A Comprehensive Evaluation of Hot Mix Asphalt versus Chemically Modified Warm Mix Asphalt

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

Warm mix asphalt (WMA) technology has now been successfully used in Ontario for a few years. This shift in usage relates to extensions in construction season, reduced emissions, larger compaction windows, and potential fuel savings. This research between Miller Paving Ltd. and the Centre for Pavement and Transportation Technology attempts to better quantify the difference in hot mix asphalt (HMA) and WMA. The object of this study was three-fold.

The first part of the research was to examine the strength characteristics of HMA and WMA as a function of storage time. The purpose of this evaluation was to quantify indirect tensile strength (ITS) and moisture susceptibility of HMA and WMA over time.

The second objective involved evaluating the performance characteristics of HMA and WMA. Resilient modulus and dynamic modulus testing were completed on plant-produced HMA and WMA material, which was used to determine long-term performance properties of both mixes.

The third and final objective of this study was an economic analysis performed to determine the difference in cost for construction and maintenance for the HMA and WMA pavements. This was completed to determine if the cost of the warm mix technology used in the production of the WMA was offset by fuel savings at the plant.

The findings of the research included:

- HMA and WMA had statistically equivalent air voids over a four-week storage period.
- Dry and wet ITS results for the WMA increased over a four-week storage period while the HMA specimens did not show this same increase.
- WMA material had slightly better workability than the HMA material although the values were statistically equivalent.
- WMA mix had higher resilient modulus values than the HMA mix.

- Dynamic modulus testing showed that at high temperatures, WMA showed to be slightly more susceptible to rutting than the HMA mix, and at lower temperatures, the HMA showed to be slightly more susceptible to fatigue cracking than the WMA mix.

- The MEPDG showed that both the HMA and WMA pavements were deemed to be structurally adequate.

- An economic analysis of the HMA and WMA pavements compared a life cycle cost analysis over a 20-year design life which included all costs associated with construction, maintenance, and rehabilitation of both the HMA and WMA and showed that the HMA was slightly more cost effective than the WMA.

- A field trial was performed by Miller Paving Limited on Highway 62 in Madoc, Ontario showed that the WMA material was more effective at maintaining the temperature of the asphalt mixture during long hauling distances.

- Overall the WMA exhibited the same performance properties as the HMA.

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I would like to extend my gratitude to Jodi Norris, and Shirley Ddamba for their assistance in the CPATT laboratory at the University of Waterloo.

Finally, I would like to thank my family for their support and enthusiasm as I further my academic and professional career.

Dedication

I dedicate this thesis to my husband, Brent, who is my biggest fan; and to my 2-month old son, Jeramus, who is teaching me to be patient.

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Chapter 1
Introduction

1.1 Background

The hot mix asphalt (HMA) industry is constantly exploring new ways and technologies that will help reduce the industry's carbon footprint, improve efficiency, save money and conserve materials without compromising the quality of the product. The use of recycled material such as recycled asphalt pavement (RAP) in HMA is showing to be a sustainable alternative [Haichert 2011] as well as the use of recycled asphalt shingles (RAS) is also being explored [Islam 2011, Rubino 2010]. Another technology that has become a "hot" topic in the industry is the idea of Warm Mix Asphalt (WMA). A common classification method of different asphalt mixture products is by production temperature and is organized as follows [Zaumanis 2010]:

- Cold mix (0 – 30^{o}C)
- Half-warm asphalt (65 – 100^{o}C)
- Warm-mix asphalt (100 – 140^{o}C)
- Hot-mix asphalt (above 140^{o}C)

WMA technology allows HMA to be produced at lower temperatures and still allow the asphalt binder and aggregate to be properly mixed at the plant. More important than mixing is that the WMA material can still be placed and compacted at the job site at the reduced temperatures. Temperatures can be reduced by as much as 30% [AI 2007]. This is feasible because the technologies work by reducing the viscosity of the asphalt binder, thus, increase the workability of the asphalt mix at the reduced temperatures [AI 2007]. Depending on the type of WMA technology used, there are many promised benefits, some of which include [Davidson 2008]:

- Reduced production and laydown temperatures
- Lowering of overall energy costs
- Reduced age hardening of mix (longer service life)
- Extended paving season
- Longer hauling distances

WMA technology has been in use in Europe since the early 1990s and gained interest in North American starting in 2002 [D'Angelo 2008]. A number of WMA technologies are currently used in industry and there have been a vast number of field trials performed to show the benefits that have been listed above. To date, there has not been extensive research into the long-term performance of pavements paved with WMA or research to quantify the economic benefits of the use of the innovative technology.

1.2 Research Objectives

The objective of this thesis is summarized as follows:

- Quantify material properties including air voids, indirect tensile strength (ITS) and moisture susceptibility of HMA and WMA over time as a measure of the strength characteristics of these materials.
- Evaluate the performance characteristics through dynamic and resilient modulus testing of HMA and WMA, which will be used to determine long-term performance
- Determine if the cost of the warm mix technology used in the production of the WMA is offset by the fuel saving at the asphalt plant

1.3 Methodology

The methodology used in this research can be summarized as follows:

- Literature review of current WMA technologies
- Select HMA and WMA to evaluate
- Collect plant produced HMA and WMA from an asphalt plant.
- Determine mixture properties such as air voids and strength characteristics of the two mixes over a period of different storage times.
- Carry out performance tests on both mixes.
- Use performance test results to create performance prediction distress models using mechanistic-empirical pavement design guide.

- Collect thermal image of a field trial and use the data to evaluate thermal characteristics of both HMA and WMA over the duration of the construction period of the project.
- Perform an economic analysis to determine cost effectiveness of WMA from data collected of the field trial.

1.4 Thesis Organization

Chapter 1 presents an introduction to WMA technology and the objectives of this research.

Chapter 2 provides a literature review of WMA in terms of the different technologies that are currently available in the industry and background information for mix design processes used for HMA and how the process is modified for WMA mix designs.

Chapter 3 explains in detail the methodology used in this research and a background of the different types of performance tests that were used.

Chapter 4 summarizes the test results from all testing carried out as part of this research project.

Chapter 5 presents an economic analysis performed using data collected from the laboratory test rests and from the field trial.

Chapter 6 describes a WMA field trial conducted by Miller Paving Limited on a Ministry of Transportation Ontario (MTO) project.

Chapter 7 provides conclusions from this research and recommendations for future research.

Chapter 2
Literature Review

2.1 Introduction

This chapter presents a literature review on HMA and WMA technology. It will investigate the different types of technology currently used in industry to achieve WMA properties. The literature review will also explore current industry practices for producing and testing WMA and the current issues being faced in terms of field performance and also look at mix design methods used for HMA and WMA.

2.2 Hot Mix Asphalt

HMA or asphalt concrete pavement is the bound layers of flexible pavement structure. It is a mixture of asphalt binder and mineral aggregate. The asphalt binder comes in the form of either asphalt cement, or modified asphalt cement and it acts as the glue which holds the aggregate particles together to form a dense, waterproof mixture [AI 2001a]. The mineral aggregate acts as the framework that provides strength and toughness to the system.

A wide variety of asphalt binders and aggregates are used in the production of HMA. However, regardless of the source, processing method, or mineralogy, the goal is to produce a mixture that will provide the strength needed to resist repeated traffic loading [AI 2001a]. Common distress types caused by the traffic loading include [TAC 1997]:

- Low temperature cracking
- Permanent deformation (rutting, shoving, etc)
- Fatigue cracking
- Moisture sensitivity and stripping
- Aging of the asphalt/aggregate system
- Durability

2.3 Warm Mix Asphalt Technologies

WMA technology allows HMA to be produced at lower temperatures and still allow the asphalt binder and aggregate to be properly mixed at the plant. In recent years, the use of WMA in Canada has grown significantly. There are over twenty different WMA additives and processes being used and it is expected this number will continue to increase [Nabhani, 2010]. For the purpose of this research, the various technologies will be categorized as organic additives, foaming processes, and chemical processes. These are described herein.

2.3.1 Organic Additives

There are two types of organic additives – synthetic paraffin waxes, and low-molecular-weight ester compounds. The paraffin waxes consist of long-chained aliphatic hydrocarbons derived from coal gasification, while the ester compounds consist mainly of esters from fat acids and wax alcohols produced by toluene extraction from brown coal [Cevarich 2003]. Organic additives have melting points below normal HMA production and they increase the viscosity of the binder at low temperatures which facilitates in the production of WMA. A common organic additive that is currently used is Sasobit®.

2.3.2 Foaming Processes

Foaming processes rely on water expansion when it turns into steam. In the foaming processes, small amounts of cold water are injected into the hot asphalt binder. This causes the water to evaporate into the binder, creating a controlled foaming effect that increases the binder volume and reduces the viscosity. This effect can improve workability and these effects remain for 7 hours or until the temperature drop below 100^{o}C and the water gradually releases from the binder [D'Angelo 2008]. Some examples of foaming processes currently used in industry are Aspha-Min®, Advera®, Double Barrel Green®, and WAM-Foam®.

2.3.3 Chemical Processes

A variety of chemical packages exist for the production of WMA. They usually include a combination of emulsification agents, polymers, and additives. These chemicals, when added to the asphalt cement, improve coating, mixture workability, and compaction [Zumanis 2010]. They are formulated to operate without changing the rheology of the asphalt cement itself; therefore, properties are not altered of the service temperature. Some chemical additives also have adhesion promoters which improve stripping resistance in the asphalt mixtures.

An example of a chemical additive currently being used is Evotherm®. Evotherm® is a chemical package that consists of emulsification agents and anti-stripping agent additives used in the production of WMA. There are different types of Evotherm® that exists to accommodate different application types, Evotherm Dispersed Asphalt Technology (DAT) and Evotherm 3G. Evotherm DAT is a chemical package that has a small amount of water diluted in it; this helps the additive to be injected into the asphalt line before the mixing chamber [Zumanis 2010]. Evotherm 3G is the same chemical package as the Evotherm DAT except it does not have any water in the additive and it is mixed into the asphalt cement prior to arrival to the asphalt plant. Evotherm 3G was used in the laboratory testing portion of this research, and Evotherm DAT system was used in the field trial portion of this research project. This has been used in Ontario previously and showed good results [Ddamba 2010].

2.4 Warm Mix Asphalt Studies

In general, some of the primary concerns related to WMA are long term performance, moisture susceptibility, rutting, and volumetric properties. These still need to be researched and benchmarked against conventional HMA. Other concerns include the effects of reduced aging of WMA due to the lower mixing temperature on mixture properties. The compatibility of the WMA additives with current aggregates and polymer modified asphalts is also a concern. Some research has been carried out. However, there is still a need to understand WMA technologies, the advantages, disadvantages, and costs/benefits.

Moisture susceptibility is an important issue for WMA because there is concern that at the lower temperatures at which WMA is produced water may be retained in the aggregates which could lead to increased susceptibility to moisture damage. This is especially a concern for the foaming processes where water injection is used as a means of increasing workability at the reduced temperatures.

In a laboratory study which evaluated the moisture susceptibility of two types of WMA technology, Sasobit® and Advera® using a Hamburg Wheel Tracking Device (HWTD), the two WMA mixtures were shown to be more susceptible to moisture damage than the control HMA material. Advera® exhibited moisture damage at a faster rate than Sasobit® [Austerman 2009]. Other research has shown that WMA, regardless of the type of WMA used, is more susceptible to moisture induced damage than conventional HMA [Nabhani 2010, Johnston 2006]. However, there has been research conducted that showed that WMA using a chemical additive performed better in terms of moisture susceptibility than the control HMA [Croteau 2010].

A research project was conducted by the Centre for Pavement and Transportation Technology (CPATT) which involved laboratory and field testing of HMA and WMA placed in Hamilton, Ontario in 2007. Based on resilient and dynamic modulus testing, it was concluded that the HMA and WMA mixes were statistically the same in terms of performance [Ddamba 2010]. Field evaluation of pavement distresses showed that the WMA was in slightly better condition than the HMA; however it could be related to the pavement performance of the underlaying material [Ddamba 2010].

The National Center for Asphalt Technology (NCAT) placed two sections of WMA and one HMA control section on the NCAT test track in Opelika, Alabama (near Auburn University) in fall of 2005. NCAT evaluated performance of the mixes through laboratory testing and monitored field performance. The results of the evaluation (Table 1) showed that only the HMA control mixture could satisfy the Superpave requirement of 80% for moisture resistance [Prowell 2007]. The field performance results (Table 2) indicated that both HMA and WMA sections showed excellent rutting performance.

Table 1: Tensile Strength Ratio Results for NCAT Test Sections [Prowell 2007]

Mix Type	Average Air Voids [%]		Indirect Tensile Strength [psi]		Tensile Strength Ratio
	Unconditioned	Conditioned	Unconditioned	Conditioned	
HMA Control	4.6	4.4	104.1	98.0	0.94
Evotherm® Surface	6.2	6.2	118.0	52.9	0.45
Evotherm® Base	7.6	7.7	98.1	32.4	0.33
Evotherm® Binder	8.0	8.1	106.9	40.6	0.38

Table 2: Field Rut Depths of NCAT Test Sections [Prowell 2007]

Surface Mix	Average Rut Depth [mm]			Standard Deviation
	LWP	RWP	Average	
Evotherm PG 67-22	0.8	1.1	0.9	1.19
Evotherm PG 67-22 +3% Latex	1.1	0.7	0.9	0.52
HMA PG 67-22	1.0	1.1	1.1	0.29

Note: LWP is Left Wheel Path
RWP is Right Wheel Path

In a field trial that was conducted by Alberta Transportation, the objective was to compare the differences between WMA and HMA with respect to production and placement by using a chemical additive WMA system. Quality control samples were obtained and tested for both HMA and WMA which showed that the volumetric properties of the two mixtures were very similar. The study also included performance testing in the form of rutting resistance, flow number, and asphalt cement testing. Both rut testing and flow number results indicated that WMA was slightly more susceptible to rutting [Croteau 2010].

Differences in field and laboratory results have resulted in challenges. However, the University of Massachusetts Dartmouth surveyed materials and construction engineers in Department of Transportation (DOT) agencies in the United States and revealed that 100% of the fifty respondents stated that although the WMA projects were relatively new, moisture damage related distresses have not been observed in the field for WMA [Mogawer 2011]. This research further explored the use of anti-strip agents coupled with longer aging period and concluded that this caused some WMA technologies to perform better in terms of moisture susceptibility [Mogawer 2011].

It is evident that there is a lot more work that needs to be done to understand the properties and performance of WMA and also to make sense of the correlation between laboratory test procedures and field activities before asphalt producers can be confident that WMA will become a substitute for HMA.

Another important issue related to WMA is how the WMA additives and processes affect the asphalt binder properties. Testing has shown that there is less binder hardening with the WMA at reduced production temperatures. This has been attributed to the fact that lower mixing temperatures for the WMA reduces the amount of hydrocarbon vapors, thus, reducing the elimination of the light end portion of the binder [Croteau 2010]. Research has shown however, that the addition of WMA additives at the recommended manufacturer dosages does not change the Performance Graded Asphalt Cement (PGAC) properties [Manolis 2009].

A study done at the University of Massachusetts evaluated the effect of adding Sasobit® at different dosage rates to a base asphalt binder. The results showed that the addition of Sasobit® had a significant impact on the asphalt binder properties depending on the amount of the additive incorporated into the binder [Austerman 2009]. There is other research that supports this finding [Hughes 2009, Nabhani 2010, and Ho 2008].

Based on the available research, it appears that the addition of certain WMA additives can significantly impact the final PGAC and therefore, testing should be done at the specified dosage rate to ensure that the desired final PGAC is met before carrying out production of WMA.

2.5 Economic and Environmental Studies

In the past few years the construction industry has seen an increase in construction prices and this is largely due to the increase in energy costs and the price of asphalt cement. Consequently, if WMA may provide energy savings for the asphalt industry, then it will be very attractive.

In 2008, Lafarge Canada examined the benefits, risks, investment and material costs, and sustainability associated with a WMA trial done in Vancouver, British Columbia. The WMA technology used in the trial was the Astec, Inc. Double Barrel® Green process. The study evaluated the economics, plant emissions, and mixture performance associated with WMA produced with the Double Barrel® Green process. The study concluded that there was a 10% reduction in carbon monoxide, carbon dioxide and nitrogen oxides in the production of WMA using the Double Barrel® Green process versus conventional HMA production and a 24% reduction in energy consumption as well [Middleton 2008].

Another paper involved a life-cycle inventory (LCI) to quantify the energy, material inputs, and emission during the aggregate extraction, asphalt binder production, and hot mix asphalt production and placement for both HMA and WMA mixes. The study concluded that WMA provided a 24% reduction on the air pollution impact of HMA, 18% reduction on fossil fuel consumption, and 10% reduction in smog formation [Hassan 2009]. Although the study showed that the use of WMA was a positive improvement, it does appear to be similar to HMA in the aggregate production, asphalt binder, transportation and construction processes [Hassan 2009].

Another study was conducted on asphalt mixtures that were designed and produced under nearly identical conditions with very similar volumetric properties. The study, summarized in Table 3, concluded that 11.2% less energy was used during the production of WMA as a result of a mixing temperature reduction of 19^{o}C when compared to the HMA [Croteau 2010]. During placements, the energy required to achieve compaction was similar for both mixtures, however HMA was placed at 125°C and WMA was placed at 111°C [Croteau 2010].

Table 3: Calculated and Field Measurements of Energy Requirements for HMA and WMA Production [Croteau 2010]

Mix Type	Energy	Calculated Fuel	Savings	Field Fuel Results	Savings
HMA	271 MJ/t of Mix	7.01 L/t of mix	-	7.10 L/t of mix	-
WMA	246 MJ/t of mix	6.35 L/t of mix	9.4%	6.30 L/t of mix	11.2%

The economic and environmental information available for HMA and WMA production thus far is limited to the reduced energy consumption or reduced emissions through the use of WMA versus HMA. This, however, does not provide a complete evaluation of the WMA technology and factors such as maintenance and rehabilitation activities are omitted in the analysis.

2.6 Mix Design Background

There are three main mix design methods: Hveem, Marshall, and Superpave methods. The Hveem and Marshall Design methods are very similar in the aggregate and asphalt binder selection process with slight differences in their strength testing methodology. The Hveem mix design method was developed by Francis Hveem in the late 1920s in California and the Marshall method was developed by Bruce Marshall in 1939 in Mississippi [AI 2001a]. Currently, in Canada, the Marshall and Superpave methods are used for hot mix asphalt mix designs.

Although aggregate and asphalt binder evaluation is a very critical part of any asphalt mix design, both the Hveem and Marshall mix design methods do not contain a procedure in their methodology for the selection of aggregates and asphalt binder.

In the Hveem method, several trials of aggregate-asphalt blends are generated and samples are compacted with the California Kneading Compactor. The optimum asphalt cement content is determined by the combined results from density and air void analysis and results from the stabilometer test [PI 2007b]. The stabilometer test applies an increasing load to the top of the compacted specimen at a predetermined rate (Figure 1).

Figure 1: Stabilometer Test [PI 2007b]

In the Marshal method, trials are generated in the same way as the Hveem method. However, samples are compacted using a Marshall hammer. The amount of blows applied by the Marshall hammer to the samples is dependent of the traffic category to which the pavement is being design for. The optimum asphalt binder content is determined by a combination of density and air void analysis and results from the Marshall Stability and Flow test (Figure 2). The Marshall Stability test measures the maximum load which the compacted specimen can support under a predetermined loading rate. During the loading, the specimen's plastic flow as a result of the load is also measured [Hoffman 2008b].

Figure 2: Marshall Stability and Flow test [Hoffman 2008b]

The Superpave mix design method was developed to replace the Hveem and Marshall design methods. However, the volumetric analysis used in both Hveem and Marshall methods provided the basis for the development of the Superpave Method.

2.6.1 Superpave Design Theory

Superpave mix design consists of selection of aggregates that meet specified physical property requirements, selection of asphalt binder grade, and finally, determination the optimum asphalt binder content that meets all the desired volumetric properties.

2.6.2 Aggregates

Aggregate properties are a very important part of asphalt mixture performance, and one of the main differences between the Marshall and Superpave mix design methods is that the Marshall method does not specify aggregate properties in the design. There are two types of properties that are determined for aggregates used in Superpave mix designs, consensus properties, and source properties. Consensus properties are those that researchers believe to be critical in achieving high performance HMA and source properties are those that are used to qualify local sources of aggregates [AI 2001a]. Consensus properties include:

- Coarse Aggregate Angularity
- Fine Aggregate Angularity
- Flat and Elongated Particles
- Clay Content

Specification for coarse and fine aggregate angularity aim to achieve a high level of internal friction while limiting the amount of flat and elongated particles will limit the degree of aggregate breakdown during transporting and handling of the aggregate. Limiting the amount of clay ensures a good adhesive bond between the asphalt binder and the aggregates [AI 2001a]. Source properties are:

- Toughness
- Soundness
- Deleterious materials

Toughness is determined through a breakdown test generally measured by the Los Angeles (LA) abrasion test. The test subjects a coarse aggregate sample that is retained on the No. 12 sieve to abrasion, impact, and grinding in a rotating steel drum (Figure 3) that contains steel spheres and then determine the weight of material still retained on the No. 12 sieve afterwards [AASHTO 2002].

Figure 3: LA abrasion testing equipment

An alternative test to the LA abrasion test is the Micro-Deval test, which is used in Ontario. Micro-Deval also uses a rotating drum (Figure 4) with steel spheres but much smaller than the LA abrasion testing equipment and testing is carried out as per MTO LS-618 [MTO 2007]. The difference between the two methods is that the Micro-Deval test polishes the aggregates while the LA abrasion test breaks the aggregates. However, both tests are used to predict toughness and abrasion resistance of aggregates.

Figure 4: Micro-Deval drum

Soundness is a method of determining weathering resistance by simulating the effects of freeze-thaw cycles. The testing is conducted as per MTO LS-606 [MTO 2006]. The test involves measuring the amount by which the aggregates crack when subjected to soaking in a sodium or magnesium sulfate [AASHTO 1999]. During the soaking, the salt is absorbed into the pores of the aggregate and subsequently crystallizes during drying [TAC 1997] which simulates ice formation (Figure 5). Typical aggregate loss ranges from 10 to 20% for every five cycles [PI 2007a].

Figure 5: Aggregates before (left) and after (right) soundness test [PI 2007a].

The amount of deleterious materials in the aggregates are determined by measuring the%, by weight, of clay lumps, vegetable matter, friable particles, or other objectionable material [AASHTO 2000]. The procedure used to determine is outlined in MTO LS-609 [MTO 2006b]. A summary of the requirements of the source properties are outlined in Table 4 below for SP 19.0mm mix design used in this research.

Table 4: Source property requirements for SP 19.0mm mix [OPSS 2006]

Laboratory Test	MTO Test Number	Superpave 19.0mm Requirement
Micro-Deval Abrasion,% maximum loss	LS-618	21
Magnesium Sulphate Soundness,% maximum loss	LS-606	15
Petrographic Number, maximum [1]	LS-609	N/A

Note: 1) Petrographic analysis is only carried out for coarse aggregate used in Superpave surface mixes, and the Superpave 19.0 mix used in this research is a base mix.

Another important aggregate property is the gradation. Aggregate gradation requirements vary depending on the required pavement performance. In Ontario, provincial standards are put in place to specify the gradation requirements for use in hot mix asphalt. A summary of the gradation requirements for the Superpave 19.0mm mix used in this research are shown in Table 5 [OPSS 2007b]. The specifications also include quality control and quality assurance testing protocols.

Table 5: Aggregate Gradation Requirements for SP 19.0mm [OPSS 2007b]

Hot Mix Asphalt Type	**% 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 19.0mm	-	-	100	90-100	23-90	-	-	23-49	-	2-8

All the physical requirements put in place along with the aggregate gradation specifications ensure that the aggregate will have a strong skeleton that will enhance resistance to permanent deformation and enhance mixture durability [AI 2001a].

2.6.3 Asphalt

Asphalt is a residue product in the refining of crude oil and is the binder used in all asphalt concrete materials [TAC 1997]. Superpave uses a unique Performance Grade (PG) system of selecting binders based on the climate and traffic in which the pavement will serve. For instance, a binder classified as a PG 58-34 will be used in an area where the average seven-day maximum pavement temperature is 58°C and the expected minimum temperature is -34°C.

Superpave binder testing and specifications address physical properties of asphalt cement that are related to field performance by engineering principles [Hoffman 2008a]. Specifications are put in place for the selection of the correct grade of asphalt based on environment, traffic, and the desired reliability factor [AI 2001a, OPSS 2007]. To adjust the PG grade to account for traffic level and speed, AASHTO has a system put in place [AASHTO 2001] to adjust the high temperature grades of the binder.

2.6.4 Optimum Asphalt Binder Content

To determine the optimum asphalt binder content that yields the desired volumetric properties, a series of trials are carried out. One trial is typically performed for the proposed design asphalt content as set out in the contract documents, and then trials are carried out at 0.5% and 1.0% above and below the design asphalt content [AI 2001a].

Samples of the mixture are compacted in the SGC and testing is carried out to determine the densities of the samples. The asphalt binder content that corresponds to 4% air voids is chosen to be the optimum asphalt binder content provided that all other volumetric properties set out in the contract documents are met.

2.7 Hot Mix and Warm Mix Asphalt Mix Design

To date, HMA and WMA mix designs are performed in the same manner as discussed in the preceding section. The main difference between the two mixes is that manufacturer’s procedures have to be followed during the application process of the WMA technology being employed. All WMA technologies aim to reduce mixing (production) and compaction (placement) temperatures. However, the temperatures are dependent on the type of WMA and thus vary. All other procedures (ie, aggregate selection and asphalt binder selection and determination of optimum asphalt cement content) remain the same as in the Superpave mix design method.

The Transportation Research Board (TRB) is currently working on updating the AASHTO R35, Standard Practice for Superpave Volumetric Design for HMA, to include an appendix that will be titled Special Mixture Design Considerations and Methods for WMA, and this will address issues such as in-laboratory aging procedures, laboratory compaction effort, and recommended performance tests.

At the time when this research project was done this document was not yet available and thus mix design procedures for HMA and WMA were carried out as per Superpave design methodology with the exception of the mixing the compaction temperatures as recommended by the manufacturer of the WMA technology used.

2.8 Summary

This chapter presented a literature review on WMA technology. There was a review of the different types of WMA technologies available and also a discussion of the current challenges involved with WMA. It was determined that there is a lot of research that has been done, however the results are conflicting between different research when it comes to the issue of moisture susceptibility. There also seems to be a gap between the laboratory and the field tests, in that the test results obtained in the laboratory do not correlate with field performance. The economic and environmental information available for HMA and WMA production does not provide a complete evaluation of the WMA technology because factors such as maintenance and rehabilitation activities are omitted in the analysis. The background of mix asphalt mix design was also presented to show how mix design procedures differ between HMA and WMA.

Chapter 3
Experimental Methodology

3.1 Introduction

The general outline of the methodology that was followed throughout the research is outlined in Figure 6 below. The purpose of this chapter is to explain the testing procedures used in this research.

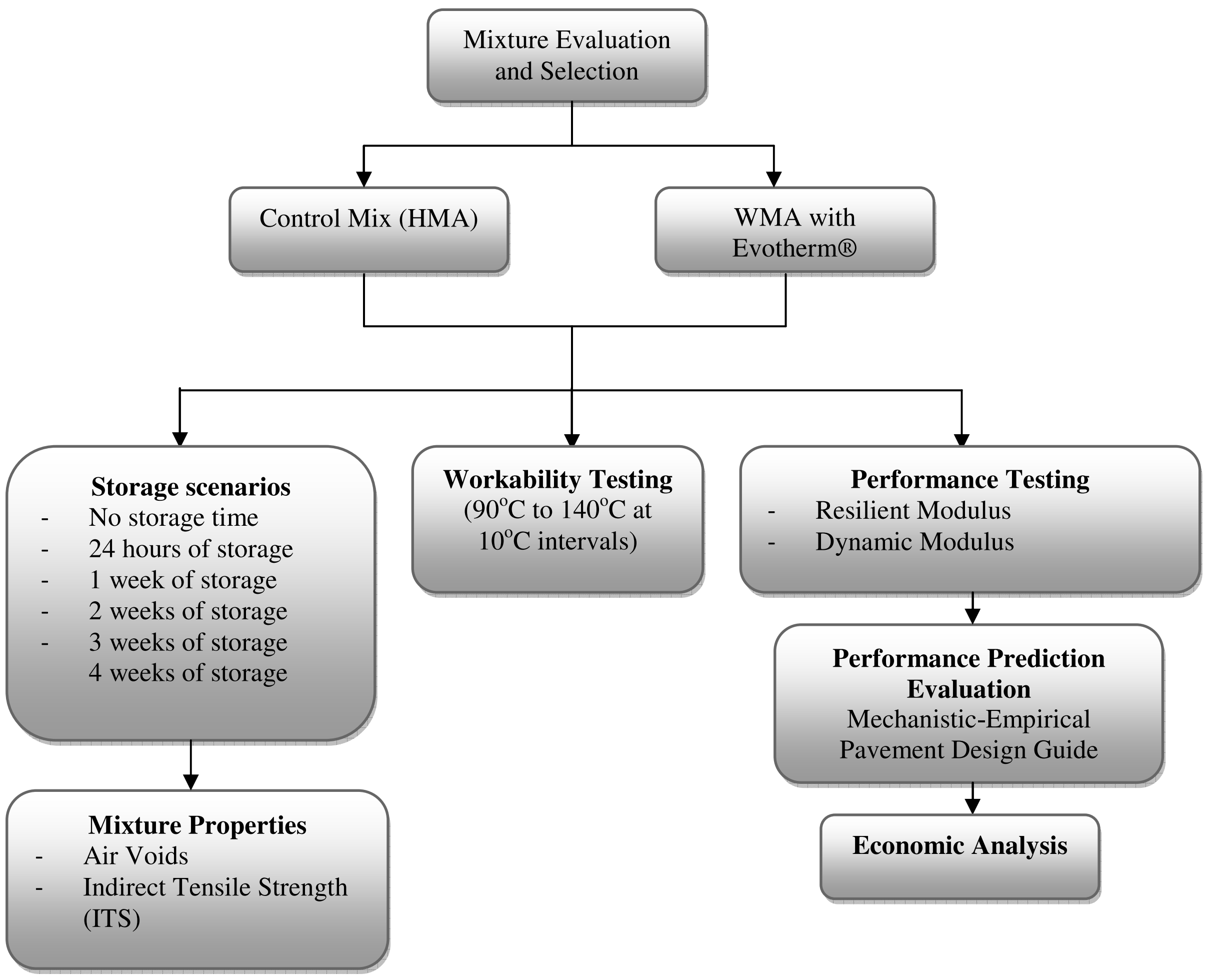


Figure 6: Research Methodology

3.2 Asphalt Mixes

The mix design used in this research was chosen based on material availability at the asphalt plant during this phase of the research. This mix is generally used as a base mix and was determined to be a Category D Superpave 19.0mm mix design. Details of the requirements of a Superpave 19.0mm mix design was described previously in Chapter 2.

The objective was to evaluate the difference in strength and performance characteristics between the HMA and WMA in the laboratory testing phase of this research. The mix was identical; however, HMA was used in one case while WMA was used in the other.

3.3 Mix Design Tests

The first part of this research involved determining mixture properties such as air voids and strength characteristics for both the HMA and WMA. Performance testing including resilient modulus and dynamic modulus tests weere also carried out for the two mixes. The performance properties determined through the testing was used as inputs for pavement performance analysis for Level 1 of the Mechanistic-Empirical Pavement Design Guide (MEPDG) design.

3.3.1 Air Void Testing

Air void (AV) testing was conducted as per Ministry of Transportation Ontario (MTO) Laboratory Testing Manual, Test Method LS-262 [MTO 1999] and LS-264 [MTO 2009]. The method involves determining the bulking relative density (BRD) of the compacted specimen in accordance with LS 262 [MTO 1999]. This value is then related to the maximum relative density (MRD) which is determined from the loose mix as described in LS 264 [MTO 2009] by Equation 3.1.

$$AV = \frac{MRD - BRD}{MRD} \times 100\% \qquad \textbf{3.1}$$

Where AV = air voids,%
MRD = maximum relative density, kg/m^3
BRD = bulk relative density, kg/m^3

The air void testing was carried out on both the HMA and WMA samples for six different scenarios, over a period of four weeks. The scenarios are as follows:

- Scenario 1 – Air voids determined as soon as samples were collected from the plant
- Scenario 2 – Air voids determined 24 hours after sampled from the plant
- Scenario 3 – Air voids determined 1 week after sampled from the plant
- Scenario 4 – Air voids determined 2 weeks after sampled from the plant
- Scenario 5 – Air voids determined 3 weeks after sampled from the plant
- Scenario 6 – Air voids determined 4 weeks after sampled from the plant

After the required storage time had elapsed, samples were fabricated using a Superpave Gyratory Compactor (SGC) and were compacted to 100 gyrations as required for a Category D mix design. The HMA samples were compacted at 134^{o}C while the WMA samples were compacted at 120^{o}C.

3.3.2 Indirect Tensile Strength (ITS)

ITS testing was conducted on the both the HMA and WMA samples as per the same scenarios followed in air void testing (Section 3.2.1). The test serves to measure the change in diametral tensile strength resulting from the effects of water saturation [AASHTO 2007b]. ITS is an important test because it is a good indicator of cracking potential of the asphalt mix [Hoffman 2009]. Results from this test are used to predict long-term stripping susceptibility of asphalt mixtures. Testing was conducted in accordance with AASHTO T 283.

Plant produced samples of the HMA and WMA were stored for the various allotted times, then six 100 mm diameter specimens of both HMA and WMA mixes were compacted to a height of 95 ± 5 mm with 7 ± 0.5% air voids. For each mix, the specimens were grouped into two subsets of three specimens each so that the air voids of the two subsets are approximately equal. One subset was used as a control, and the second subset was subjected to conditioning. The conditioning involved first vacuum saturation of the specimens to a degree of 70 to 80% then the specimens were wrapped with plastic.

The specimens were then subjected to a freeze-thaw cycle which consisted of a freeze cycle for 24 hours at -18 ± 3°C for 16 hours, followed by a soaking cycle in warm water set at 60 ± 1°C for 24 hours. At the end of the conditioning cycle of the second subset, all six specimens were placed in a heavy duty plastic bag and placed in a 25 ± 0.5°C for 2 hours ± 10 minutes as per AASHTO T 283 [AASHTO 2007b].

The specimens were tested by applying a load at a constant rate of 50 mm per minute. The maximum load at which the specimen broke was recorded and used to calculate the tensile strength of the specimen as per Equation 3.2. The tensile strength ratio (TSR) was determined by comparing the strength of the conditioned specimens (S_2) to the strength of the unconditioned specimens (S_1) as per Equation 3.3.

$$S_t = \frac{2000 \times P}{\pi t D} \qquad \textbf{3.2}$$

Where S_t = tensile strength, kPa
P = maximum load, N
t = specimen thickness, mm
D = specimen diameter, mm

$$TSR = \frac{S_2}{S_1} \qquad \textbf{3.3}$$

Where TSR = Tensile Strength Ratio, %
S_2 = Strength of conditioned specimen, kPa
S_1 = Strength of unconditioned specimen, kPa

3.3.3 Densification

Densification describes how easily asphalt mixture can be placed and compacted in the field. Mixtures that are easy to compact would exhibit good workability and mixtures that are more difficult and have poor workability [AI 2001b]. Asphalt mixtures parameters that affect workability are aggregate source, gradation, type of asphalt cement, and now, warm mix asphalt additives.

In the laboratory, workability can be measured during compaction by measuring the height changes of the asphalt specimens using a Superpave Gyratory Compactor (SGC) [Siswoso 2005]. This methodology was employed in this research to compare the workability characteristics of the HMA and the WMA. The samples were compacted with the same number of gyrations using the SGC at various temperatures (90 to 135^{o}C at 5^{o}C increments). The heights of the samples were compared at different gyration levels over the specified range of temperatures.

3.4 Performance Testing

3.4.1 Resilient Modulus

Resilient modulus testing was conducted on both HMA and WMA samples in accordance with ASTM D 7369-09 in the Centre for Pavement and Transportation Technology (CPATT) laboratory at the University of Waterloo. The ASTM summary of the test method is as follows: a repetitive application of sinusoidal compressive loads along a vertical diametrical plane of a cylindrical asphalt concrete specimen, as shown in Figure 7. The resulting horizontal and vertical deformations are measured and used to determine the Poisson's ratio. From this, the instantaneous resilient modulus is calculated using the instantaneous recoverable deformation that occurs during the unloaded portion of one load-unload cycle [ASTM 2009].

Plant produced samples of HMA and WMA were compacted to a height of 50 mm and diameter of 150 mm with air void content of 7 ± 1%. Four specimens of with mixture were made and each specimen was tested twice on two diametrical planes perpendicular to each other at 25^{o}C as per ASTM D 7369-09 and the results were averaged for each specimen.

Figure 7: Resilient Modulus Test, CPATT Laboratory

3.4.2 Dynamic Modulus

Dynamic modulus testing was conducted on both HMA and WMA samples in accordance with AASHTO TP 62-07 in the CPATT laboratory at the University of Waterloo [AASHTO 2007a]. Plant produced samples of HMA and WMA were compacted to a height of 150 mm and diameter of 150 mm with air void content of 7±1%. The specimens were cored to 100 mm diameter and 6 specimens of the mixture were made. Each specimen was tested at five temperatures (-10, 4.4, 21, 37, and 54^{o}C) and six frequencies (25, 10, 5, 1, 0.5, and 0.1 Hz).

The testing procedure involves applying a sinusoidal axial compressive stress to a cylindrical specimen (Figure 8). The stress is applied over the specified range of frequencies and temperatures. The applied stress and the resulting recoverable axial strain response was measured and used in calculating the dynamic modulus [AASHTO 2007a]. The calculated values can be used to develop a master curve. Dynamic modulus is also a required material property for Level 1 of MEPDG design.

Figure 8: Dynamic Modulus Test Setup, CPATT Laboratory

3.5 Mechanistic-Empirical Pavement Design Guide (MEPDG)

The MEPDG is a new pavement design methodology that is being adopted in Canada. It was developed by the American Association of State Highway and Transportation Officials (AASHTO) and adoption of MEPDG in Canada by all provincial transportation agencies is under way through a pooled fund study of the Transportation Association of Canada (TAC). The MEPDG program can be used to investigate the long-term performance of asphalt mixes and is able to predict the development and propagation of pavement distresses, which include rutting, thermal cracking, permanent deformation, and fatigue cracking [Goh 2007]. These predictions are made by taking into account the traffic loading, climactic effects, pavement structure, and material characteristics [Diefenderfer 2008].

There are three input levels in MEPDG that can be used based on resource and information availability and level of accuracy required. Level 1 provides the highest level of accuracy, and thus the lowest level of uncertainty and error [ARA 2004]. This level is usually used for major freeways and requires detailed materials information such as the dynamic modulus testing results as an input. Level 2 provides an intermediate level of accuracy and is used when resources or testing equipment are not available. Statistic tests are used to estimate the dynamic modulus and other material characterization parameters. Inputs can be estimated based on the transportation agency database or through correlations [ARA 2004]. Level 3, which relies on default values, provides the lowest level of accuracy and is used for low volume roads where there would be minimal consequence of early failure. The inputs for level 3 do not require any specialized testing. The analysis done as part of this research used Level 1 inputs for a rehabilitation project characterized in the program as asphalt over asphalt. The pavement structure that was used in the analysis for both the HMA and WMA is shown in Figure 9. The inputs that were constant through the analysis are summarized in Table 6 through Table 8. Some of these inputs are default values of the MEPDG program. All values have been selected based on a typical arterial road in Ontario.

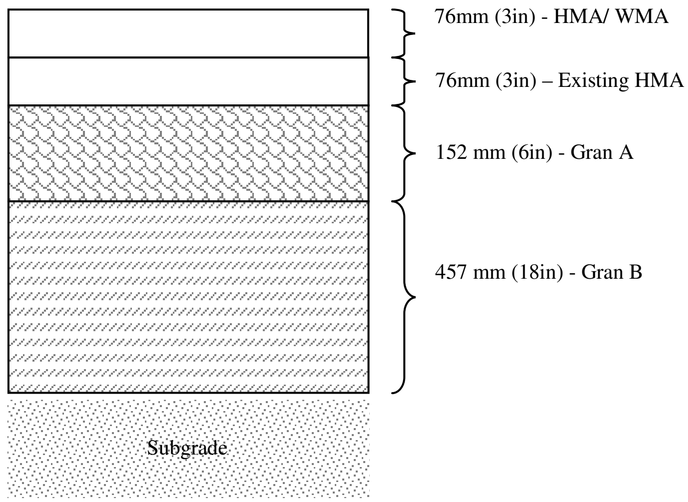


Figure 9: Cross-section of Pavement Design for Economic Analysis

Table 6: MEPDG Pavement Design Input Values

Description	Input Value
Initial Average Annual Daily Traffic (AADT)	3000
Growth rate [%]	2
Number of Lanes in Design Direction	1
% of Trucks in Design Direction [%]	40
% of Trucks in Design Lane [%]	100
Operational Speed [km/h]	80
Design Life [years]	20
Reliability Level [%]	75
Climate Data	Toronto Pearson International Airport

Table 7: MEPDG Material Gradation Input Values

Sieve Size (% Passing)	Existing HMA	Granular A	Granular B	Subgrade (Well Graded Gravel)
37.5 mm	100	100	100	100
26.5 mm	-	100	100	N/A
19.0 mm	96.8	96.4	93.1	N/A
16.0 mm	90.2	88	84.7	N/A
13.2 mm	-	79.1	-	N/A
12.5 mm	84.0	-	-	N/A
9.5 mm	74.0	61.9	56	N/A
4.75 mm	56.8	41.0	35.9	60
2.36 mm	44.9	32.8	26.7	-
1.18 mm	33.0	24.7	20.6	55
0.600 mm	25.0	17.7	16.3	-
0.300 mm	17.8	10.1	9.7	50
0.150 mm	8.4	4.6	5.8	33.5
0.075 mm	3.7	2.2	3.5	12.5
PGAC	58-28	N/A	N/A	N/A

Notes: N/A is Not Applicable
PGAC is Performance Graded Asphalt Cement

Table 8: MEPDG Material Property Design Values

Material Property	Asphalt	Granular A	Granular B	Subgrade (Well Graded Gravel)
Plasticity Index	N/A	0	0	2
Liquid Limit	N/A	0	0	8
Modulus [MPa]	N/A	207	207	200
Poisson's Ratio[1]	0.35	0.35	0.35	0.35
Coefficient of Lateral Pressure (K_o)[2]	N/A	0.5	0.5	0.5

Note: 1. MEPDG default values
N/A is Not Applicable

Design inputs for both the HMA and WMA used dynamic modulus values obtained from the laboratory testing conducted at the CPATT laboratory and asphalt cement properties obtained from McAsphalt Industries Limited who supplied the asphalt.

3.6 Economic Analysis

A life cycle cost analysis (LCCA) was performed to determine the difference in cost for construction and maintenance for the HMA and WMA pavements. The distress development and propagation results determined though the MEPDG analysis were used in conjunction with the MTO Manual for Condition Rating of Flexible Pavements to determine treatment alternatives for the pavement distresses.

Estimated costs were obtained from Miller Paving Limited for the HMA and WMA material costs as well as treatment costs for the distresses. An additional cost that was included in the construction costs was fuel consumption of the asphalt plant during the production of the two different asphalt materials. Fuel consumption was measured at the plant during the production of both material types and recorded to be used in this economic analysis.

Based on the distresses that were evident through the MEPDG analysis the maintenance treatments chosen for the economic analysis were rout and seal, and chip seal. The net present value (NPV) of the costs associated with the initial costs and maintenance schedules were determined using Equation 3.4.

$$NPV = InitialCost + F\left[\frac{1}{(1+i)^n}\right] \qquad \textbf{3.4}$$

Where n = number of years
i = interest rate
F = future cost at the end of n years

3.7 Analysis of Variance (ANOVA)

Analysis of Variance (ANOVA) was used in this research to analyze the variances present in the testing and to compare the HMA with the WMA. This is done by comparing the variances around the mean values of two or more groups on one or more variables in order to accept or reject the hypothesis.

The research hypothesis is that the HMA and the WMA have the same volumetric and performance characteristics. The statistical analysis procedures conducted in order to accept or reject this hypothesis include the F-test and Single Factor ANOVA. All analysis was conducted at 95% confidence level and is one-sided.

An F-test is designed to test if two population variances are equal [Jones 2011]. A ratio of the two variances ($F_{calculated}$) is compared against a table value ($F_{critical}$). The $F_{critical}$ is based on the degrees of freedom used to calculate both variances and a selected confidence level ($\alpha = 0.05$). The degrees of freedom (sample size – dependent variables) refers to the number of values in the final calculation of a statistic that are free to vary [Jones 2011]. If $F_{calculated} < F_{critical}$, then the hypothesis is true and the comparison groups are statistically equivalent. Alternatively, if the $F_{calculated} > F_{critical}$, then the two comparisons groups are statistically different, then the hypothesis is rejected.

Another parameter used in the ANOVA is the P-value, which is the probability of obtaining a test statistic at least as extreme as the one that was actually observed, assuming the hypothesis is true [Jones 2011]. When the P-value < α then the hypothesis is rejected, meaning it is not statistically significant.

3.8 Summary

This chapter discussed the experimental methodology employed in this research. The research included testing of mix properties such as air voids and indirect tensile strength of HMA and WMA for a period of four weeks to determine if these properties change over time. Densification was also determined for both HMA and WMA mixtures over a range of temperatures (90 to 135^{o}C at 5^{o}C increments) using a SGC to determine the workability of the mixtures at the different temperatures. Performance testing included resilient and dynamic modulus testing. Dynamic modulus results were used as inputs in a Level 1 MEPDG analysis to determine distress development and propagation for both HMA and WMA mixes. Finally, the life cycle cost analysis was conducted to determine the Net Present Value associated with the initial construction and maintenance of the HMA and WMA pavements based on the performance outputs from MEPDG analysis.

Chapter 4
HMA and WMA Test Results

4.1 Introduction

This chapter summarizes and discusses the test results obtained from all the testing in this research. Results of mixture properties for both HMA and WMA (air voids, moisture susceptibility and workability) and performance testing (resilient modulus, dynamic modulus, and TSRST) are discussed. The results from the performance testing that was carried out, combined with material properties of the aggregates and asphalt binder were then used in the pavement design, MEPDG software to predict the expected pavement performance for the two different materials. This is very important as it assists pavement designers in determining where the WMA should be used.

4.2 Air Void Characteristics

As previously discussed in Chapter 3, air voids were determined for both HMA and WMA samples under different scenarios which represent the different storage times as follows:

- Scenario 1 – No storage time (sampled from the plant)
- Scenario 2 – 24 hours after sampled from the plant
- Scenario 3 – 1 week after sampled from the plant
- Scenario 4 – 2 weeks after sampled from the plant
- Scenario 5 – 3 weeks after sampled from the plant
- Scenario 6 – 4 weeks after sampled from the plant

Air voids are the small pockets of air that exists between the asphalt coated aggregates in a pavement structure [PI 2010]. Air voids are very important to asphalt pavement because they are related to the durability and stability of the asphalt mixture. A mix design that has less than 3% air voids will result in an unstable mixture and a mix design that has more than 8% air voids will result in a water-permeable mixture [PI 2010]. Air voids is calculated from the BRD and MRD as previously explained in Chapter 3. The results of the air voids testing for the specimens are summarized in Table 9.

Table 9: Air Void Test Results

Scenario	HMA			WMA		
	BRD [kg/m^3]	MRD [kg/m^3]	Avg. Air Voids [%]	BRD [kg/m^3]	MRD [kg/m^3]	Avg. Air Voids [%]
1	2.422	2.511	3.9	2.422	2.510	3.5
2	2.422	2.512	3.6	2.427	2.523	3.8
3	2.417	2.529	4.4	2.423	2.521	3.9
4	2.426	2.520	3.7	2.426	2.511	3.4
5	2.422	2.521	3.9	2.425	2.516	3.6
6	2.417	2.526	4.3	2.413	2.518	4.2

The air voids test results for HMA and WMA did not exhibit any unusual trends over the four-week storage period. The WMA did appear to have lower air voids than the HMA with the exception of Scenario 2 in which WMA had 0.2% more air voids than the HMA.

A Single Factor ANOVA at 95% confidence as shown in Table 10 showed that the difference between the air voids results of the HMA and WMA over the different storage periods was not statistically significant as $F_{calculated} < F_{critical}$.

Table 10: ANOVA for Air Voids

Source of Variation	Sum of Squares (SS)	Degree of Freedom (df)	F	P-value	F-critical
Between HMA and WMA	0.14	1	1.23	0.23	4.96

4.3 Indirect Tensile Strength (ITS)

The results of the ITS testing are summarized in Table 11 below. The dry and wet results represent the unconditioned and conditioned specimens respectively. Testing was conducted under the same scenarios as the air void testing.

Table 11: Indirect Tensile Strength (ITS) and Tensile Strength Ratio (TSR) results

Scenario	HMA				WMA			
	Avg. Voids (Dry/Wet) [%]	Dry ITS [kPa]	Wet ITS [kPa]	TSR [%]	Avg. Voids (Dry/Wet) [%]	Dry ITS [kPa]	Wet ITS [kPa]	TSR [%]
1	6.6/6.7	703.6	515.5	73.3	7.0/6.9	483.3	372.9	77.2
2	6.6/6.7	754.7	453.3	60.1	6.5/6.7	530.0	390.0	73.6
3	7.0/7.1	833.7	371.3	44.5	7.3/7.1	532.6	336.5	63.2
4	6.6/6.8	739.5	437.6	59.2	6.5/6.7	661.2	461.2	69.8
5	6.9/6.6	765.5	515.8	67.4	7.1/7.0	646.2	509.4	78.8
6	6.7/6.9	828.2	545.9	65.9	7.1/7.1	744.1	537.1	72.2

Based on the ITS and TSR results summarized above, there does not appear to be any trend in the TSR results for either HMA or WMA over the different storage times. A Single Factor ANOVA at 95% confidence as shown in Table 12 indicated that the difference between the TSR results of the HMA and WMA was statistically significant since $F_{calculated} > F_{critical}$, thus, the HMA and WMA are statistically different.

Table 12: ANOVA for Tensile Strength Ratio

Source of Variation	Sum of Squares (SS)	Degree of Freedom (df)	F	P-value	F-critical
Between HMA and WMA	345.61	1	5.35	0.04	4.96

What is interesting to note is that the WMA dry and wet ITS increased over the four week storage period, while the results for HMA did not show an increase. This can be attributed to asphalt binder aging of the WMA over the storage time. Although these samples were only stored at room temperature, the findings correlate to other research where samples were stored in an oven at elevated temperatures (up to 146°C) for a few hours (up to 8 hours) to simulate binder aging over time [Mogawer 2011]. A Single Factor ANOVA at 95% confidence shown in Table 13 and Table 14 suggested that the difference between the HMA and WMA results were statistically different since $F_{calculatred} > F_{critical}$ for both Dry ITS and Wet ITS test results.

Table 13: ANOVA for Dry Indirect Tensile Strength

Source of Variation	Sum of Squares (SS)	Degree of Freedom (df)	F	P-value	F-critical
Between HMA and WMA	88031.07	1	14.03	0.00	4.96

Table 14: ANOVA for Wet Indirect Tensile Strength

Source of Variation	Sum of Squares (SS)	Degree of Freedom (df)	F	P-value	F-critical
Between HMA and WMA	81724.51	1	9.97	0.01	4.96

4.4 Densification

As described in Chapter 3, workability was determined by comparing the height changes in the HMA and WMA specimens while being compacted with the Superpave Gyratory Compactor (SGC). The height changes and the amount of revolutions were recorded and a workability calculation process was employed to determine a workability index (WI). In the proposed process, the WI is the inverse of the mixture's porosity value when revolutions equal zero [Siswoso 2005].

The first step in the processes involves determining the volume of the specimens for every chosen gyration level using Equation 4.1. In this case the height of the sample was measured after every 10 gyrations or one interval for a total of 12 intervals as the sample was run for 120 gyrations.

$$V_i = \left(\frac{1}{4}\pi d^2\right)h_i \qquad \textbf{4.1}$$

Where V_i = specimen's volume at the i gyrations (cm3)
h_i = specimen's height at the i gyrations (cm)
d = specimens diameter (cm)

The next step was to determine the density and porosity of the specimens at the chosen gyrations using Equation 4.2 and Equation 4.3. Determining the density of the specimens also consequently determines how much air pockets are the in the specimens at the chosen gyrations, which is also the porosity in the specimen.

$$D_i = \frac{W_a}{V_i}$$ **4.2**

$$P_i = 100\left(1 - \frac{D_i}{SG}\right)$$ **4.3**

Where D_i = density at the i gyrations
W_a = specimen weight
P_i = porosity at the i gyrations
SG = specific gravity of the specimens

A plot was then generated of the porosity values against the gyrations in the form of Equation 4.4. Since the proposed process states the WI is the inverse of mixture porosity value when revolutions is equal to zero, the y-intercepts were determined from the generated porosity plots at the various compaction temperatures.

$$P_i = A - B\log i$$ **4.4**

Where A = the intercept of the line and y-axis
B = the angle between the line and x-axis
i = the amount of gyrations

The determined y-intercept values (A) from the graphs were used to calculate the workability index (WI) using Equation 4.5.

$$WI = \frac{100}{A}$$ **4.5**

This process was carried out over a range of temperatures (90^{o}C to 140^{o}C in 10 degree intervals) as described in Chapter 3. A summary of the porosity results for the HMA are shown in Table 15 and a summary of the porosity and calculated WI are jointly presented in Table 16. Figure 10 shows the porosity as a function of gyrations at various temperatures for HMA. Table 17 and Table 18 show the similar results for the WMA whereby Table 17 shows the porosity results and Table 18 shows porosity and calculated WI and Figure 11 summarizes the results.

Table 15: Porosity results for HMA at different temperatures

Gyration Interval	HMA Porosity [%]					
	90°C	100°C	110°C	120°C	130°C	140°C
10	17.9	17.2	16.8	15.9	15.8	15.9
20	15.9	15.5	15.2	14.2	14.1	14.2
30	14.7	14.7	14.4	13.2	13.1	13.3
40	14.0	14.0	13.8	12.7	12.5	12.7
50	13.5	13.6	13.4	12.2	12.0	12.2
60	13.1	13.3	13.1	11.8	11.7	11.9
70	12.8	13.0	12.9	11.5	11.4	11.6
80	12.6	12.8	12.6	11.3	11.2	11.4
90	12.3	12.6	12.4	11.1	10.9	11.1
100	12.2	12.5	12.3	10.9	10.8	11.0
110	12.0	12.2	12.1	10.8	10.6	10.7
120	11.8	12.1	11.9	10.6	10.5	10.7

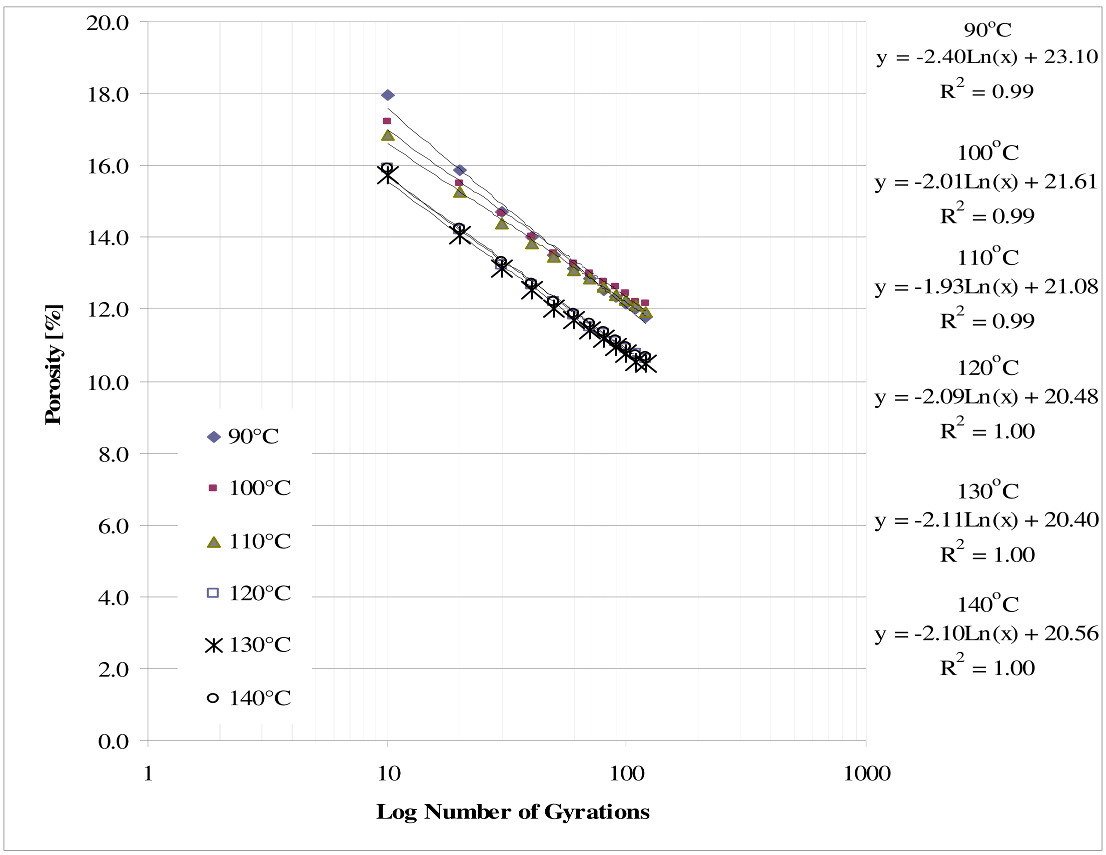


Figure 10: Porosity vs. Number of Gyrations for HMA mix

Table 16: HMA Summary Porosity and Workability Index

Temperature	Porosity Line Equation	Y-Intercept	Workability Index
90^{o}C	-2.40 ln(x) + 23.10	23.10	4.33
100^{o}C	-2.01 ln(x) + 21.61	21.61	4.62
110^{o}C	-1.93 ln(x) + 21.08	21.08	4.74
120^{o}C	-2.09 ln(x) + 20.48	20.48	4.85
130^{o}C	-2.11 ln(x) + 20.40	20.40	4.90
140^{o}C	-2.10 ln(x) + 20.56	20.56	4.86

Table 17: Porosity results for WMA at different temperatures

Gyration Interval	Porosity [%]					
	90°C	100°C	110°C	120°C	130°C	140°C
10	16.6	16.4	16.2	15.9	15.4	15.6
20	15.0	14.7	14.5	14.2	13.7	14.2
30	14.1	13.9	13.6	13.4	12.8	13.4
40	13.5	13.3	13.0	12.8	12.3	12.9
50	13.2	12.8	12.6	12.4	11.8	12.5
60	12.8	12.6	12.3	12.0	11.5	12.2
70	12.6	12.3	12.0	11.7	11.2	11.9
80	12.3	12.0	11.8	11.5	11.0	11.6
90	12.1	11.9	11.5	11.4	10.7	11.4
100	12.0	11.7	11.4	11.1	10.6	11.3
110	11.8	11.6	11.2	11.0	10.4	11.1
120	11.7	11.4	11.1	10.9	10.3	10.9

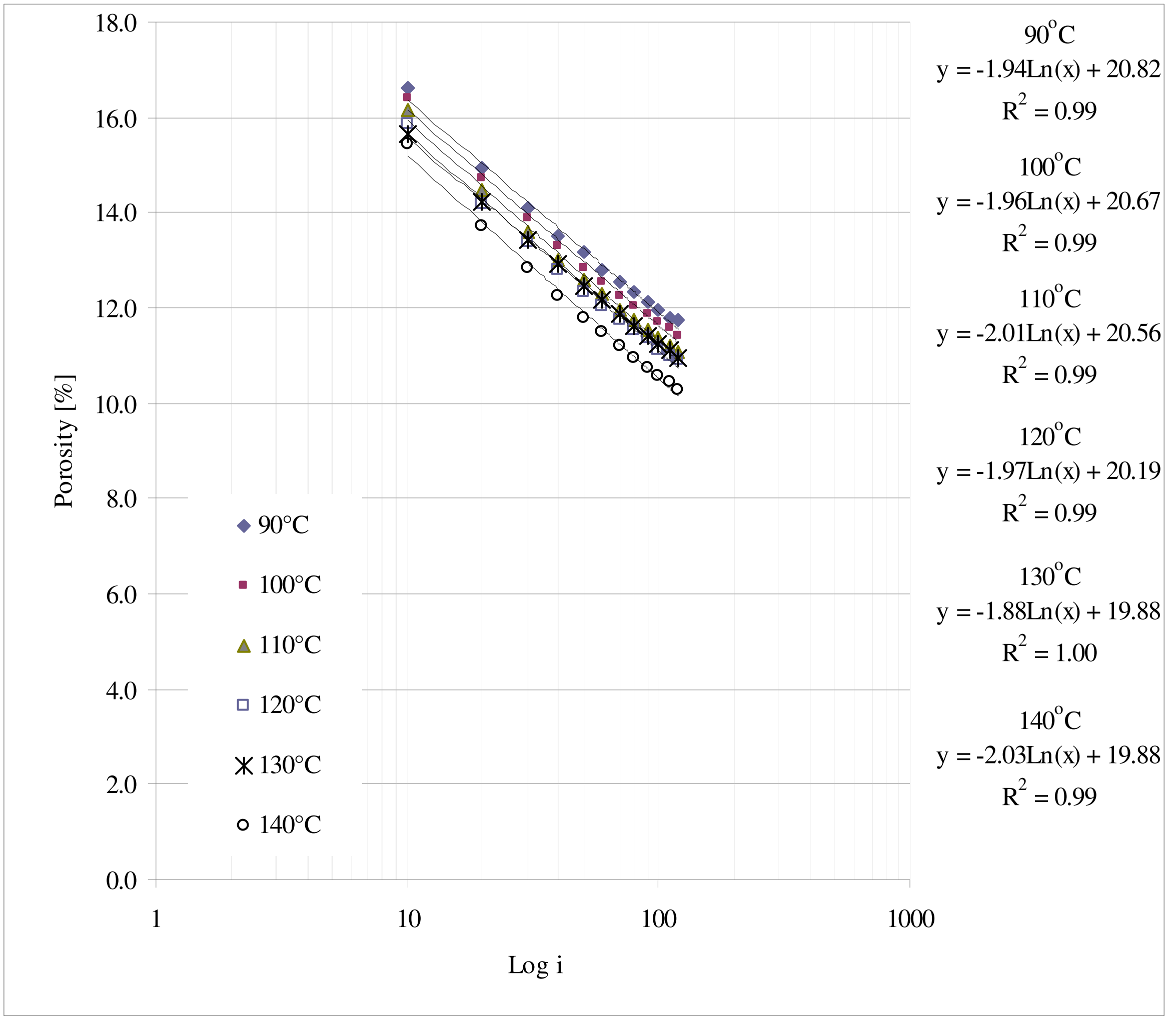


Figure 11: Porosity vs. Number of Gyrations for WMA mix

Table 18: WMA Summary Porosity and Workability Index

Temperature	Porosity Line Equation	Y-Intercept	Workability Index
90°C	-1.94 ln(x) + 20.82	20.82	4.80
100°C	-1.96 ln(x) + 20.67	20.67	4.84
110°C	-2.01 ln(x) + 20.56	20.56	4.86
120°C	-1.97 ln(x) + 20.19	20.19	4.95
130°C	-1.88 ln(x) + 19.88	19.88	5.03
140°C	-2.03 ln(x) + 19.88	19.88	5.03

Table 19 below shows a summary of WI for HMA and WMA. The workability index shows that the WMA mix consistently had better workability characteristics over the range of the chosen temperatures.

Table 19: Workability Index Values for HMA and WMA

Temp [^oC]	WI	
	HMA	WMA
90	4.33	4.80
100	4.62	4.84
110	4.74	4.86
120	4.85	4.95
130	4.90	5.03
140	4.86	5.03

A Single Factor ANOVA at 95% confidence was performed on the WI results and showed that although the WI for the WMA mix was consistently higher than that of the HMA mix, $F_{calculated} < F_{critical}$ thus difference were not statistically significant. However, it is notable that the WI of WMA at 90^{o}C was not achieved in the HMA mix until a temperature of 120^{o}C. This means it requires more energy and effort to achieve the compaction.

Table 20: ANOVA for Workability Index

Source of Variation	Sum of Squares (SS)	Degree of Freedom (df)	F	P-value	F-critical
Between HMA and WMA	0.12	1	4.34	0.06	4.96

4.5 Performance Testing

4.5.1 Resilient Modulus

Resilient modulus testing was performed on both HMA and WMA samples at 25°C, as described in Chapter 3. Four samples for each mix type were tested. Table 21 and Table 22 are a summary of average resilient modulus values and Poisson's ratio obtained from testing each specimen twice on the two diametrical planes perpendicular to each other.

Table 21: Resilient Modulus test results for HMA

Sample Number	Average Instantaneous Resilient Modulus [MPa]	Average Poisson's Ratio
1	3,035	0.06
2	3,101	0.04
3	2,795	0.07
4	2,775	0.04
Average	2926	0.05
Standard Deviation	166	0.01

Table 22: Resilient Modulus test results for WMA

Sample Number	Average Instantaneous Resilient Modulus [MPa]	Average Poisson's Ratio
1	3,401	0.22
2	3,056	0.15
3	3,059	0.12
4	3,208	0.15
Average	3181	0.16
Standard Deviation	163	0.04

The average resilient modulus results for the four samples showed that the WMA mix had a higher resilient modulus than the HMA. This can be attributed to the fact that the HMA and WMA will have different aggregate structures at the two different compaction temperatures. WMA is easier to compact to the required air voids and therefore will exhibit a stronger aggregate skeleton [You 2007], which increases its capability to transfer loads from one aggregate to another aggregate [Goh 2008]. This structure provides many advantages when placing a pavement at lower temperatures, which often occurs in the spring or fall in Canada. The resilient modulus test results found that this testing correlates with findings of other research [Goh 2008]. A Single Factor ANOVA at 95% confidence was performed on the resilient modulus test results, shown in Table 23, indicated that $F_{calculated} < F_{critical}$ and therefore the difference between them is not statistically significant.

Table 23: ANOVA for Resilient Modulus Test Results

Source of Variation	Sum of Squares (SS)	Degree of Freedom (df)	F	P-value	F-critical
Between HMA and WMA	129541	1	4.79	0.07	5.99

4.5.2 Dynamic Modulus

Dynamic modulus testing was performed on the HMA and WMA samples as described in Chapter 3 at six loading frequencies and at five different temperatures. This is an important test for asphalt as it provides a measurement of strength over a range of temperatures and loadings which relates to traffic. For example, the range would represent a static vehicle versus dynamic. It is an excellent predictor of field performance. Four samples of both HMA and WMA samples were tested and the average results are summarized in Table 24. Complete results of individual samples can be found in Appendix A.

Table 24: Dynamic Modulus results for HMA and WMA

Mix	Frequency [Hz]	Average Dynamic Modulus [MPa]				
		-10°C	4.4°C	21°C	37°C	54°C

HMA	25	12,777	9,303	5,190	2,625	1,058
	10	12,825	8,920	4,966	2,536	925
	5	12,611	8,501	4,692	2,348	828
	1	11,497	7,099	3,692	1,774	606
	0.5	11,083	6,669	3,247	1,656	577
	0.1	10,004	5,668	2,897	1,409	493
WMA	25	15,115	10,904	5,714	2,827	1,098
	10	14,862	10,486	5,465	2,502	852
	5	14,369	9,885	5,052	2,200	715
	1	12,517	8,008	3,631	1,586	551
	0.5	11,746	7,374	3,366	1,450	502
	0.1	10,220	6,002	2,735	1,174	425

Graphical representation of the dynamic modulus test results for both HMA and WMA are shown in Figure 12 and Figure 13 respectively.

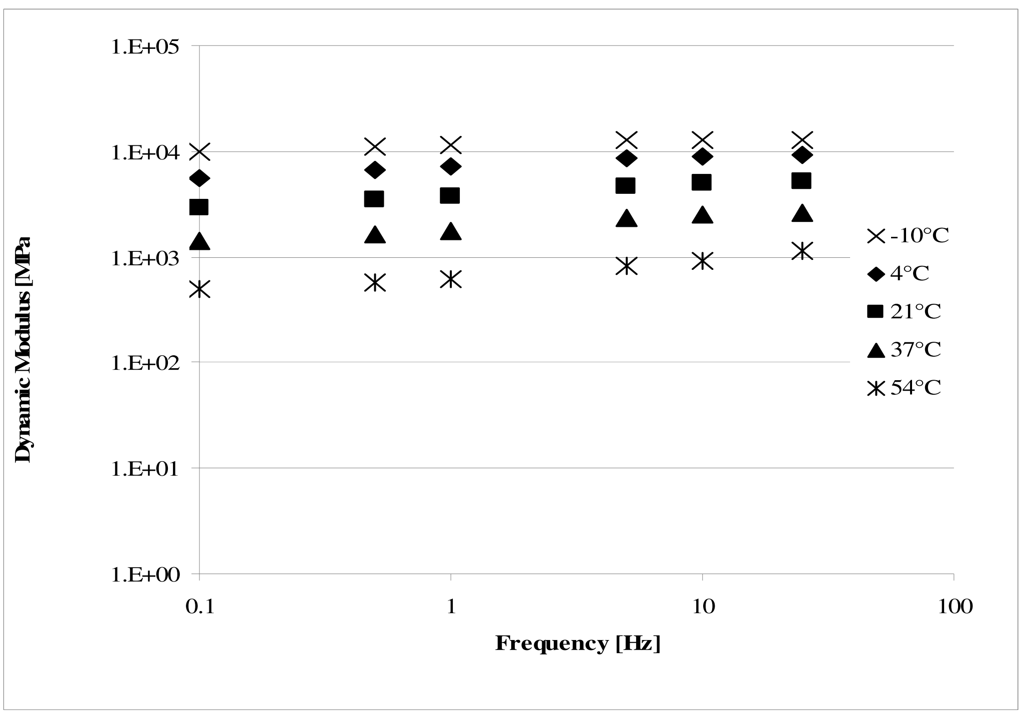


Figure 12: Dynamic Modulus Results for HMA

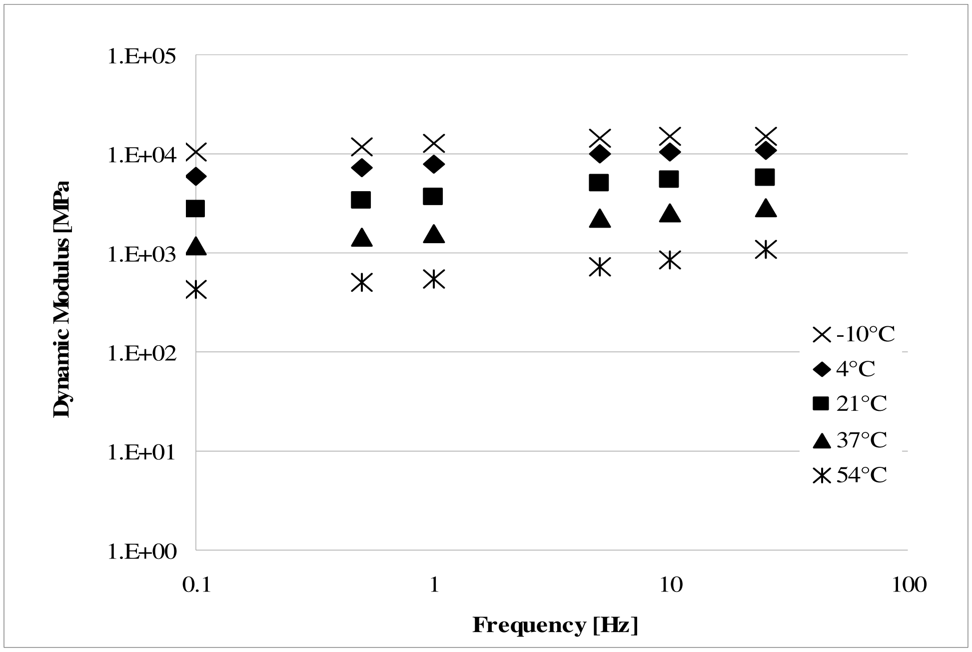


Figure 13: Dynamic Modulus Results for WMA

4.5.3 Master Curve Development

Master curves were constructed using the principle of time-temperature superposition. A reference temperature is selected; in this case 21^{o}C, then the measured data at the other temperatures are shifted with respect to loading frequency until the curves merge into a smooth function [Bonaquist 2005]. The temperature dependency of the material is described by how much shifting is required at each temperature. The greater the shift factor, the greater the temperature dependency (temperature susceptibility) of the material [Cross 2007].

The master curve helps to extend the range of data and helps in predicting the behavior of the material at temperatures which would otherwise be outside the range of the measured temperatures. For instance, the portion of the master curve that is in the low frequency region describes the dynamic mechanical behavior of the mixture at higher temperatures such as rutting, while the portion in the higher frequency region describes dynamic mechanical behavior at lower temperatures such as fatigue cracking [Shenoy 2002]. Equation 4.6 and Equation 4.7 are used to describe the rate dependency master curve.

$$\log(E^*) = \delta + \frac{\alpha}{1 + e^{\beta + \gamma(\log \omega_r)}} \qquad \textbf{4.6}$$

Where E* = dynamic modulus
ω_r = reduced frequency
δ, α, β, γ = fitting parameters

$$\omega_r = \omega \times a(T) \qquad \textbf{4.7}$$

Where ω = loading frequency
a(T) = shift faster as a function of temperature
T = temperature

Numerical optimization was performed using Microsoft Excel and the master curves along with the shift factors developed for both HMA and WMA are shown in

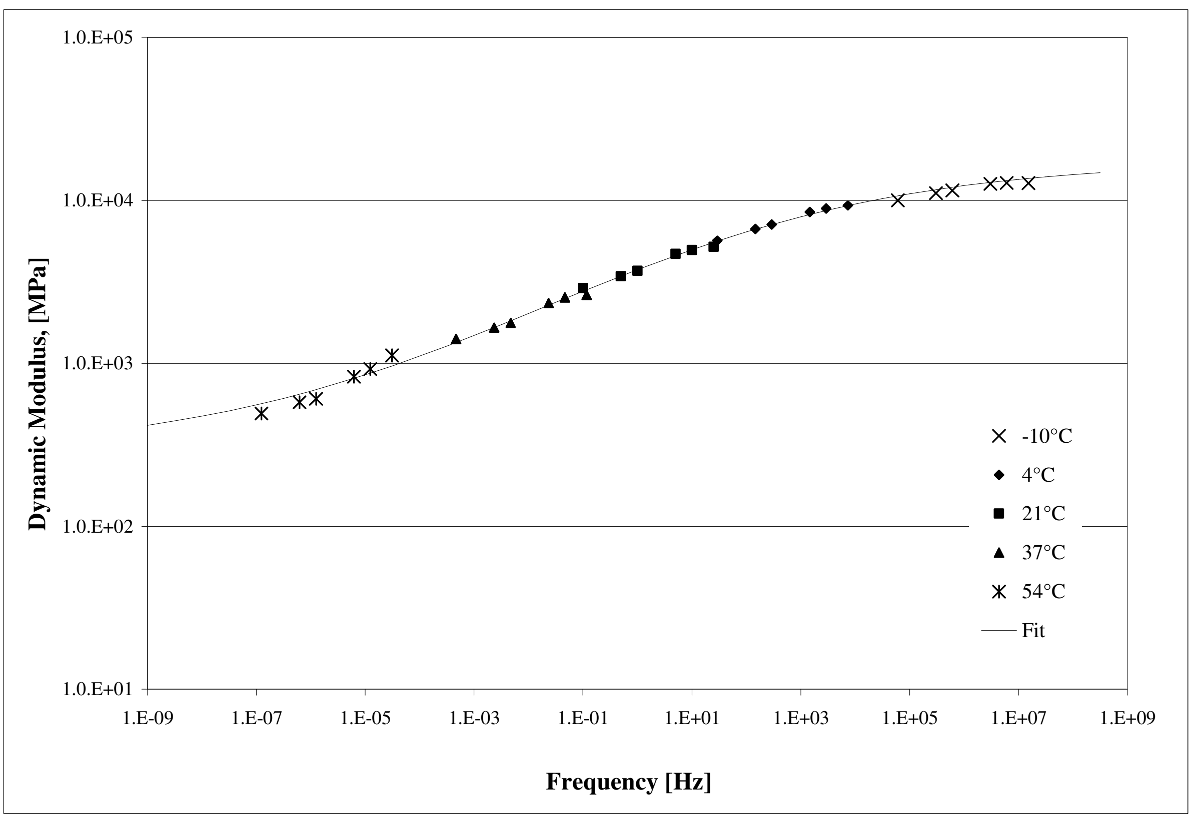


Figure 14 through Figure 17.

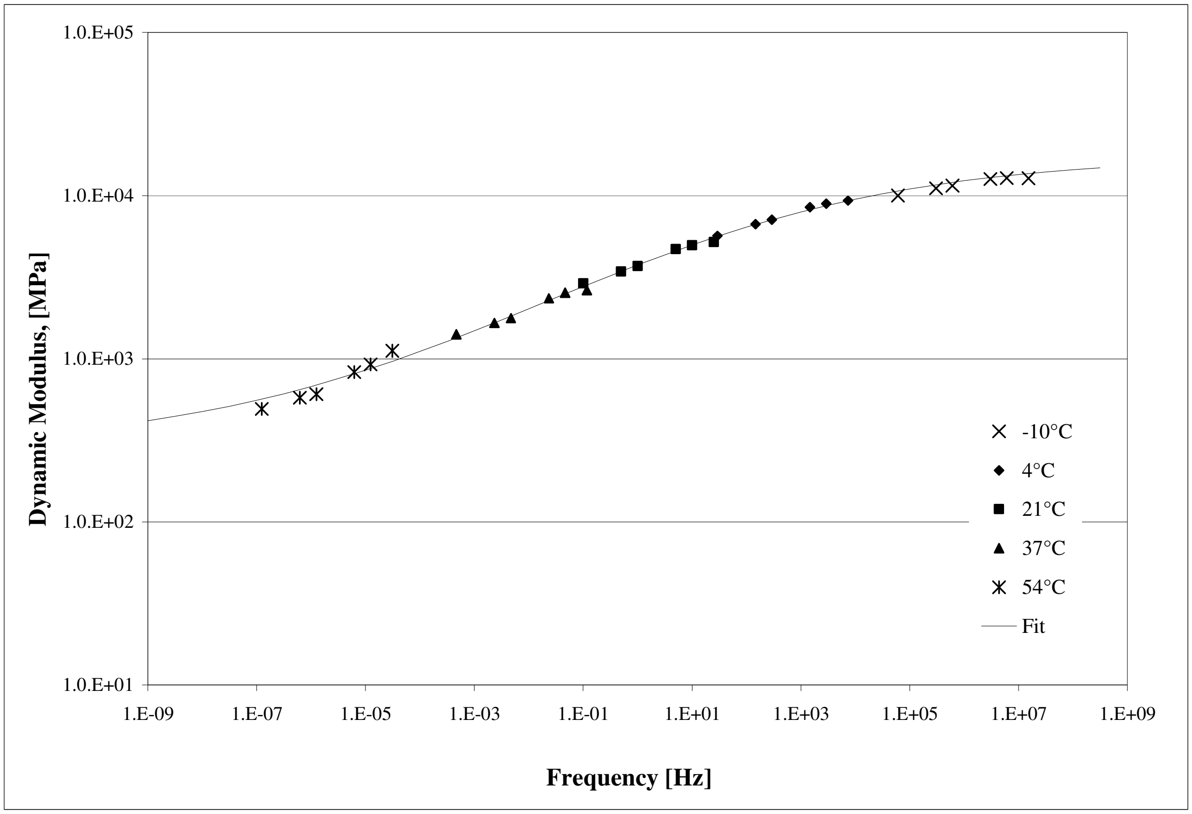


Figure 14: Master curve for HMA

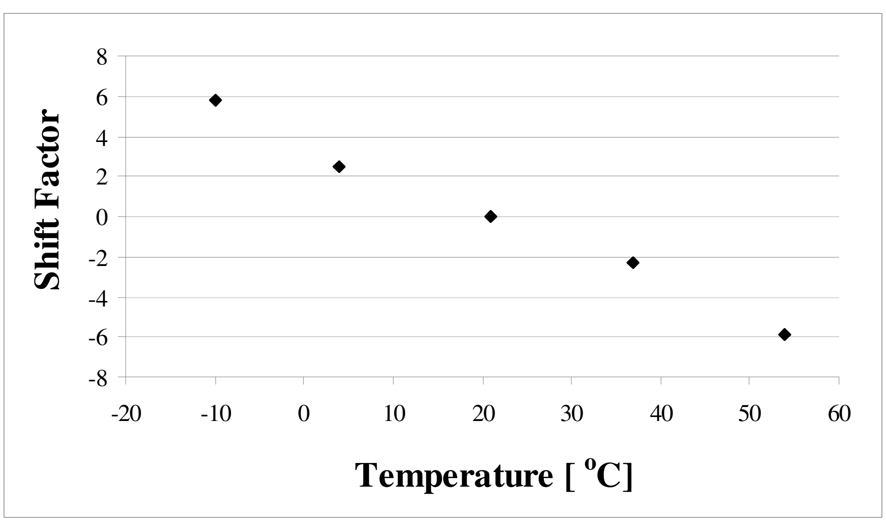


Figure 15: Shift factors for HMA master curve

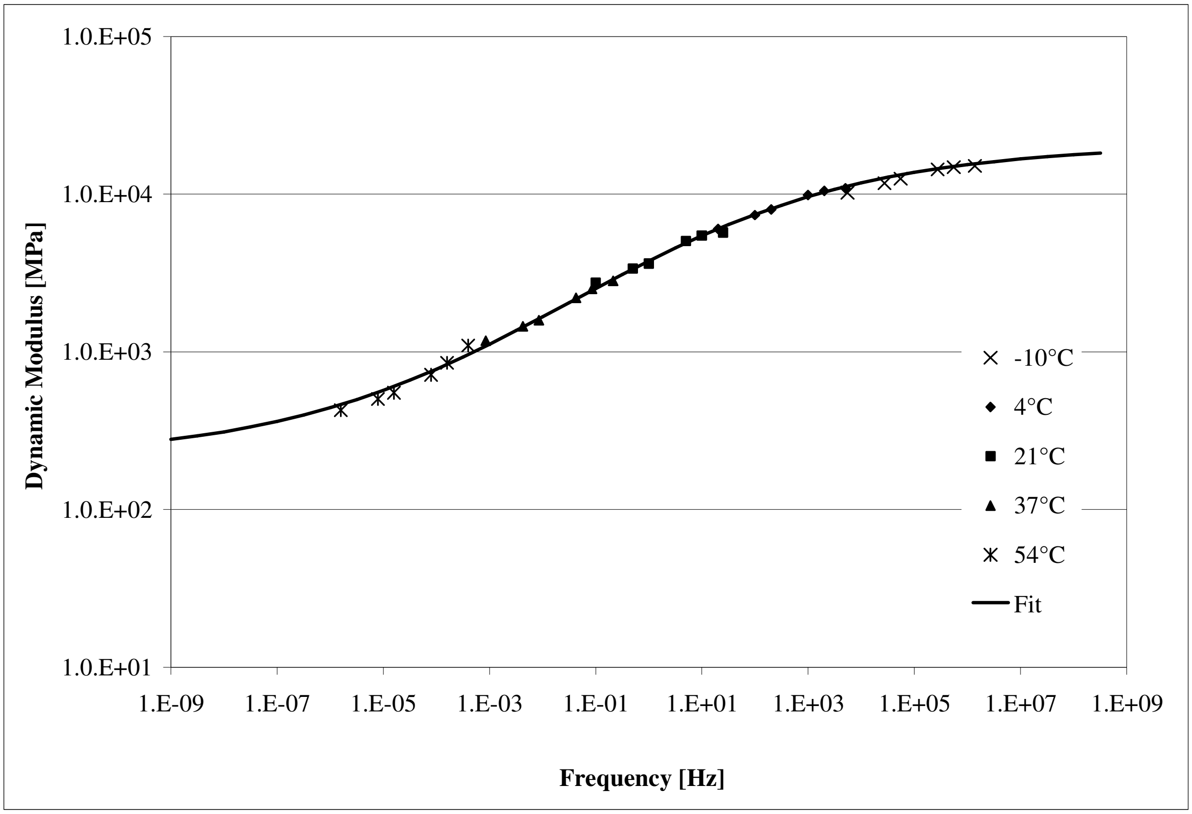


Figure 16: Master Curve for WMA

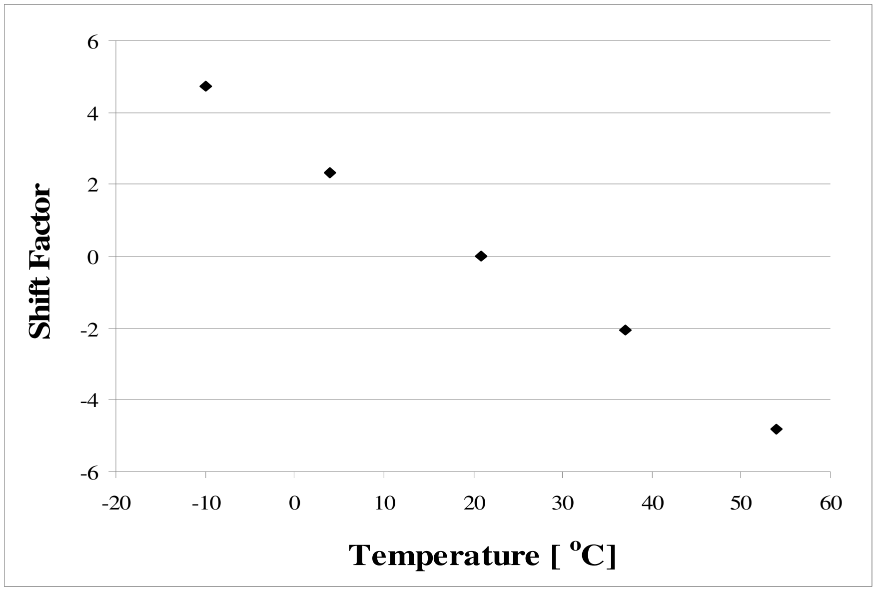


Figure 17: Shift factors for WMA master curve

Based on the shift factors shown in Figure 15 and Figure 17, it can be seen that HMA mix has slightly higher shift factors at the extreme high and low temperatures than with the WMA. This would suggest the HMA is a more temperature dependent mix than the WMA. Again this provides many advantages when placing WMA in spring and fall when temperatures are lower.

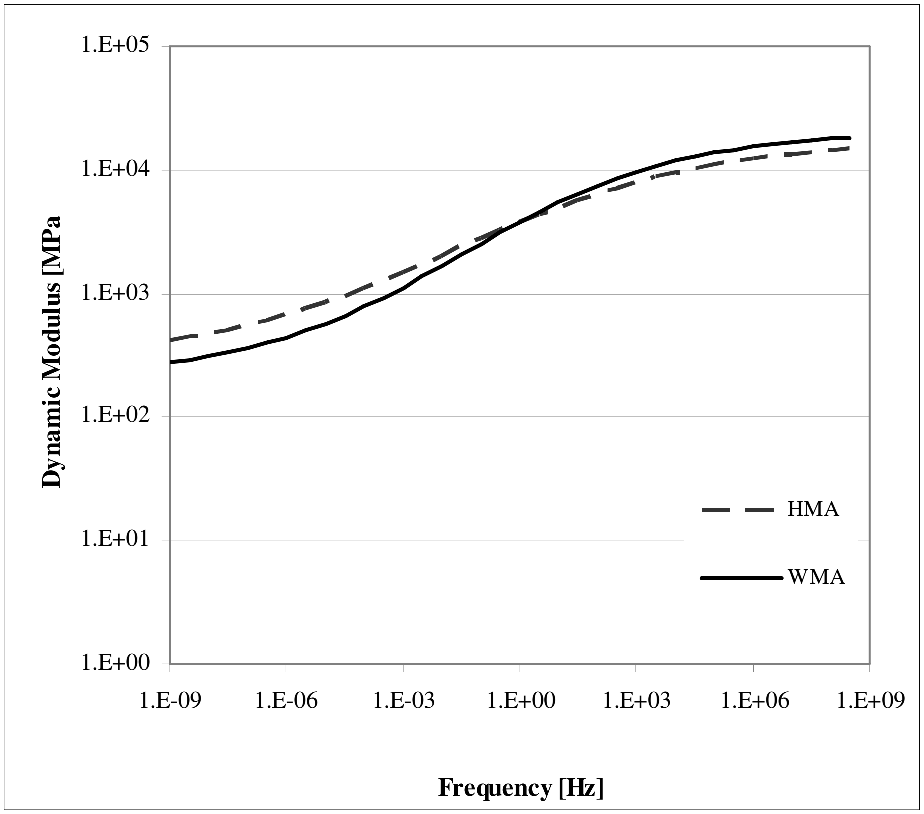


Figure 18: Maser Curve for both HMA and WMA

Figure 18 shows the combined master curves of both HMA and WMA mixes. It can be seen that at a lower frequency or high a temperature, the HMA is stiffer than the WMA and this suggests that the WMA would be more susceptible to rutting than the HMA mix. As well, at high frequency or low temperature, the WMA is stiffer than the HMA which would suggest that the HMA would be more susceptible to fatigue cracking than the WMA mix. At intermediate temperatures, the HMA and WMA appear to behave in a similar manner. However, in both extremes, the dynamic modulus results were consistent with current well performing materials.

F-Test analysis conducted on the dynamic modulus values between the HMA and WMA showed that $F_{calculated} > F_{critical}$ at all temperatures indicating that overall there is a significant variance between the values for HMA and WMA. The predicted differences indicate that the WMA would show slightly better performance at high temperature. The results are summarized in Table 25; complete analysis is in Appendix B.

Table 25: Summary Dynamic Modulus F-Test Results

Test Temperature	Source of Variation	Degree of Freedom (df)	F	P-value	F-critical
-10^{o}C	Between HMA and WMA	5	0.34	0.13	0.20
4.4^{o}C	Between HMA and WMA	5	0.54	0.26	0.20
21^{o}C	Between HMA and WMA	5	0.60	0.30	0.20
37^{o}C	Between HMA and WMA	5	0.61	0.30	0.20
54^{o}C	Between HMA and WMA	5	0.78	0.40	0.20

4.6 Mechanistic-Empirical Pavement Design Guide (MEPDG)

MEPDG software was used to investigate the long-term performance of the HMA and WMA mixes over a 20-year (240-month) design life. The dynamic modulus test results for the HMA and WMA summarized previously were used as a Level 1 input for the surface layer. All other inputs such as climate, traffic data, growth rate, pavement structure, material properties and material gradation were consistent as described in Chapter 3.

The distresses that were obtained from the MEPDG analysis are longitudinal and alligator cracking, as well as rutting and pavement roughness. Results are summarized in Figure 19 to Figure 30.

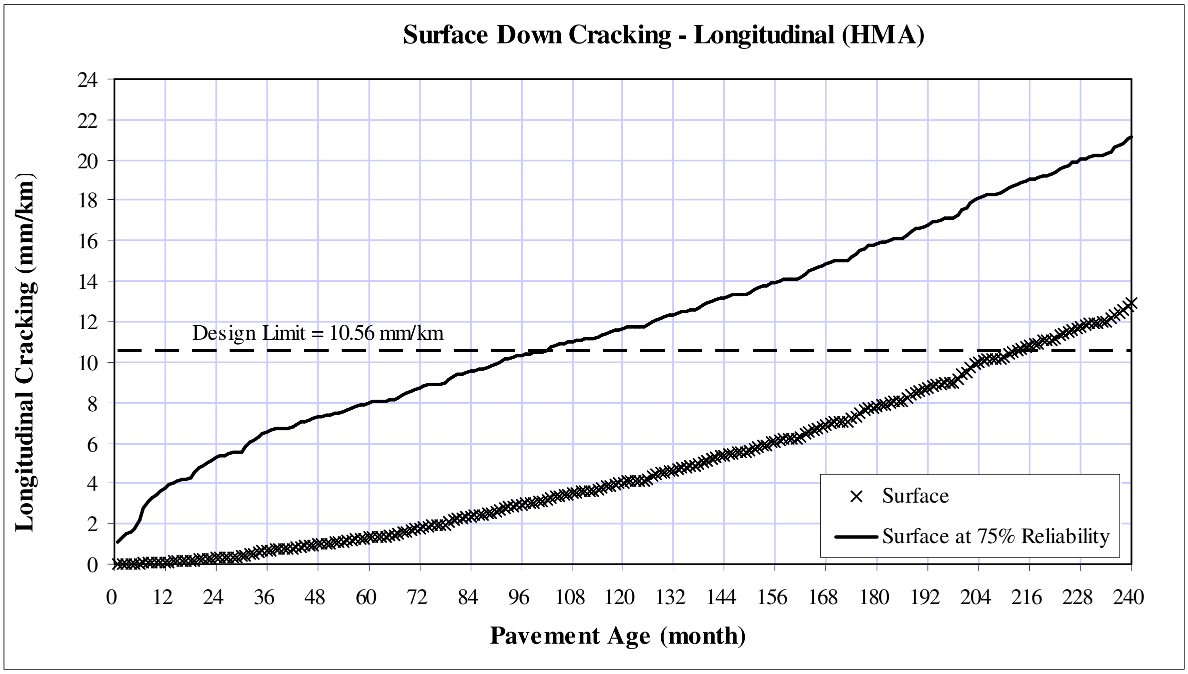


Figure 19: Longitudinal Cracking for HMA

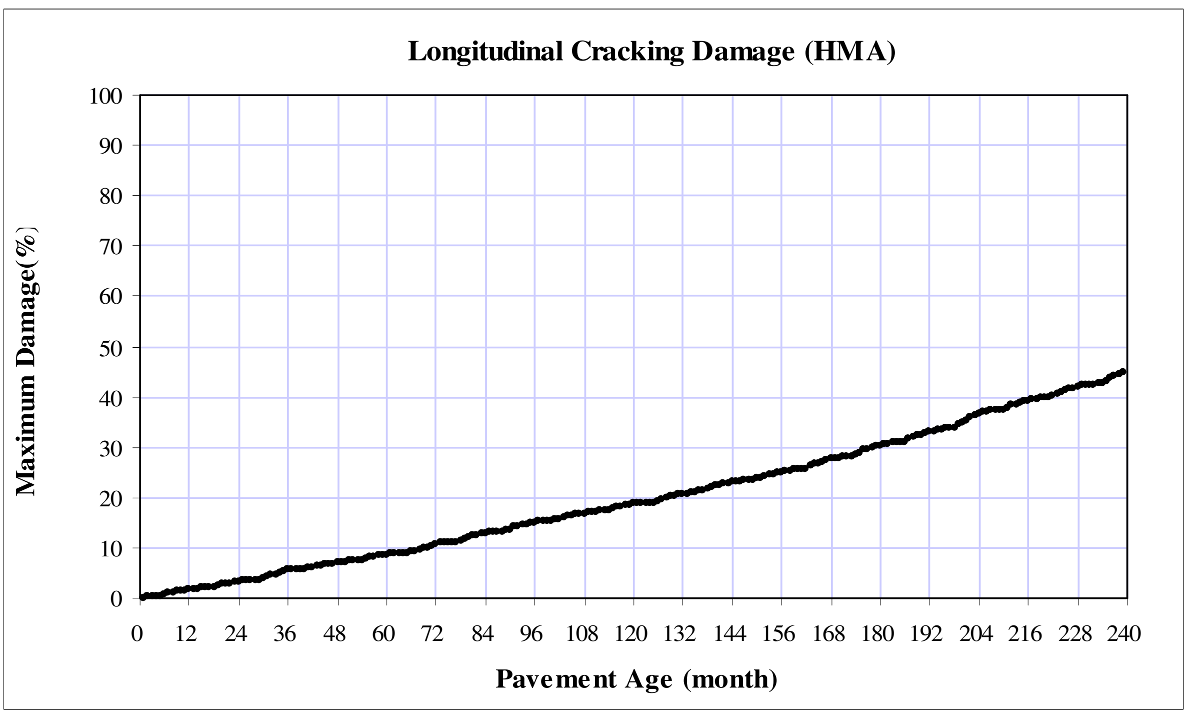


Figure 20: Maximum Damaged from Longitudinal Cracking for HMA

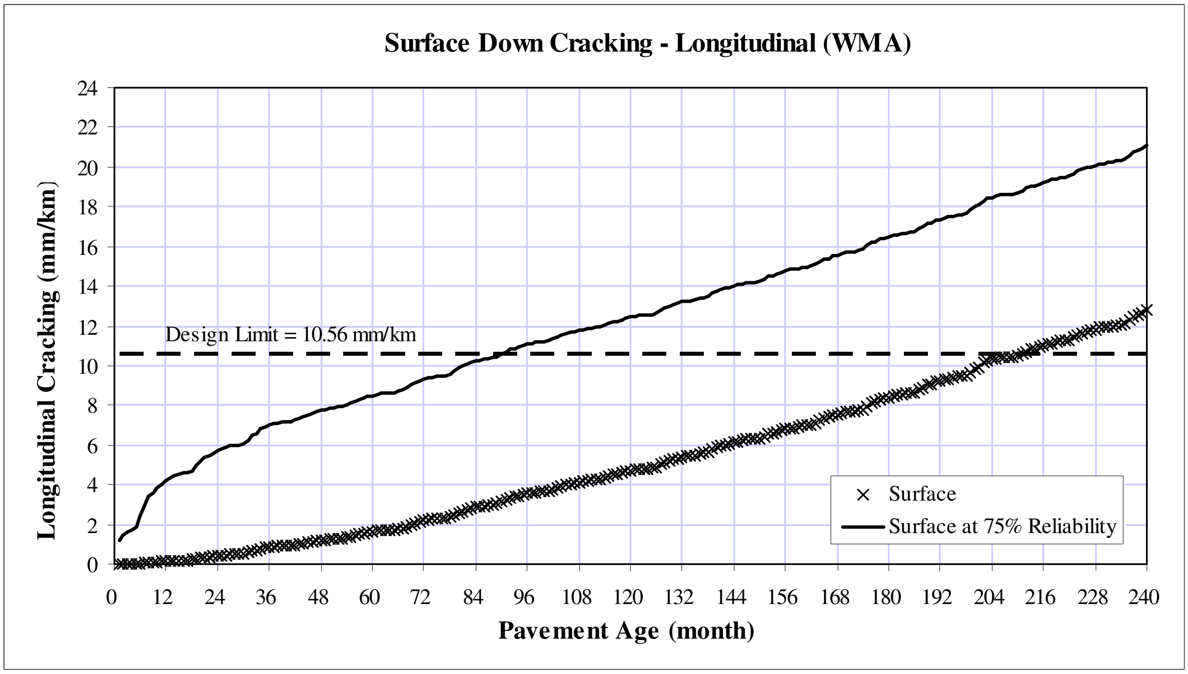


Figure 21: Longitudinal Cracking for WMA

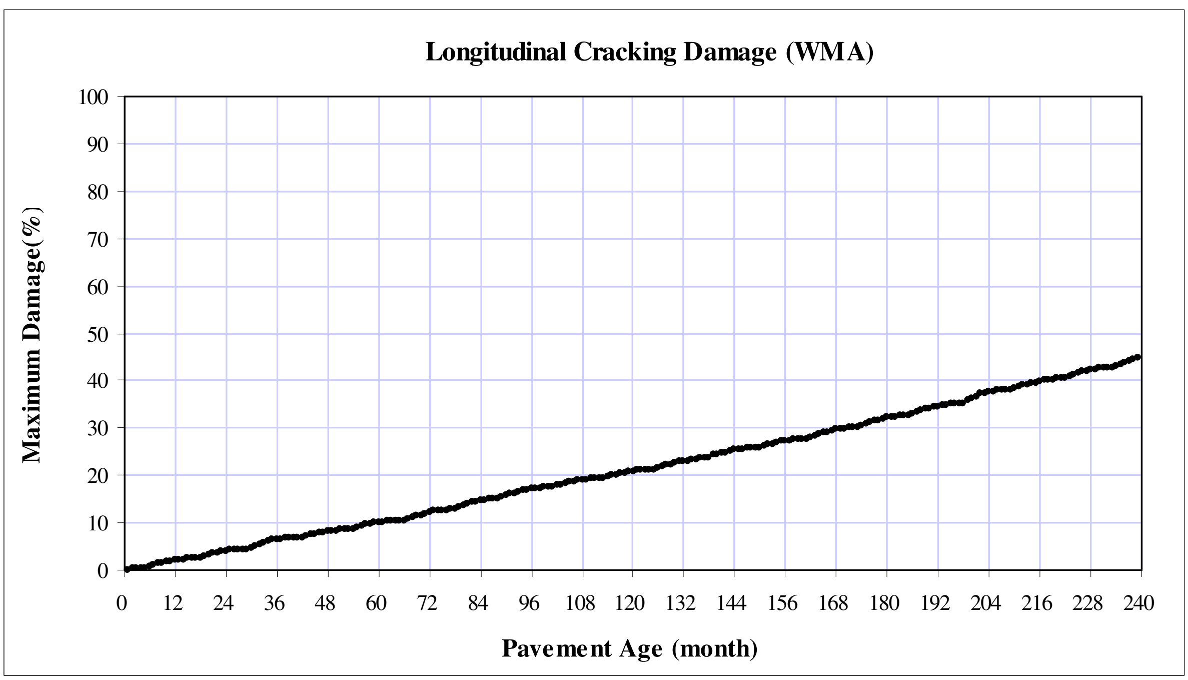


Figure 22: Maximum Damage from Longitudinal Cracking for WMA

Comparing the predicted surface longitudinal cracking propagation for HMA (Figure 19) and WMA (Figure 21), the HMA pavement reaches the design limit of 10.56 mm/km after 17.7 years (212 months). At that point 15.5% of the surface would be experiencing the maximum damage from the longitudinal cracking of the HMA pavement as shown in Figure 20. The WMA pavement reaches the design limit of 10.56 mm/km after 17.5 years (210 months) and at that point 15.6% of the surface would be experiencing the maximum damage from the longitudinal cracking as shown in Figure 22.

The longitudinal cracking reliability is an indication of when pavement rehabilitation should be performed to repair early forms of longitudinal cracking thereby extending the pavement life. Figure 19 and Figure 21 show the longitudinal cracking at 75% reliability and Figure 23 shows a comparison of both pavements for longitudinal cracking. The WMA would need to be repaired after 90 months, and the HMA would need to be repaired after 101 months for longitudinal cracking. This information is used in Chapter 5 for the economic analysis.

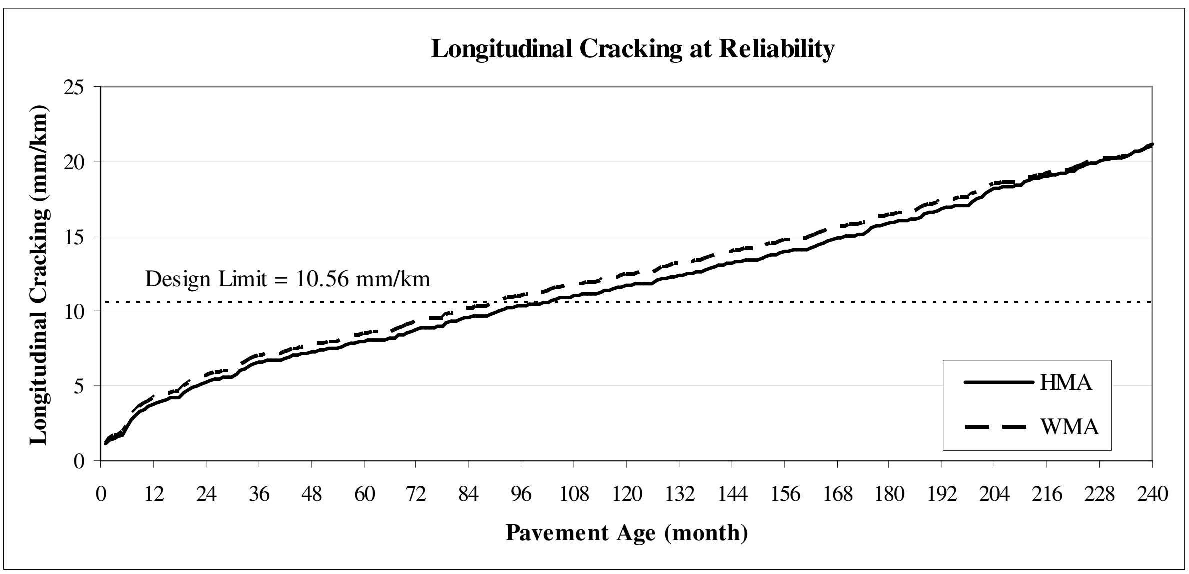


Figure 23: HMA and WMA Longitudinal Cracking at Reliability

F-Test analysis conducted on the longitudinal cracking at reliability between the HMA and WMA is shown in Table 26. The results showed that $F_{calculated} < F_{critical}$ indicating that the HMA and WMA are statistically the same.

Table 26: F-Test Results for Longitudinal Cracking

	HMA	WMA
Mean	11.85	12.39
Variance	25.11	24.78
Observations	240	240
df	239	239
F	1.01	
P(F<=f) one-tail	0.46	
F Critical one-tail	1.24	

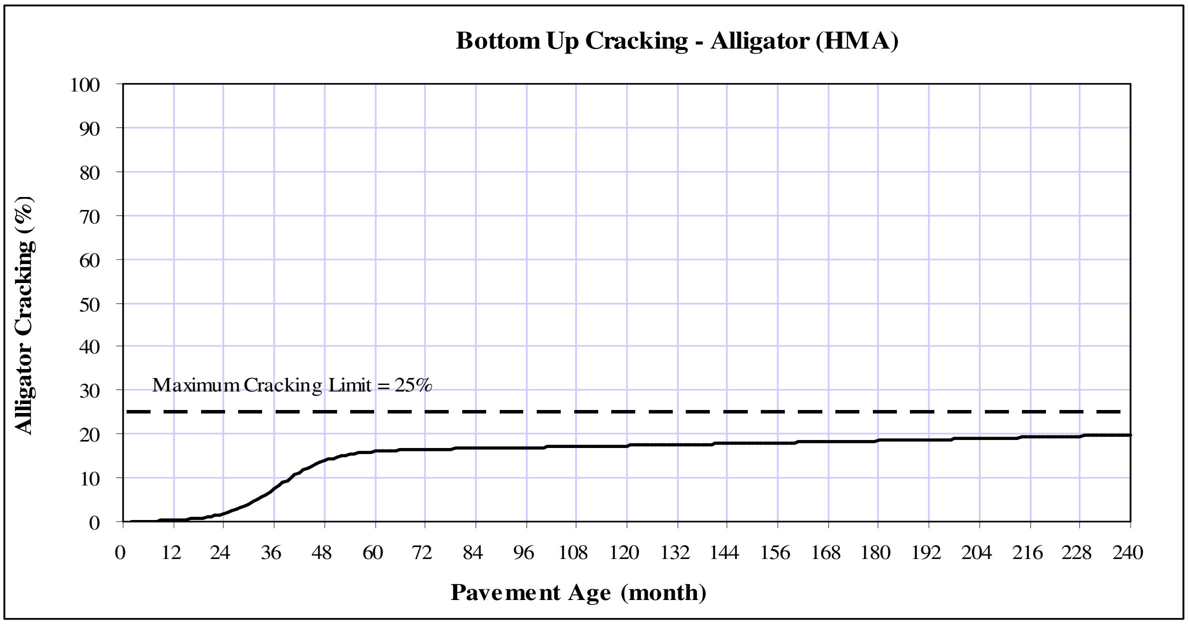


Figure 24: Alligator Cracking Prediction for HMA

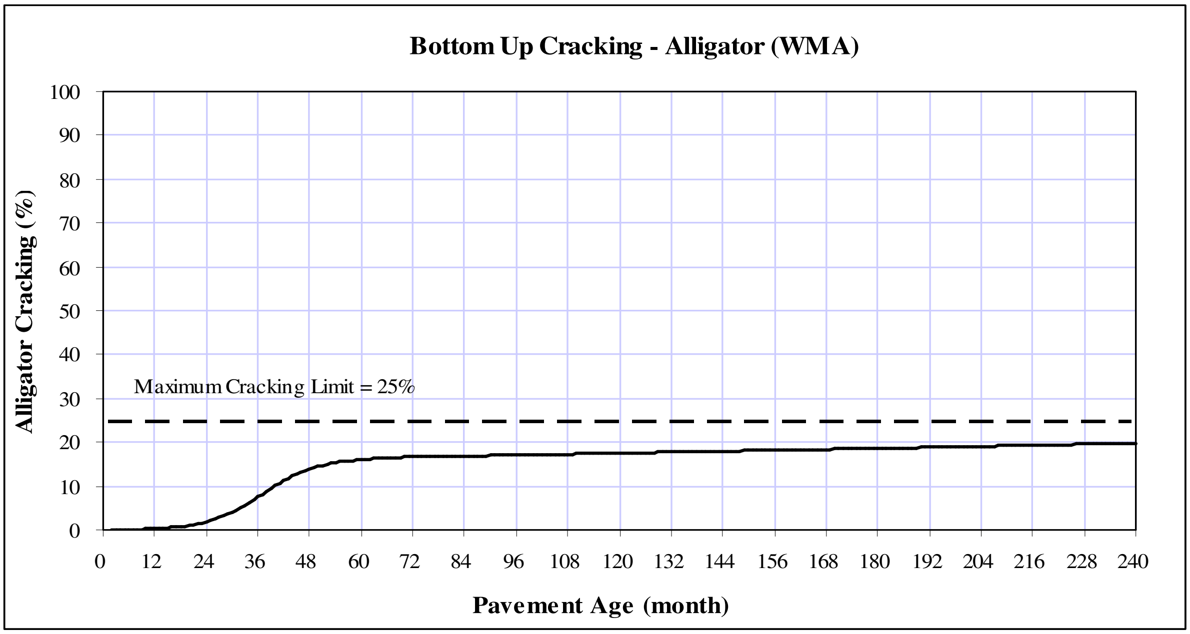


Figure 25: Alligator Cracking Prediction for WMA

Figure 24 and Figure 25 show that although alligator cracking is expected to occur in both the HMA and WMA. However, neither pavement types will reach the maximum cracking limit of 25% to require any form of rehabilitation due to alligator cracking. By the end of the 20-year design life of the pavement, both the HMA and WMA pavements would show 19.8% alligator cracking on the pavement surface.

Figure 26 and Figure 27 show the development of rutting in the HMA and WMA pavements as predicted by the MEPDG software. Both HMA and WMA pavements show that the total rutting, which includes all pavement layers, will not exceed the design limit by the end of the 20-year design life. However the reliability suggests that both pavement types will need to be rehabilitated to repair total rutting before the end of the 20 years to extend the pavement life. Both the HMA and WMA pavements show that a repair will be needed after 16.8 years (201 months). Looking at rutting experienced by the surface asphalt only, the HMA and WMA pavements will need a repair after 11.9 years (143 months) and 11.8 years (142 months) respectively. This is further illustrated in Figure 28 in a comparison of both HMA and WMA rutting of the surface asphalt material.

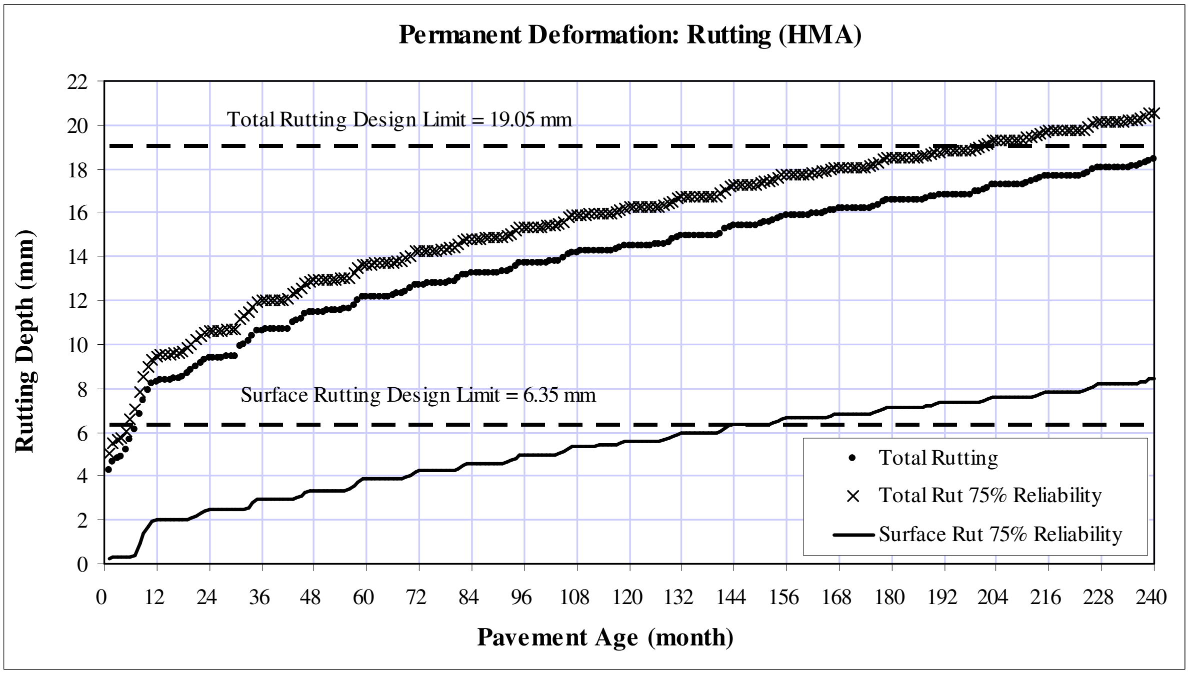


Figure 26: Rutting Prediction for HMA

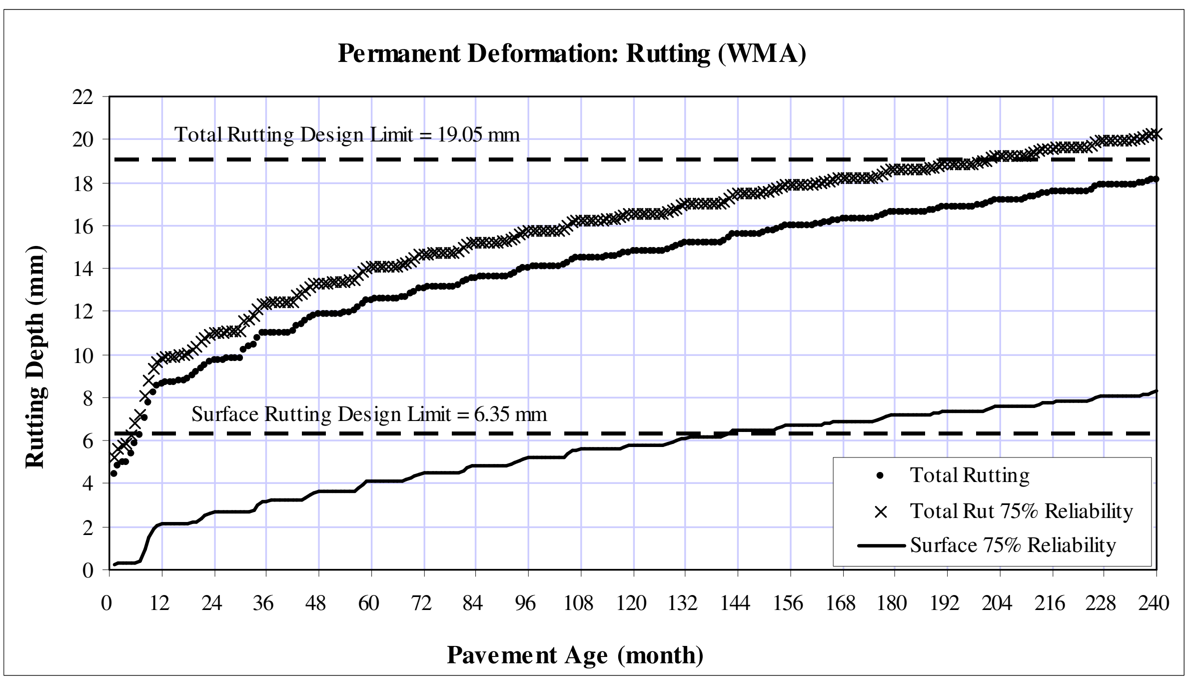


Figure 27: Rutting Prediction of WMA

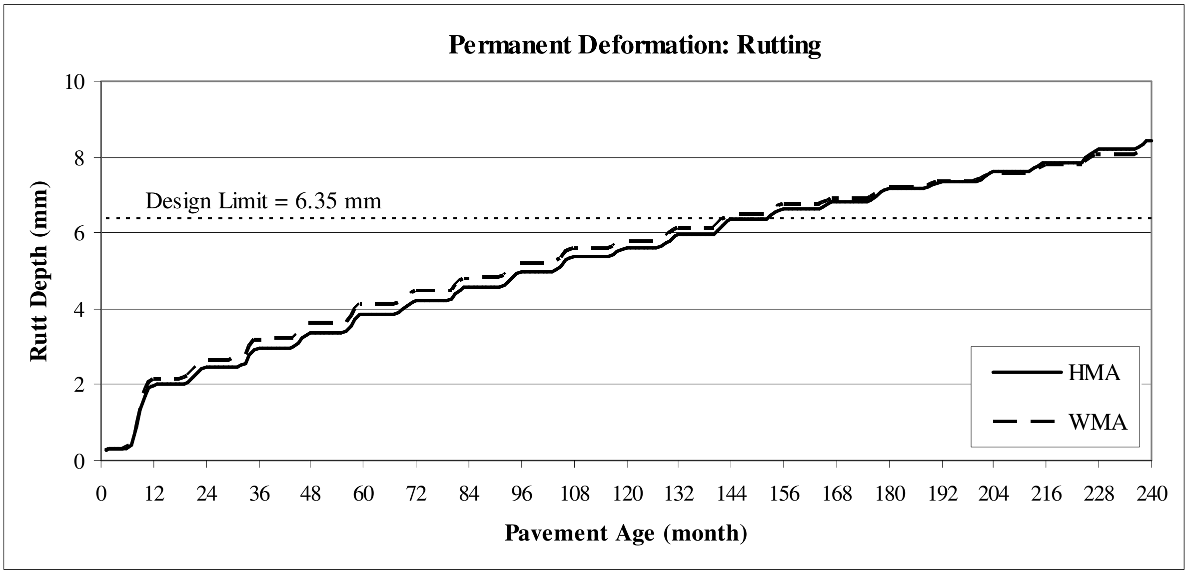


Figure 28: Surface Rutting Reliability Summary for HMA and WMA

The F-Test analysis conducted on the surface rutting at 95% confidence between the HMA and WMA is shown in Table 27. The results showed that $F_{calculated} < F_{critical}$ indicating that there is not a significant difference between the values.

Table 27: F-Test Results for Permanent Deformation: Rutting

	HMA	WMA
Mean	5.29	5.43
Variance	4.29	4.00
Observations	240	240
df	239	239
F	1.07	
P(F<=f) one-tail	0.29	
F Critical one-tail	1.24	

The pavement roughness prediction is determined through the International Roughness Index (IRI), which is a standardized roughness measurement developed by the World Bank. The predicted roughness measurements for the HMA and WMA pavements over the 20-year design life are shown in Figure 29 and Figure 30. Both HMA and WMA pavements show that IRI measurements will not exceed the design limit before the end of the 20 years.

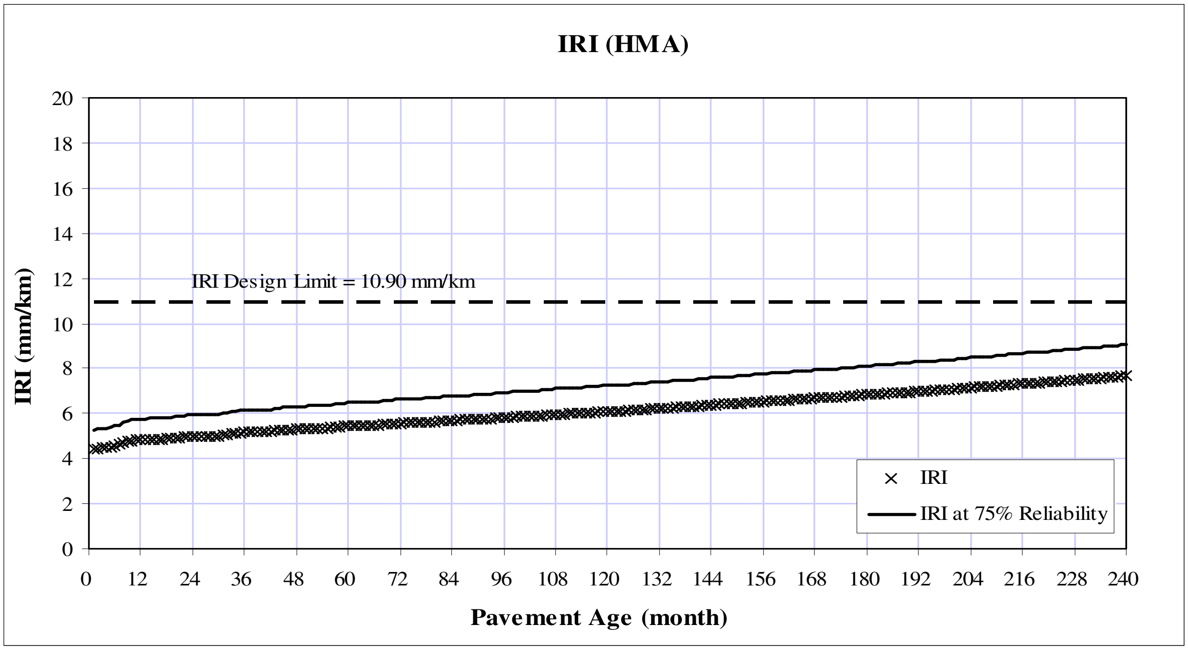


Figure 29: IRI Predicted Measurements for HMA

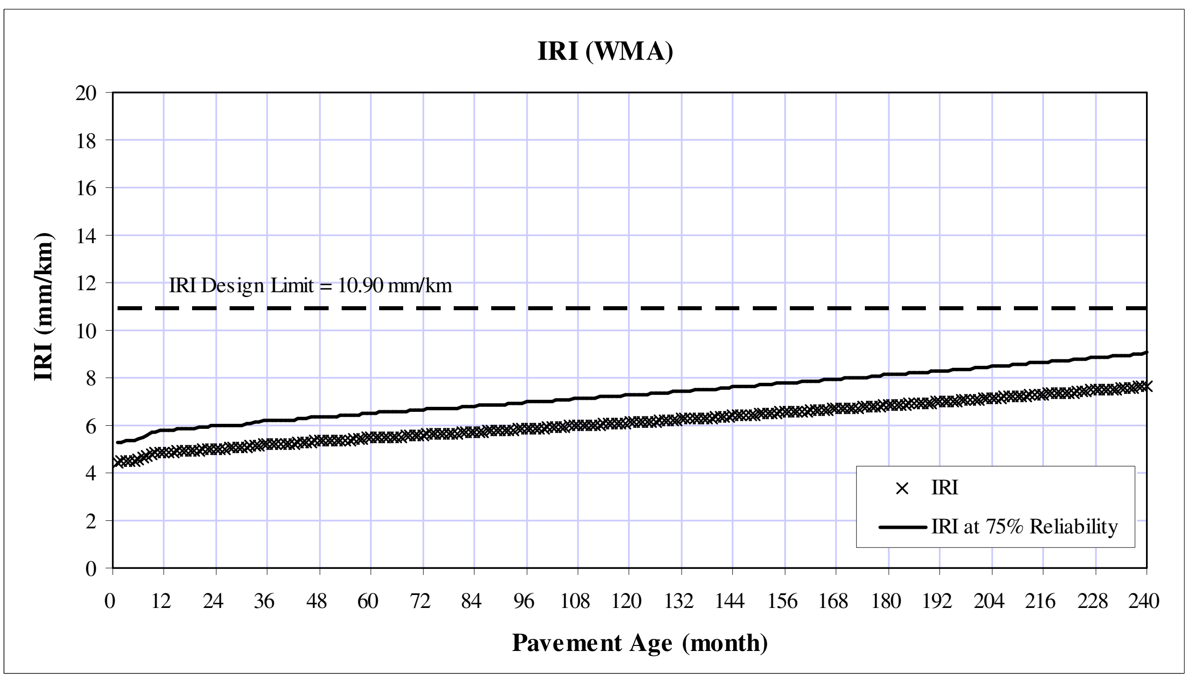


Figure 30: IRI Predicted Measurements for WMA

4.7 Summary

This chapter summarized the laboratory test results conducted on plant produced HMA and WMA samples from the Miller Paving Limited asphalt plant located in Markham.

The air voids testing over the four-week storage period showed that there was no trend in the air voids for either HMA or WMA. However, on average the WMA specimens had lower air voids than the HMA specimens. Single Factor ANOVA showed that the difference in air voids between the HMA and WMA was not statistically significant.

ITS results showed an increase in dry and wet tensile strengths for the WMA mix as the binder aged over the four-week storage time which was statistically significant. The HMA mix did not exhibit the same increase in strength.

Densification testing carried out on the two mixes to determine workability index showed that the difference in workability between the HMA and WMA was not statistically significant. However, the WMA mix consistently had better workability characteristics.

Resilient modulus test results showed that the WMA had higher resilient modulus values which can be attributed to a stronger aggregate skeleton achieved from easier compaction of the WMA mix. However, the difference in the results between the HMA and WMA values were not statistically significant.

Master curves developed from dynamic modulus test results showed that overall HMA was a more temperature dependent mix than the WMA. At high temperatures, WMA showed to be more susceptible to rutting than the HMA mix. At lower temperatures, the WMA was shown to be more susceptible to fatigue cracking than the HMA mix. The Single Factor ANOVA showed that the difference between the HMA and WMA results were statistically significant. However, the absolute values indicated they would be within typical ranges for other similar asphalt pavements used on highway and arterial roads in Ontario.

When the dynamic modulus results were used as a Level 1 input for MEPDG analysis, the HMA pavement reached the design limit for longitudinal cracking at 17.7 years, whereas the WMA pavement reached the design limit for longitudinal cracking propagation after 17.5 years. Neither of the pavement types reached the maximum design limits for alligator cracking, or pavement roughness in the 20-year pavement design life. In terms of rutting, both HMA and WMA pavements showed that total rutting would not exceed the design limit in the 20-year design life. However, rehabilitation would be needed on the HMA surface asphalt at 11.9 years and on the WMA pavement at 11.8 years in order to ensure reliability of the pavement over the 20 years. A Single Factor ANOVA showed that the difference between the HMA and WMA results for longitudinal cracking as well as permanent deformation were not statistically significant.

Chapter 5
Economic Analysis

5.1 Introduction

Maintenance programs are a critical part of any pavement structure's design life. A maintenance program is needed in order to slow down the rate of pavement deterioration and to delay the need for costly pavement rehabilitation for years [Zimmerman 2011]. The program generally includes such treatments as crack sealing, micro-surfacing, chip seals, and asphalt overlays. The maintenance treatments contribute directly to the overall pavement performance by improving the condition of the pavement and extending the pavement service life as shown in Figure 31 [Chong 1989, Zimmerman 2011].

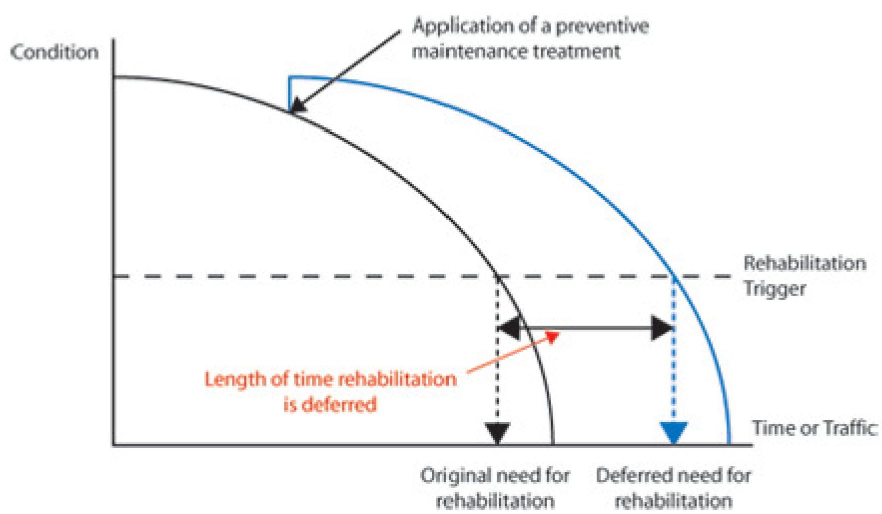


Figure 31: Effect of Pavement Maintenance on Service Life [Zimmerman 2011]

The economic analysis performed as part of this research compared the costs associated with the maintenance of the HMA and WMA pavement types over a 20-year design life. The purpose of this was to ensure that all costs involved with construction, maintenance and rehabilitation of both pavement types were considered in the life cycle assessment. The maintenance schedule was determined using the results of the predictive models generated from the MEPDG software in Chapter 4.

To determine the types of maintenance to apply to the HMA and WMA pavements, the MTO Manual for Condition Rating of Flexible Pavements was used in conjunction with the distresses that were generated from the MEPDG software. The MTO manual categorizes the distress manifestations into three groups: surface defects, permanent deformation, and cracking. The distresses generated from the MEPDG software for the HMA and WMA pavement types for this research fall under the permanent deformation (rutting), and cracking (longitudinal and alligator cracking) groups.

In the manual, the distresses are categorized into the density and severity of the occurrence of the distress. The manual also provides probable causes for the distresses and possible remedial measures to slow down the rate of deterioration of the pavement [MTO 1989].

Severity is determined by measuring "how bad" the distresses are and it is mostly based on past engineering experience and is described as follows: Very Slight, Slight, Moderate, Severe, and Very Severe [MTO 1989]. For the purpose of this research, the severity of the distresses were assumed to be moderate.

Density is also based on past engineering experience that is used to determine "how big" the distress is. The density of the distress is described as follows: few, intermittent, frequent, extensive, and throughout. Guidelines are provided in the manual to classify the various distress levels as follows:

Table 28: Guidelines for Distress Density [MTO 1989]

Class	Guidelines
Few	Less than 10% of pavement surface affected
Intermittent	10 to 20% of pavement surface affected
Frequent	20 to 50% of pavement surface affected
Extensive	50 to 80% of pavement surface affected
Throughout	80 to 100% of pavement surface affected

The results generated from the MEPDG software regarding the distress manifestation also includes the amount of damage caused by the type of distress and this was used as the density of the distresses for the purpose of this research.

5.2 Life Cycle Cost Analysis

Using the information from the MEPDG and the MTO manual, it was determined that for the longitudinal cracking for both HMA and WMA pavements, the severity and density of the distress warrants a rout and seal as the treatment. For the permanent deformation (rutting) experienced by both the HMA and WMA pavements, chip seal was the treatment alternative recommended as per the MTO manual for severity and density of rutting experienced by both pavement types.

A life cycle cost analysis (LCCA) was performed to determine the present value of all costs associated with the HMA and WMA pavements over a 20-year design period. Table 29 outlines the cost in Canadian dollars of each material used in either the initial construction or maintenance of the pavement for a 30-km section. These costs are estimated values obtained from Miller Paving Limited. Fuel cost was obtained from the MTO’s Registry, Appraisal and Qualification System (RAQS) fuel price for the month of October 2010 when the field section Highway 62 project was paved as discussed in Chapter 6.

Table 30 and Table 31 outline the schedule of activities and the year they occur for both the HMA and WMA pavements respectively.

Table 29: Material and Maintenance Cost

Material	$/unit
HMA	$ 58.61/tonne
Plant Fuel Consumption for HMA	0.157 L/tonne
WMA	$ 63.61/tonne
Plant Fuel Consumption for WMA	0.117 L/tonne
Fuel	$ 82.90/L
Rout and Seal	$ 4/m
Chip Seal	$ 7/m

Table 30: HMA Schedule of Maintenance Activities

Year (HMA)	Activity	Quantity
0	Initial Construction	16800 tonnes
8	Rout and Seal	4,500 m
11	Chip Seal	28,200 m
16	Rout and Seal	4,500 m

Table 31: WMA Schedule of Maintenance Activities

Year (WMA)	Activity	Quantity
0	Initial Construction	16800 tonnes
7	Rout and Seal	4,410 m
11	Chip Seal	29,100 m
14	Rout and Seal	4,410 m

Using the information from the above tables, a complete LCCA was performed for both the HMA and WMA pavements using a 5% discount rate for a 20-year analysis period. The results for the HMA and WMA pavements are shown in Table 32 and

Table 33 respectively. The results show over a 20-year period, the WMA pavement would cost $ 32,991 more for construction and maintenance than the HMA pavement.

Table 32: HMA Pavement Life Cycle Cost

Year	Activity	Quantity	$/unit	Cost	Present Value
0	Initial Construction	16,800 tonnes	$ 58.61/tonne	$ 1,203,305	$ 1,203,305
8	Rout and Seal	4,500 m	$ 4/m	$ 18,000	$ 12,183
11	Chip Seal	28,200 m	$ 7/m	$ 197,400	$ 115,416
16	Rout and Seal	4,500 m	$ 4/m	$ 18,000	$ 8,246
Total Net Present Value					**$ 1,339,150**

Table 33: WMA Pavement Life Cycle Cost

Year	Activity	Quantity	$/unit	Cost	Present Value
0	Initial Construction	16,800 tonnes	$ 63.61/tonne	$ 1,231,596	$ 1,231,596
7	Rout and Seal	4,410 m	$ 4/m	$ 17,640	$ 12,536
11	Chip Seal	29,100 m	$ 7/m	$ 203,700	$ 119,009
14	Rout and Seal	4,410 m	$ 4/m	$ 17,640	$ 8,909
Total Net Present Value					**$ 1,372,141**

The discount rate of 5% was selected based on current practice. However, the analysis was also run at 3% and 7%. The results for the HMA are shown in Table 34 and Table 35 and the results for the WMA are shown in Table 36 and Table 37.

Table 34: HMA LCCA at 3% Discount Rate

Year	Activity	Quantity	$/unit	Cost	Present Value
0	Initial Construction	16,800 tonnes	$ 58.61/tonne	$ 1,203,305	$ 1,203,305
8	Rout and Seal	4,500 m	$ 4/m	$ 18,000	$ 14,209
11	Chip Seal	28,200 m	$ 7/m	$ 197,400	$ 142,606
16	Rout and Seal	4,500 m	$ 4/m	$ 18,000	$ 11,217
Total Net Present Value					**$ 1,371,337**

Table 35: HMA LCCA at 7% Discount Rate

Year	Activity	Quantity	$/unit	Cost	Present Value
0	Initial Construction	16,800 tonnes	$ 58.61/tonne	$ 1,203,305	$ 1,203,305
8	Rout and Seal	4,500 m	$ 4/m	$ 18,000	$ 10,476
11	Chip Seal	28,200 m	$ 7/m	$ 197,400	$ 93,783
16	Rout and Seal	4,500 m	$ 4/m	$ 18,000	$ 6,097
Total Net Present Value					**$ 1,313,662**

Table 36: WMA LCCA at 3% Discount Rate

Year	Activity	Quantity	$/unit	Cost	Present Value
0	Initial Construction	16,800 tonnes	$ 63.61/tonne	$ 1,231,596	$ 1,231,596
7	Rout and Seal	4,410 m	$ 4/m	$ 17,640	$ 14,343
11	Chip Seal	29,100 m	$ 7/m	$ 203,700	$ 147,157
14	Rout and Seal	4,410 m	$ 4/m	$ 17,640	$ 11,662
Total Net Present Value					**$ 1,404,758**

Table 37: WMA LCCA at 7% Discount Rate

Year	Activity	Quantity	$/unit	Cost	Present Value
0	Initial Construction	16,800 tonnes	$ 63.61/tonne	$ 1,231,596	$ 1,231,596
7	Rout and Seal	4,410 m	$ 4/m	$ 17,640	$ 10,985
11	Chip Seal	29,100 m	$ 7/m	$ 203,700	$ 96,776
14	Rout and Seal	4,410 m	$ 4/m	$ 17,640	$ 6,841
Total Net Present Value					**$ 1,346,199**

5.3 Summary

This chapter presented an economic analysis of the HMA and WMA pavement types involved in this research by comparing an LCCA over a 20-year design life in an effort to take into account all costs including future rehabilitation and maintenance costs associated with the two pavement types. Pavement distresses obtained from the MEPDG results in Chapter 4 were used in conjunction with the MTO Manual for Condition Rating of Flexible Pavements to determine predicted pavement distresses that would be expected over the 20-year design life and the expected treatment alternatives that would be required to maintain the pavement at a safe standard.

It was determined that over the 20-year design life, the HMA pavement would cost $ 1,339,149 while the WMA pavement would cost $ 1,372,141, both in terms of Net Present Value. Therefore the HMA pavement is the less expensive alternative, costing $ 32, 991 less than the WMA pavement over the 20-year design life. However, as noted, some of the improved placement properties particularly at low temperatures which are experienced in spring and fall might override this cost difference if the paving season can be extended. If paving can be carried out still when temperatures get lower while maintaining a high quality product, this is desirable. Also in situations where the asphalt plant is located far away from the construction site, it is desirable as the HMA will lose heat resulting in lower temperature on site and more difficult ability to compact it and achieve a high quality material.

Chapter 6
Highway 62 Field Trial

6.1 Project Description

A section of Highway 62 was paved in the Township of Tudor and Cashel, which is approximately 37km north of Madoc in Ontario by Miller Paving Limited in October 2011. The project consisted of grading and HMA and WMA paving on Highway 62 from Highway 7 northerly for 30.7 km as shown in Figure 32.

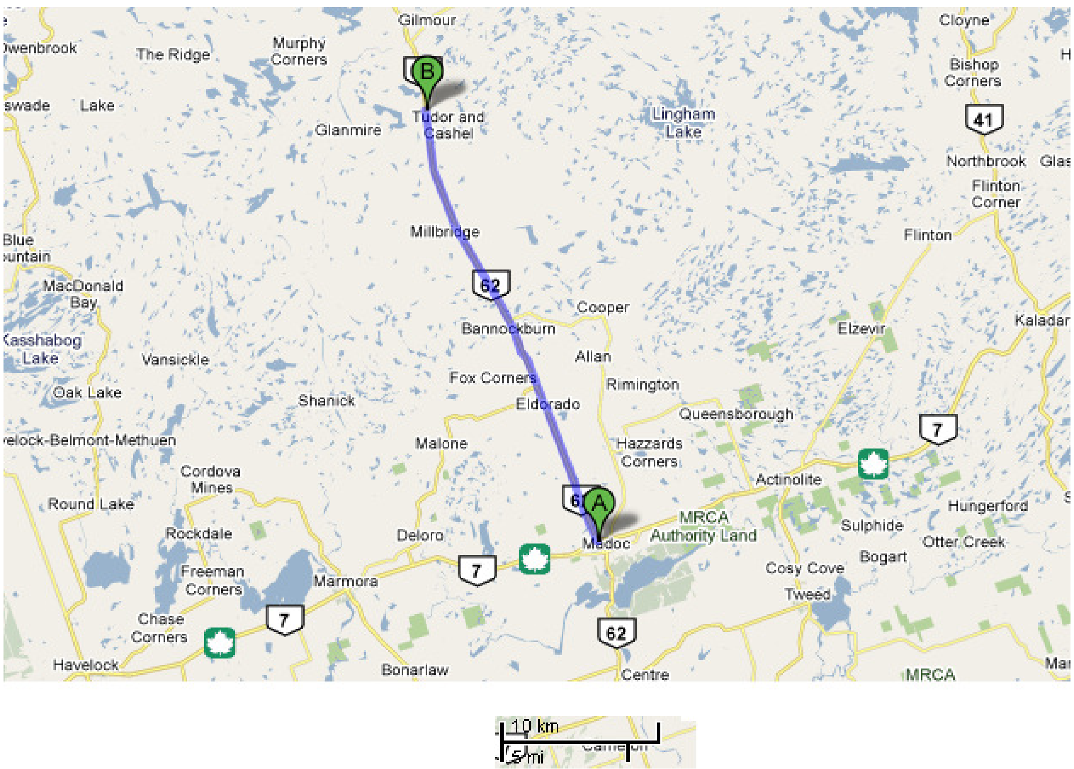


Figure 32: Map of Highway 62 project courtesy of Google Maps

The project used Superpave 12.5mm mix designed for Traffic Category C or 3 to 10 million Equivalent Single Axle Loads (ESALS) [OPSS 2010] for both the HMA and WMA sections on Highway 62. HMA was initially placed as padding at varying depths as required over the section followed by WMA as the surface material, which was compacted to a thickness of 50mm.

Approximately 1,800 tonnes and 15,000 tonnes of HMA and WMA were placed, respectively. The WMA technology used in this project was the Evotherm DAT system at a dosage rate of 0.5% by weight of asphalt cement.

6.2 Mix Design and Volumetric Properties

The mix used on Highway 62 was designed using the Superpave methodology with a gyratory compactor. The mix was designed at the Miller Materials Research Laboratory and a summary of the materials used in the mix design and volumetric properties are summarized in Table 38 and Table 39. There are some small differences in the percentages of Coarse aggregate or HL-3 Stone and manufactured sand in order to meet the required volumetric properties for the WMA due to the addition of the Evotherm (DAT).

Table 38: Mix Design Composition for HMA and WMA

Material	Source	HMA Proportions [%]	WMA Proportions [%]
HL-3 Stone	Carden	43	40.3
Asphalt Sand	CBM	13	13
Manufactured Sand	Carden	24	26.7
RAP[1]	Miller (Whitby)	20	20
PG 58-34P[2]	McAsphalt	4.7	4.7
Evotherm (DAT)	McAsphalt	--	0.5

Note: 1. RAP is Recycled Asphalt Pavement
2. Asphalt binder includes a polymer

Table 39: Volumetric Properties for HMA and WMA

Property		OPSS 1151 Specification	SP 12.5mm HMA	SP 12.5mm WMA
Mixing Temperature [^{o}C]		N/A	155	120 – 125
Compaction Temperature [^{o}C]		N/A	145	112
N_{des} [%Gmm]		96.0	96.0	96
N_{ini} [%Gmm]		</= 89.0	89	89
N_{max} [%Gmm]		</= 98.0	97.3	96.5
Air Voids at N_{des} [%]		4.0	4.0	4.0
VMA [%]		14.0	14.5	14.8
VFA [%]	Minimum	65.0	72.5	73.0
	Maximum	75.0		
Dust Proportion [%]	Minimum	0.6	0.90	0.85
	Maximum	1.2		
Tensile Strength Ratio [%]		Min 80.0%	82.4	82.5
Asphalt Film Thickness [%]		N/A	8.3	8.7

6.3 Mix Production and Placement

Production of the HMA and WMA took place over the course of two months. Over the two months period, the HMA was paved over a period of a week while the WMA was placed over a period of three weeks. Both HMA and WMA mixes were produced at the Miller Mosport plant. This plant is approximately 140km from the job site. The asphalt trucks took on average 2 hours to travel from the plant to the job site. This is a very long period considering the need for retaining the temperature of the asphalt in order to achieve quality in the field.

Both HMA and WMA mixes were placed by a conventional paver and compacted by a 12 ton breakdown vibratory roller, followed by a 20 ton pneumatic tired roller, and a12 ton static steel finish roller.

6.4 Infrared Thermal Imaging Results

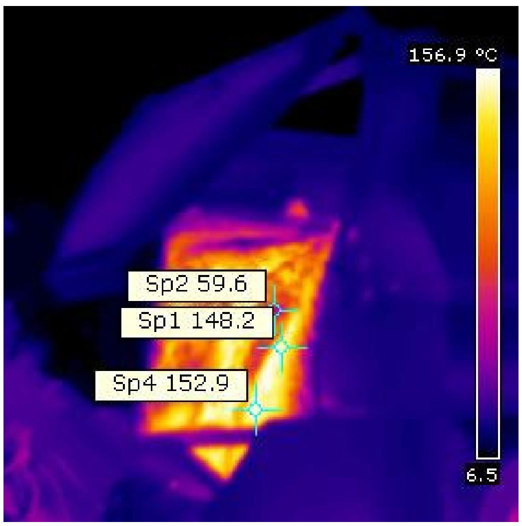
Every object, person or animal emits infrared radiation. Thermal imaging works by capturing this infrared radiation. To track production and placement temperatures of the two mixes, a thermal imaging infrared camera was used to record temperatures at various states of production and placement in this research. The results of the temperature reading are summarized in the following subsections for both HMA and WMA mixes. This was determined to be important in order to compare the HMA and WMA. The camera that was selected was a FLIR Systems b-series thermal imaging camera.

6.4.1 Hot Mix Asphalt Thermal Results

On the day of production of the HMA, the ambient air temperature was 10^{o}C, and it was partly cloudy and no wind. The first truck left the asphalt plant with HMA at 9:00am and the discharge temperature of the HMA was 150^{o}C, as shown in Figure 33, which is consistent with the recommended production temperature of 155^{o}C for the asphalt binder PG58-34P, with a polymer.

Figure 33: Discharge of HMA from silo at 150^{o}C

The truck was monitored for almost 2 hours to the job site at which point the HMA was discharged into a material transfer vehicle (MTV). The purpose of an MTV is to transfer the asphalt material from the truck to the paver while remixing it to prevent temperature and material segregation. The temperature distribution of the HMA as it was transferred into the MTV is shown in Figure 34. The temperature of the HMA ranged from as low as 59.6°C to as high as 157.7°C. The image shows several hot and cold spots in the HMA which was a result of the haulage over the 140km distance in the 10°C weather.



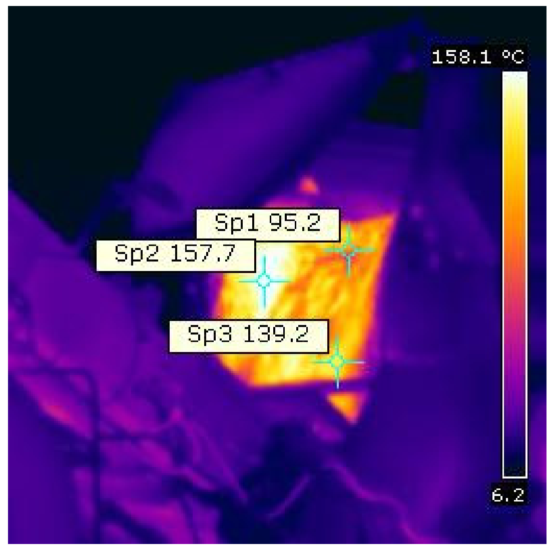


Figure 34: HMA transferred from truck to MTV

After the material was remixed in the MTV, another temperature profile was taken of the HMA and is shown in Figure 35 below.

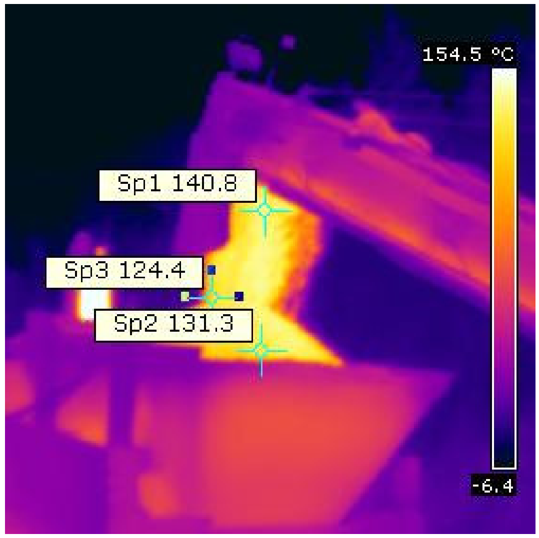


Figure 35: HMA after remixing in MTV

It can be seen that there is less temperature segregation in the material after remixing in the MTV, with a range in temperature of about 124°C to 140°C as the material is transferred into the paver (an average of 132°C). Therefore, there was a temperature loss of about 12% in the HMA material from the asphalt plant to the job site.

Temperature of the HMA, as it was placed, was also recorded and is shown in Figure 36. Temperature of the HMA, as it was placed, ranged from about 123°C to 134°C (an average of 128.5°C), which is a loss in temperature of about 3% as the material traveled through the paver. The HMA mix was compacted immediately.

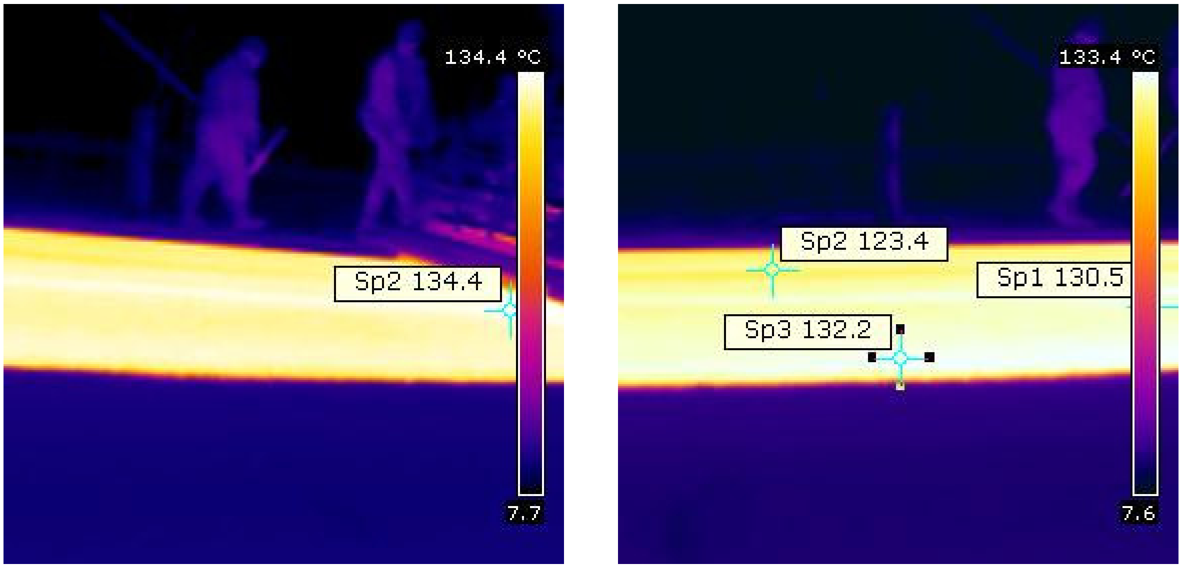


Figure 36: Temperature profile of HMA exiting paver

6.4.2 Warm Mix Asphalt Thermal Results

On the day of paving the WMA, the thermal images were recorded and, the ambient temperature was 13°C with no clouds or wind. An asphalt truck leaving the plant at approximately 9:30am was used for tracking temperatures of the WMA. Figure 37 shows a discharge temperature of 127.2°C which is consistent with the recommended production temperature of 125°C for polymer modified asphalt using the Evotherm DAT solution [Davidson 2010].

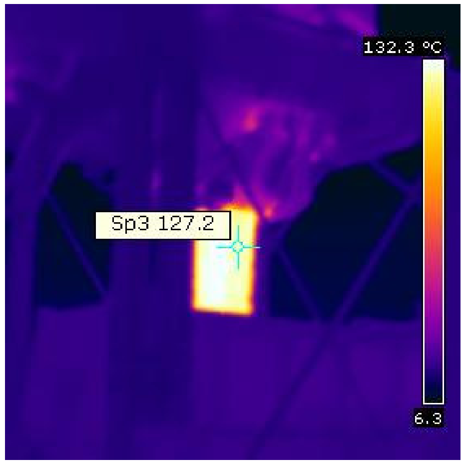


Figure 37: Discharge of WMA from silo at 127.2°C

The asphalt truck was followed to the job site where the mix was transferred into an MTV. The temperature distribution of the material as it was discharged into the MTV is shown in Figure 38. The temperature of the WMA ranged from as low as 114.3°C to as high as 126.9°C. The image shows much less temperature segregation in the WMA than compared to the HMA over the 140km haulage distance.

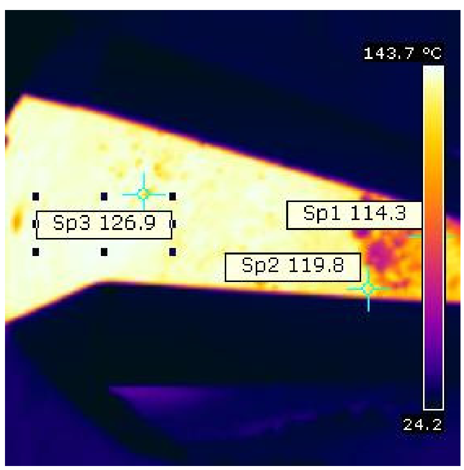


Figure 38: Discharge of WMA into MTV

After the material was remixed in the MTV another temperature profile was taken of the WMA and is shown in Figure 39. The temperature range of the material after remixing in the MTV ranged from approximately 112°C to 124°C as the material was transferred into the paver, giving an average temperature of about 118°C. This is about a 7% reduction in temperature in the mix from the asphalt plant to the job site as compared to the HMA which was 12%.

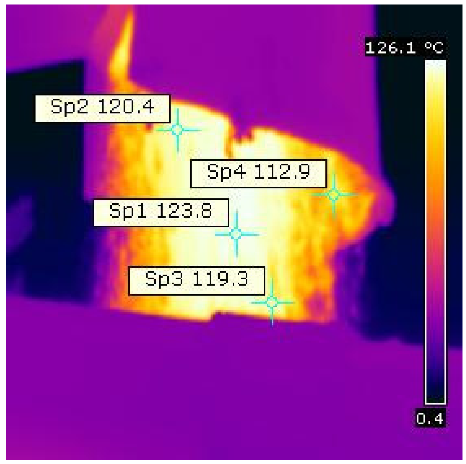


Figure 39: Temperature profile of WMA after remixing in MTV

Temperature of the WMA as it was placed was also recorded and is shown in Figure 40. The same paving and compaction equipment was used for both the WMA and HMA. Placement temperature for the WMA ranged from about 98°C to 108°C, an average of 103°C and a 13% loss in temperature as the material traveled through the paver. The WMA mix was compacted immediately following placement.

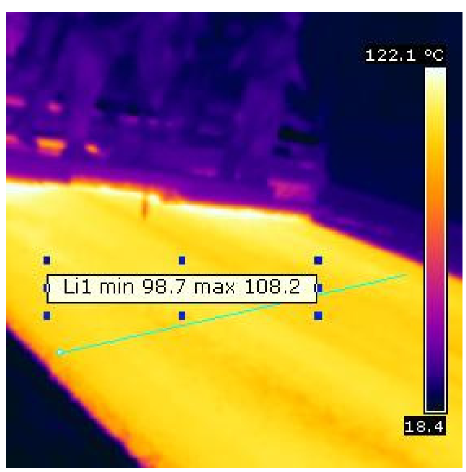


Figure 40: Lay down temperature profile of WMA

6.5 Summary

This chapter examined a field trial by Miller Paving Limited of a 30-km section of Highway 62 that was paved with HMA as padding and WMA as surface. A series of thermal images were taken with an infrared thermal imaging camera to track the temperature of the two mixes at the different stages in the paving process as shown in Table 40. Based on the temperature profiles taken, it was concluded that the WMA material was more effective at maintaining the temperature of the asphalt mixture during the hauling to the jobsite which was located 140km from the asphalt plant. The HMA showed more temperature segregation and hot spots than the WMA during the discharge from the asphalt truck to the MTV. In both mixes the MTV was effective at remixing and reducing temperature segregation of the material.

Table 40: Summary of HMA and WMA thermal analysis

Process	Average HMA Temperature	Average WMA Temperature
Ambient Air Temperature	10°C	13°C
Discharge from Asphalt Plant	150.0°C	127.2°C
At job site (transfer to MTV)	132.0°C	118.0°C
Lay down	128.5°C	103.0°C
Total Temperature drop	21.5°C	24.2°C
Total% Temperature drop	14%	19%

Chapter 7
Conclusions and Recommendations

7.1 Conclusions

Air voids test results showed that although the air voids for the WMA specimens were lower than the HMA specimens over the four-week storage period, the difference in the air voids between the HMA and WMA was not statistically significant.

Over the same storage period, there was a statistical difference in the TSR results of the HMA and WMA. The dry and wet ITS results for the WMA increased over the four-week storage period while the HMA specimens did not show this same increase. It was concluded that this increase in strength can be attributed to asphalt binder aging of the WMA over the storage time. It is expected that this will result in strengthening and improved long term performance of the WMA.

Workability testing was conducted using a workability calculation process and it was determined that the plant produced WMA material had slightly better workability than the HMA material. However, the difference between the two results was not shown to be statistically significant. The workability index of the HMA material at 120^{o}C was attained in the WMA material at 90^{o}C, which makes a significant difference in the field during placement and compaction of asphalt material. In short, WMA can be effectively used at lower spring and fall temperature while HMA can be problematic as it requires higher temperatures which can be challenging under those seasons.

Performance testing results showed that the WMA mix had higher resilient modulus values than the HMA mix. It was concluded that this can be attributed to the stronger aggregate skeleton structure achieved in the mix based on the fact that it is easier compact the WMA. Although the difference between the resilient modulus values of the two mixes was not statistically significant, the values do show improvements.

Master curves developed from dynamic modulus test results showed that overall HMA was a more temperature dependent mix than the WMA. At high temperatures, WMA showed to be slightly more susceptible to rutting than the HMA mix, while at lower temperatures, the HMA showed to slightly more susceptible to fatigue cracking than the WMA mix. F-test analysis conducted for the dynamic modulus results showed that there was a significant difference between the HMA and WMA. However the results were well within the ranges for conventional paving materials.

The MEPDG is currently being adopted by contractors and transportation agencies as was used in the research to predict field performance. This was to ensure the WMA could be easily incorporated. The MEPDG analysis showed that the HMA pavement experienced longitudinal cracking at the design limit after 17.7 years and would need maintenance in the eighth year of service in order to extend the pavement life. The WMA pavement reached the design limit for longitudinal cracking after 17.5 years but would need maintenance in the seventh year of service. In terms of rutting, both HMA and WMA pavements would need rehabilitation in year eleven in order to ensure the pavement reached the 20-year design life. Neither of the pavement types reached the maximum design limits for alligator cracking, or pavement roughness in the 20-year pavement design life. Thus, both were deemed to be structurally adequate.

An economic analysis of the HMA and WMA pavements compared a life cycle cost analysis over a 20-year design life which included all costs associated with the two pavement types over the design life. It was determined that over the 20-year design life, the HMA pavement would cost $ 32, 991 less than the WMA pavement.

A field trial was performed by Miller Paving Limited on Highway 62 in Madoc, Ontario. Based on the temperature profiles taken using a thermal camera, it was concluded that the WMA material was more effective at maintaining the temperature of the asphalt mixture during the hauling to the jobsite which was located 140 km from the asphalt plant. The HMA showed more temperature segregation and hot spots than the WMA during the discharge from that asphalt truck.

Overall the WMA exhibited the same performance properties as the HMA. The life cycle cost analysis showed that WMA was the more expensive pavement because of maintenance treatment for longitudinal cracking that needed to be performed a year sooner than on the HMA pavement. However, having the opportunity to produce the asphalt mix at temperatures over 20^{o}C less than conventional HMA has environmental and health benefits that certainly are advantageous to the asphalt paving industry. Furthermore, as demonstrated in the field trial, WMA provides opportunities for longer hauling distances while maintaining the temperature of the asphalt mixture whereas the temperature segregation was an issue in the HMA mixture. This will allow asphalt paving contractors the opportunity to haul material to jobsites where they otherwise would not have been able to due to temperature lose during longer hauling distances. WMA will also allow the paving season to be extended in Canada into the colder months since compaction can be achieved with WMA at much lower temperatures than with the conventional HMA. It can be argued that the advantages associated with the use of WMA offset the slightly higher cost especially given the current length of the Canadian paving season.

7.2 Recommendations

Based on the findings of this research, the following recommendations are made:

- TSR results should not be the only measure of moisture susceptibility for asphalt specimens, but instead minimum requirements should also be set for both dry and wet ITS values.

- The effect of reduced aging of WMA due to the lower mixing temperature on mixture properties should be explored further.

- Further testing should be conducted to evaluate how different WMA technologies perform with different asphalt cements and aggregate types since the chemistry of the aggregates may affect strength and performance properties, especially WMA technologies that incorporate chemical additives.

- More field trials should be conducted and monitored annually to collect a database of results in order to develop a better correlation between laboratory performance results and field performance.

- A more extensive economic analysis should be performed where the life cycle cost analysis also includes the emissions data taken from the asphalt plant during production of the HMA and WMA and also on the job site during placement and compaction of the mixes.

Appendix A
Dynamic Modulus Test Results

Table 41: HMA Dynamic Modulus Test Results

		E* [MPa]						
Temperature [^{o}C]	**Frequency [Hz]**	**Sample 1**	**Sample 2**	**Sample 3**	**Sample 4**	**Average**	**Standard Deviation**	**Coefficient of Variance**
-10	25	1.35E+04	1.32E+04	1.11E+04	1.33E+04	1.28E+04	1121.68	0.09
	10	1.35E+04	1.31E+04	1.12E+04	1.35E+04	1.28E+04	1074.08	0.08
	5	1.28E+04	1.30E+04	1.11E+04	1.36E+04	1.26E+04	1063.06	0.08
	1	1.15E+04	1.19E+04	9.96E+03	1.26E+04	1.15E+04	1116.85	0.10
	0.5	1.10E+04	1.16E+04	9.59E+03	1.21E+04	1.11E+04	1109.05	0.10
	0.1	9.52E+03	1.06E+04	8.73E+03	1.11E+04	1.00E+04	1076.89	0.11
4.4	25	9.85E+03	8.44E+03	8.18E+03	1.07E+04	9.30E+03	1208.70	0.13
	10	9.34E+03	8.25E+03	7.94E+03	1.02E+04	8.92E+03	1019.18	0.11
	5	8.90E+03	7.88E+03	7.55E+03	9.68E+03	8.50E+03	971.58	0.11
	1	7.31E+03	6.54E+03	6.41E+03	8.15E+03	7.10E+03	803.50	0.11
	0.5	6.74E+03	6.14E+03	6.03E+03	7.76E+03	6.67E+03	790.11	0.12
	0.1	5.55E+03	5.35E+03	5.17E+03	6.60E+03	5.67E+03	638.68	0.11
21	25	5.39E+03	4.95E+03	4.65E+03	5.77E+03	5.19E+03	492.53	0.09
	10	5.13E+03	4.69E+03	4.44E+03	5.61E+03	4.97E+03	516.65	0.10
	5	4.85E+03	4.44E+03	4.13E+03	5.35E+03	4.69E+03	528.41	0.11
	1	3.67E+03	3.52E+03	3.40E+03	4.17E+03	3.69E+03	338.69	0.09
	0.5	3.42E+03	3.23E+03	3.15E+03	3.91E+03	3.43E+03	341.94	0.10
	0.1	2.87E+03	2.76E+03	2.68E+03	3.28E+03	2.90E+03	264.39	0.09
37	25	2.83E+03	2.55E+03	2.30E+03	2.82E+03	2.63E+03	249.22	0.09
	10	2.65E+03	2.50E+03	2.15E+03	2.84E+03	2.54E+03	291.34	0.11
	5	2.42E+03	2.32E+03	2.00E+03	2.64E+03	2.35E+03	266.44	0.11
	1	1.84E+03	1.74E+03	1.58E+03	1.94E+03	1.77E+03	153.27	0.09
	0.5	1.68E+03	1.62E+03	1.48E+03	1.84E+03	1.66E+03	150.32	0.09
	0.1	1.47E+03	1.37E+03	1.27E+03	1.52E+03	1.41E+03	110.32	0.08
54	25	8.7738e	1.20E+03	9.21E+02	1.24E+03	1.12E+03	171.92	0.15
	10	7.69E+02	9.72E+02	8.11E+02	1.15E+03	9.25E+02	172.61	0.19
	5	7.30E+02	8.29E+02	7.36E+02	1.02E+03	8.28E+02	133.85	0.16
	1	5.48E+02	6.09E+02	5.64E+02	7.04E+02	6.06E+02	70.22	0.12
	0.5	5.24E+02	5.83E+02	5.33E+02	6.69E+02	5.77E+02	66.66	0.12
	0.1	4.55E+02	4.89E+02	4.64E+02	5.62E+02	4.92E+02	48.46	0.10

Table 42: WMA Dynamic Modulus Test Results

		Dynamic Modulus E* [MPa]						
Temperature [^{o}C]	**Frequency [Hz]**	**Sample 1**	**Sample 2**	**Sample 3**	**Sample 4**	**Average [MPa]**	**Standard Deviation**	**Coefficient of Variance**
-10	25	1.73E+04	1.29E+04	1.55E+04	1.48E+04	1.51E+04	1.82E+03	1.20E-01
	10	1.69E+04	1.28E+04	1.52E+04	1.45E+04	1.49E+04	1.68E+03	1.13E-01
	5	1.63E+04	1.28E+04	1.46E+04	1.38E+04	1.44E+04	1.47E+03	1.02E-01
	1	1.36E+04	1.18E+04	1.26E+04	1.20E+04	1.25E+04	8.23E+02	6.58E-02
	0.5	1.26E+04	1.13E+04	1.18E+04	1.13E+04	1.17E+04	5.96E+02	5.07E-02
	0.1	1.07E+04	1.03E+04	1.01E+04	9.77E+03	1.02E+04	3.72E+02	3.64E-02
4.4	25	1.21E+04	9.00E+03	1.18E+04	1.07E+04	1.09E+04	1.39E+03	1.28E-01
	10	1.16E+04	9.18E+03	1.09E+04	1.03E+04	1.05E+04	1.01E+03	9.62E-02
	5	1.08E+04	9.06E+03	1.01E+04	9.56E+03	9.89E+03	7.63E+02	7.72E-02
	1	8.72E+03	7.60E+03	8.02E+03	7.69E+03	8.01E+03	5.06E+02	6.32E-02
	0.5	7.98E+03	7.09E+03	7.23E+03	7.20E+03	7.37E+03	4.08E+02	5.54E-02
	0.1	6.38E+03	6.05E+03	5.86E+03	5.79E+03	6.02E+03	2.64E+02	4.38E-02
21	25	5.58E+03	5.75E+03	5.72E+03	5.81E+03	5.71E+03	9.72E+01	1.70E-02
	10	5.28E+03	5.43E+03	5.52E+03	5.64E+03	5.47E+03	1.51E+02	2.76E-02
	5	4.98E+03	5.03E+03	5.04E+03	5.16E+03	5.05E+03	7.73E+01	1.53E-02
	1	3.66E+03	3.73E+03	3.54E+03	3.58E+03	3.63E+03	8.47E+01	2.33E-02
	0.5	3.36E+03	3.50E+03	3.28E+03	3.33E+03	3.37E+03	9.26E+01	2.75E-02
	0.1	2.67E+03	2.90E+03	2.72E+03	2.66E+03	2.74E+03	1.14E+02	4.18E-02
37	25	2.78E+03	2.94E+03	2.90E+03	2.70E+03	2.83E+03	1.10E+02	3.90E-02
	10	2.46E+03	2.59E+03	2.58E+03	2.37E+03	2.50E+03	1.06E+02	4.24E-02
	5	2.20E+03	2.30E+03	2.23E+03	2.07E+03	2.20E+03	9.52E+01	4.33E-02
	1	1.61E+03	1.68E+03	1.60E+03	1.46E+03	1.59E+03	9.09E+01	5.73E-02
	0.5	1.46E+03	1.54E+03	1.45E+03	1.35E+03	1.45E+03	7.71E+01	5.32E-02
	0.1	1.20E+03	1.26E+03	1.17E+03	1.08E+03	1.17E+03	7.66E+01	6.52E-02
54	25	1.05E+03	1.33E+03	1.03E+03	9.90E+02	1.10E+03	1.55E+02	1.41E-01
	10	8.01E+02	1.00E+03	8.04E+02	8.00E+02	8.52E+02	1.00E+02	1.18E-01
	5	6.85E+02	8.34E+02	6.82E+02	6.57E+02	7.15E+02	8.08E+01	1.13E-01
	1	5.14E+02	6.04E+02	5.16E+02	5.70E+02	5.51E+02	4.42E+01	8.02E-02
	0.5	4.76E+02	5.55E+02	4.83E+02	4.93E+02	5.02E+02	3.58E+01	7.14E-02
	0.1	4.11E+02	4.47E+02	4.19E+02	4.25E+02	4.25E+02	1.56E+01	3.68E-02

Appendix B

F-Test Results for Dynamic Modulus

Table 43: F-Test for -10°C

	HMA	**WMA**
Mean	11800	13138
Variance	1298878	3846819
Observations	6	6
df	5	5
F	0.34	
P(F<=f) one-tail	0.13	
F Critical one-tail	0.20	

Table 44: F-Test for 4.4°C

	HMA	**WMA**
Mean	7693	8777
Variance	2050506	3786574
Observations	6	6
df	5	5
F	0.54	
P(F<=f) one-tail	0.26	
F Critical one-tail	0.20	

Table 45: F-Test for 21°C

	HMA	**WMA**
Mean	4114	4327
Variance	925725	1537368
Observations	6	6
df	5	5
F	0.60	
P(F<=f) one-tail	0.30	
F Critical one-tail	0.20	

Table 46: F-Test for 37°C

	HMA	**WMA**
Mean	2058	1957
Variance	259507	424150
Observations	6	6
df	5	5
F	0.61	
P(F<=f) one-tail	0.30	
F Critical one-tail	0.20	

Table 47: F-Test for 54°C

	HMA	WMA
Mean	747.8333333	690.5
Variance	49651.76667	63644.3
Observations	6	6
df	5	5
F	0.78	
P(F<=f) one-tail	0.40	
F Critical one-tail	0.20	

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