Development of Hot Mix Asphalt Using Fractionated Coarse and Fine Reclaimed Asphalt Pavement (RAP) and Pulp Aramid Fiber (PAF) for Cold Climate

by

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Développement de la performance de chaussée avec des additifs bitumineux et non bitumineux pour le climat froid

Seyed Saeed SALIANI

RÉSUMÉ

De nos jours, il est courant d'ajouter des matériaux recyclés dans les enrobés sans modifier le module et les propriétés à basse température. Il a été démontré que l'utilisation de granulats bitumineux recyclés (GBR) dans les enrobés, lorsqu'effectuée correctement, peut augmenter les performances de ce dernier. Cependant, peu d'études ont examiné comment les enrobés avec recyclés pourraient être améliorés. Cette étude se concentre sur l'utilisation de GBR et sur l'effet du fractionnement et de la taille des particules de GBR sur la performance globale des mélanges à forte teneur en GBR produits uniquement avec des particules grossières ou fines de GBR. En outre, un autre additif qui augmente les propriétés des enrobés a été étudié ici. Des fibres aramides courtes (PAF) ont été ajoutées à l'enrobé pour étudier leur efficacité. Des tests de performance et des tests thermomécaniques tels que des tests de fatigue en tension-compression, la résistance à la fissuration thermique (TSRST), la résistance à l'orniérage, le test de tension indirecte, des tests de module complexe, des tests au rhéomètre à cisaillement dynamique, etc. ont été utilisés pour caractériser les bitumes et les enrobés. Le modèle rhéologique 2S2P1D a été utilisé pour comparer les mélanges conçus dans cette étude. En conséquence, la teneur en GBR dans les enrobés a été augmentée par le fractionnement du GBR en particules fines et grossières. De plus, de meilleures performances ont été observées pour l'enrobé avec de grosses particules de GBR que pour les enrobés avec la fraction fine des GBR. En outre, la fibre d'aramide est suggérée comme alternative pour renforcer la chaussée afin d'augmenter la résistance à la tension. Une étude plus approfondie est recommandée concernant l'optimisation de la teneur en fibres de l'HMA.

Mots-clés: Enrobés, Enrobé recyclé, Granulats bitumineux recyclés; Tests Thermo-Mécaniques, Fibre d'aramide, Conception de mélange, Performance de mélange

Development of hot mix asphalt using divided coarse and fine reclaimed asphalt pavement (rap) and pulp aramid fiber (paf) for cold climate

Seyed Saeed SALIANI

ABSTRACT

Nowadays, it is common to add recycled materials in asphalt mixes without changing the modulus and the low temperature properties. It has been shown that the usage of reclaimed asphalt pavement (RAP), when done properly, can enhance the performance of Hot Mix Asphalt (HMA). However, limited studies have looked at how RAP mixes could be improved. This study mainly focused on utilizing RAP and the effect of RAP fraction and particle size on the overall performance of high RAP mixes produced solely with either coarse or fine RAP particles. In addition, other additives that increase HMA properties have been studied here. Pulp Aramid Fibers (PAF) was added to HMA to study their effectiveness. Performance tests and thermomechanical tests such as tension-compression fatigue tests, thermal cracking resistance (TSRST), rutting resistance, indirect tension test, complex modulus tests and dynamic shear rheometer test were used to characterize the bitumens and the mix. The 2S2P1D rheological model was used to compare the behaviour of the mixes that designed in this study. As a result, fractionating the RAP particle can help to increase the RAP content limits. Also, it is expected that HMA has better performance with inclusion of coarse RAP particles than fine particles. In addition, pulp aramid fiber is suggested as the green alternative to reinforce the pavement to increase the tension resistance. Further investigation recommended regarding the optimization of fiber content in HMA.

Keywords: Hot Mix Asphalt, Recycled asphalt, RAP gradation; Thermo-Mechanical tests, Pulp Fiber, Short Fiber, Mix Design, Mix Performance

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation
AFNOR	Association Française de NORmalisation
ARRA	Asphalt Recycling and Reclaimed Association
ASTM	American Society for Testing and Materials
ATR	Attenuated Total Reflection
BBR	Bending Beam Rheometer
BMD	Balanced Mix Design
Ccor	Compaction Obtained on the specimen
CGSC	Compaction obtained by the test of Gyratory Shear Compactor
CPATT	Centre for Pavement and Transportation Technology
CR	Coarse RAP
CRM	Coarse Rap Mix
CSCE	Canadian Society for Civil Engineering
DE	Dissipated Energy
DGCB	Departement Genie Civil et Batiment
DSR	Dynamic Shear Rheometer
EN	NORME EUROPÉENNE
ENTPE	Ecole Nationale des Travaux Publics de l'Etat
ESEM	Environmental Scanning Electron Microscopy
ÉTS	École de Technologie Supérieure
FHWA	Federal Highway Administration
FR	Fine RAP

FRM	Fine RAP Mix
FTIR	Fourier Transform Infrared Spectroscopy
GB	Grave Bitume
Gmm	Theoretical Maximum Specific Gravity
HMA	Hot Mix Asphalt
IDS	Indirect tensile Strength
IDT	Indirect Tension
LC	Laboratoire des Chaussées du MTQ
LCMB	Laboratoire sur les Chaussées et les Matériaux Bitumineux
LCPC	Laboratoire Central des Ponts et Chaussées
LVE	Linier Visco-Elastic
MEDG	Mechanistic Empirical Design Guide
MTQ	Ministère des Transports du Québec
MTS	Materials Testing System
NAPA	National Asphalt Pavement Association
NCHRP	National Cooperative Highway Research Program
OBC	Optimum Binder Content
OGFC	Open-Graded Friction Course
ON	Ontario
PAF	Pulp Aramid Fiber
PAV	Pressure Aging Vessel
PG	Performance Grade
QC	Quebec
RAP	Reclaimed Asphalt Pavement

RG*	Reduced G*
RTC	Regional Transportation Commission
RTFO	Rolling Thin Film Oven
SCB	Semi-Circular Bending
SGC	Superpave Gyratory Compactor
SHRP	Strategic Highway Research Program
SMA	Stone Matrix Asphalt
STA	The Strength-Time Area
TLPA	Two-layer Porous Asphalt
TMD or Gmm	Theoretical Maximum Specific Gravity
TSRST	Thermal Stress Restrained Specimen Test
TTSP	Time Temperature Superposition Principle
Va	Air voids
VAM	Volume of voids in the mineral aggregate in the compacted mixture
VFA	Voids Filled with Asphalt
VFB	Void Fills with Bitumen
VMA	Voids in the Mineral Aggregate
VTM	Voids in the Total Mix
WLF	Williams Landell and Ferry

LIST OF SYMBOLS

%	Percentage
°	Degree
°C	Degree Celsius
μ_{def}	Micro deformation
µm	Micron
A	Amplitude of signal
aF	Slope of fatigue
C1	Coefficient corresponding to the life duration of the material for an imposed strain amplitude of 1 m/m, at a given temperature and frequency
C2	Coefficient related to the slope of the fatigue right for a given material
c	Slope
Ci	Coefficient of correction
cm	Centimeter
cm^{-1}	Wave number
D	Diameter
Def	Deformation
E	Elastic modulus due to loading
E*	Complex modulus
E_0	Initial modulus
E00	static modulus
Fr	Frequency
g	Gram

G*	Shear modulus
Gmm	Aggregate’s maximum theoretical specific gravity
H	Height
Hz	Hertz
kg	Kilogram
km	Kilometer
kN	Kilo Newton
kPa	Kilo Pascal
m	Meter
mm	Millimeter
MPa	Mega Pascal
N	Newton
N	Number of cycles
N	Number of gyrations
Nf	Number of cycles at the rupture
$Nf_{50\%}$	Number of cycles at the half of the initial modulus
$Nf_{II/III}$	Number of cycles at the point of transition between phases II and III
P	Load
p.	Page
Pa	Pascal
R2	Coefficient of determination
rf	failure strength
S	slope
s	Second

St	Strenght
STD	Standard
t	Time of solicitation
T°	Temperature
T_f	failure temperature
Tg	Transition temperature
TL	binder content
T_t	transition temperature
Vbe	Volume of effective asphalt binder
Wd	dissipated energy
Wi	Dissipated energy at load cycle
δ	phase angle
ε_A	strain amplitude
ε_0	Initial Deformation
ε_6	Deformation at 1 million of cycles
ε_A	Amplitude of signal of deformation
η	Newtonian viscosity
θ	Temperature
σ	Stress
σ_0	Initial stress
σ_A	Amplitude of signal of stress
τ	characteristic time
ϕ_E	Phase angle
ω	Angular frequency

INTRODUCTION

The usage of Reclaimed Asphalt Pavement (RAP) in Hot Mix Asphalt (HMA) is proven to be a sustainable alternative to produce more environment-friendly asphalt mixes. Adding RAP in asphalt mixes is recommended in order to preserve the aggregates and bitumen, while achieving or exceeding the same performance as virgin mixes (Kandhal & Mallick, 1998). Recycling of the existing mineral aggregates and asphalt binder in RAP particles would be of great benefits to the environment by saving the non-renewable materials. Milled pavements are considered to be valuable materials after reaching the end of pavement service life. At a minimum level, RAP can play the role of virgin mineral aggregates in order to conserve the energy and save the environment. However, there is still a lot of information missing about the impact of the RAP characteristics on the mixes performance. The work presented here aims to increase the knowledge on bituminous mixes with RAP by studying the effect of the RAP size and by studying if the properties of those mixes can be enhanced with the addition of fibers. The first chapter covers a review of the literature, laboratory works, results and the analysis of the results are presented in the subsequent chapters. This is a paper-based thesis, which means that the dissertation is a series of submitted, accepted and presented papers in journals and conferences regarding investigation on HMA additives such as recycled asphalt pavement (RAP) fractions, study the impact of RAP size on long-term pavement performance and also Pulp Aramid Fiber (PAF).

CHAPTER 1

LITERATURE REVIEW

1.1 Hot Mix Asphalt (HMA) Components Characteristics

Flexible pavement or hot mix asphalt pavement consists of aggregates, asphalt binder and if needed additives which leads the pavement has better performance in various weather and loading conditions.

1.1.1 Aggregates

Conventional HMA mixes volume consists of 90% to 95% of aggregate. Basically, aggregate properties play the significant role in HMA performance especially in higher temperatures. Aggregates gradation is one of the most important properties that affect stiffness, stability, durability, permeability, fatigue resistance, fractional resistance and resistance to the moisture damage (Robert, Kandhal, Brown, Lee, Kennedy,1996).

Obtaining a desired skeleton gradation from a single aggregate stockpile is a rare practice. Therefore, blending the aggregates from different stockpile with various particle size is a common practice in super-pave mix design. Each agency recommends a specific desire size range for each particle size. The LC Test Method's Compendium, produced by the General Directorate of the Pavement Laboratory of the ministère des Transports du Québec, presents all the test methods used to measure the characteristics of aggregate and other materials used in the construction and maintenance of the roads infrastructure.

Several researches studied the aggregate impact on HMA performance. Aggregate mineral and chemical composition, exposure history have significant effects on stripping.(Robert et al., 1991).

Aggregate characteristics can change the fatigue life of HMA. In the Superpave mix design method, the aggregate gradation and the binder type selections are based on the expected mix behavior in the gyratory compactor. Performance-based tests are excluded at the design stage. Consequently, aggregate gradation is playing the important role in volumetric mix design.

HMA permanent deformation is affected by aggregate particle size. Amount of stone or coarse particle which is retained on the 5 mm sieve and aggregate angularity are controlling the HMA performance under the passing load that causes permanent deformations. Stone skeleton such as the one in stone matrix asphalt (SMA) is generally resistance to rutting due to their high stone concentration (Langlois, 2003). In general, aggregate characteristics and aggregate gradation must satisfy overall pavement performance criteria such as fatigue resistance, permanent deformation, respecting the acceptable air void and interlock. Reclaimed Asphalt Pavement (RAP) aggregates do need to meet the same requirements than virgin aggregates.

1.1.2 Asphalt binder

The second component of pavement is a highly viscous hydrocarbon material produced from petroleum distillation residue which is called asphalt cement in Canada,bitumen (Carter et al., 2018) or asphalt binder. In HMA, asphalt functions as a waterproof, isotropic, thermoplastic (become plastic on heating and harden on cooling and can repeat these processes) and viscoelastic adhesive (Krishnan and Rajagopal,2003). Viscoelasticity is the property of materials that exhibit both viscous and elastic characteristics when undergoing deformation. asphalt binder possesses elastic property of solid and the viscous behavior of liquid at different temperatures (Lee and Huang, 1973). Asphalt has the high sensitivity to the temperature.

There are several types of asphalt binder all around the world produced from the distillation process of various sources of crude oil. This distillation can occur naturally, resulting in asphalt lakes, or occur in a petroleum refinery using crude oil. A great variety of asphalt makes complexity to establish a known relationship between viscosity changes and temperature. Besides, the elastic and the viscous responses change with temperature (Krishnan & Rajagopal,2003). Pfeiffer and Saal (1940) studied the impact of oil sources on physical and chemical properties of asphalt binder. Various tests were performed to characterize different types of asphalt binder produced from different sources such as specific gravity, coefficient of expansion, specific heat, thermal conductivity, permeability to water vapor, surface tension, and total surface energy of a particular class of asphaltic bitumen. They concluded that asphalt binder characteristics especially specific gravity depends on crude oil source. High dependency to source makes the asphalt binder classification complicated. In Canada like in many other countries, before 1995, it was a common practice to classify the asphalt binder by penetration and viscosity. The Strategic Highway Research Program (SHRP) introduced a new method of asphalt binder classification known as performance grade (PG) in the mid 1990s to address three primary performance parameters of HMA: permanent deformation, fatigue cracking and low temperature cracking (Carter et al., 2018). The main purpose of grading and selecting asphalt binder using the PG system is to make certain that the binder has the appropriate properties for environmental conditions in the field. Therefore, the PG system uses a common set of tests to measure physical properties of the binder that can be directly related to field performance of the pavement at its service temperatures by engineering principles (Superpave interactive, 2021). It is one of the most important changes introduced in Superpave that acceptance limits are the same but must be met at specific pavement temperature and traffic conditions (RAHA bitumen Co, 2019).

Lethersich (1942) concluded that asphalt has a complicated response in various conditions. It is shown that the behavior of asphalt binder may be expressed in terms of a mechanical model containing two elasticity and two viscosities. Also, Maxwell studied the viscoelastic nature of asphalt binder. As a soft solid form, a sudden sufficient force is altering the form but on the viscous fluid form, given enough time with very small force will produce a sensible effect (Maxwell, 2012).

The binder viscosity should be low enough at high temperature to well coat the aggregates and to enhance the coated aggregates' movement through the HMA plant during mixing. Also, it should be stiff enough at the in-service temperature to minimize the permanent deformation. At the same time, it should be ductile at low temperature in winter time to improve the HMA resistance against the thermal cracking (Carter, 2004).

It is common to rely on physical properties of asphalt binder in the pavement industry, although physical properties are a direct result of chemical composition. So, it is important to mention some crucial physical properties such as:

Purity: Asphalt binder, as used in HMA paving, should consist of almost pure asphalt binder. Impurities are not actively cementing constituents and may be detrimental to asphalt performance.

Ductility: it is considered as a significant property which is the ability to stretch without breaking. Roberts et al. (1996) concluded that binder with lower ductility has poor performance than ductile binder even when both asphalt binder have the same penetration.

Rheology: material deformation and flow are known as rheology property. Binder rheology has an impact on HMA performance directly. Soft binder might be susceptible to permanent deformation and bleeding while too stiff binder may cause fatigue cracking.

In addition, binder durability is known as the physical changing by time. It is also called aging or hardening). In general, as an asphalt binder ages, its viscosity increases, and it becomes stiffer and more brittle.

1.2 Reclaimed Asphalt Pavement (RAP)

1.2.1 Age hardening

During pavement rehabilitation, Reclaimed Asphalt Pavement (RAP) is commonly produced by milling the existing pavement. The produced RAP is composed, like a new HMA, of aggregates and asphalt binder.. However, even if the aggregates are not too affected by the time, they were in a pavement structure, the RAP asphalt binder did change over time. Asphalt is a natural organic end product of ancient living organisms. It can be oxidized by presence of atmospheric oxygen. Asphalt oxidation is important because it leads to the hardening of asphalt and consequently, a deterioration of desirable physical properties is happening (Petersen, 2009).

Asphalt hardening, or aging, is a phenomenon caused by chemical interaction of asphalt components and the environment. Corbett and Swarbrick (1959) studies confirmed that aging is caused by increase of asphaltenes part of asphalt binder. Petersen (1984) showed the main factors of asphalt binder aging:

- Loss of oily components by volatility or absorption;
- Changes in composition by reaction with atmospheric oxygen;
- Molecular structuring that produces thixotropic effects (steric hardening).

In general, aging can be classified according to the position of the asphalt binder in the pavement structure (easy access to oxygen or not) or by time. Potentially, hardening is starting from the surface and extends to deeper layers. These two aging types happen on all kinds of asphalt binder, but the aging rate might be different due to oil source. Following the study of Krishnan & Rajagopal (2003), Aging rate and behavior of asphalt depends to a very large extent on the internal structure of asphalt binder, which in turn depends on the crude oil source.

Aging phenomena affects the pavement quality in two steps. First during the construction period and then in the service life. Vallerga, Finn, & Hicks (1967) realized that most of the short-term aging happens after subjecting asphalt binder to heat during the mix process. Roberts et al. (1996) concluded that the reason for short term aging is that lighter oils, or volatiles, are driven out of the asphalt binder through evaporation. Short term aging is a complex phenomenon as it depends on several factors. Fonseca (1995) listed significant factors as different crude oil sources, refining procedures and levels of additives (volatile oils). Other factors such as plant type, mixing temperature and time, and mix type also influence the short-term aging. On the other hand, the progressive environmental oxidation of the in-place material with time causes the long-term aging. It significantly depends on environmental conditions. Warmer temperature increases the oxidation rate. In addition to the environment, mix design has an impact on long-term aging as well for example effective asphalt content (V_{beff}) and air void (Va).

Pavement agencies developed various tests to measure the long-term and short-term aging. HMA aging can be simulated with the Rolling Thin Film Oven (RTFO) test and with the

Pressure Aging Vessel (PAV) (ASTM D2872). Superpave mix design method uses RTFO to simulate the short-term aging which happens during the construction and uses PAV to simulate the long-term aging. SHRP uses another simulating method known as SHRP long-term oven ageing (McDaniel, Soleymani,Anderson, Turner,Peterson,2006). In this process, all specimens are kept in forced draft oven for 120 hours at 85°C. He recommended using this method to simulate the 15 years of field aging in a wet, no-freeze climate.

1.2.2 Pavement Recycling

The basic concept of pavement recycling lies in the conservation of the total energy required to rehabilitate deteriorated pavement. It means the conservation of materials through reuse of old pavement and the reduced need for new materials, as well as the preservation of the environment by eliminating the necessity for disposing of old materials (Basueny, 2009).

Several parameters affect the mix design of asphalt mixes with inclusion of recycled materials such as: Construction cost, conservation of aggregate and binders, preservation of existing pavement geometrics, preservation of the environment and conservation of energy. Beside several reasons for encouraging pavement industry to use recycle asphalt, probably the largest single factor was the oil embargo of the early 1970s and the subsequent increase in the price of asphalt binder (Basueny, 2009). Several researches have been done to prove that adding more RAP has both environmental and economic benefits (Roberts et al., 1996) (Baaj, Ech, Tapsoba, Sauzeat, & Di Benedetto, 2013a).

In early 2000, performance of the HMA with RAP was unknown (McDaniel et al., 2000), but many studies have been done to prove that sufficient RAP content in HMA can satisfy the mix design criteria in terms of fatigue, rutting, thermal resistance and durability (Al-Qadi, Elseifi, & Carpenter, 2007).

Roberts et al. (1996) showed that up to 20% of aged binder does not significantly affect the properties of the blend of virgin and RAP binder. Although there is no recognized unanimity about the limit of the maximum amount of RAP in HMA, RAP percentage in HMA has been limited by many agencies, mainly, due to the unproven performance of high RAP mixes and also lack of a unified mix design (Zaumanis, Mallick, Frank, 2016). In 1997, the Federal Highway Administration's RAP expert task group developed guidelines for the design of Superpave HMA containing RAP (Federal Highway Administration, 1995). In the same year, another study by Kandhal and Foo (1997) recommended a three-tier process to deal with RAP in asphalt concrete, where a RAP content of 25% and more was defined as high RAP mix, requiring detailed evaluations (Kandhal and Foo, 1997). RAP limitation was also supported by the findings of the NCHRP research report 9–12 (MC Daniel et al., 2000) In spite of several research projects conducted on RAP incorporated mixes, still there is not a clear vision about the interaction of RAP and virgin materials in detail. Different scenarios can be considered about the interaction of virgin and aged binder: (1) there is no interaction between old and virgin materials, so RAP could be assumed as a black rock. In other words, the aged binder in RAP does not significantly contribute the total binder content. As the rheology of RAP may be affected by facing preheated aggregate and hot virgin binder, this assumption would most probably be different from what happens in reality. (2) All of the aged binder in RAP blends into the mix and with virgin materials effectively. Again, it is not clear whether this assumption is close to reality or not. Therefore, further research is needed to figure out the rate of interaction between the used and new materials and the significant parameters affecting this phenomenon. Previous study showed that, depending on the RAP size and aggregate gradation, the available binder content in RAP does vary (Saliani, Carter, Baaj, 2016). They have also concluded that there is a significant difference between large and small particles with respect to transition of the asphalt binder from RAP to virgin aggregates.

There was no guidance until early 1990s for the implementation of RAP in HMA, but based on experimental studies, FHWA Asphalt Mixture Expert Task Group defined the interim recommendations (Bukowski, 1997). Based on the performance of Marshall Mixes with RAP, and mixes designed according to the Superpave system, AASHTO Standards MP2 (now M323) describes how to design HMA with RAP (Basueny, Perraton, Carter, 2013).

Since RAP comes from different pavements and was milled by different equipment and different processes, many issues arise due to RAP variability when high percentages of RAP are used in a mix. One solution for this issue might be using RAP in different layers of pavement structure. An example of such application is discussed by Pratico et al. (2013) who describes the feasibility of building a two-layer porous asphalt (TLPA) by recycling from permeable European mixes (PEM) RAP, when highly variable RAP stockpiles are involved.

The aging level of RAP binder film thickness might be changed according to depth where the mix is placed at and presence of oxygen. Bressi , Dumont and Pittet (2015) proposed a methodology to detect the existence of a cluster phenomenon and they also proposed a first approach to show a different aging level in the RAP binder film thickness (i.e., partial differential aging). Stephens, Mahoney and Dippold (2001) also investigated the asphalt film properties for the coarse RAP aggregates. Based of these studies, aging depends on the film thickness and also particle size.

The level of interaction between old and new materials are a major factor that is still unclear. Different scenarios can be developed. However, there is no specific method to validate the level of interaction. In fact, many design procedures prefer to assume that all the aged binder is fully available and can be mixed with virgin binder and would effectively contribute to the blend ref. The Full amount of RAP binder can reduce the needed total amount of virgin binder.

Several mix designs factors like mix duration and temperature, RAP and aggregates surface area, can influence the rheology of the produced mix. So, it is important to characterize each part of the RAP precisely. Since aggregates consist more than 90% of the volume of the mixture, each particle has a great impact on RAP binder absorption.

It is desirable to have a higher recycle material content in hot mix asphalt (HMA), but incorporating large quantities can make the mix stiffer and more brittle. Consequently, these mixes are less workable, harder to compact in the field, and prone to crack (Kim et al. 2007; Mogawer et al., 2012; Munoz et al., 2015; Kaseer et al. 2017). Various strategies have been developed in order to increase RAP content such as incorporating a warm mix asphalt additive, adding rejuvenator agent, or a combination of mix designs (Kaseer, Martin, Arámbula-Mercado, 2019). In addition, RAP size is one of the critical factors that impact the level of blending between RAP and virgin particles. The degree of blending has not been quantified clearly so far. McDaniel and Anderson assumed that full blending happened during mixing

(McDaniel et al.,2000). Some investigations were carried out to evaluate the level of interaction between RAP and virgin asphalt (Bennert, Dongre, 2010; Bonaquist; 2005 Rowe, 2009), but they could not quantify it. In all above studies, the impact of RAP size by itself was not studied precisely.

In order to blend with the virgin asphalt binder, the RAP asphalt binder, or at least a part of the RAP asphalt binder need to be effective. The term available or effective RAP binder refers to the binder that is released from RAP, becomes fluid, and blends with a virgin binder under typical mixing temperatures (Kaseer, Arámbula-Mercado, Martin, 2019) . The stiffer the RAP binder, the less it will blend with the virgin binder. The binder stiffens with time mostly because of its reaction with oxygen (i.e., oxidization).

Oxygen availability for a given asphalt mix depends on the pavement structure and varies from outer layers to pavement sublayers. Oxygen diffusion depth is considerable (Jin, Cui, Glover, 2013). Also, asphalt film thickness affects oxygen diffusion. Sufficient methodology has been developed to estimate the diffusion depth (Rose, Arambula, Howell, Glover, 2014). Basically, using RAP in new mixes is a good idea for sustainable development, but information is still needed to understand the interaction between different components in a mix with RAP to optimize the properties of those recycled mixes.

In addition to the binder characterization, mix performance needs to be evaluated when a higher RAP content is used. The indirect tension and semicircular bending test results which were conducted by Huang et al. showed that RAP increases the mixture stiffness (Huang, Zhang, Kingery, Zuo, 2004). All RAP materials in Huang et al. study was screened through the No. 4 (5mm) sieve to acquire a consistent gradation. There is a possibility to increase the stiffness by adding fine RAP but it depends on job mix formula with inclusion of RAP. Huang et al. assumed that RAP binder totally contributes to the mix, an assumption which still needs to be verified.

Traditionally, black curves and white curves have been used for sieves analysis of RAP incorporated mixes. Black curves are the gradation of RAP particles from fractioned RAP and white curves are the gradation of recovered RAP aggregate after binder extraction. For a given mix, these two curves are significantly different. Al-Qadi et al. (2009) compared these two curves and concluded that; black curve tends to indicate higher amounts of large particles and lower amounts of fine particles. Therefore, to avoid the detrimental effects caused by unexpected extra fine particles, black curves are not suggested for use in job mix formula calculations. Using the white curve is common practice. However it is not the only approach being used. It should be noted that neither black curve nor white curve represents the actual gradation of the RAP material, and the real gradation lies somewhere in between (Roque, Yan, Cocconcelli, Lopp, 2015).

1.3 Short Aramid Fiber – HMA additive

Premature cracking of flexible pavements is a very common problem in Canada. Nowadays, it is common to use several types of additives in asphalt binders and asphalt mixes to improve their performance and increase the service life of flexible pavements. In order to mitigate the pavement cracking, the asphalt mixes used in the pavement structure need to have a high resistance to fatigue or thermal cracking according to their position in the pavement structure.

Several studies reporting on the use of fibers in asphalt concrete have been found in the literature. The types of fibers used in these studies can be divided into two categories, namely synthetic fibers and natural fibers. Synthetic fibers are the most common. Although there are many types of synthetic fibers, glass, carbon and aramid fibers represent the most common fibers used in asphalt mixes (Baaj, Di Benedetto,Chaverot, 2005).

Kevlar is an aromatic polyamide or aramid fiber introduced in early 1970s by DuPont. It was the first organic fiber with sufficient tensile strength and modulus to be used in advanced composite materials. Factory details of the aramid fiber show approximately five times higher tensile strength in comparison to steel (Fibremax ltd, 2021). Originally, it was developed as a replacement for steel in radial tires; Kevlar is now used in a wide range of applications.

Cellulose and mineral fibers are commonly used in gap-graded stone matrix asphalt (SMA) and open-graded or porous mixtures. Polypropylene and polyester fibers were previously used in dense-graded mixtures and are still used to some extent (McDaniel, 2015). Various polymers, steel wool, and other fibers are also sometimes added to asphalt mixtures. The relative benefits and issues with these various types of fibers are not well documented. The appropriate specifications and material characteristics to ensure the best performance in different climates, under different traffic loadings, and in different applications are also not widely recognized.

Fibers can come from various forms as well. Fibers can be short (ex. 1mm) and randomly oriented, long and unidirectional, tufts, or woven (McDaniel, 2015). The individual types of fibers can have various structures and cross-sections. Scanning electron microscopy was used by Chen & Xu (2010) investigate the structure of some fibers, including asbestos, lignin (cellulosic), polyacrylonitrile, and polyester. They found that the synthetic fibers had "antenna features" at their ends that helped anchor them in the binder phase, creating a stronger network within the binder. The asbestos fibers had a smooth texture and a thin diameter, yielding a large surface area. The cellulosic fiber had a rough texture, and the diameter varied along the length of individual fibers.

The physical dimensions of the fibers can affect how well the fibers can disperse and interact with the other components of the mixture. For example, the lengths of the fibers can be modified to relate to the maximum aggregate size in the mixture; short fibers might be used for smaller aggregate sizes. Long fibers may be difficult to mix uniformly into the mixture in the lab or plant because they can get tangled and clumped together (Wigotsky, 2002; Canadian Infrastructure ReportCard,2012). Sieve analysis canbe used to characterize fiber size. Clearly, if the length of fiber are too long (ex. 20mm), it may create the so called ‘‘balling" problem, and they may lump together, and they may not blend perfectly with the asphalt binder (McDaniel, 2015). Short fibers may just assume the role of filler in the mix.

Whichever type of fiber is used, one has to be careful with the length of those fibers. If the fibers are too long, they can clump together during mixing (Vale, Casagrande, & Soares, 2014). However, no study was found on the minimum length required to be effective. Kaloush, Biligiri, Zeiada, Rodezno and Reed (2010) have shown how the use of 19mm long mix of polypropylene and Aramid fibers is good to enhance the rutting and fatigue resistance, and they did not mention mixing problems. The physical dimension of fiber has a significant impact on the interaction and inter lock between HMA components. As an example, according to the maximum size of aggregates, length of the fibers can be modified. Long fibers may be difficult to mix uniformly into the mixture in the lab or plant because they can get tangled and clump together (Abtahi, Sheikhzadeh, & Hejazi, 2010). Additionally, surface area and absorption rate of fibers can also change the optimum binder content in the mix (Button & Lytton, 1987). So physical properties must be considered accordingly.

Fibers can be added to HMA by two potential methods:d or wet process. Prior to mixing binder with aggregate, fiber is blended with binder which is called wet process. As dry process, fiber is blended with aggregate. Dry process is preferred over the wet process because of easier mixing process and better distribution in HMA. Meanwhile since the fibers used do not melt in the asphalt, there are no apparent special benefits to the wet process. In addition, the field work done on fiber-reinforced asphalt mixtures has generally used the dry process (Echols, 1989 ; Hejazi, 2007), possibly due to the production problems of introducing the fibers directly into the asphalt (wet process). Another reason for using the dry process is that it minimizes the major problem of clumping or balling of fibers in the mixture (Labib & Maher, 1999).To sum up, several factors are taken into the account to characterize fiber. Those factors can be such as physical and chemical properties of fiber, interaction of fiber with virgin aggregate and binder and mix type. It was needed to characterize HMA due to adding fiber to mix according to failure criteria because of lack of knowledge of this particular material. According to different physical and chemical properties of Pulp Aramid Fiber (PAF) it can be used as additive in HMA in order to have last longer pavement.

1.4 Summary

As mentioned earlier in this chapter, previous studies confirm that RAP can be used successfully as an additives in HMA design. Review of the literature on RAP indicates that there is no consensus on several aspects of the RAP binder contribution to the new mix. Therefore, it is difficult to come up with a synthesis of previous work that would be unanimously acceptable. However various aspects of the impact of RAP on HMA performance have been reported, but more investigations are required to study the RAP particle size effect by itself without any changes prior to mixing with virgin aggregates and asphalt binder. It can be concluded that coarse and fine RAP fractions have relatively similar stiffness, but their contribution to the new mix is different. Also, to have more RAP in HMA, as a greener alternative to standard HMA, blending degree and aging rate of each group of RAP need to be well understood. It can be hypothesized that, by dividing a single source of RAP into two groups of coarse and fine particles, it is expected that both groups do not have a similar impact on the final performance of the HMA in which they are added. However, even if several studies have been reporting the interaction of RAP with HMA components, the comparison and the impact of the asphalt binder extracted from the coarse and fine RAP is unknown. To this end, various empirical and thermomechanical tests are adopted to validate the impact of RAP fractions on the HMA mixes.

In this chapter, it was also shown that different types of fiber have been recommended recently to reinforce the HMA against cracking. HMA performance with the inclusion of fiber depends highly on the physical and chemical characteristic of the fibers. Clumping is a common phenomenon when fiber is added to the mix. None of the studies found in the literature were performed using Pulp Aramid Fiber (PAF), which consists of tiny and short fibers (1 mm length). It is expected that the PAF, with special physical characteristic and high tensile strength, enhance tensile resistance of the HMA by providing better interlocking in mastic part of of HMA.

CHAPTER 2

APPROACH AND ORGANIZATION OF THE DOCUMENT

2.1 Research objectives

As demonstrated in the literature review, work is still needed to fully understand the effect of the addition of RAP on asphalt mixes properties. Many different aspects of the behavior of asphalt mixes with RAP need to be investigated, but the work presented here concentrate on two main aspects. The first is the interaction between RAP particles and the virgin asphalt binder and aggregate, and the second is the impact of the adding fibers to reinforced HMA to limit the cracks caused by aged asphalt binder. The main objective of this research is to develop a mix design with environmentally friendly additives such as reusing the reclaimed asphalt and adding aramid fiber. The specific objectives of this thesis are to:

- Verify the impact of separated RAP particles to Fine RAP (FR) and Coarse RAP (CR) on mix design;
- Quantify the active binder content in both groups of RAP particles (fine and coarse RAP);
- Evaluate the properties of asphalt binder in fine and coarse RAP particles from a single RAP source;
- Verify the interaction between RAP asphalt binder and virgin asphalt binder before and after mixing with fine and coarse RAP;
- Develop a HMA mix design with inclusion of FR and CR and compare the thermo-mechanical characteristics and performance;
- Determine the optimum asphalt content of the mixes reinforced with pulp aramid fibers, and
- Determine the impact of the addition of pulp aramid fibers on the volumetric and the mechanical properties of an asphalt mix.

2.2 Overall methodology

In order to reach the objectives, the work of this thesis is separated in two main parts. Figure 2.1 shows the overall methodology of this study. This dissertation is composed of two parts with two main objectives. Part A is aiming to increase the RAP content in HMA by comparing the impact of coarse RAP particles and fine particles separately in HMA. Part B is aiming to study the possibility of using specific fiber which is called Pulp Aramid Fiber (PAF).

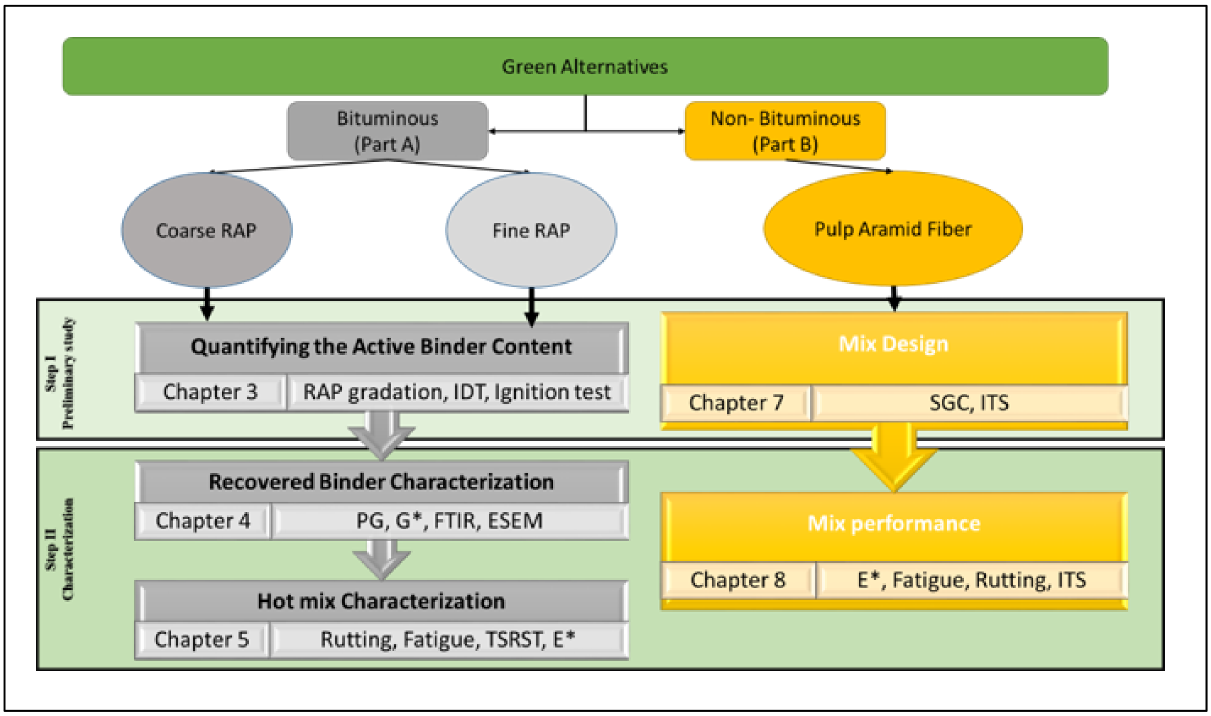

Figure 2.1 overall methodology of the study

2.3 Contribution

This research provides a contribution to sustainability in hot mix asphalt design and pavement material characteristics, particularly in road applications, and environmental engineering. Reusing waste materials or recycling materials is the best solution to save virgin materials. Also, RAP fractioning helps to develop HMA mix design with equivalent or better life expentencytahn conventional mix. The present study particularly contributes toward a better understanding of the utilizing as much recycled asphalt as possible as a greener alternative while also proposing a reinforcement additive for HMA. From environmental and economic aspects, using RAP in roadwork can reduce the demand for landfill, thus leading to lower waste, lower energy usage, and lower gas emissions. This phenomenon leads to a more sustainable environment. Furthermore, using RAP in roadwork contributes to conserving the natural aggregate and asphalt binder sources and reduces their usage, which is again beneficial to environment and economy.

2.4 Novelty of Research

Several studies have been done on reusing materials such as recycled asphalt material which leads pavement agencies to develop guidelines and methods regarding the concerns and limits of using RAP in HMA. The innovative part of this study is the thermomechanical characterization of mixes containing only coarse or fine RAP particles. The study on mixes containing finer or coarser RAP are available in the literature, but little information is available on the effect of separating the RAP into two different fractions, and that effect has on the properties other than the virgin binder content. The results show that separating RAP into coarse and fine could help to enhance the thermo-mechanical properties of the mixes and increase the amount of maximum RAP content in HMA. The RAP content in HMA increased to 54% without any specific additives.

In addition, the present study is introducing the new type of fiber as the reinforcement element in HMA. Pulp Aramid Fiber which, called PAF in this project, was applied for the first time in HMA as the novel additive based on potential physical and chemical characteristics of this special fiber. The results show that tiny and short fiber leads the pavement resist longer under the repeated tension compression loads.

2.5 Organization of the Thesis

The work presented in this thesis is organized in nine chapters, including the general introduction, summary, literature study, a series of articles and general conclusions. Since this is a paper-based thesis, chapters 4, 5 and 8 are papers published in peer-reviewed journals, and the work presented in those papers is completed by conference papers presented in chapters 3, 6 and 7.

CHAPTER 1 provides a literature review that starts with a summary regarding to bituminous mixtures components. An overview is given for conventional hot mix asphalt, Recycled Asphalt and Pulp-Aramid fiber.

CHAPTER 2 presents the research methodology, objectives, contribution, and novelty of the work.

CHAPTER 3 focuses on the preliminary study about the separation of RAP particles in two groups of fine and coarse and the quantifying the active aged binder in both groups. The chapter is based on a study presented at the Annual Conference of the Canadian Society for Civil Engineering (CSCE) - Resilient Infrastructure (London, ON, Canada, June 01-04, 2016).

CHAPTER 4 describes in detail the laboratory procedure and results from comparing the recovered asphalt binder characterization of RAP, Coarse RAP, Fine RAP, mix of coarse RAP and virgin, mix of fine RAP and virgin and regular asphalt binder. This chapter is clarifying the asphalt binder characteristics according to particle size. This chapter is based on the paper titled “Characterization of Recovered asphalt binder from Coarse and Fine Reclaimed Asphalt Pavement Particles”, and it was published in a special issue titled Recent Advances and Future Trends in Pavement Engineering in the journal Infrastructures, in 2019.

Only recovered asphalt binder of mixes were studied in previous chapter but whole mixes characterization including virgin aggregate, RAP aggregate, RAP asphalt binder and virgin asphalt binder are still missing. **CHAPTER 5** is based on the published journal article titled “Characterization of Asphalt Mixtures Produced with Coarse and Fine Recycled Asphalt Particles”, published in a special issue titled Recent Advances and Future Trends in Pavement Engineering in the journal Infrastructures, in 2019. This paper describes in detail the advantages and disadvantages of adding separated RAP particles to HMA.

The conclusions of the work presented in Chapters 3, 4 and 5 are summarized in **CHAPTER 6** Which concludes Part A of the thesis.

CHAPTER 7 covers the preliminary study of reinforcement additives, called pulp Aramid fiber, in HMA. This chapter discusses the development of mixing procedure of HMA with inclusion of pulp aramid fiber. This work was presented in the Sixty-Second Annual Conference of the Canadian Technical Asphalt Association (CTAA): Halifax, Nova Scotia, in 2017.

CHAPTER 8 is referring to the journal article published in Construction and Building Materials journal as the second step of part B of the thesis to study the impact of Pulp Aramid fiber in HMA performance in short- and long-term life.

The results and conclusions drawn from the work presented in chapters 7 and 8, which represent the work done in part B of this thesis, are summarized in Chapter 9. Finally, a summary of the conclusions of the research work, as well as recommendations that are needed for future studies and research, are discussed. In addition, the list of references is provided at the end of the document before the appendices.

CHAPTER 3

INVESTIGATION OF THE IMPACT OF RAP GRADATION ON THE EFFECTIVE BINDER CONTENT IN HOT MIX ASPHALT

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Conference proceeding and Presented in Annual Conference of the Canadian Society for Civil Engineering , (CSCE): Resilient Infrastructure
London, Ontario, Canada, June, 2016

3.1 Abstract

Nowadays, it is common to add little amount of Reclaimed Asphalt Pavement (RAP) in asphalt mixes without changing too much the properties such as the modulus and the low temperature cracking resistance. Not only will those mixes be able to make roads last longer, but they will be a greener alternative to usual mixes. In order to make a flexible pavement design, the mixture behavior is usually characterized with the complex modulus. To have a high modulus mix, you need to control the gradation precisely even when RAP is added. When performing a mix design to incorporate RAP, it is desirable to know the RAP binder characteristics and content and its gradation. In the literature, there is no clear vision of the RAP gradation impacts on the mixture properties and field performance. The objective of this study, performed at the Pavements and Bituminous Materials Laboratory (LCMB), is to evaluate the impact of RAP gradation on Hot Mix Asphalt. This is needed to understand how much binder can be transferred during mix from RAP to virgin aggregate. In this study, a single source of RAP was separated into different sizes and mixed with specific group of virgin aggregates. Then according to their size, the mixes were separated again into RAP group and virgin aggregate. While these were mixed, active RAP binder transferred to virgin aggregate. Then ignition test (ASTM D6307) was adapted to separate RAP binder from virgin aggregate. With this

procedure, it was possible to see that, for a given temperature and mixing time, activated binder amount at coarse RAP particles and fine RAP particles. The ignition test result showed that coarse RAP particles have more active binder in mix, but ITS test indicated that fine RAP particles have higher strength.

Keywords: Hot Mix Asphalt, Recycled asphalt, RAP gradation, IDT, Ignition test.

3.2 Introduction

Nowadays, it is common to add little amount of Reclaimed Asphalt Pavement (RAP) in asphalt mixes without changing too much the properties such as the modulus and the low temperature cracking resistance. Not only will those mixes be able to make roads last longer, but they will be a greener alternative to usual mixes a reusable mixture of aggregate and asphalt binder can be a worth approach for technical, economical, and environmental reasons. RAP that is consisting of aged binder and aggregate particles provides saving energy according to the various RAP content with considering the total cost (Kandhal & Mallick, 1998).

Despite the fact that there is no recognized limit for the amount of RAP that can be added to any mixes, it has been limited by many agencies and it varies between 10 to 50%. Afterward in 1997, the Federal Highway Administration's RAP expert task force developed guidelines for the design of Super-Pave HMA containing RAP (Bukowski, 1997). These guidelines have been supported by the findings of the NCHRP research report 9-12 (R. S. McDaniel, Soleymani, Anderson, Turner, & Peterson, 2000). But there is no clear vision of how it can be added to the mix and what conditions are needed to prepare RAP before mix. Level of interaction between old and new materials is a major factor that is still unclear. Different scenarios can be developed such as there is no interaction, so RAP can be called Black Rock, it means that it does not significantly change the virgin binder properties. But with heating there is the possibility that RAP binder can change the rheological properties of the mix. However, there is no specific method to see how it works. In fact, many design procedures prefer to assume that all the aged binder is fully available and can be mixed with virgin binder and would effectively contribute to the blend. The Full amount of RAP binder can reduce the needed total amount of virgin binder.

When RAP is used, many mix designs factors like mix duration, mix temperature, RAP and aggregates surface area, etc., can influenced the rheology of the produced mix. Because of this, it is important to characterize each part of the RAP precisely. Since aggregates account for more than 90% of volume of mixture, each particle has great impact on RAP binder absorption. This paper is going to characterize the impact of each particles with respect to the active binder that can cover the virgin aggregates.

In this research, the amount of RAP binder that can be mobilized from RAP aggregate to virgin aggregate during construction was analyzed. In fact, the impact of virgin aggregate size on RAP binder mobilization was investigated.

3.3 Background

Milled pavements can be considered as a valuable material after they reached the end of their service life. Reclaimed Asphalt Pavement (RAP) can be added to virgin Hot Mix Asphalt (HMA) in order to preserve materials and energy. However, it is necessary to account for old materials in the HMA design process. The rheology properties of asphalt continuously changes during the road service life and it would not be the same as virgin materials, but at least RAP can act like virgin materials even because of its new rheology properties, it can improve the performance of new pavements. Since past decades, several studies have been done to characterize the RAP and use it as proper way in mixture.

At the beginning of the implementation of RAP, there were no guidance on how to integrate RAP into the new mix design, but based on experimental research, interim recommendations were defined through the FHWA Asphalt Mixture Expert Task Group (Bukowski, 1997). Afterwards, according to the performance of Marshall's mixes with RAP, new specifications were developed in 2002 and it has been available in the Super-pave system. In addition, AASHTO Standards MP2 (now M323), standard specification for Super-pave volumetric mix design for hot mix asphalt, describe how to design HMA with RAP (Basueny, Perraton, & Carter, 2013).

As aged binder and aggregated are included so all conditions like mix temperature, Mix duration, aggregate gradation, preheating conditions can change the performance of new mix. Europe standard method EN 12697-35 presented a method on how to prepare RAP materials to add to mixture. It can be added in different manner like cold, heated in a microwave, heated in an oven in a pan covered, and heated in an oven in a pan non-covered. Preheating condition impact would be change according to the changing the amount of RAP in mix. Basueny et al.., (2013) showed that they cannot propose a specific method from the four methods to be used in the laboratory since each method has its advantage and disadvantages from the degree of handling and the required time saving.

With respects to the different conditions and mix durations and temperatures, the reaction between the RAP and the other components could change. Achieving at least same performance level of HMA without RAP is the critical aspect to reuse asphalt. Several researches have been done to understand the properties of mix with inclusion of RAP. One of the main concerns is the degree of interaction of old materials with new one, and the behavior of RAP during a new mixing procedure according to different percentage of RAP. Since a correlation was found between microstructural characteristics and mechanical properties (Nahar et al.., 2013) it becomes important to develop a deep understanding of the physical phenomena occurring during a new fabrication as well as the definition of the degree of blending to determine the rheology of the final binder (Booshehrian, Mogawer, & Bonaquist, 2012).

The interaction degree is a serious concern that directly affects the performance of HMA that incorporates RAP. The level of blending affects both the performance of the produced HMA and the economic competitiveness of the recycling process. It can be assumed that RAP totally participate in mix when it is actually behaving as a black rock, or it can be assumed that RP binder does not blend with the virgin binder when there is some evidence that proof that it does and finally complicated assumption is that the blending process may take some time to occur and is influenced by various factors (Carpenter & Wolosick, 1980). There are various possibilities according to utilize RAP in mix. It can act as a black rock; it means RAP binder has no impact in mix. Black rock in NCHRP 9-12 were fabricated by extracting binder from

RAP afterward RAP aggregate can be added to mix. Another possibility is that RAP completely participates in mix so RAP binder blends with virgin binder as well as RAP aggregate blends with virgin aggregate. Third one called practical blended which is blending unprocessed RAP with virgin material. The NCHRP 9-12 report concluded that at 40% RAP content, the black rock exhibited significant differences in laboratory performance compared with the actual practice and total blending mixtures. There were no significant differences between the total blending and actual practice mixtures (R. McDaniel, Soleymani, & Shah, 2002).

Stephens, et al. (2001) conducted an experimental program to evaluate the effects of blending between RAP and virgin binders on the resulting Super-Pave grade. To validate that RAP does not act as a black rock and has an effect on the overall blend. The difference between the prepared samples was the RAP preheating time before being added to virgin aggregates and binder, if RAP acts as a black rock, preheating time should not have any effect on the mix properties. In contrast, if long heating times facilitate the blending between aged and virgin binders, an increase in the mix strength should be detected. In addition, when comparing the mix with no preheating to the mix made with virgin materials, an increase in strength is immediately observed upon adding the RAP to the virgin materials even without any preheating.it can be concluded as RAP being added to mix the stiffness also would be changed however the preheating time and temperature can change the mix stiffness.

In order to simplify the visualization of the blending of the virgin and the reclaimed materials, different particles of virgin aggregate and RAP size mixed with virgin binder were done. Complex modulus and phase angle of reclaimed binder extracted from mix were measured, and the results show that blending is not homogeneous throughout the sample. Some locations show good blending whereas other locations appear non-blended with micro-cracks forming at the binder boundaries. (Rinaldini, Schuetz, Partl, Tebaldi, & Poulikakos, 2014).

On the other hand, the amount of RAP content can change the blend degree that involves several physical and chemical phenomena. In order to understand and control the RAP behavior Bressi et al., (2015) proposed a methodology to detect the existence of a cluster phenomenon (Figure 3.1) and she also proposes a first approach to show a different aging level in the RAP binder film thickness (i.e. partial differential aging). Quantity of new bitumen which is needed to add was determined at first step then the verification and quantification of the phenomena detected in the first part were carried out. The clusters formation might have important consequences on the RAP mixture behavior. Clustering could prevent the uniform distribution of the virgin binder, which results in an increase of the heterogeneity of the mixture.

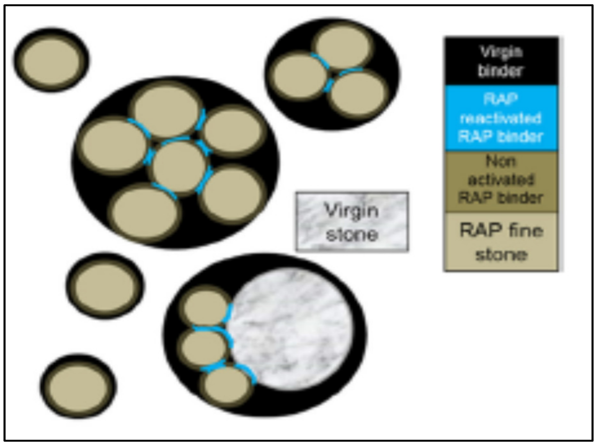

Figure 3.1 Schematic diagram of cluster phenomenon
Taken from Bressi et al. (2015)

Moreover, they indicated that the quality and quantity of virgin aggregates could play a main role in the cluster's formation, as well as aggregate shapes and aggregate texture.

In order to insert as much RAP as possible in HMA, a coarser HMA mix has been encouraged by Super-pave mix design method which requires tight control of both the overall gradation and percent passing the 0.075-mm (No. 200) screen. Implementation of RAP has been seriously limited because stockpiles of RAP may have widely variable gradations as well as high percentages of minus 0.075-mm material. One possibility for maximizing the use of RAP in Super-pave mixtures was suggested to screen out the finer RAP fractions (Stroup-Gardiner & Wagner, 1999). This would minimize the minus 0.075-mm material and help produce a more uniform coarse RAP gradation. Both of these factors should permit a higher percentage of RAP to be used and still meet the Super-pave graduation requirements (Stroup-Gardiner & Wagner, 1999).

Each RAP fractions, fine and coarse, have specific characteristics that should be taken into account when RAP is added into a mix. For instance, fine RAP fractions consistently show a higher asphalt content according to the higher surface area per unite weight. Regardless of RAP source, the portion passing the 1.2mm screen consistently has a binder content of 1.0 to 1.5 percent greater than for the coarse RAP. This agrees with previous research of Scholz et al., (1991) which indicated that higher binder content could be advantages of using this fine part as well as it may provide a noticeable savings in the amount of neat asphalt needed while using a lower percentage of RAP materials. However, it was not mentioned if all this extra binder participates in the mix or if it just glues aggregates together (cluster phenomenon). The most obvious limitation in using this material in Super-pave mixtures is the high percentage of very fine material. The analysis performed on RAP samples reveals a certain degree of variability in RAP binder content and gradation, being higher in the coarse RAP fraction. As a consequence, dividing RAP into several fractions and using higher percentages of fine RAP fraction results in less variability of bitumen content and gradation in the recycled mixtures. It was summarized, splitting the RAP stockpiles on the 1.2-mm screen results in two potential benefits: (a) increased uniformity between the RAP sources in the coarser fractions and (b) a reduction in the material passing the 0.075-mm screen. To maximize implementation of RAP, splitting RAP stockpile into fine and coarse fractions was suggested but it was not clarifying at all.

Stephens et al. (2001) also investigated the concept that asphalt films on coarse aggregates would be more prone to blending with virgin aggregates than asphalt film around fine aggregates. Recovered binder from coarse and fine particles was compared to each other using a Dynamic Shear Rheometer (DSR). They concluded that there is no correlation between variation in the binder stiffness and the asphalt coating of coarse or fine aggregates. Main issue in this domain referred to its exposure to heat and air during production, which is a random process and does not relate to whether the aggregate is coarse or fine. Laboratory testing conducted in this study also indicated that the use of RAP substantially affects the binder blend grade.

To sum up previous researches, it can be concluded that the binder film thickness around fine particles are differing from coarse particles and it was not clarified the impact of different RAP aggregate size in mix also on the other hand if whole RAP was added it can caused variability in final results and these variability made it complicated to characterized RAP in mix as black rock or semi blended materials of fully blended. To come up with that new mix designed developed to make clear vision of impacts of RAP aggregate size.

3.4 Objectives

The main objective of this paper is to evaluate the impact of RAP gradation on Hot Mix Asphalt. It is needed to determine how much binder can be transferred during mix from RAP to virgin aggregate.

In this study, a single source of RAP was separated into different sizes and mixed with specific group of virgin aggregates. Then according to their size, the mixes were separated again into RAP group and virgin aggregate. RAP active binder transferred to virgin aggregate while these groups were mixed. Then ignition test (ASTM D6307) was adapted to separate RAP binder from virgin aggregate. As a result of this project, it is possible to figure out activated binder amount in coarse RAP particles and fine RAP particles for a given temperature and mixing time, Finally Indirect Tensile Test was adapted to evaluate the quality of different mixes.

3.5 Materials

The mobilization of RAP binder defines the RAP binder that transfer from RAP aggregates to virgin stones or mix with the virgin binder. In order to evaluate the mobilization of RAP binder, a single source of RAP and a single source of aggregate are adapted to minimize the impact of different sources of material on final result. On the other hand, RAP was blended with aggregate without adding virgin binder in order to have a clear vision of RAP binder mobilization.

In this experimental program, a dense graded 20 mm HMA commonly used as a base course in Quebec (GB20) was designed with a PG 64-28 binder. The selected virgin binder (PG 64-

28) is a medium grade asphalt binder that can be used in warm climates. Three different classes of virgin aggregates were selected to produce the GB20 blend with inclusion of RAP without adding virgin binder. The control points for GB-20 pavement mixture are shown in Table 3-1.

In order to meet the hot mix requirements, the proportions of each aggregate gradation must be determined. The particular aggregates were selected based on LC method specifications such as maximum passing percentage of coarse particles, minimum Passing percentage of coarse particles and mid-point of specification.

In order to achieve desired gradation, mix design was driven from 5 different classes of aggregates and a source of RAP. The properties of different stockpile are presented in Table 3 2.

Table 3.1 GB-20 gradation specifications

	Specification
Sieve	**GB 20**
mm	
28	100
20	95-100
14	67-90
10	52-75
5	35-50
2.5	-
1.25	-
0.63	-
0.315	-
0.16	-
0.08	4,0-8,0

Table 3.2 Different aggregates used in this study

	RAP	filler	0-5	5-10	10-14	14-20
Sieves (mm)						
28	100%	100%	100%	100%	100%	100%
20	100%	100%	100%	100%	100%	85%
14	100%	100%	100%	100%	89%	13%
10	90%	100%	100%	91%	28%	2.60%
5	57%	100%	93%	4.00%	8.10%	2.00%
2.5	36%	100%	48%	1.00%	3.70%	1.70%
1.25	23%	100%	28%	1.00%	2.00%	1.50%
0.63	13%	100%	16%	1.00%	1.50%	1.40%
0.315	6.00%	99%	7%	1.00%	1.30%	1.30%
0.16	2.00%	93%	4%	1.00%	1.20%	1.20%
0.08	1.00%	79%	2.30%	0.40%	1.20%	1.10%
Filler					1.10%	1.10%

3.6 Methodology

The methodology of this project can be divided into different steps (Figure 3.2). First, a GB20 mix design with and without RAP was performed. Then, the mobilized RAP characteristic was quantified with the ignition test, and finally, the mobilized RAP evaluation was done. In this project, fine particles are defined as virgin aggregates, or RAP particles, which pass through the 5 mm sieve, and coarse particles were defined as those particles which were retained on the 5 mm sieve.

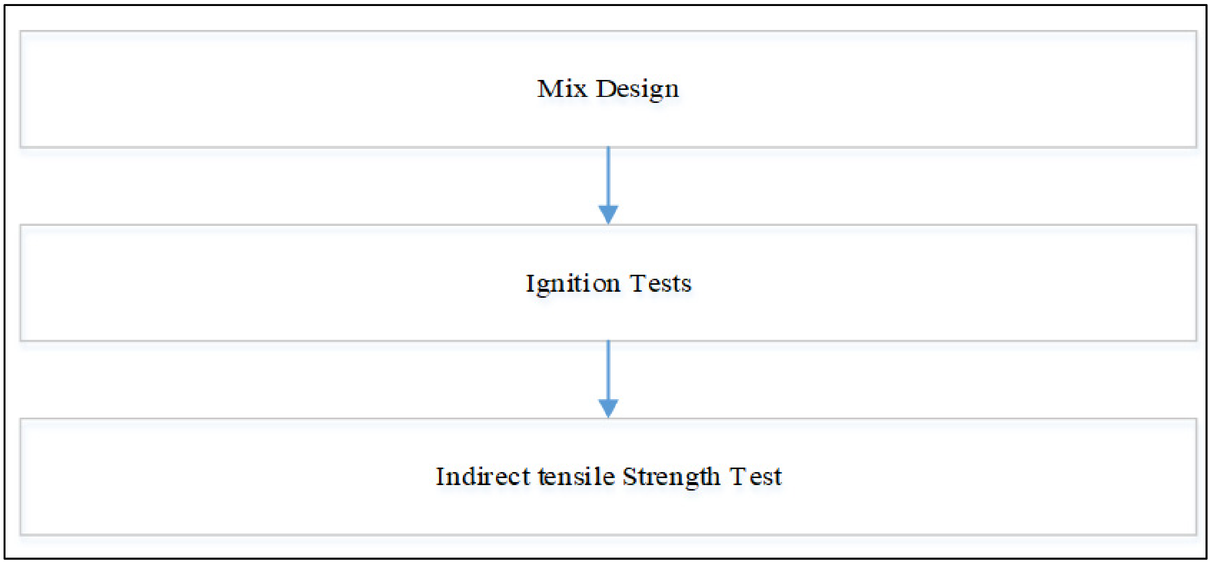

Figure 3.2 Steps of program

According to LC method specification, sieve analysis should be done to separate coarse from fine particle in RAP and virgin Aggregate. indicate different scenarios of blending virgin aggregate and RAP particles according to their size and percentage. There are two parts in each sample, one-part RAP and one part virgin aggregate as it was shown in Table 3.3 .

The mix design properties are performed step by step which are mention here:

- Prepare corresponding weigh of fine (820g, 1000g, and 700 g) and coarse (1180g, 1000g, 1300g) from RAP and virgin aggregate;
- The RAP is heated in an oven at a temperature of 60 ° C ± 5 ° C, stirring frequently. It is not possible to heat the RAP with the aggregates. It is only necessary to heat the aggregate to a temperature of 15 ° C to 25 ° C higher than the heating temperature for mixing specified in LC 26-003test, without exceeding a temperature of 190 °C. The desired binder for this mix is 64-28, so the mix temperature should be 155 °C. Consequently, aggregates preheating temperature is 180 and compaction temperature is 145°C. It should be precise that the mixing temperature was chosen according to the binder, but that no virgin binder was added;

- In this step preheated RAP was added to the virgin aggregates. The RAP being a separate raw material, it is brought via the mixer, and mixed with preheated virgin aggregate so that the end temperature is about 155 ºC. This process is referred to in the literature as hot recycling using Batch plants with a separate heating drum. After mixing, the mix is cool down back to room temperature;
- Separate coarse from fine by using the sieve 5 mm for each sample.

Table 3.3 Different Mix design of blending virgin aggregate and RAP particles

N	**%fine**		**%coarse**	
	RAP	**Virgin**	**RAP**	**Virgin**
1	41%			59%
2		41%	59%	
3	50%			50%
4		50%	50%	
5	35%			65%
6		35%	65%	

The asphalt binder content of the RAP was determined using LC26-006 or ASTM D6307 ignition test methods. The values of indirect tension (IDT) strength may be used to evaluate the relative quality of bituminous mixtures in conjunction with laboratory mix design testing and for estimating the potential for cracking. In this study, samples of 150mm in diameter were compacted with a Superpave gyratory compactor, and the indirect strength is calculated with the following equation:

$$S_t = \frac{2000 \times P}{\pi \times t \times D} \tag{3.1}$$

St = IDT strength, kPa;

P = maximum load, N;

t = specimen height immediately before test, mm;

D = specimen diameter, mm.

3.7 Results

The different GB20 designed in this project all have the same gradation. According to specified gradation Figure 3.3, three different fine RAP percentages was added to coarse virgin aggregate and three coarse RAP percentages was added to fine virgin aggregate. Weight of samples was measured before and after separation to determined amount of lost materials during this process. According to the comparison of these two conditions, around 20 % percent binder transferred to virgin fine aggregates when coarse RAP was added. Mobilized RAP includes binder and mastic which can be taken from RAP. There are several factors that can change the mobilized RAP content. According to the literature, it is supposed to get higher mobilized binder in fine RAP compared to coarse RAP. Mix temperature, time and amount of binder also can help RAP binder coat virgin aggregate properly.

As it can be seen in Figure 3.3, separated fine and coarse particles were tested in order to figure out the binder content. It was expected that fine RAP has more percentage of binder. Binder amount in particles can be increased with growth of aggregate surface area and make aggregate gradation finer. RAP binder content was determined by ignition test. Ignition oven gave us 5.22% of binder content for the total RAP (fine and coarse part together). Additionally, the binder content was found separately in fine and coarse particles (3.68% Coarse RAP binder content and 6.6% Fine RAP binder content).

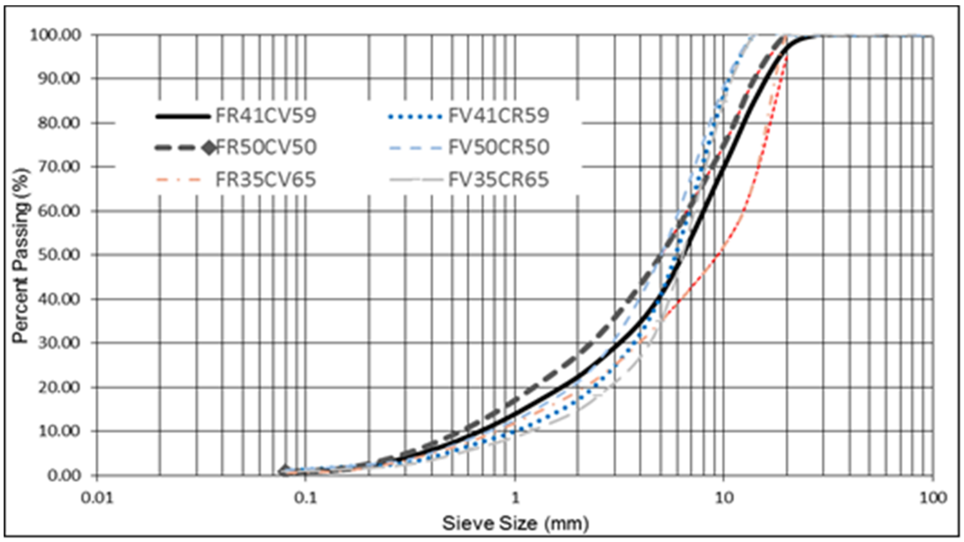

Figure 3.3 Gradation of the tested mixes

Because of higher binder content in fine RAP, it was expected that after mixing, coarse virgin aggregates would have absorbed more binder than the fine virgin aggregates that were mixed with coarse RAP. Samples with coarse RAP seem more homogenies than samples with Fine RAP (Figure 3.4).

Figure 3.4 Mixture with inclusion of Fine virgin aggregate and fine RAP

Despite higher binder content in fine RAP, results in Figure 3.5 show that the binder content in fine virgin aggregate are much more than coarse virgin aggregate.

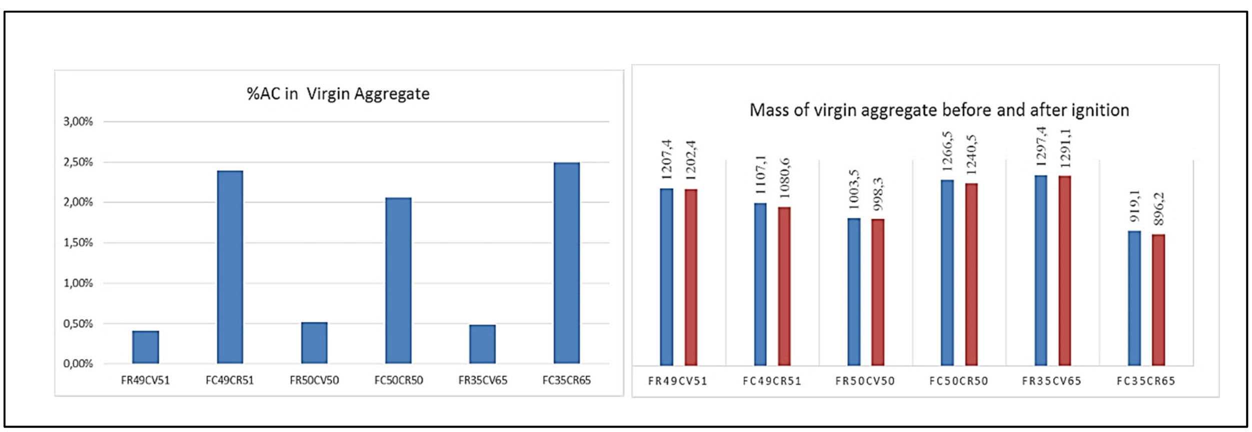

Figure 3.5 %AC of virgin aggregate by ignition test

This means that more RAP binder transferred from coarse RAP than fine RAP particles. It can be referred to surface area of virgin aggregate. Virgin aggregate has rough surface. The rough texture can cut off the RAP binder from RAP aggregate during the mix. So particles which are faced to the more rough texture could participate more in mix. Coarse RAP has more chance to face with rough texture than the fine RAP.

Surface area can play the main role in this domain. Fine graded aggregate has more surface area than the coarse. As we have different kind of gradations, so it was needed to see the surface area factor. The surface area is determined by multiplying the surface area factors (given in the Asphalt Institute Manual) by the percentage passing the various sieve sizes (Clark et al. , 2011).

However, they could not find the background research data for the surface area factors in the literature. They concluded that further research is needed to verify these surface factors and the concept of film thickness. The rough surface area or in other meaning, virgin aggregate surface area was needed. Figure 3.6 shows the surface area according to samples gradation. Almost the same surface area is reported for each group, so it can be concluded that the total surface area can not verify perfectly binder transferred binder. It was needed to present rough surface area. Rough surface area was calculated from virgin aggregate part in each sample. With respect to the Figure 3.5 and Figure 3.6 it can be summarized that the rough surface may effect on binder transferring. More RAP binder can mobilize from RAP to virgin aggregate according to the higher rough surface area.

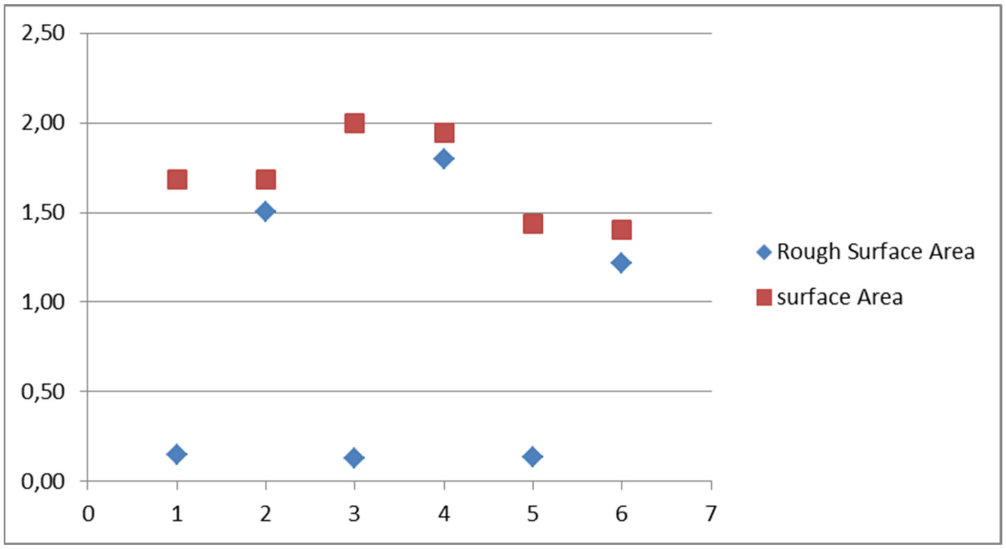

Figure 3.6 Surface Area

It was tried to see the impact of RAP gradation on mix. Previously it was understood that despite the fact that fine RAP has more binder, coarse RAP has more transferred binder according to the high virgin aggregates surface area. It was needed to compare the impact of coarse and fine RAP precisely. For that, indirect tension was performed on three different mixes for which the gradation is presented in Figure 3.7. For those mixes, 34% RAP was added as fine RAP, coarse RAP or complete RAP. In order to have the same overall gradation, the virgin aggregates gradation was adjusted for each mix. As for the first part, no virgin binder was added, but the mixing and compaction temperature were taken as if a PG64-28 binder was used.

As it can be seen in Figure 3.8, except those mixes with inclusion of fine RAP, all the rest could not hold themselves together after 200 times gyration (not enough cohesion). The main reason was lack of proper binder content. Fine RAP has enough binder content to hold virgin aggregate without adding virgin binder, but it was doubted that available binder content could change rheological properties of mix or at least it could be able to cover virgin aggregate, consequently, decreases the virgin binder content.

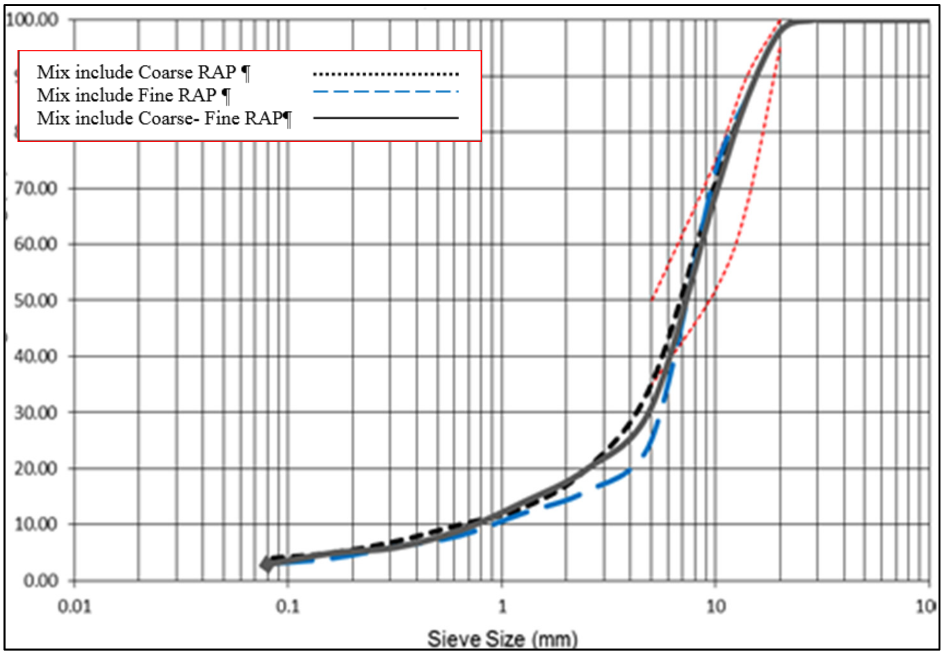

Figure 3.7 Samples Gradation

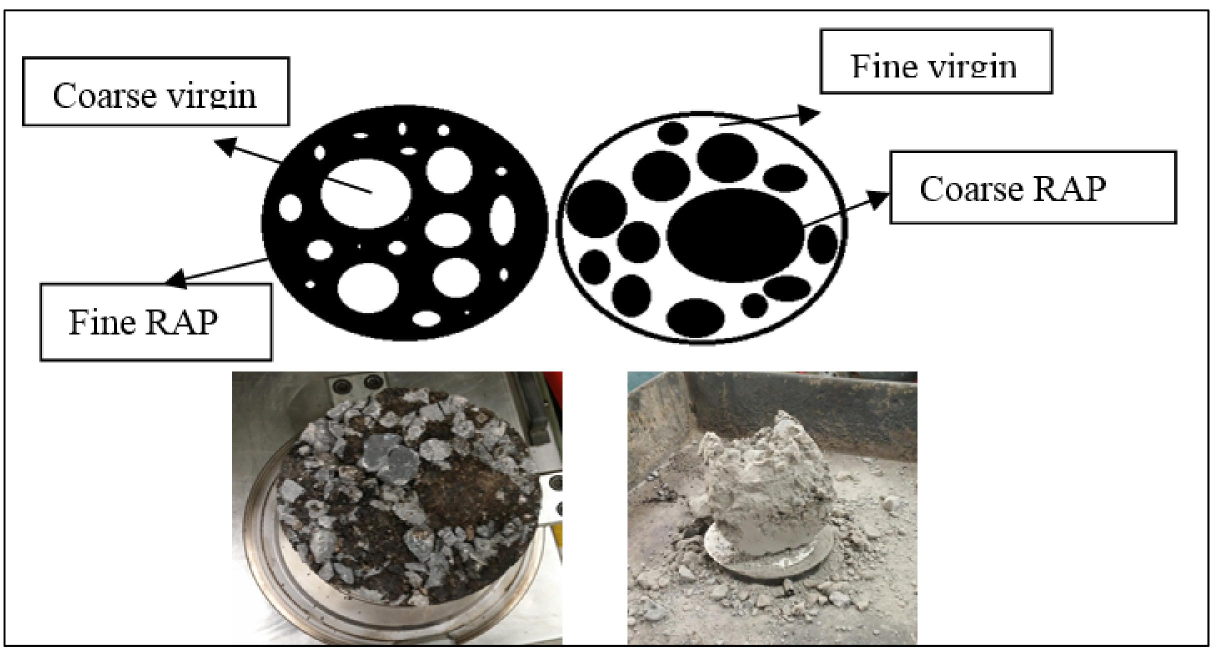

Figure 3.8 Samples cohesion after gyratory compaction

3.8 Conclusion

This paper investigates impact of RAP gradation on effective binder in HMA. Binder content in fine RAP particles is higher than in coarse RAP particles because fine particles have more surface area compare with coarse particles. It means that binder content in different RAP particles varies according to its aggregate gradation. Differences in binder content can change the amount of required virgin binder in mix. On the other hand, it was doubted that if RAP participate in mix as black rock or its binder blends with virgin binder partially or completely. The level of participation has great impact on chosen binder type and mix stiffness. Various scenarios were developed to understand the interaction of RAP with virgin aggregate.

In the first part of this study, the same quantity of RAP was added to virgin aggregates in all mixes, but as fine RAP or as coarse RAP. On second step, final gradation was fixed, and 34% RAP was added to mix. Ignition test, and compaction of samples to perform IDT. The following conclusions were drawn:

- Fine RAP has higher binder content, but the binder does not participate completely in the mix;
- Transferred binder or mastic from fine RAP to coarse virgin would not be a lot even it cannot cover virgin aggregates;
- Although it cannot cover the virgin aggregate, it can hold virgin aggregates together without adding virgin binder;
- Coarse RAP has lower binder content compare to fine RAP but when mix with virgin aggregates, more binder transfer from the RAP to the virgin aggregates;
- Although coarse RAP has more transferred binder, but it cannot hold aggregate properly according to lack of binder content.

Further studies are needed to figure out the impact of mix duration and temperature on the amount of transferred binder also it would be needed to validate these results with different source of materials.

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CHAPTER 4

CHARACTERIZATION OF RECOVERED BITUMEN FROM COARSE AND FINE RECLAIMED ASPHALT PAVEMENT PARTICLES

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4.1 Abstract

In the current era of road construction, it is common to add a small amount of reclaimed asphalt pavement (RAP) in asphalt mixes without significantly changing properties such as stiffness and low-temperature cracking resistance. Not only can these mixes be better for the environment, but they can also improve certain properties like rutting resistance. However, there is no clear understanding of how RAP gradation and bitumen properties impact the mixture properties. In this study, a single RAP source was separated into coarse and fine particles and added into a hot mix asphalt (HMA). Fourier transform infrared (FTIR) spectrometry was used to evaluate the chemical properties of the bitumen, while environmental scanning electron microscopy (ESEM) image analysis was used to visualize the differences of the virgin and RAP bitumen at a microscopic level. The observed results indicated that the recovered bitumen from coarse RAP did not have the same characteristics as the fine RAP bitumen, and the interaction of RAP bitumen with virgin bitumen significantly depended on RAP particle size. The amount of active RAP bitumen in coarse RAP particles was higher than in fine RAP particles.

Keywords: hot mix asphalt, recycled asphalt, RAP gradation, ignition test, FTIR, ESEM

4.2 Introduction

Nowadays, sustainability and environmental matter are the main issues for most research studies on construction materials. One of the solutions is to use reclaimed asphalt pavement (RAP) as the main recycled material used in hot mix asphalts. Some researchers (McDaniel et al., 2012; Mogawer et al., 2013; Sias Daniel et al., 2013; Visintine et al. 2013) have studied the use of RAP in road construction and concluded that because of the large environmental potential and cost benefits, it is important to maximize RAP content in the mix design. According to Al-Qadi et al. (2009), the performance of pavements using properly prepared RAP was found to be satisfactory in terms of fatigue, rutting, thermal resistance, and durability.

The required thickness for pavement design depends on the pavement design method and material characteristics (Perraton, Baaj, Carter, 2010). Saliani et al. (2017) concluded that the surface area of mix components, like virgin aggregates, fiber, RAP, etc., impact bitumen absorption. Mix design factors, like mix duration and mix temperature, influence the rheological properties of mixes with RAP particles. Because of this, it is important to precisely characterize RAP according to particle size. Previous studies have shown that active bitumen content in coarse RAP particles is more than that for fine RAP particles (Saliani, Carter, Baaj , 2016).

It is desirable to have a higher recycle material content in hot mix asphalt (HMA), but incorporating large quantities can make the mix stiffer and more brittle. Consequently, these mixes are less workable, harder to compact in the field, and prone to crack (Kim et al., 2007; Mogawer et al.2012; Munoz et al., 2015; Kaseer et al., 2017; Kaseer et al., 2018). Various strategies have been developed in order to increase RAP content such as incorporating a warm mix asphalt additive, adding rejuvenator agent, or a combination of mix designs (Kaseer et al., 2019). In addition, RAP size is one of the critical factors that impact the level of blending between RAP and virgin particles. The degree of blending has not been quantified clearly. McDaniel and Anderson assumed full blending happened during mixing (McDaniel, Soleymani, Anderson, Turner, Peterson, 2000). Some investigations were carried out to evaluate the level of interaction between RAP and virgin asphalt (Bennert & Dongre, 2010;

Bonaquist 2005; Rowe 2009) and enhance the understanding of the interaction degree, but they could not quantify it. In all above studies, RAP size by itself was not studied precisely.

On the other hand, film thickness of bitumen that surrounds RAP aggregate particles can significantly impact HMA performance. Bressi et al, (2015) proposed an initial approach to show different aging levels in RAP binder film thickness, and they proposed a methodology to detect the existence of a cluster phenomenon in RAP binder. Aging level and cluster phenomena can also impact the level of interaction between RAP and virgin materials.

Gardiner (Stroup-Gardiner, Wagner, 1999) concluded that the complex modulus was not solely controlled by the stiffness of the binder, whereas several other factors, including the gradation and angularity of the aggregate, played a main role in stiffness.

The term available or effective RAP binder refers to the binder that is released from RAP, becomes fluid, and blends with a virgin binder under typical mixing temperatures (Kaseer, Arámbula-Mercado, Martin, 2019). The stiffer the RAP binder, the less it will blend with the virgin binder. The binder stiffens with time mostly because of its reaction with oxygen (i.e., oxidization).

Oxygen availability depends on the pavement structure and varies from outer layers to pavement sublayers. Oxygen diffusion depth is considerable (Jin, Cui, Glover, 2013). Also, asphalt film thickness affects oxygen diffusion. Sufficient methodology has been developed to estimate the diffusion depth (Rose, 2014). Basically, using RAP in new mixes is a good idea for sustainable development, but information is still needed to understand the interaction between different components in a mix with RAP to optimize the properties of those recycled mixes. In order to have more RAP in HMA, as a greener alternative, blending degree and aging rate need to be well understood. The objective of the present research is to study bitumen characteristics of fine and coarse RAP particles from the same RAP source and to verify the interaction between RAP bitumen and virgin bitumen according to RAP particle size.

4.3 Materials and methods

In order to reach the goal of this study, a single RAP source was selected and split in two sizes, fine RAP (FR) and coarse RAP (CR), before extracting the bitumen (with solvent). The two RAPs were then used to make mixes with virgin aggregate and virgin binder before extracting the mixed bitumen. The following steps were performed:

- Performance grading (PG) for CR and FR bitumen;
- Performance grading (PG) for binder blended with CR and FR (coarse RAP mix (CRM) and fine RAP mix (FRM));
- Rheology behavior for CR, FR, CRM, FRM, and control mix;
- A Fourier transform infrared spectroscopy (FTIR)–ATR (attenuated total reflection) analysis was done on all bitumen and bitumen mixes;
- Environmental scanning electron microscopy (ESEM) image analysis was used to visualize bitumen differences on a micro scale.

In this study, a base course 20 mm mix was made with 4.5% total bitumen. The virgin bitumen used was a PG 64-28 from Bitumar (Montreal, Canada). Also, RAP was supplied by Bauval (Montreal, Canada).

To have a clear and better understanding of the differences between fine and coarse RAP, a single source of RAP from a specific area was chosen. In this way, the potential errors according to the RAP source were reduced. Two mixes were designed and tested. These mixes contained 35% fine RAP or 54% coarse RAP. Those mixes were referred to as fine RAP (FR) mix and coarse RAP (CR) mix in this paper. Initially, it was assumed that all RAP bitumen participated in the mix. The bitumen content of fine RAP was measured with the ignition oven at 6.67%, and it was 4.33% for the coarse RAP (Figure 4.1). The RAP content for each type of RAP was selected to achieve the same bitumen content of 4.5% for all mixes.

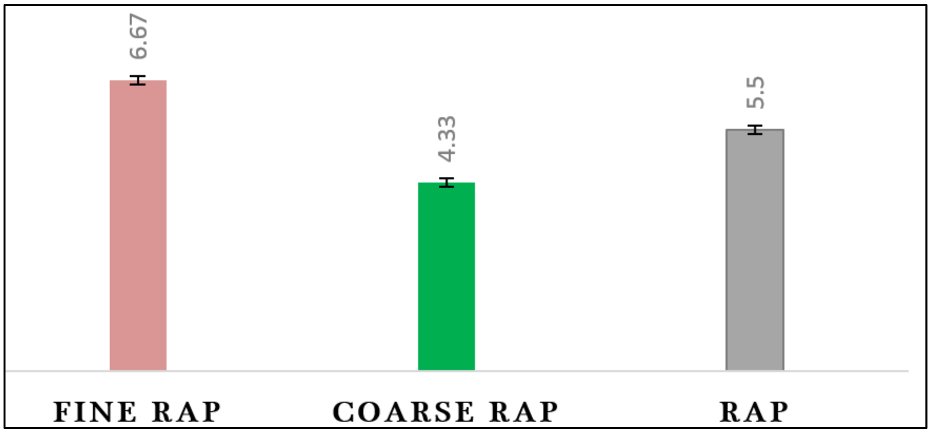

Figure 4.1 Bitumen content of fine reclaimed asphalt pavement (FR), coarse reclaimed asphalt pavement (CR), and reclaimed asphalt pavement (RAP) from the ignition test

Once the mixes were done, the bitumen was extracted and recuperated according to ASTM D2172 standards. Bitumen was also extracted from the two RAP sources. The five resulting bitumen were:

- Virgin bitumen PG64-28;
- Recovered bitumen from coarse RAP particles (CR);
- Recovered bitumen from fine RAP particles (FR);
- Recovered bitumen from a mixture of virgin bitumen with CR (CRM);
- Recovered bitumen from a mixture of virgin bitumen with FR (FRM).

The rheology of the bitumen was tested in terms of dynamic shear modulus, and their chemical characteristics were tested with infrared spectrometry. The difference between recovered coarse RAP bitumen and recovered fine RAP bitumen was also visualized by electron microscopy.

4.4 Bitumen Characterization

Extracted bitumen were classified by their performance grade (PG). Bitumen classification in North America is based on performance-grade (PG H–L), which is selected according to the temperature. The H value corresponds to the maximum pavement temperature, at which the bitumen has enough cohesion to minimize plastic deformation of the asphalt mix, and the L value corresponds to the minimum air temperature, where the bitumen has enough elasticity to keep asphalt mix flexible at low temperatures to reduce cracking potential (American Association of State Highway and Transportation 1994).

4.5 Rheological behavior

A dynamic shear rheometer (DSR) was used to determine the complex shear modulus (G*). All five bitumen were tested. AASHTO-T-315 (American Association of State Highway and Transportation, 2012) was adopted for the analysis. This test method covered the determination of the dynamic shear modulus and phase angle of asphalt binder when tested in dynamic (oscillatory) shear using parallel plate test geometry. It was applicable to asphalt binders that have dynamic shear modulus values in the range from 100 Pa to 10 MPa. This range of modulus was typically obtained between 6°C and 88 °C at an angular frequency of 10 rad/s. This test method was intended for determining the linear viscoelastic properties of asphalt binders, as required for specification testing, and was not intended as a comprehensive procedure for full characterization of the viscoelastic properties of asphalt binder.

The shear modulus responses (G*) of the bitumen measured in the laboratory were first modelled with the 2S2P1D model, a generalization of the Huet–Sayegh model (Olard & Di Benedetto, 2003). Referring to the 2S2P1D model, the values of the DBN (Di Benedetto-Neifar) bodies were fixed in the Linier Visco-Elastic (LVE) domain. An alternative general model “2S2P1D” (generalization of the Huet–Sayegh model), valid for both the bituminous binders and mixes and based on a simple combination of physical elements (spring, dashpot, and parabolic elements), was proposed.

The introduced 2S2P1D model had seven constants (Figure 4.2). At a given temperature T, the seven constants of the 2S2P1D model can be determined. G* is calculated following Olard and Di Benedetto (Olard & Di Benedetto, 2003) :

$$E^*_{(\mathrm{i}\omega\tau)} = E_{00} + (\mathrm{E}_0 - \mathrm{E}_{00})/(1 + \delta\,(\mathrm{i}\omega\tau)^{(-\mathrm{k})} + (\mathrm{i}\omega\tau)^{(-\mathrm{h})} - (\mathrm{i}\omega\beta\tau)^{(-1)}) \quad (4.1)$$

where:

- h and k are exponents such as $0 < k < h < 1$;
- E00 is the static modulus obtained when $\omega\tau \rightarrow 0$ (at low frequencies and high temperatures) with $\omega = 2\pi$ frequency;
- E0 is the glassy modulus when $\omega\tau \rightarrow \infty$ (at high frequencies and low temperatures), MPa;
- τ is the characteristic time, which this value varies only with temperature, accounting for the time–temperature superposition principle (TTSP);
- η is the Newtonian viscosity, $\eta = (E_0 - E_{00})\, \beta\tau$;
- When $\omega\tau \rightarrow 0$, the $E^*(i\omega t) \rightarrow E00 + i\omega\, (E0 - E_{00})\, \beta\tau \times \beta$.

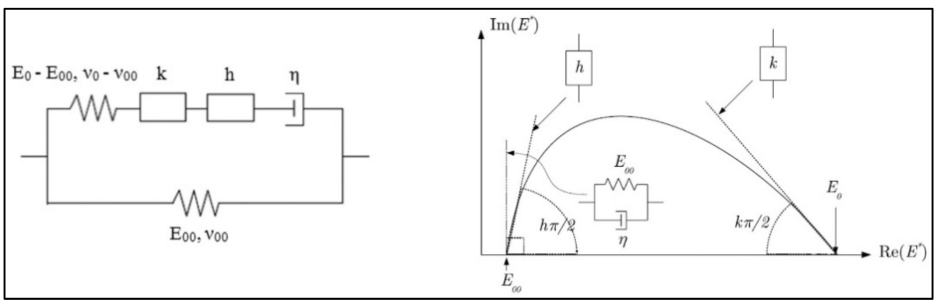

Figure 4.2 2S2P1D schematic model

4.6 Fourier Transform Infrared Spectroscopy–Attenuated Total Reflection (FTIR–ATR) Spectrometry

The FTIR–ATR analysis was performed with a Bruker Tensor 27 and a diamond ATR crystal. The analysis was performed 16 times per specimen with a 4 cm^{-1} resolution and a band range of 4000–600 cm^{-1}. Around 0.5 g of bitumen was placed with a spatula (at ambient temperature) on the crystal. The crystal was cleaned with a bitumen remover after each test followed by ethanol to remove traces of the bitumen remover.

Oxidation of hydrocarbons was associated, notably, with the increase of carbonyl C=O (around 1700 cm^{-1}) and sulfoxide S=O (around 1030 cm^{-1}) bonds. The carbonyl index (%) and sulfoxide index (%) are defined in Table 4.1 for C=O and S=O, respectively, with a higher value indicating relatively more oxidation. The peaks around 1460 cm^{-1} and 1376 cm^{-1} were for aliphatic C-CH3 groups, which served as baselines for the analysis, as they change relatively little during aging (Mikhailenko, Bertron, Ringot, 2016).

Table 4.1 Fourier transform infrared (FTIR) bands with bitumen aging
Taken from Mikhailenko et al. (2016)

Chemical Group	**Bond**	**Approximate Wavenumber (cm^{-1})**	**Change with aging**	**Intensity**	**Expression**
Sulfoxide	S=O	1030	Increases in short-term	Weak	$\frac{A_{1030}}{A_{1460}+A_{1376}}$
Carbonyl	C=O	1700	Increases in long-term	Weak to medium	$\frac{A_{1700}}{A_{1460}+A_{1376}}$
Aliphatics (Asymmetric)	C-CH_3	1460, 1376	Relatively constant, small decrease	Medium	$\frac{A_{1700}}{A_{1460}+A_{1376}}$

4.7 Environmental Scanning Electron Microscopy (ESEM) Analysis

Environmental scanning electron microscopy (ESEM) observations were conducted in accordance with the settings developed previously for bitumen by Mikhailenko et al. (2017). Bitumen specimens were observed at room temperature immediately after being removed from the cooler with a FEI Quanta 250 FEG ESEM.

4.8 Results and discussion

4.8.1 Bitumen performance grade (PG)

Rheological characterizations of all extracted and recovered bitumen from FR, CR, FRM, CRM, and control mix were performed following AASHTO T 315. Figure 4.3 presents the rheological properties of each asphalt bitumen using a dynamic shear rheometer (DSR).

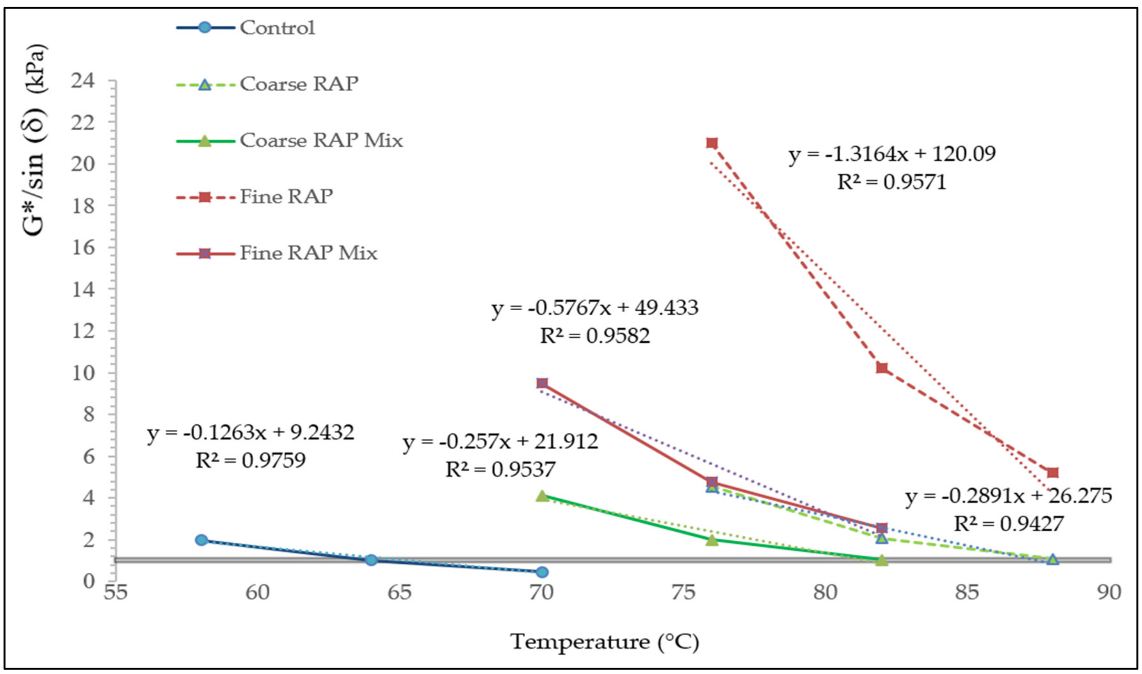

Figure 4.3 Performance grade of recovered bitumen by dynamic shear rheometer (DSR) at 1.6 Hz

It was important to note that the recovered bitumen from the fine RAP was much stiffer than the recovered bitumen from the coarse RAP, even for the same RAP source that was simply split into two different groups. For example, at 76 °C and 1.6 Hz, the fine RAP bitumen was 4.6 times stiffer than the coarse RAP bitumen. Basically, the slope of each trend showed the sensitivity of that bitumen to temperature change. The control mix had the lowest sensitivity to temperature changes. Recovered bitumen from CR and CRM showed almost the same sensitivities, but recovered bitumen from FRM had less sensitivity than FR. Both CRM and FRM had the same virgin bitumen quantity (2.2%) and RAP bitumen content (2.3%). It was seen that FRM was stiffer than CRM at 76 °C (4.8 and 2 kPa, respectively), but the CRM bitumen was less sensitive than FRM to temperature change (0.25 and 0.57, respectively). In terms of the coarse and fine RAP before and after mixing with virgin bitumen, there was a huge difference from FR to FRM. This may be due to the amount of active bitumen in FR. As it was shown in a previous work (Saliani et al., 2016), more RAP bitumen was transferred from coarse RAP than fine RAP particles because fine RAP particles were covered by a clump of mastic that did not tend to participate as active bitumen in the mixture. Despite the fact that fine RAP particles had higher aged bitumen content, participation by volume was less for the bitumen surrounding the coarse RAP particles in HMA.

One method that has been extensively used to evaluate the PG of virgin bitumen, in adding to HMA containing a high amount of RAP, is the blending chart (McDaniel & Soleymani, 2000). In blending, it is assumed that there is a linear relationship between the amount of RAP bitumen and the rigidity (or DSR results) at a given temperature. Figure 4.4 shows a blending chart for the tested mix. To get results for all mixes at the same temperature, linear best fit curves were drawn from the results shown in Figure 4.3 (all with R2 > 0.95), and results were calculated for missing temperatures. At 70 °C, G*/sin(d) value of control mix (0.4 KPa) was referred to as 0% coarse rap content, and G*/sin(d) value of CR (6.038 KPa) was referred to as 100% coarse rap content. With these two values, the linear relationship between coarse RAP bitumen content and rigidity was plotted in Figure 4.4. The linear best fit curve for fine rap content was drawn at 70°C and 1.6 Hz as well as in Figure 4.4.

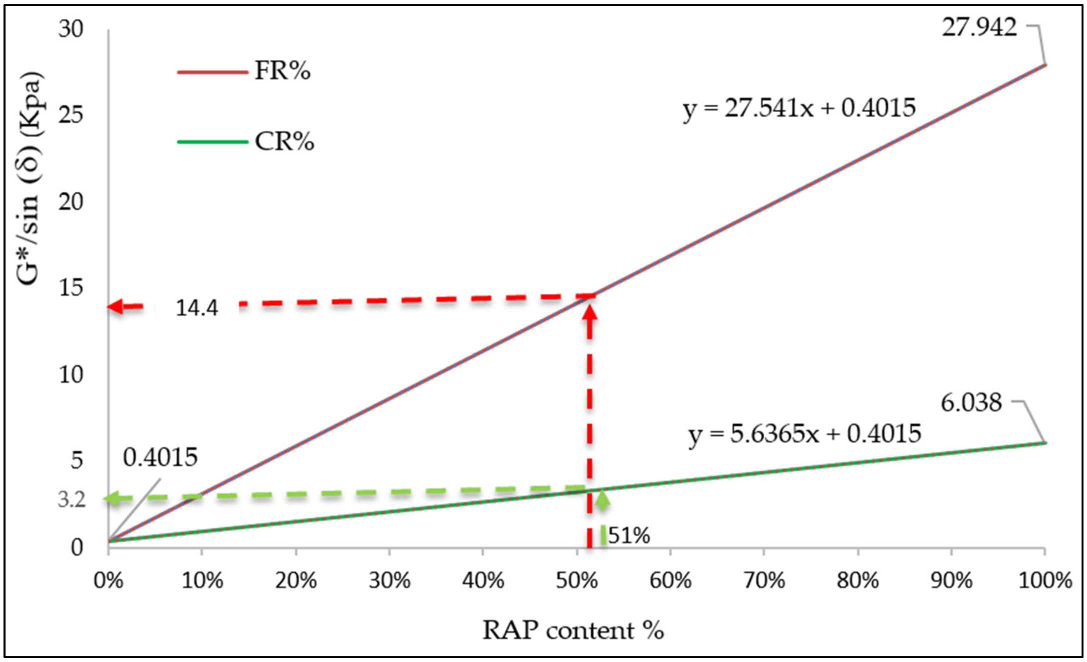

Figure 4.4 Blending charts for the fine and coarse RAP mixes at 70°C and 1.6 Hz

As shown in Figure 4.4, if we compare mixes with 51% FR or CR RAP, we have G*/sin (d) values of 14.4 and 3.2 KPa respectively. It showed that a mix with CR binder had a huge difference in shear modulus from the same mix with FR. It might be because the bitumen aging rate around finer particles is faster than the binder surrounding coarse particles, but more studies are needed to precisely verify this hypothesis. It may also be because fine particles have more surface area than coarse particles. Clearly, 51% CR had the same impact as 11% FR in mixes. Thus, it was important to clarify properly characterized RAP bitumen for a given particle size before mixing with virgin materials.

4.8.2 Shear modulus (G*)

Figure 4.5 shows the 2S2P1D Cole–Cole plot of the shear complex modulus at different temperatures and frequencies for all recovered bitumen from CR, CRM, FR, FRM, and control mix. 2S2P1D was adopted to model the shear modulus of recovered bitumen. From Figure 4.5, it seemed that the virgin bitumen (PG64-28) was more viscous than the other bitumen.

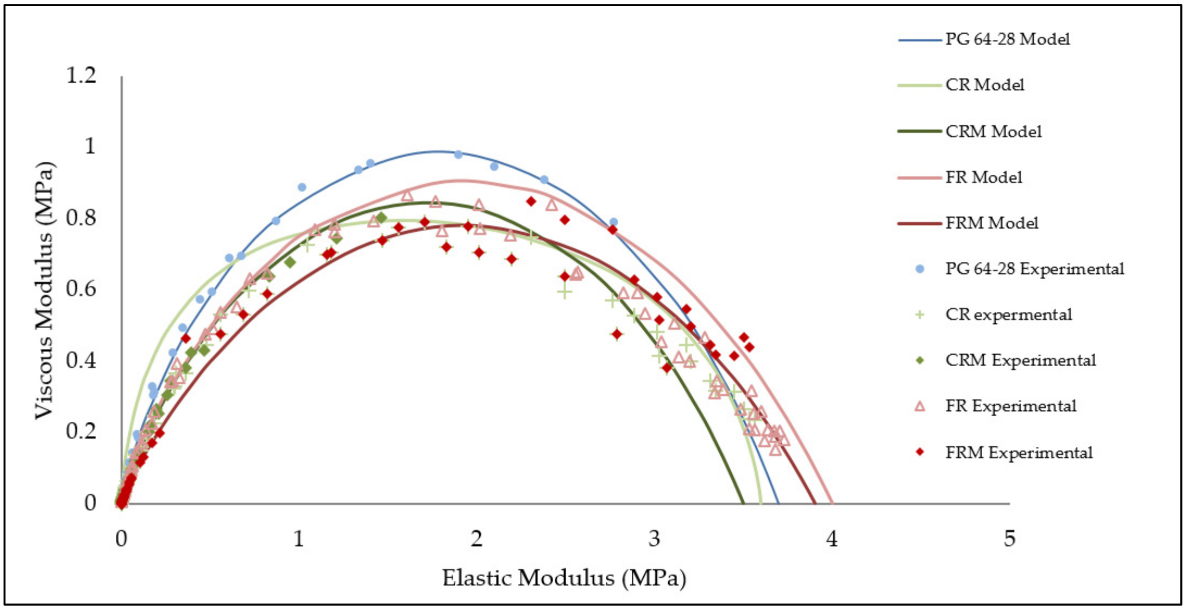

Figure 4.5 Cole–Cole shear modulus of recovered bitumen (2S2P1D model)

Table 4.2 shows the seven constant values for all four mixes. As mentioned earlier, these seven constants represented the bitumen characteristics. In general, the same binder was supposed to have equal values for the constants. Table 4.2 shows that the constants were not the same, so it could be concluded that recovered binder from FR was not the same as CR. Virgin bitumen was expected to have higher viscous components. Aged binder lost some parts of its viscous components and became brittle, so it mixed with aged bitumen and had a lower viscosity than virgin bitumen.

Table 4.2 the 2S2P1D model constants for all recovered binder

Mix	G_0 (Pa)	G_{00} (Pa)	k	h	δ	τ_E (s)	β
virgin	2	3,700,000	0.59	0.99	3.90	0.00015	5000
CR	100	5,000,000	0.38	0.86	4.90	0.00080	5000
FR	500	4,000,000	0.51	0.89	3.90	0.06000	5000
CRM	2	3,500,000	0.56	0.98	7.00	0.00250	5000
FRM	10	3,900,000	0.48	0.90	10.00	0.00700	5000

Figure 4.6 shows the master curve of shear modulus (Pa). The G* values of FR and CR were higher than the other mixes. Asphalt production, lifetime, and presence of air impacted the aging progress. A single source of RAP was used for CR and FR. Then, since all these factors were the same for CR and FR, the recovered bitumen was supposed to have the same master curve. Figure 4.6 shows that, except for the mentioned factors, particle size may change the rheology of bitumen. This was because FR and CR mixes had higher aged bitumen content than CRM and FRM at intermediate and lower frequencies. FR was stiffer than CR, which meant that the finer RAP particles were covered by a stiffer bitumen. This could be explained as bitumen that surrounded fine particles aged faster than when they surrounded coarse particles; a higher surface area of the bitumen was exposed to oxidative aging. After mixing the coarse and fine RAP with virgin materials, master curves from recovered bitumen were plotted. Here, we again saw that FRM was stiffer than CRM at intermediate and lower frequencies.

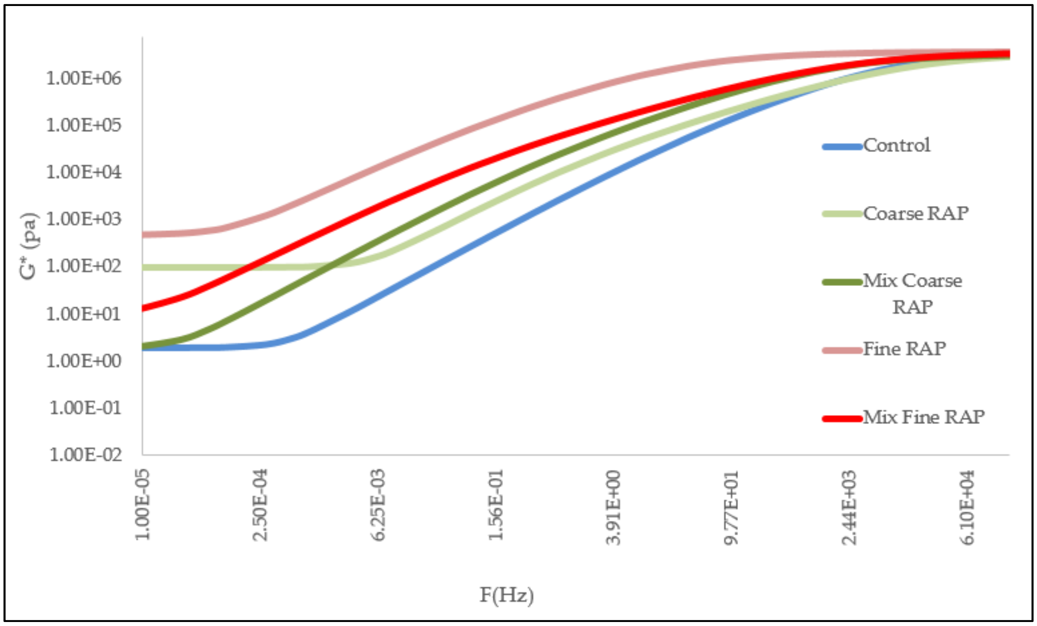

Figure 4.6 G* master curve at 10 °C of recovered binder (2S2P1D model)

The Reduced G* (RG*) index was looked at to clarify the interaction of RAP bitumen with virgin bitumen according to RAP particles. G* was determined for CR and FR as well as for CRM and FRM. According to CRM (FRM) mix design, CR (or FR) was added to the 2.2% virgin bitumen. The difference between CR (or FR) and CRM (FRM) showed the impact of virgin bitumen on total bitumen stiffness. (RG*) is calculated by Equation (3) as a percentage:

$$RG^* = ((\mathrm{G}_i{}^* - \mathrm{G}_{iM}{}^*))/(\mathrm{G}_i{}^*) \tag{4.2}$$

where RG* is the percentage of reduced G*, G^*_i is the shear modulus of pure RAP, and G^*_{iM} is the shear moduli of RAP and virgin bitumen mixes. The difference of G^*_i and G^*_{iM} showed how well the bitumen mixed together. As the aged bitumen was stiffer than virgin bitumen, eventually G^*_i was supposed to be higher than G^*_{iM}. Figure 4.7 indicates the rheology transition before and after mixing RAP with virgin materials. Both CR and FR mixed with 2.2% virgin binder. Most of the influence of aged binder was observed at low frequency. The difference between aged bitumen and mixture bitumen changed from 0% to 90%. These differences meant rigidity was changed. FRM was 90% softer than FR; it increased rapidly and was constant over a wide frequency range. The impact of CR clearly differed from FR. The different trends in CR and FR could translate to the impact of each type of RAP on virgin bitumen. CR and CRM had the same rigidities at high frequency and gradually increased at low frequency. CR and FR bitumen recovered from a single source of recycled asphalt were supposed to have the same characteristics when blended with a specific amount of virgin bitumen, but the interactions were different. Aging rates were not the same between fine gradation and coarse gradation.

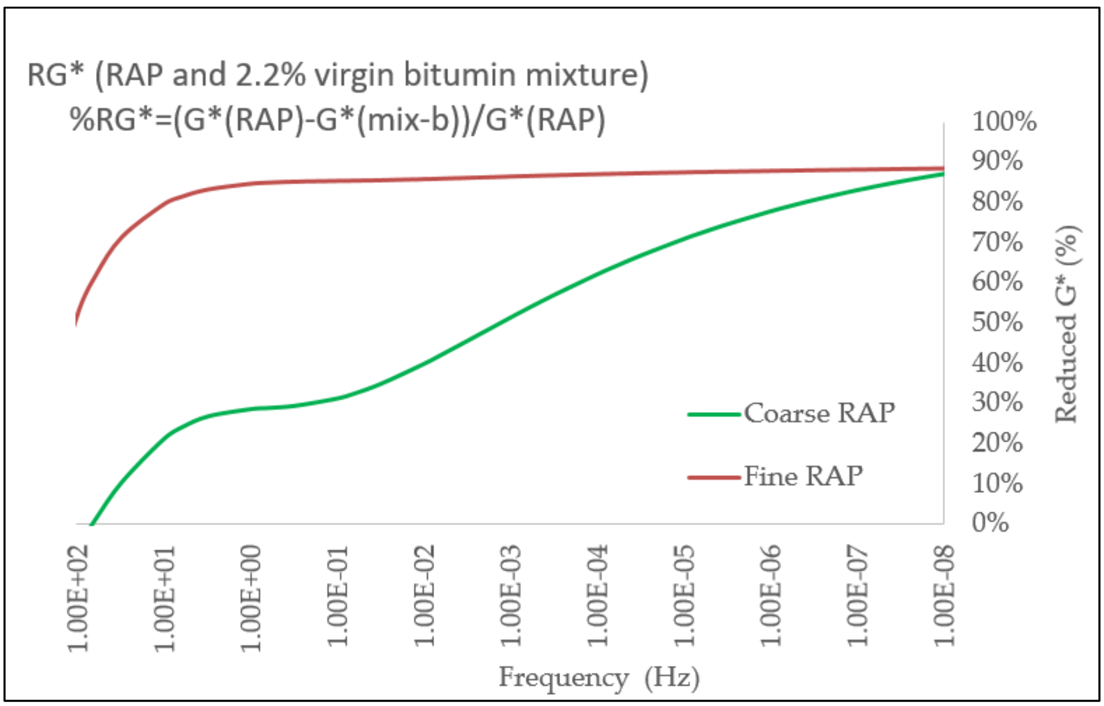

Figure 4.7 Reduced G* for CR and FR at 10 °C

4.8.3 FTIR–ATR Spectrometry

The results for the FTIR–ATR analysis is presented in Figure 4.8. Asphalt aging was characterized by oxidization indices shown in Table 4.1. The C=O and S=O indices both increased significantly with RAP and blended bitumen relative to the virgin bitumen. There was a small increase in the RAP bitumen compared to the blended bitumen in the indices. The aging indices were higher for the fine RAP, likely because of its higher surface area, compared to the coarse RAP, which allowed for a greater degree of aging. The sulfoxide indices for all bitumen were higher, as was typical for asphalt bitumen (Hofko, 2018).

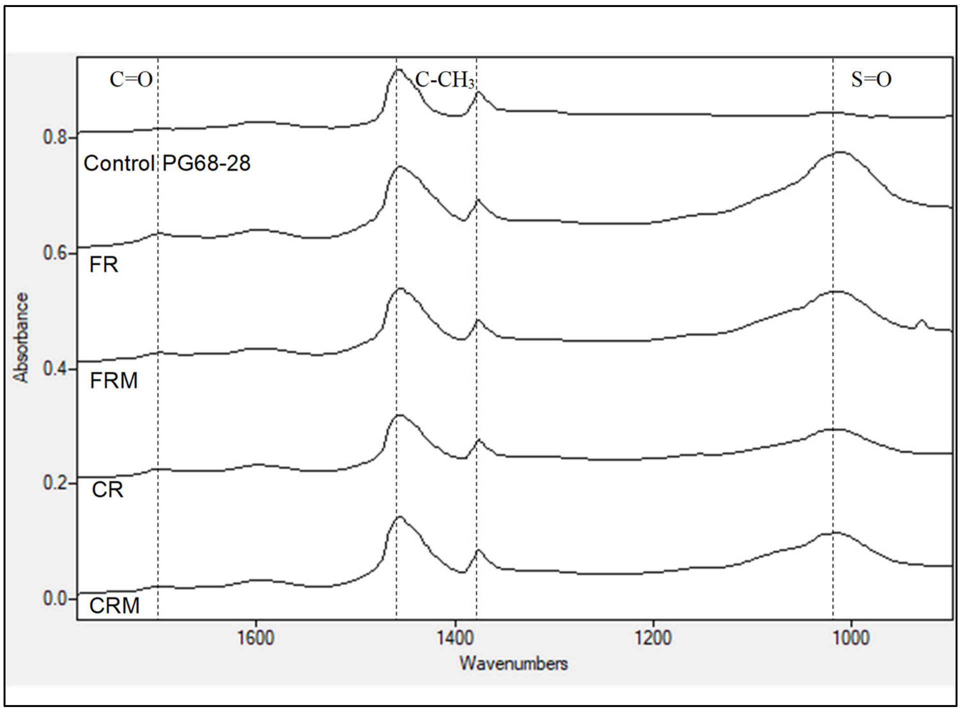

Figure 4.8 Compilation of FTIR spectra featuring C=O and S=O oxidation bands

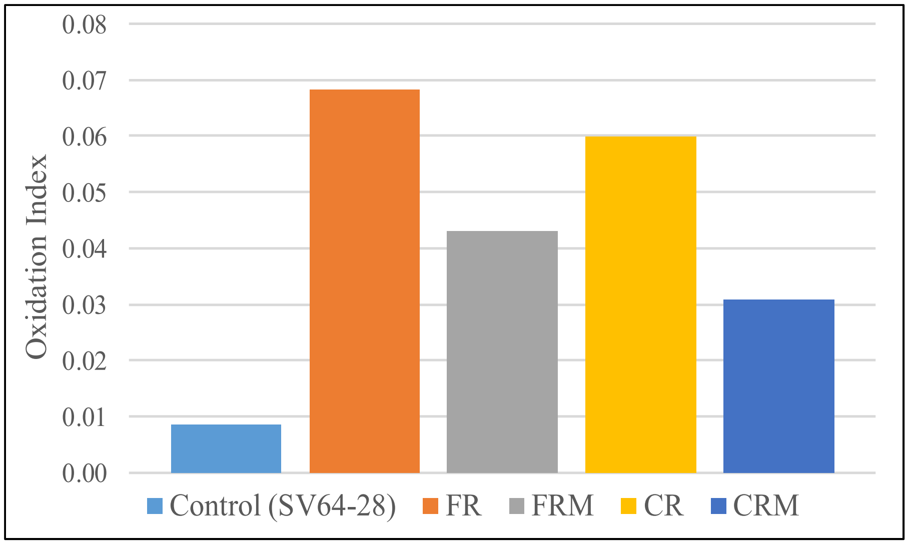

Figure 4.9 FTIR carbonyl oxidation indices

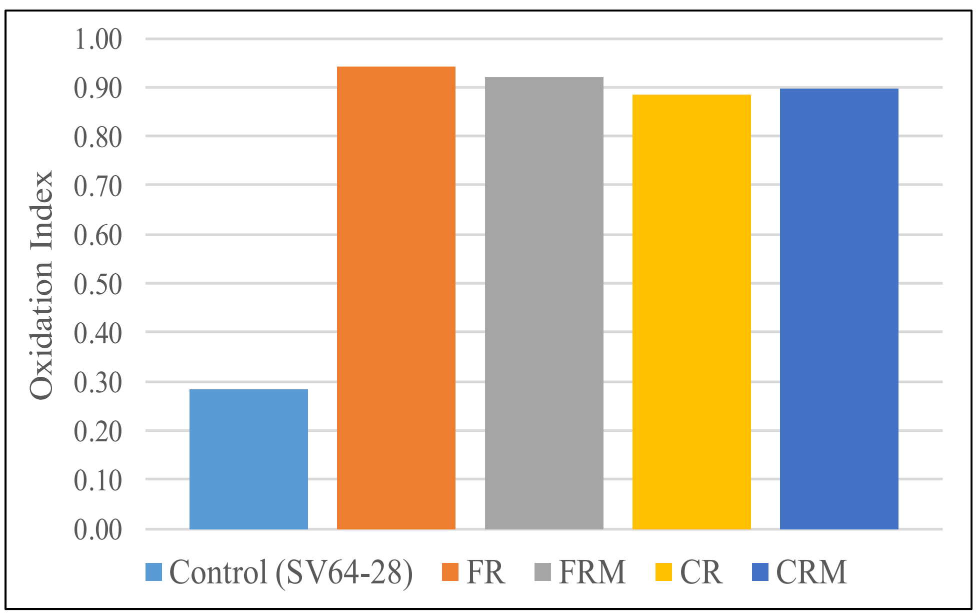

Figure 4.10 FTIR sulfoxide oxidation indices

4.8.4 Environmental Scanning Electron Microscopy (ESEM) Analysis

The images from the ESEM analysis are shown in Figure 4.11, before electron beam irradiation and after stabilization of the sample. The bitumen before irradiation showed a ‘bee’ type structure, similar to the one found in AFM observations (Das, Baaj, Tighe, Kringos, 2016; Oenen, et al., 2013), which was denser in the RAP bitumen compared to the virgin and blended bitumen.

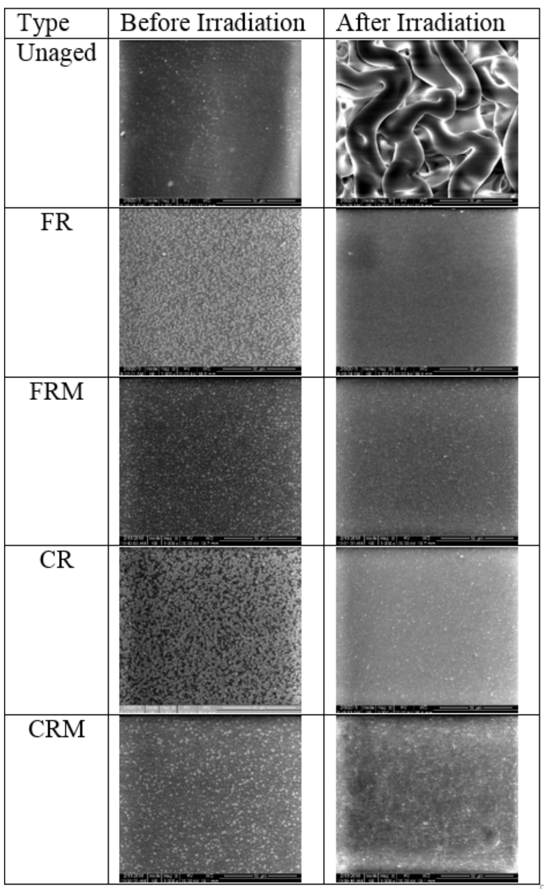

Figure 4.11 Environmental scanning electron microscopy (ESEM) Images of Samples at 1000× before Electron Beam Irradiation and After Image Stabilized

After the sample was exposed to the electron beam, and the accumulation of energy irradiated the sample, the asphalt bitumen tended to reveal a 'fibril' microstructure as lighter components of the bitumen dispersed (Mikhailenko, 2019), which was the case for the virgin bitumen. However, both the blended and RAP bitumen responded much less to beam irradiation. This was likely because the microstructure of the bitumen stiffened from aging, and the molecular mobility was reduced. This made revealing the microstructure by ESEM irradiation a slower and more difficult process with the settings that were used.

4.9 Conclusions

It is common to use green alternatives in asphalt mix designs such as recycled asphalt, crumb rubber, etc. Several studies showed the advantages of RAP in hot mix asphalt. Still, there is no clear vision of the impact RAP size has on HMA behavior. In this study, a single Quebec source of RAP was separated into coarse and fine particles and mixed in with bitumen pavement mixture. Properties of the asphalt mixes were evaluated by the complex modulus test and DSR. Also, the ignition test (ASTM D6307) was used to quantify bitumen content in the RAP, FTIR spectrometry was used for chemical properties of the bitumen, while ESEM image analysis was used to visualize the differences of the virgin and RAP bitumen at a microscopic level.

FRM was designed with 35% fine RAP particles, including 2.3% aged bitumen and 2.2% virgin bitumen. CRM was designed with 54% coarse RAP particles, including 2.3% aged bitumen and 2.2% virgin bitumen. Recovered bitumen (4.3%) was measured for CR particles, and bitumen content was 6.6% for FR particles. Designed mixes were compared with the control mix, and the following results were achieved:

- FR-recovered bitumen is more sensitive to temperature changes than virgin and CR-recovered bitumen;

- FR-recovered bitumen is stiffer than CR-recovered bitumen. Also, FRM-recovered bitumen is stiffer than CRM-recovered bitumen. This shows that the bitumen grade of coarse particles is not the same as fine particles in RAP;
- Reduced G* (RG*) was looked at to clarify the interaction of RAP bitumen with virgin bitumen according to RAP particles. The CR reduced factor was higher than FR; CR bitumen and CRM bitumen were more similar than FR and FRM. This factor might be applied in mixing designs to predict the stiffness of HMA with the inclusion of RAP, which is the case for future studies;
- Aging rates in fine particles are faster than coarse particles because fine RAP has a higher surface area;
- Results for the FTIR–ATR analysis showed aging indices are higher for the fine RAP, likely because of its higher surface area compared to the coarse, which allowed for a greater degree of aging;
- Images from the ESEM analysis shows both the blended and RAP bitumen respond much less to beam irradiation. This is likely because the bitumen microstructure stiffened from aging.

More work is needed to better understand the impact of RAP gradation. The results shown here definitely give new information about different RAP bitumen contained in different sizes, but new tests with several other RAP size are needed to confirm the results presented here.

Author Contributions: S.S.S. conceived of the presented idea. A.C. and H.B. developed the theory and verified the analytical methods and supervised the project. S.S.S. wrote the original draft; and P.M. wrote, reviewed, and edited the manuscript. All authors discussed the results and contributed to the final manuscript.

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Conflicts of Interest: The authors declare no conflict of interest.

CHAPTER 5

CHARACTERIZATION OF ASPHALT MIXTURES PRODUCED WITH COARSE AND FINE RECYCLED ASPHALT PARTICLES

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5.1 Abstract

Utilizing recycled asphalt pavements (RAP) in pavement construction is known as a sustainable approach with significant economic and environmental benefits. While studying the effect of high RAP contents on the performance of hot mix asphalt (HMA) mixes has been the focus of several research projects, limited work has been done on studying the effect of RAP fraction and particle size on the overall performance of high RAP mixes produced solely with either coarse or fine RAP particles. To this end, three mixes including a conventional control mix with no RAP, a fine RAP mix (FRM) made with 35% percent fine RAP, and a coarse RAP mix (CRM) prepared with 54% of coarse RAP were designed and investigated in this study. These mixes were evaluated with respect to their rutting resistance, fatigue cracking resistance, and low temperature cracking performance. The results indicate that although the CRM had a higher RAP content, it exhibited better or at least the same performance than the FRM. The thermal stress restrained specimen testing (TSRST) results showed that the control mix performed slightly better than the CRM, while the FRM performance was adversely affected with respect to the transition temperature midpoint and the maximum tensile stress temperature. Both of the RAP incorporated mixes exhibited better rutting resistance than the control mix. With regard to fatigue cracking, the CRM performed better than the FRM. It can

be concluded that the RAP particle size has a considerable effect on its contribution to the total binder content, the aggregate skeleton of the mix, and ultimately the performance of the mix. In spite of the higher RAP content in the CRM versus FRM, the satisfactory performance observed for the CRM mix indicates a great potential in producing high RAP content mixes through optimizing the RAP particle size and content. The results also suggest that the black curve gradation assumption is not representative of the actual RAP particles contribution in a high RAP mix.

Keywords: hot mix asphalt; recycled asphalt; RAP gradation; complex modulus; fatigue cracking; permanent deformation; thermal cracking resistance.

5.2 Introduction

Utilizing reclaimed asphalt pavement (RAP) in hot mix asphalt (HMA) is proven to be a green alternative to produce environment-friendly asphalt mixes. Adding RAP in asphalt mixes is suggested not only to conserve the aggregates and bitumen, but also to have at least the same performance (Kandhal, Mallick,1998). Recycling of the existing mineral aggregates and asphalt binder in RAP particles would be of great benefit to the environment by saving the non-renewable materials. Milled pavements are considered to be valuable materials after reaching the end of pavement service life. At a minimum level, RAP can play the role of virgin mineral aggregates in order to conserve the energy and save the environment. However, the ideal goal is to maximize reusing the waste materials in new pavement construction projects in a way that the same or even better performance as compared to the conventional materials can be achieved.

5.2.1 High rap mixes

Although there is no recognized unanimity about the limit of the maximum amount of RAP in HMA, RAP percentage in HMA has been limited by many agencies, mainly, due to the unproven performance of high RAP mixes and also lack of a unified mix design (Zaumanis, Mallick, Frank, 2016). The review of the previous research on RAP indicates use of up to 100% RAP in HMA mixes. However, most of the plant-produced 100% RAP hot mix asphalt projects date back to 1991 and earlier (Federal Highway Administration , 1995; Bloomquist, Diamond, Oden, Ruth, Tia, 1993; Little, Epps, 1980). In 1997, the Federal Highway Administration's RAP expert task group developed guidelines for the design of Superpave HMA containing RAP (Bukowski , 1997). In the same year, another study by Kandhal and Foo (Kandhal, Foo, 1997) recommended a three-tier process to deal with RAP in asphalt concrete, where a RAP content of 25% and more was defined as high RAP mix, requiring detailed evaluations (Kandhal, Foo, 1997). RAP limitation was also supported by the findings of the NCHRP research report 9–12 (McDaniel, Soleymani, Anderson, Turner, Peterson, 2000). In spite of several research projects conducted on RAP incorporated mixes, still there is not a clear vision about the interaction of RAP and virgin materials in details. Different scenarios can be considered about the interaction of virgin and aged binder: (1) there is no interaction between old and virgin materials, so RAP could be assumed as a black rock. In other words, the aged binder in RAP does not significantly contribute the total binder content. As the rheology of RAP may be affected by facing preheated aggregate and hot virgin binder, this assumption would most probably be different from what happens in reality. (2) All of the aged binder in RAP blends into the mix and with virgin materials effectively. Again, it is not clear whether this assumption is close to reality or not. Therefore, further research is needed to figure out the rate of interaction between the used and new materials and the significant parameters affecting this phenomenon. Previous study showed that depending on the RAP size and aggregate gradation, the available binder content in RAP would vary (Saliani, Carter, Baaj , 2016). They have also concluded that there is a significant difference between large and small particles with respect to transition of the asphalt binder from RAP to virgin aggregates.

There was no guidance until early 1990's for implementation of RAP in HMA, but based on experimental studies, FHWA Asphalt Mixture Expert Task Group defined the interim recommendations (Bukowski, , 1997). Based on the performance of Marshal Mixes with RAP, and mixes designed according to the Superpave system, AASHTO Standards MP2 (now M323) describes how to design HMA with RAP (Basueny, Perraton, Carter, 2013).

Recycled asphalt mixes consist of complex bituminous material. Further, sometimes unknown milling processes make it difficult to study RAP with predictable properties. Therefore, many issues arise due to RAP variability when high percentages of RAP are used in a mix. One solution for this issue might be using RAP in different layers of pavement structure. An example of such application is discussed by Pratico et al. who describe the feasibility of building a two-layer porous asphalt (TLPA) by recycling from permeable European mixes (PEM) RAP, when highly variable RAP stockpiles are involved (Praticò, Vaiana, Giunta, Iuele, Moro , 2013).

5.2.2 RAP binder characteristics

RAP mainly consists of aggregates and aged binder. When incorporating RAP in a new HMA, the aged binder can affect the mix behavior in long-term because of the diffusion of RAP and virgin binder. Generally, adding high RAP contents into new mixes can increase the stiffness significantly (Tavassoti-Kheiry, Solaimanian, Qiu, 2016). Asphalt as a petroleum-based product is an organic material which can be subjected to short term chemical oxidation due to the combined effect of heat and atmospheric oxygen during the mixing and hauling process. Characterization of oxidation is of utmost importance because this phenomenon makes the material brittle (Petersen, , 2009). This becomes even more crucial when the RAP content in a mix is more than 25%, which according to the conventional definitions mentioned earlier is known as high RAP mix. Therefore, depending on the RAP content, presence of aged binder can change the mixture performance. During the past few decades, several studies have focused on characterizing RAP and on finding the proper way of using it in producing asphalt mixtures. For example, Cosentino et al. (2003) concluded that the controlling factors in the performance of RAP are dependent on where the RAP is obtained and its gradation. However, there are still several aspects of using RAP in HMA that require further investigation such as the impact of RAP source, content, gradation, conditioning, etc.

According to McDaniel et al.(2000), less than 15% RAP has no impact on the blended binder performance in mix. Between 15% and 25% RAP, the virgin binder grade is commonly decreased by one grade (6 °C) on both ends (e.g., a PG 64-22 is changed to a PG 58-28). For more than 25% of RAP, binder needs to be graded using the performance-graded binder tests. Therefore, for the mixes with high RAP content, full characterization of binder is needed.

The aging level of RAP binder film thickness might be changed according to depth where the mix is placed at and presence of oxygen. Bressi , Dumont and Pittet (2015) proposed a methodology to detect the existence of a cluster phenomenon (Figure 5.1) and they also proposed a first approach to show a different aging level in the RAP binder film thickness (i.e., partial differential aging). Stephens et al. (2001) also investigated the asphalt films properties for the coarse aggregates. They concluded that it would be more prone to blending with virgin aggregates than asphalt film around fine aggregates. The aged binder recovered from the coarse and the fine particles was compared by conducting a series of Dynamic Shear Rheometer (DSR) tests. They concluded that there is no correlation between variation in the binder stiffness and the asphalt coating of coarse or fine aggregates. The main issue in this domain was referred to its exposure to heat and air during production, which is a random process and does not correlate with either the aggregate size or related film thickness.

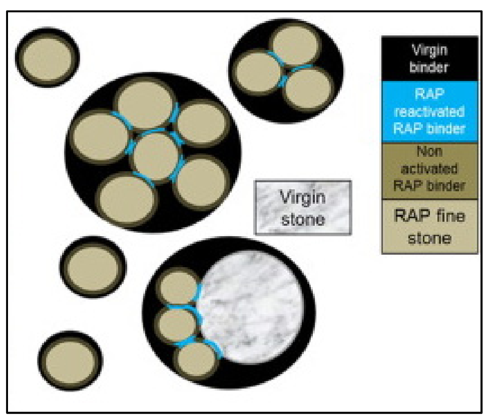

Figure 5.1 Schematic diagram of cluster phenomenon
Taken from Bressi, Dumont, Pittet (2015)

5.2.3 Effect of RAP particles size on binder blending phenomenon

In addition to the binder rheology in RAP particles several other parameters can affect the final performance of a RAP produced mix. Saliani et al, (2016) showed active binder in coarse RAP is significantly higher than the fine RAP. In their work, Saliani et al. (2016) mainly focused on virgin aggregate surface area and correlation with cutting or melting the aged binder from RAP particles. Saliani et al. (2016) concluded that virgin aggregate surface area is another factor that can have an impact on the interaction of recycled and virgin material. In addition to aggregate surface area, film thickness in RAP particles are not the same suggesting that more investigations are needed to characterize the film thickness properties properly.

Gardiner (Stroup-Gardiner, Wagner , 1999) concluded that the complex modulus is not solely controlled by the stiffness of the binder, whereas several other factors including the gradation and angularity of the aggregate have impact on it. Mixing method, heating temperature, and mixing duration of RAP need to be optimized to ensure the complete blending of the old and new binders so that the plant production process can be better simulated in the lab (Hassan, 2009). However, all of the aforementioned studies concluded that the stiffness of coarse and fine RAP would be the same, but their binder contribution to the new mix depends on some other factors, such as virgin aggregates surface area, mixing temperature, and RAP preheating temperature (if applicable).

Several researchers studied the blending of RAP and virgin binders. Chen et al. (2007) found that not only RAP does not act like a black rock but also a significant blending occurs between RAP and the virgin binder. However, Huang et al. (2005) studied the blending of RAP with virgin HMA mixtures for a given type of screened RAP. They concluded that aged binder in RAP formed a stiffer layer coating the RAP aggregate particles than the virgin binder (see Figure 5.2).

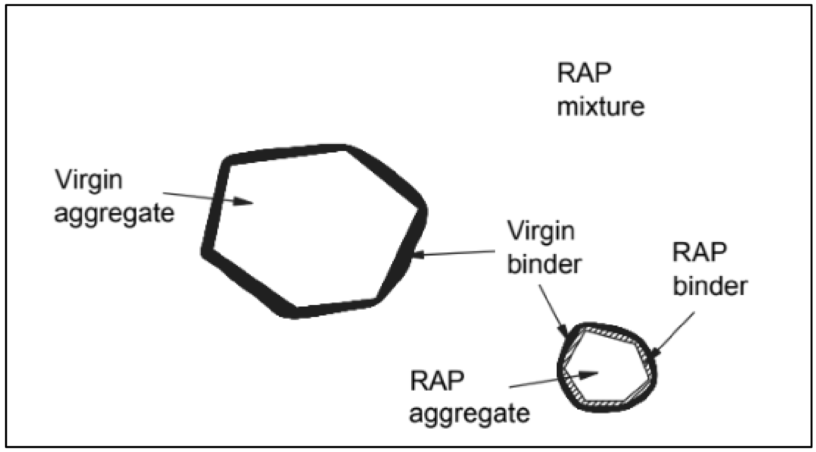

Figure 5.2 Composite-layered system in recycled asphalt pavement (RAP) Virgin
Taken from Huang et al. (2005)

Composite analyses indicated that the layered system in RAP (Figure 5.3) helps in reducing the stress concentration in HMA mixtures microstructure. The aged binder mastic layer was actually serving as a cushion layer in between the hard aggregate and the soft binder mastic (Huang et al., 2005).

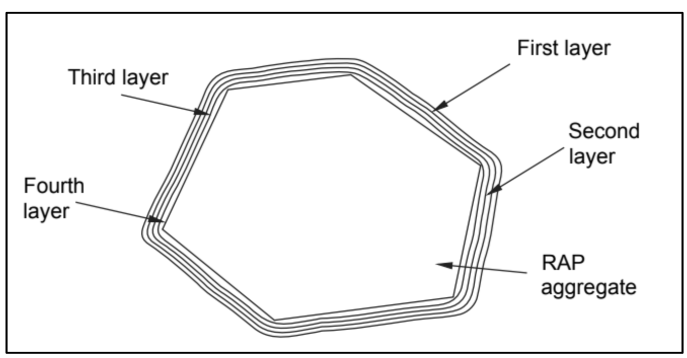

Figure 5.3 Layers of asphalt binder coating RAP aggregate
Taken from Huang et al. (2005)

As there is aged binder in RAP particles, bitumen additives might be applied to rehabilitate the aged binder which is not the case in this study. On the other hand, since there is high RAP content in HMA, rejuvenators are recommended (Yu, X.; Zaumanis, M.; Dos Santos, S.; Poulikakos, 2014; Moghaddam, T.B.; Baaj, H, 2016; Król, Kowalski, Niczke, Radziszewski, 2016). Additionally, green additives are recommended as cost effective and environmentally friendly alternatives. The green additive is obtained by a simple method from two low-cost and eco-friendly pre-cursors to restore the mechanical properties of the oxidized bitumen, acting on the structure of the bitumen, having a restructuring effect on the altered colloidal network of the aged bitumen binder (Caputo et al., 2019).

In addition to the binder characterization, mix performance needs to be evaluated when a higher RAP content is used. The indirect tension and semicircular bending test results which were conducted by Huang et al. showed that RAP increases the mixture stiffness (Huang, Zhang, Kingery, Zuo , 2004). All RAP materials in Huang et al. study was screened through the No. 4 sieve to acquire a consistent gradation which can compare to the fine aggregates group in this study. There is a possibility to increase the stiffness by adding fine RAP but it depends on job mix formula with inclusion of RAP. Huang et al. assumed that RAP binder totally contributes to the mix, an assumption which still needs to be verified.

Traditionally, black curves and white curves have been used for sieves analysis of RAP incorporated mixes. Black curves are the gradation of RAP particles from fractioned RAP and white curves are the gradation of recovered RAP aggregate after binder extraction. For a given mix, these two curves are significantly different. Al-Qadi et al. (2009) compared these two curves and concluded that; black curve tends to indicate higher amounts of large particles and lower amounts of fine particles. Therefore, to avoid the detrimental effects caused by unexpected extra fine particles, black curves are not suggested for use in job mix formula calculations. Using the white curve is common practice, however it is not the only approach being used. It should be noted that neither black curve nor white curve represents the actual gradation of the RAP material, and the real gradation lies somewhere in between (Roque, Yan, Cocconcelli, Lopp , 2015).

Previous research has shown that the bitumen recovered from coarse particles differ from fine particles (Saliani, Carter, Baaj, Mikhailenko, 2019). They concluded that RAP bitumen participation in hot mix process significantly depends on RAP size. The aging rate in fine particles is also faster than coarse particles. Therefore, it can be concluded that the RAP particle size would affect the properties of the RAP incorporated mix and can affect the overall performance of the pavements.

5.3 Research goals, scopes, and objectives

Review of the literature on RAP indicates that there is not a consensus on several aspects of the RAP binder contribution to the new mix. Therefore, it is difficult to come up with a synthesis of previous work that would be unanimously acceptable. In summary, it can be concluded that the coarse and the fine RAP fractions have relatively similar stiffness, but their contribution to the new mix is different. Therefore, the goal of this project is to understand the interaction of coarse RAP and fine RAP binder in HMA more precisely, through studying mixes prepared with either fine RAP fraction or coarse RAP fraction separately. To this end, various empirical and thermo mechanical tests are adopted to validate the impact of RAP fractions on the HMA mixes.

In this study, the RAP particles are separated in two groups by following the LC 21-040 protocol: particles passing sieve number 5 (5 mm), called fine RAP, and those retained on sieve number 5 are considered as the coarse RAP. The main objective of this research is to characterize RAP particles according to the particle sizes used to produce them. Generally, fine RAP is expected to possess a higher binder content, while there has not been any clear evidence to support this claim yet. The presence of such extra amount of binder (as compared to the coarse RAP) may potentially enhance the pavement resistance to cracking. Previous studies showed that in high RAP content mixes, the amount of binder (or mastic) that would diffuse into virgin binder from the fine RAP particles is less than that from the coarse RAP particles. Therefore, it was deemed necessary to further investigate the mix design and mix performance to characterize the fine and coarse RAP mixes more comprehensively. This research aims to characterize the mixes produced using the fine RAP and coarse RAP in terms of stiffness, fatigue cracking, permanent deformation, and thermal cracking resistance.

5.4 Materials and experimental methods

Generally, limits have been set on the maximum allowable amount of RAP in HMA mixes to avoid the loss of performance due to the impact of more aged binder content, which is present in RAP particles. However, many aspects of RAP incorporated mixes have been investigated by several past studies, the effect of increased RAP content using only coarse or fine RAP particles has not been fully explored. Therefore, in this study it was hypothesized that coarse RAP mix characteristics is different from fine RAP mix and such difference can affect the mix performance. The results might be used to identify the functional class for proper use of RAP mixes in pavement structural design in different layers. Three mixes were designed in this study, including a control mix, a fine RAP mix, and a coarse RAP mix. It was assumed that all of the RAP binder would contribute to the total binder content of the mix. Consequently, it was assumed that all three mixes have the same binder content. Furthermore, the black curve was used for the aggregate gradation in the mix design process.

Four performance tests were used to characterize the mixes in this study. These tests were selected to evaluate the mixes from the major aspects of the pavement materials characteristics, that is, high temperature performance, low temperature cracking, and fatigue cracking. They can be classified as two categories of empirical and thermo-mechanical tests. Resistance to rutting (MLPC wheel tracking rutting tester or French Wheel Tracking Test) is used in this study as the empirical test. The thermo-mechanical tests utilized in this research are as follows:

- Complex (dynamic) modulus test;
- Uniaxial tension–compression test for resistance to fatigue cracking;
- Thermal stress restrained specimen testing (TSRST) to evaluate resistance to low temperature cracking.

5.4.1 Mix design and volumetric

A control mix was designed and prepared with only virgin aggregates and virgin asphalt binder. The control mix was a 20 mm-dense graded HMA, commonly used as a base course in Quebec.

The design binder content using a PG 64-28 was determined to be 4.5% by the weight of the total mix. A bituminous mixture with a nominal maximum aggregate size of 20 mm, called Grave Bitumen (aka GB20) is mostly used in binder course layer of pavements in Quebec. The selected virgin binder (PG 64-28) is a medium grade asphalt binder that can be used in warm climates. The aggregate size and gradation were selected based on the LC method specifications. This virgin asphalt concrete mix will be referred to as the control mix hereafter in this paper. The LC Test Methods Compendium, produced by the General Directorate of the Pavement Laboratory of the Ministère des Transports du Québec, presents all the test methods used to measure the characteristics of materials used in the construction and maintenance of infrastructures. Additionally, two more mixes (fine and coarse) were also designed for comparison and validation purposes. Fine RAP mix contained 35% fine RAP content (passing sieve 5 mm) and coarse RAP mix contained 54% coarse RAP content (retained on sieve 5 mm). Initially, it was assumed that all of the aged binder in the RAP could blend into the virgin binder, so that the total binder content could be assumed to be the same for all of the mixes. These mixes would be referred to as Fine RAP (FRM) and Coarse RAP (CRM) in this paper. It should be noted that regardless of the RAP fraction sizes, the target gradation of the mixes was kept the same.

More experimental mixes were initially designed to study the active RAP binder content and their participation in the mixt. The RAP content varied from 0% to 59%. The gradation curves of the experimental mixes were all in accordance with the Ministère des Transports, de la Mobilité durable et de l'Électrification des Transports specification. The mix gradations are shown in Figure 5.4. It can be seen that the mixes have the same black curve gradation. First, there was a question of whether to use the white or black curve gradation. With respect to having the same binder content and aggregate gradation as the fixed variable, the black curve was chosen in this research. Consequently, all mixes would have the same binder content, same aged binder content and same black curve, but different RAP content.

The differences between the two RAP mixes are the RAP particle size and content. This experimental program was designed to investigate the impact of RAP size on Hot Mix Asphalt (HMA). It was envisioned that there is no advantage in looking solely into the RAP content by itself, rather considering the RAP binder content and RAP mix gradation. This plan was designed to achieve the same RAP binder content in both mixes, as well as keeping the final mix gradation the same. In this project, RAP content translates to RAP binder content and RAP size, where the former was kept constant in both mixes and studying the effect of the latter was set as the main objective of this research.

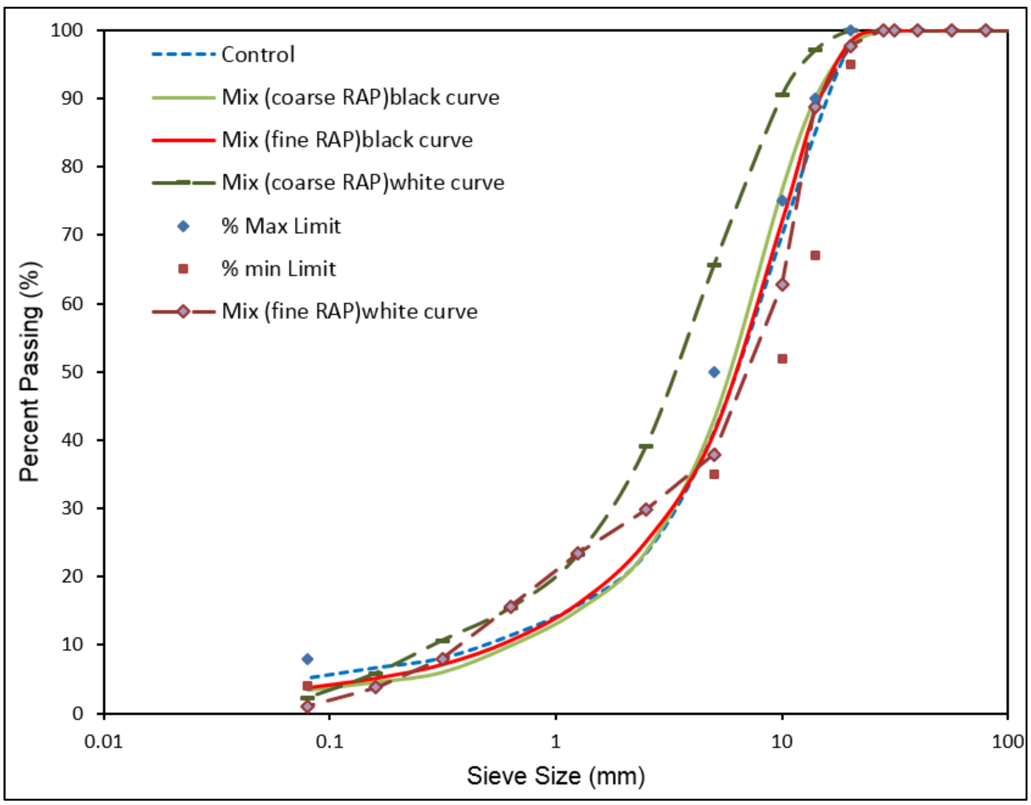

Figure 5.4 Mixes gradation

The Superpave^{TM} mix design method was followed in this study. The only design criterion was the binder content at this level of the study (see Table 5.1 Design Criterion). Using the design binder content of 4.5%, the control mix was prepared with 4.5% virgin binder content, while for the RAP mixes the RAP binder contribution was considered in determining the needed virgin binder content. The recovered RAP binder from chemical extraction showed that the

Coarse RAP (CR) had 4.3% binder, whereas the Fine RAP had 6.7%. In order to do mix design, sufficient amounts of coarse and fine RAP were chosen to have the same recycled binder ratio in the final mixes. Table 5.1 presents the summary of the virgin and aged binder contributions in the fine and coarse RAP mixes.

Table 5.1 Design criterion

Mix	**A = % RAP Content**	**B = % RAP Binder**	**% Recycled Binder Ratio = A*B**	**C = % Total Binder**	**% Virgin Binder = C-(A*B)**
Coarse RAP	54%	4.3	2.3	4.5	2.2
Fine RAP	35%	6.7	2.3	4.5	2.2

The composition of control and experimental mixes, as well as the values of the volumetric obtained from the mix design, are shown in Table 5.2. It can be realized that most of the volumetric values are within the LC method specification.

Table 5.2 The composition of control and experimental mixes.

Mix Type	**% RAP**	**% Air Void (@200 G)**	**% Void Mineral Aggregate**	**% Void Fill with Bitumen**	**Mix Specific Gravity**
Control	0	2.3	14.6	66.5	2.520
Fine RAP	35	1.5	11.6	86.8	2.606
Coarse RAP	54	2.6	12.5	79.7	2.602

The compaction process was done following to the LC 26-400 Fabrication d'éprouvettes au compacteur LCPC. The laboratory compacted cylindrical specimens were stored for a minimum of one month at room temperature in a sand bed prior to testing. Mechanical tests, including fatigue, and complex modulus were performed on cored specimens extracted from slabs as shown in the schematics in Figure 5.5. Samples were compacted by the French MLPC wheel compactor (Figure 5.5 Coring graphical illustration).

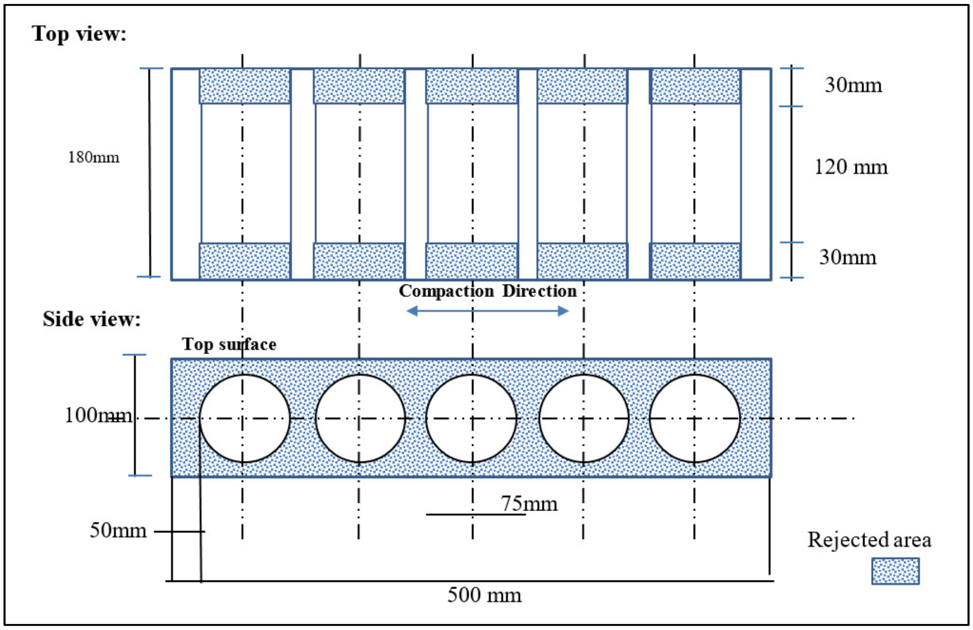

Figure 5.5 Coring graphical illustration

Figure 5.6 Photo of the Laboratoire Central des Ponts et Chaussées (LCPC) slab compactor

5.4.2 Resistance to rutting

All mixes were subjected to laboratory testing for resistance to rutting. The equipment used in this study was developed by France's Laboratoire Central des Ponts et Chaussées (LCPC) (see Figure 5.7 and Figure 5.8). The test was standardized in Europe (EN 12697-22A1) and in the province of Quebec, Canada (LC 26-410). It is also commonly used for research purposes by the asphalt industry in other countries (Gabet , 2011; Perraton et al., 2011).

Figure 5.7 The French rutting test equipment

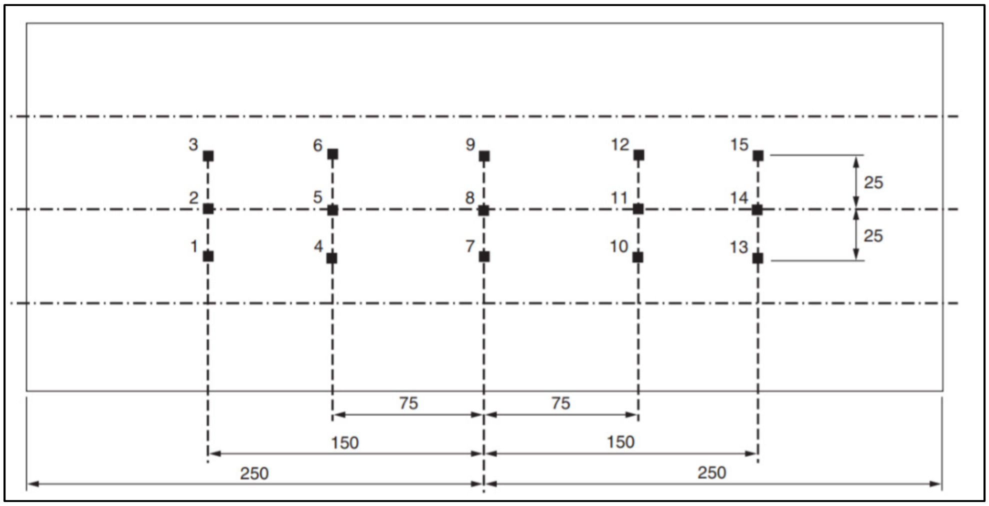

Figure 5.8 Measurement points location in mm (LC 21-410)

Slab dimensions were 500 mm by 180 mm with a thickness of 100 mm. The level of compaction must correspond to what is obtained in the field. On roadways, the required minimum compaction level is usually 92%. For most hot mixes, laboratory-manufactured specimens at the 92% level may lead to rutting after compaction. Consequently, laboratory-prepared specimens are compacted to a greater value, approximately 95%. At this level, post-compaction is generally negligible (Rahman, Hossain , 2014). Heating temperatures for mixing and compaction are indicated in the test method LC 26-003. This laboratory step was done according to AFNOR Standard P98-250-2 Préparation des mélanges hydrocarbonés; Partie 2: Compactage des plaques.

Prior to the rutting test, a preconditioning was done by rolling the pneumatic tire of the rutting tester across the specimen for 1000 cycles at the ambient laboratory temperature. The preconditioning helps with minimizing discrepancies due to the installation of the specimens in the mold. The slabs were then conditioned to reach the testing temperature of 60 °C. Once the temperature of 60 °C was reached, the rutting test was started, and rut depths were measured after 30, 100, 300, 1000, 3000, 10,000, and 30,000 cycles (as applicable). The rut is defined as the mean vertical displacement of the hot mix surface as compared to the mean height of the specimen before starting the test. As described in AFNOR P 98-253-1 Déformation permanente des mélanges hydrocarbonés; Partie 1: Essai d'orniérage, height measurements were taken at 15 locations over the slab area. The stress induced by the tires was maintained at 0.6 MPa during the tests.

Rutting generally progresses along a straight line when plotted on a logarithmic scale against the number of wheel passes. In order to have an acceptable rutting resistance, the rut depth, expressed as percentage of the specimen thickness, should be less than 10%. Yildirim et al.. (Yildirim, 2007) characterized the rutting trend by post-compaction consolidation, creep slope, stripping slope, and stripping inflection point by a typical Hamburg Wheel Tracking Device Test (see Figure 5.9). Post-compaction consolidation is the deformation (mm) at 1000-wheel passes. Creep slope is the inverse of the rate of deformation in the linear region of plot between post compaction and stripping inflection point (if stripping occurs). Stripping inflection point is the number of wheels passes at the intersection of creep slope and stripping slope. Finally, stripping slope is defined as the inverse rate of deformation after the stripping inflection point.

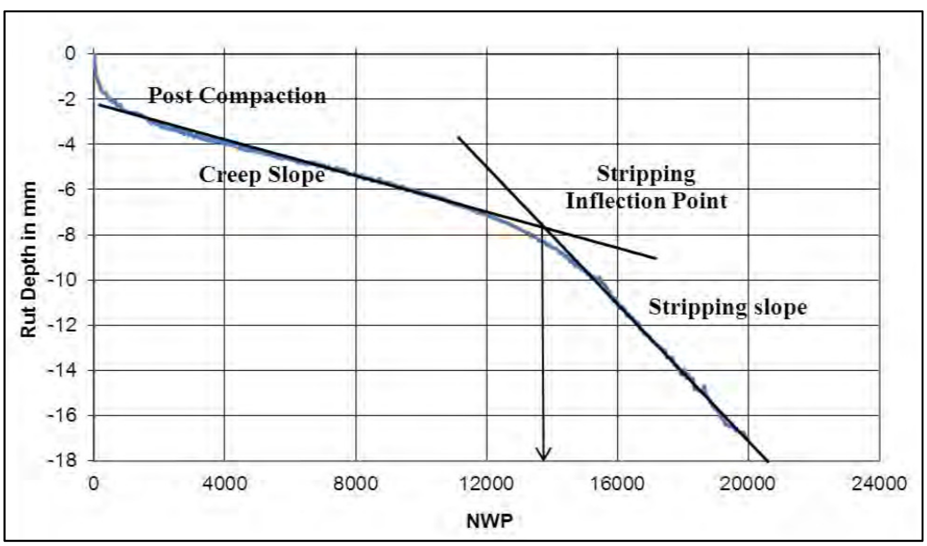

Figure 5.9 Typical hamburg wheel tracking device test results
Taken from Yildirim (2007)

Meunier (2012) characterized the rutting trend from cyclic compression-tension test in three phases as shown in Figure 5.10. They concluded that the deformation increases rapidly in first phase. In phase two, the deformation increases by a constant rate per loading cycle. It should be noted that phase three marks the failure of the material and is usually considered less accurate for the purpose of prediction process than the previous two phases.

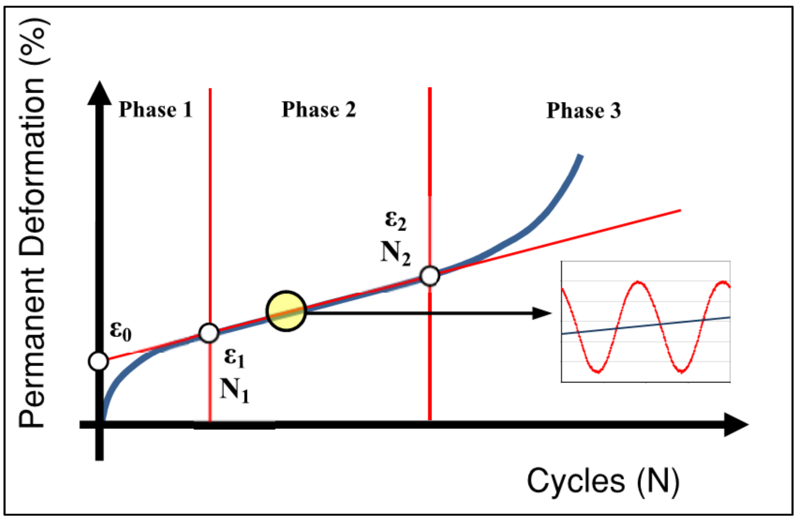

Figure 5.10 Evaluation of permanent deformation
Taken from Meunier (2012)

5.4.3 Thermo-Mechanical tests

For the purpose of thermomechanical characterization of the mixes in this study, two main tests, namely uniaxial fatigue test and TSRST, were performed by means of a 25 kN servo-hydraulic system. Figure 5.11 shows a graphical illustration of the test set-up with the specimen and extensometers. Three extensometers were mounted on the specimens, 120° apart around the sample, to measure the axial strain during the tests.

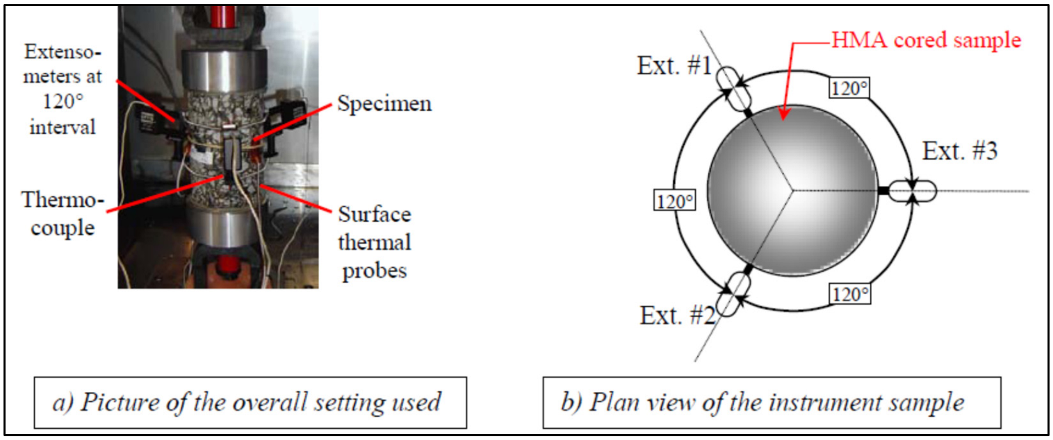

Figure 5.11 Schematics of the test setup used in this study

The set-up was enclosed in an environmentally controlled chamber with three temperature probes, capable of cooling and heating within a range of −40 °C to 80 °C. The following sections provide more details about these two tests.

Fatigue resistance

Fatigue characterization was performed by means of the uniaxial tension–compression (T–C) tests on cylindrical specimens in this study. The experimental test setup is almost the same as the complex modulus test. The fatigue test was performed at a single loading frequency of 10 Hz at 10 °C. The advantage of using this test over the other conventional fatigue tests is possibility of maintaining the homogeneous state of stress and strain in the sample during the testing process.

The cored samples from slabs were tested under uniaxial T–C condition and the axial strain values were measured using three extensometers. The average of recorded values was considered as the strain level in the sample. Data quality measures were used to ensure that the assumption of homogenous stress/strain condition has not been violated. To this end, reaching a difference of ± 25% in the recorded values was considered as an indication of highly non-homogeneous conditions for the strain field within the sample. Therefore, in such cases the test should be considered no longer valid beyond that limit (Baaj, Di Benedetto, Chaverot , 2005).

The graphical presentation of the fatigue test results is usually given by Wöhler curve or fatigue curve (see Figure 12). This curve shows the relation between the fatigue life (Nf) and the level of loading expressed by the initial strain (or stress) amplitude in a bi-logarithmic scale (Tayebali et al. 1994). A particular value of strain called ($\varepsilon 6$) can be found to correspond to the value of the strain level that would lead to a fatigue life of 1,000,000 cycles. This value is commonly used to characterize the fatigue resistance of the bituminous mixes (Di Benedetto, De La Roche , 1998). The fatigue resistance is determined through a series of laboratory tests in different magnitudes of solicitation under controlled conditions (temperature and frequency). As demonstrated by the log-log plot in Figure 5.12, Wöhler's Law is associated with a straight line, where fatigue behavior is characterized by two parameters: the slope (c_2) and the Y-intercept (c_1). Coefficients c_1 and c_2 depend on both, the material and the chosen failure criterion (Perraton, Di Benedetto, Carter , 2011). It should be noted that most of the fatigue-cracking models characterize fatigue failure in three stages: crack initiation, crack propagation, and fatigue induced fracture (Olidis, Hein , 2014). The classical fatigue failure criterion determines the fatigue life as the number of loading cycles that the specimen can take to the point that a 50 percent loss of the initial stiffness for homogeneous tests, or when a 50 percent loss of the initial sample rigidity for non-homogeneous tests is observed (Baaj , 2002; Di Benedetto, Ashayer Soltani, Chaverot , 1996).

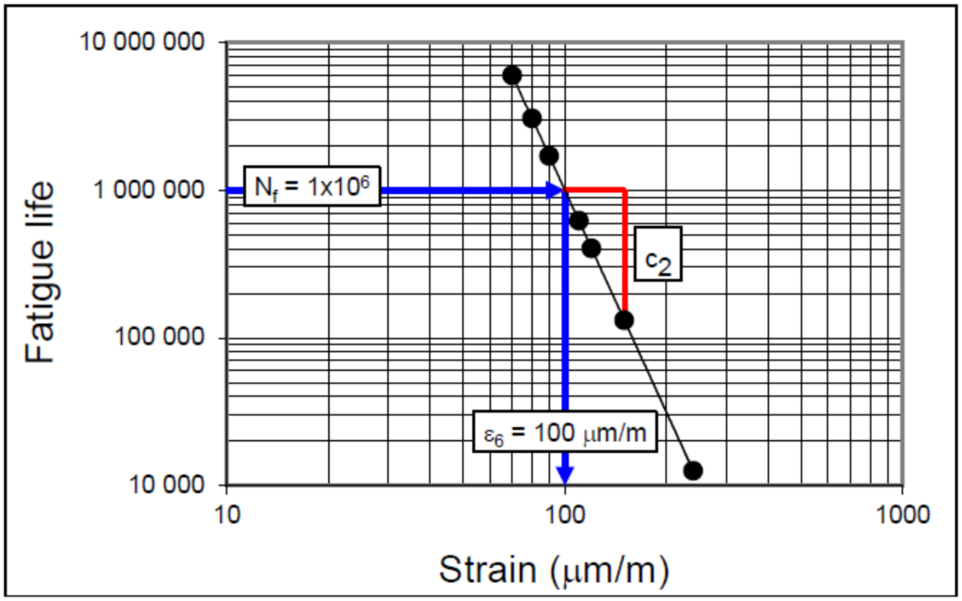

Figure 5.12 Typical fatigue test results from laboratory tests done on an asphalt mixture Taken from Olidis, Hein (2004)

Based on the Wöhler curve, the fatigue characteristics of asphalt mixtures can be expressed by Equation:

$$N_f = C_1(\varepsilon_0)^{(-c_2)} \tag{5.1}$$

- N_f is fatigue life (number of cycles corresponding to the failure point for a given criterion);
- ε_0 is applied strain amplitude (mm/mm) at a given testing temperature (θ_i) under a specific testing frequency (f);
- C_1 is coefficient corresponding to the expected fatigue life for a strain amplitude of 1 mm/mm, at the given testing temperature and loading frequency;
- C_2 is slope of the Wöhler curve when it is associated with a straight line in the log N_f–log ε_0 domain.

Resistance to Low Temperature Cracking: TSRST simulates thermomechanical response of flexible pavements during the cooling period. The principle of the test is to restrain the tested specimen from any axial deformation by keeping the total height of the specimen constant throughout the testing period. As a result of decreasing the chamber temperature at a constant cooling rate of 10 °C/h, the magnitude of thermal stress in the specimen would increases until the failure of the specimen. It is also possible to calculate the axial stress as a function of the measured temperature.

Once at failure point, the stress would reach its peak value, referred to as the failure strength (rf), whereas the corresponding temperature can be defined as the failure temperature (T_f). The slope of the stress-temperature curve increases progressively until a certain temperature where it remains quasi-constant (the stress temperature curve becomes linear). To estimate the value of the quasi-constant slope, the parameter d_{rz}/d_T is calculated by linear fitting of the curve between the failure temperature and the transition temperature. Tapsoba et al. (2016) assumed transition temperature (T_t) as the temperature where axial stress reaches 50% of the failure strength. It corresponds to the temperature where the material changes from ductile to brittle behavior and vice versa and will be used to evaluate the repeatability of TSRST**.**

5.5 Results and discussion

Three mixes were investigated in this study, including a control mix, an HMA mix with inclusion of fine RAP (FRM) and an HMA mix with inclusion of coarse RAP (CRM). The FRM mix consisted of 35% RAP with 2.2% virgin binder and the CRM mix had 54% RAP and 2.2% virgin binder. The results of the experimental studies on these three mixes are as follow:

5.5.1 Rutting resistance of FRM versus CRM

Permanent deformation of the mixes was evaluated at 60 °C using the French rut tester. All mixes (slabs of 100 × 180 × 500 mm) were subjected to repeated loading of a tire inflated to 0.6 MPa, mounted on a carriage that moves back and forth at 1 Hz with a load magnitude of 5 kN. Figure 5.13 and Figure 5.14 show the results of rutting tests. Figure 5.13 indicates the percentage of permanent deformation by straight line for all mixes in the logarithmic scales. The results confirmed that all of the mixes exhibited deformation magnitudes less than 10% after 30,000 cycles. Therefore, it can be concluded that all of the mixes in this study were strong enough to resist the permanent deformation failure. It should be noted that the mixes had the same black curve gradation, but they showed different behavior under the cyclic wheel load. Therefore, as it was expected the black curve assumption was not found to be a reliable representation of the aggregate skeleton when RAP is incorporated.

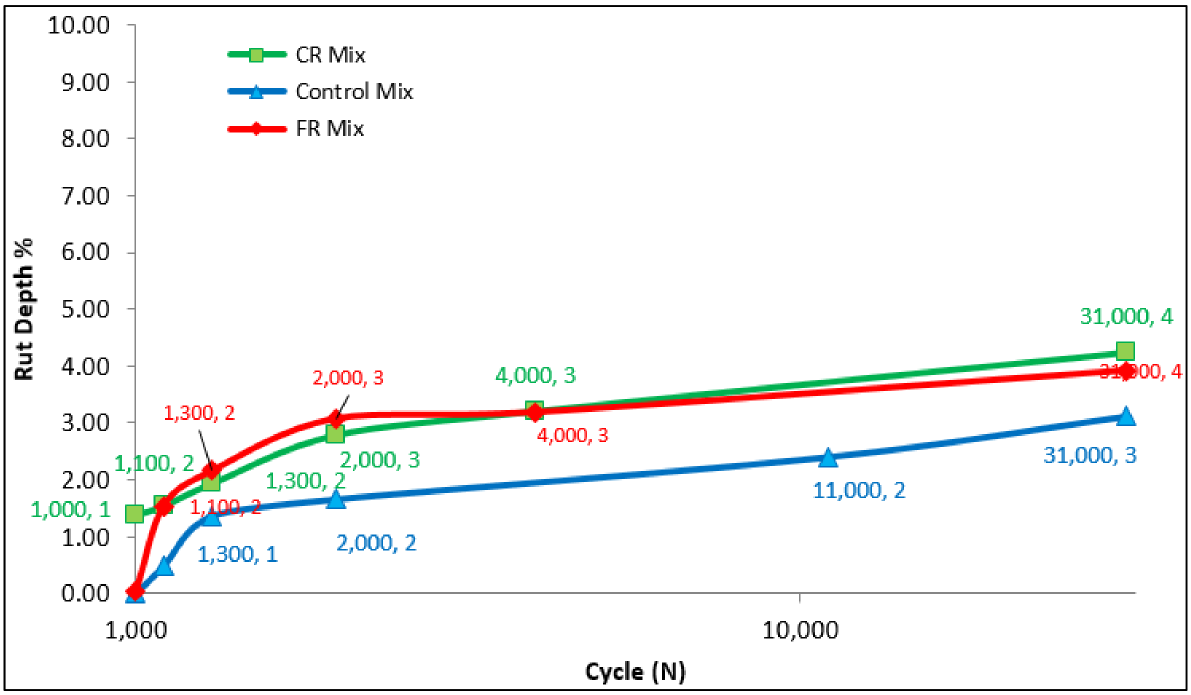

Figure 5.13 Permanent deformation result

Figure 5.14 RAP mix slabs after rutting test

A single sample of each mix type was adopted for permanent deformation validation. Basically, the first 1000 preconditioning cycles (aka cold runs) are assumed to capture the continued consolidation stage. There was a significant difference between FRM and CRM mixes. The rest of loading was performed at 60 °C. In addition, the binder exhibits a softer response at 60 °C than the cold cycles temperature. This difference in rutting might be caused by impact of aggregate gradation and air void content.

After 1000 hot cycles (post compactions), FRM was deformed almost as same as CRM. Both RAP mixes deformation were two times higher than control mix. This section could be characterized by S1 and S2. Parameter "S" represents the slope of the permanent deformation in Figure 13. For the FRM and the control mix, rut depth dramatically increased at first 300 cycles (S1) and continued at a constant slop, whereas for the CRM, these slopes increased at the same rate in both steps (i.e., S1 and S2).

The last stage represents the reaction of material to wheel passes loading which can be translated as rutting values. Table 5.3 shows the slop per section of rutting test. The CRM and control mix responded the same way to the load in rutting section, which was two times higher than FRM (see S4 in Table 5.3). The aggregate gradation plays the main role in rutting resistance. In this study, black curve was kept the same in all mixes.

CRM white curve showed finer than control mix, but the black curve was almost same as control mix. CRM deformed as same as control mix at last stage, but it could be compacted more than control mix at the beginning. Large aggregate gradation (D > 5 mm) in FRM black curve was same as the control mix but fine part of FRM white curve showed more fine content in gradation which was expected. However. It cannot be concluded whether it is more appropriate to use white or black curve up to this point.

Table 5.3 Slope for each section

Slope	FR Mix	CR Mix	Control
S1 (0 to100 cycle)	1.490%	0.165%	0.484%
S2 (100 to 300 cycle)	0.310%	0.195%	0.439%
S3 (300 to1000 cycle)	0.131%	0.122%	0.042%
S4 (1000 to 30,000 cycle)	0.003%	0.005%	0.005%

In conclusion, rutting results can be divided in three phases: deformation at the end of 1000 cold cycles, 1000 hot cycles and 30,000 cycles. First phase which was called continued consolidation earlier, suggests that the CRM gradation and air void were different form FRM mix, which was found to be true; because CRM specimen had 8% air void but FRM specimen had 6%. Second phase which was called post compaction (S1, S2, S3), suggests that the specimen binder is soft enough to indicate the difference in aggregate gradation. Flatter slope can be translated to well packing phenomenon. It was recognized that the CRM could be packed better than FRM. Last section which was called rutting, showed the rutting resistance of mixes. The results indicate that the FRM was more rut-resistant than CRM and also than the control mix. The FRM and the control mix differed only in the fine part in aggregate gradation, especially magnified by the white curve. Thus, FRM mix had stiffer fine skeleton than control mix. According to black curve, CRM and control mix had the same gradation, but considering the white curve, CRM was much finer than control mix. Basically, coarser mixes have better rutting resistance as compared to the CRM. The CRM was expected to be weaker than the control mix but exhibited the same response as the control mix. In both FRM and CRM mixes, better or at least the same resistance as the control mix was recorded in spite of the fact that there is 54% (or 35%) recycle materials in the mix. The results indicate that the FRM prepared with 35% RAP exhibited almost similar performance as the CRM prepared with 54% RAP. It can be concluded that both RAPs incorporated mixes exhibit satisfactory rutting resistance.

5.5.2 Fatigue resistance results

In this study, the classical method was used among the four types of failure criteria mentioned earlier. Table 5.4 provides the specimen details, the actual and target initial strain values, and the number of cycles to failure (aka fatigue life) for each specimen. The fatigue results are sensitive to the air void level, and hence it was attempted to maintain the same level of air void for all the specimens.

Table 5.4 Uniaxial T–C fatigue test conditions (10Hz, 10 °C)

Mix Type	Sample Name	Target Def, (µdef)	% Va	Real Def, (µdef)	Nf	Log Real Def, (def)	Log Nf II/III
Control Mix	S2C2	80	6.8	71	2,405,986	−4.15	6.38
	S1C5	100	6.8	99	61,464	−4.01	4.79
	S1C2	70	8.0	66	4,715,411	−4.18	6.67
	S2C7	90	6.5	84	2,500,000	−4.08	6.40
Coarse RAP Mix	CR1	60	7.2	53	1,324,415	−4.28	6,12
	CR3	40	7.6	30	2,145,327	−4.52	6.33
	CR2	70	8.4	61	26,211	−4.21	4.42
Fine RAP Mix	FR1	90	4.4	82	513	−4.09	2.71
	FR2	110	4.1	113	277	−3.95	2.44
	FR3	100	7.1	87	5103	−4.06	3.71
	FR4	60	5.3	59	79,608	−4.23	4.90
	FR5	50	6.3	48	22,163	−4.32	4.35
	FR6	70	5.5	58	46,000	−4.24	4.65

In Table 5.4, it should be noted that the classic failure criterion of 50% reduction in the initial stiffness was not found reliable, due to the fact that a significant loss of modulus has occurred during the first phase of the T–C test. In spite of the fact that some researchers use the 50, 100, 200, or even 1000 cycles to calculate the initial modulus, the results were not representative of the fatigue-induced damage. Therefore, the more scientific Wöhler approach was used to study the fatigue performance of the mixes in this study.

Regression based fatigue equations were developed based on the test results to quantitatively characterize the mixes (Figure 5.15). In order to develop this chart, various fatigue samples were subjected to sinusoidal load at three different strain levels in order to be able to run a linear regression. The value of ε6 corresponds to the strain level at which the asphalt mix would reach a fatigue related failure after 1 million cycles. For the sake of comparison, it can be noted that a standard asphalt base course material, made with straight run asphalt cement, usually exhibits ε6 values in the range of 70 to 90 μm/m. ε6 in this project is 81 μm/m for control mix. The value of R2 shows the quality of linear assumption. ε6 for CR in higher than that of FR. CR failed at 43.57 μm/m and FR failed at 28.94 μm/m. The slope of the trend line shows the degree of sensitivity of mix to deformation. Sharp slope is highly sensitive to deformation, it means that under a small change of deformation there would a huge difference in number of repetition that the mix can take until failure. The CR had less sensitivity to the changes in deformation, and it even surpasses the control mix with this regard.

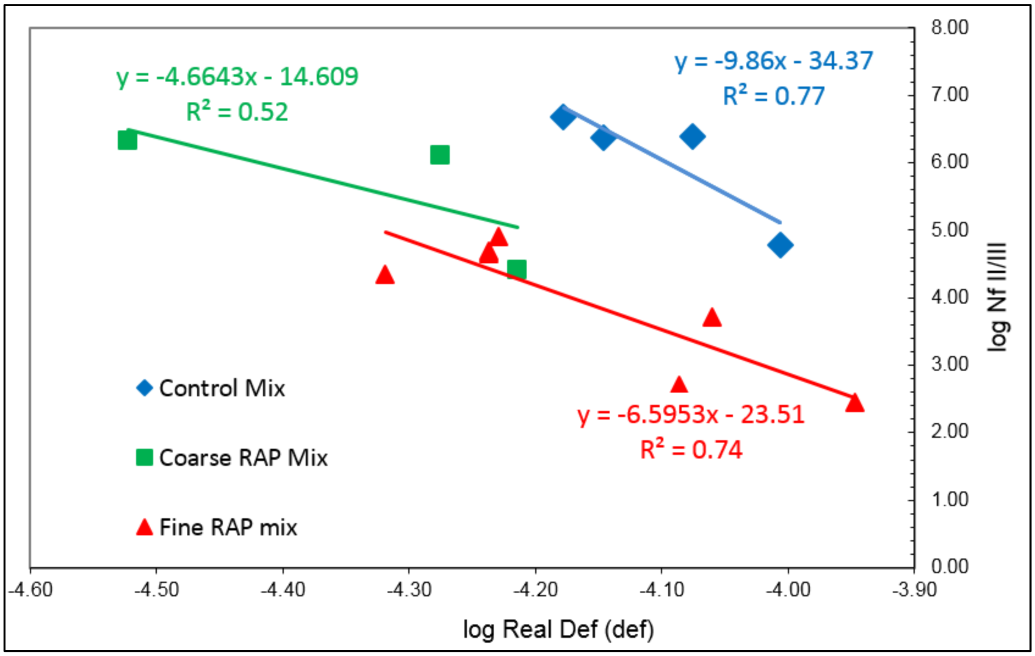

Figure 5.15 Wöhler Curve

Basueny et al.. (2016) concluded that when the percentage of RAP in the mix is considerably high, the aged RAP binder creates a significant change in the mixture properties. Therefore, it can be concluded that the influence of RAP on the final HMA property also varies with the amount of RAP. The mixes in this study were supposed to have similar recycled binder replacement ratio and black curve gradation, however the CR mix resists to fatigue much better than the FR mix. It can be concluded that black curve assumption is not the best representation of RAP gradation. Virgin binder in CRM is mostly covering the fine natural aggregates and adhesion to CR. More unaged binders in mastic increase the resistance under the tension and compression repeated loads. FRM has more aged binder in fine part of the skeleton that caused weakness of fatigue resistance. Overall, the CRM exhibited a better fatigue performance than the FRM.

5.5.3 Resistance to low temperature cracking through TSRST

In addition to fatigue cracking, another major concern for HMA mixes with RAP particles is their resistance to low temperature cracking. In general, RAP mixes are stiffer than conventional mixes, due to the highly oxidized nature of the aged binder in RAP particles. The values of the fracture temperature and the corresponding stress at failure, obtained from the TSRST tests for all the tested mixes, are presented in Figure 5.16.

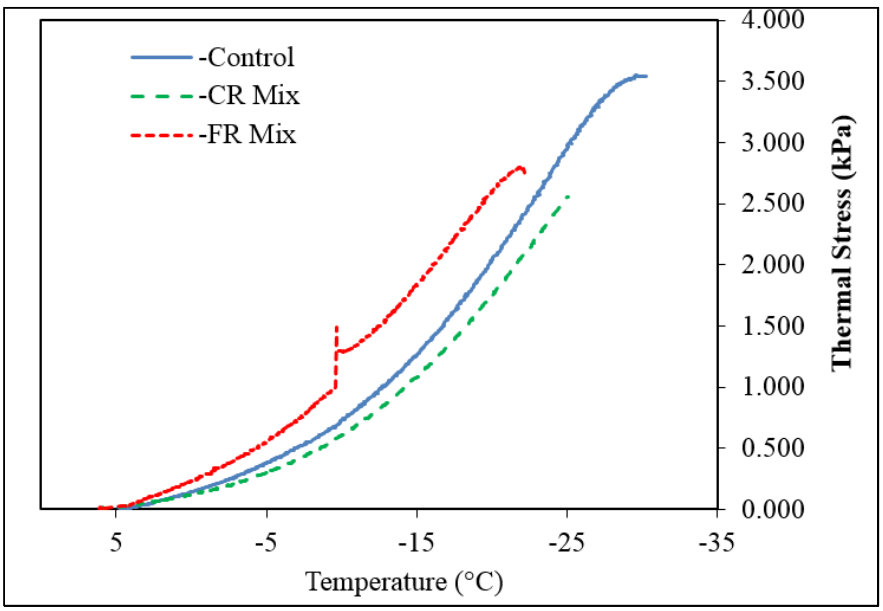

Figure 5.16 TSRST results

The description of the test progress and the associated data collected is as follows (Carter, Paradis, 2019):

- Slope no.1: This parameter represents the performance during the relaxation period;
- Slope no.2: This slope represents a value analogous to the modulus of elasticity in a diagram of stress versus deformation of an elastic material;

- Glass transition temperature (T_g): This temperature represents the end of the relaxation and the beginning of a linear change with temperature;
- Transition temperature midpoint: This temperature corresponds to the intersection of the tangent lines in section no.1 and the tangent line fitted to the section no.2 of the curve and indicates the transition temperature between two stages of the simplified bilinear response of a material;
- Maximum tensile strength: This value represents the maximum stress applied to the test specimen just before it fails;
- Maximum stress temperature: This temperature is simply the value obtained when the maximum stress is reached.

Figure 5.17 Figure 5.19 show the TSRST values derived from the results for the purpose of comparison. The maximum tensile strength values were found to be 3548, 2558 and 2799 kPa for the control, CR and FR mixes, respectively. The maximum stress temperatures were measured as −30, −25, and −22 °C for the control, CR, and FR mixes, respectively. The Transition temperature midpoint of the control and CR mix is almost the same (i.e., −11 °C) but Transition temperature of the FR was very low (i.e., −5 °C). The results indicate that the CR mix performed better than the FR mix with respect to low temperature properties. The CR had lower Tg midpoint and lower failure temperature, however FR failure stress was slightly higher than the CR. In addition, the value of Tt, calculated according to Tapsoba et al. (2016) study, was found to be the same for both mixes.

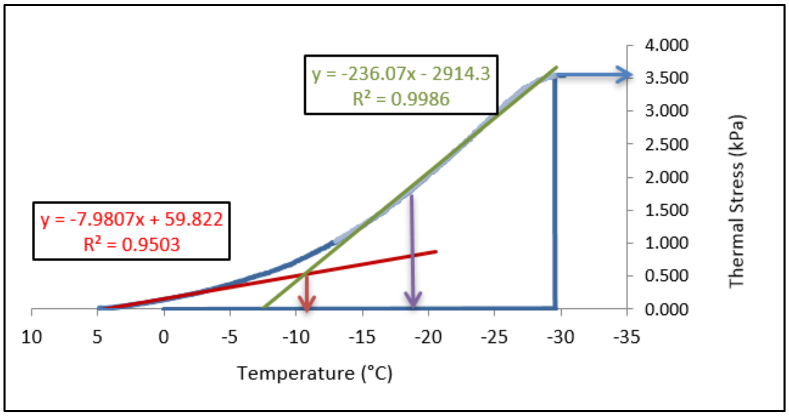

Figure 5.17 TSRST for control mix

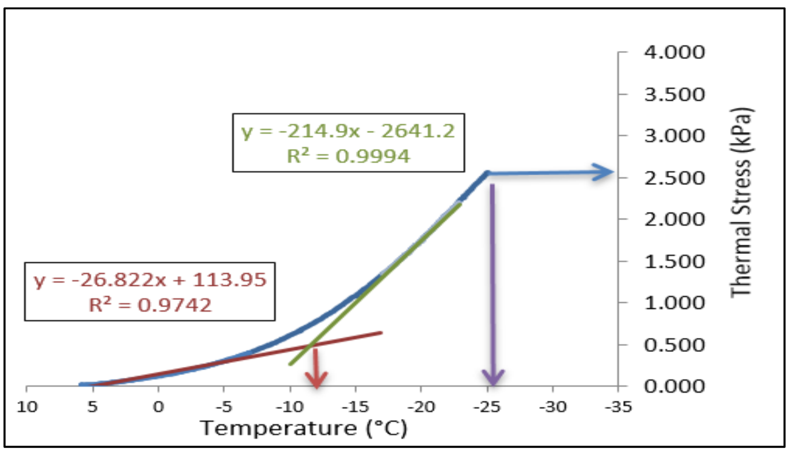

Figure 5.18 TSRST for coarse RAP (CR) mix

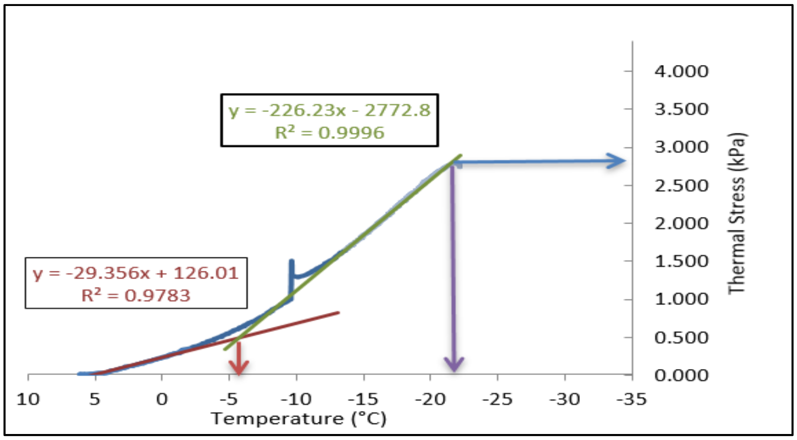

Figure 5.19 TSRST for fine RAP (FR) mix

Void fills with bitumen (VFB) represent the effective bitumen content. The decrease of VFB indicates a decrease of effective bitumen film thickness between aggregates, which will result in higher low-temperature cracking and lower durability of bitumen mixture since bitumen perform the filling and healing effects to improve the flexibility of mixture.

5.5.4 Complex modulus

Various criteria are available in order to compare the stiffness of different bituminous materials. Baaj et al.. (2013) suggested to looking into the stiffness of the materials in the following ways:

- The stiffness |E*| at −30 °C and 3 Hz: this value gives the material stiffness for a low temperature and a high-frequency condition;
- The values of |E*|/sin (ϕ) at 40 °C and 0.03 Hz: this ratio is used as an indicator of rutting resistance;
- The stiffness |E*| at 20 °C and 3 Hz frequency: The stiffness of the mix (resilient modulus) at this temperature is used in the AASHTO'93 empirical pavement design method.

In addition, Perraton et al. (2014) also used the stiffness at 15 °C and 3 Hz. typically, standard bituminous base course materials have dynamic modulus values in the range of 5000 to 7000 MPa when tested under the same conditions at 15 °C.

Figure 5-20 indicates the Cole-Cole plot for all mixes from −35 °C to +35 °C. Two replicate specimens were used for each of the mixes. The measured data was modeled with the 2S2P1D model. There is a notable difference between the control mix and RAP mixes with respect to loss (or so-called imaginary) modulus. Several factors can affect the loss modulus of a bituminous material such as air void level, bitumen content, and bitumen type. The results indicate that, generally, the two RAP mixes are the same according to the Cole-Cole diagram presented in Figure 5.20. However, this plot cannot explicitly distinguish the differences in the bitumen characteristics.

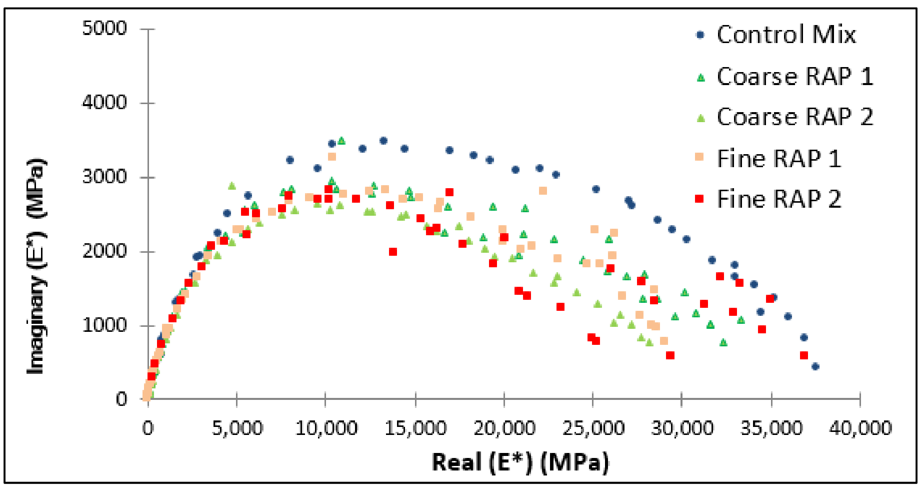

Figure 5.20 Complex modulus master curve in Cole-Cole plot

The time-temperature superposition principle (TTSP) was applied to analyze the complex modulus test data. This principle was verified by several studies dealing with the unidirectional linear viscoelastic behavior of bituminous materials (Delaporte, Di Benedetto, Chaverot, Gauthier, 2007). As shown in Figure 5.21, at high frequency, RAP mixes have lower stiffness than the control mix, and this difference becomes greater at lower frequencies. According to TTSP principle, high frequency could be translated to low temperature and low frequency could be translated to high temperature. Therefore, in a full range of temperature, RAP mixes were slightly softer than the control mix. However, the RAP content is not the same (i.e., 0%, 35%, and 54%). Therefore, the FRM with inclusion of 35% RAP content has almost the same behavior as the CRM with 54% RAP content. Consequently, it can be inferred that the binder contribution from 54% CR would be almost have the same effect as that of the 35% FR mix used in this study. In addition, FRM stiffness was found to be strongly sensitive to the testing conditions. Figure 3 shows that the same FRM specimens are variable at low frequency, but all CRM mix specimens show consistent response, suggesting that they are less sensitive to the testing conditions.

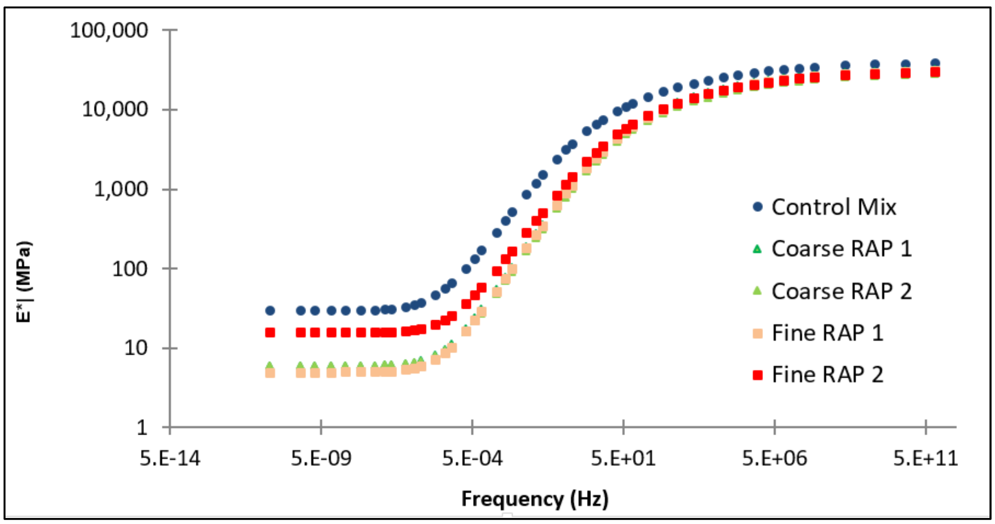

Figure 5.21 Master curve of the norm of complex modulus

The rheological properties of the mixes can also be expressed in terms of phase angle. A phase angle (δ) value of 0 degrees means a purely elastic material and 90° means a purely viscous material. Figure 5.22 shows the master curve of the phase angle for the mixes investigated in this study. The RAP mixes exhibited more viscous response than the control even though they have less virgin bitumen content (i.e., 2.2%). The FR results varied significantly, which might be associated with the higher RAP bitumen content and some unexpected phenomena in the fine RAP particles like clustering and variability in the film thickness of the particles.

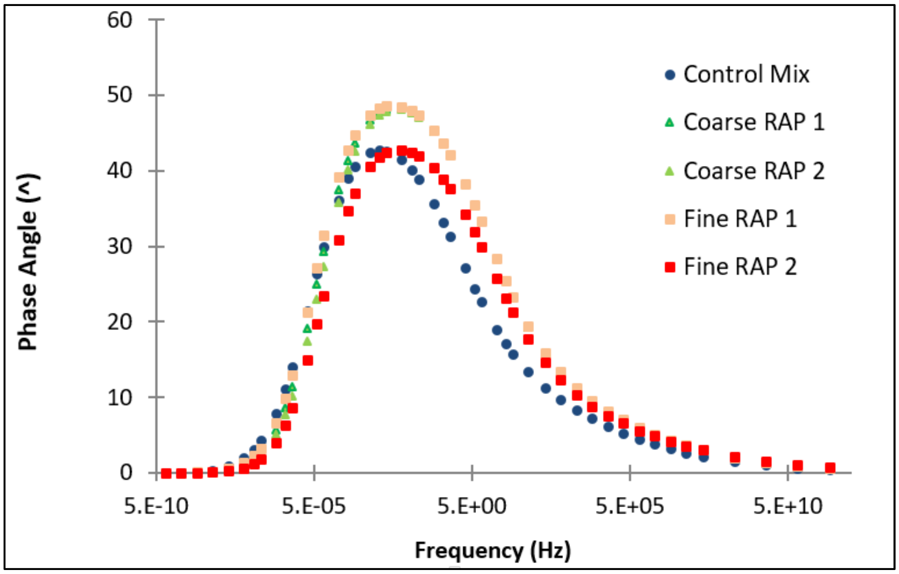

Figure 5.22 Master curve of the phase angle of complex modulus

Figure 5.23 presents a summary of the 2S2P1D model parameters in the Cole-Cole model and the corresponding values of these parameters are listed in Table 5.5.

- h, k: exponents such as $0 < k < h < 1$, related to the ratio $E_{Imaginary}$ / E_{Real} when ϖ tends to 0 (resp. to infinity);
- E_0: the glassy modulus when $\varpi \approx \infty$;
- E_{00}: the static modulus when $\varpi \approx 0$;
- η: Newtonian viscosity.

CRM is the same with control mix and FRM in black curve, but it has more active aged bitumen. FR bitumen could not increase the FRM stiffness, but CR bitumen was more active in mixture and increased the CRM stiffness.

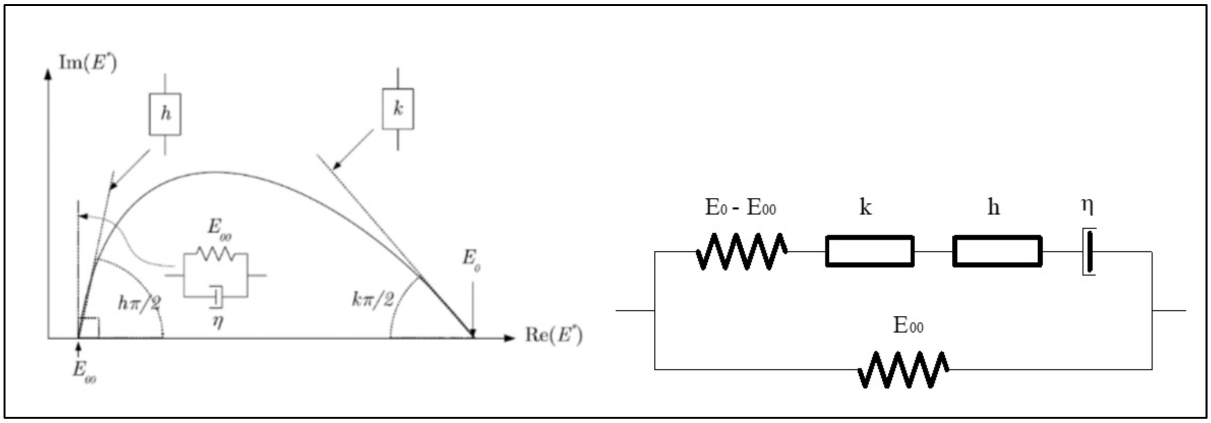

Figure 5.23 2S2P1D model parameters in Cole-Cole model

Table 5.5 The complex modulus properties of the mixes

MIX	E* at −30 °C, 3 Hz (MPa)	E*/Sin(ϕ) at 40 °C and 0.03 Hz (MPa)	E*at 20 °C, 3 Hz (MPa)	E*|at 15 °C and 3 Hz (MPa)	E_0 (MPa)	E_∞ (MPa)	k
Control	37,000	99	4000	5100	30	40,000	0.18
CR	32,762	113	5700	7554	6	33,000	0.19
FR	31,214	27	3000	4915	5	32,000	0.16

The fact that the same or even better results could be achieved using the coarse RAP at a higher rate, as compared to the fine RAP, offers significant potential advantages in producing high RAP content mixes.

5.6 Conclusions

Review of the literature indicates that research on producing asphalt concrete mixes with only fine or coarse RAP particles has been scarce. The main goal of this study was to evaluate the effect of fine and coarse RAP fractions on performance of the high RAP content asphalt concrete mixes. Accordingly, three mixes were designed and investigated in this study, including a control HMA prepared with virgin materials, a Fine RAP mix (FRM) with 35% percent RAP, and a Coarse RAP mix (CRM) with 54% RAP. The total binder content considering the different contributions from the RAP particles was maintained the same for all the mixes. The following conclusions can be drawn based on the results of this study:

- Black curve and white curve assumptions for the RAP particles were explored and it was concluded that the black rock assumption, which is commonly used, cannot be representative of the RAP contribution to the total binder content and the skeleton of the mix. The actual gradation lies somewhere in between these two extreme cases;
- The classifying the RAP particles in HMA is changing the role of virgin binder in the mix. Virgin binder role is covering the natural coarse aggregates and adhesion to aged bitumen of fine part in FRM. On the other hand in CRM, it is covering the natural fine aggregate;
- It is important to look into the binder contribution from RAP with respect to the RAP particle sizes, rather than solely considering the RAP content;
- The FRM and CRM specimens both passed the rutting resistance evaluation criterion of less than 10% deformation. The rutting performance of these high RAP mixes was found to be at least the same or even better than the control mix;
- While the control mixes surpassed both the FRM and CRM with respect to fatigue life, it was found to be more sensitive to the changes in deformation;
- The transition temperature midpoint and the maximum tensile stress temperature for the CRM was much more desirable as compared to the FRM and was slightly worse than the control mix.

Overall, the CRM mix, exhibited acceptable performance with respect to rutting, fatigue, and thermal cracking. It can be concluded that the RAP particle size can have a more significant effect on the mix performance than the RAP content. RAP content should be considered along with other important parameters such as RAP particle size and gradation, recycled binder ratio, and RAP binder content.

Author Contributions: S.S.S. conceived of the presented idea. A.C. and H.B. developed the theory and verified the analytical methods and supervised the project. S.S.S. wrote the original draft; and P.T. wrote, reviewed, and edited the manuscript and verified the analysis. All authors discussed the results and contributed to the final manuscript.

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Conflicts of Interest: The authors declare no conflict of interest.

CHAPTER 6

CONCLUSION FOR PART A

The use of Reclaimed Asphalt Pavement in asphalt mixes is a green alternative for the pavement industry to preserve resources, energy and money. During the past decades, several studies have been conducted to investigate the possibilities, pros and cons of utilizing RAP in HMA. As mentioned earlier in this document, RAP content is recommended to be limited in HMA due to negative impacts on long-term performance. This study was targeting the increase of RAP content without any additional cost for adding modifiers and additives to the RAP. A principal hypothesis in this work is that RAP's role in the HMA is significantly related to RAP particles size. Therefore, further investigation is needed to characterize RAP by size, by separating the coarse from the fine particles. The novelty of this work has been on the fractionated RAP and investigating the intrinsic differences between RAP fractions of different sizes and their impact on overall performance of a high RAP mix produced with solely coarse or fine RAP particles.

Chapters 3, 4 and 5 presented the methodology and the evaluation process of the comparison of the impact of RAP particles in HMA in terms of short- and long-term performance.

Fine RAP and coarse RAP were separated from a single source of RAP to minimize any errors caused by different sources of materials. The 5 mm sieve was chosen as the separator to obtain desired RAP particles. Retaining RAP quantities on sieve 5 mm was considered as coarse RAP (CR) and passing quantities considered as fine RAP (FR).

At the first step of the evaluation process, a method was developed to compare the existing active aged binder quantities in fine and coarse particles. Active aged binder is the amount of RAP binder that transfers from RAP particles to virgin aggregate by rubbing RAP and virgin together prior to adding virgin binder. As it was expected, FR had around 2% more binder than CR. But the results showed that the CR transfer more binder to virgin aggregate than FR in terms of volumetric measures. It is believed that the rough surface area of the virgin aggregate surface plays a key role to take off the binder from the RAP. At this time, active binder is important because more binder can cover more virgin aggregates and decrease required virgin binder in mix, subsequently it causes saving more virgin binder. It must be noted that no virgin binder was used in this step.

In the second step of the process, only recovered binder from RAP and RAP mixes were characterized. DSR was adopted to measure the shear modulus, ignition test was used to quantify the asphalt binder content, FTIR was used for chemical properties of the asphalt binder and ESEM image analysis was used to visualize the difference in binder microstructure. Four different binder samples extracted from FR, CR, and blend of FR with virgin binder and CR with virgin binder were tested. It must be noted that recycling binder rate and the virgin binder were kept constant in mix design of all blend. PG grading showed that recovered binder from FR and CR are not the same. Also it could be seen that binder from CR and FR mixed with virgin binder are not the same either. Shear modulus results showed FR stiffer than CR and aging rate is faster in fine particles probably due to the higher surface area.

Following to preliminary study and binder characterization, study of mixture of virgin and RAP (binder and aggregate) was planned as the last step of part A. Three mixes of HMA with and without RAP were designed as follows: a Fine RAP mix (FRM) with 35% fine RAP; a Coarse RAP mix (CRM) with 54% coarse RAP, and Control Mix not containing RAP. However the RAP content looks different in the mixes but the aged binder ratio in all mixes FRM and CRM are the same (2.3%). It was noticed that the black curve cannot be reliable in mix design. The actual gradation lies somewhere in between these two extreme cases. The role of added virging binder in FRM is to coat the coarse virgin aggregate and to ensure adhesion between fine RAP particles and virgin coarse aggregate, which changes the viscoelastic characteristics of the HMA. But it’s role in CRM is to coat the fine virgin aggregate and to ensure adhesion between Fine aggregate and coarse RAP.Coarse and fine aggregate has different roles in the aggregate interlock and HMA skeleton. So it is expected CRM and FRM should not have the responses to tension compression loads (fatigue resistance) . In addition, both mixes have an acceptable rutting resistance. Furthermore, the results of this part provide valuable insights about the levels of binder contribution from coarse versus fine RAP fractions with regard to the performance of the mixes.

CHAPTER 7

INVESTIGATION OF THE TENSILE STRENGTH OF HOT MIX ASPHALT INCORPORATING PULP ARAMID FIBER (PAF)

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7.1 Abstract

Premature cracking of flexible pavements is a very common problem in Canada. Nowadays, it is common to use several types of additives and modifiers to asphalt binders and asphalt mixes to improve their performance and increase the service life of flexible pavements. In order to mitigate pavement cracking, the asphalt mixes used in the pavement structure need to have a high resistance to fatigue or thermal cracking according to their position in the pavement structure. Several studies reporting on the use of fibers in asphalt concrete have been found in the literature. The objective of this project is to study the impact of the addition of Pulp Aramid Fiber (PAF) to Hot Mix Asphalt (HMA) in terms of mix design and indirect tensile strength.

The optimum asphalt content of the fiber mixes is first determined and then the impact of fiber on their volumetric properties is investigated. Finally, the behavior of PAF in HMA is

characterized by indirect tensile testing at different conditions. PAF mixes showed better ductility, even at lower temperatures, than the control mix. Therefore, PAF would lead to an improvement of the resistance to low temperature cracking and would delay crack propagation in the mix.

7.2 Introduction

Premature cracking of flexible pavements is a very common problem in Canada. Nowadays, it is common to use several types of additives to asphalt binders such as polymers (Baaj, Di Benedetto, Chaverot , 2005) or modifiers to the asphalt mixes, such as fibers used in Stone Mastic Asphalt (SMA) (Perraton, Baaj, Di Benedetto, Paradis, 2003), in order to improve their performance and increase the service life of flexible pavements (Perraton, Baaj, Carter, 2010). In order to mitigate pavement cracking, the asphalt mixes used in the pavement structure need to have a high resistance to fatigue or thermal cracking according to their position in the pavement structure.

Several studies reporting on the use of fibers in asphalt concrete have been found in the literature (McDaniel, 2015). The fibers could be introduced directly in mix like in the case of SMA or indirectly through the use fiber-containing modifiers such as post-manufacturer roofing shingles (Baaj, Ech, Tapsoba, Sauzeat, Di Benedetto , 2013). According to the literature, the types of fibers used in asphalt could be divided into two categories, namely Synthetic Fibers and Natural Fibers. Synthetic Fibers are the most common. Although there are many types of Synthetic Fibers, glass, carbon, and aramid fibers represent the most common fibers used in asphalt mixes (McDaniel, 2015) . The objective of this research project is to investigate the use of Aramid Fiber to improve the performance of asphalt mixes.

Kevlar is an aromatic polyamide or aramid fiber that was introduced in the early 1970s by DuPont. It was the first organic fiber with sufficient tensile strength and modulus to be used in advanced composites. Factory details show the tensile strength of Kevlar to be approximately five times higher than that of steel, with a corresponding tensile modulus. Kevlar, which is a trade name of Aramid Fiber, was originally developed as a replacement for steel in radial tires, but is now used in a wide range of applications. In order to characterize the performance of a modified Kevlar mixture, a set of performance tests and behaviour characterizations were performed (Wigotsky, 2002).

This research aims to provide the researchers and the pavement industry with a better understanding of the potential of using Aramid Fiber in asphalt mixtures. Further benefits that may arise from the addition of Aramid pulp or short fibers include rendering bituminous materials more durable and lending them a higher tensile strength at low temperatures.

7.3 Background

Canada ranks seventh in the world in terms of road network length and 95 percent of the paved roads in Canada are flexible pavements (The world's biggest road networks, 2017). According to the Canadian Infrastructure Report Card (2012), more than 50 percent of municipal roads are in fair, poor, or very poor condition. Consequently, there is a strong necessity for developing and using high performance asphalt mixes that would be more resistant to degradation and would extend the service life of flexible pavements. One of the options studied is the use of fibers in asphalt mixes.

Many types of fibers are available for incorporation into asphalt paving mixtures. Cellulose and mineral fibers are commonly used in gap-graded SMA and open-graded or porous mixtures. Polypropylene and polyester fibers used to be employed in dense-graded mixtures, but their use has diminished. Various polymers, steel wool, and other fibers are also sometimes added to asphalt mixtures. The relative benefits and issues facing these different types of fibers are not well-documented. The appropriate specifications and material characteristics needed to ensure their best performance in different climates, under different traffic loadings, and in different applications, are also not widely recognized (McDaniel, 2015).

Fibers can come in various forms as well. They can be short and randomly oriented, long and unidirectional, in tufts, or woven (Abiola, Kupolati, Sadiku, Ndambuki , 2014). Individual types of fibers can have various structures and cross-sections. Scanning electron microscopy was used by Chen & Xu (2010) to investigate the structure of some fibers, including asbestos, lignin (cellulosic), polyacrylonitrile, and polyester. They found that synthetic fibers had "antenna features" at their ends that helped anchor them in the binder phase, creating a stronger network within the binder. They mentioned Asbestos fibers have a smooth texture and a thin diameter, yielding a large surface area while Cellulosic fibers have a rough texture, and the diameter varied along the length of individual fibers.

The physical dimensions of fibers can affect how well they can disperse and interact with the other components of the mixture. Also the virgin aggregate characteristics can affect the interaction of additives and virgin materials in HMA (Saliani, Carter, Baaj , 2016). For example, the lengths of fibers can be modified to relate to the maximum aggregate size in the mixture, with smaller aggregate sizes using shorter fibers. Long fibers may be difficult to mix uniformly into the mixture in the lab or the plant because they can get tangled and clump together (do Vale, Casagrande, Soares, 2013; Tapkın, Uşar, Tuncan, Tuncan, 2009; Tapkın, Çevik, Uşar, 2012). Sieve analysis is also sometimes used to characterize fiber size. Clearly, if fibers are too long, this may create the so called "balling" problem, which is what causes them to clump together, and consequently, they may not blend perfectly with the asphalt cement. Short fibers may just assume the role of filler in the mix (McDaniel, 2015).

Figure 7.1 shows examples of clumping during and after compaction. Note that even some short fibers may cause clumping. It has been suggested that increasing the mixing time could help avoid this (Watson . 2003).

Figure 7.1 Clumping during and after compaction
Taken from McDaniel (2015)

Fibers may improve the rutting and cracking resistance of bituminous mixes. Generally, fibers appear to be more effective at improving the performance of marginal or lower-quality mixtures and are not generally known to be detrimental to performance in dense mixes. One of the main advantages of fibers is that they can reduce the drain down (bleeding) of a mixture (McDaniel, 2015).

Fibers have been used in asphalt mixtures for two main reasons:

- To increase the toughness and fracture resistance of Hot Mix Asphalt (HMA) (Putman & Amirkhanian, 2004). There has been limited use of fibers to increase toughness, even though research has shown that the addition of polyester fibers does increase the toughness of asphalt mixtures (Hansen, McGennis, Prowell, Stonex , 2000), and;
- To act as a stabilizer and prevent the drain down of the asphalt binder (Hansen, McGennis, Prowell, Stonex , 2000). Fibers (cellulose- and mineral-type, for example) have been used in large part as stabilizers in asphalt mixtures such as SMA and Open-Graded Friction Courses (OGFC).

Bennert (Bennert , 2012) compared a conventional mix with a similar mix containing Aramid Fiber. From the results, the Dynamic modulus (AASHTO TP 79) of the plant-produced mixtures showed that the two mixes had a similar stiffness at all frequencies. Furthermore, the fiber-reinforced mixture had a slightly better resistance to rutting in the asphalt pavement analyzer (AASHTO T 340) than the control (2.70 vs. 3.14 mm). Likewise, testing results in the repeated load flow number test (AASHTO TP 79) also showed a slightly better performance for the fiber mix (919 vs. 747 cycles) (Bennert , 2012).

Jiang (1992) evaluated 8 years of field performance for various thicknesses of asphalt overlays with and without fibers. The fibers were 10 mm long and added at 0.3 percent by weight of the mixture. He showed that the use of fibers on overlays did not decrease cracking, but that it could delay propagation. Jiang believed that cracking was caused by vertical and horizontal movements in the base and in subbase layers, and thus suggested the use of fiber in the base layer in order to reduce cracking.

There are two methods used to introduce fibers, namely, the wet process and the dry process. The wet process blends the fibers with the asphalt cement prior to the binder being incorporated into the mixture. The dry process mixes the fiber with the aggregate before asphalt is added. Generally, the dry process is preferred, as it is experimentally simpler and allows for a better fiber distribution in the mixture. Moreover, since the fibers used do not melt in the asphalt, there is apparently no special benefit brought about by the wet process. The dry process (Hejazi, 2007; Munn, 1982; Echols, 1989) is generally used in field work carried out on fiber-reinforced asphalt mixtures. Another reason for using the dry process is that it minimizes the major problem of clumping or balling of fibers in the mixture (Labib & Maher , 1999).

7.4 Materials

The fiber used in this work is Aramid fiber provided by DuPont. It is known as KEVLAR® brand pulp, with its product code being 1F361 (Figure 7.2). Table 7.1 shows the specifications of 1F361. Its high moisture content and static energy make it difficult to achieve a homogeneous mix when used in HMA.

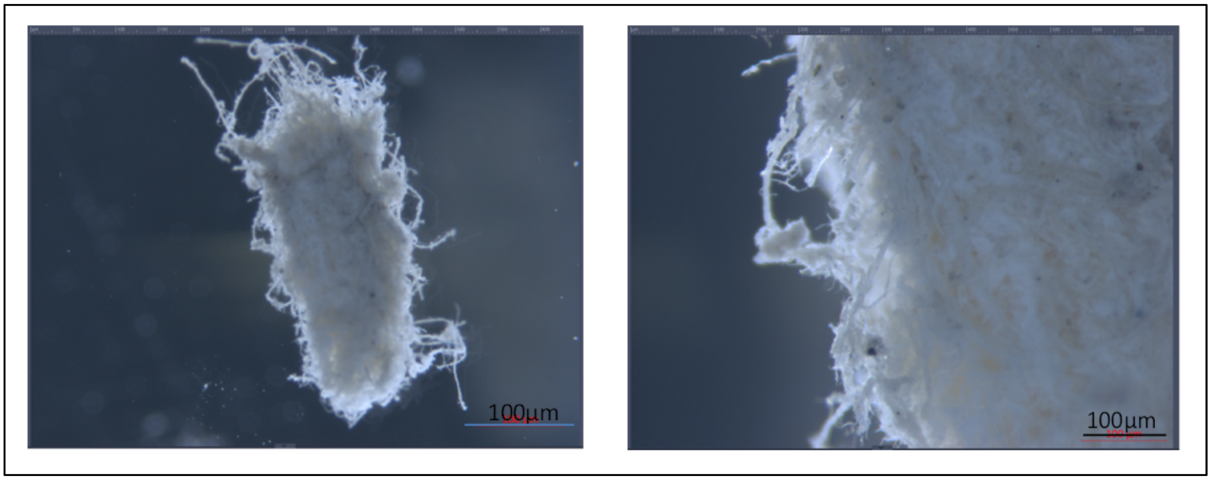

Figure 7.2 Pulp Aramid Fiber (PAF)

Table 7.1 Pulp Aramid Factory Specifications
Taken from DuPont (2015)

Property	**Specifications**
Specific Gravity	1.44-1.45 at 20 (°C)
Color	Gold
Water solubility	Insoluble
Moisture, %	45% - 55%

For this project, a dense-graded 20 mm HMA commonly used as a base course in Quebec (GB20), was designed with a 4.5 percent binder (PG 64-28). The aggregate size and gradation were selected based on LC method specifications (seeFigure 7.3).

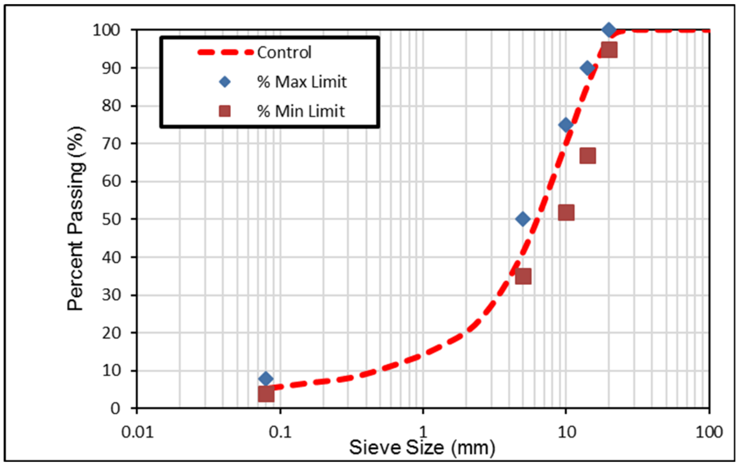

Figure 7.3 Aggregate Gradation

7.5 Methodology

Little appears in the literature regarding mix design procedures with fibers. In general, the mix design proceeds as usual with compaction, but the major issues with PAF include how much fiber needs to be added to the mix, how it can be added to the mix, and what the optimum asphalt content should be, based on the specific PAF amount.

All mix design methods use density and voids to determine the basic physical characteristics of the HMA. Here, the theoretical maximum specific gravity (Gmm) as per AASHTO T 209 and ASTM D 2041 was determined in order to find the needed mass for gyratory compactor testing. Various trials are needed to find the Optimum Binder Content (OBC) according to the desired air voids. These densities are then used to calculate the volumetric parameters of the HMA. Measured void expressions usually include:

- Air voids (Va), sometimes expressed as Voids in the Total Mix (VTM);
- Voids in the Mineral Aggregate (VMA);
- Voids Filled with Asphalt (VFA).

These values must meet certain specific criteria before an acceptable mix design can be obtained.

These mixes are also evaluated in terms of stability against the tensile strain. The values of Indirect Tensile Strength (ITS) were used to evaluate the relative quality of bituminous mixtures in conjunction with laboratory mix design testing, and for estimating the potential for cracking. In this study, specimens with a 150mm diameter were compacted using the Superpave Gyratory Compactor (SGC), and the ITS was calculated according to Equation 7.1.

$$S_t = \frac{2000 \times P}{\pi \times h \times D} \tag{7.1}$$

Where:

St is Indirect Tensile Strength (ITS) in kPa;

P is the maximum load in N;

h is the specimen height immediately before test in mm;

D is the specimen diameter in mm.

7.6 Results

To see the impact of the fiber alone in the mix, the binder content was initially kept constant. As mentioned earlier, there are two ways fibers can be introduced to the mix. However, the literature does not mention whether or not a correlation exists between the length of the fiber and the mixing process. Because we used a special type of fiber, which was very short and tiny, we faced the problem of achieving good distribution of the fiber in the mix.

Figure 7.4 shows a blend of aggregate and PAF in a dried mix procedure. The PAF did not distribute uniformly but formed into a mass in some areas of the sample due to static charges on the fibers. To solve this problem, water was pre-added to the fiber and the suspended fiber was then added to the aggregates. After that, the aggregates were left to dry for 24 hours in the oven at a mix temperature.

Figure 7.5 shows that immersed fiber can facilitate a homogeneous mix, however, this type of mixing is not environmentally friendly and increases the cost of the mix given the high amount of energy and time needed to dry the fibers.

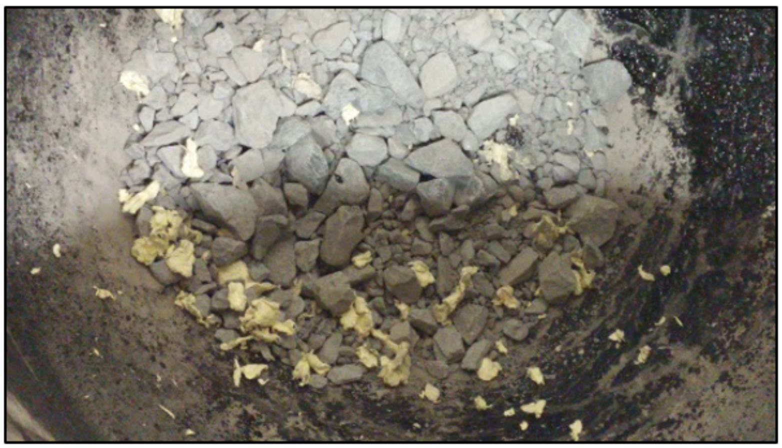

Figure 7.4 Dried Mix Process (note clumping of fibres)

Figure 7.5 Immersed Fibre Mix Process (homogeneous mixture)

In order to ensure that the manufacturing process is environmentally friendly, we tried another solution. PAF was blended with the filler in a small mixer for 2 to 5 minutes, depending on the filler and fiber content, and then added to the aggregates. The trial was successful, and this type of blending was found to be the most energy-efficient way of spreading PAF and filler in between particles as shown in Figure 7.6.

Figure 7.6 Homogenized blending of fibers with mineral filler before addition to aggregates

The PAF and aggregates were preheated for 2 hours at 170ºC prior to being mixed with the binder. The mix temperature was 155ºC, which is in keeping with the Quebec standard for this type of binders. Figure 7.7 shows 23 mixes, with 7 different combinations and repetitions of fiber with aggregate and binder. Broken down, 2F0.3%2 means second type of mix with 0.3 percent (weight of total mix) of fiber and second repetition. Various PAF contents from 0.1 to 2 percent were added to the mix in order to see the impact of fiber on volumetric properties.

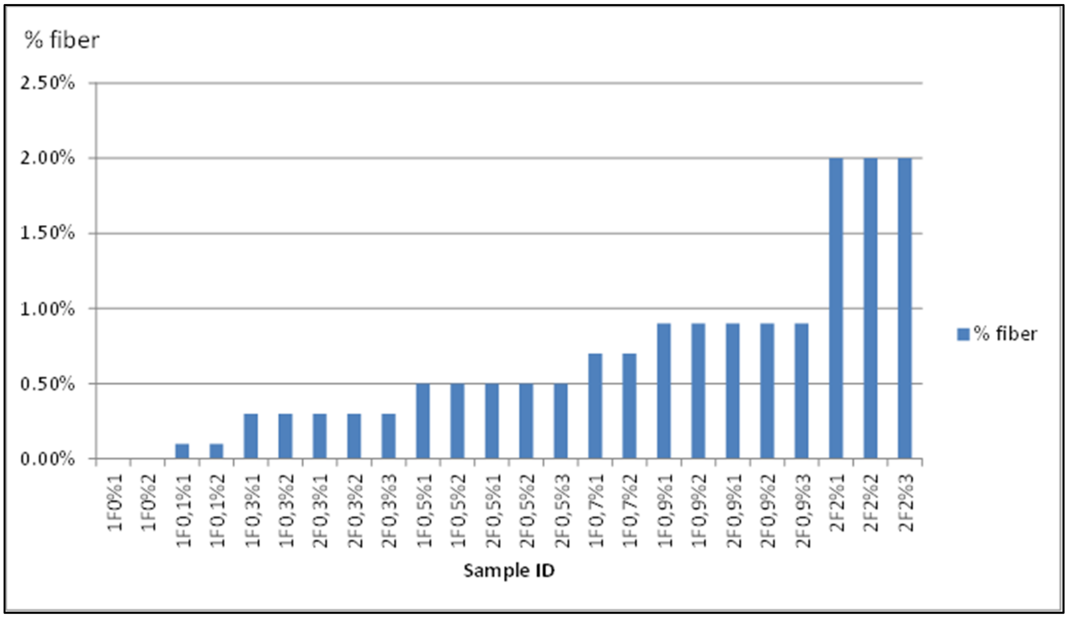

Figure 7.7 Different Combinations of Fiber

The theoretical maximum specific gravity (Gmm) is a critical mix property as it is used to calculate the percent of air voids in compacted HMA. This calculation is used both in Superpave and LC mix designs and in determining the in-place air voids in the field. Figure 7.8 indicates that added fiber decreased the Gmm of the mix. Additional fiber could make the mix lighter than the control mix, making the latter cost-effective. More research is needed though, to see if the filler can be completely replaced with fiber in order to reduce the weight of the structure, and hence, the quantity of material used. Type 1 corresponds to the mixes where the fiber is added directly to the aggregate and Type 2 corresponds to mixes where the fibre is blended with the filler in advance then added to aggregate.

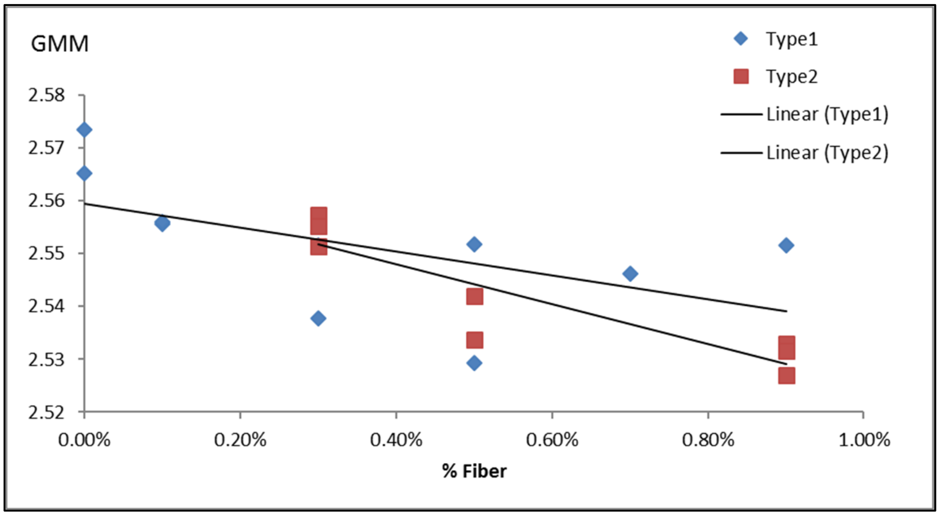

Figure 7.8 Theoretical Maximum Specific Gravity (Gmm)

Cylindrical samples with 75mm height and 100mm in diameter were compacted using a SGC to determine their stability. An ITS test was carried out for each single mix. Prior to the ITS testing, air void content was determined. Figure 7.9 shows that the air void content was increased by increasing the fiber content. This would potentially mean that PAF pushes and separate the aggregates, creating additional voids between the aggregates as more fiber is added to the reference mix with constant binder content for all mixes.

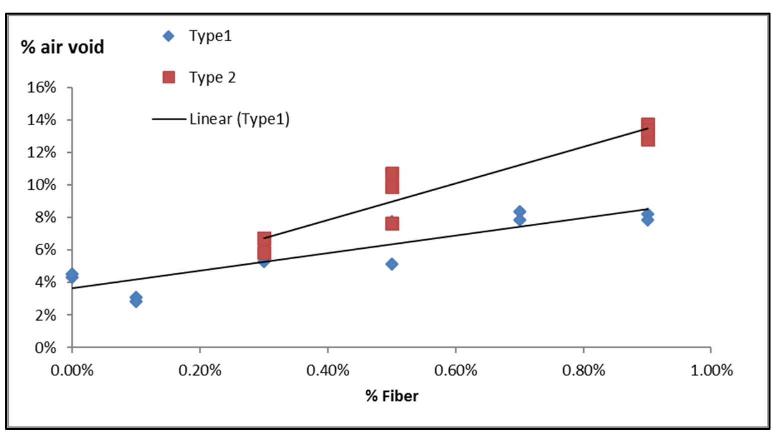

Figure 7.9 Air Void Content according to Fiber Content

Several researchers have used ITS values for cracking evaluation (Hansen , 2000; Bennert, 2012). In our case, PAF mixes were subjected to a compression load rating of 50 mm/min. Various PAF contents were used without extra binder content added, in order to figure out the impact of PAF on volumetric properties. Obviously, fiber absorbs binder, and as a result, when fiber is added, extra binder should be added as well in order to properly coat the aggregates and fiber in the mix. At this step, the impact of fiber was studied without extra asphalt cement used.

Figure 7.10 compares tensile strength results for the mixes. The behavior of the material can be divided into two phases, namely, before maximum strength is achieved and after maximum strength is achieved. Increasing the fiber content in the mix led to a change in the behavior of the material after it reached its maximum tensile resistance capacity. Figure 7.10 displays the strength of the specimen over time. It can be seen that the slope and area under the curve changed as the PAF content was increased; the area under the curve increased and the slope of the PAF mixes was flatter than that of the control mix. This clearly indicated that the behavior becomes more ductile and that the fibers help in holding the sample for a longer time before ultimate failure. Moreover, it can be seen that the maximum resistance is decreased as the fiber content increases. This is because when the binder content is insufficient, the mix becomes weaker; a proper mix design is therefore required in order to find the optimum binder content according to chosen fiber content.

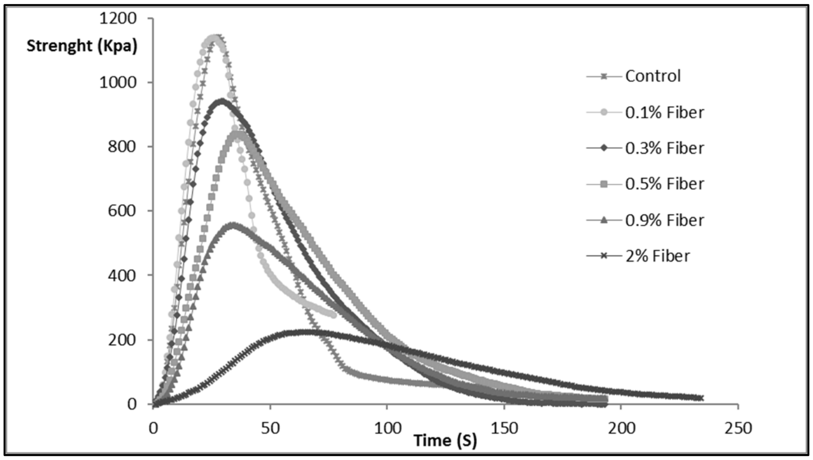

Figure 7.10 Indirect Tensile Strength Test Results

A review of the literature indicates that the fiber content in a mix depends on the physical and chemical properties of fiber, and consequently, the optimum binder content is determined from the fiber content. A review of the literature showed that a 0.1 to 0.7 percent fiber content has been used in HMA. We have the decided to use 0.3 percent as a starting point. The mix was designed to calculate the optimum binder content of the desired added PAF. Figure 7.11 shows the volumetric properties of the PAF samples. Different binder contents were added to the mix, and air void contents were determined. A 5.5 percent binder content was chosen in order to have 6.5 percent air voids, which is the same quantity as the control mix air voids.

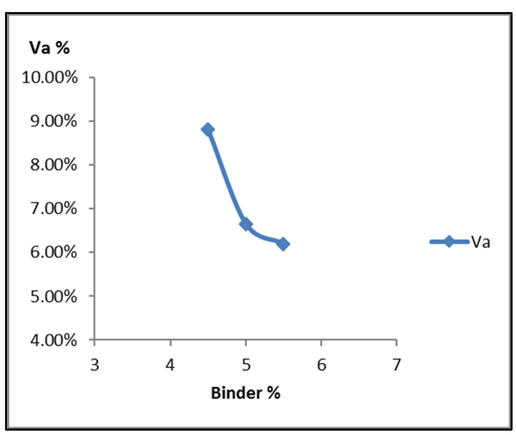

Figure 7.11 Change in Air Voids (Va) with Binder Content

Two different temperatures (-18 and 0ºC) were chosen in order to see the impact of the fiber in cold climates. Indirect tensile strength testing was performed for each single PAF mix and for the control mix, and the results are shown in Figure 7.12.

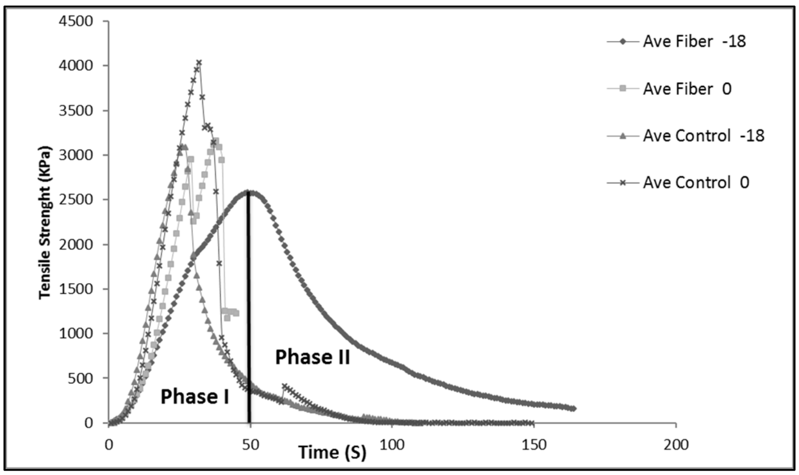

Figure 7.12 Indirect Tensile Strength at -18ºC and 0ºC

As can be seen in Figure 12, after reaching the maximum tensile strength, the control mix broke down completely and the value of the tensile strength dropped very rapidly at both testing temperatures (0 and 18°C). However, even though the value of the tensile strength was lower than that of the control mix at both testing temperatures, the tensile strength values decreased progressively after reaching the maximum and the PAF mixes showed better resistance to physical failure. This difference in the behavior can be attributed to the presence of the fiber, which contribute to improving the ductility of the mix at low temperature. This higher ductility could lead to higher resistance to crack propagation.

In order to quantify the difference in the behavior between control and PAF mixes, the area under the ITS curve was used. This parameter indicates the capacity of the specimen to bear the load until breaking occurred. The Strength-Time Area (STA) factor is the integral of a force

(F), over the time interval (t), and can represent the ductility, and indirectly the toughness of the specimen, over the test period. Equation 7.2 is the formula used to determine the STA.

$$\text{STA} = \int_0^{t_1} S_t dt = \int_0^{t_1} \frac{2000 \times P}{\pi \times h \times D} dt \qquad (7.2)$$

Table 7.2 shows the values of STA phases I and II according to Figure 7.12. The values show that the fiber mixes have higher STA values in both phases.

Table 7.2 Area under the curve

Mix ID	**STA Phase I kPa/s**	**STA Phase II kPa/s**	**Total STA kPa/s**	**Binder Content**
Control Mix -18 ºC	16,580.84	18,382.44	34,963.28	4.5%
Fiber Mix -18 ºC	32,813.18	49,304.93	82,118.10	5.5%
Control Mix 0 ºC	25,729.07	18,969.19	44,698.26	4.5%
Fiber Mix 0 ºC	26,793.36	19,853.59	46,646.95	5.5%
Control Mix 25 ºC	33,001.99	67,027.47	100,029.46	4.5%
Fiber Mix 25 ºC	7,381.85	20,446.65	27,828.50	5.5%

Table 7.2 also shows that when the control mix becomes fragile, it has a lower STA value, but that the STA values of the PAF mixes increased faster than the control mix as the temperature decreases. This means that at higher temperatures, the impact of fiber is not negligible, but the impact of PAF is mostly significant at lower temperatures, where it could improve the STA of mix to two times higher than control mix. At high temperature, the lack of added binder becomes evident.

7.7 Conclusions

The performance and the durability of flexible pavements is highly governed by the quality and the performance of the asphalt mixes used in the bound layers of the pavement structure. Nowadays, it is common to use several types of additives to asphalt binders and asphalt mixes to improve their performance. The objective of this research project was to investigate the use of potential of using Aramid Fiber to improve the performance of asphalt mixes.

Several experiments, including mix design, ITS testing and determination of the physical properties of the mix, with the PAF included, were carried out. Based on the results of the experimental study, the following conclusions were withdrawn:

- Pulp Aramid Fiber (PAF) is a short fiber. It has higher surface area, and consequently, absorbs more asphalt cement. For HMA with PAF, higher binder content is needed versus the conventional mixes. In this project, adding 0.3 percent fiber content to the HMA required an extra 1 percent of additional asphalt cement;

- Because of the special characteristics of PAF, such as a high static energy and a very short fiber length, it was difficult to blend the fibers with aggregate properly and uniformly. It is recommended to blend fiber with filler in advance before adding the mix to aggregates. Visual observations showed that this method produced a much more homogenous mix as compared to conventional mixing methods;

- PAF produced mixes with higher air voids and provided a higher optimum asphalt content as compared to the control mixes. Further investigation is needed to investigate the accuracy of this finding;

- In the long term, PAF may have a higher resistance even after reaching its maximum strength capacity. The ITS results indicate that although the control mix has a higher tensile strength, the mixes with PAF keep some strength for a longer period than the control mix;
- The analysis of the Strength-Time curves of the different mixes showed that PAF mixes are more ductile at lower temperatures than the control mix indicated a better resistance to cracking and delayed crack propagation in the mix.

This study indicated that the use of PAF would enhance the performance of asphalt mixes at low temperature. Further investigation is needed to study the impact of using PAF on air voids in the mix and on mix design.

CHAPTER 8

CHARACTERIZATION OF ASPHALT MIXTURES PRODUCED WITH SHORT PULP ARAMID FIBER (PAF)

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8.1 Abstract

Use of additives and modifiers as an effective alternative to mitigate premature cracking of flexible pavements has been gaining popularity during the past three decades. Several studies have discussed the advantages and disadvantages of using various types of fibers in pavement industry. The objective of this study is to evaluate the impact of using Pulp Aramid Fiber (PAF) in Hot Mix Asphalt (HMA) in terms of mix design and performance. Aramid fibers or aromatic polyamide fibers were the first type of organic fibers found to have sufficient tensile strength and modulus to be used in advanced composites. Although studying the effect of additives on performance of HMA mixes has been the focus of several studies, the extent of work on the use of PAF in HMA mixes using a comprehensive methodology has been limited. A control mix and a PAF modified HMA mix were designed and evaluated with respect to their rutting resistance, fatigue cracking resistance, and low temperature cracking performance. The results indicate that mixes with PAF led to higher demand for bitumen content to achieve the same air void level as compared to the control mix. However, it was concluded that this additional binder content along with the presence of PAF resulted in an enhanced performance, especially with respect to fatigue cracking as well as improved ductility at cold temperatures. In spite of the additional binder content, the PAF modified mix exhibited satisfactory permanent

deformation performance at high temperatures while considerably improving the fatigue and low temperature behavior of the mix. The results also indicate that PAF can delay cracking failure in long-term life of asphalt concrete mixes under repeated loading.

Keywords: Hot Mix Asphalt, Pulp Aramid Fiber, Complex Modulus, Fatigue Cracking, Permanent Deformation, Low Temperature.

8.2 Introduction

Bituminous flexible pavement is the most prevalent type of pavements in North America. In Canada, which ranks seventh in the world with respect to the length of its road network PIARC-Canada-national report (2015), about 95% of the paved roads are surfaced with asphalt concrete materials. According to the Canadian Infrastructure Report Card (2012), more than 50% of the municipal roads are in fair, poor or very poor conditions. Although similar condition data could not be found for the roads managed by the different departments of transportation, agency level network is believed to have a fairly better condition, in general. For example, Transport Quebec stated that only 21% of its road network was not in good conditions (www.transports.gouv.qc.ca , 2013). Degrading pavement performance can directly affect the transportation quality, leading to considerable economic impacts. Therefore, several sponsored research activities have been focused on finding bituminous materials of better durability, especially during the past five decades. Fibers can be considered as one of the possible additives for this purpose. A review study on the use of fibers to pours asphalt mixes showed that impact of fibers on pavements performance depends on the nature of the fibers used. Moreover, the combination of additives such as different types of fibers or fiber and rubber could have synergistic benefits for enhancing flexible pavements performance (Gupta , Rodriguez-Hernandez, Castro-Fresno, 2019). Results of studies on the mechanical properties of fiber reinforced asphalt mixes indicate that each specific type of fiber typically improves certain aspects of bituminous mixes (Slebi-Acevedo, Lastra-González, Pascual-Muñoz,Castro-Fresno, 2019). To this end, use of polymers and fibers have been studied by different researchers as two popular options to improve the performance of asphalt concrete mixes. While polymer modification has been extensively investigated by several studies, research on fibers has seen relatively less attention, and hence was deemed necessary in this study. Fibers can play the role of reinforcement element in a pavement structure to enhance the bearing capacity of roads against the applied tensile and compressive loads during their service life. A synthesis of the pertinent work on different aspects of fiber modification of asphalt mixes is provided in this section.

8.2.1 Fiber additives in bituminous pavements

Historically, use of fibers to improve the properties of bituminous materials can be traced back to ancient Egyptian buildings were straws were added in asphalt, according to Button and Epps (Button, 19821). Various types of fibers have been tested by different researchers in the past. However, the first documented indication of fiber modification of bituminous materials dates back as early 1900s (Kietzman, J. H., Blekicki , 1960). In this documented case, the addition of fiber was aimed to prevent bleeding, and to stabilize the bituminous mixture. Although there is no recognized consensus about fibers application in Hot Mix Asphalt (HMA), utilizing fibers in HMA is believed to be an environment-friendly approach to increase the service life of pavements, especially by improving their proneness to tensile stresses. Bituminous materials have more bearing capacity in compression than in tension (Busching, Elliott, Reyneveld, 1970). If incorporated properly, use of high tensile strength fibers can contribute to increasing the overall tensile strength, which would ultimately result in a longer-lasting material. More precisely, Peltonen (Peltonen , 1991) presented evidence which supports that the use of fibers increases mix resistance to cracking, reduces the severity of cracks, and increases resistance to fatigue cracking and rutting.

Fibers used in polymer engineering of composites can be divided into two main categories, namely synthetic fibers and natural fibers. Synthetic fibers including glass, carbon and aramid fibers are the most commonly used types of fibers for HMA reinforcement applications (V. Wigotsky , 2002). According to the NCHRP Synthesis on fiber additives in asphalt mixtures (McDaniel, 2015), the fibers can be generally classified into seven different types: cellulose, mineral, polyester, polypropylene, Aramid, Aramid and polyolefin, and fiberglass. In addition, some studies indicate that use of a hybrid fiber, composed of different fiber types, would offer benefits with respect to rutting resistance and results in less drain down (Tanzadeh, & Shahrezagamasaei , 2017). Reducing the drain down rate is known as a common advantage of using fibers in flexible pavements (Tanzadeh, Tanzadeh, Honarmand, Tahami, 2019) In addition, when utilized properly, fibers have shown considerable effects on the mechanical properties of asphalt concrete materials. Considering the temperature effects, for the areas with higher expected pavement temperatures, a higher fiber content may be needed to adequately improve the mechanical characteristics of asphalt concrete mixes (Lavasani, Latifi Namin, Fartash , 2015). All types of fibers have somewhat proven their usefulness, but for almost all of them some disadvantages have been reported in the literature. Examples include high binder absorption, relatively high specific gravity, low melting point, and moisture sensitivity. For the Aramid fibers, however, no significant disadvantages have been reported, except the fact that the cost effectiveness has yet to be proven.

The physical properties of fibers (e.g., diameter and length of strands) have a significant impact on their interaction with and interlocking between HMA components, which can ultimately affect the performance of fiber modified mixes. As an example, length of the fibers could be modified according to the maximum size of aggregates in a mix. Long fibers may be difficult to mix uniformly into the mixture either in the lab or in plant production because they can get entangled and clump together (Abtahi, Sheikhzadeh, Hejazi, 2010). Long fibers also can create the balling problem which would ultimately sacrifice the mix uniformity. On the other hand, even if short fibers have been investigated to a lesser extent, they can play the role of filler in mix design (Wigotsky, 2002). Some studies have reported that the use of 19-mm long fibers can result in better resistance of the mix to shear deformation. Furthermore, it is indicated that such reinforcement could lower accumulated permanent strain while improving fatigue cracking as compared to the conventional HMA (patents.google.com , 2020).

Regardless of the fiber type, the maximum length of fiber should be investigated to achieve the optimal performance. Fibers which are too long can result in clumping during the mixing process (Do Vale, Casagrande, Soares, 2013). While maximum length is generally considered as a factor in selecting the fiber size, too short fibers may not provide significant contribution to the matrix structure. Clumping might happen in mixes with inclusion of fiber either during or after compaction even with short fiber. Figure 8.1 shows an example of clumping in HMA mixes. Longer blending duration has been suggested to achieve uniformity and a more homogenous mix (Watson , 2003). Kaloush et al. (2010) also have shown that the use of 19mm long mix of polypropylene and Aramid fibers as a feasible option to enhance the rutting and fatigue resistance, and did not report any issues with mixing process. Studies on short fibers and minimum required length are very scarce. Nonetheless, surface area and absorption rate of fibers can also change the optimum binder content in the mix (Button & Lytton, 1987). Therefore, physical properties of the fibers must be thoroughly considered as a part of the design process.

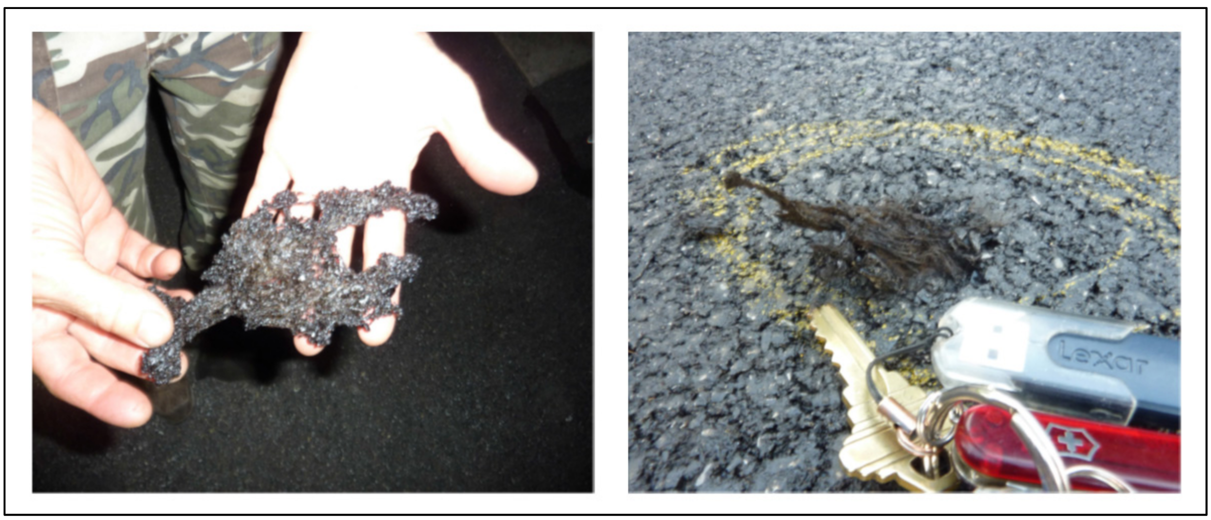

Figure 8.1 Clumping during and after compaction
Taken from McDaniel (2015)

8.2.2 Pulp aramid fiber (PAF)

None of the studies mentioned above were performed using Pulp Aramid Fiber (PAF), which consists of tiny and short fibers. Fiber reinforcement provides additional tensile strength, alleviates crack propagation, and increases strain energy absorption in bituminous mixes

(Busching, H. W., Elliott, E. H., Reyneveld , 1970) Resistance of Aramid fibers to tensile loading is approximately five times higher than that of steel. This fiber was originally used as a replacement of steel in radial tires (Wigotsky, , 2002). Aramid fiber is defined as a manufactured fiber where the fiber forming material is a long-chain synthetic polyamide with at least 85% of the amide linkages attached directly to two aromatic rings (www.globalspec.com , 2019). DuPont introduced aramid fibers for the first time in early 1970s. They introduced DuPont™ Kevlar® aramid pulps as highly fibrillated chopped fibers that can be used as specialty additives to enhance performance by helping to provide reinforcement and viscosity control under shear stress. This type of Aramid pulp is typically smaller than 1mm in length, and is commonly used in automotive brake pads, gaskets, and automatic transmission papers applications (Dupont, 2015), as well as a viscosity control additive for adhesives and sealants (Dupont, 1999).

8.2.3 Methods of fiber modification for bituminous materials

Similar to polymer modification practices, fibers can also be introduced into HMA by means of two methods, namely wet- and dry-process. In the former, the specified amount of fiber is blended with asphalt prior to mixing the asphalt binder with aggregates. Whereas in the dry-process, fiber is blended with aggregates and then binder is introduced into the mix in a conventional manner. Generally, dry process is preferred over the wet process because the mixing process is more convenient and typically a better fiber distribution can be achieved. On the other hand, since most of the fibers used for this purpose do not melt in asphalt at the mixing temperature, there would be no apparent benefits to employ the wet process. Furthermore, majority of the field work done on fiber-reinforced asphalt mixtures has generally utilized the dry (Echols, 1989; Hejazi, 2007; Munn, 1989), most probably due to the production problems of directly introducing the fibers into asphalt. Using the proper mixing duration, dry process can minimize the risk of fibers clumping or balling of fibers in the mixture (Labib & Maher , 1999). Therefore, given the inherent advantages of dry method for fiber modification, and the fact that the results can be better compared with the historical data, this method was selected for the experimental part of this research.

8.2.4 Research goals, scopes, and objectives

Review of the literature on the application of fibers in modifying asphalt concrete materials indicates that there are research gaps on several aspects of short Aramid fibers contribution to HMA performance. Therefore, it is difficult to come up with a synthesis of past studies to validate the impact of short pulp fibers in HMA, especially in cold regions. In summary, use of fibers is generally believed to be a promising approach to enhance HMA performance over the life cycle of pavements. However, such improvement is significantly dependent on fiber type and characteristics. Therefore, the goal of this project is to evaluate the properties of Pulp Aramid Fibers (PAF), as a merely investigated fiber type in asphalt modification, through the use of advanced material characterization tests. The main research hypothesis is that PAFs extend the flexibility, glassy stiffness, and ductile behavior of HMA to lower temperatures. Therefore, the experimental program was designed to included mix design, complex (dynamic) modulus, repeated load permanent deformation, and fatigue tests. The results were used to compare the performance of the fiber-modified and the control mixtures. To this end, various empirical and thermomechanical tests are employed to evaluate the impact of short Aramid fibers on the HMA mix behavior.

The main objective of this research is to evaluate the impacts of PAF modification at the optimum dosage on the properties of HMA mixes. Generally, PAF is expected to reinforce the bitumen structure through absorbing a portion of the available binder. However, no clear evidence has been found in the literature to indicate the extent of performance enhancement due to this mechanism. Presence of such extra amount of binder (as compared to the control mix) may potentially enhance the pavement resistance to cracking. Previous studies showed that reinforced HMA with fiber additives increases the resistance of mixes against tensile stresses under traffic loads. Therefore, it was deemed necessary to further investigate the mix design and performance of the mixes modified with short synthetic fibers using a relatively comprehensive experimental approach.

8.3 Materials and experimental methods

8.3.1 Methodology

Generally, limits have been set on the maximum allowable amount of fibers in HMA mixes to avoid the loss of performance due to the impact of more binder content and interaction of fibers with aggregates. Although, many aspects of fiber incorporated mixes have been investigated by several preliminary studies, the effect of short aramid fiber content has not been fully explored. Therefore, in this study it was hypothesized that Pulp Aramid Fiber (PAF) can enhance the aggregates interlocking and improve the tensile strength of the underlying layers to protect the pavement structure from cracks caused by tension or low temperature cycles. The results can be used to identify the functional class for proper use of PAF mixes in pavement structural design and in different layers. It was assumed that all of the PAF would contribute to the mix.

Four major tests were utilized to investigate the mixes in this study. The selected tests characterize the mixes with respect to the prevalent domains of failure of the pavement materials, i.e., high temperature performance, low temperature cracking, fatigue cracking, and permanent deformation. These tests can be divided into two categories of empirical and thermomechanical tests. Resistance to rutting through MLPC wheel tracking rut tester (or French Wheel Tracking Test) is used in this study as the empirical test which can simulate the permanent deformation behavior of asphalt concrete layers in the laboratory scale. The thermomechanical tests utilized in this research are:

- Complex (dynamic) modulus test (Di Benedetto, Nguyen, Sauzéat, 2011),
- Uniaxial Tension- Compression test for resistance to fatigue cracking (Di Benedetto, Nguyen, Sauzéat, 2011), and evaluation of modified mixes resistance to low temperature cracking through the Thermal Stress Restrained Specimen Testing (TSRST) was the objective of another study and has been published elsewhere (Badeli, Carter, Doré , Saliani, 2018). In this study, the authors explored an additional ductility measure based on

fracture mechanics theory through running Indirect Tensile Strength (ITS) tests on the compacted asphalt concrete specimens at cold temperatures.

8.3.2 Materials

The fiber used in this work is Aramid fiber that was provided by Kevlar DuPont, called KEVLAR pulp which is categorized in 1F361 (see Figure 8.2). Table 8.1 shows the general specifications of 1F361 fiber used in this study.

Figure 8.2 Pulp Aramid Fiber

Table 8.1 Pulp Aramid Fiber Specifications

Property	**Form**	**Specific Gravity**	**Color**	**Water solubility**	**Moisture [%]**
Specification	Pulp	1.44-1.45 at 20 (°C)	Gold	Insoluble	45% - 55%

The aggregate size and gradation were selected based on the LC specifications by the General Directorate of the Pavement Laboratory of the Ministère des Transports du Québec (MTQ), which presents the test methods used to measure the characteristics of materials used in the construction and maintenance of infrastructures. The virgin asphalt concrete mix without any fiber modification will be referred to as the Control Mix hereafter in this paper. In order to obtain a desired aggregate gradation, combination of aggregates from various aggregate stockpiles are used according to Table 8.2. This table shows the gradation of stockpiles and the mix gradation. Only one source of virgin aggregate, known as DJL from St. Phillips in Montreal, was used in this study to eliminate the impact of different material sources. A single source of performance graded asphalt binder, i.e. PG 64-28, was used in this study. The selected virgin binder is a medium grade asphalt binder that can be used in warm climates.

Table 8.2 Aggregate Gradations and specification

Sieve (mm)	**Filler**	**0-5**	**5-10**	**10-14**	**14-20**	**MTQ Specification**	**Mix Gradation**
28	100%	100%	100%	100%	100 %	100	100%
20	100%	100%	100%	100%	84%	95-100	97%
14	100%	100%	100%	89%	12%	67-90	85%
10	100%	100%	91%	28%	2.61%	52-75	70%
5	100%	93%	4%	8.1%	2.0%	35-50	41%
2.5	100%	48%	1%	3.7%	1.7%	0	23%
1.25	100%	28%	1%	1.9%	1.5%	0	15%
0.63	100%	16%	1%	1.4%	1.4%	0	11%
0.315	99%	7%	1%	1.3%	1.3%	0	8.2%
0.16	93%	4%	1%	1.2%	1.2%	0	6.8%
0.08	79%	2.3%	0.4%	1.1%	1.1%	4.0-8.0	5.3%
<0.08	-	-	-	1.1%	1.1%	-	0.4%
G_{sb}*	2.700	2.705	2.710	2.719	2.743	10.2	-
P_{wa}**	0%	0.65%	0.59%	0.49%	0.33%		
* P_{wa}: Water absorption ** G_{sb}: Bulk specific gravity							

8.3.3 Mix design

As the first step of this study, a control mix was designed and prepared using virgin mineral aggregates and a virgin asphalt binder. The control mix had a 20 mm nominal maximum aggregate size to produce a dense graded HMA, called Grave Bitumen (aka GB20), which is mostly used in binder course layer of pavements in Quebec. The design binder content using a PG 64-28 was determined, in general accordance with the Superpave^{TM} mix design method, found to be 4.5% by the weight of the total mix. Additionally, another mix with inclusion of PAF was also designed for comparison and validation purposes. Different fiber contents were used to achieve the target air void level of 4%, resulting in a new optimum asphalt binder content for the PAF modified mix.

In general, a desired mix design for the PAF modified mix should yield the same volumetric properties as those of the control mix without requiring additional compaction effort. Therefore, the revised optimum binder content for achieving the target air void level of 4% under the same number of gyrations was determined in this step. As PAF has certain physical and chemical characteristics, the mix design process should be thoroughly evaluated to determine the optimum fiber content. In this study, the PAF content was determined with respect to the target air void level, the effective bitumen content, and the mix indirect tensile strength at lower temperatures.

To properly evaluate the impact of fibers in the mix, the binder content was initially kept constant, just varying the fiber content. There has not been a clear vision as for the correlation between the length of the fiber and the mixing process in the literature. Wet and dry process were proposed in order to add PAF to HMA. As discussed earlier, the dry process is generally the preferred method, as it is experimentally simpler and allows for a better fiber distribution in the mixture. Therefore, dry process was employed throughout this study.

Initially, the fiber content was varied from 0.1% to 2% in order to study its impact on the following volumetric measures:

- Theoretical maximum specific gravity (TMD or Gmm) per AASHTO T 209 and ASTM D 2041;
- Air voids (Va) also known as voids in the total mix (VTM);
- Water absorption.

As a result, a fiber content of 0.3% was determined to be the optimum PAF content in the mix. Following to the preliminary verification of fiber content, two mixes were prepared with and without the selected PAF content. The goal was to design the experimental mix to meet the target air void level while using a binder content as close as possible to the control mix. Binder contents of 4.5% and 5.5% were determined as a result of the mix design process for the control and PAF mixes, respectively. The specimens were prepared following to the LC 26-400 using the LCPC slab compactor equipment. The laboratory compacted cylindrical specimens were cored horizontally from the compacted slab as per Quebec Standard Specifications and then stored for a minimum of one month at room temperature in a sand bed prior to testing. Mechanical tests, including fatigue, rutting, thermal cracking, and complex modulus were performed on cored specimens extracted from slabs as shown in the schematics in Figure 8.3. Samples were compacted by the French MLPC wheel compactor (Figure 8.4).

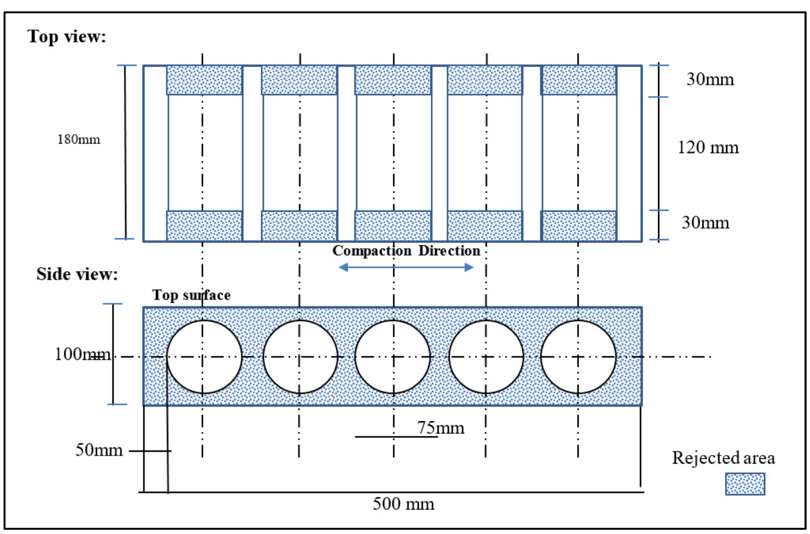

Figure 8.3 Graphical schematics of slab compaction and details of coring of specimens

Figure 8.4 View of the LCPC slab compactor

8.3.4 Thermo-Mechanical tests

For the purpose of HMA performance evaluation, a battery of thermomechanical characterization tests was carried out in this study. Uniaxial fatigue test and TSRST, were performed by means of a 25 kN servo-hydraulic system capable of applying sinusoidally varying loads in tension and compression modes. Figure 8.5 shows schematic of the test setup as well as extensometers configuration surrounding a cylindrical specimen. Three extensometers are used to measure the axial strain changes during the test. The chamber and specimen temperatures were monitored by means of three temperature probes attached to the surface of the specimens. The chamber temperature was controlled in the range of -40 ºC to 80oC according to the test specifications. More detailed description of the tests is provided in the following sections.

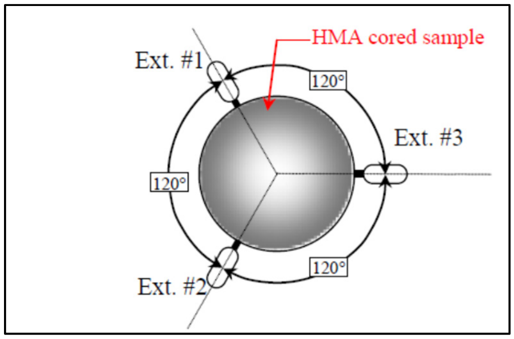

Figure 8.5 Schematics (plan view) of instrumentation in the test set-up used in this study

Complex modulus and indirect tensile strength: The mixes designed in this study were evaluated in terms of their stability against the indirect tensile loading as well as their viscoelastic properties through the use of complex modulus testing. The values of indirect tensile strength (ITS) were used to evaluate the relative quality of bituminous mixtures in conjunction with laboratory mix design testing, and for estimating the cracking potential. To this end, disk.shaped specimens of 150mm diameter were compacted using the Superpave Gyratory Compactor (SGC). The specimens were then loaded in the diametral manner and the Indirect Tensile Strength values were calculated using Equation 8.1:

$$S = \frac{2000 \times P}{\pi \times h \times D} \tag{8.1}$$

where:

St = Indirect Tensile Strength, kPa;

P = maximum load, N;

h = specimen thickness immediately before test, mm;

D = specimen diameter, mm.

On the other hand, complex modulus measurements were carried out using a uniaxial Tension-Compression (T-C) test performed on cylindrical specimens of 120 mm in height and 75 mm in diameter. The tests were done at 7 different temperatures (i.e., -35, -25, -10, 0, 10, 20, and 35 °C) and 7 different frequencies (10, 3, 1, 0.3, 0.1, 0.03, and 0.01 Hz). The materials' responses (E*) were then modelled using the 2S2P1D model, which is a generalization of the Huet–Sayegh model. 2S2P1D model was developed at the ENTPE/DGCB laboratory in France to simulate the experimental results of complex modulus test on bituminous materials, and is based on a simple combination of physical elements (i.e., springs, dashpots, and parabolic elements). The introduced 2S2P1D model has seven constants as illustrated in Figure 8.6. At a given temperature T, the seven constants of the 2S2P1D model can be determined. E* can be calculated according to Olard and Di Benedetto (2003) using Equation 8.2:

$$E^*_{(\mathrm{i}\omega\tau)} = E_{00} + (\mathrm{E}_0 - \mathrm{E}_{00})/(1 + \delta\,(\mathrm{i}\omega\tau)^{(-\mathrm{k})} + (\mathrm{i}\omega\tau)^{(-\mathrm{h})} - (\mathrm{i}\omega\beta\tau)^{(-1)}) \tag{8.2}$$

where:

h and k are exponents such that $0 < k < h < 1$;

E_{00} is the static modulus obtained when $\omega\tau \to 0$ (at low frequencies and high temperatures) with $\omega = 2\pi f$ where f = the linear frequency;

E_0 is the glassy modulus when $\omega\tau \to \infty$ (at high frequencies and low temperatures) in MPa;

τ is the characteristic time, which varies only with temperature, accounting for the time–temperature superposition principle (TTSP);

η is the Newtonian viscosity defined as: $\eta = (E_0 - E_{00})\,\beta\tau$.

When $\omega_\tau \to 0$, the $E^*(i\omega t) \to E00 + i\omega\,(E_0 - E_{00})\,\beta\tau \times \beta$.

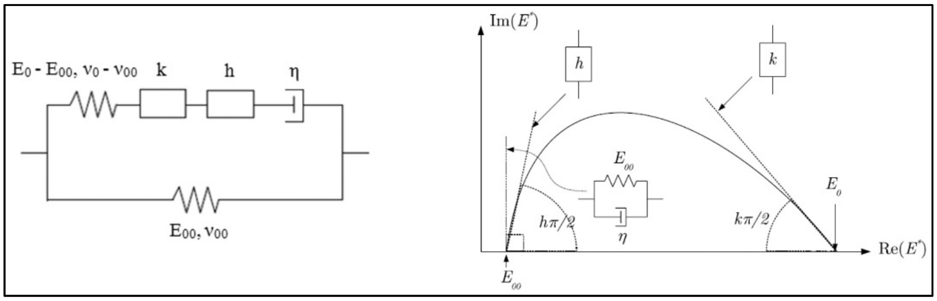

Figure 8.6 2S2P1D schematic model and its elements
Taken from Diego Alejandro Ramirez Cardona et al. (2015)

Fatigue Cracking Resistance: Fatigue characterization was carried out under uniaxial Tension–Compression (T-C) loading on cylindrical specimens in this study. The uniaxial T-C test setup is almost the same as the complex modulus test. However, in this test, only a single loading frequency (i.e., 10 Hz) and testing temperature (i.e., 10 °C) are used. The experimental setup maintains the same level of stress (or strain) during the test, as compared to the conventional non-homogeneous fatigue test. In this study, a strain-controlled mode of loading was utilized, where axial strain values were monitored using three extensometers mounted on the specimen. All recorded values were validated and specimen displacement was calculated based on the average of recorded strains from the extensometers. A difference of ±25% in the recorded values was considered as an indication of highly non-homogeneous conditions for the strain field within the sample, beyond which the test should be considered no longer valid (Baaj, Di Benedetto, Chaverot , 2005).

Wöhler's Law was used to analyze the fatigue results by studying the number of load repetitions until failure versus strain amplitudes, in a bi-logarithmic scale (Tayebali, Akhtarhusein, Bor-wen tsai, Carl, 1994). Wöhler's Law results in a linear trend, describing fatigue behavior by means of two parameters, namely the slope (c2) and the Y-intercept (c1). Coefficients c1 and c2 depend on both, the material and the chosen failure criterion (Perraton, Di benedetto, Carter , 2011). Equation 8.3 presents the Wöhler curve for the purpose of fatigue characteristics of asphalt mixtures.

$$N_f = C_1(\varepsilon_0)^{(-c_2)} \tag{8.3}$$

where:

- Nf is the number of cycles corresponding to failure (aka fatigue life) for a given failure criterion;
- ε0 is applied strain amplitude (mm/mm) at a given testing temperature (θi) under a specific testing frequency (f);
- C1 is the coefficient corresponding to the expected fatigue life for a strain amplitude of 1 mm/mm, at the given testing temperature and loading frequency; and
- C2 is slope of the Wöhler curve when it is associated with a straight line in the log N_f – log ε_0 domain.

The ε6 value is commonly used to characterize the fatigue resistance of bituminous mixes (Di Benedetto & De la roche, 1998). This particular value is the strain level at which the pavement can take up to 1,000,000 load cycles. Furthermore, the dissipated energy analysis has been used in this study using recorded values. During the tension-compression loading, part of the energy is converted into heat and raises the temperature of the specimen. Generally, in strain-controlled mode, the dissipated energy decreases with the number of cycles, whilst this pattern would be increasing in stress-controlled mode. The dissipated energy can be calculated in a simplified form by Equation 8.4:

$$W_d = \pi \times \varepsilon_A \times \sigma_A \times \sin \phi_E \tag{8.4}$$

where:

- Wd is dissipated energy;
- ε_A is strain amplitude (mm/mm) at a given testing temperature and stress level;
- σ_A is applied stress amplitude at a given testing temperature; and
- ϕ_E is measured phase angels.

In stress-controlled mode, the amplitude of the average strain may increase or decrease slightly during the test, to keep up with the target level of stress.

Resistance to Rutting: Although studying the changes in the modulus of bituminous materials and their performance at low- and intermediate-temperatures is crucial, this would not be sufficient for a comprehensive characterization of a newly proposed material. Therefore, behavior of the asphalt concrete mixes at high temperature ranges should also be studied in terms of their resistance to accumulated permanent deformation (aka rutting) which occurs beyond the linear viscoelastic range previously covered through the complex modulus testing. To this end, the French rutting test equipment — developed by Laboratoire central des ponts et chaussées (LCPC) – as shown by Figures 8.7 and 8.8 has been utilized in this study. The test is standardized in Europe under EN 12697-22A1, as well as in the province of Quebec in Canada under LC 26-410. Asphalt industry has been commonly using the French rut tester as a simulator of field compaction for research and lab purposes (Gabet et al., 2011; Perraton, , 2011).

Figure 8.7 The French rutting test equipment at ÉTS

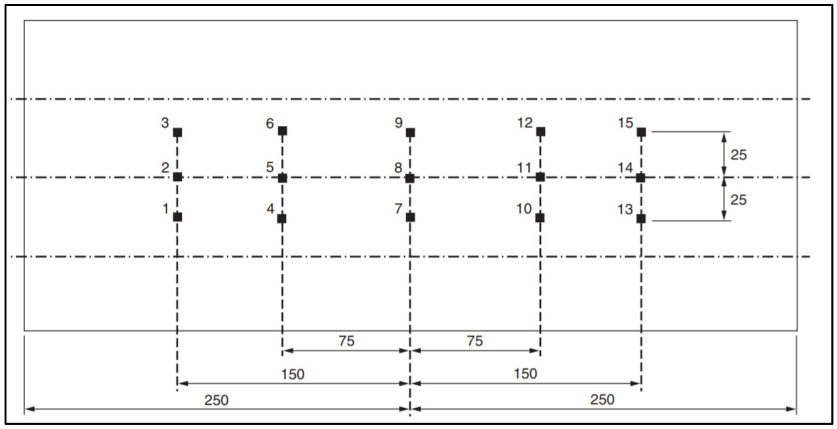

Figure 8.8 Plan view of measurement points on top of slab (mm) (LC 21-410)

In this study, 100 mm thick slabs of 500 mm by 180 mm dimension were prepared in the laboratory. For most hot mixes, laboratory-manufactured slabs at 92% compaction level may lead to post-compaction rutting. Therefore, the laboratory-prepared specimens were compacted to a greater compaction level of approximately 95%. At this level, post-compaction is generally negligible (Rahman, Hossain, 2014). Heating temperatures for mixing and compaction are indicated in the test method LC 26-003. This laboratory step was done according to AFNOR Standard P98-250-2. Post-compaction conditioning and testing were conducted as per the following three steps:

1. Pre-conditioning: rolling the pneumatic tire across the specimen for 1000 cycles at the ambient laboratory temperature to minimize any discrepancies;

2. Test temperature conditioning: heating the chamber to 60 °C;

3. Measurement: rut depths were measured at different intervals (i.e., after 30, 100, 300, 1000, 3000, 10,000 and 30,000 cycles). Rutting is then defined as the average vertical deformation of the hot mix surface as compared to the average thickness of the specimen before starting the test. As described in AFNOR P 98-253-1, height measurements were taken at 15 locations over the slab area.

For this family of rutting tests, the measurements are typically presented as a function of the number of wheels passes in semi-logarithmic scale. Different phases of the test are then identified by curve fitting using linear regression (Yildirim, 2007) and quantifying the results in terms of post-compaction consolidation, creep slope, stripping slope, and stripping inflection point. Figure 8.9 illustrates these parameters for a typical Hamburg Wheel Tracking Device Test. For this purpose, post-compaction consolidation is defined as the deformation at 1,000-wheel passes; and creep slope describes the inverse of the rate of deformation in the linear region of plot between post compaction and stripping inflection point (if stripping occurs). Stripping inflection point is the number of wheel passes at the intersection of creep slope and stripping slope. Finally, stripping slope is used as an indication of damage rate and is defined as the inverse rate of deformation after the stripping inflection point. These measures are calculated for each of the mixes investigated in this study and were used to compare the rutting resistance of fiber-modified and control mixes.

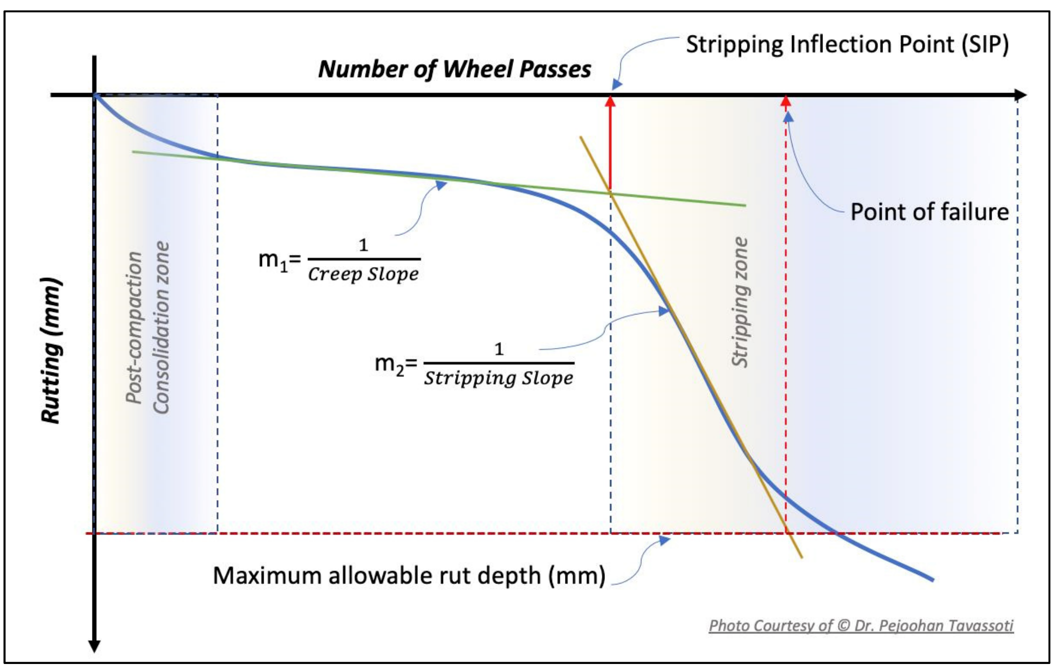

Figure 8.9 Conceptual representation of Hamburg Wheel Tracking Test Results
Taken from Tavassoti & Baaj (2020)

8.4 Results and discussion

The first step in this experimental study was to design the fiber reinforced asphalt concrete mix as well as the conventional (control) hot mix asphalt. Results of the mix design stage indicate that the optimum binder content for the Pulp Aramid Fiber (PAF) mix was found to be 5.5% as compared to the 4.5% binder content that is required to produce the control mix without any fibers. The high surface area of the fibrils, their branched structure, as well as fibrils bitumen absorption could have contributed to the increased demand for asphalt binder. The PAF mix was prepared using 0.3% of fiber content and 5.5% virgin asphalt binder. The results of the experimental studies on the fiber modified and the control mixes are presented in this section:

8.4.1 Indirect tensile strength (ITS)

The optimum fiber content in a mix depends on the physical and chemical properties of the fiber. Fiber content in HMA mixes typically varies from 0.1% to 0.7%. Preliminary testing in this study was aimed to figure out the optimum fiber content according to the desired binder content and air void level and covered a PAF content ranging from 0.1 to 2.0%. Details of this study are beyond the scope of the current paper, however, Figure 8.10 shows a summary of changes in the air void level of the compacted mixes prepared using the same binder content of 4.5% but different fiber dosages. It can be seen that for a given binder content and same compaction effort, the air void level is directly related with the fiber content. Further optimizing the mix design by varying the binder content, it was concluded that a fiber content of 0.3% provides the optimal mix. Moving forward, this fiber content was, hence, used to prepare specimens for the performance evaluation purposes. During the mix design process, the optimum binder content was calculated for the desired PAF level. The results showed that a 5.5% binder content yields an air void level of 6.5%, which is equal to the control mix air void level. Two cold testing temperatures (i.e., -18 ºC and 0 ºC) were selected to evaluate the effect of PAF on mix properties under cold- and moderate-climate conditions. To this end, indirect tensile strength test was performed on specimens of each PAF mix and the control mix and the results were used to evaluate the effect of PAF on ductility of the mixes. Figure 8.11 illustrates an example of the results at -18oC which demonstrates the difference between the PAF- and control mix with respect to their load-displacement behavior during the ITS testing. Based on the ITS results, it was observed that the maximum tensile strength for PAF mix was lower than that of the control mix at both testing temperatures. However, even by looking at the shape of the load-displacement curves it can be realized that the PAF mix exhibited a more ductile behavior. This behavior at low temperature can be attributed to both the presence of fibers in the composite matrix of the PAF modified mix as well as the extra 1% bitumen in the PAF mix. The impact of fiber was clearly observed at low temperature conditions through visual inspection, where PAF mixes after getting to its maximum capacity still kept its shape for a longer time than control mix. This is evident from the results presented in Table 8.3 which provides the fracture work calculations for two distinct stages of the test. Stage-I is identified by the area underneath the load-displacement curve up to the point of peak load, whereas Stage-II also includes the post peak portion of the curve until the point that the level of applied load drops to 20% of the peak. The fracture work was found to be consistently higher for PAF mixes, and such difference is especially pronounced at -18 ºC.

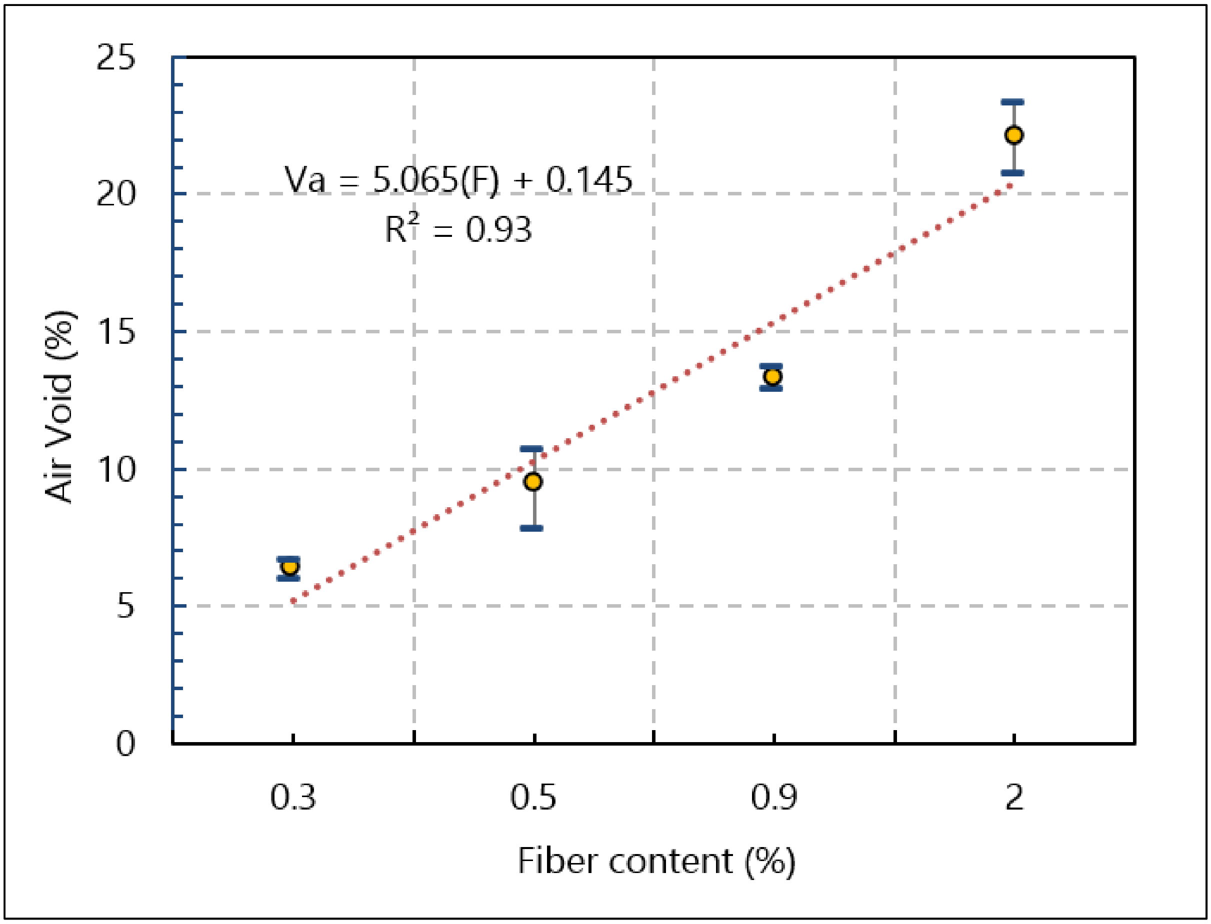

Figure 8.10 Air void vs fiber content for a constant binder content of 4.5%

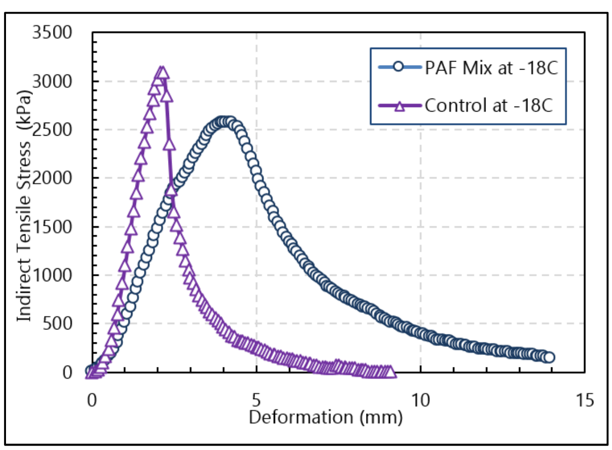

Figure 8.11 ITS results at -18 ºC for PAF and control mixes

Table 8.3 Summary of ITS testing fracture work at low temperature

	Fracture Work at Stage I (N.m)		**Fracture Work at Stage II (N.m)**	
Mix Type	**-18^{o}C**	**0^{o}C**	**-18 ^{o}C**	**0^{o}C**
PAF-Mix	626.0	433.3	1153.9	686.5
Control-Mix	272.6	420.0	464.9	606.3

Overall, it was concluded that PAF materials contribute to improving the ductility of the mix at low temperature. This higher ductility could also lead to higher resistance to crack propagation, which is investigated in Section 8.4.3.

8.4.2 Complex modulus

Given the viscoelastic nature of bituminous materials, master curves based on the norm of complex modulus and phase angle values were utilized to study the behavior of mixes in this study. The materials' responses were maintained within the linear viscoelastic (LVE) range during the tests, so that the time–temperature superposition principle (TTSP) could be applied to analyze the complex modulus test data. TTSSP principle has been verified by numerous studies dealing with the unidirectional linear viscoelastic behavior of bituminous materials (Delaporte, Di Benedetto, Chaverot, Gauthier, 2007). TTSP principle can be utilized to translate the high frequency responses of the material to those at the low temperature ranges. Similarly, low frequency data could be translated to their high temperature counterparts.

Therefore, in a full range of temperature (Figure 8.12), PAF mixes were slightly softer than the control mix. However, it should be noted that the binder contents for these mixes were not the same (i.e., 4.5 versus 5.5%).

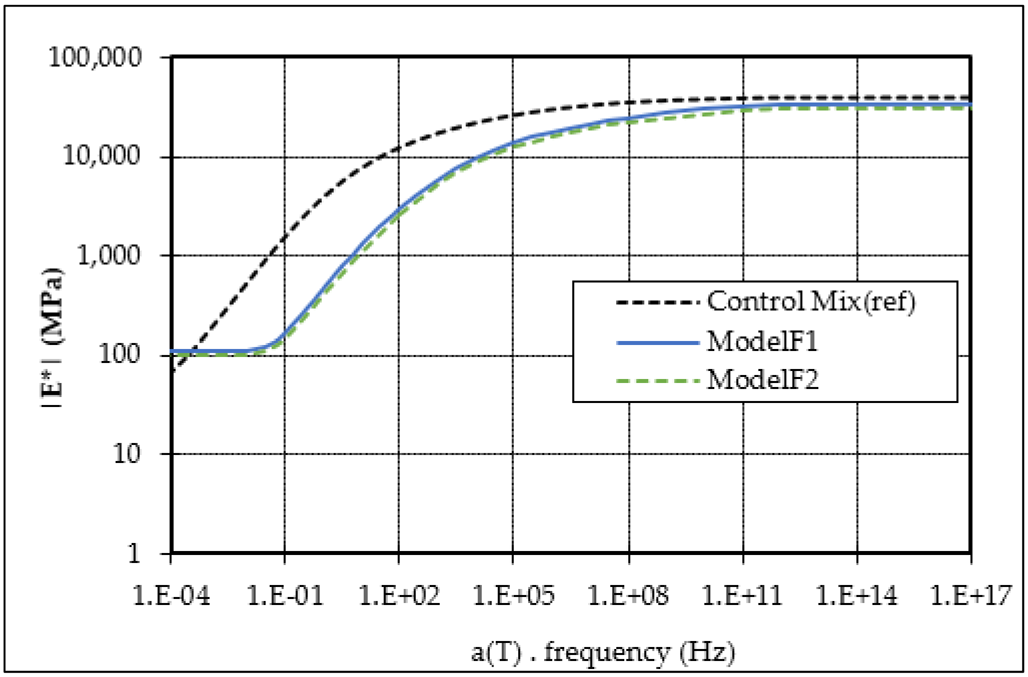

Figure 8.12 Master curve of the norm of Complex modulus at 17 ºC

The rheological properties of the mixes can also be expressed in terms of phase angle (δ). Phase angle is approximately equal to the derivative of the logarithm of the stiffness with respect to frequency, where a value of 0 degrees means a purely elastic material and 90° means a purely viscous material. Figure 8.13 presents the phase angle master curve for the mixes investigated in this study. PAF mixes exhibited more viscous response at higher frequency (lower temperature) than the control mix. Results of two replicates are shown in this figure, which confirmed that the results are reliable and repeatable. Phase angle master curve reveals that for higher temperatures (or slower loading rates) the PAF mix exhibits more elastic response as compared to the control mix. Whereas, in case of lower temperature ranges or higher loading rates the PAF mix shows more viscous response which would be desirable. This enhanced response for PAF can be recognized by looking at the phase angle results below and beyond a reduced frequency of 0.3 Hz.

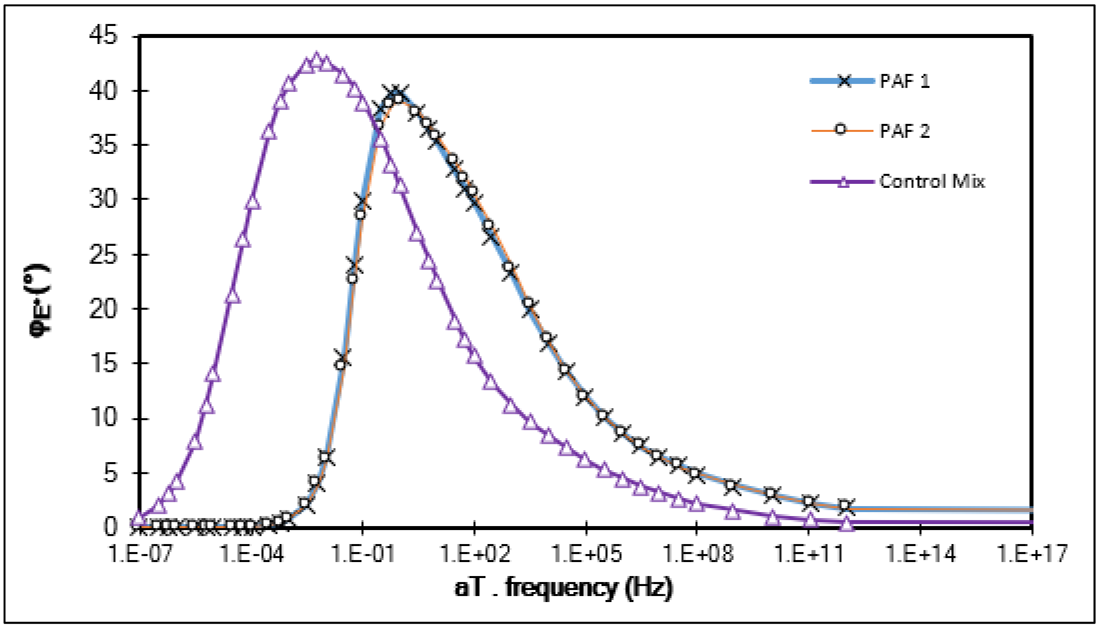

Figure 8.13 Master curve of the phase angle of complex modulus at 17 ºC

As previously discussed, the 2S2P1D model parameters in the Cole-Cole model shown in Figure 8.6 are calculated at this stage with the corresponding values of these parameters listed in Tables 8.4 and 8.5. In this model, h and k are exponents such that $0 < k < h < 1$, and are related to the ratio of E Imaginary/E Real when ϖ tends to zero (resp. to infinity). Accordingly, E0 is considered as the glassy modulus when $\varpi \approx \infty$, and E_{00} is defined as the static modulus when $\varpi \approx 0$. Newtonian viscosity is denoted by η in this model. Table 4 also presents the highest value and lowest value of the real complex modulus corresponding condition. It can be seen that at min value (or high temperature) the PAF mixes have higher stiffness than the control mix, and it can be expected that at exactly max value (or low temperature) both mixes have approximately the same stiffness.

Table 8.4 The highest and lowest value of real complex modulus

Mix Type	Temperatures(ºC)	Frequencies (Hz)	Real (E*)	
Control Mix	37.9	0.03	99	Min Value
PAF Mix1	36.7	0.01	104	
PAF Mix2	36.7	0.01	126	
Control Mix	-33.2	10	37,559	Max Value
PAF Mix1	-34.2	3	33,547	
PAF Mix2	-34.2	3	29,970	

Various criteria are proposed in the literature toward comparing the stiffness of different bituminous materials. Baaj et al. (2013) suggested the following procedure and the corresponding key parameters for the purpose viscoelastic response evaluation of bituminous mixes:

- The value of |E*| at -30 ºC and 3 Hz as an indicator for the material stiffness at a low temperature and a high Frequency condition;
- The value of |E*|/sin (ϕ) at 40ºC and 0.03 Hz, since this ratio is used as rutting indicator;

- The value of |E*| at 20oC and 3Hz, representing the mix stiffness (or resilient modulus) used in the AASHTO 1993 empirical pavement design method.

In addition to the aforementioned parameters, Perraton et al. (2014) suggested using the stiffness at 15 °C and 3 Hz. It should be noted that standard bituminous base course materials would typically have dynamic modulus values in the range of 5,000 to 7,000 MPa when tested under the same conditions at 15°C. All these criteria are summarized in Table 8.5.

Table 8.5 Key parameters from complex modulus of the mixes through 2S2D1P model

MIX	E* at -30 °C, 3Hz (MPa)	E*/Sin(ϕ)at 40°C and 0.03 Hz (MPa)	E*at 20°C, 3 Hz (MPa)	|E*|at 15 °C and 3 Hz (MPa)	E_0 (MPa)	E_∞ (MPa)	k
Control	35588.5	NA†	3771.7	6027.1	30	40000	0.18
PAF 1	31873.7	NA†	3467.3	5237.3	110	39000	0.14
PAF 2	28433.1	NA†	3828.0	4630.0	100	35000	0.14

† This criterion was not used in this study because rutting was directly evaluated using wheel tracking device.

8.4.3 Fatigue resistance results

As discussed earlier, using a homogenous loading configuration such as the uniaxial tension-compression fatigue tests in this study, can facilitate accurate analysis of the materials response during different stages of the fatigue test. Therefore, the classical method of analysis was employed in this study to investigate the impact of fibers on fatigue life of the asphalt concrete mixes. Details of the fatigue tests on both conventional HMA and PAF modified mix specimens are provided in Table 8.6. This table presents details such as the initial strain values (both the target and achieved/measured), air void level, and the number of cycles until failure which is known as fatigue life for each specimen in this study. As fatigue life results are sensitive to air void level of the specimen, the air void level was maintained within 7.0 ± 1.0 percent for at least 3 specimens per each mix. Among the eight cylindrical specimens for the fatigue evaluation phase, two specimens (one for each mix) had marginal air void levels. Therefore, for each mix corrective calculations were utilized in order to obtain an equivalent fatigue life for all specimens at the hypothetically same air void level.

Table 8.6 Details of the uniaxial Tension-Compression Fatigue tests in this study (10Hz, 10ºC)

Mix Type	Specimen ID	Target Def,(µdef)	% Va	Real Def, (µdef)	Nf	Log Real Def, (def)	Log Nf II/III
Control Mix	C-1	80	6.8	71	2,405,986	-4.15	6.38
	C-2	100	6.8	99	61,464	-4.01	4.79
	C-3	70	8.0	66	4,715,411	-4.18	6.67
	C-4	90	6.5	84	2,500,000	-4.08	6.40
Fiber Mix	FF1	80	6.3	74	5,205,000	-4.13	6.72
	FF2	90	6.6	85	637,965	-4.07	5.80
	FF3	100	6.7	94	141,876	-4.03	5.15
	FF4	80	5.6	73	3,211,000	-4.14	6.51

This correction was applied prior to developing the Wöhler curves, which is built from the fatigue results derived from specimens at similar air void levels. Moutier (1991) proposed the corrective calculation through the use of Equation 8.5:

$$\varepsilon_6 = (-125 + 72 * TL - 4.85 * TL^2 + 3.3 * \Delta C) * 10^{-6} \tag{8.5}$$

Where:

ε_6: corrected estimation of the strain level to yield a fatigue life of 106^6 cycles;

TL: binder content; and

ΔC: Difference between the compaction obtained on the specimen (Ccor) and the compaction obtained by the test of gyratory shear compactor (CGSC).

Given the fact that the binder content (TL) was kept the same among the specimens for each mix and CGSC is also the same, the corrected strain parameter can be calculated from Equation 8.6:

$$\varepsilon_{cor} = \varepsilon_m + 3.3(C_{cor} - C_{Samples}) \tag{8.6}$$

where the compaction level is defined as C = 100 – Va, which by replacing the compaction with air void content in Equation 8.7nthat can also be used in form of Equation 8.8, since in our case Vcor is the average void for all the specimens used:

$$\varepsilon_{cor} = \varepsilon_m + 3.3(V_{sample} - V_{cor}) \tag{8.7}$$

$$\varepsilon_{cor} = \varepsilon_m + 3.3(V_{sample} - \bar{V}) \tag{8.8}$$

Equation 8 was used in this study to analyze the fatigue test results by taking the air void variation into account. For this purpose, the average air void (i.e., $\bar{V}$) was found to be 6.7%. This correction was performed by using the value of ε_m, measured for a given mix, for which the air void content of a tested specimen is denoted as V_{sample}. Fatigue equations were developed by running regression models on the corrected test results to quantitatively characterize the fatigue performance of mixes (Figure 8.14). In order to run a linear regression and develop this chart, four specimens from each mix were prepared and tested using a sinusoidal load at three different strain levels. The results are then used to calculate the value of Ɛ6, which corresponds to the strain level at which the asphalt mix would reach a fatigue-induced failure after one million cycles of loading. Typical Ɛ6 values are in the range of 70 to 90 µm/m which have been also recorded for the standard asphalt base courses. The value of Ɛ6 was found to be 78 µm/m for the control mix in this study. The values of R2, being greater than 93% in this study, indicate the acceptable quality of the linear regressions. For the PAF modified mix an Ɛ6 value of 83 µm/m was found from the test results, which is greater than that of the control mix. In this figure, slope of the trendline indicates the degree of sensitivity of the mixes in terms of fatigue cracking resistance to changes in the applied strain level. In other words, a steeper slope indicates that the mix is highly sensitive to changes in the strain level, meaning that due to a small change in the strain level there would be a considerable difference in the maximum number of load repetition that a mix can take until failure. By comparing the results, it can be concluded that both mixes exhibit the same level of sensitivity to strain level, while for a given strain level the fiber modified mix would last longer.

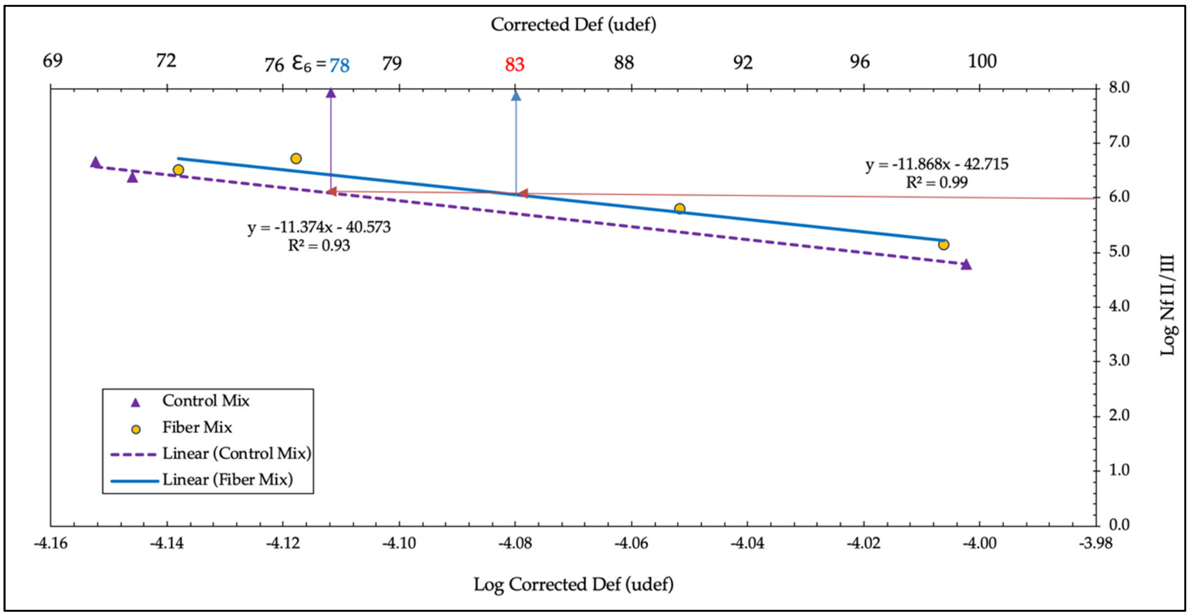

Figure 8.14 Wöhler curves for control and PAF mixes in this study

Figure 8.15 shows examples of the results demonstrating the change in the norm of complex modulus for the PAF modified mixes at three strain levels of 80, 90, and 100 microns/m. For the sake of comparison, an example of the control mix tested at 80 micro-strain level is also plotted in this figure that indicates while the initial modulus of the control mix is higher than the fiber modified mix specimens, it has a shorter fatigue life as compared to the PAF specimen at the same strain level. Furthermore, from the trend of changes in the modulus of mixes during the cyclic fatigue testing, it can be recognized that the fiber modified mixes exhibit a delayed macrocracking phase as compared to the control mix specimen. It is also recognized that part of such difference in fatigue results could be attributed to the higher binder content in PAF mixes. Nevertheless, the presence of short PAF strands has provided an enhanced microstructure which is later verified by looking into the permanent deformation characteristics of the mixes in the following section.

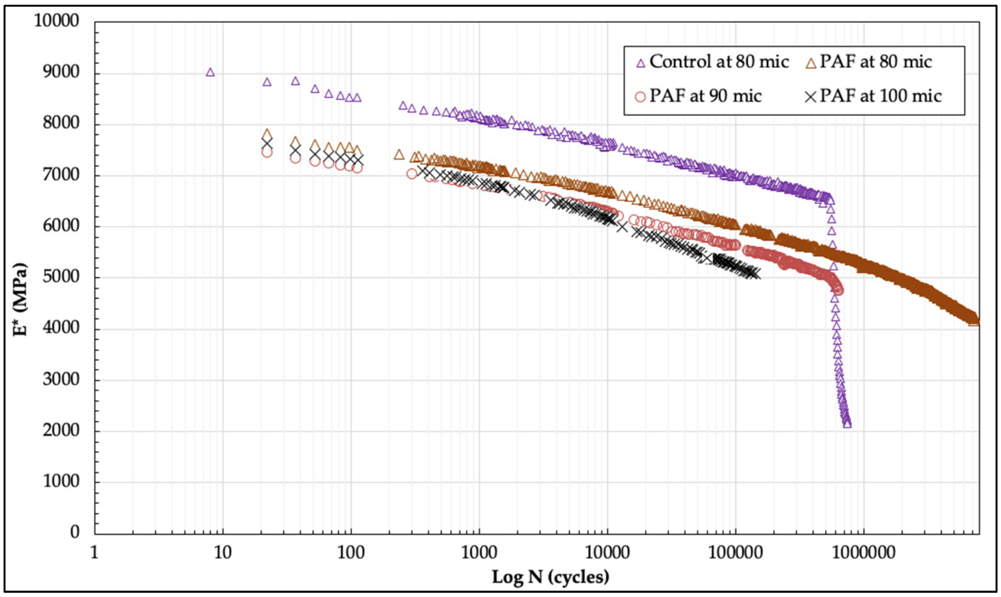

Figure 8.15 Changes in modulus during fatigue testing at variable strain levels

In addition to the conventional modulus-based fatigue failure criteria, changes in the dissipated energy was also briefly studied in this paper. Using a 50% drop in the stiffness of the specimen (with reference to the initial stiffness) has been considered as the most commonly used criteria to identify the fatigue life of asphalt concrete mixes in the laboratory. On the other hand, evolution of the dissipated energy (DE) at different cycles through a tension compression test can be used as an indicator of microcracks propagation through the specimen and development of macrocracks in the mix. To this end, dissipated energy under strain-controlled testing condition was calculated for the mixes at different strain levels. The results for the PAF mixes tested at 80, 90, and 100 micro-strain level are presented in Figure 8.16, where it can be seen that the dissipated energy evolution has three distinct stages. In the first stage, which corresponds to the initial stage in the fatigue testing, the DE value increases with a steep slope, followed by a plateau zone in the second stage where the DE maintains approximately the same magnitude for a considerably extended amount of time. Finally, the third stage can be marked with the sharp increase in the DE magnitude which indicates formation of macrocracks and ultimate failure of the specimen. This parameter can be used to further analyze the fatigue response of mixes at different strain levels, and as a supplementary measure to the arbitrary 50% loss in the initial modulus criterion.

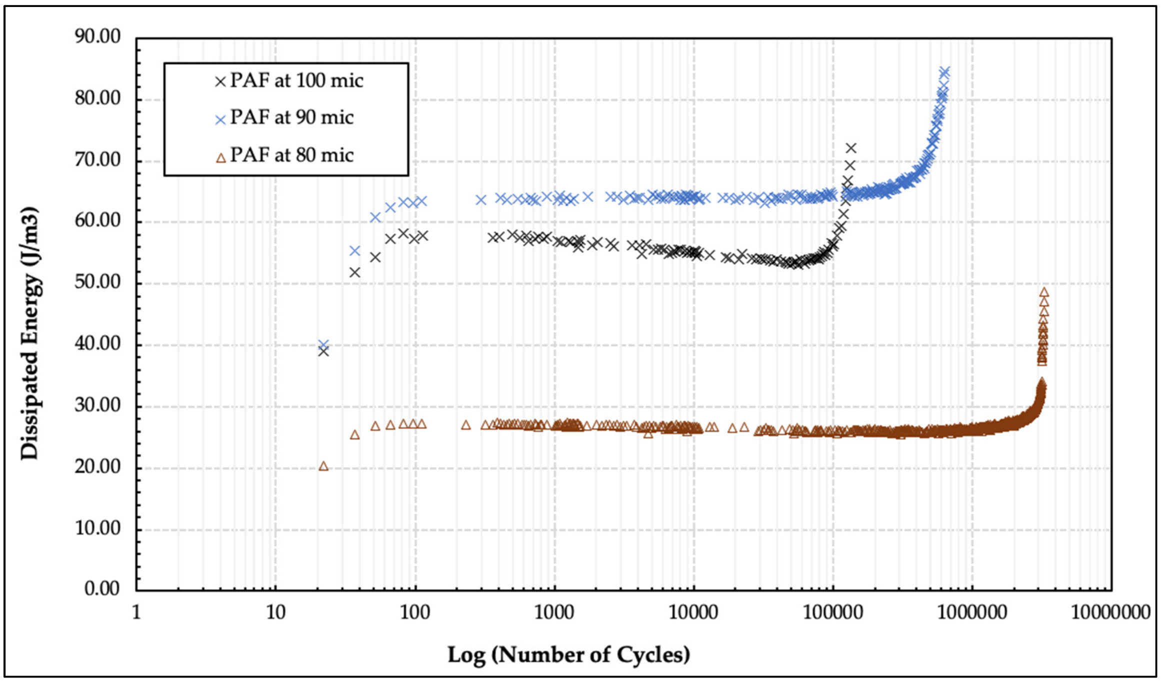

Figure 8.16 Changes in dissipated energy at different strain levels for the fiber reinforced mix

8.4.4 Rutting resistance of PAF mix versus control mix

Permanent deformation of the mixes was measured at 60 °C for slabs of 100 x180 x 500mm under repeated loading of a tire inflated to 0.6 MPa, mounted on a carriage that moves back and forth at 1 Hz rate exerting a load magnitude of 5 kN to the slab. The variability of rutting test results had been previously explored on similar materials at the ÉTS. However, due to the limited amount of the specific raw materials in this study, only one slab per mix was used for evaluating the rutting performance. Figure 8.17 shows the evolution of permanent deformation on the surface of asphalt concrete slabs versus the number of wheel passes. The results are also presented as the percentage of permanent deformation derived from linear regression for PAF- and the control mix. The results confirmed that both of the mixes exhibited an acceptable rutting resistance performance, where the deformation magnitudes remained well below the maximum 10% criterion after 30,000 cycles. Therefore, it can be concluded that both the PAF and control mixes in this study were resistant to the permanent deformation failure. It should be noted that the mixes had the same aggregate skeleton, but there was a 1% difference in the binder content. PAF mix results show slightly higher permanent deformation, as PAF mix has a higher binder content. Considering the low temperature and fatigue performance of the PAF mix, it can be concluded that the presence of PAF in the mix made it possible to increase the binder content to achieve an enhanced performance while the high temperature permanent deformation properties are maintained as well. This can be looked into as an opportunity for moving toward a more balanced mix design (BMD), as it is generally believed that the conventional SuperpaveTM mix design method can yield lean mixes that are resistant to rutting but might be prone to fatigue and low temperature cracking. To this end, PAF fibers can be thought as a promising option to bump up the binder content while avoiding any negative impacts at creep temperature ranges.

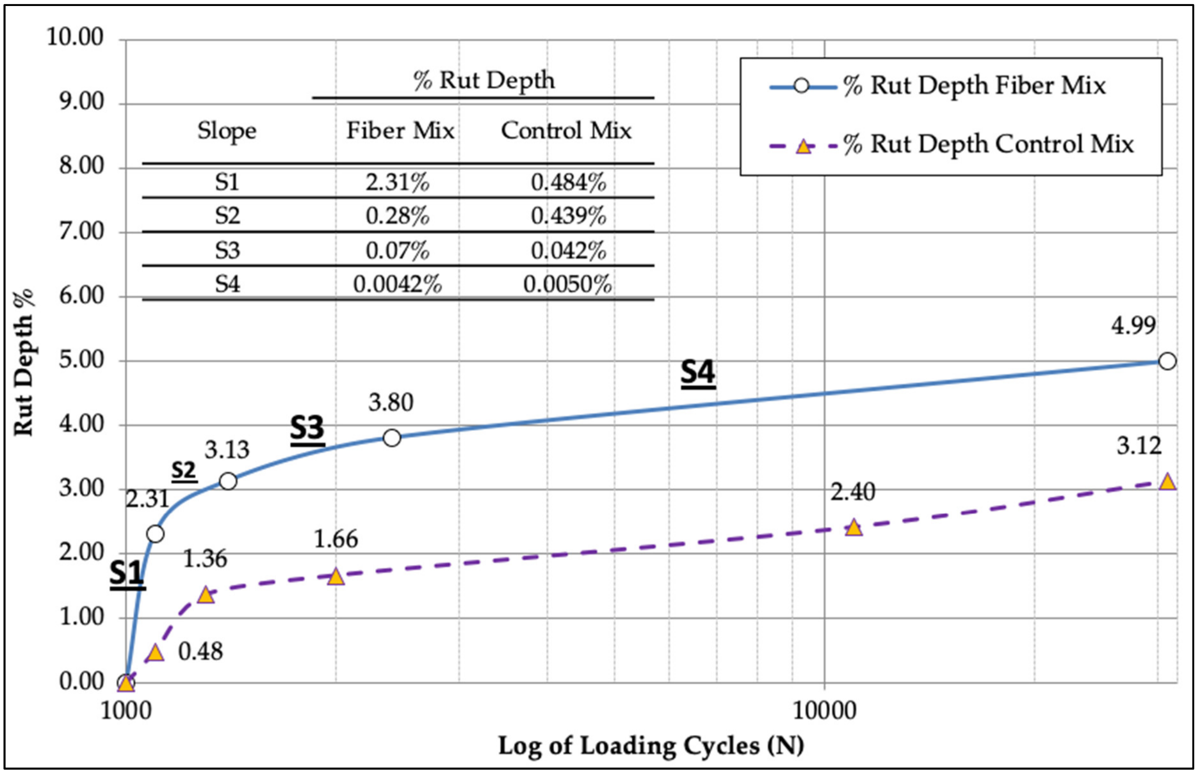

Figure 8.17 Permanent deformation evolution for the control- and PAF-mixes at 60ºC

As can be seen from Figure 8.17, the actual test begins after applying 1,000 preconditioning cycles (aka cold runs) on the slab, which is intended to capture the continued consolidation stage for bituminous layers. At this stage both mixes have the same deformation due to having identical aggregate gradation. Afterward, the rest of the loading was pursued at 60oC, where the binder exhibits a softer response as compared to the cold cycles stage.

After undergoing 1,000 hot cycles (post compaction), the PAF mix was initially deformed more than the control mix. This section could be characterized by S1 and S2, where parameter "S" represents the slope of the permanent deformation curve in Figure 8.17. For the PAF mix, rut depth dramatically increased at first 300 cycles (S1=2.31%) and slowed down at S2 (0.2%), whereas for the control mix, these slopes increased at the same rate (0.4%) in both steps (i.e., S1 and S2).

The last stage of the test represents the overall performance of the material under the loading from wheel passes which can be translated to rutting. Figure 8.17 also shows the slope per different sections of the rutting test for the PAF and control mix. It can be seen that both mixes exhibited approximately the same response to repeated loading during the rutting stage. At this stage, aggregate skeleton and interlock play the main role in rutting resistance.

In conclusion, rutting results can be divided in three phases: deformation at the end of 1,000 cold cycles, 1,000 hot cycles and 30,000 cycles. The first phase which was called continued consolidation earlier, suggests that PAF does not have a significant impact on the general skeleton of the aggregates. The second phase which was called post compaction (S1, S2, S3), suggests that the specimen binder is soft enough to indicate the difference in aggregate gradation. A flatter slope can be translated to well-packing phenomenon. It was recognized that the PAF mix with 1% extra binder has more flexibility than control mix. Also, the extra binder impact can be taken into the account at this stage. The last section which was called rutting stage, indicated the rutting resistance of mixes. The results indicate that both mixes have statistically the same rut resistance under 30,000 repetition of wheel passes.

Although the PAF mix contained an additional one percent asphalt binder content as compared to the control mix – which could have sacrificed the rutting resistance of this mix – the PAF mix exhibited the same rutting resistance as the control mix. It can be speculated that PAF absorption characteristics could have limited any negative impacts from an additional binder content at higher temperature and kept the desirable structure of pavement for a longer time.

Concurrently considering the fatigue and rutting performance of the mixes in this study, it can be concluded that the fiber reinforced mix exhibited a superior overall performance. It should be noted that while the higher demand for asphalt binder in the PAF mix could contribute to the cost of the project, presence of the fiber particles extends the fatigue life of the mixes, leading to a considerable potential saving in the life cycle cost of the modified pavements.

8.5 Conclusions

Review of the literature indicates that research on producing asphalt concrete mixes with Pulp Aramid Fibers (PAF) has been scarce. The main goal of this study was to evaluate the effect of short PAFs on the performance of Hot Mix Asphalt (HMA) mixes. To this end, mixes with and without fiber modification were designed and investigated in this study, finally resulting in one control HMA prepared with virgin materials with 4.5% bitumen content and one PAF mix with 5.5% bitumen content and 0.3% short pulp aramid fibers. The following conclusions are drawn based on the results of this study:

- Indirect Tensile Strength (ITS) tests, a 0.3% addition of short Aramid fiber was found to be optimum during the HMA mix design stage, where lower rates did not have a significant impact on HMA performance and higher dosages of fiber increased the demand for bitumen content significantly;
- ITS testing at cold, intermediate, and high temperatures indicated that the PAF reinforced HMA with inclusion enhances the ductility of the mixes, which are especially evident at lower temperatures such as -18 ºC;
- In spite of its higher design binder content, the PAF mix with 0.3% fiber content exhibited satisfactory rutting resistance while exhibiting considerably improved fatigue and low temperature performance;
- The presence of PAF in the modified mixes combined with the effect of higher binder content resulted in enhanced viscoelastic properties as demonstrated by the 2S2P1D model. The PAF mix exhibited more viscous response at higher frequencies (or lower temperatures) than the control mix, which is desirable;
- Fatigue analysis showed that the PAF mix surpasses conventional mix indicated by higher ε_6 values, leading to a better fatigue resistance;
- Short Aramid fiber reinforced mix had a delayed micro-cracking formation. It was also noticed that the control mix specimens had a higher level of variability in terms of number of cycles to failure and micro-cracking formation point.

Overall, the PAF mix exhibited acceptable performance with respect to rutting, fatigue, and stability at different temperature ranges. While conventional SuperpaveTM mix design generally is inclined to produce lean mixes with respect to binder content, presence of PAF allowed increasing the binder content to achieve the desirable ductile and elastic response over a wider range of temperature, while maintaining the same level of rutting resistance at higher temperatures. The ease of application of this material can offer considerable benefits, especially when polymer modification is not a feasible option. Further studies should be carried out to better understand the interaction between the optimum PAF content and other important parameters such as the fiber length, chemical properties, and physical characteristics.

Author Contributions: S.S.S. conceived of the presented idea. A.C. and H.B. developed the theory and verified the analytical methods and supervised the project. S.S.S. wrote the original draft; and P.T. contributed to fatigue and ITS testing analysis, wrote, reviewed, and edited the manuscript and verified the analysis. All authors discussed the results and contributed to the final manuscript.

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Conflicts of Interest: The authors declare no conflict of interest.

CHAPTER 9

CONCLUSION FOR PART B

During the past decades, various materials with different physical and chemical characteristics have been introduced by researchers to improve and enhance the pavement life. One of the novel additive is fibers. Synthetic Fibers are the most common recommendation which include different type such as glass, carbon and aramid fibers. Part B study the possibility of using a tiny, short and pulp Aramid fiber that was provided by Kevlar DuPont, called KEVLAR pulp. As mentioned in literature review, resistance of Aramid fibers to tensile loading is approximately five times higher than that of steel. This fiber was originally used as a replacement of steel in radial tires and this type of Aramid pulp is typically smaller than 1mm in length.

Due to the higher surface area and high absorption rate of fiber, mixes with PAF is expected to have higher binder content. Also there is no clear vision in literature on optimum fiber content in HMA. Different percent as trial were tested and 0.3 % was chosen as the target however further investigation required to find the optimum content. On the other hand, the method of adding PAF to HMA was unknown and different scenarios was developed and finally PAF was blended with the filler in a small mixer for 2 to 5 minutes, depending on the filler and fiber content, and then added to the aggregates. The trial was successful, and this type of blending was found to be the most energy-efficient way of spreading PAF and filler in between particles. ITS results indicated a better resistance to cracking and delayed crack propagation in the mix.

Following to preliminary study, various empirical and thermo-mechanical tests are employed to evaluate the impact of short Aramid fibers on the HMA mix behavior. These tests are including complex (dynamic) modulus, repeated load permanent deformation, and fatigue tests. PAF mixes showed the acceptable rutting resistance also The PAF mix exhibited more viscous response at higher frequencies. However both control mix and PAF has the same sensitivity to deformation but PAF mix last longer in term of fatigue life.

CONCLUSION AND RECOMMENDATIONS

As shown in previous chapters, the impact of two bituminous additives including FR and CR also non-bituminous additive called PAF have been evaluated. Following overall conclusion of the study obtained to reported in conferences CSCE, CTAA and journal Infrastructures and Construction and Building Materials journal.

- However FR has more binder content but CR contain more active binder to participate in mix with virgin aggregate and binder;
- Recovered binder from CR does not have the same characteristics as FR and aging rate is expected to be faster in FR compare to CR;
- It can be concluded that the RAP particle size can have a more significant effect on the mix performance than the RAP content. RAP content should be considered along with other important parameters such as RAP particle size and gradation, recycled binder ratio, and RAP binder content;
- PAF mixes can delay the crack propagation in the mix and improve the HMA performance at low temperature;
- the PAF mix exhibited acceptable performance with respect to rutting, fatigue, and stability at different temperature ranges.

In addition to the presented study, the following list is recommended for future investigation:

- Development on quantifying the active binder content in RAP by studying the impact of mix duration and temperature on transferred binder from RAP;
- Presented conclusion must be validated with several sources of RAP;
- More work is needed to better understand the impact of RAP gradation. The results shown here definitely give new information about different RAP asphalt binder contained in different sizes, but new tests with several other RAP size are needed to confirm the results presented here;

- Further studies should be carried out to better understand interaction between the optimum PAF content and other important parameters such as the fiber length, chemical properties, and physical characteristics.
- Further studies is recommended to validate the combination of the coarse RAP and PAF in a HMA to reinforc the fine part of the mix with PAF and coarse part of the mix with CR.

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