Innovative Evaluation of Crumb Rubber Asphalt and Recycled Asphalt Pavement

by

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Author's Declaration

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners. I understand that my thesis may be made electronically available to the public.

Abstract

Recycling asphalt pavement is consistent with the concept of sustainability and when engineered properly, it can potentially provide a more cost-effective alternative to conventional road practices. In 2011, the Centre for Pavement and Transportation Technology (CPATT) at the University of Waterloo, the Ontario Tire Stewardship (OTS), and the Ministry of Transportation of Ontario (MTO) partnered to conduct several demonstration studies on the innovative use of Crumb Rubber Modified (CRM) asphalt pavements and Reclaimed Asphalt Pavement (RAP).

In a bid to better understand and resolve the technical challenges associated with recycled Hot Mix Asphalt (HMA) mixtures as well as to advance Ontario's paving industry to a more sustainable and economically viable direction, this research involved a comprehensive laboratory testing program to characterize the behaviour and mechanistic properties and compare the overall performance of an array of typical Ontario Superpave HMA mixtures incorporating 0, 15, 20 and 40% RAP, and CRM mixtures with 20% RAP.

The laboratory testing protocols selected to characterize these mixtures include: Binder rheological assessment tests to assess failure properties and grade asphalt binders; Thermal Stress Restraint Specimen Test (TSRST) to determine fracture susceptibility at low-temperatures; the Hamburg Wheel Tracking Device (HWTD) for assessing the combined effects of rutting, stripping potential and moisture susceptibility; and Dynamic Modulus tests to evaluate the viscoelastic properties of the experimental matrixes over a range of loading frequencies and temperature scenarios.

The study also included forensic assessment of past CRM pavement sections, field monitoring of the 2011 CRM-RAP demonstration sections, and an overall cost and sustainability assessment. The main research findings are summarized as follows:

- Rheological characterization of binders indicated that the influence of RAP variation is highly related to the performance grade of the base virgin asphalt cement; while CRM binder modification significantly improved both complex shear modulus, G* and phase angle, δ parameters regardless of the binder grade. This had a significant impact on the rutting and thermal cracking performance of the evaluated HMA mixtures.
- With exceptions to the recovered binders from 20% and 40% RAP HMA mixtures with PG 52-40 and 52-34 asphalt cement, all other recovered binders were observed to be more

flexible at low and intermediate temperatures suggesting that the potential for improved resistance to fatigue failure exists.

- Assessment of dynamic modulus, |E*| and phase angle, δ data suggests that the observed mix stiffness is not exclusively a function of the improvements made by the improved characteristics of the binder, but in combination with other factors. The master curve construction using the rheological analysis software (RHEA™) confirmed these behavioural tendencies. The observed mix stiffness was further observed to correlate well with the mechanistic performance test results.
- The wet-process rubber terminal-blend HMA mixture was noted to be distinctively different from the rubber field-blend mixtures in terms of performance, but no evidence within the concerns of this research suggest that the rubber field-blend method is not effective or feasible.
- Forensic studies on extracted pavement cores indicated that the observed pavement distresses are related to aggregate segregation resulting from the effects of permeability possibly caused by poorly constructed or compacted longitudinal joints.
- In-service pavement monitoring indicated that the Rubberized-RAP sections in Ontario are all performing very well in comparison to the control sections with RAP.
- Study findings also demonstrated the potential to incorporate up to 40% RAP contents into rubberized pavements. However, such designs must take into consideration the consensus properties of the aggregates and volumetric properties of the binder.
- The 40% RAP HMA mix was found to be the most environmentally friendly pavement design alternative. However, the 20:20% CRM-RAP HMA mix was judged the most innovative and optimal sustainable option having satisfied the functional performance criteria and being the most cost-effective.

Based on these findings, the research recommends that CRM used in Ontario rubberized HMA mixtures be subjected to both cryogenic and ambient methods of grinding. This is a more effective way to ensure better or comparable performance with conventional HMA mixtures. The implication of this would be higher initial construction costs, but the many benefits associated with rubberized pavements including its prolonged service life would provide a trade-off over the pavement's lifecycle; especially in terms of maintenance or the need to carry out major rehabilitation.

Acknowledgements

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Dedication

This thesis is dedicated to all forward thinking Nigerians. Our development and road construction challenges may be complex, confounding and often times intriguing, but my conviction is reinforced in the underlying solution that if others could do it, we equally can.

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Chapter 1
Introduction

1.1 Background

The term 'recycling' applies to almost every process where waste material is re-used rather than destroyed or discarded in a land fill. Such materials - sometime of high quality have the potential to reduce the demand for other non-renewable resources. Consequently, the opportunity to find a higher value from a technical, economic and environmental perspective becomes the new challenge. The motivation for recycling may differ, but the awareness that sustainable development is needed has had a strong influence on materials recycling.

Asphalt pavement meets the description of a reusable resource since it can be recycled as part of the road maintenance, rehabilitation and new construction process. Within the scope of recycling asphalt pavements, Reclaimed Asphalt Pavement (RAP) - a resource of well-graded aggregates coated by asphalt cement or binder is considered the most recycled product in North America (NAPA, 2011). Although RAP utilization in Canada is a common practice, its use especially in surface course Hot Mix Asphalt (HMA) mixtures is limited to 20%. This trend in comparison to the available supply of RAP in the province is considered conservative. Reasons for these limitations have been attributed to the variable quality of RAP, a general lack of handling and processing expertise, mix durability and overall performance concerns.

Scrap rubber tire is another reusable resource. Over sixty million scrap rubber tires have been collected in Ontario since inception of the Ontario Used Tires Program in 2009 (OTS, 2014). This program is implemented by the Government of Ontario and operated by the Ontario Tire Stewardship (OTS). Currently, the scrap tire supply in Ontario is recycled into high-quality products such as crumb rubber, tire derived aggregates (TDA) and fabricated products. Crumb rubber is obtained by shredding and grinding scrap rubber tires to particle sizes of 3.36 mm (No. 6) - 0.075 mm (No. 200) mesh sieves (reRubber, 2014). Crumb rubber is of beneficial use when incorporated into HMA, with the potential to consume large amounts of scrap rubber tires. This application is limited in Canada due to inexperience and premature failure of initial trial sections, challenges with incorporation methods and cost effectiveness. Nonetheless, crumb rubber modified (CRM) HMA mixtures is an established practice in the United States - particularly in states like Arizona, California, Florida and Texas (Caltrans, 2005).

1.2 Research Hypothesis

Incorporating RAP and CRM into HMA mixtures changes the volumetric relationship between the three components (aggregates, asphalt binder and air voids) of the mix. These changes consequently affects the mechanistic properties and overall performance of the in-place pavement.

RAP in HMA mixture generally produces a stiffer mix that is beneficial to minimizing the effects of pavement rutting, but with a higher RAP percentage (i.e. > 25%), the resistance to thermal and fatigue cracking is often compromised. The increased stiffness of the aged RAP binder, and degree to which blending occurs with virgin binder is the primary cause of this. Blending virgin asphalt binders with crumb rubber results in a modified binder with increased viscous, relaxation and adhesive properties that are favourable to the rutting, fatigue and thermal cracking resistance of HMA mixtures. Therefore, taking the following concerns with typical Ontario HMA mixtures into consideration:

- Differences in performance grade (PG) of the resultant blended binder;
- Performance challenges with increasing RAP contents;
- The challenges with the generic processes of utilizing CRM;
- The limited investigations into typical Ontario HMA mixtures incorporating both RAP and CRM.

For this research, it is hypothesized that combining RAP and CRM in typical Ontario HMA mixtures can potentially optimize pavement performance. The rubberized binder blend is capable of compensating for RAP shortfalls such as its effects on binder aging and mix stiffness thus improving and providing safer and durable pavements. In accordance with Ontario's goals of fostering the innovation of greener roads, this study provides agencies with data which evaluates recycled materials in conventional Ontario HMA mixtures. The potentials for economic gains further confirms why a study of this nature is necessary.

1.3 Research Scope and Objectives

Through this research, recommendations to advance the sustainable use of RAP and CRM effectively, efficiently and confidently are suggested; especially in terms of selecting the binder type, crumb rubber

incorporation methods, and appropriate quantity of both materials for typical Ontario asphalt mixtures. The following are specific objectives pursued in this thesis:

- Characterize the stiffness and mechanistic properties of typical Ontario conventional, RAP, and CRM-RAP HMA mixtures through laboratory performance testing.

- Investigate the impact of the "wet-process" (Terminal and Field-blending) CRM incorporation method, change in binder grade and increase in RAP content on the performance of dense and gap-graded HMA mixtures.

- Compare and contrast performance of laboratory-prepared and plant-produced recycled HMA mixtures.

- Monitor and examine selected past and newly paved rubberized pavement sections in Ontario, and use findings to validate the performance, shortcomings, quality control and construction of such pavements in practice.

- Perform an analysis of the optimal design alternative through a simple cost and sustainability assessment involving the innovative use of RAP and CRM.

1.4 Research Methodology

The objectives of this thesis were achieved through a comprehensive laboratory testing and field monitoring program. Laboratory testing was conducted at the University of Waterloo's Centre for Pavement and Transportation Technology (CPATT) state-of-the-art pavement testing laboratory with complimentary support from DBA Engineering Limited, McAsphalt Industries Limited, Miller Paving Ltd, and Golder Associates Ltd.

In addition, past and newly paved rubberized pavement sections in Ontario were examined and monitored in collaboration with the Ministry of Transportation Ontario (MTO) and the Ontario Tire Stewardship (OTS).

The research methodology consists of a detailed literature review of asphalt pavement recycling within the context of hot mix asphalt recycling and state-of-the-practice and performance review of RAP and CRM in HMA. The literature findings justify the need to verify and evaluate the effectiveness of typical Ontario HMA mixtures utilizing RAP and CRM.

1.5 Organization of Thesis

This thesis is comprised of Seven Chapters structured as follows:

- Chapter One introduces the study with an overview of the scope, objectives and summary of the adopted methodology.

- Chapter Two presents an extensive review of the literature related to recycling asphalt pavement with intent to justify the need to verify and evaluate the effectiveness of RAP and CRM as valuable components of typical Ontario HMA mixtures. The literature review is concluded with a summary of challenges, research gaps, and the opportunities for innovation.

- Chapter Three explains the research methodology in detail, describes the laboratory testing and protocols outlined for the execution of this study including sample preparation for each test method.

- Chapter Four provides a detailed description of the experimental matrix including observations from material and mix characterizations. An inventory of the field pavement sections cored, monitored and examined in this study is also detailed.

- Chapter Five presents, analyzes, compares and discusses the results from the laboratory performance testing including findings from forensic and field pavement examination.

- Chapter Six provides an overview on sustainable pavements. This Chapter also assesses and compares the cost-effectiveness and sustainability of selected research case studies involving various configurations of RAP and CRM in HMA mixtures in order to determine which design alternative is the optimal sustainable option. A brief description of the assessment and rating tools used in the analysis are also presented in this Chapter.

- Chapter Seven concludes the study with recommendations for future research.

Chapter 2
Literature Review

2.1 Introduction

This chapter presents a literature review on asphalt pavement recycling with focus on hot mix recycling. A state-of-the-art-practice and performance review of reclaimed asphalt pavement (RAP), and crumb rubber modifier (CRM) in hot mix asphalt (HMA) in Ontario is presented. The literature review is concluded with a summary of challenges, research gaps and opportunities for innovation which provide the basis for this thesis.

2.2 Recycling and Reclaimed Asphalt Pavements (RAP)

An asphalt pavement is typically composed of fine, coarse aggregates and asphalt cement or binder. The service life of an asphalt pavement is dependent on factors such as the amount and weight of traffic, climatic zone and environment, quality of materials, subgrade strength, drainage, and quality of construction. Timely and appropriate preservation and maintenance can further extend pavement life and contribute to life cycle cost savings.

The practice of recycling asphalt pavement dates as far back as the early 1900s (ARRA, 2000a). It is considered a reusable resource in the form of RAP. RAP refers to reprocessed materials generated from milling existing asphalt pavements during maintenance, rehabilitation or reconstruction operations (Copeland, 2011). RAP contains valuable aggregate and asphalt cement that can be economically substituted to produce a new recycled hot mix asphalt (HMA). It is also used as a granular base or sub-base, stabilized base aggregate and embankment or may be suitable as a fill material (Copeland, 2011). This brings about huge savings on the extraction, transportation and use of virgin materials. The practice further results in energy conservation, eliminates waste disposal concerns and ultimately contributes to environmental and sustainable benefits. Industry experts are of the opinion that almost twice as much asphalt pavements are recycled in comparison with paper, glass, plastic and aluminum combined; and as such, RAP is considered a major recycled material in North America (OHMPA, 2007).

The current annual production estimate of new asphalt pavement material in the United States is around 500 million tonnes per year. This consists of about 60 million tonnes of reclaimed material that is reused or recycled directly into pavements (Hansen & Newcomb, 2007). Since 2007, transportation agencies have reused or recycled about 40 million tonnes of RAP into other pavement-related applications per-

year, resulting in a total use of over 100 million tonnes of RAP compared to 72 million tonnes used annually in the early 1990s (Copeland, 2011).

There are several recycling techniques to address specific maintenance, preservation, rehabilitation or reconstruction needs of a deteriorated asphalt pavement. These techniques depend on the pavement performance and can be used in conjunction with each other during roadway rehabilitation projects. The Asphalt Recycling and Reclaiming Association (ARRA) classifies these techniques into five broad categories which include: Cold Planing (CP); Hot Mix Recycling (HMR); Hot-In-Place Recycling (HIR); Cold Recycling (CR); and Full-Depth Reclamation (FDR) (ARRA, 2000a). Within these categories, a number of sub-divisions are further defined. These include (ARRA, 2000b):

1. Hot In-Place Recycling (HIR)
    a. Surface Recycling (Resurfacing)
    b. Remixing
    c. Repaving
2. Cold Recycling (CR)
    a. Cold In-Place Recycling (CIR)
    b. Cold In-Place Recycling with Expand Asphalt (CIREAM) (Chan, 2010)
    c. Cold Central Plant Recycling (CCPR)
3. Full-Depth Reclamation (FDR)
    a. Pulverization
    b. Mechanical stabilization
    c. Bituminous stabilization
    d. Chemical stabilization

Depending on how and where the RAP is produced and used, the categories of recycling asphalt pavements are further classified into ‘In-Place’ and ‘In-Plant’ method. The In-Place asphalt recycling method is one where the RAP is modified and used on-site. This enables agencies to optimize the value of on-site materials, minimize construction time and disruptions to traffic flow as well as reduce vehicle emissions from long traffic queues. When compared to the time required for conventional rehabilitation methods of milling and overlaying with HMA, the In-Place method makes it possible to return the pavement to service quicker (Harrington, 2005). The In-Plant recycling method basically transports the RAP material after milling to a central hot-mix asphalt plant where the recycled mixture is produced.

Each sub-division of asphalt recycling is a study in its own right. However, this study focuses on hot mix recycling techniques, which combines RAP, virgin aggregates, new asphalt binder, and/or other recycling agents and waste products to produce a new recycled HMA for use as pavement surface and binder course applications.

2.3 Historical Perspective of RAP Utilization in Canada

In Canada, the demand for RAP use is largely driven by the increasing cost of both materials and transportation. This includes the costs of producing asphalt concrete, scarce locally available quality aggregates, and the movement toward low energy, low-emissions and environmentally friendly pavement maintenance, preservation, rehabilitation and reconstruction treatments. The first concerted efforts to recover and reuse demolished asphalt paving materials in Ontario were conducted in 1980 based on the report of the task force set up by the Ministry of Transportation Ontario to study the recycling practices in the United States (Wrong & Oliver, 1981). This program specified a maximum recycling percentage of 70 and was considered successful since it brought about economic benefits achieved through the conservation of approximately 126,000 tonnes of virgin aggregate that would have been consumed if RAP were not used (Wrong & Oliver, 1981); (Lynch & Evers, 1981).

In 1982, the province of Alberta began its recycling program in a laboratory study that investigated several mix matrixes containing 50 – 75% RAP (Anderson & Palsat, 1982). In New Brunswick, using 40% RAP in base courses between 1985 and 1987 saved almost 40,000 tonnes of asphalt cement; see Figure 2-1 (Fleming, 1987). Table 2-1 summarizes recycling percentages used in various Canadian provinces as of 1991 (Emery, 1991). The Table indicates that out of the 12 Canadian provinces and territories, nine were incorporating RAP in new HMA while one province, Prince Edward Island, was in the trial phase of incorporating RAP in to HMA. It also indicates that the highest incorporation percentage was in Saskatchewan with the lowest being in Quebec. It is important to note that at the time these statistics were obtained, both Ontario and Quebec, had four years more experience with RAP use in HMA than Saskatchewan. The results of a 2007 survey conducted by the North Carolina Department of Transportation (NCDOT) on behalf of the Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Officials (AASHTO) to determine the level of RAP use across the United States and in Ontario, Canada indicated an average use of 12 – 15% RAP in subsurface, base, and shoulder mixtures, but with restrictions in surface courses due to performance concerns (Copeland, 2011).

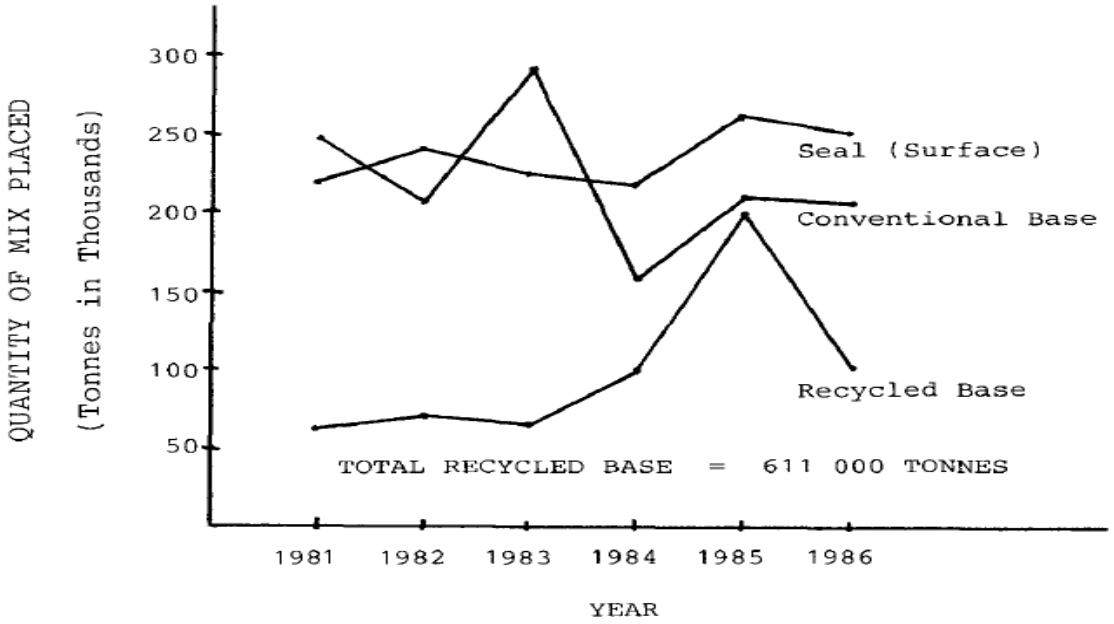

Figure 2-1: Quantity of Recycled vs Virgin Base Course Materials Placed in New Brunswick (Anderson & Palsat, Recycling of Asphalt Pavements in Alberta, 1982)

Table 2-1: RAP Percentages in New HMA in 1991 (Emery, 1991)

Province/Territory	RAP Incorporated (Y/N)	RAP Percentage
British Columbia	Y	20% - 40%
Alberta	Y	up to 40%
Saskatchewan	Y	30% - 70%
Manitoba	Y	30% - 50%
Ontario	Y	15% - 50%
Quebec	Y	15% - 30%
Prince Edward Island	N (Trial)	-
New Brunswick	Y	up to 45%
Nova Scotia	Y	up to 35%
Newfoundland	N	-
Yukon	Y	Not known
North West Territories	N	-

The United States National Asphalt Pavement Association (NAPA) established a goal to increase RAP use to an average of 25% by the end of 2013. The NCDOT survey was repeated in 2009. This survey revealed the potential for State transportation agencies to use up to 30% RAP in the intermediate pavement layers above the permitted specifications (Copeland, 2011). To validate these findings, a 2010 recycle survey conducted by the AASHTO's Subcommittee on Materials further demonstrated the interest by many transportation agencies to allow for more RAP content in the base and binder course mixes than in the surface course (AASHTO, 2010). The survey also indicated that the maximum allowable RAP content in surface mixes generally varied from 0 – 100% where testing results were adequate. In addition, the survey reported that most transportation agencies limit their surface HMA RAP content to between 11-20% with only two agencies clearly stating that no RAP was allowed in their surface course mixes. In the case of binder and base course mixes, the majority of respondents acknowledged limiting their RAP percentage to between 21-30%, while six respondents allowed greater than 30% RAP. Three survey respondents indicated that they had no limit on the amount of RAP that could be included in HMA mixes and that the percentage allowed is based on testing results. The survey further showed that the RAP percentage allowed in the shoulder HMA mixes was similar to those used in base and binder course. Ontario and Texas had the highest maximum RAP inclusion at 40%. Figure 2-2 is a histogram showing the maximum allowable RAP percentage in surface, binder and base courses (AASHTO, 2010). Figure 2-3 compares the maximum allowable and average contractor RAP use in the base course.

The amount of RAP allowed in a recycled mix and guidelines as to where such mixes can be used in a pavement structure varies by agency. Some agencies routinely allow a minimum of 15-25% RAP whereas others permit higher amounts of RAP. The use of RAP in Ontario is governed by the Ontario Provincial Standards and Specification (OPSS) 1150. The standard allows up to 20% RAP in surface course, 30% in the binder course, and up to 50% can also be used in certain situations provided testing results indicate that the recycled mix meets specifications and the contract administrator's written approval (OPSS 1150, 2010). Ontario contractors are however, usually reluctant to use more than 20% RAP by mass for surface course mixes owing to differences in asphalt cement gradation (Chan, et al., 2010). Table 2-2 shows the maximum RAP allowance in typical Ontario HMA pavements based on the design Equivalent Single Axle Loads (ESALs) (OPSS 1151, 2007). Where higher RAP concentrations

are required, it is critical that proper material evaluation, mix design, binder selection, construction, and quality control issues are addressed.

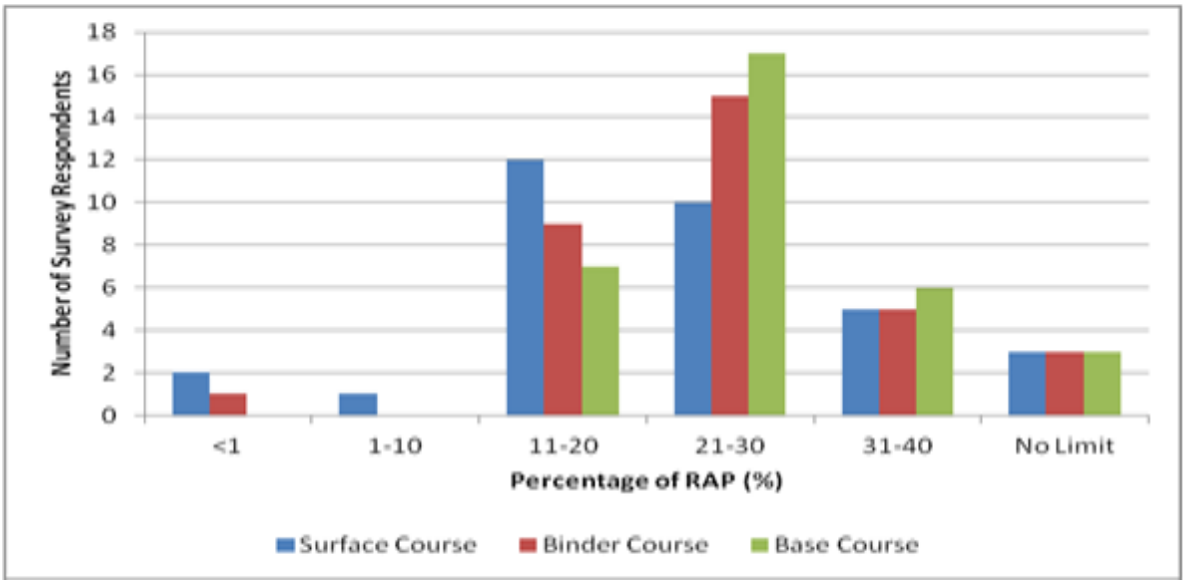

Figure 2-2: Histogram of Maximum Allowable RAP Percentage for Surface, Binder and Base Course (AASHTO, 2010)

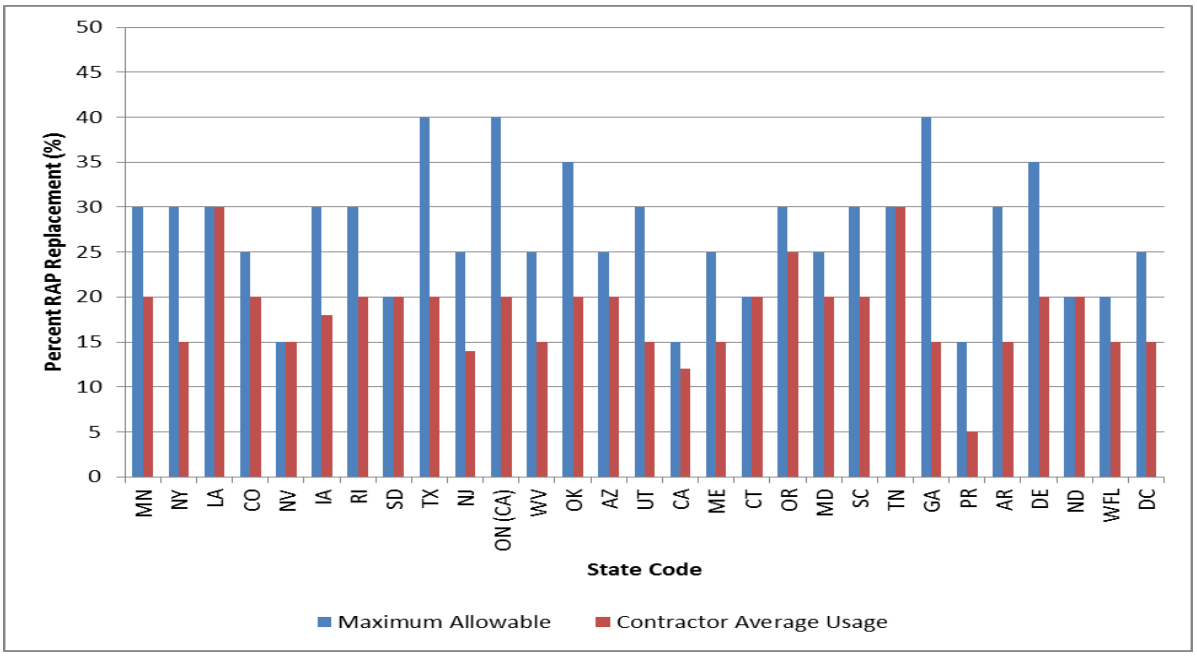

Figure 2-3: Maximum Allowable and Average Contractor RAP use in the Base Course (AASHTO, 2010)

Table 2-2: Ontario's Maximum RAP Allowance (OPSS 1151, 2007)

Traffic Category (Design ESALs)	Binder Course 150 mm or More Below Pavement Surface	Binder Course Within 150 mm of Pavement Surface	Surface Course Excluding SMA
< 3 million	40%	40%	20%
3 to 30 million	40%	40%	20%
≥ 30 million	40%	20%	20%

The current annual RAP recycling tonnage in Canada could be put at approximately 80.3 million tonnes (Aurilio, 2011). The survey of recycling state-of-practice in Canada revealed that out of the 11 provinces and territories, British Columbia, Saskatchewan and Manitoba place no limit on the usage of RAP in HMA. However, testing would be required to determine the amount of RAP that can be included in HMA. Nova Scotia and Prince Edward Island (PEI) do not consider sustainability in their pavement design and management policy nor permit RAP usage in new HMA (TAC, 2013). All five cities in the province of Ontario actively consider sustainability in their pavement design and management policy. Based on the recent survey which was conducted as part of the Transportation Association of Canada, Pavement Asset Design and Management Guide, RAP usage in Ontario is now considered common practice approximately 34 years after the first trial sections of flexible pavements containing RAP were placed (Tighe & Bland, 2010).

South Africa, Japan, Australia, United Kingdom, France, Germany and other European countries have also recorded great success in recycling asphalt pavement (Merill et al., 2004). When properly engineered and constructed, RAP performs as well as pavements built with virgin materials (Brown, 2000). Emery, also shares these thoughts based on performance and economics of existing pavement sections containing RAP. The work noted that the use of RAP in new HMA has become common practice in the Canadian pavement industry since there is no justification for HMA containing RAP to be considered inferior to conventional HMA pavements (Emery,1991). In countries such as the United States and Canada where knowledge of recycling has advanced, the present challenge is to reduce the quantity of discarded or stockpiled RAP by considering innovative means to increase the allowable RAP content in HMA. To ensure confidence in the HMA design procedure and overall success with using higher RAP percentages (> 25%) in HMA mixtures, many durability concerns related to the interaction between virgin and recycled materials must be addressed.

2.4 Recycling Scrap Rubber Tires

Scrap rubber tire is a tire that can no longer serve its original or intended purpose. It is measured in passenger tire equivalents (PTEs). A passenger tire typically consists of 47% rubber compound, 21.5% carbon black, 16.5% steel, 5.5% nylon/fibre (RPA, 2012). The historical background and development of reusing and recycling scrap rubber tires goes back more than a hundred years to a time when rubber was a scarce commodity. It is therefore fair to say that the rubber recycling industry is as old as the industrial use of rubber itself.

Each year the United States and Canada generates and stockpiles nearly 300 million scrap tires which is approximately equivalent to one passenger tire per year (RPA, 2012). While a limited number of these scrap tires are used for resource and energy recovery, the vast majority go to landfills or are disposed in an environmentally unacceptable manner (RPA, 2012). Besides occupying a large expanse of land which otherwise could be used for infrastructure and agricultural development purposes, the potential for large fires, which releases toxic chemicals and air pollutants also exists (RPA, 2012). A case in point is the February 12, 1990 Hagersviile tire fire in Ontario (Tabib et al., 2009). In addition, scrap tire stockpiles also create an ideal breeding environment for mosquitoes and other pests resulting in health complications (EPA, 2010).

The scrap tire recycling industry in Canada is managed through stewardship programs in each Canadian province. These programs are comparable in terms of structure and operation as noted in Figure 2-4 by the tire recycling industry in Canada (TRI, 2014). Since scrap tires have no economic value and recycling is not possible without government incentives, stewardship programs consisting of haulers, retailers, and processors take on the responsibility of promoting the recycling and elimination of scrap tire stockpiles. Over sixty million tires have been successfully recycled since the launch of Ontario's Used Tires Program, incorporated under the Waste Diversion Act and operated by Ontario Tire Stewardship (OTS), in September 2009 (OTS, 2014).

The recycling system begins with the generators who include tire retailers, vehicle recyclers, municipalities, landfills and tire marshalling depots (TRI, 2014). Generators collect a fee from customers for every new tire sold and remit it to the authority concerned within a stipulated time frame. Depending on the provincial regulation, the generator may or may not receive a handling allowance. As required by Ontario's amended Regulation No. 84/03, OTS has undertaken the 2014 Tire Stewardship Fee (TSF) calculation using the actual 2013 tire supply figures and Used Tires Program (UTP) costs (OTS, 2014). The next phase of the tire recycling process involves the transfer of tires from generators

to processors. Haulers are paid a fixed amount per tire by stewardship authorities depending on the type of tire and haulage distance. The processors convert the scrap tire into materials that can be used by the end-markets. The output of processors is usually crumb rubber, shredded rubber, scrap steel and fine ground rubber. Processors are paid by Stewardship Boards on proof-of-sale of products derived from scrap tires. The financial performance of processors depends on their cost structures, government support and demand from end-markets.

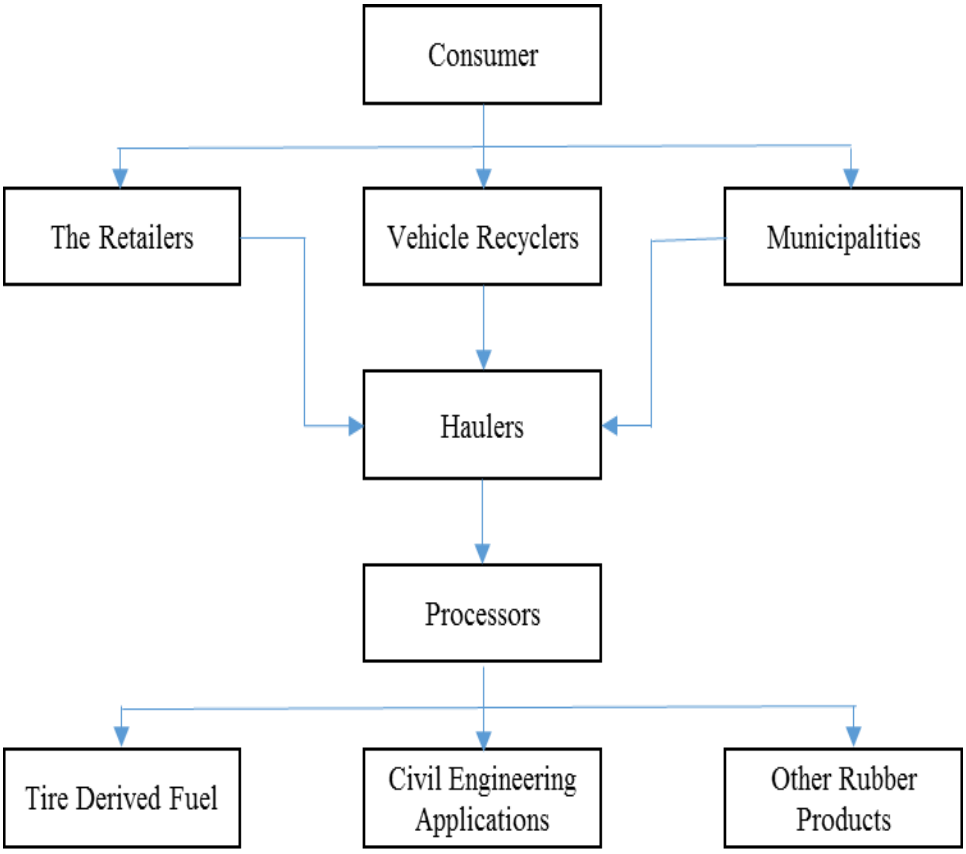

Figure 2-4: Scrap Tire Recycling Industry Structure in Canada (TRI, 2014)

Technologies such as ambient mechanical grinding, cryogenic grinding and pyrolysis are currently in use for recycling scrap tires (Caltrans, 2006). The choice of equipment depends on the desired quality of crumb rubber, which in turn depends on the end-markets that are served by the processors. The ambient technology consists of grinding scrap rubber tires at or slightly above ambient temperature using a granulator or a cracker mill. Cryogenic method consists of freezing the scrap tire rubber at temperatures near -80°C using liquid nitrogen until it becomes brittle, and then cracked into smaller particles with a

hammer mill. This process requires less energy than others and produces rubber crumbs composed of smooth, clean, flat and much finer quality. Processors who use cryogenic equipment incur higher costs both for the machinery as well as the liquid nitrogen that is used in the process.

Pyrolysis is a more technically viable recycling process aimed at recovering the original raw materials (carbon black, scrap steel, oil and hydrocarbon gases) from which the tire was made. It consists of thermal decomposition of scrap tires either in the absence of oxygen (TRI, 2014). The most common applications for scrap rubber tires use either whole or shredded tires or rubber crumbs derived from tires since the recovery of original raw materials from tires is expensive and involves an elaborate chemical process. The main end-markets for different products obtained from recycling scrap rubber tires include: crumb rubber applications; tire-derived aggregates; and fabricated products. Table 2-3 provides a description and use of each product in Canada.

Table 2-3: Scrap Tire Recycled Products and Uses in Canada (Tighe, 2011)

Product	Product Description	Uses
Crumb Rubber	Scrap tires that are processed and then ground up to various sizes to produce a coarse sand or small gravel type product	- Playgrounds instead of sand - Mulch in landscaping applications - Pour-in-place product in playground after mixing with epoxy - Raw material in molded product such as engine components - Asphalt road mixes
Tire Derived Aggregate (TDA)	Scrap tires are cut in to sizes ranging between 25 mm and 300 mm	- Subgrade fill and embankments - Backfill for walls and bridge abutments - Landfill projects - Lightweight fill - Septic system drain fields
Fabricated Products	Products that are made by cutting various parts of the scrap tire and reusing	- Base of traffic cones - Blasting mats

2.5 Recycled Hot Mix Asphalt (HMA)

A recycled HMA mixture is the product of mixing reclaimed asphalt pavement (RAP), virgin aggregates and new asphalt cement/binder (unmodified or modified), and/or other recycling agents and waste products in a hot mix plant. A wide variety of asphalt cement and aggregates are used in the production of conventional HMA mixtures. However, regardless of the source, processing method, or mineralogy, the overall goal of any HMA mix design process is to recommend a mix that can withstand the combined actions of traffic and the environment. This study is focused on HMA mixtures recycled with RAP and CRM.

2.5.1 Recycled RAP HMA Mixtures

As with the case of conventional HMA, recycled RAP mixtures should be designed properly to have similar properties and ensure proper performance. A recycling ratio is the percentage relationship between the recycled materials and virgin aggregates that make up a recycled hot mix asphalt.

2.5.2 RAP Material Characteristics

The material properties of RAP is largely dependent on the properties of the constituent materials (i.e. aggregate type, source, quality, size, and extracted binder grade). Its composition is also affected by previous maintenance and preservation activities applied to the existing pavement. Additionally, RAP from several projects is often mixed in a single stockpile where harmful materials or lower quality materials may be present. This introduces high variability in RAP material properties and thus results in a variable HMA mixture. Using low quality and/or highly variable RAP materials may eventually lead to premature failure of the HMA pavement.

RAP generally has higher dry density, California Bearing Ratio (CBR), Resilient Modulus (M_r), and field elastic modulus compared to virgin aggregates. However, the optimum water content and the maximum bulk density of RAP are lower than that of the conventional granular materials (Sayed et al., 1993).

The gradation of milled RAP is generally finer than its original gradation, and generally finer and denser than that of virgin aggregates (Mayer & Popp, 1997). In comparison to virgin aggregates, RAP aggregates have low specific gravity and high water absorption characteristics. When blended with natural aggregates for granular base use, the asphalt cement in the RAP has a significant strengthening

effect over time. It is such that specimens containing up to 40% RAP have produced CBR values exceeding 150 after one week (Hanks & Magni, 1989).

Mineral aggregates constitute the majority 93-97% by weight of RAP. Only a minor percentage 3-7% of RAP consists of hardened asphalt cement. Consequently, the overall chemical composition of RAP is essentially similar to that of the naturally occurring aggregate that is its principal constituent (FHWA, 2008). Asphalt cement is a high molecular weight aliphatic hydrocarbon compound with limited concentrations of sulphur, nitrogen, and polycyclic hydrocarbons (aromatic and/or naphthenic) of very low chemical reactivity. It contains asphaltenes and maltenes (resins and oils). Asphaltenes are more viscous than maltenes, and is a major factor in determining asphalt's viscosity. Oxidation of asphalt converts the oils to resins and the resins to asphaltenes, thus resulting in binder aging and a higher viscosity binder (Noureldin & Leonard, 1989). Table 2-4 provides a summary of the physical and mechanical properties of RAP.

Table 2-4: Physical and Mechanical Properties of RAP (FHWA, 2008)

Type of Property	RAP Properties	Typical Range of Values
Physical Properties	Unit Weight	1940 - 2300 Kg/m^3
	Moisture Content	Normal: up to 5%; Maximum: 7 - 8%
	Asphalt Content	Normal: 4.5 - 6%; Maximum Range: 3 - 7%
	Asphalt Penetration	Normal: 10 - 80 at 25°C
	Absolute Viscosity or Recovered Asphalt Cement	Normal: 4,000 - 25, 000 poises at 60°C
Mechanical Properties	Compacted Unit Weight	1600 - 2000 Kg/m^3
	California Bearing Ratio (CBR)	100% RAP: 20 - 25% 40% RAP and 60% Natural Aggregate: 150% or Higher

2.5.3 Implementing the Superpave Mix Design System for RAP HMA Mixtures

The design methods for RAP HMA mixtures are similar to conventional methods, but with necessary adjustments to best incorporate the old asphalt cement. Procedures, guidelines, and requirements for designing Superpave mixtures are described in the Superpave manual (TRB, 2005). Although the manual did not rule out the use of RAP, it also did not establish any set guidance on how to use RAP in Superpave mixtures. Consequently, the RAP Expert Task Force provided specific recommendations for inclusion of RAP in Superpave volumetric design procedure based on quantity of RAP to be used in the total mix (FHWA-ETG, 1996). Proper sampling and testing of the RAP aggregate and binder is required to determine its gradation, binder content and properties. This is to ensure that RAP materials are compatible with the virgin materials, and that the new blend satisfies the asphalt binder and resultant HMA mixture requirements. A blending chart system based on a linear relationship between the logarithm of viscosity at 60°C of the aged binder and quantity of new binder or recycling agent in the blend was developed by the Asphalt Institute as a method of estimating the RAP percentage for use in a new HMA mix (Asphalt Institute, 1989).

The selection of performance grade asphalt cement (PGAC) is based on temperature conditions, traffic loading and available materials. The Superpave system selects an appropriate binder based on the climatic conditions for a specific location with a predicted traffic speed and volume. Therefore, it is important to determine how the binder characteristics may be influenced by the percentage of RAP used. Studies on using the Superpave binder tests at high temperatures reported that a linear relationship exists between log G*/sin (δ) and percent virgin binder in a blend of virgin binder and extracted RAP binder as noted in the Pavement Recycling Guidelines for State and Local Governments Reference Book (Khandal & Mallick, 1997). Note that the expressions G^* and δ refer to the complex shear modulus and phase angle of the effective binder from the mix blend respectively.

In a similar study that investigated the relationship between G^* and percent virgin asphalt in the mix blend based on testing at 58°C, 64°C, and 70°C, a decreasing linear trend was evident for G*/sin (δ) for 0 – 75% virgin asphalt whereas it remained fairly unchanged for 75 – 100% virgin asphalt (Ceccovilli, 1996). Based on these relationships, percentage of RAP and virgin binder required to meet Superpave high-temperature binder specifications could be determined. Table 2-5 illustrates the binder selection guidelines for RAP mixtures (McDaniel & Anderson, 2001).

Table 2-5: Binder Selection Guidelines for RAP Mixtures (McDaniel & Anderson, 2001)

	RAP Percentage Recovered RAP Grade		
Recommended virgin asphalt binder grade	PG xx-22 or lower	PG xx-16	PG xx-10 or higher
No change in binder selection	<20%	<15%	<10%
Select virgin binder one grade softer than normal (e.g., select a PG 58-28 if a PG 64-22 would normally be used	20-30%	15-25%	10-15%
Recommended virgin asphalt binder grade	>30%	>25%	>15%

Typical Ontario Superpave HMA mixtures are designed in consideration of these factors whether or not recycled materials are to be incorporated. This improves durability and reduces thermal cracking potential. Based on geographical and weather information, Figure 2-5 highlights Ontario's zonal divisions with its corresponding performance graded asphalt concrete (PGAC); whereas Table 2-6 highlights Ontario's PGAC selection criteria for HMA mixtures by the design temperature and the RAP content of the mix. For consistency, the Superpave mix implementation for RAP HMA mixtures, grade change is not required for mixes with up to 20% RAP, but one grade lower PG is recommended for mixes with 21 to 40% RAP.

The PGAC designations shown in Table 2-6 are for typical traffic conditions on arterials, collectors and local roads. The PGAC high temperature grade is increased by one or two grades in order to increase the rutting resistance of HMA mixes for slow speeds, higher percentages of heavy commercial vehicles, or frequent stops and starts. The aggregates and mix properties in the Superpave design depends on the traffic loading. Ontario has five traffic categories based on the traffic loading in the amount of Equivalent Single Axle Load (ESAL) in the design lane over 20 year period: Category A (less than 0.3

million), Category B (0.3 to 3 million), Category C (3 to 10 million), Category D (10 to 30 million) and Category E (Greater than 30 million). The HMA mixtures evaluated in this study are designed for traffic category B, C and D; typically used for major collector and minor arterial roads in Ontario.

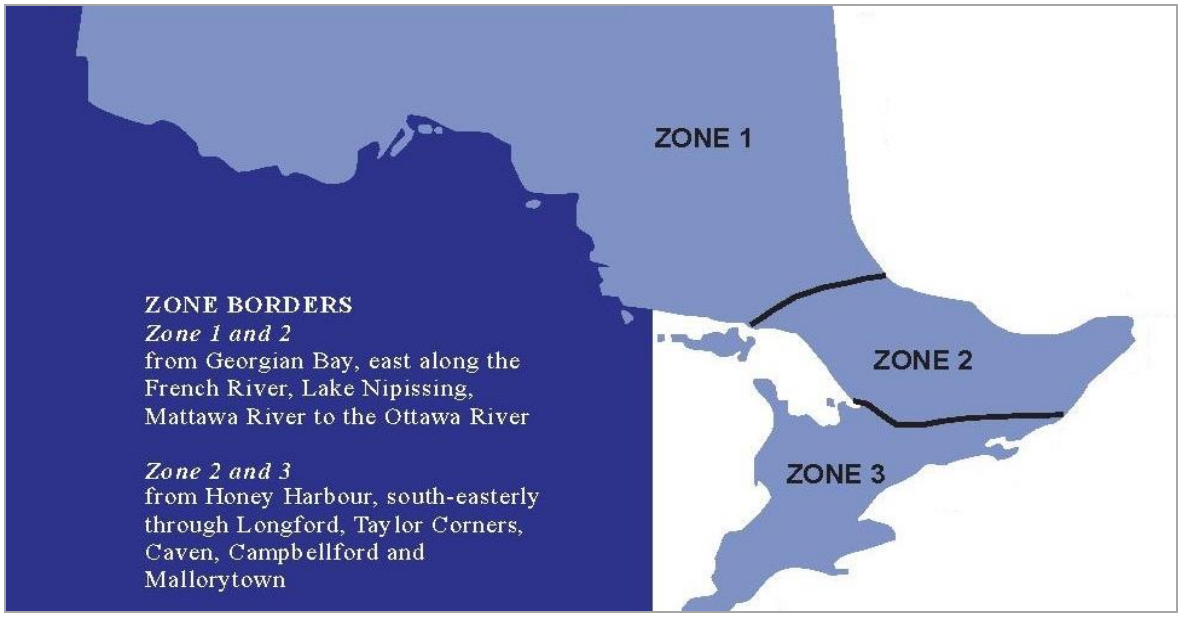

Figure 2-5: PGAC Divisions in Ontario (OHMPA, 1998)

Table 2-6: PGAC Grade Selection in Ontario (OPSS 1101, 2002)

	PGAC Zones		
	Zone 1	Zone 2	Zone 3
New Hot Mix or up to 20% RAP	52 - 34	58 - 34	58 - 28
21 to 40% RAP	52 - 40	52 - 40	52 - 34

2.6 Crumb Rubber Modifier (CRM) in HMA Mixtures

Crumb rubber modifier (CRM) is an attractive application for HMA mixtures and presents one of the most effective options for recycling scrap rubber tires. It has been used in asphalt binders for HMA since the 1960s (Epps, 1994). Incorporating crumb rubber into HMA improves the mechanical characteristics of the mix. CRM HMA mixtures are produced by thoroughly mixing the asphalt-rubber binders with other components of asphalt concrete. The two processes used to incorporate CRM into HMA are the 'wet process' and the 'dry process' (Caltrans, 2006). The wet process involves blending fine crumb rubber powders with virgin asphalt binder to form asphalt rubber binders prior to the HMA mixing operation; whereas the dry process utilizes crumb rubber particles that are coarser than those in the wet process and are considered part of aggregate gradations.

Charles H. McDonald is credited for the development of the wet process asphalt-rubber binders in the United States (Epps, 1994). During the wet process, rubber particles absorb the aromatic oils in the binder and swell, thus increasing the viscosity and stiffness of binder (Fontes et al., 2006). The wet process can accommodate dense, gap and open graded mix design. However, the dry process gives the best performance with gap-graded mixes since this distribution provides adequate space for the CRM particles in the aggregate matrix to substitute 1 to 3% of the fine aggregate (Hicks et al., 2013). Gap graded mixes do not only provide space for binder, but also increases the resistance against fatigue cracking since more binder is introduced into aggregate skeleton (Fontes et al., 2006).

The dry process does not modify the asphalt cement, but there is some potential for interactions between the CRM and the asphalt cement during mixing, silo storage, hauling, placement and compaction (Hicks et al., 2013). CRM is extensively used in Arizona, California, Florida and Texas for rubberized paving (Hicks et al., 2013). Since its early days, its use has expanded to colder regions in the United States, China, and Scandinavia (Hicks et al., 2011). Although Table 2-3 above affirms crumb rubber use for asphalt road mixes, the Canadian experience with rubberized pavements is limited since initial trial pavement sections constructed in the 1980s resulted in premature failures. This was credited to poor short-term performance of the dry process usage. However, the wet process showed potential for reduced maintenance costs for asphalt pavements and could be feasible (Emery, 1995).

Following the renewed interest into RMA pavements in Canada, the key question facing Ontario communities and, in particular, the transportation and its associated industries is not anymore whether, but rather how to eliminate, or reduce to an acceptable level, the number of scrap tires in the waste stream. Fortunately, the technology, procedures and specifications for using crumb rubber have

improved significantly since Ontario's initial trials, so the focus now is to investigate the performance and environmental soundness of using crumb rubber as a modifier in HMA as well as the recyclability of rubberized pavements.

2.6.1 Asphalt-Rubber Binder (AR)

Asphalt rubber-binder (AR) is a blend of virgin asphalt cement/binder, CRM and certain additives, in which the rubber component is at least 15 percent by weight of the total blend. Incorporating CRM in virgin binders improves the properties of virgin binder by (Roberts et al., 1996):

- Lowering the viscosity at the construction temperature to facilitate pumping, mixing and compaction of HMA.
- Increasing the viscosity at high service temperatures to reduce rutting and shoving.
- Increasing relaxation properties at low service temperatures to reduce thermal cracking.
- Increasing adhesion between asphalt binder and aggregates in the presence of moisture to reduce or prevent stripping.

Asphalt-rubber binder is also used in several surface preservation and maintenance or rehabilitation treatments in the form of Stress Absorbing Membrane Interlayer (SAMI) as rubberized fog seal and rubberized chip seal. This application has been found to minimize reflective cracking from an underlying distressed asphalt or rigid pavement and help maintain serviceability of the pavement pending rehabilitation or reconstruction (Caltrans, 2006). The performance properties of asphalt-rubber binders in HMA pavement are not always easy to predict due to the complex nature of the rubber materials and various interaction effects with different types of asphalt binders, depending on the CRM percentage, source and size of aggregates (Roberts et al., 1996).

When CRM is added to the asphalt cement, it should meet the final PG grade specified for the pavement design. Thus, the base PG asphalt cement prior to the addition of rubber may be different from the final rubber asphalt PG. One of the challenges in Ontario is related to meeting typical PG specifications for cold, moderate and warmer climate areas. It is important to note that for these areas, typical binder grades used are usually bumped due to both higher traffic levels and static or slow moving levels (Hicks et al., 2013).

The asphalt rubber binder depending on manufacturing process is classified as terminal blend or field blend. Terminal-blend refers to the type of wet process that blends about 5 to 12% CRM particle sizes less than Sieve No. 30 (0.6 mm Mesh) into the asphalt cement (AC) at temperatures between 205 and

220ºC along with a polymer; whereas field-blend asphalt rubber incorporates about 18 to 20% CRM particle sizes less than Sieve No. 10 (0.2 mm Mesh) directly into the AC before it is mixed with the aggregates (Hicks et al., 2013).

Terminal blending fully digests the rubber crumbs into the asphalt and polymer without leaving visible, discrete rubber particles or need for continuous agitation (Asphalt Institute, 2008), but in field-blending AR is maintained at temperatures between 165 and 220ºC after the mixing process for at least 45 minutes to an hour since this ensures that the blend is completely mixed (Hicks et al., 2013). This agitation process allows for the absorption of maltenes into the rubber particles and results in a binder of high viscosity. The categorization of terminal-blend as a wet process has been a controversial issue over the years. Some researchers are of the opinion that it should be grouped as an independent method of incorporating CRM into HMA mix design owing to the difference in fine rubber gradations and lesser viscosity which makes it more suitable for only dense-graded HMA mixtures (Shatnawi, 2011).

In total 5-12% CRM by total weight of binder constitutes about 0.5% by weight of the mix, and 18-20% CRM by total weight of binder is about 1% by weight of the mix. According to the "Standard Specification for Asphalt-Rubber Binder", 18-22% CRM by weight of the total asphalt binder represents high rubber content (ASTM D6114, 2009). Table 2-7 compares the various CRM incorporation technologies in terms of their advantages and disadvantages. Overall, the literature confirms that CRM-HMA typically creates more flexible, durable and safer pavements by improving frictional properties, increasing resistance against aging, reflective cracking, stripping and rutting (Fontes et al., 2006).

Table 2-7: Advantages and Disadvantages of CRM Incorporation Technologies (Tascioglu, 2013)

Technology	Advantages	Disadvantages
Wet process - Field Blend	• Offers superior performance compared with many polymer modified asphalt binders.	• Possible segregation of crumb rubber particles if not properly mixed. • Risk of mix swelling with improper crumb rubber gradation. • This affects compactability and maintain voids specifications.

Wet process - Terminal Blend	• Offers superior performance compared with many polymer modified asphalt binders. • No segregation of crumb rubber particles. • Can be hauled for long distance	• More expensive compared with the other processes. • Utilizes less crumb rubber particles.
Dry Process	• Good skid resistance and de-icing properties. • Less expensive, and utilizes more crumbs.	• Anti-oxidants are not completely mixed with the binder.

2.7 Performance Evaluation of RAP-HMA Mixes

Performance evaluation of HMA mixtures containing different RAP percentages (both field and laboratory testing) have been piloted with results indicating that properly designed recycled HMA mixtures have performed comparably to conventional HMA. RAP pavement sections constructed in 1982, 1983 and 1984 exhibited a decreasing progression in reflective cracking, and an identical rutting performance compared to virgin sections (McMillian & Palsat, 1985). However, hardening of the RAP stockpile and high emissions produced during the production of recycled mixes were issues identified at the time. Owing to the comparable performance of recycled asphalt pavements and conventional pavements, use of RAP is considered a viable option for rehabilitation of asphalt pavements in Alberta (Anderson et al., 1989). In service evaluation of 10-25% RAP pavement sections were found to be performing satisfactorily after 1.5 to 2.25 years with no significant rutting, raveling and weathering, or fatigue cracking issues (Khandal et al., 1995).

The mix design process and selecting the correct asphalt binder grade is critical to the cracking potential of mixes that do or do not contain RAP. In terms of mix durability, RAP mixtures have shown better resistance to the action of water and slower rate of ageing compared to virgin mixes since the RAP binder has already undergone oxidation and tends to retard the rate of hardening (Meyers et al., 1983); (Kiggundu, et al., 1985).

The literature confirms that RAP increases the mix stiffness and improves the rutting performance, but provides inconsistent fatigue and thermal resistive performance when compared with virgin mixes (Tam,

et al., 1992); (MacDaniel et al., 2000); (Sargious & Mushule, 1991); (Huang et al., 2004). The increased stiffness of the aged RAP binder causes an increase in the modulus of blend which in turn affects the mixtures fatigue and low-temperature cracking.

A 2005 study observed higher modulus and creep compliance curves for 15% RAP mixes; whereas at 25% and 40% RAP concentrations the modulus were comparable to those of the control mix for both tension and compression. The cause of these unexpected results were linked to a combination of gradation, asphalt content and volumetric properties (Daniel & Lachance, 2005).

An evaluation of the moduli effects when 15% RAP was added to HMA using asphalt cement type PG 64-22 and PG 58-28 for 25% and 40% RAP indicated some differences only at high temperatures for 40% RAP content and the control mix (Shah et al., 2007). The results of dynamic and resilient modulus testing indicated that while the phase angle of the mix decreased, the mix stiffness increased with increasing RAP percentages implying that a reduction in phase angle corresponds to an increase in the elastic properties and a reduction in the viscous properties of the mix (Sondag et al., 2002).

An examination of the effects of RAP percentages and sources on HMA properties revealed that as the percentage of RAP in a mix increased, there was increased variability in the dynamic modulus values at lower temperatures (Li et al., 2004). Performance findings from a 1992 RAP study within different regions in Ontario and using different asphalt binders and recycling ratios (60/40, 30/70, 50/50, 70/30 and 25/75), found five road projects constructed within the period 1981 and 1983 to be more prone to thermal cracking in comparison to conventional HMA (Tam et al., 1992).

Incorporating 50% RAP into HMA with a softer binder decreased the rutting potential and increased the potential for low-temperature cracking, but showed that an increase in RAP was accompanied by an increase in tensile strength ratio (TSR) (Gardiner & Wagner, 1999). Note that the TSR is a measure of the mix resistance to moisture damage. The TSR for the mix containing 50% RAP and a softer binder was the same as that of the virgin mix.

In 2008, Widyatmoko prepared wearing and base course mixes with 10%, 30%, and 50% RAP - contrary to what is known in existing studies, the mixtures containing RAP showed lower resistance to permanent deformation compared with control mixtures (Widyatmoko, 2008). One of the few studies that evaluated the resistance to moisture damage and thermal cracking of field and laboratory-prepared HMA mixtures containing 0%, 15% and 50% RAP noted the following observations (Loria et al., 2011):

- Laboratory-prepared samples containing 50% RAP and asphalt cement (AC) type PG 58-28 did not meet low temperature performance grade of -28°C before failure; the recorded fracture

temperatures of all samples were between 5 - 8°C lower than the critical low temperature of the recovered binder.

- For samples with 15% RAP, it was determined that no change in binder grade was necessary since both the high and low temperature performance grades were either met or exceeded.
- The fracture temperature for the 0% and 15% RAP contents were similar to the critical low temperature for the recovered asphalt binder.
- In both field and laboratory-prepared samples, fracture stress generally increased as RAP content increased.

On fatigue life of HMA mixes with RAP, tests conducted for the National Cooperative Highway Research Program (NCHRP) 9-12 study confirmed increased mix stiffness and reduction in fatigue life for asphalt concrete with RAP content greater than 20% (McDaniel & Shah, 2003).

2.8 Experiences with CRM-HMA Mixes

Considering the importance of CRM-HMA in the recycling of scrap rubber tires, U.S States such as California, Texas, Arizona, and Florida have become leaders in RMA pavement applications. New York, New Mexico, Nebraska and South Carolina are continuously improving their experiences with the technology (EPA, 2014). Maine, Colorado, Minnesota and New Jersey have also included crumb rubber in their research studies, but the initial construction cost of the CRM-HMA mixtures was too high although performance was comparable with conventional mixes.

Overall, studies into CRM-HMA mixes have reported that the wet process performs better than the dry process (Coubane et al., 1998). A summary of some state practices and experiences with RMA pavements is presented in Table 2-8.

Table 2-8: RMA Pavement Practices and Experience Summaries

State/County	Mix Type and Application	Experience/Remarks
Arizona (FHWA, 1995)	Wet Process in open and gap-graded mixes with CRM binder variations of 6 - 10%.	Excellent performance
California (Caltrans, 2006)	Wet and Dry Process in dense, open and gap- graded mixes with CRM binder contents of 6 – 8% and 14 – 20%.	Early trials showed good Performance with wet process, but sections had rutting and bleeding

		problems. Moisture damage related issues were noted with the dry process. Recent RMA pavements sections are in very good condition.
Florida (FHWA, 1995)	Wet Process in open and dense graded mixes with binder contents varying between 6.5 – 7.1%.	Excellent performance
Marion County (Amirkhanian, 2001)	Wet and Dry Process	Dry Process showed some deterioration after 8 years. However, the wet process sections are still in good condition.
New York (VanBramer, 1997).	Dry process mixes of 1, 2 and 3% rubber gradations.	These did not perform better than conventional pavement sections after 5 years of monitoring.
New Mexico (Bandini, 2011)	Dry process Rubberized open graded friction course overlays with the wet process.	These sections failed. These sections showed very good performance after 4 years with no rutting or cracking.

The experience and placement of RMA pavements in Canada is limited compared to those of California, Arizona, Florida and Texas, which span several decades. This lack of experience is summarily attributed to initial poor performance, cost effectiveness and inexperience with the technology. Initial trials in Ontario with the dry process for pavements placed between 1980 and 1995 to assess the environmental acceptability, economic feasibility and recyclability of scrap tires in to HMA showed generally moderate to poor performance (Tabib et al., 2009). Some of these failed sections were plant recycled, but results of the recycled pavement were similar to the original results, with widespread rutting, stone loss and raveling shortly after being opened to traffic (Aurilio, 1993); (Emery, 1994). A 1994 review of 11 Ontario rubberized sections noted that roller pick up had been a serious construction issue (Emery,

1994). Wet process mixes considered in the Town of Kirkland in 1994 though at an additional initial cost, indicated no significant problems and was considered a technical success (Carrick et al., 1995).

Performance evaluation of a number of wet and dry process test sections in 1997 listed the wet process to have performed as well or slightly better than the conventional HMA sections, while the dry process generally performed worse than sections constructed with conventional HMA (Tabib et al., 2009). This study which included visual inspections, distress surveys, deflection testing, frictional testing and pavement profile measurements were sponsored by the Ministry of the Environment (MOE) and the MTO. The poor performance of the dry process compared to the wet process were attributed to insufficient asphalt cement, and designers not taking the minor interactions between the virgin binder and the CRM particles into account (Tighe, 2011). The CRM-binder interaction consists of a physical exchange where the crumb rubber through diffusion absorbs the aromatic fraction of the bitumen binders resulting in the swelling of the crumb rubber particles (Nuha et al., 2014). The increased viscosity of the resulting binders is attributed to the particle swelling behaviour in combination with the reduced oily fraction of the virgin binder.

Emery, reported that the RMA pavements placed with the dry process mixes of crumb rubber concentrations 2% higher by weight of coarse aggregates retained on the 4.75 mm (No.4) sieve did not perform as well as conventional DGAC pavements since those sections showed early raveling, pop outs, and cracking along construction joints. However, the dry process mixes made with crumb rubber concentrations of 1 to 1.5% by weight of fine aggregate passing the 2 mm (No. 10) sieve were comparable in performance to conventional DGAC pavements (Emery, 1997).

In continuation of the 1997 study, a long-term performance survey of the rubberized asphalt sections focused on collecting information on the current traffic volume, the date of resurfacing and the pavement condition prior to resurfacing, as well as other general comments. The survey findings indicated that many of the pavements had been resurfaced in 2003. However, due to limited information, no firm conclusions could be drawn regarding performance (Tabib et al., 2009); (Hicks et al., 2013).

In 2008, the MTO constructed two 500 m RMA test sections incorporating the moist process with ambient and cryogenic crumb rubber on Highway 15 North of Smiths Falls. The moist process is similar to dry process, but utilizes crumb rubber particles smaller than 600 μm (0.6 mm) in size. These sections were the first rubber modified Superpave mixes in Ontario, and noted several construction challenges including slight segregation, visible fumes, increased coring difficulty, and air void inconsistencies. A longer evaluation period is needed to provide any conclusive results (Tabib et al., 2009). The province

of Saskatchewan, Alberta, and British Columbia have also indicated wide interest in using rubber tires in asphalt pavement applications. Their experiences are detailed in the Rubber Modified Technical Manual prepared for the Ontario Tire Stewardship (Hicks et al., 2013).

The general conclusion from assessing RMA pavements in Ontario is that the dry process is feasible, but would require further development of mix designs and construction procedures to achieve the desired level of performance. Also, it was concluded that CRM mixes made using wet process could be engineered to perform as well or better than conventional HMA pavements.

2.9 Laboratory Evaluation of CRM-HMA Mixes

The laboratory performance evaluation of CRM-HMA mixtures is of interest to many paving agencies and academic institutions. Significant findings from recent studies relating to the physical properties, thermal cracking, permanent deformation (rutting) and fatigue cracking behaviour of CRM-HMA mixtures are reported below.

2.9.1 Physical Properties

An Arizona study compared the material properties of gap-graded and open-graded rubberized HMA mixtures from 11 projects, and several conventional dense-graded asphalt mixtures (DGAC). The study found that CRM increased the performance grade (PG) of the virgin binder by at least one level, and had higher viscosities at higher temperatures and lower or unchanged viscosities at lower temperatures (Rodzeno & Kaloush, 2009).

Pasquini, reported that the addition of asphalt rubber to virgin binders resulted in higher softening point and higher viscosity, and better performance of the AR-mixture (Pasquini et al., 2011). Gopal, concluded that CRM could improve the low-temperature properties of the binders if carefully designed and evaluated for each combination of crumb rubber size, content and binder type (Gopal et al., 2002). Putman noted that the crumb rubber size has a strong effect on the viscosity of CRM binder produced with the ambient process, and a lesser influence on the failure temperature (Putman et al., 2005).

Dynamic modulus test results for confined and unconfined samples showed higher unconfined dynamic modulus for conventional HMA mixtures compared to gap-graded asphalt-rubber mixtures. The latter had higher modulus than asphalt-rubber open-graded friction course mixtures regardless of the test temperature and frequency. On the other hand, the confining level and temperature affected the modulus values. Confining increased the modulus of all asphalt-rubber mixtures and test conditions, but had no

significant effect on the modulus of conventional mixtures at low temperatures and a slight increasing effect at higher temperatures (Rodzeno & Kaloush, 2009). Under a confining stress of 138 kPa, the gap-graded asphalt-rubber HMA mixtures had equal or higher modulus than conventional mixtures, especially at test temperatures of 38°C and 54°C. Experimental results have further indicated that asphalt-rubber HMA mixtures have significantly less moisture sensitivity compared to conventional asphalt mixtures (Bandini, 2011).

2.9.2 Thermal Cracking

Thermal cracking is purely a tensile failure of the material, the resistance of the asphalt concrete to thermal cracking is mainly provided by the binder. Higher tensile strength has been generally associated with higher thermal cracking resistance. A study discovered that gap-graded mixes with 20% rubber in the binder had higher tensile strength using the indirect tensile strength (ITS) test, and higher energy to failure (Pasquini et al., 2011). This study also concluded that the AR mixture had a higher thermal cracking resistance compared to gap and dense graded conventional HMA mixes. However, results from other laboratory experiments have indicated that AR-mixes have significantly lower tensile strength compared to conventional dense-graded HMA mixes (Kaloush et al., 2002); (Zborowski & Kaloush, 2007). Laboratory experimental studies have equally observed better thermal cracking resistance in asphalt-rubber mixtures (Raad et al., 1993); (Epps, 1997).

2.9.3 Permanent Deformation

Documented experimental studies report that the crumb rubber content in the binder contributed to the better rutting resistance noted for comparisons between gap-graded asphalt rubber mixtures and dense-graded HMA mixtures, regardless of the relatively high asphalt-rubber content in the former (Kaloush et al., 2002); (Wong & Wong, 2007); (Fontes et al., 2010); (Pasquini et al., 2011).

In an experimental study that compared a dense-graded conventional HMA mix with four gap-graded asphalt-rubber mixtures containing 15 and 20% crumb rubber by weight in the binder, it was discovered that the asphalt-rubber mixes had better resistance to permanent deformation, with higher softening points, and much more superior cumulative plastic strain than the conventional mixes (Fontes et al., 2010). A 1996 study also noted that reducing the particle size of the crumb rubber in the asphalt-rubber modified binder in a dense-graded mix design considerably increased the rutting resistance, but resulted

in permanent deformation values that were comparable to those of mixtures designed for high-temperature regions to reduce rutting (Coomrasamy et al., 1996).

2.9.4 Fatigue Cracking

Indirect tensile tests results indicated that the fatigue resistance with respect to the number of cycles to failure for a gap-graded asphalt-rubber mixture was similar to that of a stone mastic asphalt mixture, but greater than that of a polymer-modified asphalt mixture (Pasquini et al., 2011). In another study comparing two wet process asphalt-rubber modified mixtures to a dry process mixture, and a conventional mixture, the wet process was reported to have had greater fatigue resistance in comparison to the dry process and much greater fatigue resistance to the conventional mix (Gallego et al., 2007). Miranda also reported good fatigue resistance for open-graded and gap-graded asphalt-rubber mixtures with high crumb rubber contents (Miranda et al., 2008).

2.10 Summary of Challenges, Research Gaps and Opportunity for Innovation

This chapter has provided a review of the literature relevant to the research presented in this thesis. The findings confirm that across the U.S and in Canada, recycling and use of RAP in HMA has gained widespread acceptance. However, current specifications restrictions, technical constraints, and performance concerns rather than a lack of RAP availability remain limitations to increasing the percentages of RAP in HMA.

Although substantial work has gone into the development of mix design procedures to incorporate RAP into HMA mixtures with tests to determine the effects of such inclusions on the virgin asphalt cement and overall mix performance, these tests have mostly focused on lower RAP percentages. To restore the confidence of Ontario's pavement contractors and transportation agencies in exploring higher RAP-HMA usage, it is necessary to fully characterize the extent of blending between the aged RAP and virgin binders at different RAP percentages as well as conduct investigations into how different RAP sources, contents, and quality affect the performance of the new HMA especially at low-temperatures.

As seen from the literature, test results confirm that the stiffness of RAP tends to increase resistance of an asphalt mix to rutting, but decreases resistance to thermal and fatigue cracking. The literature further confirms that several performance properties are improved by adding crumb rubber to hot mix asphalt. However, in Ontario, the challenges with CRM mix designs, incorporation method, quality control and construction practices for rubber modified asphalt (RMA) pavements still exist.

This has led to poor or less-than-expected performance of asphalt rubber pavements sections. In addition, the performance of typical Ontario HMA mixtures containing both RAP and CRM has not been extensively investigated. Confidence could also be given to higher reclaimed asphalt pavement (RAP) practices in Ontario by incorporating crumb rubber in RAP-HMA mixtures.

This combination is capable of compensating for RAP shortfalls such as its effects on binder aging and mix stiffness, thus improving the mixture's susceptibility to moisture damage and stripping and overall resistance to rutting, fatigue and thermal cracking.

The lessons learned from many pilot and routine RMA projects both successful and unsuccessful projects in addition to exploring higher RAP percentages in typical Ontario HMA using the Superpave mix design system presents opportunities for innovation. As part of the renewed interest in RMA in Ontario, loose CRM-RAP HMA mix samples have been taken from several MTO projects constructed in 2011 alongside pavement cores from Highway 15 for laboratory performance testing in this study.

Chapter 3
Research Methodology

3.1 Introduction

The primary objective of this study is to evaluate the effectiveness of varying percentages of RAP, and CRM in conventional Ontario Superpave HMA mixtures. This study has mainly been conducted through laboratory performance testing of an array of laboratory-prepared and plant-produced HMA mixes, and on cored pavement sections in Ontario. The research methodology is outlined in Figure 3-1.

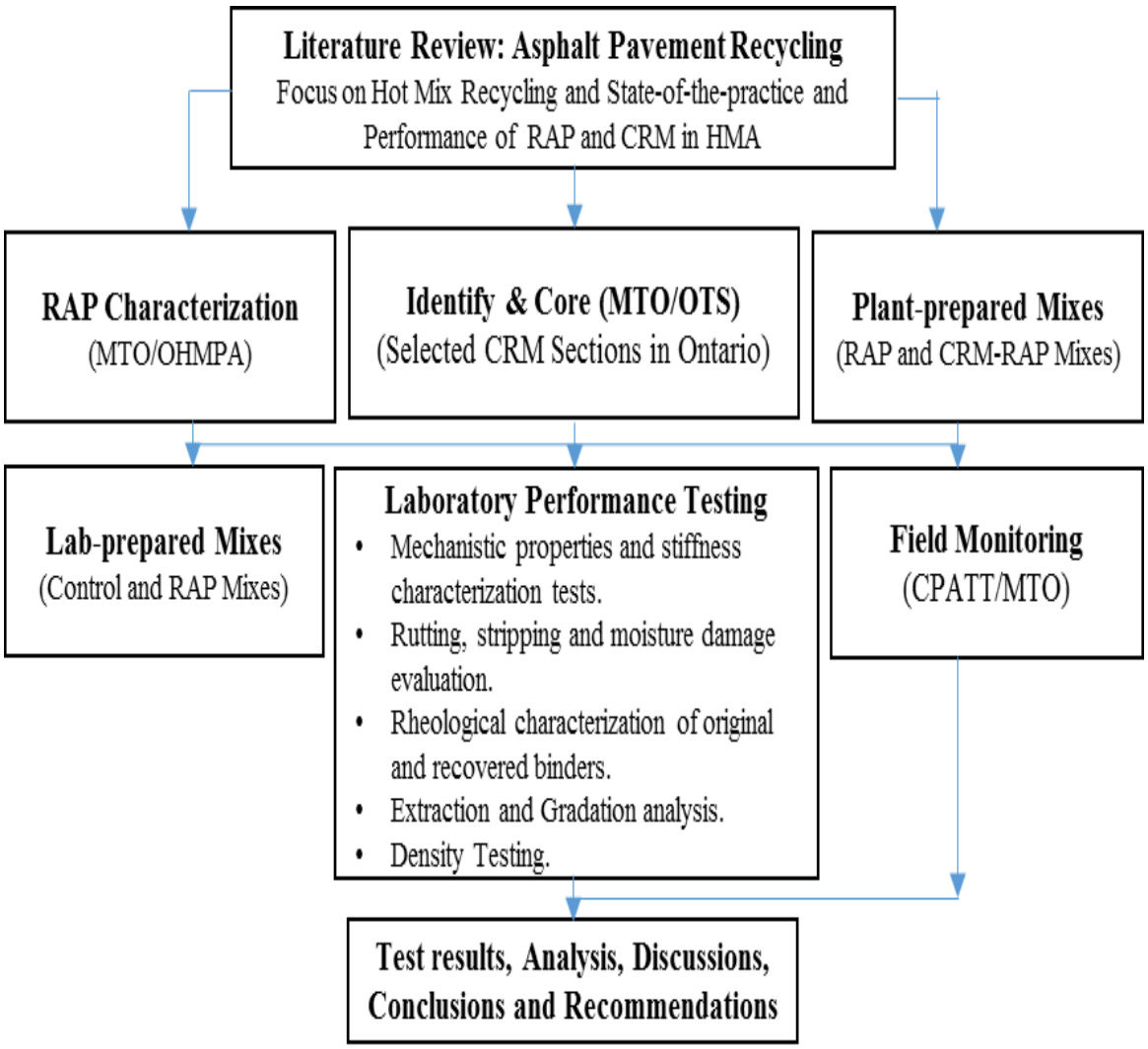

Figure 3-1: Research Methodology

3.2 Explanation of Research Tasks

This section explains the tasks associated with this study.

3.2.1 Task 1: Literature Review

The first task in this study involved a comprehensive literature review on asphalt pavement recycling with particular emphasis on hot mix asphalt recycling and state-of-the practice, and performance of RAP and CRM-HMA mixtures in North America. The intent was to identify deficiencies, research gaps and opportunities for innovation. The literature findings presented in Chapter Two of this thesis justify the need to verify the impact high RAP and CRM contents have on typical Ontario HMA mixtures. It further highlights the need for comparative studies on the performance effects of both materials, so that essential guidelines for handling, processing, and general best-practices for HMA mix design procedures can be recommended.

3.2.2 Task 2: Material and Mix Characterization

The second task involved collecting various RAP samples for characterization. This was achieved through collaboration between the Ministry of Transportation Ontario (MTO) and the Ontario Hot Mix Asphalt Producers Association (OHMPA). RAP characterization requires fundamental evaluation of the basic material properties from extraction, recovery and determination of the performance grade (PG) of the asphalt cement (AC) in the RAP including the RAP aggregate gradation and consensus properties. The continuous PG of the virgin binders were also determined. Findings from these characterizations and the impact on mix designs are presented in Chapter Four of this thesis.

3.2.3 Task 3: Identification and Coring

A third task for this research involved identification and coring of sections from selected rubber modified asphalt pavement sections in Ontario, and the associated control sections. This task was achieved through collaboration with the MTO and Ontario Tire Stewardship (OTS). The cored pavement sections, taken from Highway 15 North of Smith Falls, Ontario are detailed in Chapter Four of this thesis, and were evaluated using a rigorous material characterization framework.

3.2.4 Task 4: Plant HMA Mix Collection

Task four of this study involved collecting samples of plant-prepared HMA mixtures which incorporated RAP and CRM with its associated RAP control HMA mixtures for laboratory performance evaluation.

These mix samples were collected from MTO's 2011 demonstration sections placed on Highways 7, 35 and 115 in Ontario. A detailed description of each mix design is presented in Chapter Four of this thesis.

3.2.5 Task 5: Laboratory HMA Mix Preparation

Task number five involved laboratory preparation HMA-mixtures of varying RAP configurations (control 0% RAP, 20% RAP, and 40% RAP). These HMA mixtures were prepared with asphalt cements typically used in Southern and Northern Ontario, and in accordance with mix-designs provided by DBA Engineering Limited. A detailed description of each mix is provided in Chapter Four of this thesis.

3.2.6 Task 6: Field Pavement Monitoring

Task six involved a pavement condition survey of the in-service pavement sections on Highways 7, 35 and 115 incorporating the plant-prepared HMA mixes. This involved visual pavement monitoring by CPATT and use of the Automated Road Analyzer (ARAN) by the MTO. MTO uses the ARAN equipped vehicle to collect pavement performance data for rut depths, roughness, and cross section profiles. The ARAN research project was launched in 2004 to investigate the feasibility of incorporating automated pavement distress in collaboration with CPATT and three major Canadian pavement engineering consultants. This technology has the potential to improve pavement assessments by replacing slow, subjective manual investigations with high-speed automated surveys.

3.2.7 Task 7: Laboratory Performance Testing

This task is the most comprehensive component of the research. It involved performance testing on the laboratory and plant-prepared HMA mixtures including the cored samples. All laboratory testing were conducted at CPATT's state-of-the-art pavement testing laboratory at the University of Waterloo. These tests as identified in the research objective and structural layout were completed as follows:

1. Determine the mechanistic properties and characterize mix stiffness.
2. Evaluate the susceptibility to rutting, moisture damage and potential for stripping.
3. Characterize the interaction between aged RAP binder and crumb rubber on virgin binder to determine failure properties.
4. Evaluate the differences in performance between terminal and field-blend CRM-HMA mixtures.
5. Compare laboratory-prepared and plant-produced HMA mixtures.

The primary tests for mechanistic properties and stiffness characterization in this study include: Dynamic Modulus testing for elastic property determination at various temperatures and frequencies, Thermal Stress Restrained Specimen Test (TSRST) for fracture property evaluation at very low temperatures, and the Hamburg Wheel Tracking Device (HWTD) to investigate the combined effects of rutting, moisture damage and stripping potential. The binder properties from the various mix configurations were determined through rheological characterization after being extracted and recovered. The various test methods, protocols and procedures are detailed in section 3.3 of this Chapter. Tests on the cored pavement sections taken from Highway 15 North of Smith Falls, Ontario include:

A. Binder Extraction and Gradation analysis:

Trichloroethylene solvent was employed in extracting the binders from in-service pavement cores to determine if insufficient binder content could be attributed of the poor performance for the sections. Gradation analysis was conducted to help identify whether the cause of poor performance for the sections could be related to mix gradation.

B. Density Testing:

Density tests were completed to determine whether the poor performance of the RMA sections could be attributed to insufficient compaction resulting from either mix workability or poor construction practices. Density testing involves measurement of bulk relative density of the cores and maximum relative density of the loose mix. Air void was testing was also conducted on Highway 15 cores in accordance with 'MTO' Laboratory Testing Manual, Test Method LS-262 (MTO, 1999) and LS-264 (MTO, 2009).

The method required determining the bulk relative density (G_{mb}) of the compacted specimen in accordance with LS 262 and relating this value to the maximum relative density (G_{mm}) as determined from the loose mix sample described in LS 264 by equation 3.1 and 3.2. Note that maximum relative density, bulk relative density and air void testing was equally completed on loose HMA mixtures and compacted test specimens from the respective experimental matrix in this study.

$$AV = \frac{Gmm - Gmb}{Gmm} \text{ X } 100\% \qquad \text{(Equation 3-1)}$$

$$G_{mb} = G_{mm} (1 - AV) \qquad \text{(Equation 3-2)}$$

Where AV = Air voids (%), G_{mm} = Maximum relative density (g/cm^3), G_{mb} = Bulk relative density (g/cm^3).

C. Rutting Characterization:

Rutting resistance testing was completed to determine the combined effects of rutting, moisture damage and stripping potential. Results from this test were used to assess performance between cryogenic, ambient rubber modified sections and control sections.

3.2.8 Task 8: Analysis and Reporting

In this task, results obtained from Tasks 2, 6 and 7 are presented, analyzed, compared, discussed and reported in Chapter Five. A cost and sustainability assessment on the applicability and impacts of utilizing RAP and CRM-HMA mixtures also forms a part of the analysis and discussions, and are reported in Chapter Six of this thesis. Following the outcome of Chapters Five and Six, Chapter Seven summarizes this study with conclusions, recommendations and areas for future research.

3.3 Description of Test Protocols and Sample Preparation

This section describes the tests methods/protocols outlined to characterize the behaviour and mechanistic properties of the HMA mixtures under study. Sample preparation method for each test is also detailed.

3.3.1 Thermal Stress Restrained Specimen Testing (TSRST)

The TSRST was developed through the Strategic Highway Research Program (SHRP) and has been identified as an accelerated performance test to evaluate the low temperature susceptibility of asphalt paving mixtures. Testing was conducted in accordance with the American Association of State Highway and Transportation Officials (AASHTO) test protocol TP 10-93 *"Standard Test Method for Thermal Stress Restrained Specimen Tensile Strength"* (AASHTO TP-10, 1993), using CPATT's MTS-810 test equipment which consists of a MTS-651 environmental chamber, liquid nitrogen tank, temperature controller and a resistance temperature device.

The set-up cools down a beam specimen while restraining it from contracting. The cooling process is performed by vaporizing compressed liquid nitrogen into the environmental chamber through a solenoid valve. The cool air is circulated with a fan so air is evenly distributed whereas the resistance temperature device is connected to the controller monitors the temperature in the environmental chamber and regulates the amount of liquid nitrogen required to reach or maintain a specified temperature. As the temperature drops, thermal stresses build up until the specimen fractures.

This study conducted tests on four replicates TSRST specimens for the respective experimental mix matrixes. Each measuring 250 mm x 50 mm x 50 mm at 7± 1% air voids. These test specimens were sawn out of compacted beams measuring 390 mm × 125 mm × 78 mm using an Asphalt Vibratory Compactor (AVC) with an applied vibration force of 115 kPa. Figure 3-2 illustrates the AVC Setup. Loctite E20 NS hysol epoxy adhesive was applied on the surface of two cylindrical aluminum platens to bond top and bottom ends of the test specimen. The epoxy bond is allowed to cure for at least 12 – 16 hours prior to conditioning the test specimen at 5°C in the MTS-651 environmental test chamber for six hours. Actual testing is performed at a monotonic cooling rate of 10°C/hr.

Figure 3-2: CPATT Asphalt Vibratory Compactor (AVC) Setup with Compacted Beam Sample

Extensometers attached to the specimen senses movement during cooling and contraction, and sends a signal to the computer system, which in turn causes the hydraulic actuator to stretch the specimen to its original length. The thermal stress in the specimen increases as fracture temperature decreases gradually in the environmental chamber. Test results are reported in terms of the maximum stress at which the specimen fails (i.e., the fracture stress) with a corresponding fracture temperature. A Typical TSRST setup is shown in Figure 3-3.

Figure 3-3: CPATT TSRST Testing Setup

3.3.2 Hamburg Wheel Tracking Test Device (HWTD)

The Hamburg Wheel Tracking Device (HWTD) is used to evaluate the combined effects of rutting, stripping potential and moisture susceptibility of compacted HMA mixtures (Asphalt Institute, 2010). HMA mixtures with weak aggregate structure, inadequate binder stiffness, moisture damage, and inadequate adhesion between aggregate and binder fail prematurely with this test procedure.

The device tracks a 705 ± 4.5 N load steel wheel back and forth across the surface of a pair of 150 mm x 62 mm diameter-height ratio gyratory compacted HMA cores submerged in a hot water bath at 50°C for 10,000 cycles which is equivalent to 20,000 passes or until a rut depth of 20 mm is reached (Asphalt Institute, 2010). The wheels have a diameter of 203 mm, a width of 47 mm, and are capable of generating 50 passes per minute at a maximum speed of 0.305 m/s (AASHTO T324-04, 2008). Laboratory compacted specimens are prepared with a 7 ± 2% air voids; whereas field specimens are tested at the air void content at which they are obtained (AASHTO T324-04, 2008).

In Figure 3-4, Linear Variable Differential Transducers (LVDTs) measure the rut depth or deformation on each specimen to an accuracy of 0.01 mm. Duration of the test is approximately seven hours, including initial conditioning time of 30 minutes. However, for some tests the specimens fail early and test times are shorter. Figure 3-5 highlights a typical rut plot obtained from the HWTD. It indicates post-compaction consolidation on the HMA core after 1,000 load cycles. Post-compaction consolidation results from wheel densification (Kansas DOT, 2014).

Also noticeable on the plot is a creep slope and a stripping slope. The creep slope is the portion on the plot where rutting occurs due to consolidation and plastic flow; whereas the stripping slope is related to the severity of the damage due to moisture.

The point at the number of passes where the creep slope and stripping slope intercept is termed stripping inflection point (SIP). This is related to the mixture's potential to moisture damage, (Yildrim et al., 2007); (Asphalt Institute, 2010); (Pavement Interactive, 2011). Creep slope is preferred for evaluating rutting potential rather than rut-depth because the number of load cycles at which moisture damage begins to affect rut depth varies between HMA mixtures and cannot be conclusively determined from the rut plot (Asphalt Institute, 2010). Past studies indicate that the HWTD has been shown to have excellent correlation with field performance; especially with respect to moisture damage evaluation (Williams & Prowell, 1999), (Izzo & Tahmoressi, 1999).

The effects of rutting, moisture damage and stripping potential in this study were evaluated in accordance with the *'Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot Mix*

Asphalt (HMA)' (AASHTO T324-04, 2008). Results for rut depths measured on laboratory compacted specimens are compared with the National Center for Asphalt Technology (NCAT) suggests a rut-depth criteria of less than 10 mm after 10, 000 cycles for conventional dense-grade HMA mixes (Asphalt Institute, 2010) whereas those determined for field cores, and from pavement condition surveys using the automated road analyzer (ARAN) are compared with MTO's suggested rut-depth criteria of less than 6 mm for very slight severity on field pavements (Chong et al., 1989).

Figure 3-4: CPATT HWTD Testing Setup

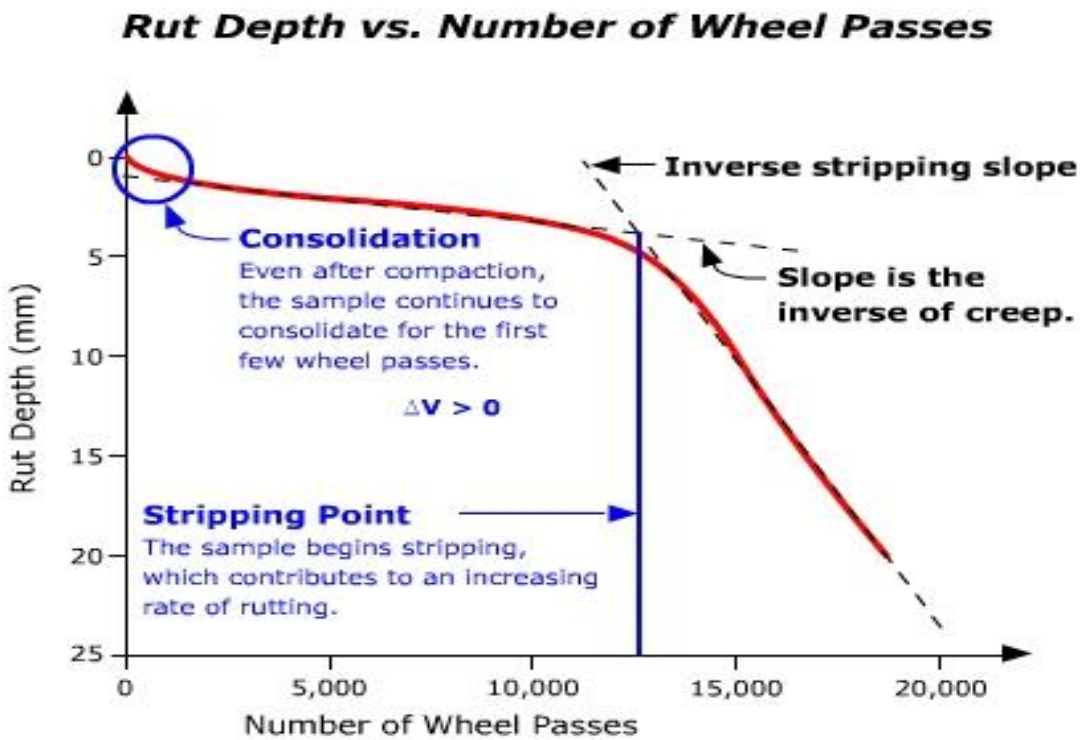

Figure 3-5: Typical Plot from a HWTD Showing Key plot Parameters (Pavement Interactive, 2011)

3.3.3 Dynamic Modulus Testing

The dynamic modulus relates the stress-to-strain performance for linear viscoelastic materials such as hot mix asphalt subjected to continuous sinusoidal loading in the frequency domain. The evaluation of complex modulus tests requires an understanding of linear viscoelasticity concepts. Ferry, (1980) describes the fundamental concepts of linear viscoelasticity. Figure 3-6 is a schematic of dynamic modulus test for one-dimensional case of a sinusoidal loading (Ferry, 1980).

The complex modulus (E*) is represented as the ratio of the amplitude of the sinusoidal stress (at any given time, t, and angular load frequency, ω), $\sigma = \sigma_0 \sin(\omega t)$ and the amplitude of the sinusoidal strain $\varepsilon = \varepsilon_0 \sin(\omega t - \delta)$, at the same time and frequency, that results in a steady state response as expressed in equation 3.3. The viscous or elastic behavior of HMA is indicated by the phase angle (δ), which is the angle by which strain (ε) lags behind stress (σ). For a pure elastic material, $\delta = 0^o$, and for a pure viscous material, $\delta = 90^o$ (Witczak et al., 2002). The in-phase and out-of phase components defines the storage

modulus and the loss modulus as expressed in equations 3.4 and 3.5 respectively. Equations 3.6 and 3.7 expresses stress and the resulting strain in complex forms. The complex modulus is defined as a complex quantity as shown in equation 3.8. The real part of the complex modulus is the storage modulus and the imaginary part is the loss modulus. The ratio of the peak stress to strain amplitudes defines the absolute value of the dynamic modulus as expressed in equation 3.9 (Pellinen & Witczak, 2002).

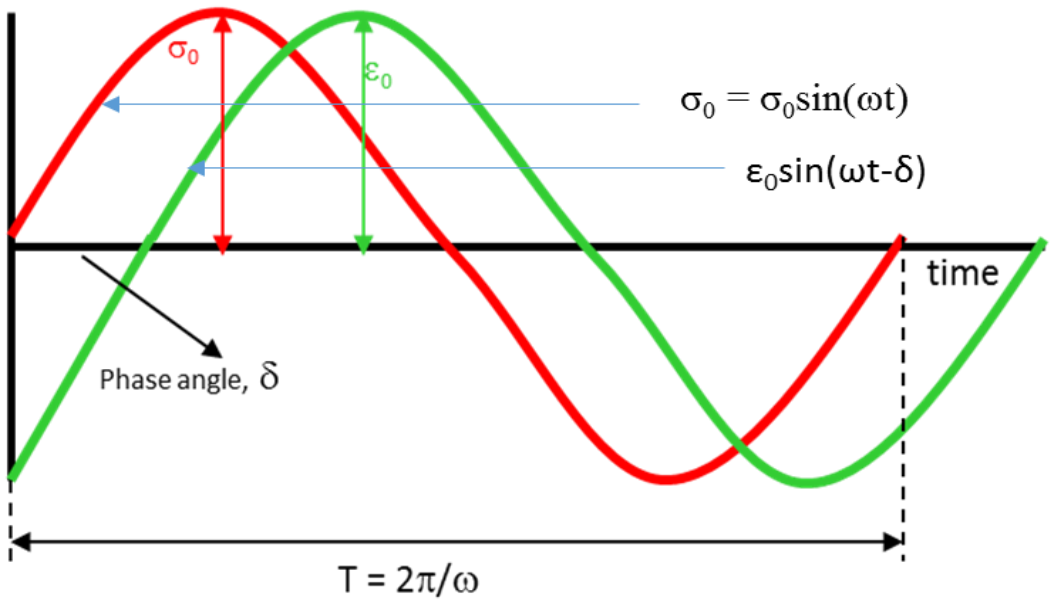

Figure 3-6: Schematic of Dynamic Modulus Test (Pavemaintainance, 2011)

$$|E^*| = \frac{\sigma}{\varepsilon} = \frac{\sigma o \sin(\omega t)}{\varepsilon o \sin(\omega t - \delta)} \qquad \text{(Equation 3-3)}$$

$$E' = \frac{\sigma o \cos(\delta)}{\varepsilon o} \qquad \text{(Equation 3-4)}$$

$$E'' = \frac{\sigma o \sin(\delta)}{\varepsilon 0} \qquad \text{(Equation 3-5)}$$

$$\sigma^* = \sigma_o e^{i\omega t} \qquad \text{(Equation 3-6)}$$

$$\varepsilon^* = \varepsilon_o e^{i(\omega t - \delta)} \qquad \text{(Equation 3-7)}$$

$$E^*(i\omega) = \frac{\sigma *}{\varepsilon *} = \frac{\sigma o}{\varepsilon o} e^{i\delta} = E' + iE'' \qquad \text{(Equation 3-8)}$$

$$|E^*| = \frac{\sigma o}{\varepsilon o} \qquad \text{(Equation 3-9)}$$

Where:
E^* = Complex modulus
$|E^*|$ = Dynamic modulus (absolute value of complex modulus)
δ = Phase angle (degrees)
i = Imaginary number
ω = Angular load frequency
t = Time of loading
σ_0 = Peak stress amplitude (applied load / sample cross sectional area)
ε_0 = Peak amplitude of recoverable axial strain ($\Delta L/L$)

The dynamic modulus test is critical because of its sensitivity to the changes in the HMA mixture volumetric and binder stiffness. An increase in stiffness of the asphalt mix is depicted by an increase in the dynamic modulus values which is a reflection of a decrease in strain corresponding to an applied load. Similarly, a decrease in the dynamic modulus results indicate an increase in strain and is interpreted as a decrease in the stiffness of the asphalt mix (El-Hakim, 2013).

In this study, dynamic modulus testing was performed to characterize stiffness for each mix matrix. Triplicate cylindrical specimens compacted, cored and cut to 150 mm x 100 mm height to diameter ratio with air void content of 7 ± 1% were test per mix. Testing was conducted in accordance with AASHTO TP 62-07 test protocol *"Standard Method of Test for Determining Dynamic Modulus of Hot Mix Asphalt (HMA)"* (AASHTO TP 62, 2007). The specimens were tested at five temperatures (-10, 4, 21, 37, and 54°C) and six frequencies (25, 10, 5, 1, 0.5, and 0.1 Hz). For each frequency, specified load cycles (200 cycles for 25 Hz, 200 cycles for 10Hz, 100 cycles for 5 Hz, 20 cycles for 1 Hz, 15 cycles for 0.5 Hz, and 15 cycles for 0.1 Hz) were applied.

The testing procedure involved applying a sinusoidal axial compressive stress to the cylindrical specimen using a Material Testing System (MTS) 810 with 22 Kip (1000KN) capacity. A MTS 651 environmental chamber provided test temperature controls; whereas applied stress measurements and the corresponding recoverable axial strain response over the specified range of frequencies and temperatures were obtained from the average of three 100 mm gage length linear variable differential transducer (LVDTs) attached to the magnetic studs on each specimen. These responses were used in calculating the dynamic modulus values. Teflon sheets lubricated with high vacuum grease were placed between the specimen ends and the 150 mm diameter steel loading platens to reduce the effects of friction and risk of lateral displacement. Table 3-1 highlights AASHTO's recommended 5% dynamic load applied for testing at the various temperatures.

A CPATT dynamic modulus test setup is shown in Figure 3-7. The set of experimental data obtained were used to generate and compare master curves between the HMA mixtures investigated based on AASHTO TP62-07 specification. The methodology of developing master curves is based on time-temperature superposition principle. This principle allows for the conversion of test data collected at different temperatures and frequencies into a specific reference temperature (usually 20 or 21ºC); thereby aligning the various curves to form a single master curve. This conversion or shifting process also results in the calculation of a reduced frequency with a corresponding shift in the stiffness values (AASHTO TP 62, 2007). In the master curve, a low reduced frequency represents high-temperature testing, whereas a high modified frequency on the master curve represents the low-temperature testing of the dynamic modulus samples (El-Hakim, 2013). The RHEA™ (Rheology Analysis) software was employed in the construction of master-curves for the various mix stiffness in the time-frequency domain based on the time-temperature superposition principle with automatic shifting routines.

Table 3-1: AASHTO's Recommended Dynamic Loading (AASHTO TP 62, 2007)

Temperature (°C)	**Dynamic Stress (kPa)**		**Dynamic Load (KN)**		**5%**
-10	1400	2800	11	22	**-1.1**
4	700	1400	5.5	11	**-0.55**
21	350	700	2.7	5.5	**-0.27**
37	140	250	1.1	2	**-0.1**
54	35	70	0.3	0.5	**-0.03**

Figure 3-7: CPATT Dynamic Modulus Test Setup

The dynamic modulus of the asphalt mixture is considered to have good correlation with the occurrence of rutting and fatigue cracking pavement distress over a diverse range of climatic and traffic conditions (NCHRP, 2002), (Pellinen & Witczak, 2002). A lower stiffness at low temperatures or high loading frequencies will help to minimize fatigue and low temperature cracking. To reduce rutting potential, a higher stiffness at high temperatures or low loading frequencies is desirable.

3.3.4 Rheological Characterization of Asphalt Binders

Asphalt binder, the principal binding agent of asphalt-aggregate mixtures is a visco-elasto-plastic material. At elevated temperatures, it is a viscous liquid and at freezing or cold temperatures it is an elastic-solid (Asphalt Institute, 2010). This behaviour is critical to the mechanical or failure properties of HMA mixtures especially in terms of rutting, thermal and fatigue cracking pavement distresses.

Rutting performance of pavements is related to the rheological properties of the binder at high temperatures whereas fatigue cracking is related to its intermediate temperatures. The low temperature properties of the binder is related to thermal cracking of the pavement.

Through rheological characterization, this mechanical or failure property is investigated by subjecting asphalt binder to various loading or deformation conditions. The rheological parameters of the original and recovered asphalt binders from mixtures in this study were determined to investigate the impact of the RAP and CRM compositions on mix stiffness properties as well as to better match this behavior to pavement distresses. To achieve this, the asphalt binder Performance Grade (PG) system for Superpave HMA mix design was employed in the grading of both original and recovered asphalt binders.

The binder grade in the Superpave system is specified by two numbers, for example PG 58-28. The first number, 58, represents the average 7-day maximum pavement design temperature in degrees Celsius, ºC at which the binder is intended to perform adequately to resist rutting; whereas, the second number, -28ºC, represents the minimum pavement design temperature at which the binder is intended to resist thermal cracking (Solyemani et al., 2004), (Varamini et al., 2014).

The rheological parameters measured from grading the binders in this research include: the total resistance to deformation represented by complex shear modulus (G*), relative non-elasticity of the binder represented by phase angle (δ), flexural creep stiffness (S), and rate of stress relaxation (m-value).

A combination of the G* and sin δ captures the contribution of asphalt binder in rutting susceptibility of HMA mixtures. Increasing the G*/sin δ parameter makes the binder stiffer and more elastic, and thus more resistant to rutting. A minimum value of 1.0 kPa and 2.2 kPa is specified for G*/sin δ for unaged and aged binders respectively (AASHTO MPI-93, 1996). G* and δ values of virgin and recovered binders from RAP mixtures in particular are also necessary to attain proper blending charts for RAP HMA mix design (McGennis et al., 1995); (OPSS 1101, 2002); (Varamini et al., 2014).

On the other hand, parameters S and m-value are related to the binder's properties to thermal cracking resistance. Binders with relatively lower values of creep stiffness will exhibit fewer thermal cracks in cold weather. Likewise, higher m-values show the ability of binder to absorb stress in the event of temperature drops, and these tend to exhibit less cracking tendency (McDaniel & Anderson, 2001).

This phase of testing was completed in collaboration with DBA Engineering Limited, Golder Associates Ltd and McAsphalt Industries Ltd. DBA Engineering Limited was responsible for determining the unaged and aged properties of the base binders employed in the laboratory-prepared HMA mixtures.

Golder Associates was responsible for determining the properties of the recovered binder for both laboratory-prepared and plant-produced HMA mixtures. Extraction of the RAP and RAP-CRM binders were done using the solvent Normal propyl bromide (nPB) while recovery consisted of heating the mixture and distilling the solvent using a Rotovap. The rheological parameters of virgin and recovered binders were measured with a Dynamic Shear Rheometer (DSR) as per AASHTO T315 (AASHTO T315, 2012). Short-term aging of the binders was accomplished in a Rolling Thin-Film Oven (RTFO) as per AASHTO T 240 (AASHTO T240, 2013) while a pressure aging vessel (PAV) in accordance with AASHTO T315 was used for tests on residue to simulate the long-term aging of the binder (AASHTO T313, 2012). The true grade of the recovered binders was determined as per AASHTO R-29 (AASHTO R-29, 2012). This allowed for characterization of the binders critical high, intermediate, and low-temperatures.

3.4 Analysis of Experimental Results

The experimental data obtained from performance characterization tests were observed, and then statistically analyzed to investigate the effects or impact of varying RAP contents, or combining RAP and CRM for the evaluated HMA mixtures. Statistical analysis employed include an Analysis of Variance (ANOVA), the F-test, and t-test. A description of these statistical analysis is provided as follows (Montgomery, 2001), (Ddamba, 2011):

A null hypothesis (H_o) is paired with an alternative hypothesis (H_1) to examine the variability of the alternative hypothesis. If H_o: $\mu_1 = \mu_2$ does not reject the null hypothesis then performance is consistent. However, if Ho: $\mu_1 \neq \mu_2$, the null hypothesis is rejected. Montgomery notes that two types of errors are possible with these hypotheses. These include:

- Type I error where null hypothesis is rejected when it is true.
- Type II error where null hypothesis is accepted when it is false.

The general procedure in hypothesis testing involves specifying a probability value for Type I error α often referred to as significance level of the test. The significance level (α) or confidence level (%) determines the degree of evidence at which the difference or variability in the variables is unlikely to have arisen by chance. A 95% confidence level ($\alpha = 0.05$) was used in the study. The level of significance and degree of freedom were also used to interpret the critical values from both the F-test and t-test tables.

Analysis of Variance (ANOVA), is a statistical tool used for measuring the relative difference between means for different data sets. This was used to analyze mixture performance. The F-test is a statistical test comprising of an F-distribution under the null hypothesis test statistics. This was used to compare the statistical models with fitting data sets. The test is designed to determine if two population variances are equal by comparing the two variances and identify the best model that best fits the population (in this case mixes with better performance) using least squares method. The F-distribution is non-negative and non-symmetrical distribution. The hypothesis testing was performed on the assumption that the null hypothesis (i.e. mixes combining RAP and CRM) will perform better than those with or without RAP.

A null hypothesis assumption was equally considered for 40% RAP mixes performing better than those with 15 and 20% RAP or without RAP. The F-test considers the variability in terms of the sum of squares reflecting the different source of variation. The sum of squares (SS) tends to be greater when the null hypothesis is not true hence SS have to be statistically independent for the F-distribution under null hypothesis to follow. F-value was calculated as shown in Equation 3.10.

$$F_{Calculated} = \frac{S^2{}_{Control\ Mix}}{S^2{}_{Alternative\ Mix}}$$

(Equation 3-10)

Where: S^2 is the variance of either control or alternative mixes.

If the $F_{Calculated} > F_{Critical}$, the H_o is rejected concluding that there were differences in the HMA mixes and it is in favor of the alternative mix whereas if $F_{Calculated} < F_{Critical}$, a weak conclusion could be drawn or indicates lack of statistical significant evidence of variation. In this case the control and alternative variables are statistically observed to be consistent with each other.

The F-Test was used in validating if there were any significant change in performance or data if recycled material is added to HMA mixes. The t-test Analysis is a statistical hypothesis test following the T-distribution and/or normal distribution (when value of the scaling term in the test statistics is known). The null hypothesis test was used to study the difference between the responses (by the design mixes) on the same statistical unit assuming the mean value was zero. An independent one-sample t-test was employed for the study whereby the null hypothesis was tested to examine whether the alternative mix mean (μ_1) was equal to the control mix mean (μ_o) and t-value was calculated using Equation 3.11.

$$t_{Calculated} = \frac{(\mu_1 - \mu_o)}{S/\sqrt{n}}$$

(Equation 3-11)

Where: μ_o – Control Mix Mean, μ_1 – Alternative Mix Mean

S – Standard Deviation, n – Sample Size

If the $t_{Calcualted} > t_{Critical}$, the null hypothesis was rejected concluding that there was difference in the mixes and that the alternative mix was better suited for use in hot-mixed asphalt pavement type whereas if $t_{Calcualted} < t_{Critical}$, then we fail to reject the null hypothesis. This indicates a weak statistical evidence of difference or lack of significant statistical difference in the mixes hence strong evidence of consistence of the alternative mix with the control mix. The t-test was used in testing the significance of increasing the percentages of RAP or combining RAP and CRM in conventional Ontario HMA mixes. Table 3-2 summarizes all variables that have been taken into consideration for this research.

Table 3-2: Research Variables

Mix Category/Type	Materials	Source/Processes/Methods	Tests/Analysis
Category: - Laboratory-prepared - Plant-produced - Pavement Cores	RAP	Source: Premium RAP	Tests: Rheological Characterization of base and recovered binders Mechanistic and Stiffness Assessments on HMA Mixtures: - Rutting - Thermal Cracking - Fatigue
Type: - Dense-Graded - Gap-Graded	CRM	Wet Process: - Field-blend - Terminal-blend	
	Binder (PG)	Method: - Ambient - Cryogenic	Cost and Sustainability Analysis: - Rating Assessment - Economics - Environmental (Co_2, Water and Energy Usage)

3.5 Summary

This chapter has detailed the methodology employed in this research to fulfill the objectives as outlined earlier. The study involves performance testing of the HMA mix and determination of asphalt binder failure properties from rheological characterization.

Master curves will be plotted and compared for each experimental matrix from dynamic modulus testing to highlight improvements made by incorporating CRM and RAP in such mixtures. Cores extracted from in-service pavement sections incorporating the moist process with ambient and cryogenic crumb rubber will be subjected to forensic testing to determine the cause of poor performance.

Field sections from which the plant HMA samples were taken have also been monitored in this study for correlation and verification purposes. The resistance to low-temperature cracking and combined effects of rutting, moisture damage and stripping potential are also investigated.

Performance testing and comparisons made with laboratory prepared HMA incorporating RAP is mainly intended at evaluating the performance of high RAP usage in such mixtures including possible incorporation of CRM into such mixtures. Statistical analysis are employed to validate experimental data as well as compare performance between the evaluated HMA mixtures. A cost and sustainability assessment of various sustainable design alternatives will also be discussed.

Chapter 4
Description of Experimental Matrix

4.1 Introduction

This chapter details the experimental matrix investigated in this research. The experimental matrix consists of loose HMA mixtures with various percentages of RAP and CRM configurations including cored pavement sections.

4.1.1 Loose HMA Mixtures

The test program considered both laboratory-prepared and plant-produced loose HMA mixtures. A total of 19 Ontario HMA mixtures consisting of 12 laboratory-prepared and 7 plant-produced with binder performance grades (PG) covering a wide stiffness range typically used across colder, moderate and warmer climate zones in Ontario were examined in this study.

The HMA mixtures include: four dense-graded conventional HMA mixtures, a dense-graded HMA mixture with 15% RAP, six dense-graded HMA mixtures with 20% RAP, four dense-graded HMA mixtures with 40% RAP, a dense-graded CRM terminal-blend HMA mix containing 20% RAP, and three gap-graded CRM field-blend HMA mixtures incorporating 20% RAP. A description of the HMA mixtures in both laboratory-prepared and plant-prepared categories are detailed in Table 4-1 and Table 4-2 respectively. Figure 4-1 and Figure 4-2 highlight plots of the gradation envelope for the respective laboratory-prepared and plant-produced HMA mixtures. It should be noted that the materials in this study satisfies all relevant Ontario Provincial Standards and Specifications (OPSS) requirements.

The laboratory-prepared HMA mixtures L1, L3, L4, L8, L10 and L12 followed a rigorous Superpave mix design provided by DBA Engineering Ltd. This design was replicated by CPATT for the HMA mixtures L2, L5, L6, L7, L9 and L11 in an order that maintained the same aggregate structure and asphalt cement content. Prior to mixing, the aggregates were sieved, dried and batched. The amount of RAP required for all batches of a single mix design was separated by size and oven dried at 30ºC over the weekend or for at least 48 hours. The virgin aggregates were oven dried at 110ºC overnight.

For ease of mixing, 15 kg batches were prepared in accordance with each mix design. The virgin batches were kept in the oven at 160ºC for 16 hours, the RAP batches were dried at 60ºC overnight while the virgin binder was pre-heated to the mixing temperature before mixing. Approximately two to three minutes were required to complete the mixing. This was completed to ensure that the aggregates were

thoroughly coated with asphalt binder. For quality control (QC) test samples were collected from random batches and maximum relative density (G_{mm}) tests were performed.

Table 4-1: Composition and Volumetric of Laboratory-prepared HMA Mixtures

Mix ID	PGAC	Mix Type	RAP Content (%)	Virgin AC (%)	RAP AC (%)	Total AC (%)	DP	VMA (%)	VFA (%)	TSR (%)	G_{mm}	Mixing Temp. (°C)	Compaction Temp. (°C)
L1	58-28	DGAC	0	5.2	-	5.2	0.7	14.8	73.1	96.2	2.532	145	134
L2			20	4.3	0.9	5.2	1.2	15.5	74.2	98.7	2.525		
L3			40	3.3	1.8	5.1	1.1	14.3	72.1	92.1	2.526		
L4	52-34		0	5.2	0	5.2	0.7	15	73.4	96	2.537	140	129
L5			20	4.3	0.9	5.2	1.2	15.0	73.3	98.7	2.521		
L6			40	3.3	1.8	5.1	1.1	14.8	73.0	94.7	2.519		
L7	58-34		0	5.2	-	5.2	0.7	14.8	73.0	96.2	2.520	151	139
L8			20	4.3	0.9	5.2	1.2	14.7	73.1	98.7	2.515		
L9			40	3.3	1.8	5.1	1.1	15.0	73.4	95.1	2.513		
L10	52-40P		0	5.2	-	5.2	0.7	14.1	71.7	96.2	2.541	150	140
L11			20	4.3	0.9	5.2	1.2	14.3	72.1	96.2	2.519		
L12			40	3.1	1.8	4.9	1.1	14.2	71.5	91.5	2.534		

Table 4-2: Composition and Volumetric of Plant-produced HMA Mixtures

Mix ID	PGAC	Mix Type	RAP Content (%)	Virgin AC (%)	RAP AC (%)	Total AC (%)	DP	VMA (%)	VFA (%)	TSR (%)	G_{mm}	Compaction Temp. (°C)
H7-C	58-28	DGAC	15	4.3	0.94	5.2	0.8	15	73.0	84.7	2.629	138
H35-C			20	4.1	0.98	5.1	0.9	15.2	74.0	83.8	2.646	135
H7-RTB				4.2		5.2	1	15.1	74.0	81.7	2.625	140
H7-RFB		GGAC		6.0		7.0	0.7	19.9	80.0	90.8	2.584	145
H35-RFB				5.6		6.6	0.6	18.6	79.0	84.5	2.59	145
H115-RFB	64-34P			5.2		6.2	0.7	18.2	78.0	93.4	2.618	155
H115-C	64-34	DGAC		4.0		5.0	0.9	15.5	74.0	101.4	2.659	150

***Note:** All mixtures are Superpave (SP) 12.5 mm Nominal Maximum Aggregate Size (NMAS) except H7-RFB which is a SP 9.5 mm mix; PGAC = Performance Graded Asphalt Cement; DGAC = Dense-graded Asphalt Concrete Mixtures; GGAC = Gap-graded Asphalt Concrete Mixtures; RFB = Wet-process rubber field-blend; RTB = Wet-process Rubber Terminal-blend; VMA = Voids in Mineral Aggregates, VFA = Voids Filled with Asphalt; L=Laboratory-prepared HMA, H = Highway No. (Plant-produced HMA); DP = Dust Proportion; and TSR = Tensile Strength Ratio.

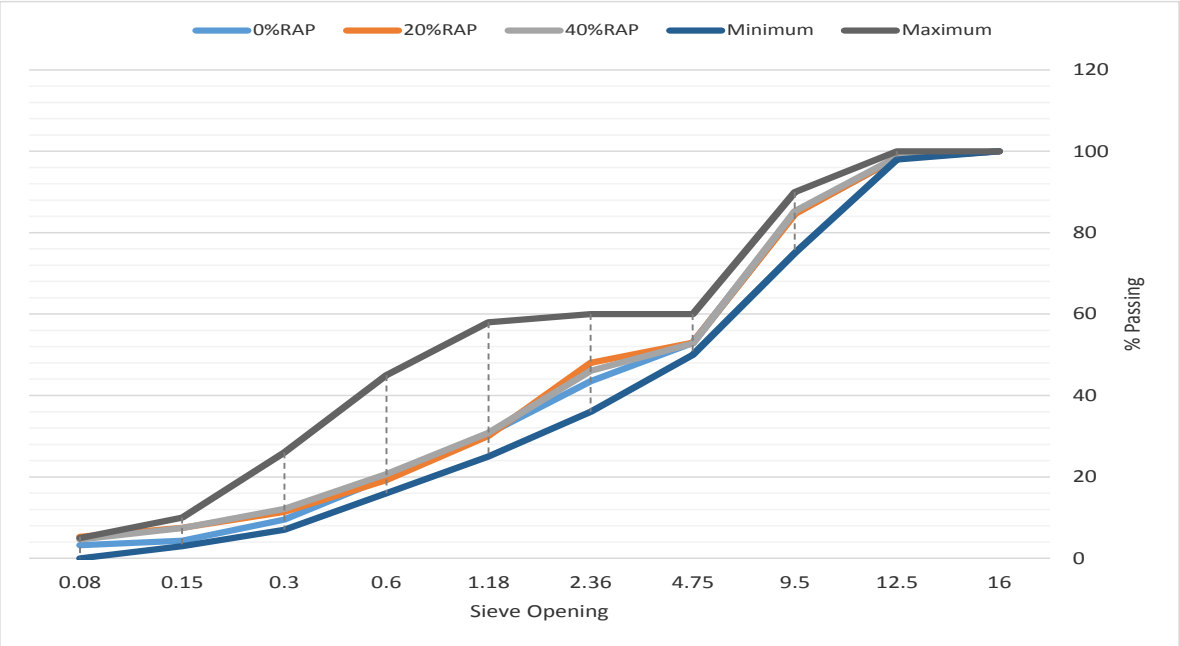

Figure 4-1: Laboratory-prepared HMA Gradation Envelope

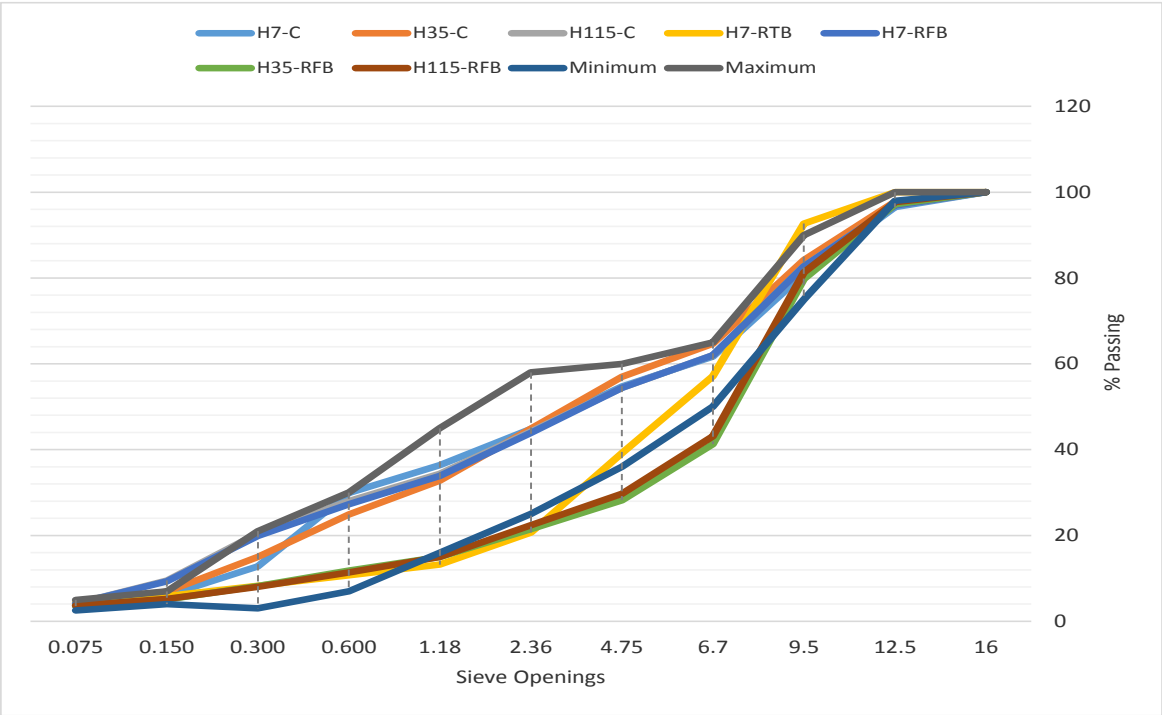

Figure 4-2: Plant-produced HMA Gradation Envelope

The plant-produced HMA mixtures were collected during construction for different demonstration sections on Highways 7, 35 and 115 in Ontario. Table 4-3 summarizes these projects. The terminal blend mixes contain crumb rubber particles smaller than the No. 30 (0.6 mm) mesh size, and constitute 10 to 12 percent by total weight of binder (about 0.5 percent by weight of the mix) whereas field blended mixes contain crumb rubber particles smaller than the No. 10 (2.0 mm) mesh size, and constitute 18 to 20 percent by total weight of binder (about 1% by weight of the mix). Table 4-4 compares the particle size distribution of crumb rubber samples for the wet-field blend HMA mixtures for tests conducted by CPATT and the MTO.

Table 4-3: MTO Rubberized Project Summaries, October 2011 (Hicks, Tighe, Tabib, & Cheng, 2013)

Highway	Location	Lane Direction	Mix Description	Lane (km) Placed
7	North Road to Sideline 22	Both	Rubber Field-blend SP 9.5 mm Gap-Graded	6
	Sideline 32 to North Road		Rubber Terminal-blend SP 12.5 mm FC2 Dense-Graded	5.6
	Sideline 22 to Brock Road		Hot Mix Asphalt SP 12.5 mm FC2 Dense-Graded	2.8
35	Drum Rd to Ballyduff Road	Both	Rubber Field-blend SP 12.5 mm FC1 Gap-Graded	6
	Ballyduff Road to Highway 7A		Hot Mix Asphalt SP 12.5 mm FC1 Dense-Graded	13.5
115	Highway 7A to Regional Road 10	South Bound Only	Rubber Field-blend SP 12.5 mm FC2 Gap-Graded	6
	Regional Road 10 southerly 1 km		Hot Mix Asphalt SP 12.5 mm FC2 Dense-Graded	2

Table 4-4: Grain Size Distribution of Crumb Rubber Samples

Process	Sieve Size	Percent Passing Specification	Percent Passing (CPATT)		Percent Passing (MTO)
			Test 1	Test 2	
Wet Process – Field Blend	2.36 mm	100	100	100	100
	2.00 mm	98 – 100	92.5	93.3	92.5
	1.18 mm	45 – 75	50.4	54.1	53.2
	600 µm	2 – 20	10.7	12.8	18.8
	300 µm	0 – 6	1.0	1.3	3.2
	150 µm	0 – 2	0.1	0.2	0.2

It should be noted that the CRM samples used in this study were supplied by the OTS and inspected by the MTO. These rubber crumbs were subjected to both cryogenic grinding and ambient grinding. Microscopic examination of the CRM samples revealed black cryogenic and ambient ground rubber with particle sizes generally less than 2.36 mm (Hicks et al., 2013). White rubber, synthetic fibers, and mineral grains were also observed; see Figure 4-3. However, no steel, plastics, cellulose fibers or other metals were observed (Hicks et al., 2013). The MTO is looking to standardize microscopic examination practices and proper Material Quality Reporting (MQR) practices for CRM supply in production of rubber modified asphalt in Ontario.

Figure 4-3: Fine CRM Samples

4.1.2 Cored Pavement Sections

Sufficient cores were extracted from Highway 15, north of Smiths Falls, Ontario, for forensic testing at the CPATT pavement testing laboratory. As previously noted, these sections were of the moist process rubber modification with ambient and cryogenic crumb rubber. Control sections were also cored and extracted for testing. Table 4-5 provides an inventory of cores received from Aecon Materials Engineering (AME) through collaborative efforts with the MTO and the OTS.

Table 4-5: Inventory of Cored RMA Sections

Control Cores	Cryogenic Rubber Cores	Ambient Rubber Cores
C-17	C-9	C-1
C-18	C-10	C-2
C-19	C-11	C-3
C-20	C-12	C-4
C-21	C-13	C-5
C-22	C-14	C-6
C-23	C-15	C-7
C-24	C-16	C-8

***Note:** C = Core

4.2 Material and Mix Characterization

As a first approach to ensuring good performance of the evaluated mixes, the RAP and virgin material in this study were characterized. This task included evaluation of the fundamental and basic Superpave consensus properties, extraction and recovery of the asphalt content (AC) in RAP, determination of the performance grade (PG) of the recovered AC, and gradation analysis. Verified results of the continuous PG for both RAP and virgin binders completed by DBA Engineering are presented in Table 4-6 and show that all materials meet the PG specifications. The consensus properties of the RAP aggregates and HMA mixes were evaluated by CPATT to determine the effects of RAP on physical properties of the mix and which mix variables were most sensible to increase in RAP contents (Sanchez, 2013). Source properties were not considered since all RAP used in this study were of high or premium quality and from a designated source in Ontario.

Table 4-6: Original Binder Continuous Grading

Binder		**High Grade (ºC)**	**Low Grade (ºC)**
RAP		77	-22
Lab Mix	58-28	61	-30
	58-34	63	-35
	52-34	55	-35
	52-40	56	-41
Plant Mix	58-28	59	-29
	64-34	65	-35
	64-34P	67	-35

4.2.1 Superpave Consensus Properties

Evaluation of the Superpave consensus properties revealed a decreasing trend in terms of crushed faces with increasing RAP content for coarse angularity of the aggregates. A higher percentage of coarse angular surfaces will potentially improve rutting resistance. This is because it provides better structure and stability between the interlocking bonds of the aggregate skeleton which induces a better load-bearing capacity. The percent crushed faces were found to decrease by up to 1.35 percent with the addition of 40 percent RAP, which is not significant taking into account that the values are kept above the lower limit (85% for 1 face and 80% for 2 faces). Figure 4.4 highlights results for percent crushed faces.

The uncompacted voids were found to increase for the 20 percent RAP, but returned to a value closer to the virgin mix for the 40 percent RAP mixes as observed in Figure 4-5. The minimum requirement for this parameter is 43. The flat and elongated particles were seen to decrease for the 20 percent RAP and increase for 40 percent RAP mixes. However, this parameter has a maximum admissible value of 10 and is not greatly affected as shown in Figure 4-6. A slight linear trend was observed for sand equivalency as shown in Figure 4-7. The value increases as the percent RAP increases, so the sand equivalent will move away from the minimum acceptable (45). This means that the RAP mixtures have a lower proportion of clay-like materials, which benefit the binding of the asphalt binder with the aggregate.

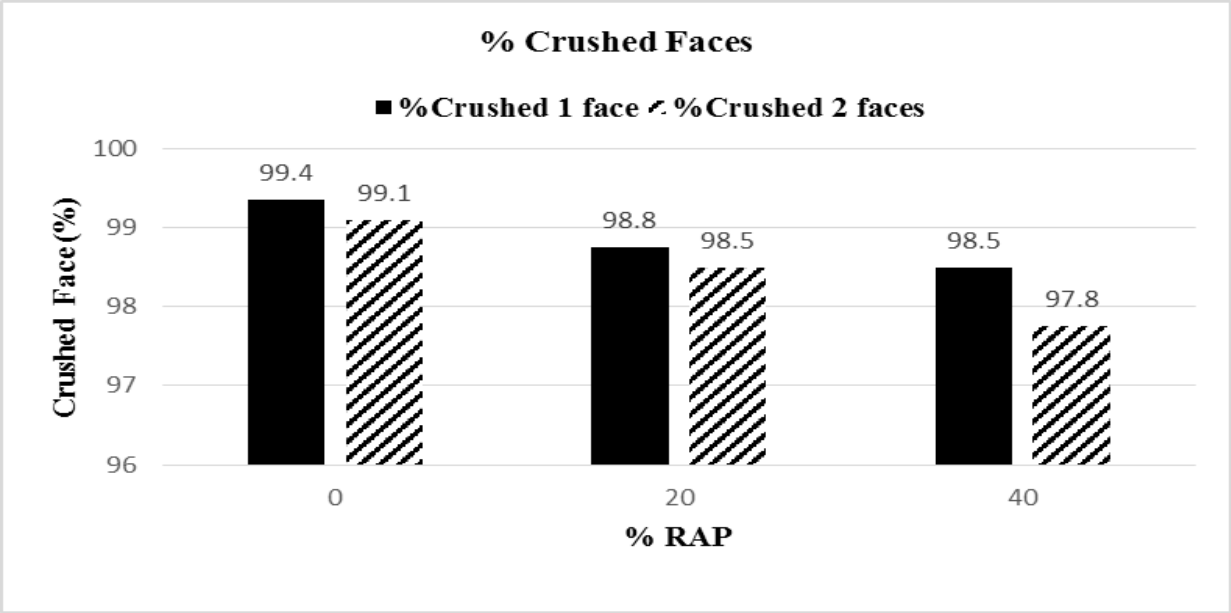

Figure 4-4: Percent Crushed Faces

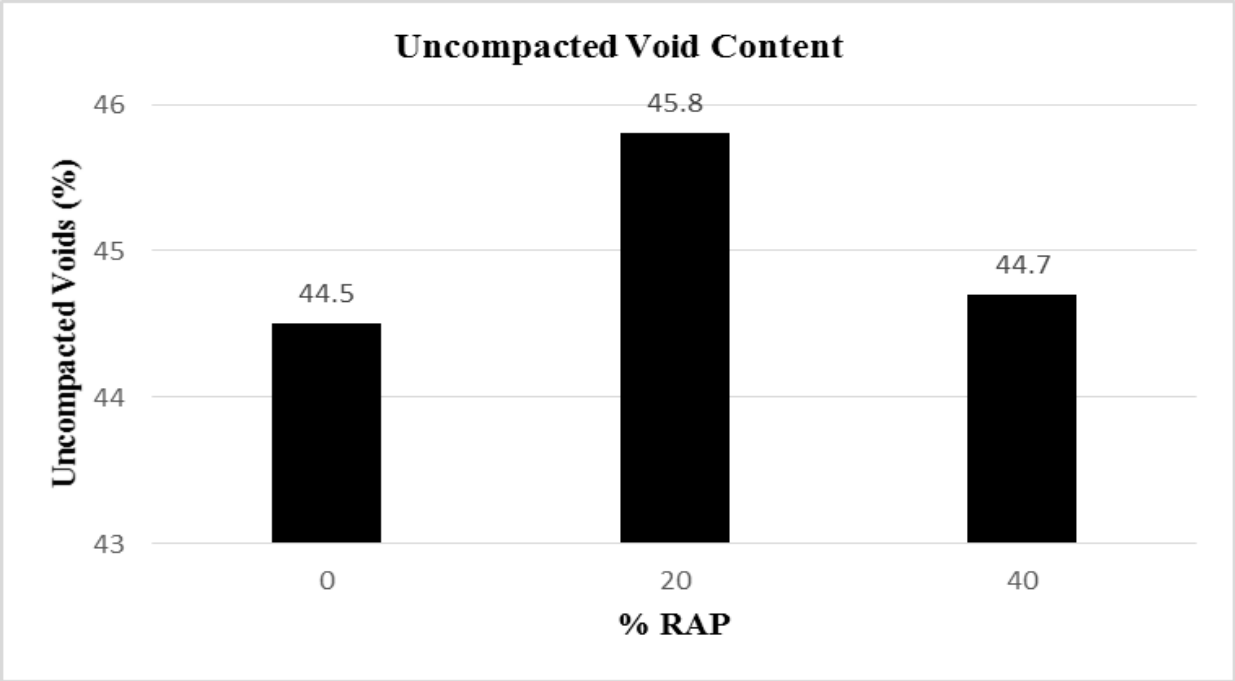

Figure 4-5: Uncompacted Voids

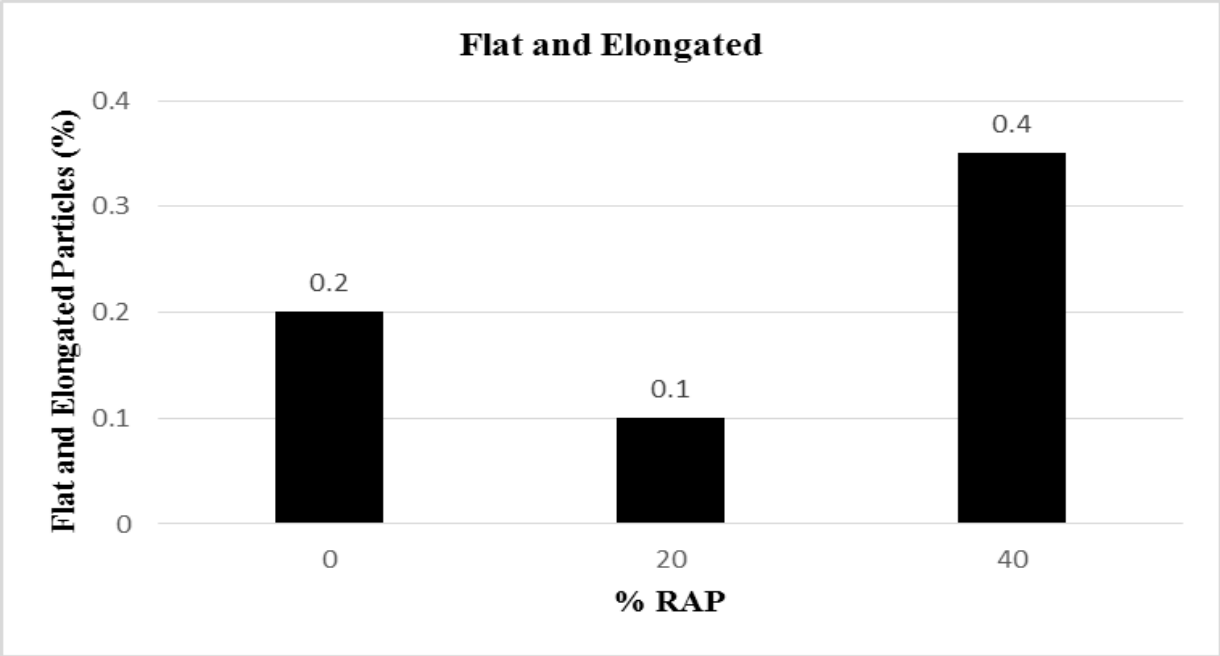

Figure 4-6: Flat and Elongated Particles

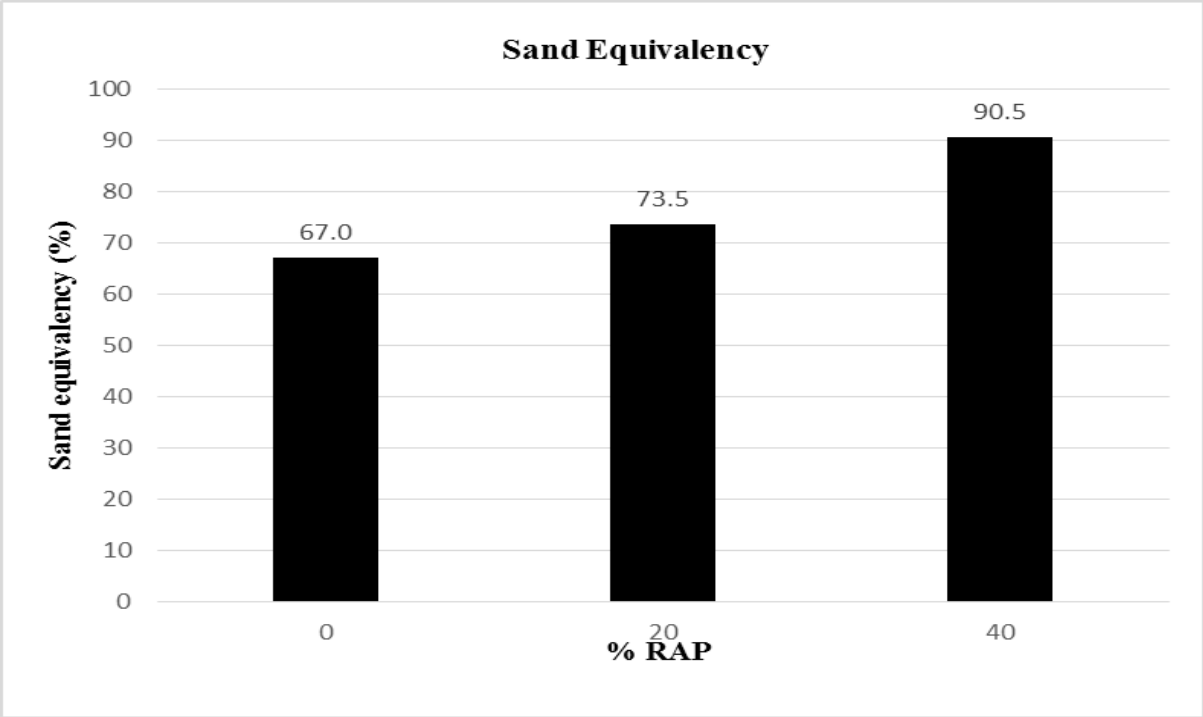

Figure 4-7: Sand Equivalency

4.2.2 Superpave Mix Design Observations

Analysis of the volumetric properties of the mixtures from each mix design revealed that for laboratory-prepared HMA mixtures, the Voids in Mineral Aggregate (VMA) and Voids Filled with Asphalt (VFA) decreased with increasing RAP content, as seen in Figure 4-8 and

Figure 4-9. An increase in VMA and VFA was also observed for field blended rubber HMA mixtures, while a decrease in VMA and VFA is observed with increasing RAP content from 15 to 20 percent as shown in Figure 4-10 and Figure 4-11 respectively. These cases of decreasing VMA and VFA explain the decrease in the percentage virgin of binder as shown in Table 4-1 and Table 4-2. This suggests that more fine aggregates from RAP would fill the spaces between particles for mixes with increasing RAP content. Minimum VMA requirement is 14 percent while that of VFA is between 65 to 75 percent. Achieving the minimum VMA and VFA requirement are beneficial to the formation of a durable binder film thickness which also aids in rutting resistance.

All values for Tensile Strength Ratio TSR values met or exceeded the minimum 80 percent requirement. The lowest TSR value were obtained from the 40 percent RAP mixes as seen in Figure 4-12. These

values are about 4 percent below those obtained for the virgin mixes. In the case of plant-produced HMA mixtures, the lowest TSR value of 81.7 percent is seen with the H7-Rubber Terminal Blend mix as shown in Figure 4-13.

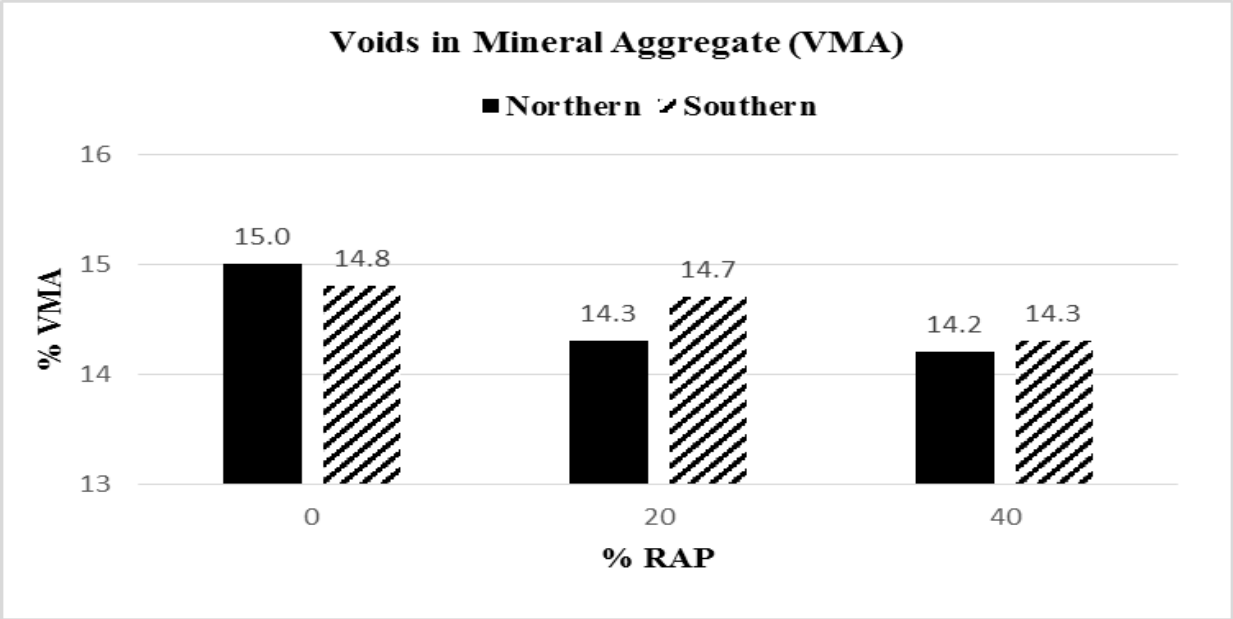

Figure 4-8: Percent Voids in Mineral Aggregate

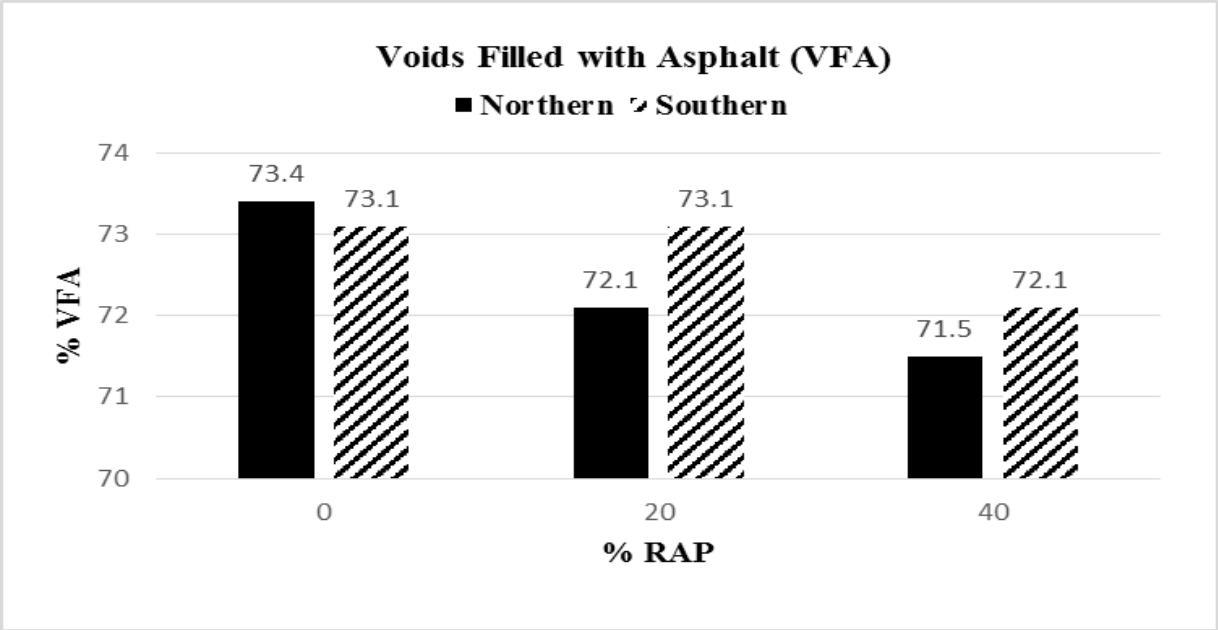

Figure 4-9: Percent Voids Filled with Asphalt

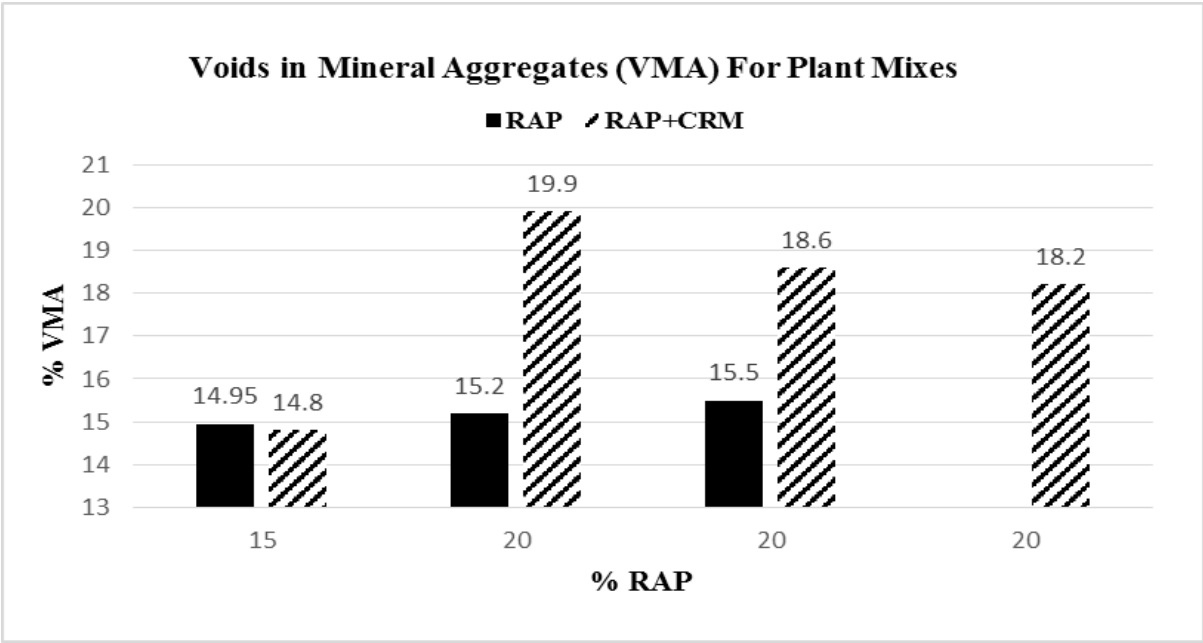

Figure 4-10: Percent VMA for Plant-produced HMA Mixtures

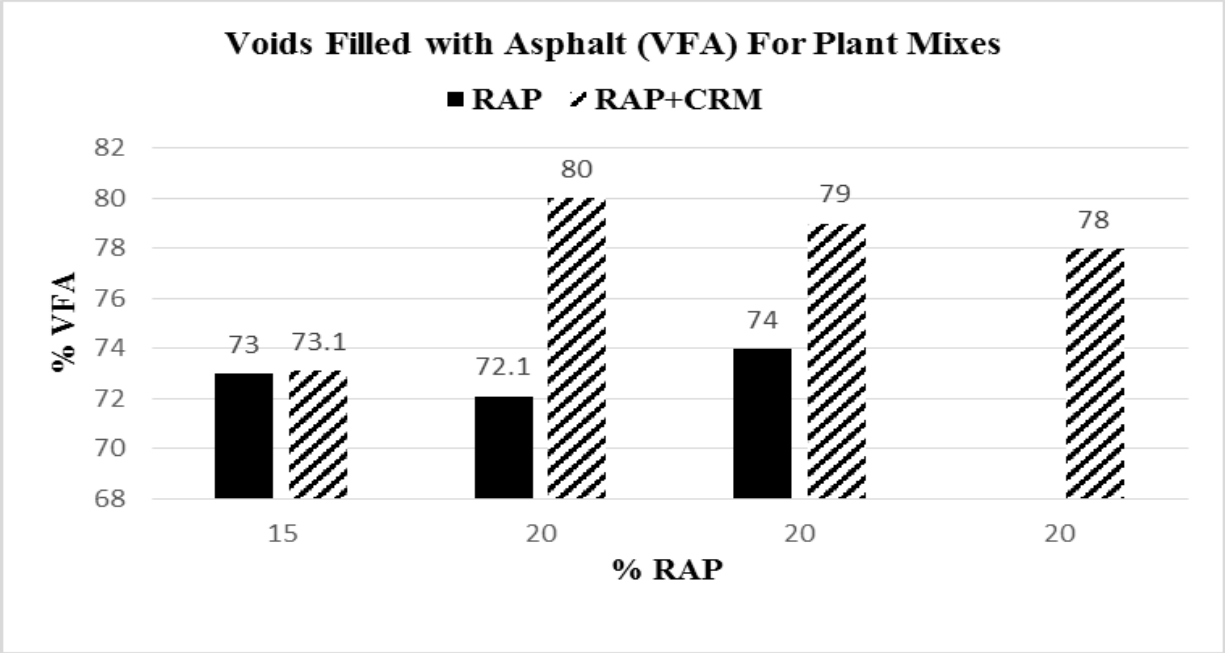

Figure 4-11: Percent Voids Filled with Asphalt for Plant-produced HMA Mixtures

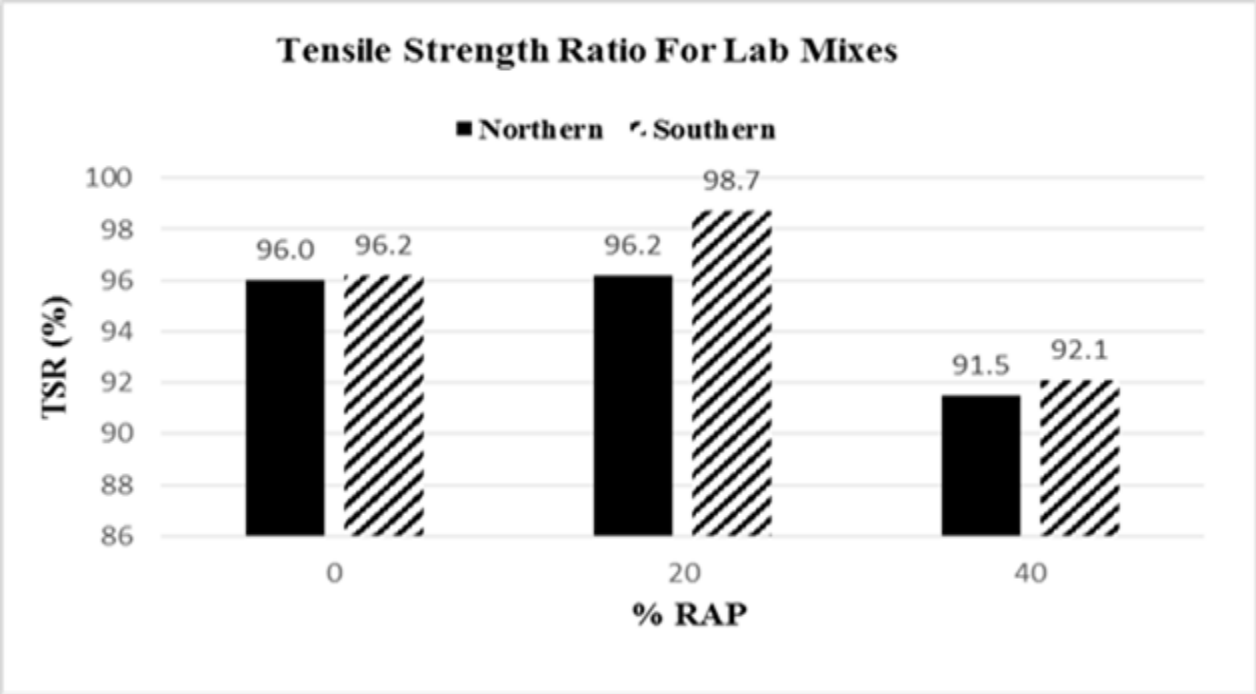

Figure 4-12: Tensile Strength Ratio (TSR) for Laboratory-prepared HMA Mixtures

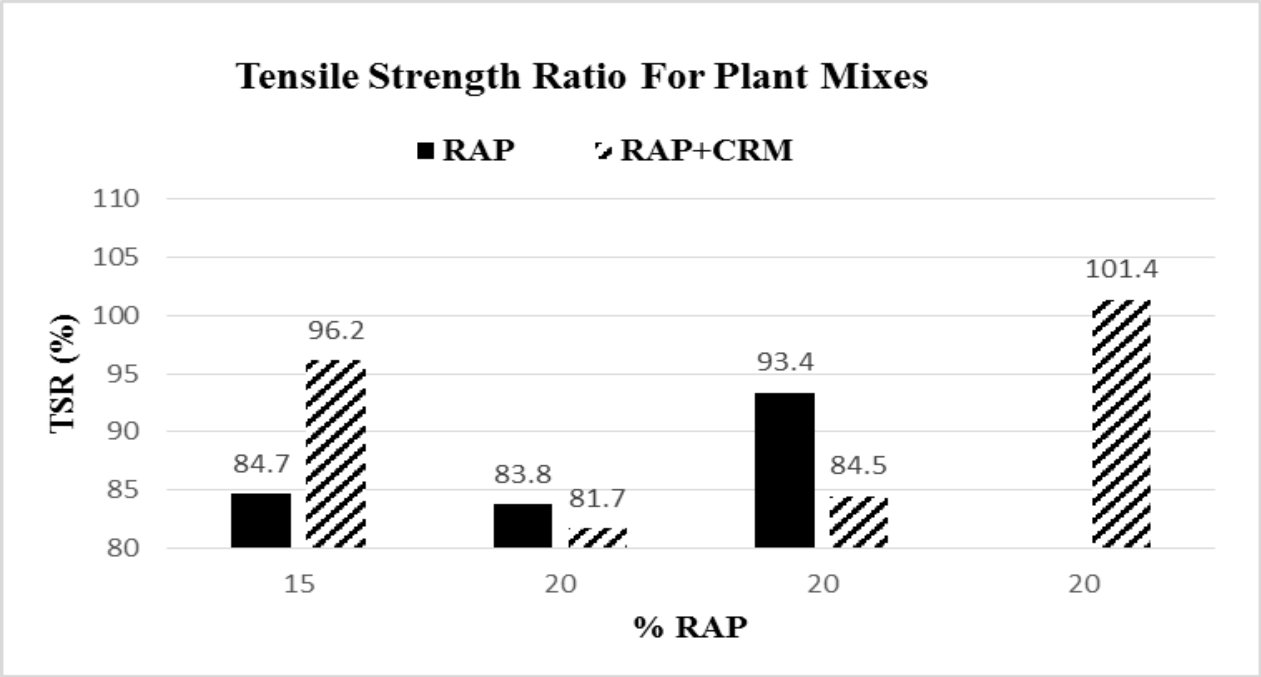

Figure 4-13: Tensile Strength Ratio (TSR) for Plant-produced HMA Mixtures

Dust Proportion (DP) as observed in Table 4-1 and Table 4-2 are within the minimum 0.6 and maximum 1.2 limits. An increase of almost 60 percent was observed for the RAP mixtures when compared to the virgin mixes. These results are related to the lower effective binder content for the RAP mixtures.

4.3 Summary

This chapter has detailed the volumetric properties and composition of the loose HMA matrixes in this study. A summary of the field pavement sections outlined for monitoring including an inventory of cored pavement sections for forensic testing has been provided.

The traditional approach associated with the design of HMA requires that asphalt cement and aggregate be blended in precise or relative proportions such that the influence from the source and volumetric properties do not compromise pavement performance. The observations noted from the material, and mix characterization as analyzed from the job mix formula showed that the consensus properties are different for the respective HMA mixtures. However, the experimental design satisfies all specified limits for the Superpave mix design including the aggregate consensus properties.

In particular, the angularity of coarse RAP aggregate could positively affect the rutting performance of the HMA mixtures. In addition to this is the effect of dust proportion resulting from adding RAP. It is expected that this would dictate the performance of the mastic in the mix and could have a potential positive impact on overall performance. TSR results suggests that the mixtures could be highly resistant to moisture damage and effects of stripping. It is therefore considered reasonable to say that the HMA mixtures in this study have been designed for stability, durability, impermeability, workability and flexibility.

The next Chapter compares, relates and verifies these observations with actual performance test results.

Chapter 5

Performance Testing, Result Analysis and Discussions

5.1 Original and Blended Binder Characterization

Binder characterization tests performed in this study provided an indication of the contributions made by the various RAP and CRM configurations to the mix performance. By employing the methodology described earlier for rheological characterization of binders, the rheological parameters and continuous true grades for both the base (aged and unaged) and recovered blended binder were determined for the respective mix blends. The findings for rheological parameters are reported in Table 5-1.

Table 5-1: Binder Rheological Test Parameters

Binder PG	Binder Blend	G*	δ	G*/sin δ (kPa)	G*sin δ ≤ 5000 (kPa)
58-28	L:58-28 (Aged-RTFO)	3.33	83.8	3.35	4195
	L2:58-28+20%RAP	2.45	83.0	2.47	3930
	L3:58-28+40%RAP	2.20	74.6	2.28	4820
	P:58-28 (Aged-RTFO)	-	-	3.23	4274
	H7C:58-28+15%RAP	3.72	74.5	3.86	4890
	H35C:58-28+20%RAP	3.49	80	3.54	4940
	H7RTB:58-28+20%RAP	2.54	64.3	2.82	4170
	H7RFB:58-28+20%RAP	3.11	75.7	3.21	3990
	H35RFB:58-28+20%RAP	2.52	79.4	2.56	4090
58-34	L:58-34 (Aged-RTFO)	4.47	79.1	4.55	4085
	L8:58-34+20%RAP	4.12	80.4	4.18	3550
	L9:58-34+40%RAP	2.43	80.6	2.46	3880
52-34	L:52-34 (Aged-RTFO)	3.20	83.3	3.23	3690
	L5:52-34+20%RAP	2.46	82.4	2.48	3830
	L6:52-34+40%RAP	2.44	80.2	2.48	3960
52-40P	L:52-40 (Aged-RTFO)	3.00	73.8	3.16	4715
	L11:52-20P+20%RAP	3.03	81.8	3.06	3580
	L12:52-40P+40%RAP	3.60	82.1	3.65	3550
64-34	P:64-34 (Aged-RTFO)	-	-	2.21	1568
	H115C:64-34+20%RAP	2.06	68.3	2.22	3740
64-34P	P:64-34P (Aged-RTFO)	-	-	2.44	1902
	H115RFB:64-34P+20%RAP	2.23	64.1	2.48	4520

***Note:** H = Highway, L = Laboratory, P = Plant, PG = Performance Grade, RTFO = Rolling Thin Film Oven, RTB = Rubber Terminal-blend, RFB = Rubber Field-blend.

Although not reported in Table 5-1, it should be noted that the specified minimum value of 1.0 kPa for stiffness parameter (G*/sin δ), is satisfied for all unaged original binders at their respective high temperature grades. Note also that a binder's G*/sin δ is a function of its shear complex modulus, G* and phase angle, δ. A higher G* will result in a binder that is less susceptible to rutting while its corresponding δ value controls its viscous or elastic behaviour. A binder of lower value δ will behave like an elastic solid while a higher value δ behaves more like a viscous liquid.

It is observed from Table 5-1 that the influence of RAP variations or a combination with CRM is highly asphalt binder-specific. In the laboratory-prepared mix category, recovered binders for PG 58-28 and 52-34 asphalt binders are observed to exhibit a decreasing G* and δ with increasing RAP contents. PG 58-34 HMA mixtures showed a decrease in G* and an increase in δ with increasing RAP contents while PG 52-40P binder exhibited an increase in both G* and δ with increasing RAP contents.

This suggests that increasing the RAP content will result in a binder that is less susceptible to rutting, but its elastic or viscous behaviour will vary depending on the type of asphalt binder. However, a recycled binder blend capable of resisting rutting and having an elastic nature is preferable. This is because a recycled asphalt binder blend exhibiting high elastic and low viscous components will be more durable during high and moderate temperatures.

In the plant-produced mix category, and for comparisons made between PG 58-28 binder with 15% RAP, and 20% RAP binders incorporating 10 - 20% CRM, a general decrease in G* and corresponding increase in δ is observed. Within this category of mixtures, the terminal-blend Rubberized-RAP mixtures exhibited lower values of G* and δ compared to the field-blend Rubberized-RAP HMA mixtures. This confirms that a terminal-blend rubber mix is less viscous and more elastic than a field-blend rubber mix. Plant-produced HMA mixtures indicated an increase in G* and corresponding increase in δ for comparisons between a 20% RAP PG 64-34 and a field-blend Rubberized-RAP mix utilizing PG 64-34P asphalt binder.

Following the observed trend in G* and δ, the actual binder stiffness parameter (G*/sin δ) was computed and compared for each mix blend. The results as shown in Table 5-1 indicates a decrease in G*/sin δ with increasing RAP content for both plant-produced and laboratory-prepared mixtures utilizing PG 58-28 asphalt binder. The lowest values in this category are noted at 40% RAP variation. A similar trend is observed for the laboratory mixes incorporating PG 58-34 binder. G*/sin δ values for plant-produced PG 58-28 HMA mixtures incorporating 20% RAP and 10 or 20% CRM suggest a compensating balance in binder stiffness when compared with the 40% laboratory-prepared RAP mix. The difference between a

terminal blend Rubberized-RAP and the field blend Rubberized-RAP mixtures are further affirmed from the observed G*/sin δ values. The binder stiffness parameter (G*/sin δ) for PG 52-34 also decreases with the addition of RAP, but values for 20 and 40% RAP content are observed to be comparable. The trend for mixes with PG 52-40P suggests an increase for 40% RAP and a reduction for 20% RAP contents. Minimal improvements in the binder stiffness parameter are observed for cases involving a 20% RAP increase, and a combination of 20% RAP and 20% CRM for mixes utilizing PG 64-34 and 64-34P. Overall, the findings reported in Table 5-1 satisfy the 2.2 kPa binder stiffness parameter (G*/sin δ) for aged base and recovered blended binder; suggesting that the respective blends will exhibit good elasticity, and thus will be more resistant to rutting. However, the Rubberized-RAP blends are expected to perform better. The observed G*/sin δ values are graphically illustrated in Figure 5-1.

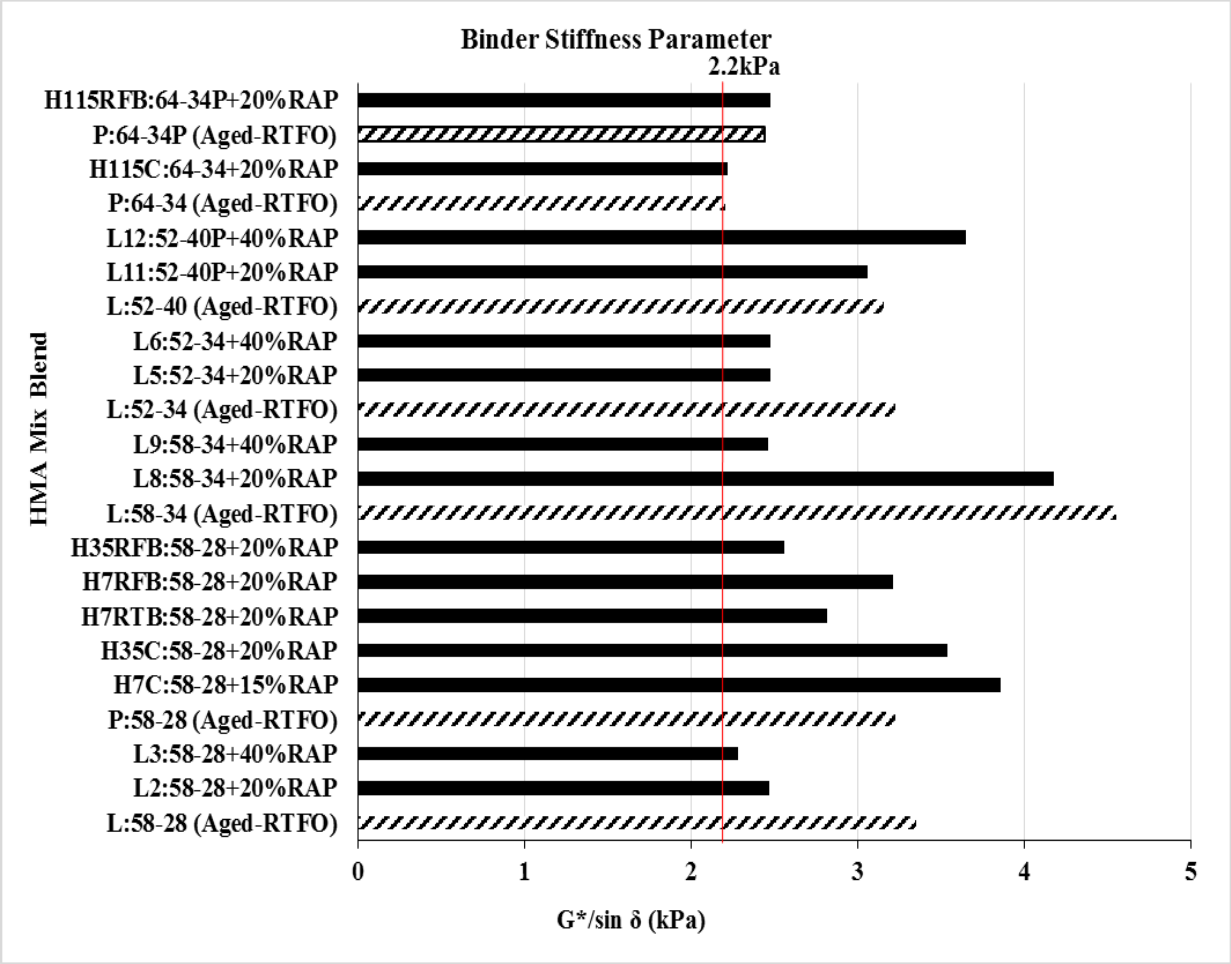

Figure 5-1: Blended Binder Stiffness Parameter (G*/sin δ)

One of the limiting factors on increasing RAP content for typical Ontario HMA is related to the performance grade (PG) difference of the resulting binder blend; especially in terms of the lower grade which controls thermal crack resistance. Earlier in the thesis, it was explained that oxidation of asphalt cement converts the maltenes (oils) component in the RAP binder to asphaltenes (resins), and vice-versa; thus resulting in a higher viscosity and aged binder which in turn affects the PG of the new mix blend. In this study, the high and low temperature continuous true grades of the respective mix blends were observed to be affected by the addition of varying RAP percentages. The grade improvements observed for HMA mixtures combining RAP and CRM could be attributed to the counter-acting effects of CRM, and its consequent stabilization of the asphaltenes and maltenes structures in the RAP binder. Figure 5-2 suggests that a terminal-blend Rubberized-RAP mix of 10:20 CRM to RAP ratio improved the critical high temperature grade for a PG 58-28 binder by as much as 13.2ºC while a field-blend Rubberized-RAP mix of 20:20 recycling ratio resulted in grade improvements between 6 to 8ºC.

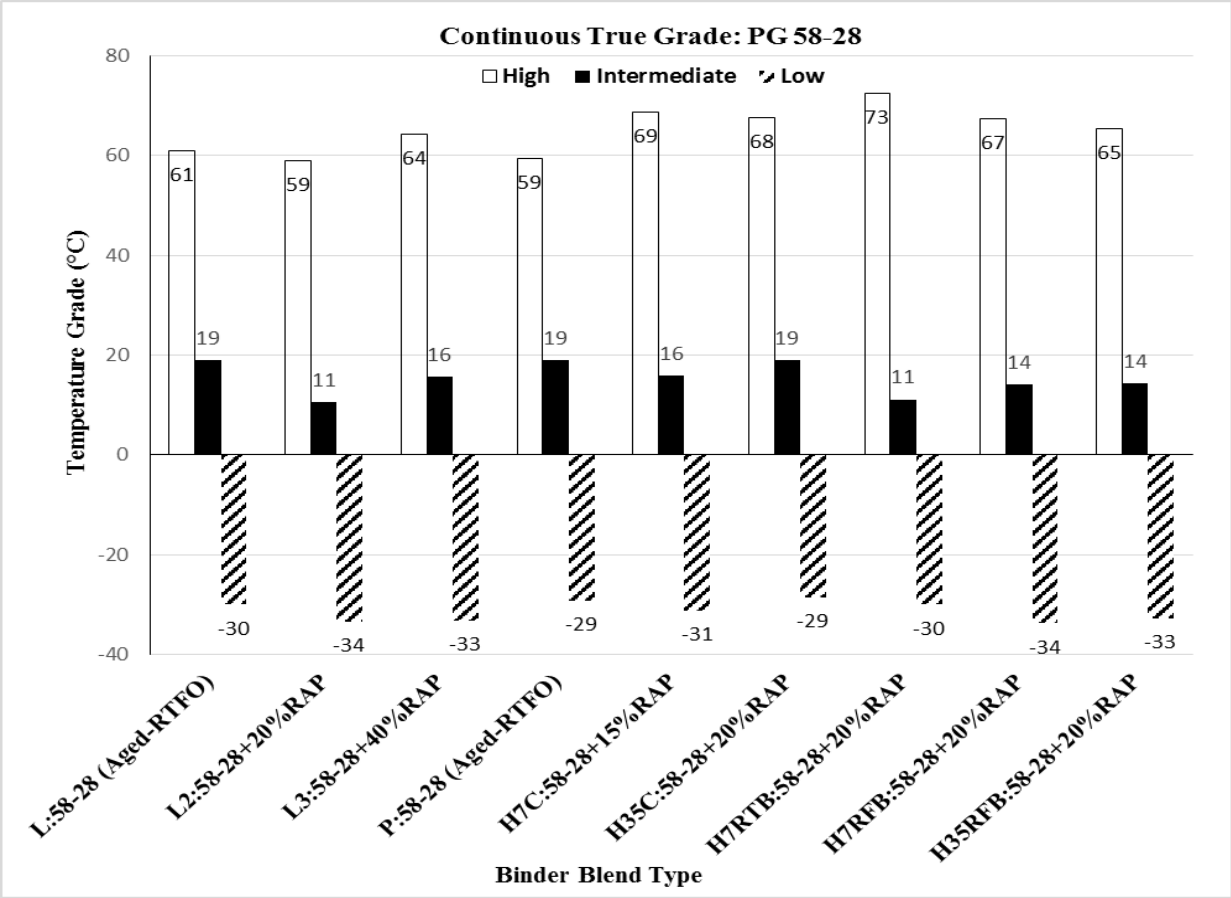

Figure 5-2: Continuous True Grade for PG 58-28

Whereas little or no critical high temperature effects are observed for a laboratory-prepared 20% RAP PG 58-28 blend, grade improvements of approximately 3.4ºC is observed when it is compared to the aged base binder. The critical low temperature grade regardless of increase in RAP variation or combination of RAP and CRM for this binder grade is improved by approximately 1 to 3.5º. This means a lower grade beneficial to the cracking resistance of the binder was obtained.

In comparison to the aged base binder, an increase in critical high temperature continuous true grade is observed with increasing RAP content for mixes incorporating PG 58-34 and 52-34. However, no change is observed for a 40% RAP mix with a PG 52-40P binder. A 4.3ºC increment is observed for a 20% RAP PG 52-40P mix. The critical low temperature grade is reduced by approximately 1ºC for 20 and 40% RAP PG 58-34 and 52-34 mix categories with exceptions to the 40% RAP PG 52-34 which gains approximately 1.2ºC compared to its base binder. These observations are graphically illustrated in Figure 5-3.

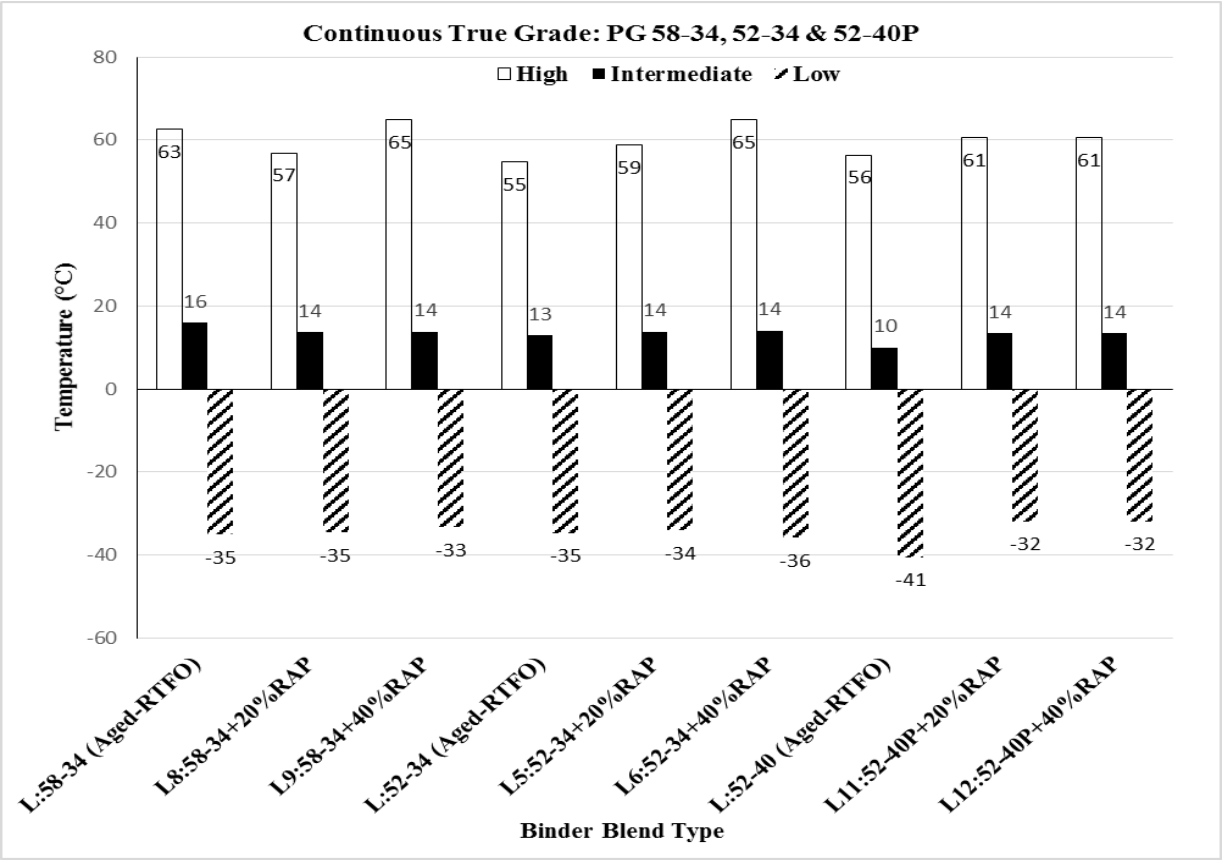

Figure 5-3: Continuous True Grade for PG 58-34, 52-34 and 52-40P

It is observed from Figure 5-4 that a 20% RAP PG 64-34 blend gains as much as 4.7ºC on the high temperature end, but loses approximately 3.7ºC in critical low temperature. However, a 20:20 RAP to CRM combination results in a high temperature gain of 4.8ºC while maintaining the low critical temperature similar to a PG64-34P blend. The low temperature properties of all binders tested were determined from creep stiffness (S) and creep rate (m-value) which is a measure of the ability to withstand thermal cracking.

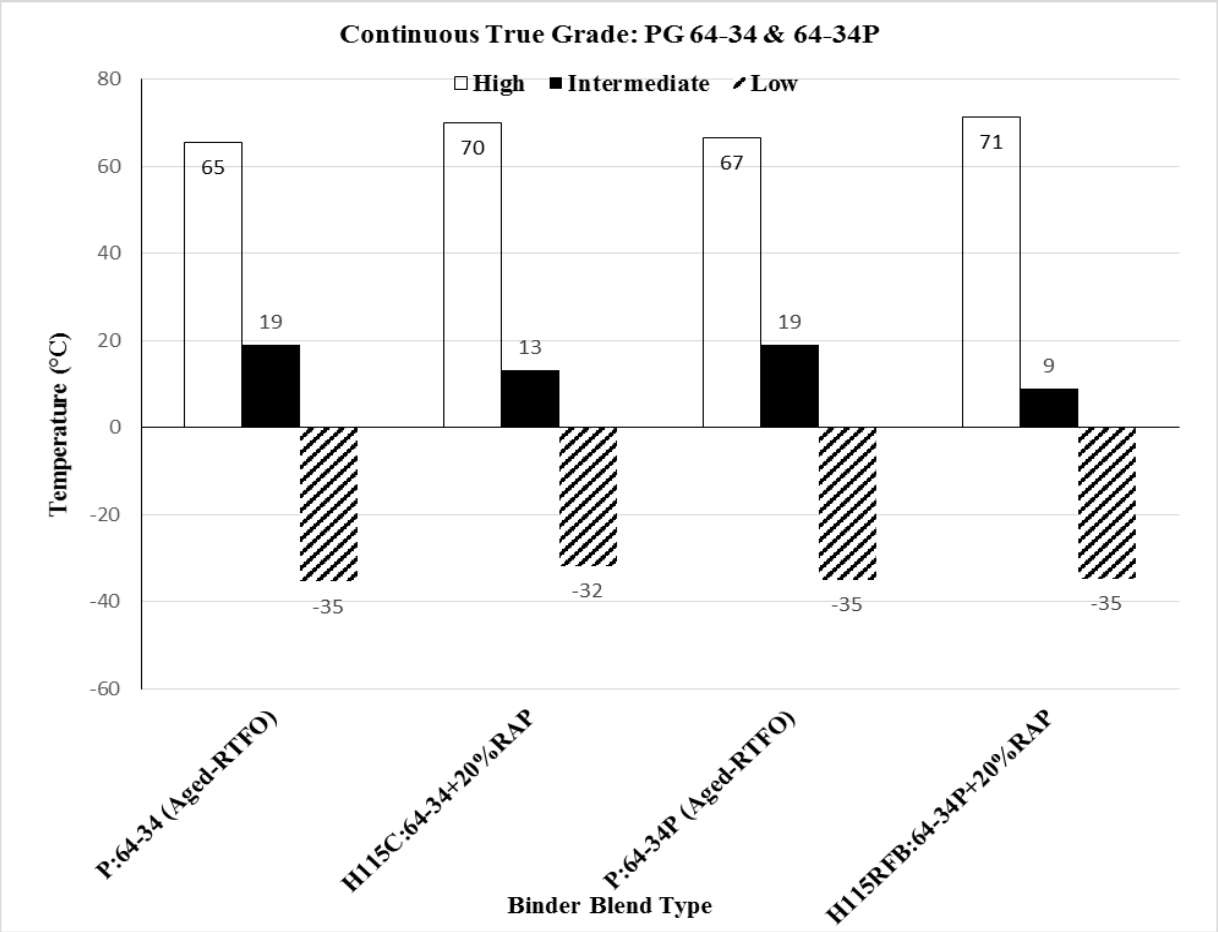

Figure 5-4: Continuous True Grade for PG 64-34 and 64-34P

The parameter G*·sin δ at intermediate pavement temperatures measures the binders ability to resist fatigue cracking at corresponding service temperatures. In accordance with binder grading criteria and specifications, the intermediate pavement temperatures where G*·sin δ ≤ 5000 KPa are shown in Figure 5-2 through 5.4 above.

All recovered binders were observed to be more flexible at lower intermediate temperatures suggesting more resistance to fatigue failure. Increasing the RAP content or combining RAP with CRM is observed to have significantly lowered the temperature at which $G^* \cdot \sin \delta \leq 5000$ KPa. The maximum allowable pavement temperature at which $G^* \cdot \sin \delta \leq 5000$ KPa for PG 58-28, 64-34 and 64-34P binder is 19ºC while those of PG 52-34, 58-34 and 52-40 are 13ºC, 16ºC and 10ºC respectively.

5.2 Thermal Crack Characterization of Evaluated HMA Mixtures

Thermal cracking is a major form of distress in asphalt concrete pavements. This form of pavement distress is visible as a set of parallel surface-initiated transverse cracks of various lengths and widths, and are mostly perpendicular to the centre line of the pavement. The existence of cracks initiates other forms of pavement distress. Water passes through the pits and crevices created by the initial cracks resulting in the weakening of the underlying pavement structural layers. A progressive deterioration of the asphalt pavement layer results from the loss of fine materials under traffic loads. The situation becomes worse with freeze-thaw cycling and associated heaving.

Thermal cracking is a tensile failure of the asphalt binder-aggregate mixture influenced by the repeated daily and seasonal temperature fluctuations; particularly those of extremely low-temperature conditions. In addition, the HMA mix volumetric, source and percentages of recycled materials are considered to have significant effects on the properties of the resultant HMA mixture. The aged asphalt binder in RAP-HMA is assumed to boost the susceptibility to thermal cracking; whereas CRM is considered to enhance the adhesive properties between the aggregates and the asphalt binder as well as increase the relaxation properties of the resulting HMA mixture at low-service temperatures to reduce thermal cracking. By incorporating RAP and CRM into HMA, it is possible to extend pavement life as well as reduce the frequency and cost of construction, maintenance, and rehabilitation. The potential for transverse thermal cracking at low temperatures were evaluated for the HMA mixtures under study based on the methodology described in Chapter Three. Tests were conducted on four replicate rectangular beam specimens for the respective experimental mix matrixes. In total, 76 specimens were subjected to the “Thermal Stress Restrained Specimen Tensile Strength Test (TSRST)”.

Appendix A details the fracture temperature and stress results determined from TSRST for each HMA mixture tested. The standard deviations, coefficient of variation and variance between tests are also detailed in Appendix A. These findings are analyzed and summarized here. The critical low true temperature grades noted from rheological binder characterization tests are matched with the mean

fracture temperatures determined from the TSRST characterization to quantify both mix performance and reliability of the rheological testing method employed. This is based on the finding that the RAP and CRM had interacted to improve the performance of the binder which primary controls thermal crack resistance at low temperatures.

The TSRST results noted in this study indicate that the colder; i.e. the more negative the fracture temperature, the greater is the potential of the asphalt mixtures to resist low-temperature thermal cracks. It is observed in Figure 5-5 that all HMA mixtures tested either met or exceeded -28ºC. However, the laboratory-prepared 20% and 40% RAP mixes failed to meet the critical low true temperature grades noted from their respective binder characterization tests. Notwithstanding, the potential for using 20% to 40% RAP or combining a 20:10/20% RAP to CRM recycling ratio with a PG 58-28 binder is exhibited in the observed performance of mixes in this category. The most significant trend is noted in rubber terminal-blend mix with 20% RAP which gained approximately 9.9ºC in comparison to the critical low true temperature grade of the blended binder.

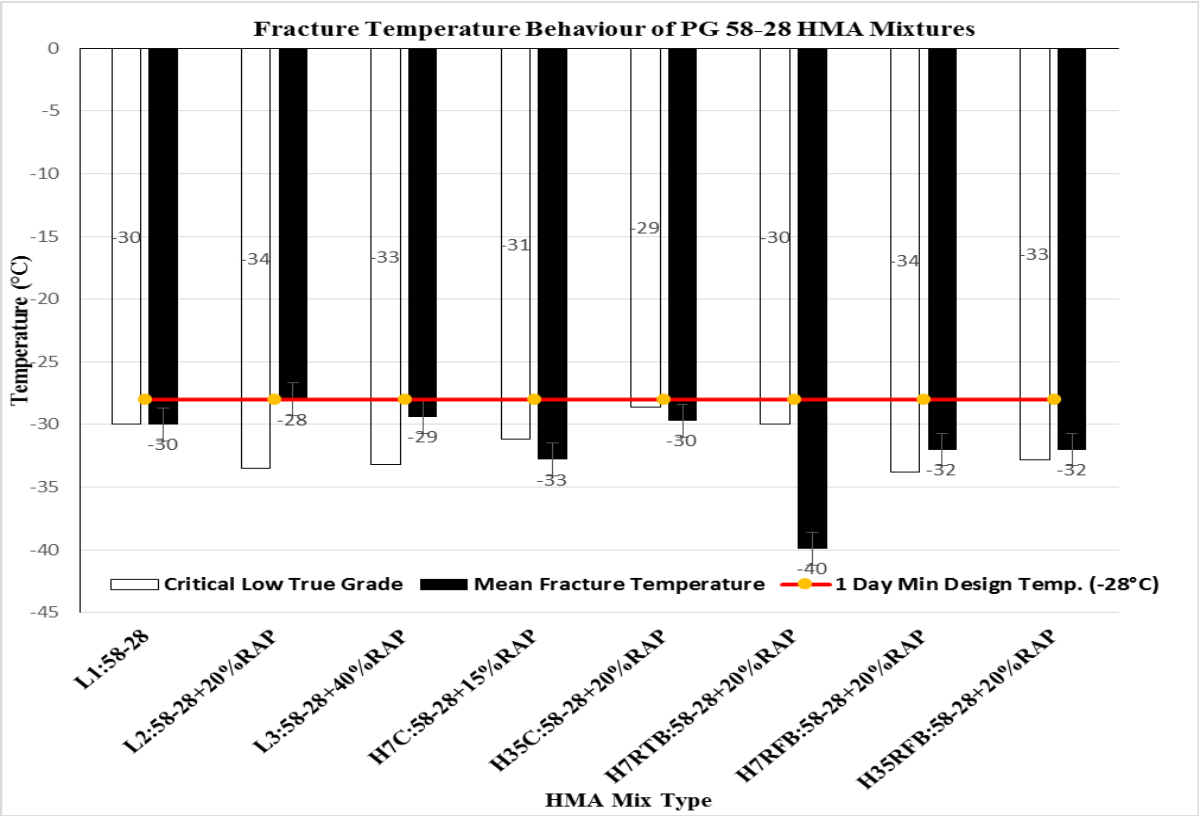

Figure 5-5: Fracture Temperature Behaviour of PG 58-28 HMA Mixtures

It is observed in Figure 5-6 that the 20 and 40% RAP mixtures do not compare favorably with their respective control mixtures for PG 58-34, 52-34 and 52-40. These mixes failed to attain the -34ºC or -40ºC requirement for mix fracture temperature, nor reach the critical low true grade of their respective recovered binders. However, a case for maintaining 20% RAP usage is exhibited with the PG 58-34 mix. These observations make it compelling to experiment with CRM for the respective binder grades and varying RAP contents in this category of the research experimental matrix.

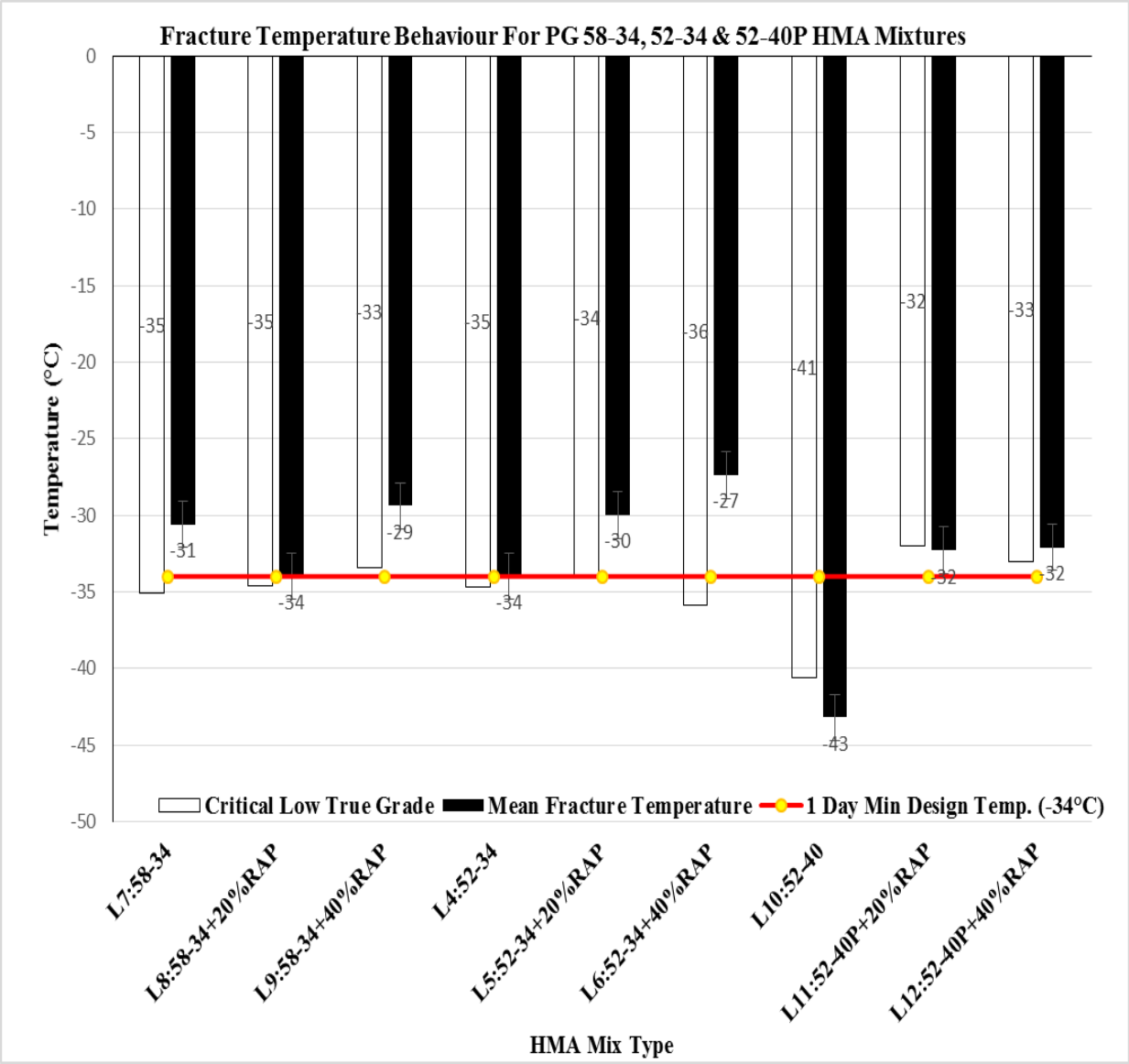

Figure 5-6: Fracture Temperature for PG 58-34, 52-34 and 52-40P HMA Mixtures

In Figure 5-7, it is observed that the fracture temperature of the Rubberized-RAP mix with 64-34P binder exceeded the critical low true grade of the recovered binder by 2.7ºC; whereas the 20% RAP mix with PG 64-34 failed to meet the -34ºC requirement for mix fracture temperature, but exceeded the critical low true grade of the recovered binder by 0.6ºC. Combining RAP and CRM for HMA mixtures with a polymer modified PG 64-34 binder increased the low temperature cracking resistance by 5ºC.

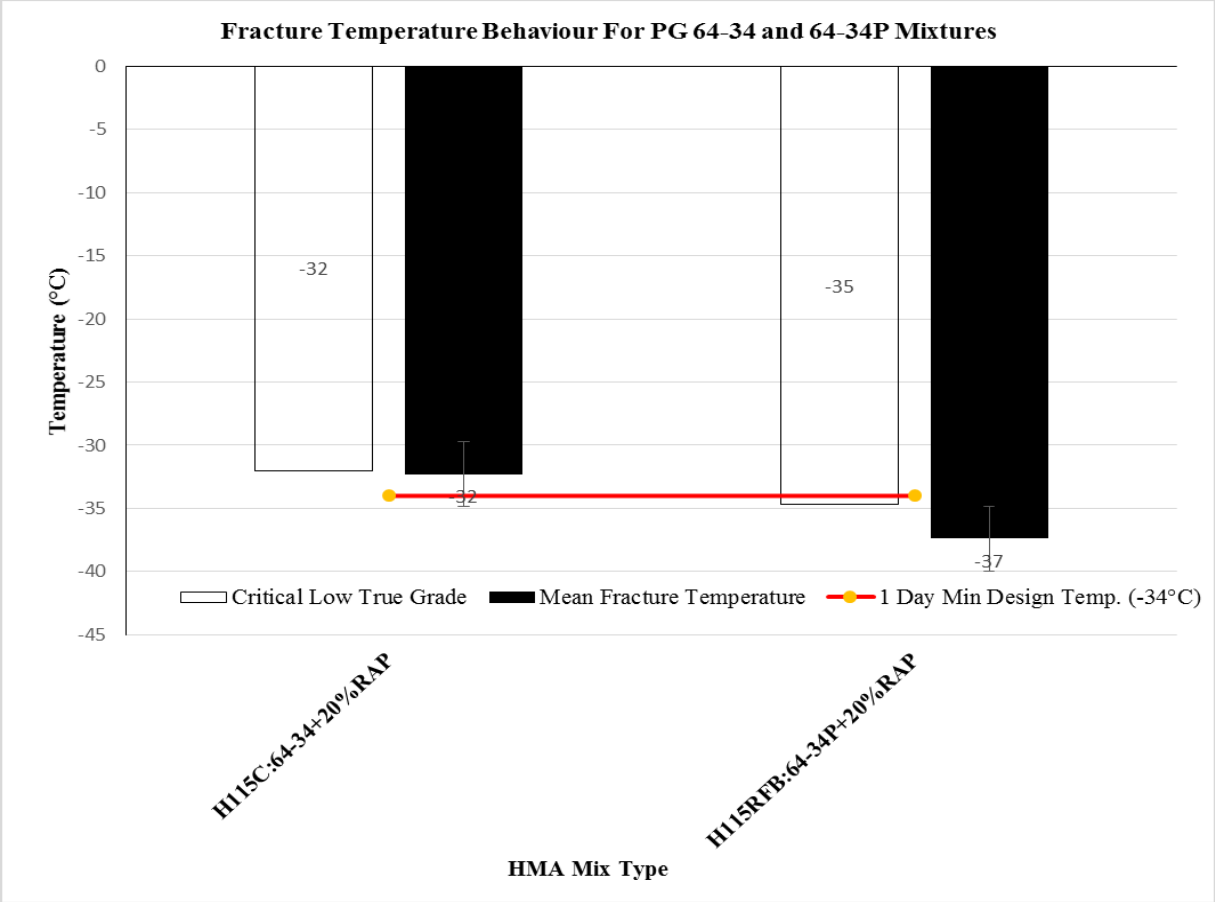

Figure 5-7: Fracture Temperature for PG64-34 and 64-34P HMA Mixtures

Comparing the mean fracture stresses for the evaluated HMA mixtures as depicted in Figure 5-8 indicates that the laboratory-prepared control HMA mix reported the least fracture stress in the PG 58-28 category. This was closely followed by the field and terminal-blend Rubberized-RAP HMA mixtures. Overall, it is observed that the fracture stresses for HMA mixtures with 15%, 20% and 40% RAP including the Rubberized-RAP HMA mixtures are comparable with the control mix.

In the PG 58-34 laboratory mix category, the fracture stress for 40% RAP mix is similar to what was obtained for the control mix and 20% RAP mix. In contrast, a lower fracture stress is noted for a 20% RAP compared to the control and 40% RAP for laboratory-prepared HMA mixtures in the PG 52-40 binder category. For laboratory-prepared HMA mixtures with PG 52-34 binder, lower fracture stress is observed for the control mix whereas those of 20% and 40% RAP are comparable. Fracture stress comparisons between 20% RAP and Rubberized-RAP plant mixes with PG 64-34 and 64-34P suggest that both mixes are not any different from each other.

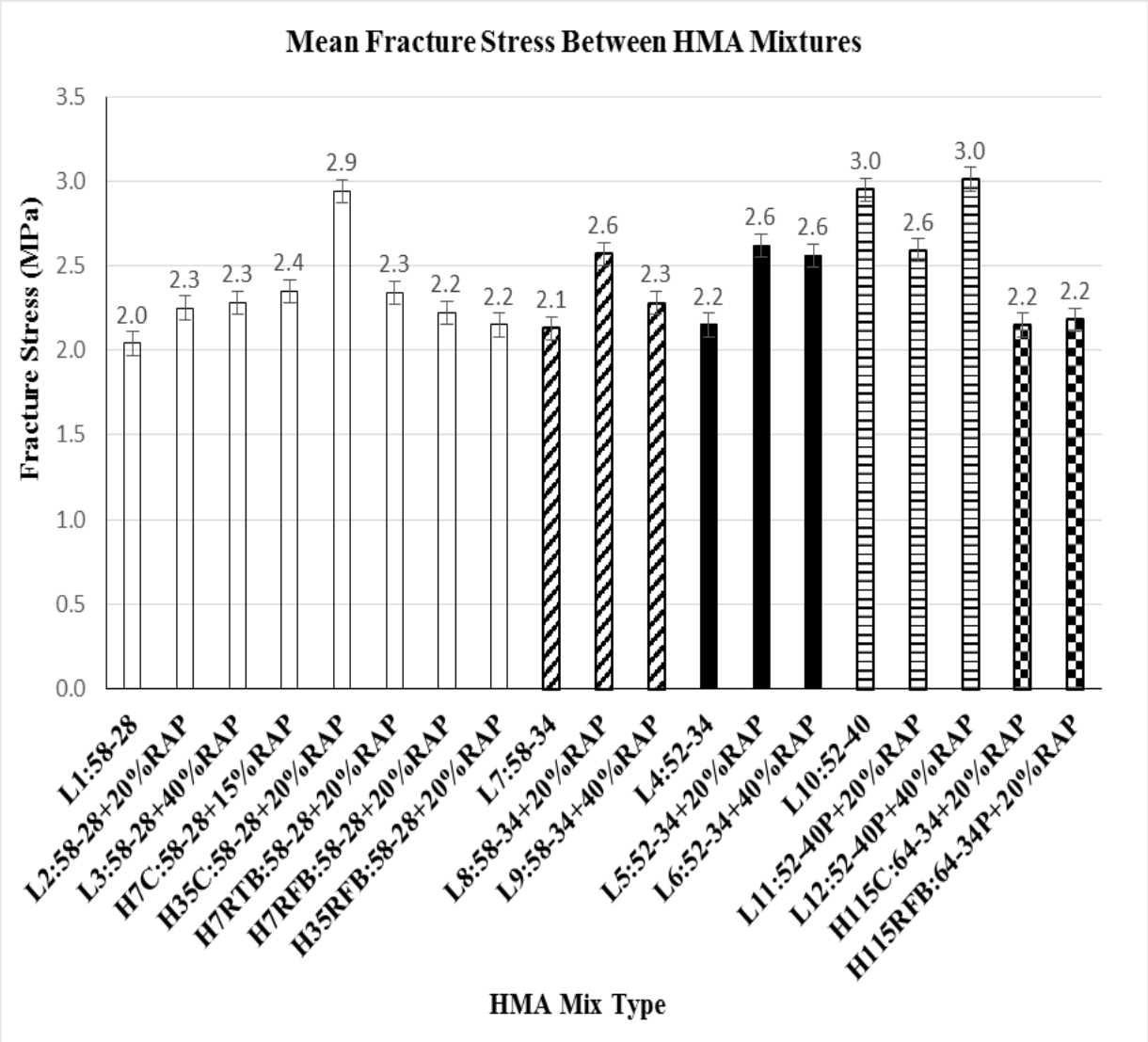

Figure 5-8: Fracture Stress of Evaluated HMA Mixtures

Recall that the fracture temperature is considered the primary indicator for thermal crack initiation in the pavement surface, while fracture stress is more indicative of the spacing between cracks after initiation. Larger crack spacing is thus attributed a higher fracture stress should the mix fail to meet its fracture temperature cracking starts to develop.

The fracture temperature for HMA mixtures in the PG 58-28 category were satisfied, and their corresponding fracture stress values are not significantly high to suggest that they will be prone to transverse thermal cracking at low temperatures. On the other hand, the control and 40% RAP PG 58-34; 20% and 40% RAP PG 52-34; and 20% and 40% RAP PG 52-40 HMA mixtures failed to attain the -34ºC or -40ºC requirement for mix fracture temperature, nor reach the critical low true grade values of their respective recovered binders. The fracture stress results for these mixtures were also not significantly high.

The fracture behaviour for the evaluated HMA mixtures is also easily explained in terms of the viscosities of their respective binder blend. This is taking into consideration the general belief that asphalt mixtures with lower viscosity are better resistant to thermal cracking than those of higher viscosity, but higher viscosity mixes could exhibit higher build-up of tensile stress during cooling which is beneficial to fracture at higher temperatures.

In the case of the terminal and field-blend RAP HMA mixtures, fracture temperature and stress is seen to be significantly influenced since their binder viscosities are improved by the rubber crumbs. Based on the fracture temperature and stress results noted for the control and 40% RAP PG 58-34; 20% and 40% RAP PG 52-34; and 20% and 40% RAP PG 52-40, it is thus suggested that their flexibilities at low temperatures could be improved for better performance if CRM were incorporated.

The coefficients of variation for fracture stress as noted in Appendix A for all evaluated HMA mixtures were significantly higher compared to those for fracture temperature. Hence, a confirmation of why fracture temperature is preferred in the ranking of low-temperature cracking resistance of asphalt concrete mixtures. Statistical analysis using a 95% confidence level were completed for fracture stress results in accordance with the hypothesis described in Chapter Three. The details from a single factor Analysis of Variance (ANOVA) for the evaluated HMA mixtures are reported in Table 5-2.

Table 5-2: ANOVA on Fracture Stress

HMA Mix Category	$F_{Calculated}$	$F_{critical}$	P-value	Remark
PG 58-28 (Lab Mix)	0.24	4.26	0.80	Statistically Insignificant
PG 58-28 (Plant Mix)	9.64	3.06	0.00	**Statistically Significant**
PG 58-28 (Lab + Plant Mix)	2.25	2.42	0.10	Statistically Insignificant
PG 58-34 (Lab Mix)	0.45	4.26	0.70	Statistically Insignificant
PG 52-34 (Lab Mix)	5.08	4.26	0.03	**Statistically Significant**
PG 52-40 (Lab Mix)	1.4	4.26	0.30	Statistically Insignificant
PG 64-34 (P) (Plant)	6.69	5.99	0.04	**Statistically Significant**

The findings show that there are no significant statistical differences for all cases with exceptions to specific scenarios between and within groups in the plant-produced PG 58-28, PG 64-34(P) and laboratory PG 52-34 mix categories. This means that performance of the evaluated HMA mixtures in terms of fracture stress is consistent and comparable for cases where $F_{Calculated}$ were found to be less than $F_{Critical}$ with a corresponding p-value greater than 0.1. An affirmation of the influence of increasing the RAP content or a combination of RAP and CRM on the binder grade of the blend, and its consequent effects on fracture properties of the HMA mixtures were determined for cases where $F_{Calculated}$ were found to be greater than $F_{Critical}$ with a corresponding p-value less than 0.1.

To further verify the deductions from Table 5-2, t-tests assuming equal variances were performed for different coupled asphalt mixtures in the respective binder categories. The values for $t_{Calculated}$ were found to be less than $t_{Critical}$ with a corresponding p-value greater than 0.1 in all cases except for specific couples detailed in Table 5-3. Again, the statistically insignificant comparisons show that the mixes are statistically similar with respect to fracture stress while the statistically significant comparisons affirmed the greater differences in effects of RAP or combining RAP and CRM. The statistically significant observations in Table 5-3 are indicative of the fact that the control mix in the paired comparison are less likely to have large crack spacings in the event that cracking occurs. That is to say for example, H115 rubber field-blend mix with 20% RAP is less likely to have large crack spacing when compared with H115C with 20% RAP.

Table 5-3: t-test on Fracture Stress for HMA Mix Couple

Binder PG	HMA Mix Couple		$t_{Calculated}$	$t_{critical\ two\text{-}tail}$	P(T<=t) two-tail	Remark
58-28	L3:40%RAP	L1:0%RAP	1.12	2.45	0.30	Statistically Insignificant
	L2:20%RAP	L1:0%RAP	0.48		0.65	Statistically Insignificant
	L3:40%RAP	L2:20%RAP	0.07		0.95	Statistically Insignificant
	H7RFB:20%RAP	L1:0%RAP	0.83		0.44	Statistically Insignificant
	H7RFB:20%RAP	L2:20%RAP	-0.07		0.95	Statistically Insignificant
	H7RFB:20%RAP	L3:40%RAP	-0.29		0.78	Statistically Insignificant
	H7RTB:20%RAP	L1:0%RAP	1.78		0.13	Statistically Insignificant
	H7RTB:20%RAP	L2:20%RAP	0.22		0.83	Statistically Insignificant
	H7RTB:20%RAP	L3:40%RAP	0.37		0.72	Statistically Insignificant
	H35RFB:20%RAP	L1:0%RAP	0.62		0.56	Statistically Insignificant
	H35RFB:20%RAP	L2:20%RAP	-0.23		0.83	Statistically Insignificant
	H35RFB:20%RAP	L3:40%RAP	-0.70		0.51	Statistically Insignificant
	H35RFB:20%RAP	H35C:20%RAP	-6.85		0.00	**Statistically Significant**
	H7RFB:20%RAP	H7C:15%RAP	-0.71		0.51	Statistically Insignificant
	H7RTB:20%RAP	H7RFB:20%RAP	0.73		0.49	Statistically Insignificant
	H7RTB:20%RAP	H7C:15%RAP	-0.06		0.95	Statistically Insignificant
	H7RTB:20%RAP	H35RFB:20%RAP	1.52		0.18	Statistically Insignificant
	H7RTB:20%RAP	H35C:20%RAP	-6.70		0.00	**Statistically Significant**
	H7RFB:20%RAP	H35C:20%RAP	-4.53		0.00	**Statistically Significant**
52-34	L6:40%RAP	L4:0%RAP	4.43	2.46	0.00	**Statistically Significant**
	L5:20%RAP	L4:0%RAP	2.33		0.03	Statistically Insignificant
	L6:40%RAP	L5:20%RAP	-0.17		0.90	Statistically Insignificant
58-34	L9:40%RAP	L7:0%RAP	0.26		0.80	Statistically Insignificant
	L8:20%RAP	L7:0%RAP	0.79		0.50	Statistically Insignificant
	L9:40%RAP	L8:20%RAP	-1.4		0.21	Statistically Insignificant
52-40P	L12:40%RAP	L10:0%RAP	0.33		0.80	Statistically Insignificant
	L11:20%RAP	L10:0%RAP	-1.21		0.30	Statistically Insignificant
	L12:40%RAP	L11:20%RAP	1.35		0.20	Statistically Insignificant
64-34(P)	H115RFB:20%RAP	H115C:20%RAP	-2.59		0.04	**Statistically Significant**

***NOTE:** Rubber Terminal-blend (RTB) contains 10% CRM, and Rubber Field-blend (RFB) contains 20% CRM

Besides the effects of RAP and CRM on binder grades, variations in fracture stress values could be attributed to changes in the air voids of the specimens or possible stone-to-stone contact within the mixture composition leading to weak spots in the compacted test specimen.

Other significant factors could be related to the differences in mixing temperature and method of compaction. HMA mixtures incorporating RAP were stiffer than the control mixtures as observed during mixing. The RAP mixtures required a longer mixing time than the control mixtures. However, there were no observable differences in compaction time between RAP and control mixtures. The 20% and 40% laboratory RAP mixtures were compacted in one direction for 12 seconds while the control mixtures were compacted for 15 seconds to achieve the target air voids content of 7±1% in TSRST specimens. Plant RAP and RAP-CRM mixes were respectively compacted for 30 and 40 seconds in both direction (Ambaiowei et al., 2013).

HMA mixtures with higher viscosity tend to exhibit larger build-up of tensile stress during cooling. This allows them to fracture at a higher temperature when compared with mixes with lower viscosity. The behavioural flexibility of various PG 58-28 asphalt mixtures incorporating RAP and CRM under low temperature conditions is highlighted in Figure 5-9.

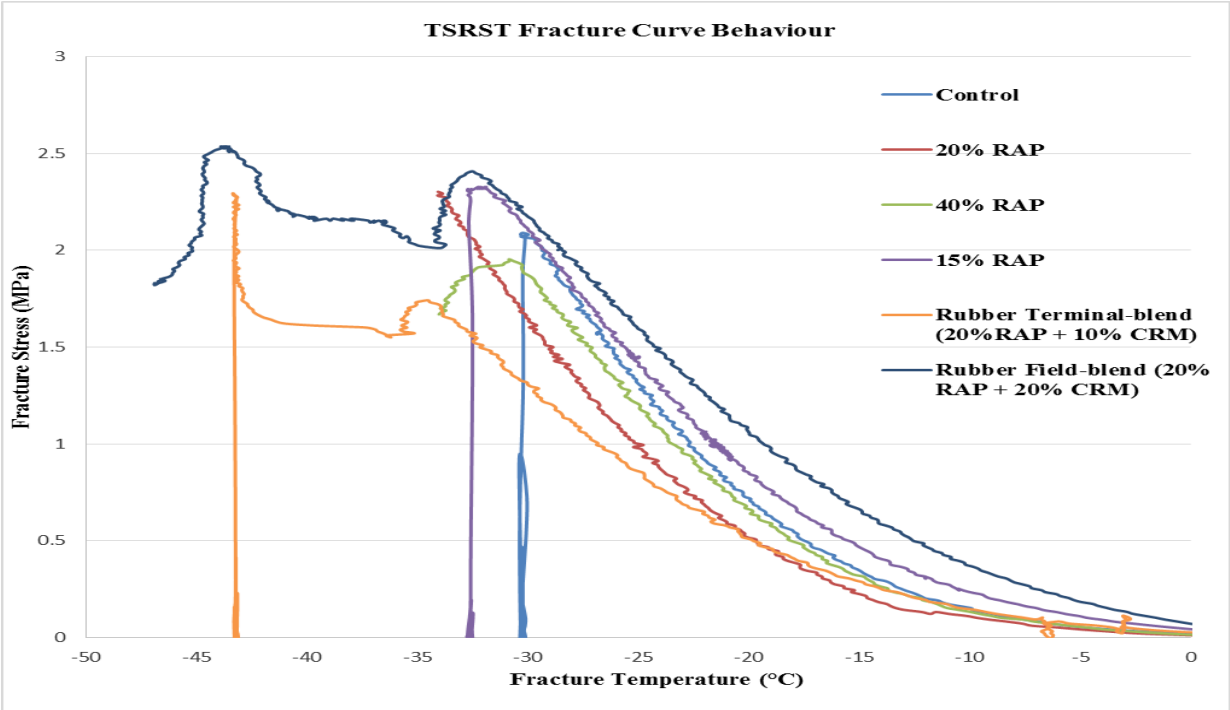

Figure 5-9: Fracture Curve Behaviour for PG 58-28 HMA Mixtures

The CRM is seen to counteract the brittleness of binder by relaxing the build-up of residual stresses in the mix during cooling thus enhancing the flexibility of the binders from temperatures below -32ºC to temperatures colder than -40ºC for terminal and field blend rubber mixes. This flexibility results from the absorption of stresses by the rubber binder and a consequent delay in thermal crack propagation. The PG 64-34P CRM mix also exhibited similar behaviour.

Based on the fracture temperature observations and statistical analysis of the reported fracture stress values, it is reasonable to say that typical Ontario conventional HMA mixes incorporating RAP, or combining RAP and CRM with the proper binder selection will not be prone to transverse thermal cracking at low temperatures or warrant wide spacing between cracks in the event that cracking occurs.

5.3 Rutting, Moisture Damage and Stripping Characterization

The resistance to rutting, moisture damage and stripping potential were conducted on two replicates of paired cylindrical specimens for each HMA mixture. Recall that stripping results when the adhesive bond (asphalt cement) between the aggregates is weakened by moisture damage.

In total, 76 specimens were tested in accordance with the methodology described in Chapter 3. The summary in Appendix B provides data on the rut testing. The rut depths measured in this research suggests that all evaluated Superpave HMA mixtures are relatively good compared to the National Center for Asphalt Technology (NCAT) suggested rut-depth criteria of less than 10 mm after 10,000 load cycles or 20,000 passes for conventional dense-graded HMA mixtures. Note that this criteria is applicable to low volume roads whereas for interstate roads, it is typically 5-6 mm.

Figure 5-10 highlights comparisons made between mixes for rut depths measured at 1,000 and 10,000 load cycles including the mean rut impression calculated as the difference between calculated averages of maximum rut depth at 10,000 load cycles and post-compaction consolidation at 1,000 load cycles.

The RAP only mixtures indicated significant rut improvement with an increase in RAP from 0 to 20 and subsequently 40%. It is observed that maximum rut depths for all mixtures did not exceed 4.1 mm after 10,000 cycles. However, rut depths exceeding 5 mm were noted for mix L8: 20% RAP PG 58-34, L4: 0% RAP PG 52-34 and L5: 20% RAP PG 52-34. For these category of HMA mixtures, Figure 5-10 indicates that incorporating 40% RAP resulted in a stiffer binder blend which was beneficial to resisting the effects of rutting.

The effectiveness of both methods of the wet-process rubberized mixtures is also demonstrated in the observed rutting behaviour of the evaluated mixtures. The combination of RAP and CRM is observed to

stiffen the base PG 58-28 and 64-34P binders resulting in a mix that is more resistance to rutting. This gives an indication that the potential to incorporate higher RAP contents into HMA mixtures using these binder grade exists.

Figure 5-11 compares mean creep slopes between the evaluated HMA mixtures in this research. The creep slope represents the slope of the first steady state portion in the deformation vs. number of passes graph. A higher creep slope suggests the mix is susceptible to rutting. Higher creep slopes in this research are consistent for mixtures with more pronounced wheel impressions. It is observed that the dense-graded PG 58-28 terminal blend mix is the least susceptible to rutting. This is closely followed by the PG 58-28 gap-graded rubber field blend mixtures in the following order: H35-RFB, and H7-RFB. H115-RFB with 20%RAP PG 64-34P had a lower creep slope compared to H115C with 20%RAP PG 64-34. It is important to note that these HMA mixtures with reduced rut depths and creep slopes had higher mix volumetric (VMA, VFA and DP) compared to others within their respective categories.

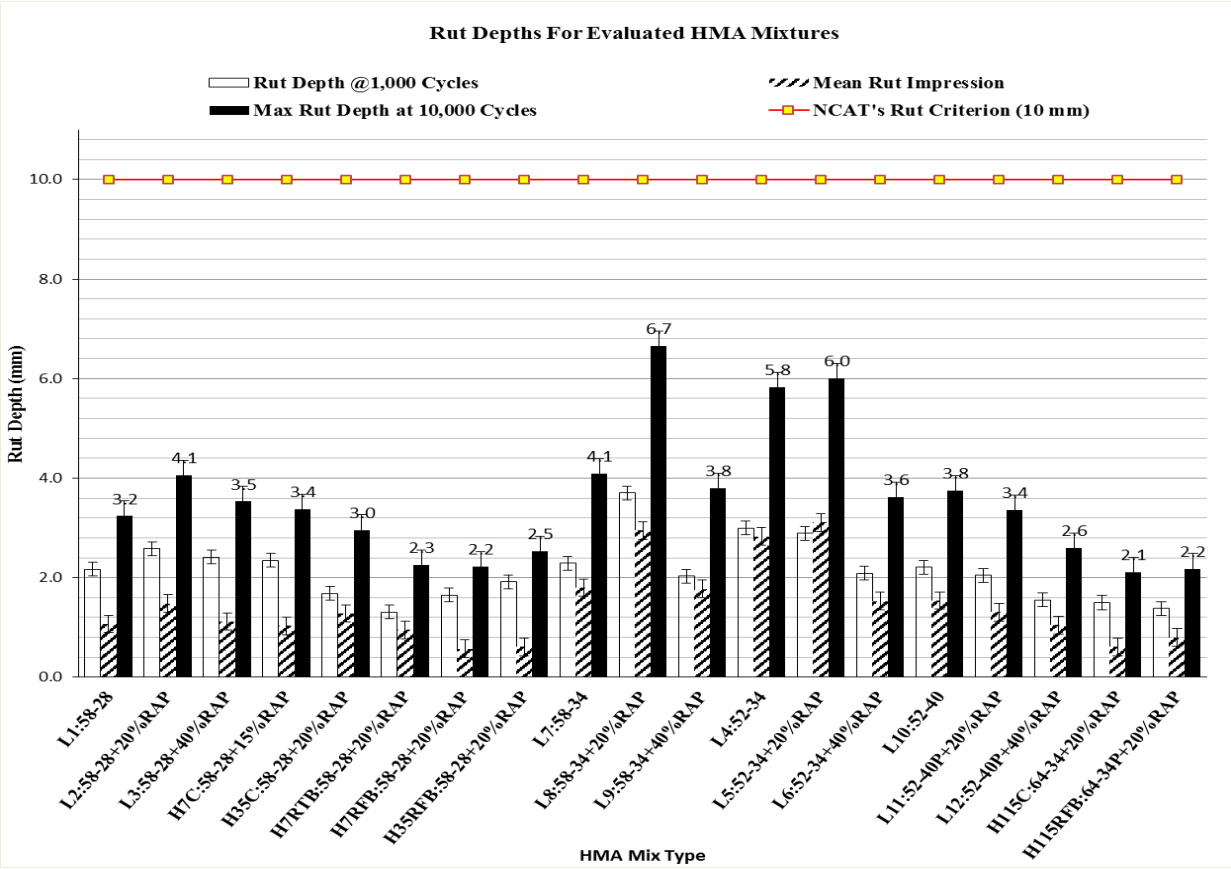

Figure 5-10: Rut Depths between HMA Mixtures at 1,000 and 10,000 Load Cycles

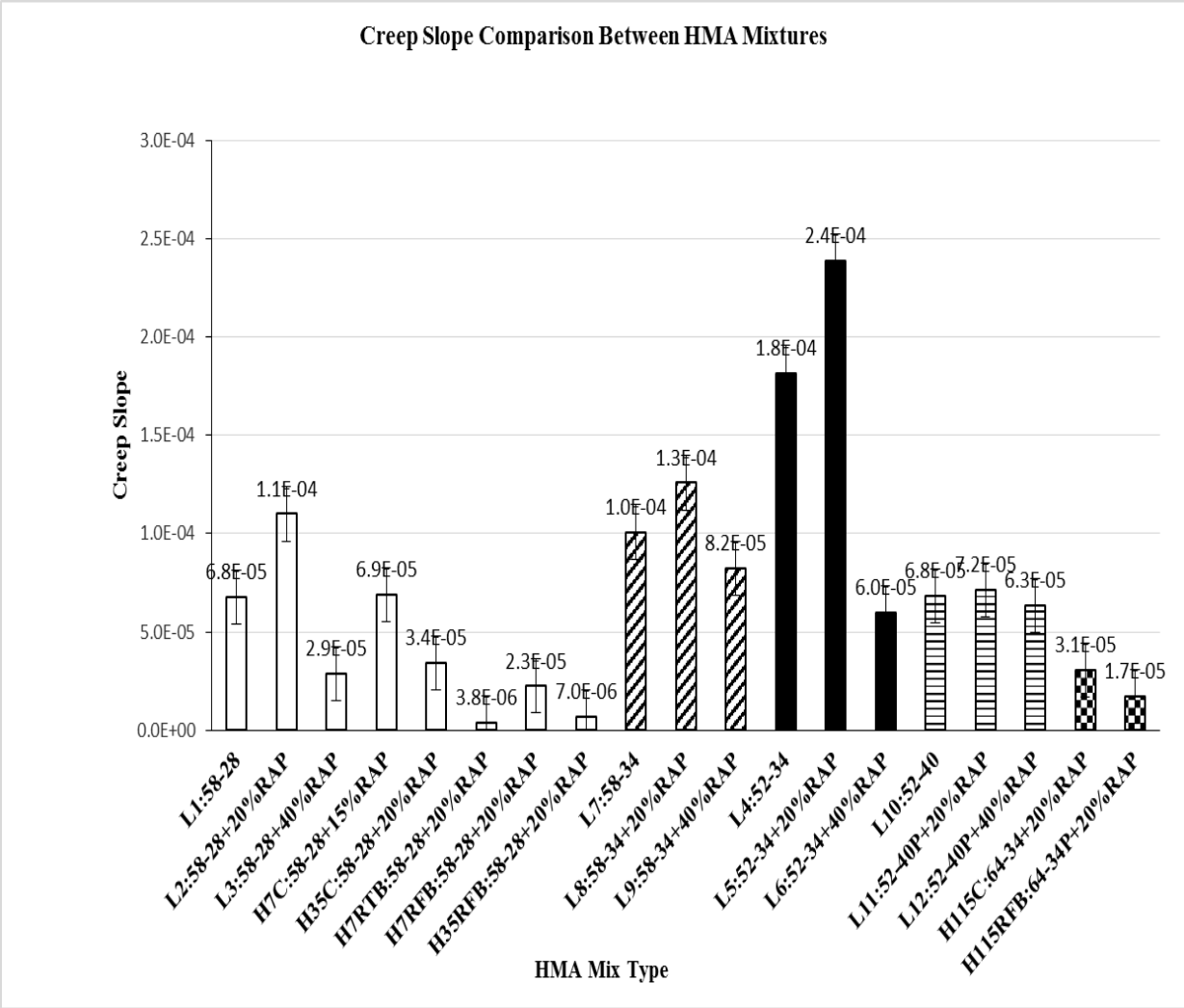

Figure 5-11: Creep Slopes between HMA Mixtures

An Analysis of Variance (ANOVA) was conducted between and within groups of the evaluated HMA mixtures for the maximum rut depths. Details of these statistical findings for maximum rut depth are shown in

Table **5-4**. Statistical differences were found in maximum rut depths for investigations made in the plant-produced PG 58-28, and laboratory-prepared PG 52-40 mix categories. Since a rigorous quality control procedure was enforced for aggregate gradation and mix volumetric in the selection and design of these mixes, the statistical observations for rut depths could be attributed to the differences in air voids between test samples.

Table 5-4: ANOVA for Maximum Rut Depth between HMA Mixtures

Mix Category	$F_{Calculated}$	$F_{Critical}$	P-value	Remark
PG 58-28 (Lab Mix)	0.49	9.55	0.70	Statistically Insignificant
PG 58-28 (Plant Mix)	5.93	5.19	0.04	**Statistically Significant**
PG 58-28 (Lab + Plant Mix)	2.71	3.5	0.09	Statistically Insignificant
PG 58-34 (Lab Mix)	7.46	9.55	0.07	Statistically Insignificant
PG 52-34 (Lab Mix)	6.43	9.55	0.08	Statistically Insignificant
PG 52-40 (Lab Mix)	11.84	9.55	0.03	**Statistically Significant**
PG 64-34 (P) (Plant Mix)	0.02	18.5	0.91	Statistically Insignificant

To affirm the differences in

Table 5-4, an Analysis of Variance (ANOVA) was conducted between and within groups of the evaluated HMA mixtures for the creep slope. Details of statistical findings for creep slope are shown in Table 5-5. It should be noted that creep slope is preferred for evaluating rutting potential instead of rut-depth because the number of load cycles at which moisture begins to affect rut depth varies between HMA mixtures, and cannot be conclusively determined from the rut plot (Asphalt Institute, 2010). Table 5-5 also suggests that the air void differences between paired test samples may have been lost over the test duration. This explains why those mixtures behaved similarly regardless of differences in mix type, addition of crumb rubber or change in the binder grade. Statistical analysis thus suggests that the rutting resistance of recycled mixtures incorporating RAP or combining RAP and CRM will result in a mix that compares favourably with conventional HMA mixtures if they are properly designed and compacted.

Table 5-5: ANOVA for Creep Slope between HMA Mixtures

Mix Category	$F_{Calculated}$	$F_{Critical}$	P-value	Remark
PG 58-28 (Lab Mix)	5.79	9.55	0.10	Statistically Insignificant
PG 58-28 (Plant Mix)	2.94	5.19	0.13	Statistically Insignificant
PG 58-28 (Lab + Plant Mix)	5.23	3.51	0.02	Statistically Insignificant
PG 58-34 (Lab Mix)	0.17	9.55	0.90	Statistically Insignificant
PG 52-34 (Lab Mix)	7.9	9.55	0.04	Statistically Insignificant
PG 52-40 (Lab Mix)	0.03	9.55	0.9	Statistically Insignificant
PG 64-34 (P) (Plant Mix)	0.39	18.5	0.6	Statistically Insignificant

It should also be noted that Stripping Inflection Point (SIP) could not be determined for any of the evaluated mixtures at any load cycle less than 10,000. A correlation analysis between all mix variables was conducted, but no strong correlations were observed. Consequently, the evaluated mixtures were not susceptible to moisture damage or stripping since stripping slope was not detected either graphically from the rut depth data or by visual inspection of the tested samples.

The resistance to rutting, stripping and moisture damage is attributed to the mix volumetric, good aggregate skeleton bond, good compactability, and the mixtures exceeding the minimum 80 percent requirement for Tensile Stress Ratio (TSR). Table 5-6 compares the laboratory rut data with those obtained from field trial sections. The field survey of rut depths were completed by the MTO in June 2013 using the Automated Road Analyzer (ARAN). The results which are observed to be below MTO's 6 mm minimum rut criterion for slight rutting confirms the effectiveness of RAP and CRM as a valuable component for typical Ontario HMA mixtures (Ambaiowei et al., 2014).

Table 5-6: Performance Comparison between Lab Mixes and Field Sections

Mix Description	Laboratory Evaluation		MTO's 2013 ARAN Survey of Field Sections		
	Mean Creep Slope	Mean Rut Depth (mm)	Length (Km)	Average IRI (m/Km)	Mean Rut Depth (mm), both Directions
H7RTB:PG 58-28 (10% CRM + 20% RAP)	3.83E-06	2.3	1.5	1.12	1.9
H7RFB:PG 58-28 (20% CRM + 20% RAP)	2.25E-05	2.2	3	1.14	2.5
H115-C:PG 64-34 (20% CRM + 20% RAP)	3.05E-05	2.1	1	1.93	2.5
H115RFB:PG 64-34P (20% CRM + 20% RAP)	1.73E-05	2.2	3	0.93	2.0

5.4 Mix Stiffness Characterization

A comprehensive stiffness assessment of the evaluated HMA mixtures was performed using the dynamic modulus test protocol described in Chapter Three. Tests were conducted in triplicates for each HMA mix evaluated. In total, 57 samples were tested. The average dynamic modulus and corresponding phase angle values from test triplicates at varying temperatures (-10, 4, 21, 37 and 54ºC) and loading frequencies (0.1, 0.5, 1, 5, 10 and 25Hz) for the respective HMA mixtures evaluated in this study are tabulated in Appendix C.

A lower stiffness at low temperatures or high loading frequencies will help to minimize fatigue and low temperature cracking. To reduce rutting potential, a higher stiffness at high temperatures or low loading frequencies is desirable. As expected, the trend in the dynamic modulus data collected between mixtures showed a reduction in mix stiffness as the temperature increased from -10ºC to 54ºC, and an increase in stiffness with increasing loading frequencies; from 0.1Hz to 25Hz. HMA mixtures incorporating 40% RAP were observed to have higher stiffness at low and high frequencies in comparison to 20% RAP,

and mixtures without RAP. This trend was consistent for all PG binders used. For comparisons made in the plant-prepared mix category, the stiffness of the Rubberized-RAP HMA mixtures were noted to be higher at all temperatures and loading frequencies than those with RAP or mixtures in the laboratory-prepared mix category.

A high phase angle at high stiffness is indicative of a binder with reduced relaxation properties thus reducing its tendency for thermal cracking; whereas a low phase angle at low stiffness is indicative of a binder capable of preventing rutting. The phase angle data between the evaluated mixtures in this study were observed to reduce with increasing loading frequency, and testing temperature. That is to say, higher phase angles were recorded at lower frequencies while lower phase angles were recorded at higher frequencies. However, the measured phase angles at 1Hz across all test temperatures were observed to be higher than those at 0.5Hz for mixtures and samples tested.

For HMA mixtures in the laboratory-prepared category, 40% RAP recorded lower phase angles in comparison to those without RAP for all loading frequencies at -10ºC and 4ºC test temperature. However, both were higher for comparisons made with 20% RAP HMA mixtures. This trend or behaviour in phase angle of the evaluated mixtures are maintained in the Rubberized-RAP HMA mixtures, but it was observed that these mixtures irrespective of the method of CRM incorporation recorded phase angles that were significantly higher in comparison to the conventional mixtures, and mixtures incorporating 15% and 20% RAP.

At 21ºC, 37ºC and 54ºC test temperatures, the performance of the recycled mixtures were better than the conventional mixes. These observations confirm the effects of RAP or combining RAP and CRM on the viscoelastic behaviour of asphalt binder, and consequently in HMA mixtures. In this case, improvements in the data for mix stiffness and phase angle are observed. In accordance with the research methodology, the F-test analysis of variance (ANOVA) assuming equal variances was employed to examine variability in the test data. These validations were compared in order to determine which mix performed the best. The details of the F-test ANOVA for mix stiffness and phase angle are presented in Table 5-7 and Table 5-8 respectively with remarks. Data validation mostly reported weak statistical evidence of differences between the HMA mixtures. In cases where the differences are statistically insignificant, the mixes are deemed to be the same. While, statistically significant validations suggests that there are differences in mix stiffness and phase angle data between respective mix pairs.

Table 5-7: Validated Stiffness Data for Evaluated HMA Mixtures

Binder PG	HMA Mix Couple		$F_{Calculated}$	$F_{critical}$ One tail	P(F<=f) one-tail	Remark
58-28	L3:40%RAP	L1:0%RAP	1.19	1.86	0.30	Statistically Insignificant
	L2:20%RAP	L1:0%RAP	1.14		0.40	Statistically Insignificant
	L3:40%RAP	L2:20%RAP	1.05		0.50	Statistically Insignificant
	H7RFB:20%RAP	L1:0%RAP	0.74	0.54	0.20	Statistically Insignificant
	H7RFB:20%RAP	L2:20%RAP	0.66		0.10	Statistically Insignificant
	H7RFB:20%RAP	L3:40%RAP	0.62		0.10	Statistically Insignificant
	H7RTB:20%RAP	L1:0%RAP	0.32		0.00	Statistically Insignificant
	H7RTB:20%RAP	L2:20%RAP	0.28			Statistically Insignificant
	H7RTB:20%RAP	L3:40%RAP	0.27			Statistically Insignificant
	H35RFB:20%RAP	L1:0%RAP	0.69		0.20	**Statistically Significant**
	H35RFB:20%RAP	L2:20%RAP	0.61		0.10	**Statistically Significant**
	H35RFB:20%RAP	L3:40%RAP	0.59		0.10	**Statistically Significant**
	H35RFB:20%RAP	H35C:20%RAP	0.57		0.10	**Statistically Significant**
	H7RFB:20%RAP	H7C:15%RAP	0.94		0.40	**Statistically Significant**
	H7RTB:20%RAP	H7RFB:20%RAP	0.43		0.01	Statistically Insignificant
	H7RTB:20%RAP	H7C:15%RAP	0.41		0.01	Statistically Insignificant
	H7RTB:20%RAP	H35RFB:20%RAP	0.46		0.00	Statistically Insignificant
	H7RTB:20%RAP	H35C:20%RAP	0.26		0.10	Statistically Insignificant
	H7RFB:20%RAP	H35C:20%RAP	0.61		0.10	Statistically Insignificant
52-34	L6:40%RAP	L4:0%RAP	1.24	1.86	0.30	Statistically Insignificant
	L5:20%RAP	L4:0%RAP	1.08		0.40	Statistically Insignificant
	L6:40%RAP	L5:20%RAP	1.15		0.40	Statistically Insignificant
58-34	L9:40%RAP	L7:0%RAP	1.23		0.30	Statistically Insignificant
	L8:20%RAP	L7:0%RAP	1.22		0.30	Statistically Insignificant
	L9:40%RAP	L8:20%RAP	1.01		0.50	Statistically Insignificant
52-40P	L12:40%RAP	L10:0%RAP	1.82		0.10	Statistically Insignificant
	L11:20%RAP	L10:0%RAP	2.43		0.01	**Statistically Significant**
	L12:40%RAP	L11:20%RAP	0.75		0.20	Statistically Insignificant
64-34(P)	H115RFB:20%RAP	H115C:20%RAP	1.13		0.40	Statistically Insignificant

***NOTE:** Rubber Terminal-blend (RTB) contains 10% CRM, and Rubber Field-blend (RFB) contains 20% CRM

Table 5-8: Validated Phase Angle Data for Evaluated HMA Mixtures

Binder PG	HMA Mix Couple		$F_{Calculated}$	$F_{critical}$ One tail	P(F<=f) one-tail	Remark
58-28	L3:40%RAP	L1:0%RAP	1.21	1.86	0.30	Statistically Insignificant
	L2:20%RAP	L1:0%RAP	1.22		0.30	Statistically Insignificant
	L3:40%RAP	L2:20%RAP	0.99	0.54	0.50	Statistically Insignificant
	H7RFB:20%RAP	L1:0%RAP	1.47	1.86	0.20	Statistically Insignificant
	H7RFB:20%RAP	L2:20%RAP	1.21		0.30	Statistically Insignificant
	H7RFB:20%RAP	L3:40%RAP	1.21		0.30	Statistically Insignificant
	H7RTB:20%RAP	L1:0%RAP	0.37	0.54	0.00	Statistically Insignificant
	H7RTB:20%RAP	L2:20%RAP	0.31			Statistically Insignificant
	H7RTB:20%RAP	L3:40%RAP	0.31			Statistically Insignificant
	H35RFB:20%RAP	L1:0%RAP	1.60	1.86	0.10	Statistically Insignificant
	H35RFB:20%RAP	L2:20%RAP	1.32		0.20	Statistically Insignificant
	H35RFB:20%RAP	L3:40%RAP	1.32		0.20	Statistically Insignificant
	H35RFB:20%RAP	H35C:20%RAP	1.90		0.10	**Statistically Significant**
	H7RFB:20%RAP	H7C:15%RAP	1.88		0.10	**Statistically Significant**
	H7RTB:20%RAP	H7RFB:20%RAP	0.25	0.54	0.00	Statistically Insignificant
	H7RTB:20%RAP	H7C:15%RAP	0.48		0.00	Statistically Insignificant
	H7RTB:20%RAP	H35RFB:20%RAP	0.23		0.00	Statistically Insignificant
	H7RTB:20%RAP	H35C:20%RAP	0.44		0.00	Statistically Insignificant
	H7RFB:20%RAP	H35C:20%RAP	1.74	1.86	0.10	Statistically Insignificant
52-34	L6:40%RAP	L4:0%RAP	1.08		0.40	Statistically Insignificant
	L5:20%RAP	L4:0%RAP	0.91	0.5	0.40	**Statistically Significant**
	L6:40%RAP	L5:20%RAP	1.18	1.86	0.30	Statistically Insignificant
58-34	L9:40%RAP	L7:0%RAP	1.13		0.40	Statistically Insignificant
	L8:20%RAP	L7:0%RAP	0.82	0.5	0.30	**Statistically Significant**
	L9:40%RAP	L8:20%RAP	0.89		0.40	**Statistically Significant**
52-40P	L12:40%RAP	L10:0%RAP	1.07	1.86	0.40	Statistically Insignificant
	L11:20%RAP	L10:0%RAP	0.82	0.5	0.30	**Statistically Significant**
	L12:40%RAP	L11:20%RAP	1.29	1.86	0.20	Statistically Insignificant
64-34(P)	H115RFB:20%RAP	H115C:20%RAP	1.15		0.40	Statistically Insignificant

***NOTE:** Rubber Terminal-blend (RTB) contains 10% CRM, and Rubber Field-blend (RFB) contains 20% CRM

Note that the validation analysis was completed to confirm the consistency of the data collected for mix stiffness and phase angles between the paired HMA mixtures. The paired t-test for sample means was used to assess and compare performance of the HMA mixtures in terms of stiffness and corresponding

phase angles after the data were validated. The details of t-test for mix stiffness are presented in Table 5-9 with remarks.

Table 5-9: Performance Assessment of HMA Mixtures Based on Stiffness

Binder PG	HMA Mix Couple		$t_{Calculated}$	$t_{Critical}$ two-tail	P(T<=t) two-tail	Remark
58-28	L3:40%RAP	L1:0%RAP	5.16	2.05	0	**Statistically Significant**
	L2:20%RAP	L1:0%RAP	6.37			**Statistically Significant**
	L3:40%RAP	L2:20%RAP	2.55			**Statistically Significant**
	H7RFB:20%RAP	L1:0%RAP	-4.44			**Statistically Significant**
	H7RFB:20%RAP	L2:20%RAP	-5.19			**Statistically Significant**
	H7RFB:20%RAP	L3:40%RAP	-5.14			**Statistically Significant**
	H7RTB:20%RAP	L1:0%RAP	-5.07			**Statistically Significant**
	H7RTB:20%RAP	L2:20%RAP	-5.27			**Statistically Significant**
	H7RTB:20%RAP	L3:40%RAP	-5.26			**Statistically Significant**
	H35RFB:20%RAP	L1:0%RAP	-4.97			**Statistically Significant**
	H35RFB:20%RAP	L2:20%RAP	-5.48			**Statistically Significant**
	H35RFB:20%RAP	L3:40%RAP	-5.41			**Statistically Significant**
	H35RFB:20%RAP	H35C:20%RAP	-5.15			**Statistically Significant**
	H7RFB:20%RAP	H7C:15%RAP	3.09			**Statistically Significant**
	H7RTB:20%RAP	H7RFB:20%RAP	-5.29			**Statistically Significant**
	H7RTB:20%RAP	H7C:15%RAP	-4.46			**Statistically Significant**
	H7RTB:20%RAP	H35RFB:20%RAP	-5.05			**Statistically Significant**
	H7RTB:20%RAP	H35C:20%RAP	-5.12			**Statistically Significant**
	H7RFB:20%RAP	H35C:20%RAP	-4.87			**Statistically Significant**
52-34	L6:40%RAP	L4:0%RAP	6.82			**Statistically Significant**
	L5:20%RAP	L4:0%RAP	7.06			**Statistically Significant**
	L6:40%RAP	L5:20%RAP	4.08			**Statistically Significant**
58-34	L9:40%RAP	L7:0%RAP	5.99			**Statistically Significant**
	L8:20%RAP	L7:0%RAP	7.34			**Statistically Significant**
	L9:40%RAP	L8:20%RAP	-6.8			**Statistically Significant**
52-40P	L12:40%RAP	L10:0%RAP	7.01			**Statistically Significant**
	L11:20%RAP	L10:0%RAP	6.97			**Statistically Significant**
	L12:40%RAP	L11:20%RAP	-6.48			**Statistically Significant**
64-34(P)	H115RFB:20%RAP	H115C:20%RAP	6.65			**Statistically Significant**

***NOTE:** Rubber Terminal-blend (RTB) contains 10% CRM, and Rubber Field-blend (RFB) contains 20% CRM

Table 5-9 shows that when the value for $t_{Calculated}$ is greater than the $t_{Critical}$ with a corresponding p-value less than 0.1 for all HMA mix pairs, this suggests there are significant differences. In short, the recycled materials in typical Ontario HMA for all categories of binders evaluated are shown to be stiffer. This shows that increasing the RAP content or combining RAP with CRM will result in a stiffer mix which is more rut resistant compared to conventional HMA mixtures. However, it is important to establish that the observed mix stiffness is not exclusively a function of the improvements made to the binder, but in combination with other factors. The improvements to the binder do have a major influencing role though. The t-test for mix phase angles were determined in this regard, and are presented in Table 5-10 and Table 5-11 with remarks.

Table 5-10: Performance Assessment of PG 58-28 HMA Mixtures Based on Phase Angle

Binder PG	HMA Mix Couple		$t_{Calculated}$	$t_{Critical}$ two-tail	P(T<=t) two-tail	Remark
58-28	L3:40%RAP	L1:0%RAP	0.39	2.05	0.70	Statistically Insignificant
	L2:20%RAP	L1:0%RAP	-0.47		0.60	Statistically Insignificant
	L3:40%RAP	L2:20%RAP	1.09		0.30	Statistically Insignificant
	H7RFB:20%RAP	L1:0%RAP	1.52		0.10	Statistically Insignificant
	H7RFB:20%RAP	L2:20%RAP	1.49		0.20	Statistically Insignificant
	H7RFB:20%RAP	L3:40%RAP	1.08		3.00	Statistically Insignificant
	H7RTB:20%RAP	L1:0%RAP	-1.05		0.30	Statistically Insignificant
	H7RTB:20%RAP	L2:20%RAP	-0.67		0.50	Statistically Insignificant
	H7RTB:20%RAP	L3:40%RAP	-0.98		0.34	Statistically Insignificant
	H35RFB:20%RAP	L1:0%RAP	0.69		0.50	Statistically Insignificant
	H35RFB:20%RAP	L2:20%RAP	0.78		0.40	Statistically Insignificant
	H35RFB:20%RAP	L3:40%RAP	0.4		0.70	Statistically Insignificant
	H35RFB:20%RAP	H35C:20%RAP	0.82		0.40	Statistically Insignificant
	H7RFB:20%RAP	H7C:15%RAP	0.89		0.40	Statistically Insignificant
	H7RTB:20%RAP	H7RFB:20%RAP	-1.71		0.10	Statistically Insignificant
	H7RTB:20%RAP	H7C:15%RAP	-1.47		0.20	Statistically Insignificant
	H7RTB:20%RAP	H35RFB:20%RAP	-1.17		0.30	Statistically Insignificant
	H7RTB:20%RAP	H35C:20%RAP	-0.89		0.40	Statistically Insignificant
	H7RFB:20%RAP	H35C:20%RAP	1.73		0.10	Statistically Insignificant

***NOTE:** Rubber Terminal-blend (RTB) contains 10% CRM, and Rubber Field-blend (RFB) contains 20% CRM

Table 5-11: Performance Assessment of HMA Mixtures Based on Phase Angle - Continued

Binder PG	HMA Mix Couple		$t_{Calculated}$	$t_{Critical}$ two-tail	P(T<=t) two-tail	Remark
52-34	L6:40%RAP	L4:0%RAP	1.14	2.05	0.30	Statistically Insignificant
	L5:20%RAP	L4:0%RAP	0.39		0.70	Statistically Insignificant
	L6:40%RAP	L5:20%RAP	1.41		0.10	Statistically Insignificant
58-34	L9:40%RAP	L7:0%RAP	-1.42		0.20	Statistically Insignificant
	L8:20%RAP	L7:0%RAP	-0.77		0.50	Statistically Insignificant
	L9:40%RAP	L8:20%RAP	-0.49		0.60	Statistically Insignificant
52-40P	L12:40%RAP	L10:0%RAP	0.73		0.50	Statistically Insignificant
	L11:20%RAP	L10:0%RAP	-0.62			Statistically Insignificant
	L12:40%RAP	L11:20%RAP	4.72		0.00	**Statistically Significant**
64-34(P)	H115RFB:20%RAP	H115C:20%RAP	-2.16		0.04	**Statistically Significant**

***NOTE:** Rubber Terminal-blend (RTB) contains 10% CRM, and Rubber Field-blend (RFB) contains 20% CRM

In Table 5-10 and Table 5-11, it is observed that values for $t_{Calculated}$ are less than $t_{Critical}$ with a corresponding p-value greater than or equal to 0.1 for all HMA mix pairs except for comparisons made between a 40% and 20% RAP mix with PG 52-40 asphalt binder. The rubberized mix with 20% RAP PG64-34P mix and 20% RAP mix with PG 64-34 asphalt binder showed similar trends.

The statistically insignificant scenarios are indicative of a lesser degree of sensitivity on effects of increasing RAP contents or combining these with CRM. That is to say, the degree to which the recycled components improved the phase angles of the respective binder grades at high and low frequencies is less. Hence the phase angle of these recycled HMA mixtures are comparable between mixes in the order in which they were paired. For example, the phase angle of a 40% RAP mix is similar to a mix without RAP in the PG 58-28 laboratory mix category.

In contrast, statistically significant scenarios are indicative of a higher degree of sensitivity on the effects of increasing RAP contents or combining same with CRM. This means that the degree to which recycled components improved the phase angles of the respective binder grades at high and low stiffness is greater. Hence the phase angle of a field blend Rubberized-RAP with PG 64-34P asphalt binder is better when compared with a 20% RAP mix with PG 64-34 asphalt binder.

Regardless of the statistical sensitivity of the phase angle to increasing RAP content or in combination with CRM for the respective binder grades, the overall behavioral characteristic of the HMA mixtures indicates that the observed improvements in stiffness of the recycled mixtures were predominately affected by the mix volumetric, good aggregate skeleton bond, and good compactability.

The main deductions from the t-test assessment of mix stiffness and phase angle characterization is that increasing the RAP content or combining RAP with CRM will result in a mix that is stiff, more elastic or flexible and capable of withstanding the effects of low temperature cracking and fatigue cracking including those of rutting or permanent deformation if such mixtures are properly designed, mixed and compacted as is the case with the HMA mixtures evaluated in this study.

5.5 Master Curve Development for Evaluated HMA Mixtures

The stress-strain behavior of HMA mixture is primarily dependent on temperature and loading rate. By constructing a master curve, it is possible to observe the behavioral changes in the asphalt binder and mixture over time. Master curves are constructed using the time-temperature superposition principle. This principle assumes that the effect of time of loading or frequency on the material properties can be replaced by the effects of temperature, and vice versa. In this way, the master curve can be defined as a function that describes simultaneously the dependency of dynamic modulus on both the temperature and the time of loading using shift factors. The amount of shifting at each temperature required to form the master curve describes the temperature dependency of the material. The greater the shift factor, the greater the temperature susceptibility of the mixture. A number of approaches exist to establish this relationship between shift factors and temperature.

In this study, stiffness master curves for the respective HMA mixtures evaluated were constructed and compared using the rheology and master curve analysis software (RHEA™, 2011). RHEA is a user friendly software that allows the conversion of dynamic mechanical data from the frequency domain to the time domain, and vice-versa (RHEA™, 2011). The benefit of using this software is that the shifting procedure is done automatically to a reference temperature of 20°C.

The RHEA™ software also provided allowance for smoothening the 1Hz bump in phase angle data collected across all HMA mixtures before the shifting task was completed. The reduced frequency and smoothened master curves were plotted to verify the behavioral observations from the t-test statistical analysis. Figure 5-12 through Figure 5-18 shows the completed master curves for all 19 evaluated HMA mixtures in the respective order of comparison.

Although statistical analysis of phase angle data suggests that the behavioral tendencies of the evaluated mixtures are similar or comparable, the master curves indicate that there are differences with respect to how each mix would respond to resisting the effects of pavement rutting, fatigue and thermal cracking at high, intermediate and low temperatures respectively. These differentials as earlier mentioned are related to the variations between mix stiffness, and influenced by the interaction between temperature changes on asphalt binder properties, mix volumetric and the aggregate structure within each mix.

The presence of RAP in increasing percentages or combining RAP and CRM in mixtures with PG 58-28 binder as seen in Figure 5-12 through Figure 5-14 suggests that these category of mixtures behaved similarly at intermediate and high loading frequencies or lower temperatures since the curves tend to come together. However at low loading frequencies or higher temperatures, a small degree of separation is observed in favour of the recycled mixtures.

The most significant differences for this category of HMA mixtures relates to the method of CRM incorporation. As shown in Figure 5-13 and Figure 5-14, the terminal blend wet-process mix (H7RTB-20% RAP+10% CRM) indicated a reasonable degree of separation by exhibiting the lowest stiffness at lower temperatures. However, its rutting behaviour is comparable to the wet-process field blend mixes with 20% RAP + 20% CRM ratio.

Another interesting observation for these types of HMA mixtures is that the master curves at low and intermediate temperatures for the control mix, 15, 20 and 40% RAP mixes respectively, including the field blend Rubberized-RAP mixtures appear to show a concave behaviour whereas their high temperature behaviour appears to be a somewhat convex. However, the master curve for the terminal blend Rubberized-RAP mix exhibits a consistent straight line at the low, intermediate and high temperatures. The concave behaviour is representative of a non-linear behavior during compression as well as an increasing influence of the aggregate-skeleton's mechanical response on the viscous asphalt binder while the convex or apparent straight line is indicative of a more viscoelastic mix that is predominantly influenced by changes in the asphalt binder type.

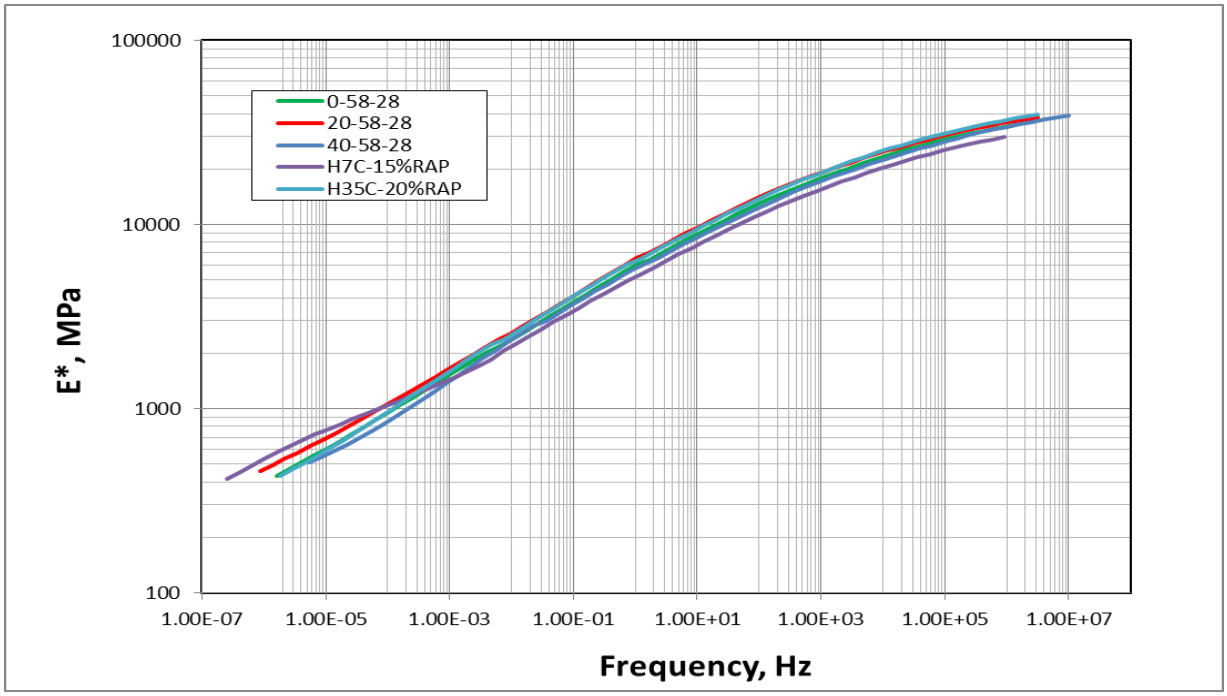

Figure 5-12: Master Curve for Lab and Plant HMA Mixtures with PG 58-28 and 0, 15, 20 and 40% RAP

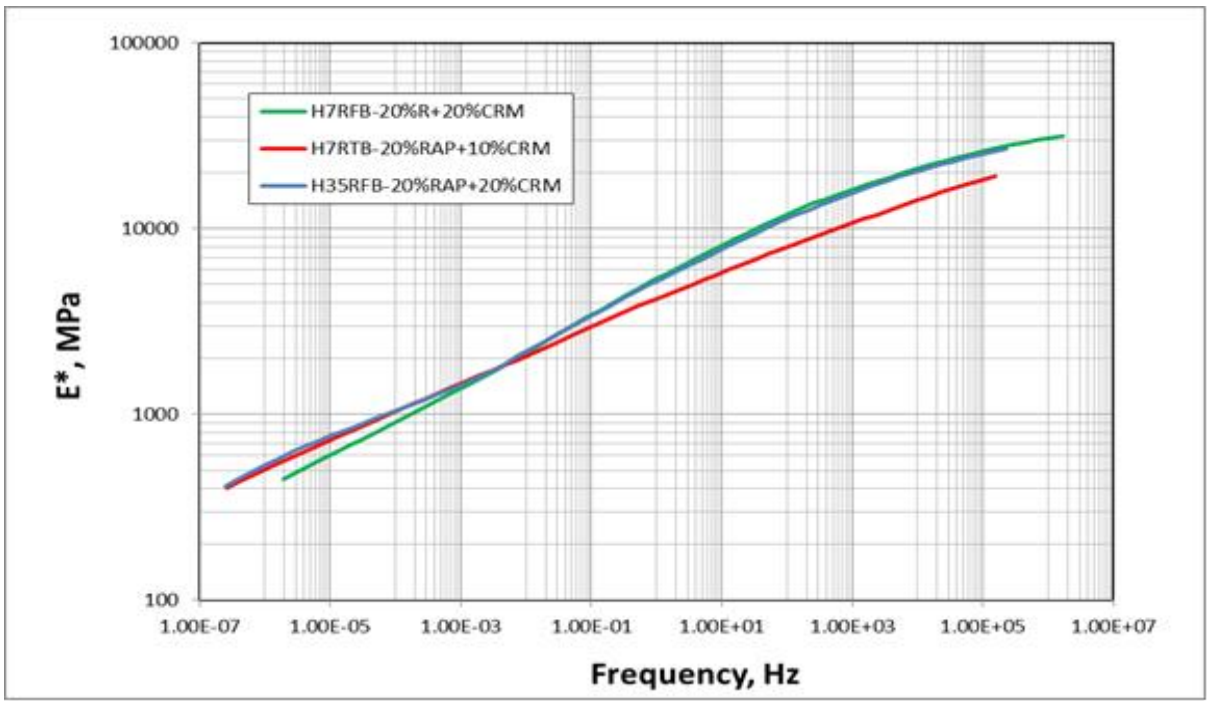

Figure 5-13: Master Curve for Plant Rubberized-RAP PG 58-28 HMA Mixtures

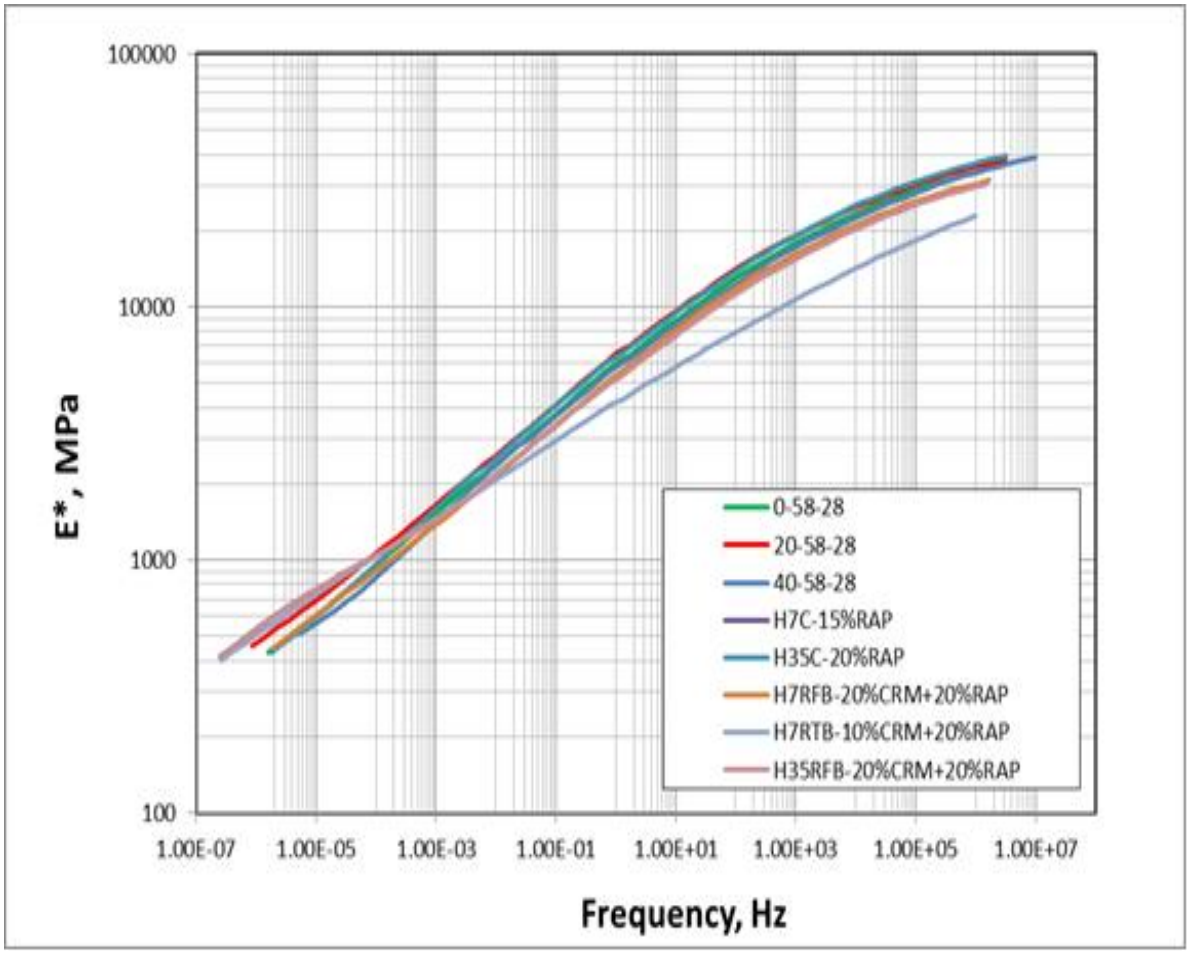

Figure 5-14: Master Curve for PG 58-28 HMA Mixtures with CRM+RAP, RAP only and No RAP

In Figure 5-15, the HMA mixtures exhibited a reasonable degree of separation. The differences in master curves are observed between PG 52-40 HMA mixtures incorporating 0%, 20% and 40% RAP. The 20% RAP mix appears to be stiffer than a 40% RAP mix which in turn was stiffer than the control mix. The recycled mixtures also exhibited a more concave behaviour compared to the control mix which is a straight line. At the low loading frequencies, the master curves suggests that a 20% and 40% RAP PG 52-40 mix will perform better in rutting respectively. At the intermediate loading frequencies, the master curves suggests that the control and 40% RAP PG 52-40 mix will perform better in fatigue cracking resistance as opposed to the 20% RAP mix since they are more flexible. At the high loading frequency, the control mix and 40% RAP mix will be more resistant to thermal cracking as compared to the 20% RAP HMA mixtures.

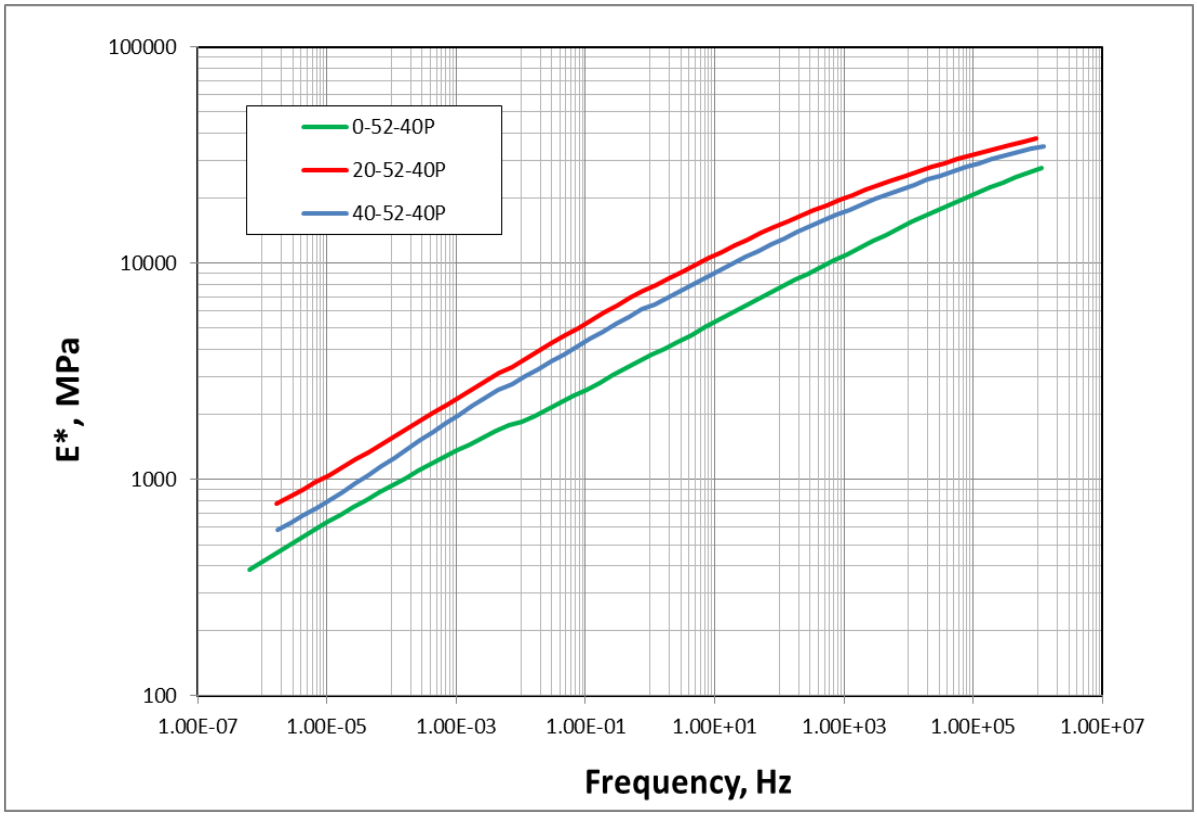

Figure 5-15: Master Curve – PG 52-40 for 0, 20 and 40% RAP

Very small differences in master curves are observed between HMA mixtures utilizing PG 52-34 asphalt binders at the intermediate and high loading frequencies. Figure 5-16 shows that at high loading frequency, the 20% and 40% RAP PG 52-34 will perform similarly, but the control mix will perform better in terms of low temperature crack susceptibility. At the intermediate loading frequency, the 40% RAP mix appears to have a slight edge in stiffness over the 20% RAP and control mix. However, the control and 20% RAP mix are more flexible in terms of fatigue resistance. At the high loading frequency, the 20% RAP mix is seen to be stiffer than the 40% RAP mix which in turn is stiffer than the control mix. This suggest that the 20% RAP PG 52-34 mix has better resistance to rutting or permanent deformation compared to the 40% RAP and control mix. A critical look at the master curve for this group of HMA mixtures shows that the concave behaviour of the curve is only consistent at the high loading frequency or low temperature region whereas the straight line were consistent at the intermediate and high loading frequencies. This confirms that good compaction aided the mix behaviour at low temperatures while influences from the asphalt binder controls the fatigue and rutting behaviour.

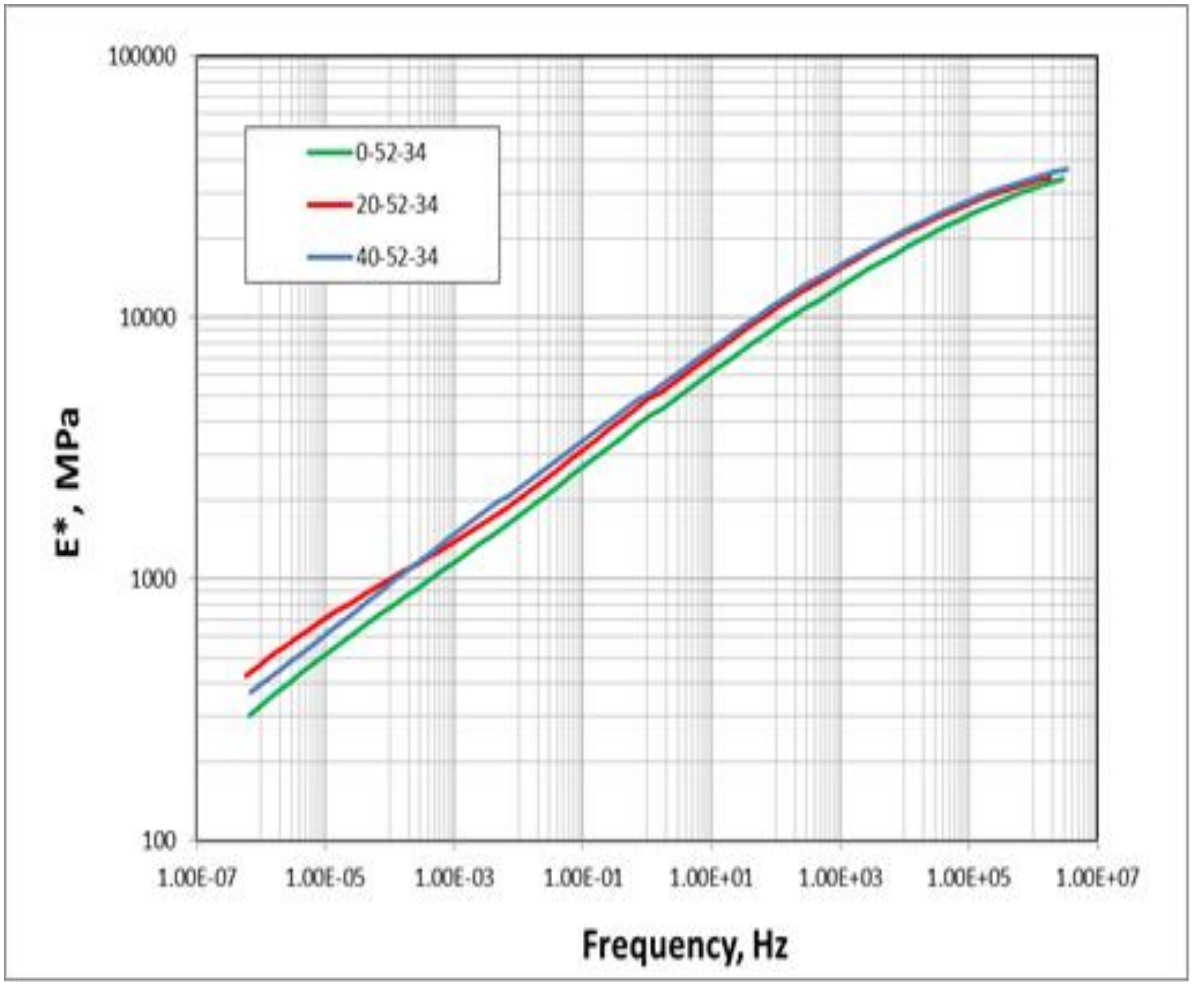

Figure 5-16: Master Curve for Lab Mix with PG 52-34 and 0, 20 and 40% RAP

In Figure 5-17, all RAP HMA mix configurations utilizing a PG 58-34 asphalt binder appeared to have a concave behaviour across the low, intermediate and high loading frequencies which is indicative of the predominant influence of the aggregate-skeleton's mechanical response on the viscous asphalt binder. The 20 and 40% RAP mix appear to be comparable in stiffness at both the low and high loading frequencies in comparison to the control mix. However, at the intermediate loading frequency, the control mixtures appears to be more flexible than the recycled mixtures. The 40% RAP mix is also observed to be slightly more flexible compared to the 20% RAP mix; hence will perform better in terms of fatigue resistance. The control mix is also observed to have a slightly better resistance to thermal cracking compared to the recycled HMA mixtures in this category.

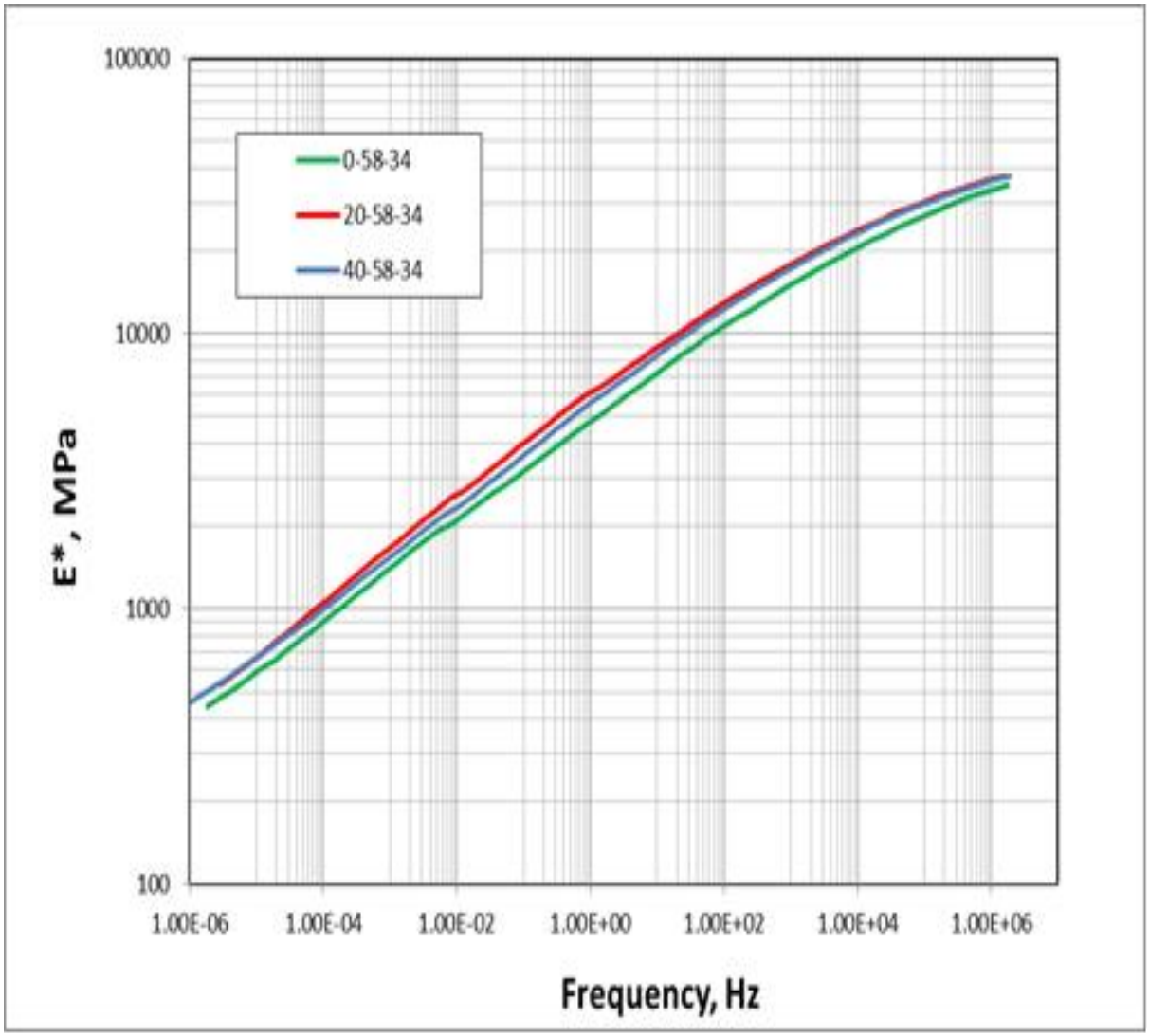

Figure 5-17: Master Curve for Lab Mix with PG 58-34 and 0, 20 and 40%RAP

Figure 5-18 indicates that the behaviour of rubberized-RAP mix with PG 64-34P and RAP mix with PG 64-34 binder may be comparable for resistance to rutting at the low loading frequencies. In terms of thermal cracking resistance, the Rubberized-RAP mix has a slight edge over the control-RAP mix. However, at the intermediate loading frequency, the Rubberized-RAP mix appears to be more fatigue resistant considering that it is more flexible. A combination of the concave and straight line behaviour is also observed for these mixtures.

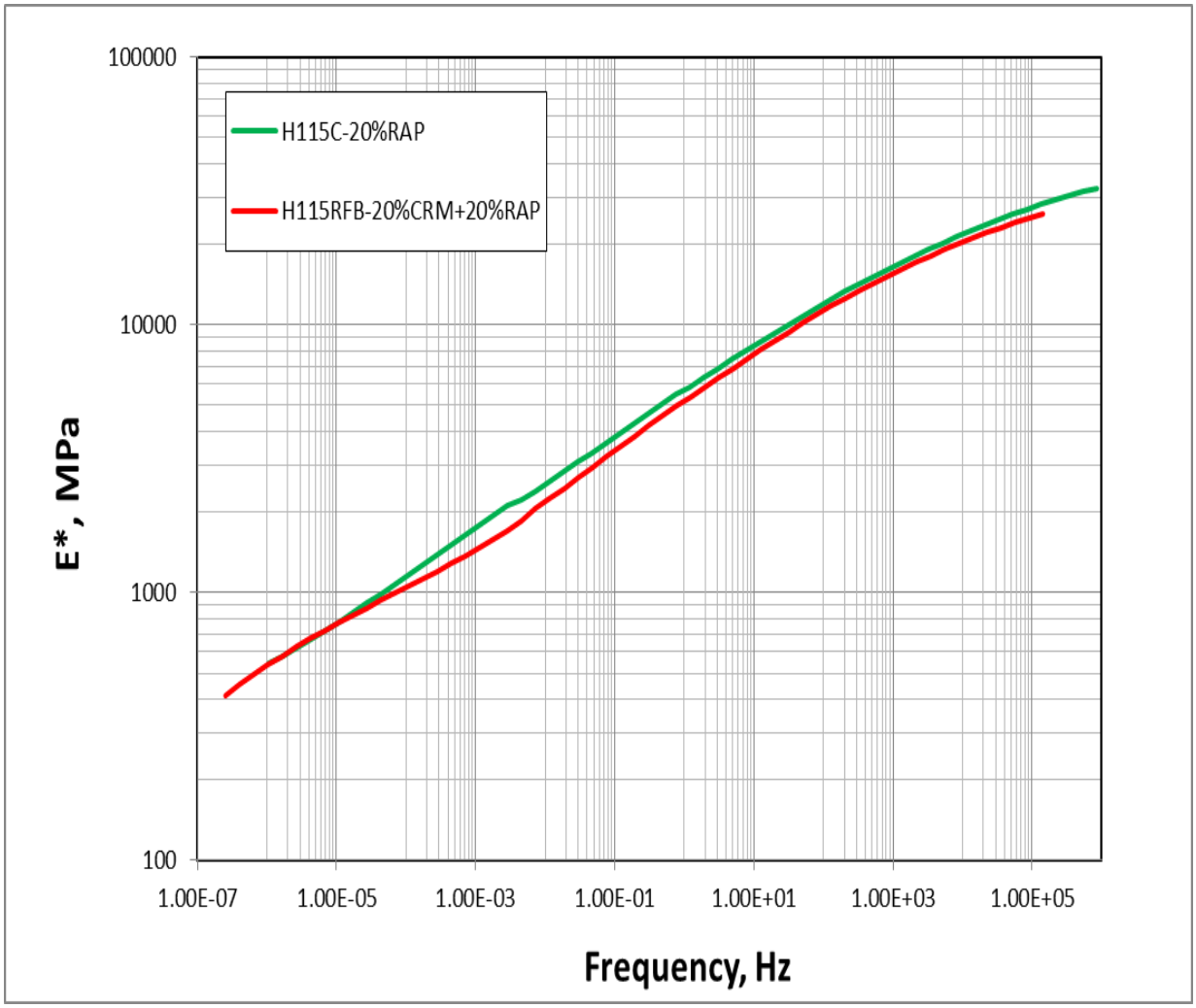

Figure 5-18: Master Curve for PG 64-34(P) mix with CRM+RAP, and RAP mix with PG 64-34

5.5.1 Evaluation of Rutting and Fatigue Factors from E* Tests

A final level of assessment of the HMA mixtures in this study involved estimating the rutting factor (E*/sin δ) and fatigue factor (E*sin δ) from the dynamic modulus (E*) test results at specific temperatures and frequencies. The E*/sin δ was computed at a loading frequency of 5Hz and test temperature of 54ºC while E*sin δ was done at 21ºC and 5Hz (Witczak et al., 2002). Note that the greater the rutting parameter (E*/sin δ), the less susceptible the mix is to the effects of rutting while a lower fatigue factor (E*sin δ) is good indicator of the HMA mixtures performance against fatigue cracking.

The findings for E*/sin δ and E*sin δ shown in Figure 5-19 and Figure 5-20 provide a good perspective of the observations in the master curves constructed. The results are a good indicator of rutting susceptibility for comparisons between the Rubberized-RAP HMA mixtures and RAP only mixtures with the former having higher rutting parameters and lower fatigue parameters. Again the terminal blend wet-process mix is noted to have ranked best compared with field blend rubberized mixtures. These results confirm that combining RAP and CRM provided a good balance between binder stiffness, mix stiffness, and durability. The results for rutting potential also appear to be consistent with those determined from the Hamburg Wheel Tracking Device (HWTD).

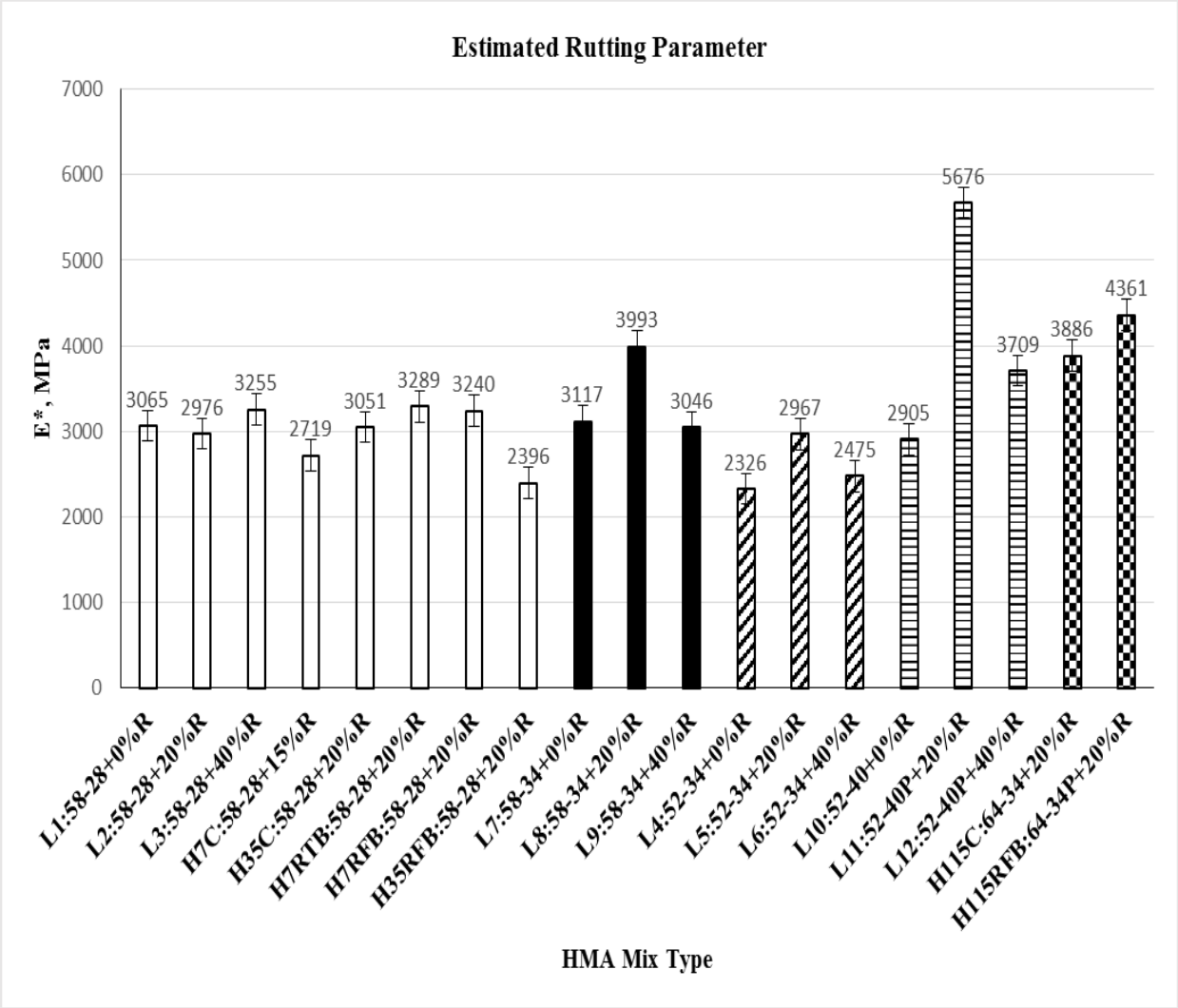

Figure 5-19: Rutting Parameter of Evaluated HMA Mixtures

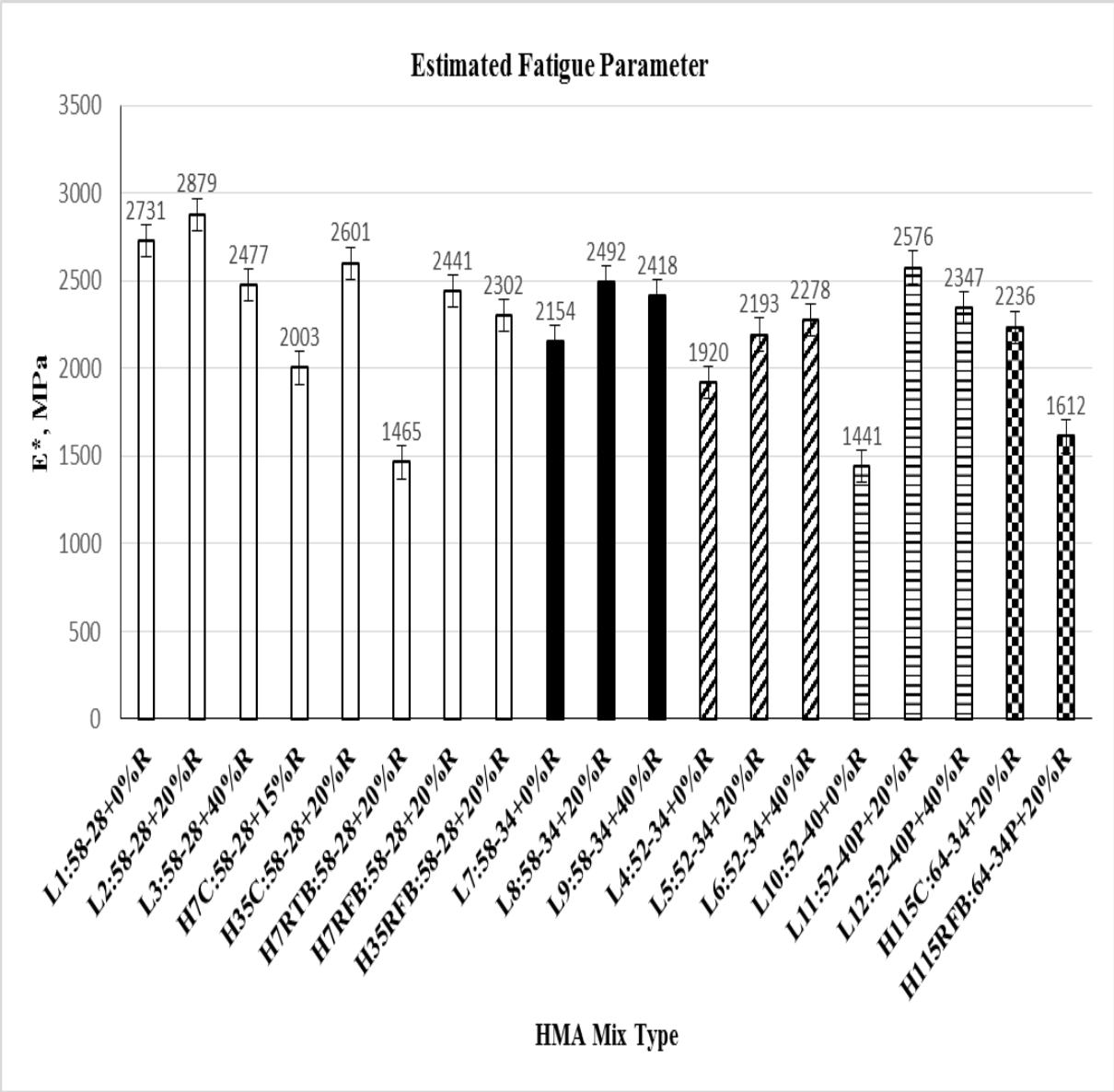

Figure 5-20: Fatigue Parameter of Evaluated HMA Mixture

5.6 Performance Ranking of the Evaluated HMA Mixtures

Based on the outcome of performance testing, statistical and master curve analysis of the evaluated HMA mixtures, rankings were completed in the order of their actual and estimated resistances to rutting, fatigue cracking and thermal cracking at low temperatures. The criteria and limits used in ranking the performance for each failure mode is discussed below.

The rankings for low temperature performance between HMA mixtures were considered on three levels. The first level was based on the ability to meet or exceed the one-day minimum Pavement Design Temperature (PDT) for the binder grade in the respective mix categories.

The HMA mixtures that satisfied this criteria were further considered to have good performance and placed in the third (3) rank if the difference between its Mean Fracture Temperature (MFT) and the corresponding one-day minimum PDT was equal to zero. HMA mixtures placed in the second (2) rank were those whose difference between MFT and one-day minimum PDT were less than the corresponding one-day minimum PDT by 1 to 5°C. Such HMA mixtures were considered to be better performing. The best performing HMA mixtures which were ranked first (1) were those whose difference between MFT and one-day minimum PDT were less than the corresponding one-day minimum PDT by 1 to 5°C.

In contrast, HMA mixtures were considered to have performed poorly and subsequently placed in the fourth (4) rank if the difference between MFT and one-day minimum PDT was greater than the corresponding one-day minimum PDT by more than 5°C. The second level of ranking considered the variation between MFT in the respective category of HMA mixtures. The ranks were placed as good (3), better (2) and best (1) depending on the degree of variation between temperatures.

The third and overall level of ranking was based on the sum of rank levels 1 and 2. The HMA mixture with the least sum of ranks was considered to be the best performing. The intermediate sum of ranks was considered better performing while the highest sum of ranks was considered to be good in performance. Table 5-12 summarizes the low temperature performance of the evaluated HMA mixtures as ranked.

In terms of actual performance, the ranking system for the laboratory-prepared PG 58-28 HMA mix category suggests that utilizing up to 40% RAP content compares favourably with the conventional mix, but the overall ranking suggest that the conventional mix is the best.

For the HMA mixtures with PG 58-34, 52-34 and 52-40 binders, the ranking system suggests that utilizing 40% RAP is not favourable to low temperature performance. These category of mixtures were found to be good in the overall rank. In the PG 58-28 plant-produced mix category, the rubber terminal-blend mix consistently ranked best in terms of performance and overall rank while the rubber field-blend was placed in better rank. For comparisons made between the 20% RAP and Rubberized-RAP HMA mixtures in the PG 64-34(P) category, the latter was ranked best.

Table 5-12: Low Temperature Performance Ranking of Evaluated HMA Mixtures

HMA Mix Type	1 Day Minimum PDT (°C)	Low Temperature Cracking					Sum of Ranks	Overall Rank
		MFT (°C)	MFT ≥ PDT	(MFT - PDT)	Rank Level 1	Rank Level 2 (Variation MFT)		
L1:58-28	-28	-30	Yes	-2	2	1	3	**1**
L2:58-28+20%RAP		-28		0	3	3	6	**3**
L3:58-28+40%RAP		-29		-1	2	2	4	**2**
H7C:58-28+15%RAP		-33		-5	2	1	3	**1**
H35C:58-28+20%RAP		-30		-2	2	3	5	**3**
H35RFB:58-28+20%RAP		-32		-4	2	2	4	**2**
H7RTB:58-28+20%RAP		-40		-12	1	1	2	**1**
H7RFB:58-28+20%RAP		-32		-4	2	2	4	**2**
L7:58-34	-34	-31	No	3	4	2	6	**2**
L8:58-34+20%RAP		-34	Yes	0	3	1	4	**1**
L9:58-34+40%RAP		-29	No	5	4	3	7	**3**
L4:52-34		-34	Yes	0	3	1	4	**1**
L5:52-34+20%RAP		-30	No	4	4	2	6	**2**
L6:52-34+40%RAP		-27	No	7	4	3	7	**3**
L10:52-40		-43	Yes	-9	1	1	2	**1**
L11:52-40P+20%RAP		-32	No	2	4	2	6	**2**
L12:52-40P+40%RAP		-32	No	2	4	2	6	**3**
H115C:64-34+20%RAP		-32	No	2	4	2	6	**2**
H115RFB:64-34P+20%RAP		-37	Yes	-3	2	1	3	**1**

***Note:** Performance - 1 = Best, 2 = Better, 3 = Good, 4 = Poor;
MFT = Mean Fracture Temperature;
PDT = Average 1 Day Minimum Pavement Design Temperature

In the case of rutting resistance, the overall ranks were based on the sum of ranks between the mean rut impression and mean creep slope as determined from the Hamburg Wheel Tracking Device (HWTD), and the estimated rutting factor (E*/sin δ) from dynamic modulus mix characterizations computed at a loading frequency of 5Hz and test temperature of 54ºC. The HMA mixture with the least sum of ranks was considered to be the best performing (1). The intermediate sum of ranks was considered better performing (2) while the highest sum of ranks was considered to be good in performance (3).

It should be noted that the mean rut impression is the difference between the maximum rut depth at 10,000 cycles and post-compaction consolidation depth at 1,000 cycles. Ranks for mean rut impression

and mean creep slope were assigned in descending order. This allowed for HMA mixtures with the highest values of mean rut impression, and mean creep slope to be ranked good (3), better (2) and best (1) respectively. In contrast, ranks for estimated rutting factor (E*/sin δ) were assigned in ascending order. This allowed for HMA mixtures with the highest values of stiffness to be ranked best (1), better (2) and good (3) respectively. Table 5-13 summarizes the rank in rutting performance of the evaluated HMA mixtures. The overall ranks shows that increasing the RAP content or combining RAP and CRM is favourable to the rutting performance of HMA mixtures.

Table 5-13: Rutting Performance Ranking of Evaluated HMA Mixtures

HMA Mix Type	Rutting							
	Hamburg Wheel Tracking Device (HWTD)				Estimated Rutting Factor		Sum of Ranks	Overall Rank
	Mean Rut Impression (mm)	Rank	Mean Creep Slope	Rank	Witczack's E/sin δ (MPa)	Rank		
L1:58-28	1.1	1	6.8E-05	2	3065	2	5	**2**
L2:58-28+20%RAP	1.5	3	1.1E-04	3	2976	3	9	**3**
L3:58-28+40%RAP	1.1	1	2.9E-05	1	3255	1	3	**1**
H7C:58-28+15%RAP	1.0	2	6.9E-05	3	2719	2	7	**3**
H35C:58-28+20%RAP	1.3	3	3.4E-05	2	3051	1	6	**2**
H35RFB:58-28+20%RAP	0.6	1	7.0E-06	1	2396	3	5	**1**
H7RTB:58-28+20%RAP	1.0	2	3.8E-06	1	3289	1	4	**1**
H7RFB:58-28+20%RAP	0.6	1	2.3E-05	2	3240	2	5	**2**
L7:58-34	1.8	1	1.0E-04	2	3117	2	5	**1**
L8:58-34+20%RAP	3.0	3	1.3E-04	3	3993	1	7	**3**
L9:58-34+40%RAP	1.8	1	8.2E-05	1	3046	3	5	**1**
L4:52-34	2.8	2	1.8E-04	2	2326	3	7	**2**
L5:52-34+20%RAP	3.1	3	2.4E-04	3	2967	1	7	**2**
L6:52-34+40%RAP	1.5	1	6.0E-05	1	2475	2	4	**1**
L10:52-40	1.5	3	6.8E-05	2	2905	3	8	**3**
L11:52-40P+20%RAP	1.3	2	7.2E-05	3	5676	1	6	**2**
L12:52-40P+40%RAP	1.1	1	6.3E-05	1	3709	2	4	**1**
H115C:64-34+20%RAP	0.6	1	3.1E-05	2	3886	2	5	**2**
H115RFB:64-34P+20%RAP	0.8	2	1.7E-05	1	4361	1	4	**1**

***Note:** Performance - 1 = Best, 2 = Better, 3 = Good

The overall ranking criteria for fatigue cracking between the evaluated HMA mixtures were based on the sum of ranks between performance from binder rheological characterization and the estimated fatigue factor (E*sin δ) from dynamic modulus mix characterizations computed at a loading frequency of 5Hz and test temperature of 21ºC.

The HMA mixture with the least sum of ranks was considered to be the best performing. The intermediate sum of ranks was considered better performing while the highest sum of ranks was considered to be good in performance. The ranks assigned to performance of the rheological characterized binders were based on the satisfaction of equation 5-1 below:

$$IT \leq T \qquad \text{(Equation 5-1)}$$

Where:

IT = Intermediate temperature of the recovered binder (ºC);

T = Temperature for $G^*\sin\delta \leq 5000$ kPa

The HMA mixtures that satisfied the criteria above were further ranked in descending order, and considered to have good (3), better (2) and best (3) performance depending on the degree of variation between the intermediate temperatures of the recovered binder.

The HMA mixtures that did not satisfy equation 5-1 were considered to have poor performance and placed in the fourth (4) rank. Ranks for estimated rutting factor (E*sin δ) were assigned in descending order. This allowed for HMA mixtures with the lowest values of stiffness to be ranked best (1), better (2) and good (3) performance respectively. Table 5-14 summarizes the rank in rutting performance of the evaluated HMA mixtures.

Table 5-14: Fatigue Cracking Performance Ranking between Evaluated HMA Mixtures

HMA Mix Type	Fatigue Cracking						Sum of Ranks	Overall Rank
	Binder Rheological Characterization				Estimated Fatigue Factor			
	Temperature, T for G*sin δ ≤ 5000 kPpa (°C)	Recovered Intermediate Temperature, IT (°C)	IT ≤ T	Rank	Witczack's E*sin δ (MPa)	Rank		
L1:58-28	19	19	Yes	3	2731	2	5	**3**
L2:58-28+20%RAP		11		1	2879	3	4	**2**
L3:58-28+40%RAP		16		2	2477	1	3	**1**
H7C:58-28+15%RAP		16		2	2003	1	3	**1**
H35C:58-28+20%RAP		19		3	2601	3	6	**3**
H35RFB:58-28+20%RAP		14		1	2302	2	3	**1**
H7RTB:58-28+20%RAP		11		1	1465	1	2	**1**
H7RFB:58-28+20%RAP		14		2	2441	2	4	**2**
L7:58-34	16	16	Yes	3	2154	1	4	**1**
L8:58-34+20%RAP		14		2	2492	3	5	**3**
L9:58-34+40%RAP		14		2	2418	2	4	**1**
L4:52-34	13	13	Yes	1	1920	1	2	**1**
L5:52-34+20%RAP		14	No	4	2193	2	6	**2**
L6:52-34+40%RAP		14		4	2278	3	7	**3**
L10:52-40	10	10	Yes	1	1441	1	2	**1**
L11:52-40P+20%RAP		14	No	4	2576	3	7	**3**
L12:52-40P+40%RAP		14		4	2347	2	6	**2**
H115C:64-34+20%RAP	19	13	Yes	2	2236	2	4	**2**
H115RFB:64-34P+20%RAP		9		1	1612	1	2	**1**

***Note: Performance - 1 = Best, 2 = Better, 3 = Good**

5.7 Field Monitoring of In-service Pavement Sections

It has been approximately four years since all of these trial sections were placed. Field monitoring results to date are consistent with the laboratory performance evaluations. The field monitoring exercise was conducted by CPATT on Highways 7, 35 and 115 pilot sections in June 2013 in accordance with the Ministry of Transportation Ontario (MTO) pavement distress evaluation considerations. Table 5-15 summarizes performance findings from the visual evaluations conducted. The Rubberized-RAP (Terminal and Field-blend) sections are all performing very well in comparison to their respective control sections incorporating 15%, and 20% RAP. It should be noted that there are no control sections without RAP for this demonstration project.

Table 5-15: Distress Manifestation on Pavement Sections

Highway Section	Distress Type	Severity	Density	Remarks
H7-C:15% RAP PG 58-28	Transverse cracks Longitudinal cracks	V V	F F	Physical appearance of pavement section is good, but with minor cracks and slight distortions. Excellent ride-ability.
H35-C:20% RAP PG 58-28	Centre line cracks Transverse cracks (1.5-2m apart) Longitudinal cracks Edge cracking Aggregate Loss	H H L V V	E Fq. F F F	Overall pavement condition and ride-ability is good, but with slightly rough and uneven sections.
H115-C:20% RAP PG 64-34	Centre line cracks Transvers cracks	V L	F Fq.	Overall pavement condition is good and ride-ability is very good.
H7R.TB:20% RAP PG 76-28	Centre line cracks at start of section. Transverse cracks	V V	F F	Pavement section is in excellent working condition. Ride-ability is smooth and noiseless.
H7R.FB:20% RAP PG 58-28	No visible distress on pavement	-	-	Overall working condition is excellent. Ride-ability is smooth and noiseless.
H35R.FB:20% RAP PG 58-28	Centre line cracks Transverse cracks Flushing	H M V	E Fq. F	Pavement section is visually gap-graded, and in better condition compared to the H35 control section. Excellent ride-ability and noiseless.
H115R.FB:20% RAP PG 64-34P	Centre line cracks Transverse cracks	V V	F F	Pavement section is in excellent working condition. Excellent ride-ability and noiseless.

***Note:** Distress Severity are noted as L = Low; V = Very Low; M = Moderate (M) and H High (H)
Distress Density are noted as F = Few; Fq. = Frequent; and E = Extensive

5.8 Forensic Testing on Cored Pavement Sections

Forensic pavement analysis provides valuable information into the potential cause of pavement failures. Prior to extracting the cores from Highway 15, North of Smith Falls, Ontario, all trial sections from field investigations were noted to have very slight to few raveling and coarse aggregate loss. There were also reported cases of few multiple centre line cracks, and slight multi-longitudinal cracks along the wheel tracks. However, the overall condition of the control and moist cryogenic sections appeared to be in good condition, and were performing well compared to the moist ambient rubber sections.

The following sections detail test findings aimed at understanding reasons for the observed distresses in the rubber modified sections.

5.8.1 Density Tests

Density analysis performed as part of this research revealed average values for the maximum specific gravity, air void content (In-place Density) and volume of water absorbed by the cores for control sections as 2.519, 6.4% and 0.4% respectively. While ambient rubber sections were 2.517, 6.9% and 0.4%; and cryogenic sections were 2.518, 5.0 and 0.4% respectively.

Water absorption rates were found to be comparable, but maximum specific gravity of the rubberized sections were observed to be lower than the control section. However the differences are not statistically significant. It is thus expected that the asphalt contents between these cored samples may be within similar range.

It is also recognized that a high in-place density can potentially result in water and air permeability which results in water damage, oxidation, raveling, and cracking while low in-place densities can potentially result in permanent deformation in the form of rutting and shoving in flexible pavements. The measured in-place densities are no more than 8% or less than the 4% required pavement densities. In this case, the observed pavement distresses cannot possibly be attributed to insufficient compaction. However, it is possible that the densities in and around the longitudinal construction joints may have been poorly constructed. This is taking into account the observed centre line cracks. Longitudinal joints are typically constructed one lane at a time. This results in low density areas and consequently increases the potential for voids to interconnect including the likelihood for permeability and possible pavement settlement.

5.8.2 Binder Extraction and Gradation Analysis

On completion of density testing, the cores were dried to a constant mass under room temperature. The asphalt cement content and aggregate gradation from selected cores for each trial section were determined in accordance with in accordance with the MTO Laboratory Testing Manual, Test Method LS-282 Rev. No. 25 (MTO, 2001). Trichloroethylene solvent was used to extract the asphalt binder from the in-service pavement cores to determine if insufficient binder content could be attributed to the poor performance of these sections. Gradation analysis was conducted to help identify whether the cause of poor performance for the sections could be related to mix gradation. The determined asphalt content and aggregate gradation for each trial section is shown in Table 5-16.

Table 5-16: Gradation Analysis and Asphalt Content of Trial Sections

Sieve Size (mm)	% Passing Sieve (mm)		
	Control Section	**Ambient Rubber Section**	**Cryogenic Rubber Section**
16.0	100.00	100.00	100.00
12.5	88.89	95.62	98.56
9.5	75.56	82.50	89.12
4.75	53.78	55.95	52.25
2.36	45.88	48.08	45.68
1.18	41.39	34.22	34.83
0.600	36.42	24.97	26.70
0.300	16.41	12.69	12.93
0.150	7.92	5.24	5.89
0.075	4.90	1.62	2.15
Pan	1.29	0.00	0.19
% Asphalt Content	6.0	5.4	6.1

The slope and the intercept constants of the coarse and fine aggregate portions of the recovered and graded aggregates were analyzed using the power's regression law. This law is expressed as shown below (Ruth et al., 2002):

$$P_{CA} = a_{CA}(d)^{n_{CA}} \qquad \text{(Equation 5.2)}$$

$$P_{FA} = a_{FA}(d)^{n_{FA}} \qquad \text{(Equation 5.3)}$$

Where:

P_{CA} and P_{FA} = Percent by weight passing a given sieve that has an opening of width d

a_{CA} = Intercept constant for the coarse aggregate

a_{FA} = Intercept constant for the fine aggregate

d = Sieve opening width, mm

n_{CA} = Slope (exponent) for the coarse aggregate

n_{FA} = Slope (exponent) for the fine aggregates

The divider sieve between coarse and fine aggregate is given by the Nominal Maximum Aggregate Size (NMAS) of the mixture. All trial sections were found to have a NMAS of 12.5 mm, so the recommended 2.36 mm dividing sieve was used for analysis. Based on Power's law, the higher the slope value, the coarser or finer that portion. Table 5-17 shows the application of the procedure for the respective trial sections.

Table 5-17: Gradation Analysis Based on Power's Law

Trial Sections	aCA	nCA	aFA	nFA
Control	31.1	0.4	34.5	0.7
Ambient Rubber	31.7	0.5	29.6	1.0
Cryogenic Rubber	31.4	0.4	29.4	0.9

The fine aggregate portion of both cryogenic and ambient rubber sections are finer, but appear segregated based on the observations in Table 5-16 and Table 5-17. This suggests a pavement with sufficient fines that are unevenly distributed to form a good mastic with the binder thus making it more permeable.

The control sections appear to have an even distribution of fines which tends to reduce permeability. The coarse aggregate portion of both cryogenic and ambient rubber sections also appear to be coarser, but is segregated compared to the control section. Ambient sections appear to have a lower asphalt content compared to cryogenic and control sections.

It is also possible that the dense-graded mix design of the moist process ambient and cryogenic mix design may have contributed to field compactability issues. Perhaps a gap-graded mix design is better suited to the moist process rubber mix which is essentially similar to the dry process rubber mix. This is because a gap-gradation is confirmed to be favorable to the compaction of dry-process rubberized asphalt mixtures. The result of good compaction is a reduction in voids and improved durability of the resulting pavement.

5.8.3 Rutting, Moisture Damage and Stripping Susceptibility

The extracted cores were also characterized for their resistance to rutting, moisture damage and the effects of stripping. Measured rut depths obtained from the control and ambient rubber trial sections cores were found to exceed MTO's minimum 6 mm criterion for a slightly rut flexible pavement; while the cryogenic section is at least 2 mm below the minimum rut criterion.

Figure 5-21 highlights these findings measured at 1,000 and 10,000 load cycles including the mean rut impressions calculated as the difference between calculated averages of maximum rut depth and post-compaction consolidation.

To confirm the measured rut depths, mean creep slopes between the trial sections were compared as shown in Figure 5-22. Higher creep slopes are observed to be consistent with the control, and ambient rubber trial sections, both of which exhibited higher rut depths and more pronounced wheel impressions.

Stripping Inflection Point (SIP) could not be determined for any of the trial sections at any load cycle less than 10,000. This is partly related to the low water absorption behaviour and in-place pavement densities. The main deduction is that a strong relationship exist between pavement density, aggregate gradation and permeability.

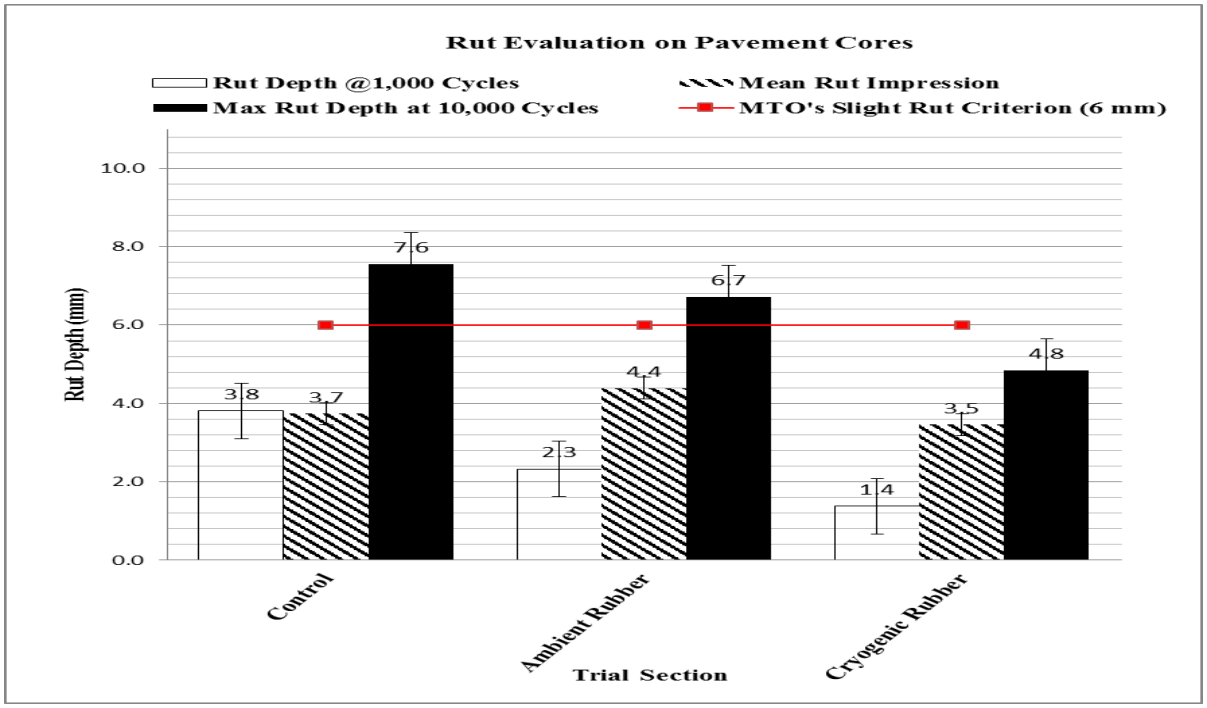

Figure 5-21: Rut Depths between Trial Sections

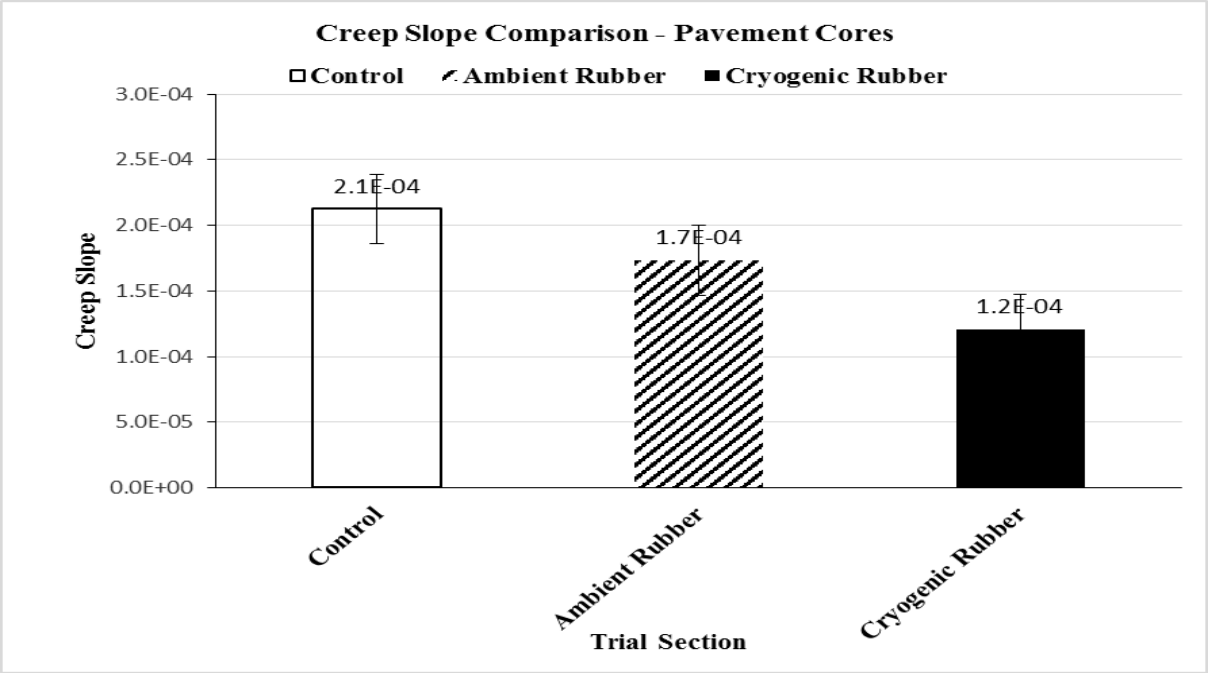

Figure 5-22: Mean Creep Slopes between Trial Sections

Forensic findings confirm that all trial sections may have been well compacted. However, the observed distress is most likely related to aggregate segregation resulting from the effects of permeability caused by poorly constructed or compacted longitudinal joints. The sections were permeable regardless of the high asphalt content and fine aggregate gradations. Regardless of the low asphalt content in the ambient rubber sections, results from rut characterization makes a statement for the continued use of crumb rubber in Hot Mix Asphalt (HMA). A properly designed, mixed and well compacted rubberized pavement will compare favorably or perform better than conventional HMA pavements.

5.9 Summary

In this Chapter, the laboratory performance testing results for all evaluated HMA samples and extracted pavement cores were presented, analyzed and discussed in accordance with the thesis methodology and research objectives. The results and observations from field monitoring of in-service pavement sections were also reported. The experimental data obtained from performance characterization tests were observed, and statistically analyzed to investigate the effects or impact of varying RAP contents, or combining RAP and CRM for the evaluated HMA mixtures. Overall, it is concluded that an engineered HMA mix with up to 40% RAP content, or a mix incorporating 20% RAP and 10 or 20% CRM will satisfy all its functional pavement performance requirements if it is properly designed, mixed and compacted. A properly designed HMA mix demands that an acceptable criteria for both volumetric and mechanical properties is reached. Finding this balance is possible with a solid understanding of the material properties of the components of the HMA mix and how interaction occurs between them. The RAP mixtures in this study have been designed keeping all Superpave volumetric properties within minimum specifications while the crumb rubber used in the field-blend Rubberized-RAP HMA mixtures were subjected to both ambient and cryogenic grinding resulting in finer crumb particles. This ensured good solubility with the virgin binder prior to mixing with the aggregates.

Chapter 6
Sustainability and Cost Assessment

6.1 Introduction to Sustainable Pavements

The general consensus on sustainability revolves around its relationship and benefits to the economy, environment and society. The consideration of these factors within the context of a sustainable pavement requires that a pavement is durable, cost effective, eco-efficient, and is of better or comparable performance to one built with virgin materials. The main indicators for a sustainable pavement should therefore include the following (Uzarowski & Moore, 2008):

- Minimizing the use of natural resources;
- Reducing energy consumption;
- Reducing greenhouse gas (GHG) emissions;
- Limiting pollution (air, water, earth, noise, etc.);
- Improving health, safety and risk prevention; and
- Ensuring a high level of user comfort and safety.

Chapter Five of this thesis has shown that the use of RAP and CRM satisfies both structural and functional pavement performance requirements. In addition, the innovative use of Reclaimed Asphalt Pavement (RAP) and Crumb Rubber Modifier (CRM) in Hot Mix Asphalt can potentially provide the above-mentioned sustainable pavement benefits. Therefore, this Chapter assesses and compares the cost and sustainability of the various pavement design alternatives to determine the optimal sustainable option.

A number of rating tools have been developed to assess the sustainability level of various pavement design alternatives. Some of which include: Leadership in Energy and Environmental Design (LEED), GreenLITES, GreenGuide, INVEST, GreenRoads, GreenPave, Sustainable Highways Self-Evaluation Tool, and Envision (Hertel, 2012). These rating tools differ in terms of applications and procedure, but the general underlying principle is point based on self-assessment. For this research, the sustainability level of a control pavement section and those incorporating RAP and CRM are assessed using Ontario's pavement sustainability rating system "GreenPave". The cost assessment considers both the environmental and economic implications of the various pavement design

alternatives. For this research, the environmental impacts are assessed using the "Pavement life-Cycle Assessment Software for Environmental and economic Effects (PaLATE)" while the economic assessment utilizes the "Life Cycle Cost Assessment (LCCA)" methodology. A brief description of each tool, and results of the completed analysis for different research case studies are presented in subsequent sections of this Chapter.

6.2 GreenPave Sustainability Rating Tool

GreenPave is a simple point based, self-evaluating rating tool to enhance the sustainability of Ontario's transportation infrastructure through designing and selecting the most economical and environmentally-friendly pavement alternative. It is modelled after the GreenLITES sustainability rating system, but focuses specifically on pavement projects rather than the entire road (Lane, 2011). There are four categories within which GreenPave assesses pavement sustainability; these alongside corresponding goals and point totals are detailed in Table 6-1.

Table 6-1: GreenPave Category Overview (Lane, 2011)

Category	Goal	Points
Pavement Design Technologies	To optimize sustainable designs. These include long life pavements, permeable pavements, noise mitigating pavements, and pavements that minimize the heat island effect.	9
Materials & Resources	To optimize the usage/reuse of recycled materials and to minimize material transportation distances.	11
Energy & Atmosphere	To minimize energy consumption and GHG emissions.	8
Innovation & Design Process	To recognize innovation and exemplary efforts made to foster sustainable pavement designs.	4
Maximum Total:		**32**

Each category is further broken down to address specific objectives, with corresponding points assigned to each subcategory. The degree to which credit objectives are met determines how projects are awarded points for each subcategory under the GreenPave rating tool. An overview of the GreenPave scorecard for each subcategory and the points associated with each criterion is highlighted in Figure 6-1. The following four awards are established in GreenPave: Bronze (7-10 points), Silver (11-14 points), Gold (15-19 points), and Trillium (20+ points) (Lane, 2011).

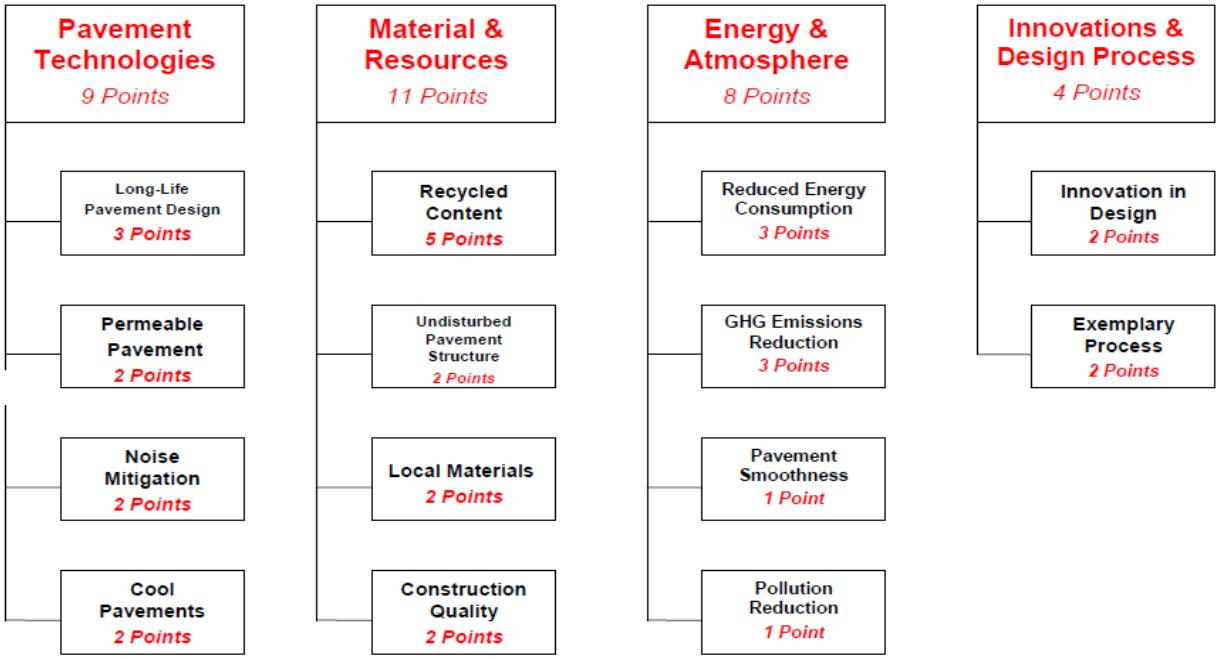

Figure 6-1: GreenPave Score Card Overview (Lane, 2011)

6.2.1 GreenPave Evaluation for Design Case Studies

To illustrate the applicability and impacts of utilizing RAP and CRM in HMA mixtures based on Ontario's pavement sustainability rating system "GreenPave", an assumed pavement structural design input is adopted for the analysis. The structure for analysis consists of two layers of asphalt; the surface course, being 40 mm of a SP 12.5 mm FC1 mix over 110 mm thick SP 19.0 mm binder course. The underlying layers are 150 mm and 450 mm for the granular A and B, respectively. The recycled components of the binder and granular layers are assumed to be 30%. The analysis are completed in the following order:

- Case A – Rubberized-RAP: 20% RAP + 20% CRM in surface course
- Case B - 20% RAP in surface course
- Case C - 40% RAP in surface course
- Case D - Control Mix with no recycled components in the surface and binder course

It should also be noted that a computer spreadsheet is available for the GreenPave rating tool, but the analysis were manually computed based on the MTO's GreenPave reference manual shown in Table 6-2. Table 6-3 highlights the pavement design section for Case A - Rubberized-RAP.

Table 6-2: MTO's GreenPave Rating System Guide (GreenPave, 2012)

Sub-Category	Points Awarded	Requirements		
		Asphalt	Concrete	Granular Layer
Long Life Pavement	2	Composite pavement, perpetual asphalt pavement, deep strength asphalt pavement	-	-
	3	-	Rigid pavement	-
Permeable Pavement	1	Roadside drainage		-
	2	Parking lot		-
Noise Mitigation	1	Superpave mixes	Longitudinal tining, Diamond grinding	-
	2	SMA mixes, HMA with rubber modified AC, Quiet Pavement Mixes	-	-
Cool Pavement	1	Quiet pavement, porous asphalt	-	-
	2	-	Concrete pavement, white cement pavement, permeable pavers, pervious concrete	-
Recycled Content	1	5-15% RAP by mass	10-15% of SCM by mass of cement	10-29% RM by mass
	2	16-20% RAP by mass	16-25% of SCM by mass of cement	-
	3	21-30% RAP by mass	-	30-49% RM by mass
	4	31-40% RAP by mass	-	-
	5	CIR, CIREAM, HIR	-	>50% RM by mass, In-place processing
	Extra Point	>1% of CR or RST by mass	Use of slurry water or treated wash water	-
Reuse of Pavement	1	Preservation treatments		-
	2	Maintainingg >80% of pavement structure, Concrete overlay		-
Local Materials	1	50-79% materials transported less than 100 km		
	2	>80% materials transported less than 100 km		
Construction Quality	1	Meets Requirements		
	2	Exceeds Requirements		
Reduce Energy Consumption	1	WMA, 5-15% RAP, >2% RST by mass	16-25% SCM by mass of cement	10-49% RM by mass
	2	HIR, 16-40% RAP by mass	-	>50% RM by mass, In-place processing
	3	CIR, CIREAM	-	FDR, EAS
GHG Emissions Reduction	1	WMA, 5-15% RAP by mass	16-25% SCM by mass of cement	10-49% RM by mass
	2	HIR, 16-40% RAP by mass	-	>50% RM by mass, In-place processing
	3	CIR, CIREAM	-	FDR, EAS
Pavement Smoothness	1	IRI < 0.65	Concrete	-
Pollution Reduction	1	>50% vehicles with diesel retrofit	>25% vehicles with alternative fuels	-
Innovation in Design	1	1 innovation in design		
	2	> 2 innovations in design		
Exemplary Process	1	1 exemplary process		
	2	>2 exemplary processes		

Table 6-3: Rubberized-RAP Pavement Section

Process	Construction Type	Pavement Layers	Material	Content (%)	Layer Depth (mm)	Width (m)
HMA with 20% RAP and 20% CRM	Initial	Surface Course SP 12.5 mm	Virgin Aggregate	80	40	8
			RAP	20		
			Asphalt Cement	6.6		
		Binder Course SP 19 mm	Virgin Aggregate	80	110	
			RAP	20		
			Asphalt Cement	5		
		Granular A	RAP to Site	30	150	
			Gravel to Site	70		
		Granular B	RAP to site	30	450	
			Rock to Site	70		

Based on recommendations in Table 6-2, no point is earned for Long Life Pavement or Permeable Pavement subcategories. The surface course is a SP mix containing rubber, so 2 points is earned under the Noise Mitigation subcategory. However, the pavement does not meet the requirements under the Cool Pavement subcategory therefore no points are awarded. Points in the Recycled Content subcategory are awarded based on weighted averages since the different pavement layers will earn varying amounts of points. Table 6-4 highlights the weighted average calculation; therefore 2.8 points are awarded under this subcategory.

Table 6-4: Recycled Content Calculation

Pavement Layers	Point Awarded	Thickness	Thickness x Points Awarded
Surface Course SP 12.5 mm	2	40	80
Binder Course SP 19 mm	2	110	220
Granular A	3	150	450
Granular B	3	450	1350
Total		750	2100
Points Earned		2100/750 = 2.8	

No points are awarded under the Reuse of Pavement subcategory since an initial construction project is being analyzed. It is assumed that all utilized materials have transportation distances less than 100km, so 2 points are awarded under the Local Materials subcategory. One point is awarded for Construction Quality subcategory since the finished pavement met requirements. A weighted average system similar to the method illustrated in Table 6-4 is also is required to award points in the Reduce Energy and Green House Gas (GHG) Emissions Reduction subcategories. These subcategories are awarded 1.2 points each. 1 points is awarded under the Pollution Reduction subcategory since use of scrap tires contributes to reducing pollution. Two points are awarded for Innovation in Design since the mix design incorporates RAP and CRM while no point is awarded to the Exemplary Process subcategory. A completed GreenPave scorecard for all design case studies are presented in Table 6-5. This simple sustainability analysis returned a score card of 13.2 points for Case A - Rubberized-RAP, so it is awarded a GreenPave "Silver" certification. Based on points earned and GreenPave rankings, Case B - 20% RAP and Case C – 40% RAP are awarded the "Bronze" certification respectively while Case D – Control Mix is certified unsustainable.

Table 6-5: GreenPave Score Card for all Pavement Sections

Category	Sub-Category	Max Points	Awarded Points			
			Set A	Set B	Set C	Set D
Pavement Technologies	Long Life pavement	3	0	0	0	0
	Permeable Pavement	2	0	0	0	0
	Noise Mitigation	2	2	1	1	1
	Cool Pavement	2	0	0	0	0
Materials and Resources	Recycled Content	5	2.8	2.8	2.9	2.5
	Reuse of Pavement	2	0	0	0	0
	Local Materials	2	2	2	2	0
	Construction Quality	2	1	1	1	1
Energy and Atmosphere	Reduce Energy Consumption	3	1.2	1.2	1.2	0
	GHG Emissions Reduction	3	1.2	1.2	1.2	0
	Pavement Smoothness	1	0	0	0	0
	Pollution reduction	1	1	1	1	0
Innovation and Design Processes	Innovation in Design	2	2	0	0	0
	Exemplary Process	2	0	0	0	0
Total Points		**32**	**13.2**	**10.2**	**10.3**	**4.5**

6.3 Life Cycle Cost Assessment (LCCA) Methodology

Life Cycle Cost Assessment (LCCA) is an analysis technique that builds on the well-founded principles of economic analysis to evaluate the over-all-long-term economic efficiency between competing alternative investment options (FHWA, 1998). The six major pavement life cycle cost components and level of influence are summarized in Table 6-6.

Table 6-6: Pavement Life Cycle Cost Components (OHMPA, 1998)

THE SIX MAJOR LIFE-CYCLE COST COMPONENTS		Influence on Life-Cycle Costs
Initial Costs	▪ design, build and construct ▪ cost of hot mix (standard mixes or enhanced pavement designs like stone mastic asphalt or modified/engineered asphalt)	moderate to high
Maintenance Costs	▪ routine maintenance such as crack sealing and patching to extend pavement service life	moderate
Rehabilitation Costs	▪ resurfacing and reconditioning to restore pavement to acceptable service levels	moderate
User Costs	▪ cost of delays due to construction and maintenance	low to moderate
Residual Value	▪ value of the remaining service life of the road (the economic analysis may cover 40 years compared to the road's expected life of 50 years).	low
Salvage Value	▪ value of reusable components at the end of the analysis period	low

A LCCA is particularly beneficial where project alternatives that satisfy similar performance requirements, but differ in terms of the relevant costs of the agency, owner, and the pavement users, are to be compared for purposes of selecting the project that maximizes net savings. The relevant costs include initial construction and project support, future maintenance and rehabilitation, and the user costs (delays and vehicle costs). The most appropriate time for conducting LCCA for pavement projects is during the design stage. This allows for necessary modifications that will ensure cost reductions, and in decision making. A summary of LCCA input variables and the general basis used to determine their values is shown in Table 6-7.

Table 6-7: LCCA Input Variables (FHWA, 1998)

LCCA Component	Input Variable	Source
Initial and Future Agency Costs	Preliminary Engineering	Estimate
	Construction Management	Estimate
	Construction	Estimate
	Maintenance	Assumption
Timing of Costs	Pavement Performance	Projection
User Costs	Current Traffic	Estimate
	Future Traffic	Projection
	Hourly Demand	Estimate
	Vehicle Distributions	Estimate
	Dollar Value of Delay Time	Assumption
	Work Zone Configuration	Assumption
	Work Zone Hours of Operation	Assumption
	Work Zone Duration	Assumption
	Work Zone Activity Years	Projection
	Crash Rates	Estimate
	Crash Cost Rates	Assumption
NPV	Discount Rate	Assumption

A LCCA is conducted in accordance with the following procedures (Hicks & Epps, 1999), (FHWA, 1998):

- Assume an analysis period and develop rehabilitation and maintenance strategies for the analysis period.
- Establish the timing (or expected life) of various rehabilitation and maintenance strategies.
- Estimate the agency costs for construction, rehabilitation, and maintenance.
- Estimate user and non-user costs and develop expenditure streams.
- Compute the present project value or worth.
- Analyze the results using either a deterministic or probabilistic approach.
- Re-evaluate strategies and develop new ones as needed.

The present value of the investment is evaluated by combining the initial project costs and the discounted future expenditures. However, for purposes of guaranteeing reasonable comparisons between options, it is important for LCCA to be evaluated based on the Net Present Value or Worth (NPV). The NPV is an economic efficiency indictor computed as shown in Equation 6.1. It takes into consideration the user costs (i.e. delay and vehicle operating costs incurred by the user of the facility) and agency costs (i.e. all costs incurred directly by the agency over the life of the project). A probabilistic LCCA approach is also recommended (Tighe, 2001).

$$NPV = Initial\ Cost + F\left(\frac{1}{(1+i)^n}\right)$$

(Equation 6-1)

Where; F – Future cost at the end of the nth year

i – Discount rate, typically 3 to 5%

n – Number of years

An Equivalent Uniform Annual Cost (EUAC) is computed using Equation 6-3 after the NPV has been determined. The EUAC represents the NPV of all the discounted costs and benefits of an alternative as if they were to occur uniformly throughout the analysis period.

$$EUAC = NPV\left(\frac{1(1+i)^n}{(1+i)^n - 1}\right)$$

(Equation 6-3)

A salvage value which takes into consideration the cost of the final rehabilitation activity, expected life of rehabilitation, and time since last rehabilitation activity is also calculated as shown in Equation 6-4. This represents the value of an investment alternative at the end of the analysis period.

$$\mathrm{SV} = 1 - \left({}^{\mathrm{L_A}}/_{\mathrm{L_E}}\right) * \mathrm{C}$$

(Equation 6-4)

Where; C – Cost of rehabilitation strategy

L_A – Portion of expected life consumed

L_E – Expected life of the rehabilitation strategy

6.3.1 Research Design Considerations for LCCA

The LCCA methodology described in section 6.3 and a framework for a 2011 CPATT study that evaluated the economic effectiveness of RAP and Recycled Asphalt Shingles (RAS) in HMA Mixtures (Ddamba, 2011) were adopted and modified to quantify and compare the economic values of the respective case studies in this research. The CPATT LCCA framework took into consideration a 2011 Ontario report on the life cycle cost analysis of municipal pavements in Southern and Eastern Ontario (ARA, 2011). In addition, material unit cost estimates reflecting current market pricing in the Greater Toronto Area (GTA) have been considered. Note that the price per tonne of each material fluctuates regularly, so all analysis done herein are simply for the purposes of this research.

The unit cost estimates for all HMA layers considered are for those incorporating a PG 58-28 asphalt binder. A tonne of the 20% and 40% RAP HMA mixtures with PG 58-28 binder were respectively assumed to be 10% and 20% less expensive than that of conventional HMA. This was considered in part because the cost for a PG 58-28 HMA mixture incorporating RAP was not readily available, but those for a PG 64-28 HMA mixture were found to be within the price range considered. However, in most cases, it was found that some municipalities in the province were paying the regular HMA price whilst allowing the contractors to elect if they want to incorporate RAP.

In the case of a Rubberized HMA mixture, no cost estimates were obtained since it is not typically in use in Ontario. The actual cost per tonne of a rubberized HMA mix used in the 2011 rubber demonstration project were also not readily available, so estimates were determined based on findings from a paper titled "*Life Cycle Costs for Asphalt-Rubber Paving Materials*" (Hicks & Epps, 1999).

The paper showed that the cost of using asphalt rubber mix could range between approximately 26 - 45% higher than the cost of a conventional HMA mix. This takes into consideration the incremental cost of acquiring, renting, or contracting the equipment used to blend the crumb rubber into the asphalt cement. In the case of the 2011 rubber demonstration project, the cost of contracting the blending unit ranged between $15 - $30 per tonne of asphalt rubber, not including the cost of mobilizing, demobilizing, and setting up/taking down the equipment (Hegazi, 2014).

Consequently, it was assumed that the unit cost for a tonne of Rubberized-RAP HMA mixture with 20% RAP and 20% crumb rubber should be 30% more expensive than a conventional HMA mixture (i.e. the difference between an assumed rubberized-HMA being 40% more expensive, and a 10% less expensive 20% RAP HMA mixture).

An analysis period of 20 years was assumed and the unit cost per tonne of each material utilized for initial pavement construction are highlighted in Table 6-8.

The expected pavement service lives shown in Table 6-9 for the initial pavement construction for the control and RAP case studies were obtained from the Transportation Association of Canada (TAC) Pavement Design and Management Guide (TAC, 2013) while the Rubberized-RAP case study was based on estimates from the Hicks paper (Hicks & Epps, 1999).

The maintenance and rehabilitation design strategies with expected life spans for the 20-year analysis period including unit costs for each activity are also shown in Table 6-10. Figure 6-2 and Figure 6-3 illustrates the time and performance impact respectively for maintaining and/or rehabilitating pavements. An increase in the life of the pavement is the result of a timely and appropriate maintenance or rehabilitation strategy. The laboratory performance assessments of the HMA mixtures for rutting, and thermal cracking as ranked in section 5.6 were the primary considerations in formulating the year of maintenance and rehabilitation strategies for the respective case studies considered. Note that an ideal approach would involve using the mechanistic-empirical pavement design guide (MEPDG) computer-based modeling software or actual field pavement distress data. The MEPDG allows for the prediction of distresses and determination of rehabilitation and maintenance programs based on the effects of the traffic, climate, existing pavement conditions and the underlying soil. It should be noted that all material quantities used in the LCCA for the hypothetical pavement geometry are typical estimates for a 1 kilometre roadway.

Table 6-8: Assumed Unit Cost for Initial Pavement Construction

Pavement Layer	Description of Pavement Layer, Amount (Quantity)	Unit Cost $/Tonne
Hot Mix Asphalt (HMA)	Superpave 12.5 mm FC1	106
	Superpave 12.5 mm FC1 (20% RAP)	96
	Superpave 12.5 mm FC1 (40% RAP)	85
	Superpave 12.5 mm FC1 (20% RAP + 20% CRM)	138
	Superpave 19 mm	90
Base	Granular A	16
Sub-base	Granular B – Type II	13

Table 6-9: Expected Pavement Service Lives for Initial Construction

Case Study	Life Span (Years)
Case A (Rubberized-RAP HMA - 20% RAP + 20% CRM)	18
Case B (20% RAP HMA)	15
Case C (40% RAP HMA)	
Case D (Control HMA)	

Table 6-10: Maintenance and Rehabilitation Strategies with Unit Costs

Years after Initial Construction	Service Life	Description of Pavement Layer, Amount (Quantity)	Quantity Per 1km of Road	Unit Cost $/ Tonne
3	5-7	Rout and Seal (200 m/km)	200	5
5	5-7	Rout and Seal (200 m/km)	200	5
8	8-10	Spot Repairs, mill 40 mm/patch 40 mm, 5% area (m^2)	750	35
12	8-10	Spot Repairs, mill 40 mm/patch 40 mm, 5% area (m^2)	750	35
16	8-10	Spot Repairs, mill 40 mm/patch 40 mm, 20% area (m^2)	3,000	45
20	8-10	Spot Repairs, mill 40 mm/patch 40 mm, 5% area (m^2)	750	35

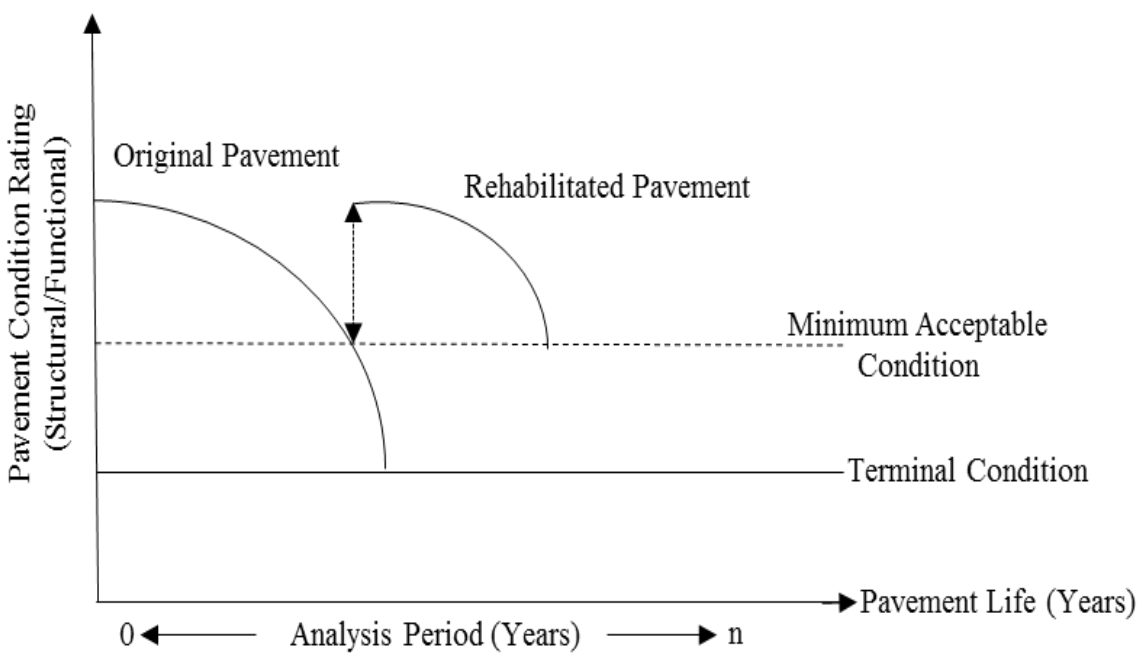

Figure 6-2: Time Impact of Pavement Maintenance and Rehabilitation Strategy

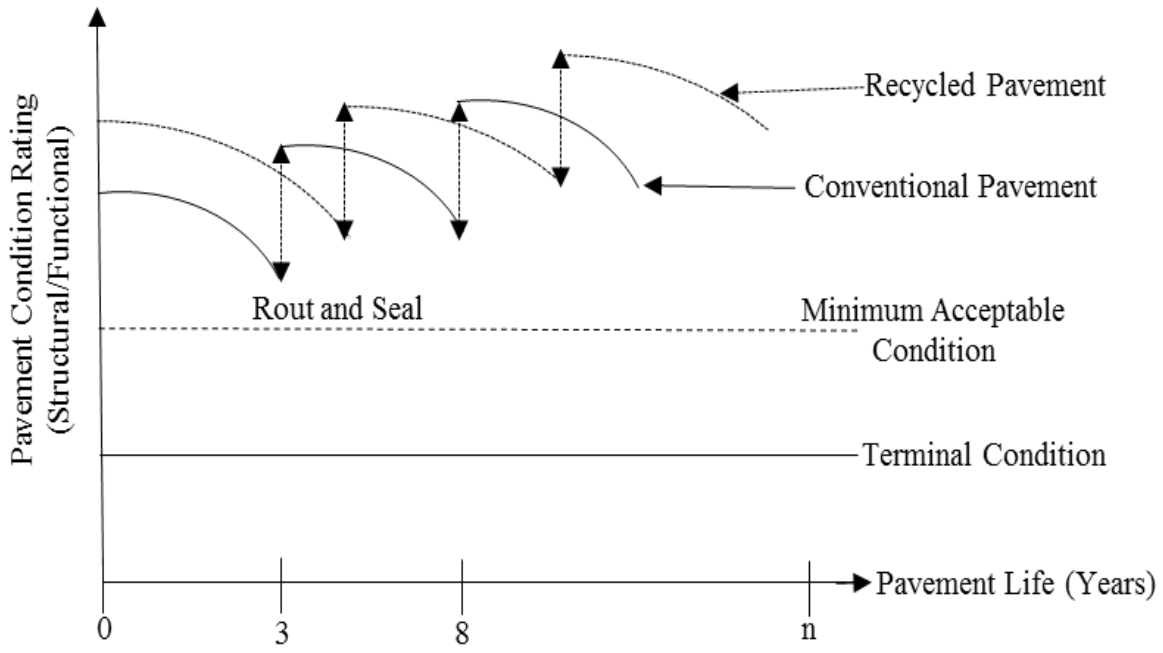

Figure 6-3: Pavement Performance Impact over Time

6.3.2 LCCA Computation and Analysis

The initial pavement construction cost and maintenance and rehabilitation schedules were calculated and annualized over the analysis period at 3, 4, 5 and 7% discount rates. The excavation costs were not included in the determination of initial pavement construction cost since it is not always necessary for a LCCA. The initial pavement construction cost determined at year zero for all case designs are shown in Table 6-11. It is observed that constructing a 1 km pavement section incorporating Case A (Rubberize-RAP HMA) is more expensive compared to a Case D (Control HMA) mix or mixtures with Case B (20% RAP HMA) and Case C (40% RAP HMA). However, the initial pavement construction cost for Case C and Case B are less expensive when compared to Case A.

The LCCA computations detailing the Present Worth Factor, Net Present Values (NPV) and Equivalent Uniform Annual Costs (EUAC) over the expenditure streams for the respective case studies are summarized in Appendix D. The rankings between case studies for total maintenance and rehabilitations costs, salvage values, and Present Worth Costs (PWC) at 3, 4, 5 and 7% discount rates are summarized in Table 6-12. Note that the typical discount rate used in Ontario is about 5.4% (ARA, 2011).

Table 6-11: Initial Pavement Construction Costs

Case D - Control HMA Mixture					
Pavement Layer	**Description of Pavement Layer**	**Amount**	**Quantity per Km**	**Unit Cost $/Tonne**	**Total Cost**
Surface	SP 12.5 mm FC1	40	1,512	$ 106.00	$ 160,272.00
Binder	SP 19, mm (t)	110	4,059	$ 90.00	$ 365,310.00
Base	Granular A, mm (t)	150	5,400	$ 16.00	$ 86,400.00
Subbase	Granular B - Type II, mm (t)	450	13,500	$ 13.00	$ 175,500.00
				Grand Initial Construction Total Cost	**$ 787,482.00**
Case B - 20% RAP HMA Mixture					
Pavement Layer	**Description of Pavement Layer**	**Amount**	**Quantity per Km**	**Unit Cost $/Tonne**	**Total Cost**
Surface	SP 12.5 mm FC1 (20% RAP)	40	1,512	$ 96.00	$ 145,152.00
Binder	SP 19, mm (t)	110	4,059	$ 90.00	$ 365,310.00
Base	Granular A, mm (t)	150	5,400	$ 16.00	$ 86,400.00
Subbase	Granular B - Type II, mm (t)	450	13,500	$ 13.00	$ 175,500.00
				Grand Initial Construction Total Cost	**$ 772,362.00**
Case C - 40% RAP HMA Mixture					
Pavement Layer	**Description of Pavement Layer**	**Amount**	**Quantity per Km**	**Unit Cost $/Tonne**	**Total Cost**
Surface	SP 12.5 mm FC1 (40% RAP)	40	1,512	$ 85.00	$ 128,520.00
Binder	SP 19, mm (t)	110	4,059	$ 90.00	$ 365,310.00
Base	Granular A, mm (t)	150	5,400	$ 16.00	$ 86,400.00
Subbase	Granular B - Type II, mm (t)	450	13,500	$ 13.00	$ 175,500.00
				Grand Initial Construction Total Cost	**$ 755,730.00**
Case A - Rubberized-RAP HMA Mixture					
Pavement Layer	**Description of Pavement Layer**	**Amount**	**Quantity per Km**	**Unit Cost $/Tonne**	**Total Cost**
Surface	SP 12.5 mm FC1 (20% RAP + 20% CRM)	40	1,512	$ 138.00	$ 208,656.00
Binder	SP 19, mm (t)	110	4,059	$ 90.00	$ 365,310.00
Base	Granular A, mm (t)	150	5,400	$ 16.00	$ 86,400.00
Subbase	Granular B - Type II, mm (t)	450	13,500	$ 13.00	$ 175,500.00
				Grand Initial Construction Total Cost	**$ 835,866.00**

Table 6-12: LCCA Summary, Comparisons and Rankings between Case Studies

Mix Discription	Discount Rate	Initial Cost	Total M&R Cost	Salvage Value	Present Worth Cost	Comparison	Rank (Least Expensive)
Case D (Control HMA)	3%	$614,736	$105,765	-$87,201.94	$806,044.73		4
Case B (20% RAP HMA)		$610,200	$105,765	-$85,527.62	$792,599.05	-1.7%	3
Case C (40% RAP HMA)		$605,664	$105,765	-$83,685.88	$777,808.80	-3.6%	2
Case A (Rubberized-RAP HMA)		$632,880	$1,539	-$92,559.75	$759,379.11	-6.1%	1
Case D (Control HMA)	4%	$614,736	$92,147	-$71,879.30	$635,003.92		4
Case B (20% RAP HMA)		$610,200	$92,147	-$70,499.19	$631,848.03	-0.5%	3
Case C (40% RAP HMA)		$605,664	$92,147	-$68,981.06	$628,830.16	-1.0%	2
Case A (Rubberized-RAP HMA)		$632,880	$1,415	-$76,295.67	$557,999.25	-13.8%	1
Case D (Control HMA)	5%	$614,736	$80,476	-$59,358.74	$635,853.19		4
Case B (20% RAP HMA)		$610,200	$80,476	-$58,219.02	$632,456.90	-0.5%	3
Case C (40% RAP HMA)		$605,664	$80,476	-$56,965.34	$629,174.59	-1.1%	2
Case A (Rubberized-RAP HMA)		$632,880	$1,303	-$63,005.82	$571,177.23	-11.3%	1
Case D (Control HMA)	7%	$614,736	$61,823	-$40,700.06	$808,605.14		4
Case B (20% RAP HMA)		$610,200	$61,823	-$39,918.60	$794,266.60	-1.8%	2
Case C (40% RAP HMA)		$605,664	$61,823	-$39,059.00	$778,494.21	-3.9%	1
Case A (Rubberized-RAP HMA)		$632,880	$1,110	-$43,200.73	$800,559.12	-1.0%	3

It is observed in Table 6-12 that Case A (Rubberized-RAP HMA) consistently returned the least expensive total maintenance and rehabilitation cost throughout the analysis period for all discount levels considered. The cost of maintaining Case C (40% RAP HMA), Case B (20% RAP HMA), and Case D (Control HMA) were observed to be comparable over the 20-year period of analysis at all levels of discount.

Table 6-12 also shows a negative salvage value for all case scenarios. This suggests that there is a value associated with the pavement at the end of the study period, and is attributed to the timely and appropriate preservation and maintenance schedule adopted. The analysis for Case A (Rubberized-RAP) is seen to return salvage values higher than Cases B, C and D across all discount levels considered. If a do-nothing approach were adopted, a more expensive rehabilitation or reconstruction in addition to a higher disposal cost would be associated with the pavement at the end of the analysis period.

Case scenarios for B, C and D are also observed to have comparable salvage values across all discount levels considered over the period of analysis. Comparisons were also made for variation in Present Worth Costs (PWC) between the recycled mixtures and the control mix. These variations across all discount rates considered are shown in Table 6-12.

On average, Case A (Rubberized-RAP HMA) is approximately 8% less expensive compared to Case D (Control HMA) while Case B (20% RAP HMA) and C (40% RAP HMA) respectively are approximately 2% and 3% less expensive compared to Case D (Control HMA) over the 20 year period of analysis. Based on the foregoing, the cost effectiveness of the respective case studies were ranked as highlighted in Table 6-12. The rankings suggests that increasing the RAP content will result in some economic savings, but combining RAP and CRM will result in significant economic savings.

6.3.3 Sensitivity Analysis

The overall best and worst design alternatives were further ranked by determining the influence of the major lifecycle input variables through a sensitivity analysis. The sensitivity analysis utilized a probabilistic approach to determine the variability and effects of the various input assumptions on the computed Net Present Values (NPV) for all case studies. These were completed over the period of analysis for all discount levels considered. Overall, Figure 6-4 through Figure 6-7 shows that the cost effectiveness of the respective design alternatives were sensitive to the discount rates applied. It is observed that the cost benefit attached to utilizing recycled materials as opposed to a virgin materials in HMA mixtures is significant. In Figure 6-4, Case A (Rubberized-RAP HMA) showed significant variation in cost when compared to the other design alternatives at 3% discount rate.

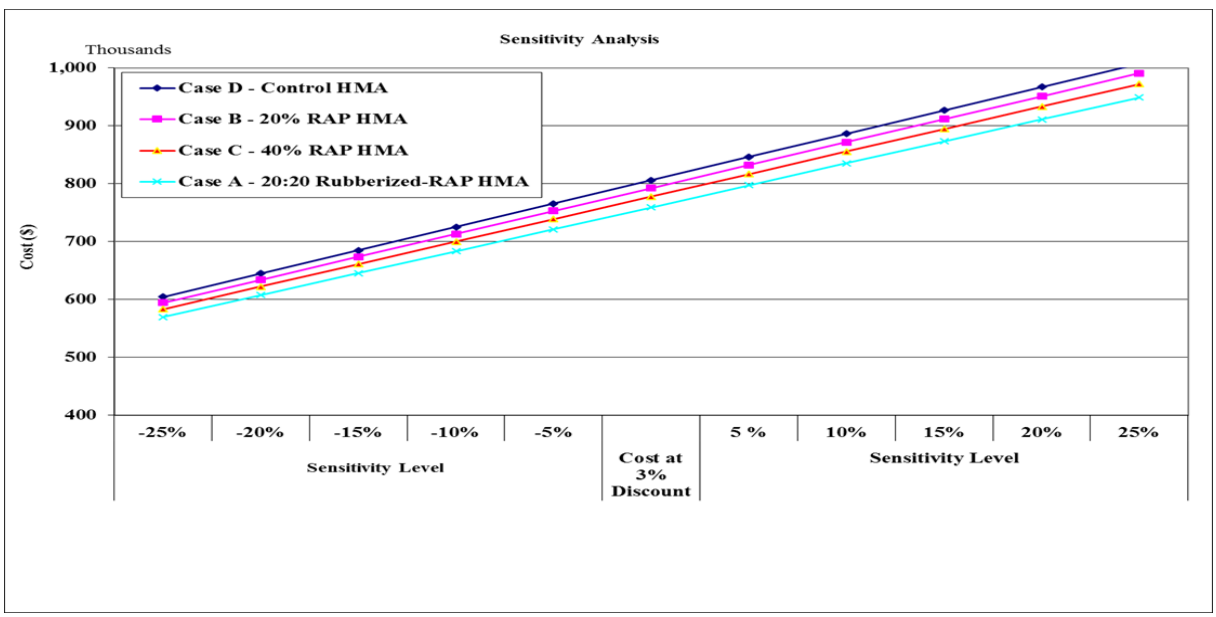

Figure 6-4: Sensitivity Analysis at 3% Discount Rate

Figure 6-5 shows that at 4% discount rate the variation in costs between all recycled design alternative are comparable.

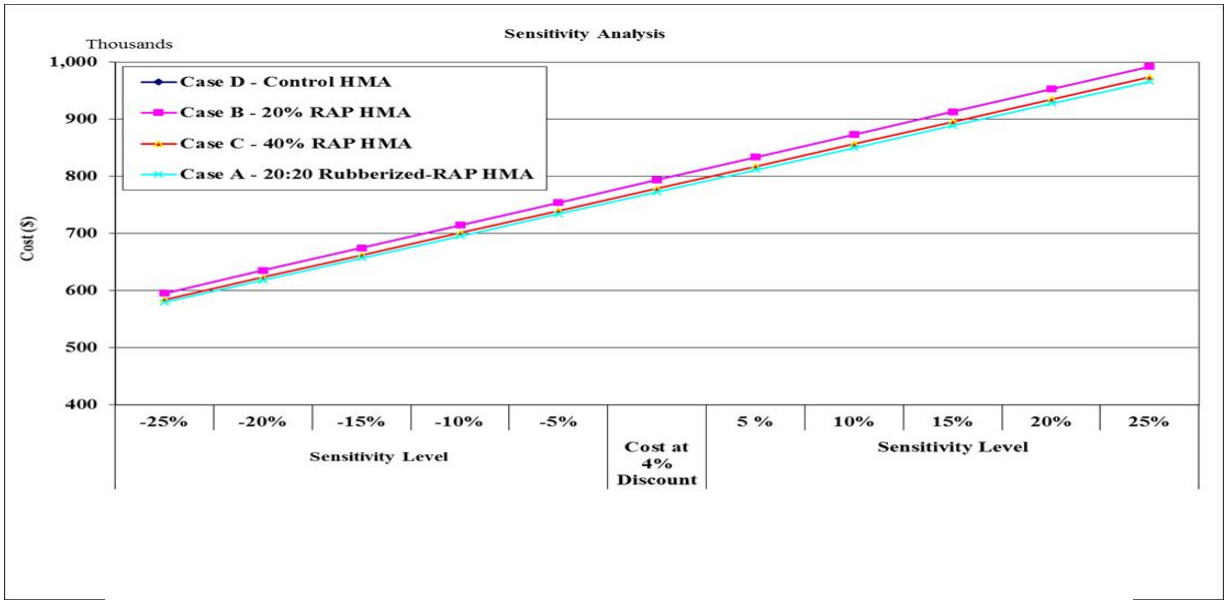

Figure 6-5: Sensitivity Analysis at 4% Discount Rate

In Figure 6-6, the variation in cost between Case A (Rubberized-RAP HMA) and Case C (40% RAP HMA) are comparable.

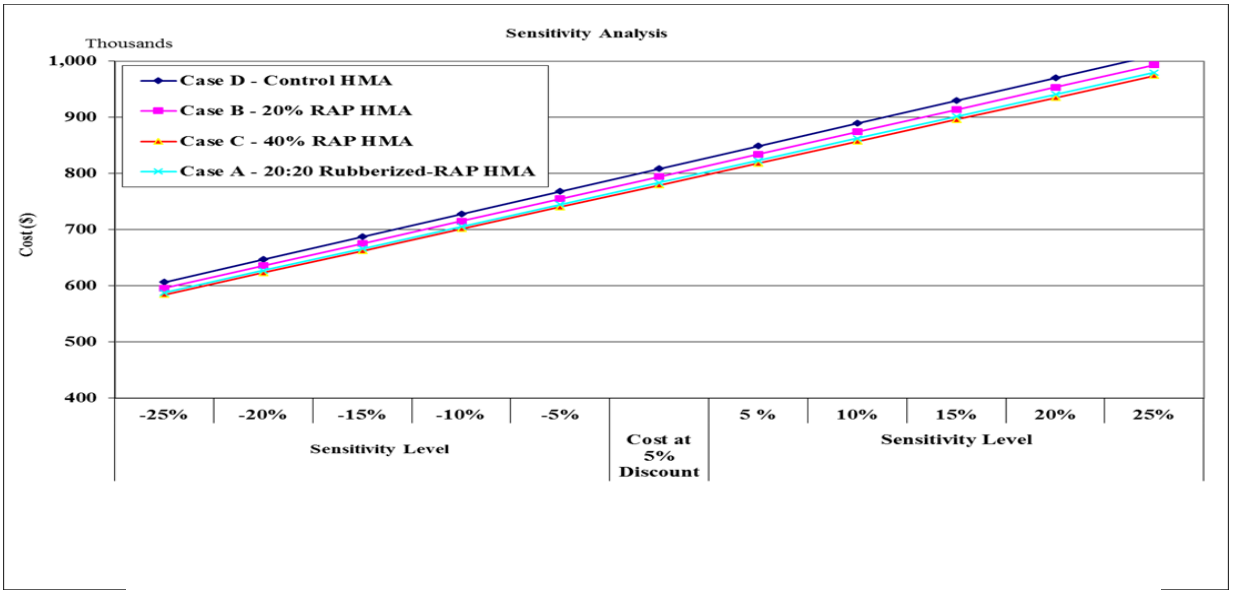

Figure 6-6: Sensitivity Analysis at 5% Discount Rate

In Figure 6-7, cost variation is mostly favorable to using Case C (40% RAP HMA) at 7% discount rate.

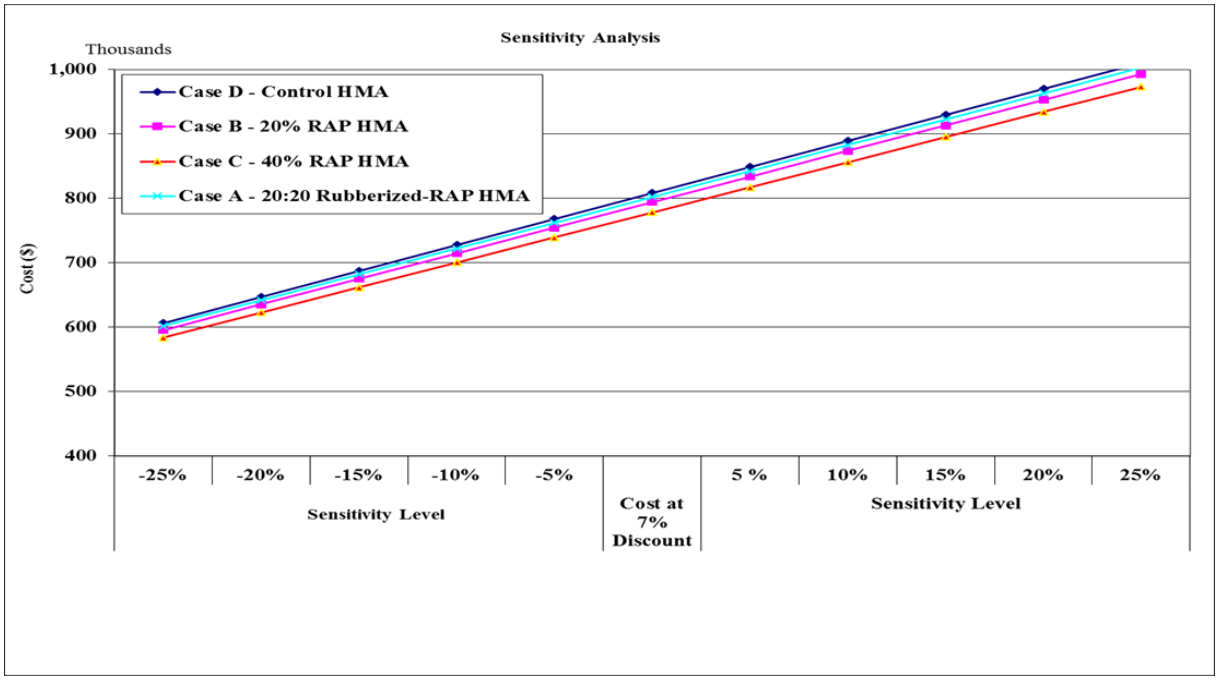

Figure 6-7: Sensitivity Analysis at 7% Discount Rate

Table 6-13 summarizes the standard deviation in cost effectiveness for each design alternative relative to the control design at the various discount rate considered over the 20-year period of analysis. At 3% discount rate for example, Case B (20% RAP HMA) is observed to be $9,507.53 less expensive when compared with the cost of maintaining Case D (Control HMA). This trend is repeated for Case C (40% RAP HMA) and Case A (Rubberized-RAP HMA) at all discount rates. However, the overall observation demonstrates that costs are very similar, but cost effectiveness changes slightly with different discount rates. The least expensive design alternative being Case A (Rubberized-RAP HMA) was consistent for all discount rates except at 7%.

Although the initial costs for Case A (Rubberized-RAP HMA) was higher than all other design alternatives; it should be noted that if HMA mixtures incorporating CRM or combining CRM with RAP is used more widely, the initial cost of construction is certain to decrease. It should also be noted that the output from sensitivity analysis which was also reflected in the calculated standard deviations for cost effectiveness is connected to the fact that a subjective approach was utilized in the selection of the maintenance and rehabilitation schedule and strategies.

A more comprehensive and qualitative approach - preferably a mechanistic-empirical based method that is capable of utilizing both laboratory performance and the statistically analyzed data is recommended.

Table 6-13: Standard Deviation in Cost Effectiveness

Discount Rate	Mix Description	Standard Deviation
3%	Case B (20% RAP HMA)	$ 9,507.53
	Case C (40% RAP HMA)	$ 19,965.82
	Case A (Rubberized-RAP HMA)	$ 32,997.58
4%	Case B (20% RAP HMA)	$ 2,231.55
	Case C (40% RAP HMA)	$ 4,365.51
	Case A (Rubberized-RAP HMA)	$ 54,450.51
5%	Case B (20% RAP HMA)	$ 2,401.54
	Case C (40% RAP HMA)	$ 4,722.48
	Case A (Rubberized-RAP HMA)	$ 45,732.81
7%	Case B (20% RAP HMA)	$ 10,138.88
	Case C (40% RAP HMA)	$ 21,291.65
	Case A (Rubberized-RAP HMA)	$ 5,689.40

6.4 Environmental Analysis using PaLATE

The environmental impacts of utilizing RAP and CRM in HMA mixtures in this research were assessed using the "Pavement Life-Cycle Assessment Tool for Environmental and economic Effects (PaLATE)". PaLATE is a Microsoft Excel-based spreadsheet program developed by Dr. Arpad Horvath to quantify the environmental consequences from constructing and maintaining pavements (Horvath, 2004). PaLATE roughly estimates the trade-offs between using virgin and recycled materials.

It calculates the cumulative environmental effects such as energy consumption, water consumption, Carbon Dioxide (CO_2), Nitrogen Monoxide (NO_x), Particulate Matter concentrations (PM_{10}), Sulfur Dioxide (SO_2), Carbonic Oxide (CO), Mercury (Hg) and Lead (Pb) emissions from input data for initial construction and maintenance material quantities, transportation distance, and equipment use (Horvath, 2004). Although PaLATE is simply meant to understand the general environmental effects of using recycled materials, it should be noted that the software can also calculate the Net Present Value (NPV) of the pavement over its life cycle. For the analysis conducted in this research, details of the PaLATE framework, input formulation and how environmental impact is quantified are explained in Appendix E.

For the purpose of the environmental quantification for the research case studies, a PaLATE input workbook was completed for a 1 Km, one-lane roadway with 4 m lane width and surface layer depth of 40 mm. Note that only the initial construction of the surface course was analyzed. A transportation distance of 120 Km (75 miles) was also assumed. The material quantities for virgin aggregate, asphalt binder, CRM and RAP for the mixes were calculated in accordance with the provisions from the respective HMA mix designs, and the material densities highlighted in Table 6-14.

Table 6-14: PaLATE Material Densities (Horvath, 2004)

Material	Suggested Density (tons/yd^3)	Density used (tons/yd^3)
Asphalt Mixture	1.23	2.16*
Asphalt Binder	0.84	0.84
RAP	1.62 - 1.89	1.85
CRM	0.97	0.97
Virgin Aggregate	1.25	2.23*

[*] Adopted from Ddamba, 2011

The total pavement asphalt (TPA) required for the analysis was calculated using Equation 6-5.

$$TPA = \text{Width (ft)} \times \text{Length (miles)} \times \text{Depth (inches)} \qquad \text{(Equation 6-5)}$$

$$TPA = 8.77\ (yd) \times 1092.96\ (yd) \times 0.044\ (yd) = 422yd^3$$

The material quantities for virgin aggregate, asphalt binder, CRM and RAP were also determined as follows:

$$\text{Weight of Mix} = \text{TPA} \times \text{HMA Mix Density} \qquad \text{(Equation 6-6)}$$

$$\text{Weight of Mix} = 422yd^3 \times 2.16\ tons/yd3 = 911.52tons$$

$$\text{Weight of Binder} = \text{Weight of Mix} \times \text{HMA Mix Density} \qquad \text{(Equation 6-7)}$$

$$\text{Weight of Binder} = 0.052 \times 911.52tons = 47.4tons$$

The total volume of asphalt binder required was calculated using the assumed binder density from Table 6-14 in Equation 6-8. Note that CRM makes up 1% of the volume of binder for Case A - Rubberized-RAP HMA mix. The percentage binder contribution from RAP is also considered in this analysis.

$$\text{Volume of Binder} = \text{Weight of Binder} \div \text{Density of Binder} \qquad \text{(Equation 6-8)}$$

$$\text{Volume of Binder} = 47.4tons \div 0.84tons/yd^3 = 56.4yd^3$$

Case D - Control HMA is made up of 94.8% aggregates; therefore, volume of aggregate was calculated using Equation 6-9:

$$\text{Volume of Aggregates} = \text{TPA} \times \text{Material Percentage} \qquad \text{(Equation 6-9)}$$

$$\text{Volume of Aggregates} = 422tons \times 0.948 = 400tons$$

The total volume of RAP in Case A, B, and C were determined using their material densities and mix design proportions using Equation 6-10; an example is illustrated for Case B – 20% RAP HMA:

$$\text{Volume of RAP} = \frac{\text{Weight of Mix x Density of HMA}}{\text{Density of RAP}} \times \text{Material Percentage} \qquad \text{(Equation 6-9)}$$

$$\text{Volume of RAP} = \frac{911.5 \times 2.16}{1.85} \times 0.2 = 212.9\text{yd}^3$$

The volume of RAP is then subtracted from the total volume of virgin aggregates to determine the required virgin aggregate.

It should be noted that the calculations illustrated above are in the Imperial system of units. This is representative of the requirements for the PaLATE software since it was designed in the United States. However, the units for all input data were converted to the SI system for the purposes of this thesis. Table 6-15 summarizes the PaLATE input data.

Table 6-15: PaLATE Input Data in SI Units

Mix Category	Asphalt Binder (m^3)	Aggregates (m^3)	RAP (m^3)	Total HMA (m^3)	Distance (km)
Case A (Rubberized-RAP HMA)	49.7	187.9	124.8	322	120
Case B (20% RAP HMA)	35.6	239.1	72		
Case C (40% RAP HMA)	27.4	181.2	145.6		
Case D (Control HMA)	43.1	296	0		

Note: 1 cubic yard is equivalent to 0.764 m^3

6.4.1 PaLATE Results

This section presents the PaLATE output data (GHG emissions, Energy and Water usage) for initial pavement construction based on the input data summarized in Table 6-16. The main outputs discussed include: energy usage, water usage, and Carbon Dioxide (CO_2) which reflects global warming potential; since these contribute the most impact on the environment. Other output data presented include CO, PM_{10}, NO_x, SO_2, Hg and Pb.

Table 6-16: PaLATE HMA Output at Initial Construction

Description	Case Comparison			
	Case A (Rubberized-RAP)	**Case B-20% RAP**	**Case C-40% RAP**	**Case D-Control HMA**
Energy [MJ]	1,303,396	1,089,025	886,754	1,270,975
Water Consumption [kg]	466	353	274	424
CO_2 [Mg] = GWP	70	59	48	70
NOx [kg]	470	669	614	718
PM_{10} [kg]	147	210	175	244
SO_2 [kg]	10,118	9,626	9,773	9,467
CO [kg]	269	223	178	263
Hg [g]	1.93	1.44	1.12	1.72
Pb [g]	91	69	54	83
RCRA Hazardous Waste Generated [kg]	19,333	14,467	11,237	17,342
Human Toxicity Potential (Cancer)	309,638	229,107	179,487	273,267
Human Toxicity Potential (Non-Cancer)	109,231,082	137,394,455	110,857,117	163,574,204

The environmental gains were evaluated in terms of the effects of increasing the RAP percentage as well as the sustainability of incorporating RAP and CRM in HMA. The PaLATE results for all research case studies considered are tabularized in Table 6-16 while the environmental gains in terms of energy usage, water usage, and CO_2 emission are graphically illustrated in Figure 6-8 through Figure 6-10. The total CO_2 emission between Case A - Rubberized-RAP HMA mix and Case D - Control HMA mix are comparable at initial pavement construction.

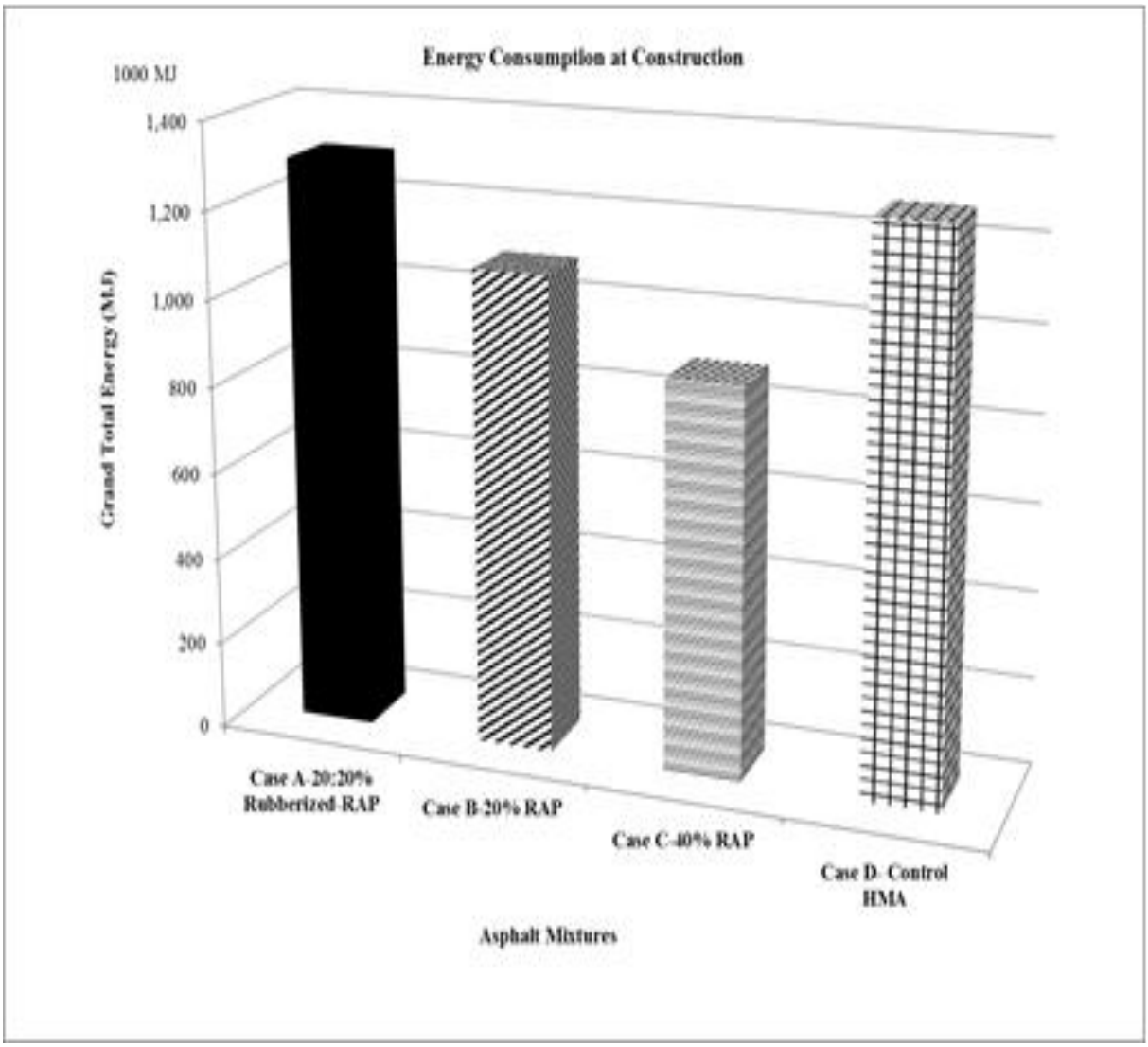

Figure 6-8: Energy Consumption at Initial Construction

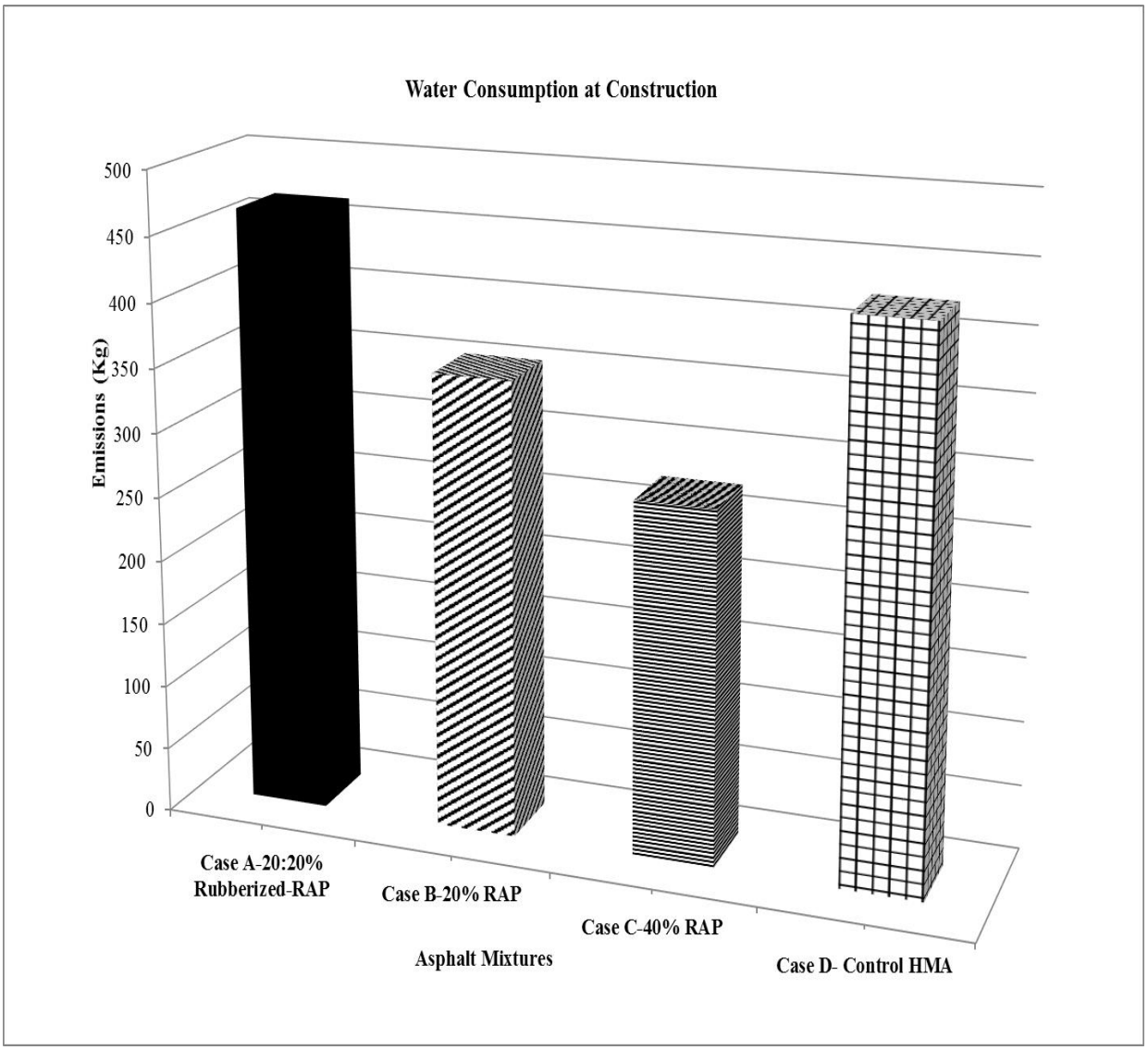

Figure 6-9: Water Consumption at Initial Construction

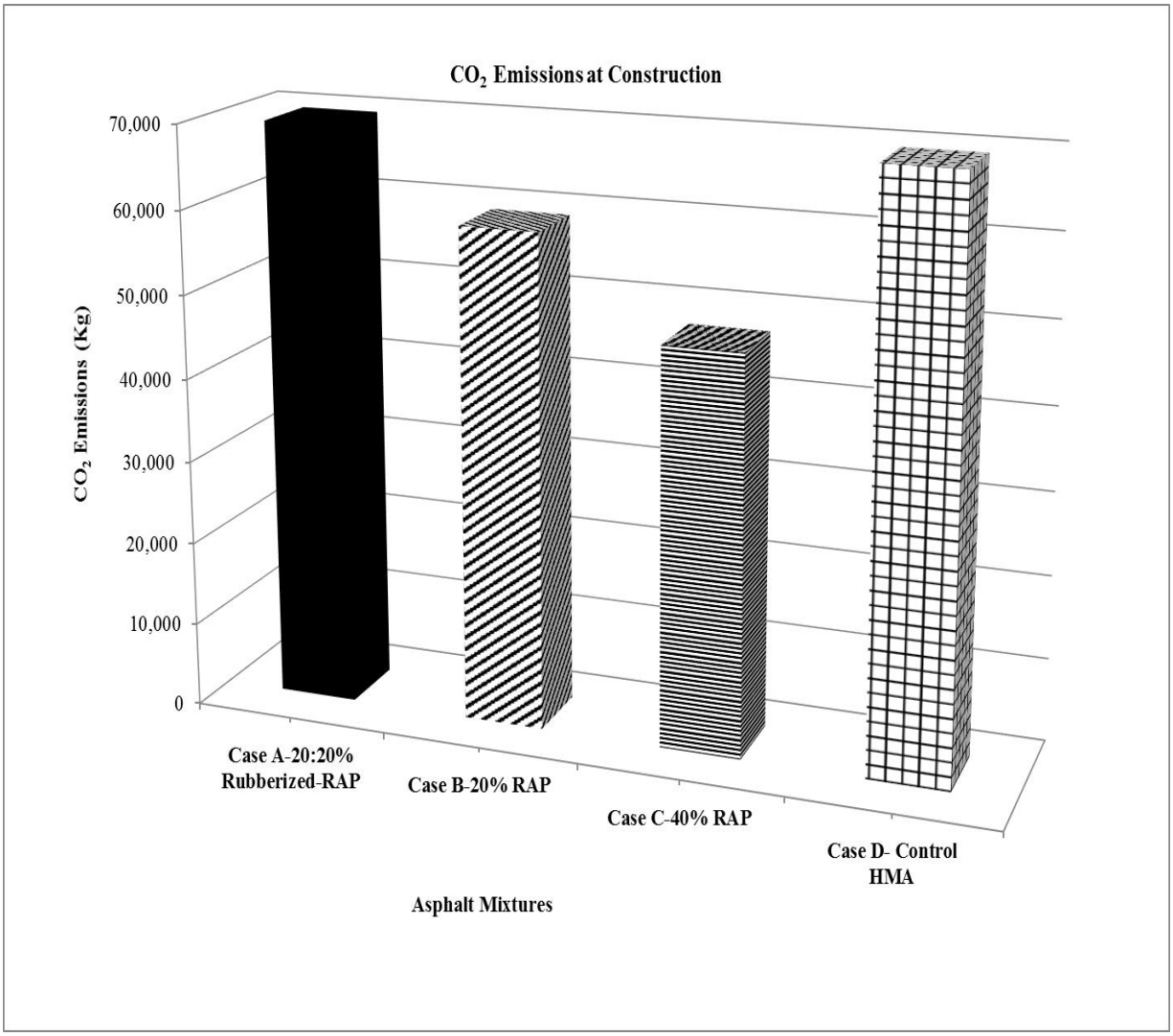

Figure 6-10: CO_2 Emission at Initial Construction

Although Case A - Rubberized-RAP HMA mix returned the least environmental savings, it should be noted that using crumb rubber in HMA contributes significantly in minimizing the negative environmental effects from accumulating large amount of scrap rubber tires, landfilling and health related concerns. In this regard, it should be noted that there is a significant cost-benefit in removing scrap tires from the waste stream. There is also a significant reduction in emissions from the equipment and facilities involved in processing scrap rubber tires compared to those from tire fires being allowed to burn uncontrollably.

According to the PaLATE findings; Case C - 40% RAP HMA mix returned the least quantity of emissions into the environment as well as consumed less water and energy at initial construction. This was closely followed by Case B – 20% RAP HMA mix, Case D - Control HMA mix and finally case study A - Rubberized-RAP HMA mix. Figure 6-11 through Figure 6-15 summarizes and compares CO, PM_{10}, NOx, SO2, Hg and Pb air emissions between the evaluated HMA mixtures. Overall, case A - Rubberized-RAP HMA mix reports the least PM_{10} and NO_x emissions while Case C - 40% RAP HMA mix is observed to have the least Pb emission. The Case A – Control had the least SO_2 emission.

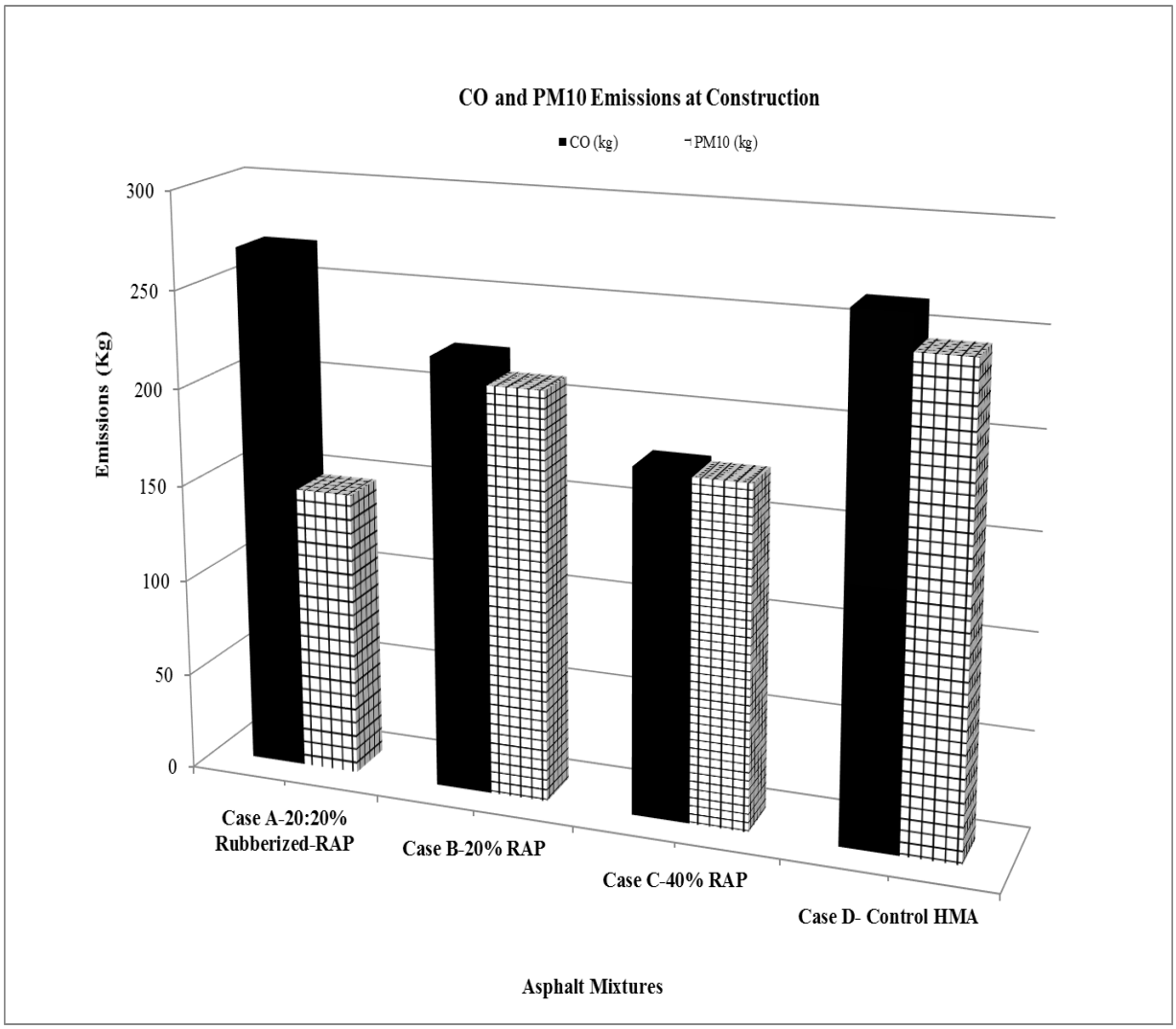

Figure 6-11: CO and PM_{10} Emissions at Initial Construction

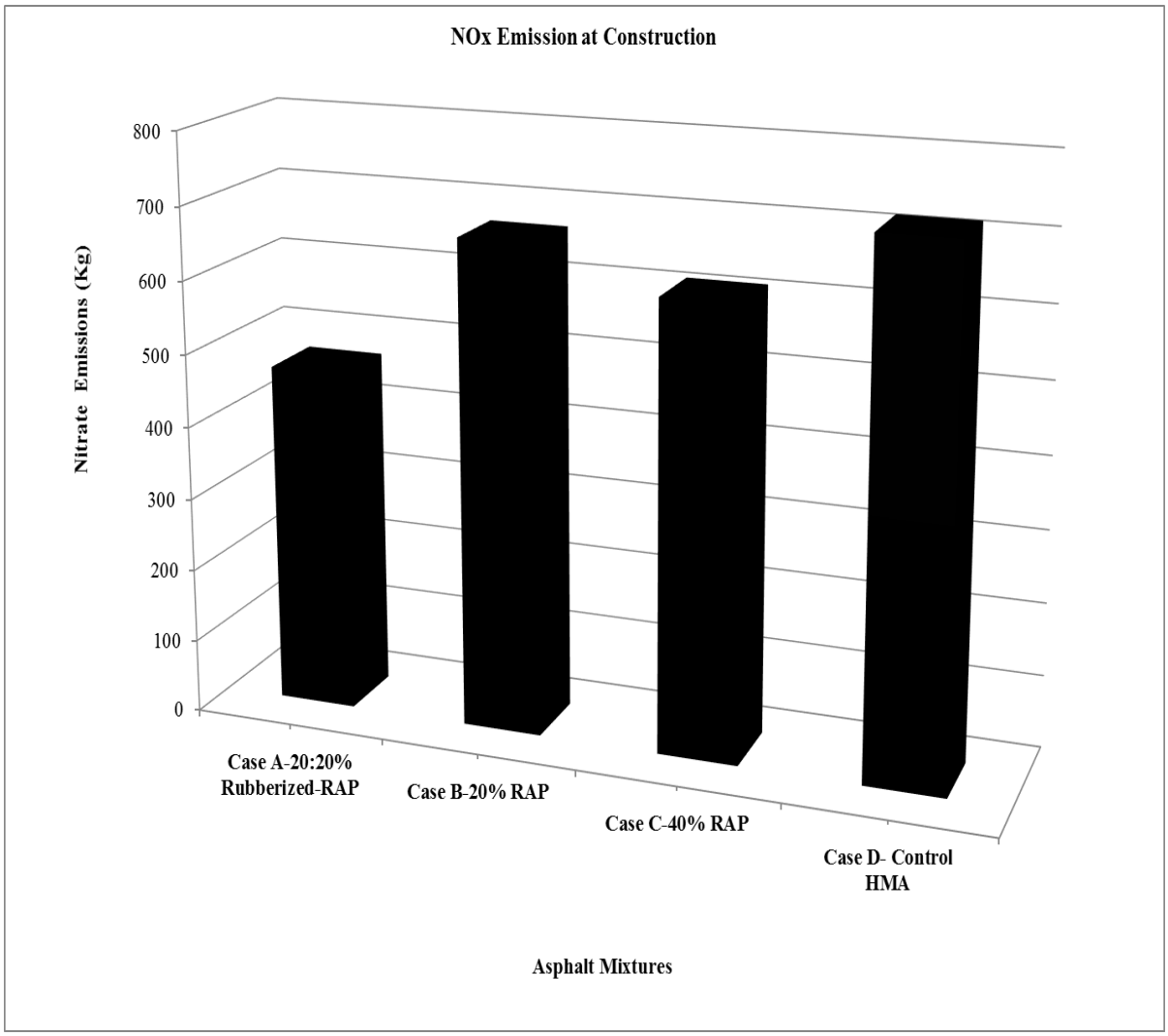

Figure 6-12: NOx Emissions at Initial Construction

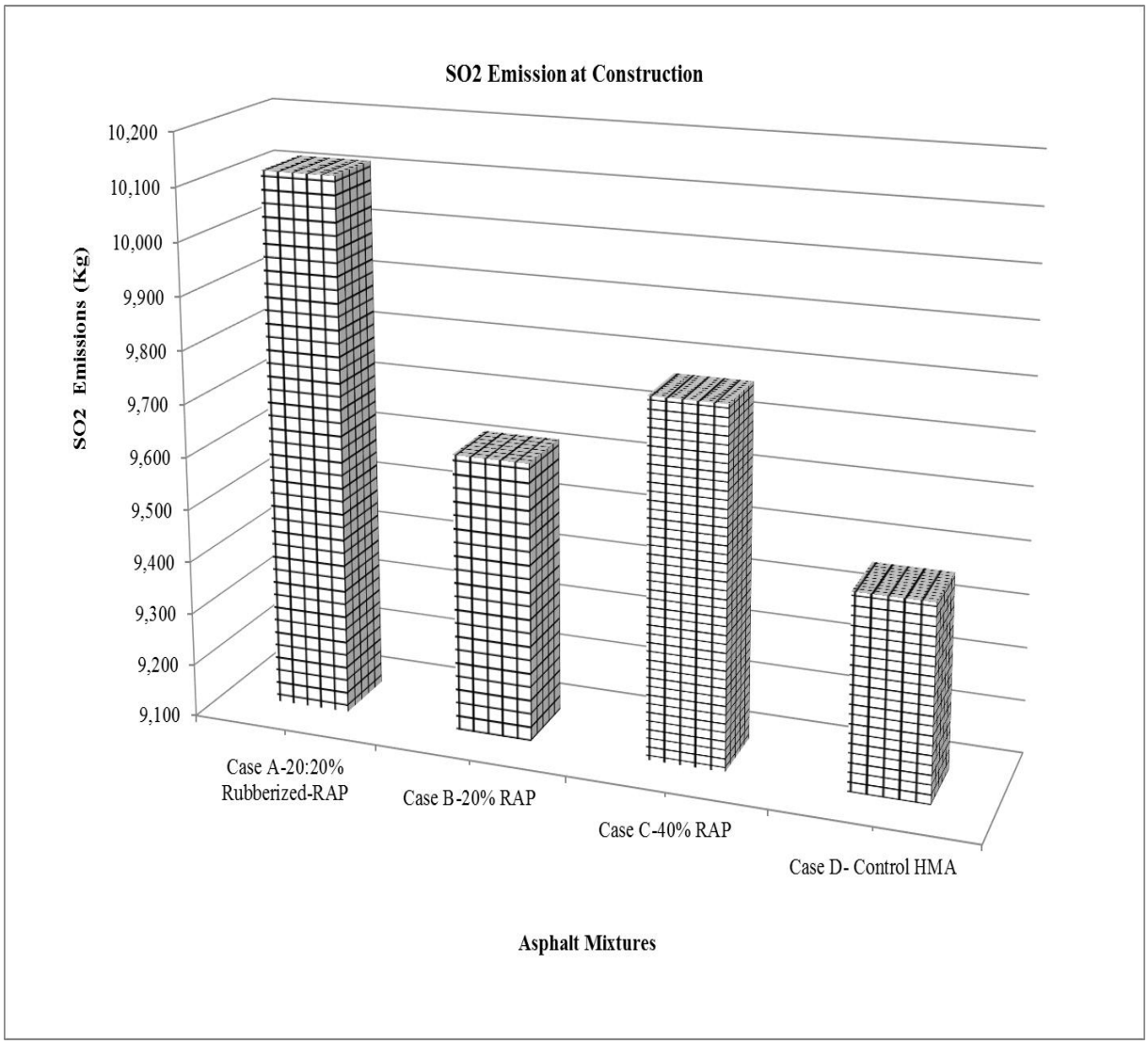

Figure 6-13: SO_2 Emissions at Initial Construction

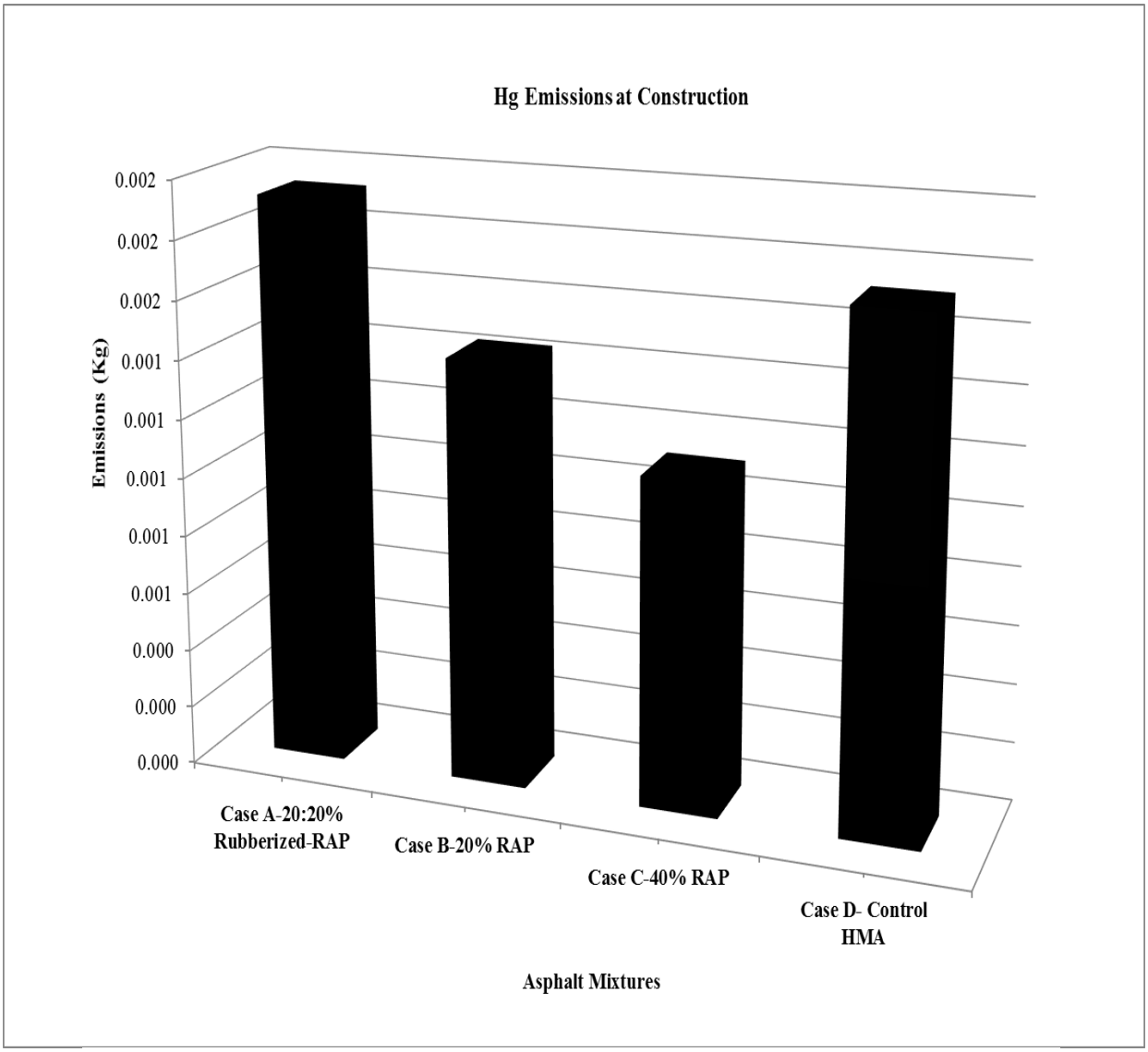

Figure 6-14: Mercury (Hg) Emissions at Initial Construction

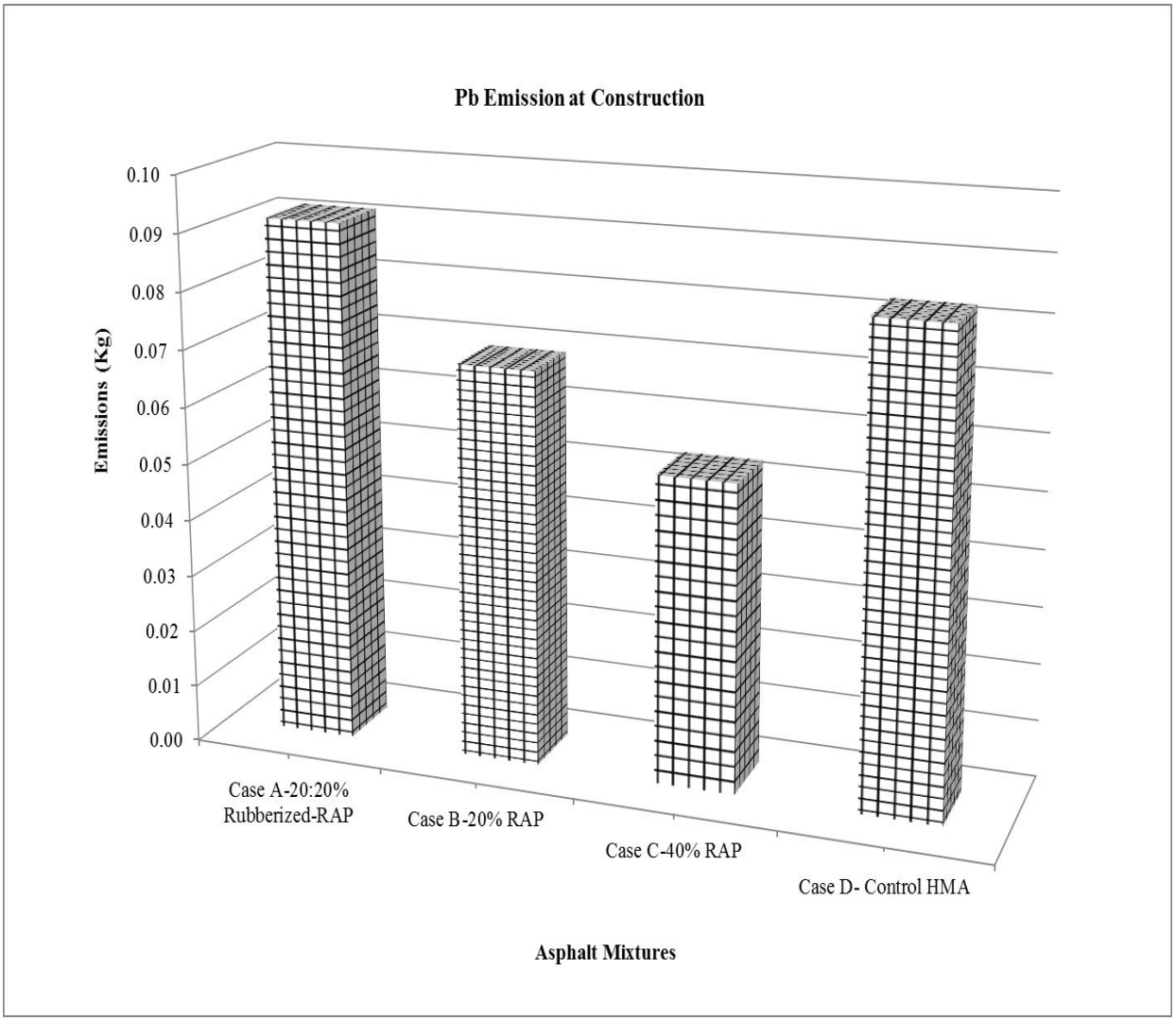

Figure 6-15: Lead (Pb) Emissions at Initial Construction

6.4.2 Relative Environmental Percentage Savings (RPS)

The relative environmental percentage savings (RPS) in air emissions, energy and water consumption incurred by the evaluated HMA mixtures were computed and compared using Equation 6-10:

$$RPS = \left({(Control\ Mix - Alternative\ Mix)}/{Control\ Mix} \right) * 100$$

(Equation 6-10)

Where; RPS = Relative Environmental Percentage Savings

Control Mix = Case D

Alternative Mix = Case A, B or C

The RPS results are tabularized in Table 6-17 and graphically illustrated in Figure 6-16 and Figure 6-17.

Table 6-17: RPS Comparison between Control and Recycled HMA Mixtures

Environmental Emission	**Relative Environmental Percentage Savings (%)**		
	Case A - 20:20% Rubberized-RAP	**Case B -20% RAP**	**Case C - 40% RAP**
Energy (MJ)	-2.6	14.3	30.2
Water Consumption (kg)	-9.7	16.8	35.5
CO_2 (Kg) = GWP	**0.1**	15.0	31.6
NO_x (kg)	**34.5**	6.8	14.5
PM_{10} (kg)	**39.8**	13.9	28.4
SO_2 (kg)	-6.9	**-1.7**	**-3.2**
CO (kg)	-2.4	15.3	32.5
Hg (kg)	-11.9	16.6	35.2
Pb (kg)	-10.3	16.6	35.3
RCRA Hazardous Waste Generated (kg)	-11.5	16.6	35.2
Human Toxicity Potential (Cancer)	-13.3	16.2	34.3
Human Toxicity Potential (Non-Cancer)	**33.2**	16.0	32.2
Average Savings	**3.4**	**12.6**	**26.7**

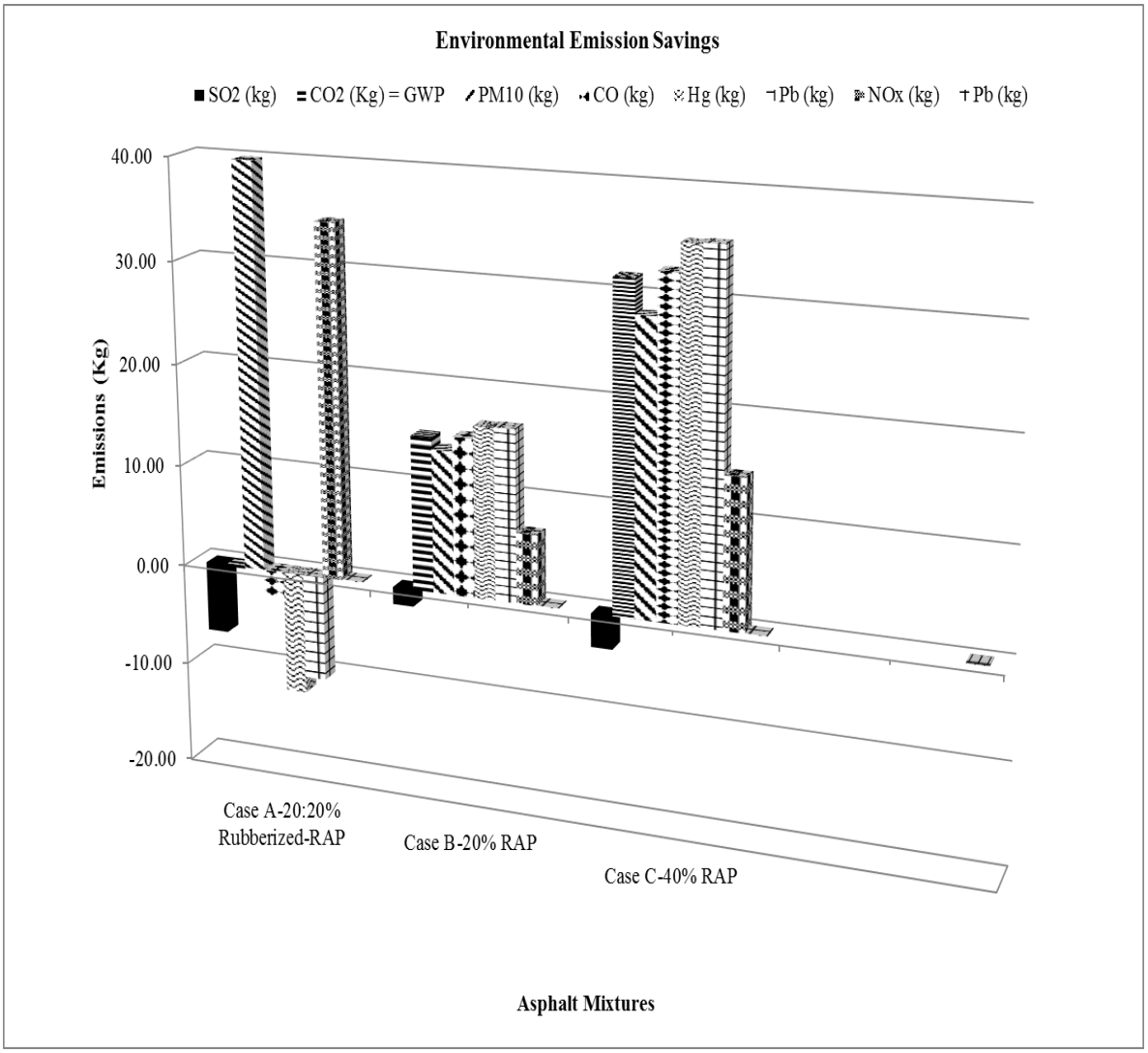

Figure 6-16: RPS, Environment Emissions Savings

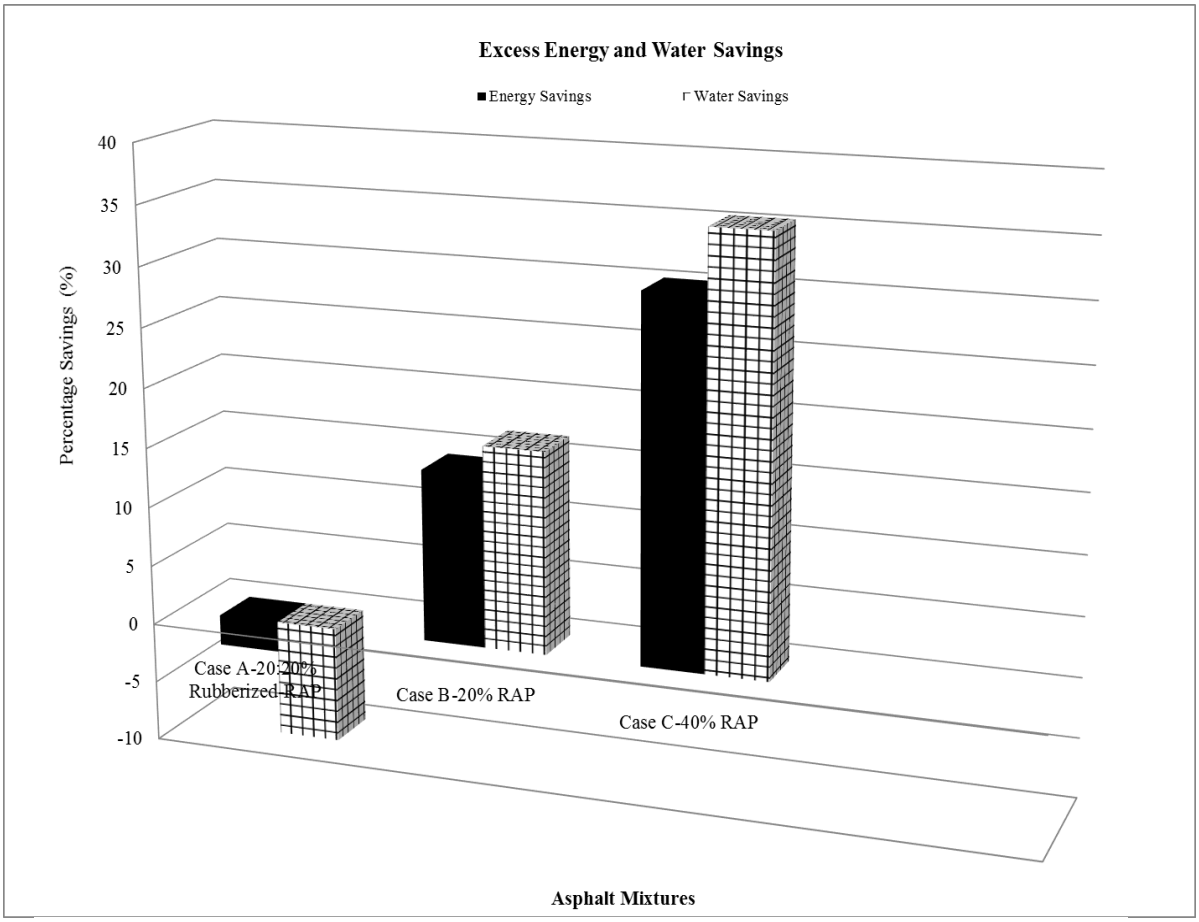

Figure 6-17: RPS, Excess Energy and Water Savings

For comparisons made between Case D - Control HMA mix and the recycled HMA mixtures, Case C - 40% RAP HMA mix is observed to have the highest average relative environmental percentage savings (RPS) of 26.7%. This is followed by Case B - 20% RAP HMA mix with 12.6%, and Case A - Rubberized-RAP HMA mix with 3.4% in total environmental savings. The results presented for relative environmental percentage savings (RPS) clearly show that Case A - Rubberized-RAP HMA mix only returned savings for Carbon Dioxide (CO_2), Nitrogen Dioxide (NOx), Particulate Matter concentration (PM10), and human toxicity potential (non-cancer) whereas Case B - 20% RAP HMA mix, and Case C - 40% RAP HMA mix recorded low savings on Sulphur Dioxide (SO_2) for comparisons made with Case study D - Control HMA mix. The savings recorded for PM_{10} are beneficial for health reasons because inhaling PM_{10} could result in harmful respiratory issues overtime.

The analysis detailed in Table 6-17 further showed that the degree to which Case A – Rubberized-RAP HMA mix negatively impacted the environment was just under 3 - 13% across the criteria considered relative to Case D – Control HMA.

Although this analysis may raise questions on the environmental impact of incorporating or increasing the crumb rubber content in HMA mixtures, it should be noted that incorporating crumb rubber in HMA mixtures actually seems to reduce certain emissions, while in others it may increase emissions over conventional asphalt. In addition, it is yet to be proven that rubberized HMA exposures are more hazardous than conventional HMA exposures. This is mostly because the compositions of crumb rubber modified asphalt which includes differences in rubber tire sources and other hydrocarbons, carbon black, extender oils, and inert fillers are also present to some extent in conventional binders, but in varying proportions (Hicks et al., 2013).

Many other studies that looked into the concerns of rubberized HMA have also indicated that there are no obvious trends to indicate a significant increase or decrease in emissions attributed to the use of CRM in HMA (Hicks et al., 2013). However, a look at Table 6-17 suggests that the trend may not be obvious in terms of CO_2, CO, SO_2, Hg, Pb and Human Toxicity Potential (cancer) air emissions, but it is significant (almost 33 – 40%) in PM_{10}, NO_x, SO_2, and Human Toxicity Potential (non-cancer) air emissions.

The analysis conducted herein is thus in agreement with the conclusions from a New Jersey (1994), Michigan (1994), Texas (1995), and California (1994 and 2001) study that evaluated and compared environmental emission issues resulting from the production of rubber modified asphalt (RMA) and dense–graded asphalt concrete HMA mixtures. The main conclusions from these studies as summarized in the Rubber Modified Technical Manual prepared for Ontario by the Ontario Tire Stewardship is that emissions from the production of RMA are not significantly different than those from the production of conventional dense-graded asphalt concrete mixtures (Hicks et al., 2013).

Given these findings, it is reasonable to conclude that the results from the analysis using the PaLATE sustainability assessment tool are both encouraging and unfavourable to the green pavements objective, but the capability of the PaLATE tool could be improved for better reliability.

6.5 Optimal Sustainable Mix Design Alternative

An optimal sustainable pavement mix design does not compromise pavement performance or possess financial constraints. The main indicators for this as earlier described is one that minimizing the use of

natural resources, reduces or limits energy consumption, greenhouse gas (GHG) emissions, pollution, and improves health, safety and risk prevention whilst ensuring a high level of user safety and comfort. Table 6-18 summarizes and ranks the optimal sustainability of the respective research case studies based on overall performance, cost and environmental assessment. Each design option is awarded points in their order of performance while the optimal sustainable pavement option is determined based on the least number of total points earned from rankings.

Table 6-18 shows that at initial construction, Case C - 40% RAP HMA mix ranked best in overall environmental savings, but this did not make it the optimal sustainable option given the other influencing factors.

The research Case A - Rubberized-RAP HMA mix is considered the most innovative and optimal sustainable option for use in flexible pavement construction given that it satisfies both structural and functional performance requirements while aiding in the social and economic development, including the potential to minimize negative environmental impacts.

Table 6-18: Optimal Sustainable HMA Mix Design

HMA Design Alternative (Case Study)	Overall Laboratory Performance	Sustainability Rating Tool			Total Points Earned	Optimal Sustainable Design Option
		MTO's GreenPave	LCCA Economic Analysis	LCCA Environmental Analysis		
A - Rubberized-RAP (20% RAP + 20% CRM)	1	1	1	4	7	**1**
B - 20% RAP	3	3	2	2	10	**3**
C - 40% RAP	2	2	4	1	9	**2**
D - Control (0% RAP)	4	4	3	3	14	**4**

6.6 Summary

This Chapter introduced sustainable pavements and its key indicators with respect to the environment, economy and society. The GreenPave pavement sustainability rating tool developed by the Ministry of Transportation Ontario was also examined and applied towards assessing the applicability and impacts of combining RAP and CRM in HMA. A life cycle cost assessment (LCCA) was also employed to

evaluate both environmental, and economic savings over a 20 year analysis period at different discount rates.

Based on the research findings, it is concluded that increasing the RAP percentage in HMA is environmentally sustainable since these resulted in the most savings in air emissions, water and energy consumption. However, the innovative use of RAP and CRM in HMA was judged to be the most optimal sustainable option for use in flexible pavement construction since this combination demonstrated better cost effectiveness in addition to an exceptional level of performance including the potential to minimize negative environmental savings. By encouraging these initiatives, the Ministry of Transportation Ontario (MTO) goal to have one of the greenest roads in North America is realizable.

Chapter 7
Conclusions, Recommendations and Future Research

7.1 Conclusions

This research has explored the feasibility of designing and constructing conventional Ontario Superpave HMA mixtures incorporating RAP and CRM without compromising pavement performance. It is intended towards advancing Ontario's paving industry to a more sustainable and economically viable direction; particularly by encouraging pavement recycling practices.

Results obtained from the comprehensive laboratory performance characterization satisfied the research hypothesis that combining RAP with CRM is capable of compensating for RAP shortfalls such as its effects on binder aging and mix stiffness, thus improving the mixture's stability, durability, impermeability, workability and flexibility. The following general and specific findings have thus been drawn:

1. Rheological characterization of binders indicated that the influence of RAP variation is highly related to the performance grade of the base asphalt cement. The high and low temperature continuous true grades were found to be significantly affected by the addition of RAP in varying percentage.

2. CRM binder modification significantly improved both G* and δ parameters of the binder. The CRM modified binder thus resulted in the balancing of the asphaltenes and maltenes components of the aged-RAP binder. This had a significant impact on the rutting and thermal cracking performance of the evaluated HMA mixtures.

3. All recovered binders were observed to be more flexible at low and intermediate temperatures suggesting that the potential for improved resistance to fatigue failure exists. This observed flexibility is attributed to a reduction in the G*•sin (δ). Increasing the RAP content or combining RAP with CRM was observed to have lowered the temperature at which G*•sin (δ) was less than or equal to 5000 KPa. The only exceptions to this were the 20% and 40% RAP HMA mixtures with PG 52-40 and 52-34 asphalt cement. It is possible that the virgin asphalt content for these category of HMA mixtures was insufficient.

4. In general, the mix stiffness characterization at high temperatures appeared to be consistent with the Superpave rutting parameter of G*/sin (δ) of the recovered blended binder. However, positive differences observed with HWTD for rut characterization suggests that the G*/sin (δ) did not accurately capture the contribution of the blended binder in rutting susceptibility. A similar trend was observed for the low temperature continuous true grade whereas a much better performance was observed with the Thermal Stress Restrained Specimen Test (TSRST) method. This could be attributed to type of solvent used for extraction and recovery and/or a validation the inadequacies of the Superpave binder characterization test method; particularly with modified or rubberized binders.

5. Assessment of mix stiffness characterization showed that increasing the RAP content or combining RAP with CRM will result in a mix that is stiff, more elastic or flexible, and capable of withstanding the effects of low temperature cracking and fatigue cracking including those of rutting or permanent deformation. The master curve construction using the rheological analysis software (RHEA™) confirmed these behavioural tendencies.

6. The analysis performed on dynamic modulus (mix stiffness, |E*|) data were found to be mostly statistically significant, thus indicating a higher degree of sensitivity on the effects of increasing RAP contents or combining same with CRM whereas those performed on phase angle, δ data were statistically insignificant. The main deductions from this is that the observed mix stiffness is not exclusively a function of the improvements made to the binder, but in combination with other factors. The observed mix stiffness was also observed to correlate well with the mechanistic performance test findings.

7. The wet-process rubber terminal-blend HMA mixture was noted to be distinctively different from the rubber field-blend mixtures in terms of performance. Whereas this performance difference could be attributed to the former having a less viscous binder, there was no evidence within the concerns of this research to suggest that the rubber field-blend method is not effective or feasible. These generic wet-processes showed the capability of providing performance improvements over strategies that use traditional, and RAP-recycled HMA mixtures.

8. Although the laboratory-prepared mixes lacked anti-stripping and other mix improving or binder rejuvenating agents, the mixtures in the PG 58-28 were found to have compared favorably with the plant-mixes of the same binder grade. This is attributed to the rigorous procedure adhered to developing the mix designs; particularly in terms of maintaining and achieving consistencies with consensus and volumetric properties. This suggests that a better design for field application is possible.

9. Although all evaluated HMA mixtures performed very well in terms of resisting the effects of rutting, stripping and moisture damage, it was observed that increasing the RAP content negatively affected the low-temperature performance HMA mixtures of the laboratory-prepared PG 52-34, 58-34 and 52-40 category. This confirmed the behavioral differences and effects between various grades of asphalt binder on mix properties.

10. Forensic studies on extracted pavement cores indicated that the observed pavement distresses could be related to aggregate segregation resulting from the effects of permeability possibly caused by poorly constructed or compacted longitudinal joints.

From a sustainability perspective, incorporating CRM in HMA can reduce the percentage of scrap rubber tires that are landfilled or indiscriminately disposed in an unacceptable environmental manner. RAP in HMA allows for the reuse of aggregates and old binder, thus reducing the need for new materials and energy required to produce asphalt mixtures. The cost and sustainability analysis completed in this study revealed as follows:

1. The GreenPave rating tool resulted in a Silver certification for a Rubberized-RAP (20% RAP + 20% CRM) pavement section while sections incorporating 20% and 40% RAP were certified with the Bronze category respectively. The conventional pavement section was found to be unsustainable.

2. Economic analysis of the research case studies considered in accordance with the life cycle cost assessment (LCCA) methodology demonstrated that the cost benefit ratio associated with HMA mixtures utilizing recycled materials as opposed to virgin materials is significant. However, variation and cost effectiveness between competing alternatives is dependent on the discount rates applied.

3. The Rubberized-RAP HMA was observed to be the least expensive option over the 20 year period of analysis, but the major disincentive to using it is the initial construction cost. It was noted that initial cost of constructing with this type of mix is certain to decrease if crumb rubber is extensively used.

4. The analysis using the PaLATE sustainability assessment tool was found to be in agreement with many previous studies which conluded that emissions from the production of rubber modified asphalt concrete are not significantly different than those from the production of conventional dense-graded asphalt concrete mixtures. This was considered to be both encouraging and unfavourable to the green pavements objective. However, the capability of the PaLATE tool could be improved for better reliability.

Given the performance outcome of the evaluated HMA mixtures in this study, it is considered reasonable and practical to conclude that RAP and CRM are valuable components of typical Ontario Superpave HMA mixtures. Increasing the RAP percentage in typical Ontario Superpave HMA is feasible and environmentally sustainable, but performance is better optimized in combination with CRM using either methods of the wet-process. This type of HMA mixtures exhibit the potential to resist low-temperature cracking and the combined effects of rutting stripping and moisture damage if properly and efficiently designed, mixed and compacted. By encouraging these innovative initiatives, the Ministry of Transportation Ontario's (MTO) goal to have one of the greenest roads in North America is realizable.

7.2 Recommendations

On the basis of the findings of this study; the need to encourage the inclusion of RAP content as high as 40 percent in typical Ontario SuperPave HMA, especially for use on low-volume roads is strongly encouraged.

For contractors or agencies looking to be more sustainable, results from this study further recommends in strong terms the innovative use of RAP in combination with CRM. For such mix designs, the following must be taken into consideration:

1. The minimum Superpave consensus volumetric and source properties should be stringently adhered to. In this study, the coarse aggregate angularity was observed to decrease with increasing RAP percentage. The potential shortfall in interlocking bond between the aggregate structures were addressed by the fine aggregate angularity portion and dust proportion which

contributed to forming a good mastic with the binder, thus providing good contact for mix stability. The sand equivalency in the mix also provided a good bond between the binder and aggregate since the RAP mixtures had less proportion of clay-like materials. In particular, the low temperature and fatigue properties of the recycled HMA mixtures in the laboratory-prepared PG 52-34, 58-34 and 52-40 category could be further improved by increasing the virgin asphalt binder content by up to 0.5% - 1%. Alternatively, such mixtures could be improved with a good RAP-binder rejuvenating agent, or 0.5% – 1% CRM by weight of the mixtures.

2. It is recommended that CRM used in typical Ontario rubberized mixtures be subjected to both cryogenic and ambient methods of grinding. This could be a more effective way of ensuring better or comparable performance with conventional mixtures. The implication of this would be higher initial construction costs, but the many benefits associated with rubberized pavements including its elongated service life would provide a trade-off over the pavement's lifecycle; especially in terms of maintenance or the need to carry out major rehabilitation. On this note, the MTO appears to be on the right track, and should continue its incentive program that encourages the demand and use of crumb rubber in HMA mixtures.
3. It has been almost Four (4) years since these trial sections were placed. Field monitoring results appear to be consistent with the laboratory performance evaluations. However, it is strongly recommended that Ontario's Highways 7, 35 and 115 be continually monitored for crack evaluation and other distress types to establish longer-term performance as well as validate the laboratory performance findings.
4. Non-destructive tests should also be conducted on these sections since such tests will reveal additional benefits of using crumb rubber modified asphalt HMA mixtures.

7.3 Future Research

Based on work done in this thesis, the following are possible areas for future studies that would be beneficial to the use of RAP and CRM in HMA Mixtures:

1. The inadequacy of the Superpave G*/sin (δ) parameter in predicting the rutting performance and low temperature true grade of some of the evaluated mixtures have been recognized in this study. Alternative test methods in capturing contribution of asphalt binder incorporating CRM and RAP is an area worthy of consideration for future studies. This can be done using the Multiple Stress Creep Recovery (MSCR) test, performed in accordance with AASHTO TP 70-

12, “The Multiple Stress Creep Recovery (MSCR) Test for Asphalt Binder Using a Dynamic Shear Rheometer (DSR)”.

2. The flexural fatigue properties of the HMA mixtures should be evaluated for future studies. Mix stiffness and binder characterization tests from this study suggest that interesting fatigue life results could be found.
3. Future studies can equally consider experimenting with HMA mix incorporating 40% RAP and 10% - 20% CRM using either methods of the wet-process rubber blend for various binder grades used in Ontario.
4. Note that dynamic modulus is also a required flexible pavement design parameter in Level 1 of the Mechanistic-Empirical Pavement Design Guide (MEPDG). The MEPDG is a pavement design strategy that calculates pavement responses (stress, strain and deflection), and uses same to compute incremental damage overtime. The level 1 analysis of this tool could thus be used in conjunction with the measured dynamic modulus, recovered binder, rutting, and low temperature characteristic data to validate the need for a modified dynamic modulus predictive equation; particularly for crumb rubber modified HMA mixtures, and an overall assessment of distress propagation and structural performance of the various HMA mix configurations. These analysis could be beneficial to addressing the many issues and limitations associated with the localized implementation of the MEPDG.
5. This study showed significant reinforcing and viscosity improvements in the rheological properties of the recovered Rubberized-RAP binders. Whereas this observation validates the research hypothesis that CRM compensates for the shortfalls and binder performance grade differences in RAP mixtures, it does not quantify the interaction and particle effects of CRM in the rubberized binder. For these reasons, it may be interesting to pursue studies that can quantify and analyze the chemical effects of CRM on the relaxation, viscosity, and diffusion properties of rubberized binders and mixtures incorporating RAP. Such studies could provide better clarifications to the mix behavioral observations; particularly those of low temperature thermal cracking of the Rubberized-RAP HMA mixtures in this study.

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Appendix A

Low Temperature: TSRST Fracture Results

Appendix A-1: Fracture Stress Results for PG 58-28 HMA Mixtures

Details	PG 58-28							
	L1	**L2**	**L3**	**H7C**	**H7RFB**	**H7RTB**	**H35C**	**H35RFB**
B1	2.05	2.68	2.66	2.33	2.47	2.3	2.89	2.27
B2	2.38	2.55	1.95	2.57	2.34	2.29	3.1	2.06
B3	1.64	2.71	2.21	2.08	2.27	2.54	2.89	1.92
B4	2.09	1.05	2.29	2.4	1.79	2.22	2.86	2.36
Average	2.04	2.25	2.28	2.35	2.22	2.34	2.94	2.15
Standard Deviation	0.3	0.8	0.29	0.2	0.3	0.14	0.11	0.2
Coefficient of Variation	14.7	35.6	12.7	8.5	13.5	6	3.7	9.3
Variance	0.09	0.64	0.09	0.04	0.09	0.02	0.01	0.04

Appendix A-2: Fracture Stress Results for HMA Mixtures with PGAC 58-34, 52-34 and 52-40

Details	PG 52-34			PG 58-34			PG 52-40		
	L4	**L5**	**L6**	**L7**	**L8**	**L9**	**L10**	**L11**	**L12**
B1	1.56	3.05	2.59	1.19	2.86	2.29	2.61	3.15	3.02
B2	1.31	1.69	2.55	1.34	2.3	2.66	3.13	2.93	3.16
B3	2.18	2.58	2.57	2.53	2.79	1.95	3.07	2.29	2.59
B4	1.86	3.16	2.54	3.47	2.33	2.21	2.97	1.99	3.26
Average	1.73	2.62	2.56	2.13	2.57	2.28	2.95	2.59	3.01
Standard Deviation	0.38	0.67	0.02	1.07	0.3	0.29	0.23	0.54	0.3
Coefficient of Variation	22	25.6	0.8	50.2	11.7	12.7	7.8	20.8	10
Variance	0.14	0.45	0.00	1.15	0.09	0.09	0.05	0.29	0.09

Appendix A-3: Fracture Stress Results for HMA Mixtures with PGAC 64-34(P)

Details	PG 64-34	PG 64-34P
	HII5C	H115RFB
B1	3.16	2.56
B2	2.64	1.98
B3	3.21	2.54
B4	2.6	1.62
Average	2.15	2.18
Standard Deviation	0.33	0.46
Coefficient of Variation	15.3	21.1
Variance	0.11	0.21

Appendix A-4: Fracture Temperature Results for PG 58-28 HMA Mixtures

Details	PG 58-28							
	L1	L2	L3	H7C	H7RFB	H7RTB	H35C	H35RFB
B1	-29.8	-27.1	-30.8	-32.4	-31.5	-36.5	-26.7	-34.1
B2	-30.6	-27.7	-30.9	-33.8	-30.8	-43.6	-32.9	-30.9
B3	-27.5	-28.7	-28.6	-33.9	-34.2	-43.6	-32.2	-31.6
B4	-30.3	-27.5	-27.10	-31.2	-32.3	-35.7	-26.9	-31.1
Average	-29.6	-27.8	-29.4	-32.8	-32.2	-39.9	-29.7	-31.9
Standard Deviation	1.41	0.68	1.84	1.28	1.47	4.34	3.33	1.48
Coefficient of Variation	-4.8	-2.4	-6.3	-3.9	-4.6	-10.9	-11.2	-4.6
Variance	1.98	0.46	3.38	1.64	2.15	18.9	11.1	2.19
PG Difference	-1.6	0.2	4.6	-4.8	-4.2	-11.9	4.3	2.1

Appendix A-5: Fracture Temperature Results for HMA Mixtures with PGAC 58-34, 52-34 and 52-40

Details	PG 58-34			PG 52-34			PG 52-40		
	L4	L5	L6	L7	L8	L9	L10	L11	L12
B1	-33.8	-32	-28	-26.1	-33.9	-27.1	-42.7	-34.9	-31.7
B2	-33.9	-23	-25.3	-26.5	-34.1	-30.8	-44.5	-33.2	-33.3
B3	-34.1	-32	-28.3	-35.4	-32.7	-30.9	-43.1	-29.3	-30.5
B4	-34	-33	-27.90	-34.4	-33.9	-28.6	-42.4	-31.6	-32.7
Average	-34	-30	-27.4	-30.6	-33.7	-29.4	-43.2	-32.3	-32.1
Standard Deviation	0.13	4.69	1.39	4.98	0.64	1.84	0.93	2.38	1.23
Coefficient of Variation	-0.4	-15.6	-5.1	-16.3	-1.9	-6.3	-2.2	-7.4	-3.8
Variance	0.02	22	1.94	24.85	0.41	3.38	0.86	5.68	1.5
PG Difference	-6			-2			6.6		

Appendix A-6: Fracture Temperature Results for PG 64-34(P) HMA Mixtures

Details	PG 64-34	PG 64-34P
	HII5C	H115RFB
B1	-31.9	-42.6
B2	-33.1	-33.3
B3	-33.9	-43.7
B4	-30.2	-30.1
Average	-32.3	-37.4
Standard Deviation	1.61	6.75
Coefficient of Variation	-5	-18
Variance	2.59	45.61
PG Difference	-4.3	-3.4

Appendix B

HWTD: Rutting Test Results

Appendix B-1: Rut Results for Evaluated HMA Mixtures

Mix Type	Max. Rut Depth (mm)		Air Void		Post-Compaction Consolidation after 1,000 Cycles (mm)		Creep Slope	
	Test 1	Test 2	Test 1	Test 2	Test 1	Test 2	Test 1	Test 2
L1:0-58-28	3.4	3.08	7.8	7.2	2.02	2.31	7.17E-05	6.40E-05
L2:58-28+20%RAP	4.84	3.28	7.7	7.6	3.15	2.00	1.20E-04	1.00E-04
L3:58-28+40%RAP	2.88	4.18	6.4	7.8	2.3	2.51	1.67E-06	5.60E-05
H7C:58-28+15%RAP	3.38	3.38	6.8	6.9	2.26	2.44	5.00E-05	8.83E-05
H35C:58-28+20%RAP	2.93	2.98	7.5	7.2	1.70	1.66	1.17E-05	5.71E-05
H7RTB:58-28+20%RAP	1.85	2.66	7.5	7.7	1.23	1.38	5.00E-06	2.67E-06
H7RFB:58-28+20%RAP	2.04	2.39	6.5	7.1	1.46	1.84	3.83E-05	6.67E-06
H35RFB:58-28+20%RAP	2.42	2.64	6.6	6.3	1.87	1.96	0.00E+00	1.39E-05
L7:0-58-34	3.31	4.87	6.4	6.6	2.38	2.19	7.17E-05	1.30E-04
L8:58-34+20%R	6.93	6.37	7.5	6.9	3.77	3.62	1.95E-04	5.67E-05
L9:58-34+40%R	3.25	4.35	6.0	6.4	1.88	2.18	2.95E-05	1.35E-04
L4:0-52-34	5.11	6.55	7.9	6.6	2.62	3.37	2.00E-04	1.63E-04
L5:52-34+20%R	6.35	5.64	7.2	8.8	3.16	2.61	2.18E-04	2.60E-04
L6:52-34+40%R	4.04	3.2	6.2	6.4	2.02	2.15	2.00E-05	1.00E-04
L10:0-52-40	3.66	3.83	6.4	6.4	1.71	2.71	9.67E-05	4.00E-05
L11:52-40P+20%R	3.5	3.22	6.6	6.7	2.05	2.05	4.80E-05	9.50E-05
L12:52-40P+40%R	2.84	2.35	7.2	7.8	1.7	1.39	7.67E-05	5.00E-05
H115C:64-34+20%R	2.31	1.90	7.3	7.9	1.56	1.44	4.50E-05	1.60E-05
H115RFB:64-34P+20%R	1.69	2.66	6.0	6.9	1.01	1.75	2.00E-06	3.25E-05

Appendix B-2: Rut Results for Extracted Cores

Trial Section	Max. Rut Depth (mm)		Air Void		Post-Compaction Consolidation after 1,000 Cycles (mm)		Creep Slope	
	Test 1	Test 2	Test 1	Test 2	Test 1	Test 2	Test 1	Test 2
Control	7.96	7.14	6.4	6.4	3.92	3.69	2.22E-04	1.25E-04
Ambient Rubber	5.71	7.73	7.7	6.0	1.31	3.35	1.38E-04	1.03E-04
Cryogenic Rubber	4.71	4.97	5.0	5.0	1.34	1.41	1.95E-04	2.30E-04

Appendix C

Average Dynamic Modulus and Phase Angle Results

Appendix C-1: Results for PG 58-28 with 0, 20 and 40% RAP

Temperature (°C)	Frequency (Hz)	L1:0-58-28		L2:20-58-28		L3:0-58-28	
		E*	δ	E*	δ	E*	δ
-10	0.1	21359.197	8.43	23037.595	7.84	23470.54	8.17
	0.5	24731.952	7.93	26452.05	7.20	27000.089	7.17
	1	26212.557	8.38	27958.69	7.67	28553.122	7.41
	5	29586.688	6.67	31331.885	6.02	32002.147	5.96
	10	30999.552	6.09	32745.05	5.45	33346.834	5.42
	25	32801.065	5.56	34575.281	4.98	35117.178	5.11
4	0.1	9930.2171	10.82	11134.878	10.23	12400.642	10.81
	0.5	12561.549	12.22	13980.83	11.45	15293.924	11.26
	1	13808.464	16.97	15370.579	16.06	16709.01	14.40
	5	17412.385	12.63	19092.937	11.92	20356.04	11.00
	10	19021.646	11.49	20746.608	10.63	21990.456	9.93
	25	20903.336	10.84	22731.369	10.11	24018.663	9.44
21	0.1	3726.7292	9.47	4038.6524	9.16	3629.356	8.85
	0.5	4874.8065	13.81	5289.9938	13.66	4718.7384	13.40
	1	5280.1877	27.89	5775.6887	27.06	5152.2742	26.08
	5	7612.6438	21.02	8246.3148	20.43	7256.5276	19.96
	10	8850.8552	19.05	9531.383	18.31	8362.2226	18.58
	25	9588.2704	18.85	10511.548	17.81	9304.309	18.19
37	0.1	1418.5553	8.33	1569.326	8.65	1812.225	9.28
	0.5	1824.9674	13.43	2022.2642	13.57	2376.4301	14.14
	1	1967.0859	25.81	2186.45	26.58	2573.7846	26.49
	5	2851.4604	22.58	3205.1286	23.13	3848.274	22.77
	10	3521.6025	22.14	3936.1514	22.89	4606.3399	22.71
	25	3940.4541	20.69	4410.4384	21.94	5106.7082	21.05
54	0.1	496.88987	9.98	523.75088	9.02	595.33881	12.67
	0.5	627.88558	13.09	658.65	11.14	700.557	14.98
	1	679.05993	21.46	683.37221	21.90	772.41455	22.65
	5	1007.0091	19.18	1009.5401	19.83	1181.3071	21.28
	10	1259.3332	19.52	1262.021	20.75	1507.454	22.35
	25	1403.8836	21.65	1328.8664	27.23	1662.4334	28.27

Appendix C-2: Results for PG 52-34 with 0, 20 and 40% RAP

Temperature (°C)	Frequency (Hz)	L4:0-52-34		L5:20-52-34		L6:40-52-34	
		E*	δ	E*	δ	E*	δ
-10	0.1	16972.652	10.15	18403.479	9.21	20058.868	9.50
	0.5	20741.211	10.23	21985.414	9.13	24024.479	9.23
	1	22520.868	12.24	23645.5	10.67	25814.415	10.54
	5	26640.584	9.15	27534.394	8.13	29824.914	8.11
	10	28404.938	8.16	29232.139	7.27	31509.318	7.33
	25	30617.143	7.34	31228.788	6.65	33350.89	6.69
4	0.1	7026.3922	9.38	8145.3025	10.59	8415.4421	9.70
	0.5	8917.1334	12.70	10343.95	12.86	10708.382	12.34
	1	9782.431	22.50	11410.835	20.19	11814.124	20.07
	5	13066.104	16.44	14964.078	14.95	15352.908	14.52
	10	14573.645	14.63	16540.969	13.23	16901.461	12.81
	25	16017.689	14.48	18496.368	13.43	18636.285	12.53
21	0.1	2648.5875	3.76	3091.2123	6.67	3335.0323	8.27
	0.5	3424.6438	11.09	3971.1878	12.58	4251.183	13.16
	1	3688.4744	28.26	4298.6519	27.01	4597.8926	26.61
	5	5231.2009	21.53	6114.2886	21.02	6473.7812	20.60
	10	6132.8162	19.66	7131.0205	19.16	7490.3335	19.13
	25	6786.8	21.23	8018.8177	20.57	8350.7683	20.76
37	0.1	1063.0078	2.61	1333.1785	7.83	1434.0506	4.55
	0.5	1396.6633	9.16	1643.9896	11.44	1785.2359	11.09
	1	1479.3774	23.42	1768.8939	22.19	1910.9053	24.37
	5	2027.2302	19.44	2428.9841	19.62	2675.5732	21.07
	10	2439.5439	19.32	2942.4643	19.37	3242.8789	20.71
	25	2868.3298	22.50	3388.717	20.96	3595.8086	21.40
54	0.1	344.43233	5.73	483.15896	5.73	426.05112	6.90
	0.5	465.35784	9.81	612.2519	10.03	539.66618	11.36
	1	479.73053	22.22	605.98913	24.46	574.86256	23.78
	5	711.06342	17.80	926.1734	18.19	864.0193	20.43
	10	897.56091	18.59	1042.3154	20.63	1107.8786	22.13
	25	907.5262	23.58	972.75584	27.18	1159.7308	31.55

Appendix C-3: Results for PG 58-34 with 0, 20 and 40% RAP

Temperature (°C)	Frequency (Hz)	L7:0-58-34		L8:20-58-34		L9:40-58-34	
		E*	δ	E*	δ	E*	δ
-10	0.1	17773.78	10.16	20390.94	9.21	20396.589	9.47
	0.5	21547.976	9.98	24157.281	8.72	24247.012	8.96
	1	23314.135	11.49	25930.803	9.75	26001.436	9.82
	5	27332.426	8.76	29988.938	7.61	29897.642	7.72
	10	29074.436	7.77	31675.455	6.88	31641.999	7.01
	25	31237.092	7.20	34003.96	6.26	33794.611	6.44
4	0.1	7425.5826	9.96	9896.228	11.13	8966.6889	11.04
	0.5	9415.7247	12.82	12384.118	12.54	11426.161	12.62
	1	10389.702	20.75	13598.115	17.69	12661.284	18.05
	5	13652.185	15.27	17301.31	13.47	16217.077	13.47
	10	15110.812	13.52	18981.117	12.14	17859.053	12.02
	25	16706.298	13.36	21000.245	11.70	20005.452	11.85
21	0.1	3132.3366	8.24	3997.7511	11.27	3580.7347	9.48
	0.5	4013.1856	12.95	5075.7917	14.25	4619.5516	13.78
	1	4361.1292	26.28	5541.7683	23.50	5035.0071	26.57
	5	6126.8241	20.58	7644.8536	19.02	7150.7473	19.76
	10	7088.0995	19.20	8738.4378	18.41	8169.0112	18.55
	25	8000.6506	20.18	9781.7303	18.18	9256.3386	20.16
37	0.1	1343.6201	7.86	1748.7739	10.75	1513.3335	7.67
	0.5	1731.3745	10.46	2181.7943	13.89	1896.1895	12.35
	1	1851.7149	23.34	2388.3247	21.65	2053.7615	24.12
	5	2541.3899	20.08	3331.1096	19.85	2903.8282	20.09
	10	3056.7663	18.85	3994.1286	19.71	3491.8149	20.46
	25	3406.1701	21.84	4752.6651	23.15	3959.8129	21.51
54	0.1	506.62128	8.62	614.03365	11.49	530.3598	10.11
	0.5	630.85604	12.09	763.94218	14.04	646.83474	12.44
	1	672.79775	21.28	836.40362	20.04	686.8596	20.62
	5	972.93999	18.19	1243.08	18.14	983.11321	18.81
	10	1190.1336	19.14	1553.4979	18.79	1209.2962	19.35
	25	1359.0952	26.11	1794.6013	25.60	1332.7863	20.28

Appendix C-4: Results for PG 52-40 with 0, 20 and 40% RAP

Temperature	Frequency	L10:0-52-40		L11:20-52-40		L12:40-52-40	
		E*	δ	E*	δ	E*	δ
-10	0.1	12394.391	9.25	23183.702	7.73	18983.304	8.72
	0.5	15418.716	10.88	26833.611	7.50	22500.524	8.45
	1	16936.26	15.85	28515.359	8.29	24078.261	9.58
	5	20826.423	11.37	32300.749	6.58	27746.667	7.53
	10	22619.326	10.00	33944.703	6.02	29430.101	6.88
	25	24712.087	9.16	36014.444	5.58	31620.21	6.37
4	0.1	5609.5384	8.67	11508.749	10.51	9815.7781	10.79
	0.5	6994.703	12.49	14091.956	11.41	12203.831	11.41
	1	7601.5706	23.06	15381.987	15.14	13386.863	15.30
	5	10088.544	17.22	18908.796	11.74	16540.16	11.81
	10	11267.837	15.77	20528.781	10.88	18122.718	10.87
	25	12439.581	16.49	22573.464	10.50	20258.307	10.63
21	0.1	2568.5646	5.95	5179.3925	12.49	4293.4657	12.14
	0.5	3201.3433	11.02	6529.574	14.21	5353.2264	14.50
	1	3450.4724	23.15	7193.5102	19.07	5847.8815	22.00
	5	4562.5154	18.41	9442.8858	15.83	7863.0274	17.37
	10	5222.2501	17.38	10561.461	15.06	8881.3789	15.84
	25	5969.1105	18.46	11913.04	15.31	9833.3912	15.30
37	0.1	1282.163	3.86	2189.8389	11.36	1848.92	12.22
	0.5	1572.009	8.58	2729.1222	14.18	2312.4274	15.15
	1	1666.4939	19.93	3000.5004	19.92	2504.743	23.78
	5	2139.2316	16.05	4103.9288	18.37	3545.6061	20.35
	10	2455.1082	15.94	4796.0338	18.24	4151.084	19.89
	25	2793.6143	17.34	5503.5746	20.26	4560.8284	21.88
54	0.1	398.86447	7.64	888.75695	10.99	673.69032	11.36
	0.5	547.72787	9.76	1082.6692	13.43	837.41732	14.73
	1	590.938	22.74	1175.1545	19.22	898.33204	23.01
	5	852.12752	17.06	1675.4949	17.17	1323.6166	20.91
	10	1056.1846	17.21	2049.4015	17.53	1645.1924	21.24
	25	1008.1599	12.19	2197.5705	22.66	1598.3008	21.29

Appendix C-5: Results for Highway 115 and 35 Plant HMA Mixtures

Temperature (°C)	Frequency (Hz)	H115C:20-64-34		H115RFB:20-64-34P	
		E*	δ	E*	δ
-10	0.1	17493.938	9.35	19595.943	8.43
	0.5	20716.906	8.82	22878.409	8.08
	1	22254.125	9.83	24427.433	8.95
	5	25695.492	7.88	27874.104	7.21
	10	27278.032	7.31	29387.789	6.62
	25	29358.806	6.94	31409.777	6.17
4	0.1	9329.7901	11.76	9782.6011	11.66
	0.5	11694.178	12.55	12165.076	12.23
	1	12848.218	16.73	13351.649	15.95
	5	16201.473	12.96	16624.801	12.43
	10	17730.351	11.94	18153.531	11.39
	25	19652.93	11.49	20108.372	11.10
21	0.1	3711.1988	13.17	4177.7381	11.46
	0.5	4739.3721	15.23	5342.0188	7.86
	1	5219.6961	22.29	5859.8064	11.64
	5	7136.6233	18.26	7995.6004	11.63
	10	8086.3276	17.15	9121.0411	13.57
	25	9078.3349	17.68	10367.652	21.34
37	0.1	1514.0623	11.57	1892.1341	16.22
	0.5	1829.6277	14.31	2379.8606	7.40
	1	2004.9439	20.37	2582.8069	10.68
	5	2714.3493	18.91	3699.2963	11.95
	10	3194.3999	18.64	4341.1305	14.32
	25	3597.3641	20.22	4623.6418	22.61
54	0.1	626.41403	11.87	716.49529	14.03
	0.5	742.92579	13.50	853.80805	15.10
	1	798.93424	19.48	929.42024	19.43
	5	1134.8425	16.98	1335.3309	17.83
	10	1364.4455	18.27	1590.5944	18.85
	25	1561.2032	17.70	1707.1734	21.27

Appendix C-6: Results for Highway 7 Plant HMA Mixtures

Temperature (°C)	Frequency (Hz)	H7C:15-58-28		H7RTB:20-58-28		H7RFB:20-58-28	
		E*	δ	E*	δ	E*	δ
-10	0.1	18100.38	8.50	11496.168	8.32	18317.668	8.40
	0.5	21413.85	8.25	13851.801	9.27	21344.794	7.70
	1	22911.83	9.32	14978.914	12.33	22683.461	8.22
	5	26370.71	7.24	17665.012	9.37	25686.431	6.55
	10	27910.84	6.52	19008.781	8.46	26971.328	5.95
	25	29883.59	6.04	20705.037	7.88	28693.16	5.47
4	0.1	8132.12	11.41	5403.4135	9.88	8907.1233	10.90
	0.5	10332.81	12.76	6597.6337	12.29	11210.575	12.00
	1	11424.92	18.68	7167.8082	19.33	12350.869	16.07
	5	14666.48	13.97	9053.6988	14.68	15430.877	12.20
	10	16168.81	12.42	9915.6094	13.50	16844.223	11.20
	25	17900.71	11.84	10984.033	13.72	18623.706	10.49
21	0.1	3087.10	11.52	2923.1147	11.16	3376.6648	8.43
	0.5	4003.70	14.80	3612.0063	13.54	4449.7059	13.71
	1	4418.45	23.39	3939.4333	20.80	4809.9663	28.12
	5	6124.59	19.09	5150.4318	16.52	6925.7895	20.64
	10	7022.02	18.00	5757.4547	15.47	7996.9541	18.43
	25	7909.52	18.57	6298.7291	15.78	9012.7603	19.44
37	0.1	1279.90	8.14	1396.2924	10.80	1394.8678	8.70
	0.5	1600.95	12.64	1709.2196	13.69	1852.197	13.97
	1	1738.02	22.04	1839.2717	21.28	1955.1349	30.87
	5	2426.93	19.50	2482.7593	17.70	3017.0839	24.87
	10	2894.17	19.56	2864.45	17.35	3735.7213	23.71
	25	3402.99	21.86	3108.1082	17.00	3937.6425	28.59
54	0.1	417.09	13.50	462.7013	12.19	513.06691	12.05
	0.5	529.89	14.38	567.0322	14.38	649.31674	14.87
	1	573.33	20.71	606.92872	20.50	676.01354	25.98
	5	858.81	18.41	888.06265	17.28	1065.0248	23.28
	10	1077.20	20.12	1036.5845	17.93	1386.6037	26.50
	25	1112.85	26.92	1018.2721	15.80	1156.4842	13.03

Appendix C-7: Results for Highway 35 Plant HMA Mixtures

Temperature (°C)	Frequency (Hz)	H35C:20-58-28		H35RFB:20-58-28	
		E*	δ	E*	δ
-10	0.1	23379.43	7.85	17511.926	8.36
	0.5	27127.577	7.15	20413.08	8.01
	1	28751.097	7.51	21764.422	8.82
	5	32362.544	5.99	24778.138	7.00
	10	33911.14	5.44	26040.465	6.42
	25	35735.925	4.99	27772.239	5.86
4	0.1	11375.076	10.81	8468.2716	10.41
	0.5	14381.392	11.83	10719.212	12.07
	1	15996.95	15.48	11848.51	16.74
	5	19790.123	11.86	14938.309	12.44
	10	21604.058	10.77	16305.197	11.33
	25	23881.282	10.30	18082.557	10.73
21	0.1	3962.4008	13.82	3296.4314	6.39
	0.5	5198.9472	15.95	4317.7505	12.84
	1	5778.4509	23.32	4695.0433	28.03
	5	8043.1777	18.87	6680.5576	20.16
	10	9199.2316	17.30	7649.5269	17.93
	25	10294.109	18.63	8426.6933	17.94
37	0.1	1489.8592	11.15	1297.4076	5.88
	0.5	1914.5068	15.25	1746.3825	12.84
	1	2117.7999	23.56	1817.8184	32.66
	5	3077.0225	21.45	2811.5506	25.30
	10	3693.3302	21.85	3327.0318	23.03
	25	4291.6287	22.06	3480.7608	24.02
54	0.1	497.68985	12.14	469.79306	8.42
	0.5	625.27102	14.84	607.76127	12.81
	1	682.70949	21.21	612.05362	27.10
	5	1034.9858	19.83	975.03526	24.01
	10	1317.9854	21.37	1229.6444	29.51
	25	1529.7306	19.48	889.21432	11.92

Appendix D

LCCA Economic Computations

Appendix D-1: Life Cycle Cost at 3% Discount Rate – Case A

Case A - 20:20 Rubberized-RAP HMA							3%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
Superpave 12.5 mm FC1 (20% RAP + 20% CRM)	0				1	$ 835,866.00	1.000	$ 835,866.00			
Rout and Crack Sealing (200 m/km)	6	$5.00	200	$1,000	1	$ 1,000.00	0.837	$ 837.48	$ 836,567.38	$5,147,608.24	$ 1,538.86
Rout and Crack Sealing (200 m/km)	12	$5.00	200	$1,000	1	$ 1,000.00	0.701	$ 701.38	$ 836,357.93	$2,800,742.07	
5% Mill and patch 40 mm	20	$35.00	750	$26,250	1	$ 26,250.00	0.554	$ 14,533.99			
Salvage Value	20					-$167,173.20	0.554	**-$ 92,559.75**			
								$ 759,379.11			

Appendix D-1: Life Cycle Cost at 3% Discount Rate – Case B

Case B - 20% RAP HMA							3%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
SP 12.5 mm FC1 (20% RAP)	0				1	$ 772,362.00	1.000	$ 772,362.00			
Rout and Crack Sealing (200 m/km)	3	$5.00	200	$1,000	1	$ 1,000.00	0.915	$ 915.14	$ 773,199.48	$9,111,649.82	$ 105,764.67
5% Mill and patch 40 mm	8	$35.00	750	$26,250	1	$ 26,250.00	0.789	$ 20,721.99	$ 788,720.13	$3,745,274.06	
20% Mill and patch 40 mm	16	$45.00	3000	$135,000	1	$ 135,000.00	0.623	$ 84,127.54	$ 824,787.50	$2,188,734.44	
Salvage Value	20					-$154,472.40	0.554	**-$ 85,527.62**			
								$ 792,599.05			

Appendix D-1: Life Cycle Cost at 3% Discount Rate – Case C

Case C - 40% RAP HMA							3%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
SP 12.5 mm FC1 (40% RAP)	0				1	$ 755,730.00	1.000	$ 755,730.00			
Rout and Crack Sealing (200 m/km)	3	$5.00	200.00	$1,000	1	$ 1,000.00	0.915	$ 915.14	$ 756,567.48	$8,915,652.59	$ 105,764.67
5% Mill and patch 40 mm	8	$35.00	750.00	$26,250	1	$ 26,250.00	0.789	$ 20,721.99	$ 772,088.13	$3,666,296.24	
20% Mill and Patch 40mm	16	$45.00	3000.00	$135,000	1	$ 135,000.00	0.623	$ 84,127.54	$ 808,155.50	$2,144,598.19	
Salvage Value	20					-$151,146.00	0.554	**-$ 83,685.88**			
								$ 777,808.80			

Appendix D-1: Life Cycle Cost at 3% Discount Rate – Case D

Case D - Control HMA							3%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
SP 12.5 mm FC1	0				1	$ 787,482.00	1.000	$ 787,482.00			
Rout and Crack Sealing (200 m/km)	3	$5.00	200	$1,000	1	$ 1,000.00	0.915	$ 915.14	$ 788,319.48	$9,289,829.12	$ 105,764.67
5% Mill and patch 40 mm	8	$35.00	750	$26,250	1	$ 26,250.00	0.789	$ 20,721.99	$ 803,840.13	$3,817,072.08	
20% Mill and patch 40 mm	16	$45.00	3000	$135,000	1	$ 135,000.00	0.623	$ 84,127.54	$ 839,907.50	$2,228,858.31	
Salvage Value	20					-$157,496.40	0.554	**-$ 87,201.94**			
								$ 806,044.73			

Appendix D-2: Life Cycle Cost at 4% Discount Rate – Case A

Case A - 20:20 Rubberized-RAP HMA							4%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
Superpave 12.5 mm FC1 (20% RAP + 20% CRM)	0				1	$ 835,866.00	1.000	$ 835,866.00			
Rout and Crack Sealing (200 m/km)	6	$5.00	200	$1,000	1	$ 1,000.00	0.790	$ 790.31	$ 836,490.60	$3,989,263.44	$ 1,414.91
Rout and Crack Sealing (200 m/km)	12	$5.00	200	$1,000	1	$ 1,000.00	0.625	$ 624.60	$ 836,256.12	$2,227,622.67	
5% Mill and patch 40 mm	20	$35.00	750	$26,250	1	$ 26,250.00	0.456	$ 11,980.16			
Salvage Value	20					-$167,173.20	0.456	**-$ 76,295.67**			
								$ 772,965.40			

Appendix D-2: Life Cycle Cost at 4% Discount Rate – Case B

Case B - 20% RAP HMA							4%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
SP 12.5 mm FC1 (20% RAP)	0				1	$ 772,362.00	1.000	$ 772,362.00			
Rout and Crack Sealing (200 m/km)	3	$5.00	200	$1,000	1	$ 1,000.00	0.889	$ 889.00	$ 773,152.31	$6,965,107.68	$ 92,147.22
5% Mill and patch 40 mm	8	$35.00	750	$26,250	1	$ 26,250.00	0.731	$ 19,180.62	$ 786,377.09	$2,919,972.11	
20% Mill and patch 40 mm	16	$45.00	3000	$135,000	1	$ 135,000.00	0.534	$ 72,077.60	$ 810,844.82	$1,739,667.55	
Salvage Value	20					-$154,472.40	0.456	**-$ 70,499.19**			
								$ 794,010.03			

Appendix D-2: Life Cycle Cost at 4% Discount Rate – Case C

Case C - 40% RAP HMA							4%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
SP 12.5 mm FC1 (40% RAP)	0				1	$ 755,730.00	1.000	$ 755,730.00			
Rout and Crack Sealing (200 m/km)	3	$5.00	200.00	$1,000	1	$ 1,000.00	0.889	$ 889.00	$ 756,520.31	$6,815,274.76	$ 92,147.22
5% Mill and patch 40 mm	8	$35.00	750.00	$26,250	1	$ 26,250.00	0.731	$ 19,180.62	$ 769,745.09	$2,858,214.23	
20% Mill and Patch 40mm	16	$45.00	3000.00	$135,000	1	$ 135,000.00	0.534	$ 72,077.60	$ 794,212.82	$1,703,983.59	
Salvage Value	20					-$151,146.00	0.456	**-$ 68,981.06**			
								$ 778,896.16			

Appendix D-2: Life Cycle Cost at 4% Discount Rate – Case D

Case D - Control HMA							4%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
SP 12.5 mm FC1	0				1	$ 787,482.00	1.000	$ 787,482.00			
Rout and Crack Sealing (200 m/km)	3	$5.00	200	$1,000	1	$ 1,000.00	0.889	$ 889.00	$ 788,272.31	$7,101,319.43	$ 92,147.22
5% Mill and patch 40 mm	8	$35.00	750	$26,250	1	$ 26,250.00	0.731	$ 19,180.62	$ 801,497.09	$2,976,115.63	
20% Mill and patch 40 mm	16	$45.00	3000	$135,000	1	$ 135,000.00	0.534	$ 72,077.60	$ 825,964.82	$1,772,107.51	
Salvage Value	20					-$157,496.40	0.456	**-$ 71,879.30**			
								$ 807,749.92			

Appendix D-3: Life Cycle Cost at 5% Discount Rate – Case A

Case A - 20:20 Rubberized-RAP HMA							5%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
Superpave 12.5 mm FC1 (20% RAP + 20% CRM)	0				1	$ 835,866.00	1.000	$ 835,866.00			
Rout and Crack Sealing (200 m/km)	6	$5.00	200	$1,000	1	$ 1,000.00	0.746	$ 746.22	$ 836,422.84	$3,295,798.19	$ 1,303.05
Rout and Crack Sealing (200 m/km)	12	$5.00	200	$1,000	1	$ 1,000.00	0.557	$ 556.84	$ 836,176.07	$1,886,838.15	
5% Mill and patch 40 mm	20	$35.00	750	$26,250	1	$ 26,250.00	0.377	$ 9,893.35			
Salvage Value	20					-$167,173.20	0.377	**-$ 63,005.82**			
								$ 784,056.58			

Appendix D-3: Life Cycle Cost at 5% Discount Rate – Case B

Case B - 20% RAP HMA							5%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
SP 12.5 mm FC1 (20% RAP)	0				1	$ 772,362.00	1.000	$ 772,362.00			
Rout and Crack Sealing (200 m/km)	3	$5.00	200	$1,000	1	$ 1,000.00	0.864	$ 863.84	$ 773,108.22	$5,677,839.16	$ 80,475.93
5% Mill and patch 40 mm	8	$35.00	750	$26,250	1	$ 26,250.00	0.677	$ 17,767.03	$ 784,387.43	$2,427,236.91	
20% Mill and patch 40 mm	16	$45.00	3000	$135,000	1	$ 135,000.00	0.458	$ 61,845.06	$ 800,693.93	$1,477,599.11	
Salvage Value	20					-$154,472.40	0.377	**-$ 58,219.02**			
								$ 794,618.90			

Appendix D-3: Life Cycle Cost at 5% Discount Rate – Case C

Case C - 40% RAP HMA							5%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
SP 12.5 mm FC1 (40% RAP)	0				1	$ 755,730.00	1.000	$ 755,730.00			
Rout and Crack Sealing (200 m/km)	3	$5.00	200.00	$1,000	1	$ 1,000.00	0.864	$ 863.84	$ 756,476.22	$5,555,690.90	$ 80,475.93
5% Mill and patch 40 mm	8	$35.00	750.00	$26,250	1	$ 26,250.00	0.677	$ 17,767.03	$ 767,755.43	$2,375,770.24	
20% Mill and Patch 40mm	16	$45.00	3000.00	$135,000	1	$ 135,000.00	0.458	$ 61,845.06	$ 784,061.93	$1,446,906.45	
Salvage Value	20					-$151,146.00	0.377	**-$ 56,965.34**			
								$ 779,240.59			

Appendix D-3: Life Cycle Cost at 5% Discount Rate – Case D

Case D - Control HMA							5%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
SP 12.5 mm FC1	0				1	$ 787,482.00	1.000	$ 787,482.00			
Rout and Crack Sealing (200 m/km)	3	$5.00	200	$1,000	1	$ 1,000.00	0.864	$ 863.84	$ 788,228.22	$5,788,883.03	$ 80,475.93
5% Mill and patch 40 mm	8	$35.00	750	$26,250	1	$ 26,250.00	0.677	$ 17,767.03	$ 799,507.43	$2,474,024.78	
20% Mill and patch 40 mm	16	$45.00	3000	$135,000	1	$ 135,000.00	0.458	$ 61,845.06	$ 815,813.93	$1,505,501.53	
Salvage Value	20					-$157,496.40	0.377	**-$ 59,358.74**			
								$ 808,599.19			

Appendix D-4: Life Cycle Cost at 7% Discount Rate – Case A

Case A - 20:20 Rubberized-RAP HMA							7%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
SP 12.5 mm FC1 (20% RAP + 20% CRM)	0				1	$ 835,866.00	1.000	$ 835,866.00			
Rout and Crack Sealing (200 m/km)	6	$5.00	200	$1,000	1	$ 1,000.00	0.666	$ 666.34	$ 836,310.01	$2,506,490.40	$ 1,110.35
Rout and Crack Sealing (200 m/km)	12	$5.00	200	$1,000	1	$ 1,000.00	0.444	$ 444.01	$ 836,063.15	$1,503,743.04	
5% Mill and patch 40 mm	20	$35.00	750	$26,250	1	$ 26,250.00	0.258	$ 6,783.50			
Salvage Value	20					-$167,173.20	0.258	**-$ 43,200.73**			
								$ 800,559.12			

Appendix D-4: Life Cycle Cost at 7% Discount Rate – Case B

Case B - 20% RAP HMA							7%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
SP 12.5 mm FC1 (20% RAP)	0				1	$ 772,362.00	1.000	$ 772,362.00			
Rout and Crack Sealing (200 m/km)	3	$5.00	200	$1,000	1	$ 1,000.00	0.816	$ 816.30	$ 773,028.34	$4,208,053.39	$ 61,823.21
5% Mill and patch 40 mm	8	$35.00	750	$26,250	1	$ 26,250.00	0.582	$ 15,277.74	$ 781,253.78	$1,869,068.90	
20% Mill and patch 40 mm	16	$45.00	3000	$135,000	1	$ 135,000.00	0.339	$ 45,729.17	$ 787,852.05	$1,191,430.93	
Salvage Value	20					-$154,472.40	0.258	**-$ 39,918.60**			
								$ 794,266.60			

Appendix D-4: Life Cycle Cost at 7% Discount Rate – Case C

Case C - 40% RAP HMA							7%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
SP 12.5 mm FC1 (40% RAP)	0				1	$ 755,730.00	1.000	$ 755,730.00			
Rout and Crack Sealing (200 m/km)	3	$5.00	200.00	$1,000	1	$ 1,000.00	0.816	$ 816.30	$ 756,396.34	$4,117,515.52	
5% Mill and patch 40 mm	8	$35.00	750.00	$26,250	1	$ 26,250.00	0.582	$ 15,277.74	$ 764,621.78	$1,829,278.56	$ 61,823.21
20% Mill and Patch 40mm	16	$45.00	3000.00	$135,000	1	$ 135,000.00	0.339	$ 45,729.17	$ 771,220.05	$1,166,279.15	
Salvage Value	20					-$151,146.00	0.258	**-$ 39,059.00**			
								$ 778,494.21			

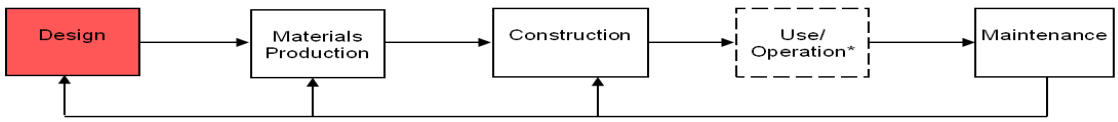
Appendix D-4: Life Cycle Cost at 7% Discount Rate – Case D

Case D - Control HMA							7%				M&R
Proposed Maintenance Activity	Year	Unit Cost	Estimated Quantity Per Km	Unit cost Per Km ($)	Length Per Km	Unit Cost	PW Factor	Present Worth Cost	Net Present Value (NPV)	EUAC	
SP 12.5 mm FC1	0				1	$ 787,482.00	1.000	$ 787,482.00			
Rout and Crack Sealing (200 m/km)	3	$5.00	200	$1,000	1	$ 1,000.00	0.816	$ 816.30	$ 788,148.34	$4,290,360.55	
5% Mill and patch 40 mm	8	$35.00	750	$26,250	1	$ 26,250.00	0.582	$ 15,277.74	$ 796,373.78	$1,905,241.94	$ 61,823.21
20% Mill and patch 40 mm	16	$45.00	3000	$135,000	1	$ 135,000.00	0.339	$ 45,729.17	$ 802,972.05	$1,214,296.18	
Salvage Value	20					-$157,496.40	0.258	**-$ 40,700.06**			
								$ 808,605.14			

Appendix E

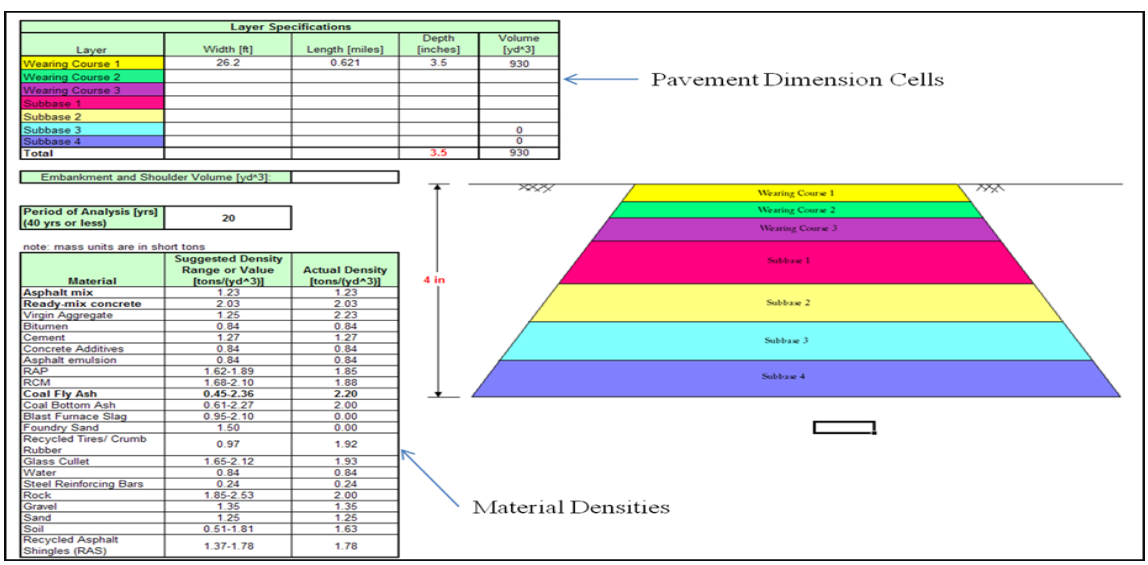
PaLATE Framework

The life cycle phases considered in the PaLATE framework is highlighted in Appendix E-1. This framework assesses the environmental effects of pavement construction and maintenance by defining the design criteria, type and volume of construction materials and equipment required. A step-by-step description of the input formulation and how the spreadsheet analyzes environmental quantification is given as follows:



Appendix E-1: PaLATE Life Cycle Phases

The first step involves completing the design worksheet as shown in Appendix E-2 is where the dimensions of pavement layers, density and volume of the construction materials, and the period of analysis are defined. The material volume for each pavement layer is combined with the material density to calculate the mass input of each material used.



Layer Specifications

Layer	Width [ft]	Length [miles]	Depth [inches]	Volume [yd^3]
Wearing Course 1	26.2	0.621	3.5	930
Wearing Course 2				
Wearing Course 3				
Subbase 1				
Subbase 2				
Subbase 3				0
Subbase 4				0
Total			3.5	930

Pavement Dimension Cells

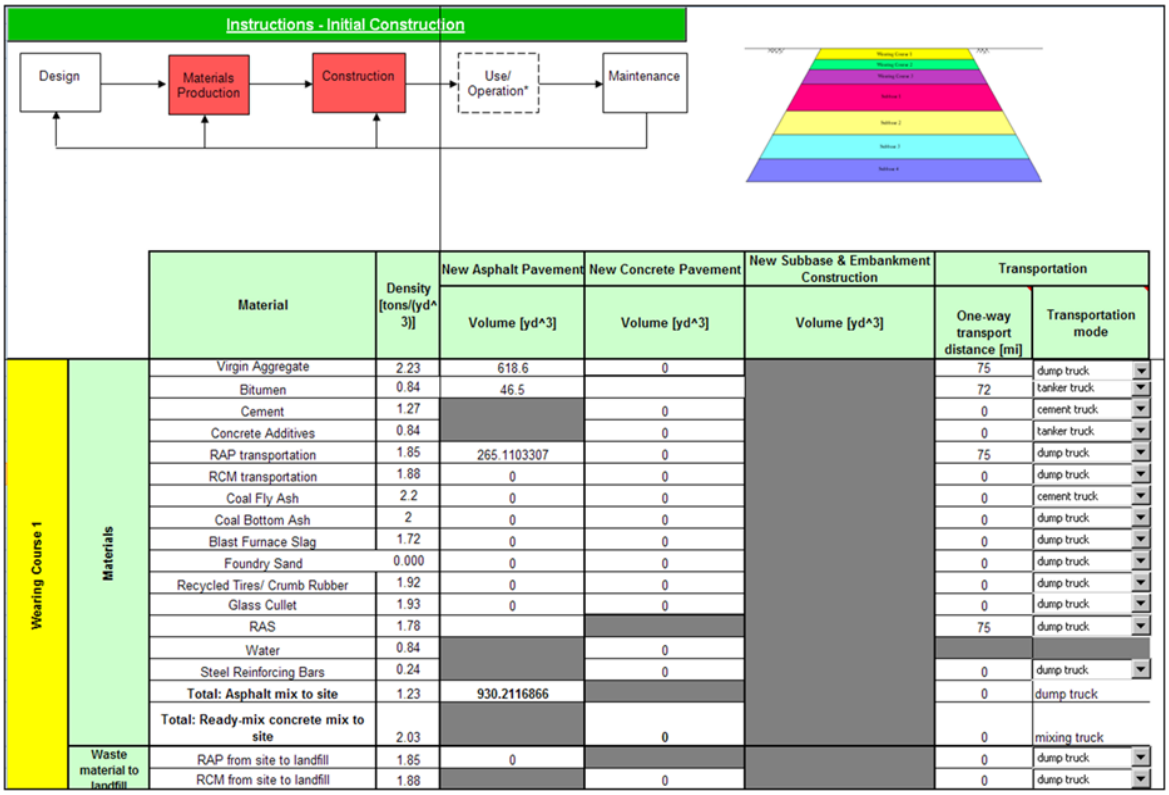
Embankment and Shoulder Volume [yd^3]:	
Period of Analysis [yrs] (40 yrs or less)	20

note: mass units are in short tons

Material	Suggested Density Range or Value [tons/(yd^3)]	Actual Density [tons/(yd^3)]
Asphalt mix	1.23	1.23
Ready-mix concrete	2.03	2.03
Virgin Aggregate	1.25	2.23
Bitumen	0.84	0.84
Cement	1.27	1.27
Concrete Additives	0.84	0.84
Asphalt emulsion	0.84	0.84
RAP	1.62-1.89	1.85
RCM	1.68-2.10	1.88
Coal Fly Ash	0.45-2.36	2.20
Coal Bottom Ash	0.61-2.27	2.00
Blast Furnace Slag	0.95-2.10	0.00
Foundry Sand	1.50	0.00
Recycled Tires/ Crumb Rubber	0.97	1.92
Glass Cullet	1.65-2.12	1.93
Water	0.84	0.84
Steel Reinforcing Bars	0.24	0.24
Rock	1.85-2.53	2.00
Gravel	1.35	1.35
Sand	1.25	1.25
Soil	0.51-1.81	1.63
Recycled Asphalt Shingles (RAS)	1.37-1.78	1.78

Appendix E-2: PaLATE Design Worksheet

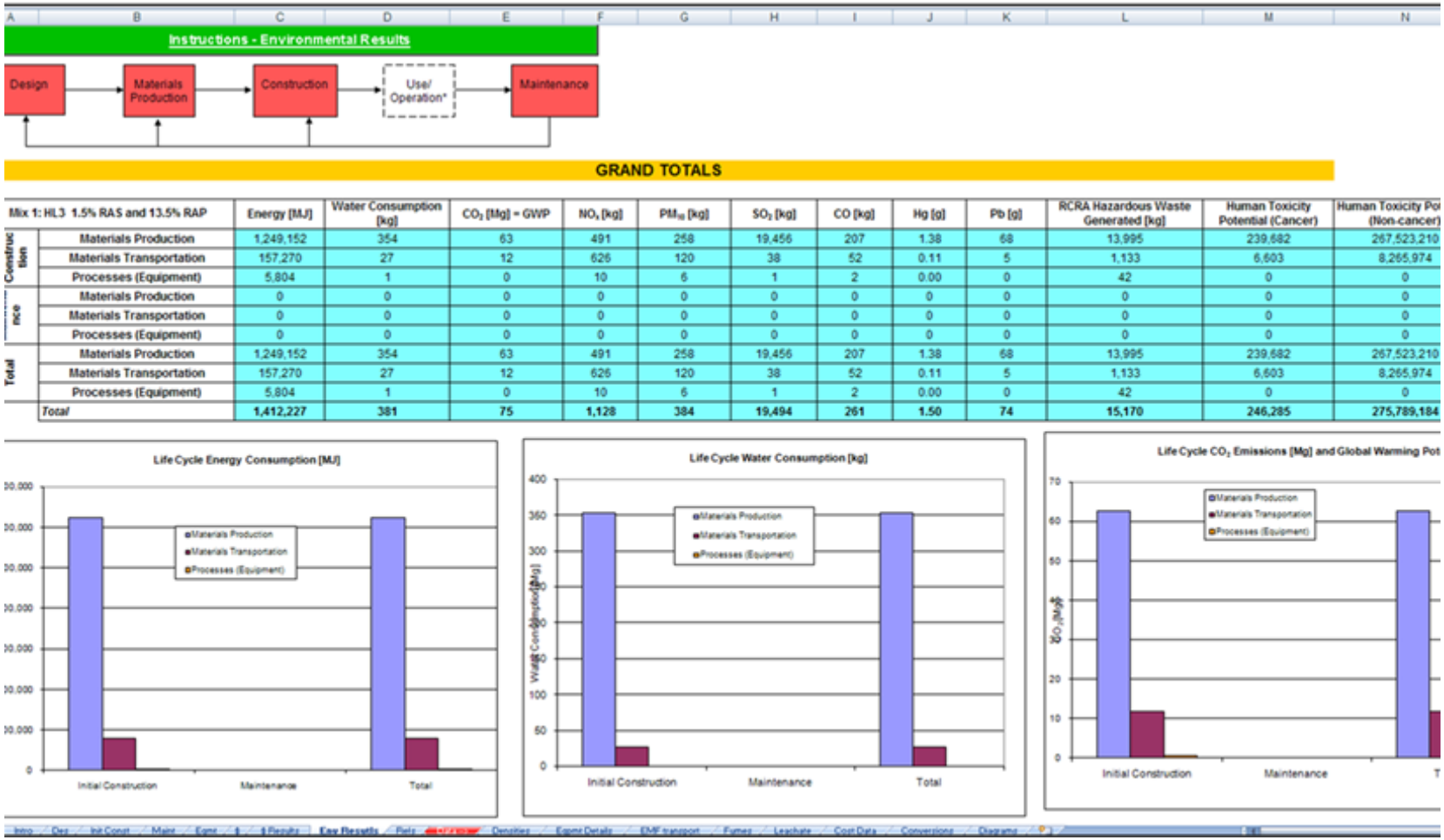
The next step involves completing the initial construction worksheet as shown in Appendix E-3. This is where the inputs for pavement material volumes, transportation distances, and method of material transportation in the different pavement layers are defined. The framework also has a worksheet for maintenance schedules, but this was not used in the environmental impact assessment in this research.



		Material	Density [tons/(yd^3)]	New Asphalt Pavement Volume [yd^3]	New Concrete Pavement Volume [yd^3]	New Subbase & Embankment Construction Volume [yd^3]	Transportation One-way transport distance [mi]	Transportation Transportation mode
Wearing Course 1	Materials	Virgin Aggregate	2.23	618.6	0		75	dump truck
		Bitumen	0.84	46.5			72	tanker truck
		Cement	1.27		0		0	cement truck
		Concrete Additives	0.84		0		0	tanker truck
		RAP transportation	1.85	265.1103307	0		75	dump truck
		RCM transportation	1.88	0	0		0	dump truck
		Coal Fly Ash	2.2	0	0		0	cement truck
		Coal Bottom Ash	2	0	0		0	dump truck
		Blast Furnace Slag	1.72	0	0		0	dump truck
		Foundry Sand	0.000	0	0		0	dump truck
		Recycled Tires/ Crumb Rubber	1.92	0	0		0	dump truck
		Glass Cullet	1.93	0	0		0	dump truck
		RAS	1.78				75	dump truck
		Water	0.84		0			
		Steel Reinforcing Bars	0.24		0		0	dump truck
		Total: Asphalt mix to site	1.23	**930.2116866**			0	dump truck
		Total: Ready-mix concrete mix to site	2.03		**0**		0	mixing truck
	Waste material to landfill	RAP from site to landfill	1.85	0			0	dump truck
		RCM from site to landfill	1.88		0		0	dump truck

Appendix E-3: PaLATE Initial Construction Worksheet

The environmental worksheet reports and summarizes all environmental effects resulting from the initial construction, maintenance phases and design input data. These results are reported in both numerical and bar charts that detail emissions let into the environment. Energy use and emissions are based on typical productivity, fuel consumption rate, and engine size of the equipment used in each activity. The environmental effects depend on the characteristics of equipment used to recover the construction material and the hauling distances of the material between processing facilities and construction site. The environmental impacts estimated by PaLATE included water and energy usage, global warming potential, pollutant emissions, RCRA hazardous waste release, human toxicity potential, fumes and leachate. A PaLATE environmental result worksheet is shown in Appendix E-4.



GRAND TOTALS

	Mix 1: HL3 1.5% RAS and 13.5% RAP	Energy [MJ]	Water Consumption [kg]	CO_2 [Mg] = GWP	NO_x [kg]	PM_{10} [kg]	SO_2 [kg]	CO [kg]	Hg [g]	Pb [g]	RCRA Hazardous Waste Generated [kg]	Human Toxicity Potential (Cancer)	Human Toxicity Po (Non-cancer)
Construction	Materials Production	1,249,152	354	63	491	258	19,456	207	1.38	68	13,995	239,682	267,523,210
	Materials Transportation	157,270	27	12	626	120	38	52	0.11	5	1,133	6,603	8,265,974
	Processes (Equipment)	5,804	1	0	10	6	1	2	0.00	0	42	0	0
nce	Materials Production	0	0	0	0	0	0	0	0	0	0	0	0
	Materials Transportation	0	0	0	0	0	0	0	0	0	0	0	0
	Processes (Equipment)	0	0	0	0	0	0	0	0	0	0	0	0
Total	Materials Production	1,249,152	354	63	491	258	19,456	207	1.38	68	13,995	239,682	267,523,210
	Materials Transportation	157,270	27	12	626	120	38	52	0.11	5	1,133	6,603	8,265,974
	Processes (Equipment)	5,804	1	0	10	6	1	2	0.00	0	42	0	0
	Total	**1,412,227**	**381**	**75**	**1,128**	**384**	**19,494**	**261**	**1.50**	**74**	**15,170**	**246,285**	**275,789,184**

Appendix E-4: PaLATE Environmental Worksheet