chip seal is applied to the damaged surface. A stress Absorbing Membrane Interlayer (SAMI) differs in that a 1.5 to 2 inch asphalt concrete layer is overlayed on the CRM asphalt concrete. A study by the Asphalt Rubber Producers Group has shown that CRM asphalt concrete used on damaged streets in Phoenix needed no subsequent maintenance for 20 years (Charania et al.-undated)

Another method of producing CRM asphalt concrete is by the dry process. This is a process that uses relatively large rubber particles to replace some of the aggregate in the mixture. The amount of crumb tire rubber typically used is between 3 to 4% by weight of the aggregate. This product was also developed in the late 1960's by Sweden and patented under the trade names of "Rubit" in Sweden and as "PlusRide" in the United States (Takallou and Hicks 1988). This process uses much more scrap rubber per mile of road, but it doesn't seem to perform as well as mixes produced by the wet process.

Other types of CRM asphalt concrete in use are Porous Asphalt Rubber Concrete (PARC) and Chunk Rubber Asphalt Concrete (CRAC) of which the former is

manufactured by the wet process and the latter by the dry process. Both mixes bear modifications to standard wet and dry process mixes. PARC was developed in France to maintain superior drainage, high skid resistance and reduce tire rolling noise while maintaining excellent durability (Sainton 1990). CRAC was created by the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL) to improve ice disbonding on roads (Eaton et al. 1991).

Though some people might debate the use of rubber in asphalt as imparting any advantages, the United States Congress seems intent on implementing this technology. Former President Bush signed a bill mandating the use of CRM asphalt concrete beginning on January 1, 1994 for any State requiring federal funds. The U.S. Congress passed The Intermodal Surface Transportation Efficiency Act (ISTEA) in 1992, which requires all states 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). Section 1038 of H.R. 2950 states "Beginning on January 1, 1994 each State must use asphalt pavement containing recycled rubber in 5 percent of the total tons of asphalt pavement laid in the state and financed in whole or in part by federal funds. The required percentage of asphalt pavement containing recycled rubber, in relationship to total tons of asphalt pavement laid in each State, increases by 5 percent each year until reaching 20 percent in 1997, where it remains for each year thereafter. If a State fails to meet this requirement, the amount of federal funds the asphalt pavement containing recycled rubber would have required will be withheld." (Manhole Messenger 1992).

2.2.2 Use of Rubber in CRM Pavements

Rubber used to mix with asphalt cement should be used from recycled tires because it is a waste product. There are two types of crumb rubber currently being used. The first is obtained from tire buffings or peelings and the second is from whole-tire processing. The former creates a much more uniform product, while the latter is more desirable since it uses the entire tire. Chehovits (1989) states that there are several characteristics of the rubber that influence properties of the CRM asphalt cement blend. Physical properties including particle size (gradation), shape (angular or elongated), surface texture (depending on grinding method), and contaminant presence (fibers, etc.) all influence properties of the CRM asphalt cement. He also states that chemical compositional characteristics influence blend properties. Rubber hydrocarbon content, specific type of rubber polymer of blends, plasticizer content, and reinforcement type and content (carbon black, etc.) must all be considered.

Salter and Mat (1990) studied the effects of three types of rubber on the behaviour of asphaltic concrete mixes. The types of rubber used were natural (not tire rubber) and are as follows:

- **Pulvatex**, an unvulcanized rubber powder manufactured from concentrated natural latex. It is composed of 60 percent natural rubber powder and 40 percent separator.
- **Crusoe Standard**, an unvulcanized spray-dried rubber powder containing approximately 6 percent silica filler and 1 percent calcium stearate as a partitioning agent.

- **LCS Revertex**, an unvulcanized latex concentrated by evaporation to 68 percent solids and stabilized with potassium hydroxide and soap.

The addition of all three types of rubber additives in the binder caused a decrease in penetration, and an increase in softening point in comparison with the unmodified binder. The Marshall testing program showed a small increase in optimum binder content for the modified mixes, however, stability values depended on the type of rubber used. The addition of rubber increased stability values for Pulvatex and LCS Revertex but not for Crusoe Standard. Similar results were obtained by the force ductility test. Fatigue testing showed improvements of up to 88 percent by CRM asphalt cement blends. Salter and Mat (1990) have indicated that the addition of rubber to asphalt has, in general, produced desirable properties but also that the type of rubber used is an important factor to be considered.

LaGrone (1981) defined the terms related to processing of scrap rubber and provided the typical composition of scrap rubbers available for the production of CRM asphalt cement binders. The type of rubber selected affects the elasticity of the resulting binder and the stability of the reacted product.

2.2.3 Use of Additives in CRM Pavements

In a report by Jimenez et al. (1979), temperature-viscosity relationships were developed for a regular asphalt and a CRM asphalt cement blend. The CRM asphalt cement blend has a ratio of vulcanized rubber to regular asphalt of 25/75. The viscosity equations are:

$$\text{regular asphalt cement} \quad \eta = 6.767 \times 10^{23} \text{ } ^\circ F^{-10.69}$$

$$\text{CRM asphalt cement blend} \quad \eta = 5.768 \times 10^{14} \text{ } ^\circ F^{-4.494}$$

where η is the viscosity in poises at a shear rate of 0.05 sec^{-1} . Test method T245-78 of the American Association of State Highway and Transportation Officials (AASHTO) specifies that the asphalt should be at a temperature that corresponds to a viscosity of 170 cSt for compaction. This requirement would correspond to a temperature of 938°C (1721°F) for the CRM asphalt cement. This temperature is surely unreasonable and if the CRM asphalt cement is too viscous and becomes a problem, extending oils can be used to soften the material for improving the degree of interaction between the asphalt and rubber and improving workability. It should be noted that a much higher shear rate will be required to properly coat aggregates with CRM asphalt cement. Other additives such as adhesion agents can be used to improve film stripping resistance.

Takallou and Sainton (1993) examined the effects of four different content levels of oil extenders on the penetration and softening point of CRM asphalt cement. Test results indicate that an increase in the percentage of oil extender will decrease the softening point, increase penetration, increase the capacity for stretching and tension, and reduce resistance to shear flow. The authors concluded that a maximum of 6 percent of extender oil provides optimal binder properties.

2.2.4 Dry Process CRM Asphalt Concrete

The method of adding rubber to asphalt mixes in a dry process was originally developed in late 1960's in Sweden and patented under the trade names of "PlusRide" in the U.S. and "Rubit" in Sweden. In this system, rubber-asphalt mixtures are prepared by a process that uses 3 to 4 percent by weight of relatively large 1.6 mm to 6.4 mm (1/16" to 1/4") rubber particles to replace some of the aggregate in the mixture.

The aggregate gradation for this type of system must be modified to accommodate the extra rubber particles. A rubber-modified pavement is gap-graded for this purpose (see Figure 2.1). The gap in the gradation is typically in the 3.2 mm to 6.4 mm (1/8 to 1/4 in.) size range. The rubber particles replace the stone aggregate that normally would occupy this space. One of the problems in producing this type of mix has been achieving the proper gap in the grading curve and obtaining sufficient fines (passing No. 200) to serve as a void filler. The lack of mineral filler causes high air voids which is not desirable in the Marshall mix design.

Takallou and Hicks (1988) performed a laboratory study on the dry process CRM asphalt concrete. The Marshall mix design was used to determine the optimum asphalt contents of the different mix combinations. Once this was accomplished, the resilient modulus and fatigue life tests were used to evaluate the mixes. It was discovered that the mixture with gap-graded aggregate and 3 percent coarse rubber required the highest design asphalt content (9.3%). The conventional dense graded asphalt mix required 5.5% asphalt at optimum content.

The fatigue results revealed that the rubber-modified mixes had a much greater

Figure 2.1: Comparative aggregate gradation curves for conventional and rubber-modified asphalt pavements (after Takallou and Hicks 1988)

fatigue life than a conventional asphalt concrete mix.

Lawrence et al., 1991, reported on Marshall tests performed on CRM asphalt concrete mix placed in Thamesville, Ontario in 1990. They noted that the addition of coarse scrap rubber to dense graded hot mix has the effect of raising the flow value and lowering the stability values, with the flow value increased by almost 80% and the stability value lowered by over 50%. The stability value for the CRM asphalt concrete mix used at Thamesville was below the minimum specified value for standard mixes.

2.2.5 Wet Process CRM Asphalt Concrete

Asphalt SAM's and SAMI's are produced by the wet process. In this process anywhere from 5% to 30% ground rubber is introduced to the asphalt cement at temperatures from 325 - 450 °F. The mix of asphalt and rubber is then continuously stirred anywhere from 1/2 to 2 hours at the elevated temperature. Extender oils may be used to reduce the viscosity and increase the workability if required. The CRM asphalt cement mixture is then allowed to cool and stored for future use.

Roberts and Lytton (1987) have compiled a list of preparation conditions of CRM asphalt cement (Table 2.1.).

The mixing and compaction temperatures used for CRM asphalt concrete are considerably higher than those used for regular asphalt concrete. The primary reasons for such high temperatures are due to the very high viscosity of the CRM asphalt cement binder at temperatures defined for the Marshall method, and due to the difficulty of wetting the aggregate surface with CRM asphalt cement at normal temperatures.

Table 2.1: CRM Asphalt Concrete Specimen Preparation Conditions
(After Roberts and Lytton, 1987).

Author	Compaction type	Mixing Time (min.)	Temperature Conditions (°F)		
			Mixing		
			Asphalt Rubber	Aggregate	Compaction
Jimenez (1979)	California Kneading Compactor	2	375	300	250
Lalwani (1982)	Marshall Hammer	N/I	N/I	N/I	N/I
Dickson (1981)	Marshall Hammer	Until coated	375	375	375
Vallerga (1981)	N/I	N/I	350	350	325

N/I: Not Included

In a report by Jimenez (1989) the viscosity effects of adding rubber to hot asphalt cement were studied. The viscometers were of the falling coaxial cylinder type and the forced flow Schweyer Rheometer. The variables studied were:

- diameter of flow-tube
- amount of rubber in the blend
- gradation of rubber particles
- type of rubber
- test temperature

Jimenez determined that the viscosity of the CRM asphalt cement increased as the amount of rubber increased and also as the range of particle sizes increased. The viscosity of one blend of rubber showed greater temperature susceptibility than other blends. It is due to the increase in viscosity that higher compaction temperatures are used for sample preparation.

Oliver (1981) reported the effect of reaction time and temperature (Figure 2.2). It is quite evident that the properties of the CRM asphalt cement binder degrade after too long a reaction time at high temperatures. Therefore, one must take precautions when selecting mixing conditions for asphalt and crumb tire rubber. Oliver also showed that synthetic rubber is much more stable than natural rubber under higher digestion temperatures.

A full Marshall mix design study on CRM asphalt concrete mixes was performed by Al-Abdul-Wahhab and Al-Amri (1991). Two aggregate gradations with hot asphaltic concrete mixes containing 10%, 20%, and 30% reclaimed rubber as part of the binder

Figure 2.2: Effect of Digestion Time and Temperature on Elastic Recovery for Asphalt-Rubber from National Rubber Tire Buffings (after Oliver 1981)

were investigated. A mixing time of 2 hours at a temperature of 200°C has been shown to be superior for elastic properties of CRM asphalt cement. The Marshall method of mix design was employed in the preparation of test specimens for both gradations in order to obtain the optimum mix. Tests were then performed at the optimal asphalt content for mix comparisons. Wahhab and Amri performed the following tests;

- softening point
- viscosity
- Marshall stability and flow
- Hveem stability
- split tensile test
- static creep test
- resilient modulus test
- fatigue test

Based on the experimentation it was determined that the addition of rubber improved the softening point by 8°C, 10°C, and 11°C for 10%, 20%, and 30% rubber content, respectively. Such an improvement on softening point might improve the rutting resistance of these mixtures. The viscosity also increased with a higher rubber content. It was also found that the addition of 10% reclaimed rubber improved the resilient modulus, M_R , stiffness modulus, as well as the stability (Marshall and Hveem) and fatigue life. Salter and Mat (1990) achieved similar results for softening point, Marshall stability, and fatigue testing for CRM asphalt concrete mixes. Improvements of up to 88 percent were noted in fatigue life.

2.2.6 Additional types of CRM Asphalt Concrete

Different variations on the basic wet and dry process blends have been studied and tested by various groups. Two of these new brands of CRM asphalt concrete are Chunk Rubber Asphalt Concrete (CRAC) and Porous Asphalt Rubber Concrete (PARC).

CRAC which was developed by the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL) differs from commercial CRM asphalt concrete products in that it contains four times as much rubber in larger pieces. CRAC is manufactured by the dry process and its goal is to enhance ice disbonding properties. Eaton et al. (1991) used the Marshall method of mix design to determine the optimal asphalt content for the different CRAC samples. They prepared pavement samples containing 0%, 3%, 6%, 12%, 25%, and 57% rubber by weight of sample. One mix was prepared with 93.5% rubber and 6.5% asphalt with no stone aggregate. Laboratory tests showed that the Marshall Stability values for CRAC doubled in strength for the 3, 6, and 12% rubber contents. Also, cracks did not develop in the ice layer, nor, was there any deterioration in the ice-pavement bond during wheel tracking tests of the samples with no rubber. Cracking in the ice layer increased with increasing rubber content. Ice disbonding for samples with rubber contents less than or equal to 12% resulted from localized deformation, whereas samples with rubber contents greater than 12% experienced disbondment through area deflection. The authors conclude that CRAC does stimulate ice disbonding to a greater extent than "PlusRide" and certainly more than regular asphalt cement.

PARC, which is manufactured by the wet process, was developed in France in an

effort to solve the problems of hydroplaning, water projection, noise reduction, and the mirror effect at night. Full-scale field experiments have been conducted on the A2 turnpike in the north of France. The French Road and Bridges Laboratory made rolling noise, roughness, and drainage tests after three years of traffic. Sainton (1990) describes the general advantages of PARC:

- high skid resistance at high speed and in rainfall
- no spray or splash
- no light reflection from wet pavement
- reduction of tire rolling noise
- low cost due to low density

CRM asphalt cement binder was chosen since pure asphalt cements have neither the cohesion nor the flexibility to give porous asphalt concrete resistance to heavy traffic. CRM asphalt cement has a high viscosity which allows a high binder content in porous asphalt mixes which leads to binding strength, fatigue resistance, and aging resistance due to the thick film. Polymer modified asphalt (PMA) displayed good mechanical properties at first however after aging it became brittle at low temperature. From his work Sainton concluded that PARC displayed constant draining properties, good fatigue and rutting resistance, reduced tire noise and insensitivity to bad weather.

2.3 Field Trials of CRM Asphalt Pavements

2.3.1 Dry Process CRM Asphalt Concrete

There are numerous cases documented of the use of this type of CRM asphalt concrete in full-scale field trials. The manufacturer of "PlusRide" claims that their product has the following benefits:

- higher friction resistance
- noise reduction from traffic
- deicing characteristics
- increased road life

The Minnesota Department of Transportation (Mn/DOT) conducted an evaluation of "PlusRide" at two selected sites. Both projects were constructed in September, 1984. The same paving equipment was used to place the rubber modified mixture as was used for the conventional mixture. Placement of the rubber modified mixture was similar to the conventional mixture except that a considerably higher temperature was used. In a report by Allen and Turgeon (1990) they noted that there is no significant difference in tire noise level, friction resistance, deicing, or ability to retard transverse reflective cracks. With these results and due to construction costs per ton being over twice the cost of the conventional mix, further test sections were not recommended.

An evaluation report for the Washington State Department of Transportation prepared by R.E. Allison (1990) produced similar results. A six year evaluation of a "PlusRide" asphalt concrete pavement laid in the city of Renton, Washington illustrated that there was no significant improvement corresponding to any of the claims made by

"PlusRide" representatives.

A test section laid in Thamesville, Ontario is performing well to date apart from some concerns (Lawrence et al. 1991). Some large coarse aggregate particles are being dislodged from the surface wherever a particle of rubber is immediately under the aggregate. Secondly, the longitudinal construction joint at centreline is showing some joint opening and ravelling, and the mat is slower to dry out after rain. A Swedish study by Sandberg et al. (1990) on tire/road noise on rubberized asphalt concluded that Rubit surfaces do not give lower noise levels than conventional asphalt surfaces. Though these cases may be disappointing, positive results with the dry process have been achieved.

Takallou H.B. et al. (1989) explain that extensive measurements by Alaska DOT&PF showed an average reduction in stopping distance of 25% for the rubber-modified pavements in icy conditions. Deicing benefits have also been reported by Mn/DOT and Swedish road authorities.

A reduction in noise level of the traffic and reflective cracking has been reported on the Jan Smuts - Johannesburg freeway in a paper by Bergh et al. (1989). The mixture was composed of an open-graded bitumen rubber asphalt with a 2.25% rubber content by weight of total mixture. Splash reduction in wet weather was also reported due to the open-graded property of the mixture and high laboratory voids achieved (20-27%).

A review of the dry process in the literature gives the reader an uncertain outlook on the promise of this product. However, results obtained from the use of CRM asphalt concrete made by the wet process are not as ambiguous.

2.3.2 Wet Process CRM Asphalt Concrete

There are numerous examples in the literature citing superior performance for CRM asphalt concrete in full scale test sections. The city of Phoenix, confident in the advantages of CRM asphalt concrete, has paved many streets with this product. One project, reported by Charania et al. (1991), placed CRM asphalt concrete on Indian School road, whose surface was in poor condition and badly cracked at the time. Application of an asphalt rubber chip seal was temporary and meant to prevent complete deterioration of the street. It was scheduled for reconstruction in two years. However, the street has performed satisfactorily with very little maintenance for over 20 years. Similar results were obtained by placing CRM asphalt concrete on the I-17 Frontage road and off ramp at Van Buren street. In a report by the Arizona Department of Transportation (ADOT 1988), it was concluded that performance data for the CRM asphalt concrete sections was promising and that "...the asphalt-rubber layers are effective in preventing reflection cracking".

A study by the Maine Department of Transportation (1990) concluded that "The initial costs of ARC are high compared to conventional asphalt pavements, estimates range from 45 to 114 percent. However, the results suggest a substantial extension in expected life of the pavement." This report summarized the CRM asphalt concrete experience of various state transportation departments. A California Department of Transportation (Caltrans) report by Doty (1988) observes "...that the service life of the (ARC overlays) under study may be considerably greater than those of equivalent thicknesses of conventional (pavement)..." Similar reports by Connecticut DOT (1989)

and New York DOT show superior performance by CRM asphalt concrete test sections.

In October 1980, the Connecticut Department of Transportation (ConnDOT) placed an experimental 274 m (900-ft) section of CRM asphalt concrete on State Route 79 in Madison, CT (Larsen 1989). The pavement was placed as a 38.1 mm (1.5 in.) thick overlay in one lift using conventional paving techniques. A standard control section was placed at the same time. Larsen concluded in an eight-year evaluation that on the basis of transverse, longitudinal and alligator cracking the asphalt-rubber mix performed better than the control mix. Skid resistance and roughness values remained acceptable and did not differ from the control section values.

South Africa constructed several bitumen-rubber test sections in 1982. The basic composition of the modified mix consisted of the following by mass;

- 78% 80/100 penetration grade bitumen
- 2% high aromatic extender oil
- 20% ambiently ground crumb rubber

Renshaw et al. (1989) claim that the performance of the bitumen rubber test sections in these projects has been very good and has "...amply demonstrated its ability to perform as originally claimed."

The Ministry of Transportation of Ontario (MTO) built trial sections in July 1990 on Highway 400, near Innisfil, to compare standard and modified HL1 mixes (Joseph and Kennepohl 1991). The modified mixes used in trial sections were: Premium asphalt, Novophalt, Neoprene, Vestoplast, and tire rubber (#80 mesh). Initial experience suggested that paving with CRM asphalt concrete was feasible using conventional equipment. The

performance of the individual test sections have not been determined to date.

The Asphalt Rubber Producers Group (ARPG) based in Phoenix, Arizona make the following claims about wet process CRM asphalt concrete:

- increases pavement life
- resists rutting, aging and reflective cracking
- reduces pavement thickness
- provides optimum skid resistance
- traffic noise reduction of 65 to 85 percent

The first three claims seem to be backed up by numerous trial sections in use today. However, upon further study of the literature, claims of increased skid resistance are inconclusive. The ARPG cites examples of CRM asphalt concrete used in Belgium, France, Germany, Austria, and Arizona where noise levels have been reduced from 50 to 90 percent. The type of CRM asphalt concrete mix commonly used for noise reduction is referred to as "Drainasphalt".

The use of wet process CRM asphalt concrete seems very promising from the results obtained to date. Further perfecting of construction procedures and the development of construction standards should increase the usage of this product.

2.4 Potential Advantages of CRM Asphalt Concrete

The advantages of CRM asphalt concrete have been formerly outlined in the chapter but are summarized in Table 2.2.

These advantages make it attractive to consider using CRM asphalt cement as a

Table 2.2: Reported Advantages of CRM Asphalt Cement

Method of Manufacture	Product Name	Advantages over regular asphalt cement
dry process	Rubit PlusRide	<ul style="list-style-type: none"> • higher friction resistance • traffic noise reduction • deicing characteristics • increased road life (debatable)
	CRAC	<ul style="list-style-type: none"> • superior ice disbonding properties
wet process	SAM SAMI	<ul style="list-style-type: none"> • retards reflection cracking • resists rutting and aging • improved skid resistance (debatable) • increased road life • improved fatigue properties
	PARC	<ul style="list-style-type: none"> • no spray or splash • improved drainage • reduced light reflection from wet pavement • reduced tire rolling noise • low cost
both processes		<ul style="list-style-type: none"> • removes a waste product (scrap tires) from landfills

binder for pavements. Although certain advantages are evident, the reasons outlined in section 2.5 discourage widespread use of CRM asphalt pavements.

2.5 Potential Disadvantages of CRM Asphalt Concrete

The main disadvantage of CRM asphalt cement is the increased cost of pavements using this binder (see next section). Other problems have been encountered in the construction of CRM asphalt concrete test sections which include:

- pick up while compacting with rubber tired rollers
- higher compaction temperatures needed
- plugging of screens due to higher viscosity binder
- chip loss in low or non traffic areas
- uncertainty of emission effects on workers during construction of the pavement
- stripping
- uncertainty of improved performance
- rubberized asphalt binder must be used within hours of production due to temperature-time properties
- lack of design criteria and nonconventional testing methods

Another concern is whether CRM asphalt concrete pavements are recyclable. If CRM asphalt concrete turns out to be unrecyclable, a solid waste problem of several magnitudes larger will be realized and will effectively kill any intended use of this product. No attempts at addressing this problem could be found in the literature although it is known that MTO has tried recycling part of their test section near Thamesville, Ontario.

2.6 Economic Analysis of CRM Asphalt Concrete

Although CRM asphalt concrete has higher initial costs it has the potential to be cost effective over the entire life cycle of the pavement. Hoyt et al., 1988 based cost estimates for CRM asphalt concrete on; (1) the cost of producing the CRM asphalt cement binder and (2) substitution of the cost of the CRM asphalt cement binder for the cost of the asphalt cement in asphalt concrete unit prices. An additional 10% was added to the construction costs due to a requirement for higher compaction temperatures and a possible greater compactive effort. The cost of CRM asphalt concrete compared to regular asphalt concrete can be seen in Table 2.3.

For a breakdown on how these numbers were calculated refer to the report by Hoyt et al., 1988. Takzailou and Sainton (1993) attribute the increase in cost to the following factors.

Wet System

- Rubberized binder must be used within hours of its production; therefore the high cost of mobilizing the specialized equipment (blending unit, metering unit, storage tanks, etc.) at the production facility must be recovered in the cost per ton of rubberized binder used on that project.
- Licence fee for using the patented process.

Table 2.3: In-Place Costs for Asphalt Concrete and CRM Asphalt Concrete (after Hoyt et al., 1988)

	Percent Binder	In-Place Costs per ton, (\$/Ton)	Density (lb/ft)	Tons per Square Yard, (Ton/S.Y.)	In-Place Costs per Square Yd. (\$/S.Y.)
Asphalt Concrete	4.80	29.44	151.2	0.851	25.04
CRM AC	5.23	47.25	144.9	0.815	38.51

Dry System

- Unique aggregate gradation.
- Introduction of crumb rubber to asphalt plants.
- Higher asphalt and filler content design requirements.
- Increase in mixing temperature
- Licence fee for using the patented process.

An equivalent uniform annual cost per square yard of each pavement over its life was calculated for four climate conditions. The projected pavement life was determined by a computer program called ILLIPAVE. It was determined that CRM asphalt concrete was more cost effective in three of the four unique climatic zones (Hoyt et al. 1988).

Jacobson and Schnormeier (1989) determined that after 15 to 20 years, the CRM asphalt concrete pavement is 40% the cost of the standard. This value was based upon the difference between the cost of a standard chip seal and an asphalt rubber chip seal for pavement rehabilitation. Several field studies in the literature have shown that asphalt rubber chip seals have outlasted the standard chip seal by a factor of 2 to 3 times. Initial CRM asphalt concrete costs are twice as much as a conventional asphalt. However, all factors must be considered in pavement rehabilitation to determine the final cost numbers. Masters (1989) states that the additional costs of the CRM asphalt concrete versus the conventional methods ranges from 29% to 162% not taken life expectancy into account. He also reached the conclusion that in all cases, savings resulted using the CRM asphalt concrete rehabilitation systems compared to conventional systems.

McQuillen et al. (1988) conducted an economic analysis of rubber-modified asphalt mixes manufactured by the dry process. The cost per ton of PlusRide at that time was \$55.68 compared to \$39.00 for conventional asphalt cement binder. Three different methods of comparing the costs were used. The first analysis used an assumed maintenance scenario and equal surfacing thicknesses to calculate the life required for equivalent annual costs. The second used equal surfacing thicknesses of rubber-modified and conventional asphalt pavements and only the capital cost to determine the required life for equivalent annual cost. The last method utilized layer equivalency ratios to compare the capital costs of rubber-modified and conventional asphalt of unequal thicknesses (McQuillen et al.). It was concluded that the rubber modified asphalt mix was more cost effective than a conventional mix. This conclusion was based upon a computed life expectancy model by the Elastic Layer System Computer Program (ELSYM5). The conclusion that CRM asphalt concrete pavements is more cost effective is based on a pavement life for the rubber modified mixes of 20 to 23 years when compared with 15 years for a conventional mix. Also, the required thickness of a rubber modified mix can be reduced by 1.2 to 1.4 times compared with the conventional mix if the equivalency factors are used.

Although initial costs are higher, the use of CRM asphalt concrete appears to be economically feasible if the life expectancy predictions are reliable.

2.7 Conclusions of the Literature Review

The preceding review of available literature has permitted conclusions to be drawn regarding the use of rubber in asphalt. The following major conclusions have been derived:

- scrap tires present a major solid waste disposal problem where CRM asphalt concrete potentially offers a feasible solution to this dilemma,
- CRM asphalt concrete mixes tend to offer higher stability and fatigue resistance in laboratory testing,
- field performance of CRM asphalt concrete mixes have shown (in some cases) an improved service life over conventional mixes,
- CRM asphalt concrete overlays are effective in retarding reflective cracking in Arizona,
- CRM asphalt concrete can be placed with existing compaction equipment,
- the initial costs of CRM asphalt concrete are higher than conventional asphalt concrete, and
- gap graded aggregate mixes are employed to allow room for the rubber particles.

With these conclusions in mind the objective of this experiment-based study was to develop two CRM asphalt concrete mixes, that will perform better or at least as good as standard mixes. The Marshall method (SMA) and CSTT methods (LAM) of mix design were used. A performance based testing plan was also devised on which to evaluate the CRM asphalt concrete mixes versus the control mix. Once the mixes have

been fully evaluated, a dependable and feasible pavement system could be designed and made available for future full-scale field testing.

Chapter 3

TESTING PROGRAM OVERVIEW

3.1 Testing Philosophy

Given the need for testing crumb rubber modified asphalt systems versus conventional flexible asphalt concrete systems a testing program was designed to obtain meaningful comparisons between the two pavement types. The following are mix designs and test methods available in engineering practice:

Mix Designs

- *Marshall mix design.* Once the aggregate gradation is selected, this method is used to determine optimum asphalt content in order to produce a mix for reliable field performance.
- *Hveem mix design.* Also used for construction design of hot-mix paving, but is not as popular.

Test Methods

- *Laboratory performance related testing.* This approach, though not yet standardized,

is rapidly gaining acceptance by research organizations to better simulate field conditions and therefore predict performance. The Strategic Highway Research Program (SHRP) project A-003A was formulated to develop test methods suitable for standardization (Monismith et al. 1991). Similar efforts are under way in Europe to standardize performance related testing systems throughout the European community.

- *Full-scale accelerated load tests.* This is the next step in the performance related testing process. Full-scale accelerated load tests such as the circular track method are in use by France (LCPC, Laboratoire Central des Ponts et Chaussées) and Spain (CEDEX pavements test track). As well, an accelerated load facility (ALF) has been developed in Australia and implemented also in the United States.
- *Full-scale field tests.* Trial test sections have been constructed and subjected to normal traffic flows. These test sections are closely monitored to determine field performance. One of the largest full-scale field test facilities in the world was constructed recently near Minneapolis, Minnesota. This facility was constructed as a traffic controlled bypass section of an interstate highway.

The approach adopted for this laboratory research program (see figure 3.1) is similar to the SHRP plan. Two different mixes have been evaluated in this research. A stone mastic asphalt (SMA) mix incorporating crumb tire rubber was designed to be used as a surface layer. A previously designed Large Aggregate Mix (LAM) was modified by adding crumb tire rubber to be used as a thick base layer for the asphalt concrete pavement system. In the initial mix design phase the Marshall method was used to select

Figure 3.1: Testing Philosophy for SMA mixes

I.T.S. = Indirect Tensile Strength

R.U.C.T. = Repetitive Uniaxial Compression Test

the optimum asphalt content for each type of SMA mix (control and CRM). When the optimum mix was determined, accelerated performance tests were conducted and comparisons made. The mix design for the LAM has already been evaluated at NRC by El Hussein et al. (1993). Therefore, the mix evaluation started in the sample preparation phase according to the testing philosophy flow chart. The LAM is subjected to the same performance tests and evaluated in the same manner as the SMA mixes.

Full-scale field test are useful for correlating laboratory data to field conditions. However, only laboratory testing was used in this research due to the extremely high costs of field tests and the time consuming nature of this approach. It is, however, expected that field tests of SMA and LAM rubber asphalt will be performed in the near future based on these results. It must be realized that a research program of this type would produce minimal results in the first 3-4 years. Also, it is difficult to draw meaningful comparisons between different roadway sections due to variability in subgrade conditions, traffic patterns, climate, and water table characteristics. In spite of these complications, the observations and results of the performance of real road sections are invaluable and can not be replaced by laboratory testing.

3.2 Aggregate Gradation Selection for use with CRM Asphalt Cement

It was noted after review of the current literature that gap-graded mix designs are most commonly used when working with CRM asphalt cement mixes. The extra space in the gradation is used to incorporate the rubber particles in the voids of the mix and not in contact between the aggregates. The incorporation of the rubber particles in the voids

helps to prevent a "spongy" mix that rebounds after compaction. Therefore, an SMA mix was selected for use with a CRM asphalt cement binder. Due to the relatively high cost of this premium mix (it might require special aggregate), it is expected that it will be used in practice as a finishing layer in the pavement system, allowing for application of thin lifts. The large aggregate mix, designed at CSTT/NRC by El Hussein et al. will be used as a base layer. This mix is also a gap-graded mix and is not expected to be as expensive as SMA mixes. Both of these mixes have shown superior rutting resistance and with the incorporation of CRM asphalt cement should provide a very sound and long-lasting roadway. A description of both stone mastic asphalt (SMA) and the large aggregate mix (LAM) follows in the next sections.

3.2.1 SMA Mix

Splittmastixasphalt (SMA) was invented in Germany about 25 years ago to better resist the wear of studded tires (Bellin 1992). Studded tires are forbidden in most of the European countries except for example in Norway, where granite based SMA is used on the entire road network. SMA has been improved and used ever since, and is excellent for resisting permanent deformation (ie. rutting) and wear. Roads in Norway must be rehabilitated every 3-4 years because they allow the use of studded tires.

SMA (generally referred to in U.S. literature as stone matrix asphalt) is a gap-graded, dense, hot-mix asphalt with a large proportion of coarse aggregates (passing 2mm) limited to about 20 percent, all crushed material) and a rich asphalt cement/filler mastic. The coarse aggregate forms a high stability structural matrix and the engineered

asphalt cement, fine aggregate, filler and stabilization additive (fibre) form a mastic binding the structural matrix together (Emery et al. 1993). The relatively uniform size and shape (preferably cubical) of the aggregate provide for excellent point to point (surface/surface) contact which resists permanent deformations (see figure 3.2). It is the SMA requirement for cubical shaped aggregates (hard to obtain locally since most local quarry sources do not crush into cubical shaped particles), and the higher costs involved with SMA (usually around 20% due to premium materials) that it is not used much in North America. However, SMA trials in Ontario have proven successful with modifications to the European requirements (Emery et al. 1993).

An SMA design modelled on a mix reported by Bellin (1992) and produced with local aggregate supplies will be developed for use with a CRM asphalt cement binder. The mix design is presented in chapter 4 while the results are documented in the fifth chapter.

3.2.2 Large Aggregate Mix

The large aggregate mix (LAM) designed at CSTT/NRC (El Hussein et al. 1993) is also a gap-graded mix using aggregate stone with sizes up to 38 mm. This mix was designed to resist premature rutting of pavements constructed using hot mix asphalt (HMA). Stone-to-stone contact of the large aggregate provides similar resistance to rutting as SMA mixes (see figure 3.3). Since Marshall compaction and mix design procedures can only support paving mixes with aggregate particles less than 19 mm (3/4 inch), it was necessary to use a gyratory compactor with 15.2 cm (6") diameter molds.

HMA

SMA

Figure 3.2: Comparison of Floating Coarse Aggregate in HMA with Stone-to-Stone Contact of SMA

Figure 3.3: Stone-to-Stone Contact of the LAM

The design of the LAM can be obtained from the report by El Hussein et al. (1993).

Aggregate gradations and mix specifications are presented in chapter 6.

Chapter 4

MARSHALL MIX DESIGN PROGRAM FOR SMA MIX

In order to design an improved asphalt concrete pavement, mixes must be developed for both the control samples and for samples containing scrap tire rubber. Two of the most popular mix design programs in use today are the Marshall and Hveem methods. A study by Kandhal and Koehler (1985) has shown that the Marshall method is the most commonly used mix design method in the United States with 76 percent of all States using it. The Marshall mix design method is employed to produce the SMA mixes for this study due to the availability of equipment and its simplicity. The Asphalt Institute (1988) states that "The overall objective for the design of asphalt paving mixes is to determine an economical blend and gradation of aggregates (within the limits of the project specifications) and asphalt that yields a mix having:

- (1) *Sufficient asphalt* to ensure a durable pavement.
- (2) *Sufficient mix stability* to satisfy the demands of traffic without distortion or displacement.
- (3) *Sufficient voids* in the total compacted mix to allow for a slight amount of

additional compaction under traffic loading without flushing, bleeding, and the loss of stability, yet low enough to keep out harmful air and moisture.

(4) *Sufficient workability* to permit efficient placement of the mix without segregation."

This chapter presents the Marshall mix design program and the criteria on how the SMA mixes were designed.

4.1 Selection of Materials

A stone mastic asphalt mix (SMA) was used in this study due to favourable characteristics of the mix. SMA, as explained earlier, is a gap-graded, dense, hot-mix asphalt with a large proportion of coarse aggregate and a rich asphalt cement/filler mastic. The coarse aggregate, through point-to-point contact, forms a high stability structural skeleton to resist load-induced shear deformation (Figure 3.2). In a review of practical experience with SMA in Denmark, Finland, Germany, Netherlands, Norway, Sweden and Japan, by Emery et al. (1993), the following advantages of SMA were listed:

- "(1) high stability (resistance to rutting) combined with good durability (20 to 40 percent longer life than conventional mixes)
- (2) good resistance to studded tire wear
- (3) good frictional properties ('skid' resistance)
- (4) thin surface course use allows relatively low costs
- (5) good placement and compaction characteristics."

The features outlined above aided in the decision to develop a suitable SMA mix, however the most important characteristic of SMA is the gap gradation of the aggregates. The concept being sought in the new design is that the rubber particles introduced into the mix will reside in the voids around the stone aggregate (without preventing stone to stone contact) and will improve the resistance to rutting of the mix due to increased flexibility. The selection of materials for designing an SMA mix are presented in the following sections.

4.1.1 Aggregates

The coarse and fine aggregates used in this investigation were obtained from the H J McFarland Construction Co. in Nepean, Ontario. The stone mix prepared by McFarland to meet MTO specifications for HL3 asphalt concrete, crusher screenings, and sand used for the mix design had the gradations shown in Table 4.1.

The bulk relative density (BRD) of the coarse aggregate is 2.657 as determined by MTO standard test LS-604 specified in the MTO Laboratory Testing Manual. The fine aggregate has a BRD of 2.658 as resolved by LS-605.

The BRD of the limestone filler, which is used to meet the high percentage of aggregate passing sieve #200 (0.09 mm), was determined using a hydrometer analysis. The resulting value was 2.76 and the grain size analysis is presented in Figure 4.1. The filler was obtained from Permanent Lafarge Co. of Navan, Ontario.

Table 4.1: Aggregate Fractions Gradation

Sieve Size (mm)	Percent Passing		
	HL3	Crusher Screenings	Sand
16.0	100	-	-
11.0	90	-	-
8.0	47	100	-
#4	4	93	100
#10	1	56	93
#20	-	34	80
#40	-	22	49
#60	-	15	21
#100	-	10	5
#200	-	5	1

Figure 4.1: Grain Size Analysis of Filler Material

4.1.2 Binder

The selected asphalt binder, 85/100 Pen Asphalt (CAS 8052-42-4), was supplied by Husky Oil of Calgary, Alberta. This type of asphalt cement was readily available and is the most commonly used type in the Ottawa region. The selected type of asphalt influences its degree of interaction with the rubber. Softer asphalt grades are preferred because of the increased viscosity resulting from the "blending" of asphalt and rubber. Typical grades used for CRM asphalt cement include 60/70 up to 200/300 penetration asphalts.

A CRM asphalt cement binder produced by Bitumar Inc. of Montreal, Quebec was also used for comparison with standard asphalt cement and the laboratory manufactured CRM asphalt cement binder. The Bitumar product is called ECOFLEX 85/100 and contains 10% recycled scrap rubber. The Ecoflex process involves the complete dissolution of the crumb rubber to ensure permanent and irreversible homogeneity.

4.1.3 Rubber

It was determined from the literature review that the selection of rubber can influence the properties of the CRM asphalt cement blend to a significant degree (Chehovits 1989). Chehovits suggested that rubber used for asphalt-rubber should primarily be made from recycled pneumatic tires. It should also be ground on appropriate systems, and should be free from contaminants including mineral matter, fibre and metal. The rubber should be sufficiently dry to prevent foaming when added to hot asphalt. He also stated that mineral contaminants should not exceed 0.25%, the moisture content must

be less than 0.75%, and the rubber hydrocarbon content should be between 40 and 50%. Cryogenically ground whole tire rubber produced by Recovery Technologies Inc. (Toronto) was selected for use in this project and meets all the former stated requirements. The suggested gradation of rubber particles by Chehovits along with the gradation chosen (GTR-10) are presented in Table 4.2.

4.1.4 Fibre

SMA employs fibrous material to prevent asphalt cement runoff during production and laydown. A cellulosic based fibre was added to the mixture in the amount of 0.3% by total weight. The addition of cellulose fibres allows for a richer mastic in the mix by soaking up excess asphalt cement. This promotes a thicker film of binder which improves mix durability.

4.2 Manufacture of Mixes

The SMA mixes were made using standard Marshall compaction equipment. In determining the optimum asphalt content for each mix, a series of test specimens were prepared for a range of different asphalt contents usually from 2% below to 2% above expected optimum in 1/2 percent increments. To provide adequate data, each specimen was made in triplicate for each asphalt content used. Therefore a total of 15 specimens were prepared for each mix (3 at 5 different asphalt contents). With six different mixes involved, a total of 90 specimens were compacted for the B.R.D., stability, and flow requirements of the mix design phase. Another 30 specimens of loose uncompacted mix

Table 4.2: Rubber Gradation Specifications

Sieve Size	Percent Passing		
	Open-graded	Dense-graded	GTR-10
No. 8	100	100	100
No. 10	100	100	97-100
No. 16	75-100	95-100	---
No. 20	---	---	0-75
No. 30	25-60	70-100	---
No. 80	0-20	0-20	---
No. 200	0-5	0-5	---

were required to calculate the maximum relative densities (MRD). The equipment required for the preparation of test specimens can be found in section 3.04 of Manual Series No. 2 of the Asphalt Institute (Asphalt Institute 1988).

4.2.1 Aggregate Gradation

The gradation selected for this experiment was modelled according to an SMA gradation presented in a report by Bellin (1992). The aggregate blend and gradation specifications are shown in Table 4.3 and Figure 4.2 respectively.

In order to achieve this gradation, the available aggregates and screenings were sieved and put into designated containers. Each stone size, defined in the aggregate blend down to 2 mm in size, was kept separately. The requirement for stone size passing the 2 mm sieve and retained on the 0.09 mm sieve was satisfied by the use of sand, while the material passing the 0.09 mm sieve was achieved using limestone filler. Therefore, the required fraction of each aggregate could be portioned and mixed as required. The dashed line in Figure 4.2 represents the resulting gradation. The gradation of the selected mix tends to the fine side of the gradation limits. This selection was dictated by the economic use of available aggregate in the Ottawa region.

4.2.2 Asphalt-Rubber Binder Preparation

Lytton and Roberts (1987) stated that reaction times in the laboratory studies have varied from 0.5 to 2 hrs. at temperatures that typically ranged from 163°C (325°F) to 232°C (450°F). A reaction time of 1 hr at 191°C (375°F) is recommended in their study.

Table 4.3: Aggregate Blend

Sieve Size	% Passing
16 mm	100
11 mm	90-100
8 mm	50-80
5 mm	30-50
2 mm	20-30
0.09 mm	8-13

Figure 4.2: Gradation Specification for Splittmastixasphalt (SMA) Mix Design

Reaction time is the length of time that the crumb rubber particles are digesting in the asphalt cement at elevated temperatures while undergoing continuous stirring. Therefore, the CRM asphalt cement was produced as follows:

- The asphalt was placed in a 2000 ml beaker and then positioned on a hot plate. The asphalt was heated and stirred until a temperature of 191°C (375°F) was achieved.
- Rubber, in the required proportion, was added to the asphalt and continuously stirred for 1 hr. This was accomplished through the use of a two bladed electric stirrer. Partial digestion of the crumb rubber particles occurred at this stage.
- The CRM asphalt cement was then portioned and stored for use at a later date.

Some researchers have been using the CRM asphalt cement as soon as it is produced to manufacture samples. While this is desirable it is more feasible to batch produce the CRM asphalt cement for reheating. This approach was adopted by Lytton and Roberts (1987) and produces a more uniform and consistent binder from sample to sample.

4.2.3 Mixing Conditions

To produce a quality rubberized SMA mix certain procedures must be followed. Firstly, the mixing temperatures required are higher than for regular asphalt cement mixes. After reviewing the literature it was decided to employ mixing temperatures used by Vallergera (1981) since approximately these conditions seemed to be used by most

researchers. The binder and aggregate were heated to 177°C (350°F). Once this had been achieved the aggregate and binder were mixed in a large metal bowl as per MTO standard LS-261. In order to prevent segregation of fines in the mix, the filler was added after partial coating of the aggregates was achieved. Adding the fines (filler) after coating the coarse aggregates with asphalt cement prevents the filler from forming a layer around the coarse aggregates. A coating of filler on the coarse aggregate would result in poor asphalt to aggregate contact which in turn results in an inferior mix.

4.2.4 Compaction Conditions

After the completion of mixing, the samples were spooned into preheated Marshall molds. The Marshall compactor used allowed for the compaction of two samples simultaneously. The mixture was compacted with seventy-five blows on each side, corresponding to the Asphalt Institute guidelines for heavy traffic conditions. Since the compaction temperature of 163°C (325°F) suggested by Vallergera could not be achieved, a temperature of 149°C (300°F) was settled on. Using Vallergera's suggested asphalt and aggregate temperatures resulted in a temperature of around 300°F when compaction was commenced. After compaction the samples were allowed to cool and then extracted from the molds. The compaction procedure also followed MTO specification LS-261. The testing procedure used for selecting the proper asphalt content is outlined in the next section (4.3).

4.3 Marshall Testing Procedure

All tests performed on the samples were conducted in accordance with MTO specifications. A bituminous mix form is shown in LS-262 (Figure 4.3). No special provisions were taken when testing CRM asphalt cement specimens.

4.3.1 Bulk Relative Density

The specimens were tested in accordance with MTO LS-262 (Bulk Relative Density for Compacted Bituminous Mixtures). This test was performed one week after compaction. All loose particles on the surface of the specimen were carefully brushed off prior to testing.

4.3.2 Maximum Relative Density

The procedure, as outlined in MTO LS-264 (Theoretical Maximum Relative Density of Bituminous Paving Mixtures), was followed for this test. One sample was tested for each different binder and binder percentage used.

4.3.3 Stability and Flow

Stability and flow tests were performed as per MTO LS-263 (Method of Test for Resistance to Plastic Flow of Bituminous Mixtures using Marshall Apparatus). Three samples were tested for each binder percentage used.

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BITUMINOUS MIX FORM

MARSHALL TEST VALUES

	BRICQUETTE NUMBER			AVERAGE
	1	2	3	
STABILITY (DIAL)				
*FLOW NO 1 (s)				
FLOW NO 2 (s)				
AVERAGE FLOW (s)				

VISUAL OBSERVATIONS

ONE APPEARANCE	D, M, B, VB
BRICQUETTE APPEARANCE	D, M, B, SF, P
COATING	P, F, G
STRIPPING	MR, SI, M, H
C AGG FRACTURE	MR, SI, M, H

SUMMARY OF TEST RESULTS

VOIDS (%)	* J D
FLOW (25 mm)	
CORRECTED STABILITY (M)	
* V M A (%)	

CONTRACT NO. _____

DATE _____

SAMPLE NO. _____

REMARKS: _____

C/A _____

SAMO _____

SA/SCR _____

HL _____ %A C

% PASS 4.75mm _____

BLEND _____ SD ☐

AGGREGATE SAMPLE NO. _____

	BRICQUETTE NUMBER			AVERAGE
	1	2	3	
A ₁ MASS OF COMPACTED SPECIMEN IN AIR				
A ₂ SURFACE DRY MASS OF SPECIMEN IN AIR AFTER IMMERSION IN WATER				
B ₁ MASS OF COMPACTED SPECIMEN IN WATER				
B ₂ VOLUME = A ₁ - B ₁				
C PURE RELATIVE DENSITY = $\frac{A_1}{B_1}$				
D MASS OF PLAS AND MIXTURE IN AIR				
E MASS OF PLAS IN AIR				
F MASS OF MIXTURE IN AIR = D - E				
F ₁ SURFACE DRY MASS OF MIXTURE IN AIR				
G MASS OF PLAS & MIXTURE IN WATER				
H MASS OF PLAS IN WATER				
I ₁ MASS OF MIXTURE IN WATER = G - H				
I ₂ VOLUME = F - I ₁				
I ₃ S D VOLUME = F ₁ - I ₁				
J MAXIMUM RELATIVE DENSITY = $\frac{F}{I_1}$				
J ₁ S D MAXIMUM RELATIVE DENSITY = $\frac{F}{I_1}$				
K PERCENT VOIDS IN MIXTURE = $\frac{J - J_1}{J} \times 100$				
K ₁ S D PERCENT VOIDS IN MIXTURE = $\frac{J - J_1}{J} \times 100$				
L VOIDS FILLED WITH ASPHALT = $\frac{V_{MA} - \text{VOIDS IN MIXTURE}}{V_{MA}}$				

* S D MEANS SURFACE DRY
 ** V M A CALCULATED ON % VOIDS IN MINERAL AGGREGATE FORM

Figure 4.3: MTO Marshall Mix Design Form

4.3.4 Air Voids (AV) and Voids in Mineral Aggregate (VMA)

The AV's were determined by method MTO LS-265 (Determination of Percent Air Voids in Compacted Dense Bituminous Pavement Mixtures). One AV percentage was calculated for each different binder percentage.

Test method MTO LS-266 (Determination of VMA in Compacted Bituminous Mixtures) was used for VMA calculations. One number was calculated for each binder percentage.

Chapter 5

MARSHALL MIX DESIGN RESULTS FOR SMA MIX

This chapter presents the major results of the Marshall mix design program for the control, Ecoflex, and CRM asphalt cement SMA mixes as described in the previous chapter.

The results of each of the mixes (Control, Ecoflex, 5%, 10%, 15%, and 20% rubber) are presented first with comparisons between the mixes documented in the latter part of this chapter. A more detailed treatment and analysis is carried out in Chapter 9.

Using the Marshall design criteria (Table 5.1) for heavy traffic and following the procedures for Marshall mix design, the optimum percentage of binder can be determined for the control, Ecoflex, and rubber mixes. The values for BRD, %AV, %VMA, stability and flow are then determined at the optimum percentage, and are presented in Table 5.3.

The design criteria have been altered to accommodate CRM asphalt cement mixes as per Chehovits (1989). Also the voids in mineral aggregate criteria may be relaxed to a minimum of 12% (Foster 1986). Foster has shown that VMA does not correlate well with performance and that specifying a minimum VMA as high as 14% cannot be justified.

Table 5.1: Marshall Design Criteria

Marshall Method Mix Criteria	Heavy Traffic	
	Min.	Max.
# of blows	75	
Stability (lbs)	1800	--
Flow (0.01 in)	8	22
% Air Voids	3	5
% VMA (3/4 NMA)	14	--

5.1 Optimum Mix Determination for Control and Rubber Samples

The mix design results for the control, Ecoflex, and CRM SMA mixes are presented in Table 5.2. These results are subsequently plotted in Figures 5.2 to 5.7 respectively and the values are connected using a "best-fit" curve. From these mix design results the optimum asphalt content for each mix was determined. Once this was accomplished, performance testing was carried out on the optimum sample of each mix and comparisons were made between the various mixes.

Using the Marshall design criteria (Table 5.1) for heavy traffic and following the procedures for Marshall mix design, the optimum percentage of binder was determined for the control, ecoflex, and CRM mixes. The optimum binder percentage was obtained by considering the following properties:

- maximum stability
- maximum BRD
- median of limits given for the percentage of AV.

The optimum asphalt content of the mix was then calculated as the numerical average of the values for the asphalt content determined for each of the above.

The optimum percentage of binder calculations are presented in Table 5.3. The mix properties, ie. BRD, %AV, etc., were then determined at the optimum percentage, and are also presented in Table 5.3. A summary of the results at this stage are outlined in the next section.

Table 5.2 - Results from Marshall, BRD and MRD tests.

0% GTR-10					
% Binder	BRD	Stability	Flow	%AV	%VMA
4.5	2.4343	2679	0.1495	2.5	12.7
5.0	2.4690	2425	0.1345	0.8	11.9
5.5	2.4623	1886	0.1879	0.2	12.6
6.0	2.4353	1597	0.2227	0.1	14.0
6.5	2.4268	1438	0.2074	0.3	14.8
5% GTR-10					
4.0	2.3500	3272	0.1039	6.8	15.4
4.5	2.3900	3111	0.1132	4.4	14.3
5.0	2.4340	3078	0.1076	2.0	13.1
5.5	2.4310	2735	0.1288	1.3	13.7
6.0	2.4220	2165	0.1528	---	14.5
10% GTR-10					
4.0	2.3200	3281	0.1222	8.1	16.3
4.5	2.3790	3423	0.1269	5.4	14.6
5.0	2.3780	3087	0.1457	3.9	15.1
5.5	2.3960	2558	0.1413	2.5	14.9
6.0	2.3760	2356	0.1478	3.5	16.1
15% GTR-15					
4.0	2.2900	2618	0.1300	9.1	17.3
4.5	2.3200	2689	0.1300	7.4	16.7
5.0	2.3100	2476	0.1500	6.9	17.4
5.5	2.3300	2414	0.1400	5.8	17.4
6.0	2.3300	2170	0.1600	4.8	17.6
20% GTR-10					
4.0	2.2400	2205	0.1920	---	19.2
4.5	2.2490	2039	0.1590	9.8	19.3
5.0	2.2805	2478	0.1371	7.8	18.6
5.5	2.2747	2057	0.1729	7.9	19.3
6.0	2.2867	2082	0.1809	6.1	19.3

Table 5.3 - Asphalt properties at optimum binder content.

	Asphalt Properties				
	5.0	5.0	5.5	6.0	6.0
	4.3	4.7	5.0	6.4	7.0
	4.5	4.0	4.5	4.5	5.0
	4.6	4.6	5.0	5.6	6.0
	14.5	11.0	14.5	15.0	18.0
	2650	3120	3110	2400	2080
	12.5	14.0	15.2	17.4	19.25
	2.25	4.0	4.0	5.5	6.0
	2.445	2.400	2.380	2.327	2.285

5.2 Summary of Results

In order to better visualize the effects of adding rubber to the SMA mix, the results of Table 5.3 are plotted in Figure 5.1. A quick discussion of the results follows. A more detailed analysis is presented in Chapter 9.

5.2.1 Bulk Relative Density

The BRD of the CRM asphalt cement samples decreased as the percentage of rubber increased. One cause may be due to higher concentrations of scrap rubber and undigested rubber particles occupying spaces meant for aggregates which creates Marshall specimens that are not as dense due to an increase in height. Although the "gap-gradation" is ideal for incorporating a certain amount of rubber particles in void spaces, there may not be enough space for higher concentrations of crumb tire rubber in this particular SMA mix. One method to alleviate such a problem is to remove aggregate fractions of the same size as the rubber gradation to make more space available for the rubber particles. Removing certain aggregate fractions is very time consuming and impracticable.

Also, the rubber particles have a lower specific gravity than stones which will reduce the BRD of the specimen.

5.2.2 Stability and Flow

Figure 5.1 shows that the stability of the CRM asphalt cement mixes increased with rubber content increases of up to about 10% and then the stability subsequently

decreased with further rubber content increases up to 15% and 20%. The increase in stability is due to the enhanced viscosity and stiffness of the binder due to the addition of rubber. This more viscous binder is at an optimum at 10% rubber and the extra rubber (ie. 15 and 20%) might act to destabilize the sample at higher concentrations.

The flow values of the mixes decreased up to approximately 10% of rubber content and then subsequently increased. This is a mirror process of what happened to the mix stability and is also attributed to a more viscous binder less susceptible to flow.

5.2.3 Air Voids

The AV increased linearly as the amount of rubber was increased. This may be attributed to the rebounding of the sample, due to the presence of rubber particles, after application of the compaction hammer. Although the rebound would be small, it still may be enough to prevent optimal compaction and thus results in increased AV's.

5.2.4 Voids in Mineral Aggregate

The VMA also increased as the percentage of rubber increased, however, this can also be attributed to the increase in AV's. Since the VMA is described as "*the volume of intergranular void space between the aggregate particles of a compacted paving mixture that includes the air voids and the effective asphalt content, expressed as a percent of the total volume of the sample*" (Asphalt Institute 1988), it can be shown that by subtracting the percent of AV's from the percent VMA, the value for the effective asphalt content remains virtually unchanged (Table 5.4). This indicates that the change

Figure 5.1: Comparison of 5 different SMA Mixes

Table 5.4 - %VMA analysis.

Optimum Asphalt Content	% Asphalt Replaced by Rubber	% Air Voids (A)	% VMA (B)	Unabsorbed Asphalt (B-A)
4.6	0	2.3	12.5	10.2
4.6	5	4.0	14.0	10.0
5.0	10	4.0	15.2	11.2
5.6	15	5.5	17.4	11.9
6.0	20	6.0	19.3	13.3

in VM . is due solely to the increase in AV's. This pattern holds reasonably true for a binder of CRM asphalt cement with up to 20% rubber particles. The change in this pattern may be attributed to a non optimum amount of rubber in the mix.

5.2.5 General Results

From Figure 5.1 it can be concluded that the mix containing 10% of rubber in asphalt is the most promising thus far. Takalou and Sainton (1993) tested the effect of three different crumb rubber contents on the viscosity, softening point, resilience, and penetrability of CRM asphalt cement binder. Their test results indicate that the optimum properties can be achieved at 10 percent crumb rubber content. It can be speculated that the binder with the optimum properties will produce the optimum mix.

The stability for a binder with 10 percent ground tire rubber is at an optimum while the flow is still quite low (14.5). The AV's and VMA are well in the range of the design criteria. If the Marshall mix design is the only criteria for selecting a mix, CRM asphalt cement containing 10% recycled crumb rubber would be favoured.

However, the Marshall mix design method is a poor predictor of performance. Performance related testing must be conducted in order to obtain a more accurate comparison between mixes. Chapter 7 outlines a performance related testing program designed for this purpose. Chapter 6 describes the specifications for the large aggregate mixes. These mixes (control and 10% rubber) will be tested for performance in the same manner as the SMA mixes.

Figure 5.2: Marshall Mix Design for SMA mix (Control)

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS
STANDARD REFERENCE MATERIAL 1010a
(ANSI and ISO TEST CHART No. 2)

Figure 5.3: Marshall Mix Design for SMA Mix (Ecoflex)

Figure 5.4: Marshall Mix Design for SMA Mix (5% Rubber)

Figure 5.5: Marshall Mix Design for SMA Mix (10% Rubber)

Figure 5.6: Marshall Mix Design for SMA Mix (15% Rubber)

Figure 5.7: Marshall Mix Design for SMA Mix (20% Rubber)

Chapter 6

LAM Specifications

The large aggregate mix (LAM) was designed at Centre for Surface Transportation Technology at the Canadian National Research Council (CSTT/NRC) by El Hussein et al. (1993). The LAM was gyratory compacted in a 15.2 cm (6") diameter mold to accommodate the large stone sizes used in this mix. This chapter presents the materials used for the mix, the design asphalt content for the control mix, as well as the selection of a CRM asphalt cement binder for the LAM.

6.1 Selection of Materials

The aggregates used in this investigation were obtained from the H J McFarland Construction Co. in Nepean. This is the same aggregate supply used for the SMA mix (section 4.1.1). The gradation of the HL3, crusher screenings and sand can be seen in Table 4.1. The 38 mm size aggregate with the gradation presented in Table 6.1 was the only other stone component used for the LAM design.

The binder selected for the LAM was also the same binder used for the SMA mixes. Refer to Section 4.1.2 for details of the binder used.

Table 6.1: Aggregate Fractions Gradation

	Sieve Size (mm)						
	(Percent Passing)						
Aggregate	37.5	25.0	16.0	11.0	8.0	#4	#10
38 mm	100	90	25	5	2	2	1

GTR-10 cryogenically ground tire rubber was used with the SMA mix. Since the size of the sample and of the aggregates was larger for the LAM a larger rubber gradation was also selected. The other reason to use larger and cheaper crumb rubber in this mix was to keep the cost of the LAM as low as possible. The gradation of the GTR-4 ground tire rubber used with the LAM is displayed in Table 6.2.

6.2 Manufacture of Mix

The manufacture of the LAM was similar but not identical to the manufacture of the SMA mix (since Marshall equipment could not be used) with some exceptions outlined in the following sections.

6.2.1 Aggregate Gradation

The gradation selected for this experiment is documented in a report by El Hussein et al. (1993). The aggregate blend and gradation specifications are shown in Table 6.3 and Figure 6.1 respectively.

In order to achieve this sort of gradation, the following fractions of aggregate used was:

- 52% 38 mm McFarland Stone
- 18% HL3 McFarland Stone
- 12% quarry screening
- 18% field sand

Table 6.2: Rubber Gradation Specifications

	Sieve Size (mm)				
	(Percent Passing)				
Rubber	No.8	No.10	No.16	No.20	No.30
GTR-4	100	0-75	--	--	--

Table 6.3: Aggregate blend for LAM

Sieve Size	% Passing
37.5 mm	100
25.0 mm	95
16.0 mm	61
11.0 mm	49
8.0 mm	40
# 4	31
# 10	24
# 20	19
# 40	12
# 60	6
# 100	3
# 200	1

As can be seen in Figure 6.1, the LAM was designed as gap-graded like the SMA mix. This design was favourable for incorporating ground tire rubber in the voids around the aggregates. The GTR 4 rubber particles are mostly the same size as the openings in the number 10 sieve. Figure 6.1 shows that there is a suitable gap in the gradation for this size of particle.

6.2.2 CRM Asphalt Cement Binder Preparation and Mixing Conditions

The preparation of the CRM asphalt cement is identical to the procedure outlined in Section 4.2.2.

The mixing conditions employed during the manufacture of the LAM samples is outlined as follows:

Binder Temp. : 175 °C

Aggregate Temp. : 175 °C

Mixing Temp. : 160 °C

When the binder reached the required temperature it was added to the coarse aggregate (32 mm and HL3) in a mixing bowl. The mixture was hand mixed with a large metal spoon until the coarse aggregate was sufficiently coated. The fine aggregates (screenings and sand) were subsequently added and mixing was continued until the aggregates were completely coated. The mixture was then spooned into a preheated mold and tamped, ready for gyratory compaction.

Figure 6.1: Gradation of gap-graded LAM

6.2.3 Compaction Conditions

Circular paper discs were used on the top and bottom of the sample to prevent the mixture from sticking to the mold and loading head. The mix was compacted using a Rainhart series 142 gyratory compactor. The gyratory compactor allows for the compaction of only one sample at a time. The following compaction conditions were present:

Ram Pressure : 600 kPa

of Gyration : 250

Angle of Gyration : 1 °

Compaction Temp. : 160 °C

The refusal density for the LAM was determined by Hussein et al. (1993) to be 250 gyrations. An equal compactive effort was produced for the CRM asphalt cement specimens. The samples were allowed to cool before extrusion from the molds. The gyratory compactor produced cylindrical cores with a diameter of 15.2 cm (6"). The cylindrical shape of the samples was preferred for the performance testing involved with this study. Refer to Figure 6.2 for a schematic of the gyratory compactor.

The gyratory compactor was employed for manufacturing the LAM for two primary reasons. First, the Marshall compactor is not capable of compacting 15.2 cm (6") cores required for the production of the LAM. Secondly, a report by Consuegra et al. (1989) shows that the gyratory compactor demonstrated the ability to produce mixtures with engineering properties nearest those determined from field cores. In a final rating of five compaction devices, Consuegra et al. gave the following ranking:

1. Texas gyratory shear compactor
2. California kneading compactor
3. mobile steel wheel simulator
4. Arizona vibratory-kneading compactor
5. Marshall mechanical hammer.

The Marshall method of compaction displayed the poorest performance with respect to properties nearest those determined from field cores. The absence of the kneading effect during the mix compaction, which is due to the uniform impact type load applied by the Marshall hammer, is probably the major reason behind the poor performance. Kneading of the mix allows for the rearrangement of aggregates which allows for a denser mix. Although Marshall compaction is inferior to the other displayed types it is still very popular in the United States and many other countries. The vast quantity of historical empirical data obtainable from the Marshall mix design make this method attractive. While gyratory shear compaction is proven to be superior, a lack of data correlating results with field trials reduces the widespread usage of this compaction method. However, increased use of the gyratory shear compactor (expected due to the SHRP program), will provide the necessary database resulting in the replacement of the Marshall method of compaction in the near future.

Figure 6.2: Rainhart Series 142 Gyratory Shear Compactor

6.3 Selection Criteria for Performance Based Testing

The optimum asphalt content for the LAM control mix was determined by El Hussein et al. (1993) to be 4% by weight of sample. After review of the Marshall mix design for SMA mixes it was determined that the CRM asphalt cement binder containing 10% ground tire rubber was the optimum asphalt rubber mix. A look at Table 5.3 shows an increase of about 0.5% for the optimum binder content for each 5% increase in the amount of rubber in the binder starting at 5%. Therefore, due to time constraints, a binder with 10% ground tire rubber was used for comparison with the large aggregate control mix. The optimum asphalt content for the CRM asphalt cement binder was selected at 4.5%.

In summary, performance tests for the LAM were conducted on a control mix with a 4% asphalt binder content and on a mix with a CRM asphalt cement binder containing 10% of recycled crumb tire rubber. The mix with the CRM asphalt cement binder had an optimum binder content of 4.5% by weight of the samples.

Chapter 7

PERFORMANCE BASED TESTING PROGRAM

Once the optimum mixes were designed (Marshall program) for the control and rubber mixes, performance tests were utilized to predict field performance of the various samples. After the initial mix design process (Figure 3.1), accelerated performance tests were carried out in order to determine the job mix formula. The Marshall mix design program was useful for determining asphalt content for mixes but it has very limited performance prediction capabilities. Performance tests, such as rutting and fatigue, are useful for predicting field performance, but they are slow, time-consuming and inappropriate for determining asphalt content. Therefore, the best approach would be to combine the Marshall mix design for the initial design phase with performance testing to determine the characteristics of the mix and predict its performance in the field.

This chapter describes the performance tests carried out on the lab specimens as well as the equipment used for this purpose.

7.1 Test Apparatus

A schematic diagram of the test apparatus is shown in Figure 7.1. The apparatus consisted of a hydraulic actuator and controller, monitoring devices, a data acquisition system, and a temperature controlled chamber. These items are described in the following sections.

7.1.1 Hydraulic Actuator and Controller

All performance tests conducted on the specimens used an MTS hydraulic actuator with an MTS 458.20 MicroConsole controller. The hydraulic actuator was equipped with a load cell and an internal linear variable displacement transducer (LVDT). The actuator controller could be used for displacement or load control depending on the test setup required.

External function controllers were also used due to the limitations of the built in function generator. A Wavetek Model 182A 4-MHz function generator along with a Hewlett Packard 8111A Pulse/Function 20-MHz generator were required to generate haversine and pulsed square waves. A Tektronix 2212 80 MHz Digital Storage Oscilloscope was employed to ensure that the proper loading function was produced.

7.1.2 Instrumentation and Data Acquisition System

The instrumentation part for the experimental investigation can be divided into two groups, load and displacement measurement. The load measurements consisted of monitoring the compressive loads by the built-in load cell of the hydraulic actuator. The

Figure 7.1: Schematic of Performance Tests Setup

stroke displacement of the hydraulic actuator was measured by the internal LVDT. The data were collected at prescribed intervals and stored in microcomputer files for further data reduction.

The load and displacement was monitored using Labtech Notebook software on a 386 microcomputer and displayed on the computer screen for direct observation. The data were also stored on the hard disk to allow further analysis.

7.1.3 Temperature Controlled Chamber

A chamber was constructed around the hydraulic actuator to allow samples to be tested under climate control. The chamber was constructed out of wood with styrofoam insulation and a window was installed for visual observation of the specimen under testing. A YSI Model 71A Thermistemp Temperature Controller was employed to control the temperature to a preset level.

7.2 Description of Testing Methods

Three different performance related tests were conducted. They were an indirect tensile strength test (ITS), a fatigue test, and a repetitive uniaxial compression test (RUCT) for evaluating rutting resistance. Resilient modulus (M_R) values were not obtained due to unavailability of the required equipment. The tests are discussed in detail in the following sections.

7.2.1 Indirect Tensile Strength Test

The ITS test was conducted by loading a cylindrical specimen with a single compressive load which acts parallel to and along the vertical diametral plane (Figure 7.2). The stress condition developed by this test acts perpendicular to the loading direction and along the diametral plane. The development of stresses within the cylindrical specimen subjected to the compressive load is documented by Kennedy and Hudson (1968a). The reader is referred to the report by Kennedy and Hudson if interest in the stress conditions is present. In order to achieve initial failure from the centre of the specimen and not along the loading zone a curved loading strip is used to distribute the load over an enlarged area.

The ITS test is a strain controlled test and is performed at a loading rate of 51 mm (2 in.) a minute (Kennedy 1977). The resulting load-deformation relationship was recorded by the data acquisition system. A sample test is displayed in Figure 7.3. The resulting tensile strength (S_T) of the specimen was calculated by the following equation from Kennedy:

$$S_T = \frac{P_{Fail}}{h} \times A_o$$

where S_T = tensile strength in kPa

P_{Fail} = maximum load at failure (kgs)

h = height of specimen (cm)

A_o = a constant given as 6.026 for a Marshall specimen (10.2 cm dia.)

4.056 for a 15.2 cm (6") dia. specimen

Figure 7.2: Loading Setup for ITS and Fatigue Tests

Figure 7.3: Indirect Tensile Strength Test

The value for the indirect tensile strength was needed to determine loading conditions for fatigue testing. Three specimens were tested for each mix type.

7.2.2 Fatigue Testing

The fatigue testing performed on the samples followed the procedure defined by Stolle (1990). Stolle's test method was selected since it incorporated testing cylindrical specimens as opposed to beams recommended by SHRP. The setup for the fatigue test was the same as the indirect tensile strength test. However, the loading for fatigue testing was a stress controlled load. A compressive Haversine wave at a frequency of 1 Hz was used to maintain uniform cyclic loading (Figure 7.4).

Specimens were failed in a fatigue mode using either 15% or 30% of the static strength determined previously. A sample fatigue test graph is shown in Figure 7.5.

Stolle defines the fatigue failure (N_f) of the specimen as the point at which the rate of accumulated irrecoverable displacement appeared to accelerate. This point corresponds to the intersection of the lines marking the line of constant accumulated deformation and a line tangent to the curve at the point of maximum curvature. At this stage of the loading history of the specimen a well-defined crack started to develop along the vertical diameter of the sample. Three fatigue tests were conducted at each load for each mix type. The results were subsequently plotted on an applied tensile stress versus cycles to failure graph for comparison (Chapter 8).

Figure 7.4: Fatigue Test Loading Function

Figure 7.5: Sample Fatigue Data

7.2.3 Repetitive Uniaxial Compression Test

There are numerous different test setups in use today to analyze permanent deformation or rutting in asphalt samples. One common type of rutting test is a wheel-tracking test that involves testing asphalt slabs whose borders are confined. A rubber tired wheel is passed back and forth over the surface and the deformation is measured. This test is intended to simulate field loading and is very time consuming in both execution and sample preparation. Other tests to measure rutting are uniaxial, triaxial, or simple shear tests. A creep test is an example of a simple shear test, however it is not entirely accurate to correlate creep data with field performance. The test selected to determine rutting resistance (through permanent shear deformation) for this study is a repetitive uniaxial compression test (RUCT) normally used at the Centre for Surface Transportation Technology of the National Research Council of Canada (CSTT/NRC). This test is explained in detail in a paper by El Hussein and Yue (1993).

The RUCT uses cylindrical cores and the MTS hydraulic actuator for testing. A cylindrical plate is placed on the flat side of the specimen and a maximum load of 690 kPa (100 psi) is applied to the sample (Figure 7.6). A thin rubber pad is placed between the loading plate and the specimen. A minimum load of 5 kg is used on the rest cycle to avoid separation of the loading head which would result in impact loading. The loading function is a square wave load application based on research work conducted at Nottingham University (Brown and Brunton 1984) used to simulate traffic loading (Figure 7.7).

Figure 7.6: Loading Setup for RUCT

Figure 7.7: RUCT Loading Function

The square wave consists of a loading period of 0.2 seconds followed by a 1.8 second rest period. The tests were conducted at 25°C and 40°C to determine the mixes rutting susceptibility to increased temperature. Each mix was tested three times and representative result was selected for comparison.

The output from the RUCT was similar to the sample output shown in Figure 7.8. A Fortran program removed the elastic portion of the deformation from the total deformation acquired during the test to provide a smooth curve. Another computer program was used to determine the rate of permanent deformation, K , during the secondary stage using linear regression. El Hussein and Yue (1993) hypothesized that the accumulated permanent axial strain undergoes three distinct stages; primary, secondary, and tertiary (Figure 7.9). These stages correspond to strain-hardening, shear flow and fracture failure. During the secondary stage (shear flow), the rate of accumulation of permanent deformation is constant and is sensitive to the rutting resistance of the asphalt mix. A brief description of each stage follows:

1. *Primary stage:* large deformation occurs upon the initial load application stage.

This stage lasts only a few cycles whereupon a rapid decrease in the rate of deformation is observed. This initial relatively large deformation may be due to the irregular surface of the sample which leads to stress concentrations at these points. Another factor may be densification of the mix as a result of reduced air voids.

Figure 7.8: RUCT output

Figure 7.9: Stages of Axial Strain (after El Hussein and Yue 1993)

2. *Secondary Stage:* During this stage the rate of accumulated permanent deformation remains constant. Once densification has been achieved and surface irregularities have been removed, shear flow is the primary mechanism of deformation. The rate of deformation during this stage is a constant (K) which indicates a mixes ability to resist rutting. A low K value indicates good rutting resistance.
3. *Tertiary Stage:* In this stage the rate of deformation increases until complete failure occurs. The formation of cracks is present in this stage which may indicate failure due to fatigue. As of this time it is uncertain what exactly occurs during this stage and further study is in progress by El Hussein and Yue.

In general, further R&D is necessary in order to improve rutting laboratory tests which could eventually replace expensive and awkward passing wheel tests.

7.3 Test Procedure Summary

A summary of the testing program is given in Table 7.1. A total of six different SMA mixes were tested. To complete the SMA testing program a minimum of ninety samples was required.

On the basis of experience with CRM SMA mixes, only a control and one CRM mix were tested for the LAM. This simplified program added another 30 samples to complete the LAM testing program. Therefore, a total of 120 samples were required for the performance testing component of the thesis.

Table 7.1: Test Summary

Test Type		Temp.	# of samples required per mix	Age of sample (days)		Approximate length of test
				SMA	LAM	
ITS		25 °C	3	28-29	8	5 min.
FATIGUE	15% max load	25 °C	3	28-32	7	1 - 4.5 hrs.
	30% max load	25 °C	3	28-32	7	10 - 25 min.
Rutting		25 °C	3	64-68	7	stopped @ 3 hrs.
		40 °C	3	62-65	7	1 - 3 hrs.

The samples were tested at the various ages due to the availability of the test equipment. Rescheduling was required a number of times to work out conflicts. The slight difference in the sample age for a particular test (SMA mixes only) was insignificant to the physical properties of a test specimen due to its advanced age.

Chapter 8

PERFORMANCE BASED TESTING RESULTS

This chapter presents the major results of the performance testing program conducted on the control and CRM asphalt cement SMA and large aggregate mixes.

The results obtained from the indirect tensile strength test, fatigue test, and compression test are presented in order. The optimum mix for each mix type was tested. Comparisons between the various optimum mixes were also performed.

This chapter also presents a preliminary discussion of the test results, a more detailed treatment and analysis are carried out in chapter 9.

8.1 ITS Results

Three specimens were tested for each SMA mix. The average indirect tensile strength for each mix is presented in Figure 8.1. Figure 8.2 shows the ITS test results for the large aggregate mixes. The following results were observed:

Figure 8.1: Indirect Tensile Strength of Optimum SMA mixes

Figure 8.2: Indirect Tensile Strength of Optimum LAM Mixes

SMA mix

The addition of crumb tire rubber to the binder has increased the indirect tensile strength for three of the four rubber mixes. The strength increased by 16.7%, 18.3%, and 7.0% for the mixes containing 5%, 10%, and 15% crumb rubber respectively. The strength value for the 20% crumb rubber mix is very close to the control mix value.

LAM

The addition of crumb rubber to the binder decreased the indirect tensile strength from 610.4 to 516.6 kPa. This 15.4 % decrease in the ITS of the large aggregate mix is not consistent with the SMA results. The addition of the larger rubber particles (GTR 4) has changed the characteristics of the binder. This behaviour is assumed because a smaller percentage of rubber in the particles is dissolved in the asphalt due to their larger size, thus, there is a failure to change the binder properties to the same extent as occurred with the smaller size particles. The undissolved rubber particles act more as an aggregate when introduced to the mix. Perhaps the introduction of the large size rubber particles changes the tendency of the resultant mix to more closely imitate a dry-process than a wet-process mix. It is fair to conclude that the influence of the size of crumb rubber particles will be important in further investigations

The maximum selected load used for the fatigue testing was based on a certain percentage of the maximum indirect tensile strength. Therefore, a greater load was used when testing the 5, 10, and 15 percent rubber content SMA mixes in fatigue compared

to the control SMA mix.. The fatigue tests for the CRM asphalt cement LAM will incorporate a slightly smaller load than the control LAM.

8.2 Fatigue Testing Results

Once again three samples were tested for each mix and for each applied tensile load. Figures 8.3 and 8.4 clearly show the effects of adding rubber to the asphalt cement in the SMA mixes. The fatigue results for the LAM are presented in Figure 8.5.

SMA mix

All the rubber modified asphalt cements display a superior fatigue performance than the control mix. The CRM asphalt cement mixes display a fatigue life of 1.5 to 3 times the life of the control mix at all stress levels. The Ecoflex mix did not show the same levels of improvement compared to the laboratory prepared CRM asphalt cement mixes. The ecoflex asphalt cement is a gel-like homogenous material with completely dissolved rubber particles. Rubber particles are not visible in the mix. This condition will be discussed further in the next chapter.

LAM

The fatigue test results for the large aggregate mix are not as conclusive as the SMA mix results. Fatigue life improvement, if any, is not obvious. The only conclusion that might be drawn is that an increase in the fatigue life for the lower stress level of the CRM asphalt cement LAM was observed.

Figure 8.3: Fatigue test results for control and rubber SMA mixes

Figure 8.4: Fatigue test results for control and ecoflex SMA mixes

Figure 8.5: Fatigue test results for LAM

8.3 Repetitive Uniaxial Compression Test Results

As stated earlier the RUCT was performed at temperatures of 25 °C and 40 °C. Three samples were tested for each test condition and a representative test sample was chosen for comparisons between mixes. The results are presented in the following sections.

8.3.1 Repetitive Uniaxial Compression Test at 25 °C

The CRM asphalt cement and Ecoflex SMA mixes were less susceptible to deformation than the control mix (Figures 8.6 and 8.7). The CRM asphalt cement LAM shows an inferior resistance to rutting than the control LAM (Figure 8.8).

SMA mix

The 5% rubber mix showed the least accumulated axial strain of all the mixes. The 10% rubber mix showed almost equal performance and the 15% and 20% rubber mix showed a slight improvement over the control sample. The slopes of the rubber mixes during the secondary stage appear to be equal and smaller than the control mix slope. However, the initial large deformation of the 15% and 20% mixes in comparison to the 5% and 10% mix seems to be the only factor for their increased permanent axial strain. This is verified in section 8.3.2. The reason for such a large initial deformation may be due to rubber particles on the surface of the mix providing surface irregularities.

The Ecoflex mix also showed considerable improvement when compared to the control sample. This improvement in permanent deformation may be due to increased

Figure 8.6: Effect of mix type on accumulated permanent axial strain tested at 25 °C (SMA mixes)

Figure 8.7: Effect of mix type (continued) on accumulated permanent axial strain test at 25 °C (SMA mixes)

Figure 8.8: Effect of mix type on accumulated permanent axial strain tested at 25 °C (LAM)

viscosity of the binder.

LAM

The CRM asphalt cement LAM shows an obvious reduction in rutting resistance. The initial deformation is much larger and the rate of deformation also appears to be greater. It is quite evident that the CRM asphalt cement LAM performs inferior to the control LAM possibly due to the use of larger crumb rubber particles as explained above.

8.3.2 Repetitive Uniaxial Compression Test at 40 °C

SMA mix

Once again the CRM asphalt cement mixes show superior performance to the unmodified SMA control mix. Although a greater initial axial deformation is observed for the 15% and 20% rubber mixes (Figure 8.9), probably due to surface irregularities caused by the presence of a greater amount of rubber particles, their K values are still smaller than the K value for the control mix (Figure 8.12). This indicates that the CRM asphalt cement SMA samples resist permanent deformation to a greater extent than the control mix even at 40 °C.

An improvement is also noted for the Ecoflex mix but not to the extent seen at 25°C (Figure 8.10). It seems that there is an improvement only in the primary stage.

LAM

The inferior performance of the CRM asphalt cement LAM is even more evident at 40°C than it was at 25°C (Figure 8.11). The CRM asphalt cement samples deteriorated very quickly under the rutting load at the elevated temperature.

Figure 8.9: Effect of mix type on accumulated permanent axial strain tested at 40 °C. (SMA mixes)

Figure 8.10: Effect of mix type (continued) of accumulated permanent axial strain tested at 40 °C. (SMA mixes)

Figure 8.11: Effect of mix type on accumulated permanent axial strain test at 40 °C (LAM)

8.3.3 K-Value Comparisons

SMA mix

Figure 8.12 verifies that the rate of permanent axial strain at 25 °C for the mixes containing 15% and 20% rubber are indeed smaller than the control sample rate though the initial deformation in the primary stage is close. The 5% and 10% rubber samples show the highest rutting resistance of the SMA mixes.

The K values for the CRM asphalt cement mixes at 40 °C are clearly superior to the control sample (Figure 8.13). The rate of permanent axial deformation for any CRM asphalt cement mix is superior at both 25 °C and 40 °C to the rate of deformation of the control mix. This result indicates an increased elastic resistance to shear flow and should provide better resistance to rutting.

LAM

Figure 8.14 shows that the rutting resistance of the CRM asphalt cement LAM is comparable to the control LAM at 25°C. However, at 40°C the CRM asphalt cement LAM shows an extremely poor resistance to rutting.

Figure 8.12: K-Values for SMA mixes (25 °C)

Figure 8.13: K-Values for SMA mixes (40 °C)

Figure 8.14: K-Values for LAM

8.4 Summary of Results

The properties demonstrated by CRM asphalt cement SMA mixes are summarized in Table 8.1. It is quite evident that CRM asphalt cement SMA mixes display superior performance to a regular SMA mix. The opposite is true for the LAM. It is, however, the author's belief that similar performance can be achieved in the case of the LAM compared to the SMA mix, if a focused development is undertaken. To succeed there is a need to further investigate the influence of the compaction method, LAM gradation, size of rubber crumbs, type of bitumen, presence of fines, fibres, etc.

Table 8.1: Summary of Asphalt Rubber Performance Compared to the Control Mix

Mix Type	I.T.S.	Fatigue Resistance	Rutting Potential	
			25 °C	40 °C
Ecoflex	=	+	++	+
5%	++	++	++	++
10%	++	++	++	++
15%	+	++	+	+
20%	-	++	+	+
LAM	-	=	--	--
-- large decrease - small decrease = no change + small increase ++ large increase				

Chapter 9

ANALYSIS AND DISCUSSION

9.1 Introduction

The purpose of this experimental work was to develop a pavement based on the asphalt rubber and stone mastic asphalt concept. The basis of the SMA concept is an increased rutting resistance by the mix provided by stone to stone contact of the aggregates. Durable (preferably granite), cubicle, one-size aggregate form the skeleton of the mix. A rich matrix, held in place with filler (passing no. 200), constitutes the remainder of the SMA mix.

A pavement structure is comprised of three layers; subbase, base, and surface. The thicker base layer needs to be resistant to permanent deformation while maintaining a relatively low cost of construction. For this purpose a large aggregate mix (LAM) developed at CSTT/NRC was employed. The LAM uses the SMA concept of stone to stone contact and is inexpensive to manufacture. The LAM was slightly modified to incorporate crumb rubber particles in the mix. For the surface layer, a finer, more durable, and more expensive SMA mix was developed in this research. This mix is

highly resistant to rutting and has been improved with the addition of crumb tire rubber.

The Marshall mix design and performance related testing program have revealed interesting results. These are described in detail in the following sections.

9.2 Discussion of Marshall Test Results for SMA

With a few exceptions all the SMA mixes met the design criteria presented in Table 5.1. The stability values are well above the minimum requirement of 1800 lbs. and flow values are within limits for all 5 mixes. The percentage of air voids (AV) is slightly low for the control mix and is high for the 15% and 20% rubber mixes. It was not possible to design a mix that would satisfy the AV requirements for the entire range of binders (control to 20% rubber). The voids in mineral aggregate (VMA) are slightly lower for the control mix than the design requirements specified by the Asphalt Institute. However, the minimum VMA requirement can be relaxed to at least 12% (Foster 1986). Foster specified in his report that VMA does not correlate well with performance. Foster states, "The literature that I reviewed indicates that if a minimum VMA is really needed, it is something less than 12%. I do not think that minimum VMA limits as high as 12% should be specified as they cannot be justified. If a limit under 12% is needed then it should be determined."

Other than the four perviously mentioned cases, all requirements for the Marshall mix design are satisfied for the five SMA mixes. The VMA for the Ecoflex mix is 12.75% which meets the requirement by Foster. All other requirements are met by the Ecoflex mix.

Figure 5.7 shows that the bulk relative density (BRD) of the Marshall cores decreases for each increase in the amount of rubber in the binder. This is possibly due to two factors: a lower specific gravity of rubber crumbs compared to stone aggregates and rebound of the mix after each blow of the Marshall hammer. The extra rubber particles in the mix absorb energy of the hammer's blow by temporary deformation. After the impact the rubber particles resume their original shape. This hypothesis was verified by an equal increase in height for the test specimen for each increase in the amount of rubber in the mix.

The stability increases with increasing rubber content up to 10% rubber in asphalt. For mixes containing 15% and 20% rubber in the binder, the stability decreased. The flow graphs shows an inverted relationship compared to the stability graphs (Figures 5.2 to 5.7). This increase in strength and decrease in flow can be attributed to the increased viscosity of the binder. The blending of the rubber into the binder at an elevated temperature increases the binder viscosity, which results in higher stability. The subsequent decrease in stability for 15% and 20% CRM asphalt cement binders may indicate that there is a limit to the amount of crumb rubber than can be properly dispersed into the gaps between the aggregates. The excess amounts of crumb rubber, for this particular SMA mix, could end up residing between the aggregates which would prevent aggregate to aggregate contact. This situation would produce a spongy mix which is not as resistant to loads. Excess rubber in the mix can lead to premature crack generation under repetitive loads. Visual observation of the samples produced with higher amounts of rubber supports the idea of the rubber particles preventing proper contact between the

aggregates (Figure 9.1). The surface of the samples bubbled up slightly and was not as compact as that of specimens with up to 10% rubber in the asphalt.

The AV increased with an increase in the amount of rubber in the mix. This behaviour is also a result of a mix that is not efficiently compacted due to the elastic properties of the crumb rubber. The decrease in the BRD as a result of energy absorption by rubber crumbs during compaction is evidence of this conclusion and is responsible for the increase in AV. The subsequent increase in the VMA is mostly due to the increase in AV (see Table 5.4). VMA is defined as the amount of AV and effective asphalt content in the mix. Table 5.4 shows that the effective asphalt content (asphalt not absorbed by aggregate) remains practically unchanged and therefore, indicates that the increase in AV is solely responsible for the increase in the VMA.

The Marshall mix design is easy and efficient at selecting the optimum asphalt content for various mixes. However, performance can not be predicted based on this procedure. Programs, such as SHRP, are developing test methods suitable for standardization and for adoption by state highway departments, other user agencies, and producers of asphalt binders and asphalt-aggregate mixtures to estimate probable field performance of asphalt-aggregate mixtures (Munismith 1991). Drawbacks of the Marshall mix design include an unrepresentative impact compaction method. Asphalt mixes in the field are compacted mostly by compression, kneading, and shear mechanisms and not by impact hammers. Stability and flow tests do not simulate any field loading conditions and cannot be accurately correlated to performance. These are some good reasons why users are trying to implement other design and performance based testing procedures.

Figure 9.1: SMA Mix Containing 20% Crumb Tire Rubber

Therefore, before any evaluation of the performance of the CRM asphalt cement mixes can be documented, performance related testing must be completed. The results of this testing is given in the next section.

9.3 Discussion of Performance Related Testing Results

Performance related testing must be completed before any comparisons between mixes can be evaluated. To achieve this aim four different performance related tests were selected for all SMA and large aggregate mixes. The testing plan was comprised of the following tests:

- indirect tensile strength test (ITS)
- fatigue test
- repetitive uniaxial compression test at 25 °C
- repetitive uniaxial compression test at 40 °C

The LAM samples were compacted by the gyratory equipment in order to better simulate field compaction. Results from these tests are discussed below.

Indirect Tensile Strength Test (ITS)

All mixes, with the exception of Ecoflex and the mix containing 20% crumb tire rubber, show improved tensile strength for the SMA mixes. This result can be attributed to the increased viscosity of the binder caused by the addition of crumb rubber. Too much rubber in the binder results in weakening of the mix due to incomplete digestion of the rubber particles. The excess rubber prevents complete coating of the aggregate and

slightly weakens the asphalt aggregate bond.

The reverse is true for the LAM. The most probable explanation for this large difference in the performance between the SMA and LAM is the large difference in the ratio between the top aggregate size and the sample diameter. The ratio for SMA samples was 0.16 (16 mm/100 mm) while for the LAM samples it was 0.25 (38 mm / 150 mm). It is possible that the strength of such different samples and materials can not be the same or even similar. In addition, laboratory samples have no horizontal confinement. This fact influences the performance of SMA and LAM sample in different ways. Another possible effect is that the addition of 10% rubber crumbs to the binder weakened the samples in the ITS test. This weakening, perhaps, can be attributed to one factor in the production of the CRM asphalt cement binder: a much larger size of rubber crumb was used for this particular rubberized binder. This variation was attempted because the stone size in the LAM is much larger than the SMA stone size. However, the increased size of rubber crumb in the binder had two effects. Firstly, due to the decrease in surface area of the crumb rubber (resulting from a coarser gradation) less rubber was dissolved in the hot asphalt cement. This lack of solution resulted in a smaller increase in viscosity than normally experienced by the CRM asphalt cement binder. Secondly, the undissolved relatively large chunks of rubber in the mix tend to act more as an aggregate. These rubber aggregates weaken the mix in the same way that a dry process CRM asphalt cement mix is weakened. It is evident that the same size crumb rubber should be used for the LAM as was used for the SMA mix, even though the aggregates in the LAM are much larger. This modification should improve the mix properties for a CRM asphalt

cement LAM. It has to be realized, however, that if smaller size rubber crumbs were to be used, a higher overall pavement cost is to be expected.

Fatigue Testing

The performance of the CRM asphalt cement SMA mixes are clearly superior to the control mix in the fatigue tests. The more elastic properties of the binder (due to the presence of rubber particles) helps in absorbing the energy imposed by the fatigue loading. The increased viscosity of the binder along with the elastic properties of the rubber particles (which imparted improved elasticity or flexibility to the asphalt concrete samples) slowed down the crack propagation process through the test specimens.

The fatigue tests for the LAM specimens are inconclusive. The improved elastic properties of the particles of rubber might offset the weakened mix due to the larger size of particles. As mentioned previously, a large aggregate design and testing program has to be undertaken in order to develop a successful crumb rubber LAM. Implementing this type of program has certain design difficulties. A mix design program must be developed for use with a gyratory compactor and a number of different variables must be addressed under laboratory testing before developing well defined laboratory testing conditions. It must be kept in mind that SMA and LAM are completely different mixes that perform differently as was explained previously.

Repetitive Uniaxial Compression Test

The repetitive uniaxial compression test was developed to simulate traffic loading. An impact load of 690 kPa is applied for 0.2 seconds of every 2 second cycle. The CRM asphalt cement SMA mixes displayed superior performance at both 25°C and 40°C compared to the control mix. Again it is postulated that the elastic properties of the CRM asphalt cement binder helped in absorbing some of the impact load energy during the tests. The CRM asphalt cement binder produced a mix with improved rutting resistant properties compared to the control SMA mix. Since the control mix has improved compression (rutting) resistance compared to conventional mixes, it is expected that a CRM SMA mix might display exceptional rutting resistance if used in the field. This expectation can be further explained by the fact that the laboratory sample under the test conditions has no horizontal confinement compared to the field situation.

The CRM asphalt cement large aggregate mix displayed the poorest performance in this test. During the test the CRM asphalt cement LAM specimens deteriorated rapidly. Pieces of the mix would crumble off the edges of the specimen after the load was removed. The larger chunks of rubber seemed to disrupt the adhesion of the mix. This problem might be alleviated by using the same CRM asphalt cement binder as was used for the SMA mix. It is obvious that this binder was not performing up to the standard. Another possible explanation of this type of failure is that some of the large rubber crumbs might have clustered together during mixing (ie. at the bottom of the flask), consequently the binder would lose its homogeneity. The resulting CRM asphalt concrete mix would be weakened by the loss of stone-to-stone contact of the aggregates

due to the rubber crumbs.

9.3.1 Variability of Test Data

Some variability in the test data is evident for each performance test. Since asphalt mixtures are a composite material with many variables affecting their performance, a variability in results is expected. There are many factors that affect the results of the testing. Even after every attempt is made to ensure consistency, it is still not possible to produce identical laboratory specimens. The following factors affect the consistency of laboratory samples and may contribute to inconsistent final results:

- *differences in asphalt, aggregate, mixture, and compaction temperatures.* Each oven has hot and cold spots. Also, different ambient temperatures could affect mixing and compaction conditions.
- *aggregate orientation.* Although every effort is made to produce identical samples the variability in aggregate shape and orientation in the compacted mix has some effect on final properties of each sample.
- *placement of the sample under the test head.* The orientation of the sample in the loading head is important since the test outcome could be affected. It is not possible to ensure placement of the sample exactly in the centre of the loading head's test after test. Even small misalignment of the specimen and loading head can be critical when employing the ITS and Fatigue tests.
- *worker variability.* Samples tested or prepared by different people could differ in properties depending on the individual's skill and procedures.

The individual sample variability was kept to a minimum by preparing more samples if necessary. The general good repeatability shown by the tests is evidence of success.

9.4 Summary

Test results reported in this chapter satisfied the main objectives of the experimental program. A CRM asphalt pavement was designed using two different mixes. The surface coarse was based on an SMA concept and small size aggregate mix while for the base coarse design, a modified large aggregate mix designed at CSTT/NRC was used. The CRM SMA surface course has shown superior performance to the unmodified mix and has also displayed extreme resistance to rutting deformation. The base course mix did not perform according to expectations, but it is believed that with further modifications and well supported R&D, this objective can be achieved. It is also believed that the use of this pavement system (with the re-design of the base coarse) will not only aid in the reduction of discarded tire stockpiles, but also will improve the performance of asphalt pavements.

Chapter 10

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This final section of the thesis summarizes the work carried out in the research program and presents the major conclusions resulting from this research. Several recommendations for further research are also presented.

10.1 Summary

The main objective of this thesis was to design a CRM asphalt pavement based on two different mixes; (a) a small top size aggregate mix (<19 mm), and (b) a large top size aggregate mix (up to 38 mm size stone). The first mix was based on the stone mastic asphalt (SMA) concept while the second was based on a CSTT LAM design. It is expected that the use of CRM asphalt in both mixes will create a durable pavement which will go a long way in helping to reduce the scrap tire disposal problem. The small stone size SMA mix was developed for use with CRM asphalt for a thin top pavement layer and was designed to provide superior rutting resistance. A crumb rubber modified

LAM was selected to be used for a base layer in the pavement structure. The SMA mixes were designed using the Marshall method of mix design. Both the SMA and LAM CRM mixes were compared with their respective control mixes using performance testing. The four tests used to relate sample performance were: indirect tensile strength test (ITS), fatigue test, and repetitive uniaxial compression (rutting) tests at 25 °C and 40 °C. These performance tests showed promising results.

The next section iterates the major conclusions of each phase of the research, including the literature review, Marshall mix design, and performance testing of both the SMA and large aggregate mixes.

10.2 Conclusions

The conclusions of this research work are divided into three sections. The following major conclusions are based on; (a) the literature review, (b) Marshall mix design phase, and (c) performance testing phase.

Literature Review

Crumb rubber modified asphalt has been successfully used since the mid 1960's. There are two distinctively different ways to manufacture CRM asphalt: the wet process and the dry process. All methods where rubber crumbs are incorporated into bitumen before mixing with the aggregates are called the wet process. Methods where rubber crumbs are first mixed with the aggregates and then mixed with hot bitumen is called the dry process. Dry process field applications of CRM asphalt do not appear to be as

successful as wet process applications. Although wet process CRM asphalt is estimated to cost 1 1/2 to 2 times the cost of HMA pavements, it has also been demonstrated to last over twice as long as regular asphalt cement pavements in Phoenix, Arizona. CRM asphalt cement pavements made by the wet process have also been shown to retard reflection cracking, resist rutting and aging, and to improve fatigue properties in both the laboratory and the field. It is because of these characteristics that the wet process was selected as the method to prepare the CRM asphalt for this thesis. Other factors that influenced the choice of the thesis topic include the author's personal interest in the development of better roads while addressing an environmental problem and the possibility that Canada might follow the lead of the US Congress which passed a bill mandating the use of CRM asphalt cement.

Marshall Mix Design

The Marshall mix design was successfully employed in the development of six different SMA mixes. The optimum asphalt content of each mix was determined and comparisons were made among them. Although this mix design method is detailed in specifications and is easy to use, several shortcomings have been noticed. They are outlined as follows:

- Marshall compaction does not simulate field compaction
- Marshall tests cannot predict field performance
- only 101.6 mm diameter samples can be made. This prevents the design of any mix using aggregates larger than 19 mm. A large aggregate mix cannot be made

using a Marshall compactor.

Once the SMA mixes had been designed, optimum samples of each mix were compacted using the Marshall compactor. The LAM samples were compacted using a gyratory shear compactor. Subsequently, the samples were subject to performance tests.

Performance Related Testing

Results of the performance related tests are very promising. The crumb rubber modified asphalt SMA mixes showed superior results compared to the control mixes in all the performance tests. The asphalt rubber with 10% crumb tire rubber by weight of binder gave the best results. The superior ITS and fatigue properties of this mix indicates an excellent resistance to cracking. Outstanding repetitive uniaxial compression test (rutting) results indicates improved rutting resistance of this asphalt rubber mix. It is clear that the addition of GTR-10 crumb tire rubber to the asphalt cement has improved the quality of this particular SMA mix.

The results of the CRM large aggregate mix are disappointing. A decrease in the performance of this mix was evident in the ITS and RUCT tests. No change was noted in the fatigue test. It is very likely that the drop in performance is attributed to the fact that relatively large chunks of rubber were used for manufacturing the asphalt rubber binder. If the same size of rubber that was used in the SMA phase of this research is also used for the large aggregate mixes, a vast improvement in performance should be expected. Further research and development is needed to develop a proper LAM using CRM asphalt.

10.3 Recommendations for Future Research

The testing program overview outlined in Chapter 3 was devised based on equipment availability and the time constraints for this thesis. Allowing more time for further basic research and determining the influence of design variable the following is recommended:

- Prepare all samples using a gyratory shear compactor. The gyratory shear compactor more closely simulates field compaction and provides superior samples. Aggregate cracking is noticeable with Marshall compaction due to the confined boundary conditions and impact load of the Marshall hammer. Even under the same boundary conditions, the gyratory shear compaction allows for aggregate reorientation providing for a denser sample. In order to eliminate the Marshall method, a new mix design program must be formulated for use with a gyratory shear compactor. Standards for this type of program are not currently available.
- Study more closely the effects of rubber particle sizes on the mixes. Changing the particle size of the crumb rubber between the SMA and large aggregate mixes has greatly changed the test results. Larger test samples might be needed to produce a good quality LAM sample.
- Study interaction conditions (ie. temperature, mixing conditions, etc.) between rubber and bitumen during manufacture of the wet process CRM asphalt. For example, mixing time and temperature have an important effect on the final bitumen viscosity (which is a function of temperature and time) and resulting CRM asphalt concrete properties.

- Introduce more performance tests to get a clearer picture of the effect of adding rubber to the binder. Other tests to consider are: resilient modulus (M_R) tests and cold temperature tests (ITS, and Fatigue) at various temperatures (down to -40°C); RUCT and, if available, wheel rutting tests at both room and high temperatures (60°C), moisture damage tests (repetition of certain tests after conditioning samples in water for various extended periods of time), and aging tests (both short and long term). The wheel rutting test has an advantage of more closely simulating field loading conditions compared to the repetitive uniaxial compression test.
- Finally, field trials should be used to correlate lab data with actual field conditions and to verify that performance has improved.
- Before this new technology can be recommended for wide field implementation, a detailed pavement life and cost analysis will have to be made.

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APPENDIX

Marshall data for control SMA mix

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
4.5	SR13	66.3	2.432	2546	0.1792		
	SR14	62.4	2.450	2998	0.1428		
	SR15	63.5	2.444	2789	0.1369		
	SR16	65.5	2.411	2382	0.1392		
	Average	64.425	2.434	2678.75	0.150	2.496	12.691
%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
5.0	SR1	64.2	2.461	2130	0.0737		
	SR2	63.5	2.467	2388	0.1554		
	SR3	61.1	2.476	2603	0.1415		
	SR4	65.9	2.472	2578	0.1674		
	Average	63.675	2.469	2424.75	0.135	0.833	11.888
%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
5.5	SR5	62.3	2.468	2197	0.1815		
	SR6	62.3	2.466	2129	0.1765		
	SR7	64.6	2.453	1649	0.2022		
	SR8	63.7	2.462	1570	0.1914		
	Average	63.225	2.462	1886.25	0.188	0.159	12.591
%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
6.0	SR9	62.5	2.432	1594	0.2236		
	SR10	68.6	2.433	1610	0.2047		
	SR11	64.3	2.435	1563	0.2097		
	SR12	63.6	2.441	1619	0.2527		
	Average	64.750	2.435	1596.50	0.223	0.114	14.007

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
6.5	SR17	64.1	2.428	1390	0.1878		
	SR18	64.6	2.423	1459	0.2171		
	SR19	60.6	2.432	1405	0.2163		
	SR20	68.3	2.424	1498	0.2085		
	Avera	64.400	2.427	1438.08	0.207	0.331	14.763

Marshall data for Ecoflex SMA mix

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
4.0	ECO 1	61.54	2.428	3579	0.0832		
	ECO 2	64.39	2.434	3528	0.0960		
	ECO 3	64.59	2.412	3178	0.1013		
	Average	63.507	2.425	3428	0.0935	2.797	12.559

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
4.5	ECO 5	60.63	2.449	3528	0.1137		
	ECO 6	63.8	2.439	2918	0.1078		
	ECO 7	63.71	2.412	3090	0.1063		
	Average	62.713	2.433	3179	0.1093	2.952	12.7

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
5.0	ECO 9	62.24	2.465	2683	0.1190		
	ECO 10	61.53	2.466	2688	0.1379		
	ECO 11	63.3	2.467	3070	0.1158		
	Average	62.357	2.466	2814	0.1242	2.468	11.99

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
5.5	ECO 13	66.1	2.449	2436	0.1411		
	ECO 14	60.71	2.451	2054	0.1221		
	ECO 15	61.64	2.459	2265	0.1537		
	Average	62.817	2.453	2252	0.1390	0.789	12.92

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
6.0	ECO 17	65.76	2.447	2043	0.1676		
	ECO 18	63.88	2.448	1613	0.1811		
	ECO 19	60.83	2.445	2039	0.1752		
	Average	63.490	2.447	1898	0.1746	0.669	13.6

Marshall data for 5% CRM SMA mix

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
4.0	s5 17	66.62	2.344	3267	0.1040		
	s5 18	63.83	2.370	3395	0.1034		
	s5 19	66.15	2.337	3154	0.1042		
	Average	65.533	2.350	3272	0.1039	6.882	15.24

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
4.5	s5 1		2.364	2893	0.1106		
	s5 2	63.36	2.372	2923	0.1215		
	s5 3	65.65	2.433	3517	0.1074		
	Average	64.505	2.390	3111	0.1132	4.351	14.270

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
5.0	s5 5	65.47	2.425	3018	0.1000		
	s5 6	59.54	2.440	3175	0.1054		
	s5 7	58.6	2.437	3042	0.1175		
	Average	61.203	2.434	3078	0.1076	2.045	13.137

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
5.5	s5 15	63.27	2.428	2565	0.1190		
	s5 16	62.85	2.428	2570	0.1453		
	s5 11	62.96	2.437	3070	0.1221		
	Average	63.027	2.431	2735	0.1288	1.311	13.700

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
6.0	s5 13	62.54	2.410	1946	0.1628		
	s5 14	65.42	2.422	2200	0.1610		
	s5 9	62.43	2.434	2351	0.1346		
	Average	63.463	2.422	2165	0.1528		14.475

Marshall data for 10% CRM SMA mix

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
4.0	s10 17	66.98	2.326	3195	0.1253		
	s10 18	64.2	2.341	3659	0.1169		
	s10 19	66.39	2.293	2550	0.1245		
	Average	65.8567	2.320	3281	0.1222	8.135	16.334

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
4.5	s10 1	67.25	2.357	3109	0.1394		
	s10 2	62.01	2.384	3454	0.1148		
	s10 3	64.27	2.397	3706	0.1265		
	Average	64.51	2.379	3423	0.1269	5.376	14.641

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
5.0	s10 5	63.07	2.376	3196	0.1436		
	s10 6	66.32	2.373	3019	0.1475		
	s10 7	66.72	2.386	3045	0.1459		
	Average	65.37	2.378	3087	0.1457	3.894	15.123

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
5.5	s10 9	63.58	2.420	2612	0.1506		
	s10 10	65.03	2.399	2614	0.1415		
	s10 11	66.38	2.370	2449	0.1318		
	Average	64.9967	2.396	2558	0.1413	2.505	14.931

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
6.0	s10 13	63.83	2.372	2772	0.1491		
	s10 14	67.77	2.338	1773	0.1506		
	s10 15	65.65	2.419	2523	0.1438		
	Average	65.75	2.376	2356	0.1478	3.455	16.087

Marshall data for 15% CRM SMA mix

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
4.0	S15-1	69.16	2.293	2376	0.1240		
	S15-2	63.11	2.285	2519	0.1263		
	S15-3	68.36	2.304	2960	0.1506		
	Average	66.88	2.293	2618	0.1336	9.12	17.32

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
4.5	S15-5	65.68	2.322	2636	0.1242		
	S15-6	67.62	2.324	2749	0.1369		
	S15-7	69.62	2.323	2683	0.1337		
	Average	67.64	2.323	2689	0.1316	7.38	16.66

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
5.0	S15-9	67.81	2.311	2413	0.1569		
	S15-10	66.95	2.325	2721	0.1411		
	S15-11	69.79	2.304	2293	0.1590		
	Average	68.18	2.313	2476	0.1523	6.88	17.44

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
5.5	S15-13	65.57	2.312	2290	0.1649		
	S15-14	62.65	2.323	2349	0.1476		
	S15-15	64.44	2.348	2603	0.1171		
	Average	64.22	2.328	2414	0.1432	5.79	17.37

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
6.0	S15-17	70.55	2.313	1903	0.1516		
	S15-18	65.43	2.318	2014	0.1760		
	S15-19	66.55	2.367	2592	0.1527		
	Average	67.51	2.333	2170	0.1601	4.81	17.63

Marshall data for 20% CRM SMA mix

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
4.0	S20-17	70.54	2.218	1937	0.2072		
	S20-18	67.58	2.262	2472	0.1767		
	Average	69.06	2.240	2204.50	0.192		19.219

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
4.5	S20-2	68.31	2.238	2229	0.1632		
	S20-21	69.67	2.259	1744	0.1537		
	S20-22	68.96	2.250	2143	0.1600		
	Average	68.98	2.249	2039	0.1590	9.80771	19.3165

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
5.0	S20-6	68.17	2.273	2652	0.1301		
	S20-7	68.79	2.288	2303	0.1440		
	Average	68.48	2.281	2477.50	0.137	7.817	18.615

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
5.5	S20-9	67.93	2.291	2248	0.1695		
	S20-10	70.12	2.259	1893	0.1798		
	S20-11	69.81	2.274	2030	0.1695		
	Average	69.2867	2.275	2057	0.1729	7.90067	19.2502

%Binder	Sample	Height	B.R.D.	Stab.	Flow	%A.V.	% V.M.A.
6.0	S20-13	68.89	2.295	2147	0.1434		
	S20-14	70.51	2.277	1910	0.2024		
	S20-15	75.79	2.288	2189	0.1969		
	Average	71.73	2.287	2082	0.1809	6.13244	19.2537

Fatigue test results for SMA

Type	Loading conditions	Sample	Nf (cycles)
Control	15%	tc11	5650
		tc13	3400
		tc15	4300
	30%	tc2	610
		tc6	600
		tc8	365
Ecoflex	15%	te7	8000
		te11	4800
		te13	4050
	30%	te2	700
		te4	650
		te6	520
	5% 15%	t59	5100
		t58	6600
		t57	8240
	30%	t52	660
		t54	575
		t56	830
10%	15%	t109	6100
		t108	7200
		t107	5050
	30%	t102	725
		t104	620
		t106	1010
15%	15%	t159	9450
		t1510	9050
		t154	860
		t156	1080
		t158	680

20%	15%	t209	13400
		t208	15800
		t207	13400
	30%	t202	1390
		t204	1570
		t206	1170

Fatigue test results for LAM

Control	15%	c10	2610
		c11	2240
		c12	2810
	30%	c4	570
		c5	447
		c6	488
10%	15%	r8	4550
		r9	5200
	30%	r10	496
		r11	1108
		r12	912

ITS results for SMA mixes (PSI)

Sample	ITS
Control	168.8
	160.4
	148.3
Ecofiex	160
	165.4
	148.7
5%	184.1
	171.9
	201.4
10%	211.1
	184.7
	169.5
15%	166.6
	172.7
	171.8
20%	165
	154.6
	145.2

ITS results for LAM mixes

Sample	ITS
Control	92.4
	80.3
	92.8
10%	56.5
	88.5
	79.7

END

21-07-94

FIN