Evaluation of Cold Weather Performance of Rubber Modified Asphalt Placed in Ontario

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

Canada has close to 900,000 km of roads, which are key to supporting the Canadian economy as they enable for efficient transportation of goods and services. The Canadian road network should be well maintained and perform to a high standard to ensure the function and growth of the economy and the Country. It is also important to consider sustainability, and more specifically, recycling, when designing the roads. In order to improve on the performance while being sustainable, research is being conducted on the feasibility of using recycled materials in asphalt pavements. One of these materials is crumb rubber, obtained from recycled rubber tires, used as an additive to conventional Hot Mix Asphalt (HMA) to produce Rubber Modified Asphalt (RMA). These tires would otherwise end up in landfills, which is undesirable when considering sustainability.

The Ontario Tire Stewardship (OTS), along with the Centre for Pavement and Transportation Technology (CPATT), the Ontario Ministry of Transportation (MTO), and the Ontario Hot Mix Producers Association (OHMPA) have combined their efforts in order to carry out a pilot study on paving roads in Ontario with crumb tire rubber. Three highways, Highway 7, Highway 35, and Highway 115, have been paved using RMA, along with a section paved with conventional HMA to conduct a comparative study on the performance of the RMA sections. During the paving, samples were collected from the pavers in order to conduct laboratory testing on the materials to determine their performance.

The material was used to test the performance using the Thermal Stress Restrained Specimen Test, which introduces a stress in the sample material by cooling it to the point of failure while restraining the specimen, preventing it from relaxing. The specimen is cooled until failure, and the temperature at failure, along with the stress in the specimen is recorded for comparison. This test was conducted to determine the ability of the mixes to withstand the low temperatures that pavements in Ontario would have to endure.

A structural evaluation was also carried out on the pavement mixes using the AASHTOWare software (formerly the Mechanistic Empirical Pavement Design Guide, MEPDG). Although there are limitations to the software, it was still important to determine the structural capacity of the mix within a pavement structure to withstand the stresses of traffic and climate. The results of the AASHTOWare software determines the performance of the rubber mixes, as compared to the conventional mixes, to ensure that utilizing the rubber mixes does not hinder the performance of the pavement.

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I would like to acknowledge the Ontario Tire Stewardship (OTS) for funding this project, specifically Andrew Horsman for giving me the opportunity to work on the rubber asphalt pilot project. I would also like to acknowledge the Ontario Ministry of Transportation (MTO), and the Ontario Hot Mix Producers Association (OHMPA) for their support.

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I would also like to thank the entire team at the Centre for Pavement and Transportation Technology (CPATT), but specifically would like to acknowledge the help of Doubra Ambaiowei for all his help. Several industry professionals have been a great assistance and support during this research, and I would like to thank them: Mr. Narayan Hanasoge (Miller Paving), Seyed Tabib (MTO), Andrew Horsman (OTS), Mark Belshe (Rubber Pavements Association), and Richard Morrison (University of Waterloo).

Dedication

I dedicate this thesis to my family, who supported me throughout the journey of achieving this milestone. I would also like to dedicate this to Mohga Abo El Soud, for constantly being there for me whenever I needed her.

Thank You.

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Chapter 1. Introduction

1.1 Background

In today's world, a country's social and economic development relies heavily on pavement infrastructure. Canada has close to 900,000 km of road, which are a key element to the Canadian economy [TC 2013]. It is therefore imperative that these roads are well maintained and always perform up to the standard at which they should. As the Canadian economy grows, the demand for better road networks increases. Maintaining and constructing the road networks to meet the demand has become increasingly difficult in more recent years as the economy fluctuates more than normal and the availability of funding and price of virgin material is constantly increasing [OHMPA 2013].

As virgin aggregate material becomes scarce, it becomes more imperative that other alternatives to virgin aggregate materials be developed and to optimize the use of the virgin materials that are currently available. Many alternatives are starting to be implemented and being studied to further develop the use and to increase the efficiency of the recycling and maximize the benefit of recycling.

Hot Mix Asphalt (HMA) pavements consist of not only aggregate material, but also Asphalt Cement (AC). AC is a petroleum product and therefore the price of AC fluctuates similarly to the price of oil. In recent years, the jump in the price of oil has affected the behavior of the pavement industry. Pairing the rising price of oil along with recent trends to create more sustainable pavements and paving practices, recycling has become an important issue to the paving industry.

Recycling involves the re-use of existing material that is often intended for waste. In the pavement industry, recycling is carried out by re-using the existing paved road that has reached the end of its service life or has deteriorated beyond being serviceable. It examines the old material and evaluates the feasibility of recycling it into the asphalt mix to create new roads. Asphalt pavements are the most recycled product in North America, and are often referred to as 'Reclaimed Asphalt Pavement' (RAP) [NAPA 2011]. Another recycling technology that has been developed for the paving industry involves the recycling of scrap tires and using the recycled material in the pavement. The recycled

tires are obtained by grinding and shredding waste tires into crumb rubber, which typically has sizes that range from a No. 6 sieve to No. 200 (3.36 mm to 0.074 mm) [reRubber, 2013]. The crumb rubber is then incorporated into the asphalt mix and used to pave the road. The use of Rubber Modified Asphalt (RMA) has been incorporated into many standard practices of agencies such as Arizona Department of Transportation (DOT), CalTRANS or California DOT, and Texas DOT. The use of recycled material changes the properties and performance of the pavement, which may result in the elongation of the service life of the pavement along with less frequent and involved maintenance strategies.

The objective of this thesis is to examine the performance of the CRM material in typical Ontario HMA mixes. Through the findings in this research, recommendations are also provided for the use and implementation of CRM mixes in the pavements in Ontario. The findings are directed at providing guidance on the use of recycled materials in Ontario, and on specifically the use of CRM, and how it can be implemented effectively.

1.2 Motivation

The purpose of this thesis is to evaluate the performance of current RMA test sections located in Ontario. The goal of the research is to evaluate performance under the typical environmental and traffic conditions in Ontario. The research involved designing a test program to evaluate performance through laboratory testing. The laboratory testing involved evaluating the Thermal Stress Restrained Specimen Test (TSRST) results of the mixes. All of the following mixes have been tested on the following materials that were placed in partnership with MTO, OTS, and OHMPA:

1. Highway 7 (From York-Durham Line to Brock Road, Durham Region): Conventional Superpave 12.5 FC1 HMA, Superpave 12.5 FC1R RMA Field Blend, and Superpave 9.5 FC1R RMA Terminal Blend
2. Highway 35 (From County Road 10 to Elevator Road, Kawartha Lakes): Conventional Superpave 12.5 FC1 HMA, Superpave 12.5 FC1R RMA Field Blend

3. Highway 115 (From Highway 7A to Tapley Quarter Line, Kawartha Lakes): Conventional Superpave 12.5 FC2 HMA, Superpave 12.5 FC2R RMA Field Blend.

The aim of this research is to produce a standard practice document for the use of RMA in Ontario and how to effectively incorporate the use of RMA with conventional HMA to increase the service life of Ontario's pavements.

1.3 Scope and Objectives

The objectives of the thesis are as follows:

1. Conduct a literature review on the current use of RMA in pavements nationally and internationally.
2. Conduct a brief literature review on the experience of RMA in Canada.
3. Discuss the recent experience with RMA in Ontario.
4. Summarize laboratory test results obtained from the samples collected during the recent RMA experience in Ontario. Laboratory results involve the thermal stress restrained specimen tests.
5. Discuss the feasibility of the use of RMA in Canada as compared to the United States, and obtain conclusions and recommendations based on the analyzed results.
6. Based on laboratory tests, predict life cycle cost and performance using MEPDG.

1.4 Research Methodology

The research methodology, presented in Figure 1.1, is as follows:

1. Conduct literature review on the use of RMA.
2. Obtain samples from the recently paved projects in Ontario that incorporated RMA.
3. Conduct laboratory testing on the samples obtained, including the dynamic modulus, fatigue beam, and thermal stress restrained specimen tests.
4. Analyze the results obtained from the laboratory tests to determine the material properties of RMA.
5. Discuss the feasibility of the use of RMA in Ontario, and in Canada.

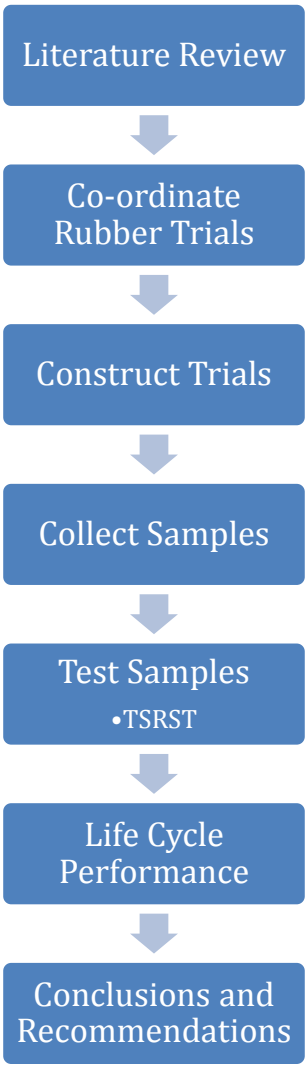


Figure 1.1 Research Methodology

1.5 Organization of Thesis

Chapter One of this thesis is an introductory chapter, explaining the scope and objectives of the thesis, along with the methodology used to obtain this thesis.

Chapter Two provides a literature review of the current and past use of RMA in the US and Canada.

Chapter Three discusses the recent experience in Ontario of sections being paved with RMA, and outlines the different sections of Highway that were paved in Ontario.

Chapter Four summarizes and discusses the laboratory testing that was carried out for the purpose of this thesis. An analysis of the results is also presented.

Chapter Five examines the rubber mixes using the AASHTOWare software and the results of the software analysis.

Chapter Six provides a summary of the key findings, conclusions based on the findings, and recommendations on the next steps for implementing and using Rubber Modified Asphalt (RMA) in Ontario.

Chapter 2. Literature Review

This chapter provides a review on Hot Mix Asphalt and the use of RMA in various jurisdictions. The processes of obtaining crumb rubber, along with crumb rubber technologies, are discussed in this chapter. The experiences in the United States with RMA are discussed in this chapter. An overview of the experiences with RMA in Canada is also presented.

2.1 Hot Mix Asphalt

Hot Mix Asphalt (HMA) consists of asphalt concrete mixed with various sizes of aggregates. The size and weight of aggregates along with the Asphalt Cement (AC) quantity is determined by the mix design that is specified for the mix [TAC 2013].

Asphalt pavements typically consist of 5 to 10 percent asphalt cement and 90 to 95 percent aggregates. The aggregates can be classified as either fine or coarse aggregate. There are many different types of asphalt pavement mixes, but the three most common mixes are dense-graded, open-graded, and gap-graded mixes.

Dense graded asphalt mixes contain a large amount of fine aggregates to fill the voids in the mix and keep it very tightly sealed. Dense graded mixes are most commonly used on pavements in the U.S. and Canada.

Open graded asphalt mixes contain a smaller percentage of fine aggregates which results in a higher percentage of air voids in the mix hence the term open graded. Open-graded mixes are best suited for a surface course, and are not intended to be used as a structural surface. The open-graded mix can accommodate higher asphalt cement content, making them highly resistant to reflective and fatigue cracking. Open-graded mixes are used for roads that have a constant flow of traffic, such as freeways, and are not suitable for parking areas or mill and overlay sections [Tighe 2012].

Gap graded mixes are between the open and dense graded mixes. Gap graded mixes contain few of the medium-sized aggregates to allow for some air voids in the mix [TXDOT 2011]. Gap-graded mixes can be used for a variety of applications, including overlays on existing pavements and new construction. Gap-graded mixes are suitable for a wide variety of traffic loading, and can be used where there is high volume, slow

moving traffic. Gap-graded mixes are not suitable for parking areas and where there exists much braking and turning [Tighe 2012].

Asphalt cement binder can be classified under the performance grade asphalt cement (PGAC) classification system. The PGAC system classifies asphalt cement binders according to performance of the binder at maximum and minimum temperatures. A PGAC binder of PG 64-34 indicates that the asphalt cement must meet the performance criteria at a maximum average temperature over seven days of 64 degrees C, and at a minimum one-day temperature of -34 degrees [TAC 2013].

Asphalt is not only designed to withstand certain levels of traffic loading, but also designed to withstand the environmental conditions it is exposed to. The Canadian climate, especially the climate experienced in southern Ontario, is extremely harsh to asphalt pavements. This is due to freeze-thaw cycles that the pavement experiences especially during winter months, as the temperature drops below freezing then rises above freezing. This freezing and thawing results in a large amount of stress induced on the asphalt pavements, and is experienced in southern Ontario extensively. It is therefore important to study the performance of asphalt as it freezes and thaws.

Factors such as the environment the pavement is exposed to, along with the traffic loading, causes the asphalt to distress and eventually fail. The pavement shows signs of fatigue and distress through many avenues, including [TAC 2013]:

- Thermal cracking,
- Fatigue cracking,
- Rutting, and
- A measure of the roughness or smoothness of the road, measured by the International Roughness Index (IRI).

2.1.1 Mix Specifications for Hot Mix Asphalt in Ontario

The Ontario Provincial Standards Specifications (OPSS) provides the standards to be used for Hot Mix Asphalt (HMA) pavements in Ontario. These specifications and standards are used to ensure the performance and suitability of pavements that are placed in Ontario.

Table 2.1 summarizes the mix gradation specifications of the various Superpave mixes used in Ontario. The table shows the typical gradations for conventional HMA in Ontario, along with the name of the Superpave mix. The mixes that have been utilized within the scope of this thesis are SP9.5, SP12.5 FC1, and SP12.5 FC2. The gradation of the SP12.5 FC1 and SP12.5 FC2 mixes are identical, as shown in Table 2.1, yet there is slight difference in the mixes. These differences are outlined in Table 2.2, which includes the Superpave mixes and their typical usage in Ontario. The explanations for the Dense Friction Course (DFC) and Hot Laid 1 (HL1) mixes are also provided in Table 2.3

Table 2.1 Aggregate Gradations for Superpave Mixes [OPSS 1151, 2006]

Hot Mix Asphalt Type	Percentage Passing by Dry Mass of Aggregates									
	Sieve Size mm									
	50.0	37.5	25	19.0	12.5	9.5	4.75	2.36	1.18	0.075
Superpave 4.75	-	-	-	-	100	95-100	90-100	-	30-60	6-12
Superpave 9.5	-	-	-	-	100	90-100	32-90	32-67	-	2-10
Superpave 12.5, 12.5FC 1 and 12.5FC 2	-	-	-	100	90-100	45-90	45-55 (Note 1)	28-58	-	2-10
Superpave 19.0	-	-	100	90-100	23-90	-	-	23-49	-	2-8
Superpave 25.0	-	100	90-100	19-90	-	-	-	19-45	-	1-7
Superpave 37.5	100	90-100	15-90	-	-	-	-	15-41	-	0-6
SMA 9.5	-	-	-	-	100	70-95	30-50	20-30	(Note 2)	8-12
SMA 12.5	-	-	-	100	90-100	50-80	20-35	16-24		8-11
SMA 19.0	-	-	100	90-100	50-88	25-60	20-28	16-24		8-11

Notes:

1. Requirements for the 4.75 mm sieve are in addition to those normally used for Superpave.
2. For the SMA 9.5 mm the maximum percentage passing the 1.18 mm, 0.600 mm, and 0.300 mm sieves is 21, 18, and 15 respectively.

Table 2.2 Hot Mix Asphalt Types and Descriptions for Ontario [OPSS 1151, 2006]

Hot Mix Asphalt Type	Typical Hot Mix Use and Properties
Superpave 4.75	Fine, surface, and levelling mixes similar to the traditional sand mixes for miscellaneous applications.
Superpave 9.5	Fine, surface, padding, and levelling mixes for Traffic Category A and B roads and driveways.
Superpave 12.5	Surface mix for Traffic Category B and C roads. Superpave 12.5 is similar to the traditional HL 3, HL 3 Fine, and HL 4 mixes according to OPSS 1150.
Superpave 12.5 FC1	Surface mix for use on Traffic Category C roads that provides superior rutting resistance and skid resistance through aggregate selection. Superpave 12.5 FC1 is similar to the traditional HL 1 mix according to OPSS 1150.
Superpave 12.5 FC2	Surface mix for use on Traffic Category D and E roads that provides superior rutting resistance and skid resistance through aggregate selection. Superpave 12.5 FC2 is similar to the traditional DFC mix according to OPSS 1150.
Superpave 19.0	Binder course mix for Traffic Category A, B, C, D, and E roads. Superpave 19.0 is similar to the traditional HL 4, HL 8, and HDBC mixes according to OPSS 1150.
Superpave 25.0 and 37.5	Large stone binder course mixes for use when thicker binder lifts are required.
SMA 9.5 and 12.5	Gap-graded premium surface course mix with high frictional resistance, enhanced rutting resistance, water spray reduction, and potential noise reduction for Traffic Category D and E roads. 100% crushed aggregates from the DSM are used for both fine and coarse fraction.
SMA 19.0	Gap-graded premium binder course mix with enhanced rutting resistance for Traffic Category D and E roads. 100% crushed aggregates are used for both fine and coarse fraction.
Note: A. The traffic categories are according to Table 1 of OPSS 1151.	

Table 2.3 Hot Mix Asphalt Types and Descriptions for Ontario [OPSS 1150, 2010]

Hot Mix Type	Abbreviation	Summary of Hot Mix Use and Properties
Dense Friction Course	DFC	A dense-graded surface course mix with high frictional resistance for high volume roads. Aggregates have an identical gradation to HL 1 aggregates with a maximum aggregate size of 16 mm. Premium 100% crushed aggregates are used for fine and coarse aggregates that are from the same source.
Hot Laid 1	HL1	A dense-graded surface course mix with a premium quality coarse aggregate. It is used on high volume roads and has a maximum aggregate size of 16 mm. Coarse aggregates are 100% crushed material.

Superpave 12.5 FC1 mixes are typically used for "Category C" traffic as defined by Ontario's OPSS 1151 [OPSS 1151, 2006], while Superpave 12.5 FC2 mixes are used typically for traffic "Category D". Table 2.2 shows that the Superpave 12.5 FC1 mix is similar to the Hot Laid (HL) 1 mix, and Superpave 12.5 FC2 mixes are similar to the DFC mixes as specified in OPSS 1150 [OPSS 1150, 2010].

Table 2.3 shows the uses for the DFC mix and HL1 mix. The DFC mix differs from the HL1 mix in that the DFC mix is used for high volume roads. The biggest difference between the DFC and HL1 mixes is that the DFC mixes utilize 100% premium aggregate for the fine and course material that are obtained from the same source, while the HL1 aggregate is only required to be 100% crushed material.

The traffic categories used in the Ontario specifications are listed in Table 2.4. The table summarizes all the traffic categories that are used for the determination of the asphalt pavement type that would be placed. The highways used for the analysis that is within this thesis are listed as "Category C" and "Category D" traffic highways. Category C highways are classified to have traffic Equivalent Single Axle Load (ESAL) for a 20-

year design of 3 to 10 million. Traffic "Category D" highways are classified with a 20-year design ESAL of 10 to 30 million.

Table 2.4 Ontario Traffic Categories [OPSS 1151, 2006]

Ontario Traffic Category	20-Year Design ESALs (Note 1)	Typical Applications
A	Less than 0.3 million	Low volume roads, parking lots, driveways, and residential roads.
B	0.3 to 3 million	Minor collector roads.
C	3 to 10 million	Major collector and minor arterial roads.
D	10 to 30 million	Major arterial roads and transit routes.
E	Greater than 30 million	Freeways, major arterial roads with heavy truck traffic, and special applications such as truck and bus climbing lanes or stopping areas.

Note:

1. Equivalent Single Axle Load (ESAL) for the projected traffic level expected in the design lane over a 20-year period, regardless of the actual design life of the pavement.

The ESAL is a cumulative traffic load summary statistic [TAC 2013]. The ESAL calculation is designed to obtain an equivalent load for all the axle loads and configurations into a calculated summary statistic to be utilized for design. In the Ontario specifications, the ESAL is obtained for 20-year cumulative traffic equivalence.

2.2 Crumb Rubber

When tires are decommissioned at the end of their useful life, they become a waste product. Tires consume a large amount of space in landfills, and it is undesirable to have tires deposited in landfills in large quantities. It is, therefore, the goal to divert these tires from landfills and have them recycled and made into other products if possible. Crumb rubber is obtained from these recycled tires. There are two methods to obtaining crumb rubber from recycled tires, ambient grinding and cryogenic grinding.

While the differences between ambient grinding and cryogenic grinding are discussed, this thesis does have a focus on the use of ambient ground crumb rubber when discussing the results of the rubber modified asphalt. While there may be differences in the methods for obtaining the rubber, it was shown previously that the ambient ground

rubber does have additional benefit and out-performs mixes made with cryogenic ground rubber [Lee 2008].

2.2.1 Ambient Grinding

Recycled tires can also be processed through multiple levels of grinders or cracker mills to obtain the desired gradation of ambient ground rubber. Ambient ground crumb rubber is kept at room temperature while being transformed into crumb rubber for possible usage into asphalt.

The process involves running tires through an initial stage of grinding, where smaller chips are obtained from whole tires. The chips are then further ground to separate the other components of a tire from the rubber to filter out undesired materials. Once the rubber is filtered from the fibers and steel, it is then run through further grinding to obtain the required gradation of crumb particles. Ambient processing is favored due to the particles of rubber having relatively large surface area and irregularly shaped edges to increase the binding between the rubber and the asphalt cement [Tighe 2012]

2.2.2 Cryogenic Grinding

Freezing the tires and breaking them after they are frozen is referred to as cryogenic ground rubber. The rubber is usually frozen to become brittle after it has been reduced to smaller chips and not whole tires. The freezing is typically conducted using liquid nitrogen and the tires are ground using a hammer mill or other impact methods. This method produces rubber particles with smooth edges and small surface areas.

The process of separating the steel from the tire involves using magnets to extract the steel from the rubber. The fibers are separated using screening to separate the unwanted materials from the crumb rubber.

2.3 Use of crumb rubber in asphalt pavement

There are two main components to asphalt pavements, the asphalt cement binder and the aggregate in the mix. Incorporating crumb rubber into asphalt pavement mixes can be achieved by adding rubber to either the asphalt cement binder or substituting virgin aggregate in an asphalt mix with larger pieces of crumb rubber. The method of adding fine graded crumb rubber to the asphalt cement binder is called the wet process. When

substituting virgin aggregate with crumb rubber, a coarser rubber is used instead of some of the aggregate that is made to be in the job mix formula.

There are two ways of incorporating crumb rubber in asphalt pavements; wet process and dry process. The wet process involves adding the rubber to the AC. The wet process requires a blending unit to be placed at the asphalt plant to mix the virgin AC with rubber. The wet process utilizes a finer grind of rubber than the dry process. The dry process adds coarse rubber as aggregate to substitute virgin aggregate.

2.3.1 Wet Process

The wet process involves adding crumb rubber from recycled tires into the asphalt cement binder. The crumb rubber can either be blended with the AC at the cement suppliers' plant or at the asphalt plant. These two processes are called terminal blend and field blend processes, respectively.

The wet process terminal blend utilizes finely ground crumb rubber particles that are mixed with the asphalt cement binder at the asphalt cement refinery or plant and then transported for use. The terminal blend mixes typically contain approximately 10 to 15% crumb rubber by weight of asphalt cement [Way 2011]. The American Standard Test Method (ASTM) defines asphalt rubber as containing 15 percent rubber by total weight and mixed enough to cause swelling of the rubber [ASTM D 8 2009], yet there are mixes that contain high rubber content, between 18% to 22%, are more frequently used [Ambaiowei 2013; Roberts 1996; Caltrans 2006].

The wet process field blend allows for use of more coarsely ground rubber particles. Mixing the asphalt cement with the rubber at the asphalt plant, and then producing asphalt mix using the blended binder produces the field blend rubber modified asphalt. The field blend method requires specialized equipment to be able to keep the asphalt cement binder and the rubber particles at a high temperature and agitated well to ensure proper bonding of the materials.

2.3.2 Dry Process

The dry process differs from the wet process in that the crumb rubber is not added to the asphalt binder but added as an aggregate in the mix. The crumb rubber is added as an aggregate before the introduction of asphalt binder to the mix at the plant. Typical values

of rubber content in a dry process mix are between 3 to 5 percent, depending on the application, gradation, and mix design. Dry process mixes can be dense graded, gap graded, or open graded, as opposed to wet process where it is required to be gap or open graded to ensure the rubber fits into the voids in the mix.

2.4 Experience with RMA in the United States

The states of Texas, California, and Arizona all use RMA extensively, and mainly incorporate the rubber through the wet process. This section discusses the added performance benefits, and the life cycle performance of the rubber modified pavement as compared to hot mix asphalt in those jurisdictions.

2.4.1 Performance

Rubberized asphalt has a quality of increased durability that allows the asphalt to be more resistant to cracking. Rubberized asphalt has been studied and shown to exhibit properties that are resistant to fatigue cracking and pavement deformation [Kaloush 2002; Mohammad 2002; Way 2000]. This allows the asphalt to be able to withstand much higher traffic loading than conventional asphalt pavements. In 2000, the Federal Highway Administration (FHWA) in the U.S. built an Accelerated Loading Facility (ALF) to test the performance of asphalt rubber, along with pavements that contain other modifiers, and compare them to a standard performance grade asphalt [Takallou 2011].

A machine that rolls a tire back and forth over the pavement repetitively carries out the ALF testing. This tire is loaded with a prescribed load. The rolling tire does these passes over the various sections of pavement to be tested, which include engineered pavements such as Styrene-Butadiene-Styrene modified pavement and ethylene terpolymer modified pavement. These are heavily modified and engineered pavements, and were tested alongside the standard asphalt mix for comparison. Each section was paved with 100 mm thickness, with the exception of the field blend rubber section, which was paved with only a 50 mm thickness over 50 mm of conventional mix.

After each 100,000 passes of the rolling tire, the pavement is checked for cracking. If the cracking is less than 10 m in total length, another 100,000 passes of the tire is put on the pavement. If the cracking exceeds 10 m in length the pavement is considered failed or insufficient to withstand more loads.

As shown in Figure 2.1, the asphalt rubber section survived 300,000 passes of the loaded rolling tire with no cracking, while the conventional pavement had 90 m of total length of cracking only after 100,000 passes [Takallou 2011]. This shows the resistance asphalt rubber has to cracking, as a core was taken from that section, and showed that the 50 mm of conventional mix under the rubber pavement had been cracked; yet the surface had no cracking. The asphalt rubber had not only out-performed the conventional pavement, it had performed much better than all the other modified pavements in the ALF.

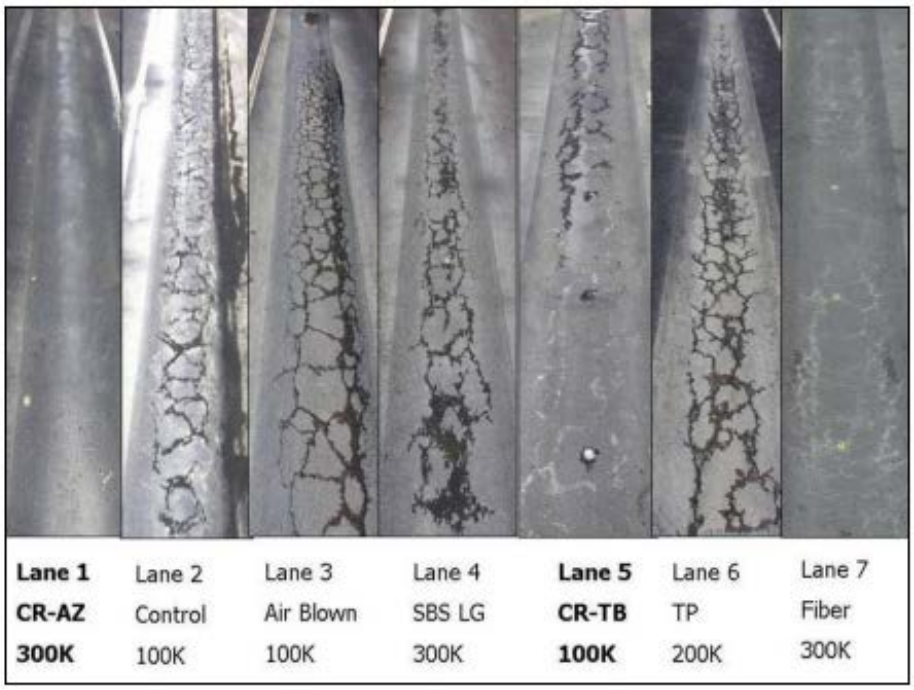


Figure 2.1 Sections Tested at the ALF (Source: [Takallou 2011])

Many laboratory tests have been conducted to determine the performance of rubber modified asphalt, with the RMA mixes consistently proving to show improved results over competing mixes, be it conventional mixes or other modified binders [Shatnawi, 2001; Kim 2001; Raad 2001; Bahia 1994; Hossein 1999; Liang 1996; Pszczola 2012]. More specifically, rubber modified asphalt has shown to better resist thermal cracking and changes in temperature than conventional HMA. RMA has consistently outperformed conventional asphalt mixes when put through thermal testing [Raad 1993; Epps 1997]. It has also been shown that rubber modified asphalt mixes have a resistance

to permanent deformation, or rutting, more so than conventional HMA mixes [Palit 2004].

2.4.2 Economics

Throughout the life of a pavement, there are four main cost components, and are summarized in Table 2.5 below [OHMPA 1998].

Table 2.5 Major Life Cycle Cost Components (Source: [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

The most important costs to consider for the purpose of RMA are the initial construction and future maintenance costs. As can be seen in the table above, these are two major cost components in the life of a pavement.

The initial construction costs of an RMA pavement are higher than that of conventional pavements due to several factors. One of the factors is that the rubber for the pavement is an added cost. Although it is much cheaper than asphalt cement, rubber is an added component to the RMA, and therefore increases the cost of the pavement.

Another added cost, and believed to be the major cost addition, is the cost of acquiring, renting, or contracting the equipment used to blend the rubber to the asphalt cement. For example, for the paving in Ontario in 2011, contracting the blending unit cost between $15-$30 per tonne of asphalt produced, not including the cost of mobilizing, demobilizing, and setting up/taking down the equipment. It is, therefore, believed that the

equipment is where the majority of the incremental cost between rubber asphalt and conventional asphalt lies.

The costs of using RMA in Arizona DOT, Texas DOT, and CalTrans are compared to the conventional alternative. These costs are shown in Table 2.6 below [Hicks 1999].

Table 2.6: Estimated Costs of Conventional vs. Rubber Alternatives (Source: [Hicks 1999])

Item		Estimated Costs		
		Average	Low	High
a) Arizona DOT				
Chip Seals ($/yd^2)				
Conventional	CS	1.00	0.75	1.50
Asphalt Rubber	AR-CS	N/A[a]	N/A	N/A
Asphalt Overlays ($/yd^2-in.)				
Conventional	ACHM-DG	1.33	–	–
	ACHM-FC	1.54	–	–
Asphalt Rubber	ARHM-GG	2.43	1.90	2.96
	ARHM-FC	2.51	1.92	3.10
b) Caltrans				
Chip Seals ($/yd^2)				
Conventional	CS	1.00	0.80	1.20
Asphalt Rubber	AR-CS	2.00	1.70	2.50
Asphalt Overlays ($/ton)				
Thick HMA Overlays	ACHM-DG	35	30	40
(4 in.)[b]	ARHM-GG	50	42	58
Thin HM Overlays	ACHM-DG	35	30	40
(1 in.)	ARHM-GG	50	42	58
c) Texas DOT				
Chip Seals ($/yd^2)				
Conventional	CS	0.65	–	–
Asphalt Rubber	AR-CS	0.80	–	–
Asphalt Overlays ($/yd^2-in.)				
Nonstructural	ACHM-FC	1.50	1.25	2.25
Friction Courses	ARHM-FC	2.05	1.55	3.08
Structural Overlays	ACHM-DG	1.50	1.15	2.37
(2 in.)	ARHM-GG	2.05	1.55	3.07

[a]Not available
[b]1 in. = 25 mm

Notes: ACHM-DG= conventional hot mix – dense-graded,
ACHM-FC = conventional hot mix – friction course,
ARHM-GG = asphalt rubber hot mix – gap-graded,
PM-CS = polymer modified chip seal,
AR-CS = asphalt rubber chip seal,
ARHM-OG = asphalt rubber hot mix – open-graded,
ARHM-FC = asphalt rubber hot mix – friction course,
CS = chip seal,

The table shows that the cost of using the asphalt rubber alternative can range between approximately 26% higher (when paving structural overlays in Texas DOT) to approximately 45% higher (when paving an asphalt overlay in Arizona DOT). Table 2.6 shows approximate average cost, in USD, of each asphalt mix and use. Although asphalt rubber has a relatively large cost associated with the initial construction of the pavement, as mentioned, there are many benefits with using asphalt rubber. These benefits include the elongated service life of the pavement, while resisting cracking. These benefits allow for less maintenance throughout the lifespan of the pavement, the pavement to perform up to the required standards for longer, and delaying the need to carry out any major rehabilitation.

Many alternatives have been considered against the asphalt rubber alternative, and it was found that for Arizona DOT the asphalt rubber alternative was cost effective in all applications [Hicks 1999]. For CalTrans, all alternatives were cost effective except for asphalt rubber chip seals, which were not covered under the scope of this thesis. In Texas DOT, most asphalt rubber alternatives were more cost effective than conventional rubber. Table 2.7 below shows the different alternatives that were being compared to conventional pavements, and the cost ratio (in the last column) showing how much more cost effective the asphalt rubber is than the conventional alternative.

Table 2.7: Cost Effectiveness of Asphalt Rubber (Source: [Hicks 1999])

Traffic Level	Scenario	Approximate % of Time AR Alternate is More Cost Effective
a) Arizona DOT		
High	A vs B	98
	C vs D	90
	E vs F	65
	J vs K	100
	L vs M	69
Low	G vs H	70
b) Caltrans – Headquarters		
High	A vs B	92
Moderate	C vs D	70
Low	E vs F	20
c) Caltrans – Districts		
High	G vs H	84
	I vs J	76
Low	K vs J	83
d) Texas DOT		
High	A vs D	50
	B vs D	80
	C vs D	36
	E vs G	99
	F vs G	95
Moderate	H vs J	5
	I vs J	13
	K vs L	99

Table 2.8 and Table 2.9 summarize the key aspects of the study [Hicks 1999]. The tables show the method of comparing the costs of the conventional pavements against the asphalt rubber alternatives. Table 2.8 shows the strategies used for comparing alternatives. Table 2.9 shows the life cycle costs of the pavement. The initial costs for the asphalt rubber are higher than the conventional pavement, yet the ongoing maintenance and rehabilitation is significantly lower throughout the life of the pavement. This causes the asphalt rubber alternative to be more cost effective over the life of the pavement, even though it has a much higher initial cost.

Table 2.8: Description of the Alternatives Considered (Source: [Hicks 1999])

Traffic Volume	Type of Activity	Existing Pavement Surface		Description of Alternatives[a]
c) Texas DOT				
High	Structural	HMA	A	2.0 in ACHM-DG
			B	2.0 in ACHM-GG
			C	2.0 in. PMHM-GG
			D	2.0 in ARHM-GG
	Nonstructural	HMA	E	0.75 in. ACHM-FC
			F	0.75 in. PMHM-FC
			G	0.75 in. ARHM-FC
Moderate	Nonstructural	HMA	H	AC-CS
			I	PM-CS
			J	AR-CS
	Structural	HMA	K	1.5 in. ACHM-DG
			L	1.5 in. ARHM-GG

Table 2.9: Life Cycle Cost Analysis for Arizona DOT (Source: [Hicks 1999])

Traffic	Scenario	Rehabilitation $	Maintenance $	Salvage $	Total Agency Costs $	Lane Rental $	Total Cost $	Ratio of Costs (Conv/AR) Agency	Ratio of Costs (Conv/AR) Total
High	A Conv	11.98	1.50	(0.58)	12.90	3.96	16.86	1.36	1.38
	B AR	9.25	0.49	(0.23)	9.51	2.71	12.22		
	C Conv	19.27	1.26	(1.52)	19.01	3.76	22.78	1.25	1.27
	D AR	15.10	0.45	(0.38)	15.17	2.71	17.89		
	E Conv	23.98	1.75	(1.72)	24.00	3.21	27.27	1.08	1.10
	F AR	24.25	0.57	(2.65)	22.17	2.47	24.64		
	J AR	5.89	0.81	(0.25)	6.45	3.33	9.78	6.77	4.69
	K Conv	43.48	0.74	(0.52)	43.69	2.17	45.87		
	L AR	18.08	0.69	(1.74)	17.04	2.52	19.56	1.08	1.11
	M Conv	19.32	0.46	(1.33)	18.45	3.25	21.70		
Low	G Conv	3.56	1.16	(0.25)	4.46	0.45	4.91	1.14	1.16
	H AR	3.79	0.49	(0.36)	3.92	0.31	4.23		
	H AR	3.79	0.49	(0.36)	3.92	0.31	4.23	2.02	1.93
	I Conv	6.37	1.70	(0.18)	7.90	0.27	8.17		

Conv = Conventional Alternate
AR = Asphalt Rubber Alternate

Although this study shows the benefits of using asphalt rubber over the life cycle of the pavement, the studies are done for those jurisdictions. While the results would be similar elsewhere, as the concept of the study is transferrable, there is still variability in different jurisdictions; therefore more detailed studies should be conducted in each jurisdiction to see the feasibility of using asphalt rubber in different environments. For this reason it is important to continue with the study of the 2011 rubber trials to determine the feasibility of using asphalt rubber in Ontario.

2.4.3 Other Key Findings

Recycling tires is a good way to contribute to the global goals of sustainability and preservation of the environment. Used tires are a waste product that get put into landfills and contribute to environmental deterioration. There are many products that come from the recycling of tire rubber, but having asphalt pavement is a very effective way of using up the majority of the tires that would otherwise sit in landfills.

In the case of Grey County, by utilizing the dry process, they used about 8% by volume of crumb rubber with coarse particles [OGRA 2009]. With over 200 km paved, they diverted approximately 660,000 tires from going to landfills. The dry process also has the environmental benefit of replacing some of the virgin aggregate in the mix. Virgin aggregate is becoming scarcer in Ontario and is a non-renewable resource; therefore using less of it is a step forward in becoming more sustainable.

The wet process uses approximately 2 to 2.5% crumb rubber in the mix, therefore it also has the potential of utilizing tires in asphalt cement, but unlike the dry process, it does not replace aggregate in the mix, rather the rubber is mixed in the asphalt cement. Although the wet process uses less rubber by volume and does not replace virgin aggregate, the wet process has been more widely used and shows more benefits than the dry process in other jurisdictions. Therefore, it makes more economic sense to use the wet process; otherwise using crumb rubber is just implementing “linear landfill” [OGRA 2009].

Grey County has used RMA for recycling with CIR. Grey County has successfully been able to pave new pavements by recycling the asphalt rubber pavement, and also used a method of spreading the crumb rubber along old pavements ahead of the CIR machines so they can include the rubber into the mix [OGRA 2009].

RMA has also been shown to to retain heat during the winter, therefore reducing de-icing chemicals. RMA has also been observed to have better ice performance with better friction during the cold months, and can reduce the need for de-icing in the winter and use less salts and chemicals on the roads [Takallou 1987].

RMA also helps reduce noise pollution. Tire noise is reduced significantly on pavements that utilize rubber. The reduction of noise pollution is an environmental benefit, as it makes the surrounding areas around the rubber pavement quieter as opposed

to loud highways [Carlson 2003]. In Arizona, an 8 dBA reduction was experienced when using asphalt rubber. For every 1 dBA of noise, a wall height of 2 feet is required to mitigate the noise of the pavement. It was shown that in Arizona, at a cost of $20 per square foot of wall, reducing the noise levels by dBA is equivalent to saving approximately $1.6 million on noise reduction walls (Rubber Pavement Association, 2010). Other studies have also shown a general improvement in the noise levels of the highways paved with RMA, along with an increase in the friction of the pavement [Losa 2012].

Some research has shown that RMA pavements also increase the safety of highways by reducing spray on the highways in bad weather conditions [Takallou 2011]. Reducing spray increases the visibility on the highways, hence increasing safety in adverse weather conditions by eliminating some of the weather related accidents.

2.5 Early experiences with RMA in Canada

While the experience in Canada with RMA are not as extensive as in the United States, there have been a few studies in the literature, and more specifically Ontario, regarding the use of RMA that started in 1980 [Tabib 2009]. Initially, the use of RMA was studied as an environmentally friendly and sustainable alternative to conventional asphalt mixes. When the performance of these rubber trials was determined, the next group of studies with RMA was conducted to test the recyclability of RMA paved sections.

Most of the initial trials with RMA were conducted using the dry process. The dry process had been used for few projects and the performance of these sections was determined to be moderate to poor. Another study was conducted in 1994 to compare the different processes for making RMA. This study was to compare the performance of the wet process with the dry process. For this study, a section in the City of Brantford was chosen for the dry process, while the wet process section was in the Town of Kirkland. The dry process' performance was considered acceptable, while the wet process section was considered a technical success [Carrick 1995]. Other sections studied had inconclusive results in terms of performance, with the rating of performance ranging from "somewhat poor to very good" [Emery 1997].

While some RMA performance were considered successful, it is still not entirely conclusive as whether or not these can be used in place of conventional mixes as California, Texas, and Arizona have done. The Ministry of Environment and the Ministry of Transportation, therefore, have conducted another study, sponsored in 1997 on 15 test sections to determine the performance of RMA [Tabib 2009]. The key findings of the study showed that the dry process performed worse than conventional HMA, which could be due to the difficulties associated with the use of the dry process [Emery 1997]. The wet process showed superior performance to the dry process sections, and met or exceeded the performance of the conventional mixes [Tabib 2009].

Grey County has taken the initiative to put in place a rubberized asphalt program. Grey County has paved over 200 km of rubber asphalt and recycled close to 660,000 tires since the introduction of the program in 1991 [OGRA 2009]

Although Grey County used the dry process in their rubber mixes, the paved sections had still performed better than the conventional mixes. The rubber surface had lasted 18 years before it had to be resurfaced [OGRA 2009].

There are three sections that were paved in Ontario in the summer of 2011 utilizing the wet process. It is important to observe the performance of these sections, as this will determine the feasibility of using asphalt rubber in Ontario to help increase the performance, longevity, and sustainability of the pavement infrastructure.

2.6 HMA Thermal Stress Restrained Specimen Test

When evaluating performance of asphalt, one important factor in the determination of the performance is the resistance of the asphalt to thermal cracking. To be able to obtain the characteristics and behavior of the asphalt material when resisting thermal stresses, various methods are employed. The methods include a regression equation, mechanistic prediction model, and laboratory testing, which involves a Thermal Stress Restrained Specimen Test (TSRST) to determine how the material behaves under thermal stress.

2.6.1 Regression Equation

A regression equation was developed based on the analysis of 26 different airfields in Canada in 1987 [Haas 1987]. The regression equation that was developed was used to

determine the spacing of transverse cracks. While the regression equation may be sufficient to model the airfields used to develop it, it may not be sufficient for evaluating pavements under differing conditions.

2.6.2 Mechanistic Prediction Model

The prediction of thermal cracking can be modeled using a beam-analysis equation derived from knowing that thermal cracks propagate from the surface, and occur when the thermal stress exceeds the strength of the asphalt cement [Hills 1966]. For the equation to work, two parameters must be measured or assumed carefully, which are the coefficient of thermal contraction and the stiffness of the asphalt concrete mix.

For this model to properly work, the rate of cooling must be matched to a rate of deformation or loading, which has not been sufficiently developed. Also, some assumptions made for this model, namely the value assumed for the thermal contraction constant and the stiffness values of the asphalt cement, are shown to be erroneous by several factors for different mixes that have modifiers introduced to them [Zubeck 1992].

2.6.3 Laboratory Testing

To determine the susceptibility of a mix to cold climate and to determine the characteristics of its performance under certain conditions, it has been suggested that laboratory testing be conducted to simulate field conditions [Monismith 1966; Vinson 1990]. The main concept behind the laboratory testing ensuring that the length of the specimen remained constant. Early testing methods that were developed included a single, fixed frame that held the specimen to restrain it from changes in length. This method was later proven to be flawed as the frame was found to deform along with the specimen under the conditions it was exposed to, and hence allowing the specimen to relax the internal stresses. Hence, a new system for testing was developed, which involved the measurement of the displacement of the specimen through a feedback loop that was installed and allowed for the adjustment of the frame according to the deformation occurring in the specimen [Arand 1987].

2.7 Mechanistic Empirical Pavement Design Guide

Computer based modeling has been implemented to assist pavement engineers with the decisions regarding pavement design and maintenance programs. There was a need to develop a mechanistic empirical design guide, which was raised by American Association of State Highway and Transportation Officials (AASHTO), and was developed and completed by a group involving research teams, academic institutions, and several consultants [Schwartz 2007].

The Mechanistic Empirical Pavement Design Guide (MEPDG), now known as AASHTOWare, is used to determine several factors when discussing a pavement design. It is used to determine the most cost effective design for a pavement given the conditions it will be placed under. The condition inputs used for the pavement designs determined by the MEPDG include traffic data, climate information for the specific region, and also underlying soil conditions and existing pavement conditions. With this information, the MEPDG is capable of determining a pavement design that is suitable for the inputs. The MEPDG is also used to determine rehabilitation and maintenance programs in order to maximize the benefit/cost ratio of the pavement.

While the MEPDG is able to calculate and predict the effects of the traffic, climate, and the underlying soil or existing pavement conditions, the limitation of the MEPDG is that it is not capable of carrying out the same analysis on modified asphalt pavements, including Recycled Asphalt Pavement (RAP), and RMA. It is also not capable of determining the effect of freeze-thaw cycles, and the effect of specialized methods of paving, including warm mix paving [Kaloush 2011]. These issues were not addressed in the current AASHTOWare version as of yet.

2.8 Chapter Two Summary

Recycling of scrap tire rubber has been used in the past, and is being currently used extensively in jurisdictions within the United States. While there is significant experience with the use of RMA in the US, there seems to be a lack of experience in Canada, more specifically in Ontario, regarding the use of this material.

Rubber Modified Asphalt is shown to add benefits to an asphalt mix when mixed and paved properly. Some of the benefits include added resistance to cracking, which

therefore decreases the need for costly maintenance and decreases the frequency and severity of the maintenance. The rubber also has the potential to extend the life of the pavement, and can potentially decrease tire noise and increase drive comfort. However, local validation of these benefits is still necessary.

RMA has been used in the United States in jurisdictions that have weather patterns that are significantly different than that faced in Ontario. Since there is little to no experience with the newer RMA mixes in Ontario, it is therefore important to investigate the proper use of RMA in Ontario and expose it to the conditions that are faced by conventional HMA mixes. One of the most important factors when designing a road in Ontario is the cold weather performance of the asphalt mix. Since the extensive use of RMA in the US is in hot and dry climate, there is a need to conduct an investigation on the performance of the RMA pavements when exposed to cold and wet weather. This is the aim of this thesis.

Chapter 3. Construction of MTO Test Sections

In 2011, the Centre for Pavement and Transportation Technology (CPATT) at the University of Waterloo, along with the Ontario Tire Stewardship (OTS), MTO, and the Ontario Hot Mix Producers Association (OHMPA) collaborated to construct trial sections utilizing rubber modified asphalt. These trials were carried out to evaluate the use of RMA in Ontario. The MTO had selected sections to be used for the trial. Table 3.1 shows the candidate sections that were selected to be paved with RMA. Only three sections were paved with RMA and incorporated into the study in this thesis, and they were Highway 7, Highway 35, and Highway 115. The other candidate sections were either switched to conventional HMA mix or delayed until the following year due to some logistical complications that were faced during that time [Hegazi 2013].

Table 3.1 Highways and mixes Selected for RMA Pilot Projects in 2011

Highway	Mix Type	Lane (km) Placed	Tonnage Placed	Date Completed	Comments
Highway 3	Field blend SP 9.5 R Gap-Graded	0	0	Rubber section was cancelled	Problems with rubber gradation and mix design.
Highway 60	Field blend SP 12.5 R Gap-Graded	0	0	Rubber section was cancelled	Problems with the mix design and blending equipment.
Highway 7	Field blend SP 9.5 R Gap-Graded	6	3600	Project Completed in October 2011	Night paving
	Terminal Blend SP 12.5 FC2 R Dense-Graded	5.6	3100		
	Hot Mix Asphalt SP 12.5 FC2 Dense-Graded	2.8	4800		

Highway 35	Field Blend SP 12.5FC1 R Gap-Graded	6	4400	Project Completed in October 2011	Paved as specified
	Hot Mix Asphalt SP 12.5 FC1 Dense-Graded	13.5	11200		
Highway 115	Field Blend SP 12.5 FC2 R Gap-Graded	6	4100	Project Completed in October 2011	Paved as specified
	Hot Mix Asphalt SP 12.5 FC2 Dense-Graded	2	1350		
Highway 24	Field blend SP 9.5 R Gap-Graded	0	0	Rubber section was cancelled	Problems with mix design and blending equipment.
Highway 60	Terminal blend SP 12.5 R Dense-Graded	4	2600	Project Completed in June 2011	Paved as specified
	Hot Mix Asphalt SP 12.5 Dense-Graded	17.6	12400		
Highway 35	Terminal blend SP 12.5FC1R Dense-Graded	1.0	800	Project Completed in June 2012	Paved as specifieds
	Hot Mix Asphalt SP 12.5 FC1 Dense-Graded	1.5	1300		

Most of the sections in Table 3.1 had either been cancelled, or paved with conventional HMA due to difficulty with the procurement of the blending unit, along with other challenges in the modification of the asphalt plants to connect the blending unit. The three highways that have been paved are Highway 7, Highway 35, and Highway 115. These sections were successfully paved with RMA during the paving season of 2011.

While the MTO had chosen the candidates to be used for the study, the OTS had provided the knowledge and also assisted in the supply of crumb rubber for the projects. Due to the experiences of RMA in Canada not positive and sometimes regarded as being negative experiences, the OTS was able to provide the channels for the experts in the US to transfer the knowledge of the US experiences. It was believed, by the OTS, that the previous Canadian experiences with RMA were not positive due to a lack of knowledge or expertise with the crumb rubber modifier. By hiring the experts from the US, the OTS was able to support the contractors in Ontario by providing them with the knowledge required for the trial sections to be properly implemented, and insuring that the trials with RMA do not fail due to a lack of knowledge regarding the material.

This chapter will discuss the sections that were paved with RMA as part of this research. All three sections were supplied asphalt from the same plant, and therefore the cost of mobilizing the field blend equipment was minimized.

3.1 Highway 7, Durham Region, Ontario

This highway consisted of three different sections. The sections were:

Field Blend Rubber Modified Asphalt SP9.5 R Gap Graded
Terminal Blend Rubber Modified Asphalt SP12.5 R Dense Graded
Control Hot Mix Asphalt SP12.5 Dense Graded

The three sections each have a different mix for the purposes of comparisons between the performance of conventional HMA (SP12.5FC2 HMA), Rubber Terminal Blend (SP12.5FC2 R RMA), and Rubber Field Blend (SP 9.5R Gap Graded). The required mix properties for each of these sections are listed in Table 3.2.

Table 3.2 Mix Properties for Highway 7 Test Sections

Hot Mix Asphalt Type	Percentage Passing by Dry Mass of Aggregates						Traffic Category	Base PGAC Grade	% Air Voids	% VMA min.	AC_{BID} (%)
	Sieve Size, mm										
	19.0	12.5	9.5	4.75	2.36	0.075					
SP 12.5FC2 R RMA	100	90-100	45-90	45-55	28-58	2-10	D	58-28	4	18	5.2
SP 12.5FC2 HMA	100	90-100	45-90	45-55	28-58	2-10	D	58-28	4	14	5.2
SP 9.5 R-Gap Graded	100	90-100	28-42	15-25	5-15	2-7	D	58-28	4	18	7.0

The mixes presented in Table 3.2 were placed in the RMA sections on the Highway 7 in Durham Region, Ontario. The first mix, the SP12.5FC2 R is the terminal blend rubber mix. The aggregate gradation for the SP12.5FC2 mixes are the same, regardless of the added rubber in the terminal blend mix, as the rubber is pre-mixed as a product of the AC. The difference between the mixes is the minimum percent Voids in Mineral Aggregate (VMA), where the SP12.5FC2 demands a minimum of 14%. The AC percent used in the SP9.5 R-Gap Graded is higher than the SP12.5 mixes. This could be due to the additional AC needed for the blending unit to ensure proper coverage of the rubber with AC. The rubber also has some absorption property that absorbs some of the AC that is added to the rubber. The rubber also has a gradation requirement, and it is listed in Table 3.3.

Typically, an SP9.5 mix would be composed of a 100 percent passing of the 12.5 mm size aggregate. However, in table 3-2, the SP9.5 R mix shows a 90-100 percent passing for this size [OPSS 1151, 2006]. While this mix is still considered a SP9.5 mix, it differs slightly in the aggregate gradation than a typical SP9.5 HMA mix. This may be

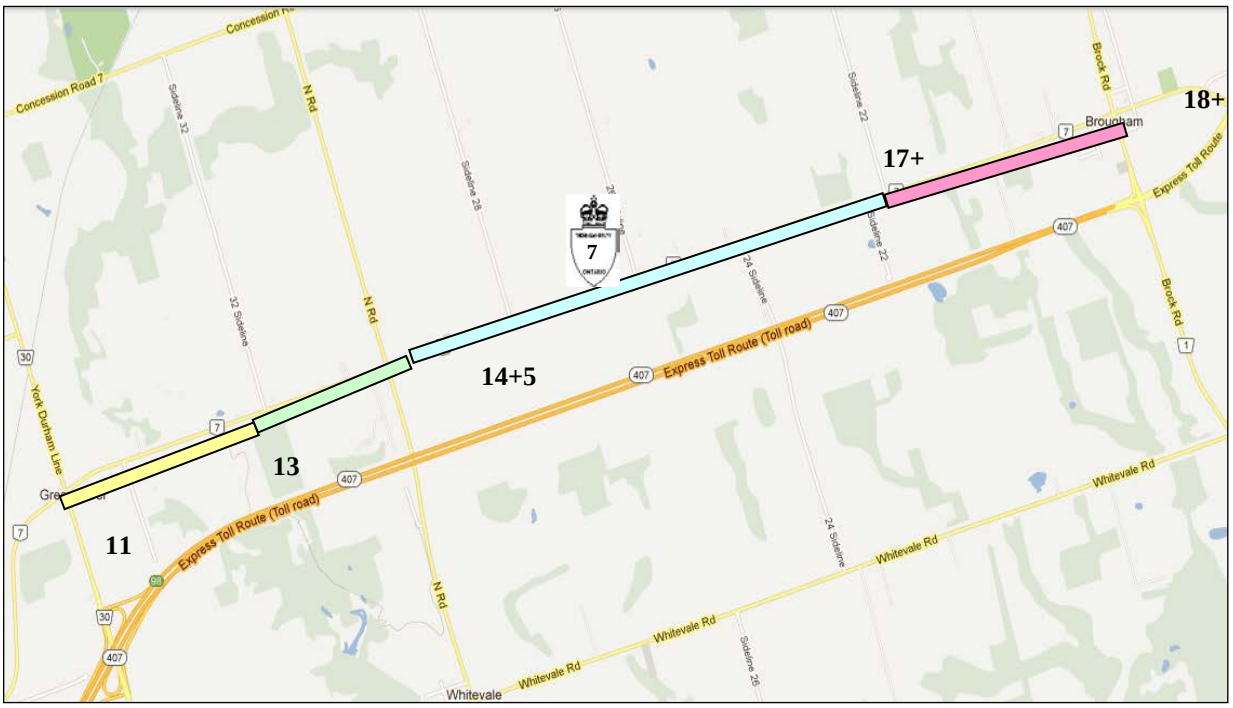
due to the addition of rubber particles to the AC, which need to fit into the matrix for the asphalt concrete. While the rubber particles are small as compared to the aggregate in the mix, it still needs to meet the Ontario pavement specifications for the SP9.5 mix.

Table 3.3 Rubber Particle Size Properties for Highway 7 Mixes

Process	Sieve	Percent Passing Sieve by Mass
Wet-Terminal Blend (see Note)	2.36 mm	100
Wet-Field Blend	2.36 mm	100
	2.00 mm	98 – 100
	1.18 mm	45 – 75
	600 µm	2 – 20
	300 µm	0 – 6
	150 µm	0 – 2
Note: The RAC supplier shall select a gradation suitable for producing a Type III Asphalt Rubber Binder according to ASTM D6114 and this non-standard special provision in addition to meeting the minimum requirements of OPSS 1101 for the PGAC grade specified elsewhere in the Contract Documents.		

Figure 3.1 shows the locations of the sections for the trials and station numbers where the different sections are located. There are four different sections that were paved with three mixes. From east to west, the first section was paved with 40 mm of SP12.5FC2 surface course with an underlying 50 mm SP19.0 binder course layer. The second section is the SP9.5 R Gap Graded (Rubber Field Blend) over 50 mm of SP19.0 binder course. The third section was paved with 40 mm of SP12.5FC2 R (Rubber Terminal Blend) on top of a SP19.0 binder course paved at 50 mm. The final section was paved similar to the third section, but differs in that the old pavement was milled 90 mm in the fourth section as opposed to only 50 mm being milled in the third. The sections are

summarized in Figure 3.1. It should be noted, though, that this section's SP9.5 R Gap Graded section (the field blend RMA) was paved at a thickness of only 30 mm as opposed to the remaining sections being paved 40 mm thick.



LEGEND	
	Mill 90mm, Pave 40mm SP12.5FC2 R Surface Course over 50mm SP19.0 Binder Course
	Mill 50mm, Pave 40mm SP12.5FC2 R Surface Course over 50mm SP19.0 Binder Course
	Mill 50mm, Pave 30mm SP9.5 R Gap Graded Surface Course over 50mm SP19.0 Binder Course
	Mill 50mm, Pave 40mm SP12.5FC2 Surface Course over 50mm SP19.0 Binder Course

Figure 3.1 Section Layout for Highway 7 [Hegazi 2013] Map [Google Maps 2014]

This section was placed with two pavers and a Materials Transfer Vehicle (MTV). Three steel drum rollers were used on the project, and a rubber tire roller was not used based on the recommendations received by experts in the US. Rubber tire rollers can potentially result in pickup from the rubber in the asphalt mix. Figure 3.2 shows the steel drum rollers operating close together. Also, can be seen from Figure 3.2 is the paving was conducted at night.

Some smoke was visible from the paver during the paving operation. The smoke is visible in Figure 3.3, which was taken during the paving of the highway. Although there was visible smoke, the fumes did not pose any immediate health risks above and beyond what exists with conventional HMA paving [Burr et 2001].

The finished surface of the SP9.5R Gap Graded mix has a more distinct look than that of the conventional HMA mix. This is due to it being a gap graded mix as opposed to dense graded. Figure 3.4 shows the matrix of the asphalt pavement after it has been paved. As can be seen, it visually differs from that of a dense graded mix. This is due to the gradation of the pavement, allowing for more visual voids to be able to accommodate for the rubber particles that have been added to the mix.

Figure 3.2 Steel Drum Rollers Used During Paving

Figure 3.3 Paver and Smoke from Paver During Night Paving

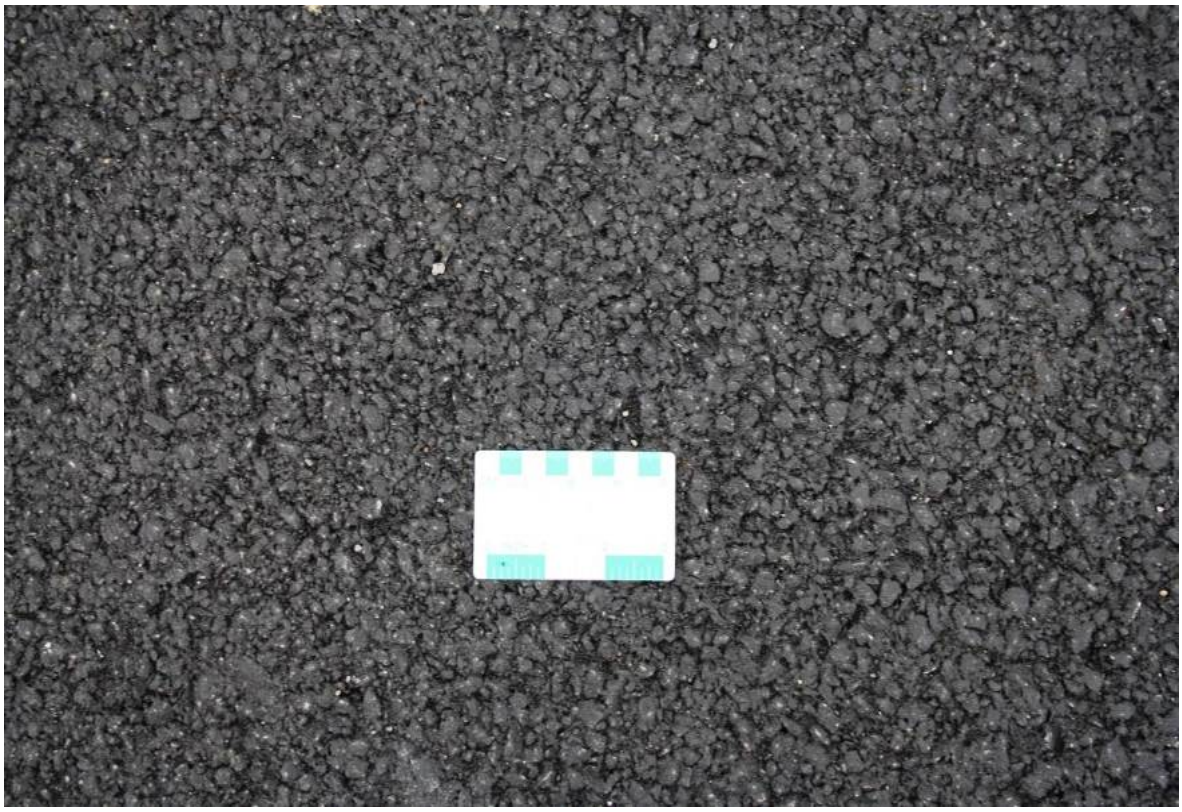


Figure 3.4 Finished RMA Surface

3.1.1 Current Condition

A field visit was conducted two years after the section had been paved. The purpose of the field visit was to conduct a visual inspection of the surface of the pavement, and to take note of any changes in ride quality or visible failures and/or distresses in the pavement.

The conventional HMA section for the highway visually looks like it has been performing up to standard and is to date performing very well. The conventional dense-graded mix shows no signs of distress, neither minor nor major distresses. It also shows no major signs of fatigue or deterioration. The only visible item on the pavement is the construction joint, which is visible at station 18+450. The pavement is performing well, and has a noticeably good ride quality.

The next section visited, the field blend RMA section, also showed very little pavement distress. Only very few, very slight transverse and longitudinal cracks were visible on the section with RMA near the Pickering Sideline 22. This occurrence is only localized though and limited to only that area, and therefore it may not be significant to the overall performance of the mix, but significant to the construction of that localized area. This section is performing well, as well as having an excellent ride quality, even compared to the conventional HMA section. It is also noticeably less noisy than that of the conventional section. The difference in ride quality and noise was only significant enough though to be just noticed without the use of any instrumentation.

The following section was a terminal blend RMA mix. This mix, visually and physically, is very similar to the dense-graded conventional HMA mix, as it is also dense-graded. This section has been and still is performing well, save a few minor distresses along the section. At the start of the section exists some centerline cracking, which is few and very slight in severity. The section otherwise is performing just as well as the other sections. As with the RMA field blend section, the ride quality on this section was and still is excellent, and also less noisy than that of the conventional HMA.

3.2 Highway 35, Kawartha Lakes, Ontario

Highway 35 was paved with two trial sections:

Field Blend Rubber Modified Asphalt SP12.5 FC1 R Gap Graded, and

Control Hot Mix Asphalt SP12.5 FC1 Dense Graded.

Table 3.4 shows the mix specification requirements for the mixes present on this Highway 35 section. Both mixes are SP12.5 mixes, although the RMA is gap graded, and the HMA is dense graded. The percent of AC for the RMA gap graded mix is again higher than that of the conventional HMA. This is to accommodate for the mixing to ensure full coverage of the rubber, and absorption of the AC by the rubber during the field blending process. As observed earlier, the aggregate gradation for the SP12.5FC1 R Gap Graded mix is atypical of that of a SP12.5 mix as specified by the MTO since the percent passing of the 19.0 mm aggregate is 90-100 percent rather than 100 percent [OPSS 1151, 2006]. The minimum percent VMA is also different between the RMA and HMA mixes, where 18 percent is demanded for RMA while 14 percent is demanded for the HMA mix. With the specified aggregate gradation, a gradation for the rubber particles being mixed into the AC is expected. The gradation for the rubber is listed in Table 3.5. The rubber gradation was selected based on the requirements for the mix and based on recommendations made from various expert advice received from those involved in jurisdictions in the U.S. that pave with RMA regularly.

Table 3.4 Mix Properties for Highway 35 Sections [Hegazi 2013]

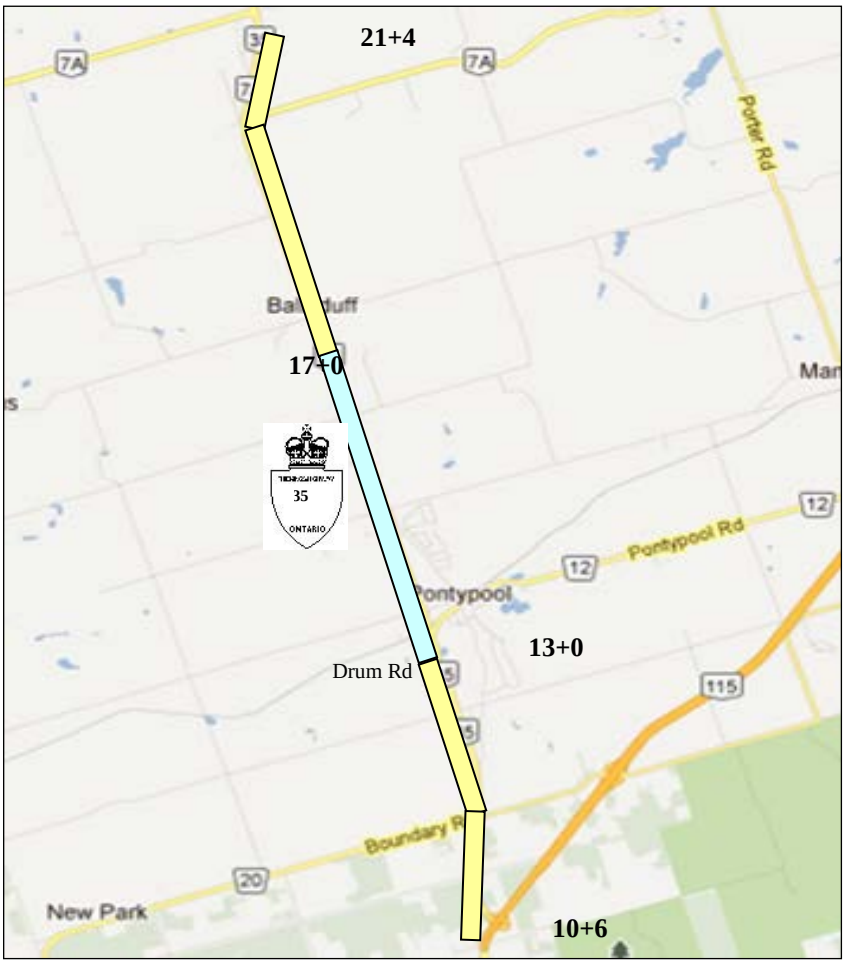
Hot Mix Asphalt Type	Percentage Passing by Dry Mass of Aggregates						Traffic Category	Base PGAC Grade	% Air Voids	% VMA min.	AC_{BID} (%)
	Sieve Size, mm										
	19.0	12.5	9.5	4.75	2.36	0.075					
SP12.5FC1 R Gap Graded	90-100	78-92	28-42	15-25	5-15	2-7	C	58-28	4	18	6.5
SP12.5FC1 HMA	100	90-100	45-90	45-55	28-58	2-10	C	58-28	4	14	5.2

Table 3.5 Rubber Particle Size Property for Highway 35 Rubber Mix [Hegazi 2013]

Process	Sieve	Percent Passing Sieve by Mass
Wet-Field Blend	2.36 mm	100
	2.00 mm	98 – 100
	1.18 mm	45 – 75
	600 µm	2 – 20
	300 µm	0 – 6
	150 µm	0 – 2

Figure 3.5 shows the start and end of the sections of two different sections of the Highway and its location on a map. There are only two different sections for this project, starting with the North most section being a section that is milled 20 mm, then paved with 50 mm of the SP12.5FC1 HMA surface course. The second section, which runs from station 17+000 to station 13+000, was milled 20 mm and paved with 50 mm of SP12.5FC1 R Gap Graded RMA field blend surface course. From station 13+000 to

station 10+650, the project was paved the same way as the first section. The sections are summarized in Figure 3.5 below.



LEGEND	
	Mill 20mm, Pave 50mm SP12.5FC1 Surface Course
	Mill 20mm, Pave 50mm SP12.5FC1 R Gap Graded Surface Course

Figure 3.5 Sections for Highway 35 Test Section [Hegazi 2013]. Map [Google Maps 2014]

During the construction process, a visit to the asphalt plant was conducted to observe the operations of the blending unit. Some of the key observations at the plant were as follows:

- The plant had to be modified to allow for the field blending unit to connect to directly into the system. Figure 3.6 shows the plant which was modified to allow for the connection of the blending unit. The oil tanks at the plant were modified to feed the AC to the rubber asphalt cement blending unit, and a valve was added to feed the plant the rubberized asphalt cement to produce the RMA.
- Additional smoke was detected at the plant and some odor was present, yet the surrounding community did not complain about any odor and it was deemed a non-issue.
- The blending unit was producing at a rate of 150 tons per hour while the plant could handle anywhere between 180 to 200 tons per hour. It was slower than capacity due to the rubberized asphalt cement being more viscous and therefore more difficult to work with than the conventional asphalt cement.

Figure 3.6 Asphalt Plant Used for Production of Highway 35 Mixes

Only one paver and one MTV were used, and three steel drum rollers. Rubber tire rollers were not used on this project to avoid pickup from the rubber in the field blended RMA section.

3.2.1 Current Condition

A field visit was conducted two years after the section had been paved. The purpose of the field visit was to conduct a visual inspection of the surface of the pavement, and to take note of any changes in ride quality or visible failures and/or distresses in the pavement.

The conventional HMA section on this project has performed well, yet with some pavement distresses present and fatigue starting to show. Moderate transverse cracking is visible and evident on the pavement that occurs every 1.5 to 2 metres. This cracking is fairly consistent throughout the section, and ranges from slight to moderate severity, with few that can be considered severe transverse cracks. Also evident in the section is centerline joint cracking that runs through most of the section. Edge cracking is also present occurring only a few times and with very low severity. The ride quality of this road is moderate, with the transverse cracking causing some slight discomfort in the ride.

The RMA section overall was performing better than the conventional HMA section. The pavement visually differs from that of the conventional HMA in that it looks gap-graded. Transverse cracking is still present in this section, yet it is less severe and less frequent than on the conventional HMA pavement. There is also centreline joint cracking, but again it is of very slight severity and few in frequency. The ride quality is greatly improved over the conventional HMA section, yet still some discomfort exists due to the transverse cracking on the highway.

Figure 3.7 Site Visit Photo

Figure 3.8 Site Visit Photo (Quarter for size reference)

3.3 Highway 115, Kawartha Lakes, Ontario

Highway 115 was paved with two trial sections:

Field Blend Rubber Modified Asphalt SP12.5 FC2 R Gap Graded, and Control Hot Mix Asphalt SP12.5 FC2 Dense Graded.

The Highway 115 mixes, located from Highway 7A to Tapley Quarter Line, Kawartha Lakes, Ontario, have similar specification requirements to that of Highway 35, as can be seen in table 3-6. The SP12.5FC2 RMA mix has a higher AC content of 6.5 percent due to the rubber absorbing the asphalt, and to ensure proper coverage of all the rubber when blended at the plant using the field blending unit. Also, the SP12.5FC2 R mix is atypical in the gradation of the aggregate percent passing. The rubber sections also require a specific gradation for the particle sizes, which is listed in Table 3.7. The gradation is required to be met in order to obtain the benefit of the rubber in the mix.

Table 3.6 Mix Properties for Highway 115 Test Sections

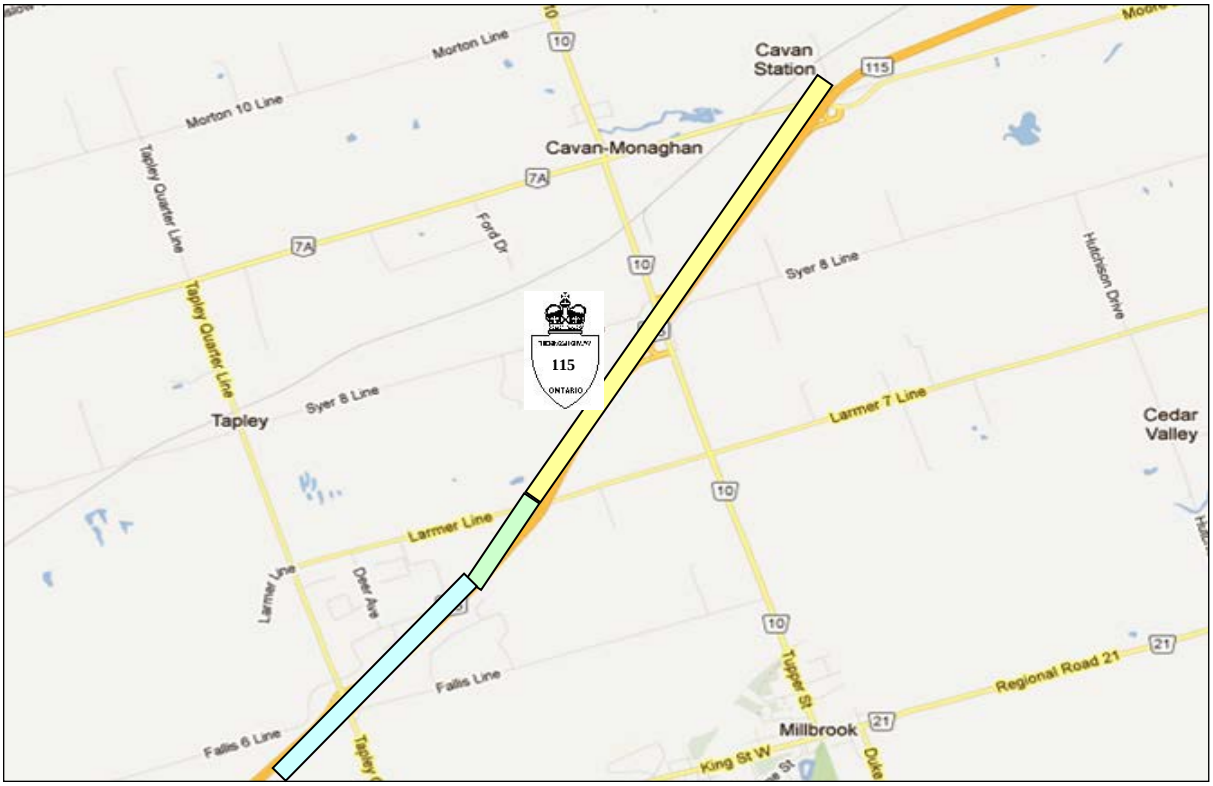
Hot Mix Asphalt Type	**Percentage Passing by Dry Mass of Aggregates**						**Traffic Category**	**Base PGAC Grade**	**% Air Voids**	**% VMA min.**	**AC_{BID} (%)**
	Sieve Size, mm										
	19.0	**12.5**	**9.5**	**4.75**	**2.36**	**0.075**					
SP 12.5FC2 R Gap Graded RMA	90-100	78-92	28-42	15-25	5-15	2-7	D	58-28	4	18	6.5
SP 12.5FC2 HMA	100	90-100	45-90	45-55	28-58	2-10	D	58-28	4	14	5.2

Table 3.7 Gradation Requirement Property for Rubber Particle Sizes for Highway 115 Rubber Mix [Hegazi 2013]

Process	Sieve	Percent Passing Sieve by Mass
Wet-Field Blend	2.36 mm	100
	2.00 mm	98 – 100
	1.18 mm	45 – 75
	600 µm	2 – 20
	300 µm	0 – 6
	150 µm	0 – 2

The traffic category D, as noted in Table 3.6, differs from the category "C" listed in the Highway 35 specification requirements. As noted earlier in this thesis, Category C represents 3 to 10 million design ESALS while Category D represents 10 to 30 million design ESALS. While the base PGAC grade is required to be 58-28 for this Highway, the actual paved PGAC was 64-34. This was selected to provide better resistance to the higher traffic loading of this highway and to ensure proper performance of the sections.

Figure 3.9 shows the configuration of the sections for Highway 115. There are three different sections paved. Starting from the north, the first section involves paving a 50 mm overlay of SP12.5FC2 R Gap Graded Surface Course. The second section is a 50 mm overlay paved with an SP12.5FC2 Surface Course. The third section is also a 50 mm paved overlay involving an SP12.5FC2 Surface Course, but as noted in the figure, this section will be outside the scope of the experiment as the underlying pavement was in worse condition than any of the other sections. With the underlying conditions being in the state it was in, it would be unwise to include this section as a control section for comparing the performance of the mixes.



LEGEND	
	Pave 50mm Overlay SUP12.5FC2 R Gap Graded Surface Course
	Pave 50mm Overlay SUP12.5FC2 Surface Course
	Pave 50mm Overlay SUP12.5FC2 Surface Course (**NOTE**: Original pavement in this section is in worse condition than the other two sections above. Therefore, this section should not be used as control for future performance comparisons)

Figure 3.9 Configuration of the Mixes for the Sections of Highway 115 [Hegazi 2013]. Map [Google Maps 2014]

One paver and one MTV were used on this paving project. Three steel drum rollers were used for compacting, two of them on vibratory mode and the third in static mode. The drum rollers were following in close proximity of the paver to ensure proper compaction temperatures, as shown in Figure 3.10. There were less hauling trucks for this project than the others.

Also, it should be noted that on this project, the underlying pavement was not milled prior to paving the overlay. While this is the case, it was noted that the original pavement

condition of the underlying pavement is worse condition than the other trial sections. This could affect the performance results of the rubber modified asphalt when it is being compared to the other sections.

Figure 3.10 Steel Drum Roller Following Closely to the Paver on Highway 115 Construction

3.3.1 Current Conditions

Highway 115 was visited during a field visit to visually inspect the condition of the pavement two years after it has been paved. The surface of the pavement is inspected visually for cues of how the pavement is performing in terms of ride quality and fatigue. Although the site investigation was brief to begin with, there was an added difficulty on this highway of it having more traffic and being a higher volume highway than the other sections. It was therefore more difficult to safely conduct the same visual inspection as on the remaining projects.

The conventional HMA section is performing well, although there are many transverse cracks that exist on this pavement. The transverse cracks are fairly severe, and occur frequently. It was noted that the northbound lane was performing worse compared to the southbound lane. The section continues to be performing well, as the ride quality on the road is still very good.

The RMA section on this highway shows some sign of distress, but it is few and very minor. There is evidence of some low severity transverse cracking, which do not occur

very often, and some centerline joint cracks that are evident yet not severe. While the pavement is starting to show slight cracking, the ride quality is still excellent, and again, the RMA section seems to have a better ride quality than that of the conventional HMA section.

Figure 3.11 Highway 115 SP12.5 Dense-Graded Conventional HMA

Figure 3.12 Highway 115 SP12.5 Gap-Graded Rubber Mix

3.4 Lessons Learned From 2011 Pilot Study

The 2011 pilot study of RMA pavements presented a number of experiences. These experiences can be summarized into lessons learned with regards to paving with RMA. Some of these experiences and lessons as follows:

- When paving with RMA, it is important to know that the RMA, specifically field blend mixes, require a gap-graded mix in order to obtain a useable mix design.
- Ensuring the rubber being supplied for the RMA mix falls within the gradation specification required is essential to obtaining a mix design that would present with no swelling and useable for paving.
- The blending unit for the mixing of the RMA mixes requires some minor modifications to the asphalt plant producing the mix. Although the modification is not extensive, the contractor should be informed well ahead of time to schedule the modifications of the plant between operations.
- The blending unit for mixing of RMA is not available locally, although it can be contracted from Alberta or New York. It was found that Alberta was easier to avoid the complications of cross-border mobilization of the plant. FATH industries from Alberta supplied the blending unit for the 2011 pilot projects, where they charged a per tonne fee for blending of mix.
- One recommendation that was concluded from the 2011 pilot study, is for either the MTO, or a private consultant/contractor to purchase one blending unit to be used in Ontario when needed, and have the paving contractors rent and/or contract the blending unit when needed to produce RMA mix.
- It is preferred to ensure a constant flow of RMA mix when paving as to not have stop-and-go paving operation. Temperature is very critical when paving RMA, and therefore it is important to have continuous operation especially with the rollers following closely behind the paver.
- Only steel drum rollers are to be used, rubber tire rollers would cause pickup from the rubber that is paved. It is also recommended to allow the paved surface to cool sufficiently before allowing traffic or any other vehicles to drive on the pavement.

The RMA, when hot, is very sticky, and driving on it before it cools can also cause pickup.

Chapter 4. Laboratory Testing

This chapter describes the laboratory testing that was performed at the CPATT laboratory to determine the performance of the field mixes in cold weather. The results of the laboratory testing were analyzed and are presented in this chapter.

4.1 Material Testing System

The laboratory testing that was carried out for this research was done on the Material Testing System (MTS) provided by CPATT. The MTS consist of an environmental chamber, a load frame, hydraulic power supply, and a control panel all controlled through the MTS computer software. Figure 4.1 below shows the MTS 810 that was used for the TSRST testing.

Figure 4.1 Material Testing System 810 with Environmental Chamber

4.2 Thermal Stress Restrained Specimen Test (TSRST)

The Thermal Stress Restrained Specimen Test (TSRST) is used to observe the behavior of the mixes at cold temperatures. This behavior is characterized by the

temperature at which the mixes fail, the stress that is endured by the mix, and the general performance of the mix as the temperature in the environmental chamber changes. These characteristics are important for pavement design and structural analysis to help reduce the susceptibility to thermal cracking and elongate the life of the pavement. The tests were performed as per AASHTO TP 10-93 ("Standard Test Method for Thermal Stress Restrained Specimen Tensile Strength").

4.2.1 Sample Preparation

The laboratory specimens were prepared using the samples that were collected in the field during the paving of the test sections. The samples were transported to the CPATT laboratory to be prepared for testing. The preparations involved compacting the specimens into large beams, which were cut down to size to meet the AASHTO TP 10-93 specifications for testing.

To compact the samples collected from the field, the AVC in the CPATT laboratory was used. The AVC compacts the beams that have dimensions of 300 mm x 125 mm x 78 mm by applying vibration and pressure for a set amount of time to obtain the beam with a target air void content of 7% +- 1%. Once the compacted beams are obtained, they are saw cut down to 250 mm x 50 mm x 50 mm with a tolerance of +- 4.0 mm for each side.

Four samples from each mix, for a total of 28 samples, were compacted for the testing. Each sample was compacted at a pressure of 110 kPa, and the time of compaction varied with the different mixes to obtain the target 7% air void content. Table 4.1 shows a summary of the air voids of the compacted TSRST samples after they have been cut to the specified sample sizes.

Table 4.1 Summary of Mixes and Air Void Percentages for TSRST Samples

Mix Identification	Mix Description	Specimen Numbers	PG Grade	Air Voids (%)
M1	Highway 7 SP12.5 Dense Graded HMA	7-C-4-1 7-C-4-2 7-C-5-1 7-C-5-2	58-28	6.1% 6.2% 7.8% 8.0%
M2	Highway 7 SP9.5 R Gap Graded RMA (Field Blend)	7-R-FB-2-1 7-R-FB-4-1 7-R-FB-5-1 7-R-FB-5-2	58-28	6.7% 7.1% 7.4% 8.5%
M3	Highway 7 SP12.5 R Dense Graded RMA (Terminal Blend)	7-R-TB-1-1 7-R-TB-1-2 7-R-TB-3-1 7-R-TB-3-2	Duratirephalt (58-28)	7.0% 7.8% 7.8% 7.0%
M4	Highway 35 SP12.5 FC1 Dense Graded HMA	35-C-2-1 35-C-2-2 35-C-5-1 35-C-3-2	58-28	6.6% 8.1% 6.4% 6.0%
M5	Highway 35 SP12.5 R FC1 Gap Graded RMA	35-R-2-1 35-R-2-2 35-R-4-1 35-R-4-2	58-28	6.6% 8.1% 6.1% 6.1%
M6	Highway 115 SP12.5 FC2 Dense Graded HMA	115-C-1-1 115-C-1-2 115-C-2-1 115-C-6-1	64-34	6.4% 7.3% 7.4% 6.0%
M7	Highway 115 SP12.5 R FC2 Gap Graded RMA	115-R-1-1 115-R-1-2 115-R-2-1 115-R-2-2	64-34	6.4% 8.1% 8.3% 7.7%

Thereafter, the samples are epoxied to two platters prior to testing using Loctite 608 Hysol Epoxy as per the specification in AASHTO TP 10-93. To carry out the test, the specimen must be securely fastened to the end platters to prevent any elongation or contraction in the specimen while the test is carried out [AASHTO TP 10-93].

4.2.2 TSRST Sample Instrumentation and Testing

The compacted and cut beams are tested in the MTS 810 after the beams have been epoxied to the end plates and fitted with instrumentation. The instrumentation used for this test involves the use of two extensometers placed on opposing sides of the sample. A thermometer is also placed on the sample for more accurate collection of temperature measurement.

The specimens are placed into the MTS 810 chamber, bolted to the hydraulic actuator. The test is designed to keep the sample at a constant length as the sample is cooled, allowing for stresses to develop in the sample. The stresses that the sample withstands are measured, and the sample is cooled at a constant 10 degrees C/hour until it fractures and/or fails.

The specimen is cooled in the environmental chamber to 5 degrees, at which the stress in the sample is negligible. At this temperature, the sample is conditioned for 6 hours, after which the temperature starts to drop at a rate of 10 degrees C per hour. The test continues until a failure is detected.

4.3 TSRST Results

The TSRST testing was carried out using the MTS 810 in the CPATT laboratory, with the specimen placed in the environmental chamber and conditioned for six hours at 5°C. After the conditioning has been completed, the specimen is cooled at a rate of 10 degrees per hour until the specimen fails. The temperature at which the specimens fail is then recorded, and is summarized in Table 4.2. Most of the mixes have exceeded their expected PG grade performance, with the exception of Mix M6 (Highway 115, 12.5 FC2 Dense Graded HMA).

Table 4.2 Failure Temperature and Failure Stress for Tested TSRST Samples

Mix	Beam	Binder PG	Failure Temp (°C)	Failure Stress (MPa)	Mean Failure Temp (°C)	Mean Failure Stress (MPa)
M1	4-1	58-28	-31.62	2.40	-33.15	2.48
	4-2		-33.46	2.62		
	5-1		-32.78	2.33		
	5-2		-34.73	2.57		
M2	2-1	58-28	-34.54	2.34	-35.95	2.47
	4-1		-32.18	2.87		
	5-1		-47.06	2.47		
	5-2		-30.01	2.19		
M3	1-1	58-28	-36.26	3.10	-41.90	2.71
	1-2		-44.24	2.90		
	3-1		-43.38	2.29		
	3-2		-43.73	2.54		
M4	2-1	58-28	-26.94	3.10	-32.96	2.85
	2-2		-33.96	3.10		
	5-2		-24.19	2.29		
	3-2		-26.84	2.90		
M5	2-1	58-28	-46.88	1.92	-43.43	2.06
	2-2		-33.24	2.05		
	4-1		-46.9	1.99		
	4-2		-46.7	2.25		
M6	1-1	64-34	-30.07	2.43	-29.87	2.64
	1-2		-28.30	2.46		
	2-1		-32.11	3.16		
	6-1		-28.99	2.51		
M7	1-1	64-34	-30.38	2.28	-38.34	2.23
	1-2		-42.82	2.56		
	2-1		-33.23	1.80		
	2-2		-46.92	2.28		

Comparing the fracture temperature of the Highway 7 mixes (M1, M2, and M3), it is evident from Table 4.2 that the performance of the rubber modified sections fracture at a lower temperature than that of the conventional HMA mix. The temperature of the mixes are shown to exhibit better performance, but also the fracture stress is lower for the rubberized mixes as compared to the HMA mixes.

The fracture temperatures for the Highway 35 mixes (M4 and M5) also exceed that of the specified PG grade for the mix. It should be noted, also, that the fracture stress of the rubberized mix is much lower than that of the conventional HMA mix.

The temperatures at fracture for the Highway 115 mixes (M6 and M7) vary greatly as seen in Table 4.2. Mix M6 had not performed up to the standard of the PG grade of -34°C, with an average fracture temperature of -30°C. it tested at 4°C lower than that specified for the mix. Although this mix is not performing as well as other mixes, similar to the other mixes, the rubberized mix fails at a lower temperature than that of the conventional HMA mix. Also, the fracture stress is lower for the rubber mix.

When observing the stress-temperature curves for the specimens, it is evident that the fracture of the RMA mixes occurs at a lower temperature than the HMA mixes, yet they behave in the same manner when cooled at a rate of 10 degrees per hour. Figure 4.2 below shows the stress temperature curve for the Highway 7 SP12.5 Dense Graded HMA mix, Figure 4.3 shows the curve for the Highway 7 SP9.5 R Gap Graded (Field Blend) RMA mix, and Figure 4.4 shows the curve for the Highway 7 SP12.5 R Dense Graded (Terminal Blend) RMA mix.

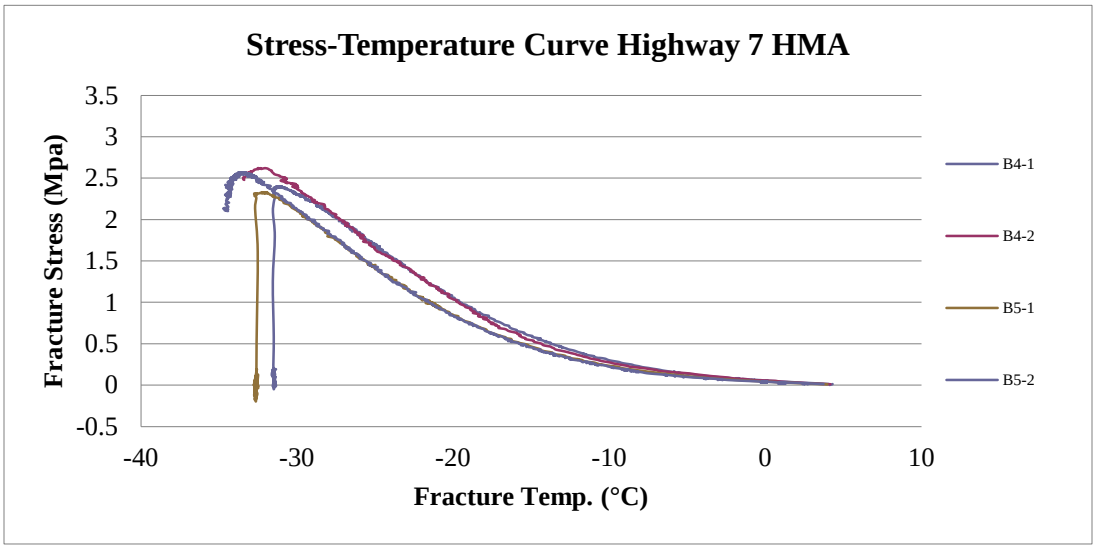


Figure 4.2 Highway 7 HMA Stress-Temperature Curve

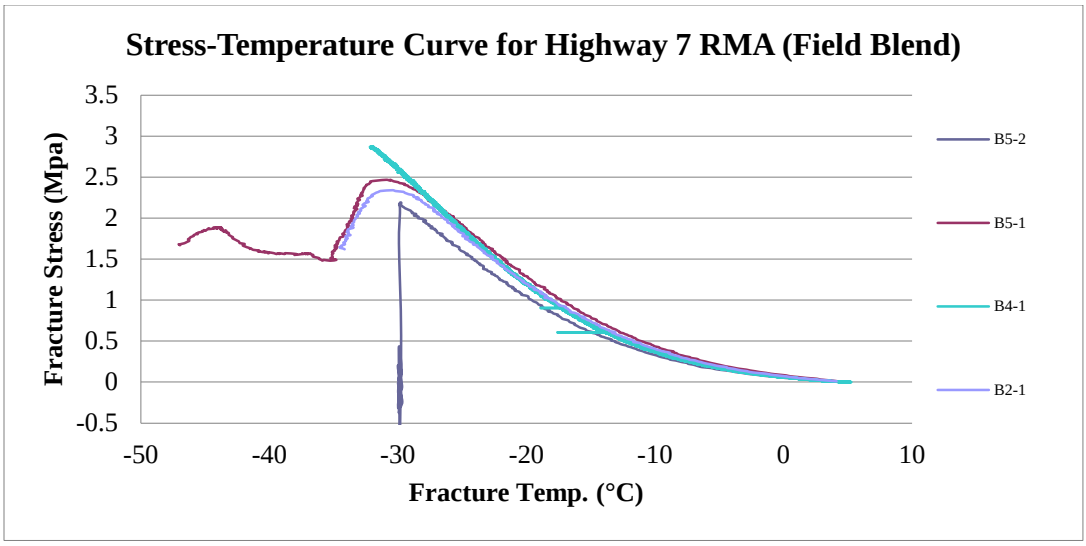


Figure 4.3 Highway 7 RMA Field Blend Stress-Temperature Curve

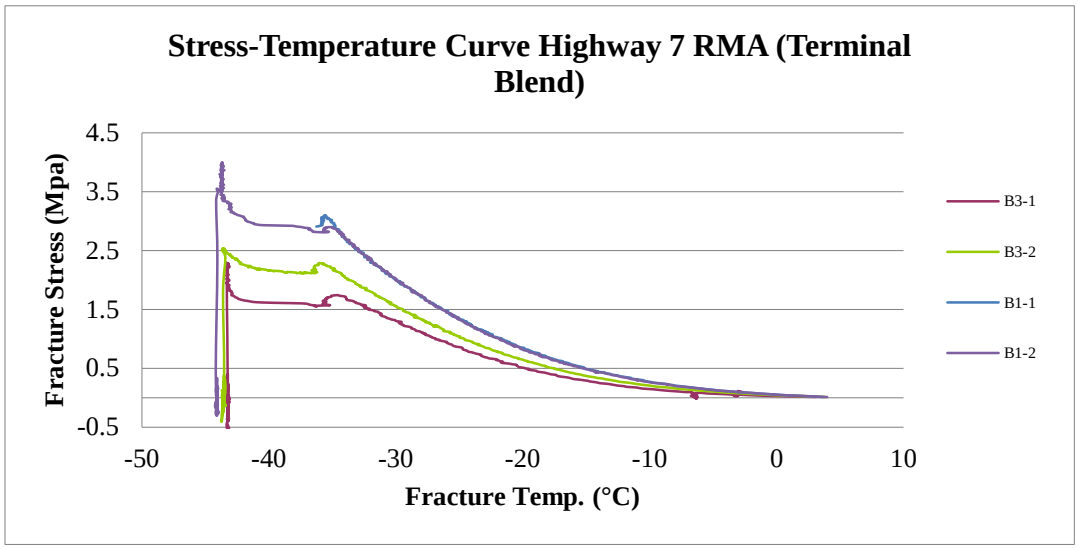


Figure 4.4 Highway 7 RMA Terminal Blend Stress-Temperature Curve

The figures for the stress temperature curves for Highway 7 HMA in Figure 4.2 exhibit typical TSRST results. The temperature decreases at a rate of 10°C per hour until the failure at approximately -33°C. Immediately following this, the curve shows a decrease in stress. At the peak of the curve is the fracture temperature and stress, and as can be seen in Figure 4.2, the curve is consistent for all four of the tests.

The Highway 7 RMA Field Blend stress-temperature curves in Figure 4.3 show the behavior of the RMA field blend mixes when cooled in the MTS chamber. The mixes

begin by behaving in the same way the conventional HMA behaves, up until it reaches approximately the temperature where the conventional HMA fails. Beam B5-1 exhibited a strange behavior after it appeared to fail at approximately -32°C, where the stress decreases as the temperature continues to drop, it then fluctuates between 1.5 and 2.0 MPa until approximately -45°C. It is not known why this beam has behaved in this manner. While this beam behaved in this manner, other beams that were tested behaved similar to the conventional HMA mixes, yet failing at a lower stress than that of HMA.

The stress-temperature curves for the Highway 7 RMA Terminal Blend mixes in Figure 4.4 look very identical for all four beams tested. As the temperature drops, the recorded stress increases in the beams slower than that of the conventional HMA. While some beams show more stress than others, the general shape of the curve remains the same for all the beams. At approximately -34°C, the stress drops, then proceeds to increase as the temperature continues to drop. This drop in temperature continues, while the stress remains constant until it reaches approximately -44°C, where the fracture stress spikes. At this point, the specimen shows a sudden failure. One theory of this phenomenon is that the asphalt cement binder is failing at the higher temperature, and then the beam is able to withstand a lower temperature drop without failing further until the rubber reaches a brittle failure. Similar to the curve for beam B5-1 for the terminal blend mix, it is not known why the beams behave with the flattening of the stress curve after -35 degrees then failing, but it is a phenomenon recommended for further investigation.

Comparatively, the conventional HMA performs very similar to the RMA field blend mixes, where the average fracture temperature is very close (-33°C versus -35°C for the HMA and RMA, respectively). The average stress is also very similar (2.48 MPa versus 2.47 MPa for HMA and RMA, respectively). Although the field blend performs similar to the HMA mix for this highway, the terminal blend does differ from both mixes, with a higher average fracture temperature and higher stress. The terminal blend mixes failed at an average temperature of approximately -41 degrees, and failed with an average stress of 2.71 MPa. Although it may seem like the RMA field blend does not perform any different from the conventional HMA mix, the RMA terminal blend did differ. It was able to withstand a lower temperature, but it had exhibited a larger stress in the beam

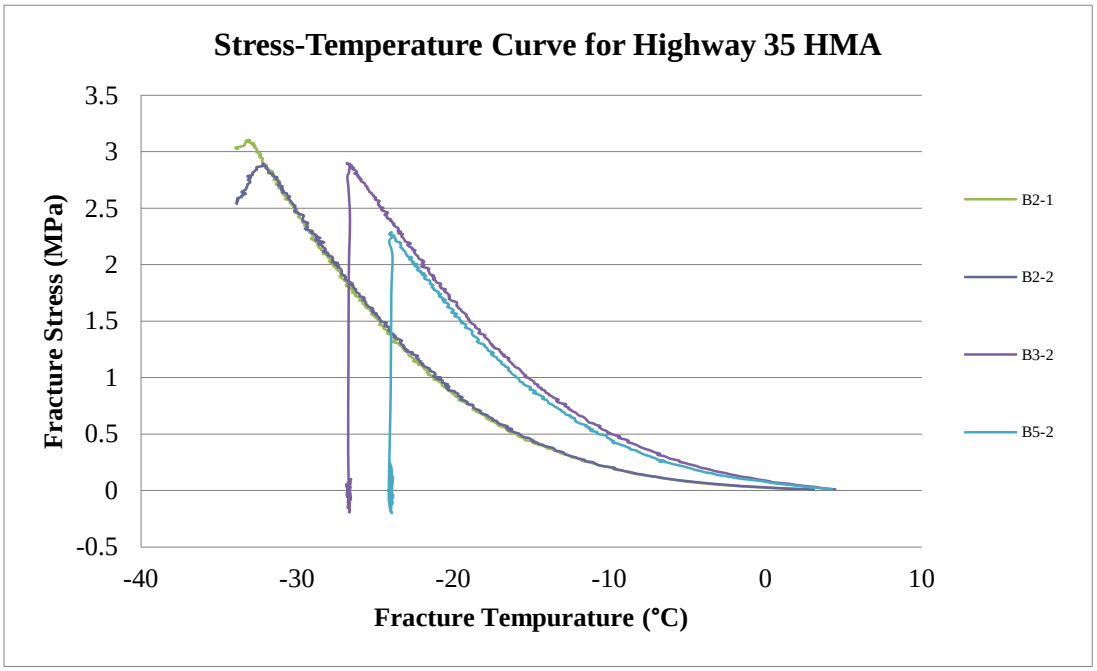


Figure 4.5 Highway 35 HMA Mix Stress Temperature Curve

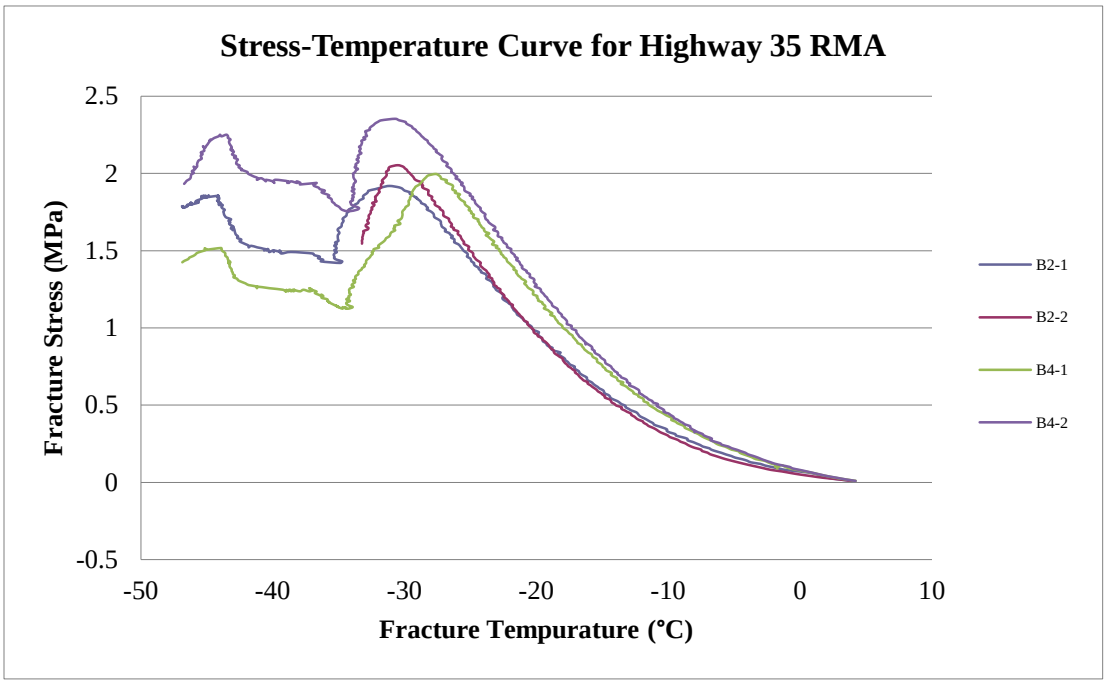


Figure 4.6 Highway 35 RMA Mix Stress-Temperature Curve

Figure 4.5 and Figure 4.6 show the stress-temperature curves for Highways 35 HMA mix RMA mix, respectively. The same pattern is observed for the Highway 7 mixes, where the conventional HMA mixes and the RMA mixes behave similarly, yet fail at different temperatures and stress values. Figure 4.5 (Highway 35 HMA mix) shows the mix being cooled until it fails at approximately -30°C average, where the curve shows failure. Although the mix is specified to be a -28°C mix, beam B3-2 and B5-2 performed under the specification for the mix. The fracture stress of this HMA mix averaged at a very high 2.85 MPa.

Figure 4.6 displays the stress-temperature curve for the RMA mix for Highway 35. This mix performed well, while exhibiting an average failure temperature of -43°C and a failure stress of 2.06 MPa. The same phenomenon was observed for the Highway 7 Terminal Blend RMA mix (M3). The beam cracks at -33°C, at which it exhibits the behavior of a failed beam. However, it is notable that the stress drops approximately 0.5 MPa and is able to withstand that constant stress while the temperature continues to drop. The internal stress in the beam is able to withstand an additional 10°C drop, after which the beam then shows a sudden failure.

Comparatively, the Highway 35 RMA mix fractures at a slightly lower temperature than that of the conventional HMA mix. The fracture stress is also much lower on the RMA mix than the conventional mix. This mix is also exhibiting a recovery type behavior as the stress decreases but the sample is able to withstand additional temperature decreases until brittle failure.

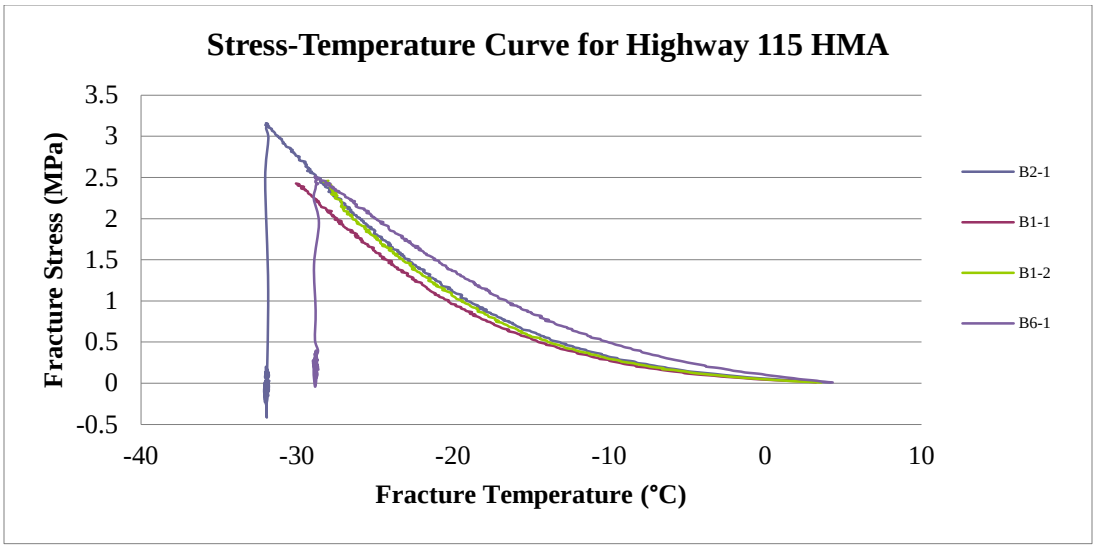


Figure 4.7 Highway 115 HMA Mix Stress-Temperature Curves

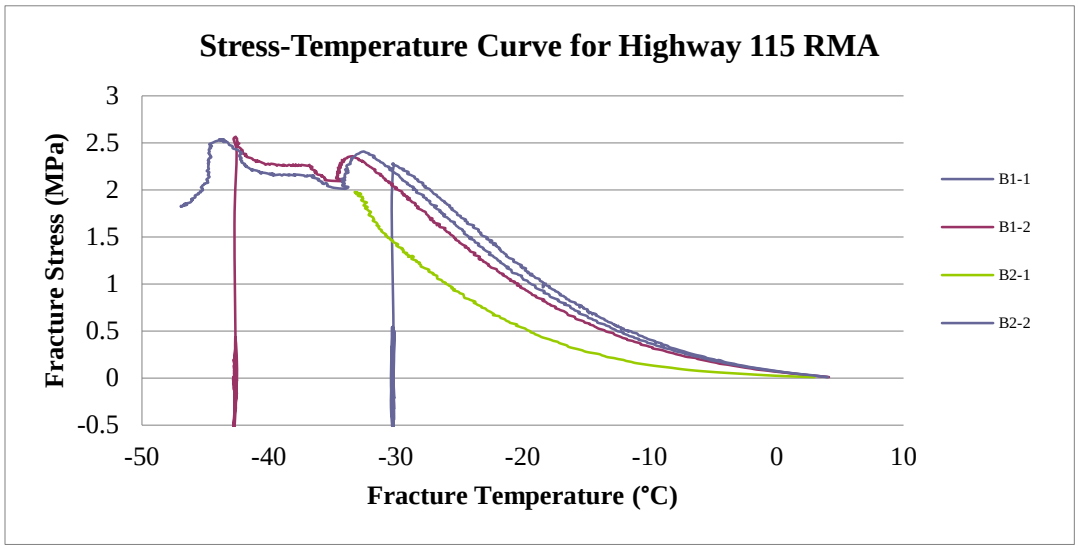


Figure 4.8 Highway 115 RMA Mix Stress-Temperature Curves

Figure 4.7 and Figure 4.8 show the stress-temperature curves for Highway 115 Conventional HMA and RMA, respectively. The conventional HMA mix for the Highway 115 section again behaves similar to the conventional HMA mixes in the previous sections. The mix is cooled until approximately -30 degrees at a failure stress of approximately 2.68 MPa. The area of concern with this mix is that the mix is underperforming, as the PG grade of the mix is PGAC 64-34. The mix is failing at a

temperature of approximately -30°C. Also, the mix fracture stress is higher than that of the RMA mix.

The RMA mix for Highway 115 behaves similar to the RMA mixes for the other highways. The mix fails at an average temperature of -38°C, with a low fracture stress of 2.23 MPa. The behavior of the beams is similar to that of the other RMA mixes, where the temperature drops to approximately -35°C, then the stress drops approximately 0.5 MPa, and continues to withstand a further drop in temperature. When the beam fails, it exhibits a sudden failure, yet the failure stress is still lower than that of the conventional mix.

4.4 Analysis and Discussion

Each highway section was paved with an RMA and a conventional HMA control mix to allow for comparison of performance. The highway sections are therefore compared side by side to determine the additional performance benefits of RMA over HMA.

The comparison of the three mixes present on the section of Highway 7 is evident in Figure 4.9. The RMA field blend mixes show very similar performance to that of the conventional HMA mixes in this case. The RMA field blend mixes do fail at a slightly lower stress and lower temperature, but it is not significant as compared to the conventional HMA. One of the differences between the Highway 7 RMA field blend mixes and the other RMA mixes on the remaining highways is the fact that it is the only mix that is a 9.5 mm nominal size mix. While this may not attribute fully to the fact that the mix is not outperforming the conventional HMA mix, but it may be a contributing factor, as other RMA mixes have outperformed the HMA mixes. While there may be other factors, it is suspected that the fact that Highway 7 uses SP9.5 mix for the RMA while the other highways use SP12.5 mixes may contribute to the performance.

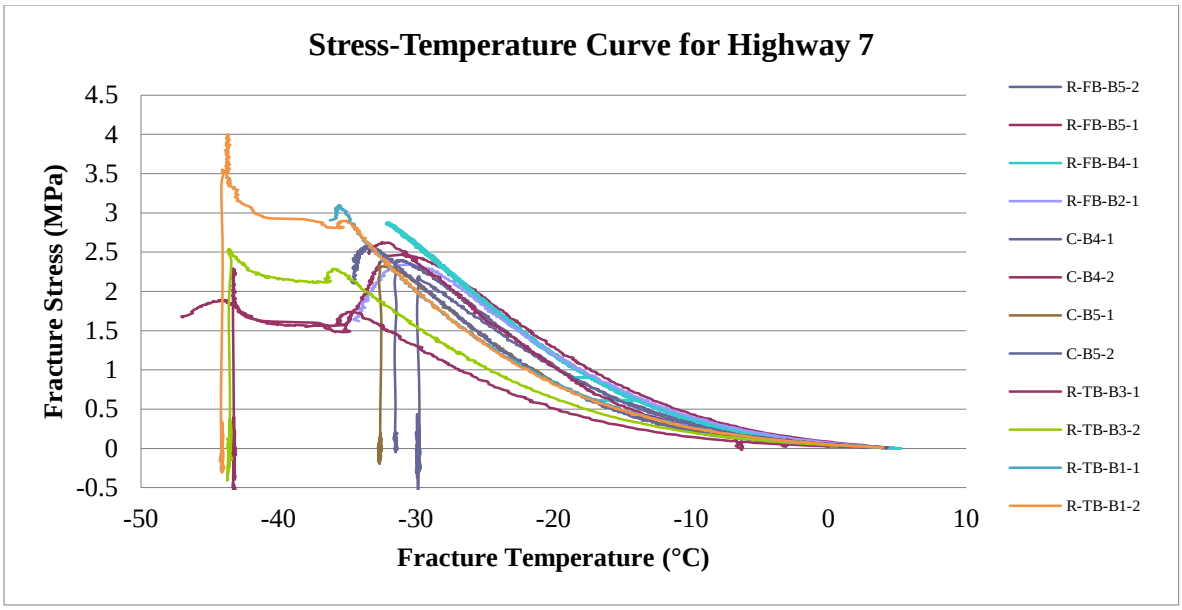


Figure 4.9 Stress Temperature Curves for All Highway 7 Mixes

The behavior of the RMA terminal blend mixes was more interesting, as it displayed a strange pattern when compared to that of a normally observed TSRST result. This phenomenon occurs with only the rubber blends, as in Figure 4.9 for the Highway 35 mixes, it exhibits the same behavior in the rubber field blend mix.

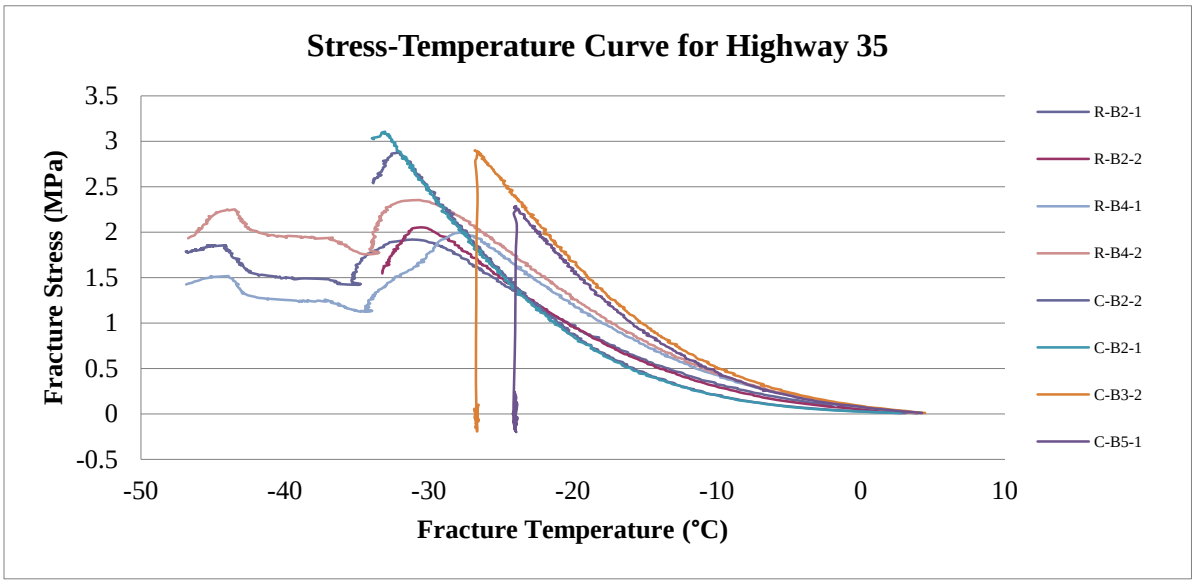


Figure 4.10 Stress-Temperature Curves for all Highway 35 Mixes

Figure 4.10 shows the comparison between the RMA field blend mix and the conventional HMA mix for Highway 35. The most evident difference between the mixes is the RMA's ability to perform at very low temperatures. As mentioned previously, the RMA mix is exhibiting a strange behavior when it approaches temperature of -35°C and lower. Although it exhibits this behavior, it is evident that the rubber mix is able to withstand lower temperatures, while keeping the internal stresses on the beam lower than that of the conventional HMA.

The behavior of the rubber mix is evident also in the Highway 115 mixes for RMA shown in figure 4-11. The stress-temperature curves for highway 115 shows the variation between the conventional HMA and the RMA mixes. The conventional HMA mixes go through a large stress before failure close to a temperature of -30°C. The RMA mixes exhibit the strange behavior of a drop in stress and withstanding lower temperatures until a brittle failure.

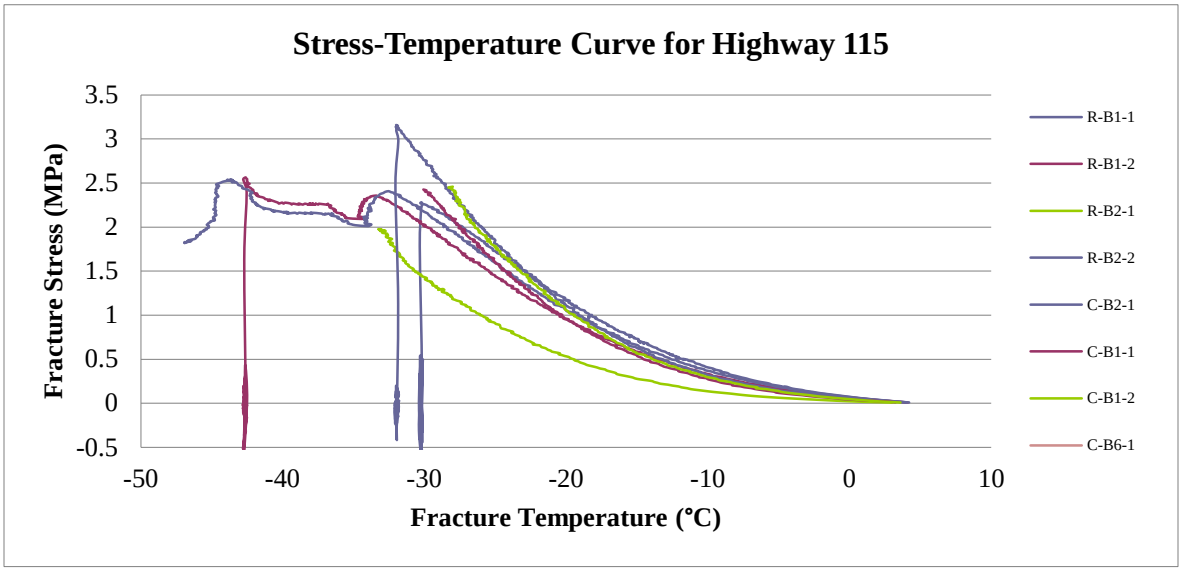


Figure 4.11 Stress-Temperature Curves for All Highway 115 Mixes

Evident in Figure 4.9 to Figure 4.11 is the pattern displayed by the rubber mixes, whereas the rubber causes the mix to withstand lower temperatures then fails suddenly after a flat line in the stress value. It is suggested that this phenomenon is due to the strength of the rubber in the matrix of the mix. It seems that the initial dip in the stress in

the rubber mixes is where the asphalt cement fails, yet the rubber is able to withstand additional temperature drop while staying under a constant stress. The rubber is able to withstand the lower temperature while keeping the stress under control because it is more ductile than the AC in the mix. This allows for the rubber particles in the matrix to absorb the additional stresses being applied to the sample, until the rubber becomes brittle and suddenly fails. This, of course, would need to be studied further to verify the hypothesis. It is therefore recommended that further studies be carried out on a molecular or smaller scale, the interaction between the rubber particles and the asphalt cement binder in the mix, and also the aggregate behavior within this interaction to determine the cause of this type of failure.

The specimen variation of the data within a mix can be attributed to many factors. Some of the more significant factors include the air voids when tested in the mix, along with interaction between the voids and the mix and weak points in the molecular level of the material interaction within the mix. It is therefore important to look statistically at the tested data to determine the significant values. Table 4.3 summarizes the statistical data that was obtained, including the standard deviation of both the fracture temperature and fracture stress, and the Coefficient Of Variation (COV). The COV that was calculated shows that the values for the fracture stress are lower than that of the fracture temperature. For this case, the fracture stress will be used for comparison, although the fracture temperature is what is normally used for the analysis of performance of a mix [Ambaiowei 2013, Velasquez 2009].

Table 4.3 Statistical Summary for Pilot Project Mixes

Mix	Mean Failure Temperature (F.T., °C)	Standard Deviation F.T.	COV (%)	Mean Failure Stress (F.S., MPa)	Standard Deviation FS	COV (%)
M1	-33.15	1.30	-3.92	2.48	0.14	5.54
M2	-35.95	7.64	-21.24	2.47	0.29	11.82
M3	-41.90	3.78	-9.02	2.71	0.36	13.38
M4	-27.98	4.18	-14.95	2.85	0.39	13.54
M5	-43.43	6.79	-15.64	2.05	0.14	6.92
M6	-29.87	1.66	-5.57	2.64	0.35	13.16
M7	-38.34	7.81	-20.38	2.23	0.32	14.15

In this case, the fracture stress is more important than the temperature. However, it is important to consider the fracture temperature when analyzing the mixes. While one may be more statistically significant, both the fracture stress and temperature for rubber mixes out-perform that of the conventional HMA, even if by small amounts.

An Analysis of the Variance (ANOVA) was carried out on the data to determine the statistical difference between the data. An ANOVA test was carried out in excel for both the fracture temperature and the stress data. The results of the ANOVA test are shown in Table 4.4 and Table 4.4 for the stress and temperature results, respectively.

Table 4.4 Analysis of Variance for Stress

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	1.74	6	0.29	3.16	0.02	2.57
Within Groups	1.92	21	0.09			
Total	3.66	27				

Table 4.5 Analysis of Variance for Temperature

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	549.69	6	91.62	2.84	0.03	2.57
Within Groups	677.82	21	32.28			
Total	1227.51	27				

Notes: SS = the sum of squares,
df = degrees of freedom,
MS = mean square,
F = ratio of mean square between groups to mean square within groups,
P-value = the probably of F occurring,
Fcrit = highest value of F that can be obtained,

For both the stress and temperature ANOVA results, the value of the F is less than $F_{critical}$ and p-value is less than 0.05, therefore it can be concluded that the results are statistically significant for both temperature and stress, and that the addition of rubber in the mix does indeed affect the stress and temperature values for the mixes.

4.5 Summary of Key Findings

This chapter has outlined the testing and results obtained for the mixes placed in the sections that were selected for testing. The results are summarized as follows:

- Most RMA sections outperformed the conventional HMA mixes. The exception is the mix placed on Highway 115, where the conventional HMA outperformed the RMA. Performance was determined to be lower than the conventional mix in the case with the PG 64-34 mix (Highway 115), where the fracture temperature was observed to be -30 degrees.
- In comparison, the RMA sections outperformed the conventional HMA sections in terms of fracture temperature. The RMA sections had been able to withstand lower temperatures than the conventional HMA sections.
- The RMA sections, when compared to HMA, show that the mix is able to absorb stresses, and therefore have exhibited a lower stress during the time of fracture. With the fracture stress being lower, the asphalt mix is able to absorb a lot of the stresses that is being applied as opposed to the HMA mix that displays a larger

stress in the mix. This may be attributed to the additional rubber that is added to the mix, giving the mix strength to be able to withstand lower temperatures.

- The mixes when compared have a visible difference in behavior between the RMA and HMA mixes. This behavior allows the RMA mixes to withstand lower temperatures than conventional mixes and allows the RMA section to outperform the conventional HMA sections.
- The RMA mixes exhibit a strange behavior when looking at the stress-temperature curves. This behavior is hypothesized to be attributed to the failure of the matrix in the mix, and the rubber then absorbs all the stress until the mix reaches a temperature of the rubber being brittle, and then ends up failing.

Chapter 5. AASHTOWare Design Analysis

The AASHTOWare software is utilized for predicting the performance of pavements using input information on the structure of the pavement, along with traffic loading and climate information to determine the performance of a pavement design over the expected design life. The AASHTOWare software outputs include pavement performance predications to surface down cracking, bottom up damage for fatigue (alligator) cracking, thermal cracking, rutting and International Roughness Index (IRI) values expected through the analysis time. The analysis assumed a pavement life of 40 years, which would be typical for these types of applications. The analysis using the AASHTOWare software determines the structural performance of RMA when designed similarly to conventional HMA. This analysis models the future performance of the RMA as a structure and compared it to the structural performance of the conventional HMA with respect to weather and traffic loading on the structure, and to ensure that the structural design of the pavement is not affected by the change in the HMA or RMA type being used in the pavement structure.

5.1 AASHTOWare Model Inputs

All of the mixes used for this research were evaluated with the AASHTOWare software; however, due to the limitations of the software and availability of some data, some assumptions were made regarding the materials being tested and the pavement structure.

The first of these assumptions were the pavement structure that is underlying the surface course of the pavement. The data that was available during this research included only surface course thickness and mix details for the surface course asphalt layer. The underlying layer information was not available for the purpose of this analysis, and therefore the layers were calculated based on AASHTO Flexible Pavement Design Method following the AASHTO pavement design equation, shown below [AASHTO 93]

$$logW_{18} = (Z_R)S_0 + 9.36log(SN+1) - 0.20 + log[\Delta PSI/4.2 - 1.5]/[0.4 + 1094/(SN+1)^{5.19}] + 2.32(logM_R) - 8.07 \quad (5\text{-}1)$$

where; W_{18} is the predicted number of equivalent single axle load applications,

Z_R is the standard normal deviate

S_0 is the combined standard error of the traffic prediction and performance prediction,

SN is the Structural Number, an index indicative of the total pavement thickness,

ΔPSI is the difference between the initial design serviceability index, p_0, and the design terminal serviceability index, p_t, and

M_R is the subgrade resilient modulus (psi).

To obtain the thickness for the structure of the pavement, the structural number (SN) needs to be determined from the above equation. The structural number is then used to calculate the required thickness of the pavement using the following equation:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \tag{5-2}$$

where a_1, a_2, a_3 are the layer coefficients of the wearing, base, and subbase layers,

D_1, D_2, D_3 are the thicknesses of the wearing, base, and subbase layers, and

m_2, m_3 are the drainage coefficients of the base and subbase layers.

The structural layers included two asphalt wearing layers (surface course and binder course, D_1), a base layer (D_2), a subbase layer (D_3), and a subgrade. The subgrade layer is the underlying soil, and therefore does not require a thickness (or considered to be "infinite"). The surface course was determined from the thicknesses of the layers that were mentioned in Chapter 3 of this thesis. The surface course thicknesses for Highway 7, 35, and 115 can be seen in Figure 3.1, Figure 3.5, and Figure 3.9, respectively. The remaining layers were calculated through the SN and layer coefficients using the excel solver function.

Figure 5.1 shows the RMA layer input for Highway 7. The Figure shows a screenshot from the AASHTOWare software input screen, where the parameters for the specific layer is inputted to be able to carry out the analysis.

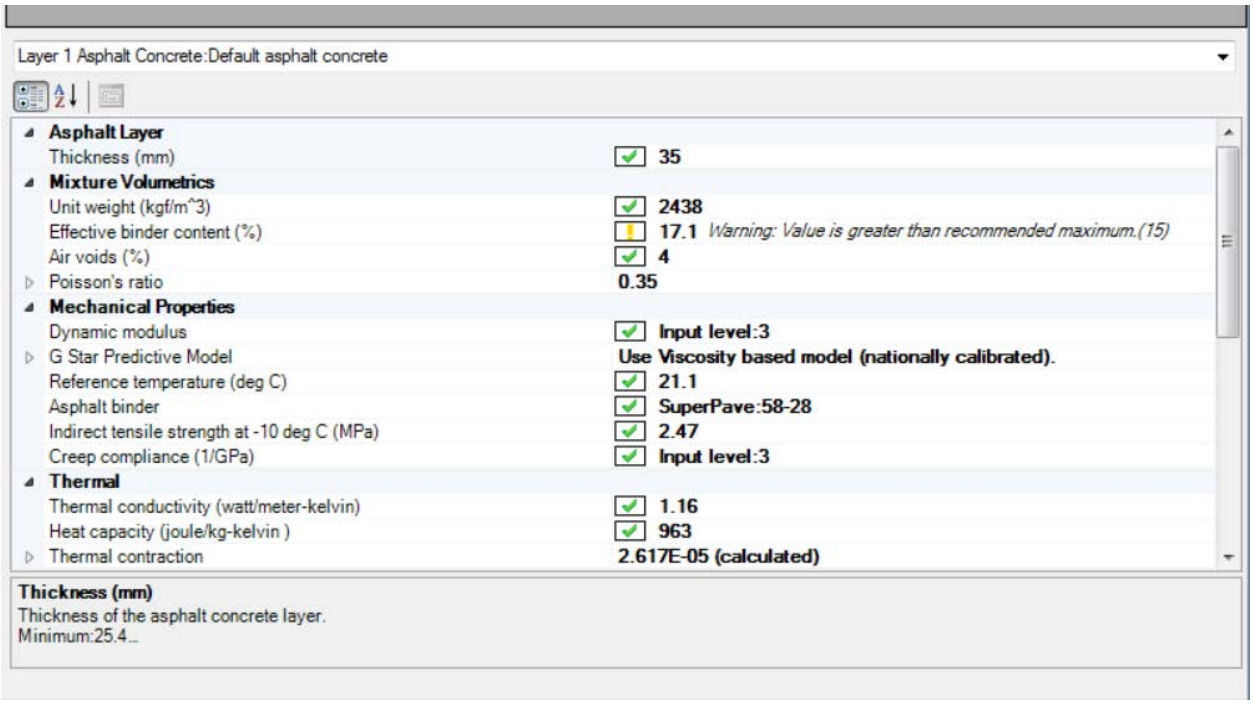


Figure 5.1: Layer Parameter Inputs for Asphalt Layer 1 (RMA Layer)

Another assumption made was the subgrade layer was considered a "strong" subgrade in terms of the soils in Ontario, and is comprised of mainly clayey gravels and sands. This assumption would yield strength of subgrade of 40 MPa for a strong subgrade, and 25 MPa for a weak subgrade [MERO 2012]. Figure 5.2 shows the input screen for the subgrade layer from the AASHTOWare software.

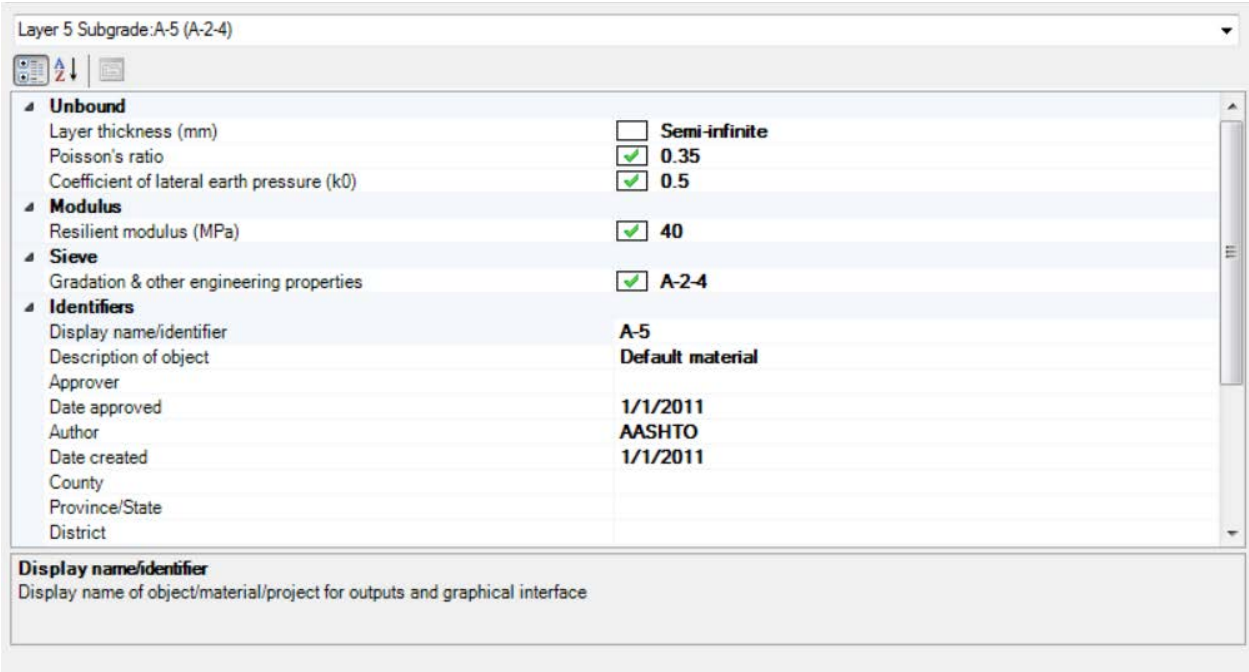


Figure 5.2: Layer Parameter Inputs for Subgrade Layer

The gradation of the base, subbase, subgrade, and binder course asphalt layers were obtained from "Ontario's Default Parameters for AASHTOWare Pavement ME Design" [MERO 2012]. This report provides default parameters to be used when utilizing the AASHTOWare software for analysis of pavements in Ontario, and had been utilized for any inputs required by the software for this thesis. Figure 5.3 shows a summary of the typical layers used in the inputs for the AASHTOWare analysis.

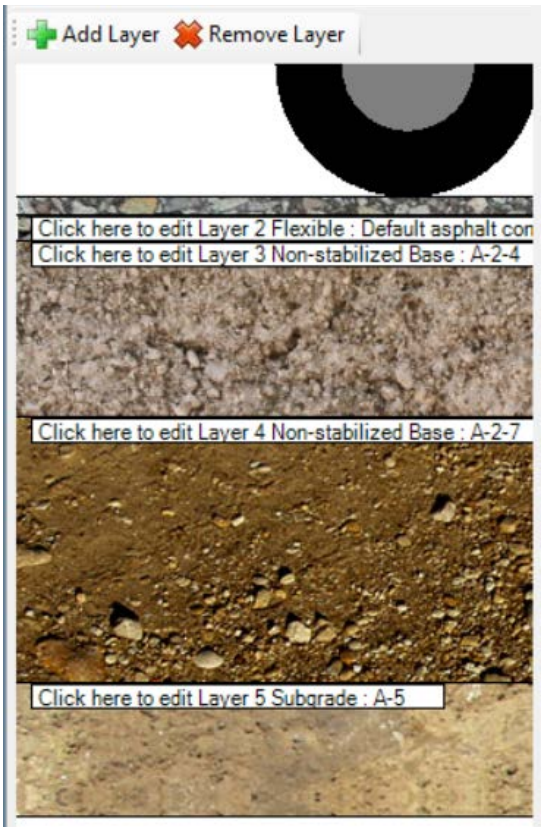


Figure 5.3: Typical Layer Structure for AASHTOWare Analysis

Traffic data was obtained from the MTO iCorridor website. The iCorridor website provides a traffic XML file that can be downloaded from each site and imported into the AASHTOWare software to ensure the accuracy of the traffic is according to the latest available data from the MTO. Figure 5.4 shows the iCorridor information available from the MTO, while Figure 5.5 shows the traffic data after it has been imported into the AASHTOWare software. Climate data that was used was obtained from the Toronto climate data provided in the AASHTOWare software, shown in Figure 5.6.

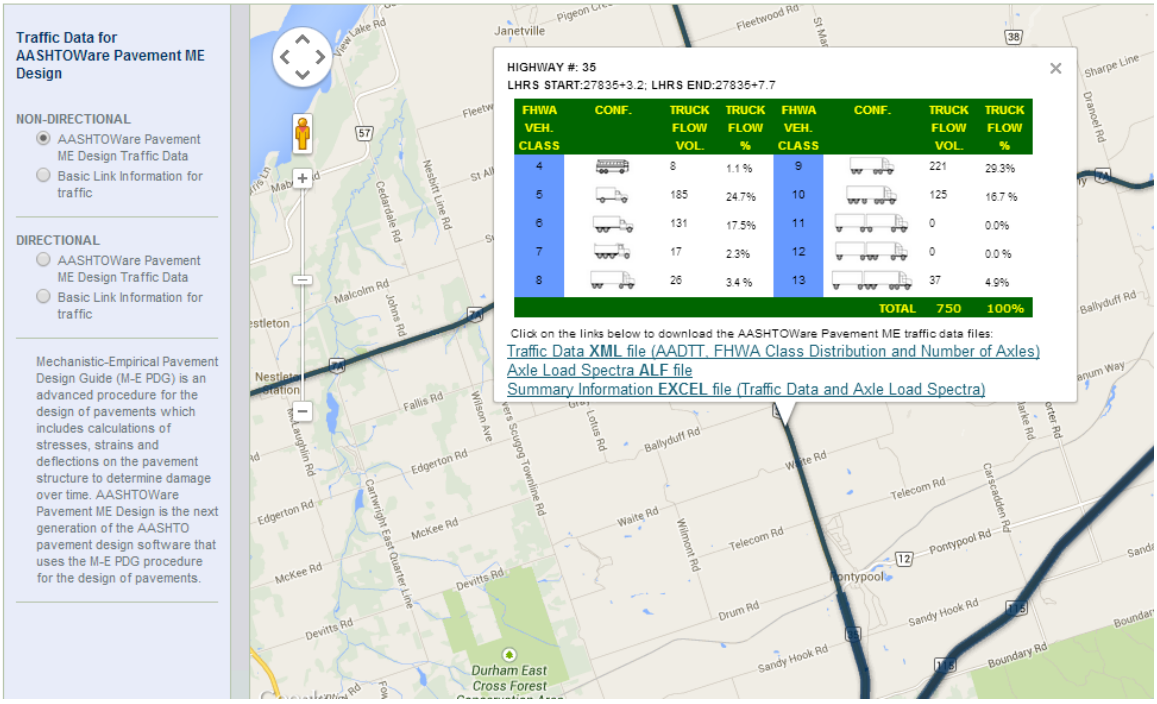


Figure 5.4: MTO Traffic Data from iCorridor [MTO 2013]

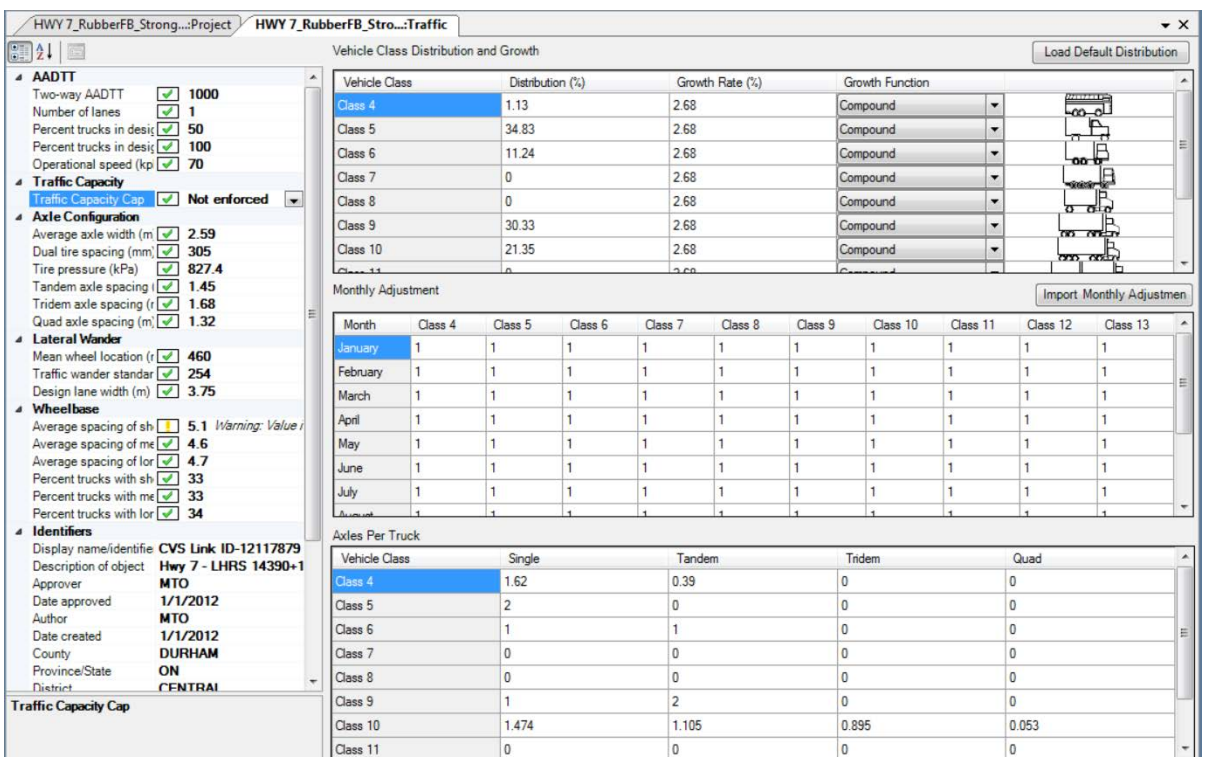


Figure 5.5: Traffic Input Data

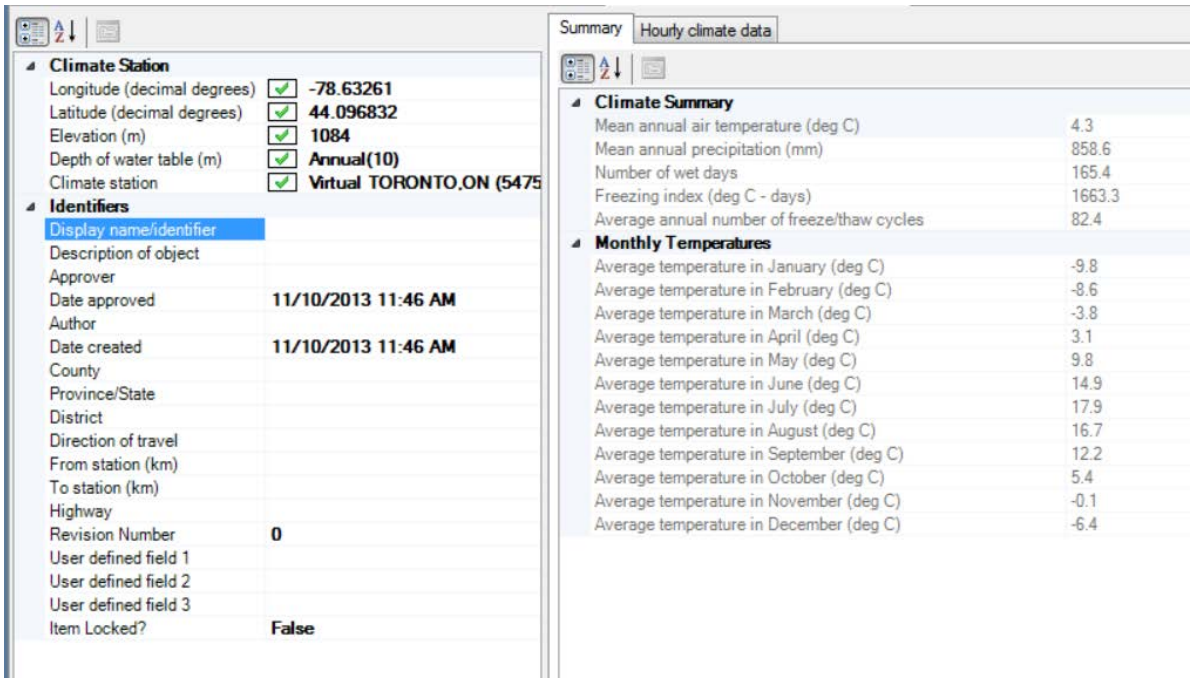


Figure 5.6: Climate Input Data from AASHTOWare

The analysis of the pavement structure was carried out for a design life of 40 years, to ensure that the pavement fails and it can be observed in the results where the failure occurs.

5.2 Analysis Results

The results of the AASHTOWare analysis are discussed below.

5.2.1 Highway 7 AASHTOWare Analysis

Highway 7 was paved with three different mixes and using two general structures: a 40 mm surface course over a 50 mm binder course, and a 30 mm surface course over a 50 mm binder course. The conventional mix, along with the terminal blend rubber mix, were paved with 40 mm surface course, while the field blend rubber mix was paving using only 30 mm thickness for the surface course.

The Highway 7 inputs included the structure shown in Table 5.1. This table shows two layers of asphalt, the surface course, being 40 mm for the conventional and rubber terminal blend, and 30 mm for the rubber field blend. The binder course under the surface course is 50 mm in thickness. The underlying layers were 335 mm and 505 mm for the granular A and B, respectively, for both the conventional and rubber terminal blend,

while for the field blend the thickness was 355 mm and 530 mm for the granular A and B layers, respectively.

Table 5.1 Highway 7 Layer Thicknesses

Rubber Field-Blend		
Layer type	**Material Type**	**Thickness(mm):**
Flexible	Default asphalt concrete	30
Flexible	Default asphalt concrete	50
NonStabilized	A-2-4	355
NonStabilized	A-2-7	530
Subgrade	A-5	Semi-infinite
Conventional HMA/Rubber Terminal Blend		
Layer type	**Material Type**	**Thickness(mm):**
Flexible	Default asphalt concrete	40
Flexible	Default asphalt concrete	50
NonStabilized	A-2-4	335
NonStabilized	A-2-7	505
Subgrade	A-5	Semi-infinite

The results from the analysis for the Highway 7 sections are presented in Figure 5.7 through Figure 5.10. These graphs show the predicted IRI, total rutting (or permanent deformation), bottom-up cracking, and thermal cracking for the design life of the pavement for the mixes used on Highway 7.

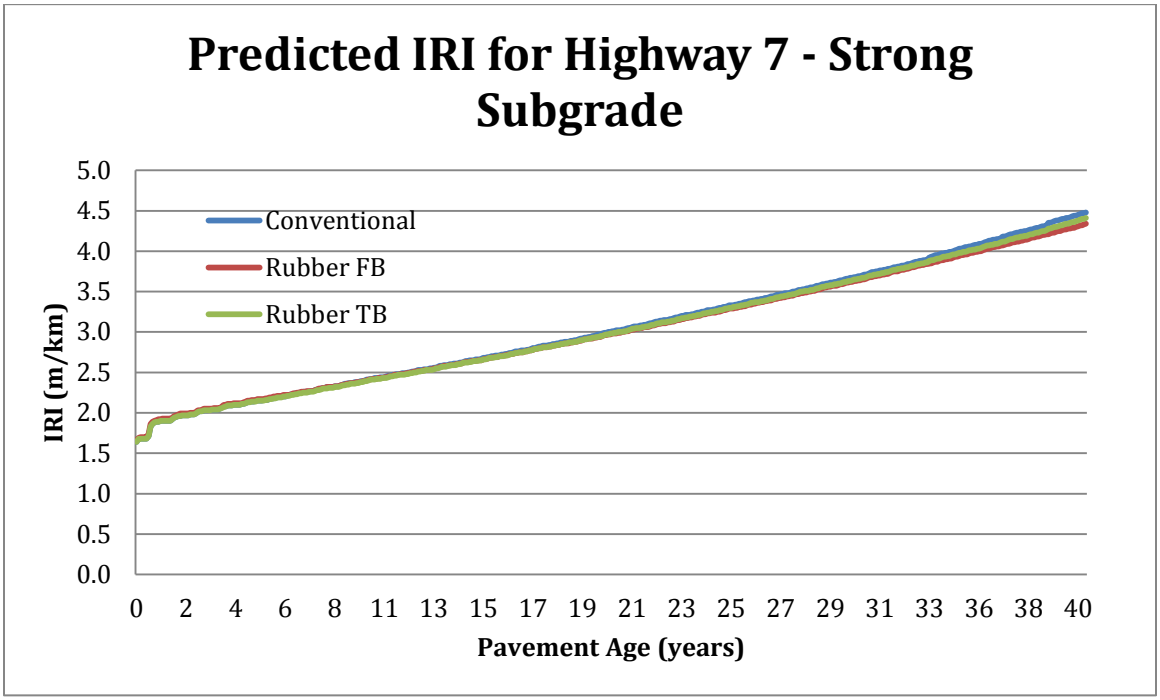


Figure 5.7 Predicted IRI for Highway 7 - Strong Subgrade

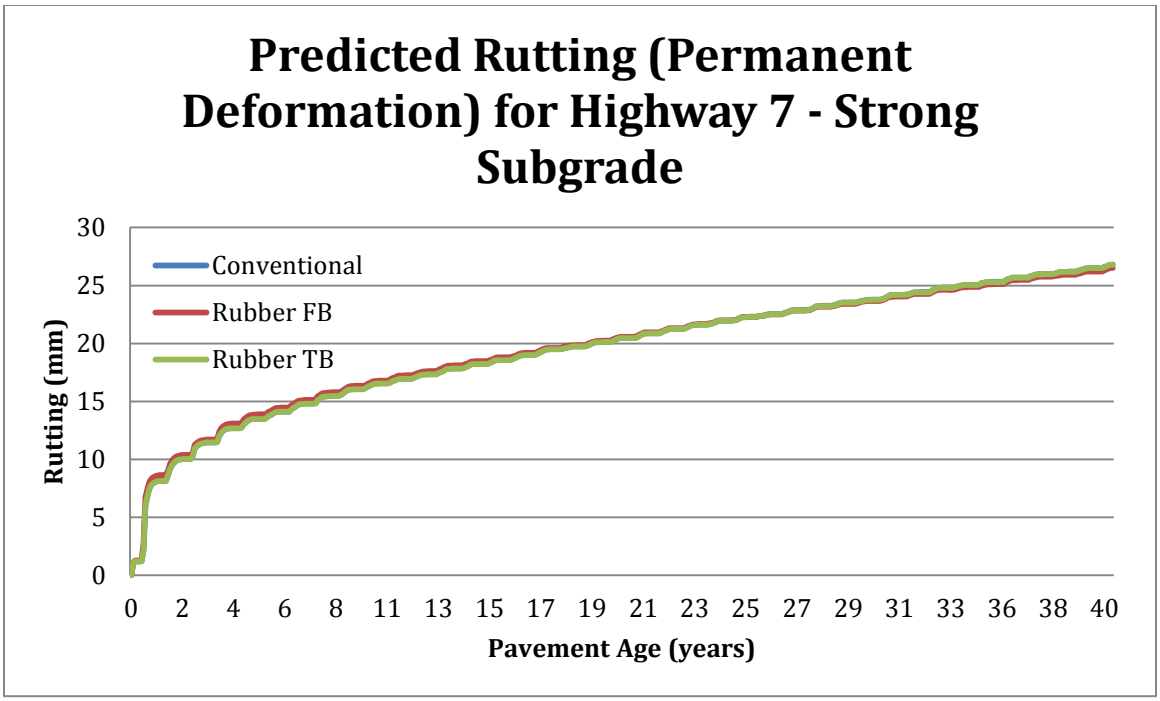


Figure 5.8 Predicted Rutting (permanent deformation) for Highway 7 - Strong Subgrade

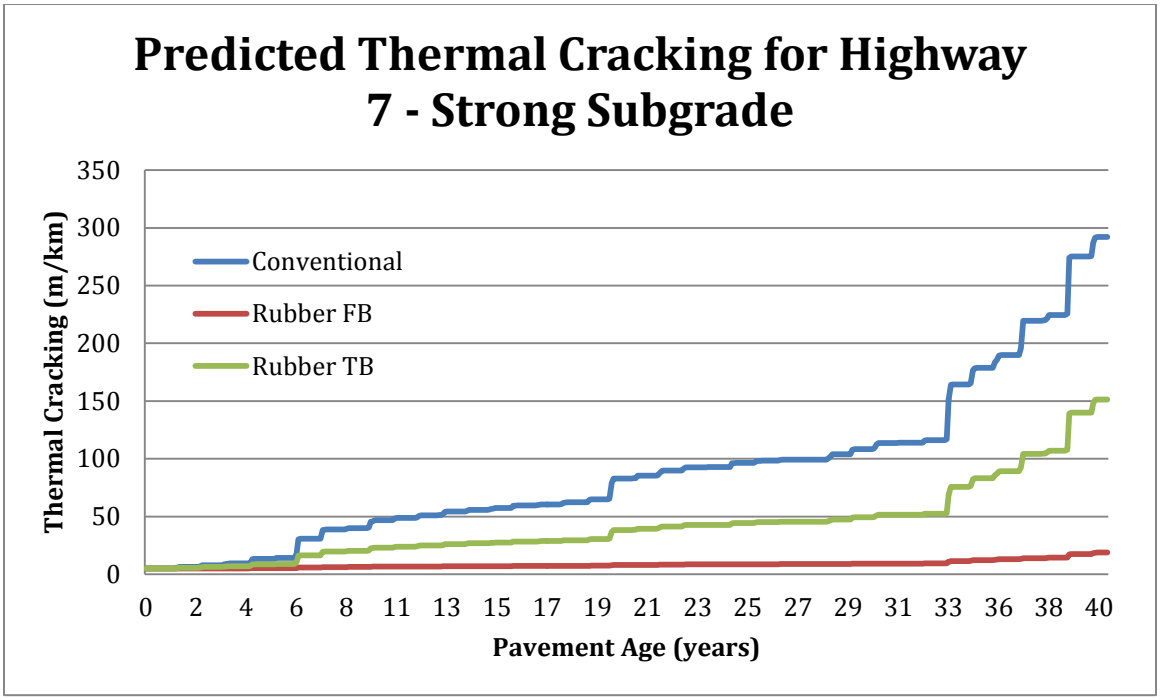


Figure 5.9 Predicted Thermal Cracking for Highway 7 - Strong Subgrade

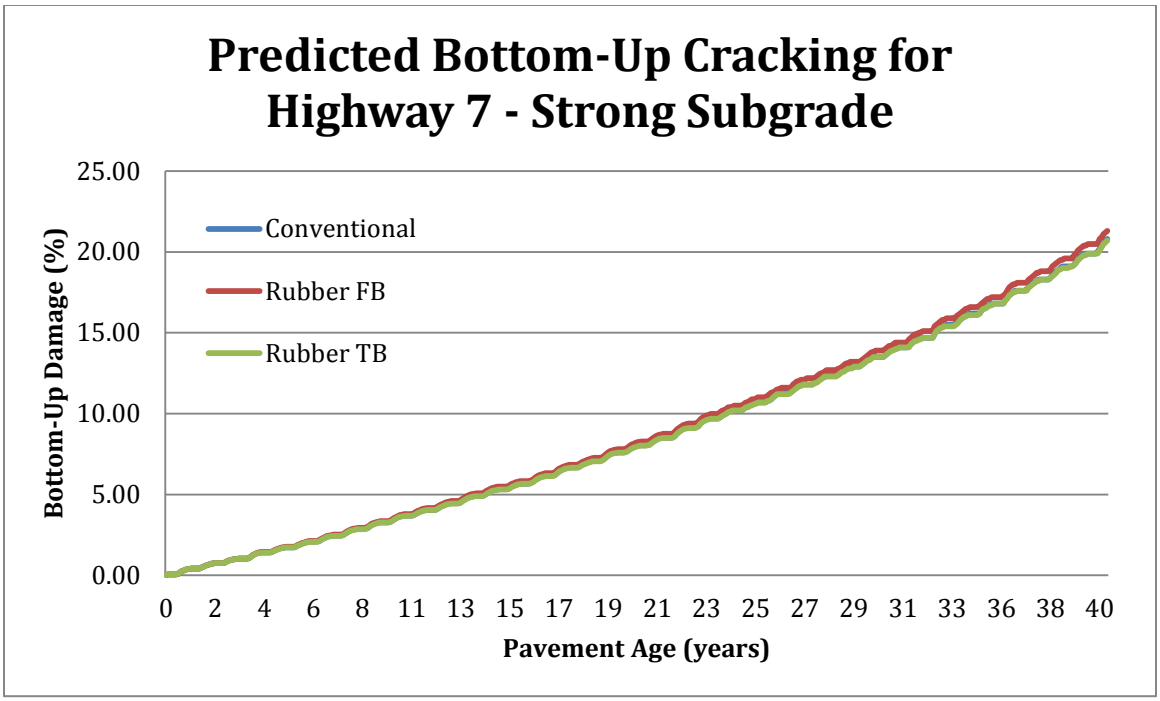


Figure 5.10 Predicted Bottom-Up Cracking for Highway 7 - Strong Subgrade

Observing the performance of the pavement mixes in terms of IRI, Figure 5.7 shows the performance of all three mixes for this section is identical when considering the structure of the pavement along with the mix properties. The rubber mixes are, at the very least, performing similar to the conventional mixes, yet the AASHTOWare software is not showing similar performance expectations for the RMA.

The predicted rutting (or permanent deformation) for the mixes on Highway 7, shown in Figure 5.8, also has matching performance. As with the IRI performance, the addition of rubber gives, at the least, similar performance to that of a conventional HMA mix. This is, again, without consideration of the effect of the addition of the rubber.

Figure 5.9 shows the predicted thermal cracking that would occur over the design life of the pavement. As observed, even with the effects of the rubber pavement not properly simulated, the thermal cracking for the rubber mixes are much lower than that of the conventional HMA. What is of interest to note is the comparison between the three RMA mixes, with the conventional mix peaking at approximately 300 m/km, while the rubber terminal blend mix showing approximately 150 m/km at the 40 year design life. The rubber field blend mix has the greatest performance, showing only approximately 25

m/km of cracking after 40 years of the pavements design life. This result can be related back to the performance of the rubber mixes in the TSRST when conducted, showing that the rubber mixes can withstand lower temperatures and failing with a lower stress than that of the HMA mixes.

The results of the AASHTOWare software also predicts bottom-up cracking for the mixes. Bottom-up cracking for the three mixes on Highway 7. All the mixes are observed to be performing similar as noted in Figure 5.10. All three mixes exhibited approximately 20% of bottom-up cracking at the end of the 40 year design life. The rubber mixes showed a slightly higher bottom-up damage percent, but the increase is very slight and therefore can be considered negligible.

While the performance of the Highway 7 mixes paved on a strong subgrade predicted the RMA to perform similar or slightly better than the conventional HMA, it is also important to determine the performance of the mixes when paved on a weak subgrade. For the purpose of comparison, the analysis was evaluated for weak subgrade conditions. The weak subgrade was determined to be a 25 MPa subgrade as per Ontario guidelines, as opposed to the 40 MPa for the strong subgrade design situation. The layers were then recalculated using the AASHTO 93 method [AASHTO 93]. Using the new pavement structures, Highway 7 was re-analyzed in the AASHTOWare software to compare the results to that of the strong subgrade.

Generally, when comparing the different strength subgrades, it was observed that there was little difference between the pavement placed on weak and strong subgrades for these Ontario pavement designs. The effect of the different mixes was consistent between strong and weak subgrades, and more specifically the thermal cracking shows similar performance between the HMA, rubber terminal blend, and rubber field blend mixes. The results for the Highway 7 AASHTOWare analysis for weak subgrades are shown in Appendix A.

5.2.2 Highway 35 AASHTOWare Analysis

Highway 35 was paved using two mixes, conventional HMA and rubber field blend. Similar to Highway 7, the information available regarding the pavement structure was very limited, and therefore the structure thicknesses had to be assumed. Also similar to

Highway 7, the layers were calculated using the AASHTO 93 method, with the assumption that the underlying subgrade is strong (approximately 40 MPa for subgrade modulus). The structure for Highway 35, as can be seen in Table 5.2, was calculated to be 50 mm of a SP12.5 surface course over 70 mm thick SP19.0 binder course.

Table 5.2 Highway 35 Layer Thicknesses

Highway 35 Layer Thicknesses		
Layer type	**Material Type**	**Thickness(mm):**
Flexible	Default asphalt concrete	50
Flexible	Default asphalt concrete	70
NonStabilized	A-2-4	240
NonStabilized	A-2-7	385
Subgrade	A-5	Semi-infinite

Highway 35 shows the same results as Highway 7, where the predicted IRI and the predicted rutting for both the HMA mixes and the rubber mixes are very close, as shown in Figure 5.11 and Figure 5.12. The results show that the addition of rubber in the mixes gives at least similar performance to the HMA mixes. Although the mixes differ in terms of materials, the AASHTOWare software cannot account for the addition of rubber in terms of the additional properties or behaviors that are associated with the addition of rubber, and therefore it can be assumed that the performance of the rubber mixes are, at the very least, performing equally as well as the HMA mixes.

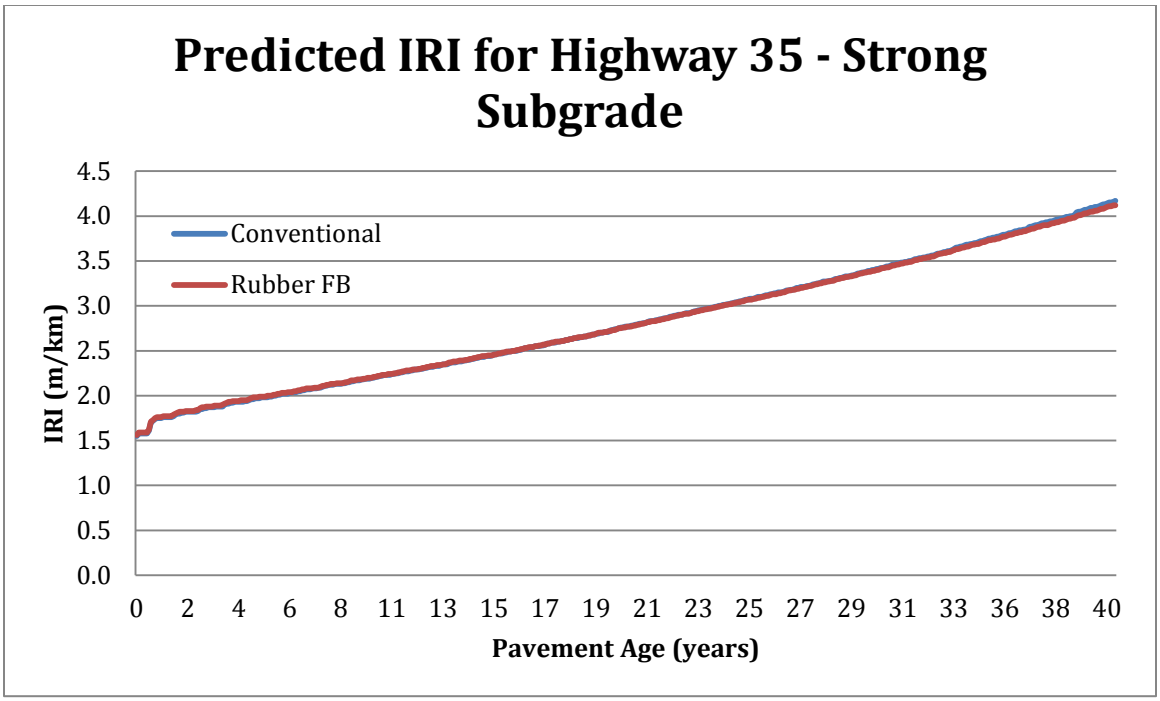


Figure 5.11 Predicted IRI for Highway 35 - Strong Subgrade

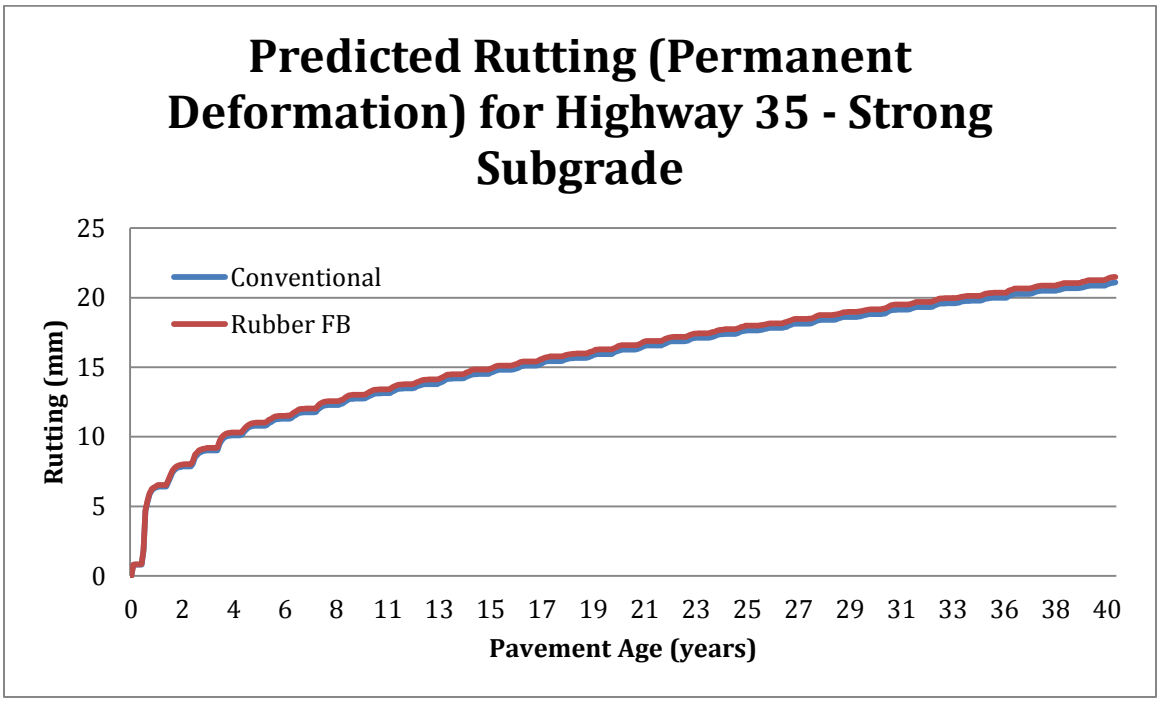


Figure 5.12 Predicted Rutting (Permanent Deformation) for Highway 35 - Strong Subgrade

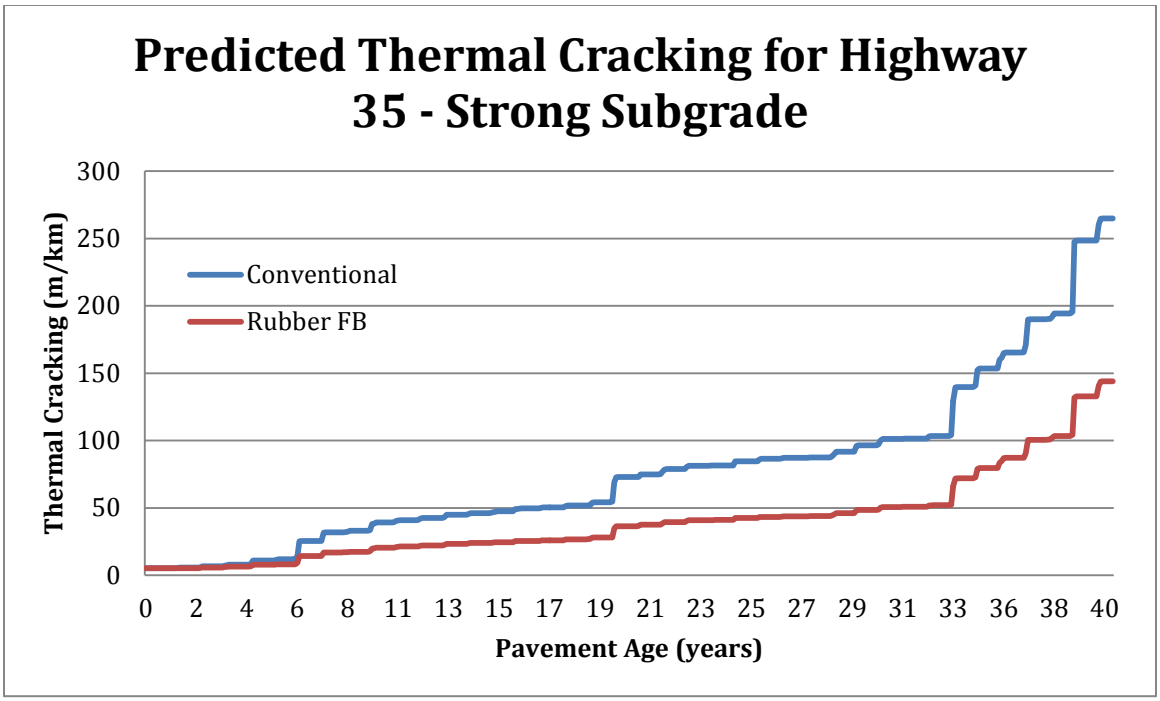


Figure 5.13 Predicted Thermal Cracking for Highway 35 - Strong Subgrade

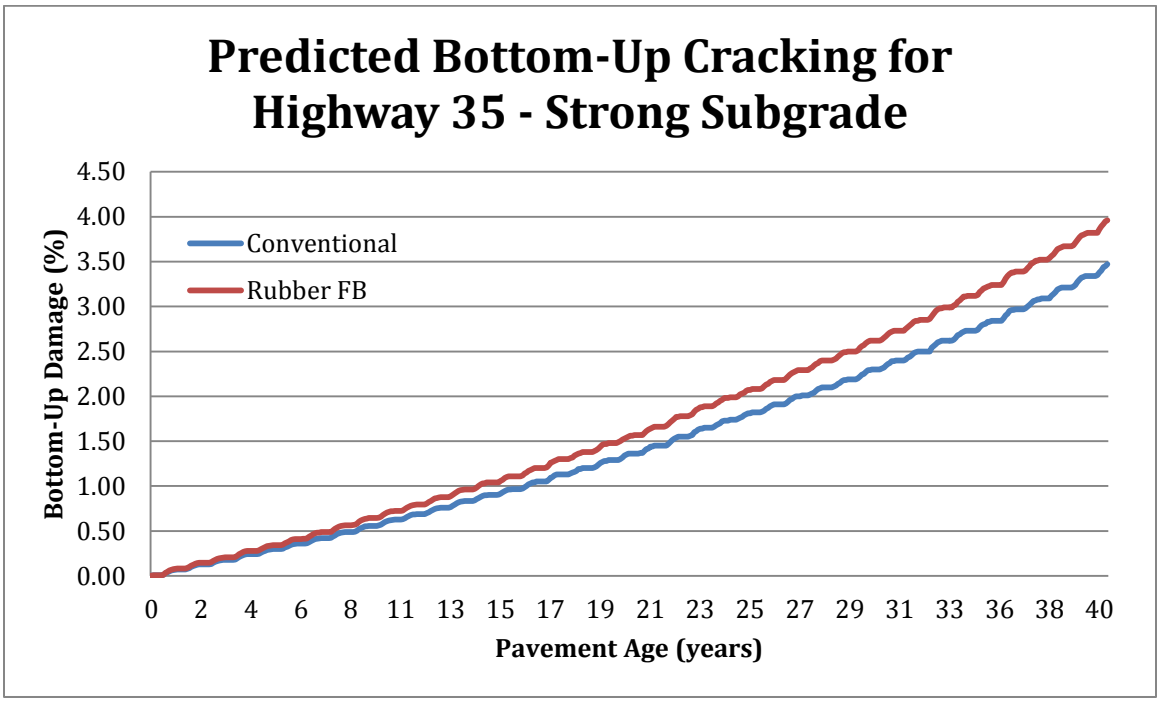


Figure 5.14 Predicted Bottom-Up Cracking for Highway 35 - Strong Subgrade

Figure 5.13 shows the comparison of the predicted thermal cracking for the Highway 35 mixes. It can be observed that the rubber field-blend mix shows less thermal cracking during the design life of the pavement, and therefore is more resistant to surface cracking than the conventional HMA mix. The result shown in Figure 5.13 is consistent with the results obtained from the TSRST testing that was carried out on the mixes, showing that the rubber mixes, being able to withstand colder temperatures, can also resist thermal cracking, more so than the conventional HMA.

For the bottom-up cracking performance for the two mixes used on Highway 35, the rubber field blend seems to have a higher percentage of damage than that of the conventional HMA, and can be observed in Figure 5.14. Although the value of the damage percentage is higher for the rubber mix, it increases to a maximum of approximately 4% damage for the rubber mix, and 3.5% damage for the conventional HMA mix. Therefore, while the rubber is higher than the conventional HMA, it exhibits an overall low percent of damage.

Figure 5.11 to Figure 5.14 show the analysis results for the mixes used on Highway 35, but under the assumption of a strong subgrade. When carrying out the analysis assuming a weak subgrade underlying the pavement layers, the analysis shows the same results. The analysis results are shown in Appendix B.

5.2.3 Highway 115 AASHTOWare Analysis

Highway 115 was paved using two different mixes, a conventional HMA mix and a rubber field-blend mix. These mixes were designed for the application of the higher traffic load experienced on Highway 115. Similar to the other highways, the structure of the highway was unknown and therefore the same assumptions were made regarding the thicknesses of the layers, and using the AASHTO 93 method of calculating the pavement structure.

The layer thicknesses for Highway 115 included a 50mm thick SP12.5 surface course with a 100 mm SP19.0 binder course underneath. The layer structure for Highway 115 is shown in Table 5.3. The assumption for the subgrade was 40 MPa, indicating a strong subgrade.

Table 5.3 Highway 115 Layer Thicknesses

Highway 115 Layer Thicknesses		
Layer type	**Material Type**	**Thickness(mm):**
Flexible	Default asphalt concrete	50
Flexible	Default asphalt concrete	100
NonStabilized	A-2-4	295
NonStabilized	A-2-7	440
Subgrade	A-5	Semi-infinite

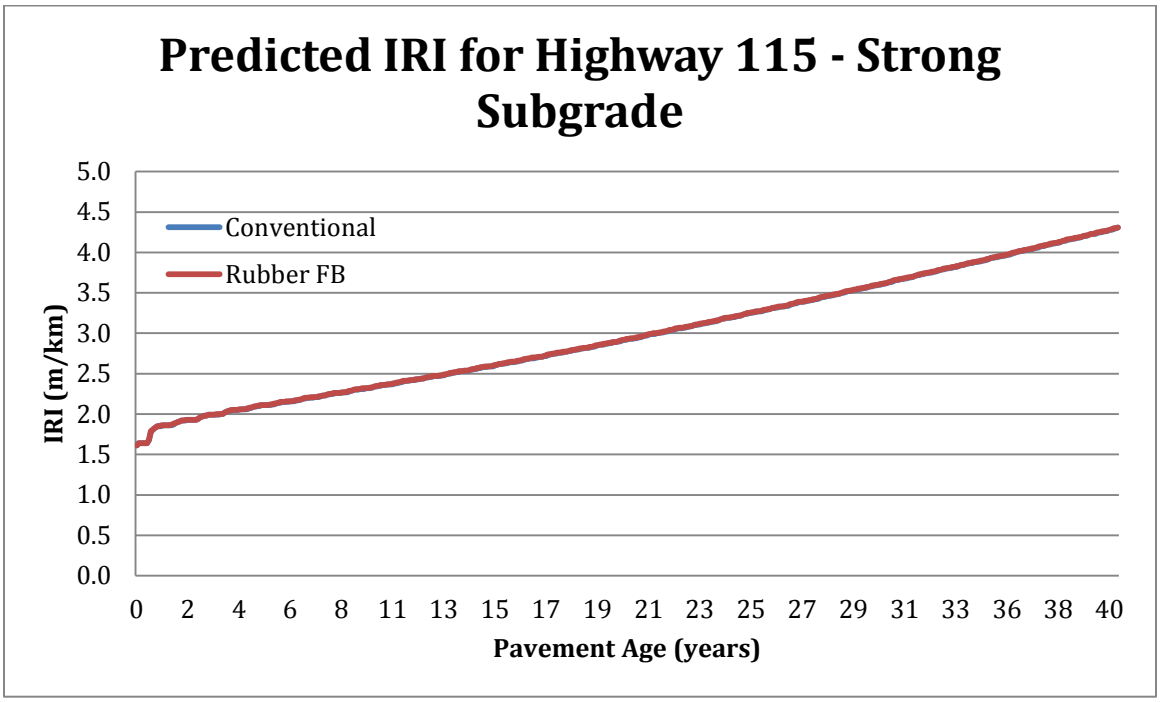


Figure 5.15 Predicted IRI for Highway 115 - Strong Subgrade

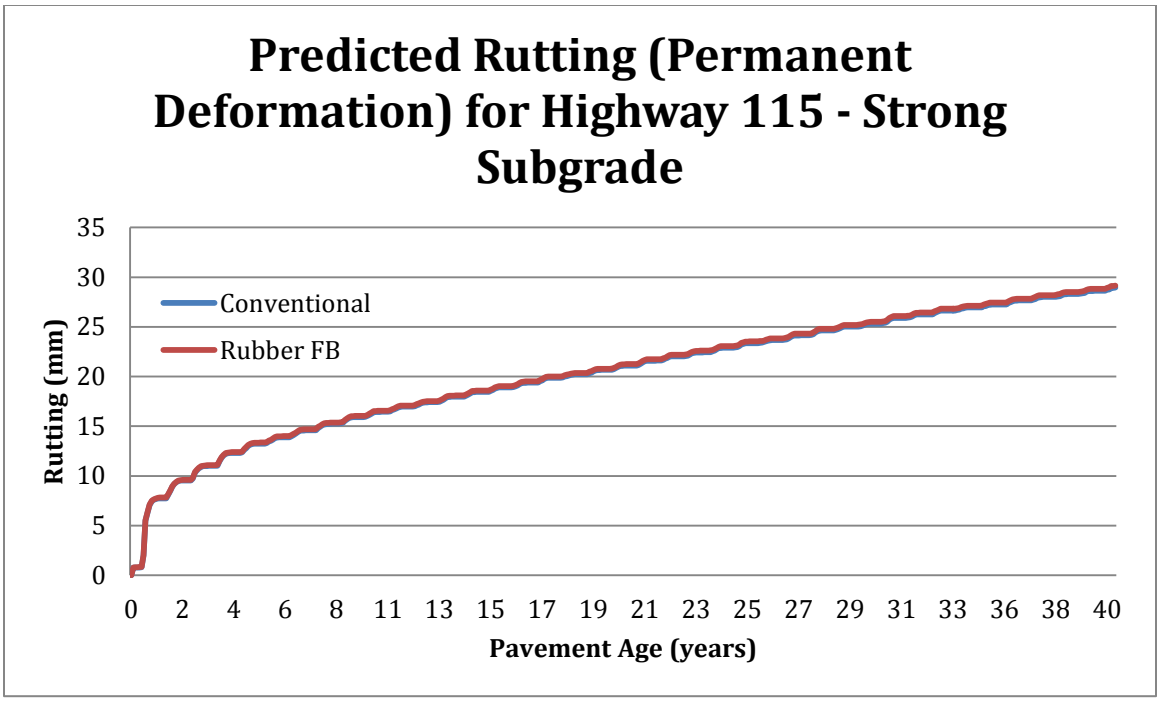


Figure 5.16 Predicted Rutting (Permanent Deformation) for Highway 115 - Strong Subgrade

The predicted IRI and predicted permanent deformation (rutting), shown in Figure 5.15 and Figure 5.16 are shown as being approximately the same for the design life of the pavement, with the IRI having a value of approximately 4.3 m/km and the predicted rutting having a value of approximately 30 mm for both mixes. Similar to Highway 7 and Highway 35, the IRI performance and rutting performance of the rubber mix is observed as being equal with the performance of the conventional mix, however, the added properties of the rubber in the mix are not accounted for due to the limitations of the AASHTOWare software.

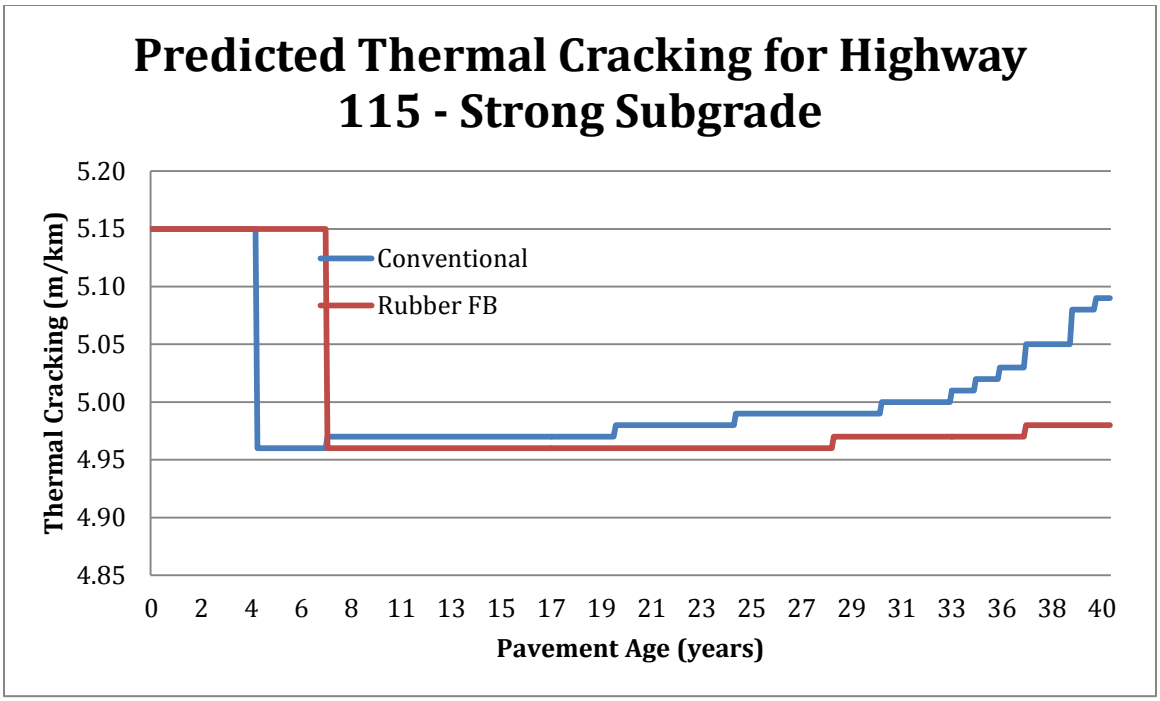


Figure 5.17 Predicted Thermal Cracking for Highway 115 - Strong Subgrade

Figure 5.17 shows the predicted thermal cracking for Highway 115. The thermal cracking graph shows a limitation of the AASHTOWare software as it seems to not be able to accurately predict the initial cracking extent, as evidenced by the drop in thermal cracking at approximately 4 years for the conventional HMA mix, and 7 years for the rubber mix. A sudden drop in the graph indicates a decrease in the amount of cracking per kilometre, which cannot be the case unless maintenance was carried out on the pavement. Although this may show a problem, the remainder of the graph follows the typical curve for thermal cracking that has been observed in Highway 7 and Highway 35 with regards to the rubber mix exhibiting less thermal cracking than the conventional HMA mix. Although the graph looks similar to the Highway 7 and Highway 35 mixes, the thermal cracking values are much lower than the mixes used on the other highways, reaching values of approximately 5.1 m/km and 5.0 m/km at the 40 year design life for the conventional HMA and rubber mixes, respectively.

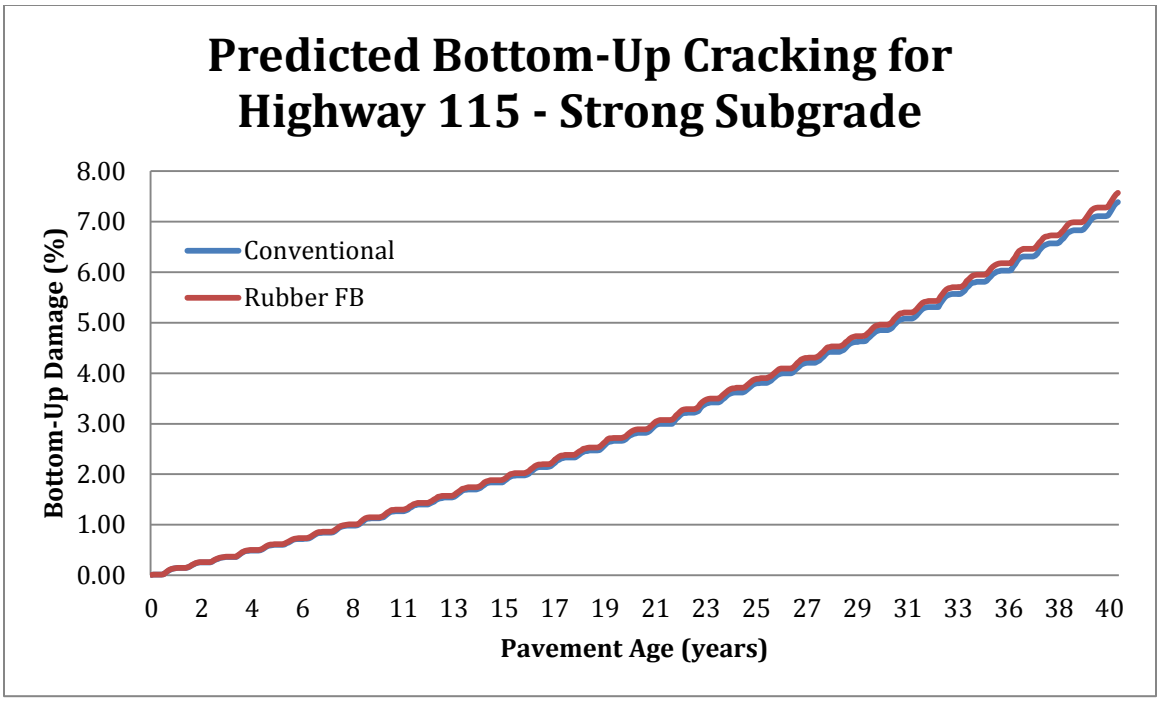


Figure 5.18 Predicted Bottom-Up Cracking for Highway 115 - Strong Subgrade

Figure 5.18 shows the percentage of bottom-up cracking on the pavement throughout the design life. Both the conventional HMA and the rubber mixes show approximately a 7.5% of bottom up damage at the 40 year design life. It can be concluded from this Figure that the rubber mix performs equally as well as the conventional HMA. Although the graph shows the rubber field blend exhibits slightly higher percentage of bottom-up cracking, the increase is in the order of 0.1%, which is minimal when looking at the pavement on a larger scale.

Since an assumption was made for Highway 115 being placed on a strong subgrade, it was important to consider the effect of placing the pavement on a weak subgrade. The design layers were calculated using the AASHTO 93 design method, and the analysis was run on the AASHTOWare software. The analysis results show similar results to those of the strong subgrade mentioned earlier. Graphs for the Highway 115 weak subgrade results can be seen in Appendix C.

5.3 Chapter 5 Summary

To determine the performance of the rubber mixes as compared to the conventional HMA mixes, the pavement structure, mix properties, and highway traffic data was entered into the AASHTOWare software to determine the prediction of the performance of the pavement in terms of IRI, rutting (or permanent deformation), thermal cracking, and bottom-up cracking. Although the software is limited to analyzing the data as a conventional mix and cannot simulate the effect of the rubber, it is important to determine the performance of the variation of the mixes to ensure that it performs at least the same as the conventional HMA mix.

To be able to utilize the software, some assumptions were made to allow for the data to be analyzed. The assumptions include the subgrade strength was "strong", and the design for the pavement follows the AASHTO 93 design method. With these assumptions, the data was analyzed over a 40 year design life, and the output indicated the following:

- The IRI performance of both the conventional HMA mix and the rubber mixes for all three highways show very similar, if not same, performance. The IRI indicates that the roughness of the rubber mix will not be worse than that of the conventional mix. Although the mixes perform the same, the software is not able to simulate the effect of the rubber, and therefore, based on the results presented in Chapter 4, it can be assumed that there would be an increase of the performance of the rubber mixes given the superior material properties.
- The permanent deformation, or rutting, performance of the mixes are also similar, if not the same, for the three highways. Although the addition of rubber does not seem to add any benefit according to the software, it does perform similar to the conventional mix.
- The thermal cracking performance of the mixes is where the analysis shows a difference between the conventional HMA and rubber mixes for all three highways. The conventional HMA predicts approximately 300 m/km of thermal cracking for Highway 7, and 250 m/km of thermal cracking for Highway 35. The RMA mixes for Highways 7 and Highway 35 are well below 50 m/km and below 150 m/km, respectively. While this behavior was observed on Highway 7 and

Highway 35, it was not observed on Highway 115. The Highway 115 figure observed very low numbers for thermal cracking, yet it followed the same prediction pattern as the other highways. It also indicated that the RMA mix performed better than the conventional HMA mix.

- The bottom-up cracking for the highways was also determined to be similar, if not the same, for all three highways. While the same discussion regarding the software involved being able to simulate the effect of rubber applies as the IRI and permanent deformation results, it is believed that the bottom-up cracking would be improved with the addition of rubber.
- While the assumption for the subgrade was determined to be “strong”, an analysis was also run with the assumption of a weak underlying subgrade. The results for this analysis showed similar predicted performance observations as a “strong” subgrade.

Chapter 6. Guidelines for use of RMA in Ontario

This chapter presents basic guidelines to follow for the implementation of RMA on Ontario roads. Based on Ontario's previous experience with RMA during the pilot study, along with the results of both the TSRST and AASHTOWare analysis, a procedure has been developed to assist in the process of selecting particular sections to be paved with RMA, along with an explanation of the process required to properly use RMA. While the pilot study discussed in this thesis was conducted on rehabilitation projects, the guidelines have also been adapted to include new construction projects as well.

RMA can be used on both maintenance or rehabilitation applications, and new construction. The mill and overlay, or reconstruction/rehabilitation activities allow for the underlying pavement to be partially or fully repaired before paving an RMA layer. Asphalt rubber mixes for rehabilitation projects provide the needed resistance to reflective cracking. For new construction projects, a gap-graded RMA application can be used at the same thickness of a conventional HMA mix. As noted in Chapter 2, an RMA application can be paved with half of the thickness of a conventional HMA to obtain similar performance, but most applications of RMA use equivalent thickness to maximize the benefit from the increased performance of the RMA.

6.1 Rehabilitation Projects

When considering a rehabilitation project, a thin RMA overlay would be an ideal application. The thin RMA overlay should be designed as a gap-graded mix, and to be between compacted to 30 to 60mm so that it may function as a structural component. Most rehabilitation projects consist of a mill and overlay and therefore an open-graded mix is not ideal. If the project is conducted for maintenance or preservation of the road, an open-graded mix may be used as a thin overlay, since open-graded mixes are not suitable to be used as a structural layer for rehabilitation projects.

For a rehabilitation project, it is important to check the condition of the existing pavement prior to paving. Any moderate or high severity cracks should be filled prior to paving an overlay. Proper application of tack coat should be inspected and insured immediately prior to paving.

In order to acquire the correct rubber for the RMA mix, the quality of the crumb rubber must first be ensured. To guarantee the optimal performance of the RMA section, it is critical for the crumb rubber to be within a specified gradation. A typical gradation used in Ontario for the 2011 pilot projects is shown in Table 6.1. The appropriate percentage of crumb rubber of rubber by weight of the AC is 18% to 20%.

The quality of the crumb rubber must also be considered, which is done by ensuring that there are no contaminants in the material. Moreover, it is preferred that the rubber be ambient ground rubber rather than cryogenic. CRM in Brantford, Ontario, the supplier that provided the rubber for the RMA pilot study, is a recently commissioned rubber tire recycling plant in Ontario capable of producing the required crumb rubber for asphalt mixes. The CRM factories in the United States have experience with supplying crumb rubber to various RMA projects located across the United States, thus providing CRM with the necessary experience and knowledge needed to address the correct rubber for RMA mixes. There are also other suppliers locally that are able to provide the crumb rubber required for the RMA mix.

Table 6.1: Crumb Rubber Gradation Requirements for RMA Mixes [Hegazi, 2012]

Process	Sieve	Percent Passing Sieve by Mass
Wet-Field Blend	2.36 mm	100
	2.00 mm	98 – 100
	1.18 mm	45 – 75
	600 µm	2 – 20
	300 µm	0 – 6
	150 µm	0 – 2

The mix design for the gap-graded mix must include considerations for air voids, AC content, density, and voids in mineral aggregate (VMA). When creating a Superpave mix design, the process includes creating briquettes, which have a tendency to swell more than is observed for conventional Superpave mix designs. Several methods were used to

attempt to mitigate the swelling during the 2011 pilot study mix design process, including placing weights on the sample. While this method may be used, it was found that ensuring the CRM is closer to the finer gradation of the supplied gradation specifications reduces swelling and allows for the creation of an appropriate Superpave mix design, and therefore it is recommended to re-check the gradation of the CRM that is obtained from the supplier when swelling is present.

During the paving of a road with RMA in Ontario, the right equipment must be available. The blending unit, which is required to make the asphalt concrete, must be procured, rented, or contracted from one of the companies that have the machine available. OTS has found that contracting FATH Industries, from Alberta, was the best option for the RMA pilot study, and could be contracted for any other RMA projects in Ontario. After the procurement of the blending unit, it is necessary for the asphalt plant to be modified in order to produce the rubberized asphalt. Typical modifications to incorporate the blending unit into the asphalt plant include installing a three-way valve to be able to pump the asphalt rubber and asphalt cement into the plant to produce rubberized asphalt, and a return line for the asphalt cement. These are fairly simple modifications that can be done without affecting the overall function of the plant.

While operating the blending unit to mix the RMA, proper agitation and temperature must be observed to ensure maximum reaction between the asphalt cement and the rubber particles, and so the rubber particle be properly coated with the asphalt cement. The mixing temperature for the CRM in the blending unit is 190°C to 224°C, for duration of 45 minutes. When the mix is being held in the tank, it should be stored at a temperature of 190°C to 218°C, and must be properly agitated [Tighe 2012]. Ensuring that all specifications and procedures are followed for the agitation and the mixing temperatures is critical in obtaining an optimal performing mix.

When paving with RMA, the standard procedure does not differ a great deal from paving with the conventional HMA. Again, temperature is the critical variable that needs to be closely monitored. It is strongly recommended that the RMA be compacted before the temperature of the RMA falls below 138°C [Tighe 2012]. Furthermore, it is recommended that steel drum rollers follow closely behind the paver to ensure that the RMA is compacted at the correct temperature and to the correct compaction level. While

following the paver with rollers, rubber tire rollers must not be used, as the rubber on the roller will stick to the rubber in the RMA causing unwanted pickup. Thus, it is important to use steel drum rollers for compaction.

While paving, collecting samples to carry out and Q/A and Q/C testing should be done. This is standard procedure for paving, and all the standard procedures for paving with the conventional HMA should be followed in the same manner. The difference when paving with RMA is that the compaction temperature is more critical, and therefore the compaction rollers need to follow the paver closely, and ensuring that no rubber tire rollers are used on the site.

6.2 New Construction Projects

When carrying out a new construction project, a gap-graded RMA mix can be used in place of a dense-graded conventional HMA mix. There are multiple options to consider when conducting a new construction. A new construction pavement can consist of a dense-graded structural mix, with a gap-graded RMA surface course of up to 60mm. A gap-graded RMA mix can also be used as a structural mix, with an RMA surface course. If the road being constructed is a freeway, an open-graded thin RMA layer can be used as a surface course. It is important to note, however, the open-graded RMA mix cannot be used as a structural layer, and therefore must be used over a dense-graded conventional HMA layer or a gap-graded RMA layer.

The remainder of the procedure is similar to that of the guidelines for rehabilitation projects. The difference is when using an open-graded mix, the AC binder content would be expected to be higher than that of dense-graded or even gap-graded mixes. It is therefore recommended to ensure that the AC content of the mix is elevated, while ensuring compliance to the mix design specifications.

6.3 Guidelines for RMA Use

When choosing to pave with RMA, Figure 6.1, developed as part of this research, provides a workflow for the paving of RMA. While the workflow presented does not include every detail of the process, it does serve as a checklist of important guidelines to abide by when paving with RMA.

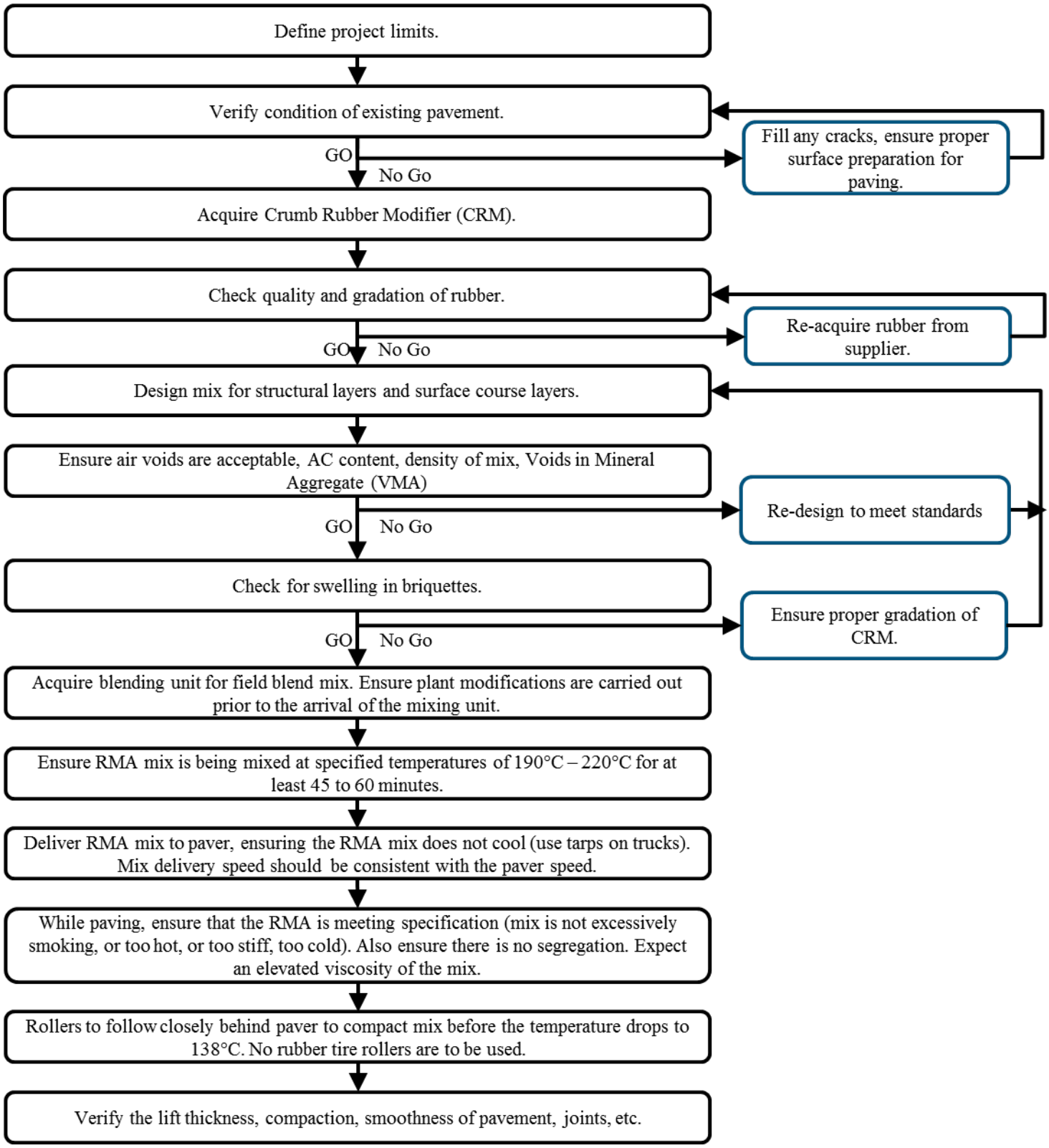


Figure 6.1: RMA Guidelines Workflow

Chapter 7. Conclusions and Recommendations

7.1 Conclusions

Crumb rubber modified asphalt has been used extensively in regions in the United States and has been proven to successfully add to the performance of the asphalt pavements. Although this may be the case, the climate in which it has been used is warm climate, and areas that do not experience the harsh weather that is faced in Ontario.

Many studies have been conducted on the use of rubber modified asphalt, and the studies have shown that the addition of rubber allows the asphalt to be more resistant to cracking. The rubber also allows the asphalt pavement to withstand additional traffic loading without undergoing fatigue cracking. This allows for a decrease in maintenance of the asphalt pavement, and hence decreasing the life cycle cost of the road.

Rubber modified asphalt has also shown to include other benefits. These benefits improve the performance of the pavement in terms of level of service to the user. The benefits of using rubber modified asphalt include reduced tire noise caused by the cars or trucks traveling on the road, reduced splash during wet weather, and increased friction to aide with braking and ensuring the road are safe.

While rubber modified asphalt has been used successfully in jurisdictions such as the State of California, the state of Texas, and the state of Arizona, where the climate is warm year-round, there has been very limited experience with the material in cold weather climate. There was some trial of rubber asphalt use in Canada previously, but the results were not conclusive as to whether it is feasible to use in the Ontario climate or not.

In 2011, the Centre for Pavement and Transportation Technology, along with the Ontario Tire Stewardship, Ontario Ministry of Transportation, and the Ontario Hot Mix Producers Association have carried out a pilot study of the use of rubber modified asphalt in Ontario, and paved three highways in Ontario using the material. The highways, Highway 7, Highway 35, and Highway 115, were sampled during paving for testing of the material at the CPATT facility at the University of Waterloo.

The test carried out on the material was the Thermal Stress Restrained Specimen Test, which fixes both ends of a prepared beam of the material onto plates and is subjected to a constant drop in temperature, which introduces stress in the beam since it is

restricted from movement. The temperature drops at a constant 10 degrees Celsius per hour until the beam fails. The results showed clearly that the rubber modified asphalt was able to withstand an average of approximately 10 degrees lower temperatures than the conventional HMA mixes. It was also observed that the stress at failure was lower for the rubber mixes. This performance indicates that the rubber is able to withstand colder temperatures than the conventional HMA, and also able to absorb more stress than the conventional HMA.

Along with determining the performance of the material itself, it was also important to determine the performance of the material as a whole pavement structure. To evaluate the performance of a pavement structure with crumb rubber modifier, the AASHTOWare software was used. While the AASHTOWare software can predict a pavement structure performance, there are limitations to the software, namely the fact that it cannot simulate the exact effect of the rubber on the asphalt mix. Also, there was a lack of available data to be input into the software, and therefore many assumptions were made with regards to the pavement structure.

The results of the AASHTOWare analysis did, however, show that as a structure, the rubber modified asphalt pavements did perform as well as the conventional HMA mix when evaluating the International Roughness Index, permanent deformation, and bottom-up cracking over a design life of 40 years. However, the performance of the rubberized asphalt in terms of thermal cracking was determined to be better than that of the conventional HMA. It can be concluded from these results that the addition of rubber does not decrease the performance of the rubber and performs at the same level of the conventional HMA, however, the additional benefit of the rubber is not modeled through this software and the rubber mixes would most likely outperform the conventional HMA if the inputs were changed to reflect the properties of the rubber material in the mix.

From the results of this thesis, it was possible to create guidelines to be used for when paving with RMA mixes. The guidelines apply to rehabilitation and new construction projects, where the mixes would be used as gap-graded or open-graded, depending on the application.

7.2 Recommendations

It is recommended that there be continuous monitoring of the pilot test sections paved on Highways 7, 35, and 115, and additional non-destructive testing be conducted on the pavement to determine the “real-world” performance of the pavement. This testing will help determine the actual performance of the pavement in a “live” environment rather than a controlled lab experiment, and can yield more revealing results in regards to the additional benefit of the crumb rubber modified asphalt mixes.

It is also recommended that further lab testing be conducted on the material collected during the paving of the test sections to also study the behavior of the mix when exposed to different climate and traffic loading conditions. Some recommended tests to carry out on the material are the dynamic modulus test and the fatigue beam tests. These tests will help determine the performance of the rubber mixes when exposed to different levels of traffic loading, and different frequencies of loading.

While the dynamic modulus test can be used to show the traffic loading performance of the mix, the results of the dynamic modulus test can be also used to input into the AASHTOWare software to more accurately model the behavior of the mix with the added rubber. The results of the AASHTOWare model can then be updated to reflect more accurately how the pavement structure would behave as a whole over a 40 year design life, and then can be compared to the performance of a conventional HMA mix.

Furthermore, it is also recommended that further testing be conducted on the specific behavior of crumb rubber in differing weather conditions, along with a study on the chemical interaction and/or the chemical processes between the asphalt cement binder and the crumb rubber particles. This testing may help determine any weaknesses with using crumb rubber modifier, and determine what can be done to overcome the weaknesses, if any. It is important to determine the interaction between the rubber and the asphalt cement binder so that crumb rubber can be more efficiently used in asphalt pavement mixes.

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Appendix A: Highway 7 AASHTOWare Results (Weak Subgrade)

HWY 7_Conv_WeakSG

File Name: C:\Users\admin\Desktop\Mo MASc Thesis\HWY 7_Conv_WeakSG.dgpx

Design Inputs

Design Life:	40 years	Base construction:	August, 2011	Climate Data Sources (Lat/Lon)	43.862, -79.37
Design Type:	Flexible Pavement	Pavement construction:	September, 2011		44.117, -77.533
		Traffic opening:	October, 2011		43.677, -79.631

Design Structure

Layer type	Material Type	Thickness(mm):
Flexible	Default asphalt concrete	40.0
Flexible	Default asphalt concrete	50.0
NonStabilized	A-2-4	415.0
NonStabilized	A-2-7	620.0
Subgrade	A-5	Semi-infinite

Volumetric at Construction:

Effective binder content (%)	13.4
Air voids (%)	4.0

Traffic

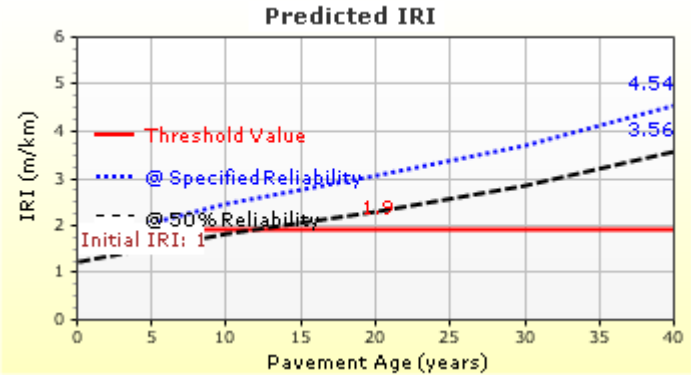
Age (year)	Heavy Trucks (cumulative)
2011 (initial)	1,000
2031 (20 years)	4,750,550
2051 (40 years)	12,812,900

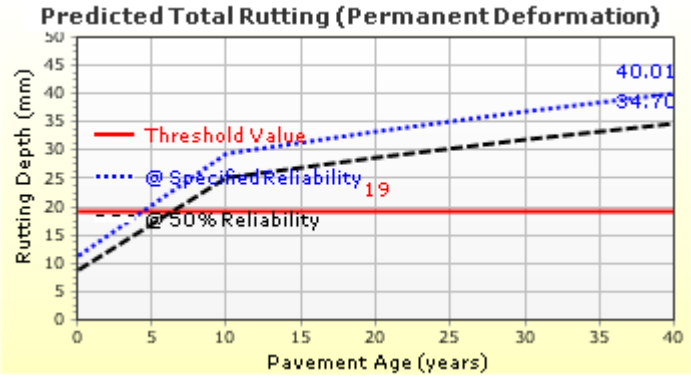
Design Outputs

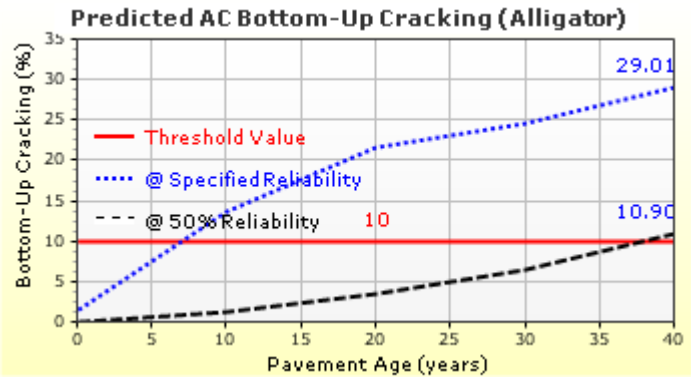
Distress Prediction Summary

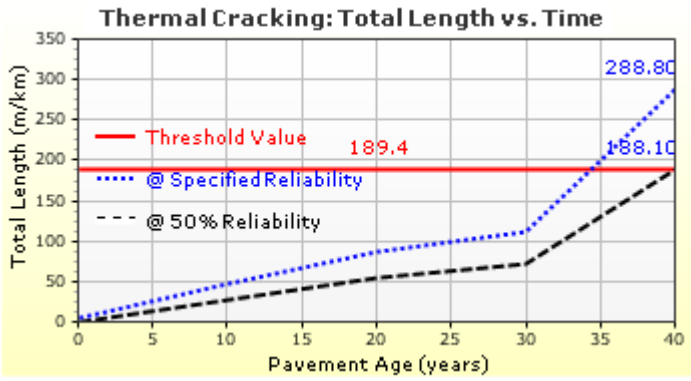
Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (m/km)	1.90	4.54	90.00	1.51	Fail
Permanent deformation - total pavement (mm)	19.00	40.00	90.00	0.01	Fail
AC bottom-up fatigue cracking (percent)	10.00	29.01	90.00	47.46	Fail
AC thermal cracking (m/km)	189.40	288.80	90.00	50.66	Fail
AC top-down fatigue cracking (m/km)	378.80	484.52	90.00	83.54	Fail
Permanent deformation - AC only (mm)	6.00	11.46	90.00	18.28	Fail

Distress Charts









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HWY 7_Conv_WeakSG

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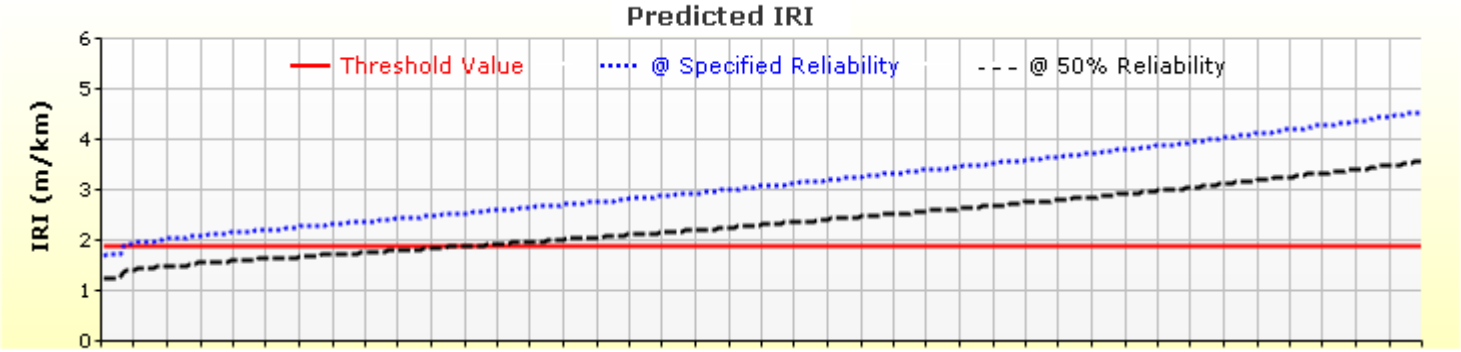
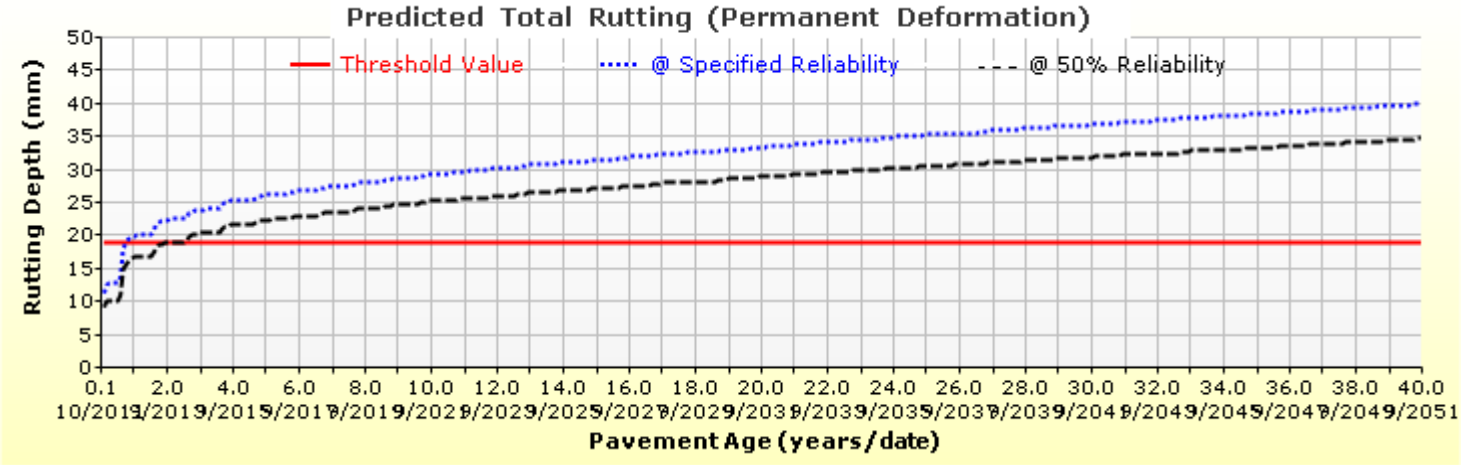
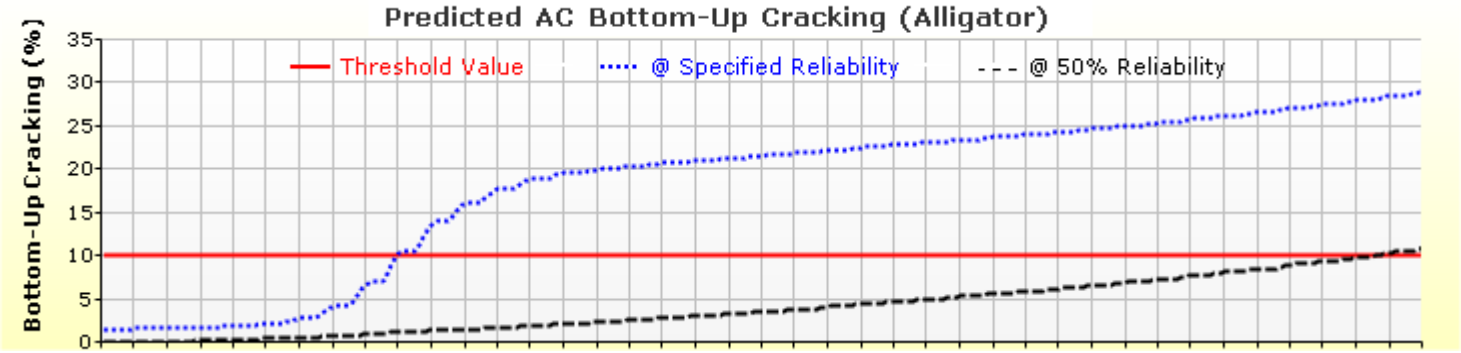
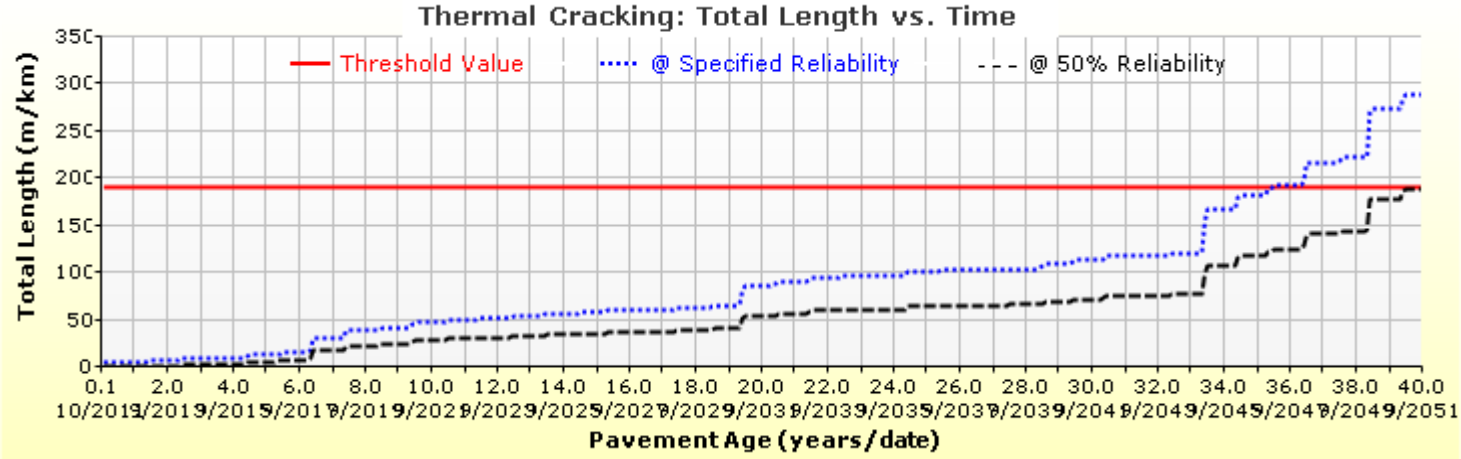
Analysis Output Charts









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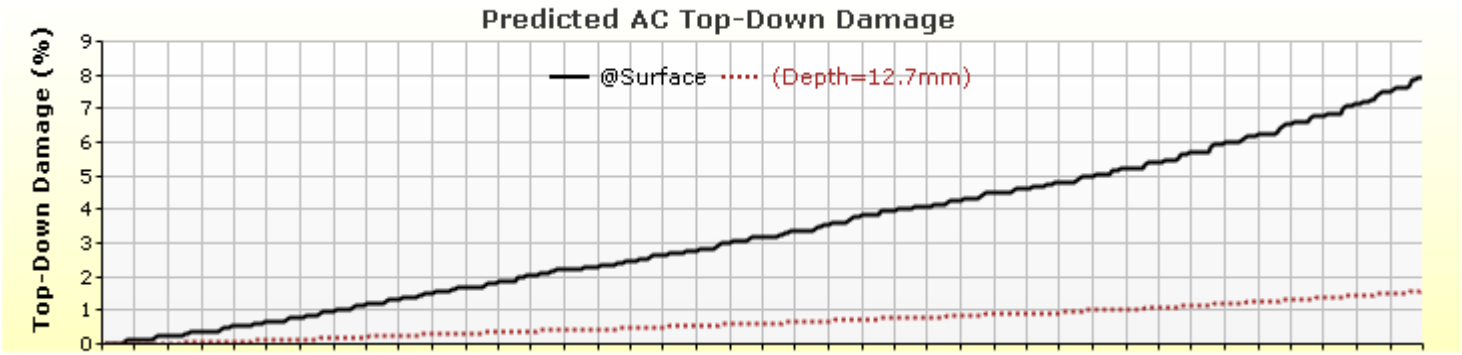

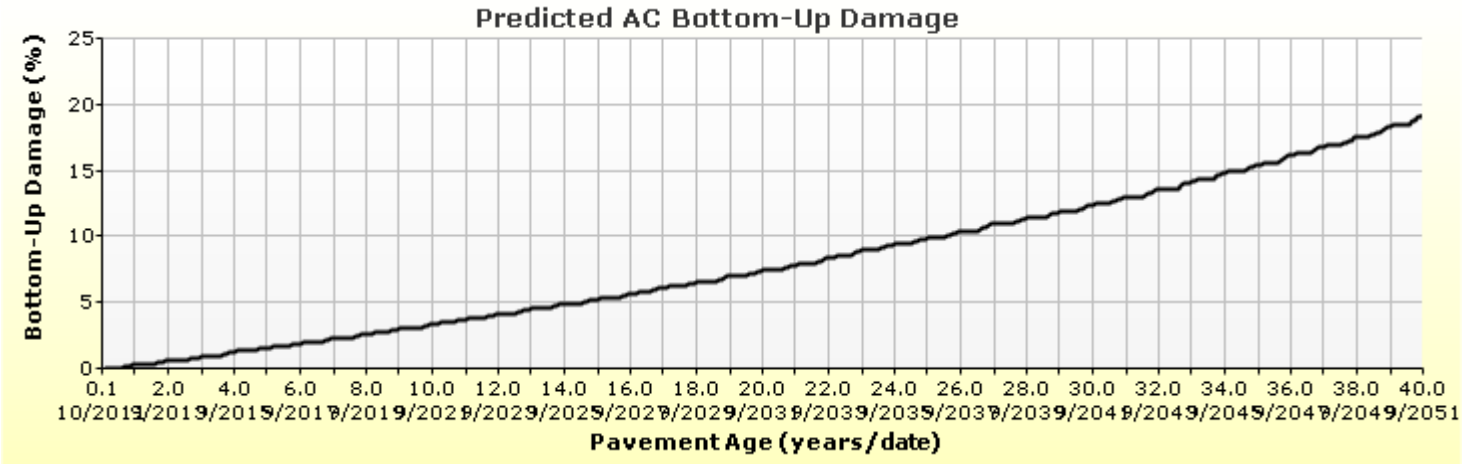

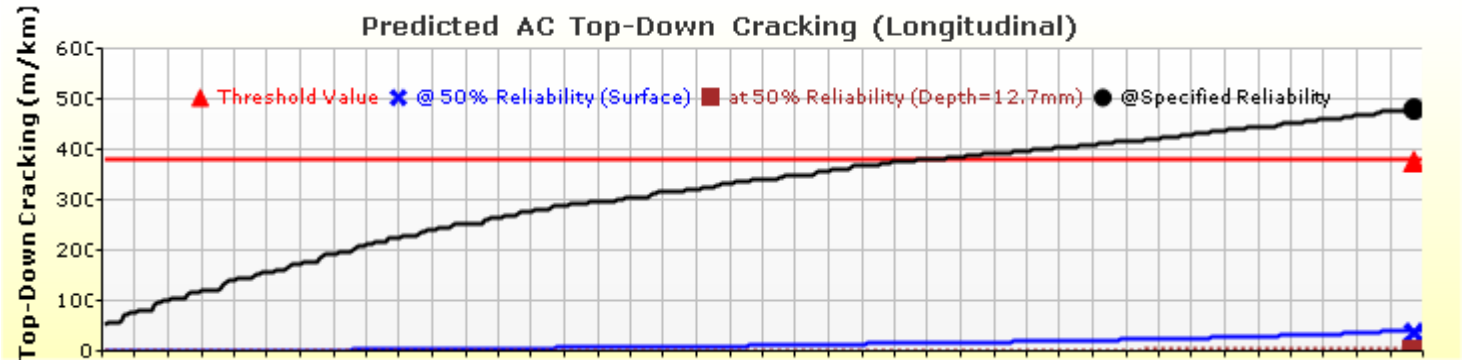

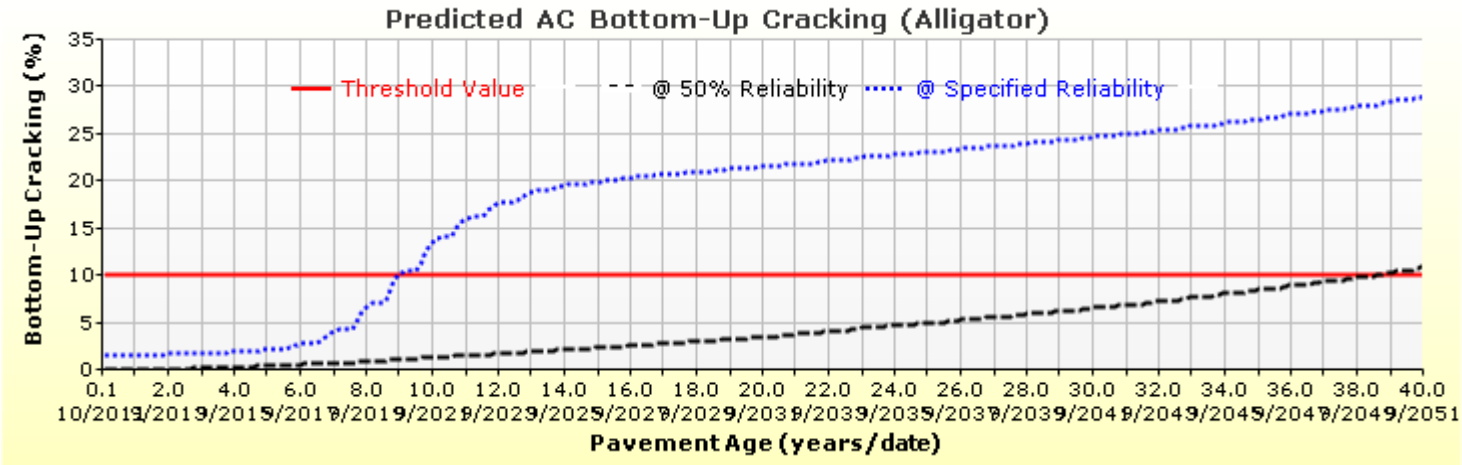

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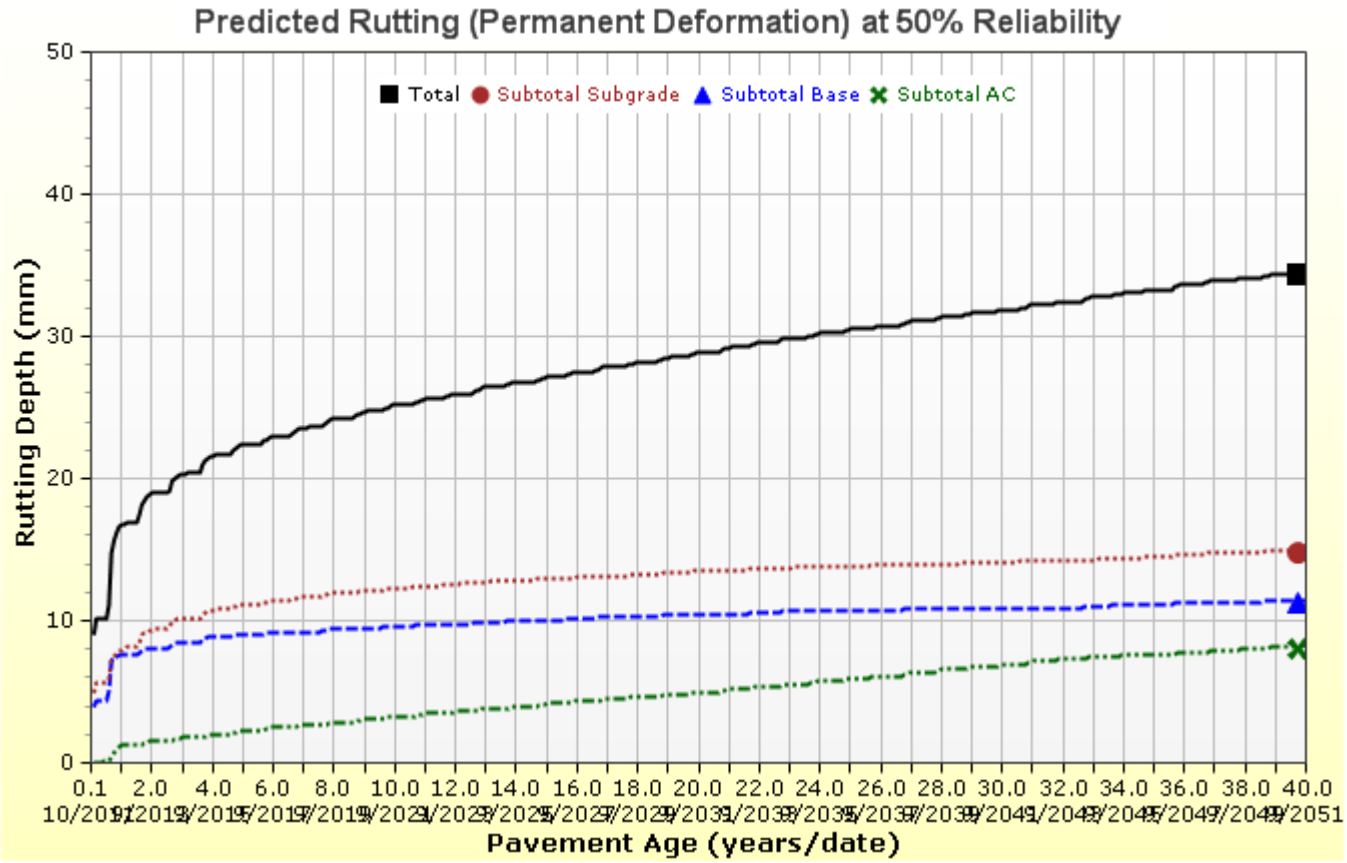

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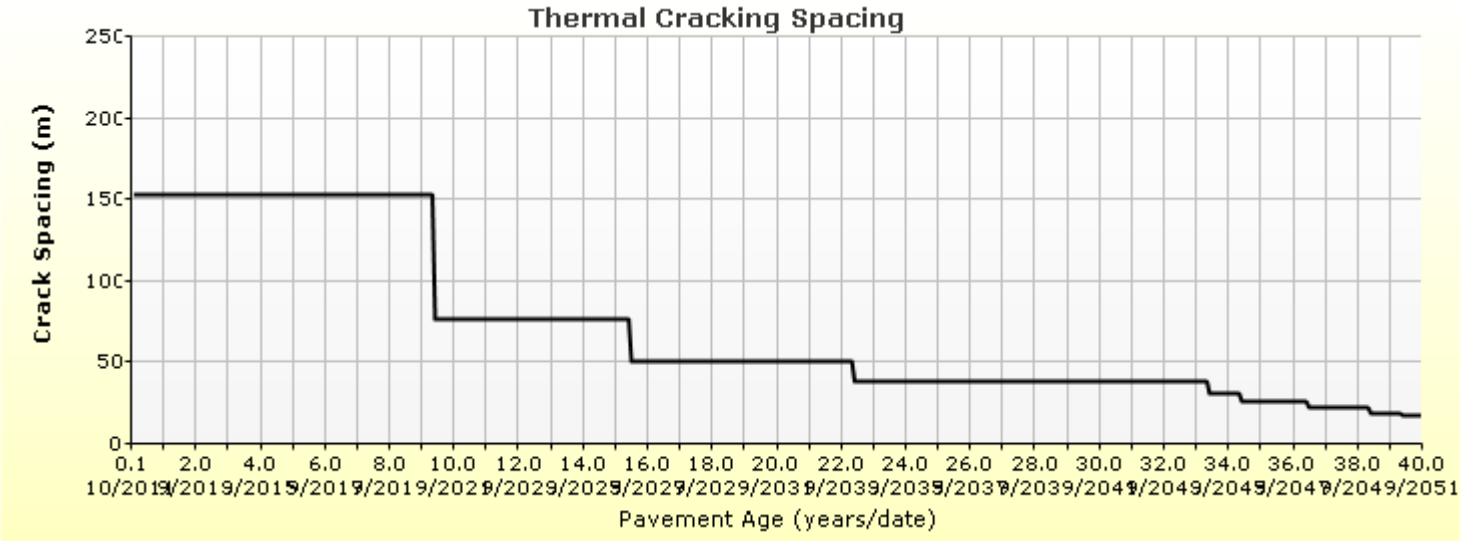

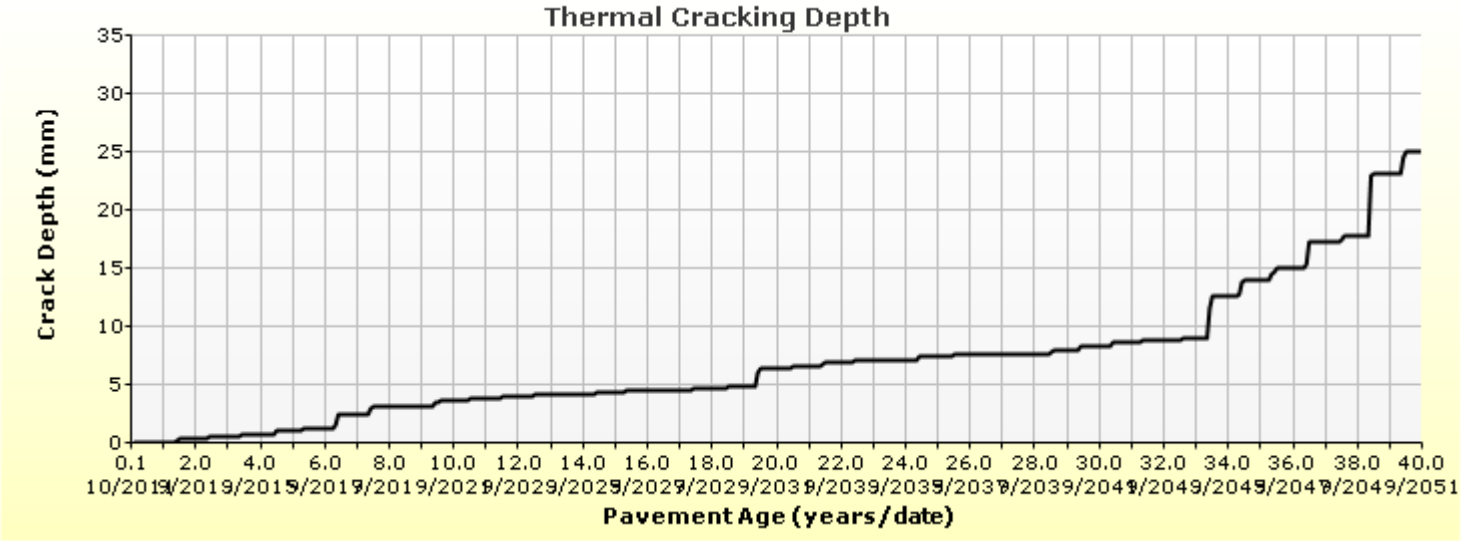

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HWY 7_RubberFB_WeakSG

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Design Inputs

Design Life:	40 years	Base construction:	August, 2011	Climate Data Sources (Lat/Lon)	43.862, -79.37
Design Type:	Flexible Pavement	Pavement construction:	September, 2011		44.117, -77.533
		Traffic opening:	October, 2011		43.677, -79.631

Design Structure

Layer type	Material Type	Thickness(mm):
Flexible	Default asphalt concrete	30.0
Flexible	Default asphalt concrete	50.0
NonStabilized	A-2-4	430.0
NonStabilized	A-2-7	645.0
Subgrade	A-5	Semi-infinite

Volumetric at Construction:

Effective binder content (%)	17.1
Air voids (%)	4.0

Traffic

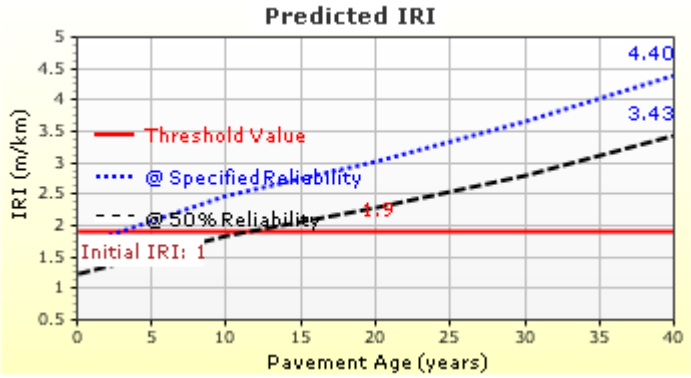
Age (year)	Heavy Trucks (cumulative)
2011 (initial)	1,000
2031 (20 years)	4,750,550
2051 (40 years)	12,812,900

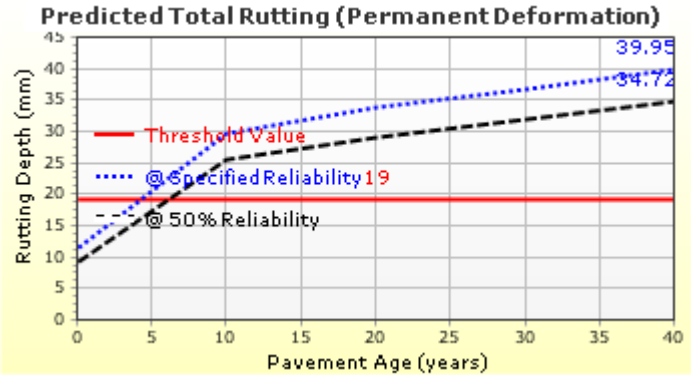
Design Outputs

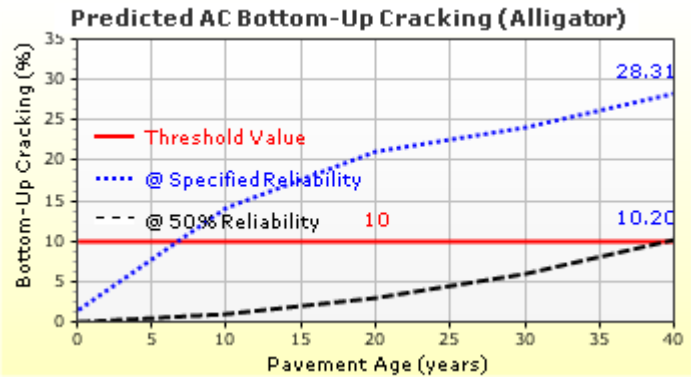
Distress Prediction Summary

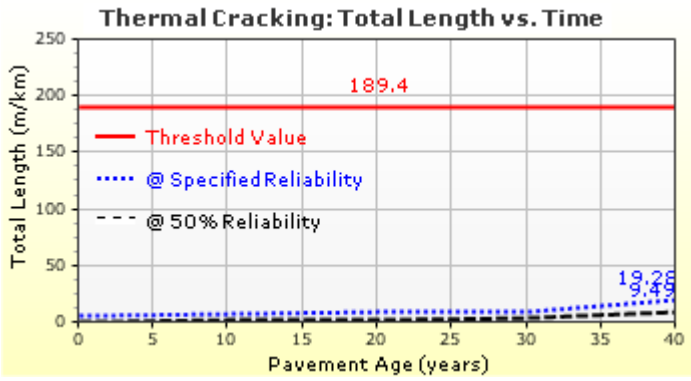
Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (m/km)	1.90	4.40	90.00	2.03	Fail
Permanent deformation - total pavement (mm)	19.00	39.96	90.00	0.01	Fail
AC bottom-up fatigue cracking (percent)	10.00	28.31	90.00	49.44	Fail
AC thermal cracking (m/km)	189.40	19.28	90.00	100.00	Pass
AC top-down fatigue cracking (m/km)	378.80	418.05	90.00	87.58	Fail
Permanent deformation - AC only (mm)	6.00	10.35	90.00	26.86	Fail

Distress Charts









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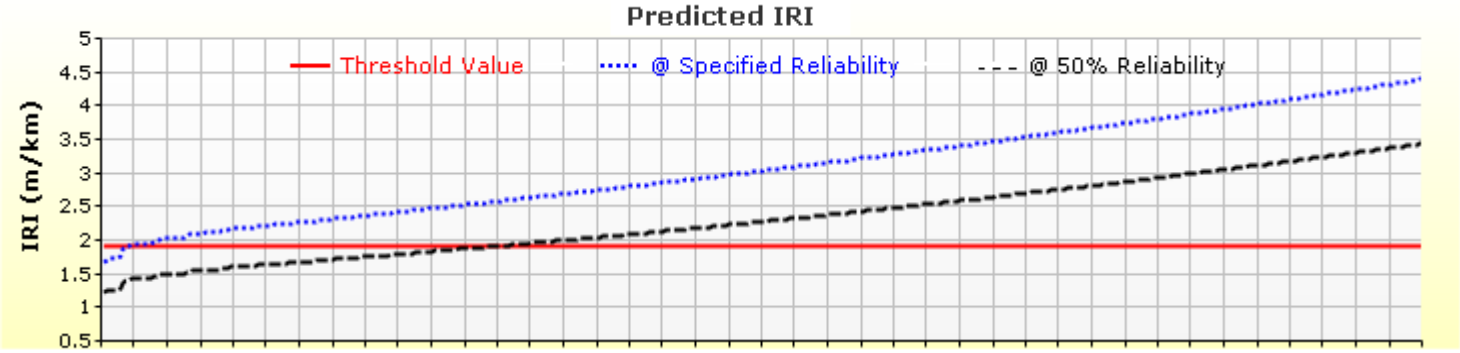
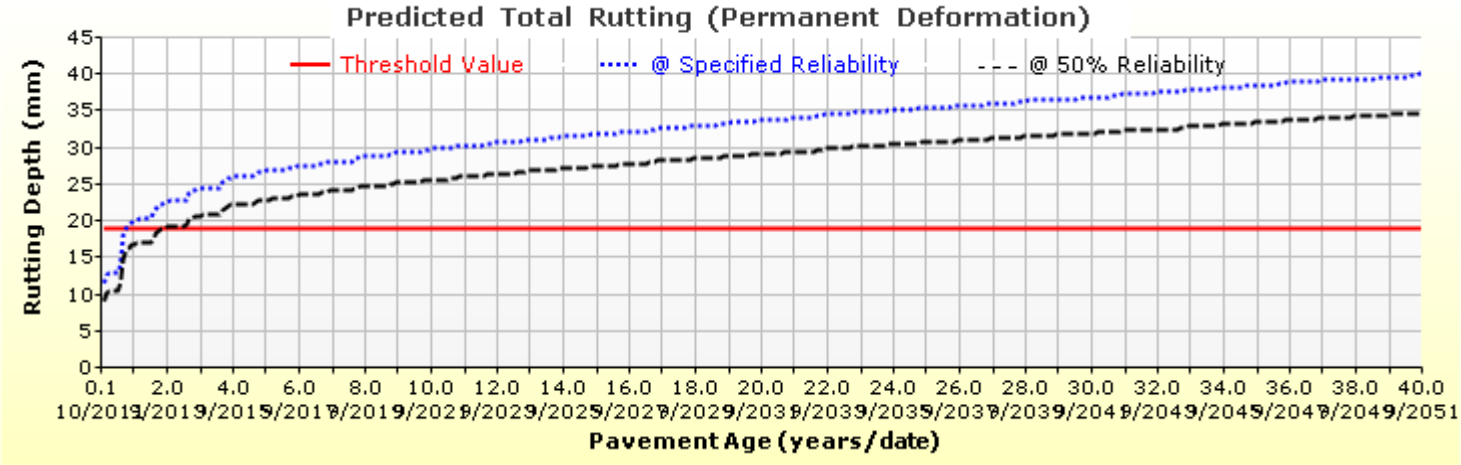
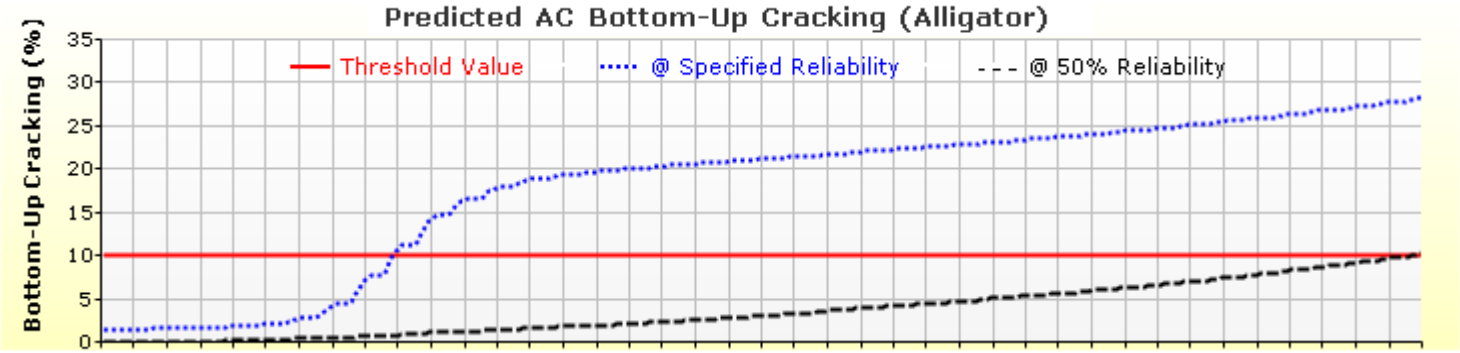
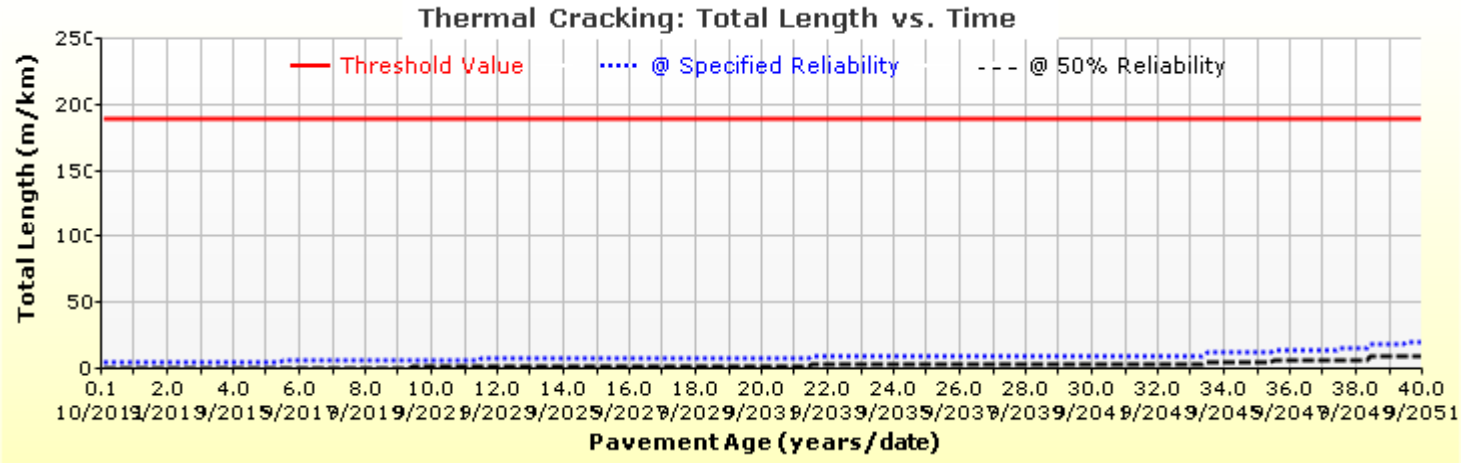
Analysis Output Charts









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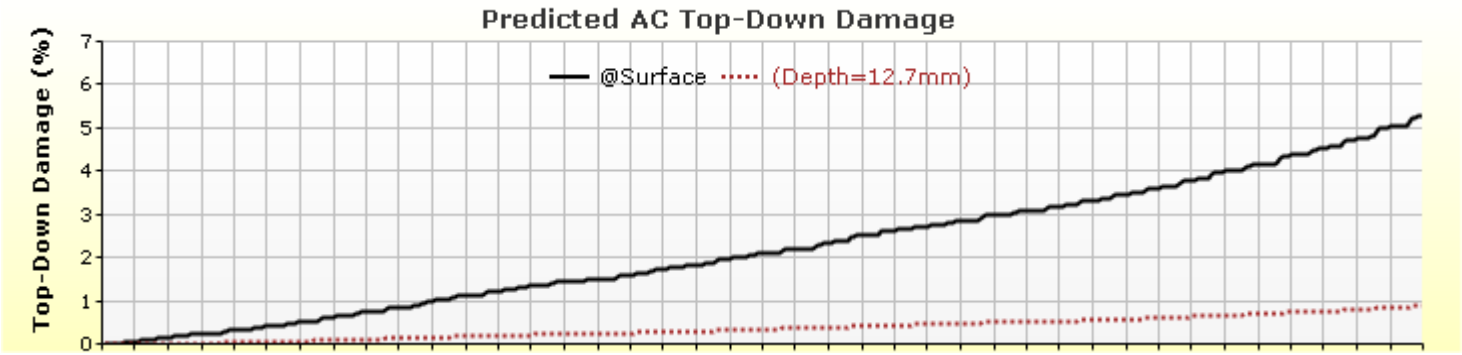

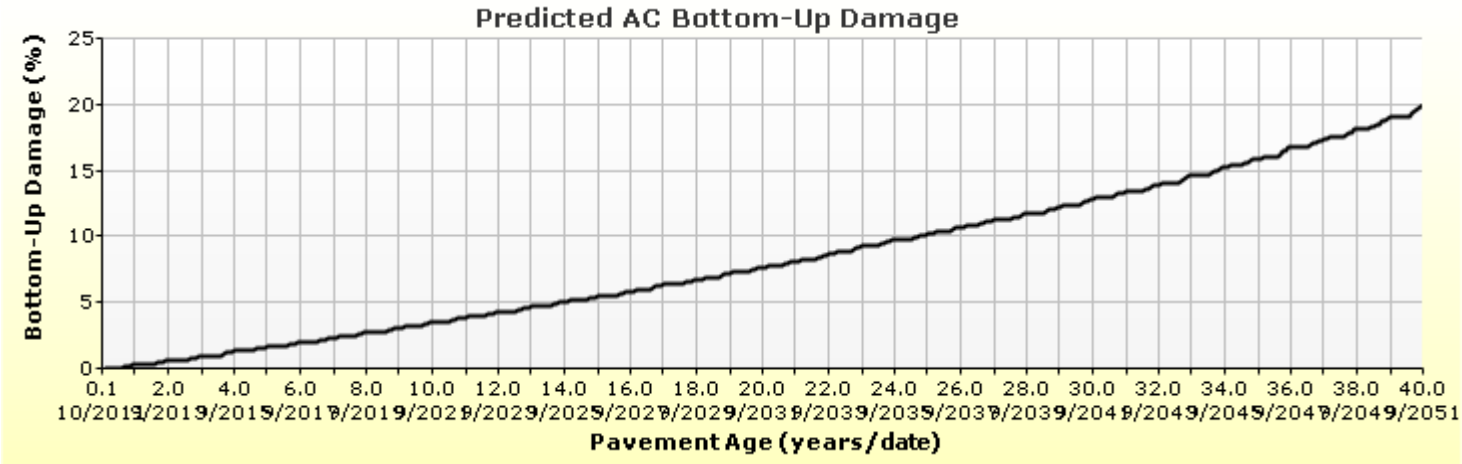

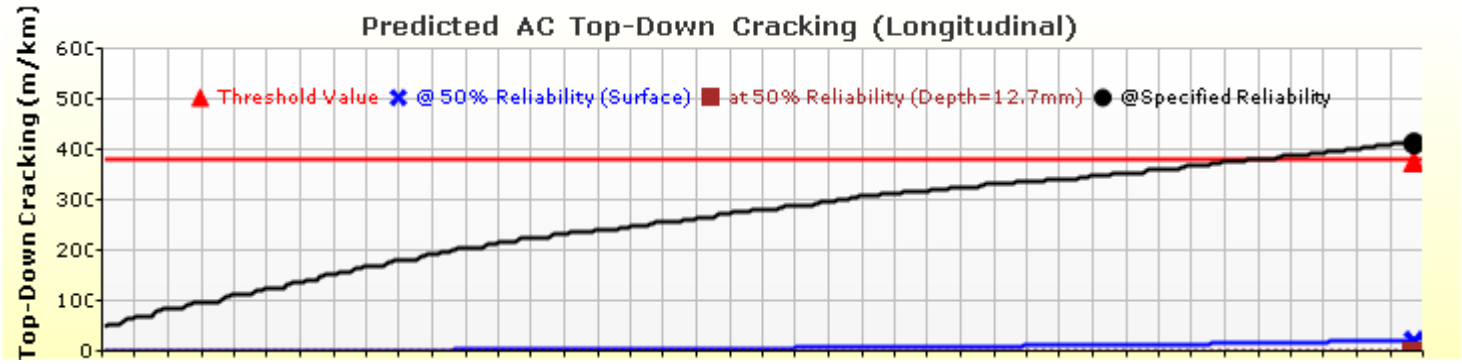

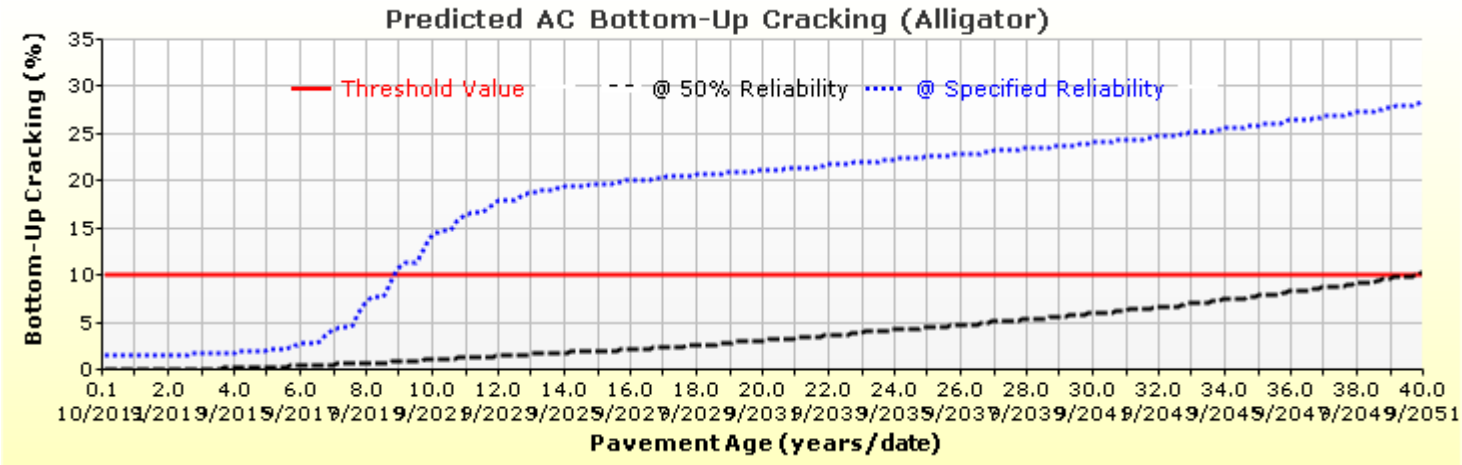

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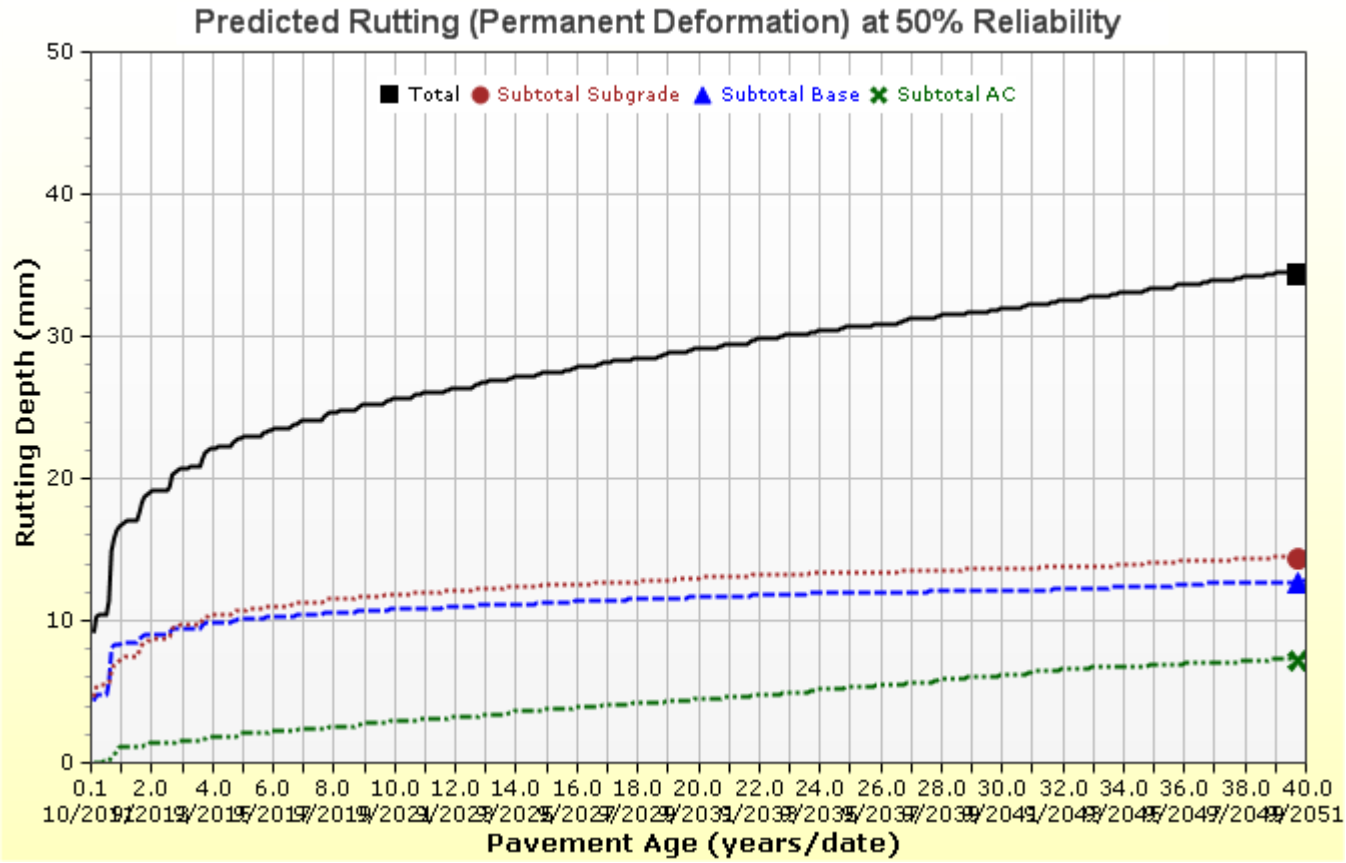

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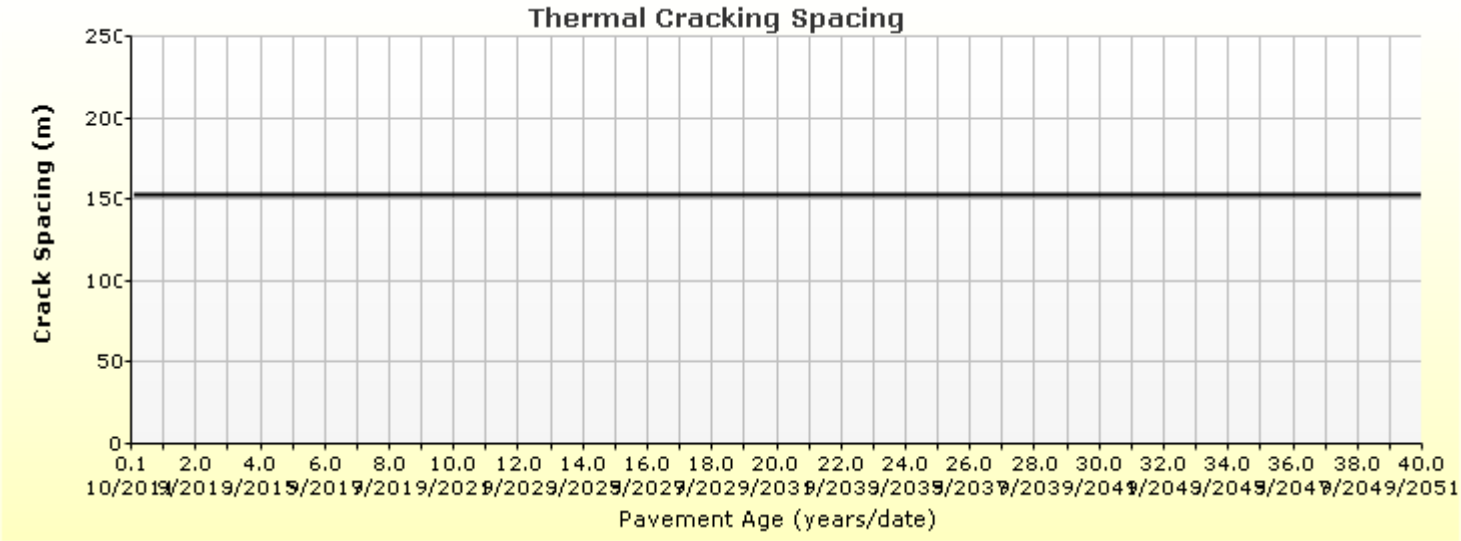

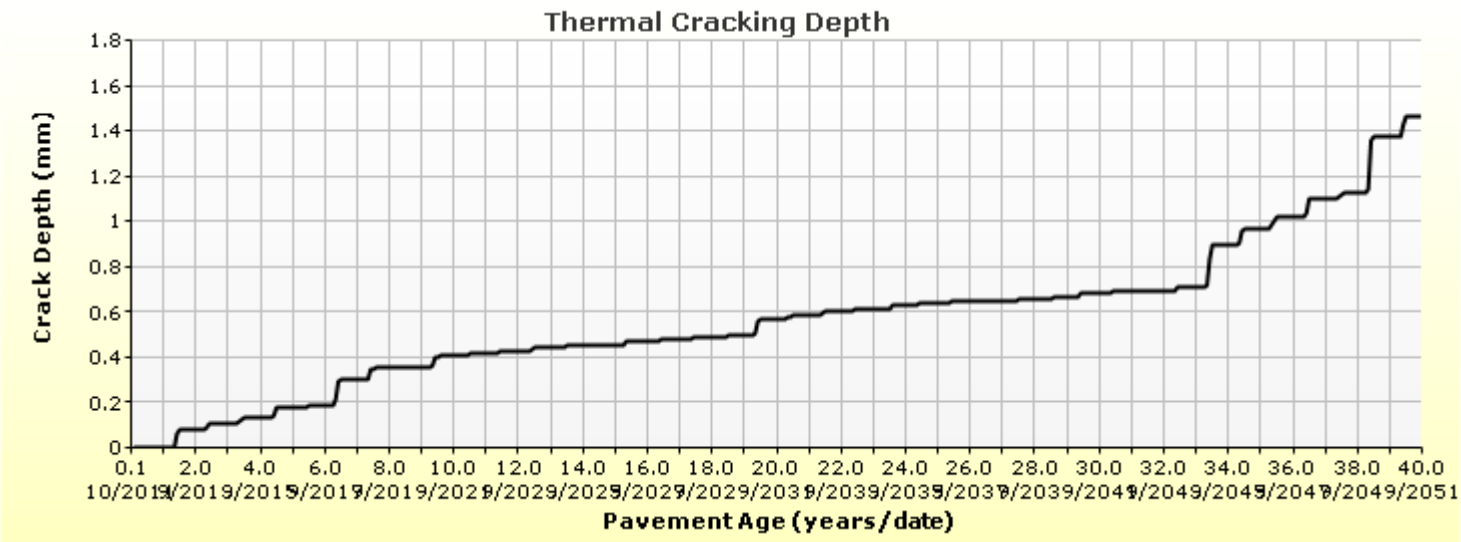

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HWY 7_RubberTB_WeakSG

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Volumetric at Construction:

Effective binder content (%)	13.3
Air voids (%)	4.0

Traffic

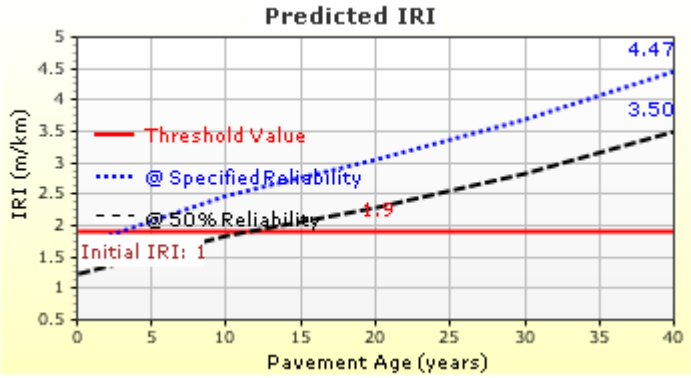
Age (year)	Heavy Trucks (cumulative)
2011 (initial)	1,000
2031 (20 years)	4,750,550
2051 (40 years)	12,812,900

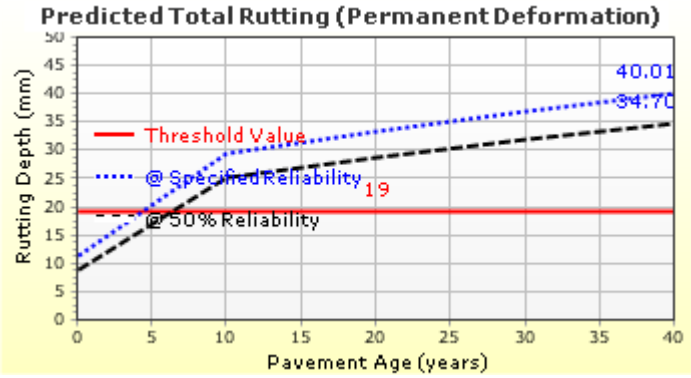
Design Outputs

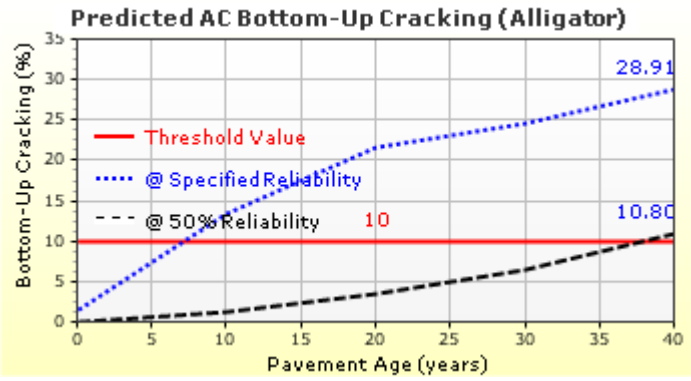
Distress Prediction Summary

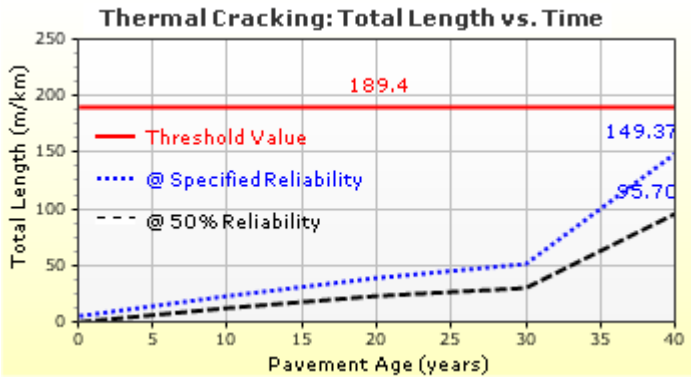
Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (m/km)	1.90	4.47	90.00	1.75	Fail
Permanent deformation - total pavement (mm)	19.00	40.00	90.00	0.01	Fail
AC bottom-up fatigue cracking (percent)	10.00	28.91	90.00	47.74	Fail
AC thermal cracking (m/km)	189.40	149.37	90.00	98.74	Pass
AC top-down fatigue cracking (m/km)	378.80	486.05	90.00	83.44	Fail
Permanent deformation - AC only (mm)	6.00	11.47	90.00	18.18	Fail

Distress Charts









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HWY 7_RubberTB_WeakSG

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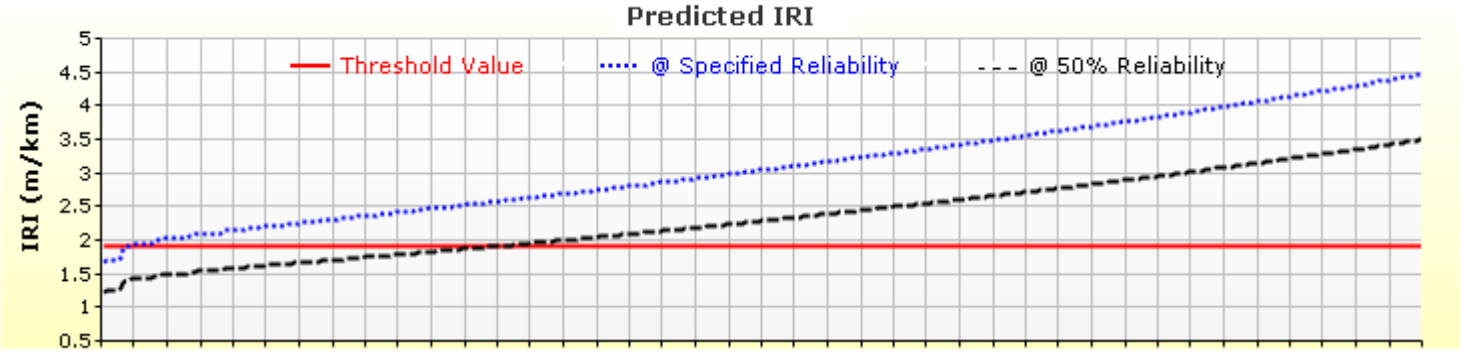
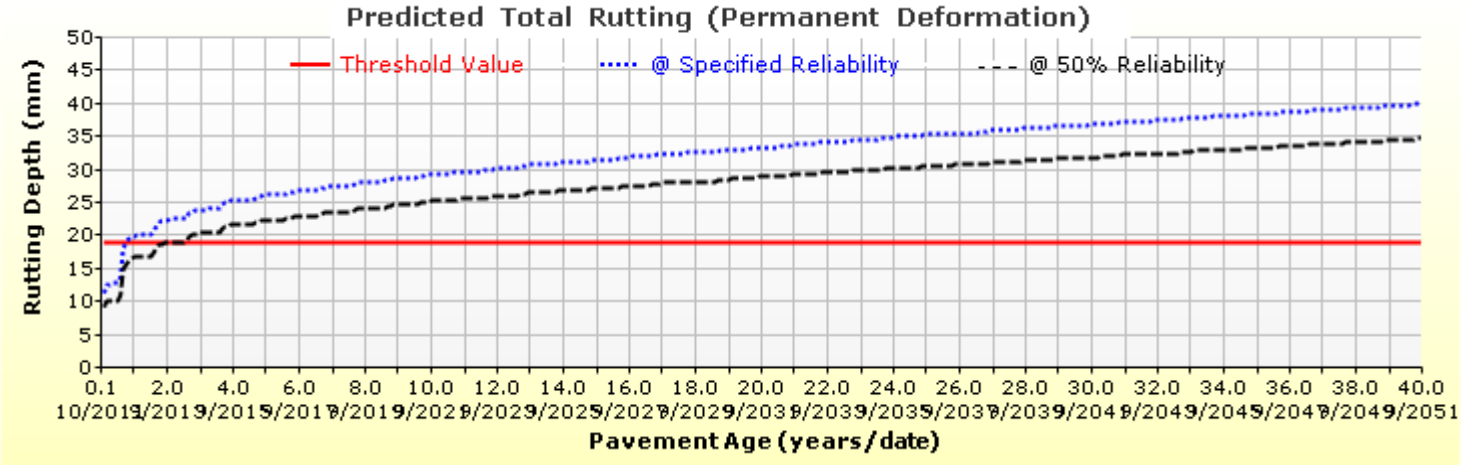
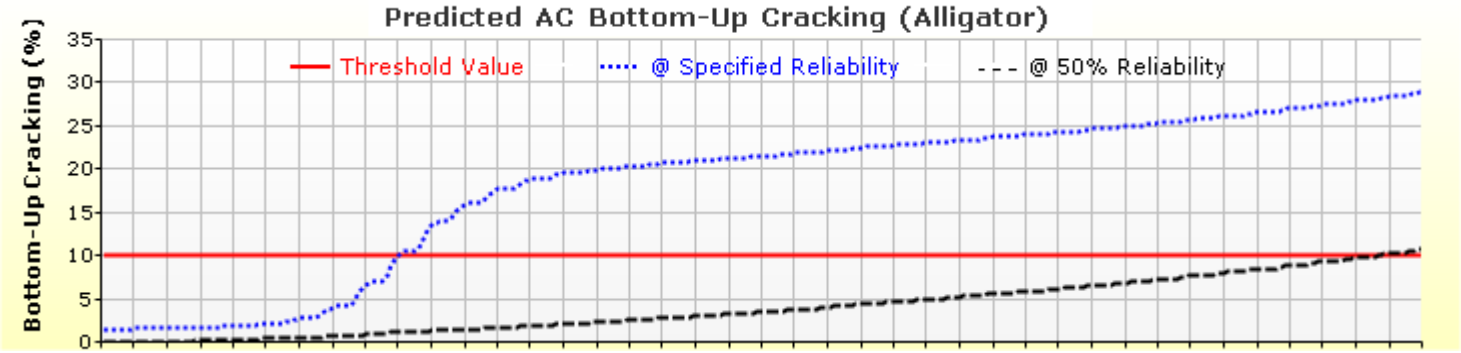
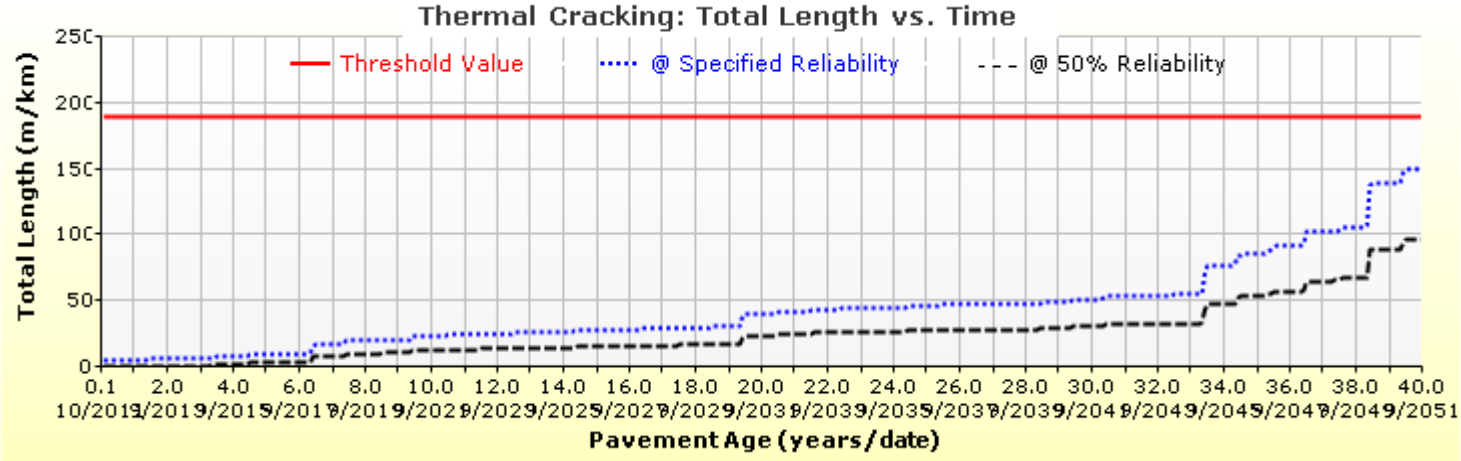
Analysis Output Charts









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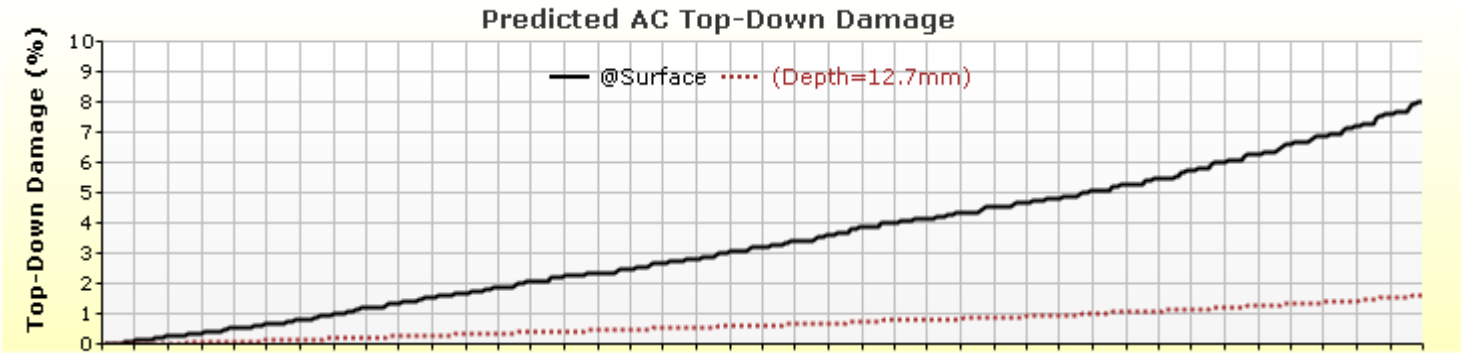

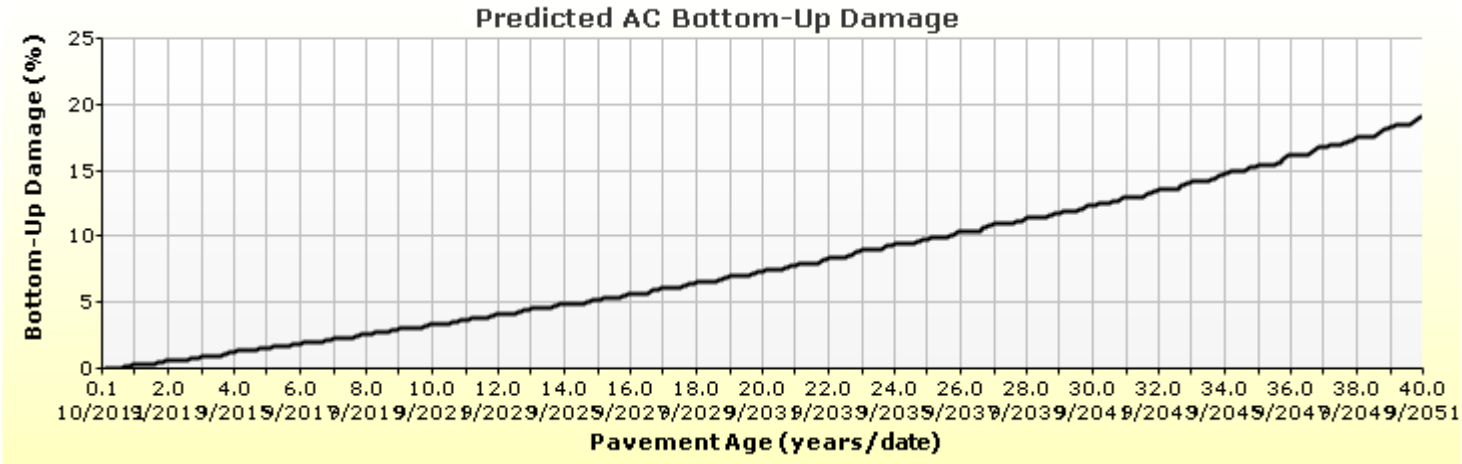

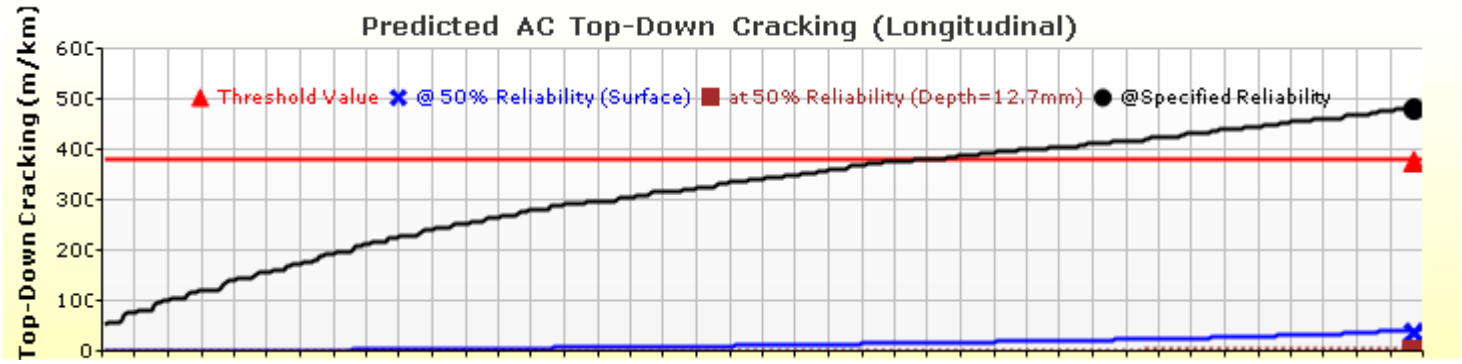

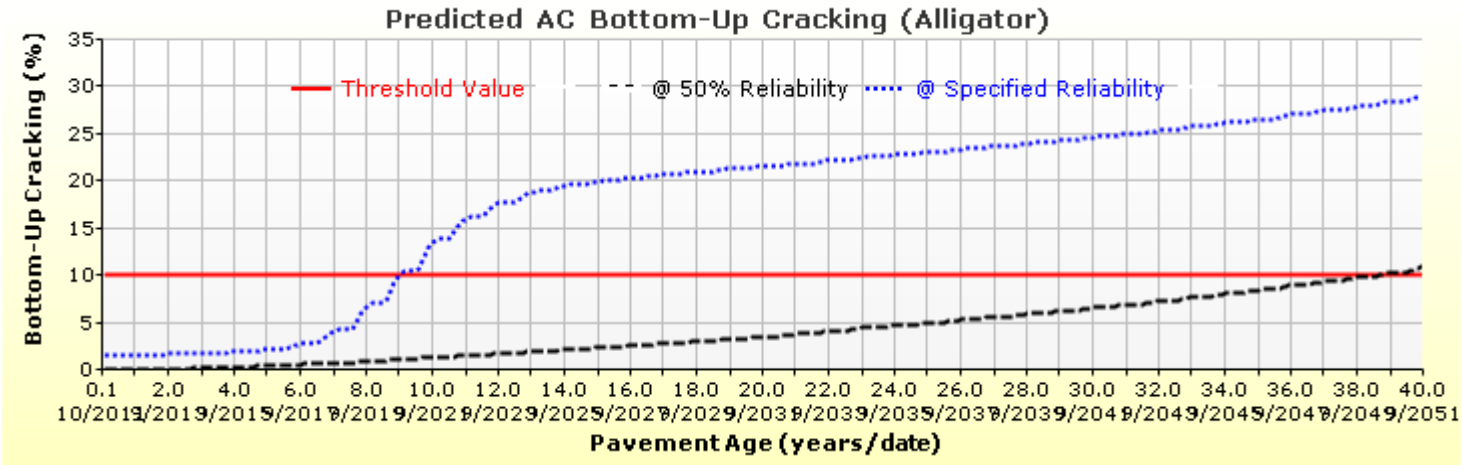

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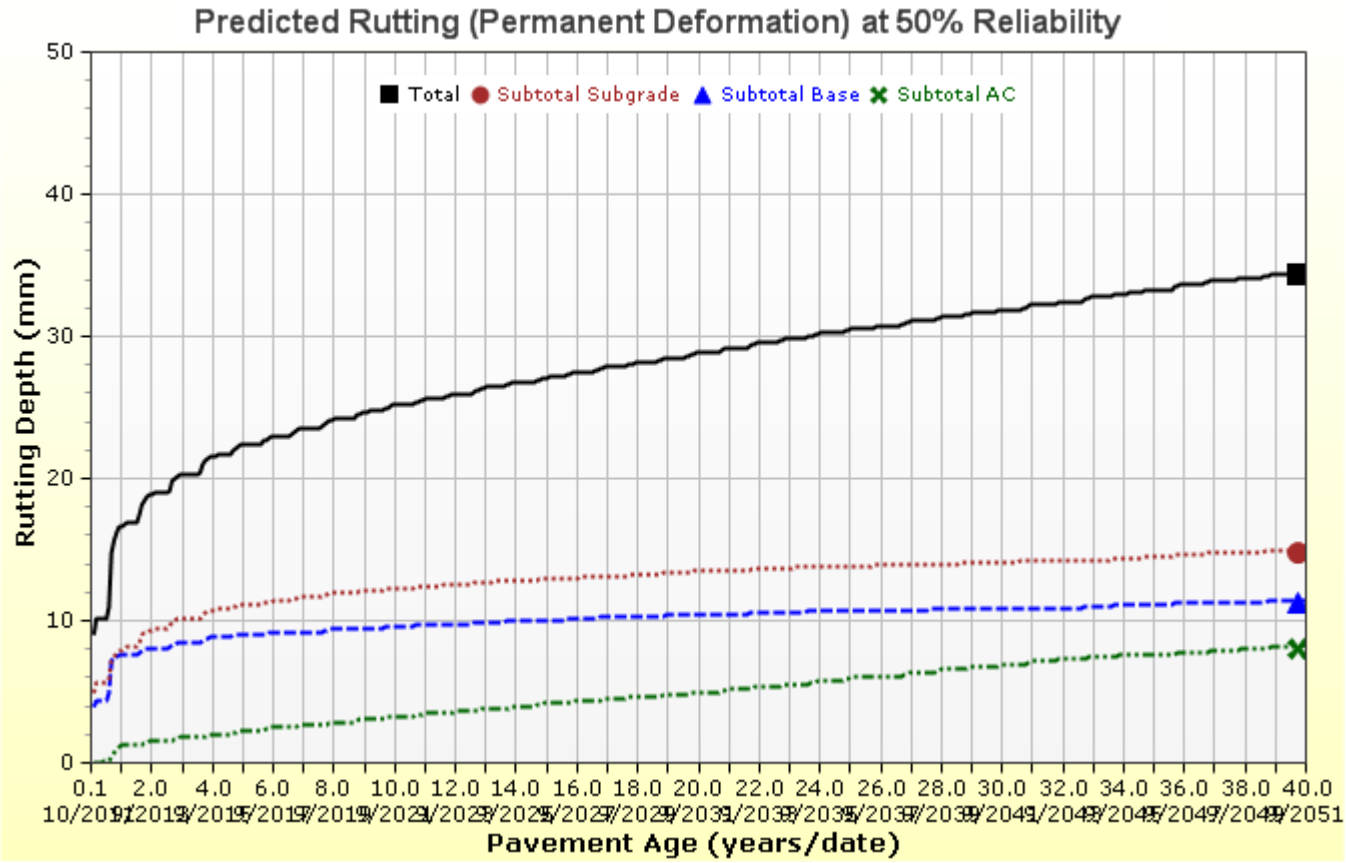
HWY 7_RubberTB_WeakSG

File Name: C:\Users\admin\Desktop\Mo MASc Thesis\HWY 7_RubberTB_WeakSG.dgpx



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HWY 7_RubberTB_WeakSG

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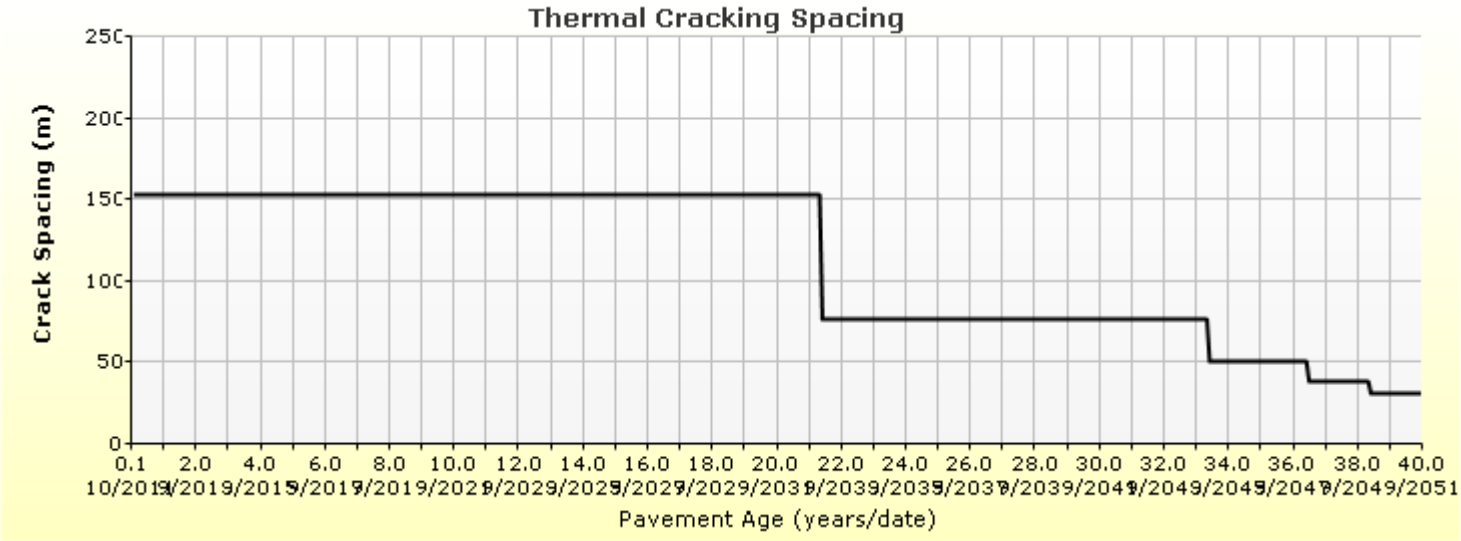

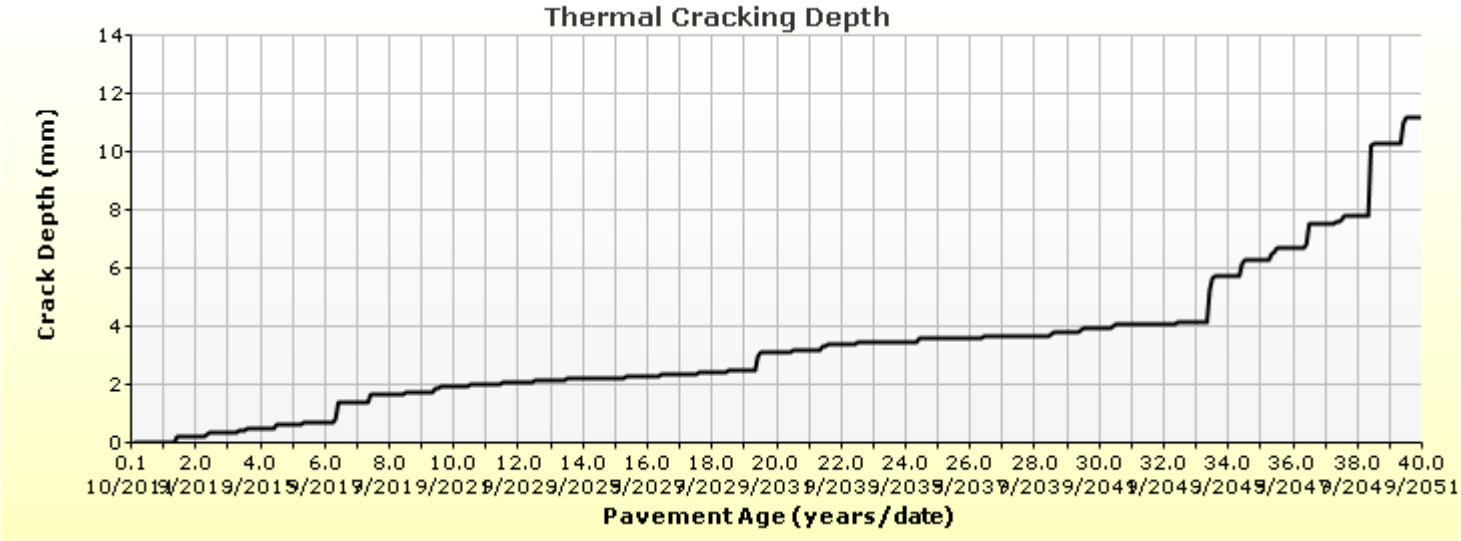

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Appendix B: Highway 35 AASHTOWare Results (Weak Subgrade)

HWY 35_Conv_WeakSG

File Name: C:\Users\admin\Desktop\Mo MASc Thesis\HWY 35_Conv_WeakSG.dgpx

Design Inputs

Design Life:	40 years	Base construction:	August, 2011	Climate Data Sources (Lat/Lon)	43.862, -79.37
Design Type:	Flexible Pavement	Pavement construction:	September, 2011		44.117, -77.533
		Traffic opening:	October, 2011		43.677, -79.631

Design Structure

Layer type	Material Type	Thickness(mm):
Flexible	Default asphalt concrete	50.0
Flexible	Default asphalt concrete	70.0
NonStabilized	A-2-4	240.0
NonStabilized	A-2-7	550.0
Subgrade	A-5	Semi-infinite

Volumetric at Construction:

Effective binder content (%)	12.9
Air voids (%)	4.0

Traffic

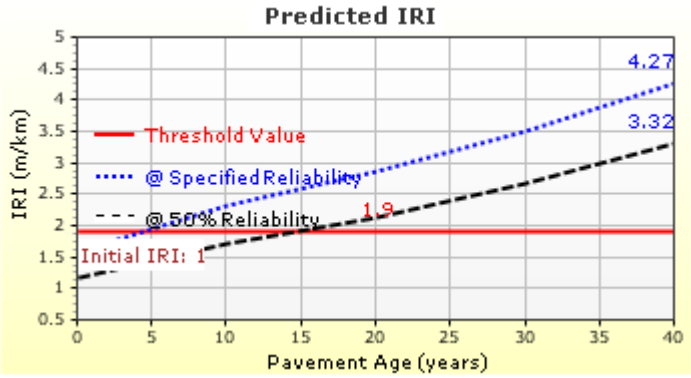
Age (year)	Heavy Trucks (cumulative)
2011 (initial)	750
2031 (20 years)	3,498,820
2051 (40 years)	9,232,040

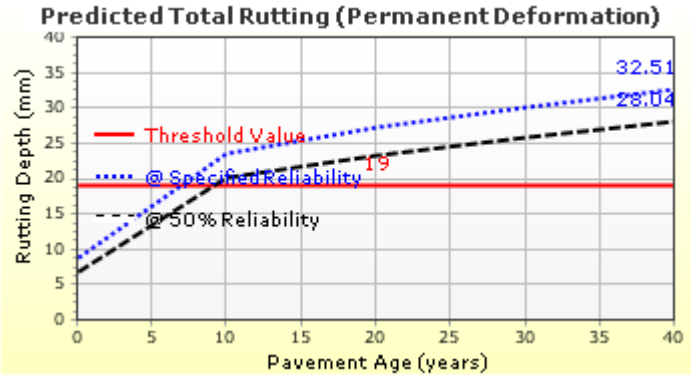
Design Outputs

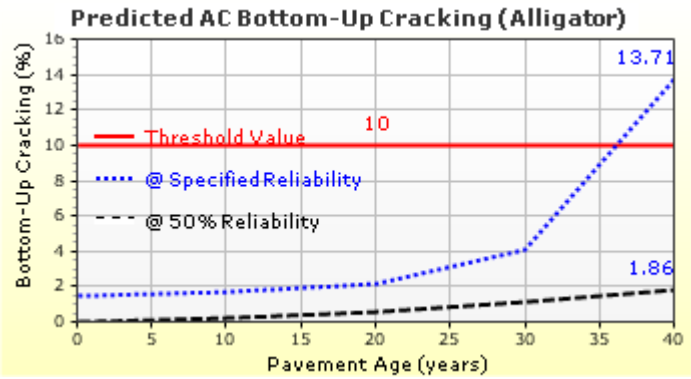
Distress Prediction Summary

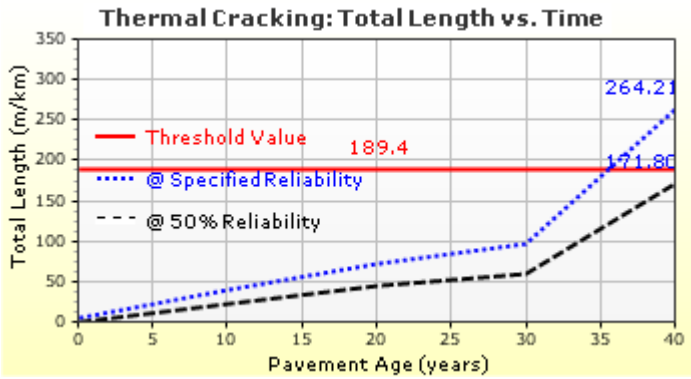
Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (m/km)	1.90	4.27	90.00	2.65	Fail
Permanent deformation - total pavement (mm)	19.00	32.52	90.00	0.48	Fail
AC bottom-up fatigue cracking (percent)	10.00	13.71	90.00	81.07	Fail
AC thermal cracking (m/km)	189.40	264.21	90.00	59.64	Fail
AC top-down fatigue cracking (m/km)	378.80	253.17	90.00	97.31	Pass
Permanent deformation - AC only (mm)	6.00	8.27	90.00	53.25	Fail

Distress Charts









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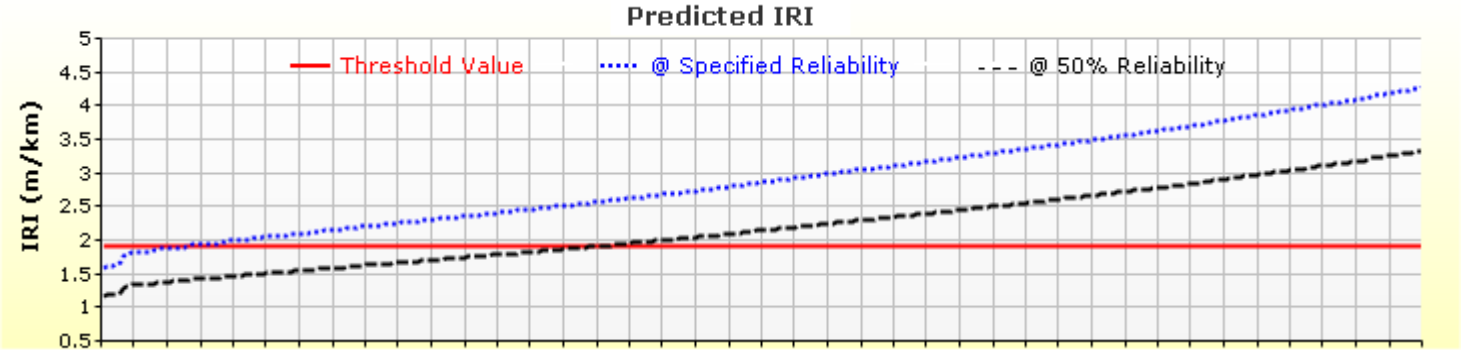
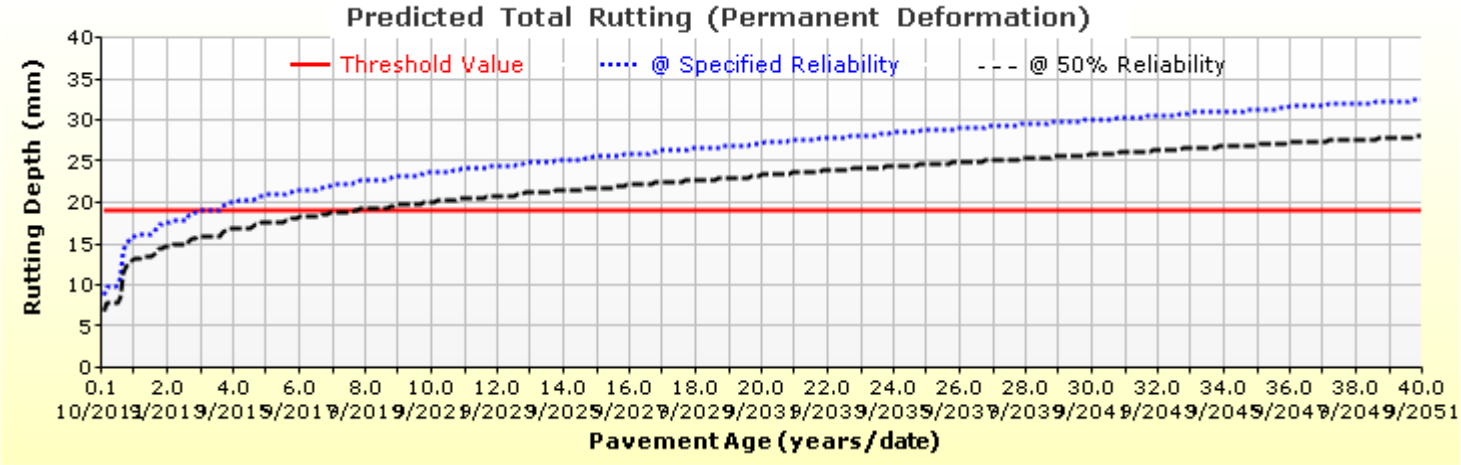
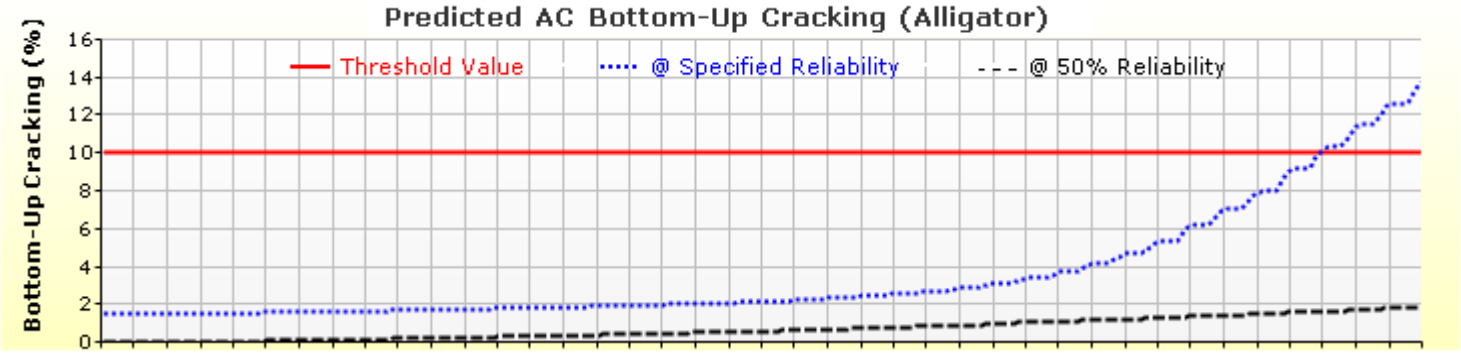
HWY 35_Conv_WeakSG

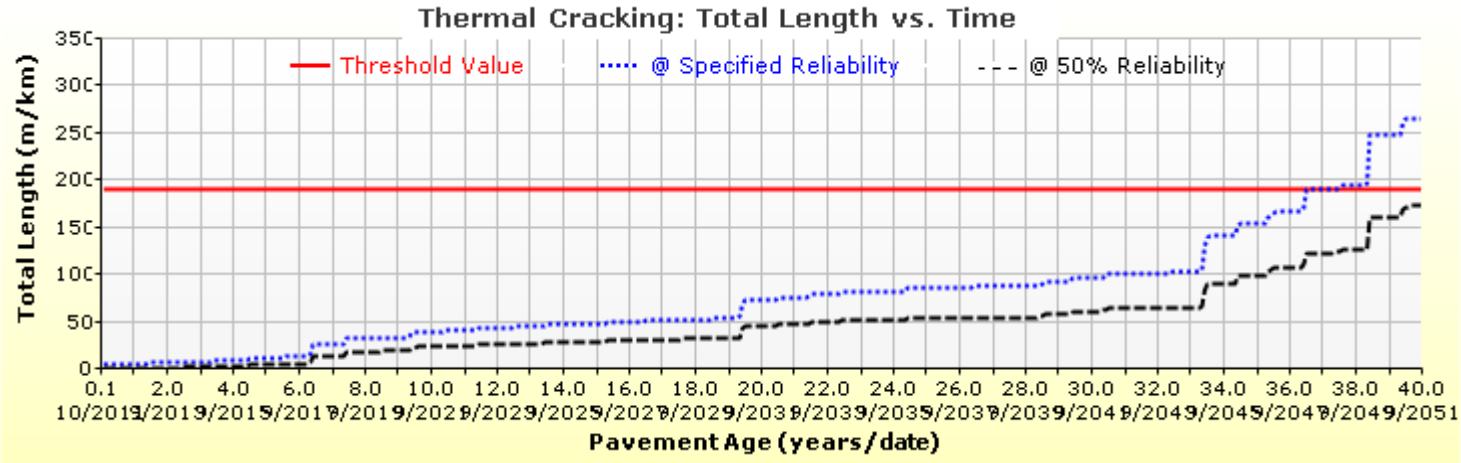
File Name: C:\Users\admin\Desktop\Mo MASc Thesis\HWY 35_Conv_WeakSG.dgpx

Analysis Output Charts









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HWY 35_Conv_WeakSG

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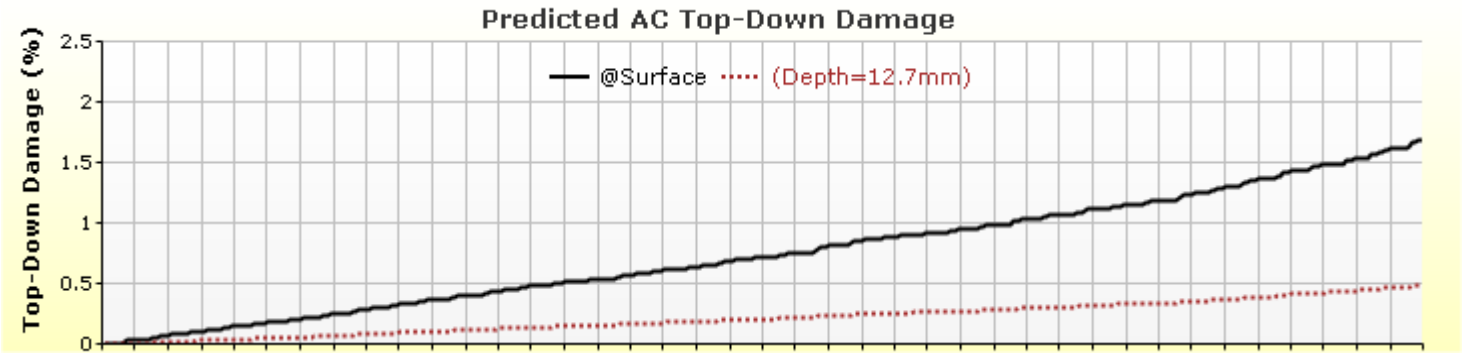

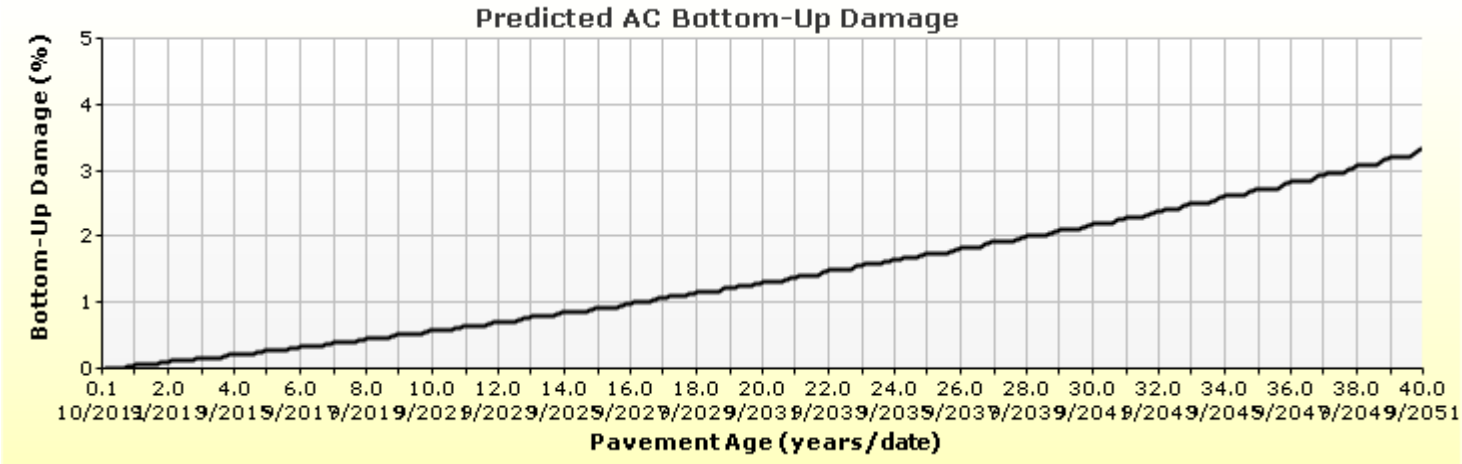

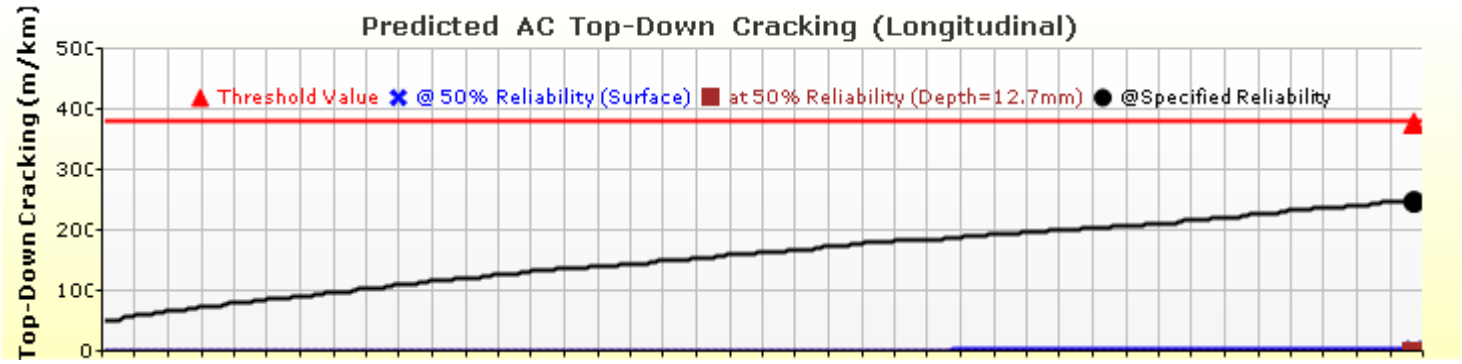

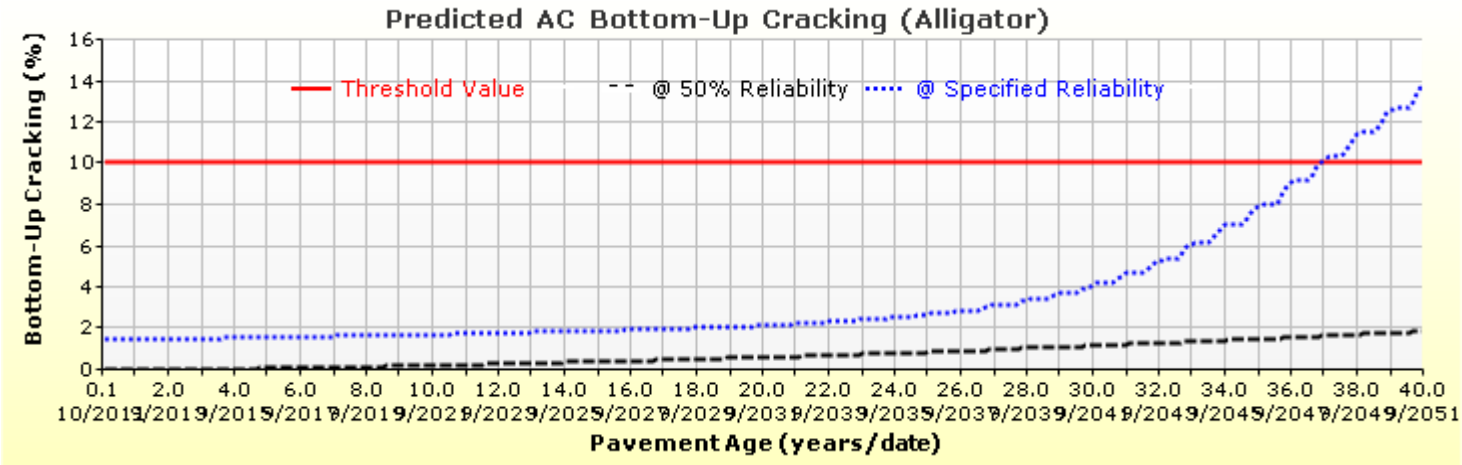

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HWY 35_Conv_WeakSG

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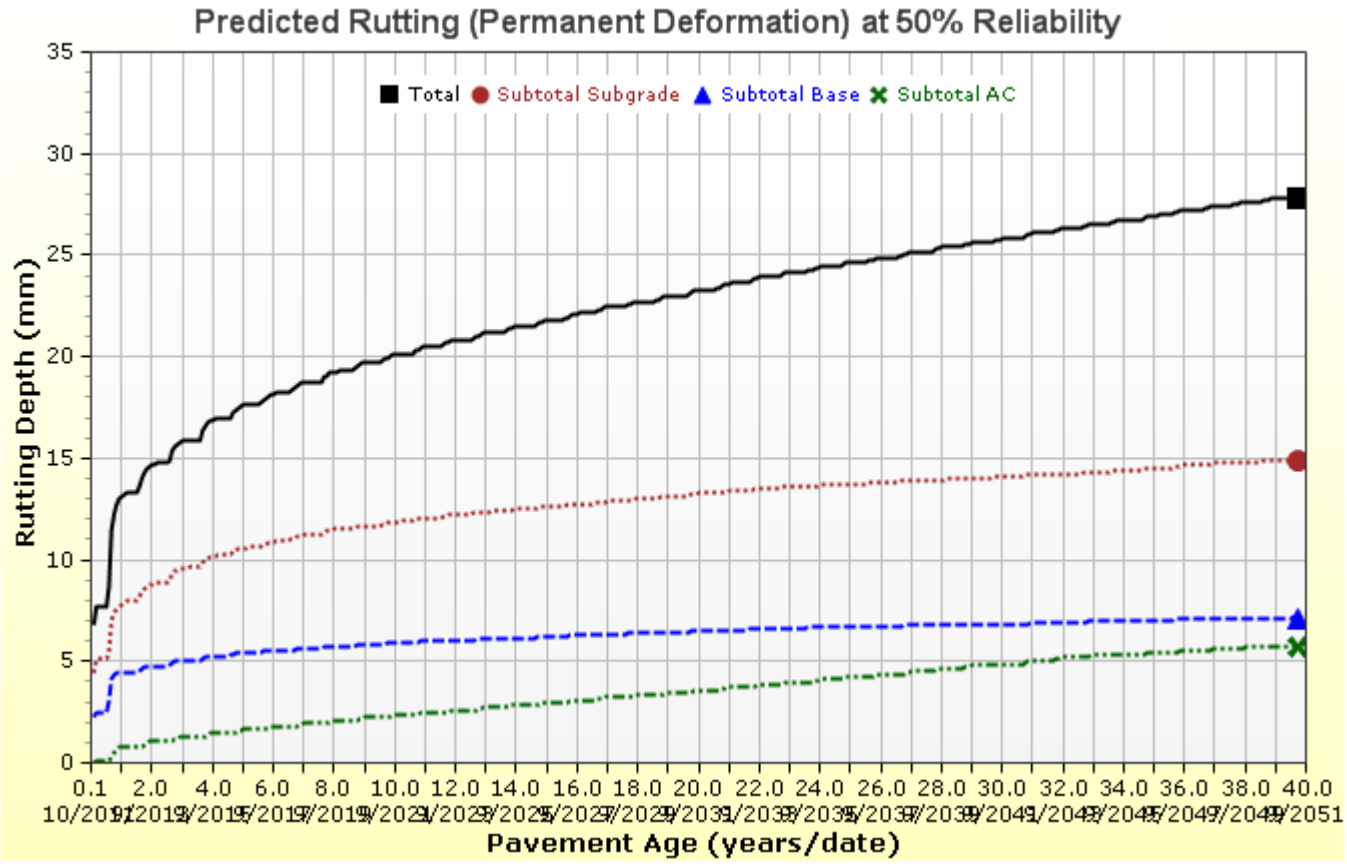

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HWY 35_Conv_WeakSG

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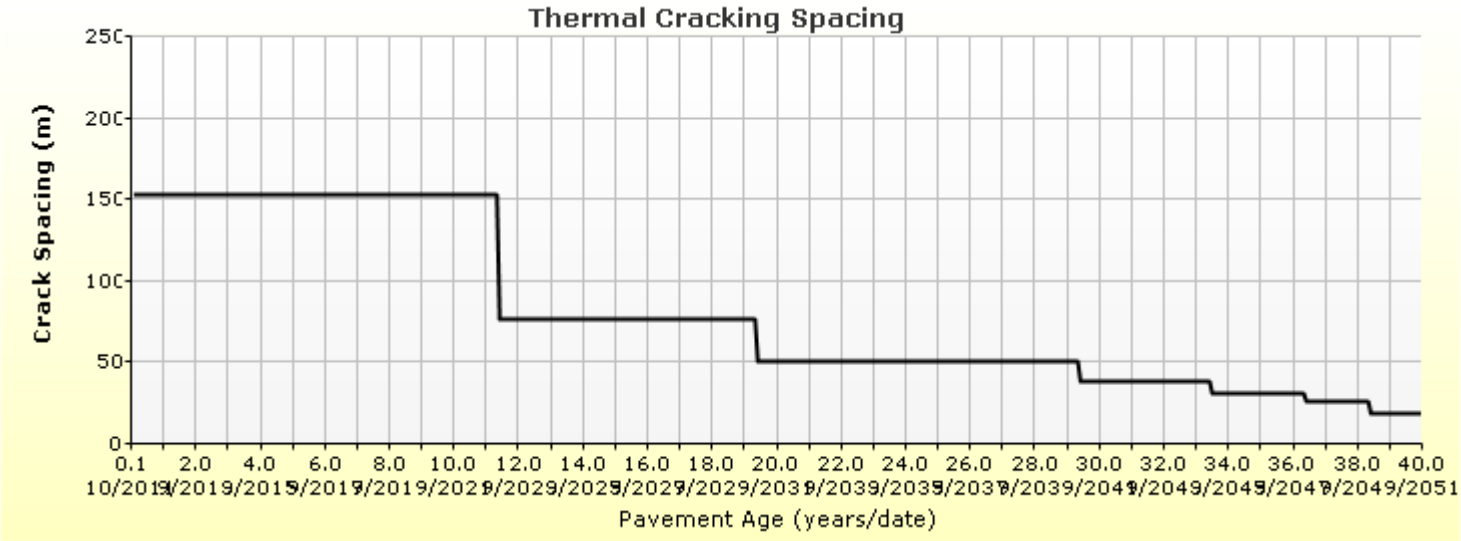

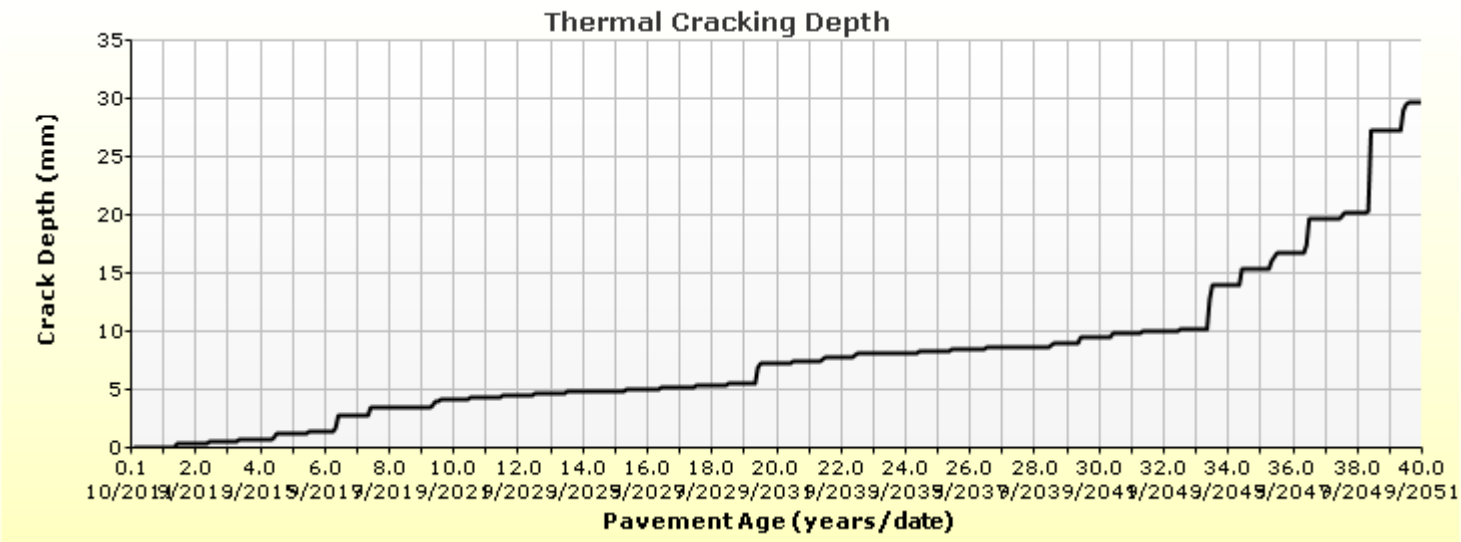

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HWY 35_Rubber_WeakSG

File Name: C:\Users\admin\Desktop\Mo MASc Thesis\HWY 35_Rubber_WeakSG.dgpx

Design Inputs

Design Life:	40 years	Base construction:	August, 2011	Climate Data Sources (Lat/Lon)	43.862, -79.37
Design Type:	Flexible Pavement	Pavement construction:	September, 2011		44.117, -77.533
		Traffic opening:	October, 2011		43.677, -79.631

Design Structure

Layer type	Material Type	Thickness(mm):
Flexible	Default asphalt concrete	50.0
Flexible	Default asphalt concrete	70.0
NonStabilized	A-2-4	240.0
NonStabilized	A-2-7	550.0
Subgrade	A-5	Semi-infinite

Volumetric at Construction:

Effective binder content (%)	16.6
Air voids (%)	4.0

Traffic

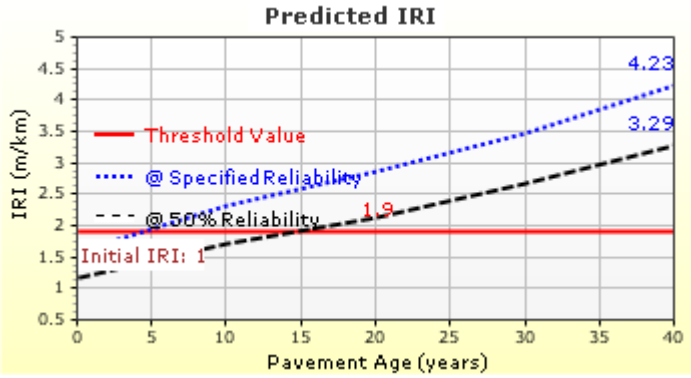
Age (year)	Heavy Trucks (cumulative)
2011 (initial)	750
2031 (20 years)	3,498,820
2051 (40 years)	9,232,040

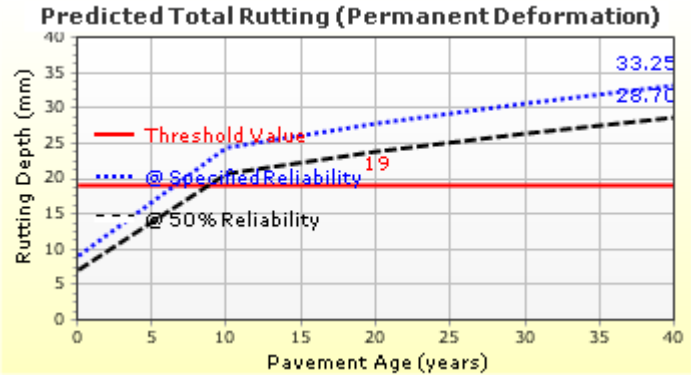
Design Outputs

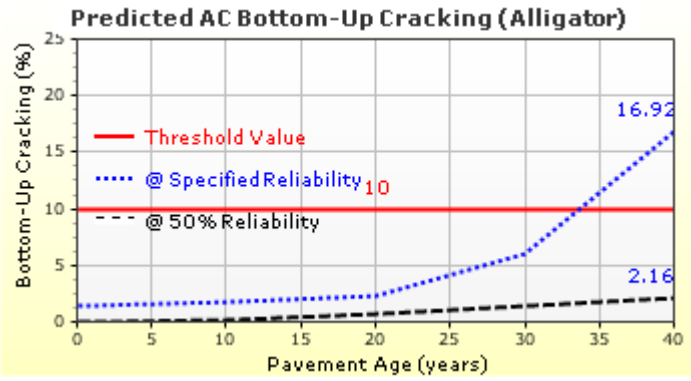
Distress Prediction Summary

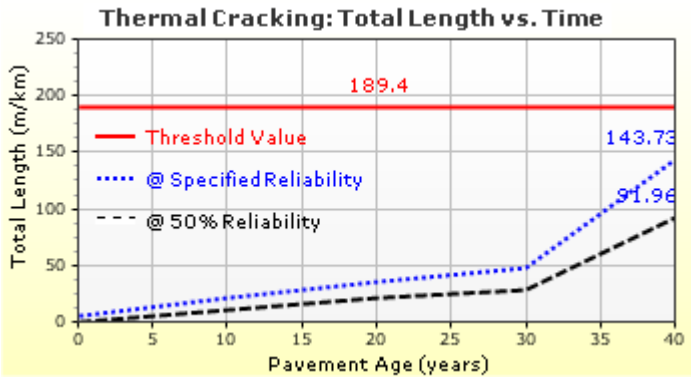
Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (m/km)	1.90	4.23	90.00	2.89	Fail
Permanent deformation - total pavement (mm)	19.00	33.24	90.00	0.31	Fail
AC bottom-up fatigue cracking (percent)	10.00	16.92	90.00	75.19	Fail
AC thermal cracking (m/km)	189.40	143.73	90.00	99.21	Pass
AC top-down fatigue cracking (m/km)	378.80	244.69	90.00	97.70	Pass
Permanent deformation - AC only (mm)	6.00	8.34	90.00	52.12	Fail

Distress Charts









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HWY 35_Rubber_WeakSG

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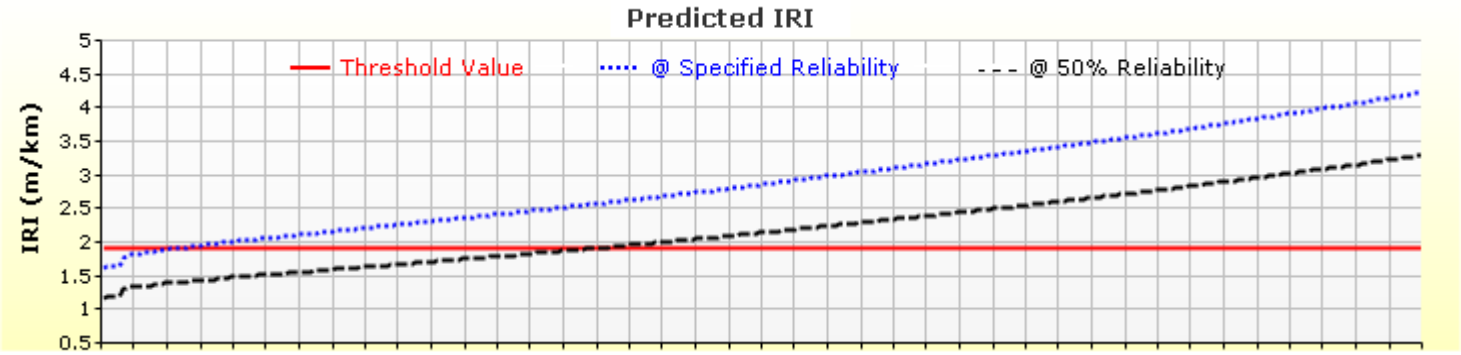
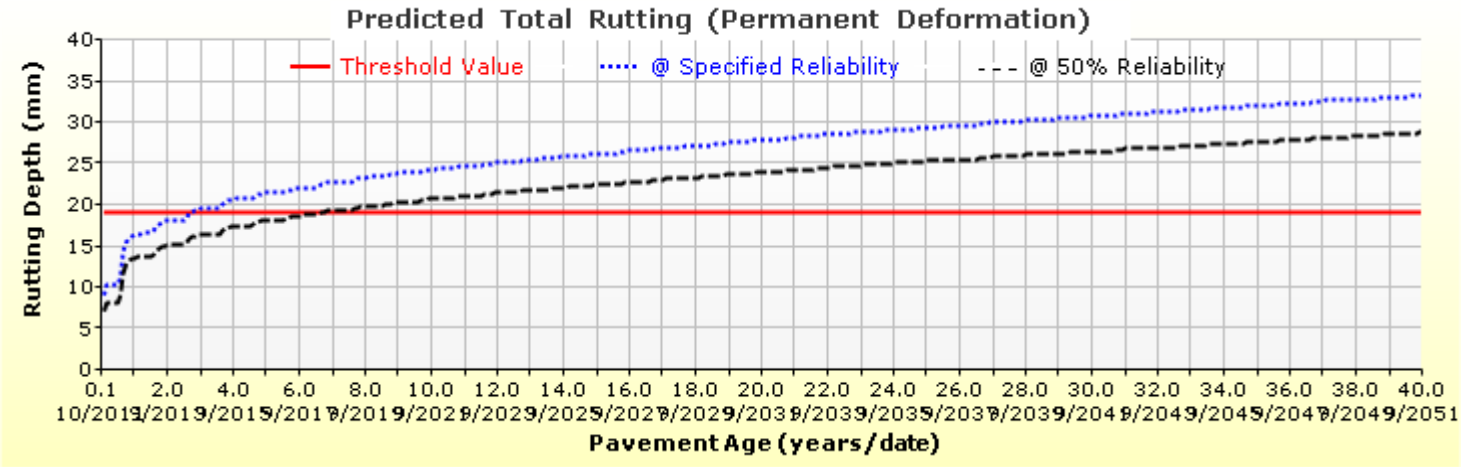
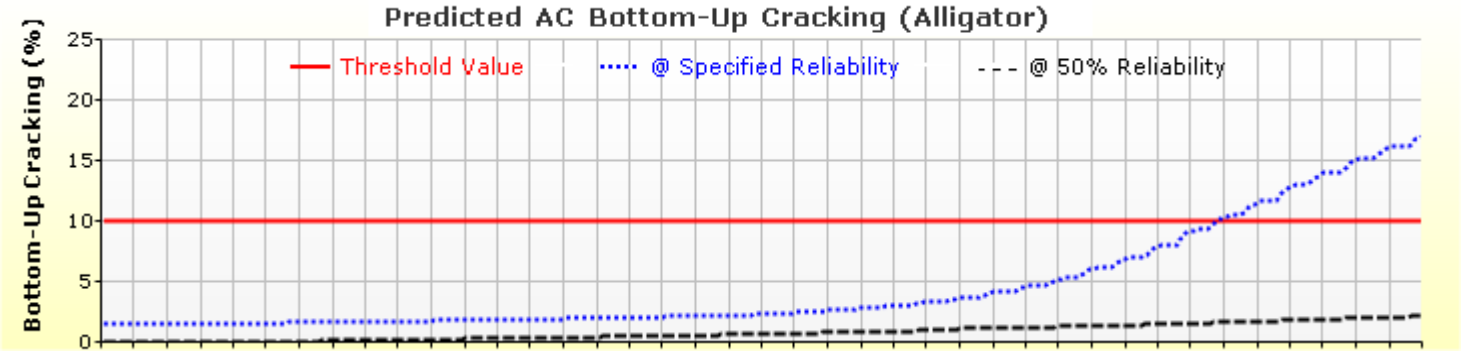
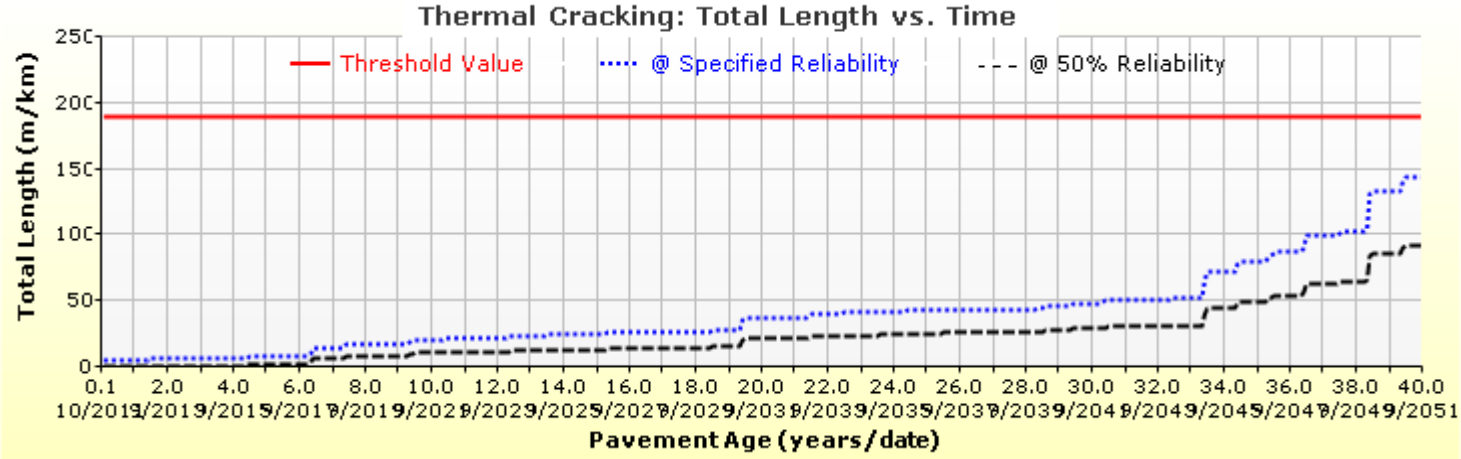
Analysis Output Charts









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HWY 35_Rubber_WeakSG

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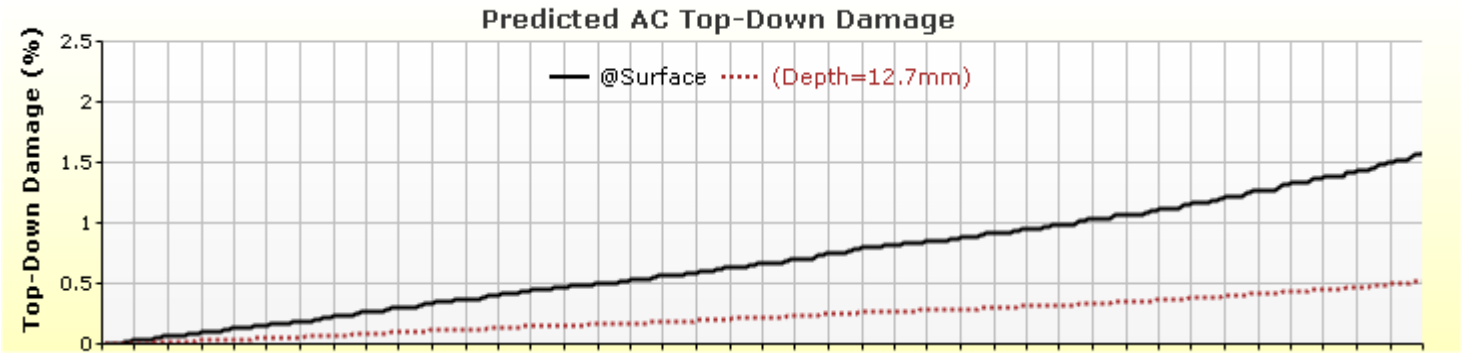

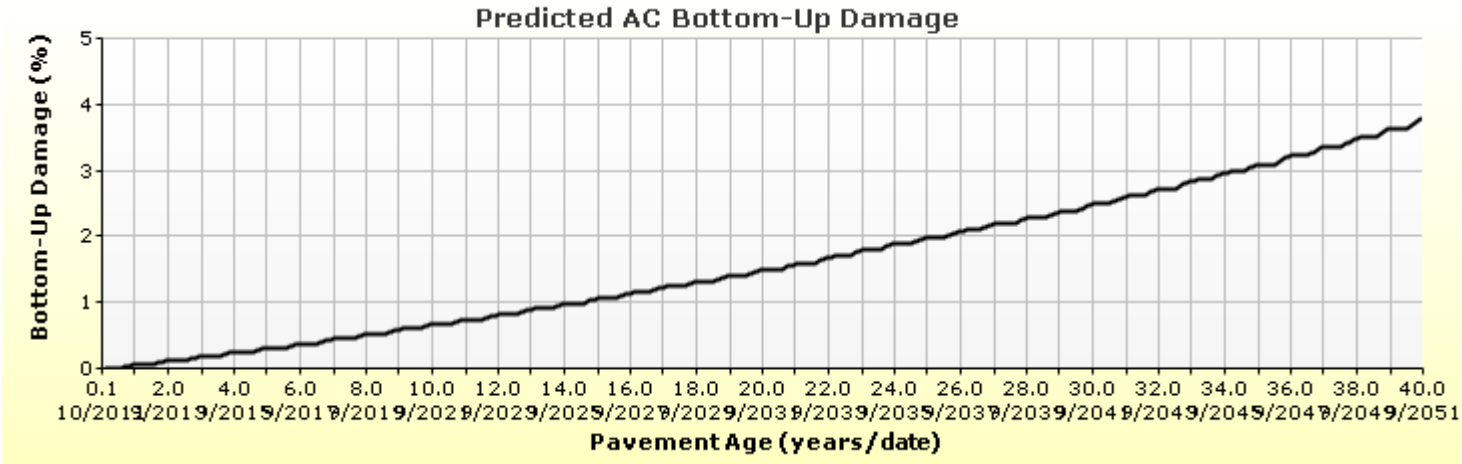

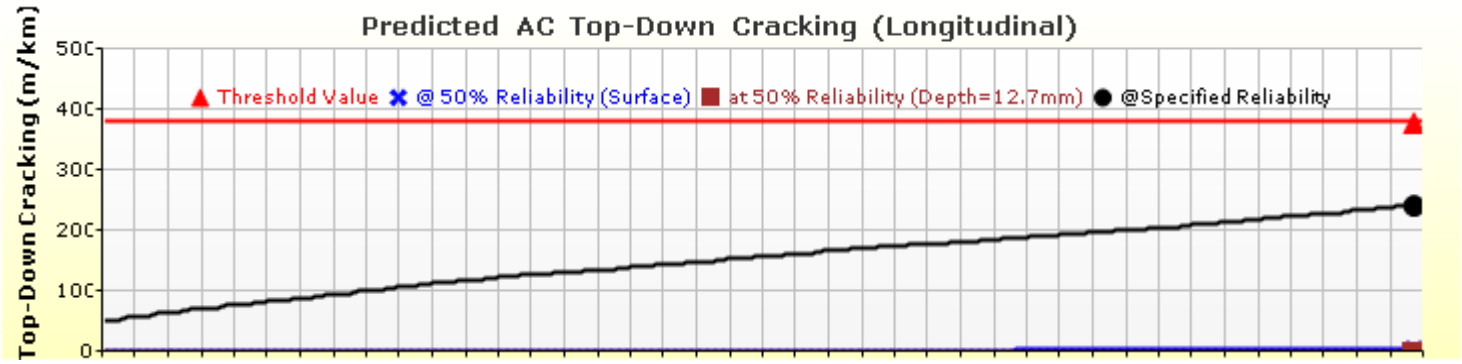

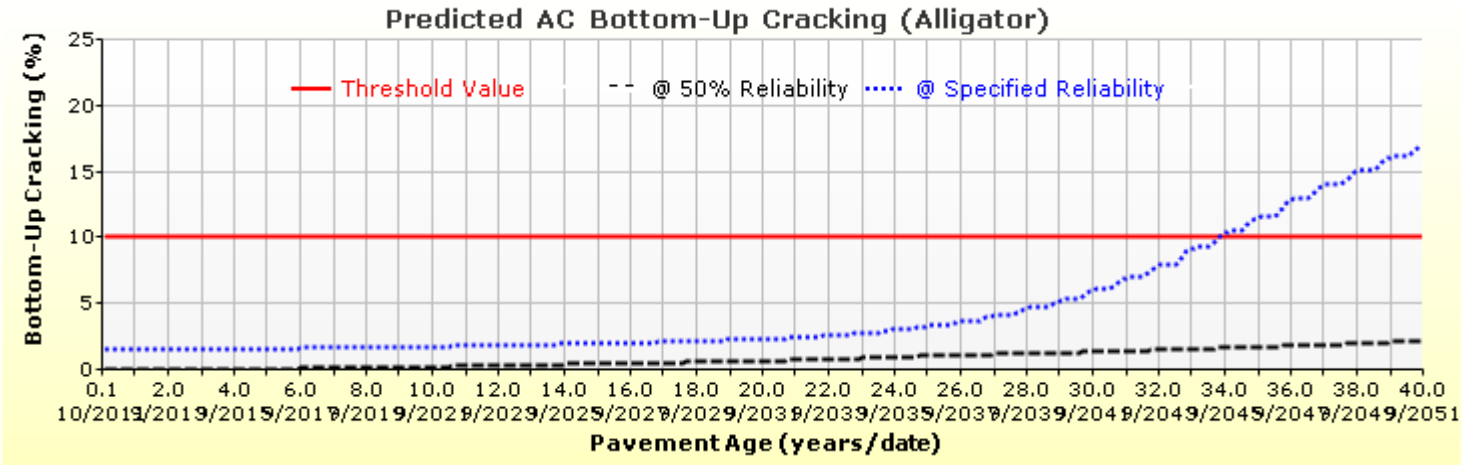

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HWY 35_Rubber_WeakSG

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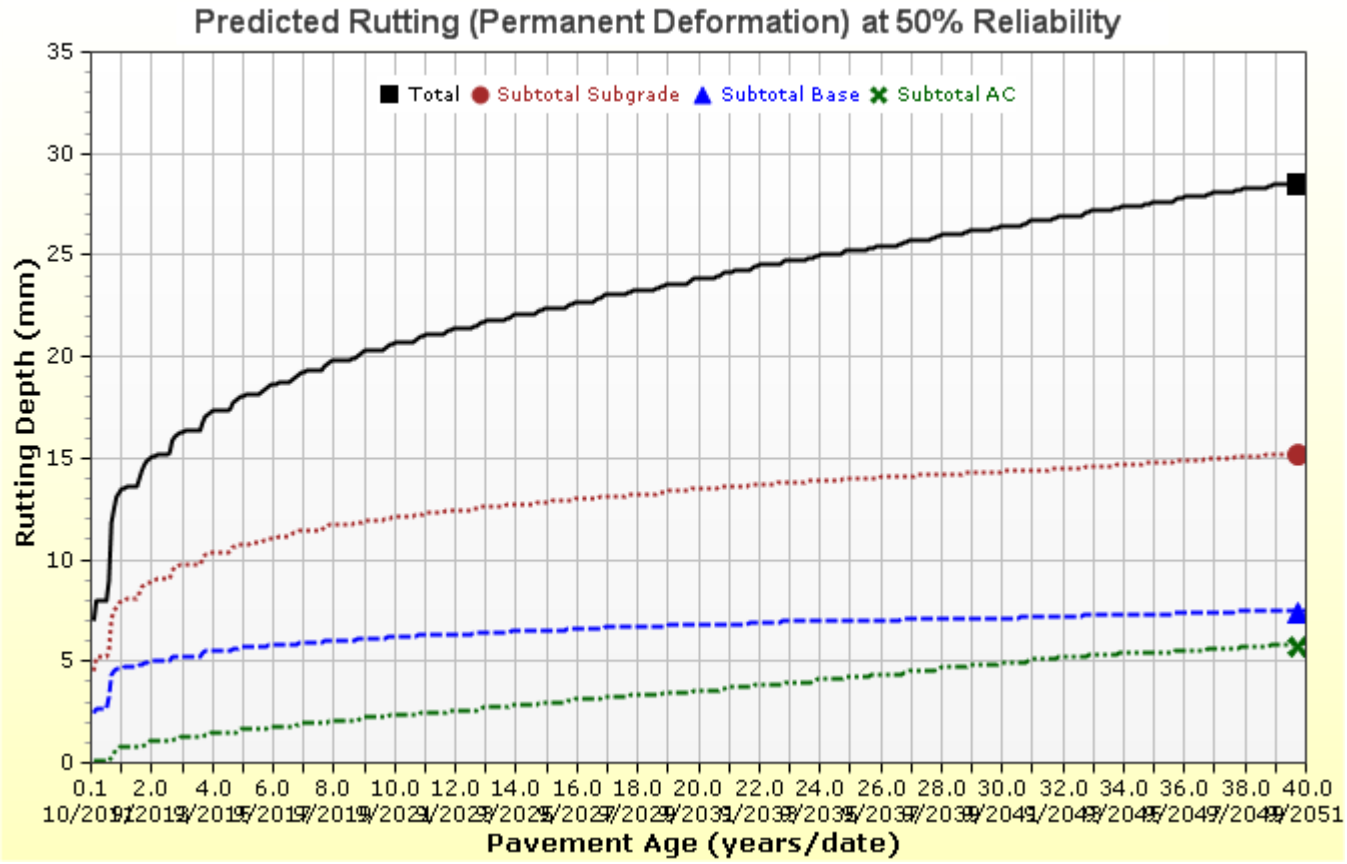

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HWY 35_Rubber_WeakSG

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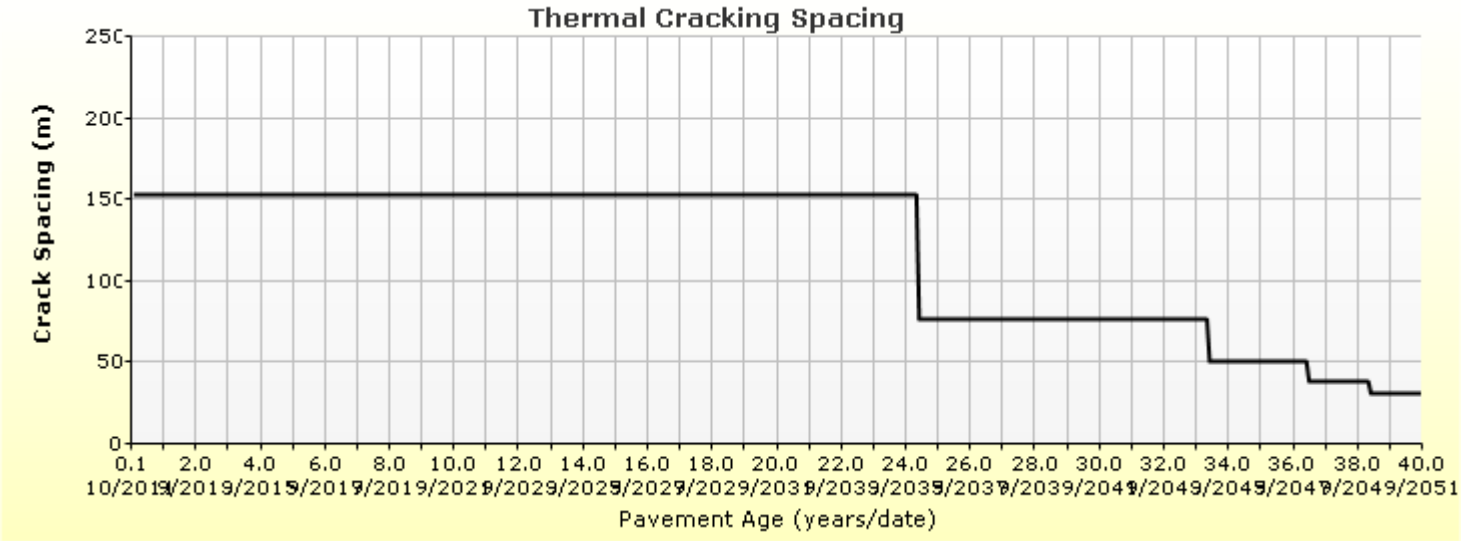

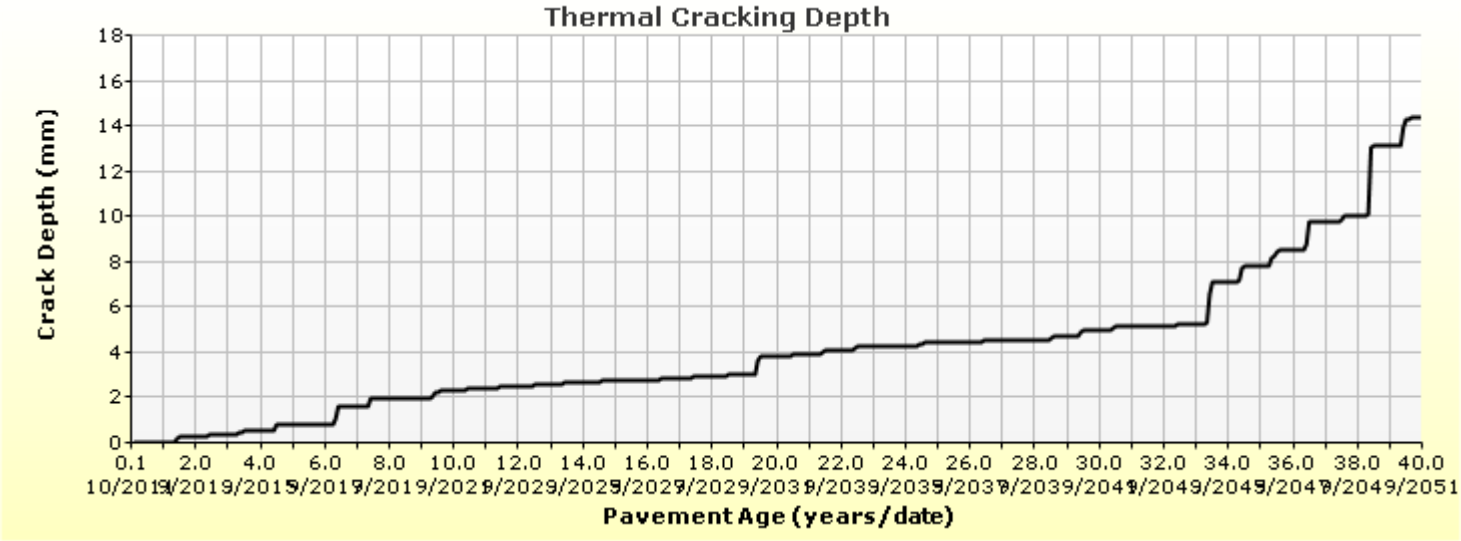

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Appendix C: Highway 115 AASHTOWare Results (Weak Subgrade)

HWY 115_Conv_WeakSG

File Name: C:\Users\admin\Desktop\Mo MASc Thesis\HWY 115_Conv_WeakSG.dgpx

Design Inputs

Design Life:	40 years	Base construction:	August, 2011	Climate Data Sources (Lat/Lon)	43.862, -79.37
Design Type:	Flexible Pavement	Pavement construction:	September, 2011		44.117, -77.533
		Traffic opening:	October, 2011		43.677, -79.631

Design Structure

Layer type	Material Type	Thickness(mm):
Flexible	Default asphalt concrete	50.0
Flexible	Default asphalt concrete	105.0
NonStabilized	A-2-4	310.0
NonStabilized	A-2-7	640.0
Subgrade	A-5	Semi-infinite

Volumetric at Construction:

Effective binder content (%)	12.7
Air voids (%)	4.0

Traffic

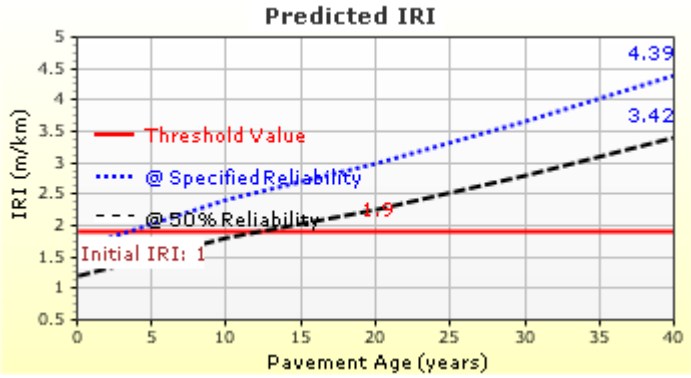
Age (year)	Heavy Trucks (cumulative)
2011 (initial)	1,920
2031 (20 years)	9,158,020
2051 (40 years)	24,822,000

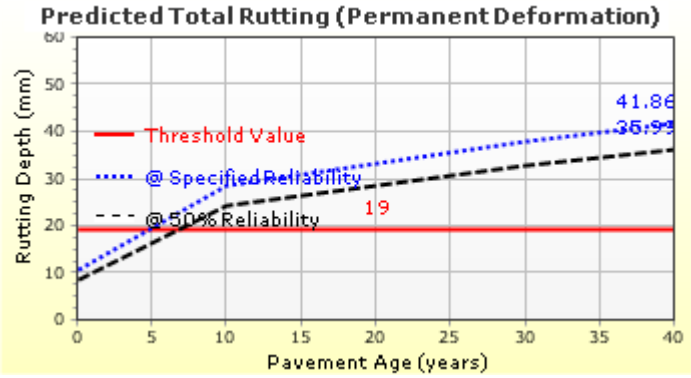
Design Outputs

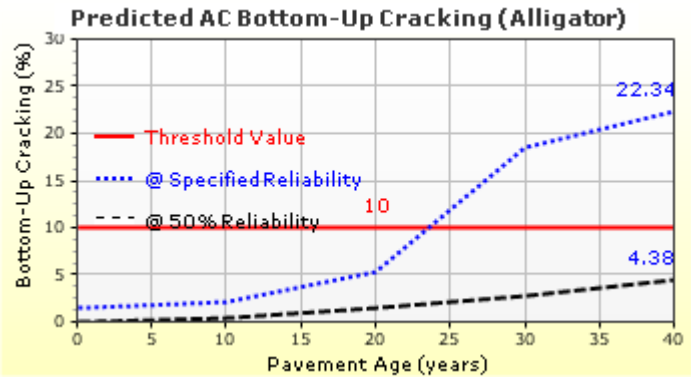
Distress Prediction Summary

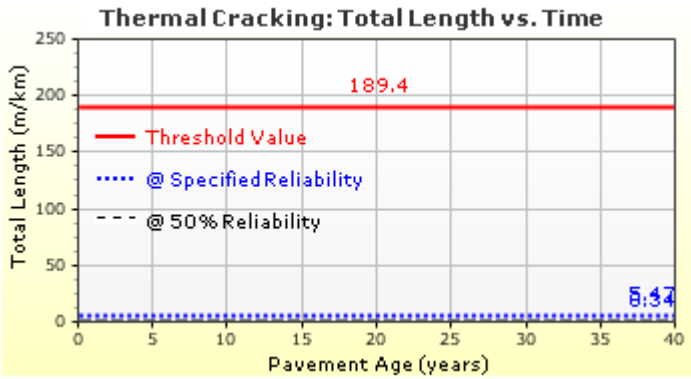
Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (m/km)	1.90	4.39	90.00	2.09	Fail
Permanent deformation - total pavement (mm)	19.00	41.87	90.00	0.01	Fail
AC bottom-up fatigue cracking (percent)	10.00	22.34	90.00	65.58	Fail
AC thermal cracking (m/km)	189.40	5.47	90.00	100.00	Pass
AC top-down fatigue cracking (m/km)	378.80	489.51	90.00	83.23	Fail
Permanent deformation - AC only (mm)	6.00	16.61	90.00	3.39	Fail

Distress Charts









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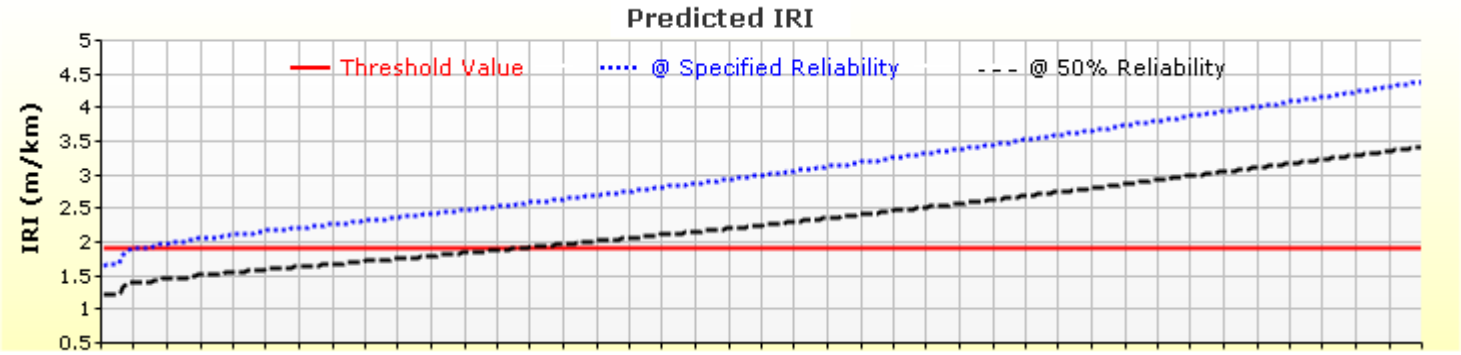
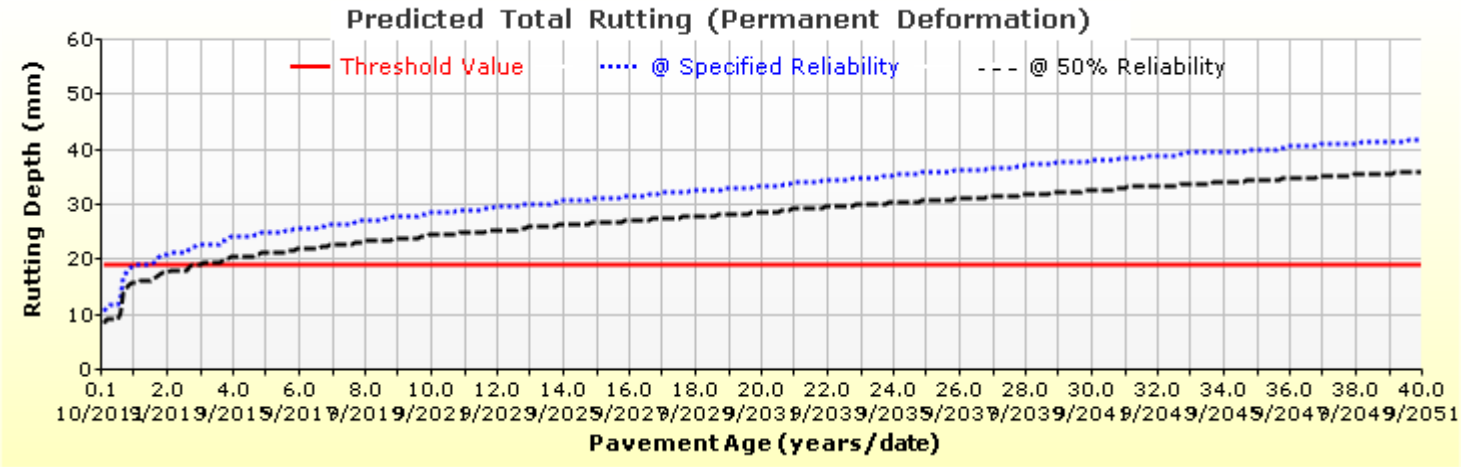
on: 10/11/2013 11:46 AM

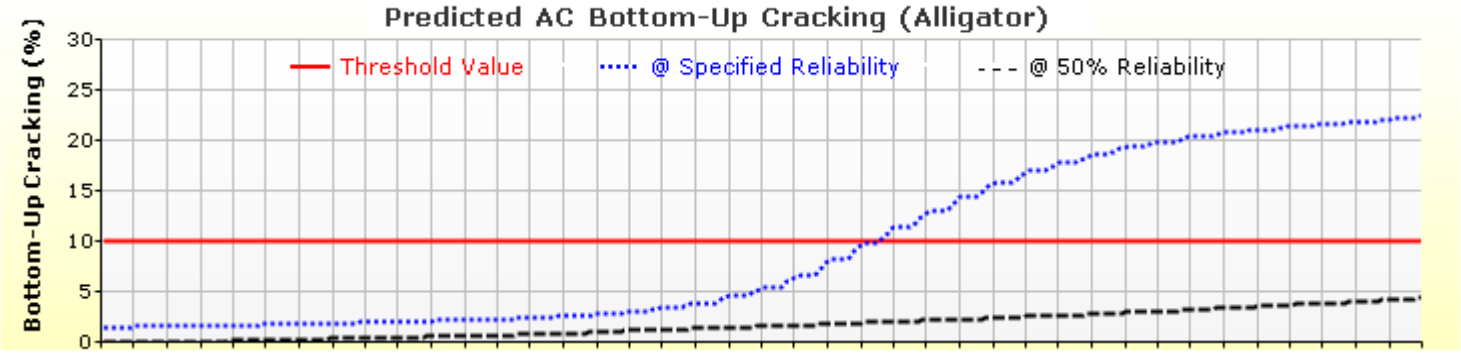
HWY 115_Conv_WeakSG

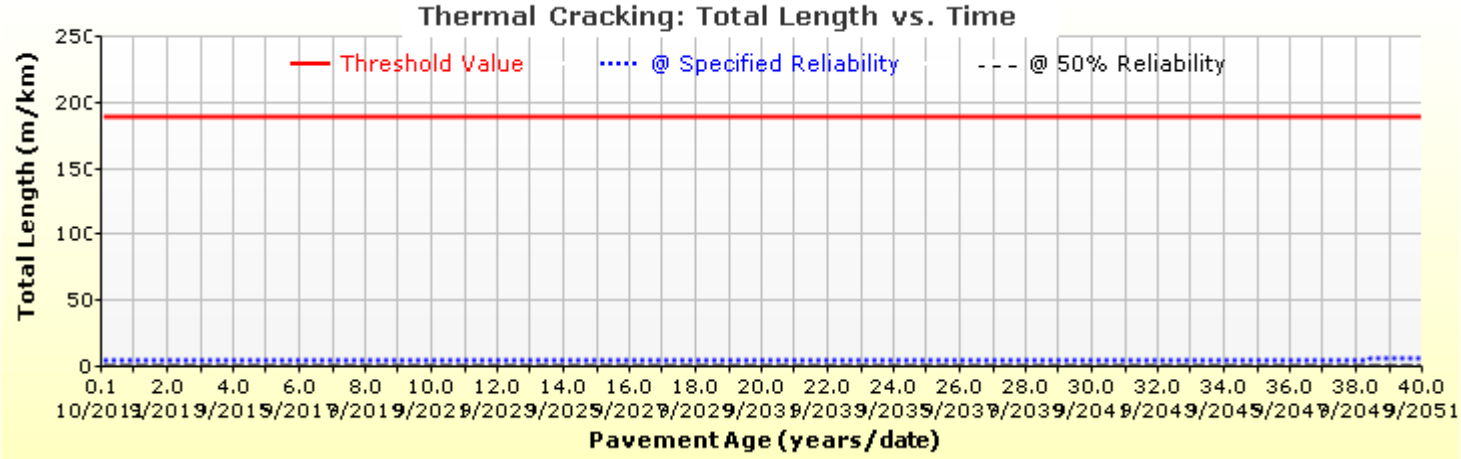
File Name: C:\Users\admin\Desktop\Mo MASc Thesis\HWY 115_Conv_WeakSG.dgpx

Analysis Output Charts









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HWY 115_Conv_WeakSG

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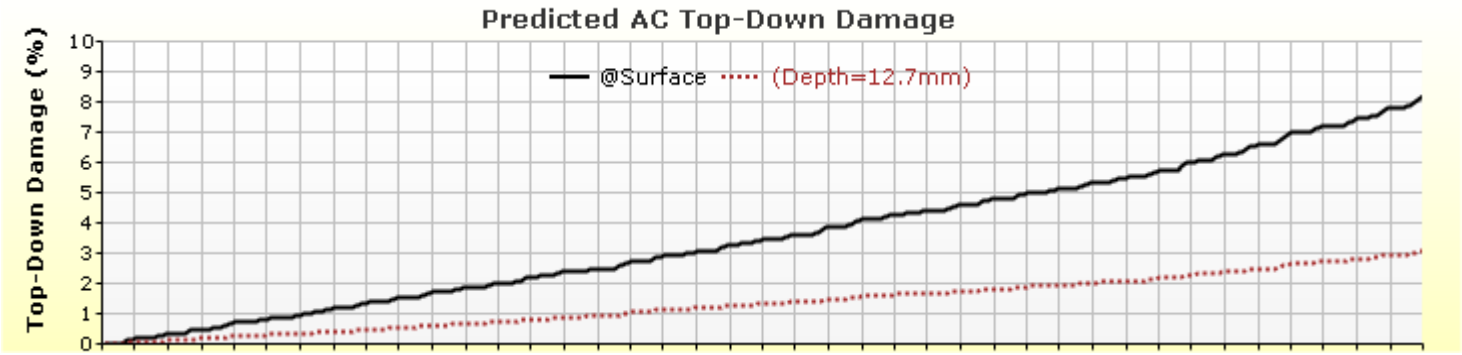

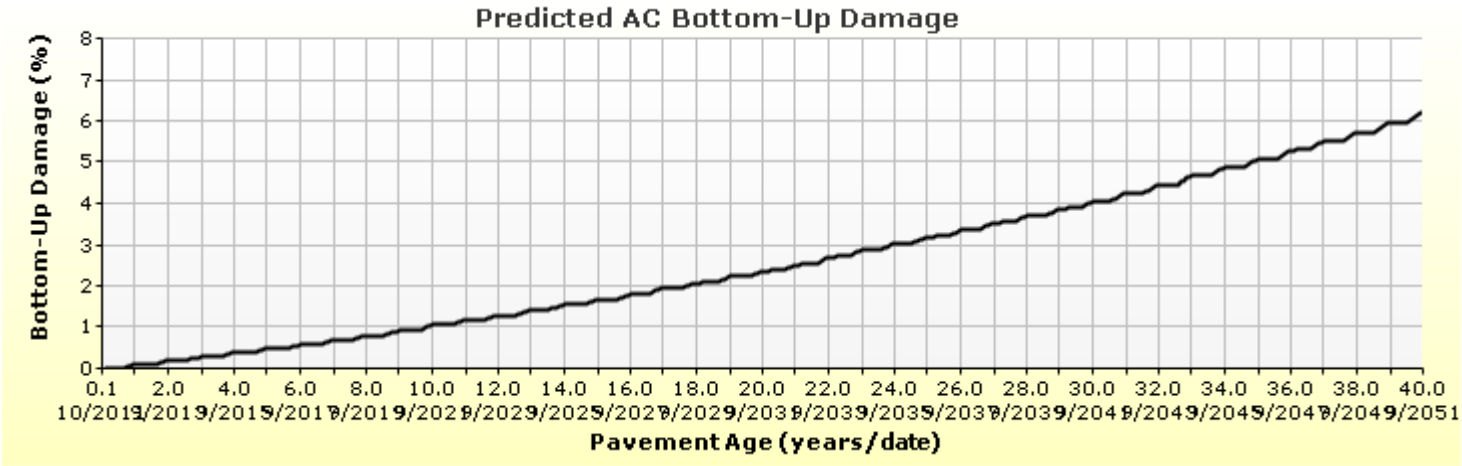

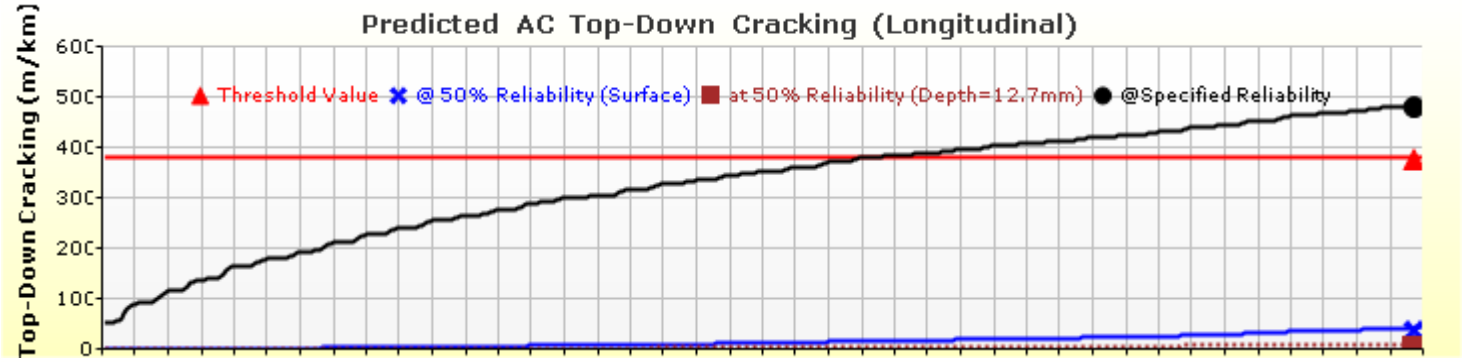

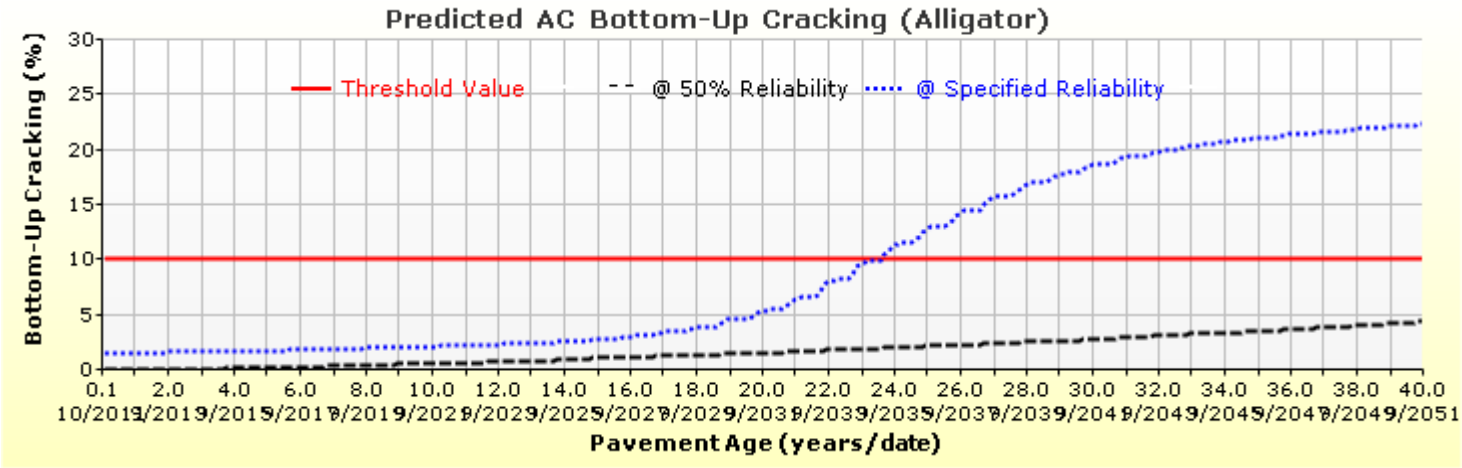

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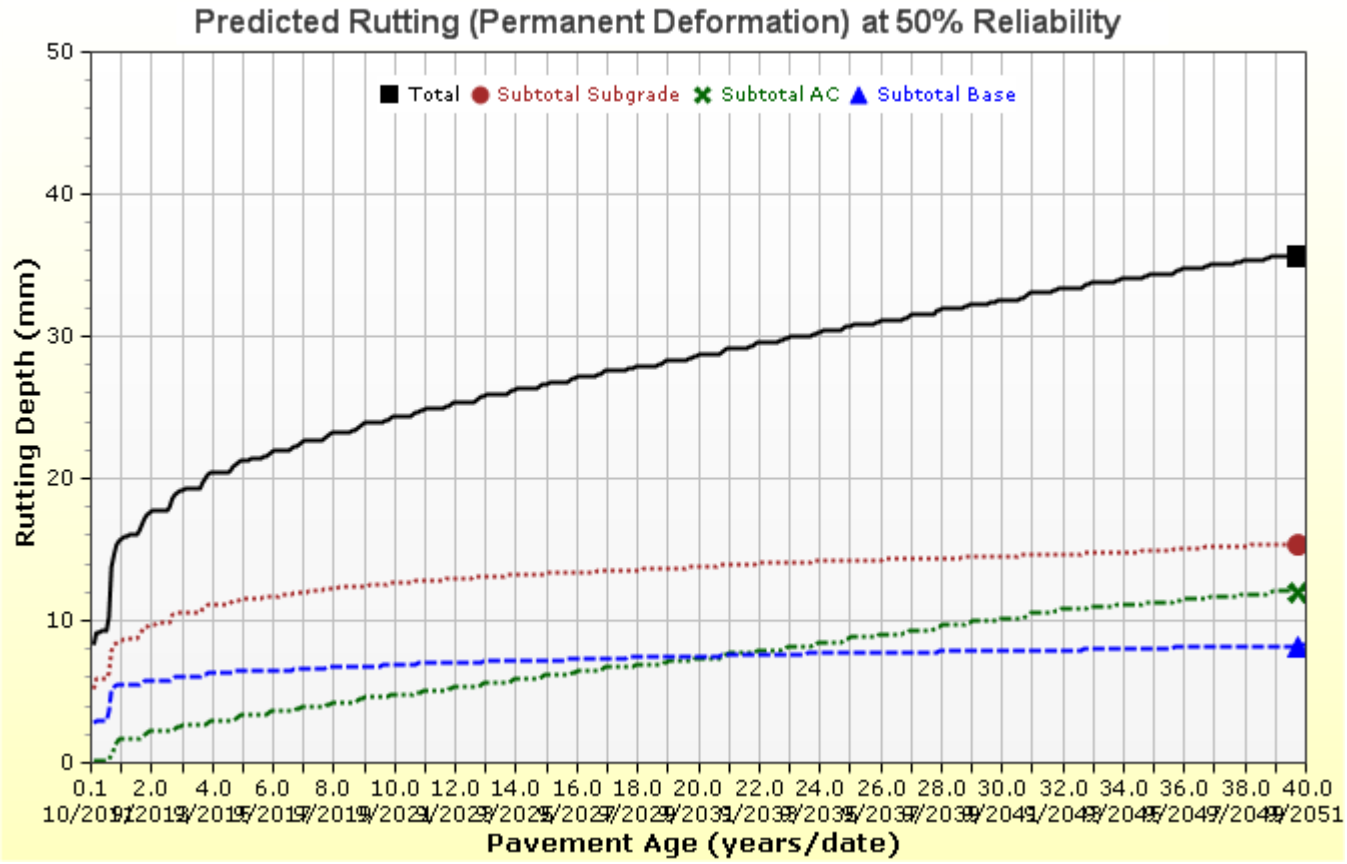

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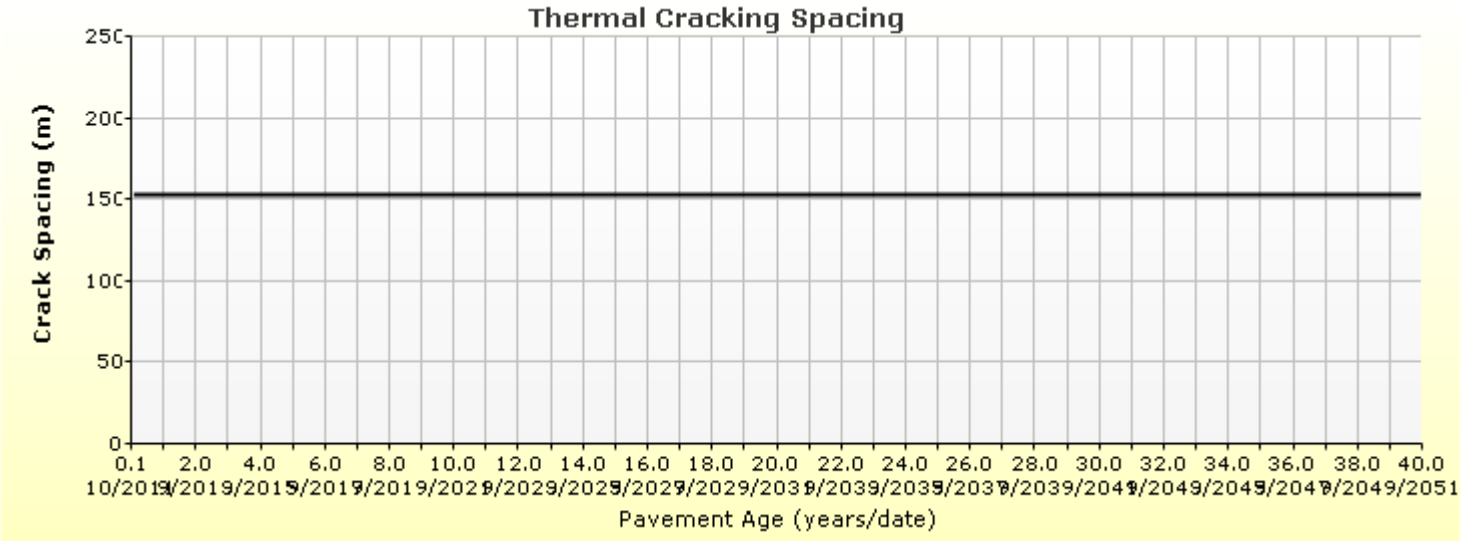

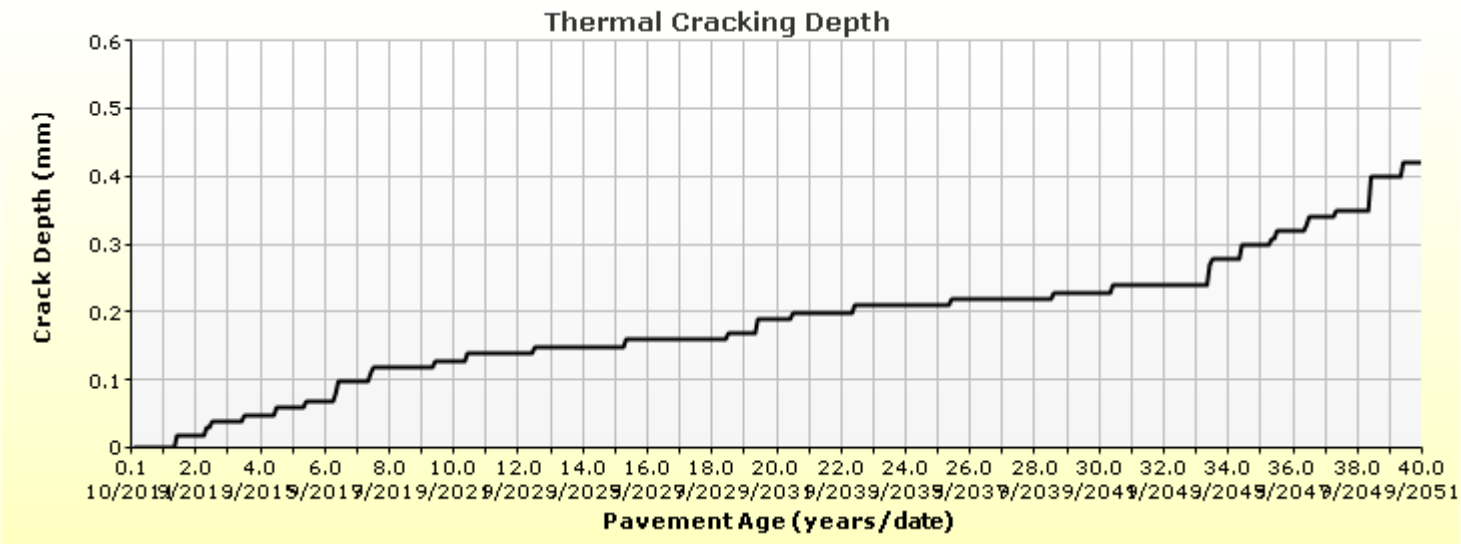

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HWY 115_Rubber_WeakSG

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Design Inputs

Design Life:	40 years	Base construction:	August, 2011	Climate Data Sources (Lat/Lon)	43.862, -79.37
Design Type:	Flexible Pavement	Pavement construction:	September, 2011		44.117, -77.533
		Traffic opening:	October, 2011		43.677, -79.631

Design Structure

Layer type	Material Type	Thickness(mm):
Flexible	Default asphalt concrete	50.0
Flexible	Default asphalt concrete	105.0
NonStabilized	A-2-4	310.0
NonStabilized	A-2-7	640.0
Subgrade	A-5	Semi-infinite

Volumetric at Construction:

Effective binder content (%)	15.6
Air voids (%)	4.0

Traffic

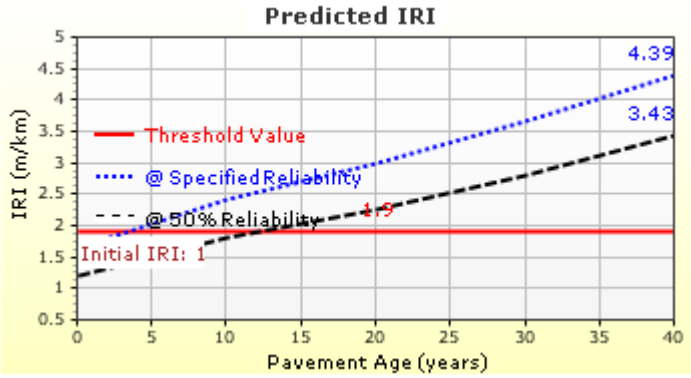
Age (year)	Heavy Trucks (cumulative)
2011 (initial)	1,920
2031 (20 years)	9,158,020
2051 (40 years)	24,822,000

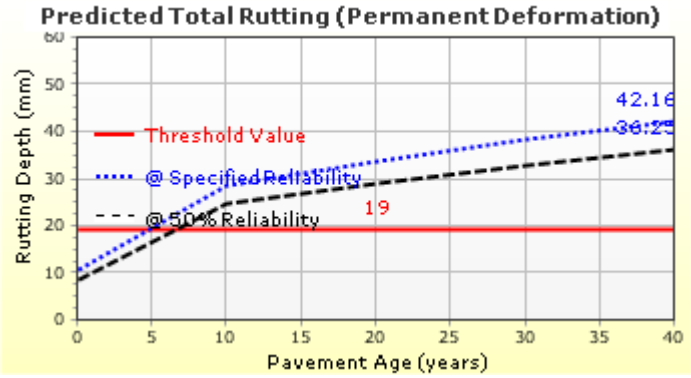
Design Outputs

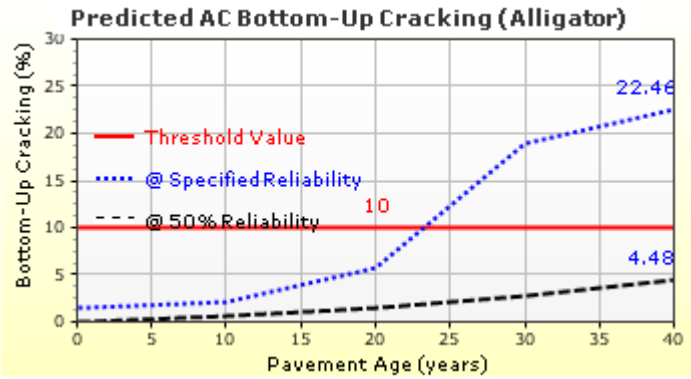
Distress Prediction Summary

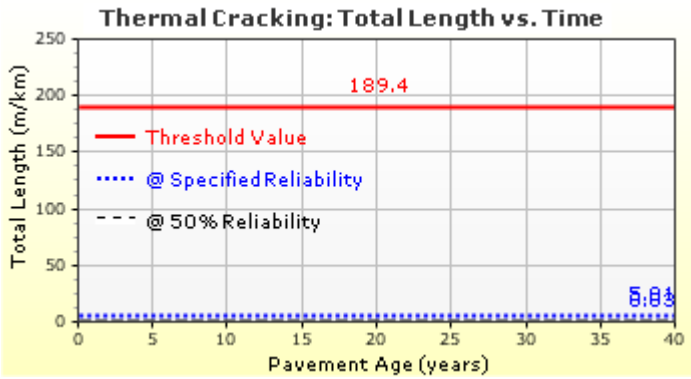
Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (m/km)	1.90	4.39	90.00	2.07	Fail
Permanent deformation - total pavement (mm)	19.00	42.15	90.00	0.01	Fail
AC bottom-up fatigue cracking (percent)	10.00	22.46	90.00	65.30	Fail
AC thermal cracking (m/km)	189.40	5.01	90.00	100.00	Pass
AC top-down fatigue cracking (m/km)	378.80	433.93	90.00	86.61	Fail
Permanent deformation - AC only (mm)	6.00	16.77	90.00	3.23	Fail

Distress Charts









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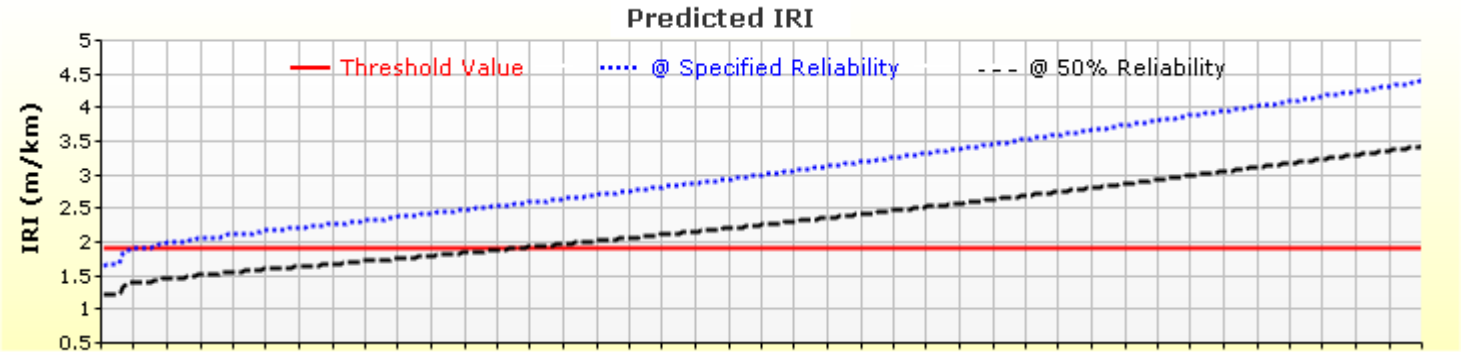
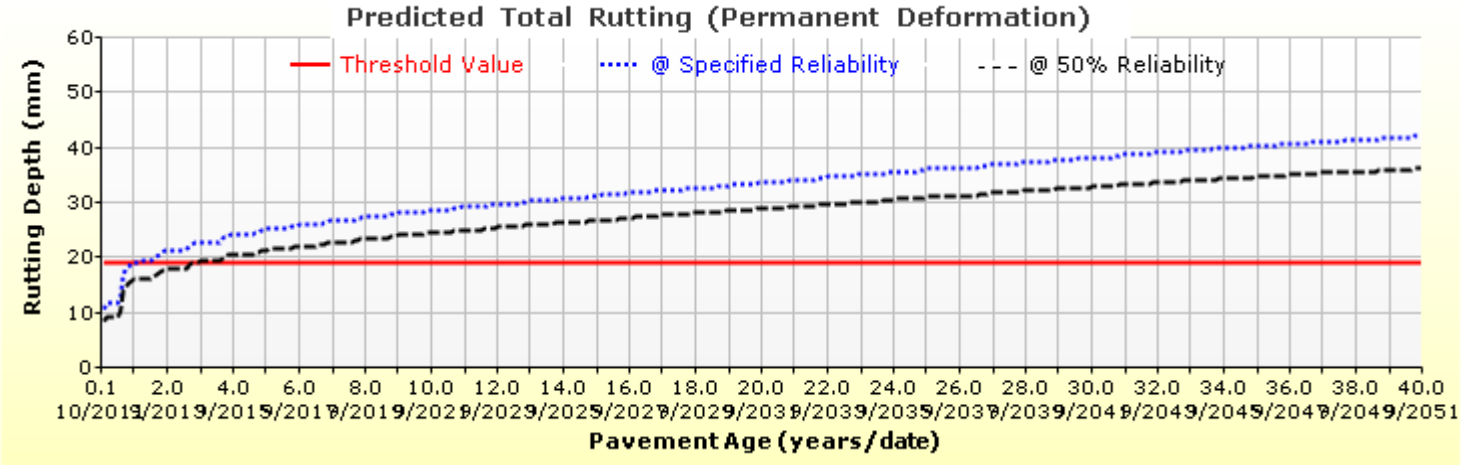
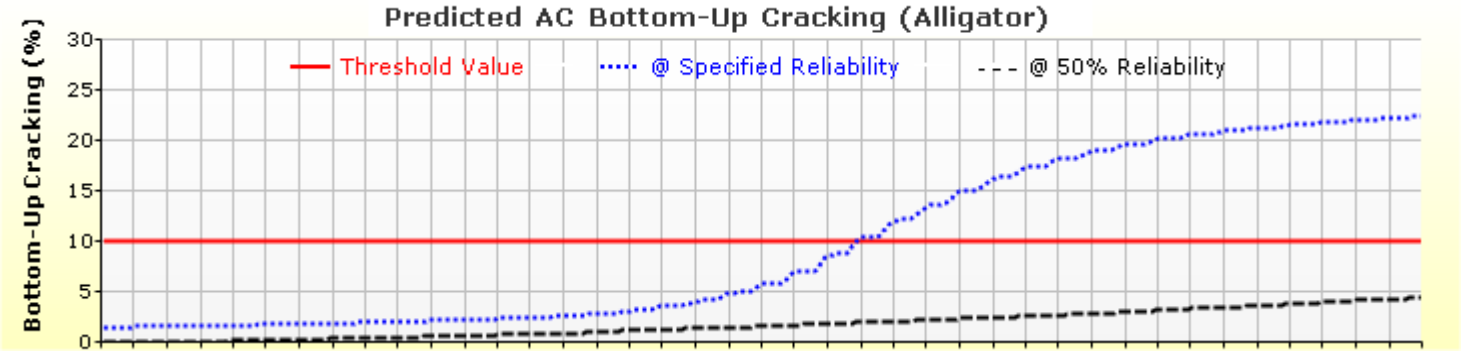
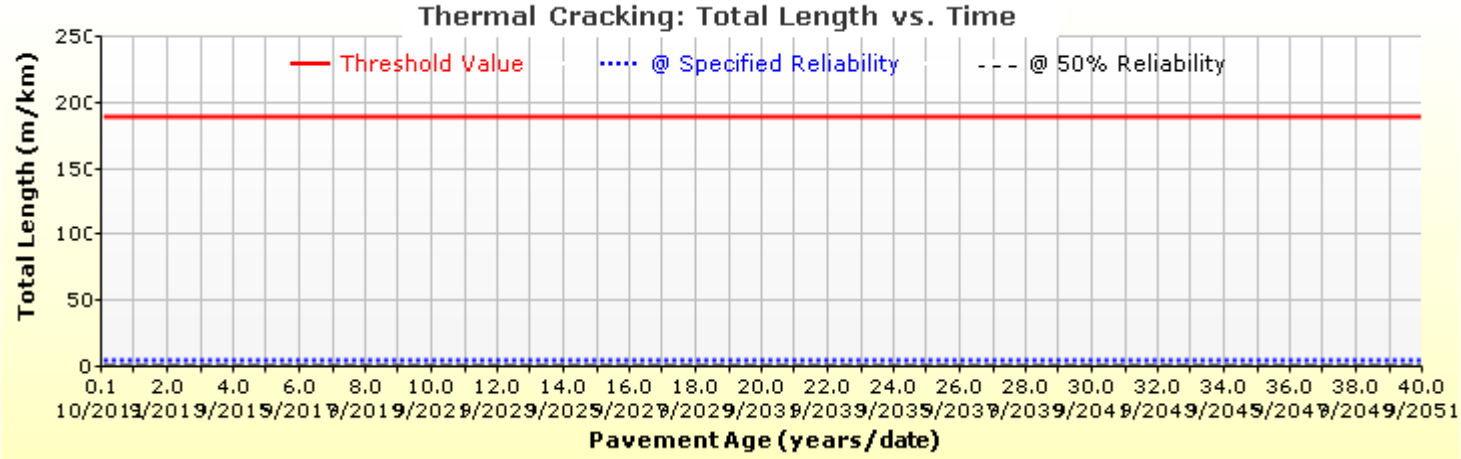
Analysis Output Charts









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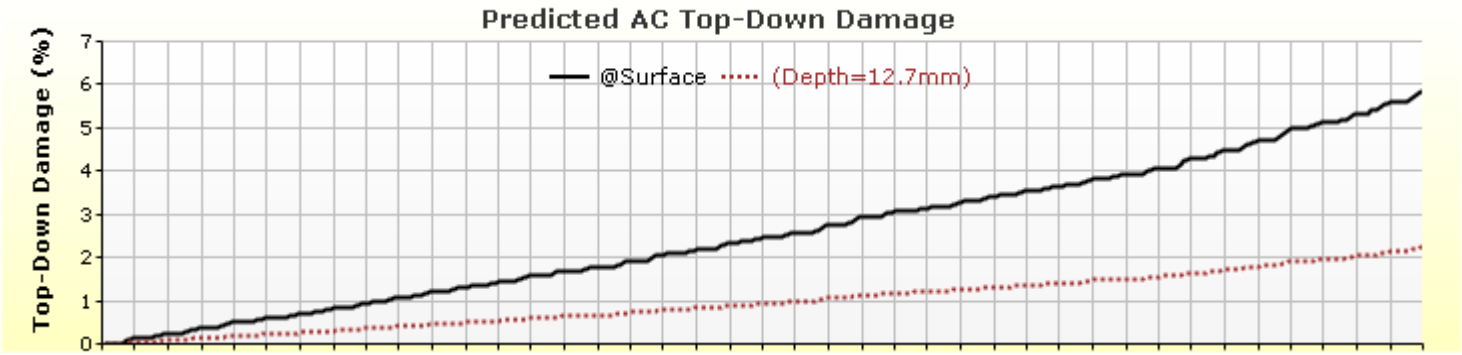

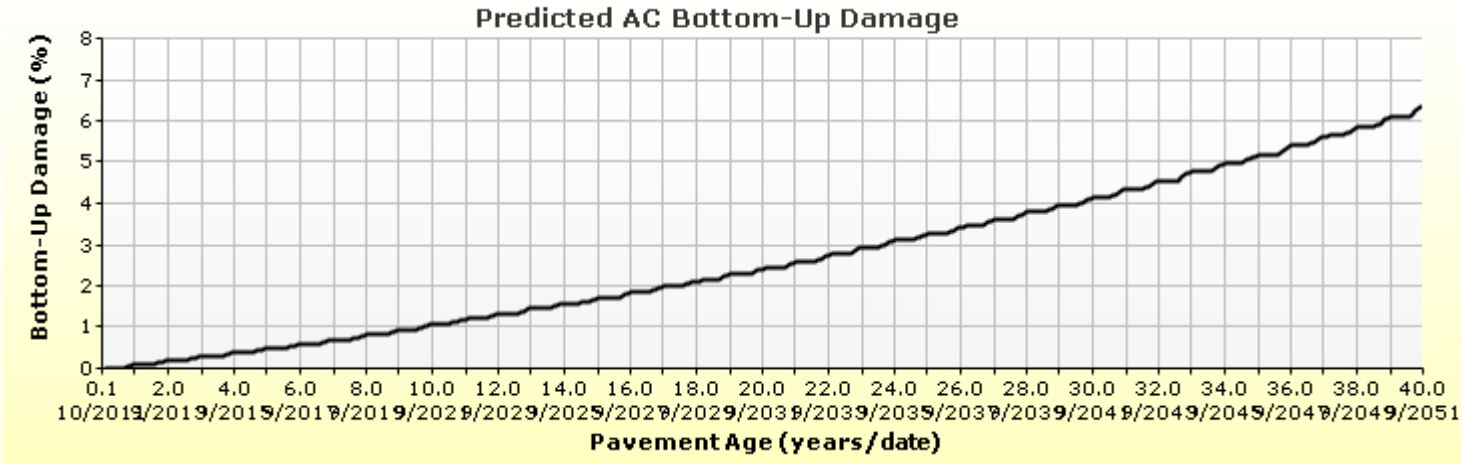

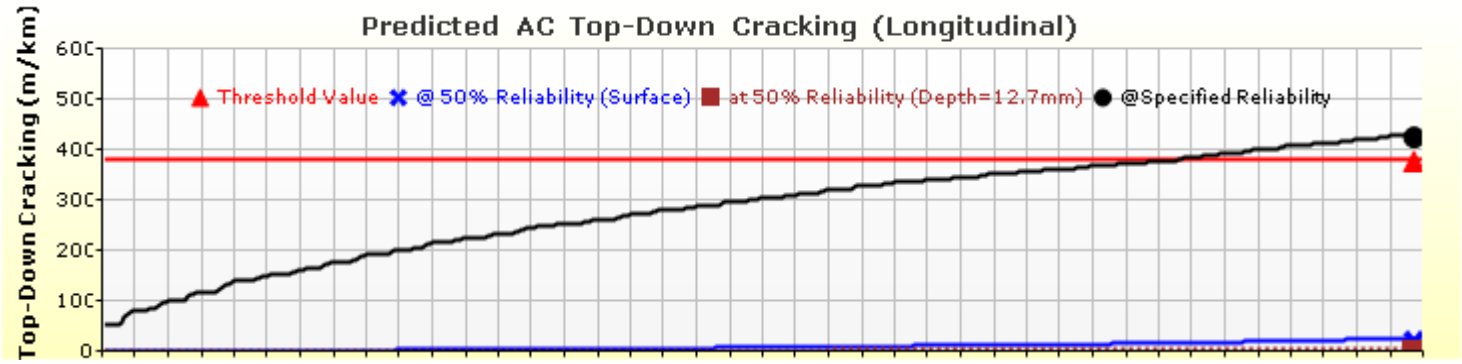

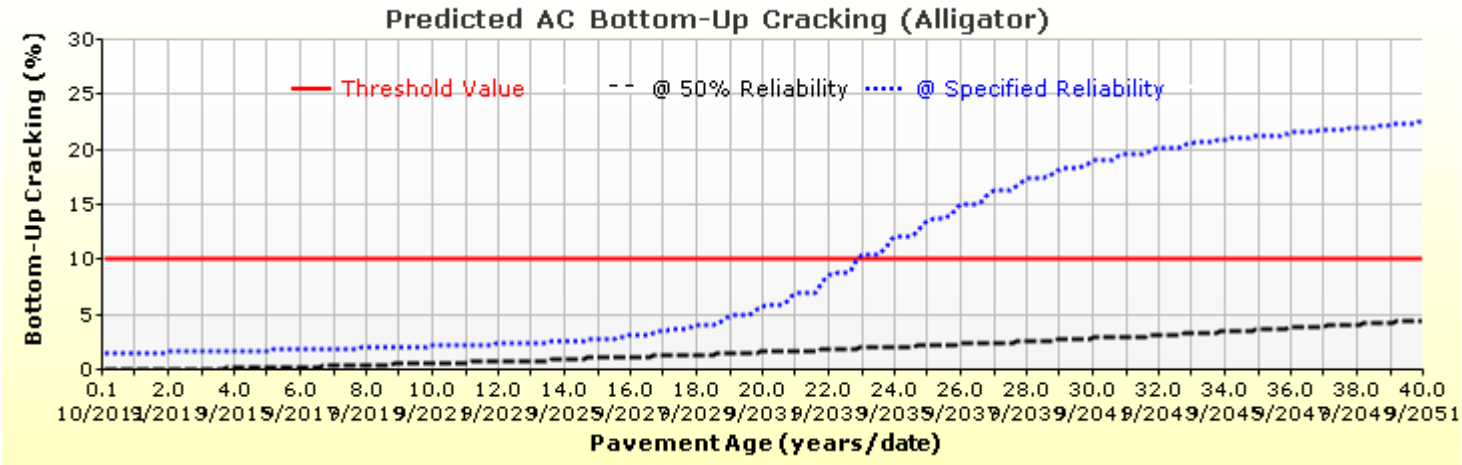

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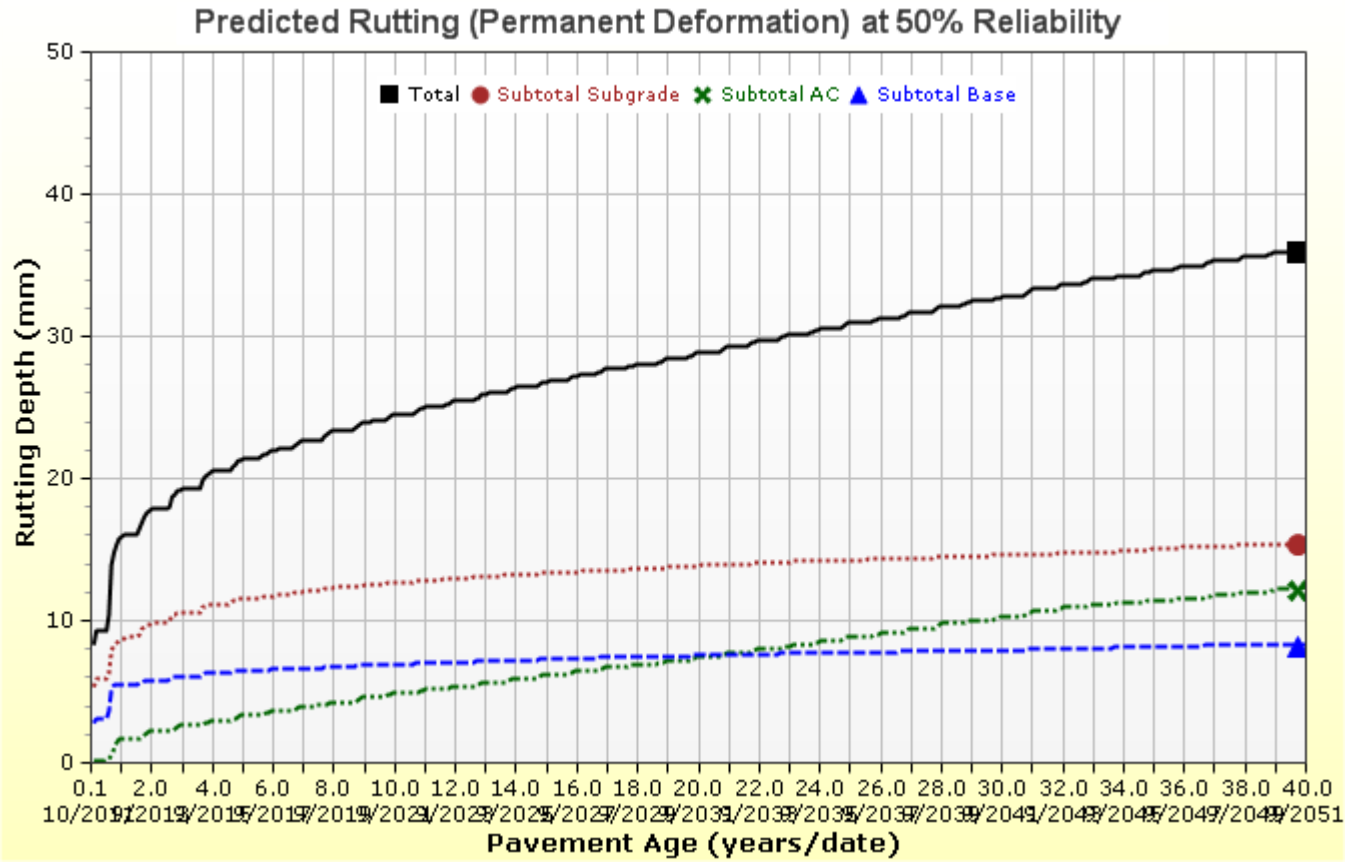

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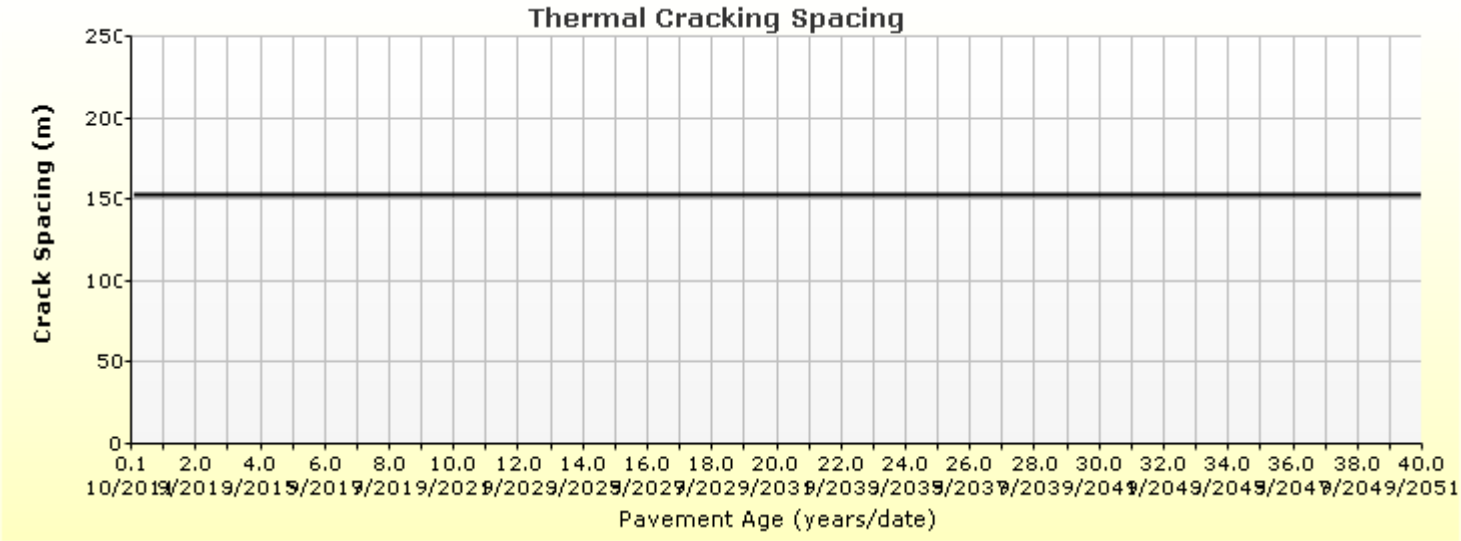

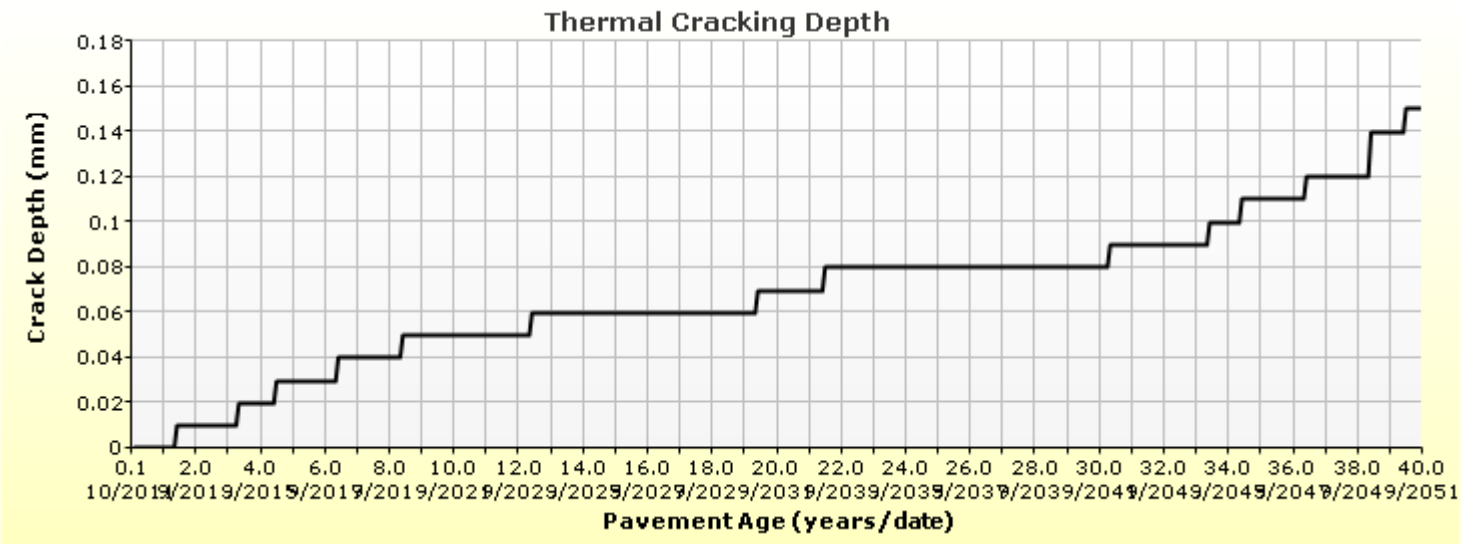

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