Evaluation of Warm Mix Asphalt Technology for Urban Pavement Rehabilitation Projects

By

Salvatory Materu

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Abstract

Warm Mix Asphalt (WMA) technology has the capability of lowering the temperature at which the asphalt is mixed and compacted by 30°C or more without compromising the performance of asphalt pavement. The reduced difference between asphalt mix and ambient temperature results in a lower cooling rate thus allows for long haul, sufficient compaction time and late season projects compared to the conventional Hot Mix Asphalt (HMA). In northern climate, asphalt paving season is relatively short and paving is often done late in the season when weather conditions are less than ideal. The potential benefit of WMA, among others, is an extended paving season for the City of Winnipeg. Reduction in production temperature also comes with other positive impacts both economically and environmentally.

The objective of this study is to evaluate the installation of WMA, compile experiences with this technology and evaluate their effects on construction methods and performance. The study further attempts to evaluate the effectiveness of the WMA chemical additives and its dosage rate as liquid anti-strip agents on the properties of WMA mixtures through field and laboratory testing programs. In addition to the overall effectiveness of WMA, the study aimed to evaluate its economic cost relative to Hot Mix Asphalt (HMA).

A chemical additive was used at three different dosages (0.3, 0.5 and 0.7 percent by weight of asphalt cement). The additive has the ability to improve mixing, aggregate coating, workability, compaction and adhesion with no change in materials or job mix formula required. The study showed that WMA could be successfully placed using conventional HMA paving practices and procedures. Among the different additive dosages used, 0.5% had a better overall performance. The moisture sensitivity tests indicated the highest Tensile Strength Ratio (TSR) at this dosage, suggesting the lowest moisture damage susceptibility. All four mixtures had low rutting resistance potential with no significant difference among them. The WMA showed a higher cracking resistance compared to HMA. The WMA price was between 2% to 11% higher than conventional HMA including the costs of additional testing as well as the WMA additives. It is expected that in future large paving contracts, the cost of WMA will decrease when the contracts do not include the additional testing and contractors realize the financial benefit of reduced energy consumption in the production of WMA.

Keywords: Warm Mix Asphalt, Chemical Additive, Pavement Performance, Moisture Susceptibility, Dynamic Modulus, Hamburg Wheel-Tracking, Cracking Resistance.

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Chapter 1. Introduction

1.1 Introduction and Background

For many years, the asphalt industry has negatively impacted the environment by consuming large amount of fossil fuels and emitting pollutant gases to the atmosphere. Due to the growing safety, health and environmental awareness of the public and the industry has led to significant efforts of reducing the use of non-renewable energy source, conserve energy and reduce emissions and exposures. This growing international pressure on energy conservation has resulted in the introduction of several technologies in the last two decades including exploring the possibility of producing and paving asphalt mixtures at intermediate temperatures (80-120 °C)(Larsen et al., 2004). The asphalt mixtures produced and compacted at lower temperatures, known as Warm Mix Asphalt (WMA) are one of the most explored and studied strategies. WMA technology aims to reduce energy consumption throughout the process without depleting the in-service mechanical performance of these asphalt mixtures (Capitão et al., 2012). The traditional asphalt mixture, known as Hot Mix Asphalt (HMA) is normally produced and compacted at temperatures above 140°C to dry the aggregates and increase the workability of the asphalt mix. There are several techniques and products that can be used to reduce production and compaction temperatures while ensuring the full coating of the aggregates, and thus achieving desired compaction (Pereira et al., 2018). The lower temperature comes with several positive impacts such as:

- Reduced thermal cracking and enhance the overall pavement performance;
- A reduction in fuel energy consumption thus fuel cost saving;
- Less emissions including greenhouse gas that contribute to health and odor problems;

- Lower emission may allow plants to be located in areas with strict air pollution regulations thus reducing the delays associated with traffic congestion;
- Improve worker safety and health due to handling mixes at lower temperatures and reduced asphalt fumes;
- Allow for longer haul distances due to the slow cooling rate; and,
- Extend construction season (more important in cold regions).

The WMA technologies are most commonly divided into three; foaming techniques, organic additives and chemical additives. Forming technology works by adding small amount of water into the asphalt mix. When water in the asphalt mix is heated, it expands which causes the binder to expand and reduces the binder viscosity to achieve proper coating of the aggregates (Larsen et al., 2004). Organic additives used are often waxes and fatty amides such as Sasobit that have the ability to reduce binder's viscosity at temperature above binder's melting point. Chemical additives are usually emulsifiers and surfactants such as Evotherm that improve coating of aggregates by reduces the internal friction between the aggregates and binders (Arega & Bhasin, 2012).

Different studies have been performed on all these technologies and products capable of reducing production and compaction temperatures of asphalt mixtures. Chemical additives have shown to be the most economic and simple to implement (Gonzalez-Leon et al., 2017). The study conducted by Gonzalez-Leon et al (2017) suggested that the use of chemical additives improved the workability of apshalt mixtures at reduced production and compaction temperature, even with the use of higher quantities of reclaimed asphalt pavement (RAP). In addition, several authors have been proving that the in-service performance of chemical additive modified asphalt mixtures is similar or even better than the conventional HMA. Morea et al. (2012) proved that the use of chemical additives at reduced temperature results to a less aged binder, and thus improving the elasticity of the binder. Furthermore, the decrease in initial ageing of the binder producing a more ductile binder leads to less thermal cracking than in the case of HMA (Hurley & Prowell, 2006). Abdullah et al. (2016) through rheological tests proved that asphalt mixtures modified with chemical additives result to lower G*/sin (δ) values that indicate an improvement in fatigue resistance at intermediate temperatures. Moreover Pérez-Martínez et al. (2014) suggested that the addition of chemical additives resulted in higher values of surface free energy which normally indicates a higher adhesion between the binder and aggregates and higher resistance to water damages.

Because of the rapid and large-scale implementation of WMA, the City of Winnipeg and the University of Manitoba have partnered under the Municipal Infrastructure Chair Funding Program to evaluate the applicability of WMA applications in the City of Winnipeg during overlay projects through a laboratory and field-testing program. The program includes evaluating the stiffness, rutting resistance and moisture susceptibility for the WMA mixtures compared to the traditional HMA mixtures. In this study, only one chemical additive was used known as Evotherm 3G. It is a water-free additive that can reduce the mixing temperature by 33°C to 45°C without compromising the asphalt performance (Kuang, 2012). Three different additive dosages (0.3, 0.5 and 0.7 percent by weight asphalt cement) were used to evaluate the effects of the dosage rate. Evotherm is the most commonly used chemical additive in Northern America (Varamini, 2016). The selection of the materials was based on typical asphalt mixes used by the City of Winnipeg (City of Winnipeg, 2015). Construction used typical mixture designs and practices so that performance under typical construction conditions could be evaluated.

1.2 Problem Statement

The reality of being located in a northern climate means that asphalt paving season is relatively short, and paving is often done late in the season when weather conditions are less than ideal. Owners and contractors have been working towards technologies that can reduce the risk associated with cool weather paving and compaction of stiff asphalt mixes. Previous studies have shown that WMA can allow paved asphalt to achieve compaction at lower mixing and compaction temperatures, while retaining better performance through its service life. This potential benefit means, among others, an extended paving season for the City of Winnipeg.

1.3 Objective

This research aims to address three main objectives:

- To evaluate the installation of WMA, compile experiences with this technology and evaluate their effects on construction methods and performance.
- To evaluate the effectiveness of the WMA additives and its dosage rate as liquid anti-strip agents on the properties of WMA mixtures through field and laboratory testing programs.
- To evaluate the economic cost of WMA relative to HMA.

1.4 Research Methodology

The research methodology of this thesis can be summarized as follows:

➢ **Project proposal**

The main focus of this phase was to prepare a detailed plan and strategies for how to carry out a successful WMA project for the city of Winnipeg. The following steps were involved:

- Conducted a literature review on the current state of knowledge and understanding of WMA.
- Conducted a literature review on WMA focusing on North America with more emphasis on Canada.
- Organized meeting with the city of Winnipeg, Manitoba Infrastructure and industry experts to discuss how to implement WMA technology in Manitoba.
- Prepared a final project description that was approved by all members attending the meetings.

➢ **Field Testing**

Two main studies were conducted in the field; temperature monitoring and measuring in-place densities. Infrared thermal camera was used to monitor the temperature behind the paver, before compaction and after compaction. A nuclear density gage was used to measure the degree of compaction.

➢ **Laboratory Testing**

To evaluate the short and long term performance of the mixtures, loose mix samples were collected for each WMA mix and the control HMA mix. The tests performed included the dynamic modulus test, moisture susceptibility test and Hamburg Wheel Track Test that evaluated the stiffness, moisture susceptibility and rutting resistance respectively.

1.5 Significance of Study

This study will provide guidance and recommendations on how to implement a Warm Mix Asphalt technology for the city of Winnipeg and Manitoba in general. It will also facilitate the approval of WMA technology and products by different agencies in Manitoba.

1.6 Thesis organization

This thesis is organized into chapters with the following arrangement:

Chapter 1: Introduction – This chapter provides the problem statement, objective, methodology and significance of this research project.

Chapter 2: Literature Review – A detailed review of the current state of warm mix asphalt application is discussed in this chapter. The advantages and disadvantages of WMA technology are also elaborated. Furthermore, the chapter provides a brief review of the Canadian experience on the WMA application.

Chapter 3: Materials and Test Methods – The properties of the materials used and laboratory tests procedures are presented in this chapter.Also, the chapter explains the field monitoring strategies that were placed during construction for quality control. This played a key role in confirming WMA temperatures and compaction degrees in the field.

Chapter 4: Evaluation of Warm Mix Asphalt – In this chapter, all the data from experimental tests and field measurement were assessed and performance comparisons between all mixtures were conducted.

Chapter 5: Conclusions and Recommendations – The conclusion of the study and recommendation for future works are provided in this chapter.

Appendix A – Shows the normalized dynamic modulus values for each mixture at different frequencies and temperatures.

Appendix B – Shows the average tensile strength values for each mixture.

Appendix C – Shows the average specimens dimension and characteristics used in the cracking test.

Chapter 2. Literature Review

2.1 Introduction

The main purpose of this chapter is to provide a detailed assessment of the current state of knowledge and understanding of concepts related to the application of Warm Mix Asphalt (WMA) focusing on the field performance, mix design, and test methods. The major asphalt pavement distresses that are evaluated include stiffness, moisture susceptibility and rutting resistance. Several studies comparing the laboratory and field performance of WMA to HMA have been conducted. In addition, the Canadian experience on WMA will be highlighted in this section.

2.2 WMA Processes and Products

2.2.1 Asphalt Mixtures

Classification of different asphalt mixtures can be achieved by the difference in temperature at which the materials are mixed and compacted. There are four classes of asphalt mixes that are generally used in the industry and referred to in researches based on their mixing and compacting temperature range (D'Angelo et al., 2008).

- Cold mix asphalt (0°C – 30°C)
- Semi warm mix asphalt (65°C – 100°C)
- Warm mix asphalt (100°C – 140°C)
- Hot mix asphalt (above 140°C)

Since achieving asphalt mixtures at high temperature requires the use of more resources such as fuel, the amount of temperature reduction supports the fundamental advantage of WMA in terms of environmental and economic benefits. Figure 2.1 illustrates the different mixing and paving temperatures for each asphalt mix class.

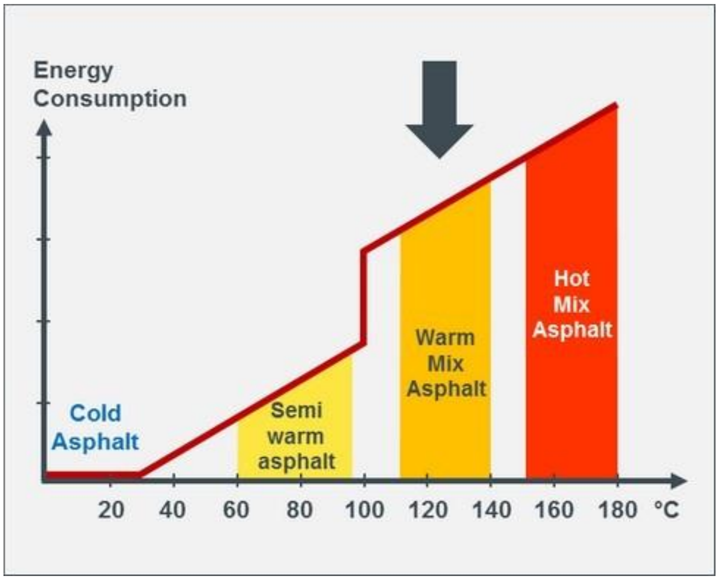


Figure 2.1 Mix type classifications by temperature range and energy usage (Kuang, 2012)

Currently, WMA technology depends on either varying the binder properties temporarily or permanently or alteration of the interaction between aggregates and binders. When the binder viscosity is reduced, it increases the workability and therefore allows better coating of aggregates at lower temperatures. In addition, other technologies improve the adhesion of binder and aggregates which aid in compaction of the mix. Both mechanisms results in less compaction effort to achieve desired densities of the placed asphalt (Yang et al., 2017).

WMA production temperature generally depends on the user due to the lack of an industry standard definition. For most contractors, a temperature reduction of 17°C has been a minimum required degree to classify a WMA technology depending on the binder type and referenced HMA mixing temperature (FHWA 2012). In the field, some contractors choose to produce WMA at the same temperature as HMA in order to achieve compaction much easier.

2.2.2 Background on WMA Technologies

In 1997, for the first time WMA technology was introduced at the German Bitumen Forum. The concept was widely adopted in Europe after different countries signed the Kyoto Agreement on reduction of greenhouse gas (D'Angelo et al., 2008). Shortly after, the United States Federal Highway Administration's International Technology Scanning Program planned a special visit to four European countries namely France, Belgium, Germany and Norway to study the feasibility of applying WMA technology in the United States. The US expert team recommended that US could greatly benefit from the WMA technology and believed highway agencies would permit WMA to be used as an alternative to HMA. From these findings, several research and trials have been performed (D'Angelo et al., 2008).

Several Warm Mix Asphalt processes and products have been identified and evaluated by different agencies such as the National Cooperative Highway Research Program (NCHRP) and the National Center of Asphalt Technology (NCAT). Table 2.1 presents the most widely used WMA technologies identified from a survey conducted by Bonaquist in the NCHRO 09-43 project (Bonaquist, 2011).

Table 2.1 Most widely used WMA technologies (Bonaquist, 2011)

Name	Process/Additive	Company
Accu-Shear Deal Warm Mix Additive system	Foaming System	Stansteel
Adesco/madsen Static Inline Vortex Mixer	Foaming System	Adesco/ Madsen
Advera	Zeolite	PQ Corporation
AQUABLACK	Foaming System	Maxam Equipment Company, Inc.
AquaFoam	Foaming System	Reliable Asphalt Products
Asphaltan –B	Montan wax	Romonta
Aspha-min	Zeolite	Eurovia
Cecabase RT	Unspecified additive	Ceca

Double Barrel Green	Foaming System	Astec, Inc.
Evotherm ET	Emulsion with unspecified additives	MeadWestvaco
Evotherm DAT	Unspecified additive	
Evotherm 3G	Unspecified additive	
Licomont BS-100	Unspecified additive	Clariant
Low Emission Asphalt	Fatty acid derivative	McConnaughay Technologies
Meeker Warm Mix Asphalt System	Foaming System	Meeker Equipment
Rediset WMX	Unspecified additive	Akzo Nobel
Sasobit	Fischer Tropsch wax	Sasobit
Terex Warm Mix Asphalt	Foaming System	Terex Road building
Thipoave	Sulfur plus compaction aid	Shell
TLA-X	Trinidad Lake Asphalt plus Modifiers	Lake Asphalt of Trinidad and Tobago
Ultrafoam GX	Foaming System	Gencor Industries, Inc.
WMA Foam	Soft binder followed by hard foamed binder	Kolo Veidekke, Shell Bitumen

There are currently three WMA production techniques in the industry and sometimes a combination of them can be used:

- Foaming technology
- Organic technology
- Chemical additives technology

2.2.2.1 Foaming technology

Forming technology works by adding small amount of water into the asphalt mix. When water in the asphalt mix is heated, it expands which causes the binder to expand and reduces the binder viscosity. This will improve aggregates coating and increase workability of the asphalt mix. The amount of water added is very critical because if too much water is added then the mix will be more susceptible to stripping (Larsen et al., 2004).

There are two factors that define the properties of a foamed warm mix asphalt namely expansion ratio and half-life of the asphalt mix. The expansion ratio is the ratio of the maximum volume of the expanded binder to the original volume of the binder. A high expansion ratio provides a large surface area for the binder to coat the aggregates. Half-life of the binder is the time in seconds it takes for the foamed binder to decrease in size from maximum volume to half of the maximum volume. A long half-life means the asphalt mix has a longer time with high workability since the viscosity is low (Frank & Prowell, 2011).

Another method of foaming the binder is by mixing water bearing minerals with the binder in the asphalt mixture. Both the binder and the minerals are to be added at the same time. The most commonly used mineral is the fine powdered synthetic Zeolite that has undergone hydro-thermal crystallization. This forms the binder and reduces its viscosity (D'Angelo et al., 2008). In the industry, the two methodologies are referred to as WAM-Foam and Zeolite forming technologies.

2.2.2.2 Organic technology

Organic or wax additives have the ability to reduce the viscosity hence increasing workability of the binder when heated above their melting point. This results to better coated aggregates even at lower temperatures. When the asphalt mix cools, the additive tends to crystallize and therefore increases the stiffness of the binder and asphalt mix resistance to load deformations. Due to the fact that the additives are sensitive to the exposed temperature, the additives must be carefully selected such that its melting point is higher than the in-service temperature. Normally, a temperature reduction between 10°C to 30°C is achieved with this technology (Frank & Prowell, 2011).

The most commonly used organic additive is Sasobit with a melting point range between 85°C and 115°C. The optimum dosage of Sasobit is 3% of the binder mass, with the potential to reduce the binder softening point by 20°C to 35°C. Previous studies have shown Sasobits–modified mixes have a higher resistance to rutting. This could be achieved due to the crystallization of the additive at a lower temperature after it cools (Frank & Prowell, 2011). Sasobit and Advera are the two most commonly used organic additives in North America.

2.2.2.3 Chemical additives technology

Chemical additives are more broadly used in Northern America which work as surfactants that reduce the friction at the interface between the binder and the aggregate. Evotherm is one alternative to achieve that purpose in the asphalt paving industry (Buss et al., 2014). Evotherm can improve mixing, aggregate coating, workability, compaction and adhesion with no change in materials or job mix formula required. The third generation of Evotherm called Evotherm M1 was introduced which is a water-free additive that can reduce the mixing temperature by 33°C to 45°C without compromising the asphalt performance (Kuang, 2012). Evotherm and Zycotherm are the two most commonly used organic additives in the industry. Chemical additive technology requires no change to the production plant as the additive can be mixed directly with the binder by the supplier.

2.3 Benefits and Challenges of Warm Mix Asphalt

2.3.1 Benefits of Warm Mix Asphalt

The implementation of WMA technology provides numerous benefits both to the industry and society in general. Different transportation agencies in Northern America have been reporting on the success of WMA applications on their local projects. The environmental benefit of WMA is the most acknowledged aspect that most agencies and academics emphasis. The advantages can be categorized into four aspects:

- Production impacts
- Compaction ability during paving
- Economical benefits
- Environmental benefits

2.3.1.1 Production Impacts

One of the most frequent concerns of asphalt producers to implement WMA has been the equipment required for production. However, WMA production can be implemented using the existing asphalt plant with little or no retrofitting of equipment required depending on the WMA technology executed. The use of chemical additives does not require any changes to the existing plant. The additives are normally added with the asphalt binder supplier. The binder is mixed with aggregates at the plant the same way as it is done with HMA production. On the other hand, foaming technology requires an additional production kit that can be fitted to contractor's plant (Frank & Prowell, 2011).

According to literature, the use of WMA potentially allows an increase in the use of Reclaimed Asphalt Pavement (RAP) and Recycled Asphalt Shingles (RAS) into the mixture (Zaumanis & Mallick, 2015). The already stiff RAS and RAP binders will age less due to a lower production temperature. An increase in workability at a lower temperature, results in a less aged binder which is essential to prevent premature and thermal cracking of the pavement.

2.3.1.2 Compaction ability during paving

In cold weather regions, the concern of paving using the traditional HMA always arise after summer. The mixing and placing temperatures of HMA are the main challenges due to the cold environment. The cooling rate of the asphalt mix increases as the difference in temperature between mix and surrounding environment increases. The desired degree of compaction becomes difficult to achieve during these conditions. However, WMA technology improves the workability by providing better interaction between the binder and aggregates even at lower temperatures. This allows an extension in paving season as desired densities are still achieved during cooler ambient temperatures. In addition, longer hauling distances and more paving hours can be allowed for nonattainment areas. This will reduce the mobilization costs and increase accessibility to urban areas. WMA technologies can also enable night–time and high altitude paving by aiding during compaction.

Furthermore, the use of WMA results in reduced time for road opening because asphalt is placed at a lower temperature. This can be greatly appreciated in projects with a very tight window for construction such as airport runways (Frank & Prowell, 2011).

2.3.1.3 Economical benefits

Depending on the circumstances, the use of WMA can result in both an increase and a reduction in asphalt production cost. There are several factors that contribute to the cost of implementing WMA technology.

- **Cost saving**

The cost saving when implementing WMA technology can be achieved either through materials selection or amount of energy used during production. The type of WMA technique used also plays a significant role in cost saving. According to literature, contractors have reported a decrease in burner fuel usage of 10 to 15 percent when using WMA compared to HMA. Other reported cost savings of \$0.1 to \$0.8 per ton depending on the reduced temperature and type of fuel (Frank & Prowell, 2011). Another cost savings technique is the use of more recycled material. As mentioned earlier, WMA allows the use of more RAP and RAS compared to HMA which reduces the amount of virgin materials required for the mix.

The extension of paving period and working during cooler temperatures, enables the contractors to complete more projects hence making more profit. In addition, the penalties for unsatisfactory compaction can greatly be avoided by contractors since WMA improves workability and required densities are better achieved.

- **Cost increase**

The additional costs of implementing WMA technology is greatly influenced by the technique executed. The extra costs normally arise from the additive and plant modification costs. The application of warm mix technology will result to an overall cost increase if the amount of energy saving is significantly lower than the cost of integrating WMA technology.

2.3.1.4 Environmental benefits

As the production temperature is reduced, less fuel consumption is required. The reduction in energy consumption when using WMA technology results in lower emissions and reduced carbon footprint from the asphalt industry. The environmental benefits can be divided into two categories:

- **Energy use**

The most important advantage of using WMA technology is the reduction in energy use. Several researchers have summarized the energy consumption from different WMA projects. Prowell and Hurley (2012) studied fifteen projects that used six different WMA technologies and reported an overall reduction in fuel consumption of 23 percent. A report published by European WMA production plants suggested a 20 to 35 percent reduction in fuel consumption when using WMA technology (D’Angelo et al., 2008). Furthermore, the NCHRP 9-47A draft final report from five different production sites suggested fuel savings of 22.1 percent for an average temperature reduction of 27°C (West et al., 2014). The calculations of energy consumption based on production temperature were performed using thermodynamics. The results are summarized in Figure 2.2.

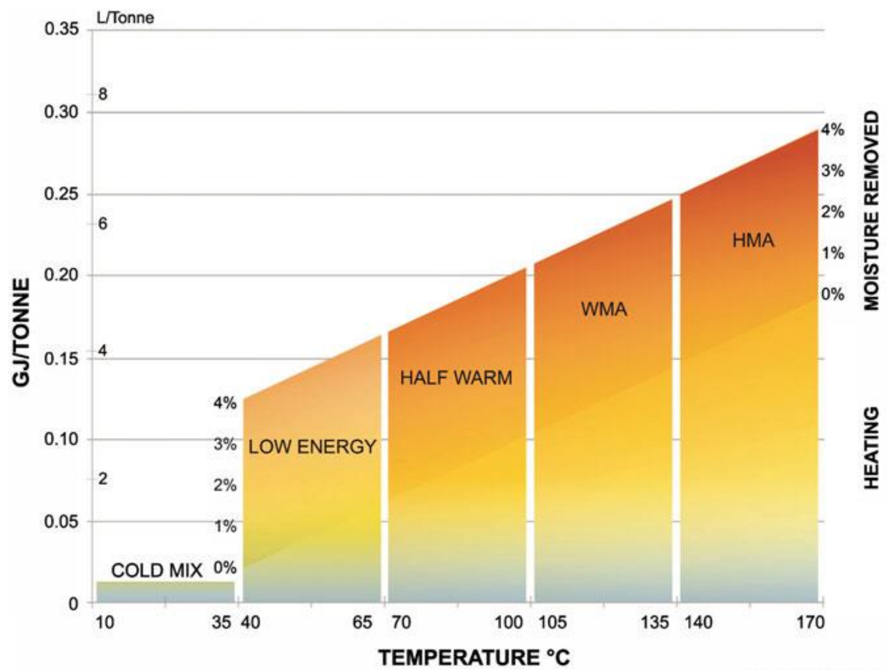


Figure 2.2 Summary of energy consumption based on production temperature (West et al., 2014)

- **Emissions Reduction**

The relationship between fuel energy and CO_2 reduction has been shown to be linear (Frank & Prowell, 2011). Most of the reports suggest a reduction in fuel consumption due to the use of WMA technology, resulting in a reduction in CO_2 emissions. The WMA technology provides a higher possibility for paving activities in non-attainment areas due to lower emissions. The NCHRP 9-47A report also indicated an average CO_2 emission reduction of 20% from seven different WMA technologies (West et al., 2014).

Figure 2.3 Paving using HMA technology (left) and WMA (right) (City of Winnipeg)

Furthermore, the use of WMA technologies reduces the worker's risk of concentrated exposure to fumes that can be toxic and associated with a wide variety of negative health effects as shown in Figure 2.3. Short term health effects include respiratory problems, eye irritation, nausea, wheezing and headache. Long term health effects include asthma, breathlessness and reduced lung capacity (Mazumder et al., 2016).

2.3.2 Challenges of Warm Mix Asphalt

Previous research has raised a concern about moisture susceptibility due to the reduction in mixing temperature. Higher moisture content in the asphalt mixture decreases the adhesion of asphalt binder to the aggregate surface, increases the moisture susceptibility of the asphalt mix which can lead to stripping and damage of the asphalt pavement. As the use of WMA technology evolves, many agencies seek a better understanding of the effects of WMA dosage rate on mixture compaction, rutting resistance and moisture susceptibility. Studies have indicated that WMA chemical additives play a significant role as an anti-stripping agent and improve the resistance to moisture damages (Kuang, 2012).

Furthermore, previous studies have indicated some challenges with determining the adequate temperature required to activate the hard and aged binder when using recycled materials (Gonzalez-Leon et al., 2017). As mentioned in the previous section, WMA technology allows the use of more recycled materials especially RAP. The use of more RAP and RAS will require a higher temperature to activate the already hardened binder. Determining the relationship between the rates of temperature increase relative to the amount of recycled material will result in better pavement performance.

2.4 Canadian Experience on WMA

In Canada, different transportation agencies have reported using WMA technology in several projects. The Centre for Pavement and Transportation Technology (CPATT) conducted a study for Canadian transportation agencies in 2015 with two main objectives: (1) to document the state-of-the-art related to WMA technologies, and (2) to identify candidate technologies or knowledge for inclusion in the work plan. All provinces except Newfoundland and Prince Edward Island and one territory responded.

From the response received, most agencies indicated routinely use of WMA as illustrated in Figure 2.4. In addition, Figure 2.5 shows the tonnage of WMA that was placed up to date when the survey was conducted.

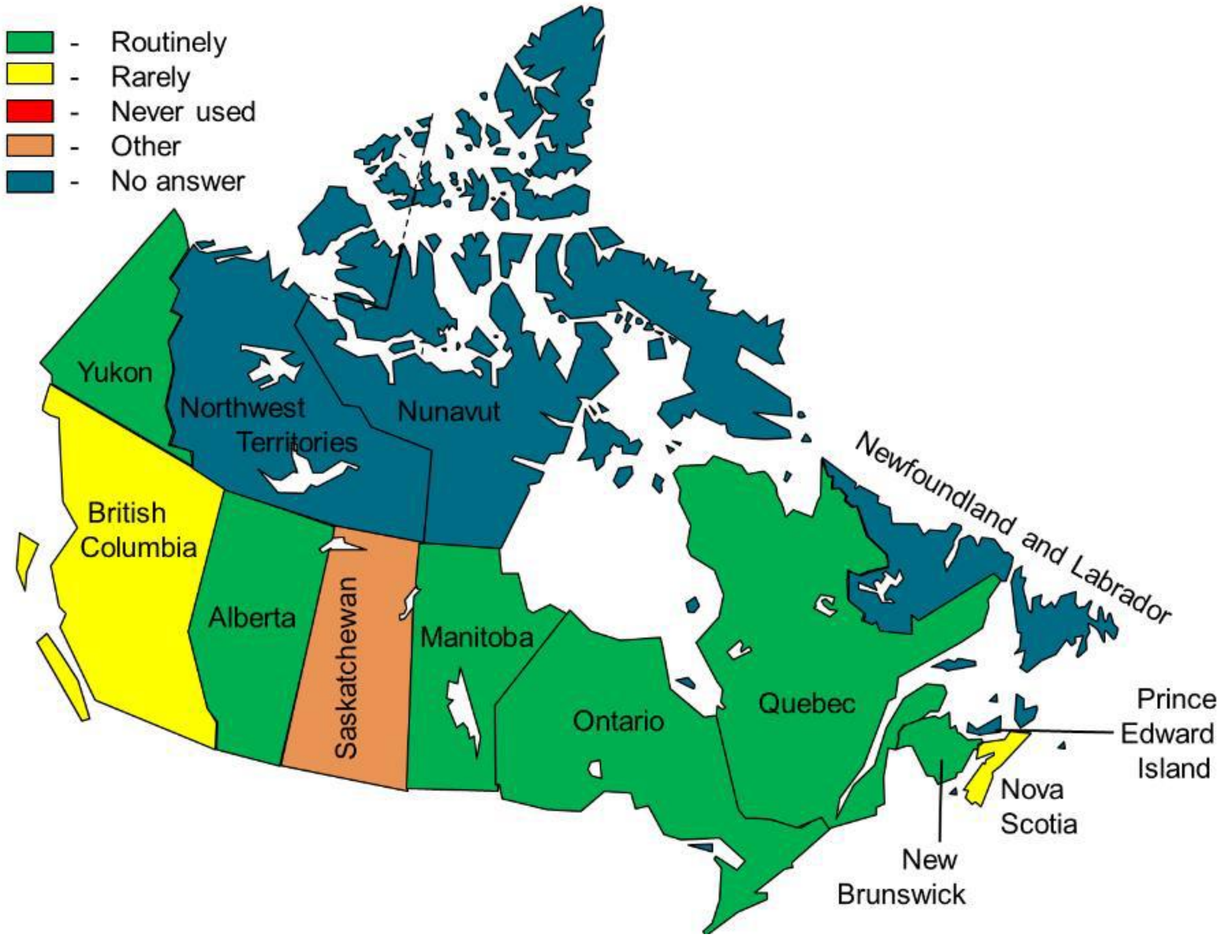


Figure 2.4 Usage of warm mix asphalt in Canada (Varamini, 2016)

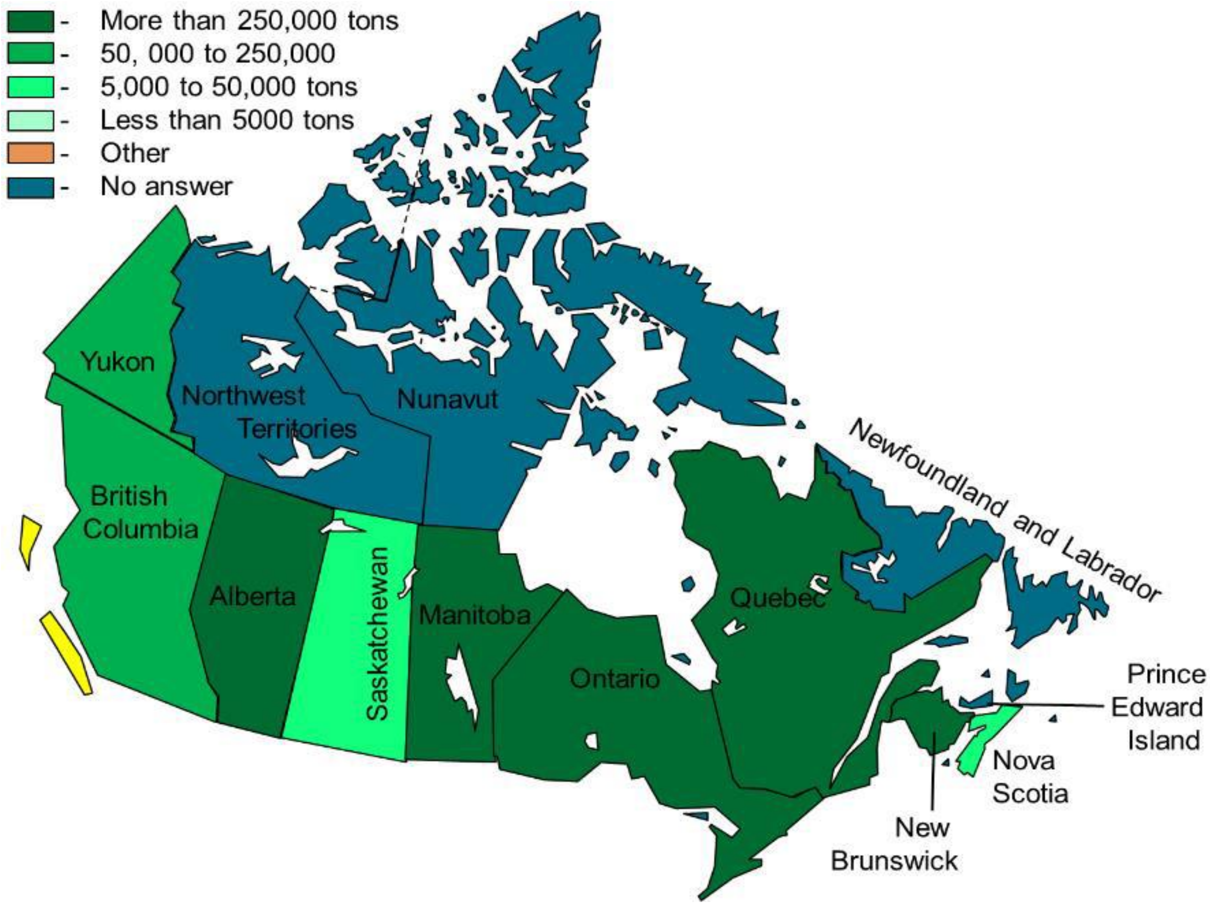


Figure 2.5 Warm mix asphalt tonnage placed as of 2015 in Canada (Varamini, 2016)

Most of the respondents reported the first use of WMA was between 2006 and 2009. The study further investigated the types of WMA technology that the agencies used, as well as the types of additives most commonly used. Table 2.2 shows the list of most commonly used additives reported by the respondents.

Table 2.2 Most commonly used warm mix asphalt additives in Canada

Category	Additive	Response
Chemical	Evotherm 3G.	82%
	Evotherm DAT	45%
	Cecabase RT	36%
	Rediset LQ.	18%
	Rediset WMX	9%
Foaming	Double Barrel Green	45%
	Ultrafoam GX	27%
	Advera WMA	18%
	Aspha-Min	9%
Organic	SonneWarmix	27%
	Sasobit	18%

Since WMA technology is commonly associated with moisture damages due to the use less dry aggregates, the survey also inquired on the use of antistripping agents with WMA. The majority of the respondents indicated the use of antistripping agents if the laboratory test results indicate presence of moisture damage. In addition, two agencies indicated the use of antistripping agents if the aggregates have a history of moisture susceptibility. However, two agencies from the survey indicated that there are some warm mix additives known to have antistripping properties, and therefore, the use of antistripping agents may not be required (Varamini, 2016).

Most agencies documented both laboratory and field performance of the WMA projects. Furthermore, 91% of the agencies indicated no premature distress or failures observed in any of the WMA projects. However, one agency detected the presence of thermal cracking in their WMA pavement.

2.5 Performance Tests Evaluation of WMA

Warm Mix Asphalt (WMA) plays a significant role in improving compaction when asphalt is placed in a cooler weather condition. It fascilitates the achievement of desired density of the pavement. Although density is normally the first criterion that have to be achieved in most paving project specifications, the pavement short-term and long-term performance is way more important when evaluating the quality of the pavement. The possibility of insuffiecient drying of the aggregates due to lower temperature used in the production of WMA allows some concerns related to aggregate stripping. Combined with heavy traffic, rutting and cracking potential also becomes a concern for the long term performance of WMA pavements. Hence, this study investigated the performance of WMA in terms of stiffness, stripping, rutting and cracking.

2.6 Challenges, Gaps and Research Opportiunities

Although there have been a wide exploration on mix design, material charatization, application and performance tests pertaining to WMA, there are still some concerns from transportation agencies and contractors that must be addressed in order to guarantee an equivalent or better performance than the convertional HMA. These concerns include:

- The long term performance of WMA especially its resistance to rutting and moisture damage. Due to the low temperature used to mix and compact, there are some concerns of presence of moisture remaining in the aggregates and mixture's resistance to permanent deformation.
- The cost effectiveness of different warm mix technologies and additives particularly in terms of energy savings and emission reduction. The additives have very different properties and some require higher temperatures than others when mixing with the binder. Further studies on environmental benefits of WMA are required to address this issue.

- The need for using an antistripping agent when using WMA is also a concern to many transportation agencies. There are reports that suggest some chemical additives such as Evotherm 3G have antistripping properties and therefore, there is no need to add another antistripping agent when using them. In addition, the compatibility of warm mix additive to antistripping agents when required remains to be a concern as well.

Chapter 3. Materials and Test Methods

3.1 Introduction

This chapter illustrates the materials selected and methods used during the study. Four mixtures were studied in this project: the control mixture (HMA) and three warm mixes at 0.3%, 0.5% and 0.7% chemical dosage by weight of asphalt cement. One of the main objectives of this study was to evaluate the effectiveness of the WMA additives and its dosage rate on the properties of WMA mixtures through field and laboratory testing programs. Figure 3.1 summarizes both field and laboratory test programs implemented during the study.

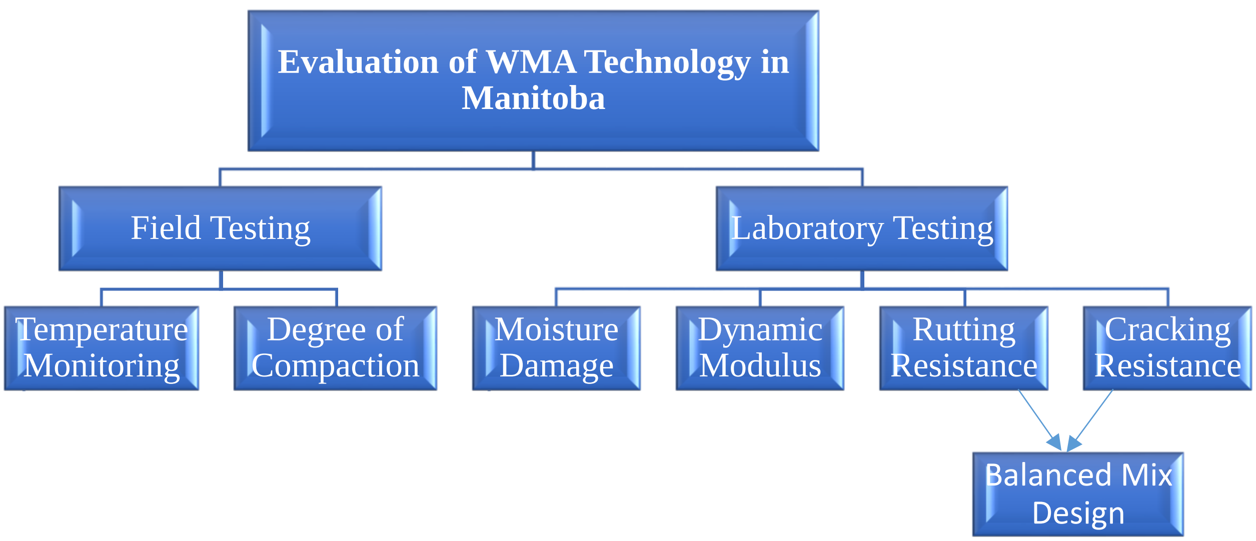


Figure 3.1 Research plan methodology

3.2 Materials

The raw materials which were used in the preparation of WMA mixtures in this study, including asphalt cement and aggregate were characterized using routine type of tests, and the results were compared with the City of Winnipeg specification requirement to evaluate their suitability for job mix (City of Winnipeg, 2015). Table 3.1 and Figure 3.2 shows the aggregate gradations of HMA and WMA mixtures collected on site during construction. The gradations were very similar and all met the specifications outlined by the city of Winnipeg (City of Winnipeg, 2015).

Table 3.1 Aggregate gradation of four mixtures (percentage passing)

SIEVE SIZE (mm)	PASSING (%) HMA	0.3% WMA	0.5% WMA	0.7% WMA	SPECIFICATION MIN (%)	MAX (%)
20.00	100.0	100.0	100.0	100.0	100	100
16.00	100.0	100.0	99.2	100.0	99	100
12.50	92.8	94.8	94.5	94.6	-	-
10.00	78.5	86.6	80.9	82.2	70	88
5.00	56.6	66.7	60.8	64.4	55	70
2.50	47.2	55.1	51.0	55.1	40	60
1.25	37.8	43.8	40.9	44.6	25	50
0.63	24.7	28.3	26.6	29.2	15	40
0.315	11.6	12.6	12.4	13.6	5	28
0.16	4.8	5.0	5.1	5.6	4	11
0.08	3.4	3.5	3.6	4.0	3	7
Crushed Content	90%	89%	89%	91%	60% minimum	

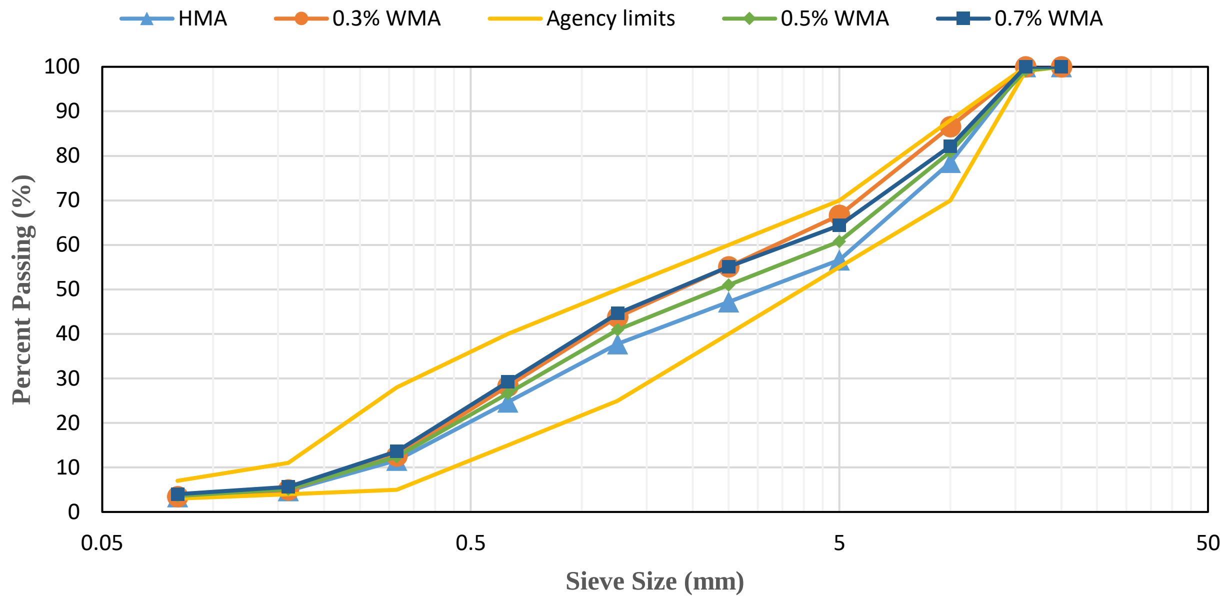


Figure 3.2 Aggregate gradation for the four mixtures

Furthermore, Marshall tests were conducted to determine the volumetric properties of the mixtures. The Marshall properties of HMA and WMA mixtures used in construction were almost similar and met the City of Winnipeg specifications, as shown in Table 3.2 (City of Winnipeg, 2015).

Table 3.2 Volumetric properties of the four mixtures

MARSHALL PROPERTIES	**HMA**	**0.3% WMA**	**0.5% WMA**	**0.7% WMA**	**CW 3410-R12 SPECS**
% AC (by total mix)	5.0	5.1	5.1	5.2	5 to 6
Air Voids (%)	4.5	4.6	4.4	4.6	3 to 5
Stability (kN)	11.7	9.9	10.6	8.8	7 minimum
Flow (mm)	8.5	8	7.2	6.8	6 to 16
VMA (%)	14.1	14.3	14.2	14.5	14 minimum
Density (kg/m3)	2389	2384	2390	2381	-
Absorption (%)	0.92	0.93	0.93	0.95	-
Gsb	2.65	2.65	2.65	2.65	-
MTSG	2.509	2.507	2.506	2.504	-

The materials and mix design for the HMA and WMA were identical except for the inclusion of Evotherm 3G in the WMA. Evotherm 3G was used at three dosages in the WMA (0.3%, 0.5%, and 0.7%). The physical and chemical properties of the additive are listed in Table 3.3 (Kuang, 2012).

Table 3.3 WMA additive properties

Properties	**Evotherm 3G**
Physical Form	Dark Amber liquid
Density at 25°C	8.35 lb/gal
Specific Gravity at 25°C	0.97
Conductivity at 25°C	2.2 μS/cm
Dielectric Constant at 25°C	2-10
Recommended Dosage Rate	0.25-0.75% by weight asphalt cement
Viscosity (Pa · S)	
at 27°C (80°F)	0.28 – 0.56
at 38°C (100°F)	0.15 – 0.30
at 49°C (120°F)	0.08 – 0.16

The HMA was used as a reference and samples were collected to compare both short term and long-term performances. The HMA was placed in August 2018, followed by the WMA that was installed from September 2018 to November 2018. Placement of the HMA and WMA used the same equipment and methods. The project specifications stated that the maximum cooking temperature shall be 160°C and 135°C for HMA and WMA, respectively, and the maximum temperature of the WMA behind the screed should be less than 130°C (City of Winnipeg, 2015). The temperatures during construction, and before and after compaction were monitored using an infrared thermal camera.

3.3 Laboratory Testing

Loose mix samples were collected from each WMA mix as well as the reference HMA mix to evaluate the stiffness, moisture susceptibility, rutting resistance and cracking resistance of the mixtures. The tests performed included the dynamic modulus test, moisture susceptibility test, Hamburg Wheel Track Test (HWTT) and cracking resistance test. All samples were compacted using a Superpave Gyratory Compactor at mixing/compaction temperatures of 160/145°C and 130/115°C for HMA and WMA, respectively. Replicate samples for each test were produced and tested for repeatability as shown in Table 3.4.

Table 3.4 Laboratory tests summary

Testing	Parameter	Equipment	Procedure	Test Specimen Air Voids	Number of Test Specimens	Specification
Stiffness	Dynamic Modulus	MTS	Temperature: -10, 4.4, , 21.1, and 37.8°C; frequency: 0.1, 1, 5,10 and 25 Hz	4 ± 0.5 %	2	AASHTO T342-11
Moisture Damage	TSR	IDT	Freeze at-18°C and Thaw at 60°C	7 ± 0.5 %	6	AASTHO T-283
	SIP, Stripping slope	HWTT	Temperature: 45°C; immersed in water	7 ± 0.5 %	6	AASHTO T324-17
Rutting	Passes to Failure	HWTT	Temperature: 45°C; immersed in water	7 ± 0.5 %	4	AASHTO T324-17
Cracking	Flexibility Index	MTS	Temperature: 25°C	7 ± 1%.	8	AASHTO TP124-18,Illinois Modified AASHTO TP 124-18

3.3.1 Dynamic Modulus Test

The dynamic modulus test was used to evaluate the stiffness of the compacted mixtures in accordance with AASHTO T342 procedure (AASHTO T342, 2011). Cylindrical test specimens (100mm (4 in.) diameter and 150mm (6 in.) height) were prepared for the test. A sinusoidal compressive stress was applied to the sample at given temperatures and frequencies. Specimens were tested under four temperatures (-10, 4.4, 21.1, and 37.8°C) and six frequencies (0.1, 0.5, 1, 5, 10, and 25 Hz) to determine the dynamic modulus (|E*|) of specimens. The laboratory setup for dynamic modulus test is shown in Figure 3.3.

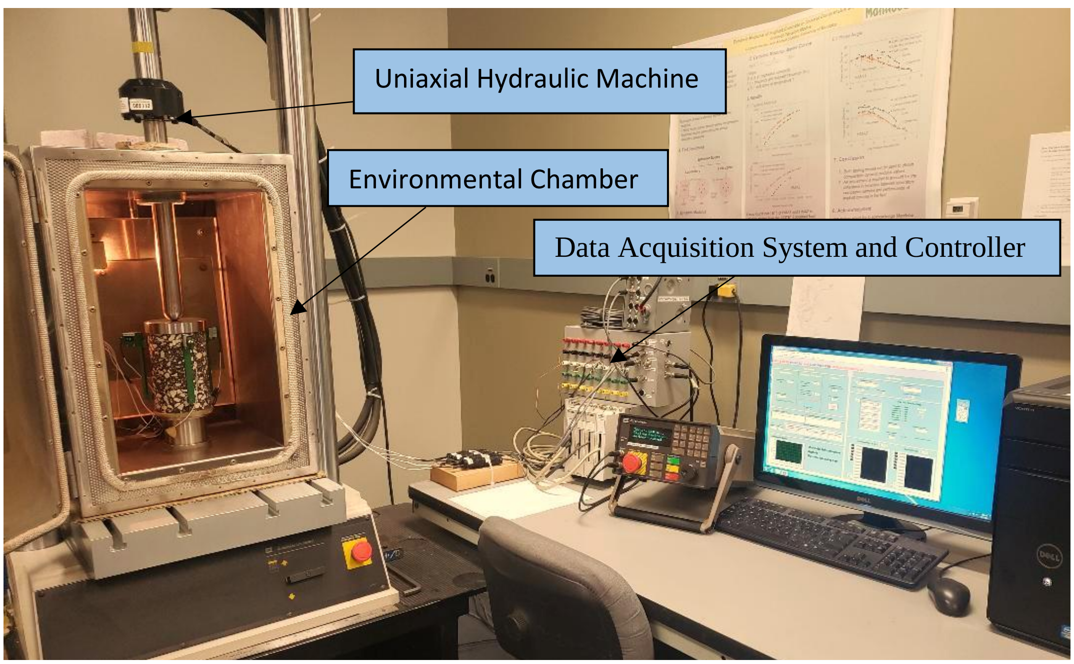


Figure 3.3 Dynamic modulus test setup

The dynamic modulus values were further combined to obtain a master curve using AASHTO PP62-09 procedure (Standard Practice for Developing Modulus Master Curve for Hot-Mix Asphalt) at a reference temperature of 21.1°C (AASHTO PP 62-10, 2012). A master curve compares the dynamic modulus values over a wider range of loading frequency and temperature. As shown in Figure 3.4, the left part of a master curve represents the stiffness at higher temperatures and lower frequencies, while the right part represents the stiffness at lower temperatures and higher frequencies.

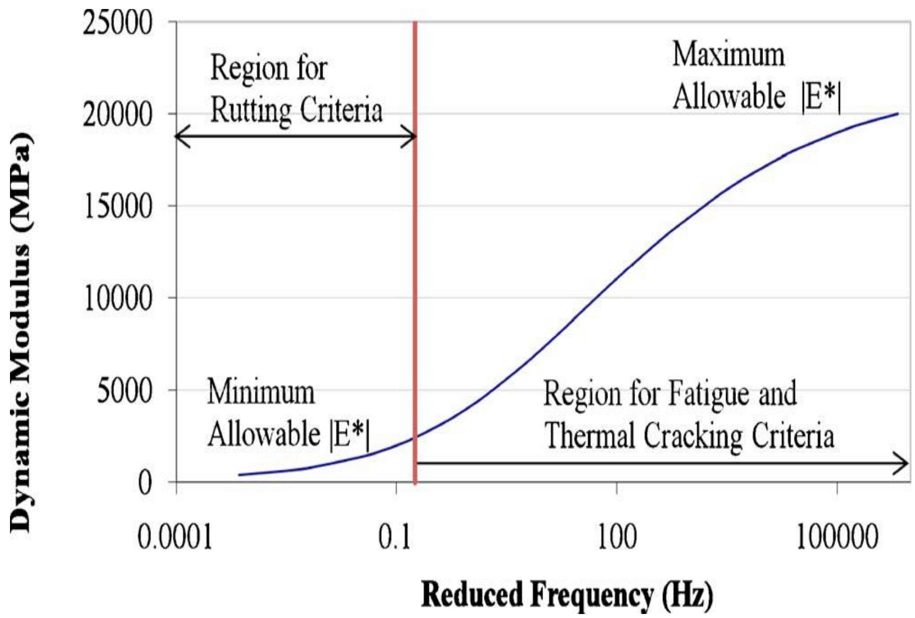


Figure 3.4 An example of a dynamic modulus master curve

In addition, the mixtures' performance was studied in terms of three main surface distresses using three dynamic moduli (|E*|) values from each mixture (Varamini, 2016):

- Thermal cracking using dynamic modulus (|E*|) value at temperature of -10°C and high frequency of 25 Hz.
- Fatigue cracking using dynamic modulus (|E*|) value at temperature of 21.1°C and medium frequency of 10 Hz.
- Rutting using dynamic modulus (|E*|) value at high temperature of 37.8°C and low frequency of 0.1 Hz.

Previous studies have shown that the normalized dynamic modulus (|E*|) values can be used to predict the mixtures performance under different conditions (Varamini, 2016).

3.3.2 Moisture Susceptibility Test

To evaluate the moisture damage, AASTHO T-283 procedure was used. Six replicate specimens (100mm (4 in.) diameter and 63mm (2.5 in.) height) from each mixture were compacted to 7% ± 0.5 air void content by using the Superpave Gyratory Compactor. Three specimens were tested under dry conditions while the other three were tested after going through moisture-conditioning. The dry specimens were conditioned at 25 ± 0.5°C (77±1°F) for two hours before performing indirect tensile strength (IDT) test, while the moisture conditioned samples were first saturated to between 70 and 80 percent then wrapped in a plastic film and kept in a plastic bag that contained 10 ± 0.5 ml of water and sealed. The specimens were then placed in the environmental chamber for a freezing cycle of 16 hours at -18 ± 3°C (0 ± 5°F). Afterwards, the specimens were removed from the plastic bag, unwrapped and then placed in a water bath at 60 ± 1°C (140±2°F) (thawing cycle) for 24 hours with 25mm (1 in.) of water above their surface. Finally, the specimens were placed in a 25 ± 0.5°C (77±1°F) water bath for 2 hours before testing (AASHTO T283, 2003). Figure 3.5 illustrates the moisture damage test procedure.

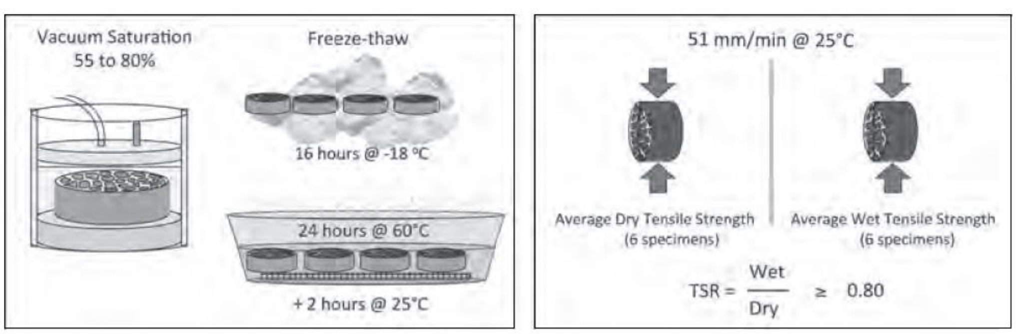


Figure 3.5 Moisture damage test (AASHTO T283, 2003)

3.3.3 Indirect Tensile Test

AASTHO T-283 was used to determine the tensile strength ratios (TSR). The strength of the specimens after freeze-thaw conditioning and dry specimens were determined and the TSR values were calculated as a ratio of the average tensile strength of the freeze-thaw subset to the dry subset. Figure 3.6 shows an example of the sample under loading at a constant rate of 50 mm/min (2 in/min).

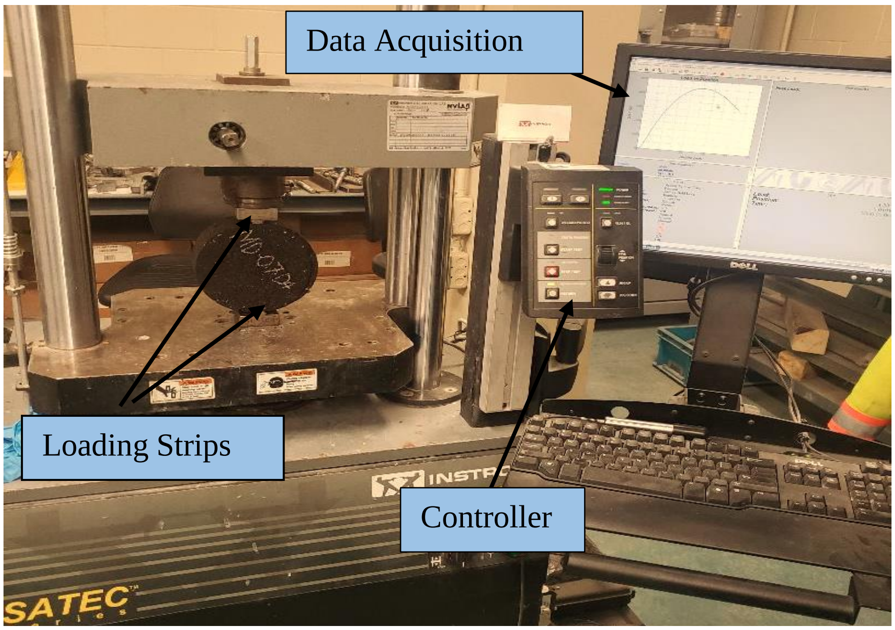


Figure 3.6 IDT test setup

The recorded maximum load was then used to calculate the tensile strength.

$$\text{Tensile Strength} = \frac{2P}{\pi t D} \qquad (3.1)$$

Where:

- P= maximum load
- t= sample thickness
- D= sample diameter

$$\mathrm{TSR} = \frac{\mathrm{S2}}{\mathrm{S1}} \tag{3.2}$$

Where:

- TSR= tensile strength ratio
- S1= average tensile strength of unconditioned sample
- S2= average tensile strength of conditioned sample

3.3.4 Hamburg Wheel-Tracking Test (HWTT)

The Hamburg wheel-tracking tests (HWTT) was used to evaluate the permanent deformation characteristics of the HMA and WMA mixtures. The HWTT was conducted in a water immersed state at elevated temperature to induce both permanent deformation and moisture damage in accordance with AASHT0 T324 (AASHTO T 324-17, 2018). To simulate the pavement performance in the field, two steel wheels apply force of approximately 702.8 N (158 lbs) on each specimen and use linear variable differential transducers (LVDT) to measure the rut depth after every wheel pass. The HWT test frequency is about 0.86 Hz (52 passes per minute).

Currently, the AASHTO standard does not specify the maximum rut depth for the HWTT. According to the literature, the test criteria need to be customized to fit the local conditions. Studies have shown that the total number of wheel passes, creep slope, and stripping slope of mixes in HWT tests are significantly affected by Recycled Asphalt Pavement (RAP) content, aggregate quality, binder grade, and asphalt sources (Rahman & Hossain, 2014). Several Department of Transportations (DOTs) in the United States have localized their criteria based mostly on the binder grade. Three DOTs in particular that have considered the Performance Grade (PG) of the binder similar to the one used in this study were Utah DOT (Utah Department of Transportation, 2013), Colorado DOT (Colorado Department of Transportation, 2015) and Texas DOT (Texas

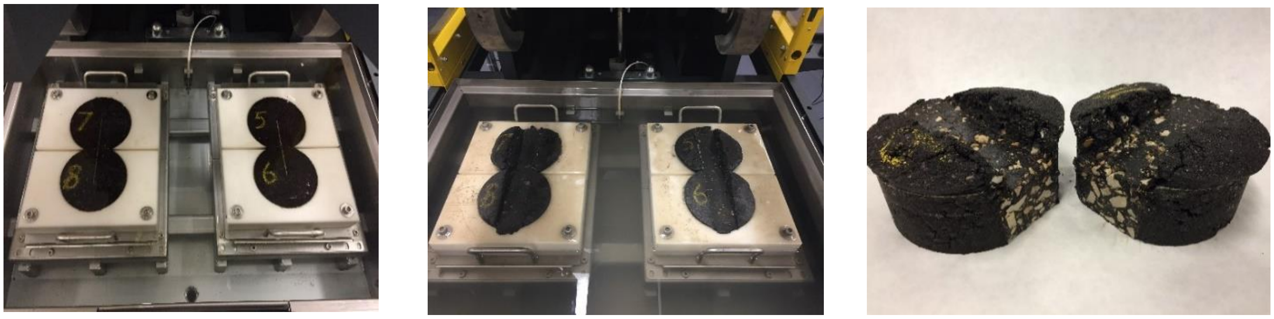
Department of Transportation (TxDOT), 2014). Table 3.5 shows the localized HWTT criteria for the DOTs.

Table 3.5 Localized HWTT criteria for different DOTs

Localized standard	**Maximum rut depth at failure (mm)**	**Maximum number of Passes at failure**	**Temperature**
Utah	20	20,000	PG58 - 46°C PG64 - 50°C PG70 - 54°C
Colorado	Rut depth > 4mm before 10,000 passes	20,000	PG58 - 45°C PG64 - 50°C PG70 - 55°C PG76 - 55°C
California	12.5	25,000	N/A
Illinois	12.5	20,000	N/A
Texas	12.5	PG <=64 - 10,000 passes PG 70 - 15,000 passes PG 76 - 20,000 passes	50°C

PG 58-28 binder was used in both HMA and WMA mixtures. The test criteria used in this study were the maximum rut depth of 12.5 mm at the center of the sample, maximum number of passes of 20,000 at failure, and test temperature of 45°C.

Specimens were compacted to 7±1 percent air void using the Superpave Gyratory Compactor. The dimensions of the cylindrical specimens were 150 mm (6 inches) in diameter and 64 mm (2.5 inches) in height. The specimens were cut in such a way that they can be paired to form a suitable pathway for the wheels, as shown in Figure 3.7. Six pairs of specimens were replicated for each mixture for repeatability.



(a) (b) (c)

Figure 3.7 HWTT setup a) Specimens before the test, b) Specimens after the test, c) Specimens separated after the test

The HWT test outputs five main parameters as follows:

1. Creep Slope: The average number of passes per 1mm deformation before stripping occurs, which indicates the rutting susceptibility.
2. Stripping Slope: The average number of passes per 1mm deformation during stripping, which suggests the rutting behaviour under extreme moisture damage.
3. Number of Passes at Failure: The total number of passes required to reach the maximum rut depth criteria in the middle of the travel pass.

The intersection between creep and stripping phases is the stripping inflection point (SIP) which can be obtained using the following equation (all parameter expressed in passes):

$$\text{Stripping Inflection Point (SIP)} = \frac{\text{intercept (stripping phase)} - \text{intercept (creep phase)}}{\text{slope (creep phase)} - \text{slope (stripping phase)}} \qquad (3.3)$$

4. Number of Passes to SIP: The total number of passes prior to the start of stripping.
5. Rut depth to SIP: The deformation in the middle before the start of stripping.

The moisture damage indicator from the HWT test is the stripping slope and SIP. It is a point from which moisture damage tends to dominate the performance of the mixture. The number of wheel passes to SIP as well as the rut depth at SIP can also be determined, as shown in Figure 3.8.

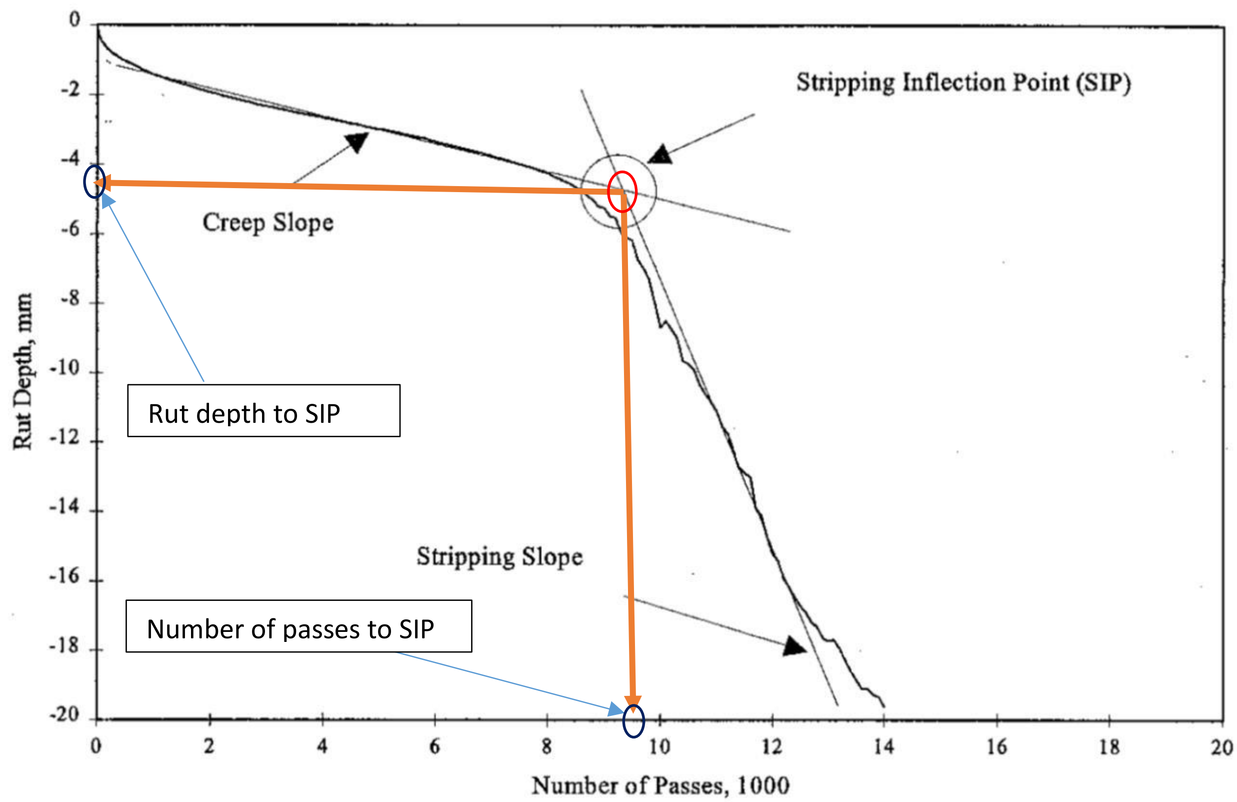


Figure 3.8 Graph of number of passes versus rut depth (AASHTO T 324-17, 2018)

3.3.5 Illinois Flexibility Index Test (I-FIT)

The cracking performance of asphalt mixtures has been measured using a variety of methods. Most of these methods were developed by different states in the United States (US). The five most common cracking tests include Semicircular Bend Test (Minnesota), Illinois Flexibility Index Test (Illinois), Texas Overlay Tester (Texas), Indirect Tensile Asphalt Cracking Test and Disc-Shaped Compact Tension Test (Newcomb & Zhou, 2018). In this research, two cracking tests were used to predict cracking performance of the four mixtures. Illinois Flexibility Index Test (I-FIT) and Indirect Tensile Asphalt Cracking Test (IDT) were conducted. For IDT cracking analysis, similar results obtained from the IDT values for dry specimens in moisture susceptibility test are used to evaluate the cracking performance.

The I-FIT was proposed by the University of Illinois as a method of measuring cracking performance by making a few alterations from the Semicircular Bend Test (SCB). The test represented in this paper was conducted in accordance with AASHTO TP124-18 standard (AASHTO T 124-18, 2018), and the Illinois Modified AASHTO TP 124-18 – Determining the fracture potential of asphalt mixture using the Flexibility Index Test (FIT) (Illinois Modified AASHTO TP 124, 2019). The I-FIT is a simple, reliable, yet robust and affordable test. The outcome of this test includes fracture energy, flexibility index (FI), post-peak slope (m), critical displacement and tensile strength (peak load related), which enable agencies and contractors to rank AC mixtures based on its cracking performance.

The test specimen was prepared using Superpave Gyratory Compactor (SCG) following AASHTO T 312 (cylindrical specimen with 150 mm diameter and final compaction of 160 mm or 140 mm height). Figure 3.9 below illustrate how the I-FIT specimens was prepared, and Figure 3.10 shows its dimensions. A total of 32 I-FIT specimen were produced, 8 I-FIT specimens for each mix. Finally, a notch with a depth of 15 ± 1 mm and width of 2.25 mm were cut in the middle of each I-FIT specimen. AASHTO TP124-18 and the IL MOD 2019 required the air void before notching of the I-FIT specimen to be 7 ± 1%.

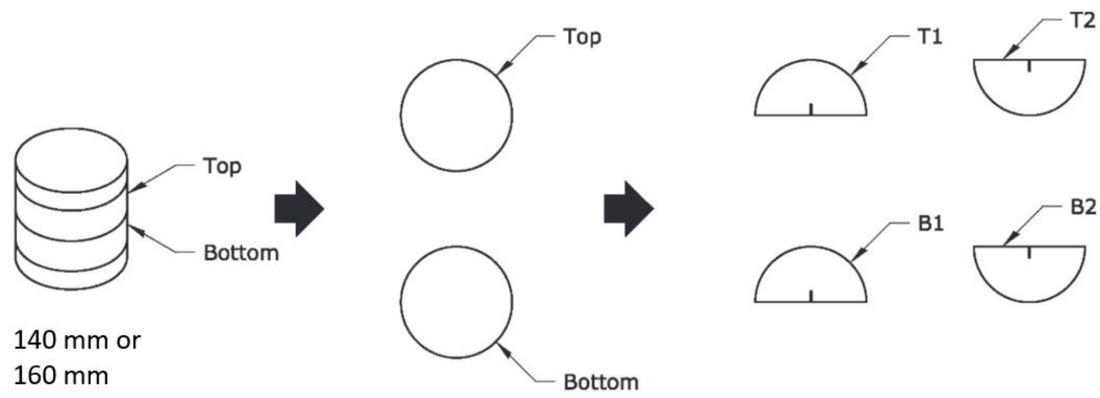


Figure 3.9 I-FIT specimen cutting location (Illinois Modified AASHTO TP 124, 2019)

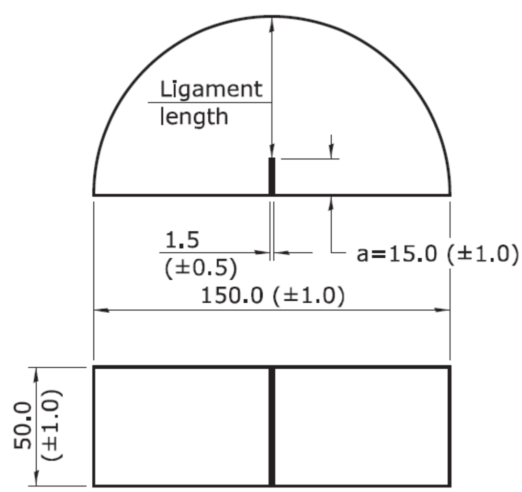


Figure 3.10 I-FIT specimen dimension (Illinois Modified AASHTO TP 124, 2019)

After air void was measured, the I-FIT specimen was left to air dry for approximately 18 hours before being conditioned in an environmental chamber at 25˚C for a minimum of 2 hours. After conditioning for a minimum of 2 hours, the I-FIT specimen was then aligned on the test apparatus.

Figure 3.11 illustrates the apparatus setup used to perform the cracking test as potrayed in AASHTO TP 124 – 18.

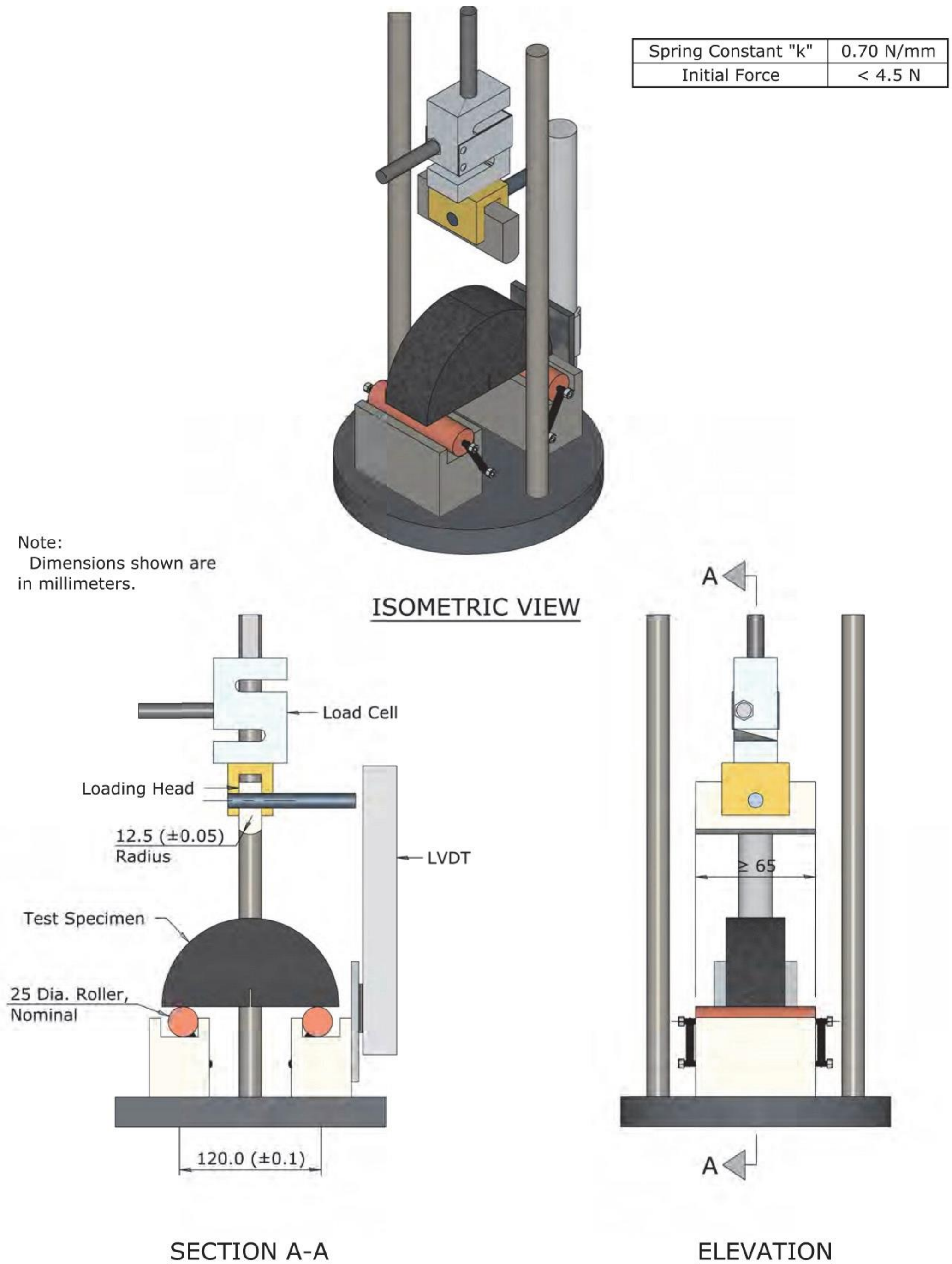


Figure 3.11 I-FIT setup (Illinois Modified AASHTO TP 124, 2019)

3.4 Field Testing

As part of the study, field monitoring during and after paving was critical. There were two main aspects of the project that required to be verified in the field; temperature behind the paver and field compaction densities. The two aspects were monitored by the use of equipment such as infrared thermal camera and nuclear density meter.

3.4.1 Infrared Thermal Imaging

The placement and compaction temperatures of WMA mixtures were monitored using the Infrared thermal camera. The main purpose was to verify that the temperature behind the paver does not exceed 130°C as required in the project specifications (City of Winnipeg, 2015). The temperature monitoring involved taking instant pictures as well as videotaping different stages of the construction. As shown in Figure 3.12, a FLIR T650 thermal camera was used to monitor the temperatures during paving.

Figure 3.12 Infared thermal camera

3.4.2 Degree of compaction

To analyze the effect of the additive to the workability of the mixtures, in-place densities of the paved sections were measured. The degree of compaction was determined using a nuclear density meter (densometer) in accordance with ASTM Standard D2950, Standard Method of Test for Density of Bituminous Concrete in Place by Nuclear Method (American Society for Testing and

Materials ASTM D2950/D2950M - 14, 2014). As required by the City of Winnipeg Specification, the measured in-place density of the completed paved sections shall be an average of ninety-seven (97%) percent of the 75 Blow Marshall Density of the paving mixture, with no individual test being less than ninety-five (95%) percent (City of Winnipeg, 2015). Table 3.6 shows the field conditions during paving. The ambient temperature was decreasing as construction progressed towards the fall season.

Table 3.6 Field conditions during construction

Mix	**Date of Construction**	**Ground Temp during compaction (°C)**	**Ambient Temp during compaction (°C)**	**Weather condition**
HMA	August 23, 2018	30	28	Sunny
0.3% WMA	September 27, 2018	11	14	Little rain
0.5% WMA	October 4, 2018	8	7	Cloudy
0.7% WMA	November 2,2018	3	0	Cloudy

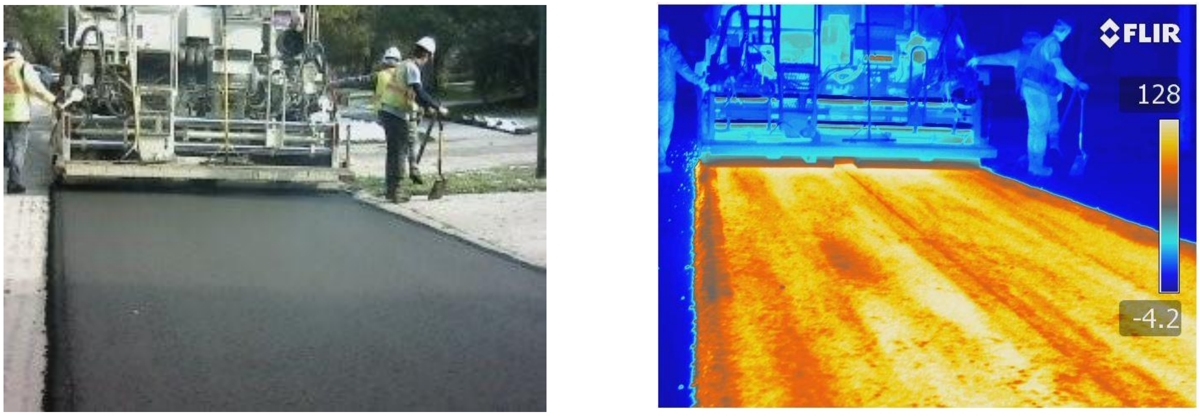
Chapter 4. Evaluation of Warm Mix Asphalt

4.1 Introduction

In this chapter, the analysis of data is divided into two major parts. First, field data that involved temperature behind the paver and degree of compaction for the paved sections was analyzed. Secondly, asphalt loose mixtures were collected from the field and taken to the laboratory to conduct different performance tests such as dynamic modulus, Hamburg wheel-tracking, moisture susceptibility and cracking resistance tests. The following subsections will provide the finding from each testsconducted.

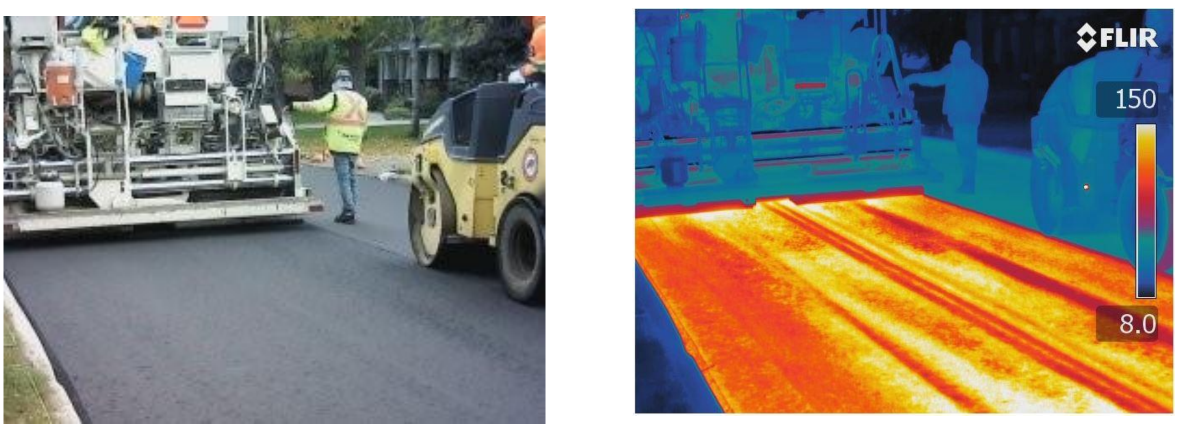
4.2 Infrared Thermal Imaging

The highest recorded temperature behind the paver for the WMA mixtures and HMA mixture was 128°C and 150°C respectively. This meant that the temperature requirement for WMA was achieved during the entire study. By having lower asphalt temperature during paving for WMA led to a lower cooling rate of the mixture compared to HMA when exposed to cooler weather and ground. Lower cooling rate of asphalt mixture provides some advantages such as allowing longer compaction period and density can still be achieved even at lower ground temperature. Figures 4.1 and 4.2 show examples of the thermal images of WMA and HMA paved sections behind the paver respectively.



a) Site image during paving b) Thermal image during paving

Figure 4.1 Paving using WMA technology showing max and min temperatures



a) Site image during paving b) Thermal image during paving

Figure 4.2 Paving using HMA technology showing max and min temperatures

4.3 Compaction Density

Densities of the compacted sections were determined using a nuclear density gage and the results were shown in Table 4.1. In general, WMA mixtures gained higher density than HMA. Also, there was a direct relationship between the additive dosages and compaction density values of the mixtures. As the chemical additive dosage increased, the degree of compaction increased even though the ambient temperature was decreasing. The increased density may be associated with the improved compactability for the WMA mixtures due to the increased workability.

Due to the fact that the project was executed by one contractor, it was not feasible to place different dosages at the same time to compare the effect of dosage under the same weather conditions. It would require the use of two or more storage tanks.

Table 4.1 Compaction density of mixtures at different dosage

Mix	Ambient Temp (°C)	Temp behind paver (°C)		Compaction Density (%)	
		Min	Max	Min	Max
HMA	28	141	150	97	98
0.3% WMA	14	104	125	97	99
0.5% WMA	7	102	128	98	100
0.7% WMA	0	109	127	98	100

4.4 Mixture Stiffness

The average dynamic modulus values for all the mixtures at the tested temperatures and frequency are shown on Table 4.2. The stiffness of the mixtures was compared using the dynamic modulus master curve presented in Figure 4.3. The master curve has the ability to characterize the performance of the mixture at different frequencies and temperatures (Williams et al., 2011). According to the master curve, WMA mixtures had slightly higher stiffness values at low and high temperatures compared to the HMA samples. However, the addition of WMA additive resulted in lowering stiffness values at intermediate temperatures. In general, all mixes showed very similar patterns in the mixture’s stiffness and ability to recover from induced stress.

Table 4.2 Average dynamic modulus values for each mixture

Temp, °C	Loading Freq (Hz)	Average Dynamic Modulus (MPa)			
		HMA	WMA (0.3%)	WMA (0.5%)	WMA (0.7%)
-10	**25**	25,696	25,802	26,100	26,350
	10	24,509	24,902	26,055	26,037
	5	22,889	23,227	24,565	24,142
	1	18,595	18,811	20,543	19,579
	0.5	16,977	17,160	18,666	17,470
	0.1	13,242	13,374	14,883	13,374
4.4	**25**	15,416	16,383	15,708	13,738
	10	13,277	13,964	14,016	11,259
	5	11,986	12,194	12,206	9,227
	1	8,468	8,555	8,534	6,312
	0.5	7,210	7,168	7,186	5,620
	0.1	4,866	4,753	4,786	3,858
21.1	**25**	5,548	5,054	5,106	5,664
	10	4,221	3,653	3,937	4,055
	5	3,250	2,966	3,134	3,165
	1	1,732	1,714	1,739	1,729
	0.5	1,351	1,387	1,404	1,366
	0.1	778	873	847	798
37.8	**25**	1,700	1,715	1,623	1,556
	10	1,090	1,165	1002	994
	5	818	866	752	704
	1	476	452	399	386
	0.5	372	356	332	315
	0.1	215	213	217	216

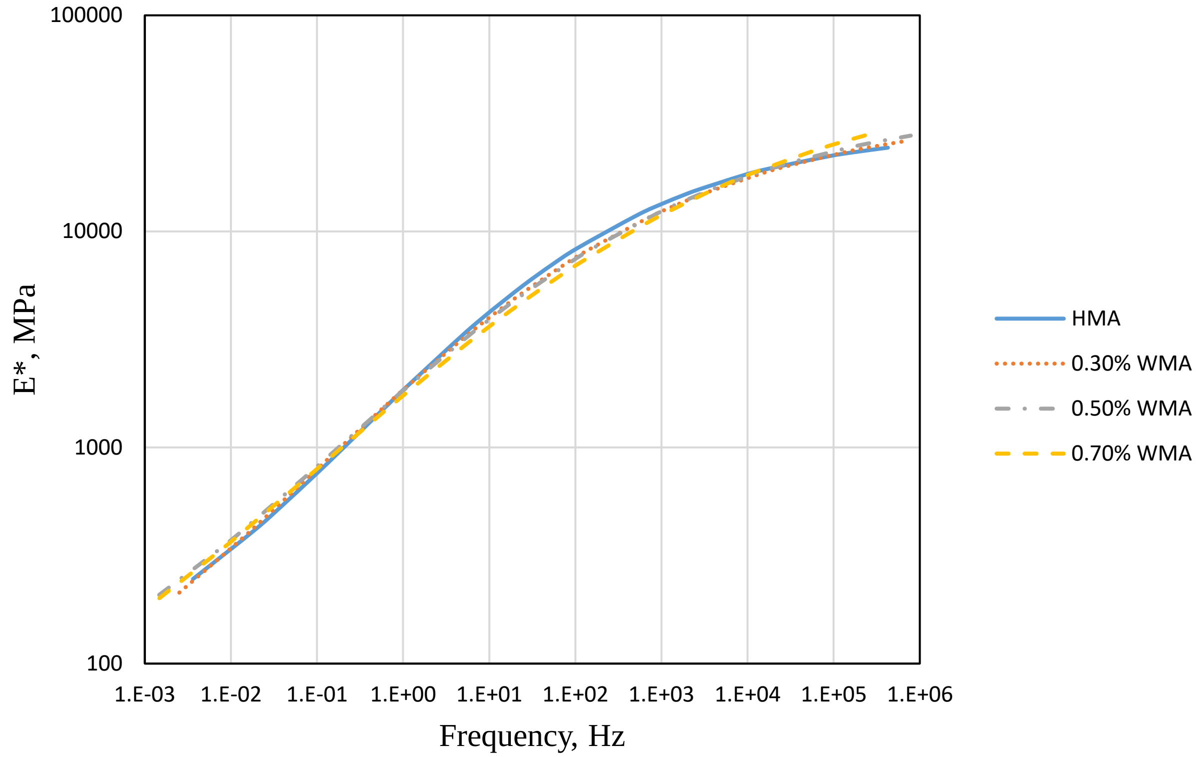


Figure 4.3 Dynamic modulus master curve of all mixtures

According to the literature, a comparison of the dynamic modulus values (|E*|) at three specific temperatures and frequencies could be used to predict the performance of the mixtures to thermal cracking, fatigue cracking and rutting (Varamini, 2016). The ratio of each warm mix to hot mix dynamic modulus values was determined. If the ratio is greater than 1, it indicates that the |E*| value of WMA is higher than that of HMA and vice versa. The temperature degrees (frequencies) used are shown in Table 4.3.

Table 4.3 Temperature and frequency values for distress prediction

Distress Resistance	Temperature	Frequency	WMA/HMA (|E*|)
Thermal cracking	-10°C	25 Hz	Better if < 1.0
Fatigue cracking	21.1°C	10 Hz	Better if < 1.0
Rutting	37.8°C	0.1 Hz	Better if > 1.0

From the normalized dynamic modulus values, the following was observed:

- The resistance to thermal cracking may not be improved by the use of warm mix technique as shown in Figure 4.4. This resulted from a slight increase in stiffness of the WMA mixtures compared to HMA at colder temperatures.
- The resistance to fatigue cracking may be improved using warm mix technique as presented in Figure 4.5. The addition of WMA additives led to a decrease in stiffness compared to the control mixture at intermediate temperatures. Lower stiffness indicates a higher ductility of the mixture.
- The resistance to rutting may not be improved by the use of warm mix technique as revealed in Figure 4.6. This is due to similar stiffness values between using WMA and HMA.

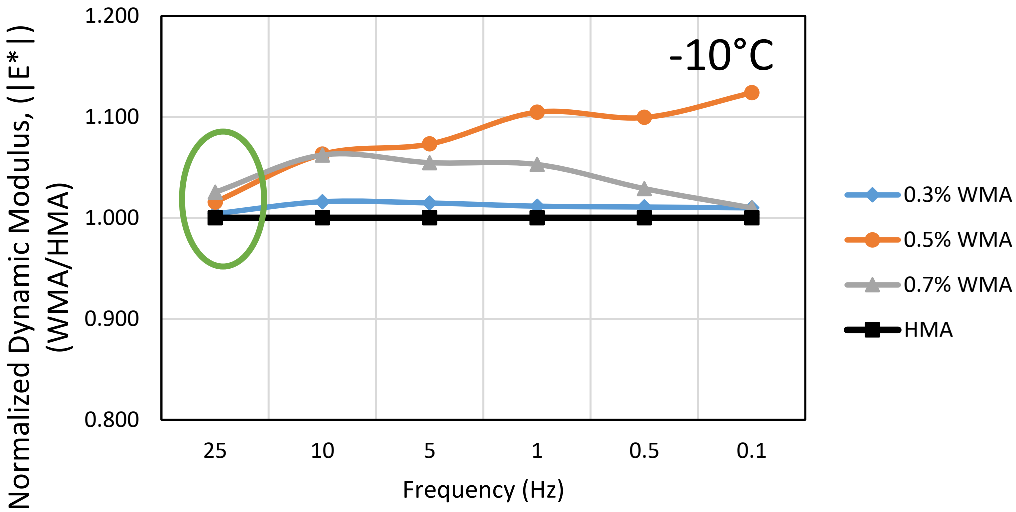


Figure 4.4 Normalized dynamic modulus results in terms of resistance to thermal cracking

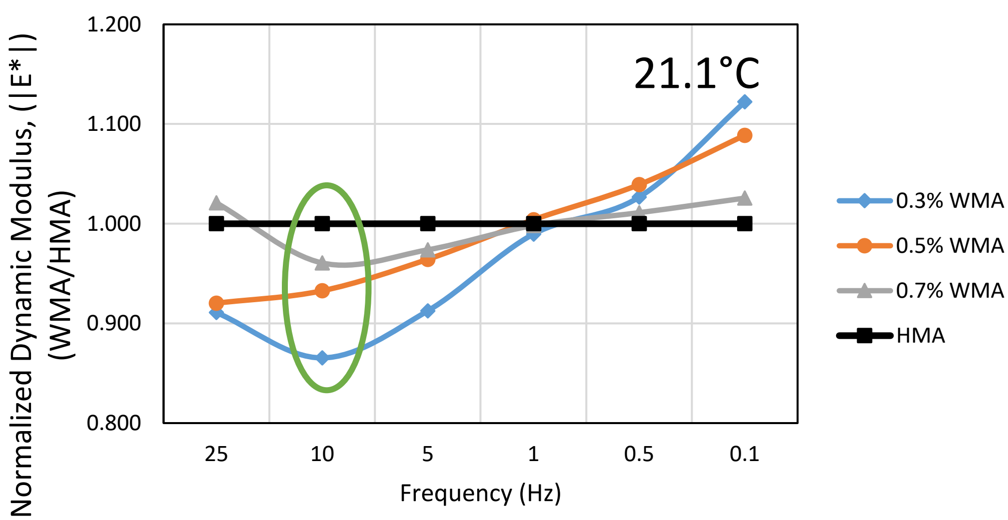


Figure 4.5 Normalized dynamic modulus results in terms of resistance to fatigue cracking

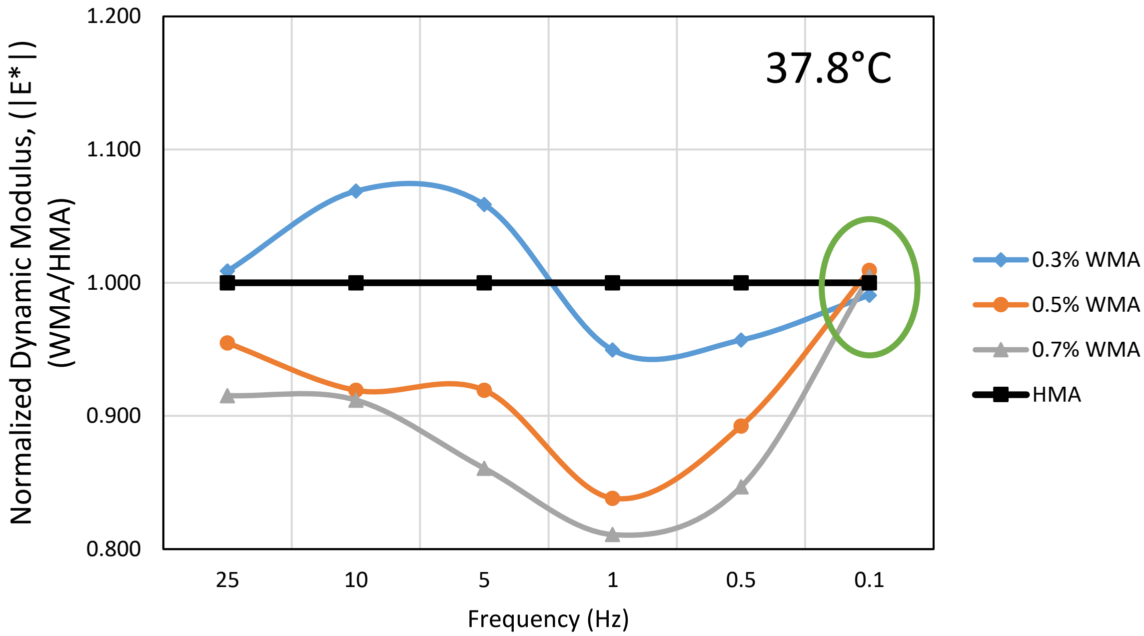


Figure 4.6 Normalized dynamic modulus results in terms of resistance to rutting

4.5 Moisture Susceptibility

Asphalt moisture damage causes loss in adhesion that directly reduces the strength of the asphalt mixture. When the strength of the asphalt mixture is compromised, other pavement failures such as rutting and raveling are likely to occur (Park et al., 2017). The resistance to moisture damage of the four mixtures was evaluated as the percentage of indirect tensile strength ratio between moisture-conditioned samples and dry samples. Figure 4.7 shows the average tensile strength for all dry specimens. The highest tensile strength observed was of HMA specimens at 525 kPa while the lowest was of 0.7% WMA at 482 kPa. Analysis of Variance (ANOVA) with 95% confidence interval confirmed that there are statistical differences between the tensile strength values of the HMA and WMA mixtures with a trend for all dosage rates. The high IDT values of HMA dry specimens can be attributed due to a more aged binder hence high stiffness. Previous studies have shown that the moisture damage resistance of WMA are very dependent to the curing time of the WMA specimen. In addition, the moisture damage resistance of WMA tends to improve as the ageing time increases (Mogawer et al., 2011).

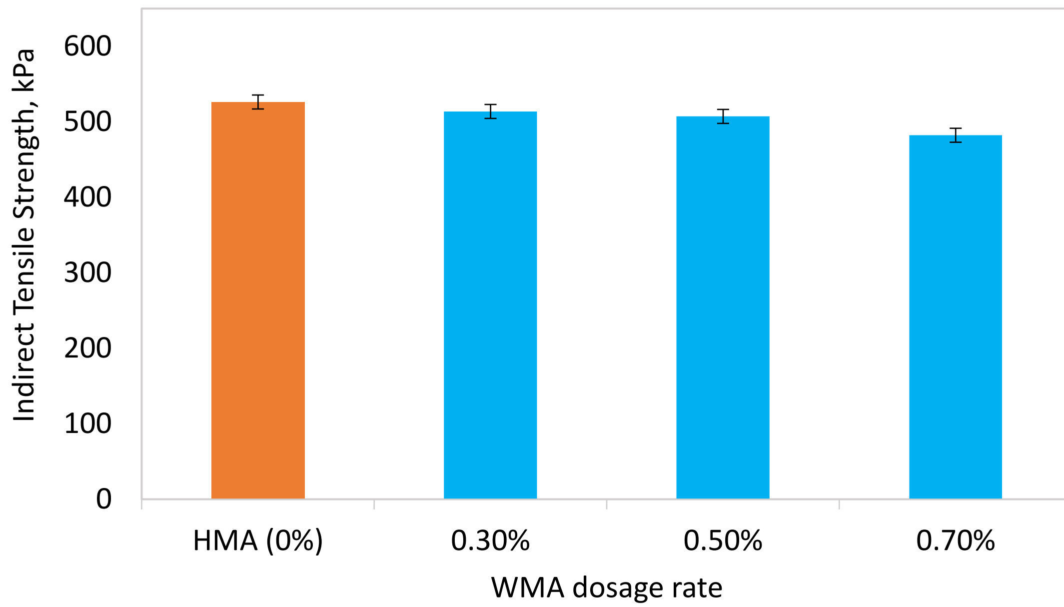


Figure 4.7 Indirect tensile strength values for dry samples

In addition, the tensile strength ratio (TSR) values of the four mixtures met the 80 percent minimum required value as shown in Figure 4.8. In general, all WMA mixtures led to higher TSR values compared to the HMA mixture. The ANOVA analysis suggested that there is a statistically significant difference between the TSR values for the HMA compared to the WMA. However, no statistical difference between the three WMA mixtures was found. Among the three warm mix asphalts, the 0.5% dosage had the highest average TSR value which indicates a better resistance to moisture damage.

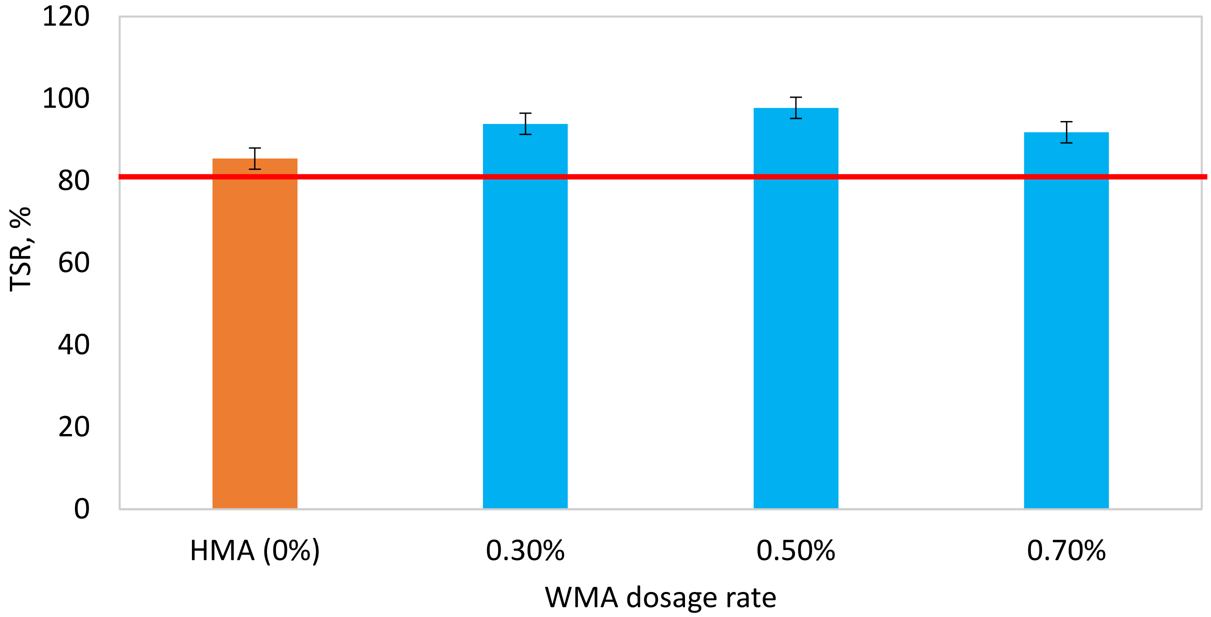


Figure 4.8 TSR ratios between indirect tensile strength of conditioned and unconditioned specimens

Although there is wide use of AASTHO T-283 to predict moisture implications to asphalt pavement within the industry, there are some studies that suggested deficiencies of using this approach. The major setback is the poor correlation between the TSR values and field performance reported in some projects. In addition, the TSR value is very sensitive to even small differences in air voids, aggregate arrangements, specimen size, degree of saturation and conditioning temperatures (Varamini, 2016) . Furtheremore, it has been argued that the test procedure does not accurately simulate traffic loading that pavements experience in the field (Kandhal & Rickards, 2002). Therefore, there has been an increased need to find a replacement for this test.

Previous studies have suggested the use of Dynamic Modulus Ratio (DMR) instead of TSR. This is mainly because of the sinusoidal loading used in dynamic modulus test which better represents the field pavement loading. One test temperature of 21.1ºC and frequencies of 0.1, 1 and 10 Hz were used for better representation of the results. This temperature is close to the 25ºC test temperature used in the tensile strength test. Similarly, the specimens will undergo one freeze and thaw cycle before being tested. The DMR measures the percentage of retained modulus of elasticity . However, there is no standard for this test and therefore it was not part of the study.

Identifying the challenges with TSR, further assessment of moisture damage was performed using the hamburg wheel tracking test.

4.6 Rutting Resistance

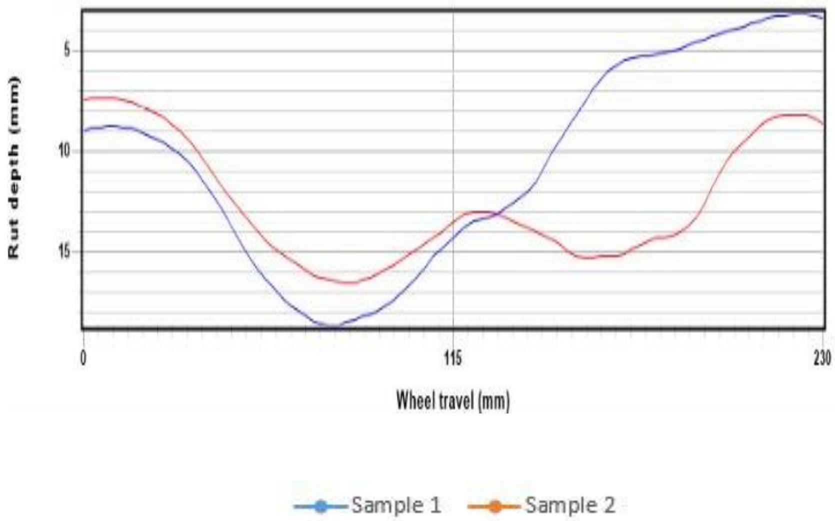
4.6.1 Hamburg Wheel-Tracking Test (HWTT)

The rutting and moisture damage analysis of the four mixtures was conducted using the HWTT. Five main parameters are usually evaluated which include, the number of passes to SIP, rut depth at SIP, creep slope, stripping slope and number of passes to failure. The data analysis of HWTT depends on the end criteria of the test, which is either the maximum allowable rut depth at the center of the sample or the maximum number of wheel passes. Three replicates were tested for each mixture and the average values were used for comparison.

Fugure 4.9 shows an example of the pictures and rutting profiles of tested samples at the end of the test for each mixture. The length of travel for the wheels was 230mm, making the centre of the tested sample be at 115mm from either ends of the wheel path. The test was set to stop when both samples reach a rut depth of 14mm at the centre of the wheel path. The HWT machine does not have the ability to stop one wheel when the end criteria is reached, therefore, one of the samples will normally rut more than the other by the end of the test.

Sample 1

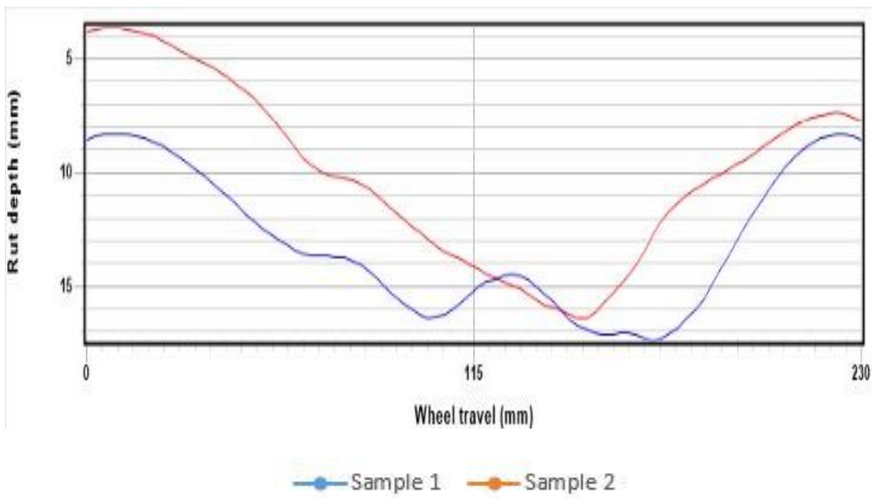
Sample 2



HMA rutting profile

Sample 1

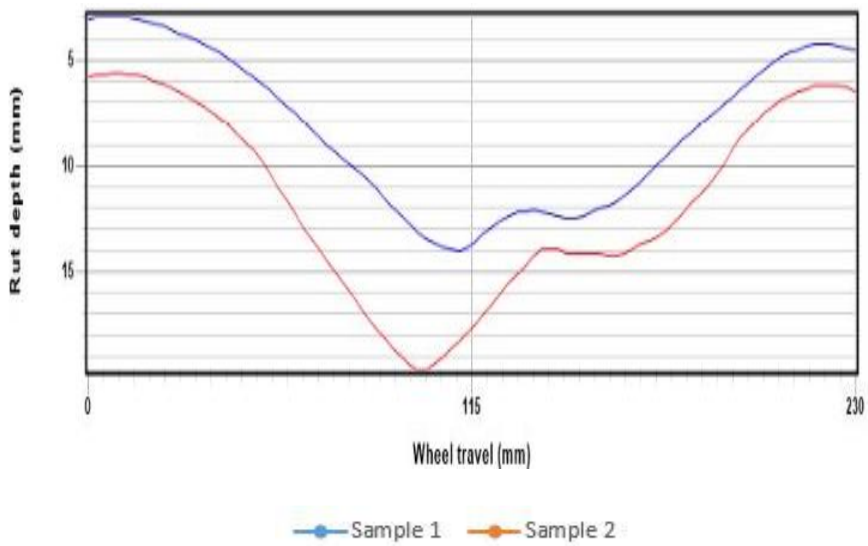
Sample 2



WMA 0.3% rutting profile

Sample 1

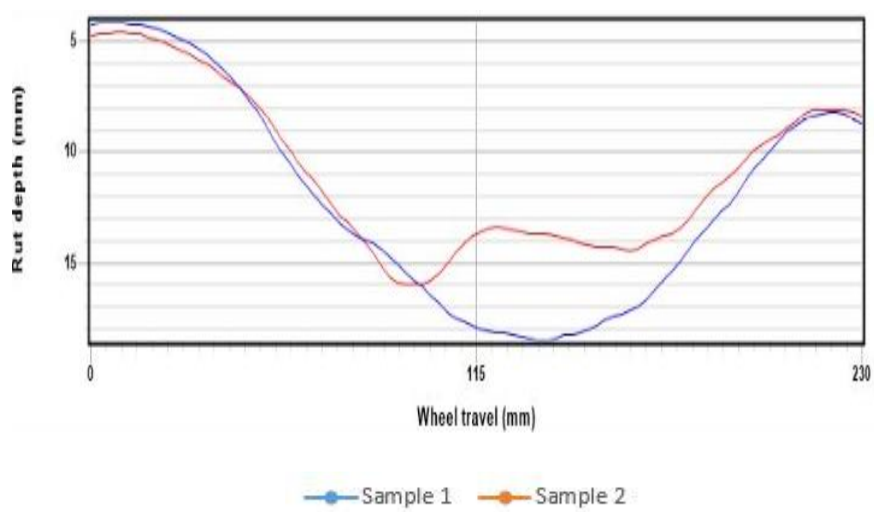
Sample 2



WMA 0.5% rutting profile

Sample 1

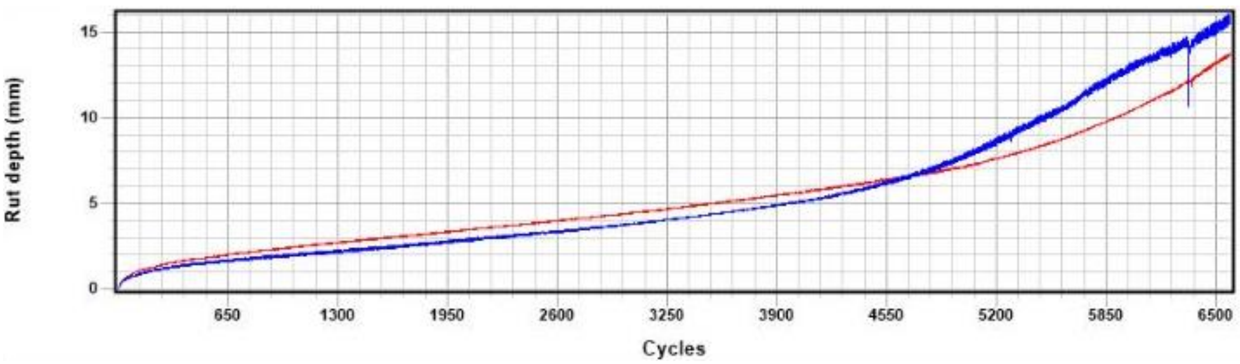
Sample 2



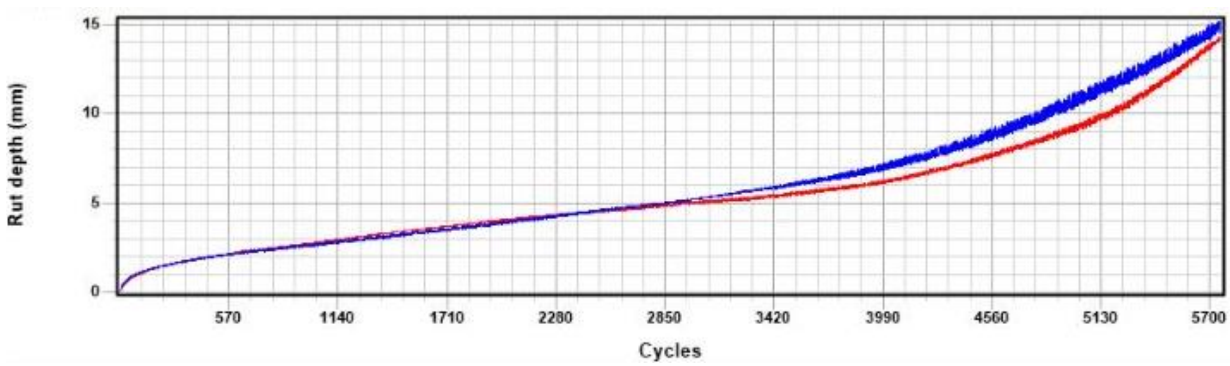
WMA 0.7% rutting profile

Figure 4.9 Rutting profiles at the end of the tests

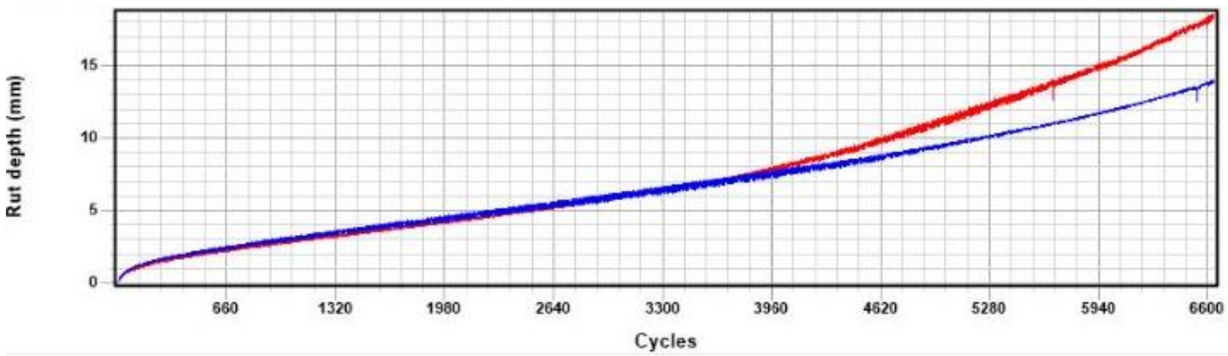
Figure 4.10 illustrates the rutting curve for each mixture which consists of two samples. The average for each mixture was used to compute the output parameters.



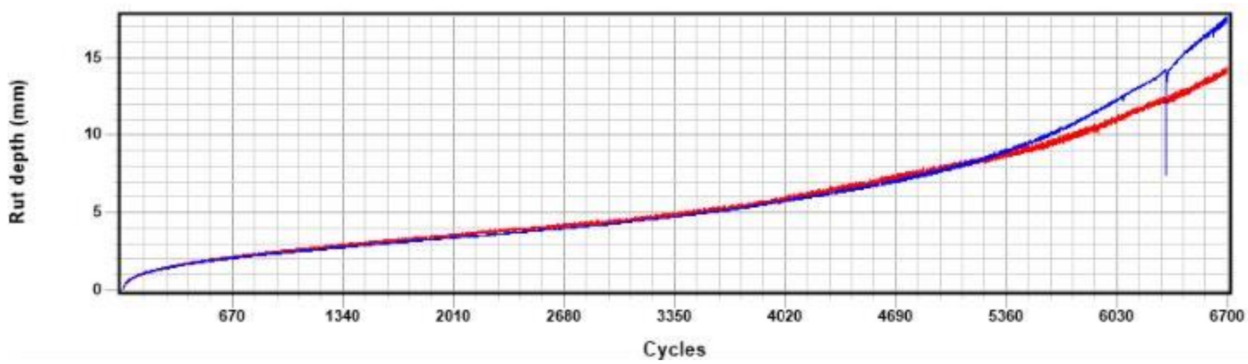
HMA



WMA 0.3%



WMA 0.5%



WMA 0.7%

Figure 4.10 The rutting graphs of HWT test for the four mixtures

Figure 4.11 portrays the rut depth versus number of wheel passes of the four mixes, and Table 4.4 shows the HWTT parameters. All the mixtures had low to fair resistance to permanent deformation and the test was stopped when the maximum allowable rut depth (12.5mm) was reached. Although all the mixtures failed at the maximum allowable rut depth, the mixtures can still perform well because they were placed on local street roads with low traffic-load. The most important parameter used to compare the results for each mixture was the number of wheels passes to failure (12.5 mm rut depth).

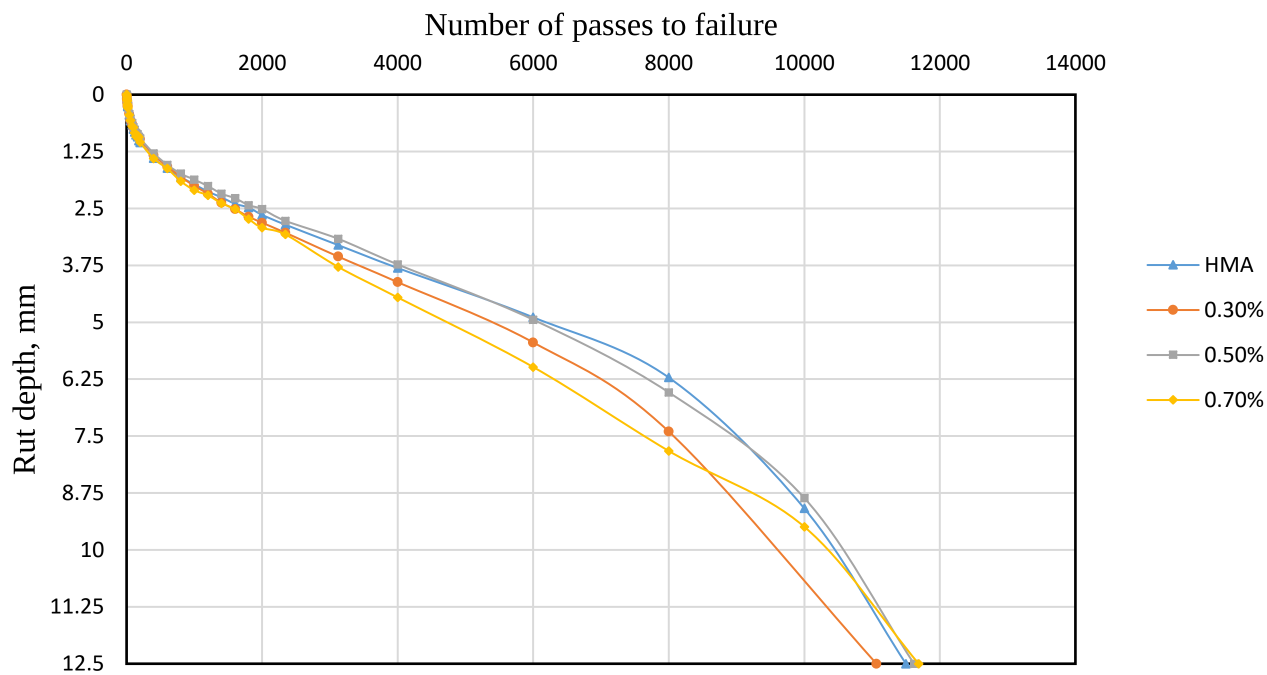


Figure 4.11 HWTT rut depth versus number of passes

Table 4.4 HWTT parameters results

Mix	**Number of passes to SIP**	**Rut depth to SIP (mm)**	**Creep slope (mm/pass)**	**Stripping slope (mm/pass)**	**Number of passes to failure (12.5mm)**
HMA	9394	6.61	0.000530	0.00284	11499
WMA (0.3%)	8978	7.00	0.000619	0.00269	11064
WMA (0.5%)	9513	6.73	0.000573	0.00267	11619
WMA (0.7%)	9201	7.39	0.000657	0.00210	11687

The results show that the 0.7% WMA and 0.5% WMA had better resistance to rutting as shown by the higher number of passes at failure than the other two mixture as shown in Table 4.4. To evaluate the statistical difference among the four mixtures, the ANOVA analysis with 95%

confidence level was utilized based on each HWTT parameter. The results suggested that there is no statistically significance difference among the mixtures for all parameters.

The hamburg wheel was also used to evaluate the moisture susceptibility of the mixtures. The results showed a slight reduction in stripping slope when WMA additive was used compared to the HMA. This indicated that the chemical additive played a role in reducing the moisture damage rate for the WMA specimens by acting as an antistripping agent.

4.6.2 Correlation between HWTT and dynamic modulus values at 37.8°C

According to the literature, the dynamic modulus (|E*|) values tested at high temperatures and low frequencies have shown a strong correlation with the HWTT (Safiuddin et al., 2014; Walubita et al., 2019, 2013). The highest temperature that was used in this study to test the dynamic modulus of all mixtures was 37.8°C. In addition, the HWTT parameter that showed highest correlation with the |E*| values from most studies is number of passes to failure. To study the correlation of these two tests, the coefficient of determination (R^2) was calculated between the $|E^*|_{37.8°C}$ at all frequencies and number of passes to failure. The calculated R^2 values are shown in Table 4.5.

Table 4.5 Correlation (R^2 values) between |E*| values at 37.8°C and number of passes to failure

HWTT parameters	**0.1 Hz**	**0.5 Hz**	**1 Hz**	**5 Hz**	**10 Hz**	**25 Hz**
Number of passes to failure	0.8605	0.3278	0.368	0.8236	0.9082	0.6369

The highest correlation between the |E*| values at 37.8°C and number of passes to failure was found at frequencies of 0.1 Hz, 5 Hz and 10 Hz. The highest R^2 values observed was 0.9082 at 10 Hz. The R^2 values of 0.8605 at 0.1 Hz and 0.8236 at 5 Hz were the second and third highest respectively. From the literature, the frequency showing the highest correlation varied in different studies. Safiuddin et al (2014) suggested the highest correlation between HWTT and |E*| values at 0.1 Hz while Walubita et al (2019) suggested the highest correlation at 10 Hz. Both studies concluded that the number of passes to failure as the HWTT parameter that had the highest correlation to dynamic modulus values at high temperatures.

This strong relationship between |E*| values at high temperature and HWTT parameters could be associated with the fact that the test temperature of HWTT (45°C) is very similar to the highest test temperature used in the dynamic modulus test (37.8°C). Furthermore, the reason for a higher correlation between $|E^*|_{37.8°C}$ values at lower frequency could be linked to the rate at which the wheels pass through the specimens during HWTT. The wheel's speed of 52 passes per minute is equivalent to 0.86 Hz that is close to the low frequencies used in dynamic modulus.

4.7 Cracking Resistance

Although the IL – SCB test is fast and simple, the results from the test is significant. The results from this test include FI value, fracture energy, post-peak slope, critical displacement, peak load and tensile strength. Currently, the City of Winnipeg and the Manitoba Infrastructure do not have an acceptance/rejection ranges for these values. More testing needs to be carried out to develop these ranges. Nevertheless, the IL – SCB test enable agencies to rank the asphalt mixture in term of cracking performance.

The I-FIT outputs six main parameters as follows:

1. Peak Load and Tensile Strength

Peak load is the maximum load that is applied to the specimen during testing. Tensile strength (F_f) is calculated using equation 4.1.

$$F_f = \frac{\mathrm{P}}{2\mathrm{rt}} \tag{4.1}$$

Where P is the applied load, r: the radius of the specimen, t: the thickness of the specimen.

A higher peak load results in a higher tensile strength of the mixture when comparing samples with similar dimensions. The peak load and tensile strength can be used in conjunction with the post-peak slope to predict the rutting performance of the mixture. In general, higher tensile strength (peak load related), combining with higher post-peak slope, increases the rutting resistance of the asphalt mixture (Al-Qadi et al., 2015).

2. Post-Peak Slope and Critical Displacement

Post – peak slope (m) is the slope at the first inflection point of the load-displacement curve after the peak load. Post – peak slope can be used as an energy dissipation index. A lower post – peak slope represents a low rate of energy dissipation (ductile material), while a higher post – peak slope represents a high rate of energy dissipation (brittle material) (AASHTO T 124-18, 2018). Critical displacement is the interception between the post – peak slope and the displacement axis.

3. Fracture Energy

Fracture energy is the total energy require to fail the specimen completely. Fracture energy is calculated by dividing the work of fracture (area under the load-displacement curve) by the ligament area (a product of ligament length and the thickness of an I-FIT specimen) (AASHTO T

124-18, 2018). Previously, fracture energy was used to rank asphalt mixes in terms of cracking performance. However, fracture energy does not consider the rate of energy dissipation after cracking (post – peak slope). Therefore, fracture energy alone may not be adequate to distinguish asphalt mixes.

4. Flexibility Index

Flexibility Index (FI) is obtained by dividing fracture energy by the post – peak slope. Since FI consider the post – peak slope FI, it can precisely rank asphalt mixes in term of long-term cracking performance. Mixture with higher FI value can resist crack propagation for longer time duration under tensile stress (AASHTO T 124-18, 2018). Figure 4.12 illustrates an example of the load – displacement curve developed by this test.

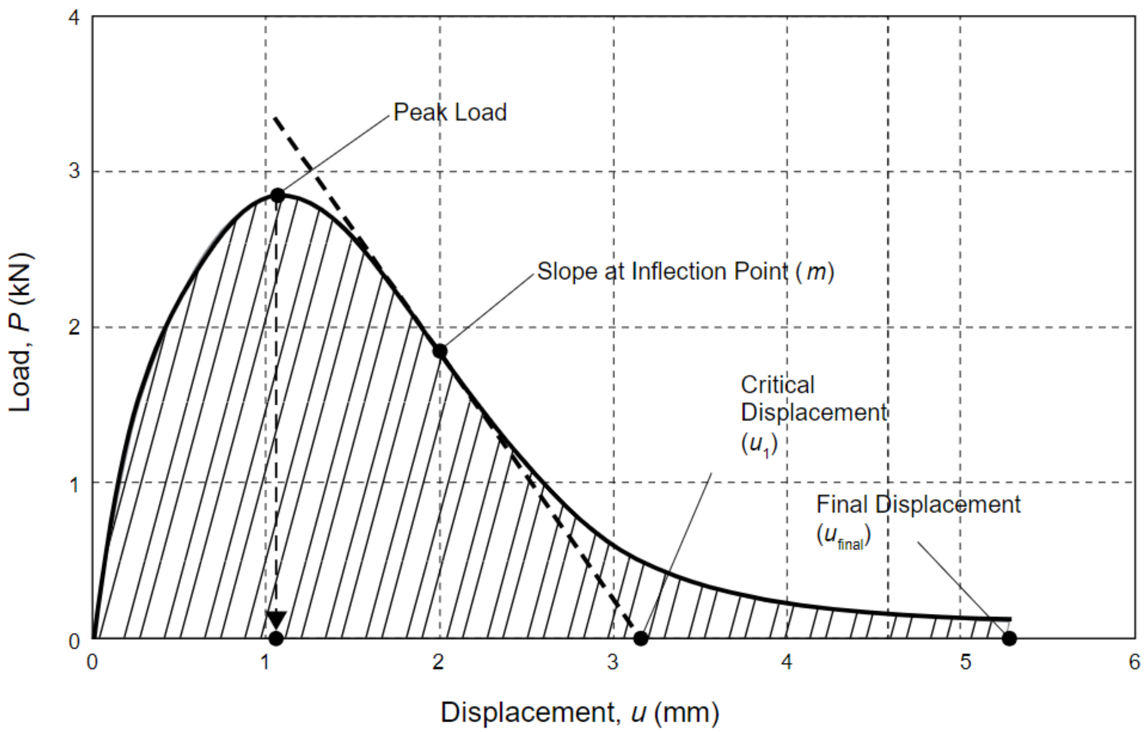


Figure 4.12 Typical load – displacement curve of an I-FIT specimen (AASHTO T 124-18, 2018)

Figure 4.13 illustrates the apparatus used to perform the cracking test and Figure 4.14 illustrates the specimen condition before and after a test.

The fixture was comprised of two main assemblies: top and bottom halves.

- The top half consisted of the loading head with a contact curvature radius of 12.5mm, able to rotate about a pivot to conform to slight specimen variations.
- The bottom half consisted of a base plate, two V-blocks spaced at 120mm (centre to centre), and two 25 mm diameter rollers.

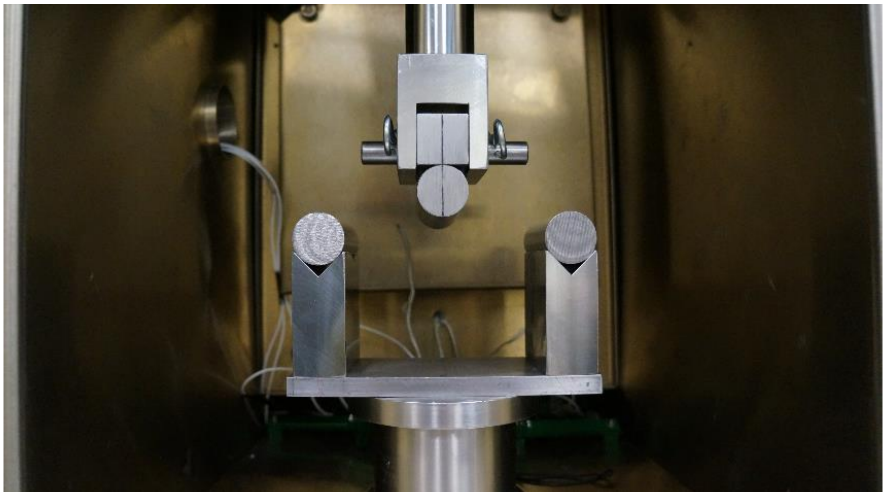


Figure 4.13 Laboratory I-FIT setup

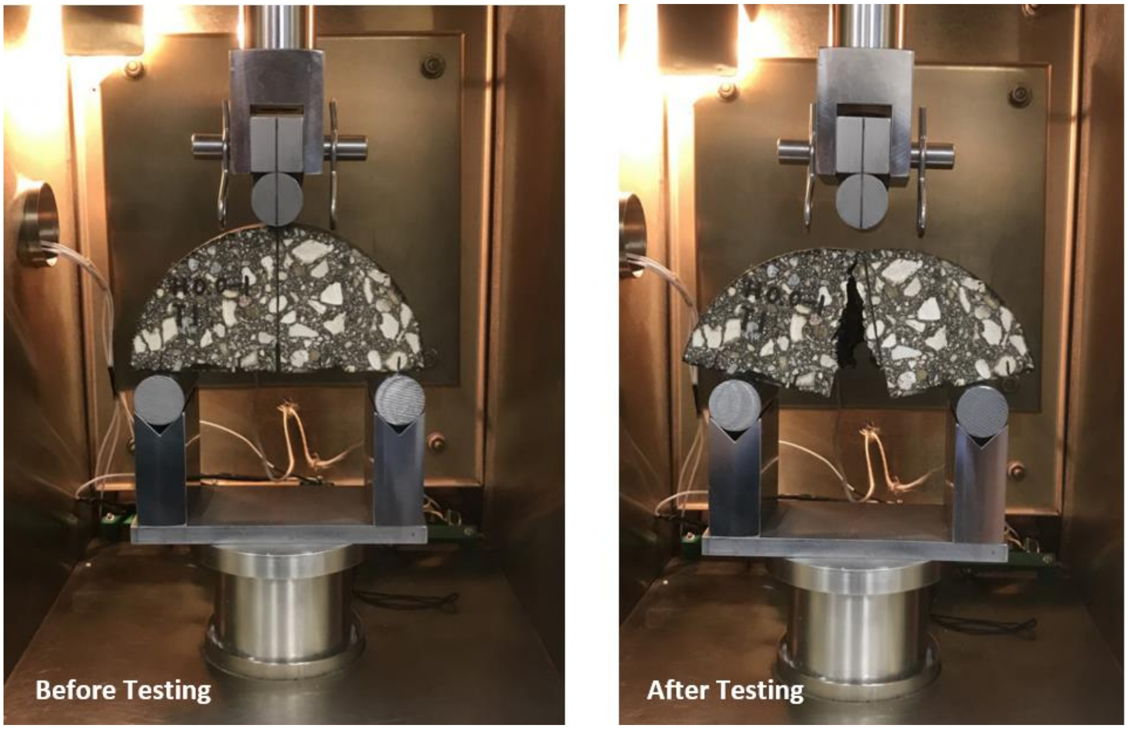


Figure 4.14 Before and after testing of specimen

Table 4.6 shows the results of the I-FIT in terms of all key parameters and Figure 4.15 shows the load-displacement curve of all four mixtures. The warm mixtures have higher FI values compare to HMA. Although HMA has a higher fracture energy compared to the warm mixes, it dissipates energy at a higher rate making it a more brittle material. Similar to post – peak slope, critical displacement is also a ductility indicator. The warm mixes have higher critical displacement (lower slope) which indicates a more ductile asphalt mix.

Table 4.6 Illinois modified semi-circular bend test result

Cracking Parameters		HMA 0.0%	WMA 0.3%	WMA 0.5%	WMA 0.7%
Flexibility Index	Average	10.11	13.11	12.59	12.38
	Standard Deviation	0.87	4.46	1.52	2.76
	COV %	8.60	34.03	12.05	22.30
Post-Peak Slope	Average	-1.73	-1.11	-1.10	-1.04
	Standard Deviation	0.24	0.24	0.10	0.27
	COV %	14.06	21.13	9.01	26.37
Fracture Energy (J/m^2)	Average	1698.00	1431.80	1390.30	1218.30
	Standard Deviation	153.92	230.02	109.06	97.24
	COV %	9.06	16.07	7.84	7.98
Strength (Psi)	Average	46.28	36.69	35.54	31.78
	Standard Deviation	4.47	4.46	2.47	2.80
	COV %	9.66	12.17	6.95	8.82
Critical Displacement (mm)	Average	2.99	3.35	3.26	3.16
	Standard Deviation	0.16	0.45	0.19	0.23
	COV %	5.20	13.31	5.74	7.19
Peak Load (KN)	Average	2.32	1.83	1.80	1.59
	Standard Deviation	0.25	0.23	0.12	0.16
	COV %	10.84	12.34	6.68	10.30

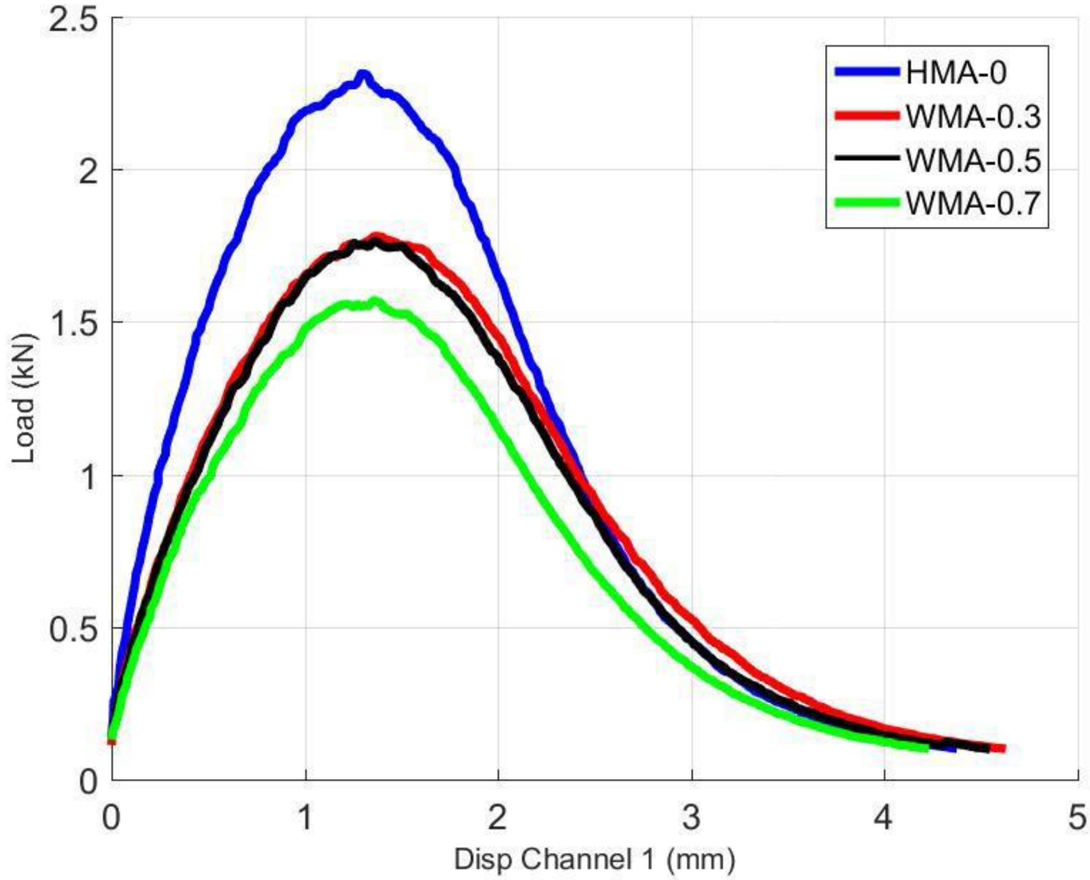


Figure 4.15 Load-displacement curve of all mixtures

One of the factors that directly affect the FI result and distinguish the mixes in terms of cracking performance in this project is the chemical additive used for WMA. As mentioned earlier, the amount of modifier added to each warm mix was different (0.3%, 0.5% and 0.7% by weight of the binder content). Cracking propagation typically occurs at the weakest link in the mixture which is the transition zone around the aggregates. Since the chemical additive used have the ability to increase the coating of binder to aggregates and acts as an adhesion enhancer, it led to an improvement in cracking resistance of the mixtures containing chemical additives. Figure 4.16 shows the cracking propagation around the aggregates.

Figure 4.16 Cracking propagation around aggregates

However, some specimens might contain weaker aggregates that would yield as cracking progressed. This was rarely observed during the cracking analysis of the mixtures because typically aggregates are the hardest constituents of asphalt mixture due to their high modulus of elasticity (stiffness). Figure 4.17 illustrates a cracking propagation through an aggregate.

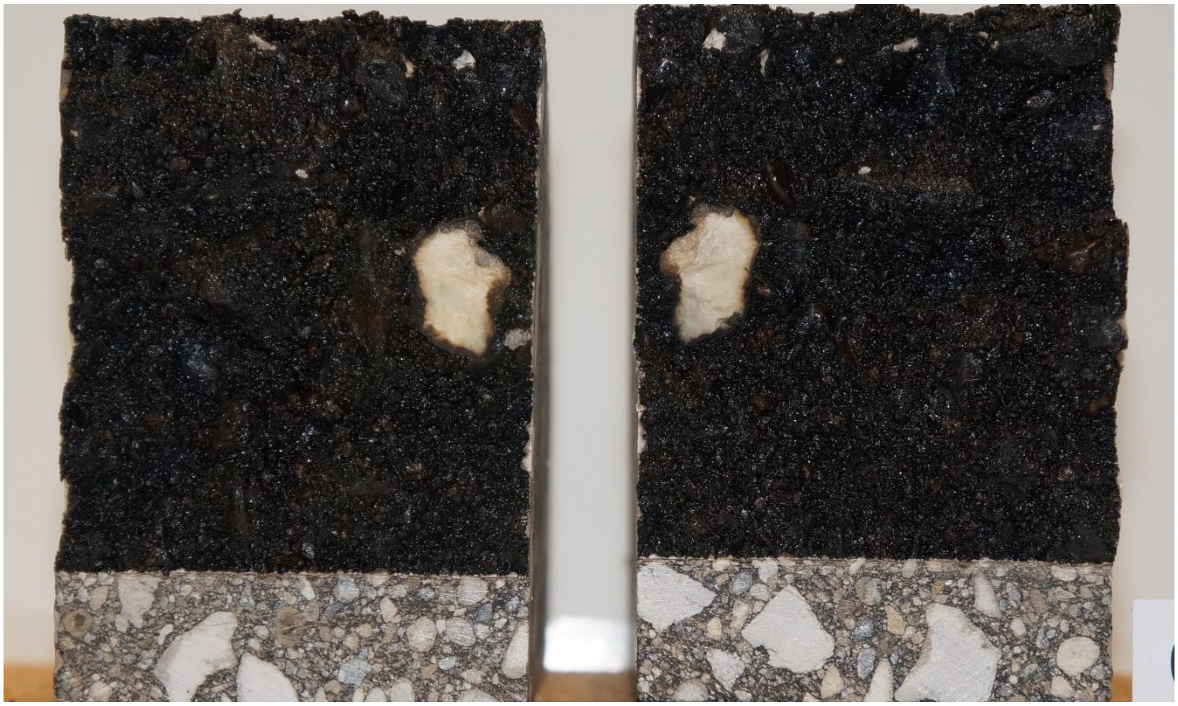


Figure 4.17 Cracking propagation through an aggregate

Another aspect that affected the FI result, and ultimately the cracking performance of the asphalt mixture is the temperature in which the mixtures were produced at the plant and compaction temperature. Mixing and compacting temperature affect the penetration grading and the overall ductility of the asphalt mixture. Higher mixing and compacting temperature cause the binder to loose a significant amount of asphaltene and maltene which causes the binder to age. As a result of an aged binder, the penetration grading is reduced and making the mixture stiffer. This results to a mixture with a lower FI value. Hot mix asphalt is typically mixed and compacted at approximately 140˚C - 160˚C, while warm mix asphalt is typically mixed and compacted at approximately 110˚C - 130˚C.

4.7.1 Correlation between Tensile Strength values of I-FIT and Moisture damage test

Furthermore, comparison can be made between the tensile strength values observed in the cracking test and the dry specimens IDT values in the moisture susceptibility test as shown in Table 4.7. To study the correlation between these two tests, the coefficient of determination (R^2) was calculated to be 80 percent, indicating a very high agreement between the two tests. Both tests show a slight reduction in the tensile strength as the chemical additive dosage increased. The main differences between the two tests that resulted in lower tensile strength values for the cracking test are:

- The specimen size used in I-FIT is half the size used in moisture damage test.
- There is a cracking initiating required for I-FIT before the test begins which reduces the strength of the specimens.
- The base support on the I-FIT is designed to roll which propagates cracking at a faster rate.

Table 4.7 Tensile strength values for I-FIT and moisture susceptibility test

Additives Content	**HMA (0%)**	**WMA (0.3%)**	**WMA (0.5%)**	**WMA (0.7%)**
Moisture Damage	526	514	507	482
I-FIT	319	253	245	219

4.8 Balanced-Mix Design

The new supepave asphalt mix design system was introduced in order to design asphalt pavements based on both volumetric properties and acceptable mixture performance tests. Initially, most agencies focused on improving rutting resistance by using more angular aggregates, changing binder grade and improving compaction. As a result, many departments of transporatation in North America reported an improvement in rutting resistance for projects built under Superpave design approach. However, most agencies have reported other pavement distresses such as cracking have emerged as the main factors with an adverse effect on pavement service life. As the challenges associated with the use of Superpave design approach increased, many transportation agencies started seeking a new asphalt mix design approach. The balanced mix design (BMD) is a design approach that integrates two or more performance tests such as the rutting test and cracking test to assess the mixture's ability to resist different forms of distress (Al-Qadi et al., 2015). This approach can be used to address and design an asphalt mix that has good performance in both rutting and cracking resistance. There are many factors that affect the performance of an asphalt mix such as binder type and binder content (%), aggregate type and gradation. Changing one of these factors may improve one design parameter, nevertheless it may negatively impact other design parameters. Balance-mix design approach allows agencies and contractors to develop a mix design that has the potential to satisfy two or more performance criteria (Newcomb & Zhou, 2018). Figure 4.18 illustrates an example of a balance-mix design using HWTT and IL-SCB test results of the four mixes used in this project.

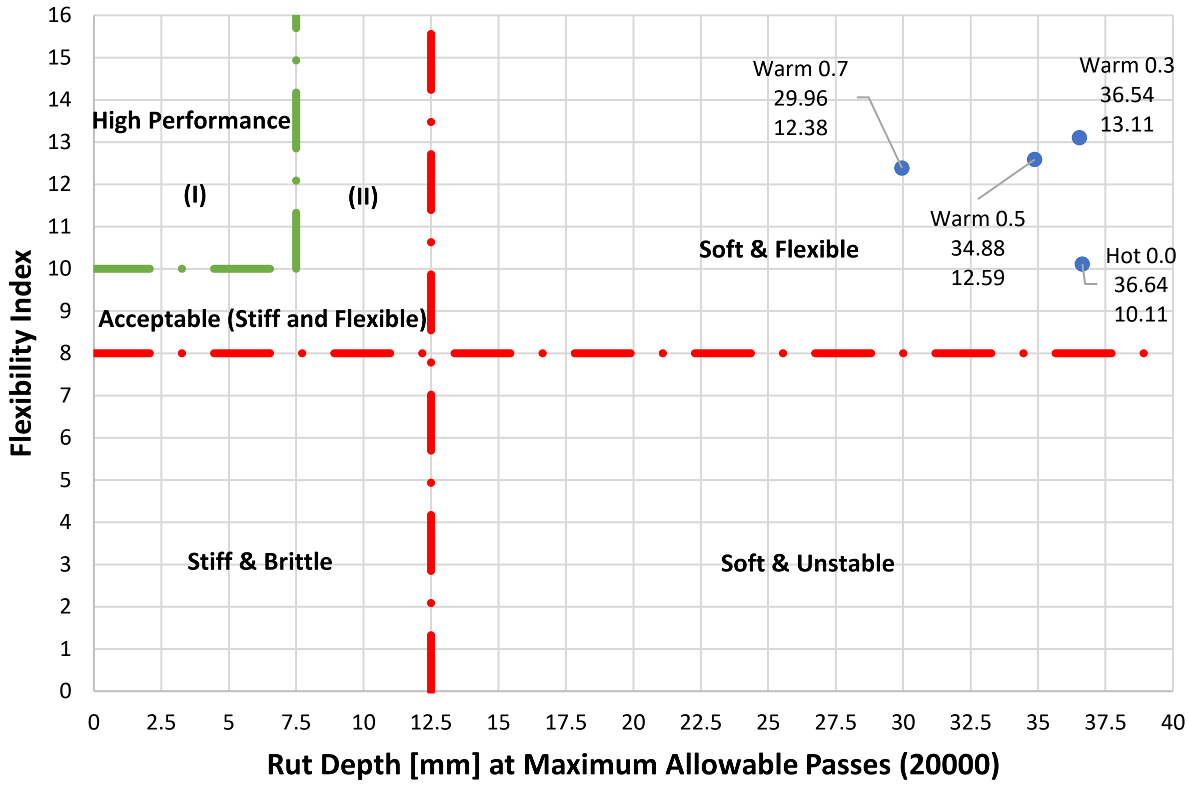


Figure 4.18 Performance quadrant for rutting and cracking performance

Performance quadrant criteria are adopting the Illinois Center for Transportation draft manuscript 16-6872. FI and rut depth threshold should be specified depending on local conditions, plant mixes and different applications.

- For Flexibility index:
    - High performance: FI > 10
    - Acceptable: FI > 8
- For rut depth at 20000 passes
    - High performance: Rut depth < 7.5 mm
    - Acceptable: Rut depth < 12.5 mm

Rut depths are at 20,000 passes. All rut depths are extrapolated using the stripping slope.

The performance quadrants in Figure 4.17 can be explained as follows (Ozer et al., 2016):

- Stiff and flexible: low cracking potential (flexible) and high rutting resistance (stiff). All mixes in this quadrant are acceptable. This quadrant is further subdivided into high performance (I), and standard performance (II). The location of these subdivisions depends on cracking and rutting resistance thresholds.
- Soft and flexible: mixes have high cracking resistance, but with high rutting potential (soft).
- Stiff and brittle: Mixes have high cracking potential (brittle), but high rutting resistance (stiff).
- Soft and unstable: mixes have very low cracking and rutting resistance.

All the four mixtures fall into the soft and flexible category that represent mixes with high cracking resistance, but with high rutting potential. Given these mixtures were used on local street roads, makes rutting a non-issue. However, the range of an acceptable FI and rut depth values will need to be developed based on local mix designs and properties such as norminal maximum aggregate size, asphalt binder content, binder performance grade and type of aggregate. More tests on various types of asphalt mixtures will need to be performed to establish the acceptable FI and rut depth values. Furthermore, a correlation factor might be required depending on the performance variation between laboratory samples and field cored samples.

The implementation of balanced mix design has some challenges to many transportation agencies. The most reported concern is the validity of current performance tests. This includes the selection of appropriate performance tests for each distress and criteria selection for specification purposes. Most transportation departments are concerned about the agreement of laboratory tests to field performance, pass/fail threshold and test variability (West et al., 2018). Other challenges regarding the use of BMD approach include:

- Cost-benefit analysis comparing volumetric approach versus BMD approach.
- Availability of laboratories to provide performance testing services.

4.9 Financial Implications

To evaluate the overall effectiveness of WMA, it is important to consider its economic cost relative to HMA. The bid price for WMA was between 2% to 11% higher than conventional HMA. However, these bid prices included the costs of additional testing as well as the WMA additives. In addition, these contracts involved relatively small quantities of WMA that might not provide the benefit of economies of scale. It is expected that in future large paving contracts the cost of WMA will decrease when the contracts do not include the additional testing, and the savings from reduced energy consumption in the production of WMA are realized.

Furthermore, Table 4.8 shows the ranking of all performance tests conducted. 1 being the best and 4 is the worst performing mixture. The stiffness of all four mixtures was similar and it differed slightly depending on the temperature and frequency. Therefore, all four mixtures were given a similar rankings. The total points for 0.3% and 0.7% of WMA were the same, but 0.3% additive was given a better ranking than 0.7% due to the economic benefit of using a lower dosage.

Table 4.8 Ranking of performance test for different additive dosage

Additives Content	**HMA (0%)**	**WMA (0.3%)**	**WMA (0.5%)**	**WMA (0.7%)**
Stiffness	1	1	1	1
TSR	4	2	1	3
Rutting	3	4	2	1
Cracking	4	1	2	3
Ranking	4	2	1	3

Mixtures with 0.5% warm mix additive demonstrated the highest overall performance among all mixtures. Followed by the mixtures with 0.3% additive, then 0.7% additive. Hot mix asphalt showed an overall lowest performance especially in terms of the resistance to moisture damage and cracking.

Chapter 5. Conclusions and Recommendations

5.1 Conclusions

This research was focused on the evaluation of warm mix asphalt to improve pavement performance, and also provide environmental and economic benefits. The Warm Mix Asphalt (WMA) technology was evaluated using a chemical additive for usage in Winnipeg focusing on urban pavement rehabilitation projects. The laboratory experiments and field test findings provided the following conclusions:

- WMA mixtures were placed and compacted at lower temperatures than HMA mixtures and achieved the required degree of compaction. Although the ambient temperature was decreasing, the degree of compaction increased as the chemical additive dosage increased due to the improved workability for these mixtures.
- The results showed that the dynamic modulus master curves for the WMA and HMA were similar. Thus, the four asphalt mixtures showed the same performance in terms of the mixture’s stiffness.
- Among the different additive dosage used, 0.5% had a higher overall stiffness for the WMA mixtures.

- In general, increasing the WMA additive resulted in an improvement in the TSR value and lower tensile strength values for the dry samples. This indicates that the used additive had anti-stripping properties which may increase the resistance to moisture damage.
- The normalized dynamic modulus (|E*|) values showed that the use of WMA technology might improve fatigue cracking resistance. Minor improvements were observed in terms of resistance to thermal cracking and rutting when implementing warm mix.
- The TSR values of all mixtures met the required 80 percent minimum value and the highest TSR value was achieved at 0.5% additive.
- The HWT test showed low rutting resistance for all mixtures since none of them reached the maximum allowable number of wheel passes at failure. The one-way ANOVA analysis with 95% confidence level suggested no statistically significant difference among the mixtures for all HWTT parameters. In addition, the reduction in stripping slope when using WMA indicates the additive had some antistripping properties.
- The highest correlation between the |E*| values at 37.8°C and number of passes to failure was found at frequencies of 0.1 Hz and 10 Hz.
- The WMA showed a higher cracking resistance compared to HMA. Although HMA required higher energy to initiate cracks, it dissipated all the energy at a higher rate than WMA. HMA proved to be a brittle material, while WMA showed more ductility properties.
- The WMA cost was between 2% to 11% higher than conventional HMA including the costs of additional testing as well as the WMA additives. It is expected that in future large paving contracts, the cost of WMA will decrease when the contracts do not include the additional testing, and the savings from reduced energy consumption in the production of WMA are realized.

5.2 Recommendations

The following recommendations for practice and future work are achieved based on the successful execution of the project and limitations encountered during the study:

5.2.1 Recommendations for Practice

- The use of warm mix is strongly encouraged, however it is critical to ensure that the additive or technology used does not adversely affect the performance of the mixture. Mixture performance should be verified using different laboratory tests such as HWTT and I-FIT. Currently performance tests are not part of the COW specifications, but should be considered to be included.
- The dynamic modulus results can be more accurately used as the expected mixture's stiffness during the design stage using Mechanistic-Empirical Pavement Design (MEPDG) method.
- The recommended dosage when using Evotherm 3G as the chemical additive is 0.5% by weight of asphalt cement. There was no significant benefit of using a higher dosage. The 0.3% dosage could also be used when the ground temperature is above 10°C.
- A better understanding of WMA can be achieved using different warm mix additives, mix designs and testing programs. Future projects should incorporate the use of other chemical additives and different mix designs to investigate the effect of WMA technology on a wider scale.
- To accomplish a better balance-mix design, more tests on various types of asphalt mixtures should be performed to establish the acceptable FI and rut depth values.

5.2.2 Future Research Opportunities

- Even though the project specification for WMA limited the aggregate moisture content to 2%, aggregates were not tested for moisture content after mixing and compaction. A quality assurance test for aggregate moisture content should be performed to evaluate its effect on pavement performance both short and long term.
- Further investigation is required in determining the rate of temperature increase required depending on the amount of recycled materials used. This will optimize the use of fuel energy and mixture bonding by ensuring the aged binder has completely melted.
- Since the WMA technology is fairly new, the study of reclaimed WMA to be used as RAP should be conducted and its effect on short and long-term performance of the pavement.
- The long-term performance tests on this project should be carried out yearly to track the distress accumulation and compare the four mixtures performance. Field cores can be collected for each mixture and laboratory performance tests such as HWTT, I-FIT and IDT can be performed. The stiffness of the pavements can be analyzed using the Falling Weight Reflectometer (FWD).
- As mention in chapter 2, there is still a concern with the compatibility between warm mix additives and other asphalt admixtures such as antistripping agents. There is a need for further exploration of the level of compatibility and cost effectiveness in using both WMA additives and antistripping agents in the same asphalt mix.

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Appendix A

Table A-1 Normalized dynamic modulus (WMA/HMA)

Temp, °C	Loading Freq (Hz)	Normalized Dynamic Modulus (WMA/HMA) 0.3%/HMA	0.5%/HMA	0.7%/HMA
-10	**25**	1.004	1.016	1.025
	10	1.016	1.063	1.062
	5	1.015	1.073	1.055
	1	1.012	1.105	1.053
	0.5	1.011	1.099	1.029
	0.1	1.010	1.124	1.010
4.4	**25**	1.063	1.019	0.891
	10	1.052	1.056	0.848
	5	1.017	1.018	0.770
	1	1.010	1.008	0.745
	0.5	0.994	0.997	0.779
	0.1	0.977	0.984	0.793
21.1	**25**	0.911	0.920	1.021
	10	0.865	0.933	0.961
	5	0.913	0.964	0.974
	1	0.990	1.004	0.998
	0.5	1.027	1.039	1.011
	0.1	1.122	1.089	1.026
37.8	**25**	1.009	0.955	0.915
	10	1.069	0.919	0.912
	5	1.059	0.919	0.861
	1	0.950	0.838	0.811
	0.5	0.957	0.892	0.847
	0.1	0.991	1.009	1.005

Appendix B

Table B-1 Dry specimen indirect tensile strength for each mixture in kPa

	HMA (0%)	**0.30% WMA**	**0.50% WMA**	**0.70% WMA**
Sample 1	521	516	510	485
Sample 2	528	512	506	480
Sample 3	529	514	505	481
Average	526	514	507	482

Table B-2 TSR values for each mixture in percentage

	HMA (0%)	**0.30% WMA**	**0.50% WMA**	**0.70% WMA**
Sample 1	92	95	97	90
Sample 2	84	94	98	95
Sample 3	81	93	98	91
Average	86	94	98	92

Appendix C

Table C-1 Cracking test specimen average dimensions and characteristic

	HMA 0.0%	WMA 0.3%	WMA 0.5%	WMA 0.7%
Ligament (mm)	58.06	57.41	57.77	57.83
Thickness (mm)	49.71	49.94	50.38	49.64
VMA (%)	17.23	17.04	17.20	17.35
NMAS (mm)	12.50	12.50	12.50	12.50
Binder (%)	5.00	5.10	5.10	5.20
Actual Air Void (%)	6.76	6.73	6.90	6.97