Technical, Economic and Environmental Evaluation of Warm Mix Asphalt and Coloured Asphalt for Usage in Canada

by

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Author's Declaration

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

I understand that my thesis may be made electronically available to the public.

Abstract

Transportation services play an important role in the Canadian economy, and social activities as well as Canada's competitiveness in the global economy. As one of the most valuable systems of transportation in Canada, 90 percent of all goods and services are transported via trucks over an extensive land area characterized by diverse landscapes and harsh environments. These unique characteristics of Canada, coupled with other challenges including an aging road network and highway infrastructure, limited finances, and environmental considerations provide great incentive to decision makers at federal, provincial/territorial, and municipal government levels to consider new and innovative ways to fund road transportation infrastructure. In an effort to evaluate two innovative pavement technologies applicable in both urban and rural areas, this research project is focused on (1) Coloured Hot Mix Asphalt (CHMA) and (2) Warm Mix Asphalt (WMA).

The intent of the CHMA research study was to characterize the structural, functional, and environmental characteristics of the coloured asphalt design by analyzing laboratory and field performance. This research was focussed on providing innovative and sustainable solutions, which can be effectively used in Canada as means of ensuring durability and high performance throughout the material's life cycle.

To achieve the research objectives, materials collected during paving operations and materials produced under controlled laboratory conditions were systematically evaluated at CPATT to capture the impact of colouring pigment on the mixture's strength. Results provided in this thesis suggest that pigmentation can adversely affect the performance and proper steps have to been taken to mitigate such effect: including using softer binder and lower Dust Proportion (DP) in the mixture. The state-of-the-art AASHTOWare Mechanistic-Empirical (M-E) Software was employed to complete the most accurate level of analysis, referred to as "Level 1". ME analysis outputs were then used to develop prediction models for a design life of 50 years that can be used to establish Life Cycle Cost Analysis (LCCA). Based on LCCA analysis Bus Rapid Transit (BRT) lane structure surfaced with CHMA was found to be significantly more expensive to construct and maintain than a similar structure surfaced with HMA located in York Region. However, this cost difference is expected to decrease in near future as contractors are becoming more familiar with the mixture's design and production techniques.

This research further evaluated the performance of WMA technology by using different Performance-Graded Asphalt Cement (PGAC) sources modified with three types of WMA additives (Evotherm 3G, Rediset LQ, and SonneWarmix) in combination with two types of aggregate of pink granite and trap rock diabase. Results obtained in this comprehensive research were statistically analyzed to verify the significance of the results. All information collected from a combination of qualitative and quantitative laboratory test methods and M-E long-term prediction were then ranked in ascending order for each combination of aggregate, additive and binder type.

This ranking suggests that certain warm mix technologies such as Evotherm 3G and Rediset LQ can be effectively used to lower the asphalt mixture production and construction temperatures, as well as improving the performance (i.e. moisture susceptibility) in both laboratory and field.

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Last but not least, I would like to thank Sahar whose continuous love and support cannot go unmentioned, as well as her lovely parents Mozaffar and Nassrin.

Dedication

To my beloved sister, Solmaz, whose love is always with me – Rest in Peace.

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Chapter 1 Introduction

The transportation services play an important role in the Canadian economy, and social activities as well as Canada's competitiveness in the global economy. As one of the most valuable systems of transportation in Canada, 90 percent of all goods and services are transported via trucks over an extensive land area characterized by diverse landscapes and harsh environments (TAC, 2013). These unique characteristics of Canada, coupled with other challenges including an aging road network and highway infrastructure, limited finances, and environmental considerations provide great incentive to decision makers at the federal, provincial/territorial, and municipal government levels to consider new and innovative ways to fund road transportation infrastructure. However, such spending at the three levels of government is mostly limited by current fiscal situations and competition for funding health and education (Transport Canada, 2011).

Despite the aforementioned challenges, the increase in international trade and globalization are also challenges that lie ahead of Canada’s transportation systems. As stated by the most recent annual report published by Transport Canada in 2011 (Transport Canada, 2011), Canada’s trade with the rest of the world has grown by 65% between 1995 and 2009 and is expected to grow substantially due to Canada’s geographic advantage, strong economy, and abundant resources. However, to seize upon these opportunities, the capacity of Canada’s transportation system has to be enhanced and optimized by using innovative engineering techniques to design, construct, maintain, manage, and predict the performance of transportation infrastructures.

In an effort to evaluate two innovative pavement technologies applicable in both urban and rural areas, this research project is focused on two technologies of Coloured Hot Mix Asphalt (CHMA) and Warm Mix Asphalt (WMA). These technologies are selected to provide improved safety under heavy loading and to reduce emissions. The research involves collaboration with the Centre for Pavement and Transportation Technology (CPATT) at the University of Waterloo. The work is directed at providing innovative solutions which save money, lives, and provide a high level of service to the users.

1.1. Background

Denoting dedicated bus lanes in the right-of-way (ROW) has been implemented globally to allow buses to move out of congestion, enabling travellers to get around the busiest corridors faster by using transit. This solution is known as Bus Rapid Transit (BRT), and endeavors to create a more vibrant, livable, and sustainable urban development by providing a Pedestrian and Transit Oriented Development (PTOD). This design moves away from car dependency and orients the residential and commercial buildings around active modes of transportation (i.e. public transit and pedestrian facilities). However, developing a BRT system that is easily understood by ROW users is a necessity in order to maintain a high level of safety.

BRT system is traditionally accomplished through signage and lane markings, but the most effective solution is through having a different surface colour for designated lanes (Seattle Department of Transportation, 2011). Lane colouring can be accomplished through one of three methods: painting, applying a coloured thermoplastic, or laying a thin wearing course of Coloured Hot Mix Asphalt (CHMA). One of the major concerns is their durability under significant volumes of vehicle traffic and winter maintenance operations (Birk et al., 1999).

A solution to the durability issue is to colour the entire surface course of a HMA pavement. This can be accomplished through a number of methods, depending on the desired colour of the pavement, including using coloured aggregates, adding pigments to conventional binders, adding pigments and using a clear synthetic binder, or a combination thereof (Asphalt Applications, 2009). From these options the most vibrant colours can only be obtained by using a clear synthetic binder. However, these technologies have not been studied for their long-term performance characteristics, nor been considered for the long-life pavement designs.

As the home of a well-established service sectors including engineering, Information Technology (IT), and finance, York Region in Greater Toronto and Hamilton Area (GTHA) is the sixth largest municipality in Canada by population (Table 1-1), with 16% share of 2014's GTHA population shown in Figure 1-1. The Region's 2014 population is expected to grow to 1.8 million in 2041 (York Region, 2014).

Table 1-1 Canada's Largest Municipalities by Population as of December 2014 (York Region, 2014)

Rank	Municipality	Population
1	City of Toronto	2,808,500
2	Greater Vancouver Regional District	2,474,100
3	City of Montreal	1,988,200
4	Peel Region	1,418,000
5	City of Calgary	1,195,200
6	York Region	1,144,800
7	City of Ottawa	944,900
8	City of Edmonton	877,900
9	City of Quebec	765,700
10	City of Winnipeg	708,400

To meet its rapidly increasing need for public transit, York Region has used a combination of coloured aggregate and red pigment as surface course for its dedicated Bus Rapid Transit (BRT) lanes (Figure 1-2) The initial BRT lanes are located along the three most heavily travelled roads in the Region; Yonge Street, Highway 7, and Davis Drive.

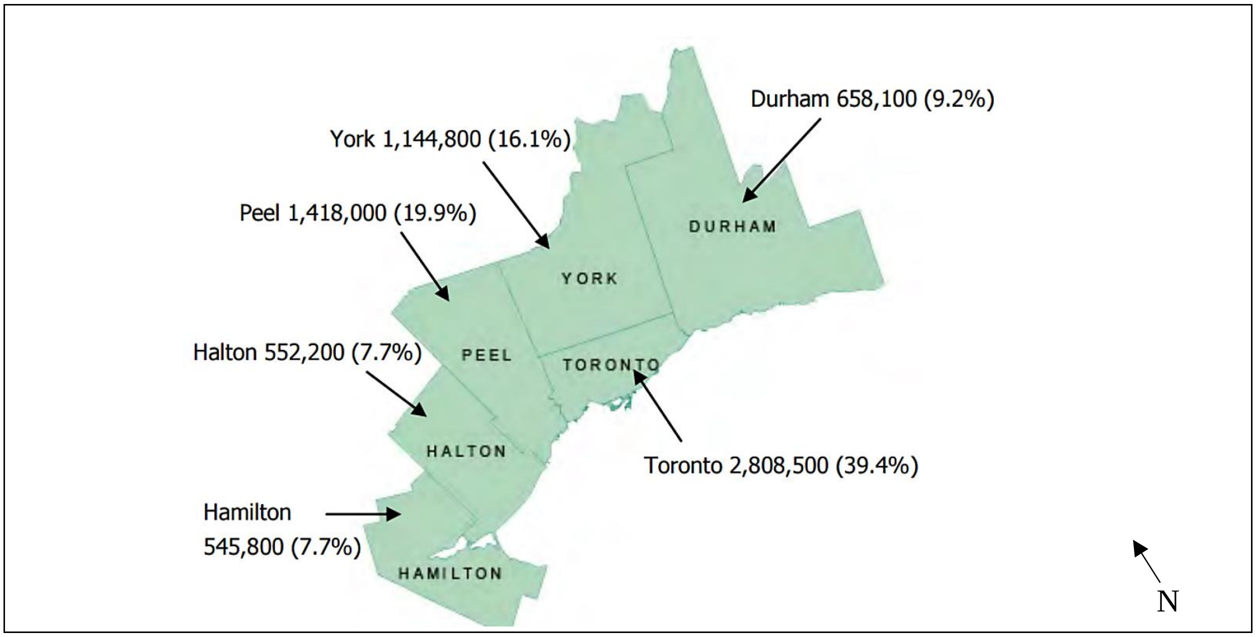


Figure 1-1 Location and Share of Greater Toronto and Hamilton Area (GTHA) Population by Municipalities as of 2014 (York Region, 2014)

Figure 1-2 A section of Highway 7 BRT-Lane, York Region Ontario, August 2014

One of the main objectives of this research was to characterize the structural, functional, and environmental characteristics of the coloured asphalt design for BRT lanes in York Region. The research provided innovative and sustainable solutions, which can be effectively used as means of ensuring durability and high performance throughout the material's life cycle.

Additionally, this research involved the evaluation of the Warm Mix Asphalt (WMA) technology. The Ministry of Transportation of Ontario (MTO) has implemented the use of Warm Mix Asphalt (WMA) on Ontario's highways and roads since 2008 in an effort to provide the asphalt industry with the incentive and opportunity to invest and build confidence in WMA. As stated by MTO (Politano, 2012):

> *"Warm Mix Asphalt is defined as a group of technologies that allow for a reduction in the temperatures at which asphalt mixes are produced and placed relative to traditional Hot Mix Asphalt (HMA). WMA is produced and placed at temperatures 20º to 50º C less than conventional HMA. The production and paving of asphalt at these reduced temperatures generates fewer emissions and requires less energy while maintaining or enhancing pavement performance"*

Many types of WMA technologies have been successfully used to produce and place close to one million tonnes of WMA in Ontario with proven environmental, economical and safety benefits. WMA technologies were first employed in Europe in late 1990s, later gained interest in the United States since 2002 in response to environmental pressures related to greenhouse gas emissions. Given the positive feedback on usage of WMA in the U.S., Canadian agencies started allowing contractors to use WMA in lieu of conventional HMA. Since then several WMA technologies have been developed with the following proven benefits (Tabib et al., 2014):

- Reduced GHG emissions at the asphalt production and during paving operations
- Reduced fuel consumption at the asphalt plant
- Improved worker health and safety due to reduced asphalt fumes and lower temperature at paving sites
- Improved compaction, and longitudinal joint quality
- Less potential to cracking due to reduced asphalt cement aging
- Potential to extend the paving season due to increased workability at lower compaction temperatures
- Facilitating longer haul distances from the production facility to the paving site
- Potential for higher reclaimed asphalt pavement (RAP) content

Despite the aforementioned environmental, economical, and safety benefits of WMA, there are still challenges and concerns with warm mix technologies in Ontario. The main concern is related to reduction in the production temperature and effects of some WMA additives on certain functional considerations, namely moisture susceptibility and rutting resistance.

These concerns needed to be addressed for typical Ontario climatic conditions in order to achieve equivalent performance to conventional Hot-Mix Asphalt (HMA) with the additional benefits of reduce emissions.

Another objective of this research was to investigate the moisture susceptibility of asphalt concrete mixtures containing different types of WMA additives. This research evaluated engineering properties of the asphalt cement[1] modified with WMA additives as well as the mechanical properties of asphalt concrete mixtures produced using the prototype modified binders.

Above all, this study provides recommendations to improve the current American Association of State Highway and Transportation Officials (AASHTO) designation T283 procedure and criteria for evaluating moisture susceptibility of WMA mixtures in Ontario.

1.2. Research Hypothesis

The hypothesis for this research were as follow:

- A combination of coloured aggregate and pigment for usage in BRT lanes can increase the level of safety for ROW users without adversely affecting the structural and functional characteristics of the asphalt concrete mixture.
- WMA additives may decrease the rutting resistance of asphalt concrete mixtures, while improving other properties of mixtures such as fatigue and thermal cracking.
- Excessive aggregate moisture content is present due to the lowered production and compaction temperatures, which may increase the moisture susceptibility of the mixture.
- Current test methods being used in the industry may not adequately able to evaluate the moisture susceptibility within reasonable confidence.

1.3. Research Objectives and Motivations

This research was directed at the evaluation of innovative pavement materials that can improve performance and safety. For this matter, the performance of Coloured Hot Mix Asphalt (CHMA) pavements for BRT lanes in York Region was evaluated. The second material evaluated is the usage of Warm Mix Asphalt (WMA) additives for usage in Ontario with particular interest on provincial and municipal roads such as York Region.

The partnerships involved in this research included York Region, Metrolinx, and the Ministry of Transportation of Ontario (MTO). Both projects are in partnership with the Centre for Pavement and Transportation Technology (CPATT), located at the University of Waterloo. Firstly, evaluation of the CHMA was focused on determining the performance characteristics of CHMA pavements and coloured surface treatments for municipal applications with specific interest in York Region. However, it is important to note that Metrolinx is interested in using CHMA in

[1] The term *Asphalt Cement* (AC) is mostly used in Canada and North America. Outside North America, especially Europe, the term *asphalt* is used to describe the asphalt concrete mixtures and the terms *bitumen* and *asphaltic bitumen* are used in place of asphalt cement.

other areas of Southern Ontario. The other product, WMA also has the potential to provide many benefits and is also evaluated in this research.

The main objectives of this research are as follows:

- To characterize the structural, functional, and environmental characteristics of the CHMA and WMA by analyzing laboratory and field performance.
- To develop best practice guidelines for the usage of these materials in Southern Ontario.
- To evaluate the life cycle period and associated life cycle costs of both materials.
- To examine future maintenance and rehabilitation practices appropriate for CHMA and WMA pavements for use in Southern Ontario.

1.4. Research Methodology

The objectives of this thesis were achieved through a well-designed comprehensive laboratory testing of CHMA and WMA technologies. Field performance of CHMA was also monitored. Majority of laboratory testing was performed at the state-of-the-art testing facility at the Centre for Pavement and Transportation Technology (CPATT) located at the University of Waterloo. Complimentary access to testing equipment and material donation were provided by McAsphalt Industries and Miller Paving in Toronto. More details on the research methodology is provided in chapter three of this thesis. Also, complimentary access to testing equipment was also provided by the Ministry of Transportation of Ontario. In addition to the laboratory testing, field evaluation of coloured BRT pavement sections were examined in collaboration with York Region (Asset Management Department).

1.5. Thesis Organization

This thesis is organized into chapters with following contents:

Chapter 1: *Introduction* –This chapter provides the scope and overall objectives of this research project.

Chapter 2: *Relevant Literature* – A comprehensive review into details on current state of knowledge on concepts and approaches related to materials characterization, mix design, test methods, and field performance pertaining to coloured and warm mix asphalt application of pavement.

Chapter 3: *Research Methodology* – The methodology employed to evaluate coloured hot mix asphalt and warm mix asphalt is explained in this chapter in terms of: (1) details of performing laboratory and field testing and protocols, and (2) sample preparation methods.

Chapter 4: *Evaluation of Warm Mix Asphalt* – Laboratory evaluation and long-term field performance prediction by using Mechanistic-Empirical Pavement Design Guide (MEPDG) for 20 years of service.

Chapter 5: *Effect of Pigment on Mixture Performance* – Laboratory and field performance evaluation, as well as field performance prediction by using MEPDG.

Chapter 6: *Life Cycle Cost Assessment of Coloured Hot Mix Asphalt* – Developing expected path of deterioration over time, and to establish Life Cycle Cost Analysis (LCCA) framework over 50 years of services.

Chapter 7: *Conclusions, recommendations, and future research*

Chapter 2 Literature Review

2.1. Introduction

The purpose of this chapter is to provide a detailed summary on current state of knowledge on concepts and approaches related to materials characterization, mix design, test methods, and field performance pertaining to Coloured Hot Mix Asphalt (CHMA) and Warm Mix Asphalt (WMA) application of pavement.

2.2. Background on Flexible Pavement

Flexible pavement shown in Figure 2-1 is a type of pavement structure that is composed of asphalt-bound layers over unbound drainage layers and prepared subgrade. This pavement structure distributes stress caused by traffic loads downward to the underlying soil foundation in an acceptable level of stress at different seasonal environmental conditions. Key functional performance, also referred to as serviceability, is that the pavement's surface must be smooth and provides adequate skid resistance. However, over time with usage, a number of pavement distresses can contribute to reducing the serviceability and cause deterioration of asphalt-bound layer. Permanent deformation, fatigue cracking, low-temperature cracking, moisture damage, and aging are proven to be contributing distresses (TAC, 2013).

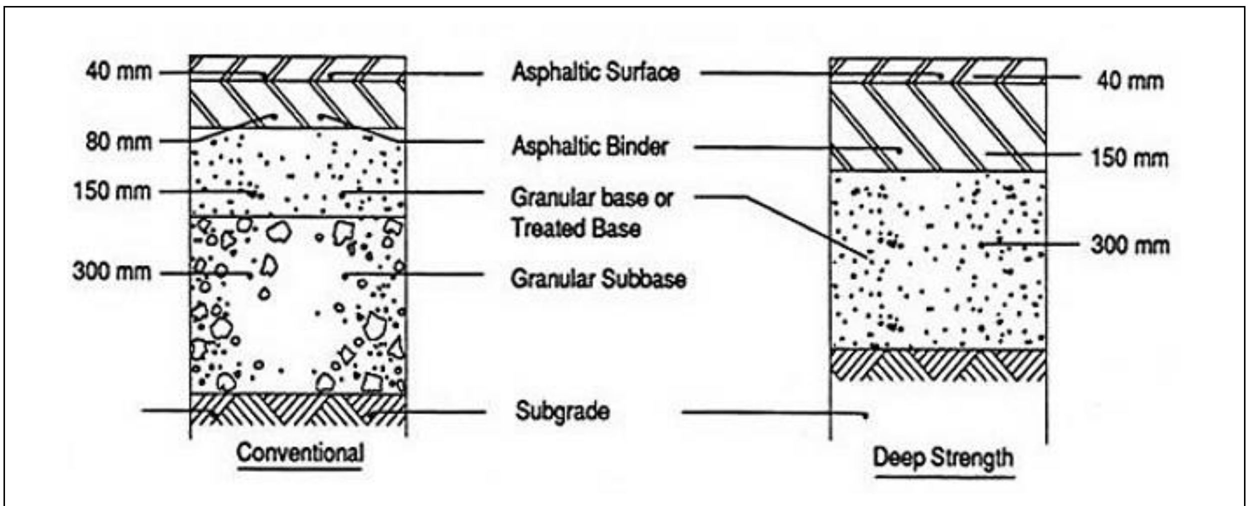


Figure 2-1 Typical Flexible Pavement Structures in Ontario (MTO, 2013)

There are various asphalt surface types including hot mix asphalt, warm mix asphalt, half warm mix, and cold mix asphalt depending on temperature of production as illustrated in

Figure **2-2**. Hot Mix Asphalt (HMA) is the most common surface type used for medium to high volume roads. This type of asphalt mixture is referred to as HMA mainly because of an elevated temperature range of 135 to 160ºC required to plant produce the mixture. The HMA mixture usually consists of 94 to 96 percent of mineral aggregates and 4 to 6 percent of asphalt cement by weight of the mixture.

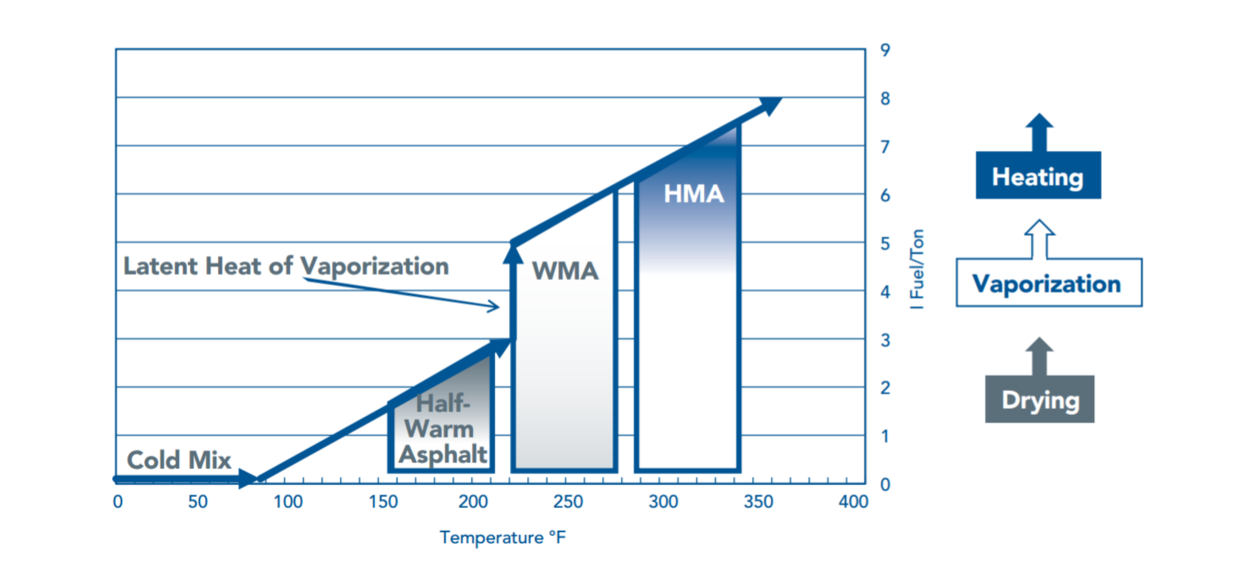


Figure 2-2 Asphalt Mixtures Classification by Production Temperature (FHWA, 2008)

Permanent deformation (also referred to as "rutting") is the most common type of asphalt-bound layer distress. It manifests as accumulated longitudinal dispersions or grooves of wheel paths because of repetitive traffic-loading coupled with environmental effects. At early stages of pavement service-life, some negligible amount of rutting occurs in the surface-layer due to continued densification under repetitive traffic-loading. The densification gradually causes more compaction of the mixture, leading to a decrease in the mixture volume.

After the mixture reaches a limit, that volume does not change instead, plastic deformation (or also referred to as plastic flow) starts to occur (Asphalt Institute, 2001). During plastic deformation, a shear plane (as shown in Figure 2-3) starts to develop. When the shear strength of the mixture becomes less than applied shear stress by a wheel load, the mixture starts to deform permanently from the wheel path to the small upheavals beside the wheel paths (Asphalt Institute, 2001).

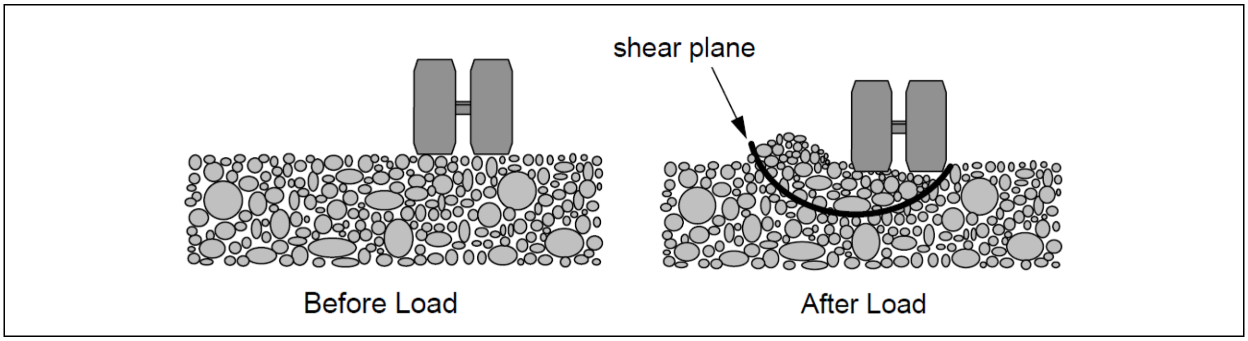


Figure 2-3 Typical Shear Loading Behaviour of Asphalt Mixture (FHWA, 2000)

The shear strength of a mixture is affected significantly by the asphalt cement physical properties. As shown in Figure 2-4, asphalt cement is a visco-elastic material. It behaves like a viscous cementitious liquid at higher temperatures, leathery/rubbery semi-solid at intermediate temperature, and very stiff and brittle at colder temperatures. The behaviour of asphalt cement also is dependent on the rate of loading. At higher temperature with slower rate of loading, the asphalt cement becomes relatively softer. In contrast, the asphalt cement becomes stiffer at colder temperature and faster rate of loading. Although aggregate angularity and shape play an important role in rutting resistance, the stiffness of asphalt cement is a significant factor as well (McGennis et al., 1995).

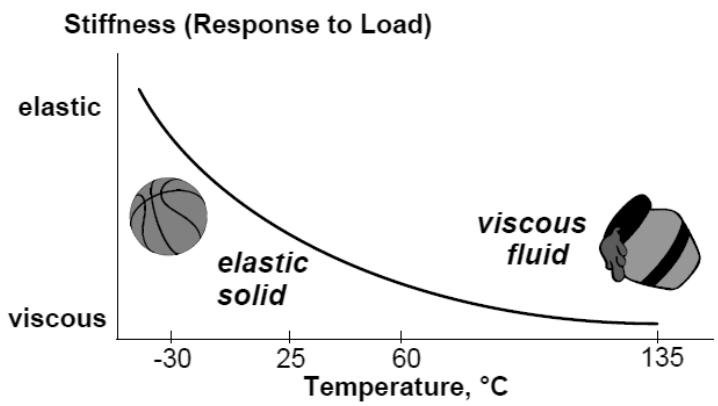


Figure 2-4 Typical Visco-Elastic Behaviour of Asphalt Cement (FHWA, 2000)

Similar to rutting, fatigue cracking is caused progressively by a large number of repetitive traffic-loading stressing a pavement to the limit of its life. However, fatigue cracking tends to form at intermediate (i.e. moderate) pavement service temperature. Because asphalt cement acts more stiff and brittle at moderate service temperatures compare to relatively higher service temperature, it tends to cracks rather than deform (NCHRP, 2011).

Considering the fatigue cracking as the progression of a pavement's design strategy, fatigue cracking often occur sooner than the design life. Regarding the formation of different types of fatigue cracks shown in Figure 2-5, National Cooperative Highway Research Program (NCHRP, 2011) stated that:

> *"Traditionally, pavement engineers believed that fatigue cracks first formed on the underside of the HMA layers, and gradually grew toward pavement surface. It has become clear during the past 10 years that pavements are also subject to top-down fatigue cracking, where the cracks begin at or near the pavement surface and grow downward, typically along the edges of the wheel paths".*

Figure 2-5 Types of Fatigue Cracking in Asphalt Pavement (NCHRP, 2011)

Roberts et al. (1996) stated that inadequate pavement drainage is one of the main causes of premature fatigue failure. Because underlying layers are weakened by the excessive moisture in underlying layers, the HMA layers experience more than anticipated tensile strains that are more than the strength of the mixture. In a manual for design of HMA prepared by NCHRP (2011), the stiffness of the surface layer binder is also stated as a contributing factor to fatigue resistance. This relationship is further stated to be dependent on the pavement structure: for HMA layers with thickness of less than 76 mm (3 in), increasing the high temperature binder stiffness is stated to decrease the resistance to both bottom-up and top-down fatigue cracks. On the other hand, increasing the high temperature stiffness is stated to increase the resistance to bottom-up fatigue cracking for HMA layers thicker than or equal to 127 mm (5 in).

Unlike fatigue cracking and rutting, thermal cracking is caused by adverse environmental conditions rather than traffic loading. Thermal cracking displays itself as consistently spaced transverse cracks perpendicular to the traffic direction, as shown in Figure 2-6.

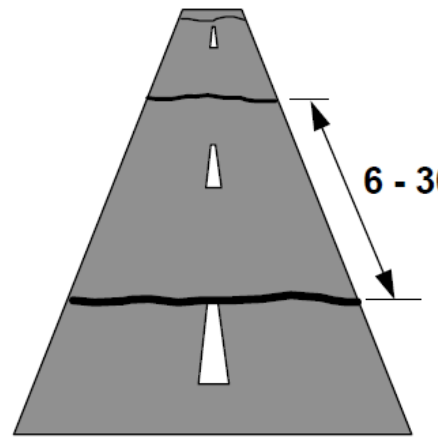


Figure 2-6 Thermal Cracking in Asphalt Pavement (FHWA, 2000)

NCHRP (2011) stated that: *"the low-temperature thermal cracking performance of asphalt pavements is almost completely controlled by the environmental conditions and the low temperature properties of the asphalt binder"*. Bahia et al. (1995) suggested that thermal cooling cycle shrinkage in an asphalt bound layer restrained by friction with the underlying layers can cause a tensile stress development. It is susceptibility with reasonable confidence that such developed stress in the asphalt-bound layer should be relaxed by ability of asphalt cement to flow readily and have less elasticity in its response, which if not relaxed, the cracking will be resulted.

In general, asphalt cement is a visco-elastic material, which its behaviour significantly depend on in-service temperature as well as rate of loading. Such behaviour is further altered during hot-mixing with aggregate at the production plant as well as deterioration due to traffic and environmental loadings during pavement's in-service stage. This alteration is referred to as aging, and is believed to be related significantly to pavement performance (Anderson & Kennedy, 1993). An example of the aging effect on the asphalt cement physical properties is shown in Figure 2-7. As shown, viscosity of the asphalt cement is significantly affected during the production (referred to as "mixing" on the graph). The aging effects continue at slower rate during in-service stages.

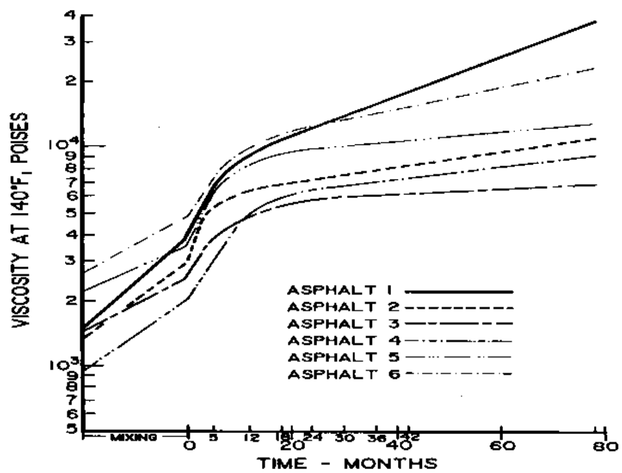


Figure 2-7 Effects of aging on viscosity (Kandhal et al., 1973)

2.2.1. Structural Design

There are four methods of pavement design: (1) experience-based, (2) empirical, (3) mechanistic, and (4) mechanistic-empirical methods. All these methods present complex process of selecting the optimum design after taking number of factors into account such as: anticipated traffic, environment, available materials, use of by-products, local contractor, resources, size of project, cost, required performance, agency policies, established practices, and sustainability (TAC, 2013). The section below provides a brief background on these types of design methods.

<u>Experience-Based Methods</u>

In the experience-based method, past performance of standard sections are used to select the most optimum design for a particular situation. Standard sections use a form of factorial approach that includes layer types and thickness values corresponding to various traffic levels, and subgrade types. This type of design method is limited to developed design conditions, and might not be applicable to future design conditions in case of incorporating different or new materials. Experience-based method is common for low-volume roads (TAC, 2013).

<u>Empirical Design Methods</u>

Empirical design methods use the results of measured pavement responses (i.e. surface deflection) to establish the limits for the optimum thickness design under varying traffic levels and subgrade structural capacity. A conceptual framework for empirically-based design method is presented in Figure 2-8. Empirical methods are more accurate than experience-based methods, but both are similar in terms relaying on the extrapolation of past experience to future conditions (TAC, 2013).

AASHTO 1993 Guide for Design of Pavement Structures and its associated design software, DARWin, is the procedure used by the majority of the road agencies in Canada to design new and rehabilitated pavements (AASHTO, 1993; TAC, 2013). The design procedure is the result of an extensive road test (known as “AASHTO Road Test”) conducted in Ottawa and Illinois, in the later 1950s and early 1960s. After number of revisions in different years, the design guide was last revised in 1993. The design method includes empirical models developed for observed performance of the road test sections to the accelerated traffic loading that can be used to predict the functional performance at desired level of reliability (as shown in Figure 2-9).

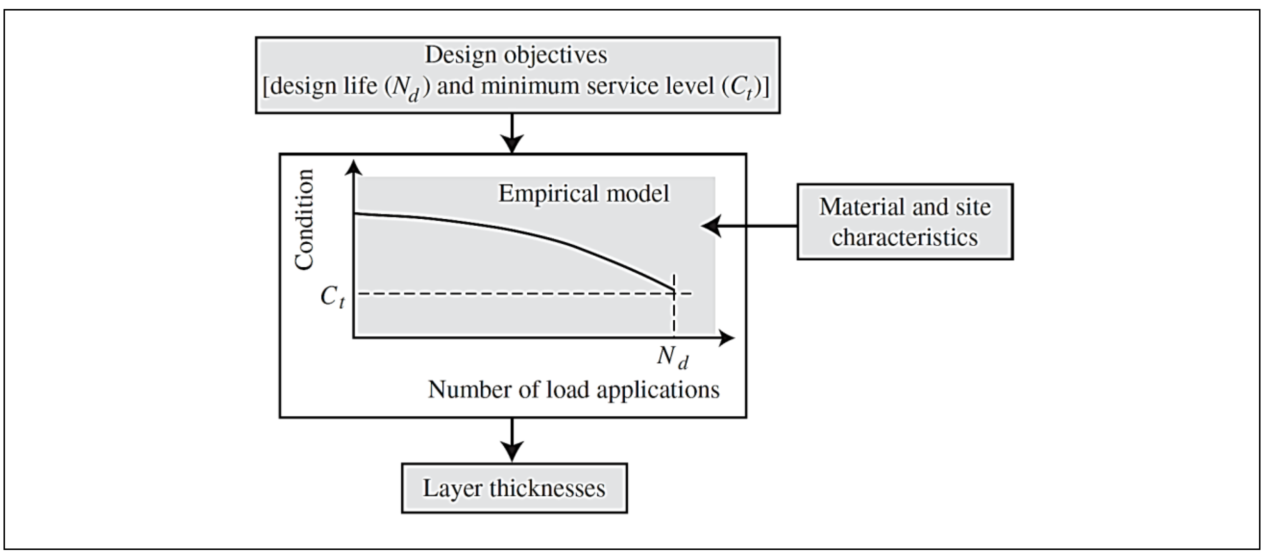


Figure 2-8 Principle of Empirical Pavement Design (Dore & Zubeck, 2009)

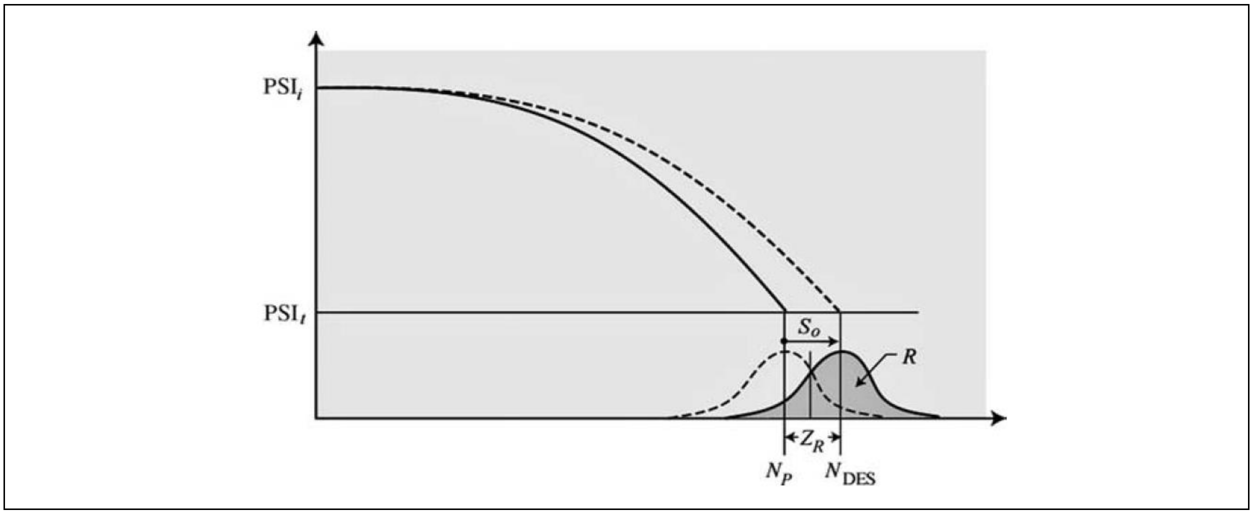


Figure 2-9 Probabilistic Approach Used in AASHTO 1993 Design Method (Dore & Zubeck, 2009)

Mechanistic Methods

Mechanistic methods are based on theoretical analysis, and modeling of static and cyclic loading in pavements to model and predict pavement performance. Stresses, strains, and deformations in pavements are determined in a multi-layered system with performance input parameters assigned to each layer, as illustrated in Figure 2-10.

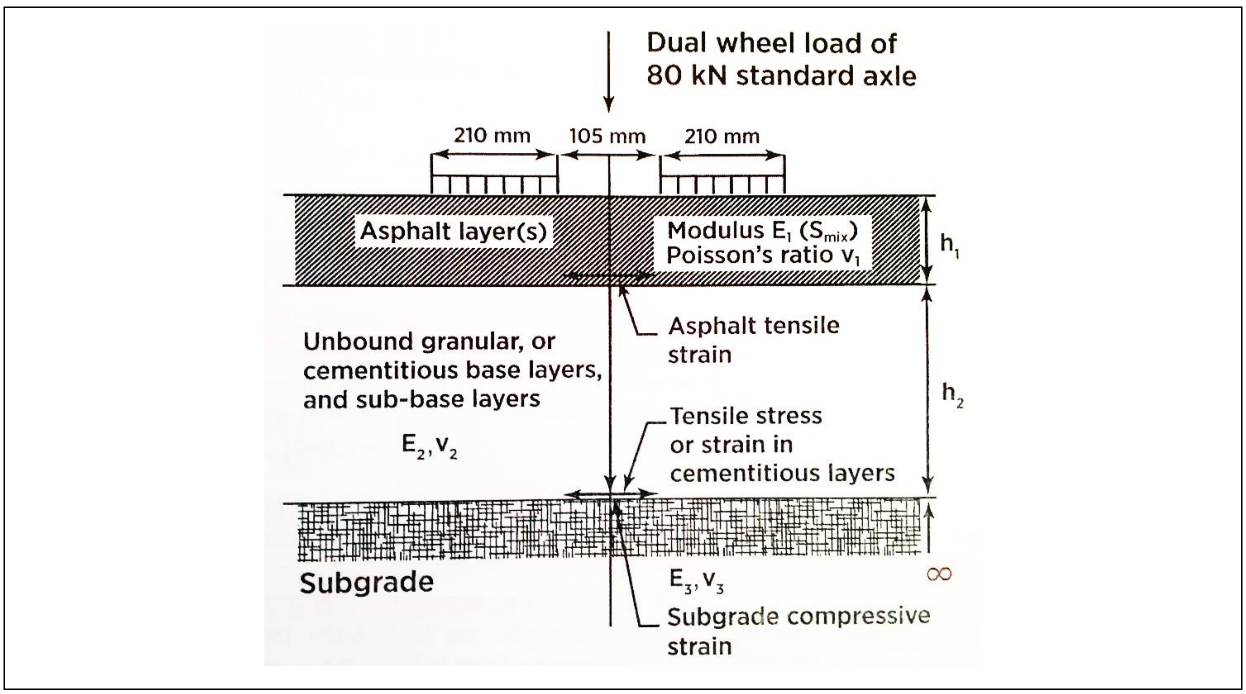


Figure 2-10 Multi-Layer Model of A Flexible Pavement Structure (Yoder & Witczak, 1975)

Mechanistic-Empirical Methods

As mentioned earlier, for mechanistic methods, the input parameters can be derived from laboratory testing of materials. However, these inputs require to be corrected to allow for in-service variations. The correction is based on pervious pavement performance observations, and because of this, this type of pavement design methods are referred to as "mechanistic-empirical methods". Figure 2-11 illustrates principle of mechanistic-empirical methods (TAC, 2013).

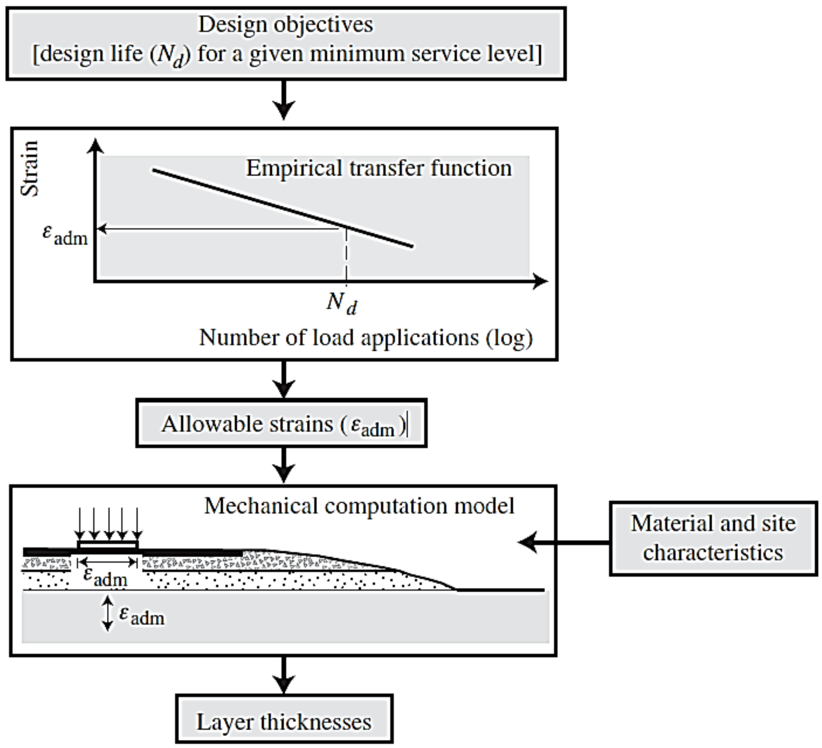


Figure 2-11 Principle of Mechanistic-Empirical Methods (Dore & Zubeck, 2009)

While there are a number of mechanistic-empirical methods, including the Ontario Pavement Analysis of Costs (OPAC) method developed by Ministry of Transportation of Ontario (MTO, 2010), the most comprehensive and widely used method is known as the Mechanistic-Empirical Pavement Design Guide (MEPDG). Developed in 2002 under the National Cooperative Highway Research Program (NCHRP) Project 1-37A (NCHRP, 2004), MEPDG was introduced to address the limitations of the AASHTO 1993 method. After considering a number of technical deficiencies regarding the accuracy of MEPDG in predicting pavement performance, in 2011, the revised version of the MEPDG was introduced under the name of DARWin-ME (also known as AASHTOWare). Depending on the availability of data and importance of the project, AASHTOWare method can be applied in three levels of performance analysis such as (TAC, 2013) following levels, which are explained more in details in Table 2-1.

- **Level 1:** the most accurate and reliable of all levels. It is commonly used for the most heavily trafficked projects where user safety and economic consequences of early pavement failures are severe.
- **Level 2:** the inputs are based on limited testing and/or are selected from the values provided by the agency. Such values are usually estimated empirically.
- **Level 3:** the least accurate of all levels. Inputs are user selected default values.

Table 2-1 Input data for each level in the MEPDG (ARA, 2004)

Input Level	Description
1	− Conduct |E*| (dynamic modulus) laboratory test (NCHRP 1-28A) at loading frequencies and temperatures of interest for the given mixture − Conduct binder complex shear modulus (|G*|) and phase angle (δ) testing on the proposed asphalt cement (AASHTO T315) at 1.59 Hz (10 rad/s) frequency over a range of temperatures. − From binder test data estimate Ai-VTSi for mix-compaction temperature. − Develop master curve for the asphalt mixture that accurately defines the time-temperature dependency including aging.
2	− No |E*| laboratory test required. − Use |E*| predictive equation. − Conduct |G*| on the proposed asphalt cement (AASHTO T315) at 1.59 Hz (10 rad/s) frequency over a range of temperatures. The binder viscosity of stiffness can also be estimated using conventional asphalt test data such as Ring and Ball Softening Point, absolute and kinematic viscosities, or using the Brookfield viscometer. − Develop Ai-VTSi for mix-compaction temperature. − Develop master curve for asphalt mixture that accurately defines the time-temperature dependency including aging.
3	− No |E*| laboratory testing required. − Use |E*| predictive equation. − Use typical Ai-VTS – values provided in the Design Guide software based on PG viscosity, or penetration grade of the binder. − Develop master curve for asphalt mixture that accurately defines the time-temperature dependency including aging.

2.2.2. Mixture Design and Laboratory Testing

Two common methods for the design of asphalt mixtures: Marshall, and Superpave method of mix designs. While all these methods have been widely used for the past 60 years in the United States and Canada, the Superpave method is the most common due to its integrated approach to addressing traffic loading and climate. The section below provides a brief background on the Superpave mixture design method, in order to provide a better perspective to the procedures described in later chapters of this thesis.

The Superpave (*Superior Performing Asphalt Pavements*) is a product of the Strategic Highway Research Program (SHRP) initiated by the United States Department of Transportation Federal Highway Administration (FHWA) during the late 1980s, and was intended to be an improvement over the Hveem and Marshall methods. In direct response to the SHRP, The Canadian Strategic Highway Research Program (C-SHRP) was launched in 1987 by the Council of Deputy Ministers Responsible for Transportation and Highway Safety to extract benefits of the Superpave concepts applicable for Canadian roads. As a result, the Superpave concepts gained acceptance and become accepted as a standard procedure of designing asphalt mixtures in Canadian pavement industry (CSHRP, 2013).

The Superpave is a system of mixture design for asphalt mixtures based upon mechanistic concepts, which includes: (1) an asphalt-grading system called Performance Grading (PG) with intention of matching the physical binder properties to the desired level of resistance to rutting, fatigue and low temperature cracking, subjected to local climate and environmental conditions, and (2) an approach to help designing the aggregate structure based on volumetric analysis and requirements.

The pavement temperatures used for performance grades are determined by converting historical air temperatures into maximum pavement temperature at depth of 20mm below the surface and minimum pavement temperature at the surface of pavement. The conversion is performed as per algorithms given in AASHTO M 323-13 (AASHTO, 2013). A computerized method of AASHTO M323 is also available in a form of software (named as "*LTPPBind*") provided by the FHWA, in which more than 6500 weather stations data from the United States and Canada are compiled.

The design reliability level is also incorporated in the process of selecting pavement service temperatures. Reliability is defined as the percent probability that the average 7-day maximum and minimum pavement temperature will not exceed the corresponding PG temperatures in a single year. The design reliability level is calculated according to a standard deviation that describes the every year variation in the average. Figure 2-12 illustrates a sample of calculation for determination of PG based on different reliability levels.

The PG grades selected by Superpave system apply for typical highway loading conditions. In case of standing or slow traffic, Superpave requires an additional shift in the selected high PG grade to avoid permanent deformation. Also, an additional shift is required for high volume of design Equivalent Single Axle Loads (ESAL). This practice of adjusting high PG grade for traffic loading and speed is referred to as "grade-bumping" (NCHRP, 2011).

Table 2-2 summarizes guidelines provided by Ontario Provincial Standard Specification (OPSS) on adjustment of PG grade based on roadway classification, as part of the Ontario Provincial Specification Standard (OPSS, 2013).

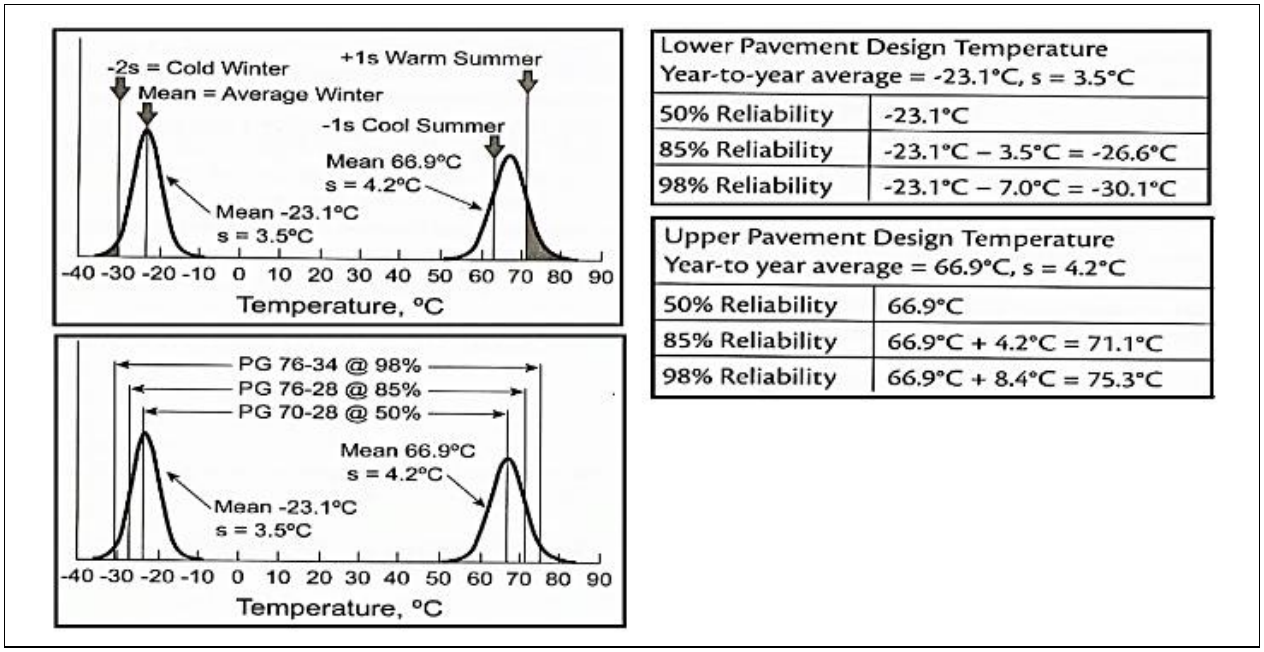


Figure 2-12 Performance Grade Selection for Different Reliabilities (Asphalt Institute, 2008)

Table 2-2 OPSS recommended grade-bumping guidelines (OPSS, 2013)

Highway Type	**Increase From Standard PG**	**Optional Grade Increase**
Urban Freeway	2 Grades	N/A
Rural Freeway Urban Arterial	1 Grade	1 Grade
Rural Arterial Urban Collector	Consider 1 grade increasing if heavy traffic is greater than 20% AADT	1 Grade
Rural Collector Rural Local Urban/Suburban Collector	No Change	1 or 2 Grades

Notes: Consideration should be given to an increase in the high temperature grade for roadways which experience a high percentage of heavy truck or bus traffic at slow operating speeds, frequent stops and starts, and historical concerns with instability rutting.

In Ontario, standard PG grade is selected depending on the in-service location of a pavement. For this matter, Ontario has been divided into three zones as follow, as stated in OPSS.MUNI 1101 (OPSS, 2013). Table 2-3 provides the grades required for each zone.

- **Zone 1**: The area north of the boundary formed by the French River, Lake Nipissing, and the Mattawa River.
- **Zone 2**: The area south of Zone 1, and north of a line from Honey Harbour, to Longford, Taylor Corners, Cavan, Campbellford, and Mallorytown.
- **Zone 3**: The area south of Zone 2.

Table 2-3 OPSS.MUNI 1101 PGAC Zones (OPSS, 2013)

	Performance Graded Asphalt Cement Zones		
	Zone 1	**Zone 2**	**Zone 3**
New Hot Mix or Up to 20% RAP	52-34	58-34	58-28
21 to 40% RAP	46-40	52-40	52-34

Note: RAP is Reclaimed Asphalt Pavement

As mentioned in Table 2-2, one or two grade adjustment is recommended for roadways experiencing heavy bus traffic at slow operating speeds. Such adjustment is often made to help mitigating rutting, and improve fatigue cracking. This adjustment requires modification of the standard asphalt cement due to limitations in production practices. Asphalt modification (also referred to as “binder modification”) can be performed in a number of production techniques by using various modifiers.

In addition to climatic conditions, the Superpave PG system also accounts for the effects of asphalt cement aging by adopting two procedures simulating two stages of binder’s life. The first stage simulates the short-term aging of the binder due to heat and air exposure during mixing at the HMA plant, transportation, and placement. The second stage simulates the long-term aging of asphalt cements that occurs by UV exposure, oxidization, and hardening of asphalt cement after several years of service.

The Superpave includes a set of specifications that provides guidance on selecting aggregate structure based on the anticipated traffic loading and thickness of the layer. After selection of materials (aggregate structure and asphalt cement grade), the Superpave method creates several trial aggregate-asphalt cement mixtures with different asphalt cement contents and compacted at three defined compaction points by using a gyratory form of compaction. The gyratory compaction is to produce specimens considered more representative of as-constructed pavement in terms of aggregate orientation and compaction (TAC, 2013). After compaction, volumetric composition and moisture resistance of mixture trials with varying asphalt cement content are evaluated in details and the most suitable design of mixture is selected.

2.2.3. Maintenance and Management

After years of being in-service, coupled with exposure to climatic conditions, cracks begin to appear on the pavement surface in different forms of distresses, which allow water to infiltrate and further weaken the pavement surface. Ultimately, pavement is no longer able to support heavy loads, and lead to pavement failure.

A number of actions can be taken to repair the distresses that occur in asphalt pavements, depending on the degree to which the pavement has deteriorated. Agencies across Canada describe these actions as emergency, routine, reactive, minor and major maintenance, preventive maintenance, corrective maintenance, preservation, restoration and rehabilitation (TAC, 2013). The selection of these activities is generally based on the type and classification of the roadway (i.e. rural versus urban), as well agency policies and available funding. The selection is further influenced by the availability of materials, contractor capability and cost effectiveness (TAC, 2013). While agencies across Canada may have different decision matrix in selecting these activities, Table 2-4 provides a simplified decision matrix for flexible pavement routine maintenance, preservation, and rehabilitation treatments.

Routine Maintenance

Routine maintenance includes daily operations, and periodical small scale corrective actions, such as pothole repair, and shallow patching. Steps involved in pothole repair and shallow patching can be performed in similar manners: (1) cleaning out of the area to be treated from any loose material and debris, (2) filling with either cold or hot mix Asphalt, (3) compaction using a small vibratory compactor, or the asphalt supply truck tires, or vehicular traffic. Cold or hot mix Asphalt can be applied in different colours at varying cost.

While not mentioned in Table 2-4, routine maintenance might include cleaning and/or repairing spots that are damaged by oil, grease, fuel, or other automotive fluids dripped onto the pavement. Automotive fluids contain minerals that dissolve or soften the asphalt bind and can result in surface deterioration and defects. These spots are often referred to as “oil spots” and can be found most commonly in bus stops, parking lots, and airport taxiways. Oil spots can be cleaned by (1) burning the oil spot with a propane torch, (2) rescrubbing with an industrial grade soap or degreaser, and (3) high pressure washing.

Preservation

Preservation includes major preventive maintenance routines that are conducted before pavement commences deterioration and failure. These major preventive maintenance routines are commonly applied in form of surface treatment and overlay options, as listed in Table 2-4. It should be noted that all treatments summarized in Table 2-4 are capable of being applied in different colours at varying cost. A brief description of most commonly used treatments is given in following sections.

Rehabilitation

Pavement rehabilitation is selected when other alternatives or routine maintenance and preservation are no longer cost-effective. Pavement rehabilitation can be performed in forms of treatments presented in Table 2-4.

Table 2-4 Decision Making For Routine Maintenance, Preservation, And Rehabilitation Treatments For Flexible Pavement (TAC, 2013)

Treatment Actions		Restoring or Improving Pavement Surface In Terms of :						Expected Service Life (Years)
		Preventing Water Infiltration	Localized Severe Distress	Bleeding Raveling, or Poor Skid Resistance	Ride Quality	Environmental Deterioration	Structural Capacity and Traffic	
Routine Maintenance	Pothole Repair	●	★		●			<1
	Shallow Patching	○	★	○	●			2-4
	Drainage Improvement	●	○			★		1-5
Preservation	Crack Sealing	★						1-5+
	Spray Patching	★		○	○			2-5
	Full Depth Patching	○	★		○		●	5-10
	Heater Scarification		●	●		●		1-3
	Hot In-Place Recycling			○	●	○		6-15
	Thin Asphalt Overlay	●		★	○			5-12
	Resurfacing – Functional			●	○	○	○	8-12
	Milling and Resurfacing – Functional	○	○	●	★	●	○	8-12
	Bonded Concrete Overlay					○	●	8-15
	Slurry Sealing	●		●		○		3-5
	Seal Coat	○		★		○		3-7+
	Micro-surfacing	○		★	●	○		3-7
Rehabilitation	Resurfacing – Structural			●	○	○	★	12-15
	Milling and Resurfacing – Structural	○		●	★	●	★	12-15
	Cold In-Place Recycling			○	○	●	○	12-15+
	Bonded Concrete Overlay					○	●	15-25
	Un-bonded Concrete Overlay			○	○	○	★	15-50
	Full Depth Reclamation		●	○	●	★	★	12-15+
★ - Primary Application				● - Commonly Used			○ -May be considered	

2.3. Warm Mix Asphalt (WMA) Technologies

WMA technologies were first employed in Europe in the late 1990s, and since 2002 gained interest in North America in response to environmental pressures related to greenhouse gas emissions. Since then several number of WMA technologies were developed. In the most recent survey conducted by Bonaquist in the NCHRP 09-43 project (Bonaquist, 2011), over 20 most widely used WMA technologies were identified as presented in Table 2-5.

Table 2-5 Summary of WMA Processes Identified During NCHRP Project 09-43 (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

WMA technologies are broadly categorized as organic additives, foaming processes, and chemical processes, which a brief description of each is provided in following sections.

Organic Additives

As stated by (Cervarich, 2003), two types of organic additives are commonly used in the asphalt industry including synthetic paraffin waxes, and low-molecular-weight ester compounds. It is further stated that: "*The paraffin waxes consist of long-chained aliphatic hydrocarbons derived from coal gasification, while the ester compounds consist mainly of esters from fat acids and wax alcohols produced by toluene extraction from brown coal*". Additionally, Hurley et al (2005) stated that organic additives have melting points below normal HMA production temperatures, hence such additives are able to increase the viscosity of the binder at low temperatures facilitating the WMA production (Hurley & Prowell, 2005a).

A common organic additive that is currently used at its potential is Sasobit®. In commercial applications in Europe, South Africa, and Asia, Sasobit® is added directly onto the aggregate mixture as solid pellets or as molten liquid via a dosing meter (Hurley & Prowell, 2005a). However, In the United States, Sasobit® has been blended with the asphalt cement at the asphalt terminal with addition rates ranged from 0.8 to 3 percent by mass of binder without any adverse effect on the performance grades (PG) of the base asphalt cement (Hurley & Prowell, 2006).

Foaming Processes

During the foaming process, small amounts of cold water are injected into hot asphalt cement, causing the water to expand and creates a controlled foaming action. This foaming action acts as a temporary asphalt volume extender and mixture lubricant, enabling the aggregate particles to be rapidly coated and the mix to be workable and compactable at temperatures significantly lower than those typically used for HMA (D'Angelo, 2008). The most commonly used foaming processes include Aspha-Min® zeolite, Advera®, Double Barrel Green®, and WAM-Foam®.

Aspha-min® contains approximately 21 percent water by mass, which can be released in the temperature range of 85 to 182ºC (185 to 360°F respectively). When Aspha-min® is added to the mixture with the asphalt cement at the same time, water is released to create foaming action. In North America, Aspha-min® zeolite is available in form of fine powder (0.300 mm). The Aspha-min® can be added directly to the pugmill of a batch plant, through the RAP collar or pneumatically fed into a drum plant using a specially built feeder.

Chemical Processes

A variety of chemical packages exist that can be used to improve coating, mixture workability, and compaction. Such chemicals are formulated to not result in any adverse effect on the rheology of the asphalt cement itself. Evotherm® is the most commonly used chemical additive in North America.

Evotherm® DAT is a high-residue cationic emulsion (approximately 70 percent asphalt cement). The Evotherm® can be pumped directly off a tanker or be stored in a storage tank. When mixed with the aggregate at the plant, the water in the emulsion is liberated from the Evotherm® in the form of steam (Hurley & Prowell, 2006).

2.3.1. Canadian WMA Experience

Information on type of additives used in Canada was found to be scatter. In 2015, CPATT distributed (Varamini & Tighe, 2015) a survey to participants who were representing various transportation agencies in Canada to: (1) document the state-of-the-art related to WMA technologies, and (2) identify candidate technologies or knowledge for inclusion in the work plan. Table 2-6 shows the agencies which responded to the survey. This survey was reviewed and received ethics clearance through a University of Waterloo Research Ethics Committee. The following is a brief summary of key findings of the survey.

Table 2-6 Summary of Warm Mix Asphalt Respondents

No.	Organization	City	Province
1	Alberta Transportation	Edmonton	Alberta
2	British Columbia Ministry of Transportation and Infrastructure	Victoria	British Columbia
3	The City of Calgary	Calgary	Alberta
4	Manitoba Infrastructure and Transportation	Winnipeg	Manitoba
5	Ministry of Transportation Ontario	Toronto	Ontario
6	Nova Scotia Transportation and Infrastructure Renewal	Halifax	Nova Scotia
7	Ministry of Highway and Infrastructure, Saskatchewan	Saskatoon	Saskatchewan
8	Transports Quebec	Quebec City	Quebec
9	Halifax Regional Municipality	Halifax	Nova Scotia
10	Government of Yukon Highway and Public Works	Whitehorse	Yukon
11	New Brunswick Department of Transportation and Infrastructure	Fredericton	New Brunswick

To briefly summarize the survey responses; majority of the provincial agencies indicated routine use of WMA as illustrated in Figure 2-13, While illustrates the tonnage of WMA that has been placed to date by different provincial agencies, Figure 2-14 provides tonnage of WMA placed up to the date of conducting the survey by respondents.

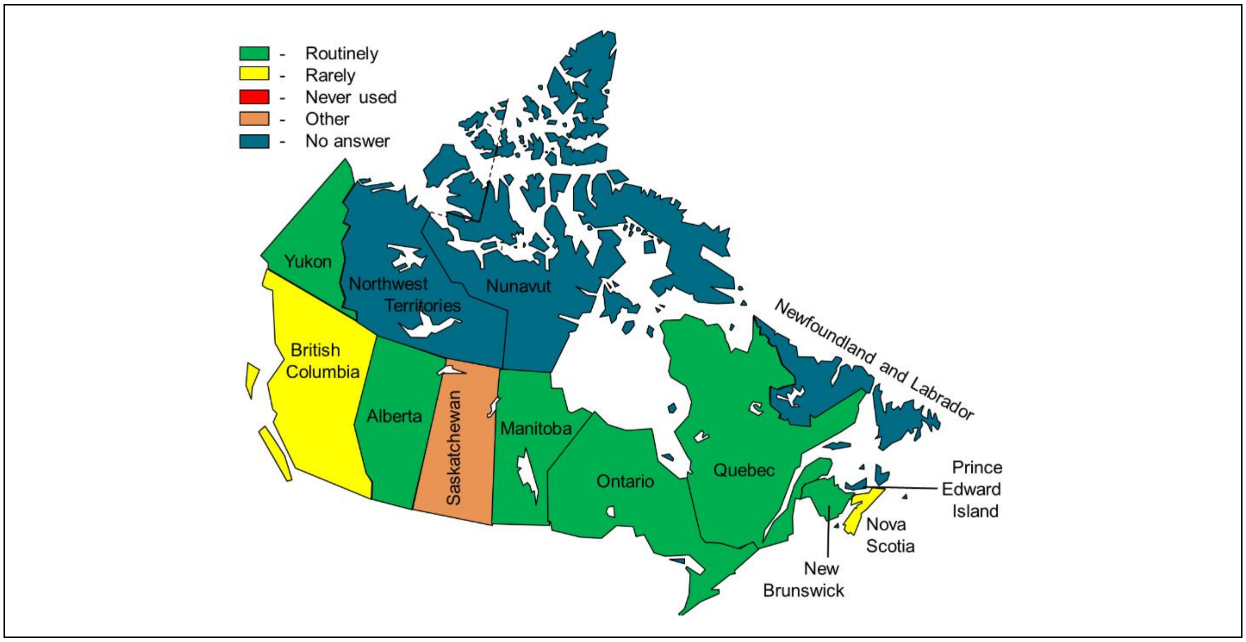


Figure 2-13 Current Usage of Warm Mix Asphalt in Canada

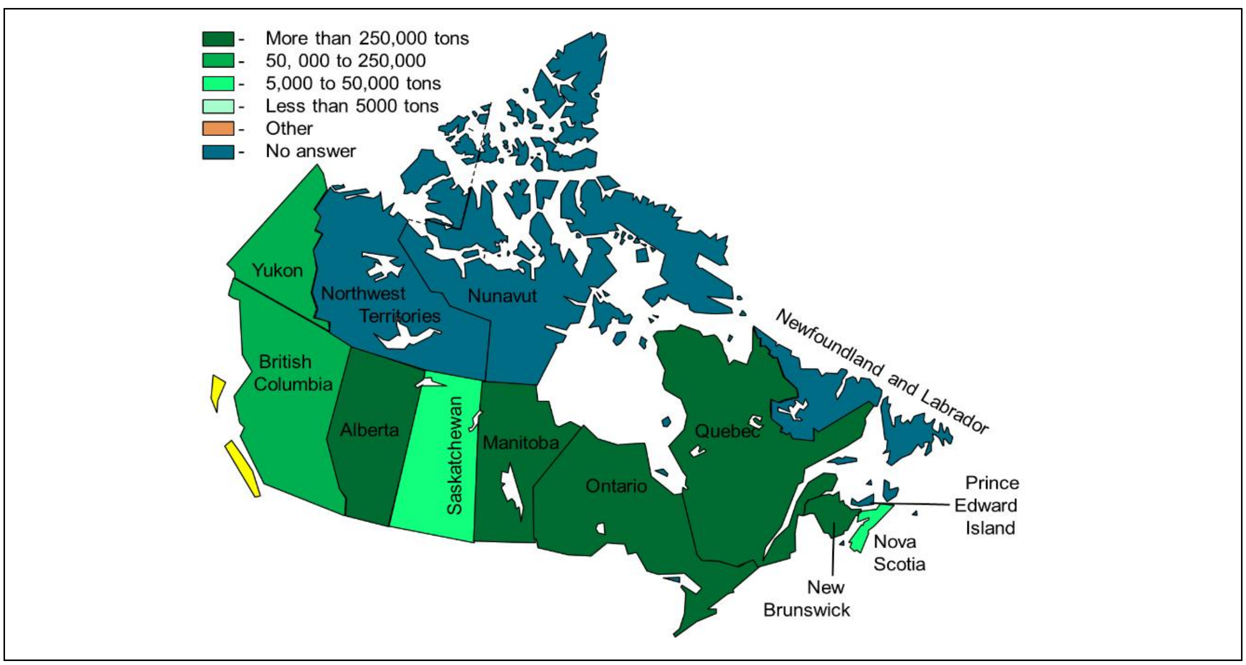


Figure 2-14 Warm Mix Asphalt Tonnage Placed as of 2015 in Canada

Two of respondents reported the first use of WMA between 2006 and 2007, while majority of the respondents indicated that they first started using WMA technology between 2008 to 2009. Table 2-7 presents the list of most commonly used additives by all respondents.

Table 2-7 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%

The majority of respondents indicated mandatory use of anti-stripping agents with warm mix technology, if the laboratory test results indicate presence of moisture damage (73%); while 18% of agencies require the use of anti-stripping agents when employing aggregates with a history of moisture susceptibility. Furthermore, 18% of agencies indicated that there are warm mix additives known to have anti-stripping properties, and because of this, use of anti-stripping agents may not be required. Being asked if they can provide name of anti-stripping agents that are commonly used for their projects, 64% of the agencies replied, the agencies named at least one of the following anti-stripping agent names:

- Hydrated Lime
- Zycosoil
- AD-Here LOF 65-00 and AD-Here 77-00
- Morlife 5000
- Indulin 814A and Redicote C2914

The laboratory and field performance of WMA has been documented by 75% of the respondents. Moreover, the vast majority of agencies (91%) indicated that no premature failures or distresses had been observed for any WMA projects/trials. However, one agency (9%) observed the appearance of thermal cracking in WMA pavements.

2.3.2. Canadian Environmental Studies on WMA

WMA technology is becoming more common for usage by Canadian transportation agencies due to the pressures from environmental agencies and environmental protocols such as Kyoto to reduce Green House Gas (GHG) emissions generated by production of paving mixtures.

The Kyoto protocol is an international agreement that forced fighting climate change by setting targets to reduced GHG emissions. The Kyoto protocol was introduced in 2005 to the Canadian government and was one of the effective means of pressuring provincial transportation agencies to allow paving industry to use WMA as an alternative to accommodate production and placement of conventional asphalt mixtures at lowered levels of GHS emissions.

In an effort to evaluate the effectiveness of WMA in reducing emissions at the production plant, a collaborative study was completed in 2006 (Davidson, Tighe, & Croteau, 2006). As part of this study, emission testing was performed at a hot mix plant in Ontario to obtain data for combustion gases during the production of conventional hot mix asphalt and warm mix asphalt. The sampling of gases was performed at a single point near the centre of the plant exhaust. Combustion gases included oxygen (O_2), carbon dioxide (CO_2), carbon monoxide (CO), sulphur dioxide (SO_2), and oxides of nitrogen (NO_x). The summary of the combustion gas data for HMA and WMA production are listed Table 2-8 and graphically shown in Figure 2-15, which proves the effectiveness of WMA in reducing harmful GHG emissions in considerable amount.

Table 2-8 McAsphalt-CPATT WMA study – Emission Results (Davidson, Tighe, & Croteau, 2006)

Combustion Gas Name	Concentration		Percent Reduction
	HMA	WMA	
Oxygen	14.6 %	17.5 %	19.9
Carbon Dioxide	4.8 %	2.6 %	45.8
Carbon Monoxide	70.2 %	25.9 %	63.1
Sulphur Dioxide	17.2 ppm	10.1 ppm	41.2
Oxides of Nitrogen	62.2 ppm	26.1 ppm	58.0
Average Stack Gas Temperature	162 deg.C	121 deg.C	25.3

Another similar study was completed by MTO in 2010 (Tabib et al., 2014). In this study, asphalt plant stack emissions were measured during production of nine contracts across the province of Ontario; six plants used natural gas and, of the remaining, three contracts used fuel oil or diesel. Figure 2-16 shows only the CO2 concentration levels reported by MTO. This study concluded slight reduction in CO_2 reduction. MTO explained the significant difference in CO_2 between natural gas and diesel operating plants as follow (Tabib et al., 2014):

"Emissions concentrations were significantly higher for diesel fuel compared to when natural gas was used as the fuel. Natural gas is normally considered to be the cleanest fuel source and produces far less air pollutants than diesel or oil."

It is further stated by MTO that:

"With increased WMA use in Ontario, it is anticipated that the plant burners will be properly tuned up for WMA production which will reduce incomplete combustion and reduce stack emissions."

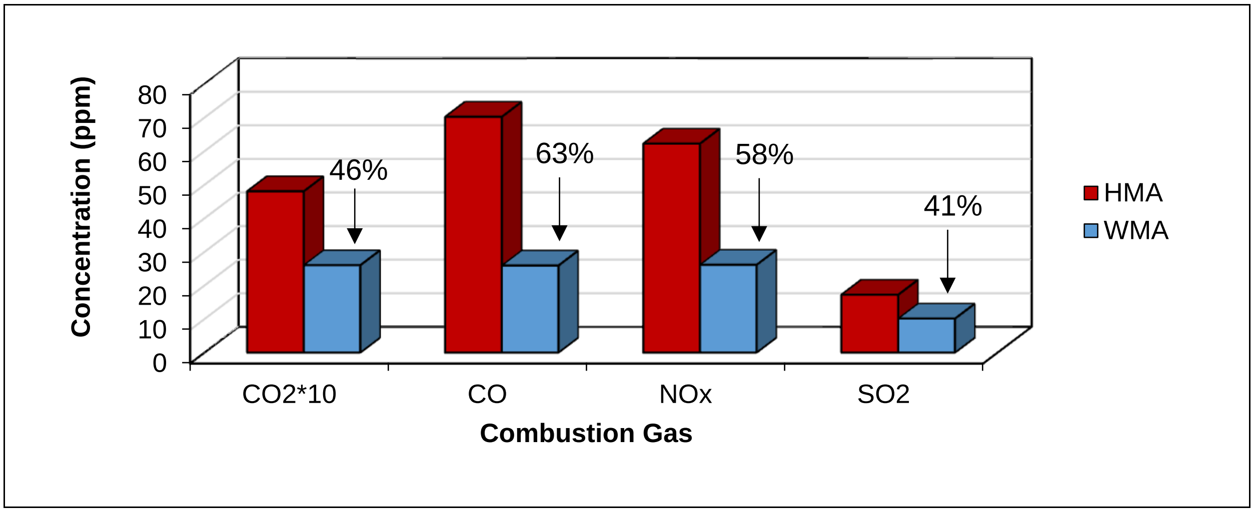


Figure 2-15 Emission Data Obtained During Production of A Typical Asphalt Mixture in Ontario (Davidson, Tighe, & Croteau, 2006)

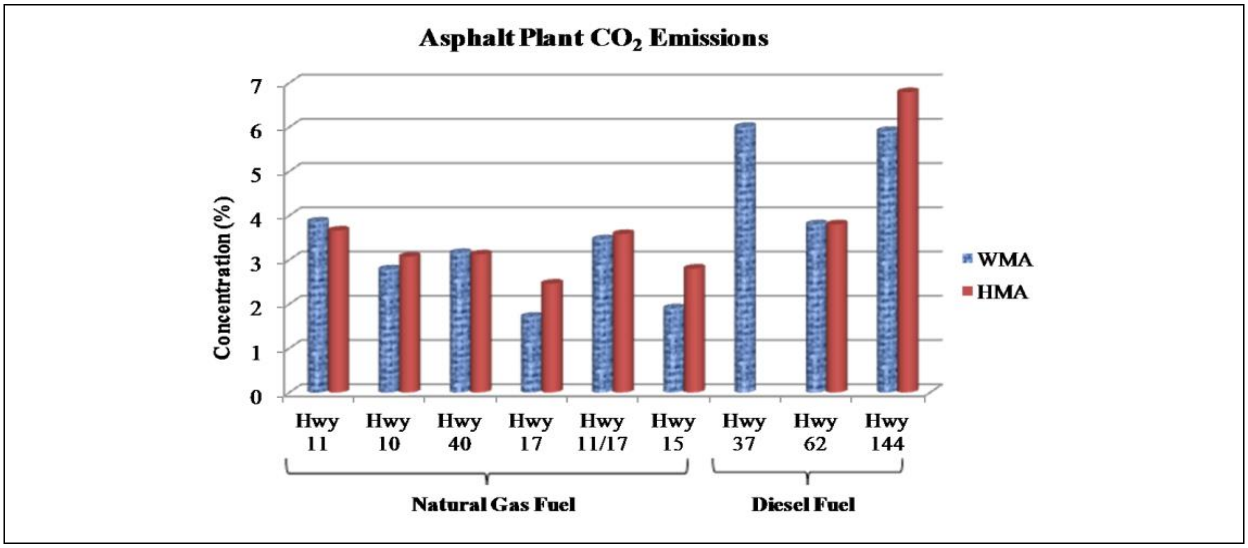


Figure 2-16 Emissions of WMA and HMA Contracts in Ontario During 2010 Season (Tabib et al., 2014)

In MTO’s study, asphalt fumes and odours at the paving site was also measured for five contracts. The study concluded that warm mix additives can be successfully used to reduce opacity (visible smoke) during the paving operations by 75 percent compared to HMA, as shown in Figure 2-17. Furthermore, the study stated that dust and benzene soluble fractions were found to be significantly lower for WMA as shown in Figure 2-18.

Figure 2-17 No Visible Fumes During WMA Paving

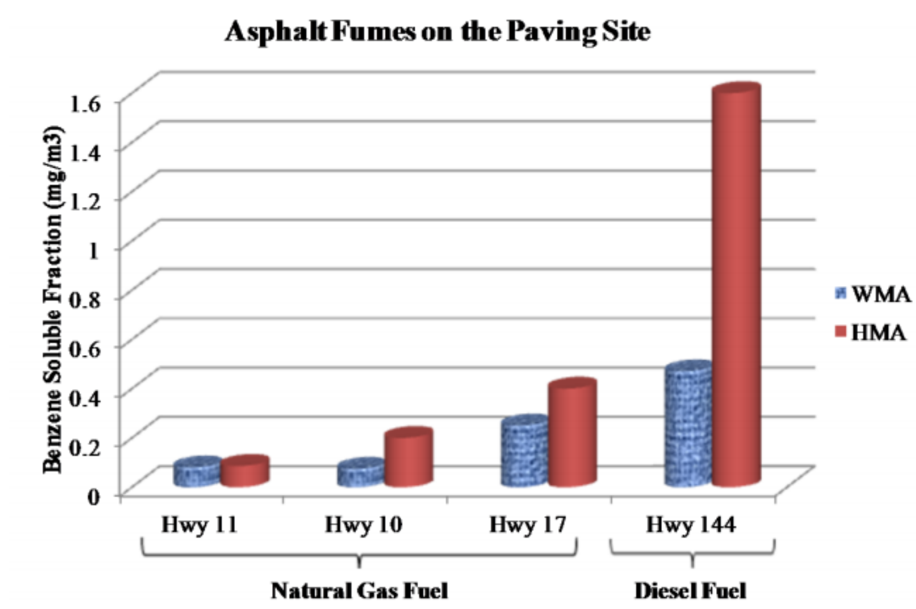


Figure 2-18 Asphalt Fumes on the Paving Site (Tabib et al., 2014)

2.3.3. Effect of WMA on Flexible Pavement Performance

Despite the potential environmental, and safety benefits of WMA, changes in the production process as compared to conventional HMA have raised concerns in regards to the long-term performance of WMA, particularly moisture susceptibility. In an effort to address these concerns, for the past few years, several studies were conducted evaluating different WMA technologies. A brief summary of conclusions drawn from these studies is provided in following sections. Additionally, literature regarding the possible increased susceptibility to rutting due to reduced aging is also provided in following sections.

Moisture damage occurs due to a loss of adhesion (the bonds between asphalt and aggregate) and/or cohesion (the bonds between asphalt molecules), which subsequently results in progressive strength reduction and decreases in stiffness of the mixture. Several mechanisms have been cited to be contributing factors to moisture damage including detachment, displacement, spontaneous, emulsification, film rupture, pore pressure, and hydraulic scouring (Solaimanian et. al., 2003). However, not all these mechanism are well understood due to the complexity of describing the level of impact of individual or combined mechanisms on the moisture sensitivity of a given mixture, as stated by (Solaimanian et. al., 2003).

It is generally accepted that the mineralogy and chemical composition of the aggregate type are the prominent factors that affect asphalt stripping. The general trend for these characteristics illustrate that rocks that are more acidic in composition have a greater affinity towards water than asphalt cement. The converse idea is believed to be true for more basic rocks (greater affinity for asphalt cement than water). Furthermore, researchers (Santucci & Aschenbrene, 2003) have found in both laboraotry experiments and field studies that moisture damage can be accelerated by mixture design or construction issues, including those given in Table 2-9.

From a review of the literature (FHWA, 2014) (Bonaquist, 2011) (Brits, 2004), the main factor that might contribute to the moisture susceptibility of WMA is the excessive aggregate moisture content due to the lowered production and compaction temperatures. This could possibly affect reduction of binder absorption by the aggregate, as well as binder-aggregate coating.

Rutting is also a concern for WMA mixtures. As stated by Anderson et al. (Anderson et al., 2008):

> *"The use of WMA technology has some potential engineering challenges. Since the asphalt cements may not harden as much at the lower production temperatures, a softer binder will likely be in the HMA mixtures when the pavement is opened to traffic and the mixture may have a potential for rutting".*

Table 2-9 Factors Contributing to Moisture-Related Distress (Santucci & Aschenbrene, 2003)

Mix Design	• Binder and aggregate chemistry • Binder content • Air voids • Additives
Production	• Percent aggregate coating and quality of passing the No. 200 sieve • Temperature at plant • Excess aggregate moisture content • Presence of clay
Construction	• Compaction – high in-place air voids • Permeability – high values • Mix segregation • Changes from mix design to field production (field variability)
Climate	• High rainfall areas • Freeze –thaw cycles • Desert issues (steam stripping)
Other Factors	• Surface drainage • Subsurface drainage • Rehab strategies – ship seals over marginal HMA materials • High truck traffic

2.4. Coloured Hot Mix Asphalt (CHMA)

Many metropolitan areas around the world have included coloured pavements in their infrastructure to denote dedicated lanes for buses and bicycles. These include cities such as New York, London, Ottawa, Sydney, and Auckland, which all have colouring applied to all or portions of their dedicated bus lanes (Seattle Department of Transportation, 2011). The transit benefits of these installations are well documented in terms of vehicle violation of the lanes; however, structural and functional performance has not been investigated systematically, and data are scarcer.

In collaboration with the University of North Carolina Highway Safety Research Center (UNCHSRC), the City of Portland investigated colour options for bike lane identification in the 1990s, and their analysis provides a broad analysis of material durability. These aspects are summarized in Table 2-10. Their research expected that the most durable solution would be a dyed asphalt wearing course; however, this was not tested due to the high cost of implementation. Portland installed test sections of painted and thermoplastic colours and found that while the painted material wore away after the first winter, the thermoplastic proved to still be in good condition after one year (Birk et al., 1999).

Table 2-10 Material Considerations for the City of Portland's Bike lane colouring research (Birk et al., 1999)

Material	Known Vendors	Approximate Materials Cost	Durability	Availability of colours
Paint	Local paint supply stores	$.04-$.10/ lineal foot	Poor	Wide variety
Thermoplastic	Flint Trading (Premark®)' 336-475-6600	$2.66/sq.ft	Good. Withstands significant volume & turning movements	Blue, red, yellow, white
Methyl methacrylate-based marking	Morton Traffic Markings (Dura Stripe®) 800-835-3357	$.50-$.60/ lineal foot	Potentially good	Yellow, White, red
Cold plastic	3M (Stamark®) 800-362-3455	$4.50 sq.ft.	Durable with inlay, not as good with existing asphalt; unlikely to hold up to heavy turning volumes	Blue, red, yellow. white
Dyed asphalt	Asphacolor® 800-258-7679	Very costly. Must apply fresh, treated asphalt.	Excellent	Earth-tones
Imprinted and sealed asphalt	Integrated Paving Concepts (Street Print) 800-688-5652	Costly. Must apply fresh asphalt.	Unknown	Earth-tones
Colored acrylic coating	Traffic Safety Systems (Zebraflex®) 407-348-2624	Unknown	Potentially good	Blue, Green, red, yellow

Note: All statements on this table were based on research work presented in (Birk et al., 1999). It is stated in the research that "*actual costs may vary widely depending on quantities, etc. Installation costs are not included. Other vendors may sell similar products*".

Coloured surface treatments for bus lanes were investigated by the New York City Department of Transportation (NYDOT) in 2010 in collaboration with Penn State University, as part of a study to improve the durability and cost-effectiveness of colouring treatments. NYDOT has implemented red-coloured bus lanes since 2008. Figure 2-19 shows one of the major bus corridors that was outfitted with red bus lanes. This study focused on long-term field observation and laboratory testing of various colouring treatments with a minimum durability and skid-resistance target of 3-years. These treatments presented in Table 2-11, which were selected to be applied on different road surfaces such as: (1) existing asphalt, (2) new asphalt, (3) existing concrete, and (4) new concrete (Carry, 2012).

Figure 2-19 Red bus lane on 1st Avenue in Midtown, Manhattan, United States (Carry, 2012)

Table 2-11 Colouring Products Evaluated for Use in NYDOT Red Bus Lanes (Carry, 2012)

<table>
<tr><th>Product</th><th>Product Type</th><th>Field Test</th><th>Lab Test</th></tr>
<tr><td>1</td><td>Red Street Paint, Brand A</td><td>No</td><td>Yes</td></tr>
<tr><td>2</td><td>Red Street Paint, Brand B</td><td rowspan="8">Yes</td><td>No</td></tr>
<tr><td>3</td><td>Epoxy with Red Aggregate (anti-skid), Brand B</td><td rowspan="6">Yes</td></tr>
<tr><td>4</td><td>Epoxy with Red Aggregate (anti-skid), Brand C</td></tr>
<tr><td>5</td><td>Red MMA with Aggregate (anti-skid), Brand D</td></tr>
<tr><td>6</td><td>Red-Tinted Portland Cement Micro Surface, Brand E</td></tr>
<tr><td>7</td><td>Red-Tinted Portland Cement Micro Surface, Brand F</td></tr>
<tr><td>8</td><td>Red Asphalt Concrete Micro Surface, Brand G</td></tr>
<tr><td>9</td><td>Chip Seal with Red Binder, Brand G</td><td>No</td></tr>
</table>

Overall, their work found that epoxy street paints produced the most durable solution, while micro surfaced HMA surfaces were promising and required further investigation (Carry, 2012). Moreover, the study concluded that, regardless of age/condition of the asphalt road surface, treatments experience intense wear at bus stops. This wearing was suggested to be due to factors such as; (1) friction caused by buses stopping and starting, and (2) prolonged heat exposure from bus engines. Figure 2-20 shows a comparison between wearing of a same product in bus stop and non-bus stop location after 3 years.

Laboratory evaluation of HMA overlays incorporating coloured synthetic binders for use in bus lanes in Seoul, South Korea have been studied. This work focused on an unnamed synthetic resin binder and performed Marshall stability test, indirect tensile strength, and modified Lottman moisture sensitivity tests to evaluate the strength and moisture susceptibility of the overlays. The results showed that the designed overlay was of higher strength and lower moisture susceptibility than traditional HMA mixtures, indicating the technical potential of coloured HMA overlay and structure solutions (Lee & Kim, 2007).

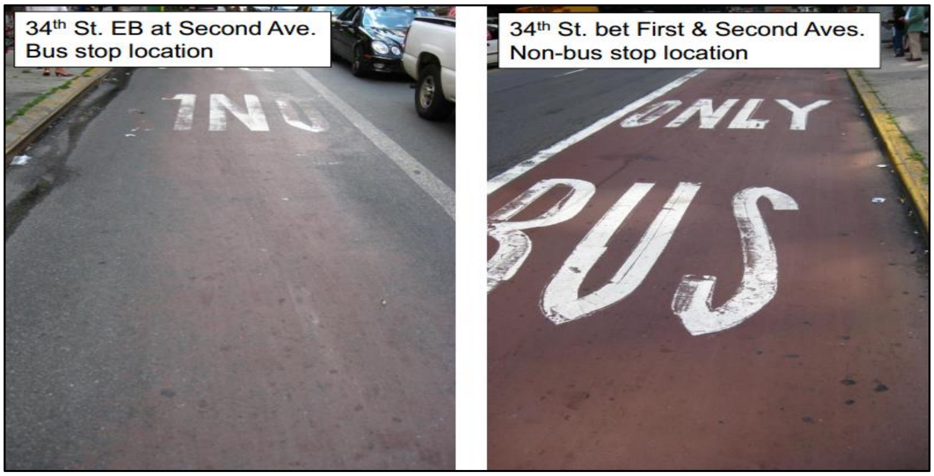


Figure 2-20 Effect of Stopping/Starting of Buses and Engine Heat Exposure (Carry, 2012)

The friction, thermal, and UV characteristics of coloured asphalt pavement surfaces have been studied to some extent. Friction is of high importance due to the minimum requirements for safe vehicle operation, and adding paints and polymers to the roadway surface increases the risk of making a driving surface too smooth (Carry, 2012). Thermal properties have also been extensively studied as one of the major benefits of coloured asphalt is reduced heat absorption, reducing the impact that the pavements have on urban heat island generation (Synnefa, 2009). Coloured asphalt reduces the amount of thermal energy absorbed compared to black asphalt. This can reduce the urban heat island effect which has major benefits for cities. Other parameters that affect the thermal and UV absorption of asphalt are permeability, thermal conductivity, convection and heat capacity.

A challenge with coloured asphalt might be an increased solar reflectance, causing glare problems for drivers. Since light coloured layers reflect some light that is in the visible part of the solar spectrum, some of the reflected light could be reflected at lower angles and travel into the sight of drivers and pedestrians. Glare is not often reported as a major cause of accidents, but might play a significant role in the case of rain events, early morning and late afternoon. It is reported that designing the pavement's texture to a coarser texture might reduce surface glare (AAPA, 2014).

The UV properties of a coloured surface are important as UV radiation will degrade the colour overtime and make it less effective for lane designation. While these factors have been widely analyzed, it is important to test these factors for any potential new installation in a new environment to ensure compatibility.

2.5. Summary of Challenges, Research Gaps and Opportunity for Innovation

This chapter provided a comprehensive review of current state of knowledge on concepts and approaches related to materials characterization, mix design, test methods pertaining to Coloured Hot Mix Asphalt (CHMA) and Warm Mix Asphalt (WMA) application of pavement. However, there are still questions and concerns that require to be addressed to ensure equivalent performance to conventional Hot-Mix Asphalt (HMA) with the additional benefits. A brief summary of these concerns are as follow:

Warm Mix Asphalt

Despite environmental and safety benefits of WMA, there are still challenges with warm mix technologies in Ontario, including:

- Effectiveness of different technologies – question still remains on cost effectiveness of different additives in reducing emissions. All additives available in the market are not the same.
- Ensuring long term performance including moisture susceptibility and rutting.
- Combination with antistrip additive – needed to ensure that the WMA additive was compatible with the antistrip additive when antistrip was needed

Coloured Hot Mix Asphalt

One of the challenges is finding scientific literature on the usage of colouring pigment in HMA, as all major chemical suppliers have their own proprietary products. The products tend to be either synthetic binders or pigment solutions. In addition, the usage of these materials in Canada has been extremely limited. All of these additives may have a direct impact on the structural, functional, and environmental performance of the pavement surface. So when selecting the ideal pigmentation option for a given region like York Region, questions remain on the importance of identifying the best options and perform thorough laboratory and field testing before implementation. Concerns and questions also remain on the best maintenance and rehabilitation practices to ensure colour and longevity of the pavement throughout the intended life-cycle.

Chapter 3 RESEARCH METHODOLOGY

3.1. Introduction

The primary objective of this study was to evaluate the performance of Coloured Hot Mix Asphalt (CHMA) pavements for usage in BRT lanes, and evaluate the effect of Warm Mix Asphalt (WMA) additives on the strength of compacted asphalt mixtures for usage in pavements in Southern Ontario. To achieve overall objectives, research plan shown in Figure 3-1 was developed to systematically evaluate CHMA and WMA in a similar manner.

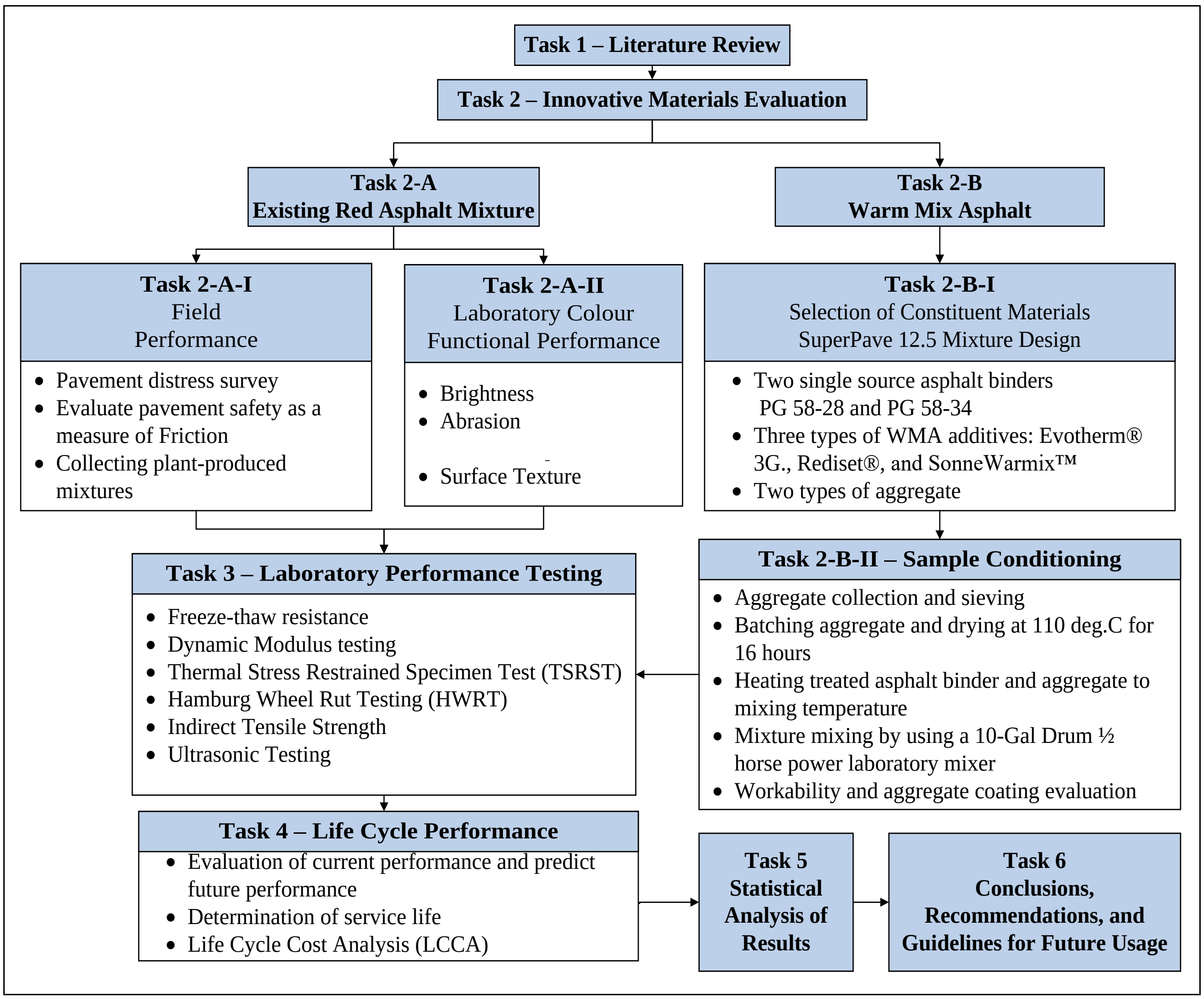


Figure 3-1 Research Plan Methodology

3.2. Experimental Materials

An array of materials was evaluated including those produced under controlled laboratory conditions and collected during plant production. A description of each material is provided in following sections.

3.2.1. Warm Mix Asphalt (WMA) Mixtures

To produce WMA mixtures under controlled laboratory conditions, Modified Binder (MB) prototypes were first produced following a consistent approach using a single-source PG 58-28 and 58-34 Polymer-modified base asphalt cements in combination with three types of warm mix additives: (1) Evotherm 3G, (2) Rediset LQ, and (3) SonneWarmix. More information on additives are given in Table 3-1. This information was retrieved from the Material Safety Data Sheet (MSDS) provided by the manufacturer. The selection of these additives were based on the preliminary literature review, availability to the paving industry, feedback obtained from a survey distributed by CPATT (referred to as CPATT-WMA 2015 survey, hereafter), and guidance from MTO. For each additive, supplier's recommended dosage rate (as presented in Table 3-1) was used to treat molten base binders with different type of additives.

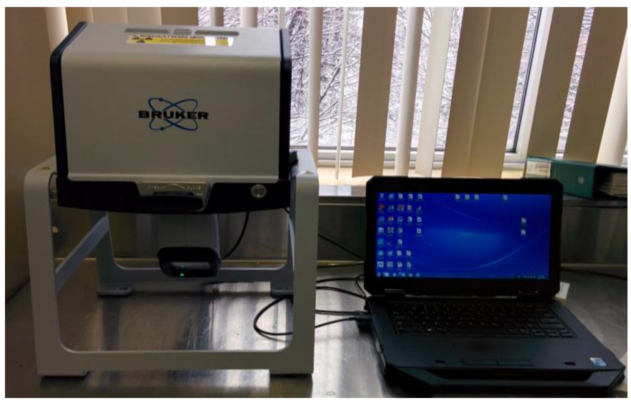
Due the relatively large number of additives available in the industry, the CPATT-WMA 2015 survey was designed to provide insights in the type of additives and evaluation procedures Canadian agencies use for WMA. More details on this survey is provided in Section 2.3.1 of this thesis. It should be reiterated that main objectives of this survey were to (1) document the state-of-the-art related to Warm Mix Asphalt technologies, and (2) identify candidate technologies or knowledge for inclusion in the work plan. This survey was distributed in a mail-back form in January 2015 to participants who were representing various transportation agencies. This survey was reviewed and received ethics clearance through a University of Waterloo Research Ethics Committee.

Table 3-1 Warm Mix Additive Information

WMA Additive	Type	Colour	Addition rate (% by binder weight)	Physical State at 25ºC
Evotherm 3G	Chemical (Fatty amine derivative)	Amber Dark	0.3	Liquid
Rediset LQ	Chemical (Surfactant blend)	Brown	0.5	Liquid
SonneWarmix	Wax/Organic	Brown	1.0	Solid

Two aggregate sources were used in this study to produce asphalt mixtures: trap rock diabase, referred to as aggregate A, and granite, referred to as aggregate B. Aggregate types and sources were selected based on the MTO and contractors past experience and historical records on their composition and level of resistance to moisture damage without use of anti-stripping agent: Type A trap rock with acceptable moisture performance and Type B pink granite with relatively weaker resistance to moisture damage.

Information regarding aggregate mineralogy and physical properties are listed in Table 3-2. Both aggregate types are listed in the MTO's Designated Sources for Material (DSM). It should be noted that composition of each aggregate type was confirmed by using an X-ray fluorescence (XRF) analyzer at MTO's bituminous laboratory shown in Figure 3-2 (a). For this test, 50-grams of material retained on different sieve sizes were batched and crushed by using two types of crushers to achieve a fine powder of passing 75-μm sieve size for XRF analysis, as shown in Figure 3-2 (b). Given in Table 3-2, XRF analysis verified that type B aggregate contained relatively higher percent of silicon dioxide (SiO_2) compare to type A, which is expected to cause type B aggregate source to be more susceptible to moisture damage.



(a) X-ray fluorescence (XRF) analyzer

(b) Fine powder of passing 75-μm sieve size used for XRF analysis

Figure 3-2 MTO's X-Ray Fluorescent (XRF) Analyzer and Sample Size Used For The Analysis

Table 3-2 Warm Mix Asphalt Aggregate Blends Physical Properties

Laboratory Test and Number[1]	Coarse (12.5 mm)			Washed Fines		Unwashed Fines	
	OPSS[2] Requirement	Type A[4]	Type B[5]	Type A	Type B	Type A	Type B
Flat and Elongated Particles (4:1 ratio, % maximum) LS-608	15	4.5	4.4				
Unconfined Freeze Thaw Loss (%maximum) LS-614	6	3.6	2.2				
Micro-Deval Abrasion Loss (% maximum) LS-618	10	4.3	7.2				
Micro-Deval Abrasion Loss (% maximum) LS-619	15			7.3	5.8	8.5	5.7
Silicon dioxide Content by XRF[3], passed 75-µm (% of weight)	-	42.5	57.0				

Note: [1]LS is MTO's Laboratory Standards, [2]OPSS is Ontario Provincial Standard Specification, [3]XRF is X-ray Fluorescence,[4]Type A is trap rock diabase, and [5]Type B is pink granite

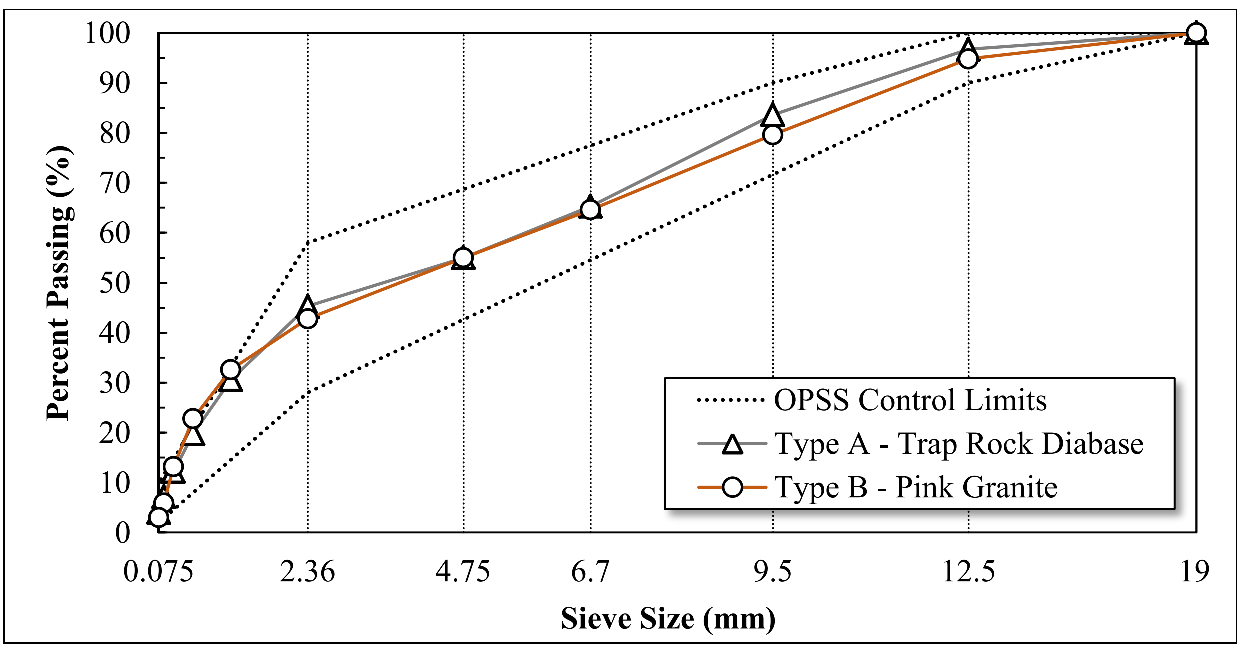


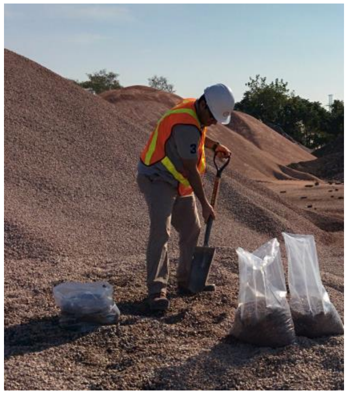
Figure 3-3 Aggregate Blend Chart of WMA Mixtures

Table 3-3 Warm Mix Asphalt Properties

Property		12.5 FC-2 OPSS[1] Requirement	Type A[2] Blend	Type B[3] Blend
Gradation (% Passing)	Sieve Size (mm)			
	16.0	-	99.0	99.9
	12.5	90 – 100	96.7	94.8
	9.5	45 – 90	83.6	79.6
	6.7	-	65.3	64.6
	4.75	45 – 55	55.0	55.0
	2.36	28 – 58	45.3	42.8
	1.18	-	30.6	32.6
	0.600	-	19.8	23.8
	0.300	-	12.2	13.2
	0.150	-	7.2	5.9
	0.075	2 – 10	4.0	3.0
N_{des} (%G_{mm})		96.0	96.0	96.0
N_{ini} (%G_{mm})		≤ 89.0	88.8	89
N_{max} (%G_{mm})		≤ 98.0	97.2	97
Air Voids (%) at N_{des}		4.0	4.0	4.0
Voids in Mineral Aggregate, VMA (% minimum)		14.0	14.7	14.3
Voids Filled with Asphalt, VFA (%)		65 – 75	73.2	72.2
Dust Proportion, DP		0.6 – 1.2	1.0	0.7
Asphalt Film Thickness (μm)		-	8.7	9.0
Asphalt Cement Content (%)		-	4.7	5.0

Note: [1]OPSS is Ontario Provincial Standard Specification, [2]Type A is trap rock diabase, [3]Type B is pink granite, N_{des}, N_{ini}, N_{max} are number of gyrations at different compaction levels (design, initial, and maximum), and G_{mm} is theoretical maximum specific gravity.

Asphalt mixtures were produced in the CPATT's laboratory at the University of Waterloo by following a consistent approach as shown in Figure 3-4 by using modified asphalt binders. All mixtures were short-term aged prior to testing by using a forced draft oven: HMA control mixtures for 4 hours at 135ºC as per AASHTO R30 and WMA mixtures for 2 hours at field compaction temperatures as per AASHTO R35.



(a) Aggregate stockpile sampling

(b) Sieving oven-dried aggregate

(c) Batching the aggregate blend according to the JMF

(d) Adding heated aggregate blend to the mixer drum

(e) Adding required amount of asphalt binder treated with warm-mix additive

(f) Mixing for 90 as per AASHTO R35 at recommended mixing temperature

(g) Mixture discharging on flat, non-abrasive, non-absorptive, and warm surface area

(h) Flattening and quartering with warmed and oil-sprayed tools to obtain desired testing sample size.

(i) Evaluating coating of particles with asphalt binder in accordance with AASHTO T-195 (AASHTO, 2011)

Figure 3-4 Laboratory Procedure for WMA Production in CPATT Laboratory

3.2.2. Coloured Hot Mix Asphalt (CHMA) Mixtures

For this study, both plant and laboratory produced CHMA are evaluated. Plant-produced mixtures were collected during number of site visits conducted by the CPATT research team in different dates. A brief description of each site visit and type of material sampled are provided in the following sections. It should be noted that during all the visits liaisons from York Region were present.

The first site visit was conducted on August 21, 2014 to observe the plant production of a coloured asphalt mixture (referred to as Initial red asphalt, hereafter), as well as paving of a section of Highway 7 BRT lane (Figure 3-5). During this visit, samples were collected for further testing. Initial mixture consisted of a pink granite aggregate blend, red proprietary pigment and polymer modified PG 70-28 asphalt binder. The aggregate blend consisted of 12.5 mm coarse aggregate, and crusher fines (washed, and unwashed) to meet physical requirements of Superpave 12.5 FC2 mixture type (as listed in Table 3-5) for use in Traffic Category ‘D’ as per Ontario Provincial Standard Specification (OPSS). This type of mixture is intended to provide adequate rutting and skid resistance for a 20-year ESALs level of 10 to 30 million. It should be noted that the pink aggregate used in the initial mixture is the same source as the aggregate Type B with aggregate mineralogy and physical properties listed in Table 3-2. Physical properties of the colouring pigment are listed in Table 3-4.

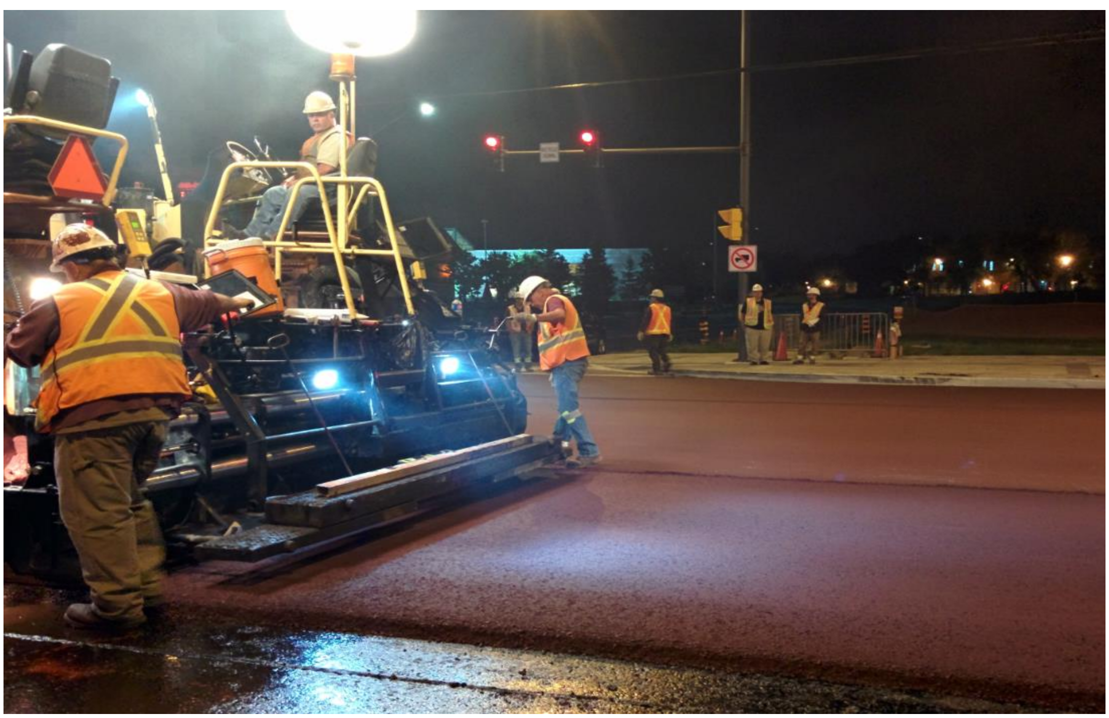
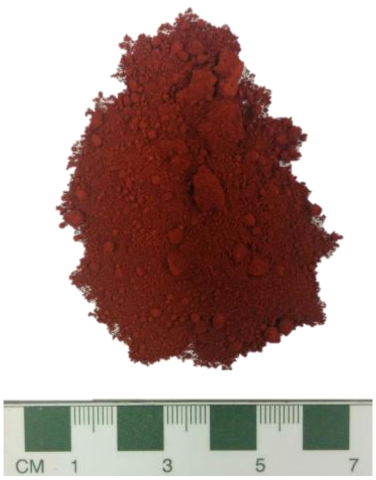


Figure 3-5 Red Asphalt Paving on a Section of Highway 7 BRT Lane York Region ON, August 21st 2014

Table 3-4 Colouring Pigment Properties

Colour/ Texture	Property		Red Pigment
		Sieve Size (mm)	
	Gradation (% Passing)	16.0	100
		12.5	100
		9.5	100
		6.7	100
		4.75	100
		2.36	100
		1.18	100
		0.600	100
		0.300	95.0
		0.150	92.0
		0.075	77.0
Absorption (%), LS-605			0.065
Apparent Specific Gravity, G_{sa}, LS-605			2.725
Bulk Specific Gravity, G_{sb}, LS-605			2.720

In practice, there are two methods of sampling asphalt paving mixtures: (1) behind the paver and (2) from the hauling truck. Due to the relatively large amount of materials required for this project, these two methods were found to be time-consuming and not practical to sample the initial mixture. Alternatively, a loader was used to collect close to one tonne at the batch discharge of the production facility. For this type of sampling following steps was taken, as illustrated in Figure 3-6:

1. Cleaning and lubricating the loader bucket to prevent the introduction of foreign substances into the paving mixture and to prevent the mixture sticking to the bucket.
2. Piling the material at a safe location, and flatten the pile to form a pad with a depth of approximately 0.5 meters.
3. Using a flat, square end shovel to sample around the pad into sample boxes. Extra care was taken to ensure that each box is filled with samples taken from at least three locations of the pad. Extra care was taken to avoid segregating the material while sampling. Approximately, each box was filled with 20 kilograms of material.

(a) Paving mixture piling and creating a pad

(b) sampling of the plant produced red Asphalt

Figure 3-6 Steps of Sampling Plant Produced Initial Red Asphalt

Second site visit was conducted on November 25, 2014 to observe paving of a section of Newmarket BRT-lane and also collect samples of the SP 19 binder course. Sampling was performed by using the behind the paver method. The performance evaluation of SP 19 binder course is not part of the scope of this thesis; however, the result of the binder's performance testing was used as input for The AASHTOWare's Mechanistic-Empirical (M-E) Software to increase the reliability of the software's output in regards to the overall pavement's structure performance. Sectios 3.5.2 provides more details on how the AASHTOWare's Mechanistic-Empirical (M-E) Software was used in this research.

After few years, sections of BRT lanes paved with initial red mixture exhibited cracks that raised concern about the integrity and performance of the initial red mixture. To address these concerns, performance evaluation performed as part of this thesis provided insight to the decision makers to modify the initial red mixture to a mixture referred to as New Red Mix. This mixture is also evaluated in this thesis.

Pigmented and no pigment laboratory-produced mixtures were also included in this study, which were produced in CPATT laboratory by following JMFs presented in Table 3-5. This was to capture effect of pigmentation on the mixture's performance. These mixtures were produced following a mixture design provided by the manufacturer of CHMA and were produced by following a consistent approach similar to steps shown in Figure 3-4 with additional steps shown in Figure 3-7.

(a) Adding heated red granite aggregate blend

(b) Adding red pigment

(c) Adding polymer modified asphalt binder

(d) Mixing for 90 seconds to achieve consistent colour and coating

Figure 3-7 Steps of Producing Red Asphalt Mixture in Laboratory

Table 3-5 Coloured Hot Mix Asphalt and Binder Course Asphalt Properties

Property		OPSS Requirement	Initial Red Mix	New Red Mix	Binder Course
	Sieve Size (mm)				
Gradation (% Passing)	19.0	(90 – 100)[1]	-	-	95.9
	16.0	-	100	100	88.4
	12.5	90 – 100 (23 – 90)[1]	98.2	94.7	79.5
	9.5	45 – 90	83.4	79.1	70.8
	6.7	-	65.2	64.0	60.3
	4.75	45 – 55	56.2	55.0	53.2
	2.36	28 – 58 (23 – 49)[1]	48.0	43.0	41.1
	1.18	-	36.9	33.0	29.3
	0.600	-	27.9	24.3	22.1
	0.300	-	17.5	13.7	15.6
	0.150	-	10.2	6.4	8.20
	0.075	2 – 10 (2 – 8)[1]	5.8	3.3	4.20
N_{des} (%G_{mm})		96.0	96.1	96	96
N_{ini} (%G_{mm})		≤ 89.0	89.1	89	88.5
N_{max} (%G_{mm})		≤ 98.0	97.8	97	97.3
Air Voids (%) at N_{des}		4.0	3.9	4.0	4.0
Voids in Mineral Aggregate, VMA (% minimum)		14.0 (13)[1]	14.1	14.3	13.0
Asphalt Binder Performance Grade		-	PG 70-28 P[2]	PG 64-34P	PG 64-28 P
Voids Filled with Asphalt, VFA (%)		65 – 75	72.1	72.2	72.0
Dust Proportion, DP		0.6 – 1.2	1.3[3]	0.7	1.0
Tensile Strength Ratio, TSR (%)		80	97.6	91.3	83.7
Asphalt Film Thickness (μm)		-	6.8	9.0	7.9
Asphalt Cement Content (%)		-	4.9	5.0	4.65

Note: [1]limits for binder course only, [2]P means polymer modified asphalt binder, and [3]selected values are slightly larger than specified limits to promote finer gradation and texture as requested by the agency owner

3.3. Modified Binder Characterization

The Superpave PG binder specification of AASHTO M320 (AASHTO, 2010) was used to characterize each modified binder used to produce warm mix asphalt mixtures. This was to ensure recommendations provided by the additive suppliers were appropriate for this study and all binders are exhibiting similar high and low temperature performance grades. This was also to ensure that the targeted PG grades were not adversely affected by additives.

The AASHTO M320 includes an array of rheological tests associated with the control of workability, rutting (permanent deformation), fatigue cracking, and thermal cracking at specific temperatures and aging conditions. While Figure 3-8 illustrates the way such rheological tests are used in combination to establish the PG grades, Table 3-6 provides a brief description of each test. More details on each test is provided in results section. The binder modification and characterization was performed at AASHTO's AMRL certified and Canadian Council of Independent Laboratories (CCIL) accredited research and development laboratory of McAsphalt Industries located in Toronto.

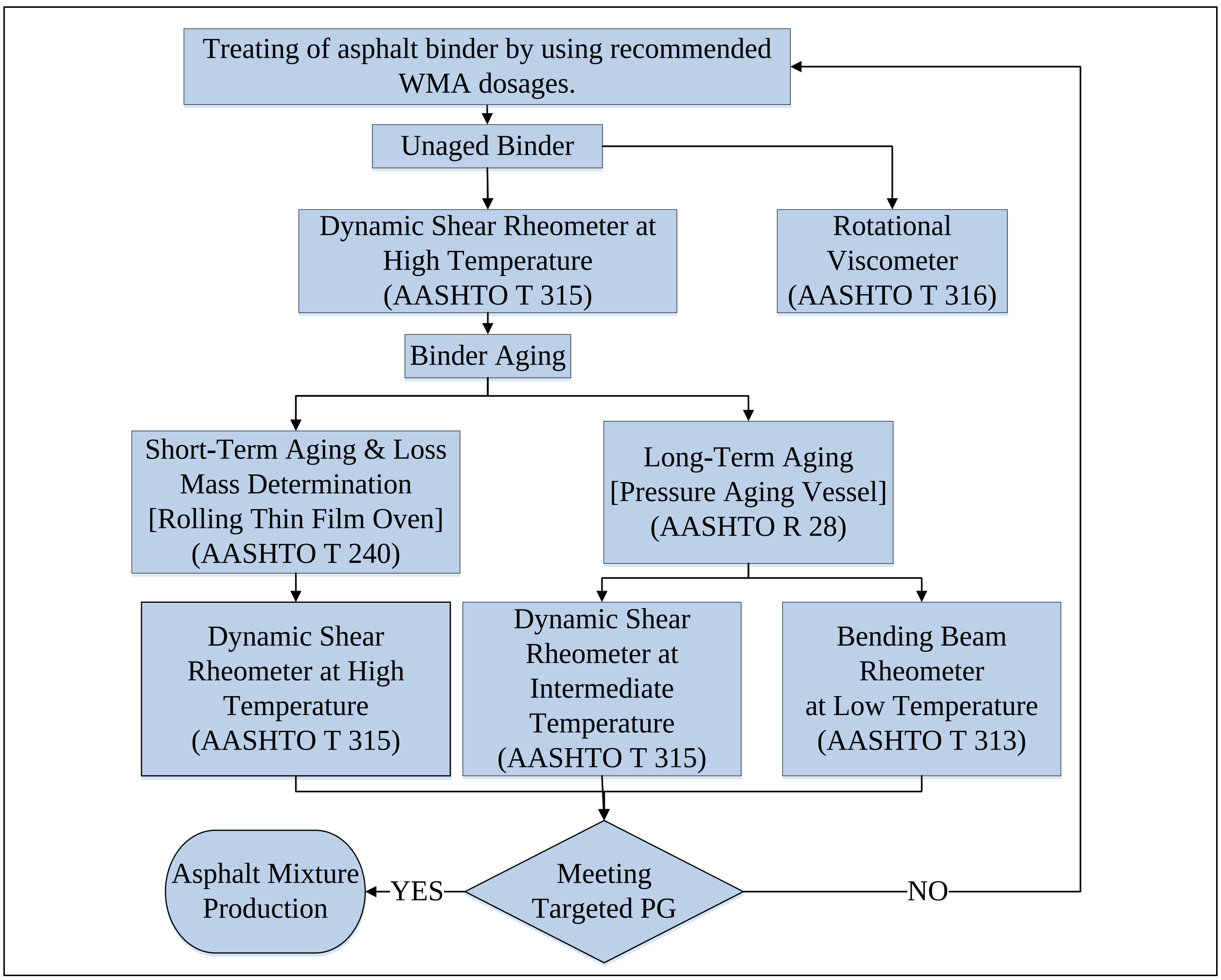
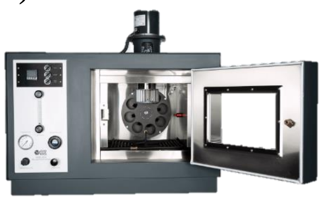


Figure 3-8 Modified Binders Characterization Flow Chart

Table 3-6 Test Method and Equipment Description of Asphalt Binder Characterization

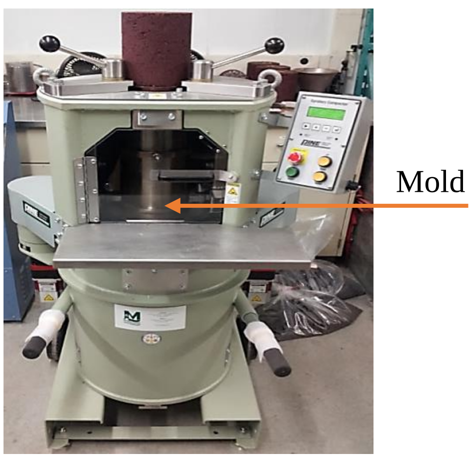
Binder Test	Purpose	Equipment
Rotational Viscometer (RV)	Measuring viscosity of asphalt binder at different temperatures to establish mixing and compaction temperatures. Also, used to ensuring binder can be pumped through the plant.	Brookfield© RV 
Dynamic Shear Rheometer	Measuring physical properties of asphalt binder at high and intermediate pavement temperatures to ensure adequate level of resistance to rutting and fatigue cracking, respectively.	Anton Paar© DSR 
Short-term aging	Simulating hardening (aging) characteristics of asphalt binder during mixture plant production. Aging is due to exposure to elevated temperature of mixing (i.e. more than 150 deg.C for conventional HMA)	Despatch© Rolling Thin Film Oven (RTFO) 
Long-term aging	Simulating aging characteristics of asphalt binder during years of in-service. Aging is due to exposure to UV light and climatic conditions.	Prentex© Pressure Aging Vessel (PAV) 
Bending Beam Rheometer	Measuring physical properties of asphalt binder at low temperatures to ensure adequate level of resistance to thermal cracking.	Cannon© Bending Beam Rheometer 

3.4. Asphalt Mixture Characterization

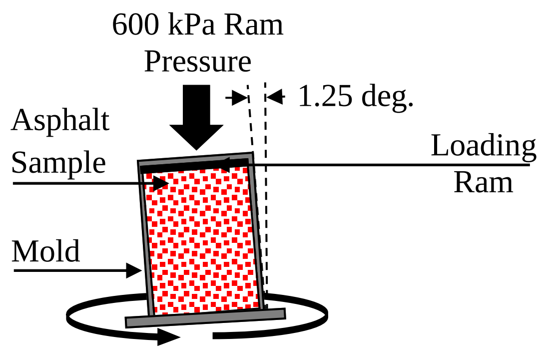
This section describes the test methods employed to characterize mixtures containing colouring pigment and warm mix additives in the state-of-the-art pavement laboratory at CPATT, located at the University of Waterloo. Methods of fabricating specimen for these tests are also provided in this section.

3.4.1. Specimen Fabrication

The CPATT Superpave Gyratory Compactor (SGC) and Asphalt Vibratory Compactor (AVC) as shown in Figure 3-9 were used to fabricate compacted specimens. The specimens measuring 152 mm in diameter and approximately 180 mm in height were compacted at pressure of 600 kPa in accordance with AASHTO PP 60-13, "Standard Practice for Preparation of Cylindrical Performance Test Specimens Using the Superpave Gyratory Compactor (SGC) (AASHTO, 2013). The CPATT Asphalt Vibratory Compactor (AVC) was used to compact specimens measuring 380 mm in length and approximately 125 mm in height.



(a) AFG2 Pine Superpave Gyratory Compactor

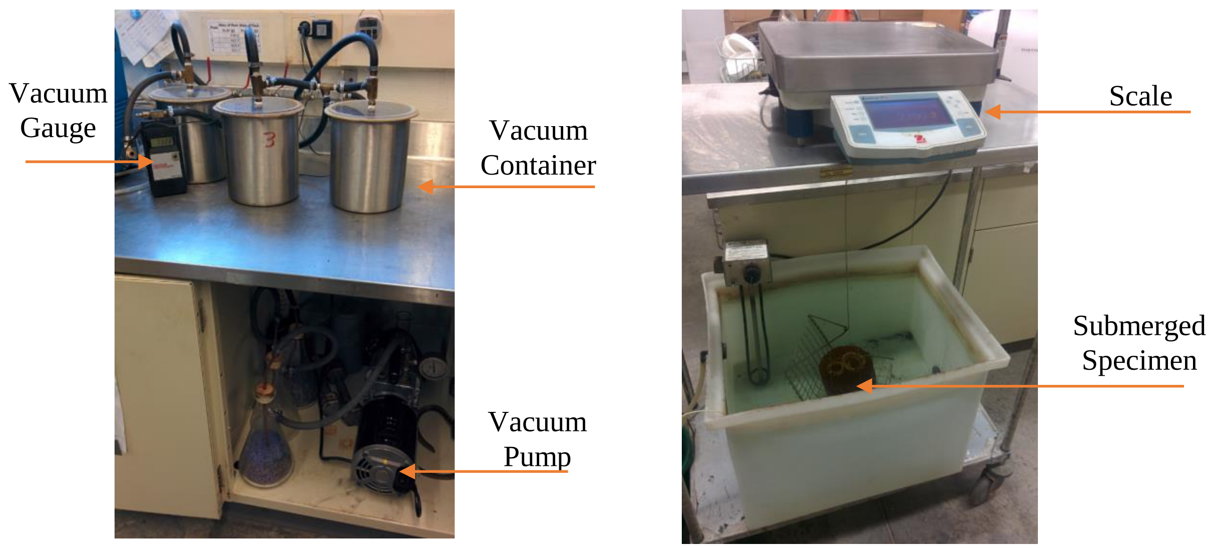


(b) Schematic Inside View of Gyratory Compactor and method of compaction

(c) Asphalt Vibratory Compactor

Figure 3-9 CPATT Compaction Equipment

The SGC is capable of recording the position of the loading ram as well as specimen height throughout the compaction. This measurement was used to achieve a target air void of 7 ± 1 percent (93 percent G_{mm}) for each specimen. In order to do so, trial specimens for the mixture was compacted to 100 gyrations (N_{100}). Volumetric properties such as bulk specific gravity (G_{mb}), and theoretical maximum specific gravity of loose mixture (G_{mm}) were then calculated for this specimen by apparatus shown in Figure 3-10.



(a) Maximum Relative Density (MRD) Apparatus (loose mixture)

(b) Bulk Relative Density (BRD) Apparatus (Compacted Mixture)

Figure 3-10 CPATT Apparatus to Determine Volumetric Properties of Loose and Compacted Mixtures

G_{mm} was determined by measuring the specific gravity of loose mixture after removal of the air entrapped in the mixture by using a vacuum saturation, as shown in Figure 3-10(a). Equation 3-1 was used to determine G_{mm} values in accordance with LS-264, "Theoretical Maximum Relative Density of Bituminous Paving Mixture" (MTO, 2012a). Compacted bulk specific gravity (G_{mb}) refers the specific gravity of compacted specimen, which includes the volume of air voids within the specimen. Determination of G_{mb} was performed in accordance with LS-262, "Bulk Relative Density of Compacted Bituminous Mixes" (MTO, 2012b). G_{mb} is calculated by using Equation 3-2. Figure 3-10(b) shows an apparatus employed to weight compacted specimen in water.

$$G_{mm} = \frac{A}{A + B - C} \tag{3-1}$$

where

G_{mm} = theoretical maximum specific gravity of loose mixture
A = mass of oven-dry specimen in air (g)
B = mass of container filled with water at 25ºC (g)
C = mass of container with specimen filled with water at 25ºC (g)

$$G_{mb} = \frac{A}{D - E} \quad \textbf{3-2}$$

where

G_{mb} = bulk specific gravity of compacted specimen
A = mass of dry specimen in air (g)
D = mass of the saturated surface-dry specimen in air (g)
E = mass of the specimen in water at 25ºC (g)

After performing necessary volumetric calculations, a plot of percent air void versus number of gyrations was developed by using Equation 3-3 and 3-4. Figure 3-11 shows an example of a plot constructed for a mixture tested for this study.

$$G_{mb@Nx} = \frac{h_{@N100} \times G_{mb@N100}}{h_{@Nx}} \quad \textbf{3-3}$$

where

$G_{mb@Nx}$ = estimated bulk specific gravity of compacted specimen at X gyrations
$G_{mb@N100}$ = bulk specific gravity of compacted trial specimen at 100 gyrations
$h_{@N100}$ = final height of trial specimen compacted at 100 gyrations (mm)
$h_{@Nx}$ = height of specimen throughout the compaction at x gyrations (mm)

$$VTM_x = 100\left(1 - \frac{G_{mb@Nx}}{G_{mm}}\right) \quad \textbf{3-4}$$

where

VTM_x = estimated air voids in compacted mixture at x gyrations (%)
G_{mm} = theoretical maximum specific gravity of loose trial mixture

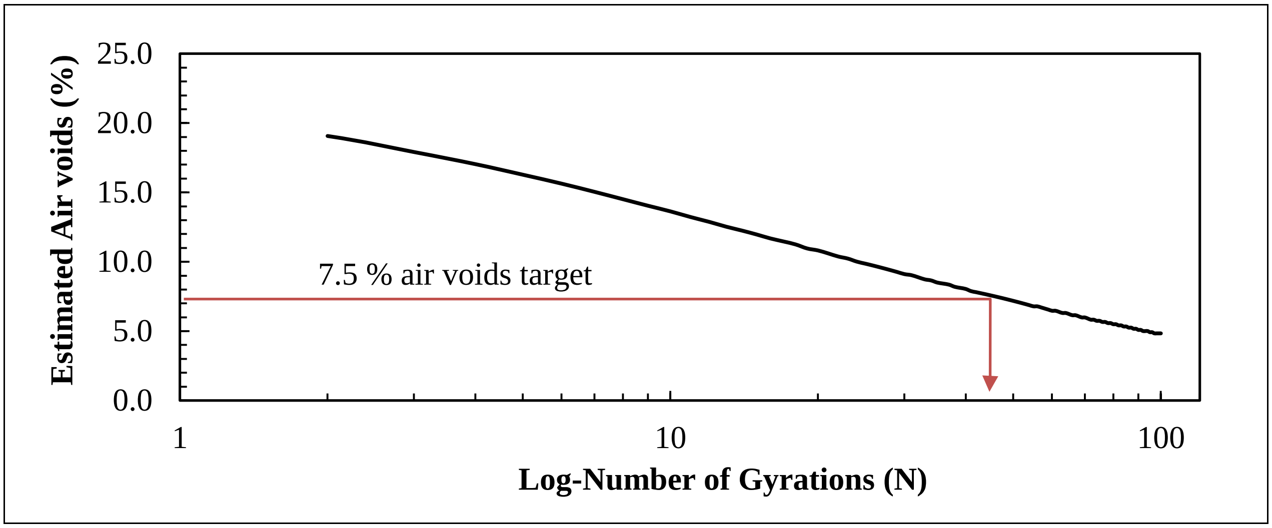


Figure 3-11 A sample plot of SGC Trial Specimen Compaction Characteristic

For specimens compacted by using the AVC, similar aforementioned steps were followed to obtained slabs with air voids target level of 7.5 percent.

3.4.2. Dynamic Modulus Test

Dynamic Modulus test was used to determine cracking distress development and propagation related to CHMA and WMA mixtures. Dynamic Modulus can be used as a material property for Level 1 Mechanistic-Empirical Pavement Design Guide (MEPDG) as well. Dynamic modulus is abbreviated as E* (pronounced as E-star), where E for elastic modulus and star for dynamic (NCHRP, 2011).

E* is a complex number that is used to relate stress to strain for visco-elastic materials, like asphalt mixtures. This relationship is determined by performing a laboratory test under a sinusoidal stress-controlled mode, in which the stress applied is given by:

$$\sigma = \sigma_o \sin(\omega t) \qquad \textbf{3-5}$$

where

$\sigma_o =$ peak (maximum) stress (kPa)
$\omega =$ angular velocity (Hz)
$t =$ time (seconds)

The corresponding strain is then expressed by:

$$\varepsilon = \varepsilon_o \sin(\omega t + \delta) \qquad \textbf{3-6}$$

where

$\varepsilon_o =$ peak (maximum) strain
$\phi =$ phase angle (degrees)

In the above equations, ω is related to the frequency (f) of applied stress given by $\omega = 2\pi f$. Also, in the Equation 3-6, phase angle (ϕ) is related to the visco-elastic bahavior of the asphalt mixture and it is the lagging time of the measured strain to the stress applied.

If the asphalt mixture were a perfectly elastic material, the reponse would immediately match with the applied stress and ϕ would be zero degree angle as shown in Figure 3-12(a). On the other hand, a perfectly viscous material would have a large ϕ of 90 degree angle as shown in Figure 3-12(b). At pavement in-service temperatures, asphalt mixtures behave both viscous and elastic, and the relationship between the maximum stress and strain is commonly used to quanitfy such behavior in terms of complex modulus |E*|. This relationship is defined by Equation 3-7.

$$|E^*| = \frac{\sigma_0}{\varepsilon_o} \qquad \textbf{3-7}$$

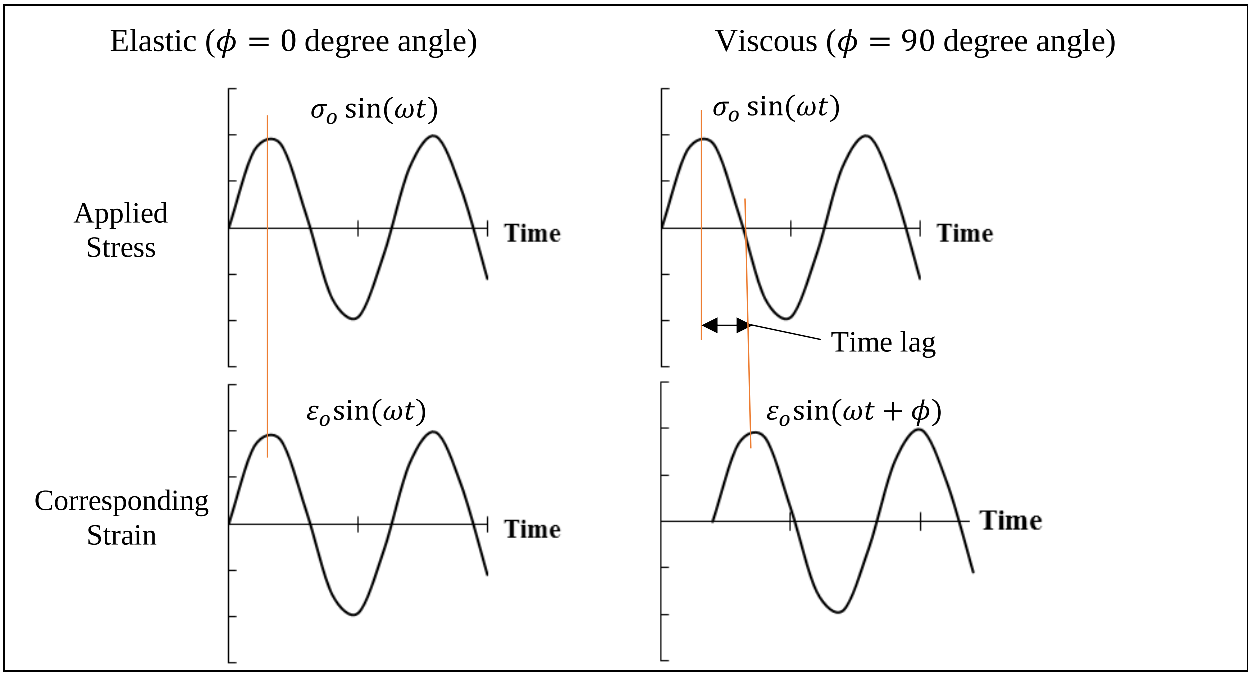
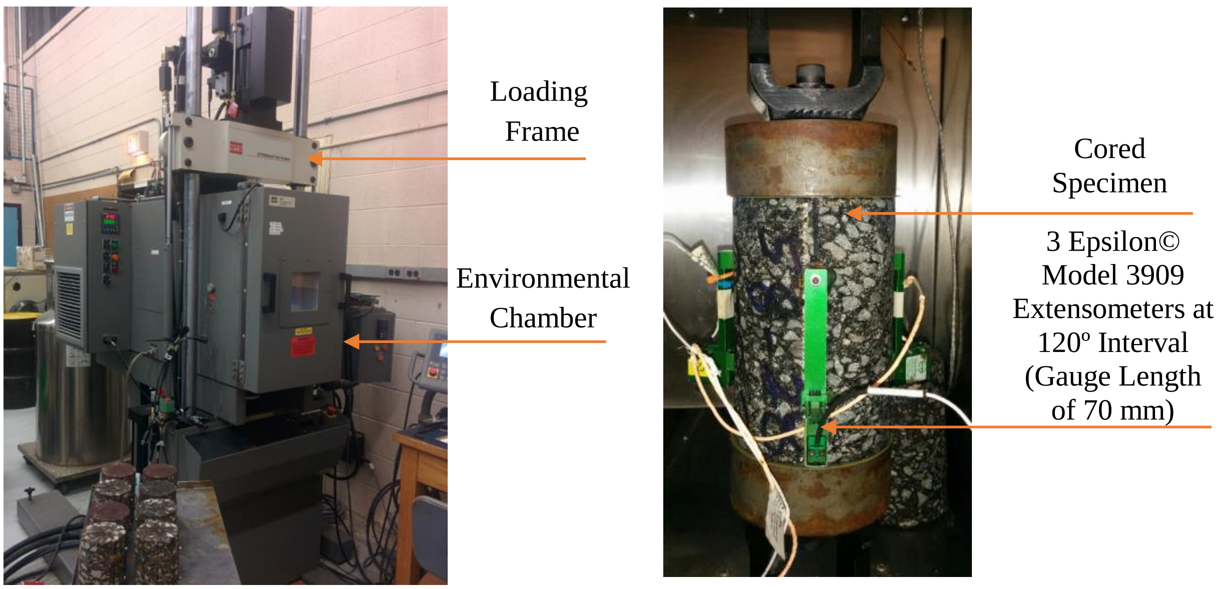


Figure 3-12 Graphical Illustration of Dynamic Modulus Test Parameters

A Material Testing System (MTS) at CPATT as shown in Figure 3-13 was used to determine dynamic modulus (|E*|) of specimens, each measuring 100 mm in diameter and 150 mm in height. Specimens were cored and cut from the middle of a SGC Compacted specimen measuring 150 mm in diameter by 180 mm in height, as illustrated in Figure 3-14.



(a) 810 MTS loading frame and 651 MTS environmental chamber

(b) Inside view of the environmental chamber

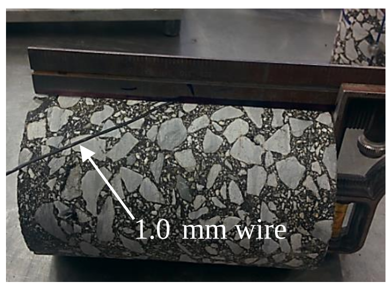
Figure 3-13 CPATT Dynamic Modulus Test Setup

(a) Coring a SGC specimen

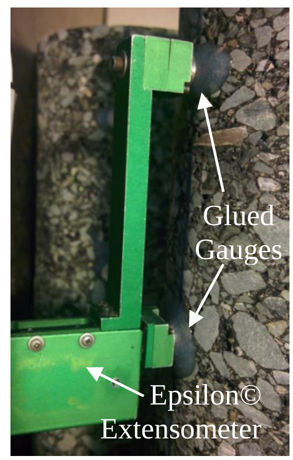
(b) Saw-cutting both ends of the specimen to achieve desired testing height

(**c**) Using Marui© G-402 diamond grinder to flatten and smooth two ends

(d) Inspecting the flatness of each end at 120º Interval by using a precision level



(e) Inspecting the perpendicularity by using a precision square and 1.0 mm carbon steel wire at different locations



(f) Drawing guiding lines at 120º Interval to glue steel gauges that allow attachment of magnetic ends of the extensometer

Figure 3-14 Preparing a Dynamic Modulus Testing Specimen

The cored specimens were tested at six loading frequencies (0.1, 0.5, 1, 5, 10 and 25 Hz) and five different temperatures (-10, 4.4, 21.1, 37.8 and 54.4 degree Celsius) to obtain |E*| in accordance with AASHTO TP 62-07 “Standard Method of Test for Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures” (AASHTO, 2007). Measurements obtain from this test were further combined to obtain a master curve in accordance with AASHTO PP62-09 procedure, “Standard Practice for Developing Modulus Master Curve for Hot-Mix Asphalt” (AASHTO, 2009).

Fitting the |E*| results into the master curve was performed by using "Solver" analysis tool in Microsoft Excel© program spreadsheet based on second-order polynomial. The procedure of developing a master curve involved applying a shift factors that transferred the |E*| results obtained at different temperatures to a reference temperature of 21.1ºC, as illustrated in Figure 3-15. Shifting was constructed using the principal of time-temperature (Equation 3-8 and 3-9) with respect to time until the curves merge into a single smooth function (Witczack, 2005).

$$\log|E^*| = \delta + \frac{\alpha}{1 + e^{\beta + \gamma \log(t_r)}} \tag{3-8}$$

where

$\delta =$ minimum value of E* (MPa)
$\alpha =$ span of modulus values
$\beta , \gamma =$ shape factors
$t_r =$ time of loading at reference temperature of 21.1ºC

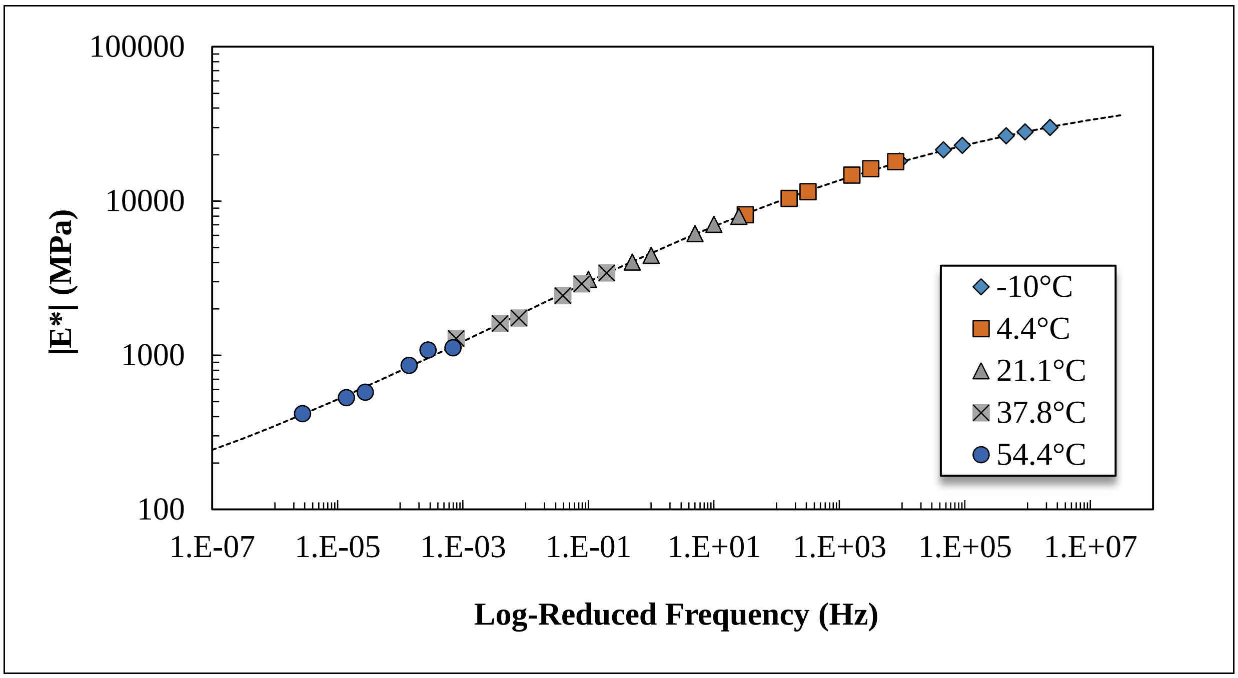
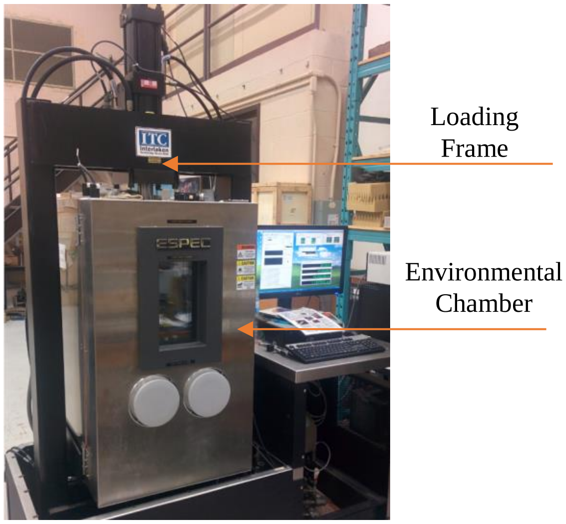


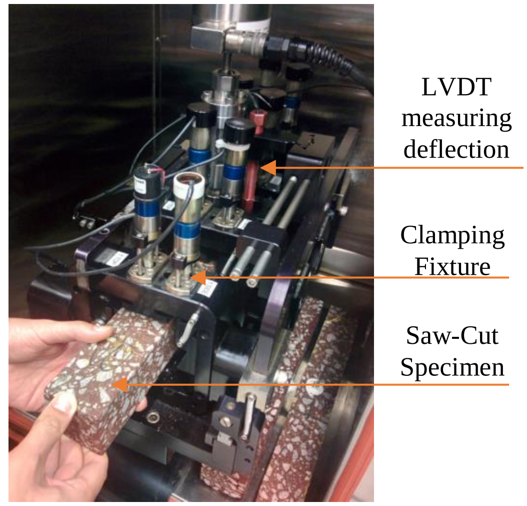
Figure 3-15 Dynamic Modulus Master Curve 12.5FC2 Mixture Example

3.4.3. Four-Point Flexural Beam Fatigue Test

Effect of colouring pigment on the level of resistance to fatigue cracking was evaluated by performing the flexural beam fatigue test in accordance with the AASHTO T 321-07, "Determining the Fatigue Life of Compacted Hot Mix Asphalt (HMA) subjected to Repeated Flexural Bending" (AASHTO, 2007). The CPATT test setup used for this test is shown in Figure 3-16.



(a) ITC loading frame and ESPEC environmental chamber

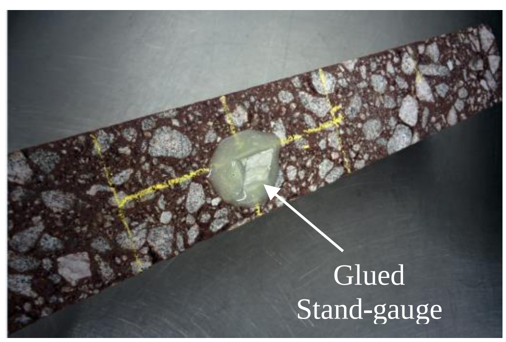


(b) Inside view of the environmental chamber

Figure 3-16 CPATT Repeated Flexural Fatigue Bending Test Setup

For this testing program, beams measuring 380 mm long by 63 mm wide by 50 mm thick were saw-cut from asphalt slabs compacted by using Asphalt Vibratory Compactor (AVC). Slabs were compacted to a targeted air void content of 7 ± 1 percent. Steps involved in preparing a beam for testing are shown in Figure 3-17.

(a) Saw-cutting two beams from a slab



(b) Inspecting the straightness of beams by using a precision square and 1.0 mm carbon steel wire at different locations Drawing guiding lines to glue a stand-gauge for LVDT measurements

Figure 3-17 Preparing a Fatigue Beam Testing Specimen

The flexural beam testing procedure involved subjecting an asphalt beam to flexural loading applied in a cycle manner with loading frequency within at 10 Hz frequency at a specified deflection level (also known as “micro-strain level”). The fatigue failure was then defined as the number of load cycles until initial stiffness is reduced by 50 percent.

3.4.4. Thermal Stress Restrained Specimen Test (TSRST)

The effect of colouring pigment on the durability of the asphalt mixture at lower temperatures was further evaluated by using the same CPATT equipment used for dynamic modulus test with exception of different testing setup as shown in Figure 3-18. TSRST was performed in accordance with the AASHTO TP 10-93, “Standard Test Method for Thermal Stress Restrained Specimen Tensile Strength” (AASHTO, 1993).

(a) 810 MTS loading frame and 651S environmental chamber

(b) Inside view of the environmental chamber

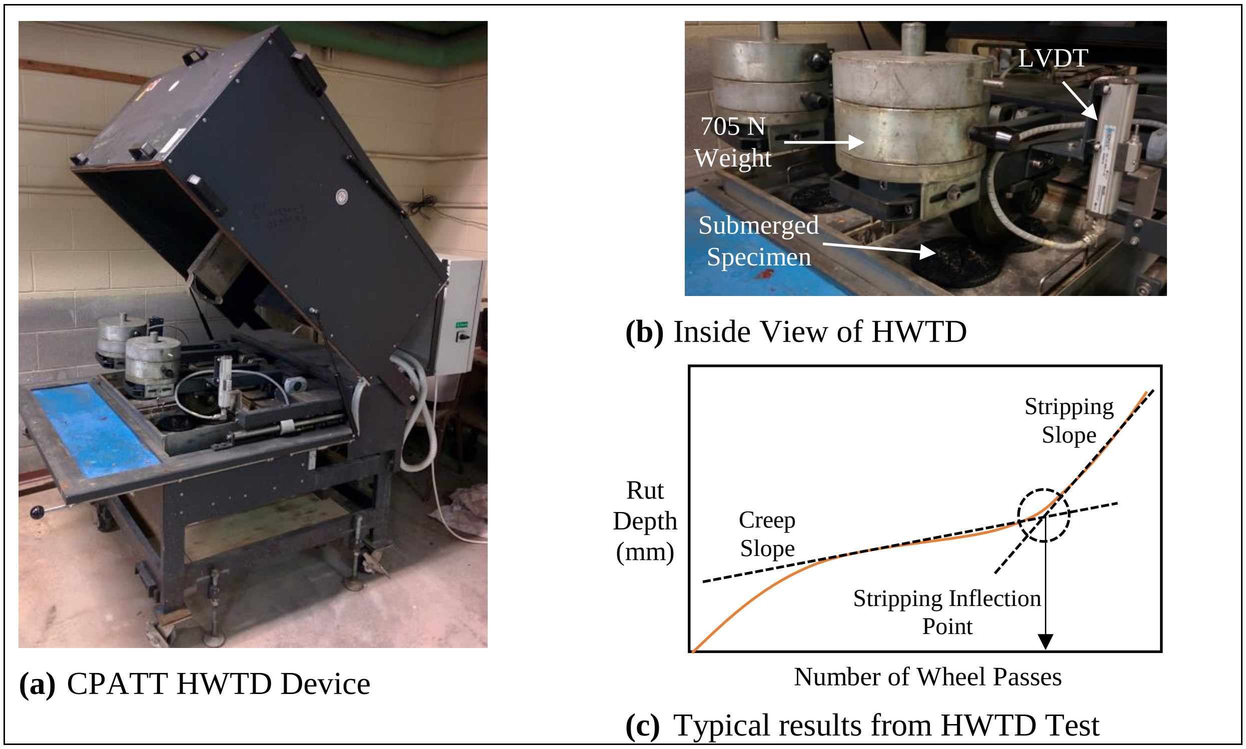
Figure 3-18 CPATT Thermal Stress Restrained Specimen Test Setup

The testing procedure involved restraining a rectangular beam from contraction while being simultaneously subjected to a constant cooling rate of -10ºC (14ºF) per hour. Resistance to thermal cracking is then evaluated as the temperature in which a fracture is developed within the length of specimen. Beams measuring 50 mm by 50 mm in cross-section and 250 mm in height were prepared similar to fatigue beams. Slabs were compacted to a targeted air void content of 7 ± 1 percent.

3.4.5. Hamburg Wheel Tracking Device (HWTD)

The resistance of compacted asphalt mixtures to rutting and moisture damage was evaluated by using a Hamburg Wheel Tracking Device (HWTD) in accordance with AASHTO T324-04 “Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA)” (AAHTO, 2011).

The device tracks a 705 N load hard-rubber wheel across the surface of gyratory compacted specimens submerged in a hot water bath at 50°C. During the test, the deformation of specimens under the wheelpath was recorded as a function of the number of passes by using linear variable differential transducers (LVDTs). The moisture susceptibility is then evaluated based on the total rut depth as well as the stripping inflection point, which is defined as the intersection of the slopes of stripping and rutting as shown in Figure 3-19.



(a) CPATT HWTD Device

(b) Inside View of HWTD

(c) Typical results from HWTD Test

Figure 3-19 CPATT Hamburg Wheel Tracking Device

3.4.6. Moisture Sensitivity Testing

Moisture susceptibility in pavement materials is a major factor that affects pavement life. Stripping which is the term used to describe the loss of adhesive bonding force between binder and aggregates is generally the result of moisture damage in asphalt. Moisture damage in asphalt mixtures occurs due to loss of adhesion (the bond between asphalt and aggregate) and/or cohesion (the bond between asphalt binder molecules), which subsequently results in progressive strength reduction and decrease in stiffness of the mixture. Several mechanisms have been cited as contributing factors to moisture damage including detachment, displacement, spontaneous, emulsification, film rupture, pore pressure, and hydraulic scouring (Solaimanian et al., 2007). However, not all of these mechanisms are well understood due to the complexity of the impact of individual or combined mechanisms on the moisture susceptibility of a given mixture, as stated by Solaimanian et al. (Solaimanian et. al., 2003). Furthermore, researchers have found that moisture damage can be accelerated by improper mix design or production.

Through the years, several testing procedures have been proposed in North America to evaluate the moisture susceptibility of asphalt mixtures. However, majority of these tests could not gain acceptance within the industry as routine tests to evaluate moisture susceptibility. This was mainly due to level of complexity of these tests, as well as relatively long period of time required to perform these tests. The Canadian CPATT-UW 2015 survey results indicated that almost 64 percent of the provincial transportation agencies prefer moisture sensitivity testing using the Tensile Strength Ratio (TSR) of the AASHTO T 283 and 18 percent of respondents preferred the Hamburg Wheel Tracking Test (AASHTO T 324). Nine percent of respondents indicated that they prefer Asphalt Pavement Analyzer (APA) (AASHTO TP 63) and the Immersion-Compression Test (AASHTO T 165). Of the remaining, 9 percent of the respondents had no requirement for moisture sensitivity testing.

A combination of qualitative and quantitative laboratory test methods shown in Figure 3-20 were used to assess moisture susceptibility of mixtures containing different types of binder, WMA additive and aggregate blends. The main objective of this assessment is to establish a reliable ranking system for moisture susceptibility of WMA mixtures. Also, the assessment will be used to provide recommendations to improve the current procedure and criteria for evaluating moisture susceptibility of WMA mixtures.

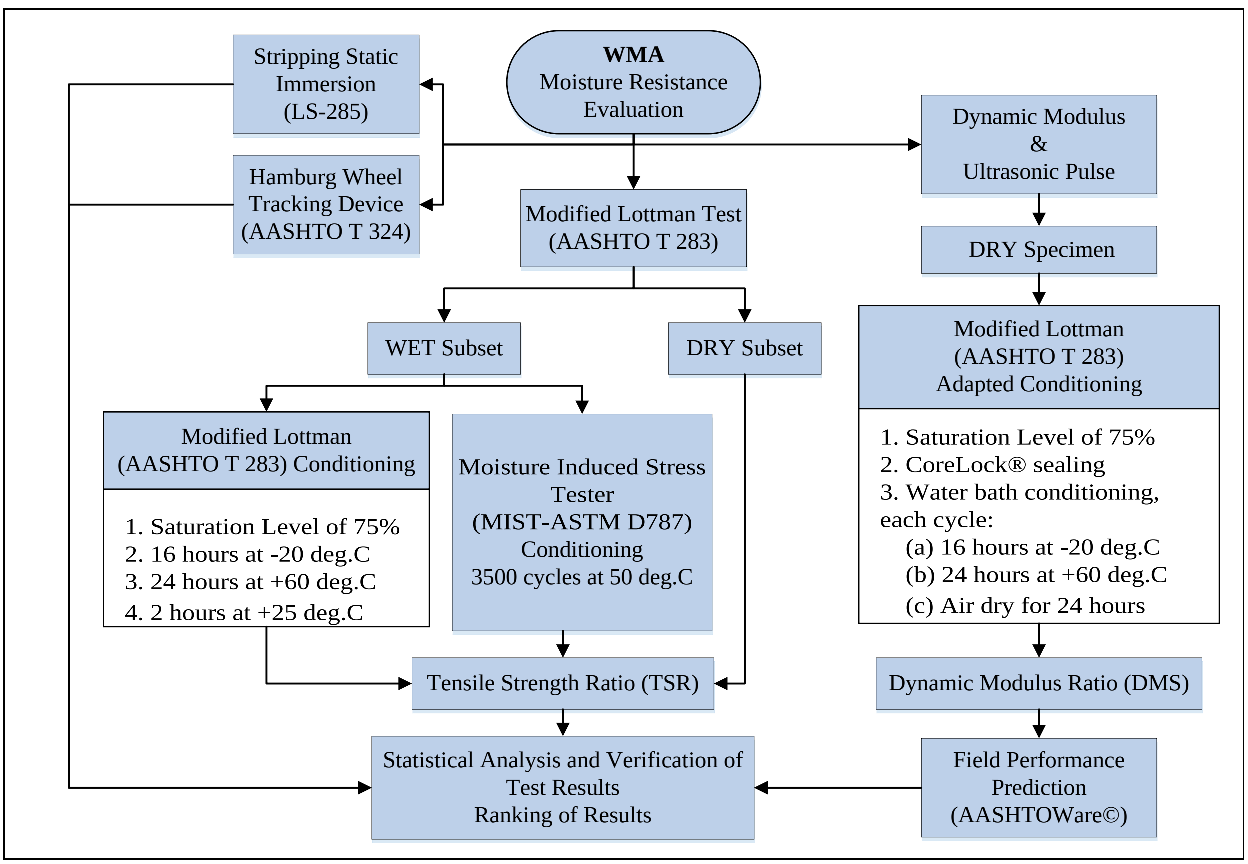


Figure 3-20 WMA Moisture Sensitivity Evaluation Flow Chart

3.4.6.1.Tensile Strength Ratio (TSR)

Moisture sensitivity of compacted mixtures was quantified as the percentage of tensile strength retained after conditioning which is referred to as the Tensile Strength Ratio (TSR). The tensile strength was determined by using the Indirect Tensile Strength (IDT) apparatus in accordance with ASTM D6931-12, "Standard Test Method for Indirect Tensile Strength of Bituminous Mixtures" (ASTM, 2012). Two moisture conditioning alternatives were considered for this study to quantify the resistance of mixtures to moisture damage: (1) AASHTO T283 conditioning, and (2) moisture conditioning performed by Moisture Induced Stress Tester (MIST) as per ASTM D 7870-13, "Standard Practice for Moisture Conditioning Compacted Asphalt Mixture Specimens by Using Hydrostatic Pore Pressure" (ASTM, 2013).

The strength testing was performed by applying an axial force at a rate of 50 mm/min until the maximum load was reached. The indirect tensile strength was then calculated by using Equation 3-10 as follow:

$$S_t = \frac{2000P}{\pi t D} \quad \textbf{3-9}$$

where

$S_t =$ IDT strength, kPa
$P =$ maximum load, N
$t =$ sample thickness immediately before test, mm
$D =$ sample diameter, mm, and
$\pi =$ 3.14

For both of T283 and MIST conditioning alternatives, a minimum of six specimens were compacted using the CPATT Superpave gyratory compactor to a target percentage of air voids (7 ± 0.5 percent), each measuring 150 mm in diameter and 100 ± 5 mm in height. The compacted specimens were then separated into two subsets: conditioned and unconditioned. For T283, a minimum of three specimens were vacuumed to saturation range of 75 ± 3 percent, and subjected to a freeze-cycle (16 hours at -20ºC) followed by a thaw-cycle in water bath (24 hours at 60 ºC).

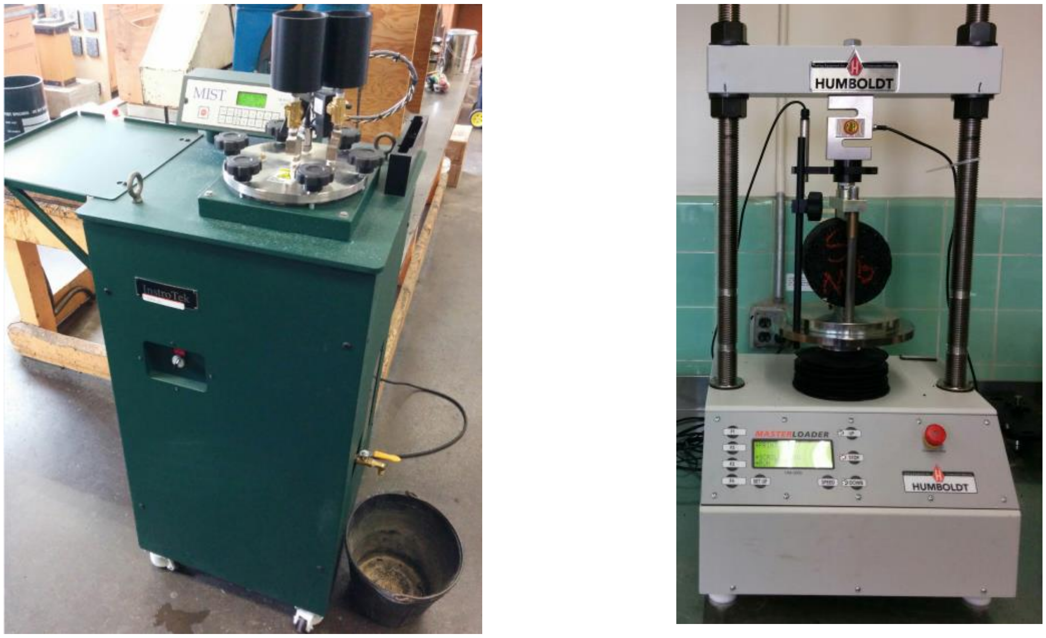
The desired level of saturated air voids was achieved by applying vacuum partial pressure of 13 to 67 kPa for 3 to 10 minutes to saturate the air voids within the specimen. The saturation level was calculated as ratio of volume of absorbed water (J') over volume of air voids (V_a):

$$\%\,Saturation\ Level = \frac{100J'}{V_a} = \frac{B' - A}{P_a(B - C)/100} \quad \textbf{3-10}$$

where

$B' =$ mass of the saturated surface-dry specimen in air after saturation, g,
$A =$ mass of dry specimen in air, g
$P_a =$ air voids in compacted mixture, %
$B =$ mass of the saturated surface-dry specimen in air before saturation, g, and
$C =$ mass of the specimen in water at 25ºC,g

MIST conditioning was performed by applying 3500 cycles of 276 kPa (40 psi) pore pressure at 50°C. Pore pressure cycling was applied immediately after specimens in the chamber reached conditioning temperature of 50°C. This temperature was maintained by the equipment. After cycling, specimens were cooled to 25 ± 1°C in a water container for 2 hours prior to strength testing.



(a) Moisture Induced Stress Tester **(b)** IDT testing equipment

Figure 3-21 MTO's MIST and Indirect Tensile Tester

3.4.6.2.Dynamic Modulus Ratio (DMR)

Similar to TSR, moisture sensitivity of compacted mixtures was quantified as percentage of dynamic modulus retained after freeze-thaw cycling. The dynamic modulus was determined by using a Material Testing System (MTS) setup shown in Figure 3-13 and the test method described in Section 3.4.2 of this thesis.

After tested unconditioned for dynamic modulus, each specimen was conditioned in accordance with a modified AASHTO T 283 (modified Lottman test) test method. For this conditioning approach: (1) specimens were vacuumed to saturation range of 75 ± 3 percent, (2) covered tightly with plastic film, (3) placed inside a polymer bag containing 10 mL of distilled water, (4) sealed by using a controlled vacuum system, and (5) subjected to one freeze-thaw cycle. Each cycle consisted of a freeze-cycle (16 hours at -20ºC) followed by a thaw-cycle in water bath (24 hours at 60 ºC). After freeze-thaw cycle, specimens were removed from the bag and left to dry for 24 hour at room temperature before being tested for dynamic modulus to determine if any changes in |E*|. Figure 3-22 illustrates steps of conditioning a dynamic modulus specimen.

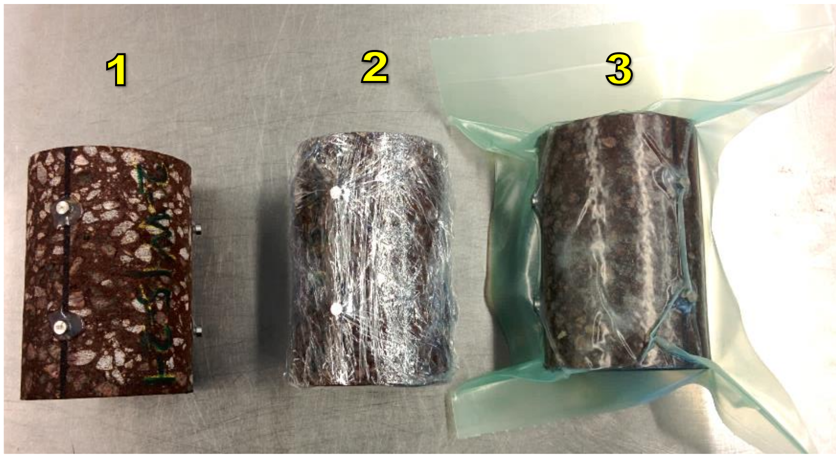


Figure 3-22 Steps of Dynamic Modulus Specimen Preparation for Freeze-Thaw Conditioning: (1) Unconditioned, (2) Vacuum-Saturated wrapped, and (3) Vacuum-Saturated Wrapped & Sealed

3.4.6.3.Static Immersion Stripping Test

To assess quality of chemical compatibility and bonding between binder and aggregate, static immersion test was performed at MTO's bituminous laboratory in accordance with LS-285, "Method of Test for Stripping by Static Immersion" (MTO, 2011).

According to this test, 100 grams of dry coarse-aggregate blend was prepared by mixing 50 grams of aggregate retained on 9.5-mm sieve size, 35 grams of retained on 6.7-mm sieve, and 15 grams of retained on 4.75-mm sieve size. The aggregate blend was placed in an oven at specified temperature prior to mixing with 4.0 ± 0.1 grams of heated asphalt binder. The loose mixture was then transferred to a 600-mL beaker to allow cooling to room temperature. After cooling, the beaker was filled with distilled water to the ¾ full mark to submerse the mixture, sealed, and placed into a water bath at 49 ± 0.5°C for 24 hours. The beaker was then removed and placed under a lamp for evaluation of the extent of retained asphalt coating on the aggregate as a percentage.

3.4.6.4.Ultrasonic Pulse (UP) Test

Ultrasonic Pulse (UP) method was employed to evaluate the impact of conditioning on development of micro cracks and defects and overall durability of dynamic modulus specimens after freeze-thaw cycles. Ultrasonic Pulse echo method is a non-destructive test commonly used for quality control and condition assessment of different materials and structures, especially concrete structures. The method consists of measuring the time of travel of an ultrasonic waveform passed through a dynamic modulus specimen. Higher time travel indicates good quality and continuity, while slower time travel may indicate presence of voids and micro-cracks within the specimen. Figure 3-23 shows a laboratory setup employed at University of Waterloo to perform UP test on dynamic modulus specimens before and after freeze-thaw cycling.

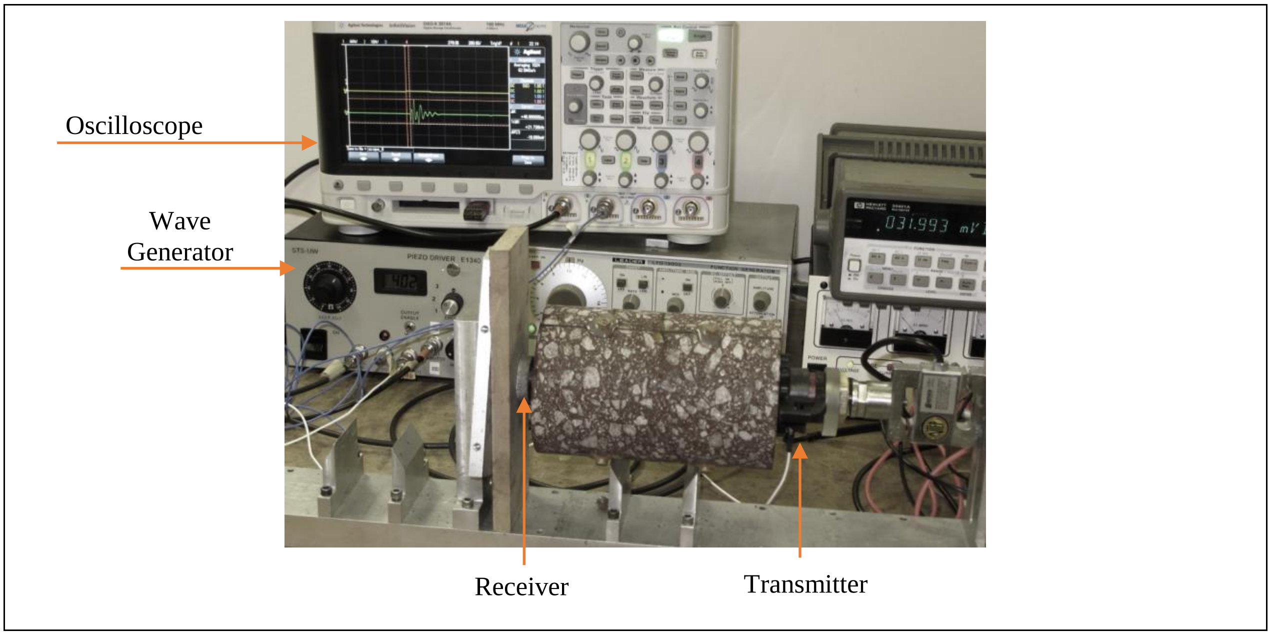


Figure 3-23 University of Waterloo Ultrasonic Pulse Test Setup

3.4.7. Multiple Freeze-Thaw Resistance

Similar to evaluating DMR for WMA, dynamic modulus specimen were used to capture any impact of use of colouring pigment on the level of resistance to multiple freeze-thaw damage and overall long-term durability of the mixture. The level of resistance was quantified as percentage of dynamic modulus retained after freeze-thaw cycling. The dynamic modulus was determined by using the MTS setup shown in Figure 3-13 and the test method described in Section 3.4.2 of this thesis. After tested unconditioned for dynamic modulus, each specimen was conditioned in accordance with conditioning approach explained in Section of 3.4.6.2 of this thesis.

After conditioning, vacuum-saturated and sealed specimen were placed in a walk-in freezer capable of applying multiple freeze-thaw cycles shown in Figure 3-24(c). The freezer was programmed to automatically apply multiple cycles of freeze-thaw; each cycle consisted of 16 hours of freezing temperature of -20ºC followed by 8 hours of thawing temperature of +25ºC. Specimens were subjected to one month of freeze-thaw cycles (31 cycles).

During the freeze-thaw, specimens were laid along their sides placed on spacers as shown in Figure 3-24 (b) to allow consistent circulation of air around the specimen. Such position was selected to allow uninterrupted expansion and contraction. After being subjected to multiple freeze-thaw cycles, specimens were then removed from the bag and left to dry for 24 hour at room temperature before being tested for dynamic modulus to determine if any changes in elastic modulus results obtained before freeze-thaw cycling. Similar to TSR value, the deterioration was determined as percentage of elastic modulus retained after freeze-thaw cycling.

(a) CoreLock® Vacuum

(b) Specimen placed on spacers to allow consistent air circulation

(a) CPATT Walk-in Freezer

Figure 3-24 CPATT Equipment Used for Freeze-Thaw Durability Test

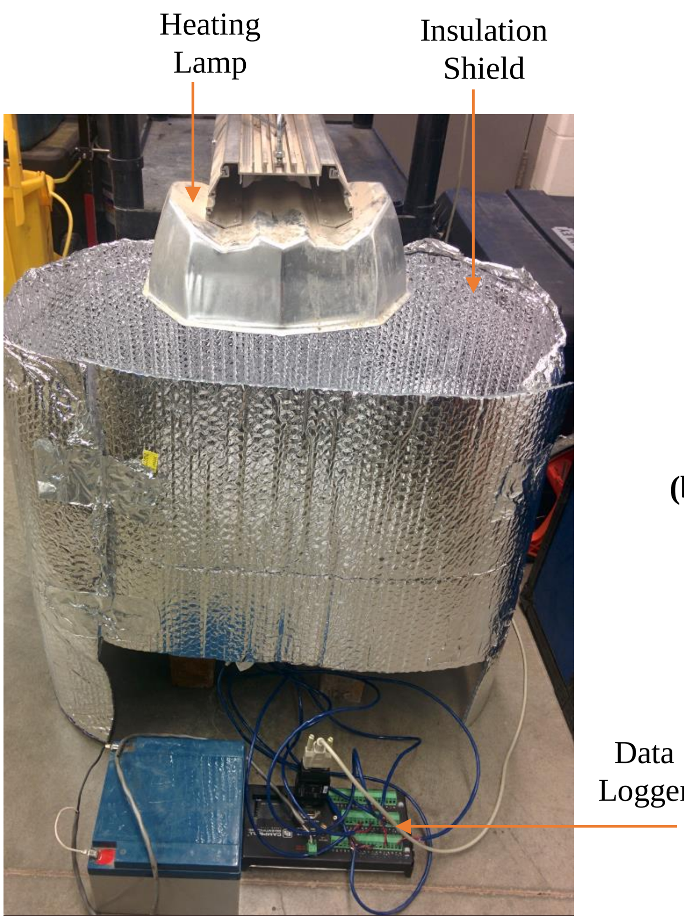
3.4.8. Heat Absorption Test

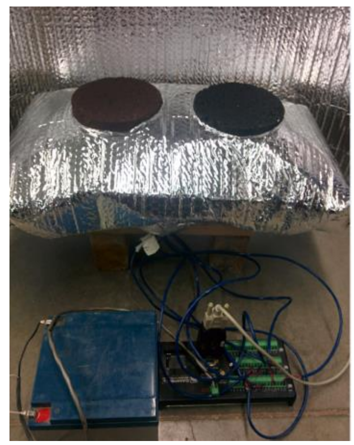
Literature review suggested that the colour of material exposed to sunlight affects its thermal properties due to solar radiation absorption. Darker colours relatively absorb more solar radiation, which causes temperature increase; while lighter colours absorb less radiation, leading to cooler temperatures. This theory have been evaluated by many researchers on pavements with lighter surface colours compare with so called “black topped” asphalt pavements. Lighter surface colour pavements are commonly referred to as “cool pavements”, which have been demonstrated by many researchers to provide major benefits for cities by reducing heat absorption and ultimately reducing the possible impact of black surfaced pavements on urban heat island generation. In addition to the environmental benefits, lighter colour pavements may provide economic benefits as reduction in heat absorption rate may lead to reduction in the in-service temperatures through different seasons.

This allows for usage of less expensive Performance Grade (PG) binders. Moreover, surface layers within the pavement structure may not experience as high temperature which causes damages such as rutting and bleeding. This might lead to more durable and longer lasting road, which might require less maintenance.

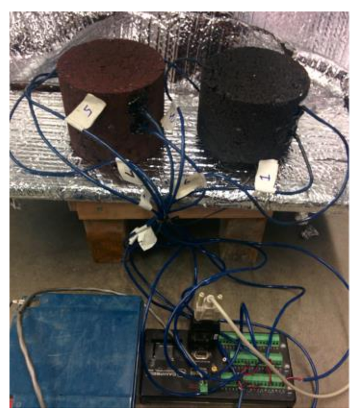
A laboratory setup shown in Figure 3-25 (a) was used to evaluate the effect of colour change due to pigmentation on the rate of heat absorption. This test was performed on two laboratory produced cylindrical specimens at the same time: (1) pigmented and (2) non-pigmented asphalt specimen. Specimens were prepared by using the CPATT Superpave Gyratory Compactor compacted to similar level of in-filed compaction level (4% Air or 96% G_{mm}).

Specimens were instrumented with thermocouples connected to an automated data collection unit (also referred to as "data logger") capable of collecting data every two minutes. Thermocouples were installed at various depths of specimen by drilling holes extended 75 mm (2 in) into the specimen. To ensure accurate temperature measurement and tight fit, the holes were then backfilled with the same PG grade asphalt binder used in producing the specimens. Instrumented specimens are shown in Figure 3-25 (c).





(b) Inside view of insulation shield



(a) UV heating CPATT laboratory setup

(a) Instrumented Specimens

Figure 3-25 UV Heat Absorption Laboratory Setup

3.4.9. British Pendulum Friction Testing

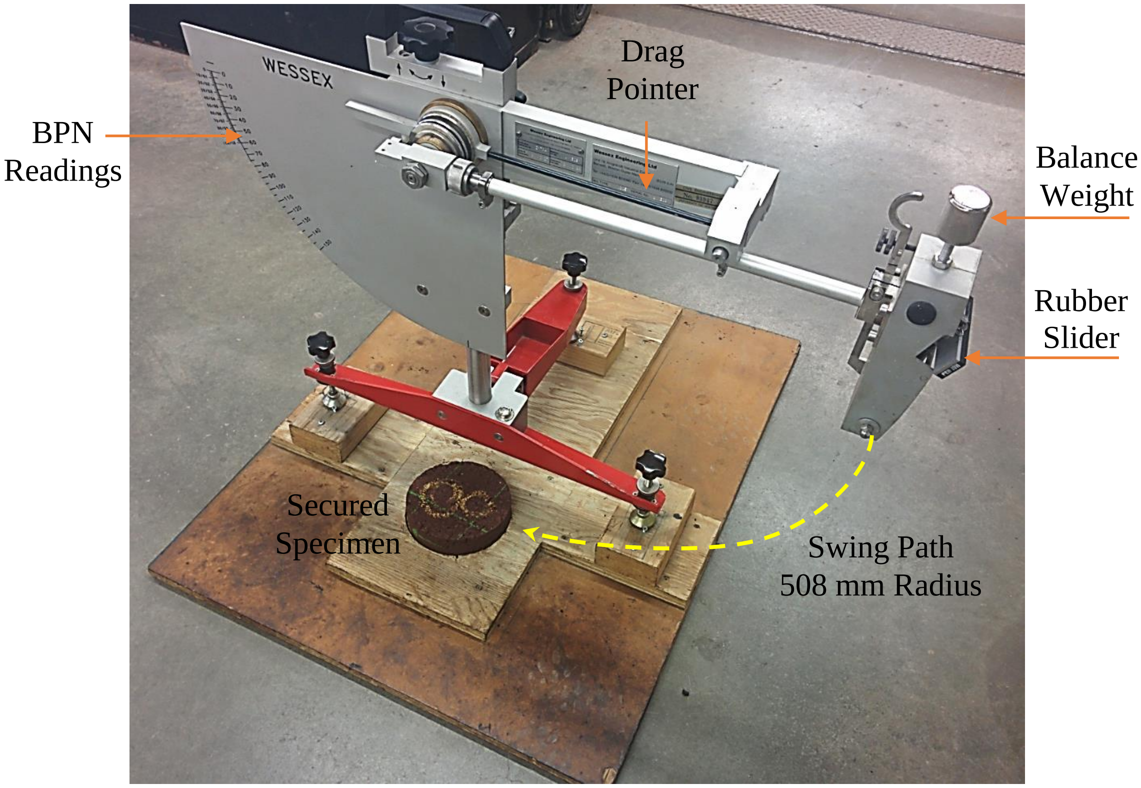
The effect of colouring pigment on the surface frictional properties was measured both in laboratory and field by using the British Pendulum Skid Resistance Tester in accordance with the ASTM E 303-93, "Standard Test Method for Measuring Surface Frictional Properties Using the British Pendulum Tester" (ASTM, 2013). In the laboratory, a test setup shown in Figure 3-26(a) was used at CPATT to capture the effect of pigment on the friction response due to micro-texture modification.

The testing procedure involved using a dynamic pendulum impact-type tester (also called "British Pendulum") to swing a rubber slider over a contact path marked on the surface of a specimen, as shown in Figure 3-26(b). Then, the surface friction was measured as the amount of energy loss during the contact between the slider and the test surface. A drag pointer on the British Pendulum as shown in Figure 3-26(a) was used to indicate the energy loss in terms of a British Pendulum Number (BPN). The greater the friction between the slider and the test surface, the more the swing is retarded, and the larger the BPN reading.

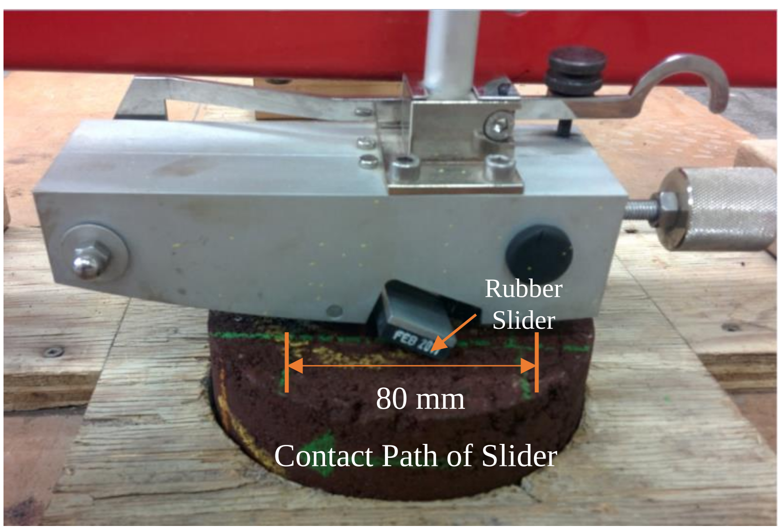
Specimens were prepared by using the CPATT SGC, each measuring 150 mm in diameter. Specimens were compacted to a targeted air void content of 4 ± 0.5 percent to better represent the field condition. Five swings of the pendulum were made for each test dry surface conditioned at five different temperatures (0, 4, 21, 37 and 54 degree Celsius) to obtain BPNs. Conditioning was performed by using the environmental chamber located at CPATT as shown in Figure 3-13(a). Furthermore, the testing was performed on wet surface for each of the testing temperatures. To wet the test surface, approximately 45 mL of distilled water was sprayed across the specimen in the beginning of each set of data collection and 5 mL of water was sprayed on the specimen surface to replace the lost water between swings.

3.5. Field Pavement Distress Survey and Testing

Numbers of site visits were conducted in different dates to perform manual pavement condition survey of different sections of Highway 7 BRT lanes, as shown in Figure 3-27. This was to document the performance of these sections and also have a better understanding of automatic distress data made available to CPATT research team. Automated distress data were collected on only 2-year old sections by a consultant hired by York Region. Numbers of site visits were also conducted to perform field testing which are explained more in details in the following sections.



(b) Wessex British Pendulum and CPATT test setup



(c) Close-up view of contact path

Figure 3-26 CPATT British Pendulum Tester

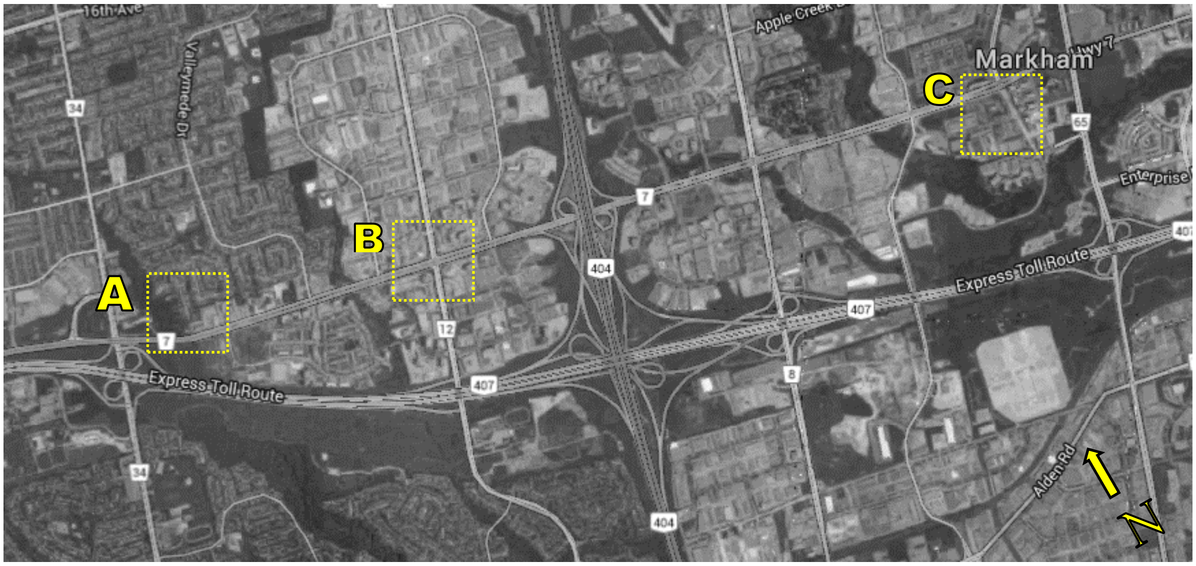
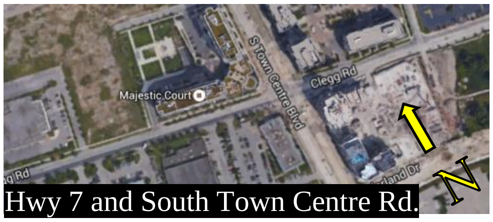


Figure 3-27 Distress Survey and Field Testing Key Map

Table 3-7 Details of Distress Survey and Field testing Locations

Site ID	**As-Built Material**	**Purpose**	**Location**
A	-Epoxy paint -Red Asphalt (Initial Mix)	Comparison of Painted and red asphalt paved sections in terms of (*1*) friction, (*2*) conducting manual distress survey	
B	Red Asphalt (Initial Mix)	(*1*) surficial friction testing, (*2*) conducting manual distress survey	
C	Red Asphalt (New Mix and No red intersection)	(*1*) surficial friction testing, (*2*) conducting manual distress survey	

3.5.1. Safety Evaluation

Two site visit were conducted on different dates to measure the surficial friction properties of sections of BRT lanes shown in Figure 3-27, as well as a section of Highway 7 conventional black surface. The surface frictional properties was measured by using equipment shown in Figure 3-28: (1) the CPATT British Pendulum Skid Resistance Tester in accordance with the ASTM E 303-93, “Standard Test Method for Measuring Surface Frictional Properties Using the British Pendulum Tester” (ASTM, 2013), and (2) ASFT© T2-GO surficial friction tester (ASFT, 2016). It should be mentioned that measurements for both equipment were taken both in wheelpath and centre lines of lanes.

(a) Using CPATT British Pendulum Friction Tester

(b) Using Portable/continuous ASFT T2-GO Friction Tester

Figure 3-28 Field Friction Testing Equipment

3.5.2. Long-term Performance Prediction Using Mechanistic-Empirical Pavement Design Guide (MEPDG)

The AASHTOWare Mechanistic-Empirical (M-E) Software was used to investigate the long-term effect of warm mix additives on the long-term performance of a pavement structure designed for Ontario climate and traffic conditions with WMA used as surface course. For this purpose, a level 1 design was performed for the surface course by using dynamic modulus results and asphalt binder properties. All other inputs were retrieved from the Ministry of Transportation Ontario recommended inputs (MTO, 2012) and were maintained constant in all the designs for other layers.

The AASHTOWare was also employed to investigate the effect of using coloured hot mix asphalt on the long-term performance of an as-built BRT pavement structure located on Highway 7. For this purpose, a level 1 design was performed for the surface and binder course by using dynamic modulus results and asphalt binder properties. All other inputs were retrieved from the

Ministry of Transportation Ontario recommended inputs (MTO, 2012) and were maintained constant in all the designs for other layers. Furthermore, the MEPDG distress outputs were group into an overall composite index which was integrated into pavement management system.

3.6. Life-Cycle Cost Assessment

The intent of main objective of the research was to characterize the structural, functional, and environmental characteristics of the coloured asphalt design by analyzing laboratory and field performance. The laboratory and field results collected during the research was used to create performance predictions models for pavement designs incorporating the red asphalt material and this formed the basis of the life cycle assessment. These models are expected to provide better understanding of the in-situ materials' performance and long-term behaviour that can be used as inputs into the overall pavement management plan shown in Figure 3-29 to improve the structural performance of the asphalt and maintain the colouring and functional performance of the surface.

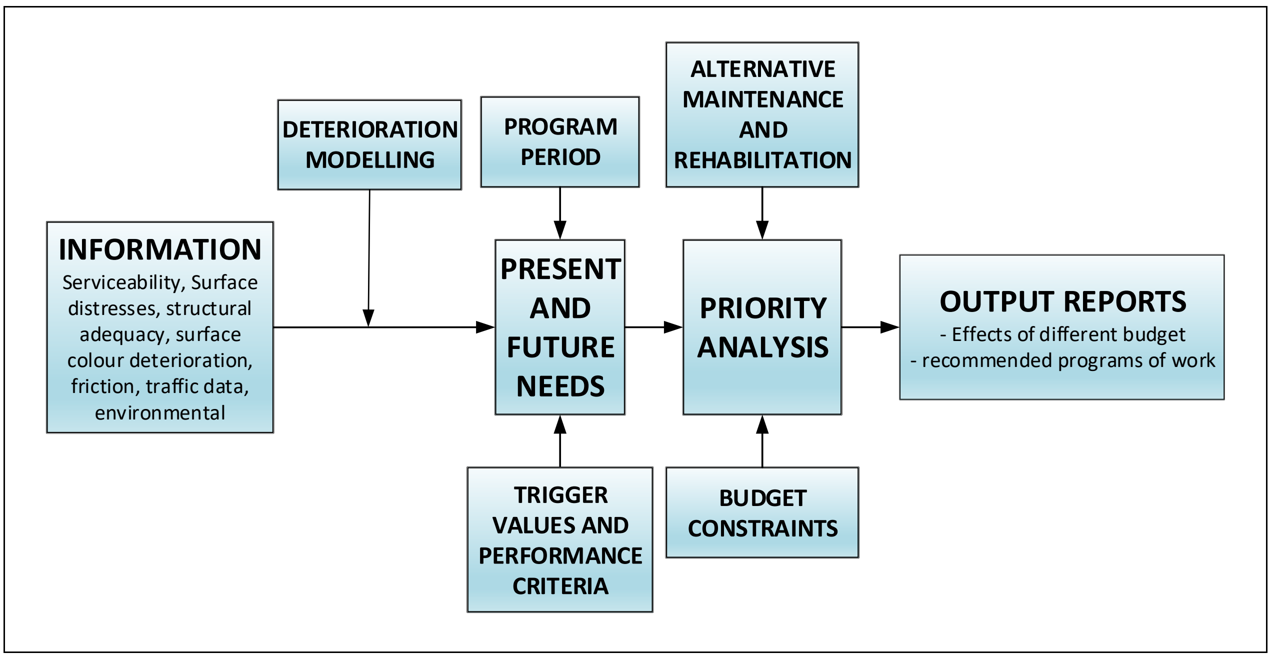


Figure 3-29 Conceptual Framework of Pavement Management (TAC, 2014)

3.7. Summary

This chapter provided details and steps involved in the methodology employed in this research. This methodology was developed to fulfill the two main objectives of: (1) evaluating the performance of Coloured Hot Mix Asphalt (CHMA) pavements for usage in BRT lanes, and (2) evaluate the effect of different warm mix additives on the strength of compacted asphalt mixtures for usage in pavements in Southern Ontario. The evaluation included performing an array of test methods with purposes such as those briefly summarized in Table 3-8.

Table 3-8 Summary of Test Methods Included in The Research Methodology

Material Type	Test Method	Purpose
WMA	Superpave PG Rheological Binder Tests	Capturing the effect of warm mix additives on rheological properties of asphalt cement (AC) at specified in-service temperatures related to controlling the level of resistance to different types of pavement distresses: rutting, thermal cracking, aging, and fatigue cracking
	Moisture Sensitivity Tests	Array of quantitative and qualitative tests to evaluate the moisture susceptibility of WMA mixtures. Number of tests with different capabilities were employed to provide a comprehensive evaluation.
WMA/CHMA	Hamburg Wheel Tracking Test	Quantifying the level of resistance to rutting and moisture damage at high in-service temperature by using a wheel tracking device.
WMA/CHMA	Dynamic Modulus	Evaluating cracking distress development and propagation related to CHMA and WMA mixtures at different in-service temperatures and different frequencies simulating traffic loadings at varying travelling speeds.
CHMA	Flexural Beam Fatigue Test	Evaluating if colouring pigment has an effect on the level of resistance to fatigue cracking at intermediate in-service temperature.
	Thermal Stress Restrained Specimen Test	Evaluating if colouring pigment has an effect on the level of resistance to thermal cracking at low in-service temperature.
	Friction Tests	Testing the CHMA in laboratory and field to evaluate the surficial friction of CHMA in comparison to a typical friction surface course asphalt mixture commonly designed for Ontario roads; Superpave 12.5mm Friction Course (FC) type II.
	CPATT Developed Heat Absorption Test	A test setup developed for this research to capture the effect of colour change on the level of heat may absorbed by the mixture.
WMA/CHMA	Long-Term Performance Prediction	Investigating the long-term effect of WMA and Colouring Pigment on the long-term performance of as-built pavement structures designed for Ontario climate and traffic conditions. The performance prediction was performed by integrating results obtained in this research into the most advance computer-based modeling software available in the paving industry: The AASHTOWare's Mechanistic-Empirical (M-E) Software.

Chapter 4 Evaluation of Warm Mix Asphalt

4.1. Introduction

One of the main objectives of this research was to investigate the effect of different warm mix additives on the strength of asphalt mixtures. Results of this investigation are provided in this chapter accompanied with the respective statistical analysis to verify the significance of the results. Furthermore, information collected from each test were ranked in ascending order for each combination of aggregate and binder type: first for the best performance and last for the weakest performance.

4.2. Effect of WMA on Mixture Performance

Asphalt cement prototypes treated with different types of warm mix additive listed in Table 3-1 were produced following a consistent approach using a single-source PG 58-28, and PG 58-34P base asphalt cements. The Superpave PG binder specification of AASHTO M320 (AASHTO, 2010) was used to characterize each modified binder through an array of rheological tests described in Figure 3-8. The AASHTO M320 was employed to ensure: (1) recommendations provided by the additive suppliers did not cause adverse effect on the targeted high and low PG grades and (2) all binder are exhibiting similar high and low temperature performance grades. It can be seen from Figure 4-1 that the PG grades were not adversely affected by warm mix additives.

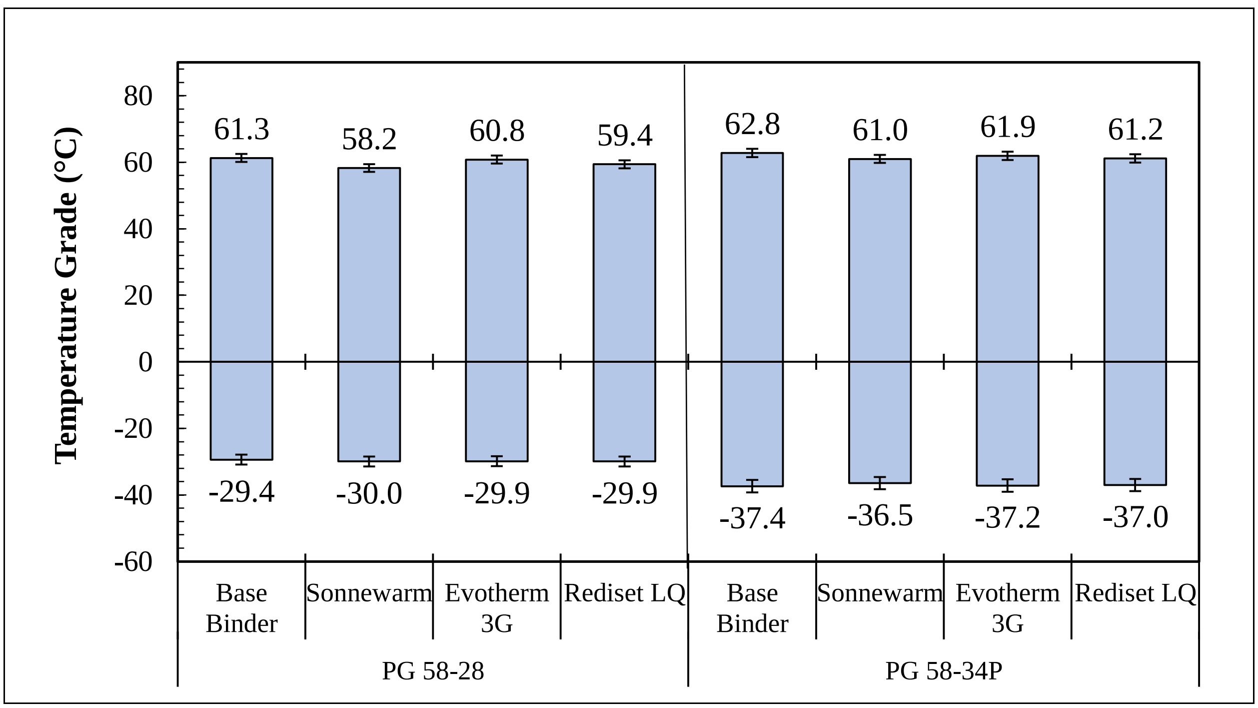


Figure 4-1 Continuous Performance Grade (PG) of Asphalt Binders Treated with Warm Mix Additives

It should be noted that suggested mixture mixing and compaction temperatures were provided by the additive supplier or manufacturer. However, ASTM viscosity-temperature plot was used to ensure that recommended temperatures are appropriate. For this purpose, a Rotational Viscometer (RV) was used to determine temperatures related to mixing and compaction viscosity values of 0.17 ± 0.02, and 0.28 ± 0.03 Pa.s respectively in accordance with AASHTO T 316-11, "Viscosity Determination of Asphalt Binder Using Rotational Viscometer" (AASHTO, 2011).

The linearity of the resulting viscosity-temperature curves were verified for all binders, ensuring binders behave Newtonian. The viscosity values at 135ºC were also verified to desired flow characteristics of binders at pumping and handling stages. For this study, all modified binders exhibited viscosities that were well below the AASHTO M320 criterion for the proper pumping and handling. It should be also noted that all modified binders exhibited a linear viscosity-temperature curve. Table 4-1 shows determined mixing and compaction temperatures.

Table 4-1 WMA and HMA Mixing and Compaction Temperature Range

Binder	Mixing Temperature (°C)	Compaction Temperature (°C)
WMA PG 58-28	125 – 130	115 – 120
WMA PG 58-34P	135 – 140	120 - 125
Conventional PG 58-28	145 – 150	132 - 135
Conventional PG 58-34 P	150 – 155	135 - 140

4.2.1. High Temperature Performance

To capture the effect of warm mix additives on high temperature properties of the base asphalt binder, the rutting parameter of $|G^*|/\sin(\delta)$ on both unaged and RTFO-aged binders was measured in accordance with AASHTO T 315-12, "Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR) (AASHTO, 2012).

Rutting (also known as "permanent deformation") of visco-elastic materials such as asphalt binder is considered as a stress-controlled cyclic loading phenomenon, which during each cycle of reversible loading, a certain amount of work is being done to deform the material. A portion of this work is recoverable due to the material's elasticity, and of the remaining is dissipated energy in forms of permanent deformation and heat. The amount of dissipated energy per loading cycle can be determined by using Equation 4-1.

$$W_c = \pi . \sigma_0^2 . \left(\frac{1}{|G^*|/sin\delta}\right) \quad \textbf{4-1}$$

where

$W_c =$ work dissipated per loading cycle
$\sigma_0 =$ stress applied during the loading cycle
$|G^*| =$ complex shear modulus
$\delta =$ phase angle

The parameter |G*|/sin(δ) in the above Equation is selected to relate asphalt binder's physical properties to rutting resistance. This parameter is a ratio of the total resistance to rutting (|G*|) and relative non-elasticity of the binder (sin(δ)). Higher the |G*|/sin(δ) parameter causes the binder to behave stiffer and more elastic, and thus more resistant to permanent deformation at higher pavement temperature. The AASHTO M320 specifies this parameter to be measured at maximum pavement temperature at a frequency of 10 radians per second (1.59 Hz), simulating the stress caused by a vehicle travelling at speed of 70 kilometres per hour.

To measure |G*|/sin(δ) parameter, a certain amount of sample is sandwiched between a fixed plate and an oscillating plate as shown in Figure 4-2(b). The oscillating plate traversed by applying torque to complete one cycle of oscillation, in which the resulting strain is recorded and then used to determine the G* by using Equation 4-2.

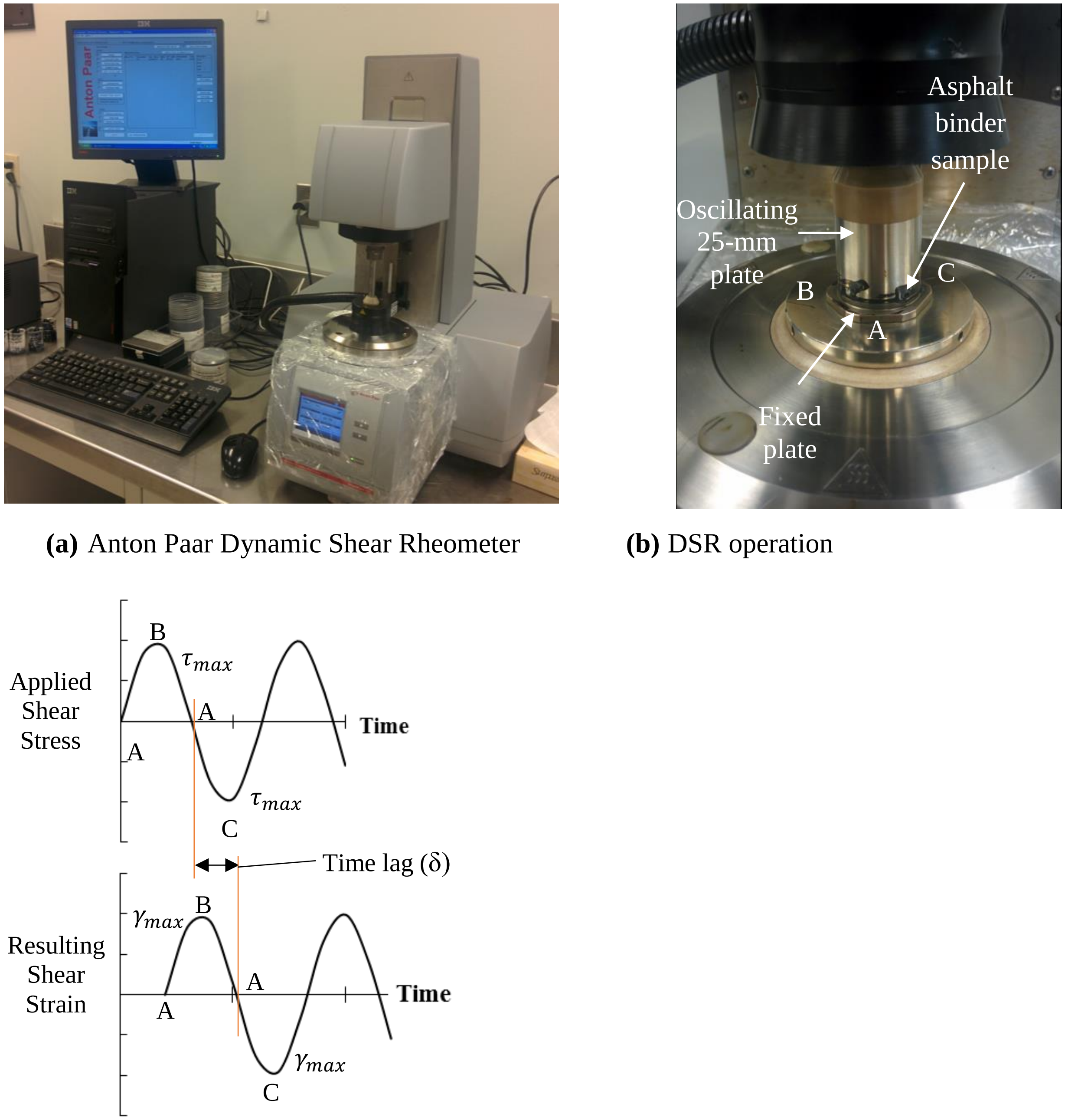
$$|G^*| = \frac{\tau_{max} - \tau_{min}}{\gamma_{max} - \gamma_{min}} \quad \textbf{4-2}$$

where

$|G^*| =$ complex shear modulus
$\tau =$ shear stress
$\gamma =$ shear strain

Figure 4-3 illustrates |G*|, δ, and |G*|/sin(δ) values measured for each unaged binder at maximum temperature of 58°C, but are normalized with respect to those values obtained for the control binders by using Equation 4-3. In all figures, error bars represent two standard deviation from the average value of two replicates tested, with |G*|/sin(δ) normalized values shown above the bars.

$$Normalized\ Parameter = \frac{Modified\ Binder\ Parameter}{Parameter\ Obtained\ for\ the\ Control} \quad \textbf{4-3}$$



(a) Anton Paar Dynamic Shear Rheometer

(b) DSR operation

(c) DSR resulting graphs

Figure 4-2 Dynamic Shear Rheometer Equipment and Operation

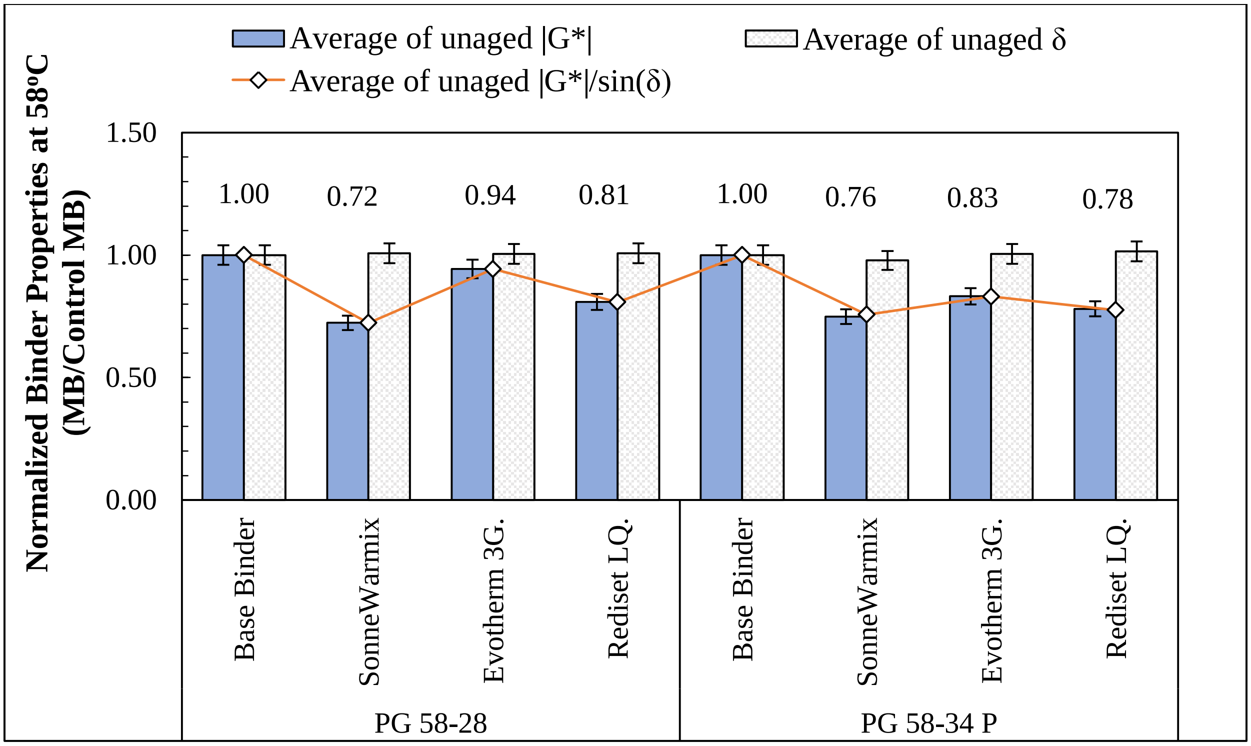
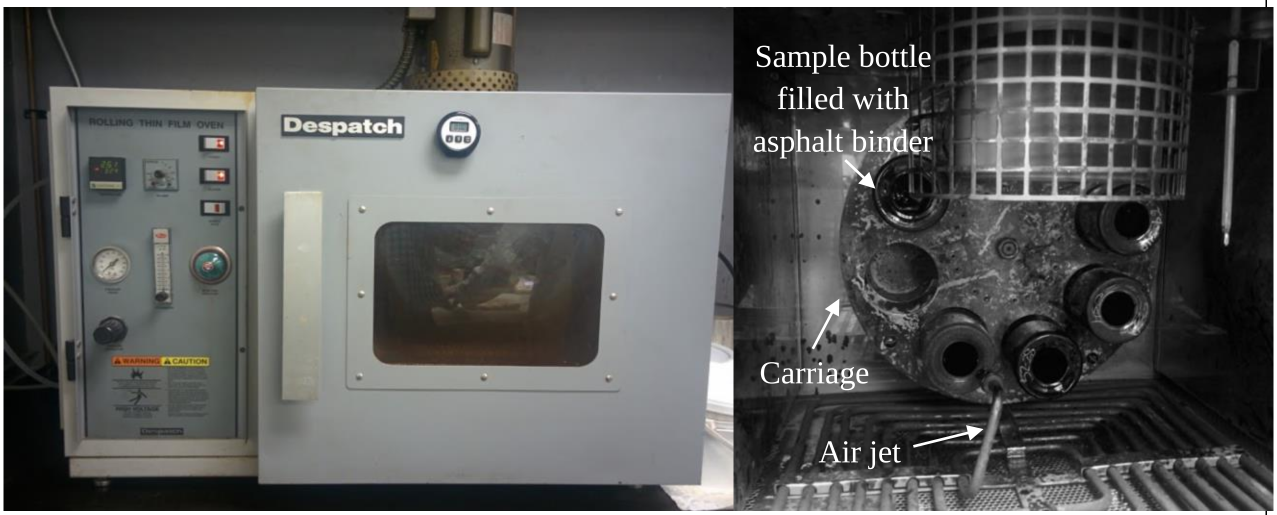


Figure 4-3 Comparison of Asphalt Binder's Rutting Parameter for Unaged G*/sin(δ) Parameter

As observed in Figure 4-3, the addition of the warm mix additives resulted in decreasing the |G*|/sin(δ) parameter; suggesting lowered level of resistance to rutting. It was also observed that binders modified with Sonnewarmix additive provided the least value of |G*|/sin(δ) compared to other modified binders. Although not illustrated in this figure, all binders met the lower stiffness limit of 1.0 kPa at 58ºC, as per the AASHTO M320 criterion for the |G*|/sin(δ) value for unaged binder.

To determine the RTFO |G*|/sin(δ) parameter, all binders were RTFO aged in accordance with AASHTO T240-09, "Effect of Heat and Air on a Moving Film of Asphalt Binder (Rolling Thin-Film Oven Test) (AASHTO, 2009). RTFO aging procedure simulates the short-term aging of the binder due to heat and air exposure during mixing at the HMA plant, transportation, and paving.

For RTFO aging, for each asphalt binder, five cylindrical glass bottles were filled with asphalt binder, each measuring with 35 ± 0.5 grams. After 60 to 180 minutes of cooling, all bottles were placed horizontally in a vertically rotating frame (also called "carriage") as shown in Figure 4-4, rotating at speed of 15 ± 0.2 revolutions per minute. The rotation causes the sample to flow along the wall of glass bottle. During each rotation, taking few seconds, air was blown once into each glass bottle. This action continued for 85 minutes in an oven with a constant operating temperature of 163ºC. Bottles were then removed, and the aged asphalt sample was poured into thin cans for further rheological testing and long-term aging in Pressure Aging Vessel (PAV).



(a) Despatch Rolling Thin Film Oven (b) Inside view of RTFO

Figure 4-4 Rolling Thin Film Oven Short-Term Aging

Figure 4-5 compares the |G*|/sin(δ) parameter after RTFO aging, but normalized with respect to those values obtained for the control binders by using Equation 4-3. In Figure 4-5, similar to trends observed in Figure 4-3, that addition of warm mix additives resulted in decreasing the |G*|/sin(δ) parameter. All RTFO-aged binders met the lower stiffness limit of 2.2 kPa at 58^{o}C, as per the AASHTO M320 criterion for the |G*|/sin(δ) value for RTFO-aged binder.

It is important to quantify the susceptibility of the binders to aging. This provides insight into the level of increase in stiffness after plant production, transportation, and paving. For this reason, an index was used to quantify aging after production only by using Equation 4-4. Figure 4-6 illustrates aging index values measured for each binder at maximum temperature of 58^{o}C.

$$Aging\ Index = \frac{Aged_{RTFO}\ |\mathrm{G}*|/\sin(\delta)}{Unaged\ |\mathrm{G}*|/\sin(\delta)} \qquad \textbf{4-4}$$

where

$|G^*| =$ complex shear modulus

$\delta =$ phase angle

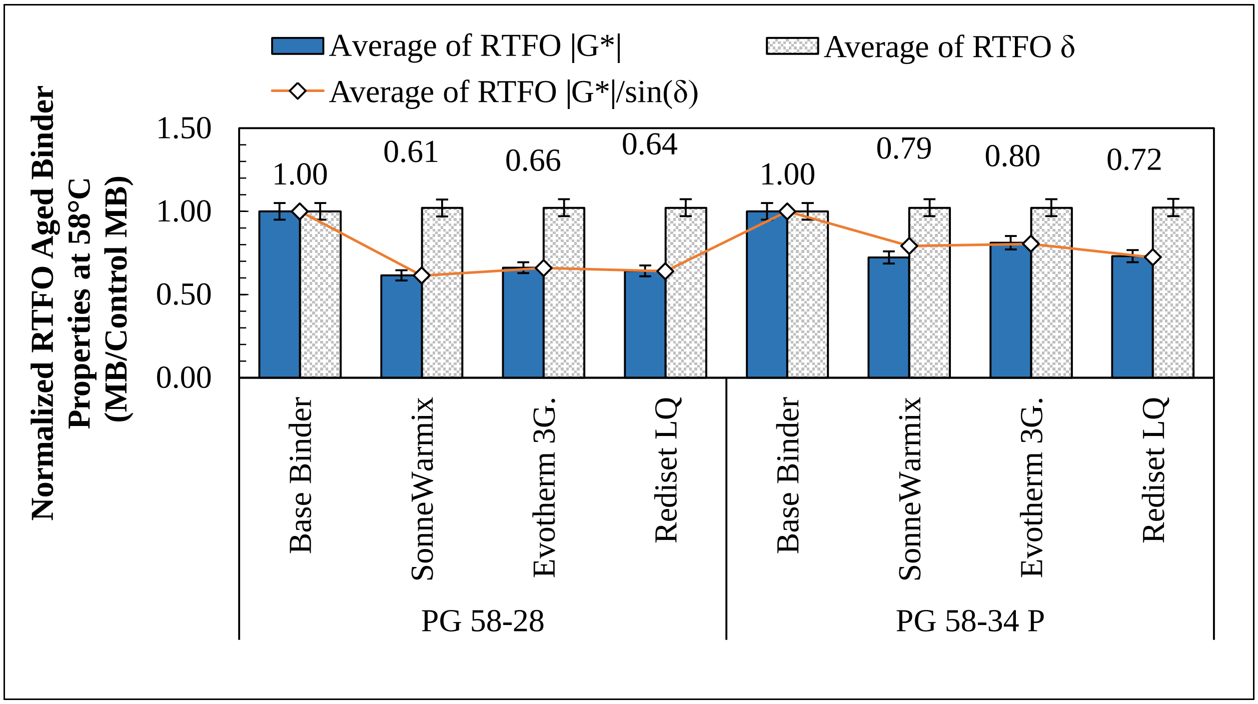


Figure 4-5 **Comparison of Asphalt Binder's Rutting Parameter for RTFO-aged G*/sin(δ) Parameter**

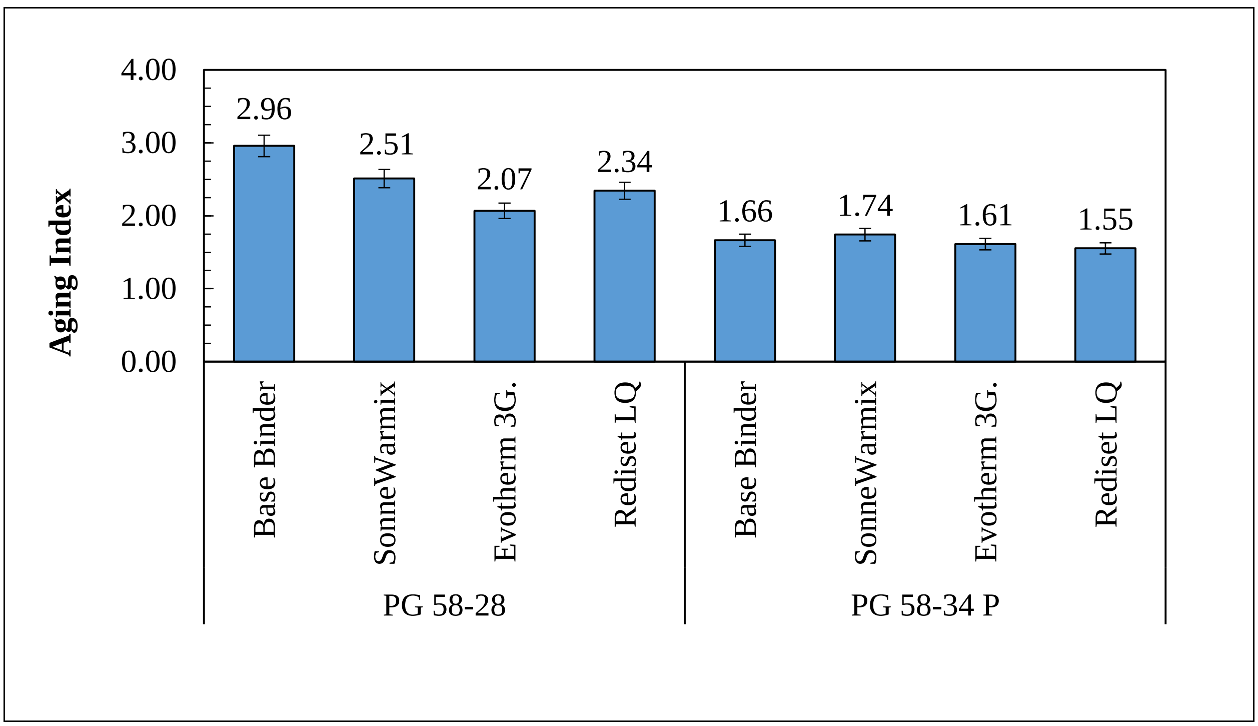


Figure 4-6 **Ratio of RTFO-aged to Unaged G*/sin(δ) Parameter (Aging Index)**

As shown in Figure 4-6, the addition of the warm mix additives to the base asphalt cement PG 58-28 resulted in decreasing short-term aging susceptibility. This suggests that pavement layer consisted of WMA may have reduced stiffness after production, which may translate into slower rate of deterioration as compared with a conventional HMA. The addition of the warm mix additives to PG 58-34 P did not improve aging susceptibility. But, polymer modified binders exhibited better aging resistance compared to PG 58-28 binders.

In order to verify the aforementioned observation and trends in the |G*|/sin(δ) rutting parameter, a Hamburg Wheel Tracking test (HWT) was used to measure rutting susceptibility of asphalt mixtures produced from the binders modified with warm mix additives. It should be noted that this equipment was also used to evaluate moisture susceptibility of warm mix asphalt mixtures. The rutting resistance of compacted asphalt mixtures was evaluated by tracking a 705 N (158 lb) load hard-rubber wheel across the surface of gyratory compacted specimens submerged in a hot water bath at 50°C. Test results of Hamburg rutting test for the various WMA mixtures are presented graphically in Figure 4-7 to Figure 4-10.

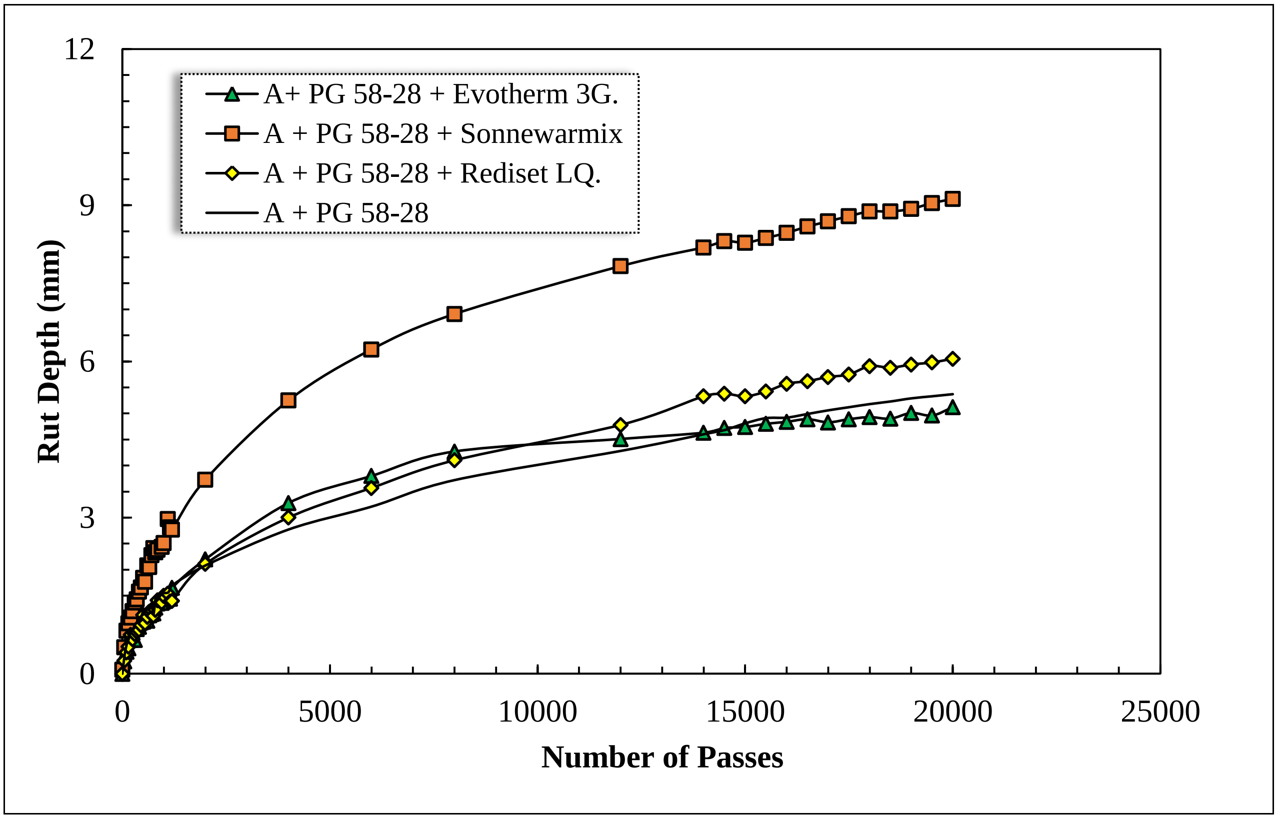


Figure 4-7 Hamburg Rutting Results For Mixtures Containing Type A Aggregate (Trap Rock Diabase) With PG 58-28

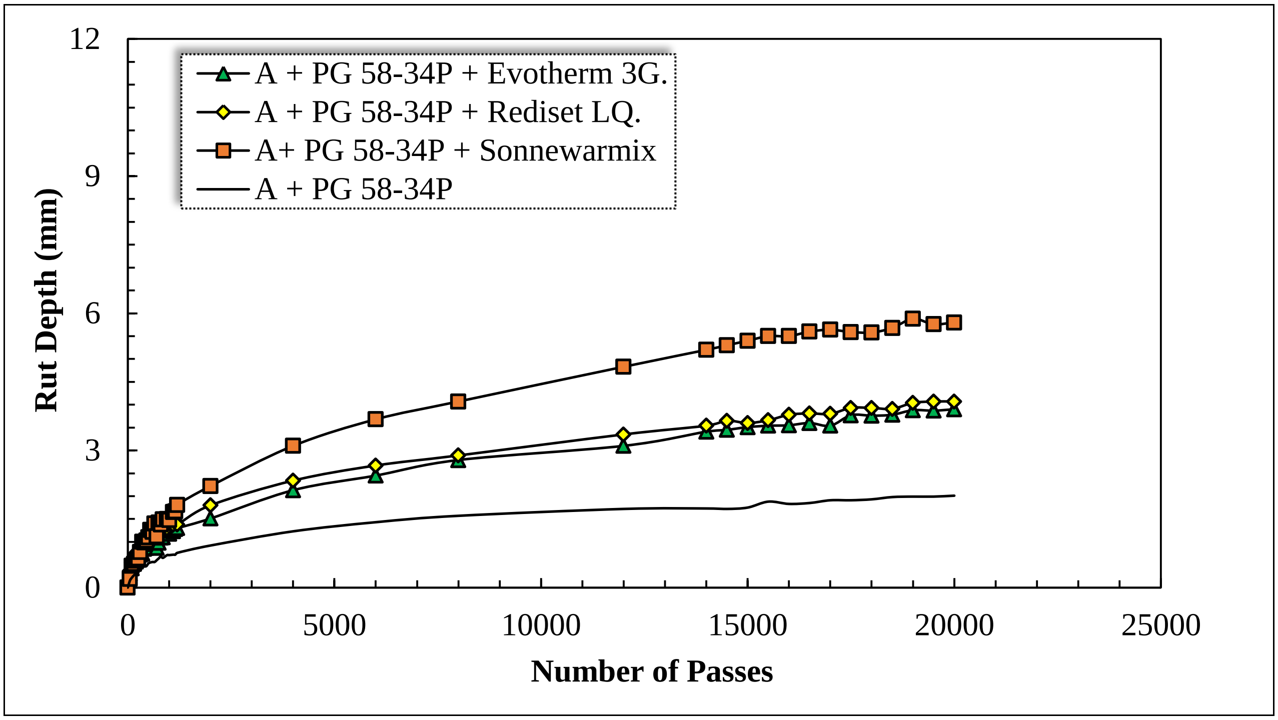


Figure 4-8 Hamburg Rutting Results For Mixtures Containing Type A Aggregate (Trap Rock Diabase) With PG 58-34P

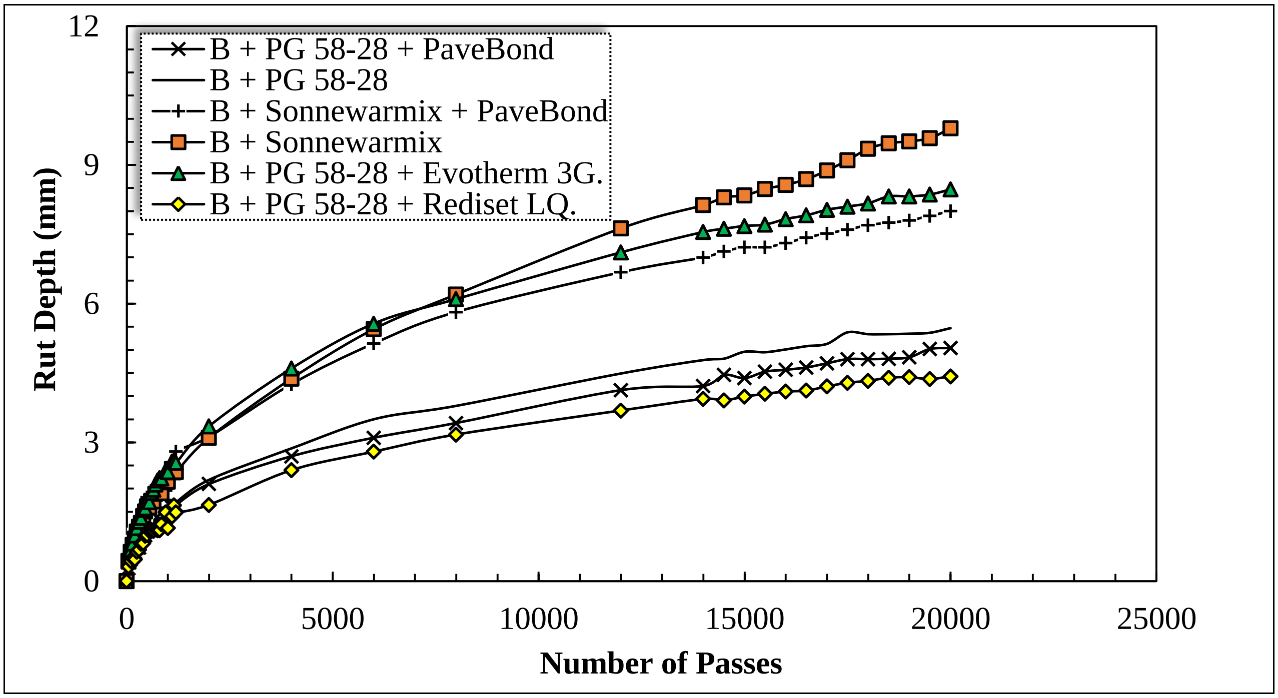


Figure 4-9 Hamburg Rutting Results For Mixtures Containing Type B Aggregate (Pink Granite) With PG 58-28

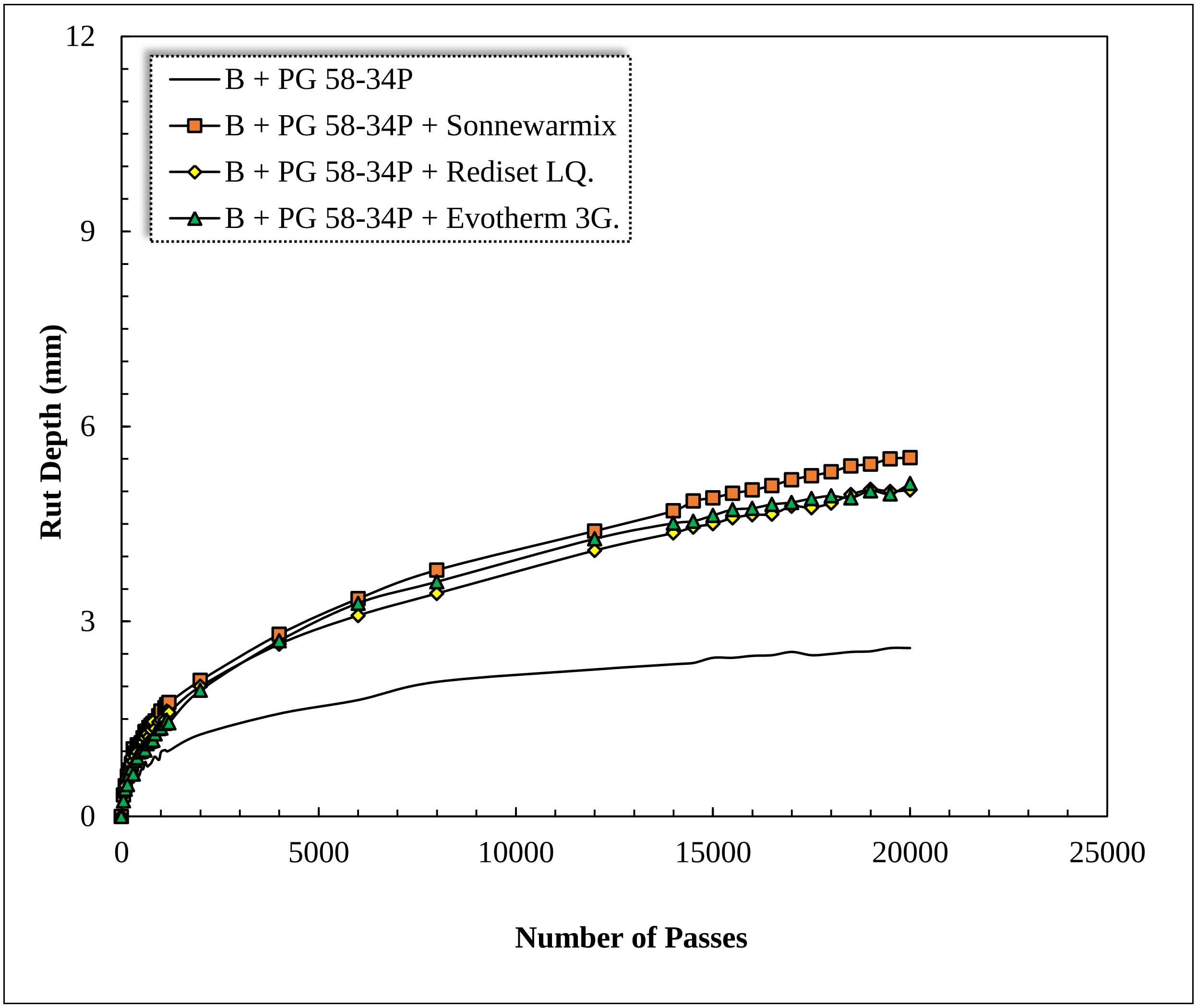


Figure 4-10 Hamburg Rutting Results For Mixtures Containing Type B Aggregate (Pink Granite) With PG 58-34P

The HWT results suggest that addition of warm mix additives in general resulted in an decreased level of resistance to rutting. However, the analysis of variance (ANOVA) presented in Table 4-2 (α = 0.05) indicates that binder PG grade and aggregate type were also significant sources of variation between the rutting behaviour of the mixtures. Furthermore, the interaction between these sources were found significantly important as presented in Table 4-2, except for the interaction between the binder PG grade and aggregate Type. It should be mentioned that all mixtures exhibited an acceptable level of resistance to rutting as compared to the upper limit of 12 mm, as per MTO's criterion for surface rut depth. This is consistent with what would be expected for a premium surface course mix designed in Ontario.

Table 4-2 Analysis of Variance (ANOVA) for Hamburg Rutting of All Mixtures

Source	DF	Adj SS	Adj MS	F Value	P Value	Statistically Significant
Binder (A)	1	47.5	47.5	1582	0.000	**YES**
Warm Mix Additive (B)	3	57.5	19.2	638	0.000	**YES**
Aggregate (C)	1	2.92	2.92	97	0.000	**YES**
A & B	3	10.7	3.57	118	0.000	**YES**
A& C	1	0.01	0.00	0.12	0.733	NO
B & C	3	5.68	2.33	77.7	0.000	**YES**
A & B & C	3	0.48	1.89	63.1	0.000	**YES**
Error	16	0.48	0.03	-	-	-
Total	31	132	-	-	-	-

Note: DF = Degree of Freedom, Adj. SS = Adjusted Sum of Squares, and Adj. MS = Adjusted Mean of Squares

The resistance of all mixtures to rutting were visually compared and the following trends were observed.

1. Addition of warm mix additives in general resulted in an decreased level of resistance to rutting, except for when Evotherm 3G was used with Type A aggregate (trap rock) and PG 58-28. Also, addition of Rediset LQ. to the combination of Type B aggregate (pink aggregate) and PG 58-28 improved the rutting resistance. The decreased level of resistance to rutting was expected for WMA mixtures, as rheological testing presented earlier on both unaged and short-term aged indicated reduction in binder's rutting parameter of $|G^*|/\sin(\delta)$. Furthermore, improvement of rutting susceptibility by using Rediset LQ. suggests that this additive may contain anti-stripping agent that can improve moisture susceptibility.
2. It was observed that mixtures containing Sonnewarmix provided the least level of resistance to rutting. This was hypothesized to be related to the melting point of this wax type additive which causes asphalt mixture to behave relatively softer at the testing temperature of 50°C and lower the resistance to rutting. However, the melting point of this additive was reported by the manufacturer to be 80°C on the Material Safety Data Sheet (MSDS) determined by using ASTM D-127 test method. No further testing was performed to verify this temperature.
3. For all mixtures, use of warm mix additives in combination with polymer modified asphalt binder (PG 58-34P) resulted in increased resistance to rutting. This suggests using stiffer binder to improve the rutting performance of WMA mixtures.

It should be noted that none of mixtures exhibited stripping inflection point. But, two mixtures exhibited severe visual stripping in the wheelpath after completion of the Hamburg rutting test: (1) conventional HMA containing PG 58-28 and type B aggregate, and (2) WMA mixture containing Sonnewarmix with PG 58-28 and type B aggregate. Furthermore, these mixtures did not exhibit such visual stripping after being treated by a liquid anti-stripping agent.

Furthermore, the relationship between the binder's rutting parameter of |G*|/sin(δ) was compared to the mixture's rutting performance as shown in Figure 4-11 and Figure 4-12. This relationship was compared for unaged and RTFO-aged binders. A linear relationship between the unaged |G*|/sin(δ) and the average rut depth exhibits relatively strong coefficient of determination (R^2) equal to 0.82 for mixtures containing aggregate Type A trap rock. However, mixtures containing aggregate Type B showed weaker correlation with R^2 equal to 0.45. Modeling these relationships using a logarithmic function did slightly improve R^2 values, but there was still scatter in the data. For both aggregate types, no relationship could be established between the binder's rutting parameter and rutting depth as there was a great scatter in data. This may imply that the current short-term RTFO aging temperature of 163°C and/or aging duration of 85 minutes may require reconsideration since WMA is produced at lowered temperatures than HMA.

Table 4-3 lists different rankings for the asphalt mixtures based on the average rut depth. It is observed that in general that addition of warm mix additives in general resulted in an decreased level of resistance to rutting.

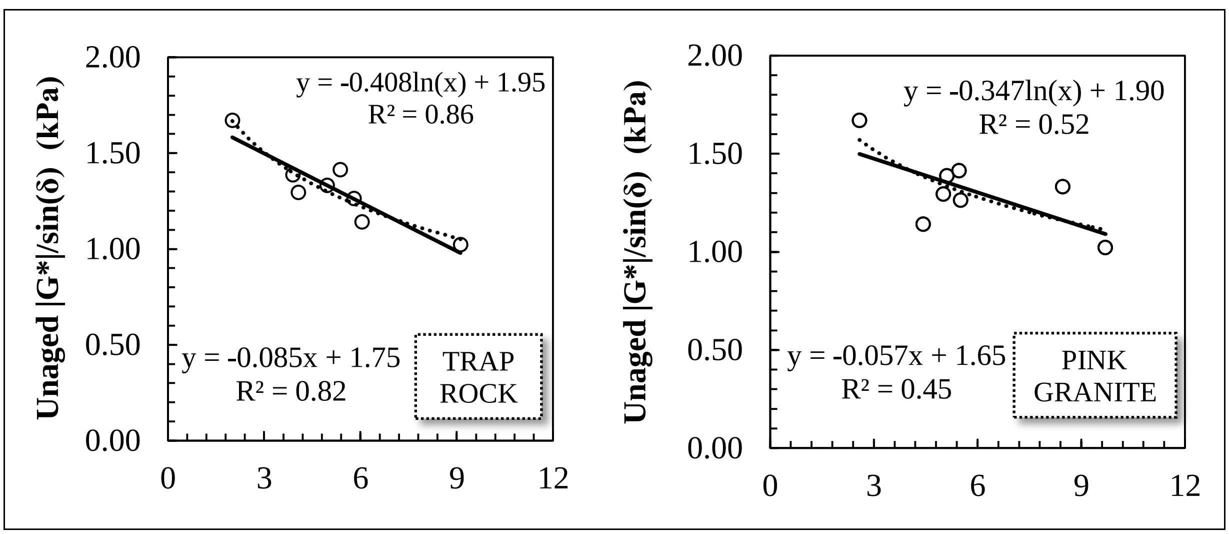


Figure 4-11 Models Describing Relationship Between Asphalt Binder's Unaged Rutting Parameter and Asphalt Mixture Rutting Depth

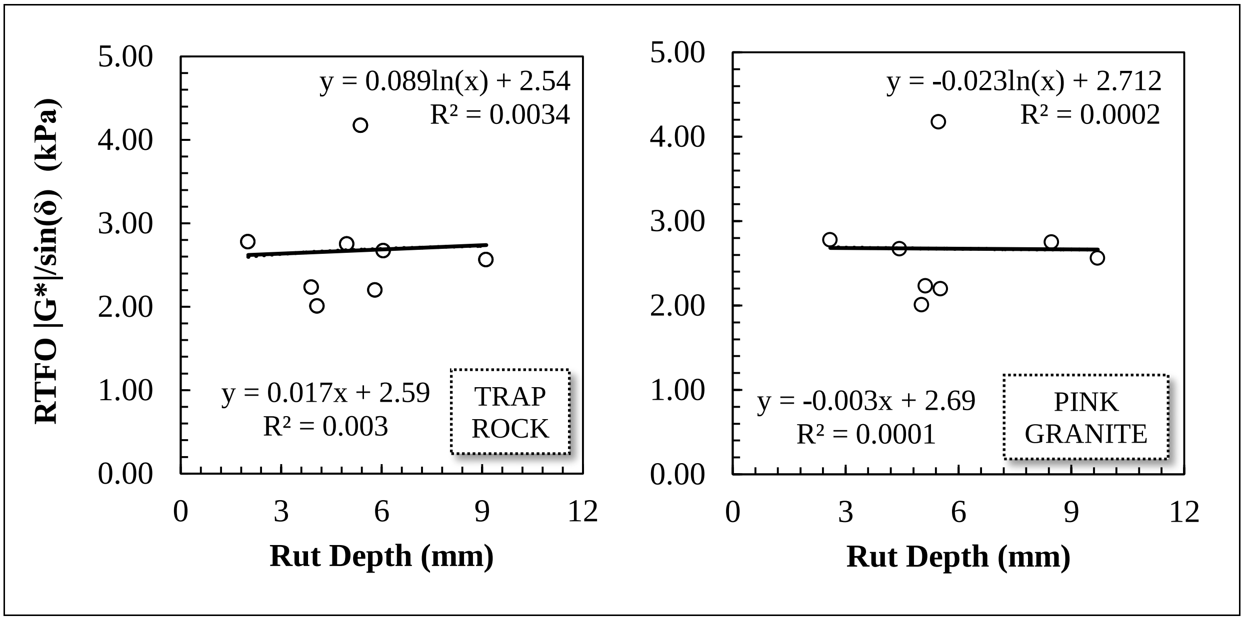


Figure 4-12 Models Describing Relationship Between Asphalt Binder's RTFO-Aged Rutting Parameter and Asphalt Mixture Rutting Depth

Table 4-3 Hamburg Asphalt Mixture Rutting Depth Ranking

Aggregate Type	Binder Grade	Warm Mix Additive	Average Rut Depth (mm)	Rutting Rank[1]
Trap Rock	PG 58-28	Control	5.37	2
		Evotherm 3G.	5.12	1
		Rediset LQ.	6.05	3
		SonneWarmix	9.02	4
	PG 58-34P	Control	2.03	1
		Evotherm 3G.	3.99	2
		Rediset LQ.	4.07	3
		SonneWarmix	5.77	4
Pink Granite	PG 58-28	Control	5.47	2
		Evotherm 3G.	8.22	3
		Rediset LQ.	4.43	1
		SonneWarmix	9.79	4
	PG 58-34P	Control	2.59	1
		Evotherm 3G.	5.12	3
		Rediset LQ.	5.02	2
		SonneWarmix	5.52	4

Note: [1]Ranking is in ascending order for each group of aggregate/binder combination: 1 for the best performance and 4 for the weakest performance.

4.2.2. Intermediate Temperature Performance

The parameter |G*|sin(δ) was selected to capture the effect of warm mix additives on fatigue cracking resistance at intermediate temperatures. Fatigue of visco-elastic materials like asphalt binder is considered as a strain-controlled cyclic loading phenomenon, which during each cycle of reversible loading, a certain amount of work is being done to fatigue the material and cause cracking. A portion of this work is recoverable due to elasticity rebound of the material, and of the remaining is dissipated energy in forms of cracking. The amount of dissipated work per loading cycle can be determined by using Equation 4-5.

$$W_c = \pi . \in_0^2 . (|G^*| \times sin\delta) \qquad \textbf{4-5}$$

where

$W_c =$ work dissipated per loading cycle
$\in_0 =$ strain applied during the load cycle
$|G^*| =$ complex shear modulus
$\delta =$ phase angle

Decreasing the |G*|sin(δ) causes the binder to behave less stiff (more flexible), and thus able to deform without storing relatively large stresses. This parameter was measured at intermediate pavement's in-service temperature at a frequency of 10 radians per second (1.59 Hz) on long-term aged binder, as per AASHTO M 320. Similar to rutting parameter determination, the fatigue parameter was also determined by using a Dynamic Shear Rheometer (DSR). Long-term aging was simulated by using a Pressurized Aging Vessel (PAV) as shown in Figure 4-13.

To perform PAV aging, residue collected from RTFO aging was poured on three stainless pans weighing 50 ± 0.5 grams each. The pans were placed vertically on a pan holder, which then placed in a sealed pressure vessel (Figure 4-13). The aging conditioning was then performed by 20 hours ± 10 minutes of constant pressure of 2.10 MPa at conditioning temperature of 100 ºC. it should be noted that the PAV test procedure does not produce a test result; it outlines details on how to produce residue, which can be used for additional rheology tests described in following sections.

Figure 4-14 illustrates |G*|sin(δ) values measured for each binder at their intended intermediate temperature after aged in Pressure Aging Vessel (PAV), as well as normalized with respect to those values obtained for the control binders. In this figure, error bars represent two standard deviation from the average value of two replicates tested, with |G*|sin(δ) normalized values shown above the bars. Intermediate temperature of 19 and 16°C were selected for binders modified with base binders of PG 58-28 and 58-34 P, respectively.

(a) Pressure Aging Vessel **(b)** Sample holder

Figure 4-13 Pressure Aging Vessel Long Term Aging

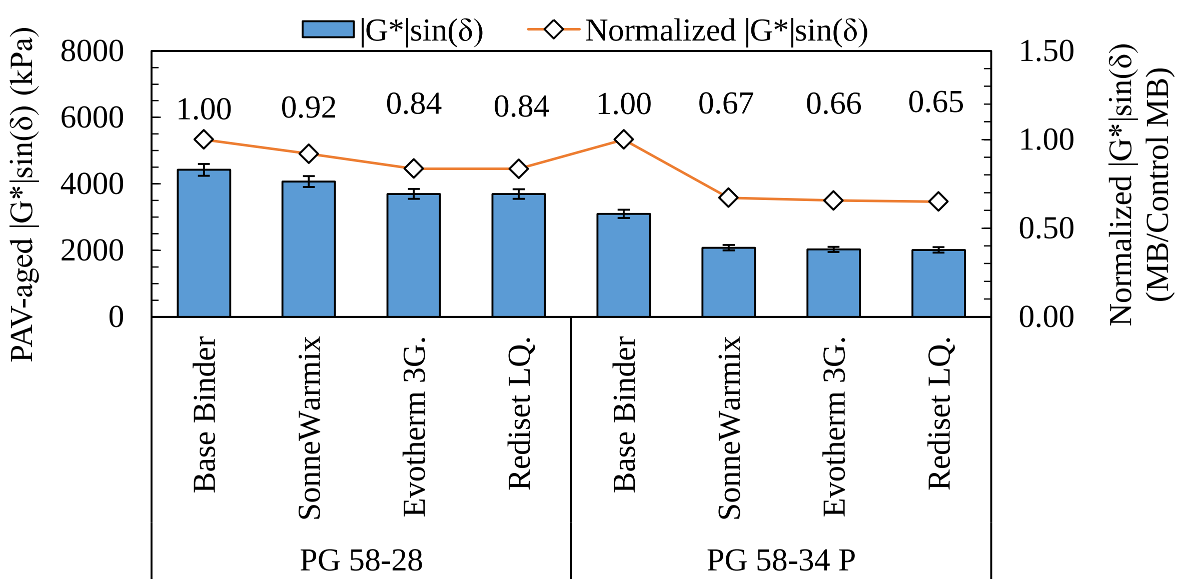


Figure 4-14 PAV-aged |G*|sin(δ) Asphalt Binder Fatigue Parameter

As shown in Figure 4-14, addition of warm mix additives resulted in decreasing the |G*|sin(δ) parameter; suggesting improved level of resistance to fatigue. This decrease was found to be significant by performing statistical T-test presented in Table 4-4 at significance level of 95%. It was also observed that binders modified with Sonnewarmix additive provided the least fatigue resistance improvement compared to other warm mix additives, when added to base PG 58-28. All binders met the upper stiffness limit of 5000 kPa at intermediate testing temperature, as per the AASHTO M320 criterion for the |G*|sin(δ) value after RTFO and Pressure Aging Vessel (PAV).

Table 4-4 T-Test of PAV-aged |G*|sin(δ) Asphalt Binder Fatigue Parameter

Paired Mixes		P-Value*	Statistically Significant
PG 58-28	PG 58-28 + SonneWarmix	0.000	**YES**
	PG 58-28 + Evotherm 3G.	0.000	
	PG 58-28 + Rediset LQ.	0.000	
PG 58-28 + SonneWarmix	PG 58-28 + Evotherm 3G.	0.000	
PG 58-28 + SonneWarmix	PG 58-28 + Rediset LQ.	0.000	
PG 58-28 + Evotherm 3G.	PG 58-28 + Rediset LQ.	0.080	NO
PG 58-34P	PG 58-34P + SonneWarmix	0.000	**YES**
	PG 58-34P + Evotherm 3G.	0.000	
	PG 58-34P + Rediset LQ.	0.000	
PG 58-34P + SonneWarmix	PG 58-34P + Evotherm 3G.	0.000	
PG 58-34P + SonneWarmix	PG 58-34P + Rediset LQ.	0.000	
PG 58-34P + Evotherm 3G.	PG 58-34P + Rediset LQ.	0.000	

Note: *P-Value is the probability of $|T_{observed}|>t_{critical}$ at significance level of 95% (α=0.05)

In order to verify observed trends in the |G*|sin(δ) fatigue parameter, an Indirect Tensile Strength (IDT) tester at CPATT was used to measure tensile strength of asphalt mixtures listed in Table 4-5 at 25°C that can be related to fatigue performance. Mixtures with relatively higher tensile strengths tend to exhibit higher resistance to fatigue loading, resulting in the development of less cracks. It is observed that in general that addition of warm mix additives in general resulted in an decreased level of resistance to rutting; which may result in improved level of resistance to fatigue cracking over long-term performance.

Figure 4-15 illustrates the correlation between the IDT results and binder's fatigue parameter values. Asphalt mixtures were tested at room temperature of 25°C. As shown in Figure 4-15, a power relationship between the PAV-aged |G*|sin(δ) and the average IDT exhibits satisfactory coefficient of determination (R^2) equal to 0.62 and 0.59 for mixtures containing aggregate Type A trap rock and Type B granite, respectively. But, there was still scatter in the data.

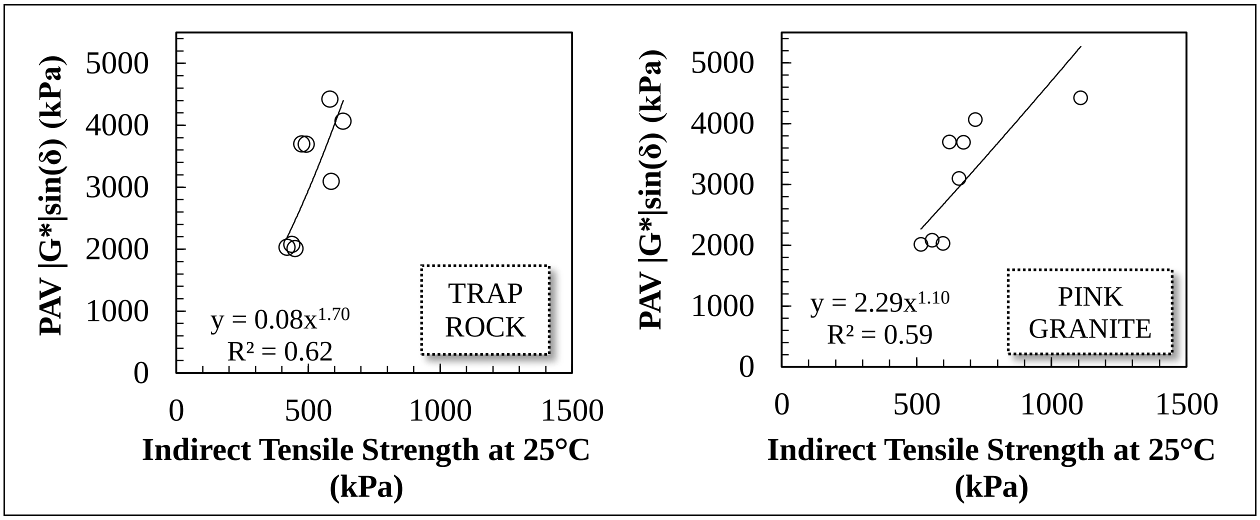


Figure 4-15 Models Describing Relationship Between Asphalt Binder's Fatigue Parameter and Asphalt Mixture Indirect Tensile Strength

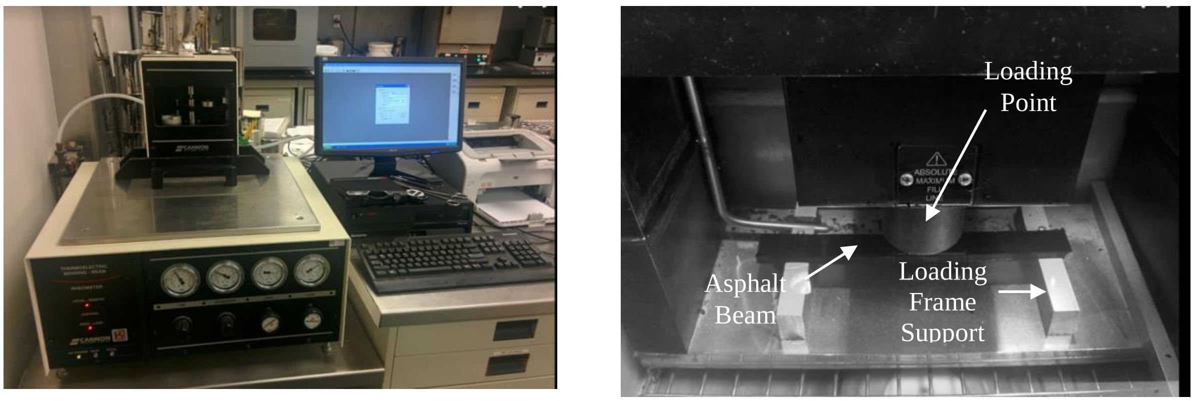
Table 4-5 Mixture Indirect Tensile Strength at 25°C Ranking

Aggregate Type	Binder Grade	Additive	Indirect Tensile Strength at 25°C (kPa)	Standard Deviation	Rank[1]
Trap Rock	PG 58-28	Control	583	122	3
		Evotherm 3G.	476	35.0	1
		Rediset LQ.	493	98.2	2
		SonneWarmix	632	20.5	4
	PG 58-34P	Control	587	76.3	4
		Evotherm 3G.	420	16.8	2
		Rediset LQ.	450	63.3	3
		SonneWarmix	340	68.0	1
Pink Granite	PG 58-28	Control	1109	60.0	4
		Evotherm 3G.	718	49.0	3
		Rediset LQ.	622	59.3	1
		SonneWarmix	674	41.4	2
	PG 58-34P	Control	658	18.4	4
		Evotherm 3G.	598	64.5	3
		Rediset LQ.	516	28.4	1
		SonneWarmix	558	28.1	2

Note: [1]Ranking is in ascending order for each group of aggregate/binder combination: 1 for the best performance and 4 for the weakest performance.

4.2.3. Low Temperature Performance

Creep response (S) and relaxation (m-value) of asphalt binder to a constant applied load were used to evaluate the effect of warm mix additives on performance of the binder at colder temperatures. These values were measured by using a Bending Beam Rheometer (BBR) shown in Figure 4-16 to apply a constant creep load of 100 grams (980 mN) to an asphalt binder beam specimen measuring 125 mm in length, 6.35 mm in width, and 12.7 mm in height for 240 seconds. The creep response was then calculated by using the elementary Bernoulli-Euler theory of bending prismatic beams as presented by Equation 4-6, while the m-value is the slope of the logarithmic stiffness versus logarithmic time curve at time.



(a) Cannon Bending Beam Rheometer **(b)** Inside View of BBR

Figure 4-16 Bending Beam Rheometer Equipment

$$S(t) = \frac{PL^3}{4bh^3\delta(t)} \qquad \text{4-6}$$

where

$S(t) =$ creep stiffness at time of t seconds
$P =$ applied constant load
$\delta(t) =$ deflection at time of t seconds
$L, b, and\ h =$ dimension of asphalt binder beam

To relate the creep stiffness to thermal cracking resistance of a binder, the AASHTO M320 specifies an upper limit of 300 MPa for the creep stiffness measured at 60 seconds (S(60)), and a lower limit of 0.300 for creep rate (m-value). It should be noted that decreasing the creep stiffness (or increasing m-value) causes the binder to behave less stiff (more flexible), and thus able to deform without storing relatively large stresses. Creep stiffness and m-value were measured at testing temperatures of -18°C and -24°C to simulate the minimum pavement temperature of -28°C and -34°C respectively. Furthermore, the BBR test was performed on PAV-aged binders.

It should be noted that the testing temperature for creep stiffness and m-value is 10˚C higher than the lowest pavement service temperature for loading time of 60 seconds. This offset was verified, at the time of AASHTO M320 development, to be sufficient to equate the S(60) to the asphalt binder stiffness at two hours loading time in the field at lowest pavement service temperature (Bahia & Anderson, The New Proposed Rheological Properties of Asphalt Cement, 1995).

Figure 4-17 illustrates S and m-value measured for each binder at their intended low temperature after aged in Pressure Aging Vessel (PAV), but are normalized with respect to those values obtained for the control binders. Low temperature of -18°C and -24°C were selected for binders modified with base binders of PG 58-28 and 58-34 P, respectively. As shown above, warm mix additives did not affect the stiffness and m-value of the base binders. This suggests that addition of warm mix additives may not affect low temperature properties of the base binders (PG 58-28 and PG 58-34P).

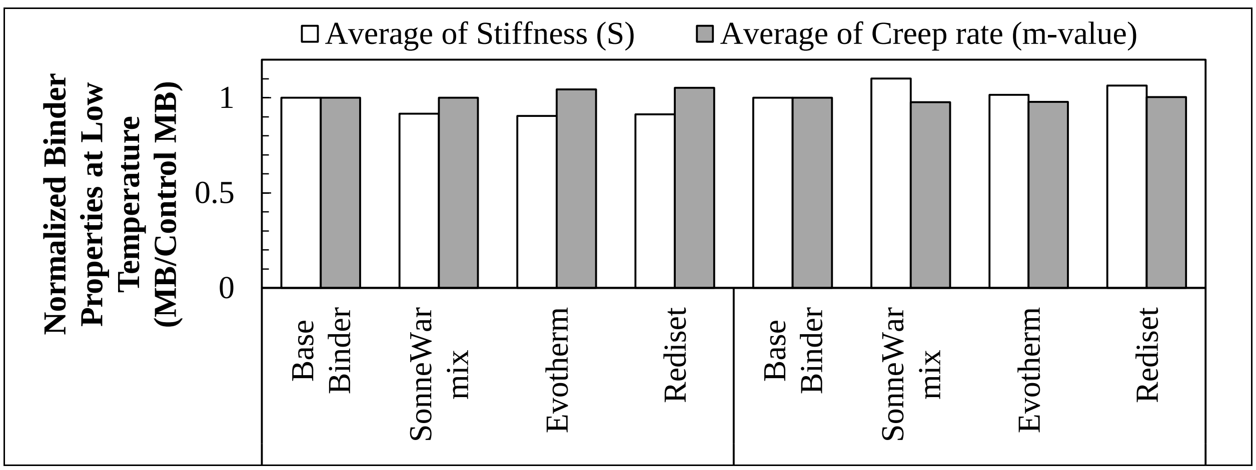


Figure 4-17 Stiffness and m-value Asphalt Binder Low Temperature Parameters

4.3. Dynamic Modulus

To evaluate stiffness of compacted mixtures containing WMA additives, Dynamic Modulus test setup at CPATT was used to determine dynamic modulus (|E*|) of specimens. The cored specimens were tested at six loading frequencies (0.1, 0.5, 1, 5, 10 and 25 Hz) and five different temperatures (-10, 4.4, 21.1, 37.8 and 54.4°C) to obtain |E*|. Figure 4-18 to Figure 4-21 illustrate |E*| values measured for each mixture type, but are normalized with respect to those values obtained for the control mixture. Furthermore, to rank the mixtures based on their performance as listed in Table 4-6, three |E*| values were selected from each mix to evaluate the stiffness at temperatures and frequencies that can be used as indicators of three main surface distresses: (1) thermal cracking at temperature of -10°C and high frequency of 25 Hz, (2) fatigue cracking at 21.1°C and intermediate frequency of 10 Hz, and (3) rutting at high temperature of 54.4°C and low frequency of 0.1 Hz.

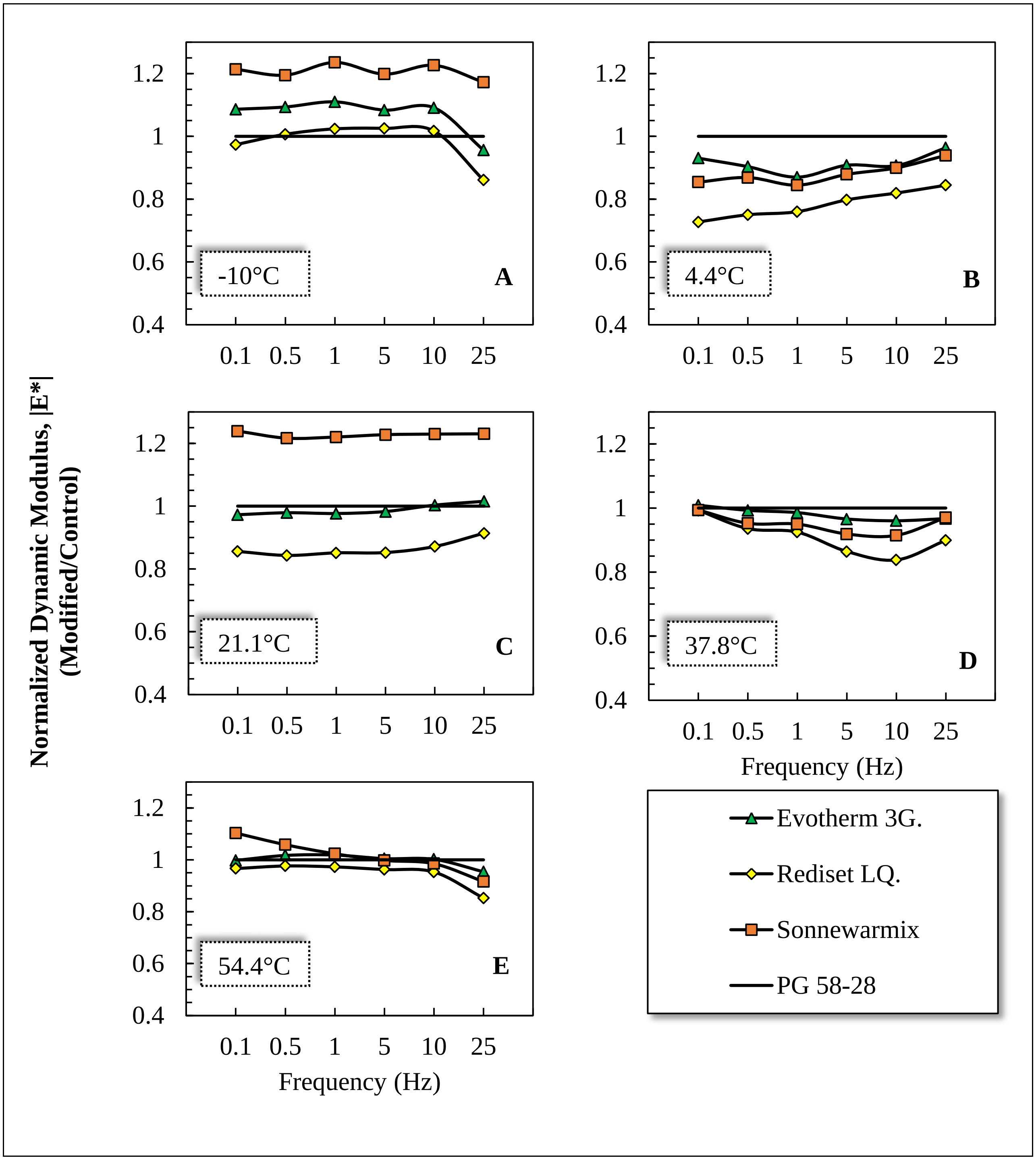


Figure 4-18 Dynamic Modulus Results For Mixtures Containing Type A Aggregate (Trap Rock Diabase) With PG 58-28

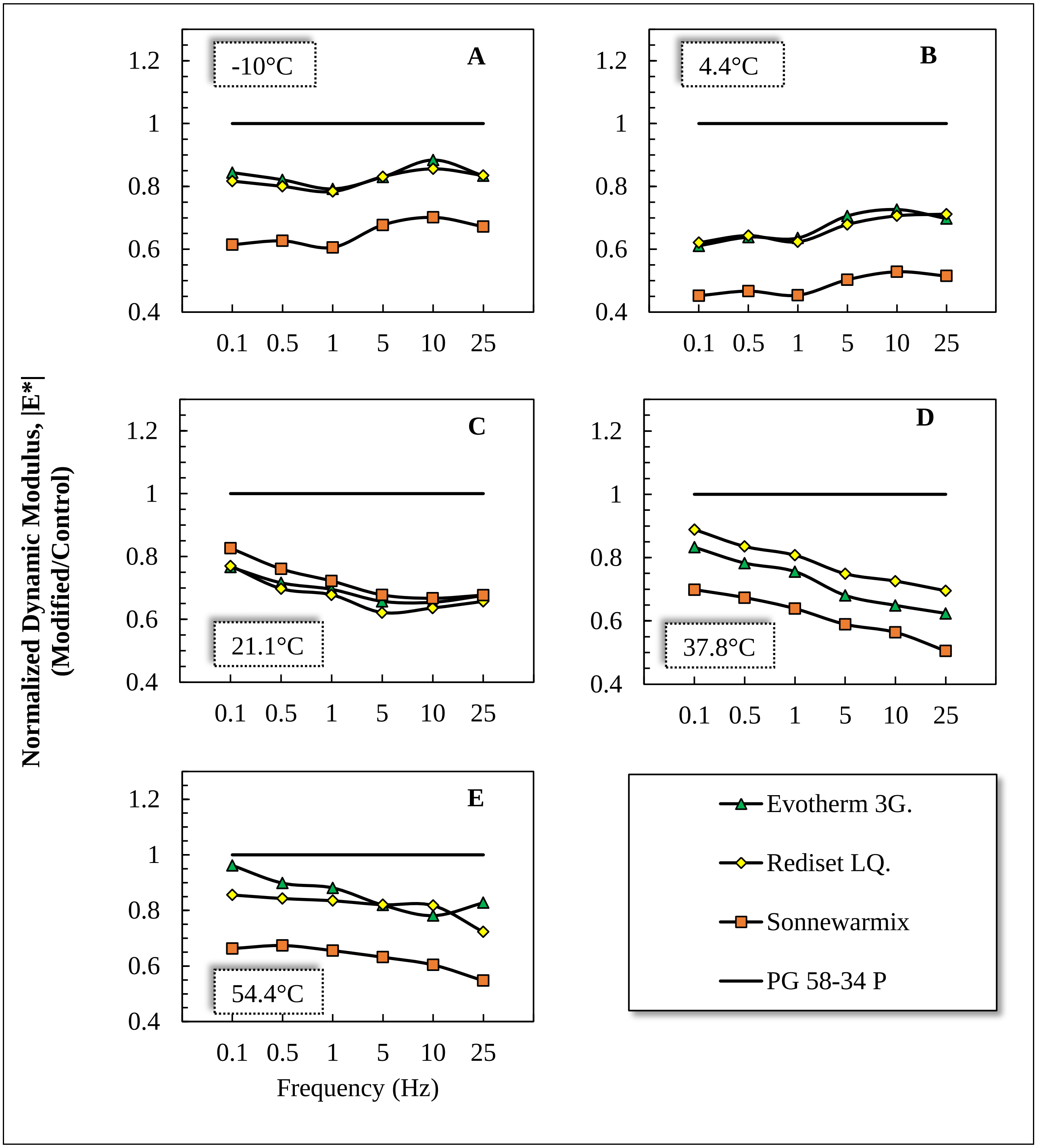


Figure 4-19 Dynamic Modulus Results For Mixtures Containing Type A Aggregate (Trap Rock Diabase) With PG 58-34 P

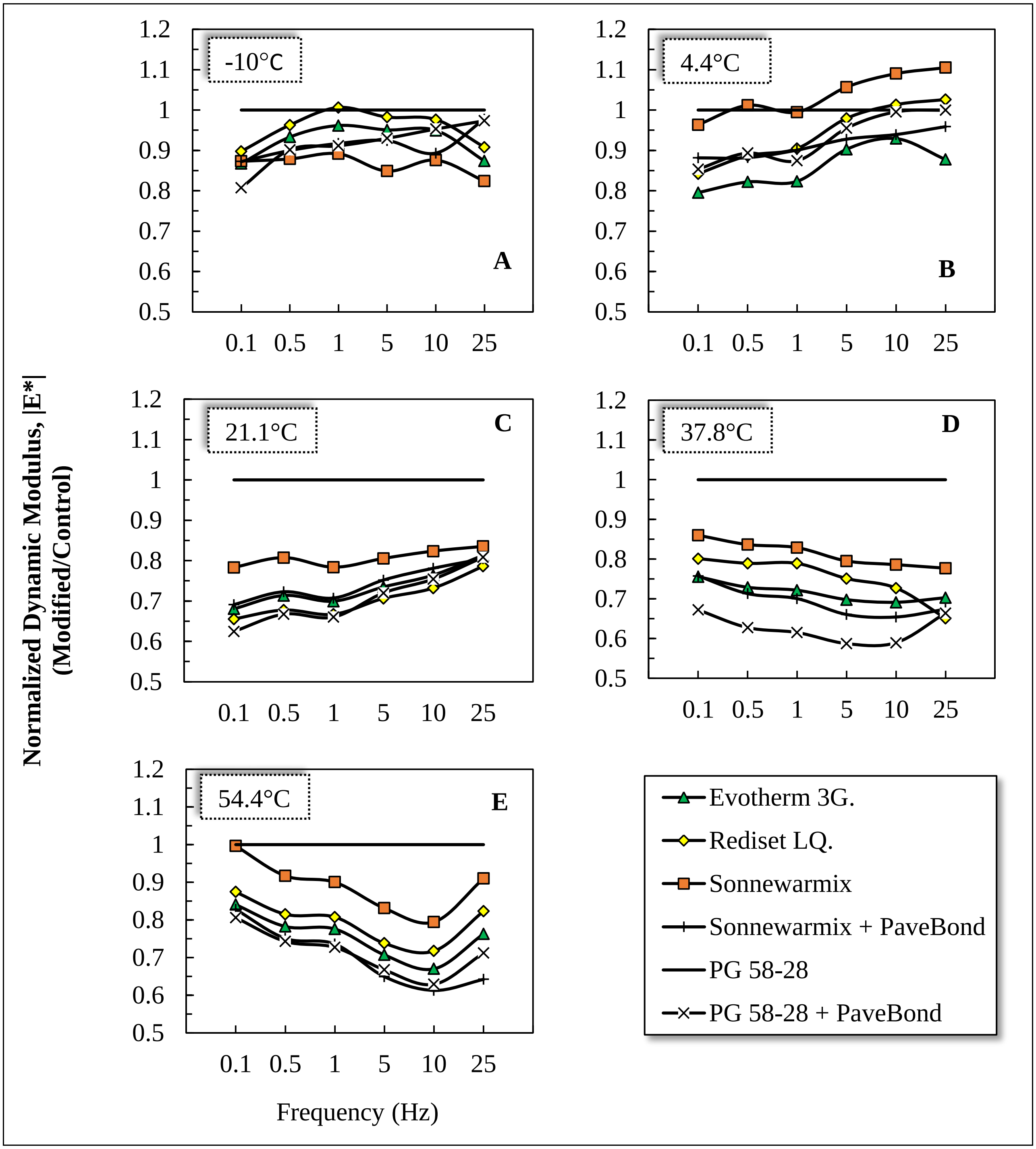


Figure 4-20 Dynamic Modulus Results For Mixtures Containing Type B Aggregate (Pink Granite) With PG 58-28

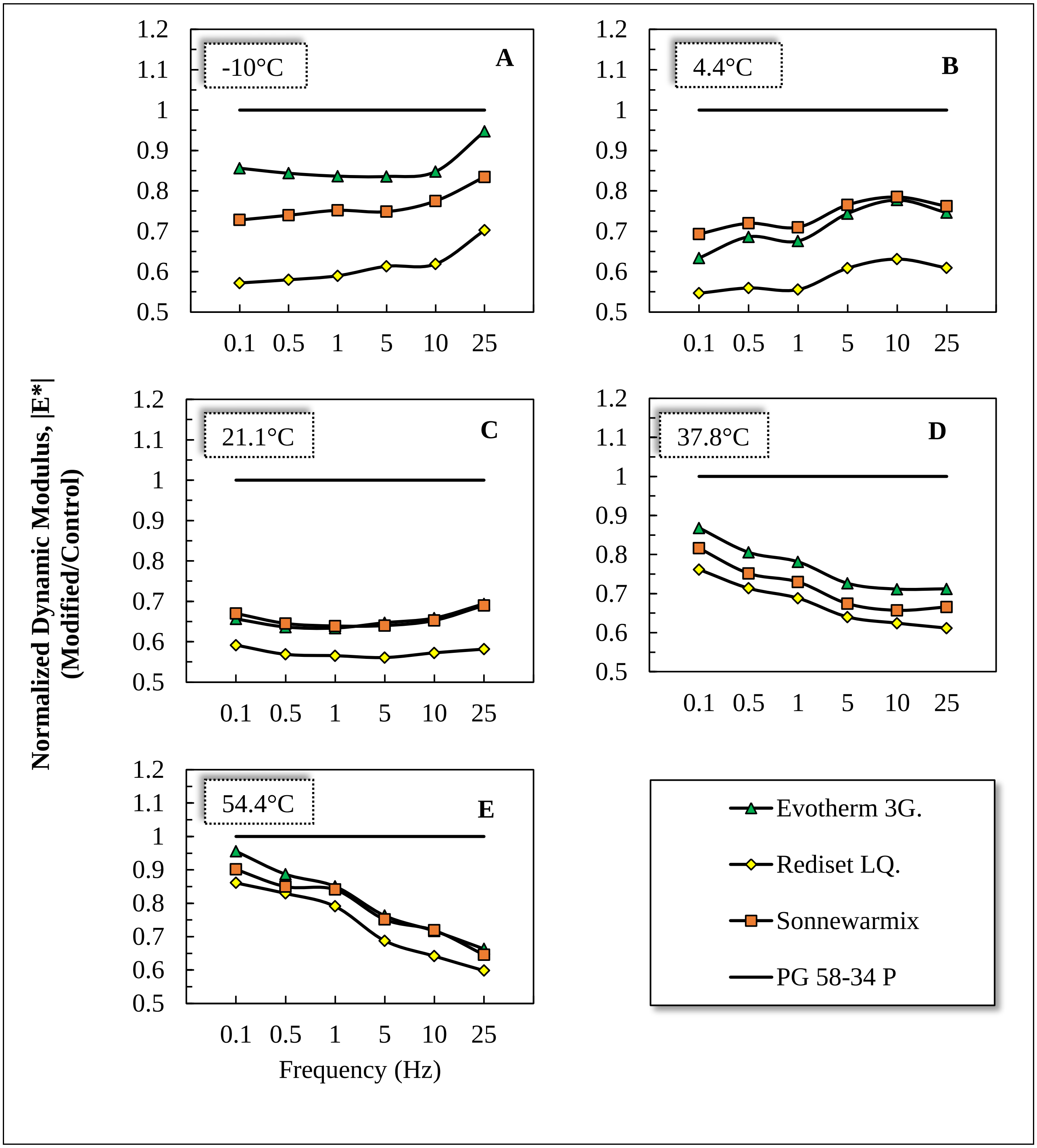


Figure 4-21 Dynamic Modulus Results For Mixtures Containing Type B Aggregate (Pink Granite) With PG 58-34 P

The following general trends were observed from the dynamic modulus testing: °C

1. As shown in Figure 4-18(a), addition of Evotherm 3G. and Sonnewarmix additive caused an increase in the cold temperature stiffness of the control mixture containing PG 58-28 and Type A aggregate. This suggests that addition of these additives may reduce the level of resistance to thermal cracking at colder temperatures. Rediset LQ., however, was found to have marginal effect on the control mixture's stiffness at this temperature. Furthermore, it was observed that such increase in stiffness can be mitigated by using warm mix additives in combination with polymer modified asphalt binder (PG 58-34P) as shown in Figure 4-19(a). This may suggest improved level of resistance to thermal cracking.
2. In general, it was observed that addition of warm mix additives decreased stiffness of the control mixtures containing asphalt binder types of PG 58-28 and 58-34P in combination with Type A aggregate at intermediate in-service temperatures of 4.4 and 21.1°C, except when Sonnewarmix was added to the PG 58-28. This may suggest improved level of resistance to fatigue cracking.
3. Addition of warm mix additives was observed to decrease the high temperature stiffness of the control mixtures containing asphalt binder types of PG 58-28 and 58-34P in combination with Type A aggregate at in-service temperatures of 37 and 54.4°C. This may suggest lowered level of resistance to rutting. This correlates well with trends observed in mixture's rutting results (Figure 4-7 and Figure 4-8).
4. Similar aforementioned trends were observed when warm mix additives were used in combination with Type B pink granite and PG 58-28 and 58-34P asphalt binder types; except the thermal cracking resistance was not affected by additives.

Measurements obtain from dynamic modulus testing were further combined to obtain a master curve in accordance with AASHTO PP62-09 procedure, "Standard Practice for Developing Modulus Master Curve for Hot-Mix Asphalt" (AASHTO, 2009). Figure 4-22 to Figure 4-25 illustrates the |E*| measurement from different frequencies and temperatures merged into master curve for all mixtures under study at reference temperature of 21.1°C.

The middle portion of the dynamic modulus master curve is usually employed for comparison, as it is the most sensitive part of the curve to changes in binder stiffness. Similar to trends observed previously in dynamic modulus results, master curve results also suggest that addition of WMA resulted in lower overall stiffness of mixtures, except when Sonnewarmix was added to PG 58-28 base binder in combination with Type A aggregate Trap Rock. This is correlated with results obtained from IDT testing at 25˚C as well. Moreover, it is observed that the addition of Rediset LQ. generally resulted in lower stiffness than mixtures containing Evotherm 3G. and Sonnewarmix. In general, majority of the rankings provided in Table 4-6 were observed to correlate well with results previously analyzed from IDT testing at 25°C and Hamburg rutting test at 50°C.

Table 4-6 Asphalt Mixture Dynamic Modulus Ranking

Aggregate Type	Binder Type	Additive Type	-10°C (25Hz)			21.1°C (10Hz)			54.4°C (0.1Hz)		
			|E*| (MPa)	Standard Deviation	Rank	|E*| (MPa)	Standard Deviation	Rank	|E*| (MPa)	Standard Deviation	Rank
Trap Rock	PG 58-28	Control	24404	2735	3	5889	347	2	313	24.2	2
		Evotherm 3G.	23334	1173	2	5908	321	3	313	15.1	2
		Rediset LQ.	21004	3095	1	5133	257	1	302	4.09	3
		SonneWarmix	28612	1419	4	7241	196	4	345	21.4	1
	PG 58-34P	Control	22854	5360	4	5221	256	4	392	49.8	1
		Evotherm 3G.	19055	3447	2	3415	262	2	377	52.7	2
		Rediset LQ.	19076	5459	3	3321	-	1	336	22.2	3
		SonneWarmix	15356	3652	1	3482	744	3	260	6.20	4
Pink Granite	PG 58-28	Control	23441	1066	4	6888	149	4	335	41.1	1
		Evotherm 3G.	20484	1793	2	5264	48.2	2	283	14.7	3
		Rediset LQ.	21285	7129	3	5044	149	1	294	4.68	2
		SonneWarmix	19312	6489	1	5674	69.1	3	335	24.0	1
	PG 58-34P	Control	20461	1255	4	5313	218	4	355	54.6	1
		Evotherm 3G.	19382	2349	3	3499	175	3	339	14.4	2
		Rediset LQ.	14374	9526	1	3042	97.4	1	306	62.2	4
		SonneWarmix	17075	2483	2	3468	241	2	320	8.71	3

Note: Ranking is in ascending order for each group of aggregate/binder combination: "1" for the best performance and "4" for the weakest performance.

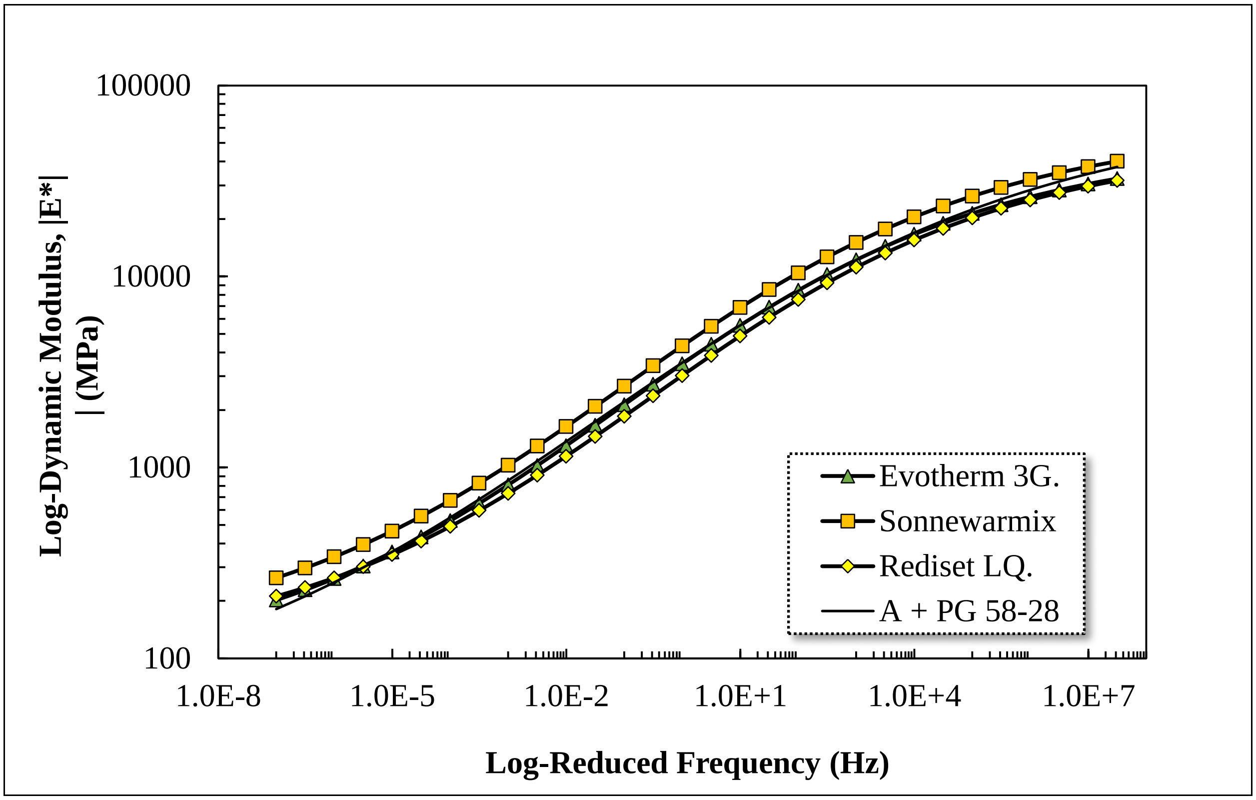


Figure 4-22 Master Curve Results for Mixtures Containing Type A Aggregate (Trap Rock Diabase) with PG 58-28

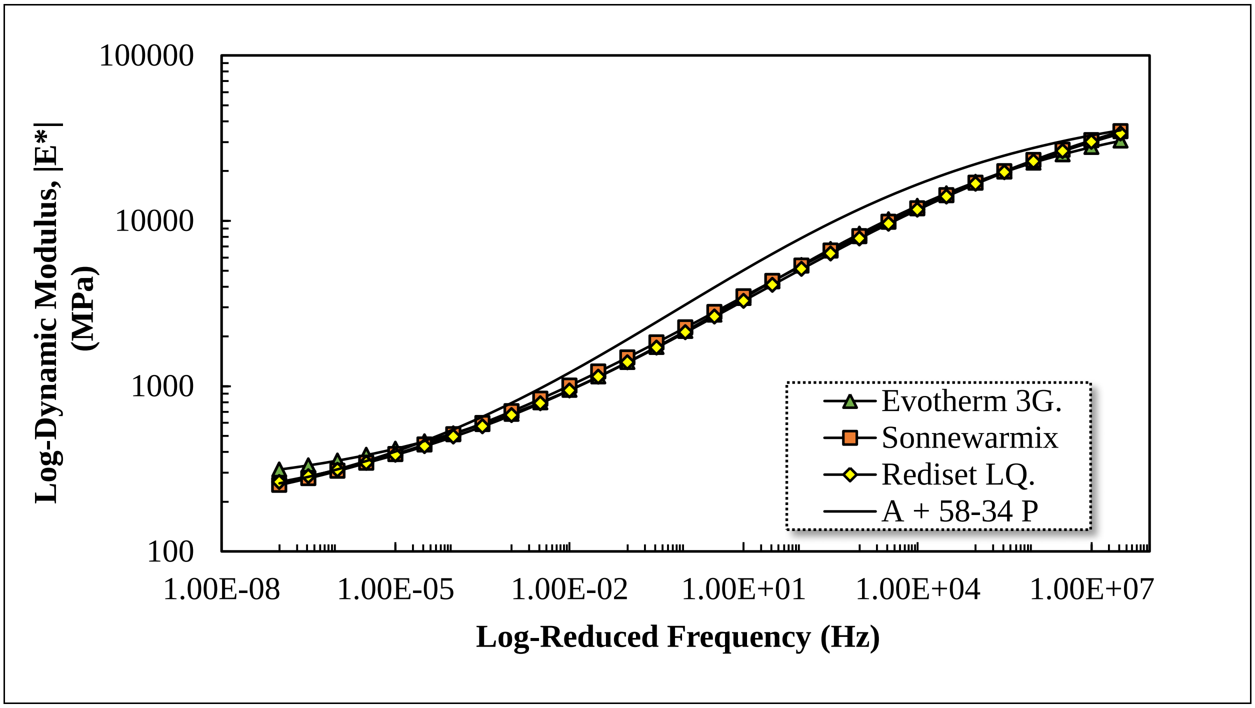


Figure 4-23 Master Curve Results for Mixtures Containing Type A Aggregate (Trap Rock Diabase) with PG 58-34P

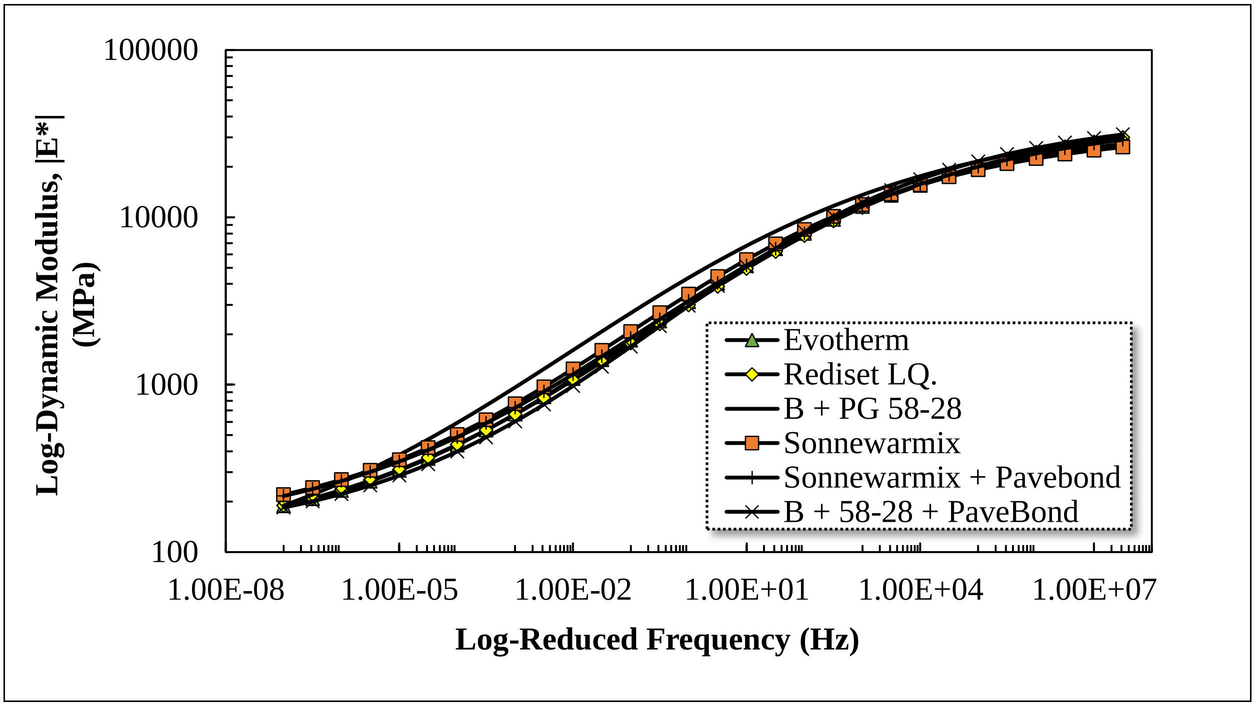


Figure 4-24 Master Curve Results for Mixtures Containing Type A Aggregate (Pink Granite) with PG 58-38

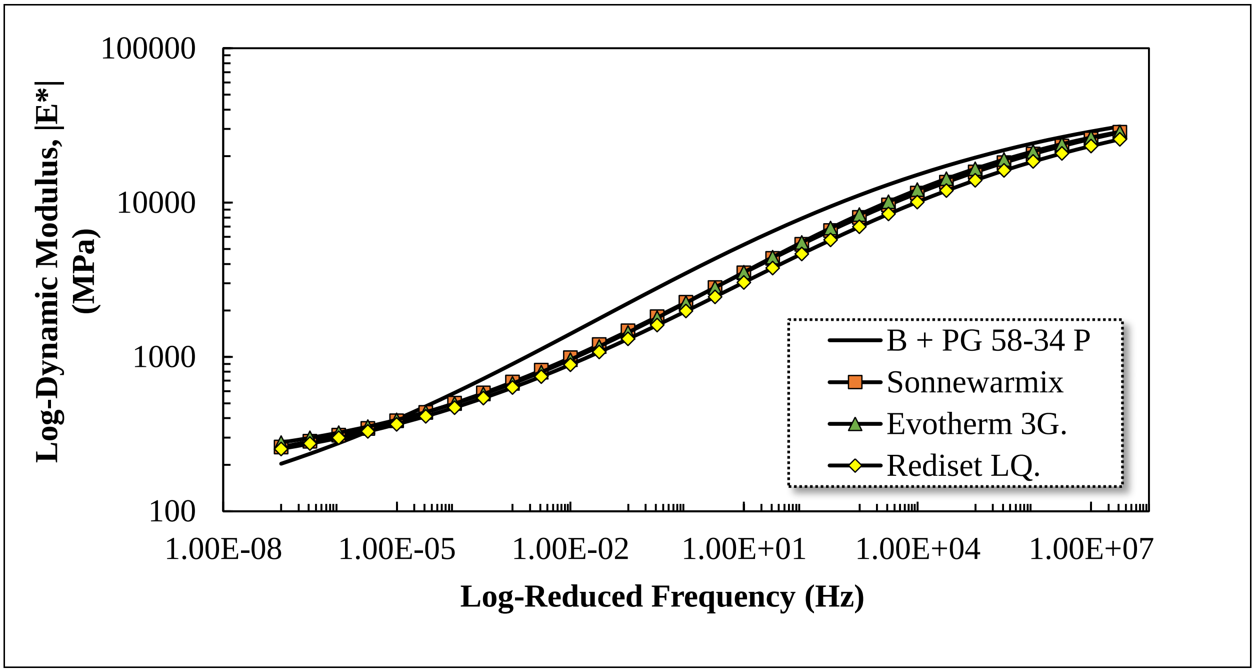


Figure 4-25 Master Curve Results for Mixtures Containing Type B Aggregate (Pink Granite) with PG 58-34P

4.4. Moisture Sensitivity Evaluation

A combination of qualitative and quantitative laboratory test methods shown in Figure 3-20 were used to assess moisture susceptibility of mixtures containing different types of binder, WMA additive and aggregate blends. The main objective of this assessment was to establish a reliable ranking system for moisture susceptibility of WMA mixtures.

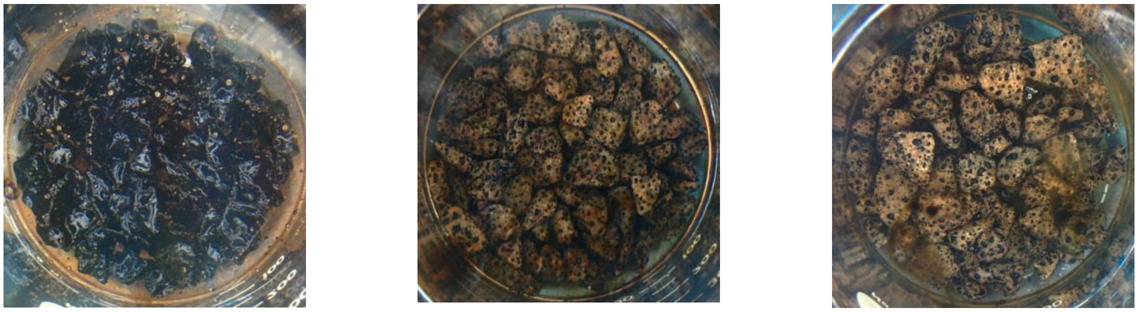
4.4.1. Static Immersion Test

To assess quality of chemical compatibility and bonding between modified binders and aggregates, static immersion test was performed. For this test, it was observed that all combinations with Type A aggregate resulted in average percent retained coating of more than 95 percent similar to Figure 4-26 (a). However, combination of Type B aggregate and PG 58-28 base binder resulted in severe stripping as shown in Figure 4-26 (b). Similar severe stripping was also observed when Sonnewarmix additive was used with Type B aggregate and PG 58-28, as shown in Figure 4-26 (c). This suggests requirement of anti-striping agent when Sonnewarmix is used with an aggregate source with known history of moisture susceptibility. This recommendation was further validated by adding an anti-stripping additive, as shown in Figure 4-26 (d). Moreover, results obtained from static immersion test implies that Evotherm 3G and Rediset LQ. may not require anti-stripping additive (Figure 4-26 (e) and (f)).

4.4.2. Tensile Strength Ratio (TSR)

The resistance of compacted mixtures to moisture damage as the percentage of indirect tensile strength ratio was evaluated by employing two moisture conditioning protocols: (1) vacuum saturation followed by one freeze-thaw cycle as per AASHTO T283 procedure, and (2) moisture conditioning performed by MIST. Figure 4-27 and Figure 4-28 present the IDT strength test results for T283 and MIST conditioned specimens containing Type A aggregate and different additive types. IDT strength of mixtures containing Type B are shown in Figure 4-29 and Figure 4-30. In all figures, error bars represent one standard deviation from the average value of three replicates tested, with TSR results shown above the bars of each conditioning protocol.

In general, for all mixtures, addition of WMA resulted in lower tensile strength of dry mixtures, except when Sonnewarmix additive was used in mixture containing PG 58-28 base binder with Type A aggregate. This was expected, similar trend was observed in dynamic modulus results. Addition of warm mix additives improved TSR value, except when Sonnewarmix was used. However, usage of anti-stripping agent and modified binder (PG 58-34P) improved TSR values as shown in Figure 4-31.



(a) PG58-28 + trap rock diabase Type A
(>95% retained coating)

(b) Base PG58-28 + granite Type B
(10% retained coating)

(c) PG58-28 + Sonnewarm + granite Type B
(10% retained coating)

(d) PG58-28 + Sonnewarm + PaveBond+ granite Type B
(87% retained coating)

(e) PG58-28 + Rediset + granite Type B
(>95% retained coating)

(f) PG58-28 + Evotherm + granite Type B
(>95% retained coating

Figure 4-26 Static Immersion Test (LS-285) Visual Ratings

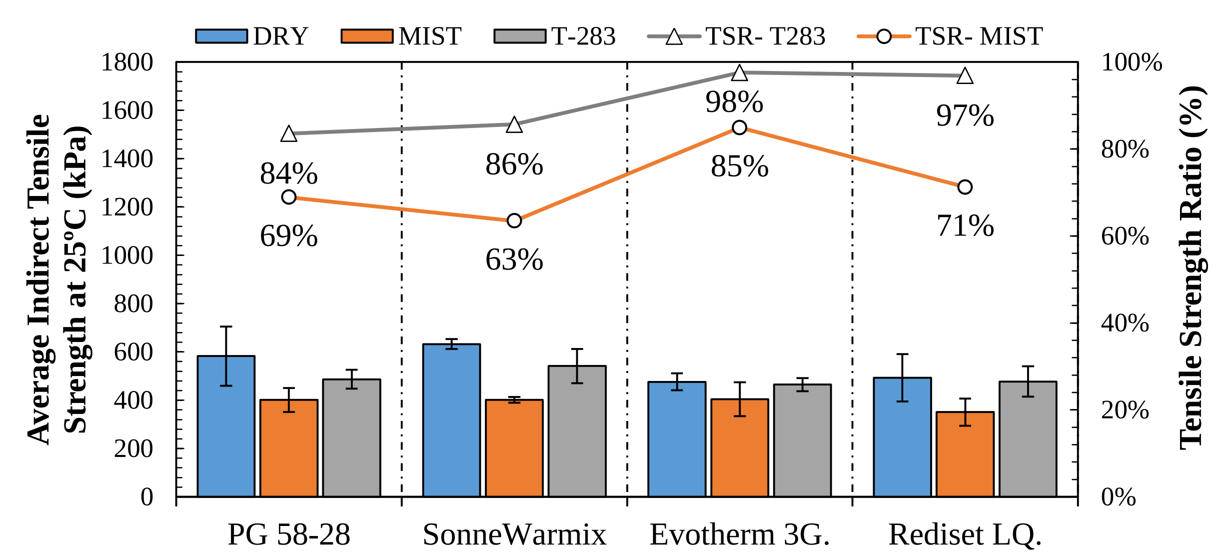


Figure 4-27 Effect of Warm Mix Additives on Indirect Tensile Strength of Mixtures Containing Type A Aggregate (Trap Rock Diabase) and PG 58-28 Base Binder

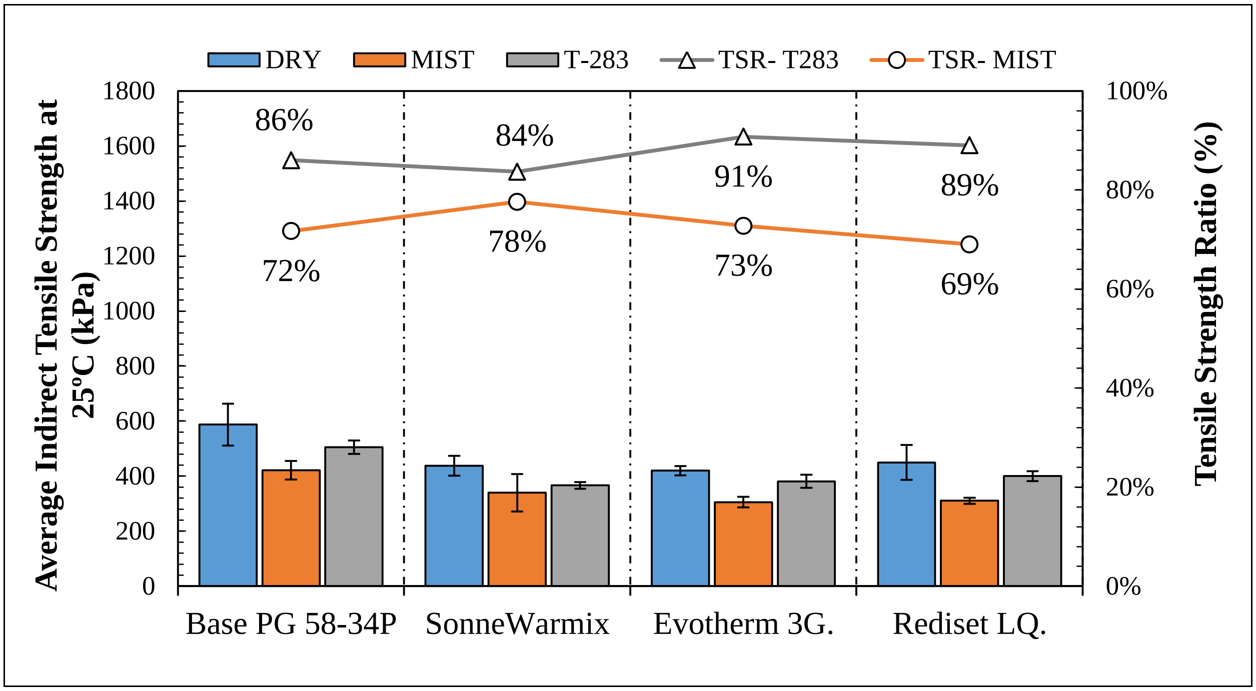


Figure 4-28 Effect of Warm Mix Additives on Indirect Tensile Strength of Mixtures Containing Type A Aggregate (Trap Rock Diabase) and PG 58-34P Base Binder

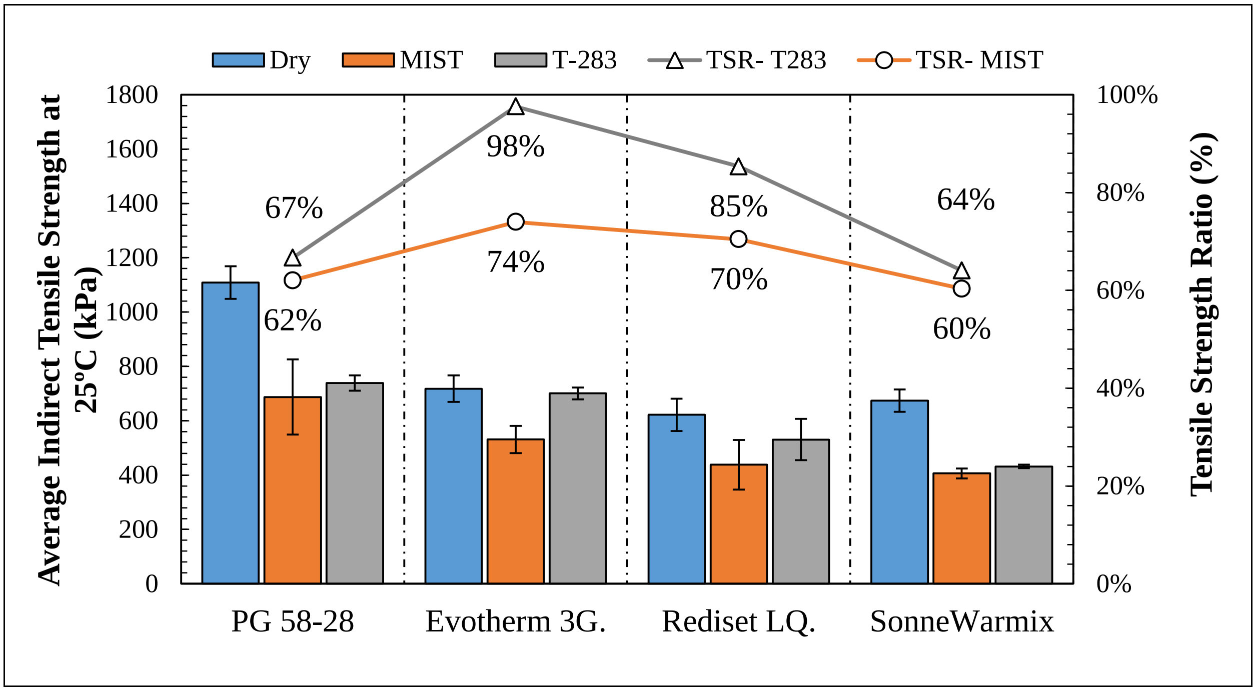


Figure 4-29 Effect of Warm Mix Additives on Indirect Tensile Strength of mixtures containing Type B aggregate (granite) and PG 58-28 base binder

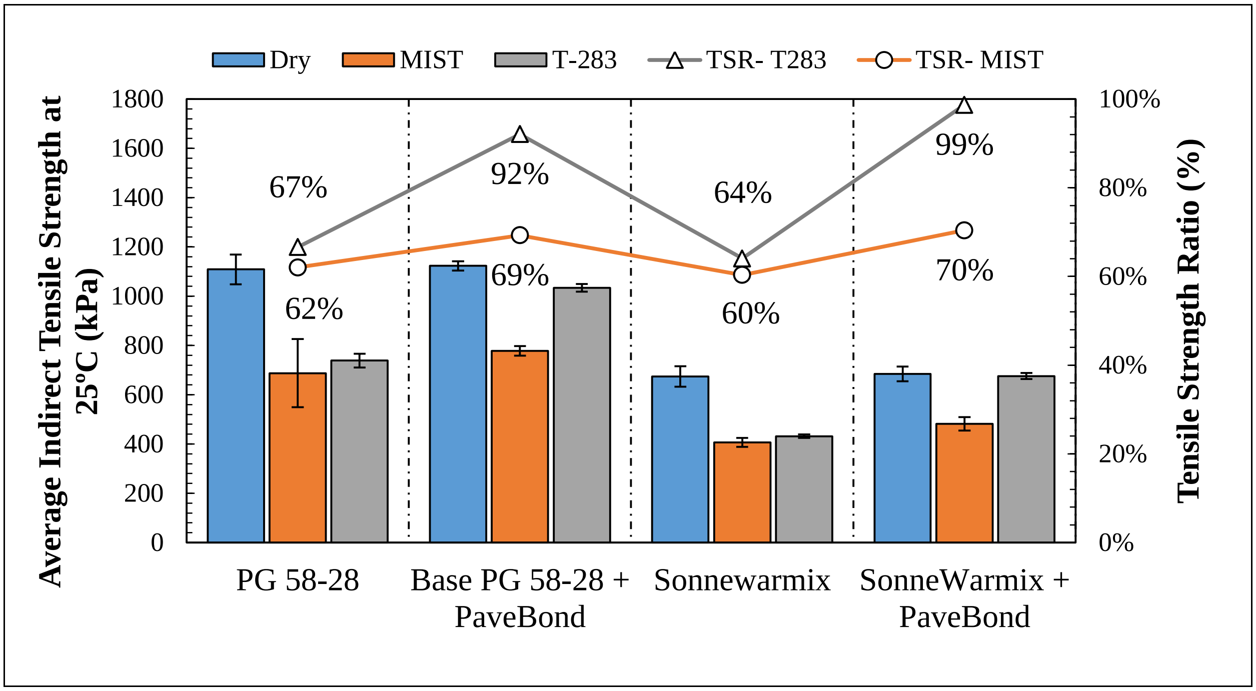


Figure 4-30 Effect of Liquid Anti-Stripping Agent on Indirect Tensile Strength of Mixtures Containing Type B Aggregate (Granite) and PG 58-28 Base Binder

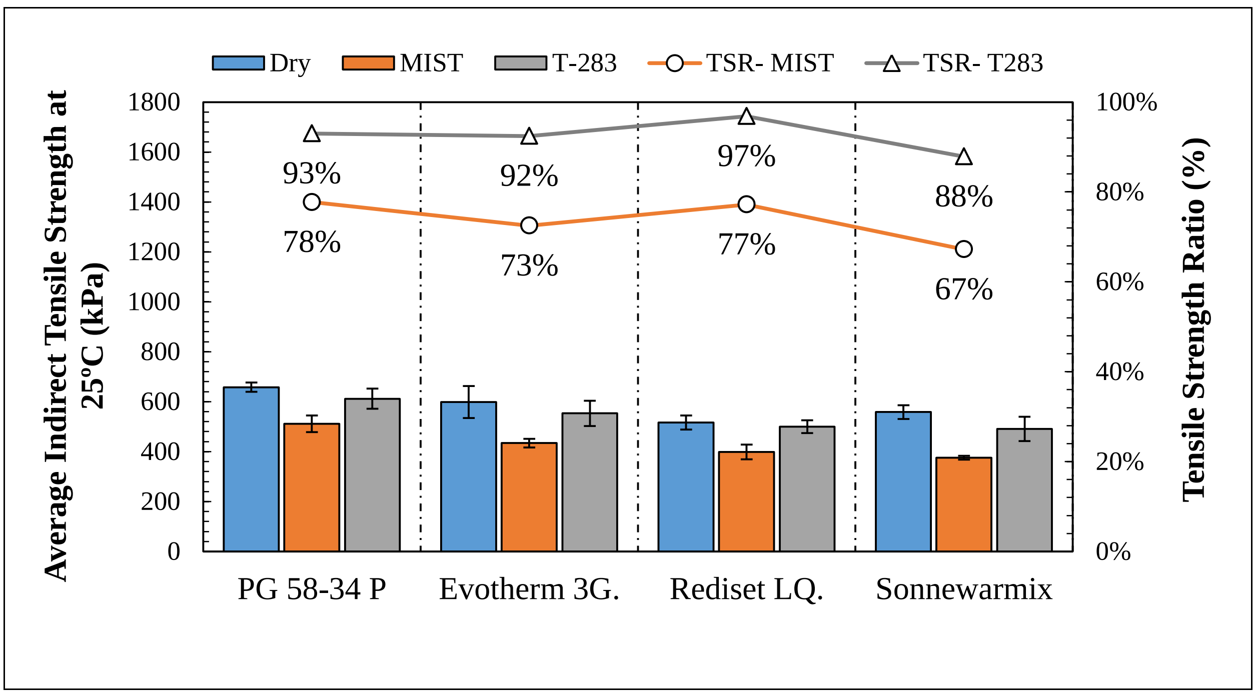


Figure 4-31 Effect of Warm Mix Additives on Indirect Tensile Strength of Mixtures Containing Type B Aggregate (Granite) and PG 58-34P Base Binder

According to TSR values obtained from T283 conditioning protocol, Evotherm 3G provided higher level of resistance to moisture damage compared to Rediset LQ and Sonnewarmix, expect when Evotherm 3G was used in combination with PG 58-34P and Type B aggregate. Furthermore, it was observed that TSR values of all mixtures are more than threshold of 80% specified by MTO, except when Sonnewarmix was used with PG 58-28 and Type B aggregate.

Conventional HMA containing PG 58-28 and Type B aggregate also exhibited TSR value of less than 80%, however, the TSR exceeded the threshold after addition of PaveBond liquid anti-stripping agent, as depicted in Figure 4-31. Similar observation was made for static immersion test. It was also observed that mixtures containing Type B aggregate resulted in higher dry tensile strength compare to those containing Type A aggregate.

Overall, MIST conditioning protocol resulted in lower TSR values for all mixtures compare to T283. The significance of this difference was evaluated by a statistical pairwise comparison known as "Tukey" method at 95% confidence interval. The analysis was performed by Minitab© statistical software and results of this method is presented in Table 4-7 in terms of connecting letters group. Tukey's results indicate that MIST protocol significantly produced the most severe moisture damage compared to T283 protocol.

Table 4-7 Tukey Statistical Analysis Results at 95% Confidence Interval for Different Moisture Conditioning Protocols

	Number of specimens	Average Indirect Tensile Strength at 25ºC (kPa)	Grouping Letter[2]
Unconditioned (dry)	48	602	A
T283 ("modified Lottman")	48	512	B
MIST[1]	48	420	C

Note: [1]MIST = Moisture Induced Stress Tester, and [2]protocals that do not share a letter are significantly different

The TSR results obtained from T283 and MIST conditioning were further examined for correlation with percent retained coating obtained from static immersion testing. As shown in Figure 4-32, a good correlation (R^2 value of 0.67) was observed for TSR results obtained from T283 conditioning and retained coating. However, the correlation between TSR results obtained from MIST conditioning and retained coating was found to be relatively lower (R^2 value of 0.269).

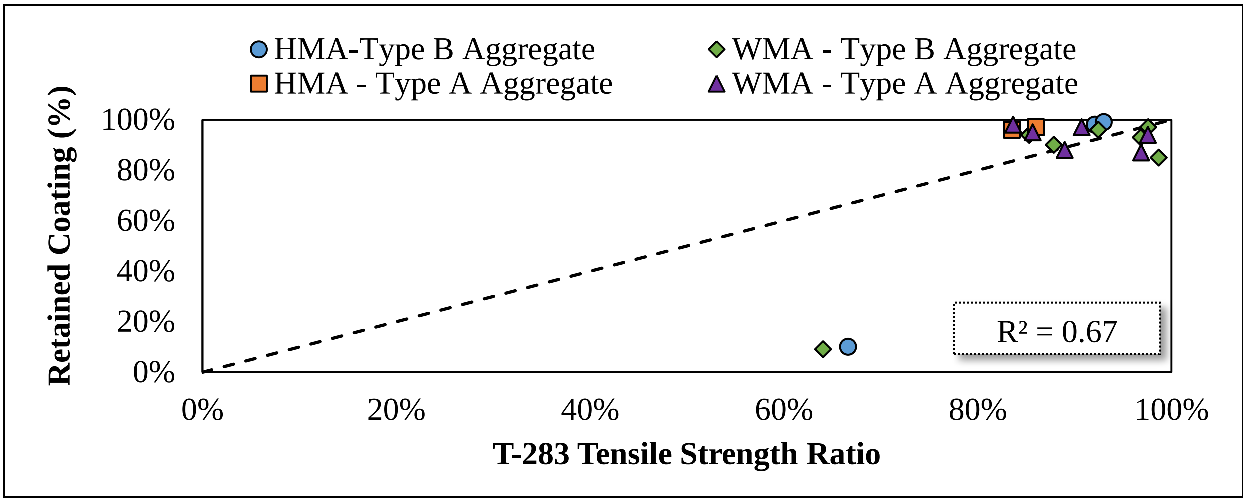


Figure 4-32 Relationships Between T283 Conditioned TSR Results And Retained Coating Obtained From Static Immersion Test

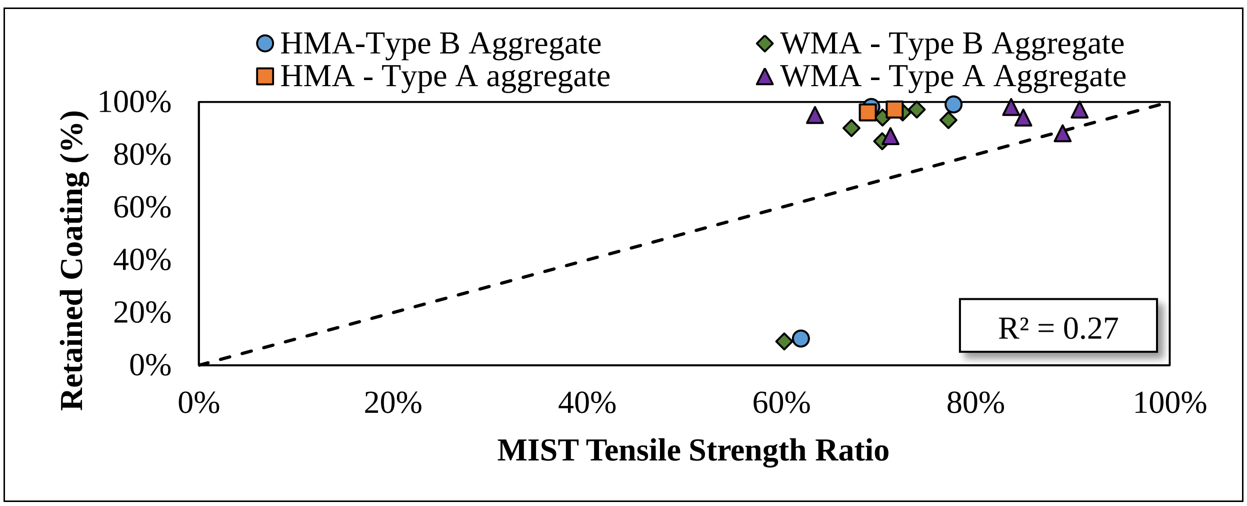


Figure 4-33 Relationships Between MIST Conditioned TSR Results And Retained Coating Obtained From Static Immersion Test

To further evaluate the moisture susceptibility of asphalt mixtures, change in density (also known as "swelling") was calculated for each specimen after MIST conditioning. Swelling was calculated by using Equation 4-6, by measuring Bulk Relative Density (BRD) of each specimen before and after MIST conditioning. Measurement of specimen's BRD was performed in accordance with procedure explained in section 3.4.1 of this thesis.

$$swell\ (\%) = \left(\frac{BRD_{initial} - BRD_{after\ MIST}}{BRD_{initial}}\right) \times 100 \qquad \textbf{4-6}$$

The results of average swelling for each mixture are shown in Figure 4-34 and Figure 4-35. In general, it was observed that Evotherm 3G and Rediset LQ effectively reduced swelling by 23% to 40% for all mixtures regardless of the aggregate type and asphalt binder grade. However, Sonnewarmix was observed to cause increased swelling for all mixtures. To further verify the observed trends, statistical t-tests assuming equal variances were performed for asphalt mixture pairs in the respective aggregate and binder type as presented in Table 4-8 and Table 4-9. Results presented in these tables verify that Evotherm 3G. and Rediset LQ. significantly reduced swelling.

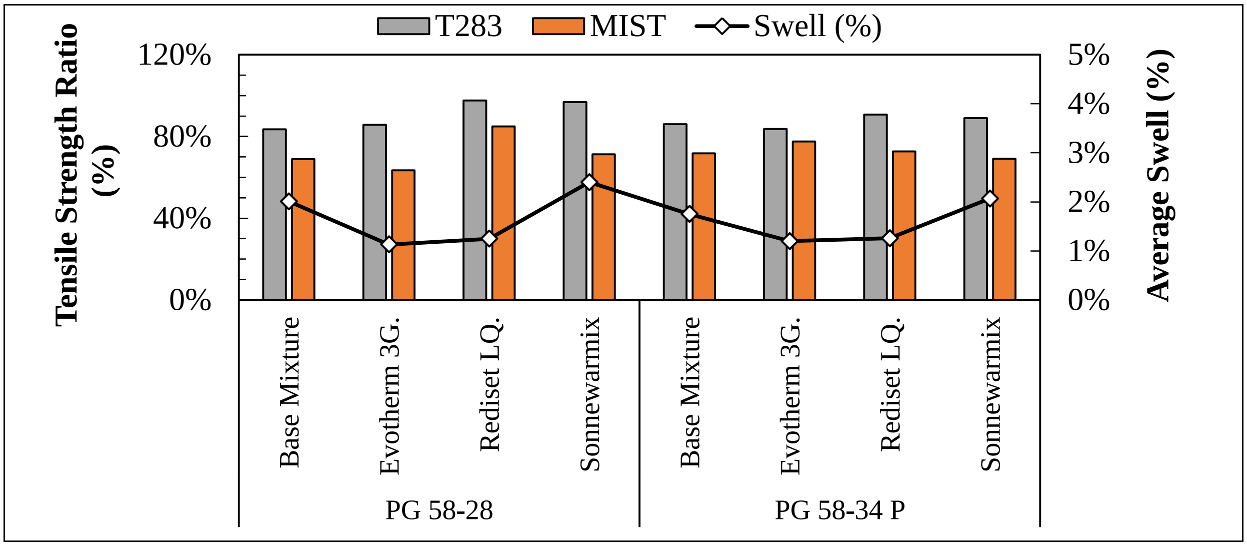


Figure 4-34 Effect of Warm Mix Additives on Volumetric Properties of Mixtures Containing Type A Aggregate (Trap Rock Diabase)

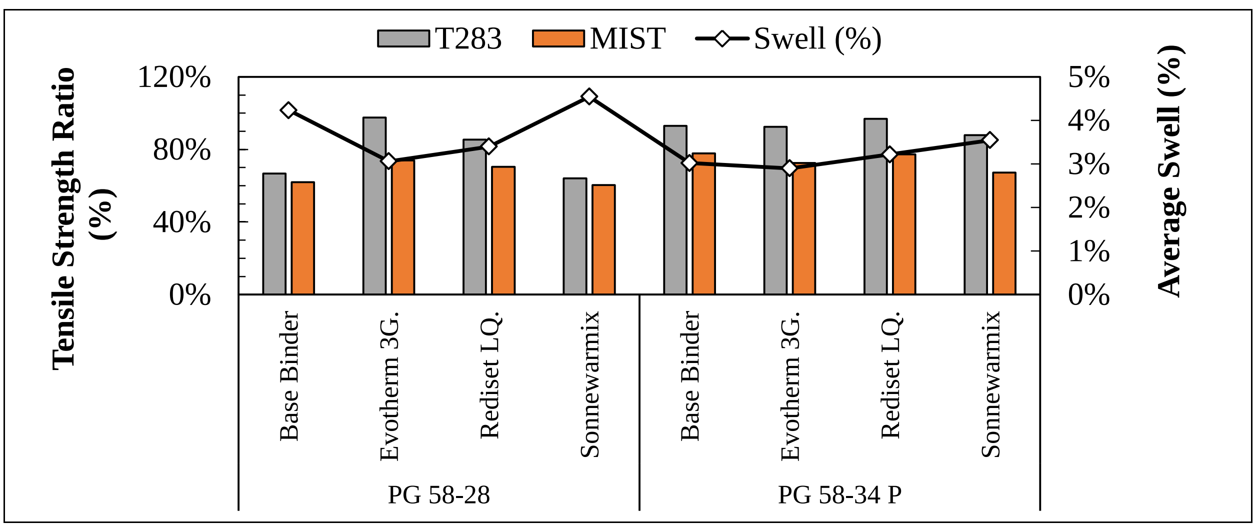


Figure 4-35 Effect of Warm Mix Additives on Volumetric Properties of Mixtures Containing Type B Aggregate (Pink Granite)

Table 4-8 Paired T-Test on Swelling of Mixtures Containing Type A Aggregate (Trap Rock Diabase)

<table>
<tr><th colspan="2">Paired Mixes</th><th>P-Value*</th><th>Statistically Significant</th></tr>
<tr><td rowspan="3">PG 58-28</td><td>PG 58-28 + SonneWarmix</td><td>0.002</td><td rowspan="5">YES</td></tr>
<tr><td>PG 58-28 + Evotherm 3G.</td><td>0.000</td></tr>
<tr><td>PG 58-28 + Rediset LQ.</td><td>0.000</td></tr>
<tr><td>PG 58-28 + SonneWarmix</td><td>PG 58-28 + Evotherm 3G.</td><td>0.000</td></tr>
<tr><td>PG 58-28 + SonneWarmix</td><td>PG 58-28 + Rediset LQ.</td><td>0.000</td></tr>
<tr><td>PG 58-28 + Evotherm 3G.</td><td>PG 58-28 + Rediset LQ.</td><td>0.088</td><td>NO</td></tr>
<tr><td rowspan="3">PG 58-34P</td><td>PG 58-34P + SonneWarmix</td><td>0.001</td><td rowspan="6">YES</td></tr>
<tr><td>PG 58-34P + Evotherm 3G.</td><td>0.000</td></tr>
<tr><td>PG 58-34P + Rediset LQ.</td><td>0.000</td></tr>
<tr><td>PG 58-34P + SonneWarmix</td><td>PG 58-34P + Evotherm 3G.</td><td>0.000</td></tr>
<tr><td>PG 58-34P + SonneWarmix</td><td>PG 58-34P + Rediset LQ.</td><td>0.000</td></tr>
<tr><td>PG 58-34P + Evotherm 3G.</td><td>PG 58-34P + Rediset LQ.</td><td>0.037</td></tr>
</table>

Note: *P-Value is the probability of $|T_{observed}|>t_{critical}$ at significance level of 95% (α=0.05)

Table 4-9 Paired T-Test on Swelling of Mixtures Containing Type B Aggregate (Pink Granite)

<table>
<tr><th colspan="2">Paired Mixes</th><th>P-Value*</th><th>Statistically Significant</th></tr>
<tr><td rowspan="3">PG 58-28</td><td>PG 58-28 + SonneWarmix</td><td>0.002</td><td rowspan="7">YES</td></tr>
<tr><td>PG 58-28 + Evotherm 3G.</td><td>0.000</td></tr>
<tr><td>PG 58-28 + Rediset LQ.</td><td>0.000</td></tr>
<tr><td>PG 58-28 + SonneWarmix</td><td>PG 58-28 + Evotherm 3G.</td><td>0.000</td></tr>
<tr><td>PG 58-28 + SonneWarmix</td><td>PG 58-28 + Rediset LQ.</td><td>0.000</td></tr>
<tr><td>PG 58-28 + Evotherm 3G.</td><td>PG 58-28 + Rediset LQ.</td><td>0.000</td></tr>
<tr><td rowspan="3">PG 58-34P</td><td>PG 58-34P + SonneWarmix</td><td>0.000</td></tr>
<tr><td>PG 58-34P + Evotherm 3G.</td><td>0.096</td><td rowspan="2">NO</td></tr>
<tr><td>PG 58-34P + Rediset LQ.</td><td>0.675</td></tr>
<tr><td>PG 58-34P + SonneWarmix</td><td>PG 58-34P + Evotherm 3G.</td><td>0.000</td><td>YES</td></tr>
<tr><td>PG 58-34P + SonneWarmix</td><td>PG 58-34P + Rediset LQ.</td><td>0.496</td><td rowspan="2">NO</td></tr>
<tr><td>PG 58-34P + Evotherm 3G.</td><td>PG 58-34P + Rediset LQ.</td><td>0.509</td></tr>
</table>

Note: *P-Value is the probability of $|T_{observed}|>t_{critical}$ at significance level of 95% (α=0.05)

Figure 4-36 shows a severe swelling observed after MIST conditioning of a mixture containing a combination of Sonnewarmix and PG 58-28 with aggregate Type B (pink granite). Such severe swelling was further reduced by using PaveBond liquid anti-stripping agent by 16% as shown in Figure 4-37. Similar trend was also observed for the combination of Type B moisture susceptible aggregate source and PG 58-28, which swelling in specimen volume was reduced by 19% after using anti-stripping agent (Figure 4-37).

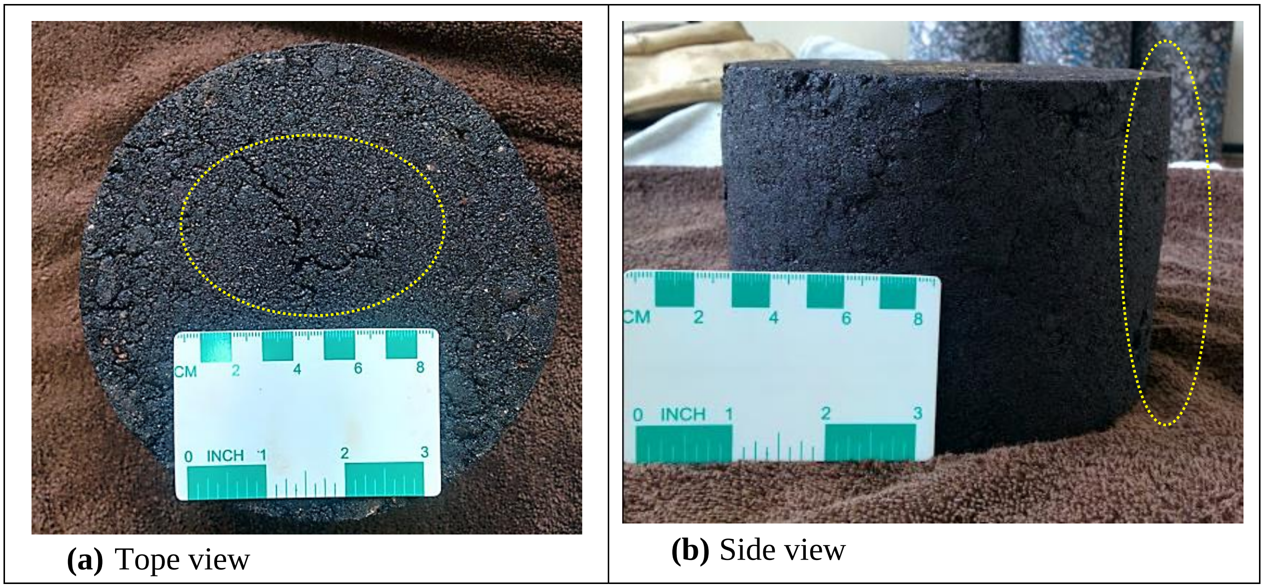


Figure 4-36 Severe Specimen Swelling After MIST Conditioning

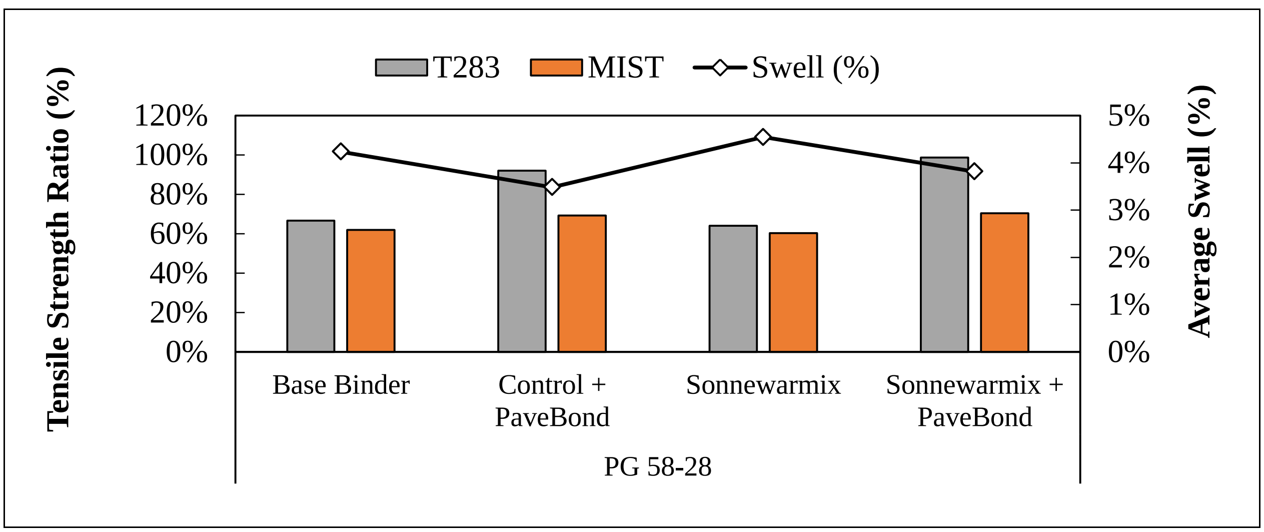


Figure 4-37 Effect of Liquid Anti-Stripping on Volumetric Properties of Mixtures Containing Type B Aggregate (Pink Granite)

4.4.3. Dynamic Modulus Ratio

Despite the wide acceptance of T283 within the industry, several studies have reported shortcomings of the test method. One of the major shortcomings of this test is the lack of ability to predict moisture susceptibility with reasonable confidence as TSR does not always correlate closely with the field performance. Furthermore, the TSR is sensitive to minor changes in conditioning temperatures (freeze and thaw), level of air voids, saturation level, specimen size, and aggregate orientation. This may make the results biased and unreliable, as suggested by Bahia and Ahmad (Bahia & Ahmad, 1999). It is reported by Kandhal and Rickards (Kandhal P. a., 2002) that Indirect Tensile Strength (IDT) apparatus used in the test does not accurately simulate the pumping action of traffic load. Finally, the T283 is mainly used as a criterion and index for strength retention, which does not provide useful information in long-term performance and deterioration modelling.

Recognizing the aforementioned shortcoming and limitations with the T283, dynamic modulus testing has been evaluated by number of researchers. These studies suggest that the dynamic modulus test may potentially be a better test to predict field moisture susceptibility. Furthermore, dynamic modulus is a material input in AASHTO's MEPDG software.

To assess the level of resistance to moisture damage, the percent of |E*| retained after one freeze-thaw cycle was calculated for values obtained at testing temperature of 21.1ºC and frequencies of 10, 1 and 0.1 Hz. This temperature was similar to the IDT testing temperature of 25ºC. Also, this temperature was selected to produce less variability in results compared to other temperatures such as 37 ºC or 54 ºC. For this reason, only results of 21ºC were included in the work. Figure 4-38 and Figure 4-39 present the dynamic modulus results conditioned specimens containing Type A trap rock aggregate and different additive types. Dynamic modulus results of mixtures containing Type B are shown in Figure 4-40 through Figure 4-42. In all figures, error bars represent two standard deviation from the average value of two replicates tested, with Dynamic Modulus Ratio (DMR) results shown above the bars of each conditioning protocol.

In general, the trend previously observed in indirect tensile strength values is also shown that the addition of warm mix additives resulted in lower |E*| of dry mixtures, except when SonneWarmix additive was used with PG 58-28 base binder and Type A trap rock aggregate. DMR values obtained after subjecting dynamic modulus specimens to one freeze-thaw cycle demonstrate that dynamic modulus test was able to effectively differentiate the level of resistance to moisture damage. For instance, DMR determined for conventional HMA mixtures containing Type A trap rock were higher than those values obtained for mixtures containing Type B granite aggregate. Furthermore, DMR was able to capture the effect of warm mix additives on improving the level of resistance to moisture susceptibility.

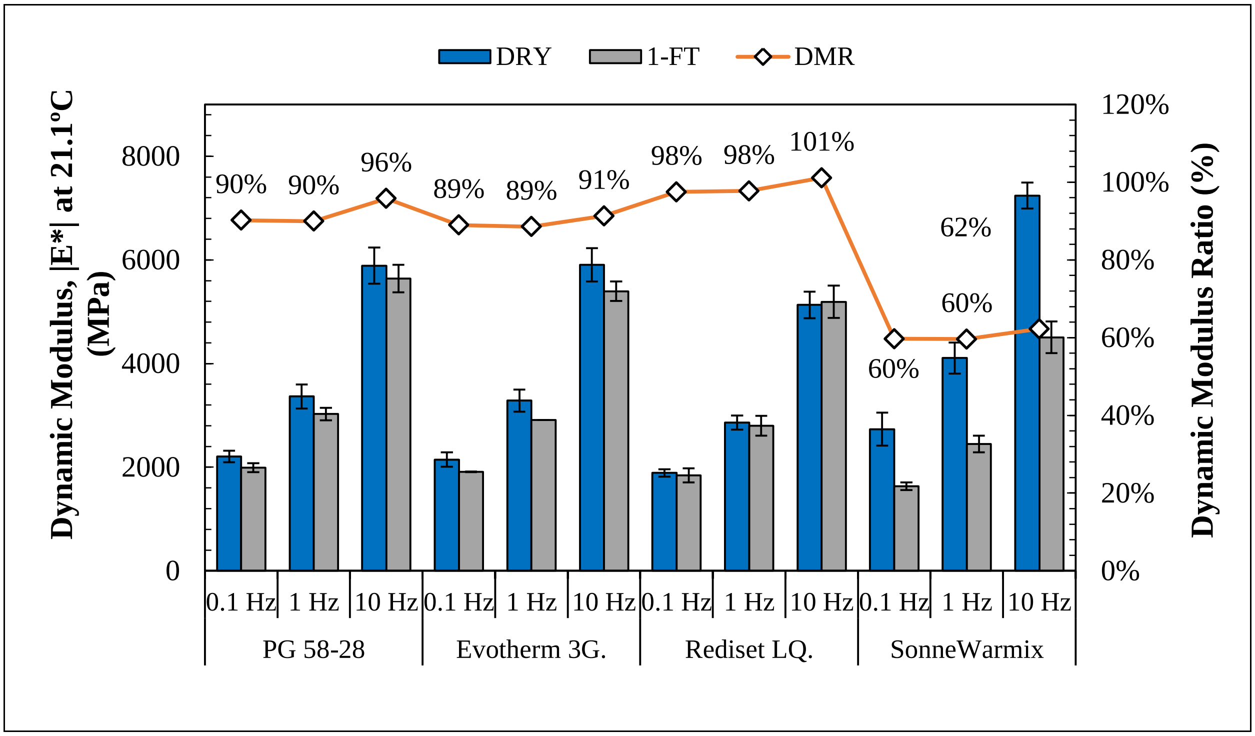


Figure 4-38 Effect Of Warm Mix Additives on |E*| of Mixtures Containing Type A Aggregate (Trap Rock Diabase) and PG 58-28 Base Binder

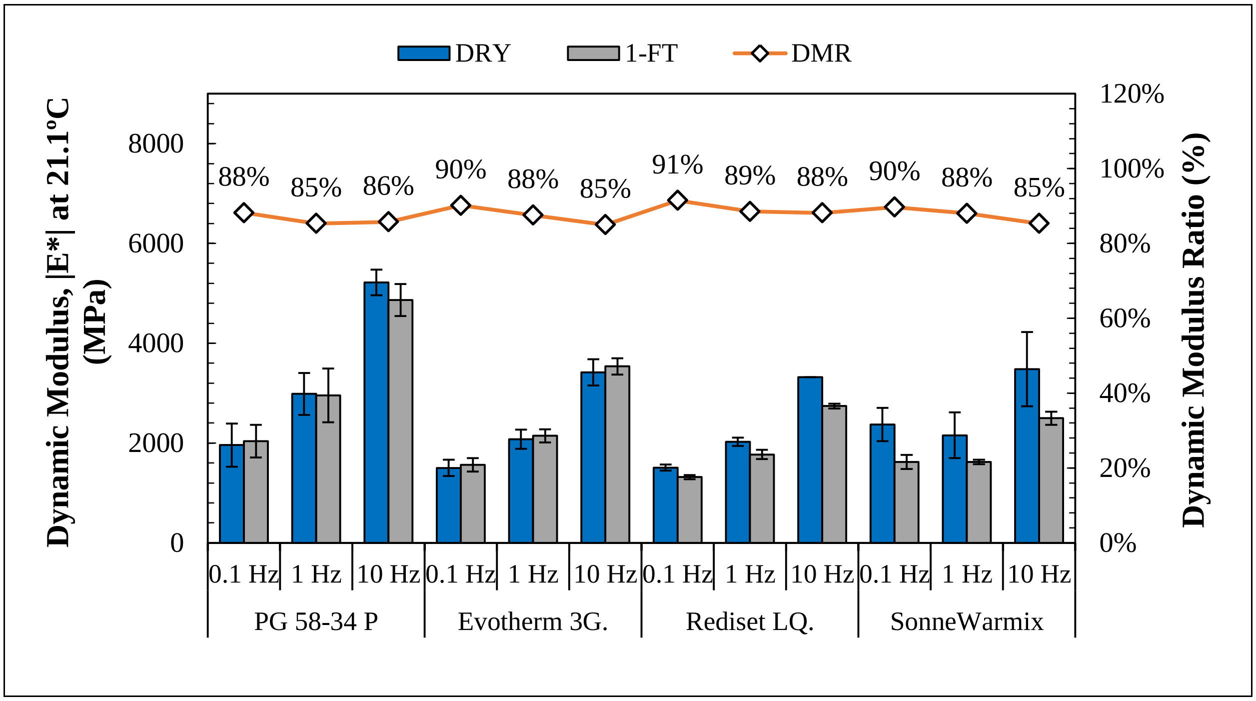


Figure 4-39 Effect Of Warm Mix Additives on |E*| of Mixtures Containing Type A Aggregate (Trap Rock Diabase) And PG 58-34P Base Binder

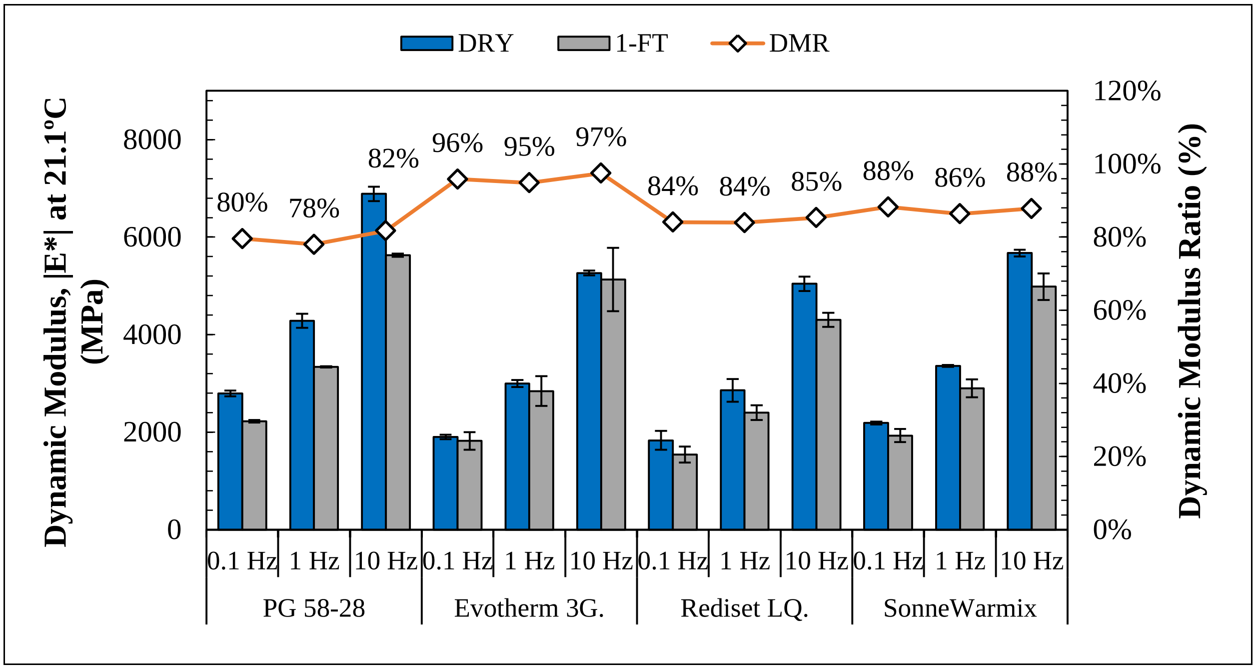


Figure 4-40 Effect Of Warm Mix Additives on |E*| of Mixtures Containing Type B Aggregate (Pink Granite) And PG 58-28 Base Binder

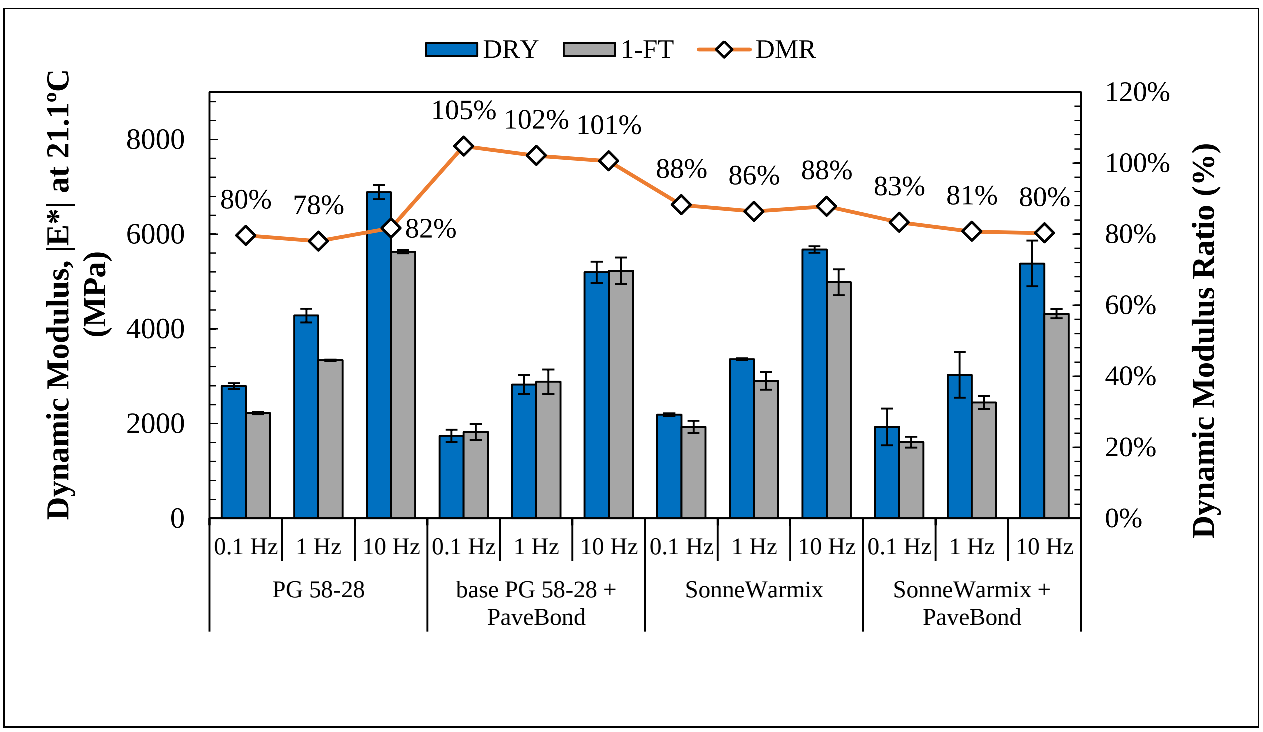


Figure 4-41 Effect Of Liquid Anti-Stripping Agent on |E*| of Mixtures Containing Type B Aggregate (Pink Granite) And PG 58-28 Base Binder

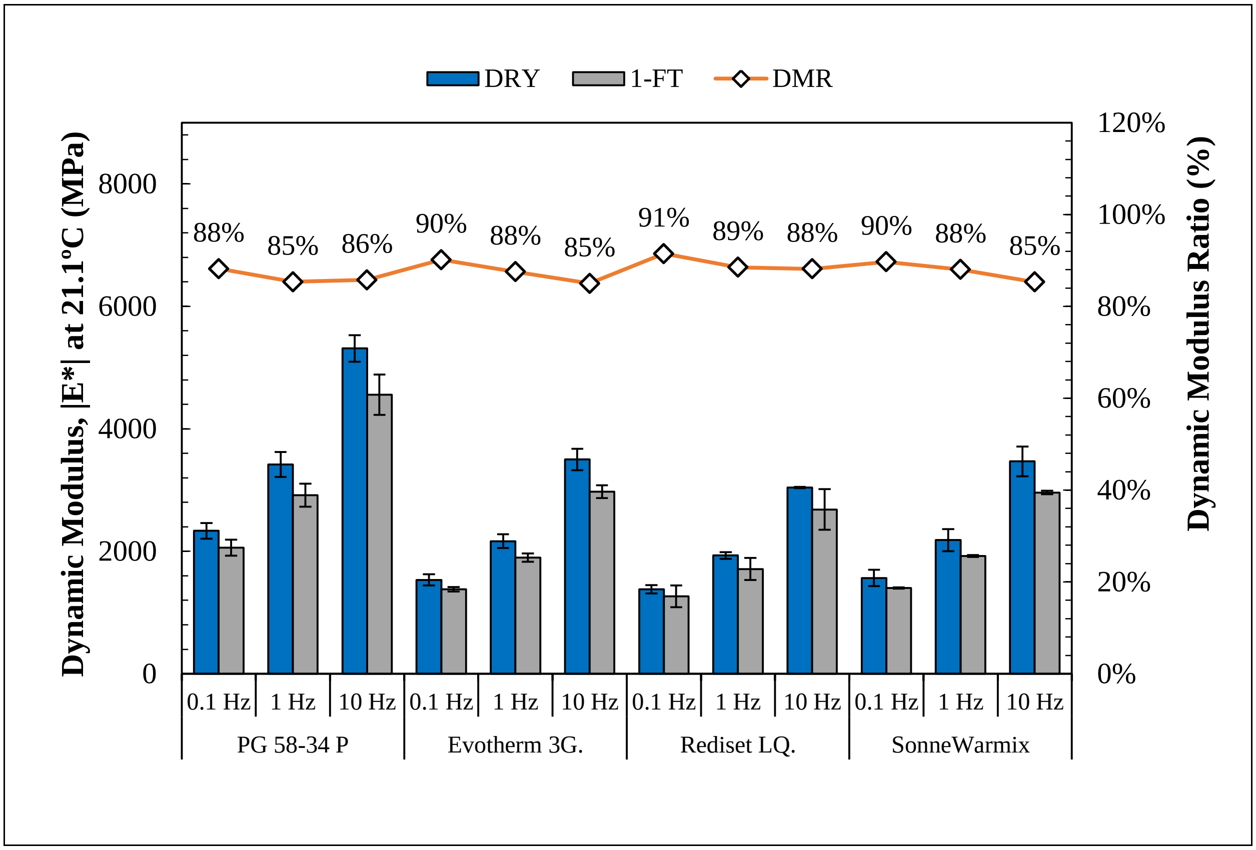


Figure 4-42 Effect Of Warm Mix Additives on |E*| Strength of Mixtures Containing Type B Aggregate (Pink Granite) And PG 58-34P Base Binder

4.4.4. Ultrasonic Pulse Ratio

Ultrasonic Pulse (UP) method was employed to evaluate if there is any benefit associated with use of WMA in reducing development of micro cracks and defects and improving overall durability of asphalt mixtures. For this reason, the evaluation included measuring the time of travel of an ultrasonic waveform passed through dynamic modulus specimen, as explained in details in Figure 4-43.

Similar to TSR, moisture sensitivity of compacted mixtures was quantified as the ratio of Peak to Peak (PTP) amplitude loss during the freeze-thaw over the dry PTP. However, to better evaluate the effect evaluate PTP loss, Figure 4-44 and Figure 4-45 show normalized PTP loss ratio with respect to those values obtained for the control mixtures. Error bars represent two standard deviation from the average value of two replicates tested.

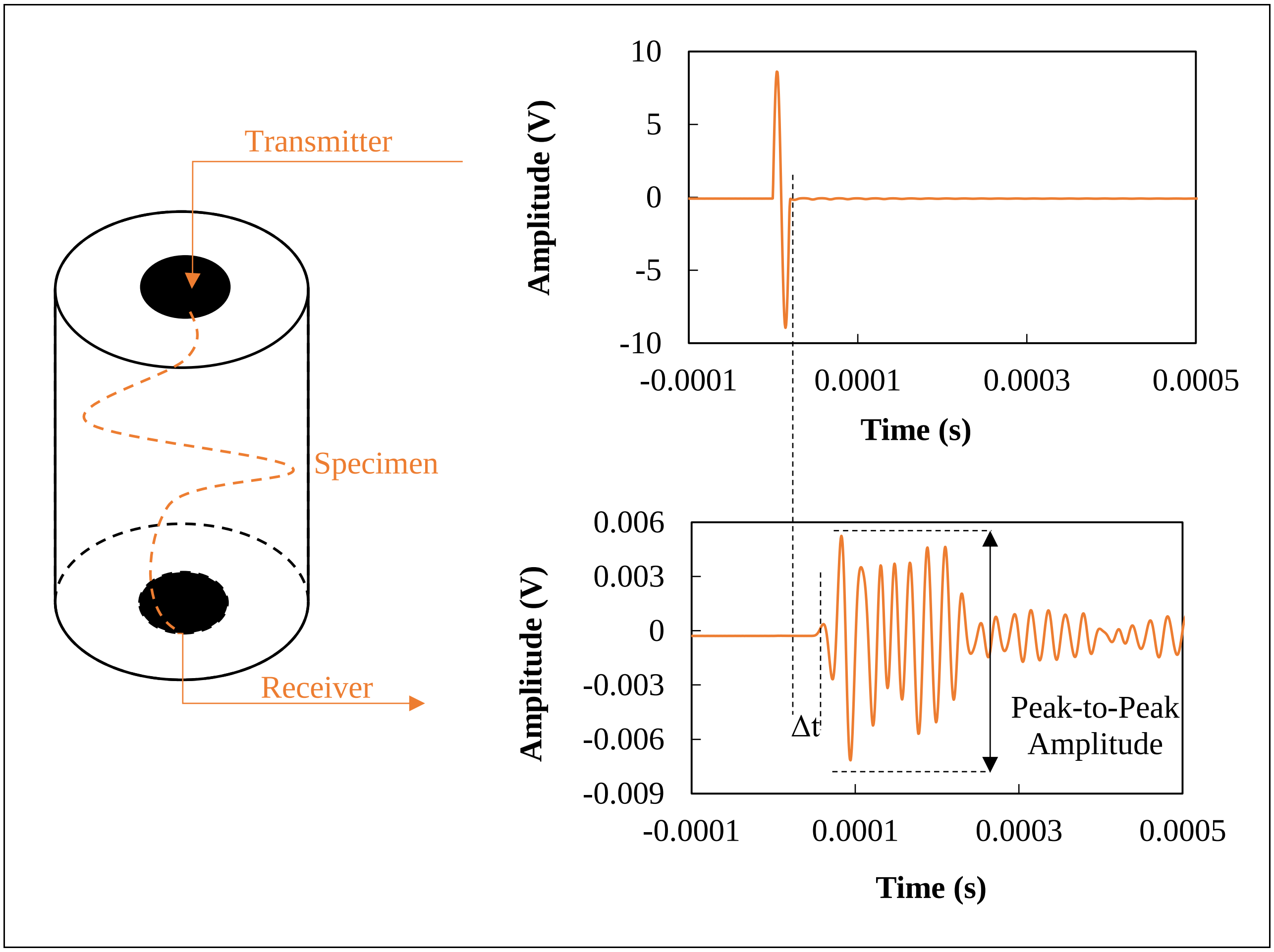
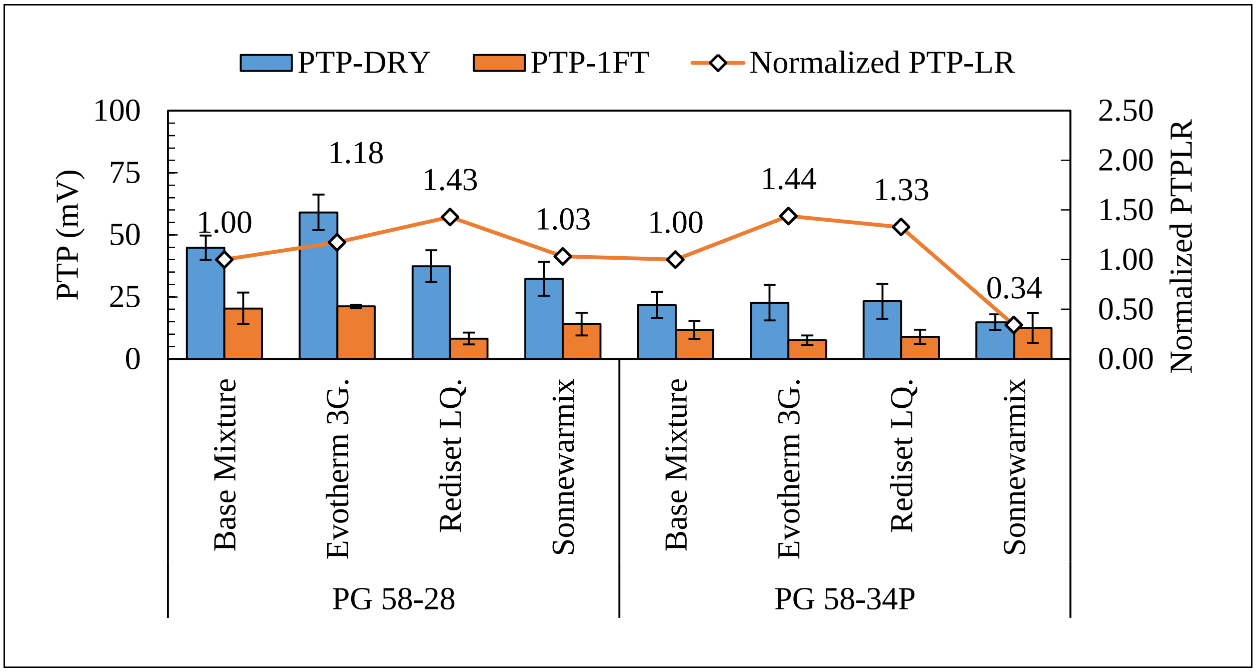


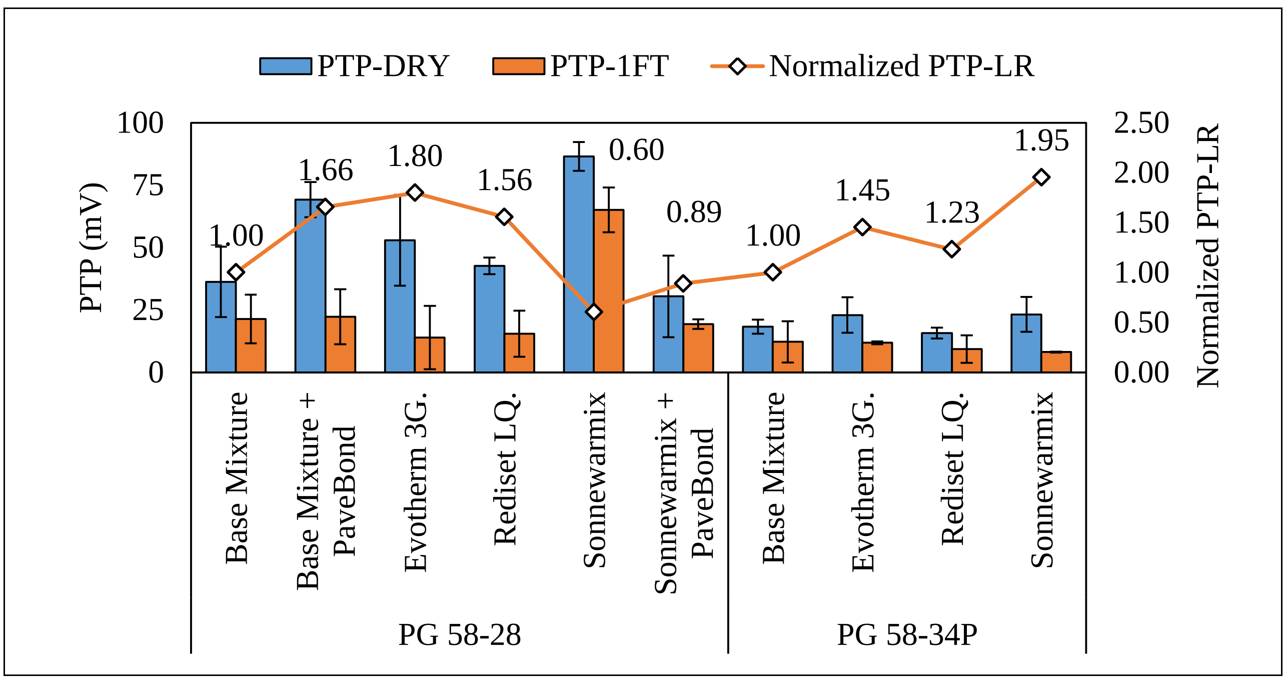
Figure 4-43 Ultrasonic Pulse Testing Schematic and Parameters

Increase in normalized-PTPLR is an indication that ultrasonic wave passed through the specimen was less interrupted by micro cracks developed inside the specimen. By considering this statement, it was observed that in general warm mix additives improved the resistance of the mixture to the development and propagation of the internal micro-cracking, except when Sonnewarmix was added to the PG 58-28 base binder regardless of the aggregate type. However, addition of Sonnewarmix to the combination of PG 58-34P and aggregate Type B improved the resistance of the mixture to development of internal cracking.



Note: PTP is Peak-to-Peak and PTPLR is Peak-to-Peak Loss Ratio

Figure 4-44 Effect of Warm Mix Additives on Internal Micro Crack Development of Mixtures Containing Type A Aggregate (Trap Rock)



Note: PTP is Peak-to-Peak and PTPLR is Peak-to-Peak Loss Ratio

Figure 4-45 Effect of Warm Mix Additives on Internal Micro Crack Development of Mixtures Containing Type B Aggregate (Pink Granite)

4.4.5. Summary of Moisture Susceptibility Testing

One of the main objectives of this research was to investigate the effect of different warm mix additives on the strength of asphalt mixtures in Ontario; with focus on moisture susceptibility. For a better systematic evaluation, test results presented from each test in pervious sections of this thesis were ranked in ascending order as listed in Table 4-10 for each combination of aggregate and binder type: first for the best performance and last for the weakest performance. Then, for each mixture, a total rank was calculated by adding ranks from each test as listed in Table 4-11.

In addition to providing aid in lowering production and placement temperatures of asphalt mixtures, rankings provided in Table 4-11 confirm that Evotherm 3G. and Rediset LQ. warm mix additives have anti-stripping properties and can be effectively used to improve the moisture resistance of mixtures containing aggregate types and binder grades studied in this thesis. However, Sonnewarmix seem to not have such anti-stripping properties and, in fact, lowered the moisture resistance in most of the aggregate/binder combinations.

Table 4-10 Mixture Moisture Susceptibility Rankings

Agg. Type	Binder Grade	Additive Type	TSR T283 (%)	Rank	TSR MIST (%)	Rank	MIST Swell (%)	Rank	DMR (%)	Rank	PTP-LR	Rank
Trap Rock	PG 58-28	Control	84	2	69	3	2.1	3	92	2	1.00	3
		Evotherm 3G.	85	1	85	1	1.13	1	90	3	1.18	2
		Rediset LQ.	71	3	71	2	1.25	2	99	1	1.43	1
		SonneWarmix	63	4	63	4	2.40	4	61	4	1.03	3
	PG 58-34P	Control	86	3	72	3	1.75	3	86	3	1.00	3
		Evotherm 3G.	91	1	73	2	1.20	1	88	2	1.44	1
		Rediset LQ.	89	2	69	4	1.26	2	89	1	1.33	2
		SonneWarmix	84	4	78	1	2.70	4	88	2	0.34	4
Pink Granite	PG 58-28	Control	67	3	62	3	4.23	3	80	4	1.00	3
		Evotherm 3G.	98	1	74	1	3.06	1	96	1	1.80	1
		Rediset LQ.	85	2	70	2	3.40	2	84	3	1.56	2
		SonneWarmix	64	4	60	4	4.55	4	87	2	0.60	4
	PG 58-34P	Control	93	2	78	1	3.20	3	86	3	1.00	4
		Evotherm 3G.	92	3	73	3	2.90	1	88	2	1.45	2
		Rediset LQ.	97	1	77	2	3.22	2	89	1	1.23	3
		SonneWarmix	88	4	67	4	3.55	4	88	2	1.95	1

Note: Agg. = aggregate, TSR is Tensile Strength Ratio, MIST is Moisture Induced Stress Tester, DMR is Dynamic Modulus Ratio, and PTP-LR is Peak-to-Peak Loss Ratio

Table 4-11 Overall Moisture Susceptibility Ranking

Aggregate Type	Binder Grade	Additive Type	Moisture Resistance Rank
Trap Rock	PG 58-28	Control	13
		Evotherm 3G.	8
		Rediset LQ.	9
		SonneWarmix	19
	PG 58-34P	Control	15
		Evotherm 3G.	7
		Rediset LQ.	11
		SonneWarmix	15
Pink Granite	PG 58-28	Control	16
		Evotherm 3G.	5
		Rediset LQ.	11
		SonneWarmix	18
	PG 58-34P	Control	13
		Evotherm 3G.	11
		Rediset LQ.	9
		SonneWarmix	15

Note: Lower the ranking, better the anticipated resistance to moisture damage.

4.5. Long-Term Performance Prediction

The AASHTOWare's Mechanistic-Empirical (M-E) Software was used to investigate the effect of warm mix additives and moisture conditioning on the long-term performance of a pavement structure designed for Ontario climate and traffic conditions with WMA used as surface course. For this purpose, a level 1 design was performed for the surface course by using dynamic modulus results and results of asphalt binder testing. All other inputs were retrieved from the Ministry of Transportation Ontario recommended inputs (MTO, 2012) and were maintained constant in all the designs for other layers.

An as-built pavement structure near London, Ontario, was selected from the MTO's asset management system. This structure was selected to presents pavement structures that could be routinely surface or re-resurfaced with WMA mixtures. The pavement structure comprised of a total asphalt thickness of 170 mm, underlain by 200 mm of Granular Base and 150 mm of Granular Subbase. The asphalt layer was separated into two layers with a 40 mm surface course, 130 mm binder course. The subgrade soils were clayey silt with the resilient modulus of 36 MPa. The granular base and subbase materials were comprised of crushed stones with as-built material properties given in Table 4-12.

Table 4-12 Granular Base and Subbase Inputs

Property		Granular Base Type A[1]	Granular Subbase Type B-I[1]
	Sieve Size (mm)		
Gradation (% Passing)	25.0	100	75.0
	19.0	92.5	-
	9.5	61.5	-
	4.75	45.0	60.0
	1.18	27.5	-
	0.300	13.5	33.5
	0.075	5.0	4.0
Maximum dry unit weight (kgf/m^3)		2038.2	2012.4
Liquid Limit (LL)		6.0	11.0
Plasticity Index (PI)		0.0	0.0
Modulus (MPa)		250.0	150.0

Note: [1]These are types of granular materials that are commonly specified and used throughout the province of Ontario

4.5.1. Traffic Inputs

Site specific traffic data (level 1) were retrieved from the MTO's web-based mapping program referred to as *iCorridor* (MTO, 2016). Traffic data included as listed in Table 4-13: (1) two-way Annual Average Daily Truck Traffic (AADTT), (2) operational vehicle speed, (3) Federal Highway Administration (FHWA) vehicle class distribution, (4) number of axle per truck, and (5) axle load distribution. It should be noted that other vehicle classes (e.g. light passenger vehicles) were ignored in the analysis. The traffic inputs resulted in 2.91 million Equivalent Single Axle Loads (ESALs) over 20 years of design life.

4.5.2. Climate Inputs

In the Pavement M-E software, climate inputs are incorporated into the performance prediction analysis through the Enhanced Integrated Climatic Model (EICM) which is a product of a computer program that calculates the effect of changes in moisture within the pavement materials over time and depth combined with the effect of freezing and thawing on the pavement response. Inputs such as latitude, longitude, elevation, and depth of water table for a selected weather station are required in order to generate climatic file. For the analysis, closest station to the site location was selected to generate the climate conditions presented in Table 4-14. Figure 4-46 shows the approximately of selected weather station to the site location.

Table 4-13 Warm Mix Asphalt Long-Term Performance Prediction Traffic Inputs (MTO, 2016)

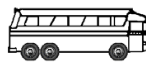
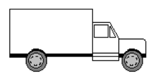
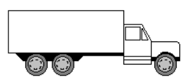
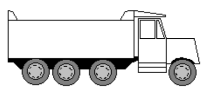
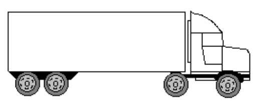
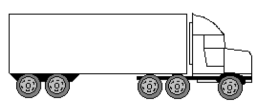
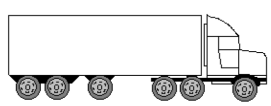
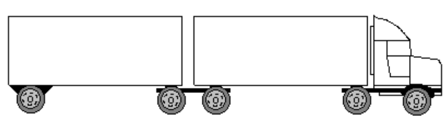
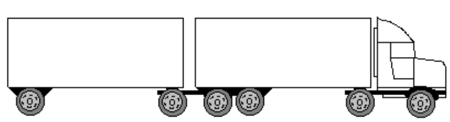
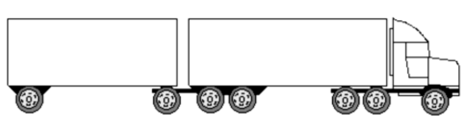
FHWA Vehicle Class	Commercial Vehicle		Truck Flow Volume	Truck Flow Percentage
4		Two or Three Axle Buses	8	1.5
5		Two-Axle, Six-Tire, Single Unit Trucks	72	13.8
6		Three-Axle Single Unit Trucks	49	9.5
7		Four or More Axle Single Unit Trucks	5	0.9
8		Four or Less Axle Single Trailer Trucks	19	3.7
9		Five-Axle Single Trailer Trucks	227	43.6
10		Six or More Axle Single Trailer Trucks	121	23.3
11		Five or Less Axle Multi-Trailer Trucks	0	0.0
12		Six-Axle Multi-Trailer Trucks	0	0.0
13		Seven or More Axle Multi-Trailer Trucks	19	3.7

Table 4-14 Warm Mix Asphalt Long-Term Performance Prediction Annual Climate Inputs

Parameter	Annual Statistics
Mean annual air temperature (°C)	7.7
Mean annual precipitation (mm)	1008
Freezing index (°C – days)	1009.4
Average Number of freeze-thaw cycles	67.3

Figure 4-46 Approximate Location Of Weather Station To Site Selected For Warm Mix Asphalt Surfacing

4.5.3. Terminal Service Levels

In the MEPDG, structural adequacy is evaluated by the ability of a design to meet sets of targeted threshold (also known as Terminal Service Levels). For this study following thresholds at 90% reliability were retrieved from the Ministry of Transportation Ontario recommended inputs (MTO, 2012).

Table 4-15 Warm Mix Asphalt Long-Term Performance Distress Prediction Target Values

Performance Criteria	Targeted at 90% reliability
Permanent Deformation - Total Pavement (mm)	19.00
AC Bottom-up Fatigue Cracking (%)	25.00
AC Thermal Fracture (m/km)	189.4
AC Top-down Fatigue Cracking (m/km)	378.8
Permanent Deformation – AC only (mm)	6.00

4.5.4. Performance Predictions

Figure 4-47 to Figure 4-50 present the relative difference in predicted distresses for mixtures containing warm mix additives, as normalized by the predictions obtained for control HMA mixtures. It should be noted that the analysis was performed at 90 percent design reliability.

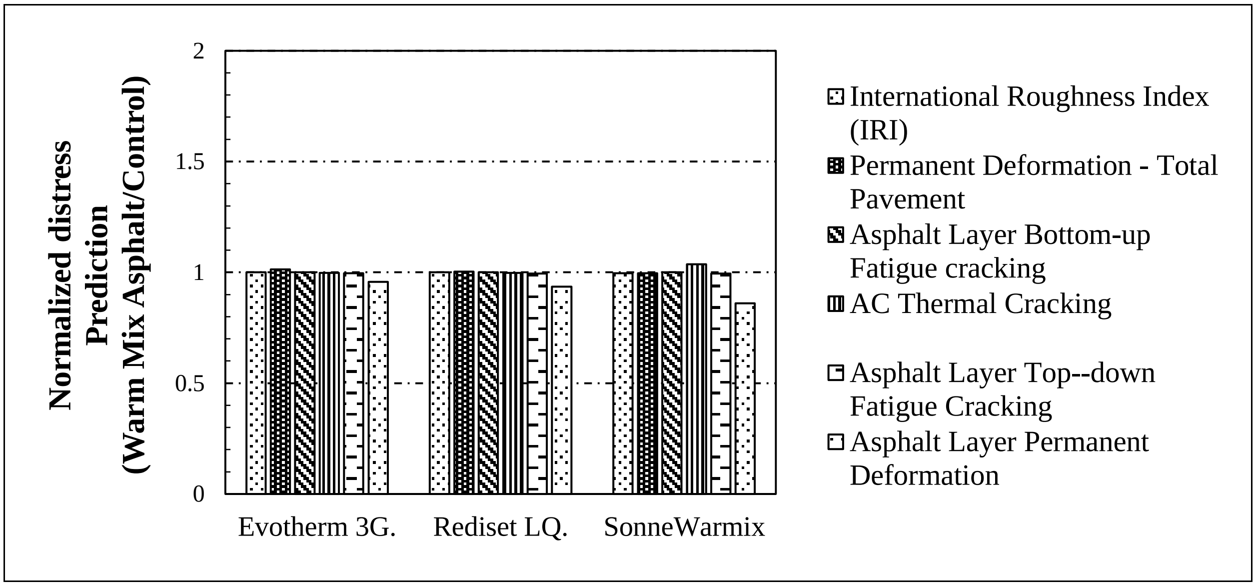


Figure 4-47 Effect Of Warm Mix Additives on Long-Term Performance Of Mixtures Containing Type A Aggregate (Trap Rock Diabase) And PG 58-28 Base Binder

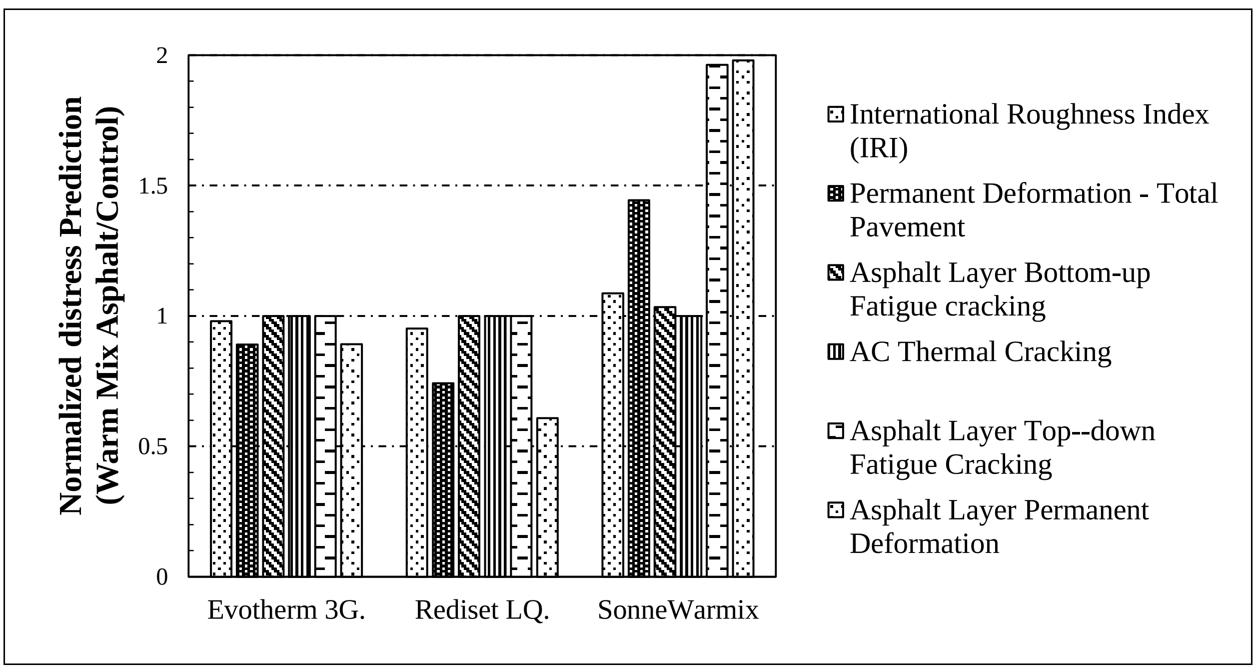


Figure 4-48 Effect of Warm Mix Additives on long-term performance of mixtures containing Type A aggregate (trap rock diabase) and PG 58-34P base binder

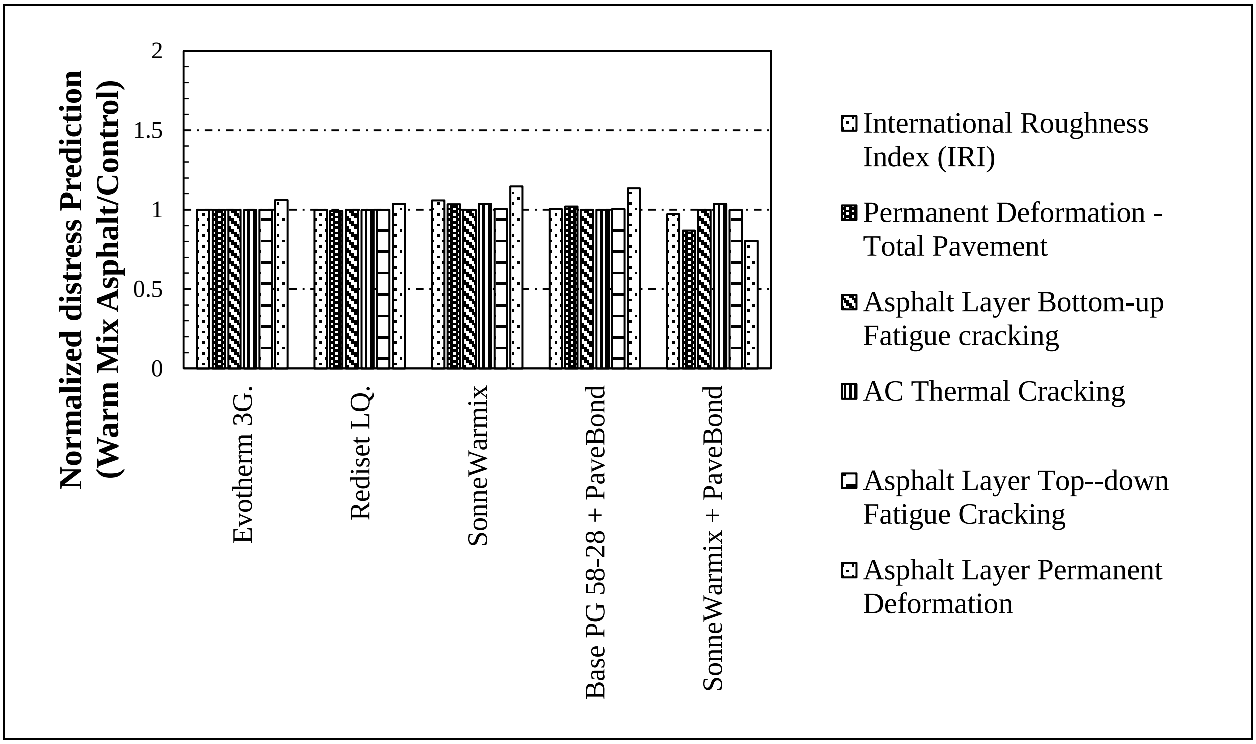


Figure 4-49 Effect of Warm Mix Additives on long-term performance of mixtures containing Type B aggregate (granite) and PG 58-28 base binder

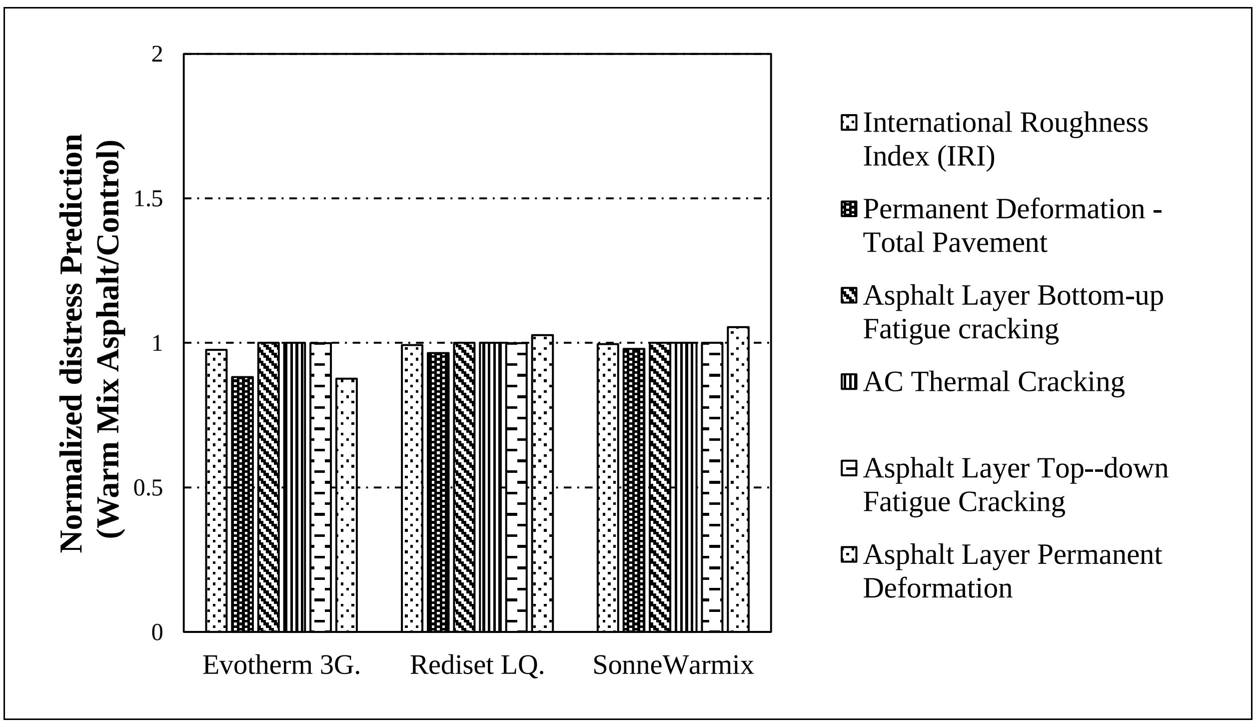


Figure 4-50 Effect of Warm Mix Additives on long-term performance of mixtures containing Type B aggregate (granite) and PG 58-34P base binder

Following trends can be observed from the MEPDG analysis:

1. In general, the MEPDG analysis indicates that warm mix additives can be used in asphalt mixtures as surface layers without affecting smoothness of the pavement and the level of resistance to fatigue cracking; except when Sonnewarmix was used in combination of asphalt grade PG 58-34P and Type A Trap Rock aggregate.
2. Usage of warm mix additives may decrease level of resistance to permanent deformation or rutting. Reduction in rutting resistance was expected as previously presented Hamburg rutting test and dynamic modulus results at higher temperatures indicated reduced level of strength for WMA mixtures.
3. The lowered rutting resistance may be improved by using warm mix additives in combination with polymer modified asphalt binder. Sonnewarmix additive, however, seemed to be exempted from this assumption.

One of the objectives of this research was to capture the long-term performance of a pavement structure designed for Ontario climate and traffic conditions with WMA used as surface course. For this purpose, previously presented relative difference in predicted distresses for mixtures containing warm mix additives were ranked in an ascending order for each combination of aggregate type and binder grade as listed in Table 4-16: first for the best performance and last for the weakest performance. Then, for each mixture, a total rank was calculated by adding ranks from each test referred to as “Long-Term Predicted Field Performance Rank”.

In addition to proven environmental and safety benefits of WMA, rankings provided in Table 4-16 suggest that Evotherm 3G and Rediset LQ can be effectively used in Ontario’s asphalt surface mixtures without affecting the long-term performance. In some cases, these additives were found to improve, in fact, performed better than HMA.

Table 4-16 WMA Long-Term Predicted Field Performance Rank

Aggregate Type	Binder Grade	Additive Type	International Roughness Index	Permanent Deformation Total Pavement	Permanent Deformation AC Layer	AC Layer Fatigue Bottom-up Cracking	AC Layer Fatigue Top-down	AC Layer Thermal Cracking	Long-Term Predicted Field Performance Rank
Trap Rock	PG 58-28	Control	1	1	4	1	1	1	9
		Evotherm 3G.	1	2	3	1	1	1	9
		Rediset LQ.	1	1	2	1	1	1	7
		SonneWarmix	1	1	1	1	1	2	7
	PG 58-34P	Control	3	3	3	1	1	1	12
		Evotherm 3G.	2	2	2	1	1	1	9
		Rediset LQ.	1	1	1	1	1	1	6
		SonneWarmix	4	4	4	2	2	1	17
Pink Granite	PG 58-28	Control	1	1	1	1	1	1	6
		Evotherm 3G.	1	1	3	1	1	1	8
		Rediset LQ.	1	1	2	1	1	1	7
		SonneWarmix	2	2	4	1	1	2	12
	PG 58-34P	Control	2	4	2	1	1	1	11
		Evotherm 3G.	1	1	1	1	1	1	6
		Rediset LQ.	2	2	3	1	1	1	10
		SonneWarmix	2	3	4	1	1	1	12

Note: Lower the ranking, better the anticipated long-term field performance.

4.6. Summary and Conclusions

In this chapter, a combination of qualitative and quantitative laboratory test methods, and the state-of-the-art long-term prediction software were employed to address questions and concerns that have been raised in regards to effect of warm mix additives on the overall long-term durability of surface mixtures designed for Ontario roads and highways. For this reason, asphalt mixtures containing two most commonly used grades of asphalt cement (PG 58-28 and PG 58-34P), three types of WMA additives (SonneWarmix, Rediset LQ, and Evotherm 3G), and two aggregate blend types (trap rock diabase and granite) were produced under controlled laboratory conditions at CPATT.

Results obtained from each proposed tests outlined in the methodology were then statistically analyzed and ranked in an ascending order; first for the best performance and last for the weakest performance. Then, for each mixture, an overall rank was calculated by adding ranks from each test, as listed in Table 4-17.

Table 4-17 WMA Overall Laboratory and Field Performance Ranking

Aggregate Type	Binder Type	Additive Type	Mixture Rutting[1]	Resistance to Fatigue Cracking[2]	Overall Strength[3]	Moisture Damage Resistance[4]	Long-Term Field Performance[5]	Overall Rank[6]
Trap Rock	PG 58-28	Control	2	3	7	3	9	24
		Evotherm 3G.	1	1	7	1	9	19
		Rediset LQ.	3	2	5	2	7	19
		SonneWarmix	4	4	9	4	7	28
	PG 58-34P	Control	1	4	9	3	12	29
		Evotherm 3G.	2	1	6	1	9	19
		Rediset LQ.	3	3	7	2	6	21
		SonneWarmix	4	2	8	3	17	34
Pink Granite	PG 58-28	Control	2	4	9	3	6	24
		Evotherm 3G.	3	3	7	1	8	22
		Rediset LQ.	1	1	6	2	7	17
		SonneWarmix	4	2	5	4	12	27
	PG 58-34P	Control	1	4	9	3	11	28
		Evotherm 3G.	3	3	8	2	6	22
		Rediset LQ.	2	1	6	1	10	20
		SonneWarmix	4	2	7	4	12	29

Note: [1]Rutting was evaluated by Hamburg rutting test, [2]Resistance to fatigue was evaluated by performing Indirect Tensile Strength at 25˚C, [3]Overall strength was evaluated by Dynamic Modulus test at different frequencies and testing temperatures (ranging from -10 to 54.4˚C), [4]Moisture damage resistance was evaluated through an array of quantitative and qualitative tests, and [5]Long-term performance was predicted by using "Level 1" MEPDG analysis. [6]Lower the ranking, better the overall laboratory and predicted field performance.

Chapter 5 Effect of Pigment on Mixture Performance

One of the main objectives of this research was to provide an assessment on the performance of the in-situ materials and its expected long-term behaviour. This research was collaboration with partners in the paving industry, with the intention of advancing knowledge of using coloured asphalt mixtures in Canada. To achieve this objective, materials collected during paving operations and materials produced under controlled laboratory conditions were systematically evaluated at CPATT to capture the impact of colouring pigment on the mixture's strength. Following sections provide results of this assessment.

5.1. Dynamic Modulus

The dynamic modulus testing a range of frequencies and temperatures was used in this research to capture the effects of simulated vehicle speeds and varying seasonal temperatures. For this test, asphalt mixtures were compacted in the CPATT laboratory, cored, and tested at six loading frequencies (0.1, 0.5, 1, 5, 10 and 25 Hz) and five different temperatures (-10, 4.4, 21.1, 37.8 and 54.4°C) to obtain |E*|. Measurements obtain from this test were further combined to obtain a master curves shown in Figure 5-1 in accordance with AASHTO PP62-09 procedure, "Standard Practice for Developing Modulus Master Curve for Hot-Mix Asphalt" (AASHTO, 2009). To better analyze the results, Master Curves were divided into three zones illustrating lower, intermediate and higher frequencies. Previous studies at CPATT demonstrated that different frequency ranges can be used to compare the potential for different types of distresses (Tighe & El-Hakim, 2010).

Development and comparison of the master curves was followed by a statistical analysis of dynamic modulus values to determine if any significant difference between the mixes. With the combination of five testing temperatures and six frequencies, 30 values of |E*| needed to be analyzed for each mixture. Comparing all these values was found intensive and rather not informative. For this reason, three |E*| values were selected from each mix to evaluate the stiffness at temperatures and frequencies that can be used as indicators of three main surface distresses: (1) thermal cracking at temperature of -10°C and high frequency of 25 Hz, (2) fatigue cracking at 21.1°C and intermediate frequency of 10 Hz, and (3) rutting at high temperature of 54.4°C and low frequency of 0.1 Hz. The statistical analysis was performed as presented in Table 5-1 using paired t-test method. The statistical software of Minitab© was used to conduct the analysis at a 95% significance level.

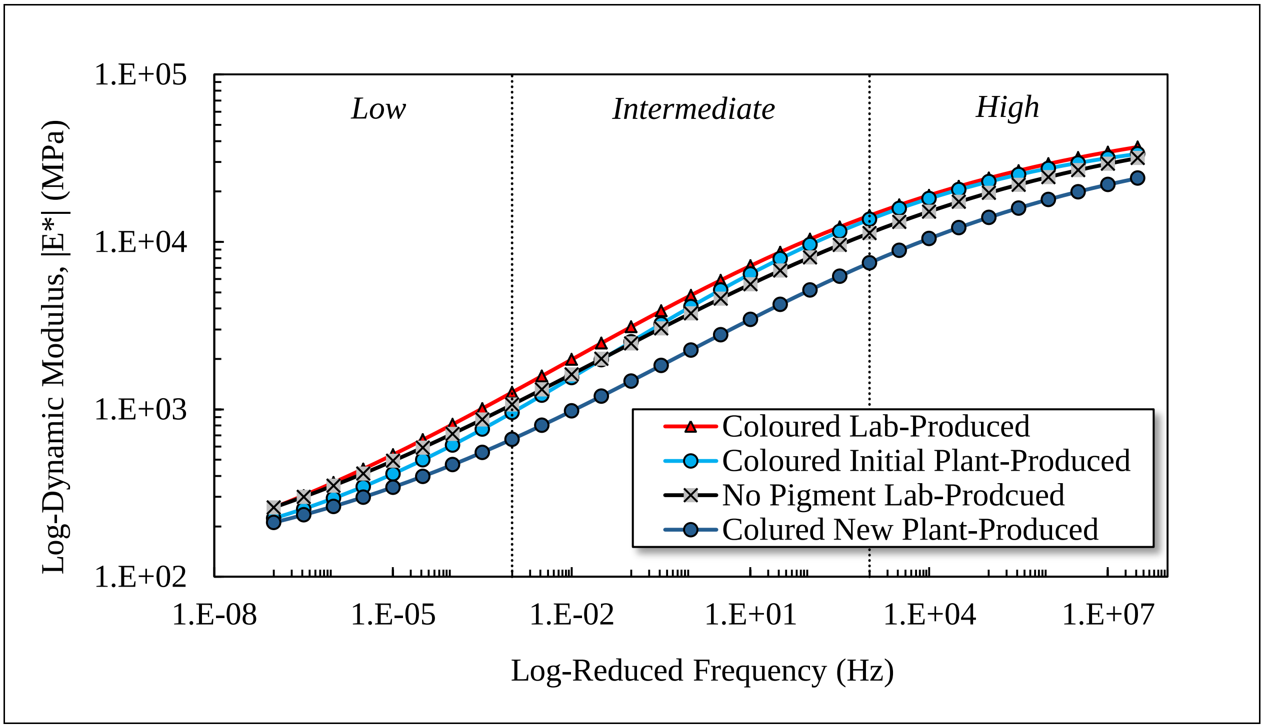


Figure 5-1 Master Curve Results for Coloured Hot Mix Asphalt Mixtures

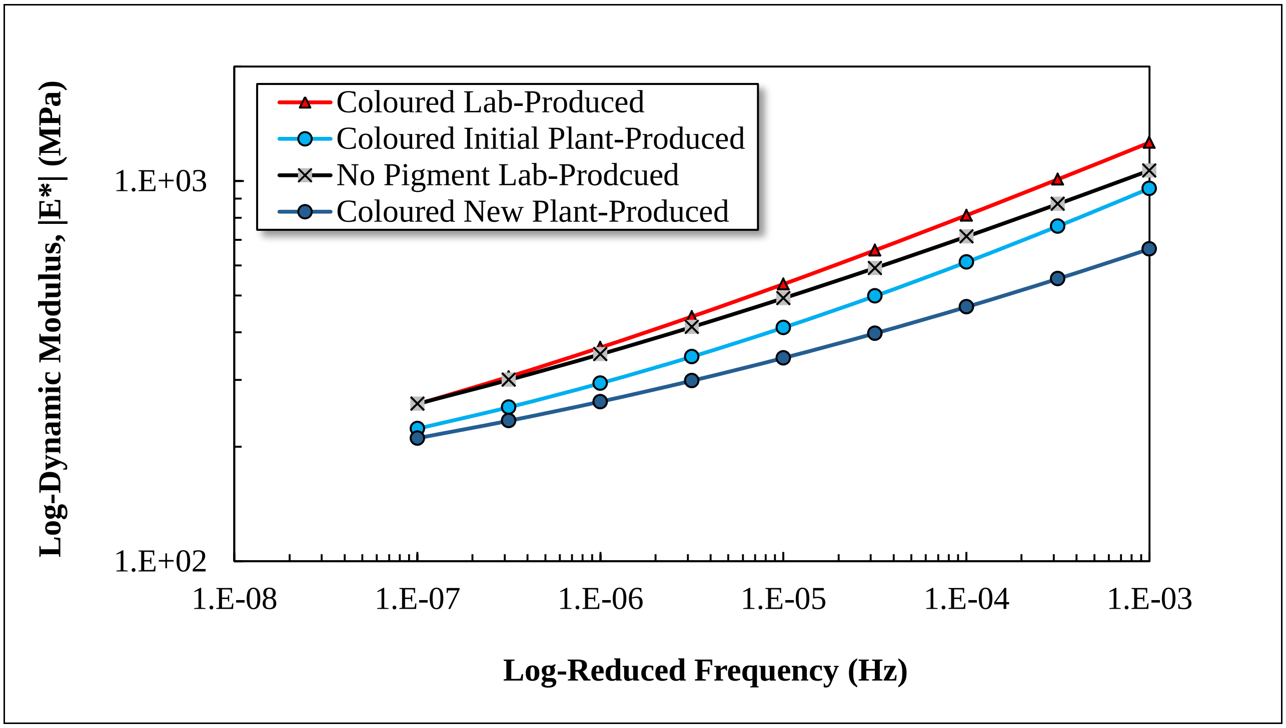


Figure 5-2 Low Frequency Zone Master Curve Results for Coloured Hot Mix Asphalt Mixtures

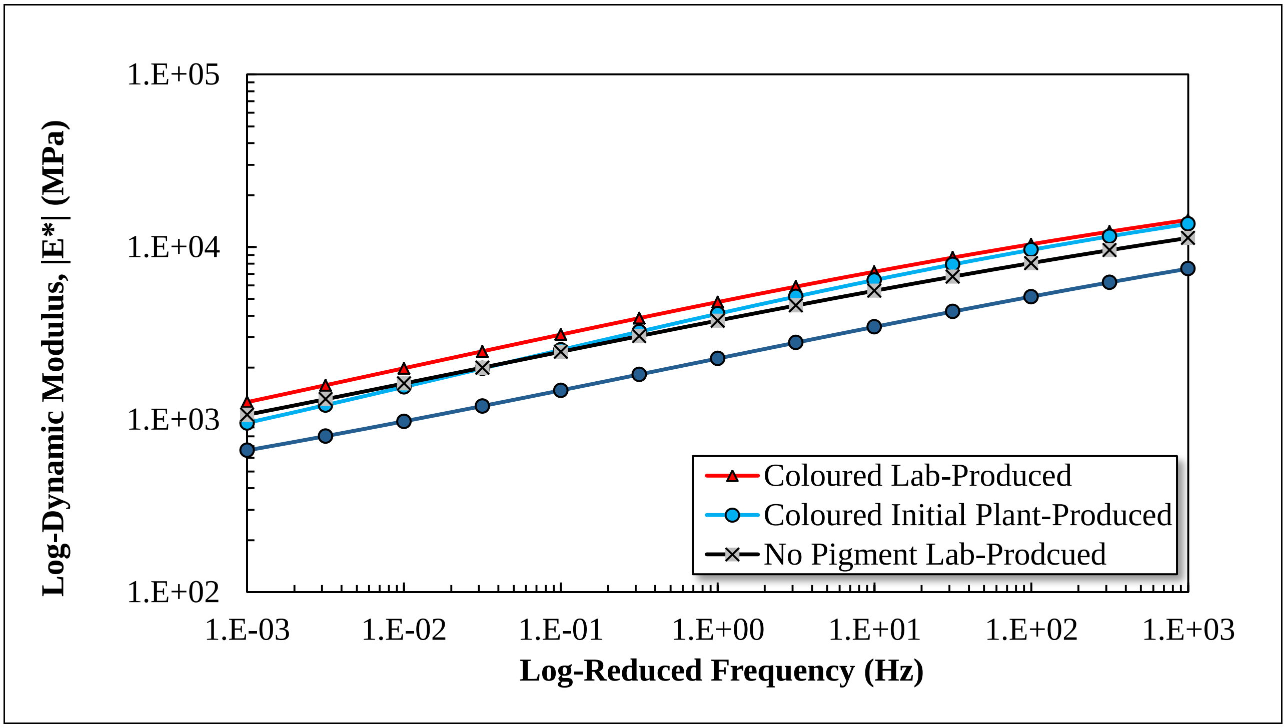


Figure 5-3 Intermediate Frequency Zone Master Curve Results for Coloured Hot Mix Asphalt Mixtures

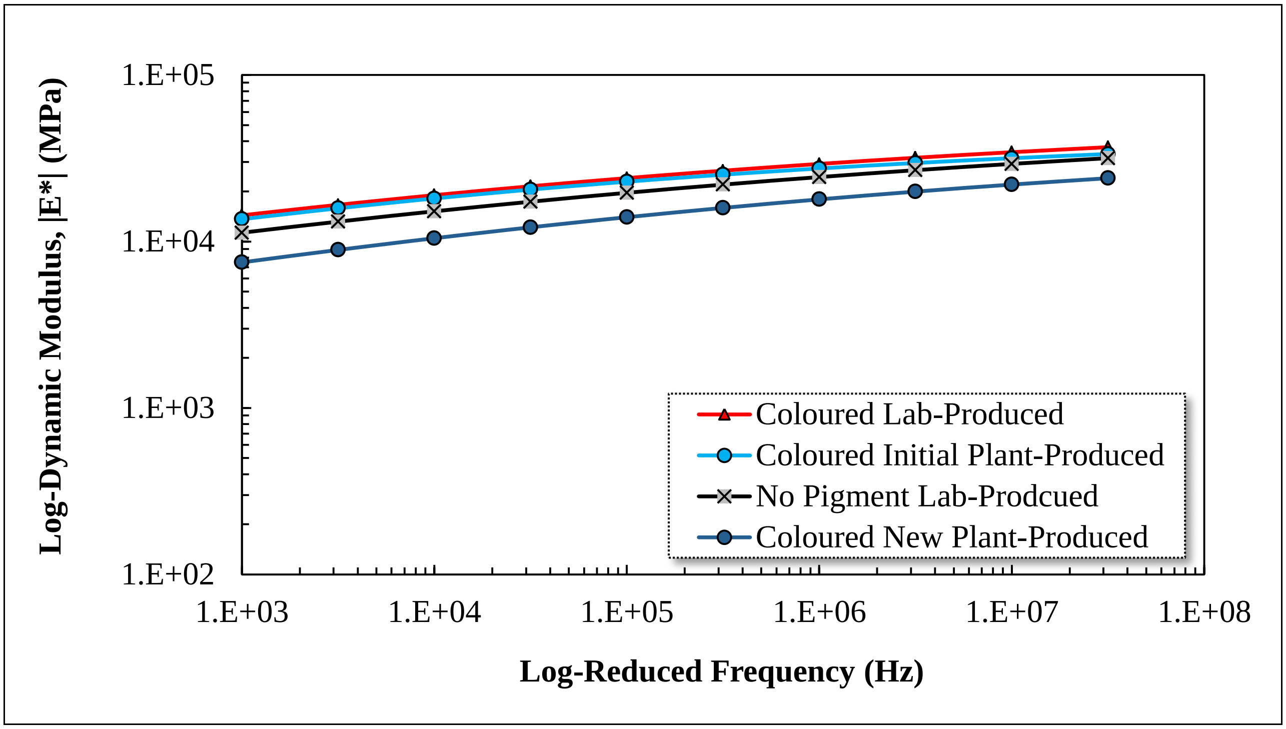


Figure 5-4 High Frequency Zone Master Curve Results for Coloured Hot Mix Asphalt Mixtures

Table 5-1 Statistical Analysis of CHMA Stiffness

Temperature (°C)	Frequency (Hz)	Paired Mixes		$\lvert T_{observed} \rvert$	$t_{critical}$	Significantly Different?
-10	25	CIPP	NPLP	3.81	2.57	**YES**
		CLP	NPLP	0.41		NO
		CIPP	CNPL	9.58		**YES**
		CIPP	CLP	2.8		**YES**
21.1	10	CIPP	NPLP	0.87		NO
		CLP	NPLP	0.15		NO
		CIPP	CNPL	9.73		**YES**
		CIPP	CLP	0.48		NO
54.4	0.1	CIPP	NPLP	1.74		NO
		CLP	NPLP	0.63		NO
		CIPP	CNPL	0.85		NO
		CIPP	CLP	0.24		NO

Note: CIPP = coloured initial plant-produced, CLP = coloured lab produced, NPLP = no pigment lab produced, CNPL = coloured new plant-produced

As shown in Figure 5-2, Coloured New Plant-Produced (CNPP) mixture exhibited lower elastic modulus compare to all other mixtures. This might suggest lower level of rutting resistance for this mixture compare to the others. It should be noted that CNPP was produced by using a softer grade of asphalt binder (PG 64-34P), which explains slightly lower level of resistance to rutting in comparison to other mixtures containing stiffer asphalt binder grade (PG 70-28P). However, this difference in stiffness was found to be insignificant as presented in Table 5-1. All other surface layer mixtures exhibited similar elastic modulus at lower frequency zone; suggesting similar level of rutting resistance for these mixtures. Results presented in Table 5-1 confirms this observation that all mixes are not significantly different in terms of resistance to rutting.

In Figure 5-3, it was observed that CNPP exhibited lower stiffness in compare to the other mixtures. This might be because of softer grade asphalt binder used in the mixture which may have improved the level of resistance to fatigue cracking. This difference in stiffness was found to significant as presented in Table 5-1. It was also observed that mixture containing no-pigment exhibited slightly lower stiffness compared to Coloured Initial Plant-Produced (CIPP) and Coloured Lab Produced (CLP). This might suggest that pigmentation may have affected the level of resistance to fatigue cracking by increasing the mixture's stiffness. However, results presented in Table 5-1 suggests that all mixes are not significantly different in resisting fatigue cracking.

Pigmented mixtures of CIPP and CLP exhibited slightly higher stiffness compared to No-pigment Lab-Produced mixture (NPLP) in higher frequency range, as shown in Figure 5-4. This suggests that pigmentation has affected the level of resistance to thermal cracking compared to non-pigmented mixture. However, paired statistical analysis presented in Table 5-1 found only CIPP to be significantly different compared to NPLP. However, this difference seem to be significantly mitigated by using a softer grade of asphalt binder as CNPP exhibited lower stiffness compared to CIPP and NPLP.

5.2. High Temperature Performance

The CPATT Hamburg Wheel Tracking tester was used to measure rutting susceptibility of coloured asphalt mixtures. From previously presented results of HMA Hamburg testing, it was observed that mixtures containing PG 58-34P exhibited relatively small rutting depth after being loaded for 20,000 wheel passes (i.e. Figure 4-8). Since under study coloured hot mix asphalt mixtures contained much stiffer asphalt binder (i.e. PG 70-28P) compared to PG 58-34P, it was decided to tortured test such mixtures to 60,000 wheel passes. This was to ensure that specimens will deform enough to enable better comparison. To further rutting resistance of mixtures, the total rut depth was manually evaluated by following similar technique employed in field; using a straight edge, and a calliper. Then, the difference between the rut depth measured by HWTD and manual rut depth was used to quantify the so-called "shear upheave" height. Test results of Hamburg rutting test for CHMA mixtures are presented in Table 5-2, while Table 5-3 presents paired t-test statistical analysis of these results to determine if any significant difference between the mixes.

Table 5-2 Rutting Depth Results for Coloured Hot Mix Asphalt Mixtures

Mixture Type	LVDT		Manual Total Rut Depth (mm)	Shear Upheave (mm)
	Average Rut Depth (mm)	Standard Deviation (mm)		
Coloured Initial Plant-Produced (CIPP)	1.47	0.30	2.31	0.84
Coloured Lab-Produced (CLP)	1.49	0.23	2.40	0.91
Coloured New Plant-Produced (CNPP)	3.91	0.45	10.77	6.86
No Pigment Lab-Produced (NPLP)	2.15	0.28	3.58	1.43

Table 5-3 Statistical Analysis of CHMA Rut Depth Results

Paired Mixes		$\lvert T_{observed} \rvert$	$t_{critical}$	Statistically Significant
CIPP	NPLP	45.7	3.18	**YES**
CLP	NPLP	19.9		**YES**
CIPP	CLP	0.20		NO
CIPP	CNPL	23.1		**YES**

Note: CIPP = coloured initial plant-produced, CLP = coloured lab produced, NPLP = no pigment lab produced, CNPL = coloured new plant-produced

As listed in Table 5-2 both pigmented CIPP and CLP mixtures exhibited a slightly higher level of rutting resistance compared to the mixture with no pigment. This minimal difference was found to be significant based on results presented in Table 5-3. However, CIPP, CLP, NPLP mixtures generally provided excellent level of resistance to rutting. The rutting performance of the CIPP was monitored in field throughout number of site visits conducted, which also showed relatively good rutting performance of this mixture as shown in Figure 5-5.

Figure 5-5 Close-up View of Rutting Depth of a Section on Highway 7 BRT-lane May 2016, York Region, ON

It should be noted that the CNPP exhibited lower level of resistance to rutting compare to other mixtures. This difference between CIPP and CNPP was found to be significant based on results presented in Table 5-3. The difference is expected to be due to the softer binder used in producing CNPP mixture (i.e. PG 64-34P). However, CNPP mixture still provided more than adequate rutting resistance. Similar trends can be noted for manual rut measurements, as well as shear upheave measurements.

5.3. Low Temperature Performance

TSRST test setup at CPATT was used to evaluate the resistance to thermal cracking developed at lower pavement temperatures. The TSRST was performed by using same MTS loading frame and environmental chamber used for dynamic modulus in accordance with the AASHTO TP 10-93, “Standard Test Method for Thermal Stress Restrained Specimen Tensile Strength” (AASHTO, 1993). The TSRST testing procedure involved restraining a rectangular beam from contraction while being simultaneously subjected to a constant cooling rate of -10ºC per hour. Resistance to thermal cracking was then evaluated as a temperature in which a fracture is developed within the length of specimen. TSRST test results are listed in Table 5-4.

Table 5-4 Thermal Cracking Results for Coloured Hot Mix Asphalt Mixtures

Mixture Type	Fracture Temperature (ºC)	
	Mean	Standard Deviation
Coloured Initial Plant-Produced (CIPP)	-29.57	2.29
Coloured Lab-Produced (CLP)	-35.02	5.16
No Pigment Lab-Produced (NPLP)	-42.13	2.01
Coloured New Plant-Produced (CNPP)	-32.84	2.60

The TSRST results indicated that adding pigment caused a decrease in the level of resistance to thermal cracking compare to the no-pigment mixture produced in the laboratory. It is also noted that the CIPP mixture exhibited slightly less resistance compare to the pigmented mixture produced in the laboratory. This might be due to higher degree of aging during production process, which may require further investigation study on short-term aging protocols in the laboratory for mixtures containing pigment and/or modified asphalt binders.

In general, TSRST results indicate that all mixtures exhibited adequate level of resistance to thermal cracking by meeting the lower PG grade requirement of -28 ºC. However, resistance of the CIPP mixture to the thermal cracking was questioned to be not adequate in the field. This was because of observing thermal cracks forming few years earlier than expected. Figure 5-6 shows an example of a thermal crack observed on a pavement section paved with CIPP surface mixture. The section was roughly three years old at the time.

Note: The white colour of cracks is because of a chalk used for mapping

Figure 5-6 Premature Thermal Crack Observed on a section on Highway 7 BRT-lane May 2016, York Region, ON

5.4. Fatigue Performance

The Flexural beam fatigue test was performed to evaluate the effect of pigment on the fatigue performance at the intermediate pavement temperature. The testing procedure involved subjecting an asphalt beam to flexural loading applied in a cycle manner with loading frequency of 10 Hz at a 700 micro-strain level. Then, the fatigue failure is defined as the number of load cycles until initial stiffness is reduced by 50 percent. It should be noted that the testing frequency and strain level were selected to simulate heavy traffic loading similar to those expected for bus lanes. Test results of fatigue testing performed at 700-micro strain level at intermediate temperature of 21ºC listed in Table 5-5.

Table 5-5 Fatigue Results for Coloured Hot Mix Asphalt Mixtures

Mixture Type	Average Number of Cycles To Failure[1]	Standard Deviation
Coloured Initial Plant-Produced (CIPP)	15,410	2,594
Coloured Lab-Produced (CLP)	34,421	12,221
No Pigment Lab-Produced (NPLP)	111,245	19,176
Coloured New Plant-Produced (CNPP)	115,312	14,074

Note: [1]Four replicates were tested for each mix at 700-micro strain level at intermediate temperature of 21ºC and testing frequency of 10 Hz.

The fatigue test results at the above specified testing conditions (strain level, temperature, and frequency) indicated that adding pigment decreased the average fatigue life by almost 70 percent for laboratory-produced and 86 percent for plant-produced mixtures. It is also noted that the pigmented plant-produced mixture exhibited less resistance to fatigue cracking compared to the pigmented mixture produced in the laboratory. This might be due to higher degree of aging during production process (also referred to as "short-term or production aging"). It should be noted that conclusions drawn from the fatigue test is only valid for the tested strain level, temperature, and frequency.

It has been well reported that asphalt aging during production is mainly associated with asphalt binder oxidation at the molecular level. This phenomenon is an irreversible chemical reaction between the asphalt binder components and the atmospheric oxygen. This reaction depends greatly on the chemical composition of asphalt binder and temperature.

To further understand the problem, an x-ray fluorescence spectrometer shown in Figure 3-2 was used to analyze the composition of the pigment. Iron was found to be the predominant element present in the pigment. When iron is oxidized, it becomes iron oxide, more commonly known as rust. Rust has been known to be detrimental to asphalt as it may cause swelling of the aggregate leading to raised bumps or micro-cracks. Furthermore, iron is also a strong catalyst that contributes to accelerated rate of aging for asphalt binder. In the pigment, other elements and compounds were also found to be abundantly present such as: Magnesium Oxide, Aluminum Oxide, Chromium, and Manganese. These compounds are also known to act as catalysts during the oxidation process of asphalt; thus contribute notably to accelerating the aging of the asphalt binder. In general, all these minerals and compounds found in the pigment may has resulted in excessive oxidative aging of CIPP, thereby increasing the stiffness of the binder excessively. This could result in decreased level of resistance to fatigue and thermal cracking. This excessive aging was also evident in the field, as same section shown in Figure 5-7 exhibited early signs of cracking.

Note: The white colour of cracks is because of a chalk used for mapping

Figure 5-7 Severe Wheelpath Fatigue Cracks Observed on a section on Highway 7 BRT-lane, May 2016, York Region, ON

5.5. Freeze-Thaw Durability of CHMA

The materials used within the pavement layer need to resist not only the construction and traffic loading, they also require to resist stresses caused environmentally such as freeze-thaw. To evaluate if the pigmentation has impacted the mixture's strength to resist freeze-thaw, previously presented dynamic modulus specimens were saturated in accordance with steps explained in section 3.4.7. Then, specimens were subjected to one month of freeze-thaw cycles; each cycle consisted of 16 hours of freezing temperature of -20ºC followed by 8 hours of thawing temperature of +25ºC.

The selection of temperature range was based on previously presented field and laboratory observations to capture effect of pigmentation on the long-term behaviour of the mixture.

To quantify the freeze-thaw durability, the percent of |E*| retained after one month freeze-thaw cycle was calculated for values obtained at testing temperature of 21ºC and frequencies of 0.1, 1, and 10 Hz. This temperature was selected to translate the impact of pigmentation on the long-term behaviour. This temperature was also selected to produce less variability in results compared to other temperatures such as 37 ºC or 54 ºC. For this reason, only results of 21ºC are only presented in this thesis as shown in Figure 5-8. In this figure, error bars represent one standard deviation from the average value of two replicates tested, with Dynamic Modulus Ratio (DMR) results shown above the bars of dynamic modulus values.

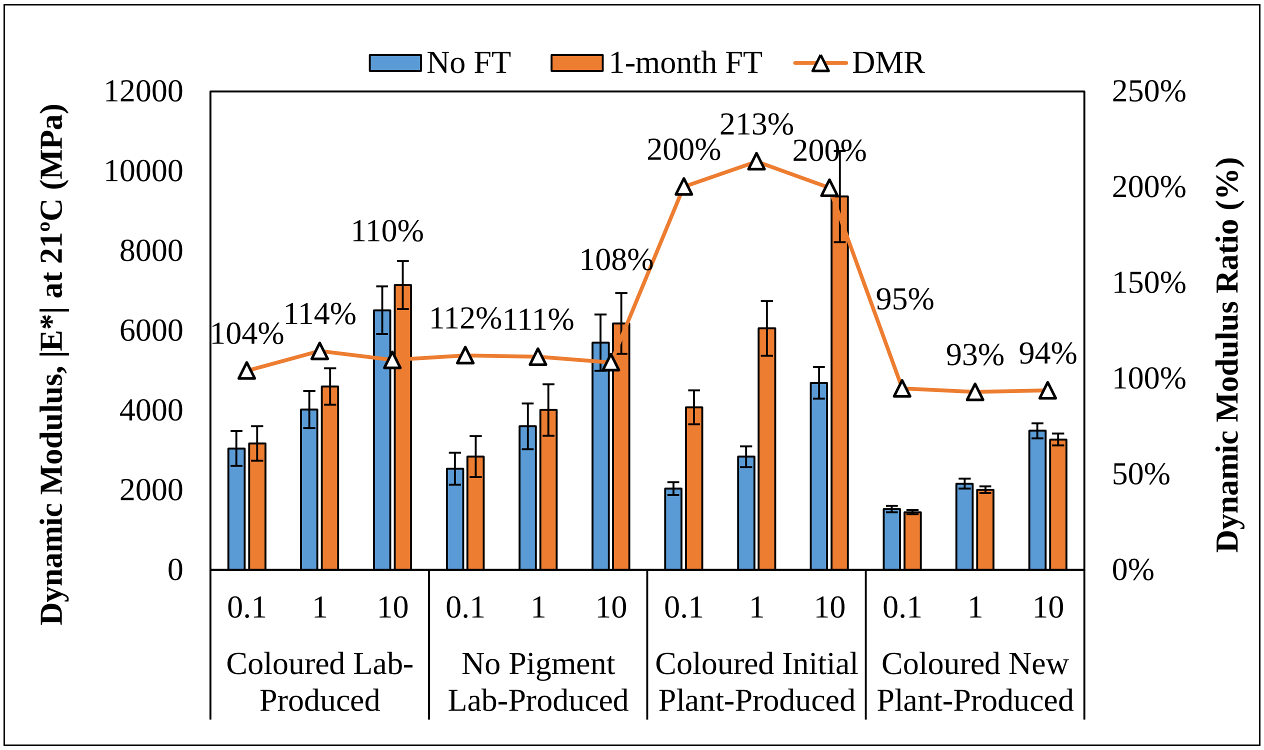


Figure 5-8 Effect Of Pigmentation on Durability of Coloured Hot Mix Asphalt Mixtures

As shown in Figure 5-8, it is evident that CIPP mixture has been excessively hardened more than any other mixtures. This may suggest lowered level of resistance to fatigue cracking, as CIPP might not have the same level of flexibility as other mixtures of CLP and NPLP. It is also observed that using softer grade of asphalt binder in CNPP has helped the mixture to exhibit better performance in multiple freeze-thaw cycles.

5.6. Safety Evaluation

While many factors can affect the roadway safety, pavement characteristics have been reported to be a contributing factor. Pavement surface characteristics are broadly defined in texture scales of micro, macro, and mega. The scale represents the surface deviations respect to a planner surface, which is conceptually illustrated in Figure 5-9. It is reported by (Hall et al., 2009) that micro-texture provides a significant contribution to surface friction of pavements in dry conditions in both slow and high vehicle speeds. Micro-texture is also reported as a factor that is directly related to the surface friction of wet pavements at lower vehicle speed.

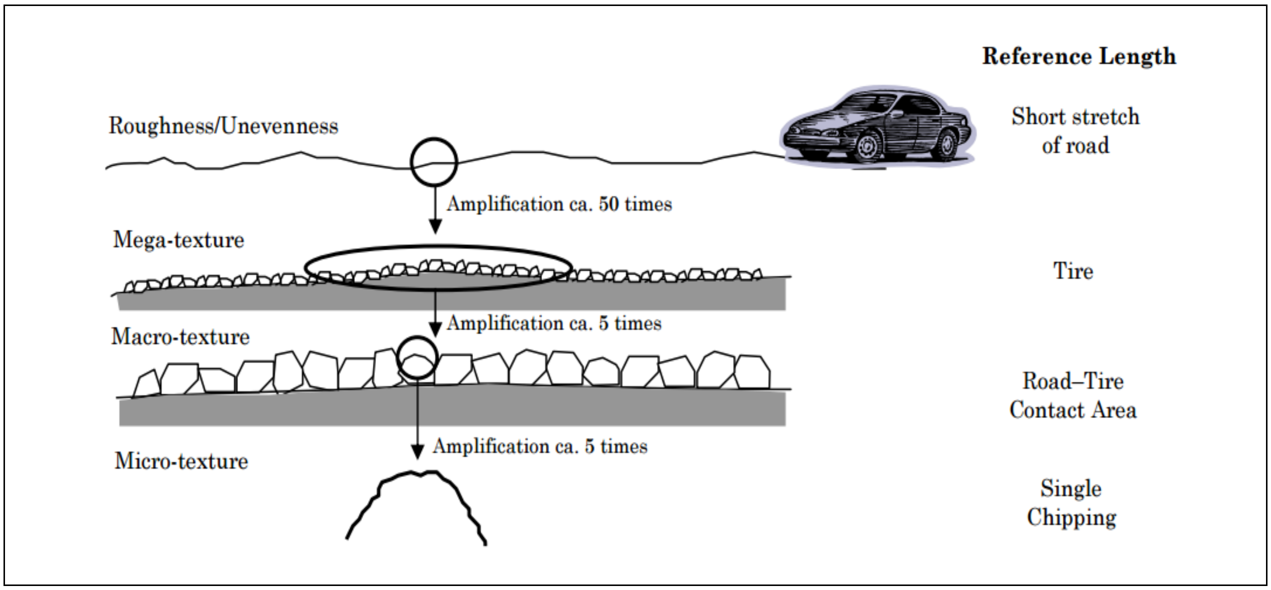


Figure 5-9 Various texture scales that exist for a given pavement surface (Hall et al., 2009)

Effect of colouring pigment on the surface frictional properties was measured in a laboratory by using the British Pendulum (BP) Skid Resistance Tester to capture the effect of pigment on the friction response due to micro-texture modification. The testing procedure involved using a BP to swing a rubber slider over a contact path marked on the surface of a specimen. The surface friction was then measured as the amount of energy loss during the contact between the slider and the test surface in terms of a British Pendulum Number (BPN). The greater the friction between the slider and the test surface, the more the swing is reduced and the larger the BPN reading.

Two swings of the pendulum were made for each test dry surface conditioned at five different temperatures (0, 4.4, 21.1, 37.8 and 54.4°C) to obtain BPNs. Conditioning was performed by using the same environmental chamber used for dynamic modulus testing located at CPATT. Furthermore, the testing was performed on a wet surface for each testing temperature.

To wet the test surface, approximately 45 mL of distilled water was sprayed across the specimen in the beginning of each set of data collection and 5 mL of water was sprayed on the specimen surface to replace the lost water between swings. The average results for the measured skid resistance are shown in Figure 5-10, while error bars represent two standard deviation from the average values.

Furthermore, a statistical analysis of the BPN results was conducted using Analysis of Variance (ANOVA) and paired t-test. The analysis was performed to determine if there is any difference among the mean values of friction numbers at 95% significance level because of: (1) change in the surface temperature (0.00 to 54.4°C), (2) change in surface condition (wet or dry), and (3) change in micro-structure due to pigmentation. The statistical software of Minitab© was used to complete the analysis. Table 5-6 and Table 5-7 present results of ANOVA and paired t-test, respectively.

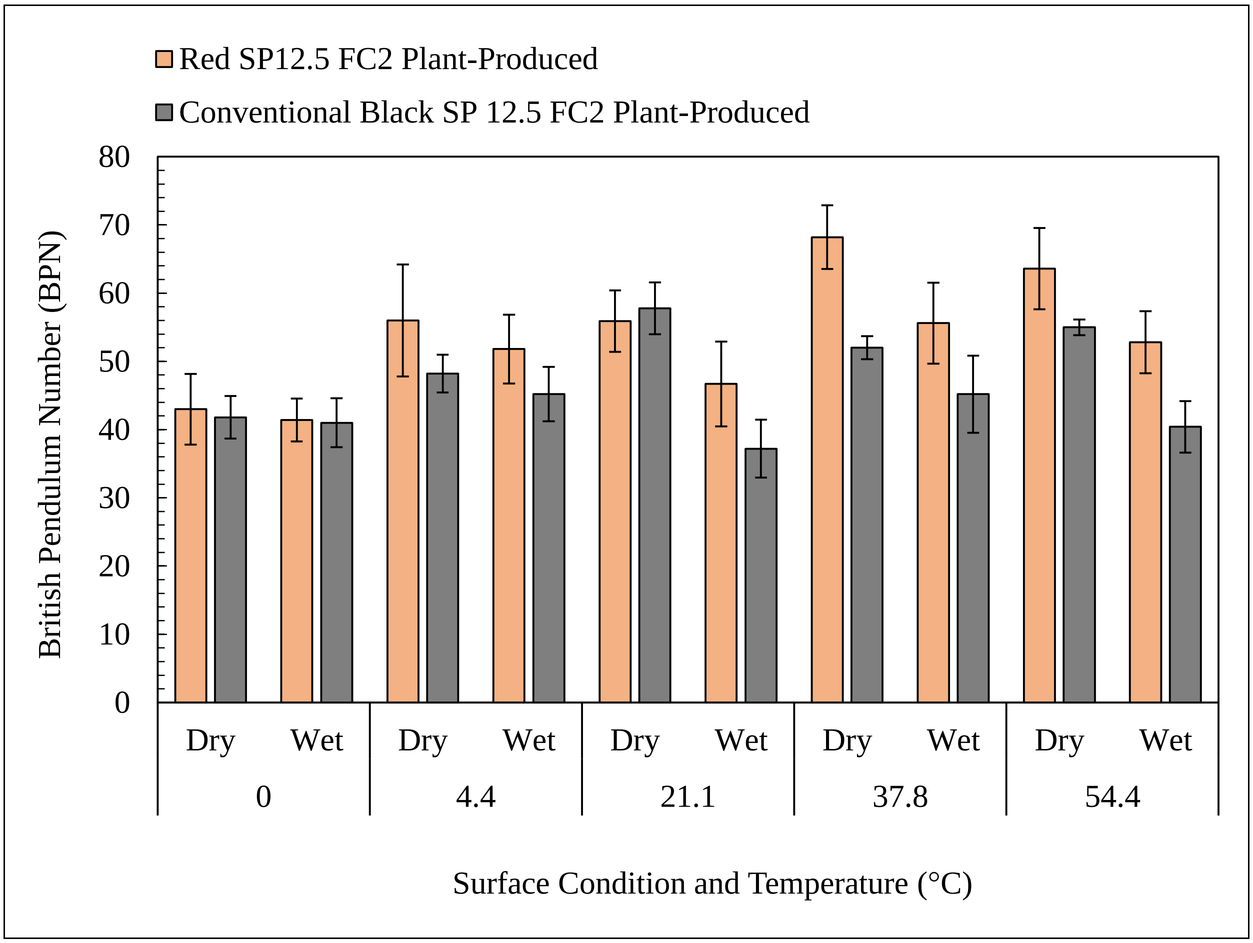


Figure 5-10 British Pendulum Number Laboratory Results

Table 5-6 Analysis of Variance of British Pendulum Test Results

Source	Degree of Freedom (DF)	Sum of Squares (SS)	Mean of Squares (MS)	F Value	P Value	Statistically Significant
Surface Temperature	4	526.50	131.63	3.85	0.011	**YES**
Surface Condition	1	538.76	538.76	15.74	0.000	**YES**
Mixture Type	1	364.82	364.82	10.66	0.003	**YES**
Error	33	1129.49	34.23			
Total	39	2559.57				

Table 5-7 Paired t-test of conventional HMA and CHMA Surficial Friction Properties

Surface Temperature (°C)	Surface Condition	$\|T_{observed}\|$	$t_{critical}$	Significantly Different?
0	Wet	2.58	2.62	NO
4.4		17.25		**YES**
21.1		13.57		**YES**
37.8		104		**YES**
54.4		46.57		**YES**
0	Dry	1.66		NO
4.4		4.07		**YES**
21.1		7.63		**YES**
37.8		15.43		**YES**
54.4		5.07		**YES**

The ANOVA results indicate that all factors of surface temperature, surface condition, and mixture types were significant in this experiment. Furthermore, results of paired t-test in Table 5-7 suggests pigmented mixture significantly exhibited a higher number of BPN in both dry and wet conditions at different temperatures, except at 0.0°C surface temperature. In general, this suggests a higher level of friction and safety for pigmented mixture in compare to the tested conventional HMA mixture.

The surface frictional properties of pavement sections surfaced with coloured hot mix asphalt was also monitored in the field. Friction was measured by using same CPATT British Pendulum and the University of Toronto's ASFT© T2-GO surficial friction tester. It should be mentioned that measurements for both equipment were taken both in wheelpath and centre lines of lanes at surface temperature of 21 ± 1 deg.C. Similar measurements were also performed for pavement sections surfaced with conventional HMA. As shown in Figure 5-11, both Coloured and conventional black exhibited similar BPN numbers on average for different sections with similar age, suggesting similar level of friction and safety in field in intermediate temperature of 21°C. Results of paired t-test presented in Table 5-8 confirms the insignificance difference between the two mixtures as well. Other temperatures were not tested. Similar to BP results, T2-GO showed similar friction properties for black and coloured sections.

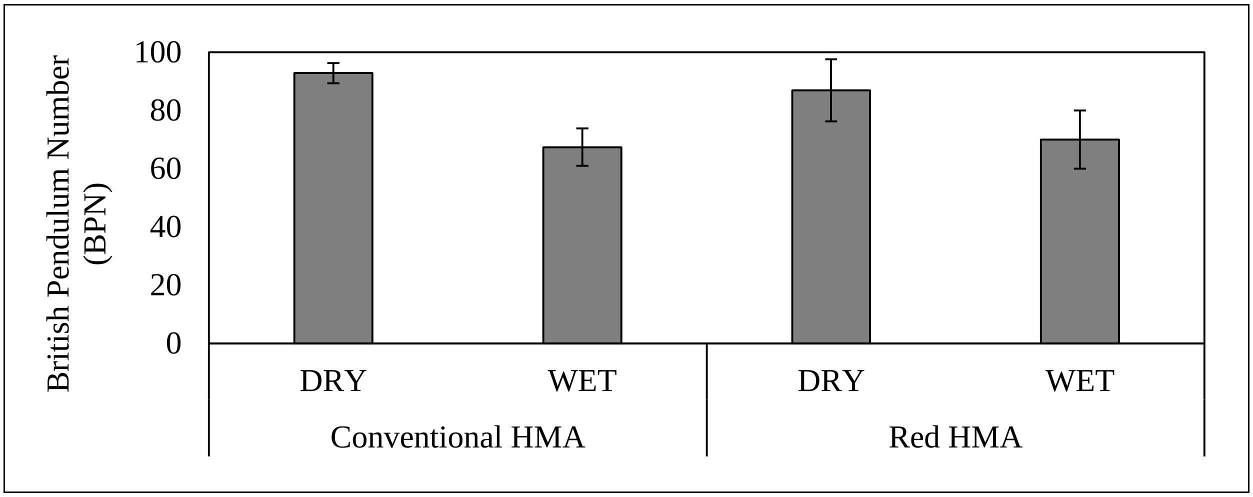


Figure 5-11 British Pendulum Number Field Results

Table 5-8 Paired t-test of conventional HMA and CHMA In-field Surficial Friction Properties

Surface Condition	**$\lvert T_{observed} \rvert$**	**$t_{critical}$**	**Statistically Significant**
Dry	2.08	2.228	NO
Wet	0.87	2.262	NO

5.7. Heat Absorption

Literature review suggested the colour of material exposed to sunlight affects its thermal properties due to solar radiation absorption. Darker colours relatively absorb more solar radiation, which cause temperature increase; while lighter colours absorb less radiation, leading to cooler temperatures.

This theory has been evaluated by many researchers on pavements with lighter surface colours compared with so called “black topped” asphalt pavements. Lighter surface colour pavements are commonly referred to as “cool pavements”, which has been demonstrated by many researchers to provide major benefits for cities by reducing heat absorption and ultimately reducing the possible impact of black surfaced pavements on urban heat island generation.

In addition to the environmental benefits, lighter coloured pavements may provide economic benefits as reduction in heat absorption rate may lead to a reduction in the in-service temperatures through different seasons. This allows for usage of less expensive Performance Grade (PG) binders. Moreover, surface layers within the pavement structure may not experience as high a temperature which causes damages such as rutting and bleeding. This might lead to a more durable and longer lasting road, which might require less maintenance.

A laboratory setup shown in Figure 3-25 was used to evaluate the effect of colour change due to pigmentation on the rate of heat absorption. This test was performed on two laboratory-produced cylindrical specimens at the same time: (1) pigmented and (2) non-pigmented asphalt specimen. Figure 5-12 illustrates measurements collected during one heat-cycle performed in 210 minutes, which indicates coloured asphalt absorbed less heat resulting in roughly 2°C of reduction in temperature at 20 mm below the surface. Similar trend was also observed for the depth of 40 mm. This temperature reduction might be significant in field, which might suggest lowered level of oxidation and colour degradation compared to conventional black asphalt.

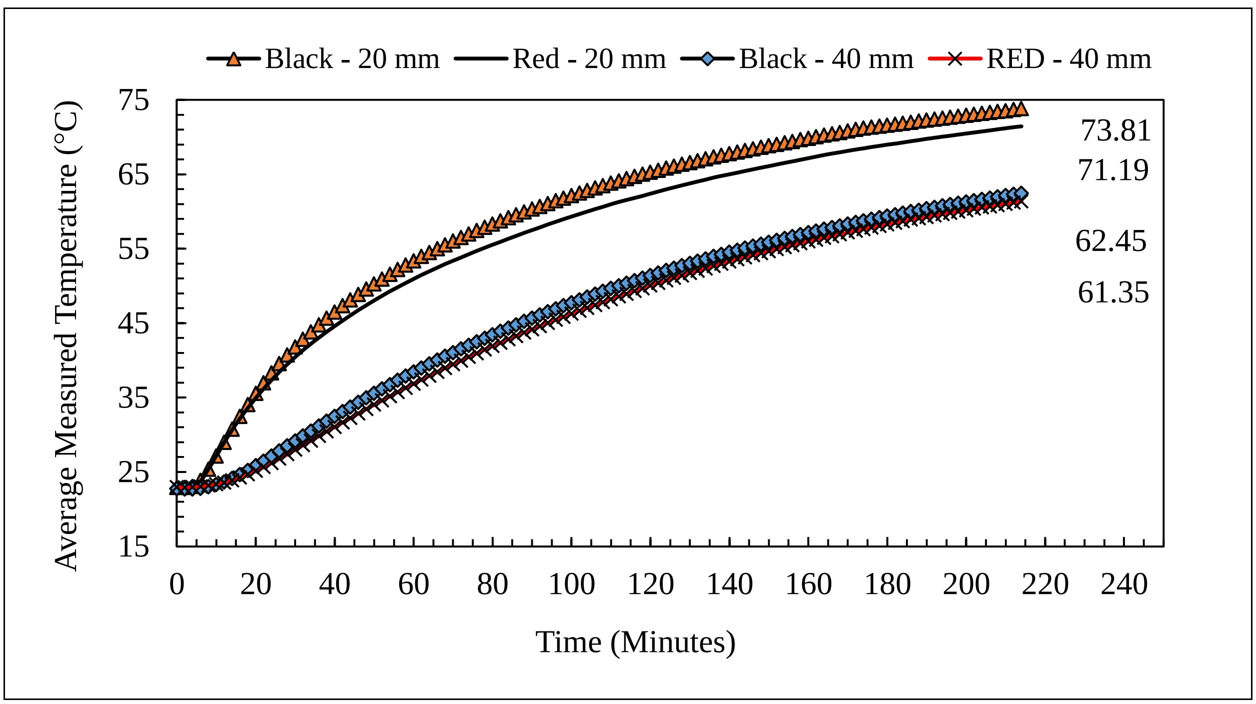


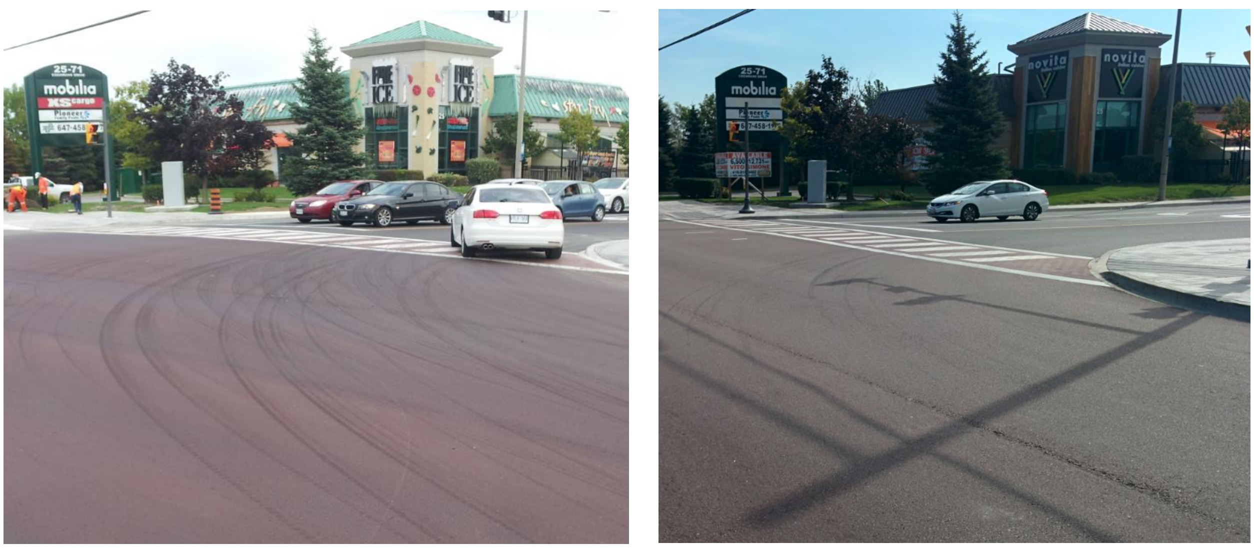
Figure 5-12 Heat Absorption Comparison of Red And Black Asphalt After One Heating Cycle

5.8. Field Manual Distress Survey

A site visit was conducted on August 21, 2014. During this visit, it was observed that the newly paved red asphalt sections exhibited surficial tire scuff marks particularly at the intersections as shown in Figure 5-13 (a). Although the appearance of scuffing and tire marks were found aesthetically unpleasant, they did not seem to affecting the overall performance of the pavement sections, nor indicate a sign of poor workmanship or improper materials.

After reviewing the aggregate blend chart (Figure 3-7), it was noted that more than 50 percent of the aggregate blend was consisted of a fine aggregate (passed sieve size of 4.75 mm) combined with a proprietary red pigment. This resulted in promoting a tighter surface texture and more aesthetically pleasing finish, which may cause the surface texture to be more sensitive to tire scuffing. This sensitivity is even expected to be higher during warm periods (i.e. summer times).

The tire scuffing was monitored more closely and pictures were taken in a site visit conducted on May 8, 2015 as shown in Figure 5-13(b). It seems that the tire scuffing disappeared in time under normal traffic conditions and becoming less visible. At this point, no action was taken to mitigate this problem. But, further research is recommended on the possibility of using coarser aggregate blend and coarser surface texture (i.e. Stone Matrix Asphalt) that may result in more resistant to scuffing.



(a) August 22, 2014 **(b)** May 08, 2015

Figure 5-13 Tire Marking Monitoring at an intersection located on Highway 7 York Region, ON

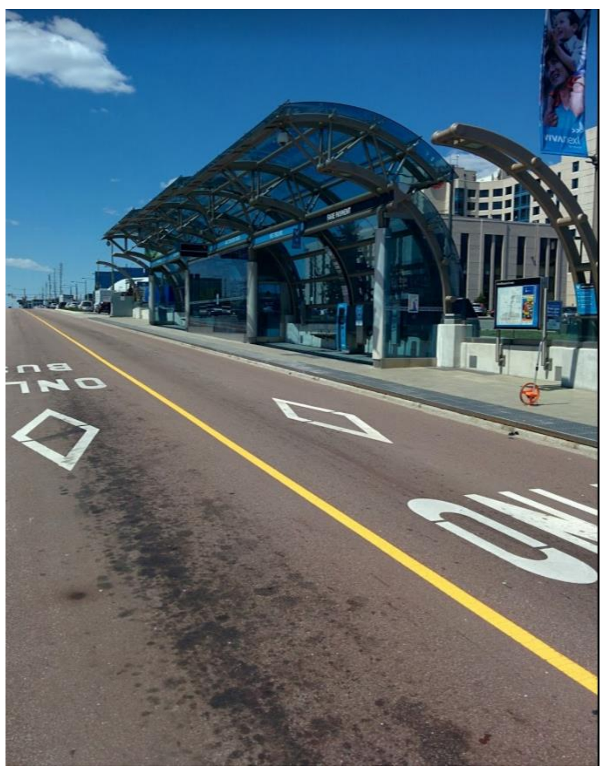
Another visible aesthetic problem was found to be the appearance of oil, grease, fuel, or other automotive fluids dripped onto the pavement. These oil spots were found to be dominate across the Highway 7, especially at sections that carries higher rate of heavy commercial truck traffic (Figure 5-14). Accumulation of these drips over time has created a continuous streak that cause discoloration of the pavement as shown in Figure 5-15. Besides being aesthetically unpleasant, oil drips/streaks was suspected to contain minerals that dissolve or soften the asphalt binder and can result in surface deterioration and defects during in longer period of time.

Figure 5-14 An Example of off-peak Heavy Truck Traffic at Leslie/ Highway 7 Intersection, York Region, ON

Two site visits were conducted on different dates to perform a visual pavement condition survey, and also have a better understanding of field performance of coloured asphalt BRT sections. The assessment indicated distresses in the following forms (Figure 5-16):

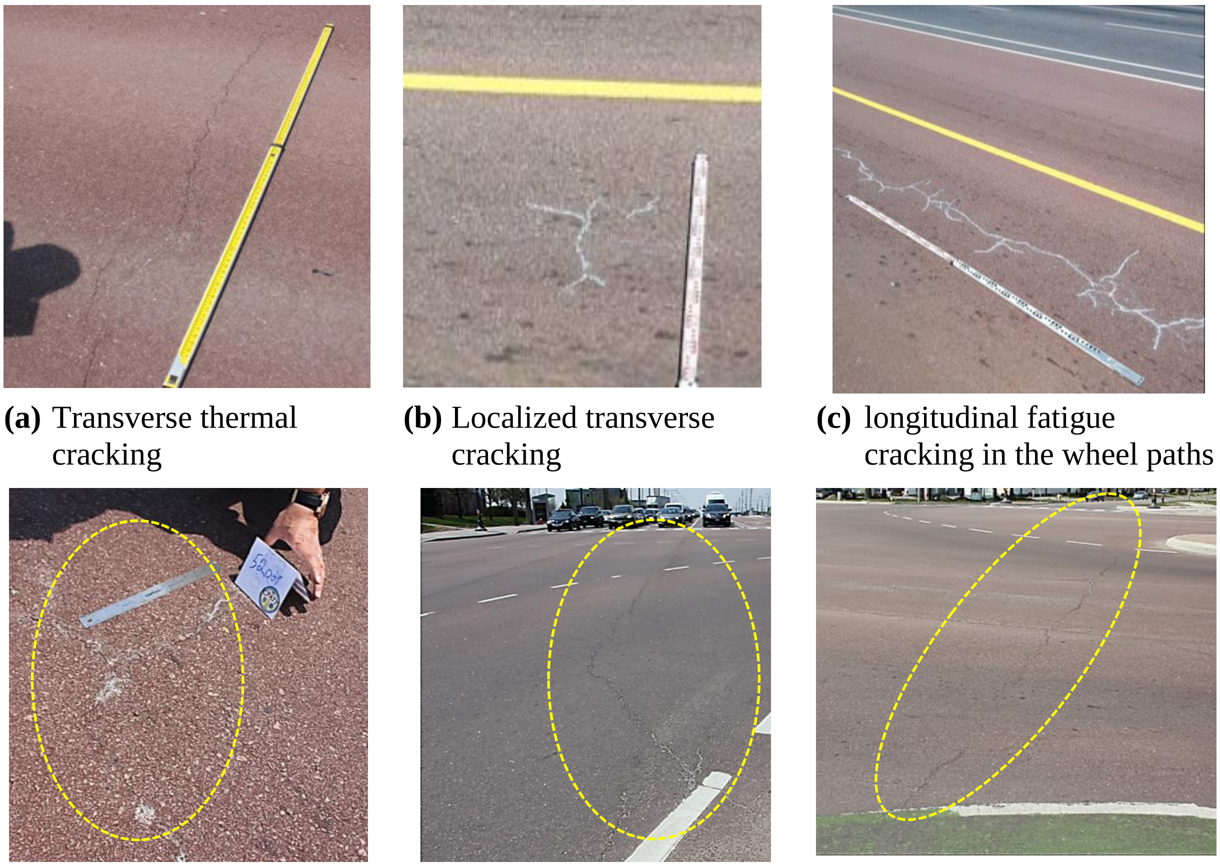
- Transverse thermal cracking along the BRT lanes
- Localized transverse cracking along the BRT lanes
- Longitudinal fatigue cracking in the wheel paths along the BRT lanes
- Localized irregular shape cracks
- Longitudinal joint cracking at the intersection
- Transverse cracking at the intersections

(a) Leslie/Highway 7 intersection, May 2015, York Region, ON



(b) Leslie/Highway 7 intersection, May 2016, York Region, ON

Figure 5-15 Monitored Discoloration Caused by Oil Drips and Formation of Oil Streaks at Leslie/Highway 7 intersection, York Region, ON



(a) Transverse thermal cracking

(b) Localized transverse cracking

(c) longitudinal fatigue cracking in the wheel paths

(d) Localized irregular shape cracks

(e) Longitudinal joint cracking at the intersections

(f) Transverse cracking at the intersections

Figure 5-16 Type of Distresses Observed Along Sections of Highway 7 York Region, ON, August 2015

The UV properties of a coloured surface are important as UV radiation will degrade the colour over time and may make it less effective for lane designation. This was observed during the field test conducted on October 26, 2015. During this visit, the image in Figure 5-17 was taken from a section of BRT lanes located near the Highway 7 and Woodbine Avenue intersection, which consisted of two sub-sections paved with coloured asphalt two years apart. The UV properties of a coloured surface are important as UV radiation will degrade the colour overtime and make it less effective for lane designation.

(a) New and old coloured surface

(b) Close-up view of new and old coloured asphalt texture

Figure 5-17 Surface colour change over two-year period

5.9. Long-Term Performance Predictions

The AASHTOWare's Mechanistic-Empirical (M-E) Software was used to investigate the effect of use of CHMA as surface course on the long-term performance of an as-built pavement structure located on Highway 7 was selected to develop prediction models for a design life of 50 years. The pavement structure comprised of a total asphalt thickness of 210 mm, underlain by 200 mm of Granular Base and 600 mm of Granular Subbase. The asphalt layer was separated into three layers with a 40 mm surface course, 100 mm binder course, and a 70 mm Rich Bottom Mix (RBM). All other inputs were retrieved from the Ministry of Transportation Ontario recommended inputs (MTO, 2012) and were maintained constant in all the designs for other layers.

Traffic Inputs

The selected BRT section is a two-lane road. An initial two-way Annual Average Daily Truck Traffic (AADTT) of 120 was used, with 100 percent of buses in the design lane. Operational vehicle speed in the analysis was selected as 60 kilometres per hour. A linear traffic growth of 1.0 percent was used for analysis, with the bus distribution given in Table 5-9.

It should be noted that other vehicle classes (e.g. light passenger vehicles) were ignored in the analysis. The traffic inputs resulted in 3.20 million Equivalent Single Axle Loads (ESALs).

Table 5-9 Traffic Inputs for Long-term Performance Evaluation of a Section Located at Highway 7 BRT Lanes

Viva Bus Type		Equivalent MEPDG Vehicle class		Distribution
	Van Hool 40-ft A-330	4	Two or three axle buses	40%
	Van Hool 60-ft Articulated AG-300	8	Four or less axle single trailer trucks	60%

The bus distributions were used in conjunction with axle load weights provided by the manufacturer (Table 5-10) when buses are operating laden (all passengers seated).

Table 5-10 Viva Buses Axle Load Inputs with All Passenger Seated

Bus Type	Axle load (kg)			
	Front Axle (Single Tire Single Axle)	Rear Axle (Dual Tire Single Axle)	Tag Axle (Dual Tire Single Axle)	Total
A-330	4846	11501	-	16,347
AG-300	6850	9625	6485	22,960

Climate Inputs and Terminal Service Levels

For the analysis, data from the Lester B. Pearson Airport in Toronto were selected to generate the climate conditions presented in Table 5-11. This weather station was found to be the closet station to the site location, and also the most updated station. Figure 5-18 shows the approximate distance between the site and weather station. For the terminal service levels, similar thresholds listed in Table 4-15 were used at 90% reliability.

Table 5-11 Annual Climate Inputs

Parameter	Annual Statistics
Mean annual air temperature (deg.C)	8.1
Mean annual precipitation (mm)	825.2
Freezing index (deg.C – days)	1028.5
Average Number of freeze-thaw cycles	77.6

Figure 5-18 Approximate Location of Weather Station to the Red Asphalt BRT Lane Job Site

Performance Predictions

The detailed outputs of the MPEDG for all evaluations and materials are presented in Table 5-12, while Figure 5-19 to Figure 5-21 are the graphical damage accumulation over the service life.

In general, the performance prediction obtained from the MEPDG did not match the expected distress propagation based on practical experience gained by pavement engineers in York Region. The inaccurate prediction stems out of the need to perform local calibration of the individual distress models for Southern Ontario. The default MEPDG models were used in this project due to unavailability of local calibration coefficients in the meantime. Although this method does not offer accurate prediction of pavement performance, the utilization of default MEPDG models is useful in comparing the performance of several pavement designs, mix designs and traffic spectrums. The MEPDG was able to effectively differentiate the effect of using different materials as surface courses. These predicted trends were also observed in laboratory performance testing performed and presented in pervious sections.

Table 5-12 MEPDG Output Table For BRT-lane Sections Surfaced with Coloured Hot Mix Asphalt

Performance Criteria	**Surface Mixture Type**			
	Initial Coloured Plant Produced	**New Coloured Plant Produced**	**No Pigment Laboratory Produced**	**Coloured Laboratory Produced**
Terminal IRI (m/km)	4.13	4.11	4.37	4.14
Permanent Deformation (total Pavement, mm)	9.36	8.48	9.56	9.66
AC Bottom-up Fatigue Cracking (%)	1.45	1.45	1.45	1.45
AC Thermal Fracture (m/km)	5.15	5.15	5.15	5.15
AC Top-down Fatigue Cracking (m/km)	48.63	48.62	48.64	48.64
Permanent Deformation (asphalt layers AC only, mm)	0.13	0.11	0.13	0.13

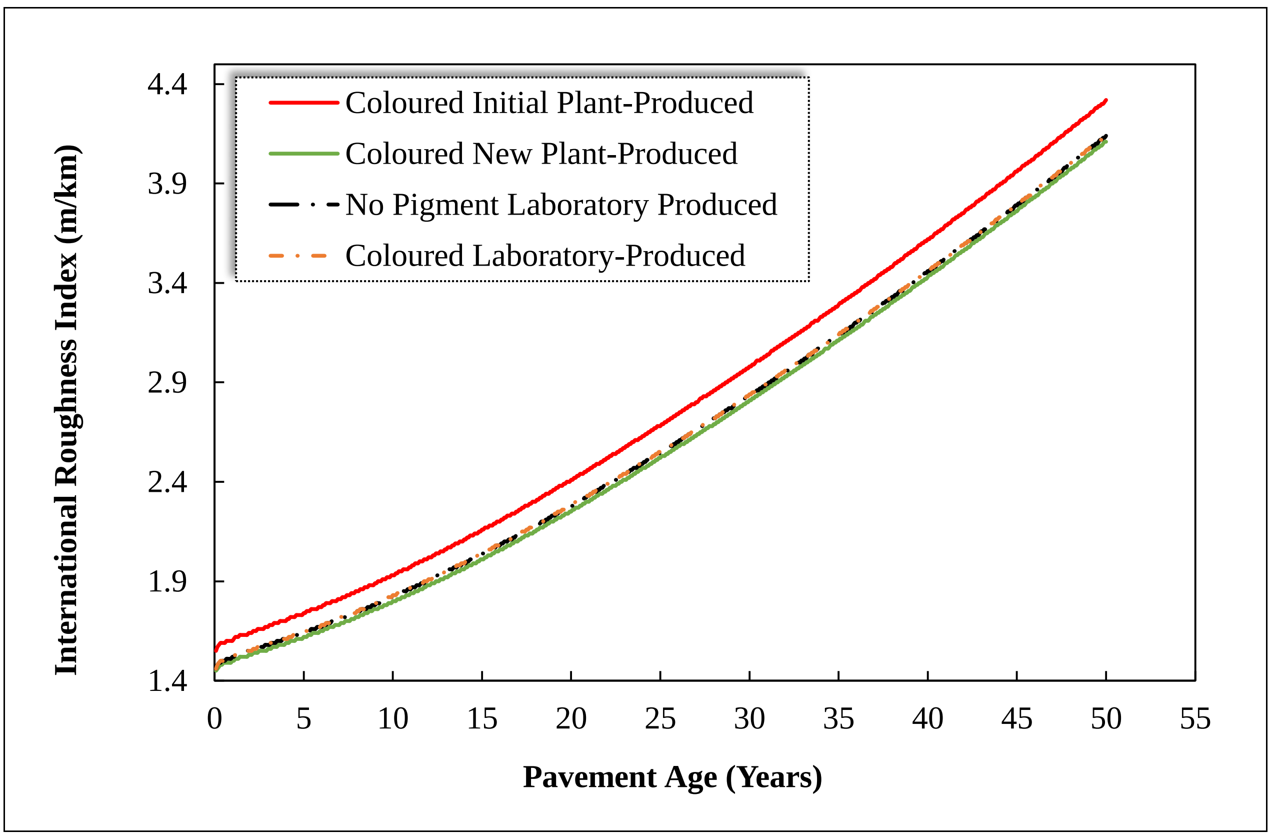


Figure 5-19 International Roughness Index (IRI) Deterioration Over the Pavement Life

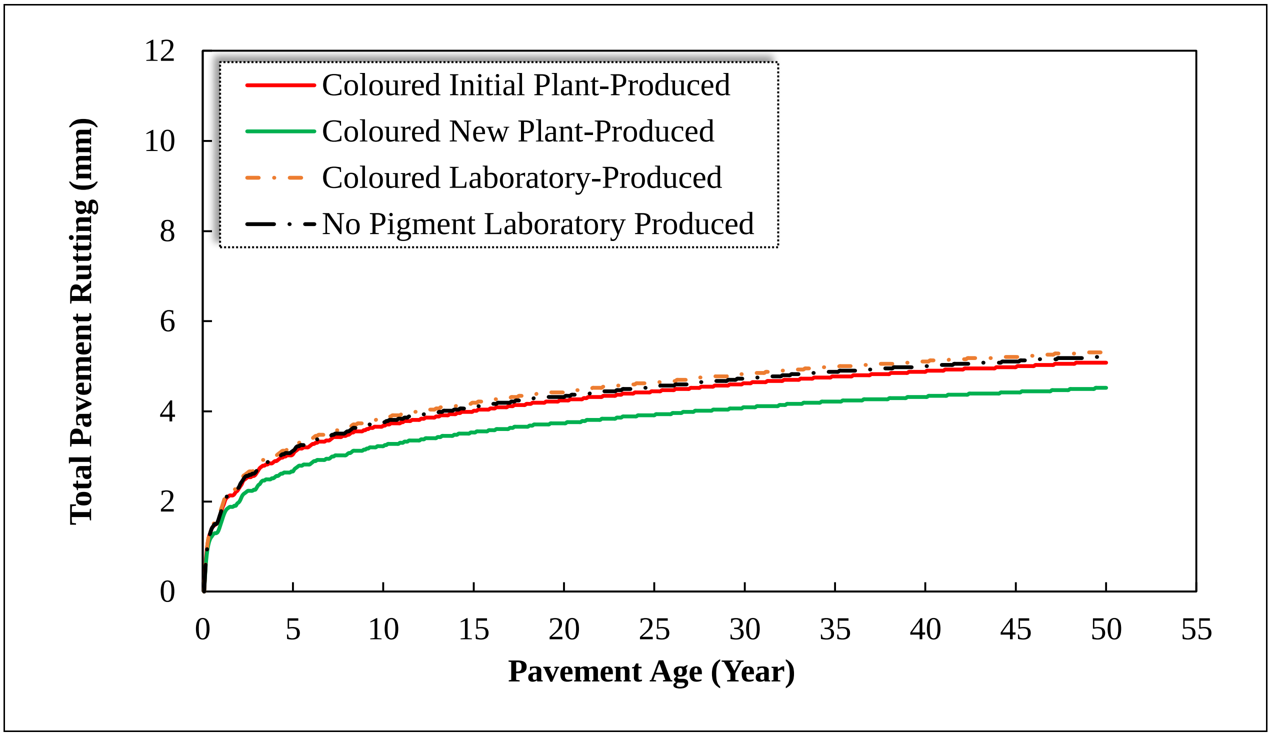


Figure 5-20 Total Pavement Rutting Over the Pavement Life

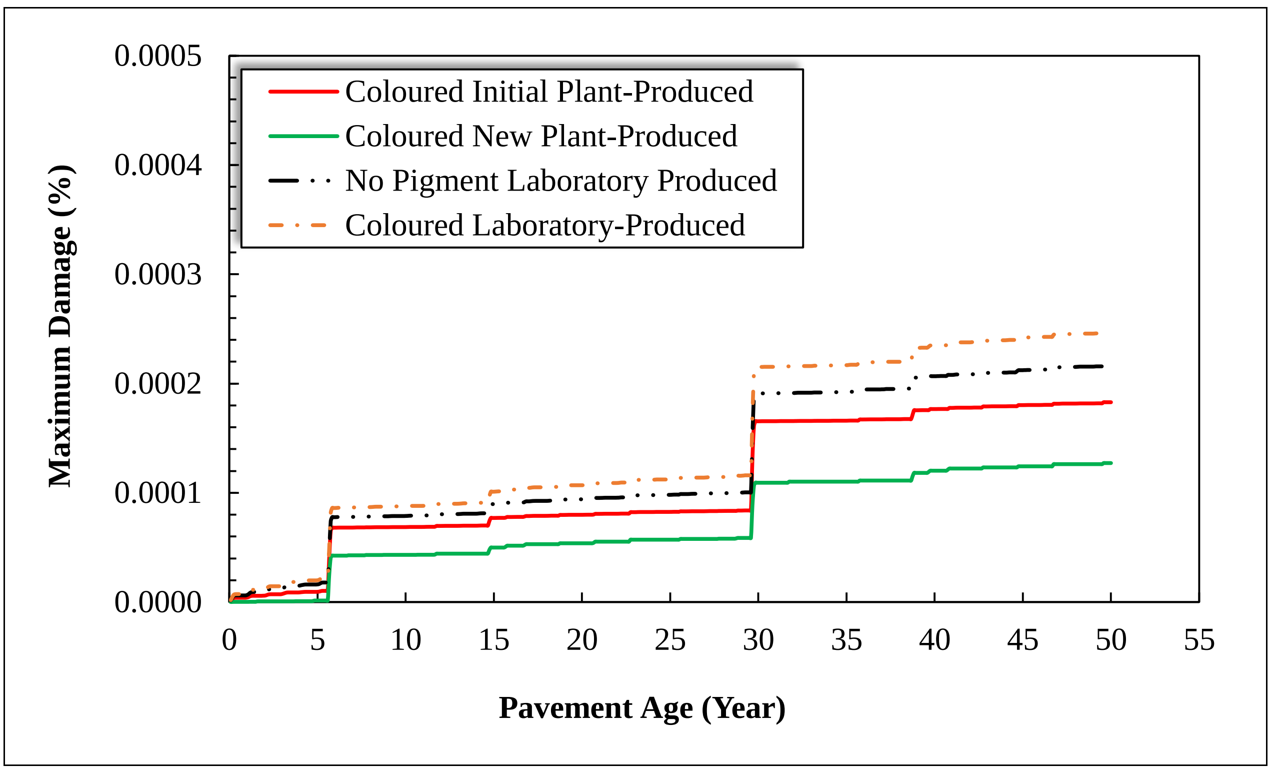


Figure 5-21 Damage Accumulation Over the Pavement Life

In general, the performance prediction obtained from the MEPDG did not match the expected distress propagation based on practical experience gained by pavement engineers in York Region. The inaccurate prediction stems out of the need to perform local calibration of the individual distress models for Southern Ontario. The default MEPDG models were used in this project due to unavailability of local calibration coefficients in the meantime. Although this method does not offer accurate prediction of pavement performance, the utilization of default MEPDG models is useful in comparing the performance of several pavement designs, mix designs and traffic spectrums. The MEPDG was able to effectively differentiate the effect of using different materials as surface courses. These predicted trends were also observed in laboratory performance testing performed and presented in pervious sections.

5.10. Summary

The chapter provides an assessment on the short-term performance of expected long-term behaviour of Coloured Hot Mix Asphalt (CHMA) used as surface mixture in a dedicated Bus Rapid Transit (BRT) pavement structure in York Region, Ontario. Performance testing of plant-produced and laboratory-produced mixtures suggests that pigmentation affected the level of resistance to thermal cracking and fatigue cracking. However, such effect was mitigated by using the new CHMA design which was developed after concerns being raised regarding the old CHMA mix design. Results obtained from multiple freeze-thaws (1-month) also confirms that new mix design has better resistance to fatigue and thermal cracking. Rutting performance of all mixture under study were found to be more than satisfying with exhibiting minimal rut depth after being torture tested for three times the testing limits.

Results obtained from friction testing suggest that pigmented mixture significantly exhibited a higher level of friction in both dry and wet conditions at different temperatures in compare to conventional black HMA surface. In general, this suggests a higher level of friction and safety for pigmented mixture in compare to the tested conventional HMA mixture. CPATT developed setup was to evaluate the effect of colour change due to pigmentation on the rate of heat absorption. Results of this test indicated coloured asphalt absorbed less heat resulting in roughly 2°C of reduction in temperature at 20 mm below the surface. Similar trend was also observed for the depth of 40 mm. This temperature reduction might be significant in field, which may result in lowered level of oxidation and colour degradation compared to conventional black asphalt.

Chapter 6 Life Cycle Cost Assessment

Life Cycle Cost Assessment (LCCA) is a systematic process employed to evaluate economic efficiency of design, maintenance, and rehabilitation alternatives over a pavement's life. LCCA is conducted in accordance with steps given in Figure 6-1. In this thesis, LCCA assessment was performed with two main objectives of (1) evaluating the life cycle period and associated life cycle costs of pavement structures surfaced with CHMA, and (2) examining future maintenance and rehabilitation practices that maintain the coloured surfaces. MEPDG distress outputs were used to develop a deterioration model that can be used to establish LCCA framework. To establish the deterioration model, distress outputs were combined into an overall composite index, such as Overall Condition Index (OCI) by using Equation 6-1.

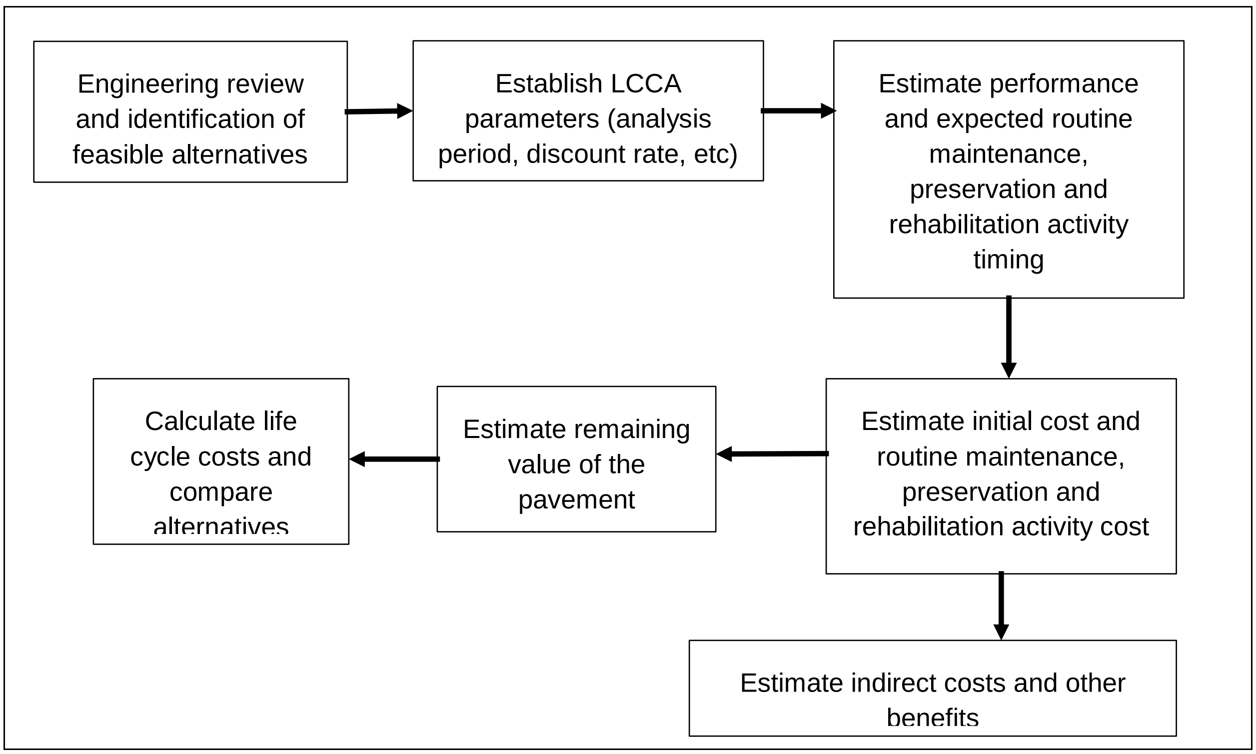


Figure 6-1 Life Cycle Cost Assessment Framework (TAC, 2014)

$$OCI = a(RCI) + b(SDI) \qquad \textbf{6-1}$$

where

$OCI =$ Overall Condition Index
$RCI =$ Riding comfort index
$SDI =$ Surface distress index
$a, b =$ Weight constants

Surface Distress Index (SDI) included distresses predicted by MEPDG were grouped by using Equation 6-2. Since the units were not the same for all predicted distresses, Equation 6-3 was used to scale each distress from 0 to 100-point. In Equation 6-3, Riding Comfort Index was equated to International Roughness Index (IRI), which was scaled from 0 to 100-point similar to distresses used for SDI development.

$$SDI = c(PD_T) + d(FC_{TD}) + e(TC) + f(MC) \tag{6-2}$$

where

$SDI =$ Surface distress index
$PD_T =$ Total Permanent deformation of all layers (including granular layers)
$FC_{TD} =$ Fatigue cracking at the surface
$TC =$ Thermal cracking, and
$MC =$ Maximum damage resulted from Bottom-up cracks and top-down cracking at 12.7 mm downward from the surface
$c, d, e, f =$ Weight constants

$$D_i = [1 - \left(\frac{D_i}{D_m}\right)] \times 100 \tag{6-3}$$

where

$D_i =$ Distress at age i
$D_m =$ Terminal service threshold value at specified reliability (Table 6-1)

Table 6-1 Coloured Hot Mix Asphalt Long-Term Performance Distress Prediction Target Values

Performance Criteria	**Targeted at 90% reliability**
Permanent Deformation - Total Pavement (mm)	19.00
AC Bottom-up Fatigue Cracking (%)	25.00
AC Thermal Fracture (m/km)	189.4
AC Top-down Fatigue Cracking (m/km)	378.8
Permanent Deformation – AC only (mm)	6.00

To develop the deterioration model for BRT-lane, only distress outputs of surfacing option Coloured New Plant-Produced was selected for further development. This was due to the Region's interest on only using this mixture type. Weight constants in above equations were manually derived from a process that ensures reconstruction trigger will be met at the end of pavement's life. Weight constant derived from this process are listed in Table 6-2. Triggers shown in Figure 6-2 were established by consulting York Region's asset management office.

Table 6-2 Weight Constants in Developing BRT-lane Pavement Deterioration Model

Weight constant	Value	Weighting Distress
a	0.75	Riding comfort index
b	0.25	Surface distress index
c	0.70	Total Permanent deformation of all layers (including granular layers)
d	0.25	Fatigue cracking at the surface
e	0.025	Thermal cracking, and
f	0.025	Maximum damage resulted from Bottom-up cracks and top-down cracking at 12.7 mm downward from the surface

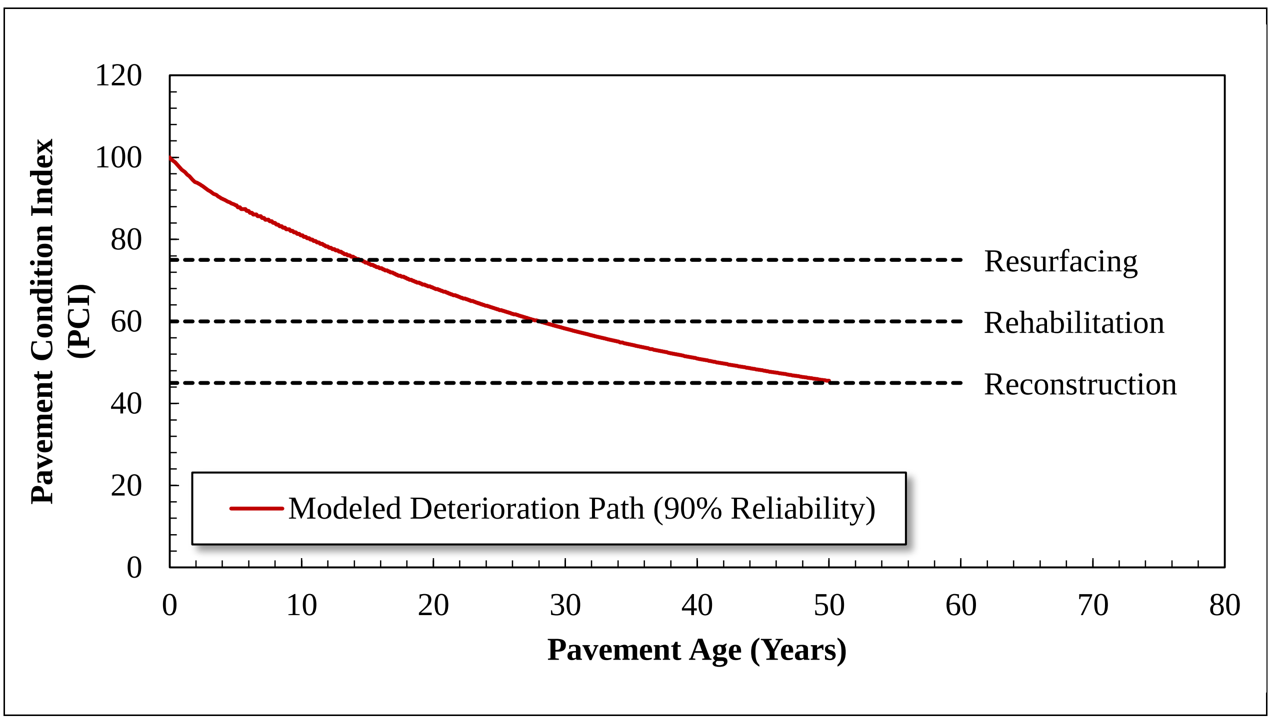


Figure 6-2 Modeled Deterioration Path For a Section of Highway 7 BRT-Lane Pavement Structure

An analysis period of 50 years was assumed. Unit cost per tonnage of each material used for initial pavement construction are listed in Table 6-3, while developed maintenance and rehabilitation (M&R) activities including unit costs for each activity are listed in Table 6-4. Figure 6-3 shows graphically the performance impact after applying each activity. It should be noted that each activity was selected based on feasibility of plant production and construction. For this matter, industry partners were consulted to ensure products required for these activities can be produced on industrial scale and can be available to the market. Furthermore, developed M&R activities for BRT-lanes were compared to a strategy developed (Table 6-5) by York Region on a heavy-duty conventional HMA surfaced pavement structure with same layer thickness.

Table 6-3 Assumed Unit Cost for Initial Pavement Construction

Item	Material	Unit	Thickness	Unit Cost
Earth Work and Grading	-	m^3	-	$ 20
Sub-Base Construction	Granular Type B	m^3	600	$ 35
Base Construction	Granular Type A	m^3	200	$ 33
Binder Course Paving	Superpave 19.0	m^3	170	$ 83
Surface Course Paving (Coloured HMA)	Superpave 12.5 FC-2	m^3	40	$ 450
Surface Course Paving (HMA)	Superpave 12.5 FC-2	m^3	40	$240

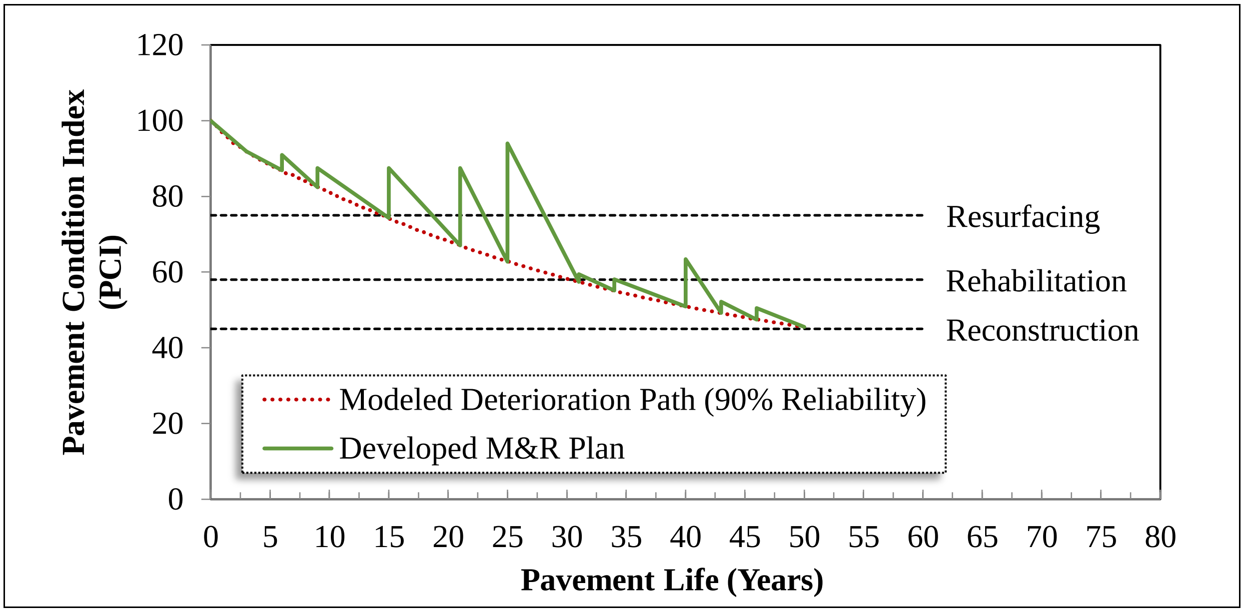


Figure 6-3 Maintenance and Rehabilitation Strategy for a Section of Highway 7 BRT-Lane Pavement Structure

Table 6-4 Developed M&R Strategies for Highway 7 BRT-Lane

Year	Activity	Unit cost	Present Worth Cost (Per Lane.Per km)
0	Initial Construction	-	$495,987
3	Crack Seal	12.5	$3,135
6	Grind and Patch	10.0	$ 35,000
9	Crack Seal + Mill and Pave (10%)	18.0	$63,000
12	Crack Seal	12.5	$6,270
15	Mill and Pave 50 mm	40.0	$140,000
18	Crack Seal	12.5	$ 9,405
21	Mill and Pave 50 mm	40.0	$140,000
23	Crack Seal	12.5	$140,000
23	Grind and Patch	10.0	$35,000
25	Pavement Rehabilitation	62.0	$217,000
28	Crack Seal	12.5	$43,890
31	Mill and Pave 50 mm	40.0	$140,000
34	Crack Seal + Mill and Pave (10%)	18.0	$63,000
37	Crack Seal	12.5	$43,890
40	Mill and Pave 50 mm	40.0	$140,000
43	Crack Seal + Mill and Pave (10%)	18.0	$63,000
46	Crack Seal + Mill and Pave (10%)	18.0	$63,000

Table 6-5 M&R Strategies for a Typical Heavily Loaded Conventional Pavement in York Region, Ontario

Year	Activity	Unit cost	Present Worth Cost (Per Lane.Per km)
0	Initial Construction	-	$466,587
4	Crack Seal	9.30	$2,325
8	Micro-Surfacing	11.0	$38,500
12	Crack Seal	9.30	$32,550
12	Crack Seal	9.30	$4,650
16	Mill and Pave 50 mm	31.5	$110,250
19	Crack Seal	9.30	$6,975
23	Grind and Patch	4.93	$17,255
26	Crack Seal	9.30	$17,255
30	Pavement Rehabilitation-U	54.5	$190,750
34	crack seal	9.30	$32,550
38	Micro-Surfacing	11.0	$38,500
42	Mill and Pave 50 mm	31.5	$110,250

The present value of the investment was evaluated by combining the initial construction costs and the discounted future expenditures by using Equation 6-3. While in Ontario discount rate is between 5 to 6 percent, for the purpose of this thesis, discount rates of 4, 5, and 6 were used for the further analysis and comparison.

$$NPW = IC + \sum_{j=1}^{k} (M\&R_j \times \left(\frac{1}{1 + i_{Discount}}\right)^{n_j}) \qquad \textbf{6-3}$$

where

NPW=	Net Present Worth ($)
IC=	Initial Cost ($)
K=	Number of future maintenance, preservation and rehabilitation,
$M\&R_j$=	Cost of j^{th} future maintenance, preservation and rehabilitation activity ($)
$i_{Discount}$=	Discount rate,
n_j=	Number of years from the presents of the j^{th} future maintenance, preservation and rehabilitation treatment,

To The cost associated with pavement construction, maintenance, and rehabilitation activities are listed in Table 6-6 for BRT-lane pavement and conventional HMA pavement at different discount rates. It is observed that BRT-lane structure surfaced with CHMA, on average, is more costly to maintain than a conventional HMA surfaced road.

Table 6-6 Initial Cost, Total M&R Cost, and NPW Cost Comparison Between BRT-Lane Pavement Structure and A Heavy-Duty HMA Structure

Pavement Structure	Discount Rate	Initial Cost	Total M&R Cost	Net Present Worth
BRT-Lane	4	$495,987	$633,037	$1,129,024
Heavy-duty Conventional HMA		$466,587	$323,562	$790,149
BRT-Lane	5	$495,987	$487,861	$983,848
Heavy-duty Conventional HMA		$466,587	$243,420	$710,007
BRT-Lane	6	$495,987	$383,232	$879,219
Heavy-duty Conventional HMA		$466,587	$187,567	$654,154

Note: All costs are per Lane.per km section

6.1. Summary

This chapter provides details on how the laboratory test results and long-term Mechanistic-Empirical (M-E) predictions obtained in Chapter five of this thesis were used to develop a model describing the expected path of deterioration over time. This model was then used to establish Life Cycle Cost Analysis (LCCA) framework that can be used in development of maintenance and rehabilitation strategies. Based on research work and results presented in this chapter, BRT-lane structure surfaced with CHMA was found to be more expensive to construct and maintain than a similar structure surfaced with HMA located in York Region, Ontario.

Research work presented in this thesis is anticipated to advance the knowledge of using coloured asphalt mixtures in Canada, which can help contractors to be more familiar with the material. This will ultimately result in decreased costs associated with the construction of the coloured asphalt materials and also pave the way to further advancements in developing more cost-effective M&R activities.

Chapter 7 CONCLUSIONS, RECOMMENDATIONS, AND FUTURE RESEARCH

7.1. Conclusions

This research was directed at the evaluation of two innovative pavement materials that can improve performance and safety. For this matter, the performance of Coloured Hot Mix Asphalt (CHMA) pavements for BRT lanes in York Region was evaluated. The second product to be evaluated was the Warm Mix Asphalt (WMA) additives for usage in Ontario with particular interest on provincial and municipal roads such as York Region.

At early stages of research and base on literature review, it was hypothesised that:

- A combination of coloured aggregate and red pigment for York Region's BRT lanes increase the level of safety for ROW users without adversely affecting the structural and functional characteristics of the asphalt concrete mixture.
- WMA additives may decrease the rutting resistance of asphalt concrete mixtures, while improving other properties of mixtures such as fatigue and thermal cracking.
- Excessive aggregate moisture content is present due to the lowered production and compaction temperatures, which may increase the moisture susceptibility of the mixture.

To investigate aforementioned hypothesis, WMA and CHMA were evaluated systematically in laboratory and field. Following sections provide general trends and conclusions were drawn for each technology.

7.2. Warm Mix Asphalt (WMA) Technology

One of the main objectives of this research was to investigate the effect of different warm mix additives on the strength of asphalt mixtures used in Ontario roadways. Followings provide general trends and conclusions drawn from this research work:

- Rheological characterization of modified asphalt binders indicated that the effect of warm mix additives is highly related to the performance grade of the base asphalt cement. But, warm mix additives can be successfully used without any adverse effect on the targeted high and low Performance Grades.
- Temperature measurements obtained from Rotational Viscometer (RV) showed that warm mix additives can be effectively added as an aid to decrease temperatures related to production and construction of asphalt mixtures. But, steps have to been taken to confirm these changes would not cause adverse effect on the Performance Grade Asphalt Cement (PGAC).
- Stiffness characterization of asphalt binders at high in-service temperature showed significant decrease in Superpave rutting parameter of $|G^*|/\sin(\delta)$ binders compare to control binders. Similar trend was observed in mixture's rutting performance tested by HWTD. However, all WMA mixtures exhibited acceptable level of resistance compared to

the upper limit of 12 mm, as per MTO's criterion for surface rutting depth. This might implies that premium surface course mixtures typically designed in Ontario provide adequate level of rutting resistance. Finally, it was observed that usage of polymer modified asphalt binder in WMA mixtures may improve rutting performance.

- Comparison of unaged and short-term aged asphalt binder physical properties showed that addition of warm mix additives can significantly result in less aged asphalt binder. This suggests that pavement layer consisted of WMA binders may become less stiff during in-service years, and might deteriorate slower than conventional HMA.
- Improvement in aging susceptibility was well translated through results obtained from different rheological parameters studied in this thesis. These parameters suggest that WMA may have potential to improve resistance to fatigue and thermal cracking, which is supported by results obtained from rheological tests included in this thesis.
- Dynamic modulus testing results showed similar trends to binder's rheological tests, which addition of warm mix additives can result in a mix that is less stiff at higher temperatures, more elastic or flexible at intermediate and colder temperatures. This also suggest that WMA is capable of improving the level of resistance to thermal temperature cracking and fatigue cracking, but may decrease the resistance to rutting.
- Through comprehensive qualitative and quantitative laboratory test methods performed on compacted and loose asphalt mixtures, it was observed that Evotherm 3G. and Rediset LQ. warm mix additives have anti-stripping properties. These additives can be effectively used to increase the level of resistance to moisture damage, as well as lowering production and construction temperatures of asphalt mixtures. However, Sonnewarmix was found to not have anti-stripping property.
- Comprehensive Mechanistic-Empirical (ME) analysis performed by using AASHTOWare© software showed that the some warm mix additives can be used in asphalt mixtures as surface layers without affecting smoothness of the pavement and the level of resistance to thermal cracking and fatigue cracking. ME analysis suggests that usage of warm mix additives may decrease level of resistance to permanent deformation or rutting.

Results obtained in this comprehensive research were statistically analyzed to verify the significance of the results and information collected from each test were further ranked in ascending order for each combination of aggregate and binder type: first for the best performance and last for the weakest performance. Then, for each mixture, an overall rank was calculated by adding ranks from each test. Overall ranking was then averaged for each mixture, as listed in Table 7-1. This ranking proves that certain warm mix technologies such as Evotherm 3G. and Rediset LQ. can be effectively used in HMA mix design to improve the performance in both laboratory and field in addition to other proven benefits associated with usage of WMA.

Table 7-1 WMA Overall Laboratory and Field Performance Ranking

Surface Course Mixture Type	Final Rank[1]
HMA Superpave 12.5 FC-Type II	26
WMA (Evotherm® 3G.)	21
WMA (Rediset® LQ.)	19
WMA (SonneWarmix™)	30

Note: [1]Lower the ranking, better the overall laboratory and predicted field performance.

7.3. Coloured Hot Mix Asphalt (CHMA)

The second main objective of this research was to provide an assessment on the performance of the in-situ materials and its expected long-term behaviour of Coloured Hot Mix Asphalt (CHMA) used as surface mixture in a dedicated Bus Rapid Transit (BRT) pavement structure in York Region, Ontario. This assessment was then used to develop performance prediction models that can be used as inputs into the overall asset management plan for BRT lanes.

This research was a collaborative study with public and private partners, with the intention of advancing knowledge of using coloured asphalt mixtures in Canada. At the start of this research, knowledge of CHMA in Canada was found to not been studied for short-term and long-term behaviour mainly due to extremely limited usage of this material.

To achieve research objectives, materials collected during paving operations and materials produced under controlled laboratory conditions were systematically evaluated at CPATT to capture the impact of colouring pigment on the mixture's strength. Following sections provide results of this assessment.

At the beginning of the research, it was hypothesised that red pigment used to produce CHMA only works as a filler material and would not adversely affect the structural and functional characteristics of the asphalt concrete mixture. However, comprehensive performance testing in laboratory and field monitoring completed for this thesis suggested that the pigment composition play an important role in the mixture. X-ray fluorescence analysis on the pigment showed iron oxide (Fe_2o_3) to be the predominant element present in the pigment. Iron is a catalyst that could contribute to accelerating the rate of aging in the asphalt cement, and also swelling of the aggregate leading to raised bumps or micro-cracks. Such effect was observed in few sections of BRT-lanes surfaced with CHMA, which raised concerns regarding the mixture design. These concerns led this research work to help industry partners in developing a mixture with improved level of resistance to surface distresses which was confirmed by using the state-of-the-art

AASHTOWare's Mechanistic-Empirical (M-E) Software at the most accurate level of analysis, referred to as "Level 1". ME analysis outputs were then used to develop prediction models for a design life of 50 years that can be used to establish Life Cycle Cost Analysis (LCCA). Based on LCCA analysis BRT-lane structure surfaced with CHMA was found to be significantly more expensive to construct and maintain than a similar structure surfaced with HMA located in York Region. However, this cost difference is expected to decrease in near future as contractors are becoming more familiar with the mixture in terms of design and production.

7.4. Recommendations

Usage of warm mix is strongly advised, but specifying agency or mixture designers should ensure that the additive (or technology) is not adversely affecting the mixture's performance. This can be verified through different tests at different levels of reliability depending on the location and importance of the project. It should be mentioned that laboratory tests such as HWTD or other rutting tests similar to Asphalt Pavement Analyzer (APA) or Flow Number (FN) should be considered. This performance testing is not currently part of OPSS specification, but should be considered for inclusion. Results of dynamic modulus and FN can be also used for MEPDG in designing the pavement structure.

If CHMA is being designed using colouring pigment; it is strongly recommended to verify the composition of the pigment. Extra care should be taken in selecting the PG grade and mixture volumetrics to compensate for any interaction between the pigment and AC. Mixture performance testing is strongly advised including dynamic modulus, flexural fatigue testing, TSRST, and multiple freeze-thaws. For a better understanding of the fatigue behaviour, flexural testing may be performed at different levels of micro-strain, temperatures, and testing frequency.

7.5. Future Research Opportunities

Based on research work presented in this thesis, the followings are possible areas for future research that would be beneficial to the better use of WMA and CHMA:

- Warm mix additives studied in this thesis were used at supplier's recommended dosage to produce WMA. But, it should be noted that same warm mix additives are often used at lowered dosage rates as a compaction aid to help contractors achieve targeted in-field density during paving operations late in the construction season (during fall season). To be considered as compaction aid, warm mix additives are added at HMA construction temperatures. Future study should be performed to quantify the effect of these additives on the mixture's performance.
- The poor correlation of the Superpave RTFO-aged $|G^*|/\sin(\delta)$ rutting parameter with mixture's rutting performance should be further investigated by evaluating different aging temperature or duration during the RTFO short-term aging.
- Effect of warm mix additives on the mixture's resistance to fatigue and thermal cracking should be studied for mixtures designed for Ontario climate.

- The difference between Tensile strength Ratio (TSR) obtained from MIST and T283 moisture conditioning protocols should be studied further. This study should include field observation that can be used to correlate with TSR value and establish a reliable threshold for unconditioned and conditioned strength.
- In Ontario, majority of roads surfaced with WMA are not old enough to produce Reclaimed Asphalt Pavement (RAP). But, in near future, most of these sections will be available for reclamation. Since, there is not yet knowledge of RAP produced from WMA, research should be considered in producing accelerated WMA-RAP in the laboratory controlled conditions. Such RAP can be further studied for its effect of short-term and long-term performance of the asphalt mixture. Similar study is strongly advised for CHMA.
- Future research is required on quantifying aging resulted from the asphalt cement and pigment interaction on the mastic level. This may help in developing different asphalt cement modification techniques that can help eliminating any premature aging or accelerated oxidation resulting in excessive stiffness.

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