DEVELOPMENT OF HIGH MODULUS ASPHALT CONCRETE MIX DESIGN TECHNOLOGY FOR USE ON ONTARIO'S HIGHWAYS

by

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Author's Declaration

This thesis consists of material all of which I authored or co-authored: see Statement of Contributions included in the 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.

Statement of Contributions

This thesis is partially the product of co-authored publications as follows:

Chapter 3

- Baghaee Moghaddam T, Baaj H. (2018). Application of compressible packing model for optimization of asphalt concrete mix design. Construction and Building Materials. 159: 530-539. https://doi.org/10.1016/j.conbuildmat.2017.11.004.

Chapter 4

- Baghaee Moghaddam T, Baaj H. (2018). The use of compressible packing model and modified asphalt binders in high-modulus asphalt mix design. Road Materials and Pavement Design. https://doi.org/10.1080/14680629.2018.1536611

Chapter 5

- Baghaee Moghaddam T, Baaj H. (2018). Rheological characterization of High-Modulus Asphalt Mix with modified asphalt binders. Construction and Building Materials. 193: 142-152. https://doi.org/10.1016/j.conbuildmat.2018.10.194.

Abstract

Asphalt pavement is subjected to external loads including mechanical loading induced by traffic and thermal loading induced by thermal variations. The last decades have witnessed a significant rise in number of heavy vehicles especially commercial trucks with higher axle loads on rural and arterial roads in Ontario. Consequently, by increasing the number and amplitude of traffic loading and severe environmental condition, servile life of asphalt pavements has been adversely affected. In many cases, premature distresses were occurred before expected service life of asphalt pavements reaches to its end.

On the other hand, new pavement materials, design procedures and construction technologies have been developed worldwide. One of these technologies is "Enrobé à Module Élevé- (EME)" or "High-Modulus Asphalt Mix". EME is a type of asphalt concrete that represents high modulus/stiffness, high durability, superior rutting performance and good fatigue resistance. This type of mix was developed in France in the 1980's. EME is a very good option to be used in lower and upper binder courses in the pavement structure which are subject to the highest levels of tensile and compressive stresses. EME offers several advantages over conventional binder course materials including reducing the thickness of the pavement structure with improved service life and reduction in raw materials consumption. Despite the excellent performance at higher and intermediate temperatures, traditional EME mixes would be very susceptible to low-temperature cracking which is associated to using very hard grade asphalt binder. In addition to the cold climate condition, some other aspects such as traffic volume, vehicle attributes, properties of raw materials, construction methods, and testing standards are specific to Ontario.

Based on the aforementioned reasons, adopting EME technology will be beneficial to Ontario's highways. However, development of a suitable EME mix design procedure in Ontario cannot be a duplicate copy of the French method, or any other methods used in other countries or jurisdictions. This study, funded by the Highway Infrastructure Innovation Funding Program (HIIFP-2015), aims to introduce a new approach to EME mix design that contributes to good performance at high, medium and low temperatures. This could be achieved by using premium aggregate particles with dense structure (high packing density), along with utilizing high quality asphalt binder with precise content in the mix.

A performance-based mix design approach is developed for EME mix design in Ontario which is a modified version of Superpave mix design procedure. Compressible Packing Model (CPM) was used for the first time to optimize the packing density of aggregate particles for two categories of mixes (12.5 mm and 19 mm Nominal Maximum Aggregate Size (NMAS)). Three types of modified asphalt binders were also considered: PG 88-28, PG 82-28 and PG 58-28 + modifiers (Elastomer additives). In addition to measuring

compaction ability (compactibility) of the developed mixes, several thermo-mechanical testing methods were designated to be used in this study to evaluate the performance of asphalt mixes at different levels.

Results of this study showed that the CPM-obtained gradation limits were within the grading control points of EME mixes recommended by French specification. The asphalt mixes had higher compactibility than the conventional mix, and, EME 19 was more compactible than EME 12.5 although it had less binder content than EME 12.5. Complex modulus test results illustrated that the mixes had high modulus values, and that the values of EME 19 were generally higher than those of EME 12.5. Hamburg wheel track rutting test results showed both mix types had superior rutting performance. Fatigue performance of developed mixes was assessed using four-point bending beam fatigue test at different strain levels to develop fatigue curves. The test results showed that the minimum strain level to meet 1,000,000 cycles of fatigue life (ε_6) was more than 300 μm/m for all the mixes. Additionally, Thermal Stress Restrained Specimen Test (TSRST) results showed that the cracking temperatures of the developed mixes were less than -25˚C; and that EME 12.5 performed slightly better than EME 19.

Binder microstructure and rheological properties were assessed using environmental scanning electron microscope (ESEM) and dynamic shear rheometer (DSR) equipment respectively. Two springs, two parabolic elements and one dashpot (2S2P1D) rheological model is used to model and compare the viscoelastic behavior of the binders as well as the mixes. ESEM test results showed that microstructure of PG 88-28 binder was the densest and connected with thicker fibril size. PG 58-28 + Elastomer additives had highly intertwined structural network with the thinnest fibril size among the binder types. 2S2P1D results showed it is a powerful tool for modeling highly polymer modified asphalt binders as well as EME mixes. According to developed master curves the mixes' moduli have followed the same pattern as for the binders' although phase angles' patterns were different. Correlations were found between the binders' microstructures and their rheological properties. Binders with denser structure and stronger bonds showed to have lower phase angles. Although binders with more intertwined structural network had higher modulus particularly at higher frequencies.

The EME mix design approach was validated by using the second source of aggregate materials and PG 82-28 asphalt binder. The SGC compactibility test results showed that the mixes were more compactible than the conventional Superpave mix. According to the rutting test results, the mixes had almost not rut after 20,000 wheel passes on the submerged specimens at 50°C (rut-depth < 1 mm). In addition, the developed mixes with the second source of aggregates had relatively higher fatigue resistance where ε_6 values were greater than 550 μm/m for both EME 12.5 and EME 19. TSRST results also depicted that the cracking temperatures of both mixes were below -30°C.

Acknowledgments

I wish to express my sincere thanks to my supervisor Professor Hassan Baaj for his support, guidance and encouragement during my Ph.D. study. I would also like to express my sincere appreciation to my Ph.D. defence committee members Prof. Susan Tighe, Prof. Kaan Inal, Dr. Ludomir Uzarowski from the University of Waterloo and Prof. Daniel Perraton from École de Technologie Supérieure (ETS), Montréal.

This study was funded by Ministry of Transportation Ontario (MTO) through the Highway Infrastructure Innovation Funding Program (HIIFP-2015). I would like to thank Imran Bashir from MTO for his useful feedbacks during meetings we had in the past years. I am also so thankful to materials suppliers McAsphalt Industries Ltd, Miller Group, Bitumar Inc, Roadway Solutions and Dufferin Construction which made this research possible.

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Last but not least, I am so grateful to my parents for their love and support that helped me to get to where I am today.

Thank you all

To My Family

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List of Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
BBME	Bétons Bitumineux à Module Élevé
BBR	Bending Beam Rheometer
BBTM	Béton Bitumineux Très Mince
BRD	Bulk Relative Density
CA	Course Aggregates
CPATT	Centre for Pavement and Transportation Technology
CPM	Compressible Packing Model
CUW	Chosen Unit Weight
D	Nominal Aggregate Size
DGCB	Département Génie Civil et Bâtiment
DSR	Dynamic Shear Rheometer
DTT	Direct Tension Tester
EME	Enrobé à Module Élevé
ESEM	Environmental Scanning Electron Microscope
FA	Fine Aggregates
HDM	Heavy Duty Macadam
HMA	Hot Mix Asphalt
HPC	High-Performance Concrete
HWTT	Hamburg Wheel-Track Tester
LCPC	Laboratoire Central des Ponts et Chaussées
LUW	Loose Unit Weight
LVE	Linear Viscoelastic
MAS	Maximum Aggregate Size
MEPDG	Mechanistic-Empirical Pavement Design Guide
MTO	Ministry of Transportation Ontario
NAS	Nominal Aggregate Size
NCHRP	National Cooperative Highway Research Program
N_{des}	Design Number of Gyrations
N_{ini}	Initial Number of Gyrations
NMAS	Nominal Maximum Aggregate Size
NMPS	Nominal Maximum Particle Size
N_{max}	Maximum Number of Gyrations
PAV	Pressure Aging Vessel
P_{be}	Effective Binder Content
PCG	French Gyratory Shear Compactor
PCS	Primary Control Sieve
PD	Packing Density
PE	Polyethylene
PG	Performance Grading
RTFO	Rolling Thin Film Oven

RUW	Rodded Unit Weight
RV	Rotational Viscometer
SABITA	South African Bitumen Association
SBS	Styrene-Butadiene-Styrene
SCC	Self-Consolidating Concrete
SGC	Superpave Gyratory Compactor
SHRP	Strategic Highway Research Program
SMA	Stone Mastic (Matrix) Asphalt
TSRST	Thermal Stress Restrained Specimen Test
V_a	Air Voids
VFA	Voids Filled with Asphalt
VMA	Voids in Mineral Aggregate

CHAPTER 1
INTRODUCTION

1.1 Preface

In addition to a general introduction, a background chapter and a general conclusion, this thesis consists of three journal articles and one unpublished chapter that focus on development and characterization of a new paving material known as Enrobé à Module Élevé (EME) or High-Modulus Asphalt Mix. The first paper, which was published in the Journal of Construction and Building Materials, presents application of Compressible Packing Model (CPM) as a new technique in optimization of asphalt concrete mix design by optimizing aggregate packing densities in the mix. The optimization was performed for two different EME types based on Nominal Maximum Aggregate Size (NMAS) and different asphalt binders. The second paper evaluates performance of EME mixes in terms of permanent deformation, fatigue properties and low-temperature cracking resistance of EME mixes. The paper is published in the Journal of Road Materials and Pavement Design. The third paper, also published in the Journal of Construction and Building Materials, investigates the rheological characterization of EME mixes. In this paper, 2S2P1D rheological model was used to determine rheological parameters of EME binders as well the mixes. In addition, morphological properties of EME binders were determined using environmental scanning electron microscope (ESEM) equipment, and correlation between morphological properties of EME binders and the rheological properties of EME mixes were discussed in the paper. The unpublished chapter investigates the effects of second aggregate source on EME mix design. This chapter provides experimental results for the two EME mix types.

The work presented in this thesis was conducted under supervision of Professor Hassan Baaj who provided tremendous help and support during my Ph.D. study. This project was funded by Ministry of Transportation Ontario (MTO) through Highway Infrastructure Innovation Funding Program (HIIFP-2015). Imran Bashir from MTO also provided very useful comments in the meetings we had in the past. All the materials in composition of the original articles provided in the thesis are the sole production of the primary investigator listed as first author in the journal publications. The research presented in this thesis is result of collaboration with materials suppliers: McAsphalt Industries Ltd, Bitumar Inc., the Miller Group, Roadway Solutions and Dufferin Construction.

1.2 Motivation

Road pavement design and construction technologies are being developed worldwide. On the other hand, as result of severe environmental conditions and increasing traffic intensity especially increasing number of passing heavy trucks with higher axle loads on Ontario's roadways, service life of flexible (asphalt) pavements could be reduced considerably. In such situation, transferring technologies that can help to enhance pavement performance to Ontario and adopting them according to the need would be very beneficial.

One of these technologies is Enrobé à Module Élevé (EME). EME was developed in the 1980s in France to address the problems of rutting and premature fatigue cracking in flexible pavements. The continual increase in traffic and the fact that the legal axle loads are very high in France (the single axle legal load in France is 13 tonnes) have indeed been the two major factors behind the development of this type of materials. EME represents a category of asphalt mix with high stiffness modulus (dynamic or complex modulus) together with excellent resistance to rutting and fatigue cracking. EME mixes are mainly used in the binder and the base course (upper and lower binder courses) of a pavement structure. They can be used in both new and rehabilitation pavement projects. In France, two classes of EME mixes exist. EME Class 2 has an excellent resistance to fatigue and rutting while EME Class 1 is a degraded "low cost" class that has similar stiffness and rutting resistance to class 2 but with a relatively lower fatigue resistance.

The use of EME mixes in the pavement structure would lead to a better distribution of stresses and strains in pavement layers. The use of EME mix in the binder course should limit the effect of compressive stresses in this layer and restricts deformation or rutting. The base course (layer beneath the binder course) is subject to repeated tensile stresses which would lead to premature fatigue cracking. The use of EME mix in this layer would decrease significantly the risk of fatigue. Some research projects have however led to the development of EME mixes with higher levels of recycled materials (Baaj et al., 2013, Olard, 2013) and a Warm Mix Asphalt version of EME (Baaj et al., 2012).

The use of EME became common in several European countries such as the UK (Sanders and Nunn, 2005), Belgium (De Backer, 2007), Switzerland (Junod and Dumont, 2005) and this technology is now in development in several other countries worldwide such as Australia (Petho and Denneman, 2013), South Africa (Denneman and Nkgapele, 2011). In Canada, Bitume Quebec has recently published a technical bulletin on High Modulus Asphalts adapted to cold climates (Bitume Québec, 2014). The primary aim of this research project is to transfer EME technology to Ontario as part of collaboration between Centre for Pavement and Transportation Technology (CPATT) at the University of Waterloo and Ministry of Transportation Ontario (MTO).

1.3 Problem Statement and Objectives

Some aspects such as the cold climate, traffic, properties of available construction materials, construction methods and standards are specific to Ontario and therefore, development of a suitable EME mix design in Ontario cannot be a duplicate copy of the French method or any other method used in another country or jurisdiction. The project should adequately address the needs and specificities of Ontario while preserving the authenticity of the concept and the advantages of the original technology.

The success of EME mixes relates to the use of premium angular aggregates with dense structure (i.e., high packing density), along with utilizing high quality binder. The binder content, grade, and quality ensure excellent resistance to fatigue, rutting, and low-temperature cracking while offering a very high stiffness of the mix at intermediate and high temperatures. The criteria for the selection of aggregates and the right mix design are then the key elements to reach very high-performance requirements of the mix. In addition, the climatic context of Ontario dictates another performance criterion to take into consideration which is the low-temperature cracking resistance.

The main challenge is then to be able to design a mix with enough binder content to ensure good fatigue resistance while adding too much binder can adversely affect the stiffness and permanent deformation resistance of the mix. In addition, the binder should be hard enough to ensure good resistance to rutting but at the same time, it should be flexible and ductile at lower temperatures to avoid low-temperature cracking.

The global objective of this study is assisting MTO with transferring and adoption of EME technology to Ontario based on Ontario's conditions including: climatic and traffic conditions, available materials and currently used test procedures. The outcome of the study is:

I. The feasibility study and mix designs for different categories of EME (Two categories 0/12.5 and 0/19);
II. Adoption of a new aggregate packing method based on Compressible Packing Model (CPM) technique.
III. Developing mix design approach and providing recommendations on design procedure and specifications (design parameters, gradation envelopes, volumetrics, advanced testing).
IV. Rheological analysis of modified asphalt binders as well as asphalt mixes using 2S2P1D rheological model.
V. Validating EME mix design approach for a different source of aggregate.

1.4 Thesis Contribution

The first contribution of this thesis is providing insight into development of high modulus asphalt mixes with acceptable performance for regions with cold climatic conditions. In this regard, using highly polymer modified asphalt binders and elastomer additives (pellets) are investigated. Two mix categories based on NMAS are developed. According to the literature, depending on what type of mix is produced, different aggregate gradation envelopes should be used. The second contribution of this thesis is the use of CPM as a potential technique for optimization of asphalt mix design which was conducted for the first time. And the last contribution of this thesis is evaluation and modeling of rheological properties of highly polymer modified binders, the developed mixes with low air void contents and finding correlations between asphalt binders' microstructures and their rheological parameters. Schematic representation of thesis contribution is provided in Figure 1 – 1.

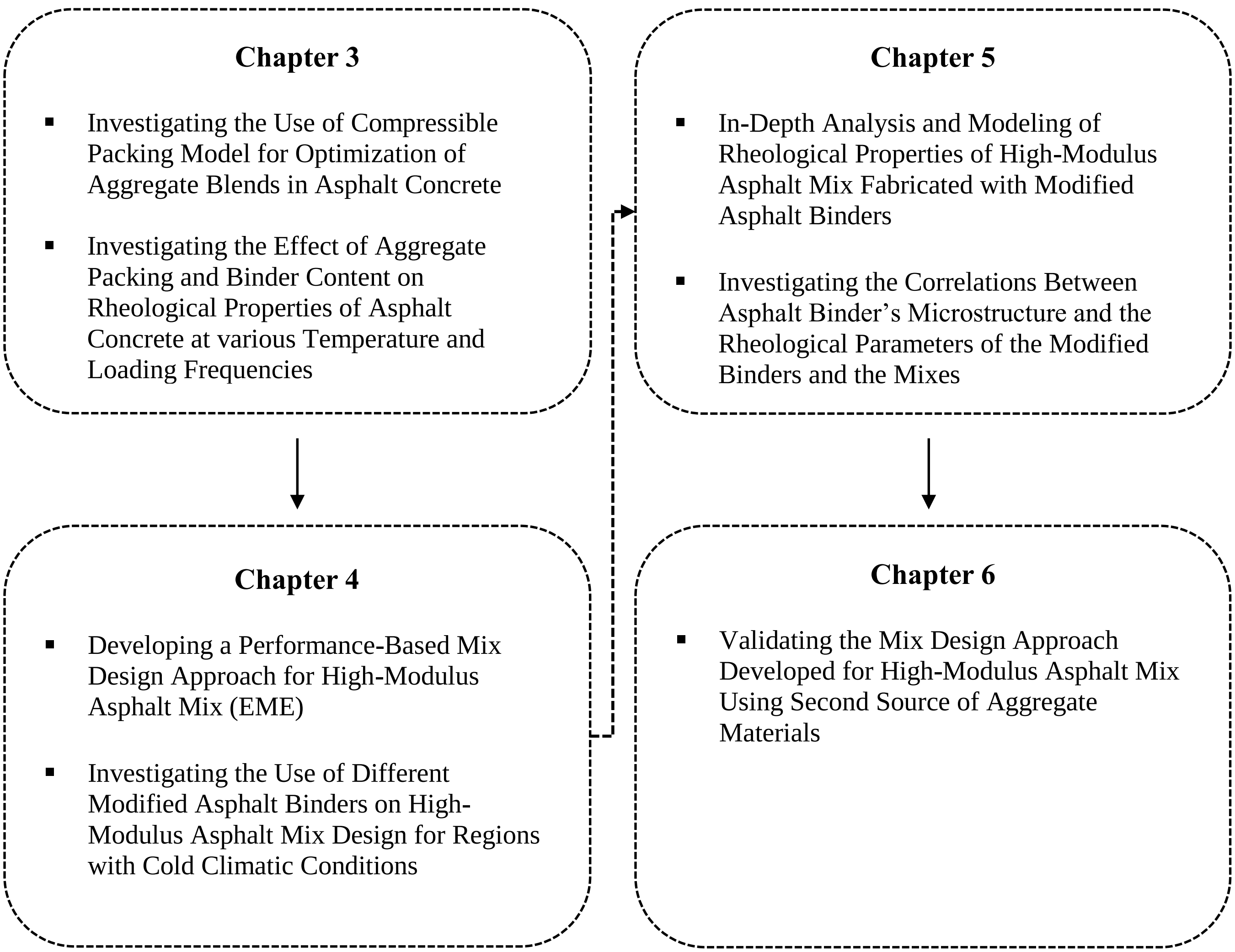


Figure 1 - 1: Schematic flowchart of the thesis contributions

1.5 Thesis Organization

This manuscript-based thesis consists of seven chapters as follows:

Chapter 1*:* General scope and overall objectives of the research are explained in this chapter.

Chapter 2: This chapter reviews general and important information related to flexible pavement materials and structure. Information about HMA designs, past and current state of practice for EME mix designs are also discussed in this chapter.

Chapter 3: This chapter addresses the possibility of using Compressible Packing Model (CPM) as a tool to optimize asphalt mix design by optimizing the packing density of aggregates. Aggregate gradation envelopes for high performance asphalt mixes were determined using this approach.

- ***Baghaee Moghaddam T***, Baaj H. (2018). *Application of compressible packing model for optimization of asphalt concrete mix design.* Construction and Building Materials. 159: 530-539. https://doi.org/10.1016/j.conbuildmat.2017.11.004.

Chapter 4: This chapter evaluates performance of the developed mixes at different levels. This includes permanent deformation (rutting) performance, fatigue properties and low-temperature cracking resistance.

- **Baghaee Moghaddam T**, Baaj H. (2018). *The use of compressible packing model and modified asphalt binders in high-modulus asphalt mix design.* Road Materials and Pavement Design. https://doi.org/10.1080/14680629.2018.1536611.

Chapter 5: This chapter evaluates morphological (microstructure) and rheological properties of used asphalt binders as well as rheological behavior of EME mixes.

- **Baghaee Moghaddam T**, Baaj H. (2018). *Rheological characterization of High-Modulus Asphalt Mix with modified asphalt binders.* Construction and Building Materials. 193: 142-152. https://doi.org/10.1016/j.conbuildmat.2018.10.194.

Chapter 6: The validation of EME mix design for another aggregate source is discussed in this chapter.

Chapter 7: This chapter provides a general conclusion and summary of key findings of the research. Recommendations for future research directions are also presented in this chapter.

Figure 1 - 2 provides a schematic illustration of the thesis contents. It should be noted that some basic information (e.g. used materials properties, testing methods) are repeated occasionally in the chapters.

Figure 1 - 2: Schematic flowchart of the thesis content

CHAPTER 2
BACKGROUND

2.1 Introduction

The background chapter consists of seven sections. Each section provides relevant information with regards to the objectives of this research. Section 2.2 discusses the flexible pavement structure and its characteristics. Section 2.3 provides information on HMA material, components and mechanical properties. Flexible pavement distresses are discussed in Section 2.4. HMA design methods (Superpave and French methods) are discussed in section 2.5. Information on aggregate packing density and its influence on mix performance are then provided in section 2.6. Section 2.7 provides information on EME materials, history, design procedure as well as international implementation of EME.

2.2 Flexible Pavement Structure

The purpose of pavement structure, such as any other types of structures, is to transfer deduced loads and stresses to the ground in a safe manner. For instance, when a truck applies a load on the pavement surface, that load is distributed and transmitted through the pavement layers to the ground. The term “flexible” in flexible pavements has been adopted from the property of this type of structure under loading. In flexible pavements, unlike rigid (Portland cement) pavements, the total pavement structure deflects, or flexes, under loading. A flexible pavement structure is typically composed of several layers of materials each of which receives the loads from the above layer, spreads them out, and then passes them onto the sublayer. Thus, the further down in the pavement structure a layer is, the less amount of load (in terms of force per area) it must carry.

The top layer in flexible pavement structure (layer one) is called wearing layer. The function of this layer is providing characteristics such as friction, noise control, smoothness, rut resistance and drainage. Depends on the type, it may also prevent penetrating surface moisture into sub layers. The top layer of flexible pavement structure sometimes subdivided into two layers namely: surface course (top) and binder course (bottom). A prime function of the binder course is to help dissipate the high stress close to tire. Following attributes should be considered for binder course materials:

a) high stiffness modulus
b) high resistance to deformation
c) good cracking resistance

d) impermeability
e) not to prone to segregation (Sanders and Nunn, 2005).

To provide the above attributes Hot Mix Asphalt (HMA) is often used to construct the binder course.

Under the wearing layer the “base course” layer is usually placed. Base course can be constructed either by crushed aggregate or HMA. Base material is subjected to lower shear stresses than the binder course which implies that the binder course should have a higher internal stability than base layer to resist higher shear forces. Traditionally, the materials used in the base course have been generally similar to binder course material, but employ a bigger nominal aggregate size and have less binder content (Sanders and Nunn, 2005). The third main layer of the pavement structure is “subbase”. Placement of subbase can be optional and depends on severity of traffic loading. The primary role of subbase layer is using it as structural support of upper layers. It can also be used to minimize the intrusion of fines from the subgrade into the pavement structure. Subgrade is usually the existing natural soil (compacted if necessary) and is not considered as pavement structural layer.

2.3 Hot Mix Asphalt

Hot Mix Asphalt (HMA) is a composite material mainly composed of aggregate particles and asphalt cement or binder (modified asphalt). Typically, 95% of asphalt mixture (by weight) consists of aggregate particles and the rest of 5% is asphalt cement. By volume, a typical asphalt mixture is 85% aggregate, 10% asphalt cement and 5% air voids. The following sections provide a short description of each element.

2.3.1 Asphalt Binder

Asphalt binder (cement) is a thick, heavy residue remaining obtained from refining crude oil, and consists mostly of carbon and hydrogen. It is viscoelastic thermoplastic material. Physical behavior of asphalt binder considerably depends on temperature variation. At higher temperatures, it usually behaves like a fluid, although at room temperature it is more likely to behave like soft rubber. In addition, it becomes very brittle when temperature drops below zero (Jenks et al., 2011). The function of asphalt binder is holding the aggregate particles together.

Asphalt binder is known as a complex construction material in terms of behavior. Its behavior is very susceptible to temperature and loading time. That is why without accompanying a specific temperature it is difficult to interpret the measured characterization. Additionally, asphalt binder behaves differently under the same amount of loading but different duration (loading frequency). Loading time and temperature are two factors that can be used interchangeably, Figure 2 - 1. Due to asphalt binder time-temperature dependency a slow loading rate can be simulated by high temperatures and fast loading rate can be simulated by low temperature (McGennis et al., 1994).

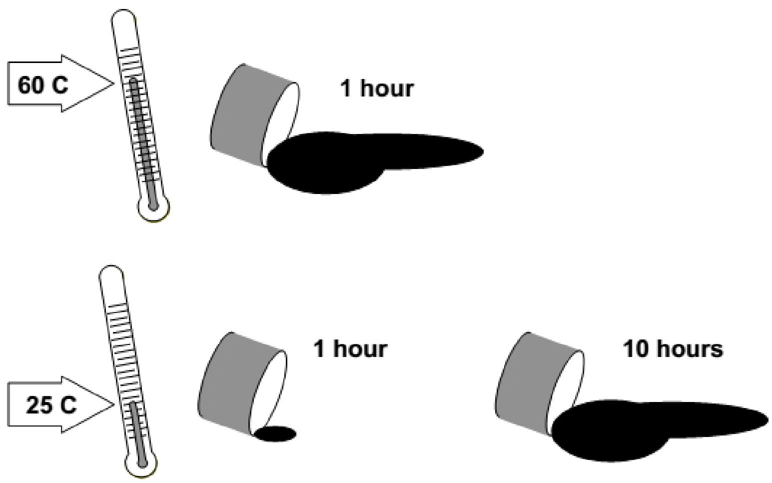


Figure 2 - 1: Asphalt binder Time-Temperature dependency (McGennis et al., 1994)

2.3.2 Aggregate Particles

Aggregates form skeleton of asphalt mixture and play an important role in carrying the loads in HMA. Aggregate particles should be tough and abrasion resistance. Additionally, shape of the aggregate particles can significantly influence the HMA performance. When aggregates are flaky or elongated, they can be easily crushed under compaction/traffic loads which would affect HMA performance over pavement service life.

Different types of aggregates such as natural aggregates, processed aggregates, synthetic and waste aggregates can be used in production of HMA. Natural aggregates are simply mined from river or glacial deposits and are used without further processing to manufacture HMA. Processed aggregates are referred to the natural aggregates that have been separated into distinct size fractions, washed, crushed, or otherwise treated to enhance certain performance characteristics of the finished HMA. However, in most cases processed aggregates are quarried and the main processing consists of crushing and sizing. Synthetic aggregates consist of any material that is not mined or quarried and, in many cases, represents an industrial by-product. In this regard, blast furnace slag is one example. Waste products are increasingly used as replacement of aggregates in pavement structure. Scrap tires and glass are the two most well-known waste products that have been successfully utilized in asphalt pavement construction (Huang et al., 2007).

Regardless of the source, processing method, or mineralogy, aggregates expected to provide a strong, stone skeleton to resist repeated load applications. Cubical, rough-textured aggregates provide more strength than rounded, smooth-textured aggregates. Even though a cubical piece and rounded piece of aggregate may possess the same inherent strength, cubical aggregate particles tend to lock together resulting in a stronger mass of material. Instead of locking together, rounded aggregate particles tend to slide over each other (Asphalt Institute, 2001).

2.3.3 Air Voids

Air voids, which is defined as small airspaces or pockets of air that exist between the coated aggregate particles throughout compacted paving mixture, is an important parameter in HMA design. The amount of air voids (or air voids content) in a mix needs to be precisely determined. Depending on the type of asphalt mixture, a certain percentage of air voids is necessary in the mix. Higher mount of air voids can contribute to higher permeability of the mix which can adversely affect the mix durability. On the other hand, if the air voids content is too low, it would result in bleeding when asphalt binder squeezes out of the mix to the surface. Bleeding leads to less pavement surface friction which increases risk of skidding.

2.3.4 Mechanical Properties of Hot Mix Asphalt

HMA, such as asphalt binder, is viscoelastic thermoplastic material that exhibits complex behavior. It can be considered as elastic material at lower temperatures while at higher temperatures it behaves more like a viscous fluid (see Figure 2 - 2). Viscous fluids and elastic solids behave differently; however, combining these two behaviors in one is possible. Most HMA mixes have viscoelastic behavior at usual pavement service temperature which means they behave like elastic solid and viscous fluid simultaneously.

When a sinusoidal load is applied to an elastic solid it deforms immediately. But, in case of viscous fluid there is a phase lag of 90˚ from the moment the load was applied, and the time deformation begins. For viscoelastic materials the phase lag (phase angle- φ) is between 0˚ and 90˚, Figure 2 - 3.

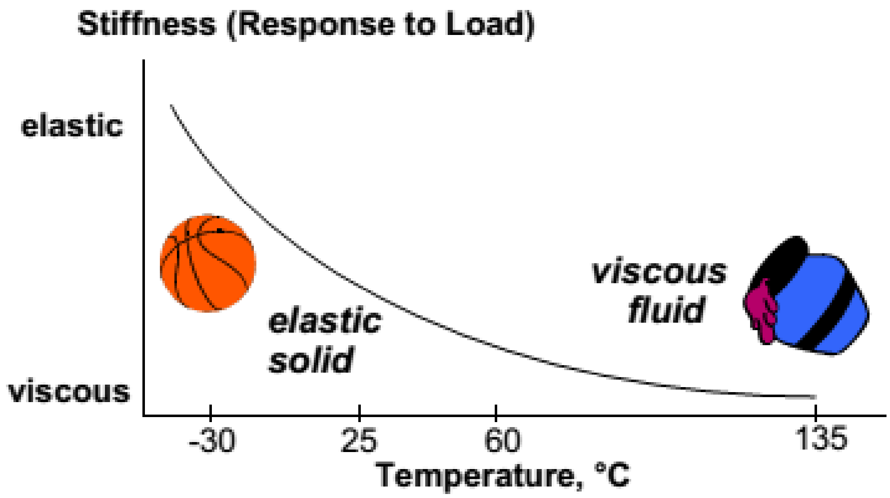


Figure 2 - 2: Visco-Elastic behaviour of asphalt binder (Superpave Fundamentals, 2000)

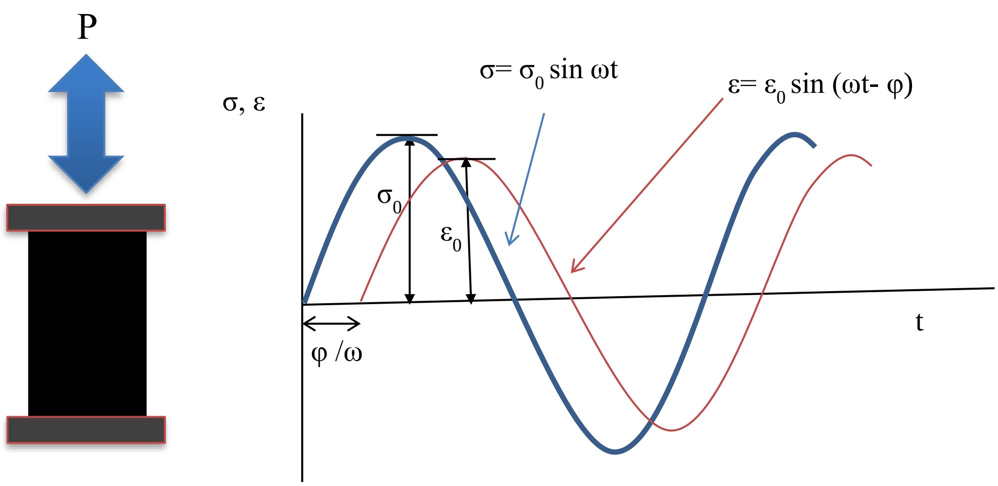


Figure 2 - 3: Stress-strain response of viscoelastic material

2.4 Pavement Distresses

Road pavement is subject to external loads including mechanical loading induced by heavy traffic and thermal loading induced by thermal changes. The applied loads, along with environmental conditions result in pavement deterioration which, in some cases, occurs even before its expected service life. Pavement damage usually occurs in the form of permanent deformation (surface rutting), fatigue failure and low-temperature cracking. Figure 2 - 4 illustrates schematic of pavement distresses. Every year, large amount of money is spent for pavement repairs and rehabilitation (Soltani et al., 2015). Information about three major distresses of flexible pavement is provided in the following sections.

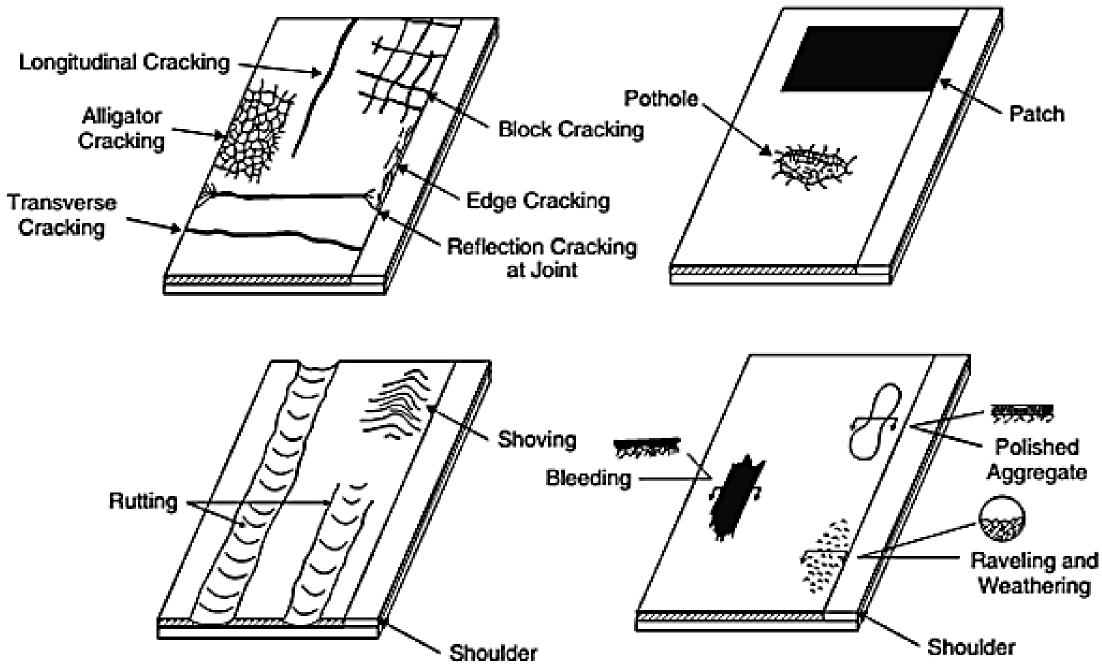


Figure 2 - 4: Schematic of distresses of flexible pavement (Miller et al., 1993)

2.4.1 Fatigue Cracking

Fatigue is one of the most significant distress modes in flexible pavements. This distress manifests itself in the form of cracking, Figure 2 - 5. It is associated with repetitive traffic loading and pavement thickness (Roberts et al., 1991, McGennis et al., 1994). Fatigue life of pavement is affected by different properties of the mixture including type and amount of binder used in the mix; temperature as well as air voids content (SHRP, 1994, NCHRP, 2004). It was also observed that aggregate gradation is an effective factor for fatigue resistance of asphalt mixture (Wen, 2001).

Fatigue resistance is the ability of the asphalt mix to resist repeated bending forces without fracture and cracking. According to structural analysis, fatigue cracks are initiated at the bottom of asphalt layer where the maximum tensile strains accrue, thereafter these cracks propagate to the surface of asphalt mixtures (Baghaee Moghaddam et al., 2011). According to the literature, three phases are defined for propagation of fatigue cracks, namely: crack initiation, stable and unstable fatigue crack growth (Liang and Zhou, 1997). Fatigue cracks usually initiated in the form of microcracks and proceed to macrocracks, these cracks grow due to shear and tensile stresses in the road pavement. Fatigue life of asphalt concrete has an inverse correlation with the amount loads applied by vehicles on road pavements. Further, fatigue life differs significantly among types of asphalt mix (Baghaee Moghaddam et al., 2012).

Figure 2 - 5: Fatigue (alligator) cracking (Miller et al., 1993)

2.4.2 Permanent Deformation (Rutting)

Progressive accumulation of permanent deformation is defined as rutting (Figure 2 - 6). Rutting, which is caused by repeated traffic loading, is sum of the total deformation occurs in each layer of pavement structure. In this regard, asphalt layer has shown a prominent magnitude in rutting (Khodaii and Mehrara, 2009). Ambient temperature and loading magnitude are the two important factors affecting the rutting performance of HMA when asphalt mixtures are likely to be deformed at higher temperatures and under severe loading conditions (heavy and slow moving trucks) (Baghaee Moghaddam et al., 2014). Rutting performance of asphalt mix has close relation with type of road construction, type of mix and air voids content in the asphalt mix. Rheological properties of asphalt such as penetration and viscosity could be influential factors in estimating the rutting performance of HMA mixtures (Muniandy and Huat, 2006, Lu and Redelius, 2007, Fontes et al., 2010).

Figure 2-6

Figure 2 - 6: Rutting damage (Ministère des Transports du Québec, 2007)

2.4.3 Low-Temperature Cracking

Low-temperature cracking (Figure 2 - 7) is one of the pavement distress modes mostly occurs in regions with cold climatic conditions (Das et al., 2013). In such environmental conditions, restrained asphalt mixture contracts which results in building up induced tensile thermal stress in the asphalt layer. When the amount of induced tensile stress exceeds the tensile strength of material, it fractures, and, as a result, transverse crack appears on the surface of the pavement. The temperature at which the fracture happens is called fracture or cracking temperature (Kanerva et al., 1994). There are some regions experiencing large daily temperature fluctuations. In these regions, thermal cracking might also occur (Gajewski and Langlois, 2014).

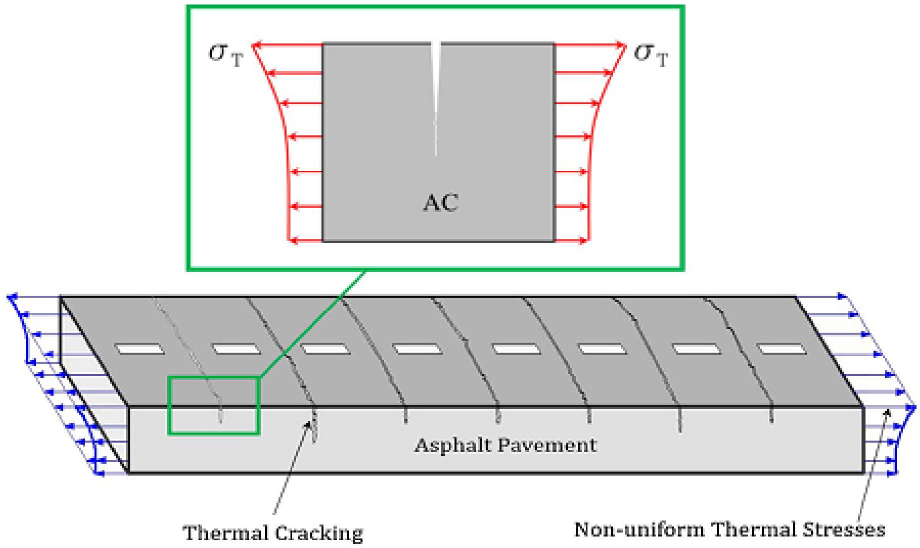


Figure 2 - 7: Low-temperature (transverse) cracking (Behnia et al., 2016)

2.5 Hot Mix Asphalt Design Methods

To manufacture a durable asphalt mix with acceptable performance, parameters such as aggregate type and fraction, type and amount of asphalt binder need to be precisely determined. Different methods have been previously developed and used for HMA mix design. According to the literatures, three most well-known methods are Marshall, Hveem and Superpave design procedures.

Marshall and Hveem mix design methods have been used extensively over the past few decades. Each procedure uses a series of laboratory tests to select the optimum asphalt content. Hveem Mix design method was developed by Francis Hveem when he was a Resident Engineer in the California Division of Highways in the late 1920s and 1930s. The Hveem stabilometer is used in this method which measures asphalt mixture's ability to resist lateral movement under a vertical load. This method is currently in use in several western states (Superpave Fundamentals, 2000).

Marshall mix design procedure has been adopted by U.S. Army Corps of Engineers (USACE). The Marshall mix design consists of three major steps: (1) aggregate selection, (2) asphalt binder selection, and (3) optimum asphalt binder content determination. Nowadays, Marshall method, despite its shortcomings, is probably the most used mix design method worldwide (Baghaee Moghaddam et al., 2015b). Marshall compactor has been losing popularity in the pavement laboratories because it relies on falling weight which does not simulate the field compaction (Tapkın and Keskin, 2013).

In 1987, the Strategic Highway Research Program (SHRP) began developing a new system for specifying asphaltic materials. The final product of the SHRP asphalt research program is a new system called Superpave, short for Superior Performing Asphalt Pavements. Superpave was intended to be an improvement over the Hveem and Marshall methods by adopting a new system for selecting and specifying asphalt binders (Asphalt Institute, 2001).

2.5.1 Superpave Mix Design Method

Superpave is a system of mixture design for asphalt mixtures based on 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. Superpave mix design procedure includes five different steps as follows:

i. Selecting asphalt binder and aggregate particles to meet the Superpave testing requirements;
ii. Developing several aggregate trial blends to meet the Superpave gradation requirements;
iii. Blending asphalt with the trial blends and short-term oven aging the mixtures;
iv. Mixture compaction using Superpave Gyratory Compactor (SGC) and measuring the volumetric of the trial blends;
v. Selecting the best trial blend as design aggregate structure; and compacting samples of the design aggregate structure at several asphalt contents to determine the design asphalt content.

2.5.1.1 Asphalt Binder Performance Grade (PG)

A unique binder grading method is developed in Superpave system which is known as binder Performance Grade (PG). In this approach, binders are selected with respect to climate and traffic conditions in which pavement is intended to serve. PG consists of two parts or performance temperatures (PG HH-LL). The first two digits from the left shows the highest temperature at which physical property requirements need to be met, and the second two digits shows the lowest temperature at which physical properties must be met. For instance, a binder which is classified as PG 76-22 means that the binder must meet high-temperature physical property requirements at least up to temperature of 76˚C, and low-temperature physical properties must be met down to minus 22˚C.

The high pavement design temperature is obtained at a depth of 20 mm below the pavement surface using the seven-day average high air temperature. However, for low pavement design temperature the lowest pavement surface temperature is considered. It should be noted that the PG grades selected by Superpave system apply for typical highway loading conditions. In case of standing or slow-moving 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 (ESALs) (Asphalt Institute, 2001, Jenks et al., 2011).

Table 2 - 1 lists the binder tests that are typically conducted to determine PG. These tests can be related directly to the field performance by engineering principles. Figure 2 - 8 also presents temperatures at which the tests are conducted and the property that can be evaluated using each test method.

Table 2 - 1: Superpave binder tests

Superpave Binder Test	**Purpose**
Dynamic Shear Rheometer (DSR)	Measure properties at high and intermediate temperatures
Rotational Viscometer (RV)	Measure properties at high temperatures
Bending Beam Rheometer (BBR) Direct Tension Tester (DTT)	Measure properties at low temperatures
Rolling Thin Film Oven (RTFO) Pressure Aging Vessel (PAV)	Simulate hardening (durability) characteristics

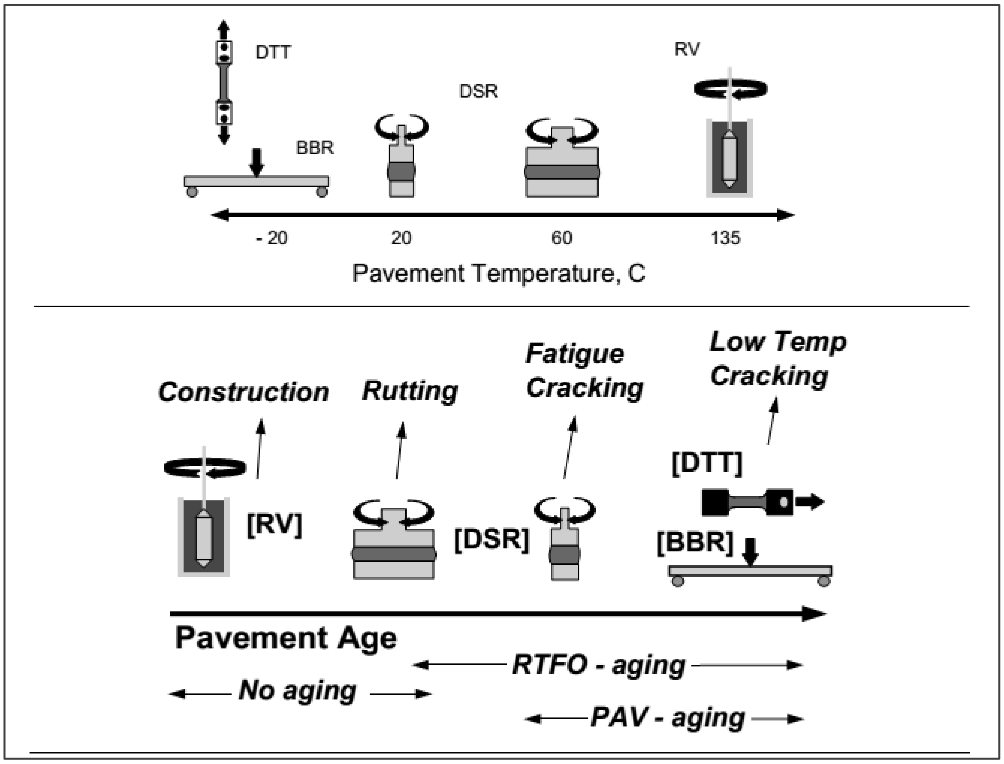


Figure 2 - 8: Schematic view of binder tests set up (Superpave Fundamentals, 2000)

2.5.1.2 Superpave Gyratory Compactor

One of the main differences between Superpave mix design and the other mix design procedures is the compaction method. Superpave Gyratory Compactor (SGC) is used in superpave mix design procedure. The major difference between SGC and other compaction methods is that SGC can provide information about the compactibility of the mix by capturing data (specimen height) during compaction. The notion behind that is realistically measuring the densities achieved under actual pavement climate and loading conditions. In addition, SGC can accommodate large aggregates, and can measure compaction ability (compactibility) of the mix. The specimen density can also be estimated during the compaction knowing the mass of material placed in the mold, inside diameter of the mold and the specimen height. Figure 2 - 9 provides a schematic view of mold configuration for a typical SGC. Three parameters affect the compaction effort in SGC including:

i. amount of applied vertical pressure (Ram pressure),
ii. angle of gyration (compaction angle) and,
iii. number of gyrations.

According to Superpave design procedure, the vertical pressure and compaction angle have constant values of 600 kPa and 1.25˚ respectively. However, design number of gyrations (N_{des}) is determined according to the traffic level. It means at higher traffic level more compactive effort should be performed to achieve a higher mix density. Initial number of gyrations (N_{ini}) is also used to measure the compactibility of mixture. In addition, N_{max} which is the maximum number of gyrations provides indication of the highest mix density which should not be exceeded in the field.

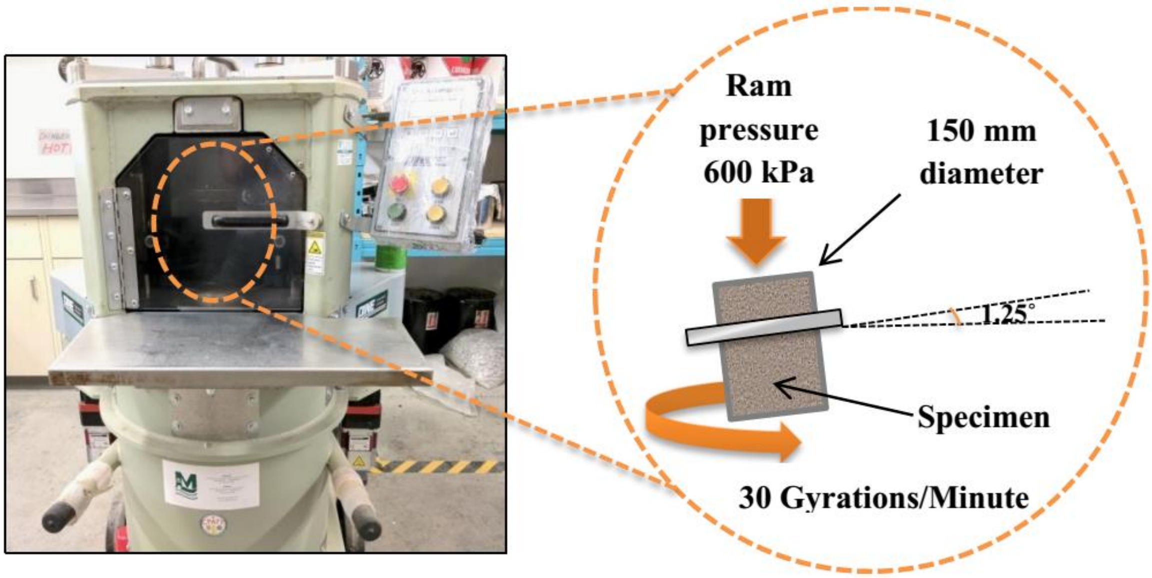


Figure 2 - 9: Superpave Gyratory Compactor (SGC)

2.5.2 French Mix Design Method

French mix design approach is a performance-based which consists of five different levels as shown in Figure 2 - 10.

- ***Level 0:*** The minimum binder content is determined empirically based on the gradation and a value called "richness factor or modulus" to insure a minimum thickness of the binder film in the mix.
- ***Level 1:*** The compaction aptitude and the moisture resistance are assessed using the French Gyratory Shear Compactor (Called PCG) and the Duriez moisture resistance test. If the designed mix meets the requirements, the designer moves to the next level. Otherwise, the binder content or the gradation should be adjusted, or an anti-stripping agent can be used. In the French mix design method, the design air voids is different from a mix to another unlike the Superpave method where the design air voids is considered 4% for all mix types.

- ***Level 2:*** The mix is tested for rutting resistance at 60°C using the French LCPC rutting tester.
- ***Level 3:*** The value of the complex modulus of the asphalt mix is determined using the two-point bending complex modulus test at 15°C and 10 Hz.
- ***Level 4:*** When required, the fatigue resistance of the asphalt mix should be assessed using the two-point bending fatigue test. Trapezoidal specimens are tested at different strain levels to determine the parameters of Wöhler (or fatigue) curve. The slope of the fatigue curve and the value of (ε_6), which is the strain that leads to the failure at 1,000,000 cycles, are then determined and used in the pavement structural design.

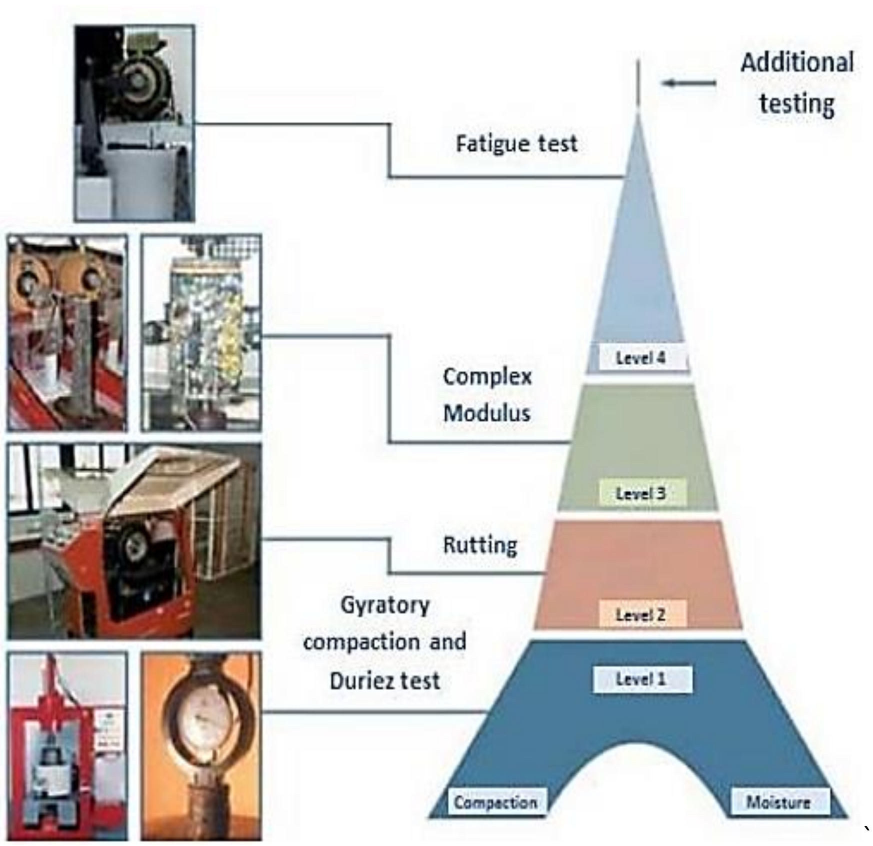


Figure 2 - 10: French mix design approach (Delorme et al., 2007)

In fact, French asphalt mix design approach is an enhanced version of Superpave method which includes performance testing parameters into design consideration (Pereira and Pais, 2017).

2.6 Aggregate Packing

Optimization of particle size distribution of asphalt mixes has been done empirically for years. The mix design methods allow the designers to select the gradation of the asphalt mix with respect to gradation envelopes developed empirically for different mix types. As a result, the mix design became an art as an experienced designer would be able to optimize the gradation and enhance the compaction. There exists however some basic and advanced tools and procedures to optimize the gradation. Bailey method is one of the common empirical methods aiming to optimize the asphalt mix design (Vavrik et al., 2002). Baron model is another approach that has been successfully used to optimize the gradation and mix design of Stone Mastic Asphalts (Perraton et al., 2007). Another avenue is using a more advanced and analytical approach based on granular packing theory. This approach has been introduced by De Larrard (1999) for Portland cement concrete mix design optimization.

The three illustrations in Figure 2 - 11 show how the optimization of the gradation would increase the packing and reduce the risk of segregation. In the following subsections, Bailey theory as one of the most used packing methods in the field of asphalt technology is explained; followed by introducing Compressible Packing Model (CPM) as a potential technique in optimization of asphalt concrete mix design by maximizing the Packing Density while meeting good mix stability.

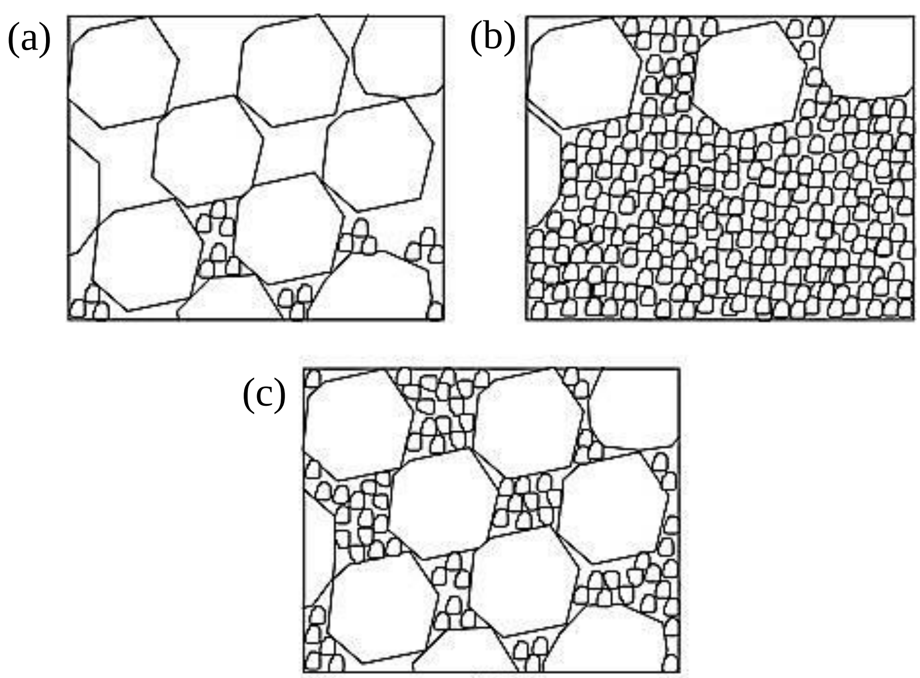


Figure 2 - 11: impact of gradation on compaction and segregation of the mix (a and b: poorly graded mixes, c: well graded mix)

2.6.1 Bailey Method

However, introducing Superpave method was a turning point in HMA mix design in Ontario, in some cases designers had to perform numerous trials to get a proper aggregate blend. In addition, some questions still had to be answered. For instance, should the mix be designed on coarse or fine side? How will this mix work in the field? Will it segregate? Will it be difficult to compact, or difficult to achieve sufficient VMA? In such cases, Bailey method could be used to provide a good starting point when adjustment was required. The method was developed by Robert. D. Bailey in the early 1980s. This method is one of the empirical methods aiming to optimize the asphalt mix design (Vavrik et al., 2002). The primary purpose of this methodology is to control the mix properties such as volumetrics, segregation, workability, and compactibility during construction by focusing on aggregate packing. Bailey method consists of four basic principles as below:

i. How to differentiate between coarse and fine aggregate particles. The coarse fraction creates voids and the fine fraction fills in the voids.
ii. How to analyse coarse fraction which influences the packing of fine fraction.
iii. How to analyse coarse part of the fine fraction, which relates to the packing of the overall fine fraction in the blend.
iv. How to analyse fine part of the fine fraction, which relates to the packing of the fine portion of the gradation in the blend.

According to Bailey method procedure, the result of 0.22×NMAS (or NMPS) of a mix is threshold between coarse and fine in a combined blend gradation. Based on definition, the Primary Control Sieve (PCS) is the closest sieve to the result of 0.22×NMAS.

In this method, determining unity weight and consequently air voids in aggregate mix is vital. In order to achieve this aim, Loose Unit Weight (LUW) and Rodded Unit Weight (RUW) of aggregate may need to be calculated according to AASHTO T 19. Thereafter, amount of Chosen Unit Weight (CUW) needs to be determined. The CUW value should be picked up according to mix type and must be within ranges provided in Table 2 - 2.

Table 2 - 2: Chosen unit weight (CUW) requirement

Mix type	CUW	
	%	Reference Unit Weight (LUW or RUW)
Fine-graded	90 or less	LUW
Coarse-graded	95 to 105	LUW
SMA	110 to 125	RUW

Figure 2 - 12 summarizes the steps for coarse/fine-graded mixes. Additionally, Table 2 - 3 and Table 2 - 4 respectively provide ratio guidelines for coarse-graded and fine-graded mixes (Aurilio et al., 2005).

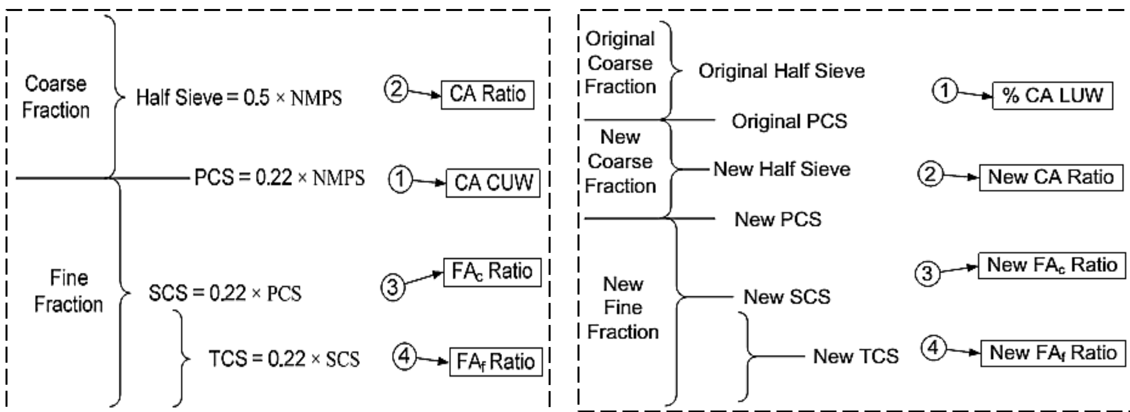


Figure 2 - 12: Combine blend evaluation for coarse-graded mixes (Left) and fine-graded mixes (Right) (Xiao, 2009)

Table 2 - 3: Ratio guidelines for coarse-graded mixes

NMAS	37.5 mm	25 mm	19.0 mm	12.5 mm	9.5 mm	4.75 mm
CA Ratio	0.80 - 0.95	0.70 - 0.85	0.60 - 0.75	0.50 - 0.65	0.40 - 0.55	0.30 - 0.45
FA_c Ratio	0.35 - 0.50					
FA_f Ratio	0.35 - 0.50					

Table 2 - 4: Ratio guidelines for fine-graded mixes

NMAS	37.5 mm	25 mm	19.0 mm	12.5 mm	9.5 mm	4.75 mm
New CA Ratio	0.6 - 1.0					
New FA_c Ratio	0.35 - 0.50					
New FA_f Ratio	0.35 - 0.50					

Although Bailey method is intuitive, there are several points that still need clarification (Shashidhar and Gopalakrishnan, 2006):

a. It is considered that coarse aggregates form the aggregate skeleton, and that idea behind definitions of coarse and fine aggregates is not clear (generally, coarse aggregates are considered to be greater than Sieves No. 4 (4.75 mm) or No. 8 (2.36 mm).

b. The origin of the cut-off for the coarse aggregates (0.22 × NMAS) is uncertain, and it is unclear that the aggregates below this cut-off would participate in aggregate skeleton. Role of aggregate gradation on this cut-off is also uncertain.

c. Role of fines is not clear. Do they just fill the voids in the skeleton and make the mix more impermeable?

2.6.2 Compressible Packing Model (CPM)

The Compressible Packing Model (CPM) was introduced by de Larrard, 1999 to predict the packing density of aggregates, cements and cementitious materials along with properties of fresh and hardened concrete. CPM is developed based on the concept of virtual packing density which is defined as "the maximum packing density which can be achieved by placing the grains one by one while keeping their original shape".

Imagine a mix of n classes of monosize grains. In this mix the partial volume ϕ_i is the volume occupied by class i in a unit bulk volume of the granular mix. ϕ_i^* is the maximum volume that particles i may occupy in the mix, given the presence of the other particles, or, in other words, the maximum value of ϕ_i if the mix was fully packed by an excess of i grains. Residual packing density of each class of grains is shown as β_i. In any mix of grains, one class, e.g. class i, may be dominant. In this case, β_i is the residual packing density which is the virtual packing density displayed when the class is isolated and fully packed.

To have better understanding about the packing density calculation, let's consider a mix of three grain classes, n=3 (Figure 2 - 13). In this case, the middle class (class 2) is dominant and we have $d_1 \geq d_2 \geq d_3$. To calculate the packing density of the overall mixture we consider that the bulk volume of the class 2 fills the space between the class 1 grains. Similarly, the volumes of the class 3 grains inserted into the voids of class 2 grains. To get the optimum value, the effect of two interactions must be considered including the wall effect and loosening effect (also called as interference effect). The wall effect happens in the vicinity of coarser grains though the loosening effect occurs as result of loss of stone-to-stone contact due to exerting too much fine in a mix. In CPM model, these two interactions are considered as additives. This means a possible intersection between the perturbed zones is neglected (De Larrard and Sedran, 2002).

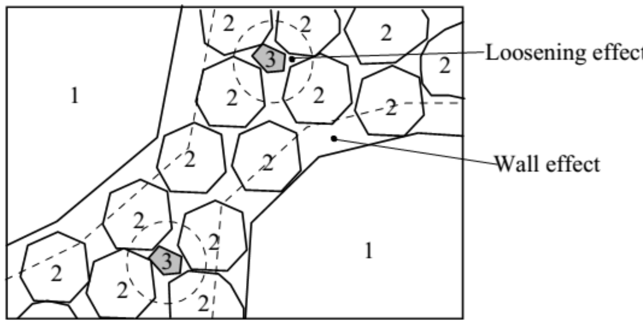


Figure 2 - 13: Ternary packing of particles, where the intermediate class is dominant (De Larrard and Sedran, 2002).

For this case, the virtual packing density can be calculated using the following Equation 2.1:

$$\Upsilon = \Upsilon_2 = \frac{\beta_2}{1-\left[1-\beta_2+ b_{21}\beta_2\left(1-{}^{1}/_{\beta_1}\right)\right]y_1-(1-{}^{a_{23}\beta_2}/_{\beta_3})\, y_3} \qquad \text{Equation 2.1}$$

In this equation, β is virtual packing of each aggregate class compacted alone; y is the volume of the fractions in the mix; Υ_2 is the virtual packing of the blend when class 2 is dominant; a_{23} is interaction coefficient describing the “loosening effect” existed between class 2 and class 3; and b_{21} describes the “wall effect” interaction between class 1 and class 2. Therefore, when *n* classes of aggregates are used, depending on which class of grain is dominant, *n* equations of virtual packing density can be developed, and the one that gives the lowest amount is chosen as the real virtual packing density. Up to our knowledge, CPM has not been used for the optimization of high modulus asphalt mixes before. In this thesis, the use of CPM was investigated on optimizing (maximizing) the packing density of aggregate blend in HMA without lowering the mix stability. Complete and detailed information on CPM is provided in the Chapter 3 of this document.

2.7 Enrobé à Module Élevé (EME)

“Enrobé à Module Élevé” or “Asphalt with an Elevated Modulus” (EME) is a type of asphalt concrete that represents high modulus/stiffness, high durability, superior rutting performance and good fatigue resistance. It has been developed in 1980’s in Laboratoire Central des Ponts et Chaussées (LCPC), France, in co-operation with road enterprises (Sybilski et al., 2010). EME was firstly designated to reinforce old pavement structure and reconstruct thinner layers in urban areas due to having underground facilities such as pipes and curbs which restricts the pavement thickness to a specific value (Corte, 2001). Additionally, it has the advantage of avoiding the complete removal of old asphalt layer as it contributes to lower pavement thickness (Capitão and Picado-Santos, 2006). In subsequent, it was utilized to reduce the pavement construction cost by reducing the thickness of road pavement especially when the aggregate used had low (weak) crushing index value or in case the traffic was intense, slow and channeled (Caroff and Corté, 1994).

The first type of EME was patented in 1980 and just after about five years of first implementation, in 1985, significant number of applications was reported. The oil crisis in 1980s was another reason that helped the reputation of this type of mixture because less amount of asphalt binder could be incorporated in construction of EME mixes. Since the first application of EME, it has been included in several manuals, including: Pavement Design for Motorways, 1994, Road Directorate's catalogue of new pavements, 1998. Over the years and because of the level of development and diversification of EME mixes, it was decided to be included and codified by AFNOR standard, published in October 1992, under reference number NF P 98-140 (1992).

According to the European specifications, EME mixes have continues aggregate gradation which usually contains 32- 35% of material smaller than 2 mm and about 7-8% of material smaller than 0.075 mm. Additionally, the asphalt layer thickness can vary with respect to maximum aggregate size of the mix. Maximum aggregate sizes of 10, 14 or 20 mm can be used for the layers thickness between 6-10 cm, 7-12 cm and 10-15 cm respectively (Picado-Santos et al., 2003, Espersson, 2014).

Hard asphalt binder is used in construction of EME. EME is manufactured using high amount of asphalt with lower air voids (close structure) to assure workability, durability and fatigue resistance of the mixture (Sybilski et al., 2008, Haritonovs et al., 2013).

Based on the successful application of EME in binder and base course layers, some researchers have successfully tried to use high stiffness mixes for wearing course layers. In France, this type of wearing course layer is called "bétons bitumineux à module élevé" or BBME. BBME is less stiff in comparison with EME. The main purpose of using BBME was to reduce the thickness of the wearing course layers while maintaining the same mechanical performance under traffic loads (Marsot, 1993). BBME was used in the wearing course layers of urban arterial roads, toll gate and fuel supplying spots. BBME's characteristics were regulated in the French standard NF P 98-141 (Capitão and Picado-Santos, 2006).

EME base layers are usually covered with Béton Bitumineux Très Mince (BBTM). BBTM has similar composition to Stone Mastic (or Matrix) Asphalt (SMA) mixture. The basic difference between BBTM and SMA mixtures is that BBTM is a non-continuous gap-graded mixture with lack of fine aggregate (sand) fraction. As it was mentioned by Sybilsky et al., (2010) BBTM layer is thinner than SMA mixture; it has thickness less than 35 mm, mostly 20 mm.

2.7.1 EME Binders

Technically, high stiffness asphalt mixture can be designed using lower binder content or utilizing stiffer asphalt binder; however, the first option may not be appropriate because durability of mixture will adversely affected if lower amount of binder is used (Maupin and Diefenderfer, 2006). Therefore, utilizing hard grade asphalt binder (e.g. 15/25, 10/20 or even 5/15 penetration grade asphalt at 25˚C) in manufacturing high modulus asphalt is inevitable to assure high mixture stiffness and resistance to permanent deformation (Rohde et al., 2008). The mechanical properties of high grade asphalt binder heavily depend on binder manufacturing process. Hard grade asphalt binders can be obtained using different methods. These types of asphalt binders were firstly produced using blowing process. The binder produced using this method showed to be very brittle and vulnerable to fatigue and low-temperature cracking. As a result, other techniques such as vacuum distillation and propane-precipitated-asphalt have been designated to produce the hard grade asphalt binder (Corte, 2001).

Polymer modified asphalt binders can be used as alternative to the use of hard grade virgin asphalt binder. Using polymer modifiers, such as styrene-butadiene-styrene (SBS) and polyethylene (PE), in developing hard grade asphalt binder is promising which can help to improve the cracking resistance of the binder. EME mixes are then fabricated using high grade asphalt binder to assure the rutting resistance of the mix. In addition, higher amount of binder content is used to assure the durability of the mixes. It is worth mentioning that using hard grade asphalt binder with high viscosity needs higher mixing temperature and time to coat the aggregate particles (e.g. around 170˚C to 180˚C for PG 10/20 asphalt binder) (Corte, 2001).

One of the leading countries in producing hard grade asphalt binder is France. According to the literature, in France, production of hard grade binder was initiated in 1980, and it reached to 39,000 tonnes and 100,000 tonnes in 1990 and 2000 respectively (Corte, 2001). It is good mentioning that in 1998, France was placed in leading position for the use of hard grade binder by the World Road Association.

Table 2 - 5 lists the requirement for hard penetration grade binder according to EN 13924 (2006).

Table 2 - 5: Requirements for hard grade penetration binder according to the European Standards

Requirement	Property	Unit	Method	Penetration grade	
				10/20	**15/25**
Consistency at mid-temperatures	Penetration at 25°C	0.1 mm	EN 1426	10 to 20	15 to 25
Consistency at high temperatures	Softening point	°C	EN 1427	58 to 78	55 to 71
	Dynamic viscosity at 60°C	Pa.s	EN 12596	≥ 700	≥ 550
Long-term performance (resistance to hardening)	Mass change	%	EN 12607-1 or -3	n/a	≤ 0.5
	Retained penetration	%	EN 1426	n/a	≥ 55
	Softening point after hardening	°C	EN 1427	n/a	≥ original minimum +2
	Increase in softening point	°C	EN 1427	≤ 10	≤ 8
Other properties	Kinematic viscosity at 135°C	mm^2/s	EN 12595	≥ 700	≥ 600

2.7.2 EME Aggregates

European standard has specified particle size limits for EME mixes which depends on the nominal aggregate size (NAS) of the mix (D). Four sieve sizes are used for the grading envelope including: 1.4D, D, 2 mm and 0.063 mm. As is mentioned in EN 13043, D and an optional sieve size between D and 2 mm should be chosen using specific sieve sizes. Aggregate gradation limits of three different EME categories based on NAS are provided in Table 2 - 6. The particle gradation limit of each category is provided in Table 2 - 7.

Table 2 - 6: Gradation limits for AC EME-mixes according to the nominal aggregate size of the mix (D) (NF EN 13108-1, 2007)

D (mm)	10		14		20	
Sieve size (mm)	**Passing (%)**					
	Min	**Max**	**Min**	**Max**	**Min**	**Max**
1.4D	100	100	100	100	100	100
D	90	100	90	100	90	100
2	10	60	10	50	10	50
0.063	2	12	0	12	0	11

Table 2 - 7: Grading control points of EME mixes (Delorme et al., 2007, NF EN 13108-1, 2007)

Sieve (mm)	0/10		0/14		0/20	
	Minimum (%)	Maximum (%)	Minimum (%)	Maximum (%)	Minimum (%)	Maximum (%)
31.5	-	-	-	-	100	100
20	-	-	100	100	90	100
14	100	100	90.0	100	-	-
10	90	100	-	-	-	-
6.3	45.0	65	50.0	70.0	45.0	65.0
4	-	-	40.0	60.0	40.0	60.0
2	28.0	38.0	25.0	38.0	25.0	38.0
0.063	6.3	7.2	5.4	7.7	5.4	7.7

The quality of aggregates is an influential factor of the mix stiffness as well as its performance. EME mix should be manufactured with crushed aggregates which have high aggregate angularity. The flakiness index of aggregates should be restricted to 25 (Delorme et al., 2007).

The possibility of using aggregate particles with lower quality is a primary advantage of EME. This advantage is adopted from inherited structure of EME mixes which have close structure and incorporation of stiffer asphalt binder with higher content. These characteristics result in better resistance to induced stresses and strains at road base layer, and higher durability of these types of mixes.

Among the aggregate types, limestone or dolomite aggregates have weak mechanical properties. Thus, the use of these types of aggregates has been reduced even at places where they are locally available. Incorporating limestone aggregates in asphalt mixture can contribute to better binder-aggregate affinity, better moisture resistance (Solaimanian et al., 2006) and improved resistance to aging (Shamsi et al., 2006). It was reported by Birgisson et al., (2003) that basic aggregates such as limestone aggregates may cause microcracking in the course layer. On the other hand, granite (acid aggregate) has shown good mechanical properties and higher resistance to low-temperature cracking; however, it was observed that granite had weaker adhesion which could cause ravelling and separation of aggregates from the binder in presence of water (Sybilski et al., 2010).

2.7.3 EME Mix Design

EME mix design approach is combination of empirical and performance-based test methods; which is more costly and time consuming than the conventional mix design. The most recent specification for EME mix is given in French specification: (NF EN 13108-1, 2007). According to NF EN 13108-1 two classes of EME are defined known as; EME Class 1 and EME Class 2. The difference between these two classes is in richness modulus value as representative of binder film thickness, Table 2 - 8.

Table 2 - 8: Richness modulus of EME classes

EME	**Richness Modulus**	
	Minimum	**Maximum**
Class 1	2.5	3.4
Class 2	3.4	-

There are basically five steps for EME mix design based on French specification. The first step is determining aggregate grading curve and binder content according to the richness modulus. Second step focuses on the compactibility of the mix based on air voids content in compacted mix. In France, French Gyratory Compactor (also called PCG) is used, and the air voids content is measured as the basis of specimen height. EME has relatively lower air voids compared to conventional asphalt mix used in base course layer. Having lower air voids, EME mix would have better durability. Additionally, moisture sensitivity of EME mix should be addressed as the part of the second step using unconfined compression tests (Duriez test) test. The ratio of results before and after conditioning should meet the minimum requirement of 75 % for EME Class 2.

Permanent deformation (rutting) performance of EME mixes is then evaluated in the third step. Rutting test is performed using French LCPC rutting tester at temperature of 60˚C and 30,000 cycles. Afterwards, stiffness of the mixes is characterised at temperature of 15˚C and at 25 Hz loading frequency using two-point bending test on trapezoidal specimens according to EN 12697-26 (2004), method A.

Finally, fatigue characteristics of the mix are evaluated. Two-point bending beam fatigue test is performed on trapezoidal specimens according to EN 12697-24 (2004), method A. This test is performed at 10°C. Different strain levels are selected to determine fatigue curve parameters, slope of the curve as well as ε_6 which is the strain value at 1,000,000 loading cycles. Table 2 - 9 shows the minimum stiffness and ε_6 values for EME classes (Delorme et al., 2007, NF EN 13108-1, 2007).

Table 2 - 9: Stiffness and fatigue requirements for EME classes

EME	**Minimum stiffness modulus at 15 °C and 10 Hz (MPa)**	**Fatigue resistance at 10°C, 25 Hz (microstrain) - ε_6**
Class 1	14,000	100
Class 2	14,000	130

2.7.4 International Implementation of EME

Some countries have attempted to transfer and standardize EME technology based on their climatic and traffic conditions and with respect to the available test methods and locally available materials. The United Kingdom (UK), South Africa, and Australia are among the countries that have done extensive research in the past few years. This section provides information on implementation of EME in these countries.

Manufacturing impermeable base course material with high durability and the superior stress distribution was the main reason behind transferring EME technology to the UK. The LCPC design approach (French method) was used along with locally available materials. In the first trial, EME Class 2 was fabricated with high binder film thickness as a binder course layer. Thin surface course (SMA 14-based on NMAS) was used above the EME layer; and under the layer, Heavy Duty Macadam (HDM) was laid on a granular subbase. The UK trial was a success and the developed mix showed great permanent deformation resistance with excellent durability compared to HDM binder course material. It was also concluded that EME Class 2 is great option for fabricating modern impermeable high-performance flexible pavements (Sanders and Nunn, 2005). Further, according to the British design manual for roads and bridges, an EME foundation needs to meet the minimum surface stiffness of 120 MPa at time of construction (Highway Agency, 2006).

South Africa is a country with a hot climate and consequently permanent deformation of asphalt layers becomes a serious issue in that region. Due to the high permanent deformation resistance of EME mixes, in 2008, a project was initiated by the South African Bitumen Association (SABITA) to transfer EME technology to the region. This project was named "High Modulus Asphalt (HiMA) Technology Transfer (T^2)" (Denneman et al., 2011). The first field trial of EME in South Africa was implemented on a heavily trafficked access road in 2011. Upon the first successful experience of manufacturing EME, an interim report was released in the same year (Denneman and Nkgapele, 2011). SABITA Manual 33 (2013) is the first South African interim EME design procedures.

Potential transfer of EME to Australia was started in 2012 as part of a project by Australian road transport and traffic agencies (Austroads TT1353). The Australian final mix design development and specifications of EME was released in 2017; Austroads publication number AP-T323-17 (2017).

In both the South Africa and Australia experiences, comparative testing was conducted using both the French and local testing procedures. Using this approach, it was possible to compare the relative performance of the mixes. In the Australian experience, the existing and widely used EME materials with known characteristics were shipped from France to Australia to be tested using Australian testing equipment. In addition, EME mixes that were developed with Australian procedures were sent to France for additional testing and evaluation. Using this approach, the design team was able to benchmark EME mix performance test results obtained from Australian test methods with those of the French. Information about the testing results was provided in the Australian interim mix design guide (Petho et al., 2014). Table 2 - 10 summarizes EME design requirements based on the above experiences.

Table 2 - 10: EME performance requirements (Sanders and Nunn, 2005, Denneman et al., 2015)

Country	Test	Standard Method	EME Performance Requirements	
			Class 1	Class 2
France	Gyratory compactor, air voids after 100 gyrations	EN 12697-31	≤ 10%	≤ 6%
	Moisture sensitivity, Duriez	EN 12697-12	≥ 0.7	≥ 0.7
	Rutting, Wheel tracking (large device) at 60°C and 30,000 cycles	EN 12697-22	≤ 7.5% strain	≤ 7.5% strain
	Stiffness, Two point bending flexural modulus 15°C, 10 Hz	EN 12697-26	≥ 14 GPa	≥ 14 GPa
	Fatigue, Two point bending 10°C, 25 Hz to 50% stiffness reduction	EN 12697-24	$\varepsilon_6 \geq 100\mu\varepsilon$	$\varepsilon_6 \geq 130\mu\varepsilon$
The United Kingdom	Gyratory compactor, air voids after 100 gyrations (0/14 mix)	EN 12697-31	N/A	≤ 6%
	Moisture sensitivity, Duriez	Based on NF P 98 251-1	N/A	≥ 0.75
	Rutting, Wheel tracking (large device) at 60°C and 30,000 cycles	EN 12697-22	N/A	≤ 7.5% strain
	Stiffness, Indirect Tensile Stiffness Modulus	DD 213: BSI 1996	N/A	5.5 GPa
	Fatigue, Two-point bending 10°C, 25 Hz to 50% stiffness reduction	NF P 98-261-1*	N/A	$\varepsilon_6 \geq 130\mu\varepsilon$
South Africa	Gyratory compactor, air voids after 45 gyrations	ASTM D6926	≤ 10%	≤ 6%
	Moisture sensitivity, Modified Lottman (including freeze-thaw)	ASTM D4867	≥ 0.8	≥ 0.8
	Rutting, RSST-CH, 55°C, 5,000 repetitions	AASHTO T320-03	≤ 1.1% strain	≤ 1.1% strain
	Stiffness, Dynamic modulus test at 15°C, 10 Hz	AASHTO TP 79	≥ 16 GPa	≥ 16 GPa
	Fatigue, Four point bending at 10 Hz, 10°C, to 50% stiffness reduction	AASHTO T 321	$\varepsilon_6 \geq 210\mu\varepsilon$	$\varepsilon_6 \geq 260\mu\varepsilon$
Australia	Gyratory compactor, air voids after 100 gyrations	Based on EN 12697-31	N/A	≤ 6%
	Water sensitivity, Modified Lottman (including freeze-thaw)	AGPT T232	N/A	≥ 0.8
	Rutting, Wheel tracking (small device) at 60°C and 30,000 cycles	AGPT T231	N/A	≤ 4.0 mm
	Stiffness, Four-point bending flexural modulus 15°C, 10 Hz	AGPT/T274	N/A	≥ 14 GPa
	Fatigue, Four-point bending at 20°C, 10 Hz to 50% stiffness reduction	AGPT/T274	N/A	$\varepsilon_6 \geq 150\mu\varepsilon$

* Note: It was recommended that the minimum ε_6 value for EME Class 2 (130$\mu\varepsilon$) could be used without the need for testing. This was decided due to good fatigue performance of EME 2 mixes with high binder content.

CHAPTER 3
APPLICATION OF COMPRESSIBLE PACKING MODEL FOR OPTIMIZATION OF ASPHALT CONCRETE MIX DESIGN

This chapter is based on the following published article in the Journal of Construction and Building Materials. Baghaee Moghaddam T, Baaj H. (2018). Application of Compressible Packing Model for Optimization of Asphalt Concrete Mix Design. Construction and Building Materials 159, pp. 530-539. DOI: 10.1016/j.conbuildmat.2017.11.004. Some minor modifications may have been applied to satisfy the examiners' comments.

Summary

Packing of an aggregate blend is a measure reflecting how solid part and air voids would share the volume occupied by the blend. It is usually measured in terms of "packing density". In this paper, Compressible Packing Model (CPM) is described as a potential technique to optimize aggregate blend by optimizing the packing density in asphalt mixes. Gradation envelops for high-performance asphalt mixes or Enrobé à Module Élevé (EME) were determined for two different mix types (12.5 mm NMAS and 19 mm NMAS) using CPM. Further, asphalt mixes were fabricated using two types of modified asphalt binders. Compactibility and volumetrics of the mixes were assessed. Dynamic modulus test was performed to evaluate the rheological behavior of the mixes at elevated temperatures as well as loading frequencies to develop master curves. Results of this study showed that the gradation limits obtained from CPM were very close to the grading control points of EME mixes and that the asphalt mixes had higher compactibility than the conventional mix. Dynamic modulus test results also depicted the designed mixes could meet stiffness requirement of EME mixes, and the mixes behaved more elastically.

3.1 Introduction

Asphalt Concrete (AC) is a composite material composed mainly of aggregate particles and asphalt binder. Large amount of aggregate particles, around 95% by weight, or 85% by volume, is used in AC. Different fractions or sizes of aggregate materials each with specific quantity are used in asphalt mix to assure required mix design (volumetrics and performance) parameters. In traditional concept, large aggregate sizes provide skeleton of the mix that transfer and distribute stresses induced by traffic loads from vehicles to sublayers of pavement structure. In addition, fines together with asphalt binder form mastic which fills the voids between coarse aggregates and make the mix more durable by providing adequate bonding between them.

There has been always a need to determine optimum aggregate gradation that should be used with respect to available aggregate sources/types and fractions to achieve the optimum mix performance in filed. In this regard, Packing Density (PD) of an aggregate blend, which is a measure of how good the aggregate particles would fill up the volume of the blend, can play a key role (Bressi et al., 2016).

According to the literature, much attention has been paid to this concept in the field of Portland cement concrete (PCC) materials specifically in case of Self-Consolidating Concrete (SCC) and High-Performance Concrete (HPC) (Mangulkar and Jamkar, 2013).

There are different methods that have been used for PCC mix proportioning based on PD. One of the methods is Compressible Packing Model (CPM) which was introduced by de Larrard (1999) to predict the packing density of aggregates, cements and cementitious materials along with properties of fresh and hardened concrete. CPM is a mathematical based model and considers combined effects of shape, texture and grading of particles. In addition, CPM includes the compaction method to describe the actual packing density of the mix. Thus, compared to other mixture proportioning methods, CPM is relatively complex. The main objective of this paper is utilizing CPM as potential method to optimize aggregate blend in AC by optimizing the packing density.

3.2 Packing Density (PD)

Packing Density (PD) is the ratio of solid volume of aggregates to bulk volume of them and can be measured under compacted or uncompacted conditions. It is determined by Equation 3.1:

$$\text{Packing density (PD)} = \frac{\text{Solid volume of particles}}{\text{Bulk volume of aggregates}} \qquad \text{Equation 3.1}$$

Further, voids ratio can be calculated using Equation 3.2:

$$\text{Voids ratio} = 1 - \text{PD} \qquad \text{Equation 3.2}$$

Therefore, as PD increases, the voids ratio or mix porosity is reduced. PD can be determined under either condition. If it is determined under compacted condition, the method of compaction needs to be mentioned since compaction energy applied in each method is different. PD value can be very close to one if particles in blended aggregates are mixed together such that smaller particles fill the voids created by larger aggregates indefinitely. However, it is somehow unrealistic to achieve a packing density very close to one since there is always a limitation in particle size distribution. In addition to that, the fine particles cannot be too fine and, therefore, there is always voids remained unfilled. Shape of aggregates also plays important role in PD of aggregate blend, specifically particle Shape Factor and Convexity ratio. A low shape factor and/or a low convexity ratio would adversely affect the packing density since they contribute to large aggregate interlocking and higher voids ratio in the blend (Kwan and Mora, 2002). According to the literature, the major factors affecting the packing property of blended aggregates are:

i. Gradation (e.g. continuously-graded, gap-graded);
ii. Shape of the particles (e.g. cubical, round and flat and elongated particles);
iii. Texture of aggregate surface (e.g. rough, smooth);
iv. Type and amount of compaction effort;
v. Aggregate strength;
vi. Layer thickness (Olard, 2012, Corté and Di Benedetto, 2004, De Larrard and Sedran, 2002, Chanvillard, 1999, De Larrard et al., 1994).

3.3 Compressible Packing Model (CPM)

CPM can predict the packing density of polydisperse blend using three known parameters:

1) packing density of monosize aggregate;
2) size distribution of aggregates and;
3) used compaction energy.

In theory, CPM calculates Virtual packing density of the blend. It is defined as the maximum packing density which can be achieved by placing the grains one by one while keeping their original shapes (De Larrard, 1999). Actual packing can be determined using the virtual value with respect to the compaction method.

In general, blending different classes of monosize aggregate particles would result in higher packing density. However, two interactions between these classes or sizes of aggregates should be considered. These are “wall effect” and “loosening” or “disturbing effect” as illustrated in Figure 3 - 1.

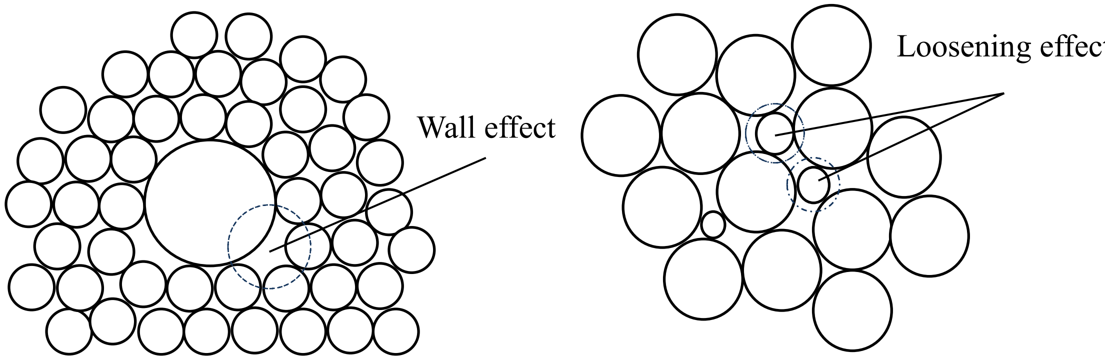


Figure 3 - 1: Wall effect and loosening effect

The wall effect is defined as the interaction of aggregate particles in presence of any type of wall such as mold and pipe. This can also be referred to as interaction exists between coarse aggregates and fines. Additionally, if the amount of fine particles increases in the blend, at some point the courser particles are pushed away by fines due to the loosening effect (Olard, 2012, De Larrard, 1999). Both the wall and loosening effects depend on size ratios of the particles interacting with each other as well as volumetric proportions of the different size particles. This implies that the grading of the aggregate is a controlling factor of these two effects (Wong and Kwan, 2005). In order to obtain virtual and consequently actual packing density of a blend, CPM considers these two interactions as additives. In the literature, several studies have investigated use of CPM in mixture proportioning of Portland cement concrete (HPC and SCC) (Sebaibi et al., 2013, Nanthagopalan and Santhanam, 2009, Kwan and Fung, 2009, de Larrard and Sedran, 1994).

3.4 Experimental Procedures

3.4.1 Materials

In this study, five classes of aggregate particles were obtained from Havelock Quarry located in Northern Ontario, Canada. Gradation curve of each class is plotted in Figure 3 - 2.

Two types of asphalt binders were used (PG 82-28 and PG 58-28). High performance elastomer additives (modifiers) were used to modify PG 58-28 asphalt for enhancing the binder properties. The main advantage of using this type of modification is in adding the additives directly to the mix as method of dry process which is less demanding and more environmentally friendly compared to conventional binder modification in which the additives are mixed with virgin binder using a high shear mixer. It is worth mentioning that 10% of these additives by weight of total modified asphalt binder were used.

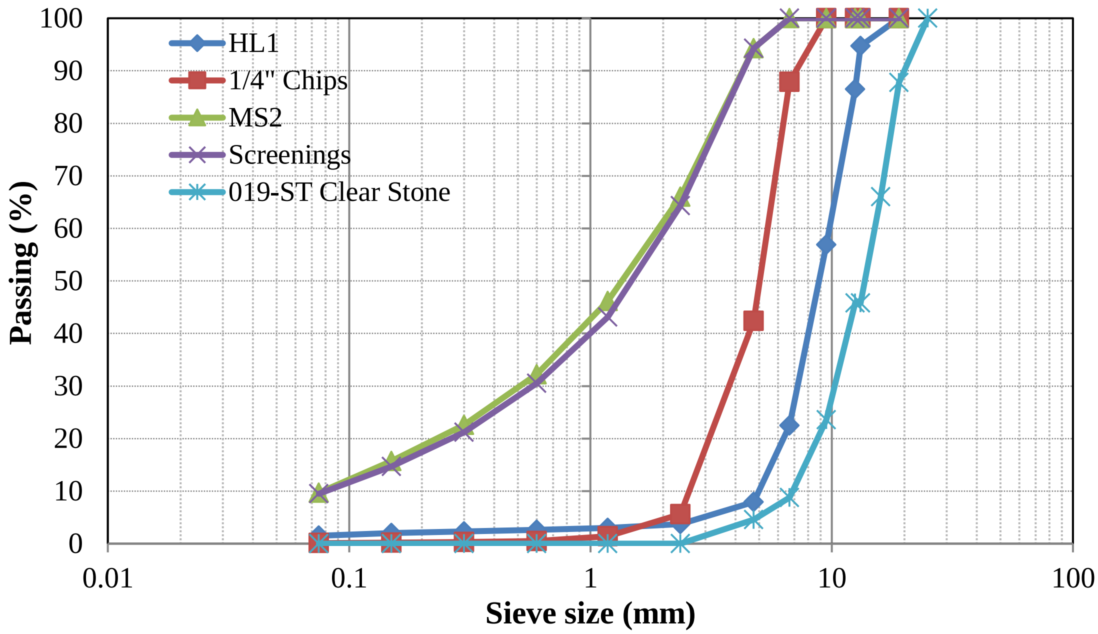


Figure 3 - 2: Gradation curve of each class of aggregate

3.4.1.1 Determination of Aggregates Shape Parameters Using Image Analysis

As explained earlier, the morphology of the aggregates would significantly affect the packing of the aggregates, the compaction and the stability of the asphalt mix under heavy traffic.

Morphological parameters of aggregate particles greatly affect the compaction ability (compactibility) of the mix. Aggregate fractions with the same gradation sizes which obtained from different sources or processing methods would unlikely have the same compaction behavior due to different induced internal friction energy. That is to say different compaction efforts need to be applied for the same aggregate fraction with different morphological parameters to reach the same PD.

The morphological parameters of midsize and fine aggregates were obtained using OCCHIO belt aggregates image analyser (Tierrie et al., 2016) see Figure 3 - 3, and the results are listed in Table 3 - 1. It is worth noting that Concavity and Elongation of particles are gradually reduced for bigger particle sizes while Shape Factor is increased. It can also be noticed that midsize particles have higher Roundness.

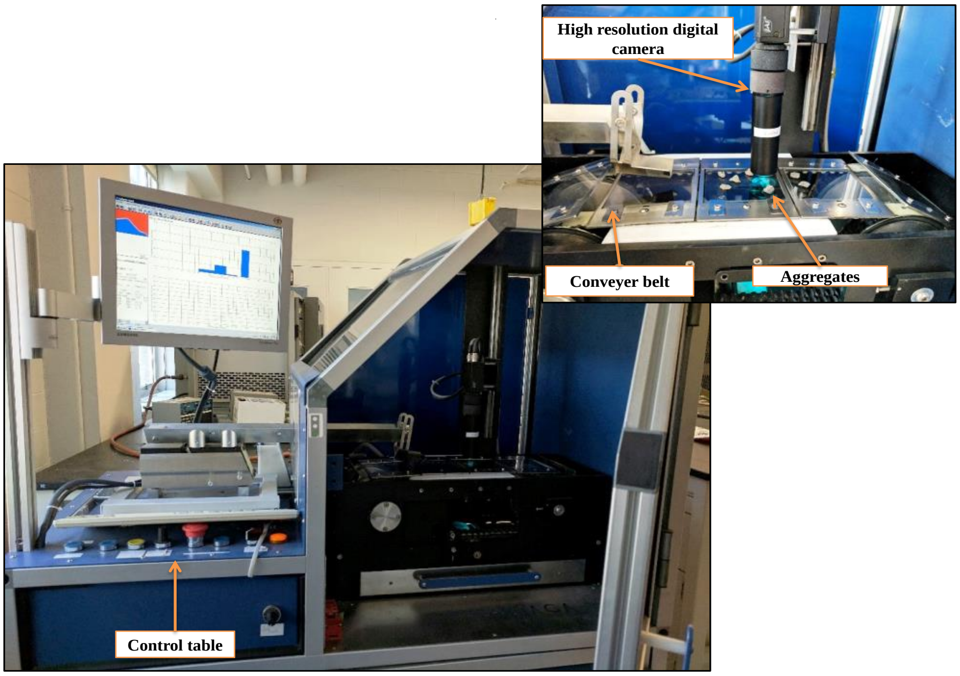
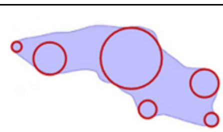
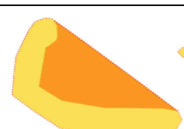
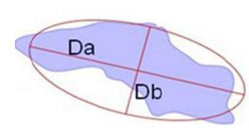


Figure 3 - 3: OCCHIO belt aggregates image analyser

Table 3 - 1: Morphologic parameters of aggregate particles

Fraction (mm)	**Mean Roundness** $Rd = \frac{100 \times \sum(\pi \times D_0)^2}{\sum(P^2)}$	**Mean Concavity** $C = 100 \times \frac{A_{Cvx} - A}{A_{Cvx}}$	**Mean Shape Factor** $F = \frac{4.\pi.A}{P^2}$	**Mean Elongation** $EL = 100 \times (1 - \frac{D_b}{D_a})$
0.075 - 0.30	*31.64*	*15.87*	*52.07*	*43.25*
0.30 - 0.60	*50.05*	*8.21*	*77.02*	*38.46*
0.60 - 1.18	*49.26*	*7.05*	*78.97*	*36.14*
1.18 - 2.36	*45.58*	*6.23*	*79.28*	*36.39*
2.36 - 4.75	*18.81*	*5.95*	*81.11*	*29.83*
				

$\boldsymbol{D_a, D_b}$: *two-dimension diameters of an ellipse having the same area as the particle*
A: *area of the particle,* ***P:*** *perimeter of the particle*
$\boldsymbol{A_{Cvx}}$: *convex hull area (yellow + orange)*
$\boldsymbol{D_0}$ **:** *diameter of equivalent inner circle at each peripheral point*

3.4.2 Dry Packing Density of Aggregates

In order to determine PD of an aggregate blend in a laboratory, the basic procedure is mixing the aggregate particles, put them into a container of known volume, and weigh the aggregate particles in the container. PD, which represents how well the aggregate are packed together, can then be measured by knowing the aggregates weight, density and the volume of the container. From this, the voids content, the volume of voids in the bulk volume of aggregate to be filled up with asphalt, may also be determined.

In this study, in order to determine more realistic values of PD of aggregate blend for asphalt mix, it is measured using Superpave Gyratory Compactor (SGC), according to a method used by Perraton et al., (2007). During the compaction, the SGC base rotates at a constant rate of 30 revolutions per minute and a 600-kPa compaction pressure was applied on the specimen. The angle of gyration was kept constant at 1.25 degrees. It is good mentioning that, using SGC on aggregates only, without adding asphalt binder lubricating effect, may cause attrition, segregation and abrasion when higher number of gyrations (e.g. 100 gyrations) is used. According to this, in later study, Olard reduced the number of gyrations to 20 in order to determine the respective void index of coarse, intermediate and fine aggregate particles (Olard, 2012).

The same approach was used in this study. PD was measured experimentally for each class of aggregate using SGC, and accordingly Virtual PD of blended aggregate was calculated as discussed in the following section.

3.4.3 Optimization of Aggregates Size Distribution Using CPM

In order to make use of CPM, some parameters need to be determined initially including: density, mean size, and packing density of each fraction. These components can be determined experimentally. Thereafter, the Virtual PD of the blend can be calculated using Equation 3.3 (De Larrard, 1999):

$$\Upsilon_i = \frac{\beta_i}{1-\sum_{j=1}^{i-1}\left[1-\beta_i+b_{ij}\beta_i\left(1-{}^{1}/_{\beta_j}\right)\right]y_j-\sum_{j=i+1}^{n}\left[{}^{1-a_{ij}\beta_i}/_{\beta_j}\right]y_j} \qquad \text{Equation 3.3}$$

In this equation, βi is virtual packing of the i-class compacted alone; yi is volume of each fraction in the mix, or the ratio of the volume of size class i to the total solid volume; ϒi is the virtual packing of the blend when class i is dominant and a_{ij} and b_{ij} parameters are interaction coefficients describing the "loosening effect" and "wall effect" of the particles respectively, and can be determined using the following Equations:

$$a_{ij} = \sqrt{1-(1-\frac{d_j}{d_i})^{1.02}} \quad \text{when} \quad d_j \leq d_i \qquad \text{Equation 3.4}$$

$$b_{ji} = 1-\left(1-\frac{d_i}{d_j}\right)^{1.50} \quad \text{when} \quad d_i \leq d_j \qquad \text{Equation 3.5}$$

where d_i and d_j are the diameters of the granular classes i and j as defined by sieve sizes.

3.4.4 Asphalt Mix Fabrication

To fabricate asphalt mixes, aggregates and asphalt binders were heated in oven to reach to the required mixing temperature. Mixing temperatures of 155˚C and 165˚C were selected for PG 58-28 + 10% modifier and PG 82-28 respectively. After they reached to the required temperatures, they were taken out form the oven and mixed using a mixing drum for about 90 s. Thereafter, loose mixes were compacted at their compaction temperatures using SGC. The compaction temperatures were respectively 145˚C and 155˚C for PG 58-28 +10% modifier and PG 82-28. To evaluate the compaction ability of mixes, each mix was conditioned at the compaction temperature for two hours to simulate the short-term aging of asphalt mixes in field then was compacted to the maximum number of gyrations (N_{max}) of 205 according to "Superpave mix design requirement for high traffic roadways" (Asphalt Institute, 2001).

3.4.5 Dynamic Modulus Test

Modulus or stiffness is a fundamental design parameter of flexible pavement (Baghaee Moghaddam et al., 2015a). Asphalt mix is a viscoelastic material and the modulus value is affected by time of loading (loading frequency) and the ambient temperature. Additionally, for a sinusoidal loading, given the viscous properties of asphalt mixes, there exists a phase lag (φ) between stress and strain (Ramirez Cardona et al., 2015)

Non-destructive dynamic modulus test is used to obtain stress-strain relationship of asphalt concrete in pavement laboratories. In this study, dynamic modulus test was conducted according to AASHTO T 342-11 (2011). Dynamic modulus samples were fabricated using SGC. The gyratory compacted samples were cored and cut to produce Ø100×150H mm cylindrical specimens. During the test, a sinusoidal axial compressive stress with different loading frequencies (0.1, 0.5, 1, 5, 10 and 25 Hz) is applied to the specimen at specific temperatures (-10, 4, 21, 37 and 54˚C). The applied stress and the resulting strain response of the specimen were measured continuously during the test using a data acquisition system. The dynamic modulus values were then calculated using Equation 3.6:

$$|E^*(\omega)| = \frac{|\sigma^*|}{\overline{|\varepsilon^*|}}$$ Equation 3.6

where:

$|E^*(\omega)|$ = dynamic modulus for pulsation ω, kPa;

$|\sigma^*|$ = stress magnitude, kPa;

$\overline{|\varepsilon^*|}$ = average strain magnitude.

3.5 Results and Discussion

This section discusses aggregate packing results after using CPM for asphalt mixes. In addition, the compactibility and dynamic modulus test results are presented and discussed.

3.5.1 Aggregate Packing Density Results

Five classes of aggregates were used, namely: 019-ST-MTO-Clear Stone, HL1, ¼" chips, MS2 and Screenings. Table 3 - 2 lists average particle size and specific gravity of each class. Further, PD of each class was obtained using SGC as explained in section 3.4.2. Having these values, CPM was used and the virtual packing densities of blends were calculated where Class i was dominant. The optimum quantity (volume) of each class in the mix was then determined to get the highest PD. This was conducted using Excel Solver and the results are summarized in Table 3 - 3. In this table, the volume ratios (y_i) of each class in the mix were determined after the optimization process. This was done when the largest and the smallest classes, in terms of average size, are dominant in the mix. The calculations were conducted for two different blends based on Nominal Maximum Aggregate Size (NMAS) of the blend, 19 mm and 12.5 mm. As can be seen in Table 3 - 3, the virtual packing densities vary between 0.86 and 0.75, and 0.80 and 0.71 for NMAS 12.5 mm (Mix A) and 19 mm (Mix B) respectively.

Table 3 - 2: Properties of aggregate classes

Class i	**Class name**	**Specific gravity (Dry)**	**Average diameter (d_i-mm)**	**β_i (20 gyrations)**
1	019-ST-MTO-Clear Stone	2.919	13.50	0.623
2	HL1	2.873	8.98	0.616
3	1/4" Chips	2.889	4.53	0.604
4	MS2	2.882	2.41	0.668
5	Screenings	2.895	2.41	0.681

Table 3 - 3: The optimum percentage of each aggregate class in the mix

Mix type	**Class i**	**Dominant class**	$\boldsymbol{y_j}$	$\boldsymbol{\Upsilon_i}$
19 mm NMAS	1	Class 1 (Lower limit)	0.50	0.86
	2		0.02	
	3		0.10	
	4		0.09	
	5		0.29	
	1	Class 4 & Class 5 (Upper limit)	0.32	0.75
	2		0.13	
	3		0.05	
	4		0.25	
	5		0.25	
12.5 mm NMAS	2	Class 2 (Lower limit)	0.50	0.80
	3		0.15	
	4		0.16	
	5		0.19	
	2	Class 4 & Class 5 (Upper limit)	0.31	0.72
	3		0.15	
	4		0.27	
	5		0.27	

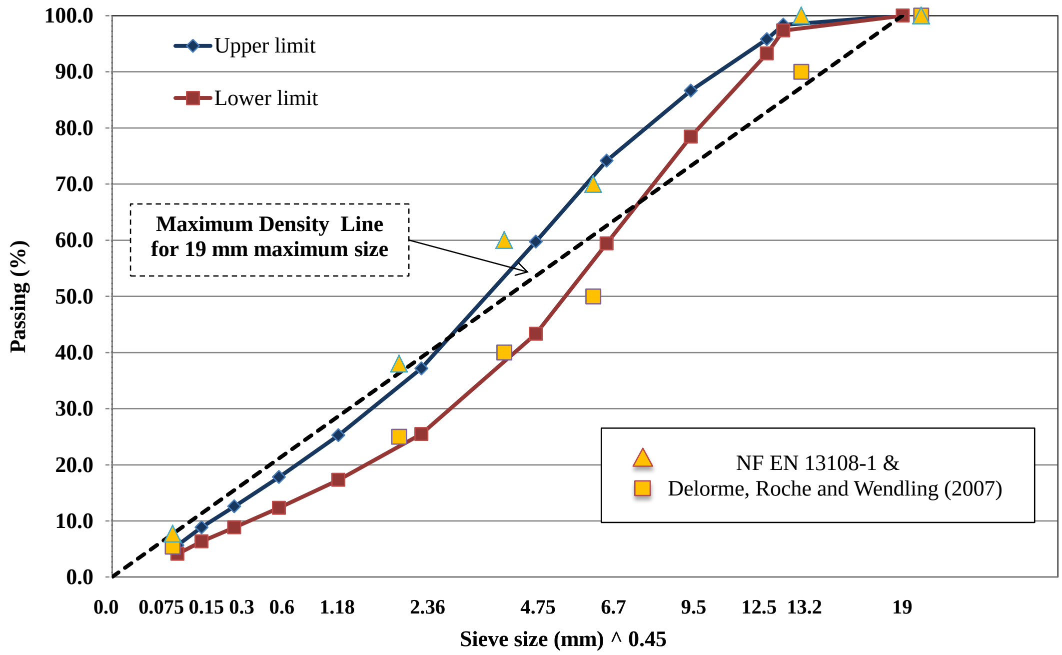
$\boldsymbol{y_i}$ = volume of each fraction in the mix

$\boldsymbol{\gamma_i}$ = virtual packing of the blend when class i is dominant

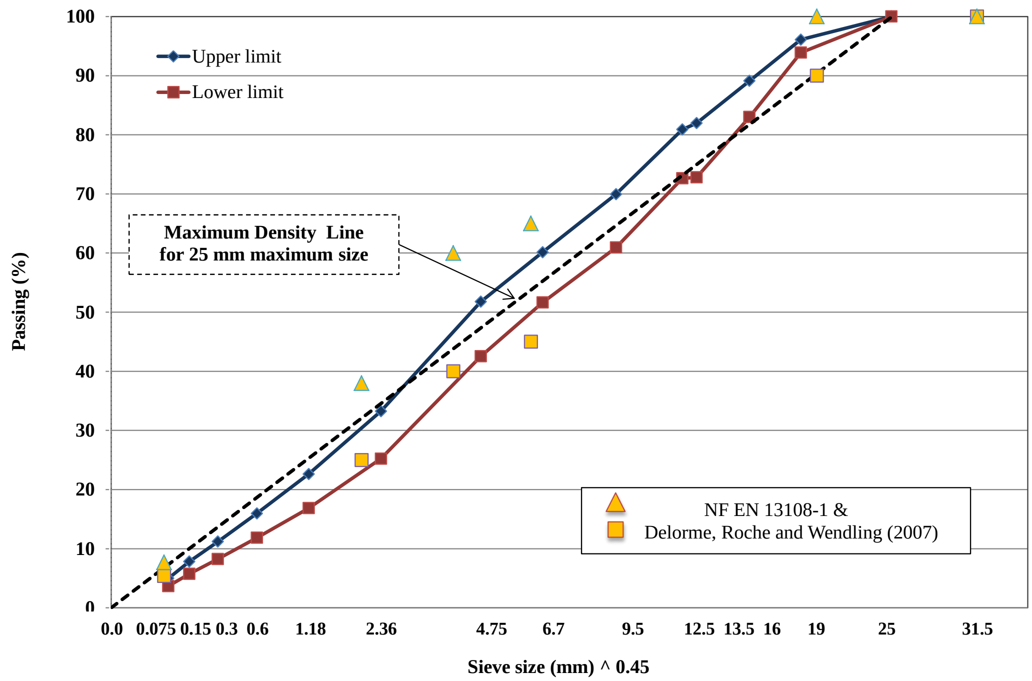
Having the volume ratios, upper and lower gradation limits (or gradation envelopes) are plotted in Figure 3 - 4. For a better comparison, the gradation curves are compared with the theoretical maximum density curve as an accepted traditional method where the passing percentage is plotted against sieve size raised to 0.45 power (Asphalt Institute, 2001). In this concept the Maximum Aggregate Size (MAS) which is one sieve size larger than NMAS is used. It is traditionally accepted that the maximum density gradation curve represents the tightest arrangement that the aggregate particles can fit together (Asphalt Institute, 2014); however, it may not be very accurate approach since regardless of the gradation, other parameters affect aggregate packing as described above in the paper. It can be seen in Figure 3 - 4 (a) that there was considerable difference between the gradation curves obtained from CPM and the maximum density curve which could be due to limitation of aggregate size distribution in the blend as well as the effect of aggregate shape parameters; however, Figure 3 - 4 (b) shows a much better fit for 19 mm NMAS blend.

As another comparison, the obtained results are compared to gradation limits recommended for High Modulus/Performance Asphalt Mix, or Enrobé à Module Élevé (EME 14 and EME 20) in French, by European/French standard (NF EN 13108-1, 2007). EME has closed structure and according to the European specifications it has continuous aggregate grading which usually contains 32-35% of material smaller than 2 mm. It is worth mentioning that EME mixture has higher stiffness modulus compared to conventional mix and is a very good option to be used in binder and base courses of pavement layers which are subject to the highest amount of stress in the pavement structure. Figure 3 - 4 also depicts that the gradation envelope obtained in this study is close to the restriction values recommended for EME 14 and EME 20 mixes (NF EN 13108-1, 2007, Delorme et al., 2007). Therefore, it could be concluded from the results that CPM can be used for optimization of mix gradation for high modulus/performance asphalt mixes.

It should be noted that the packing degree of an aggregate blend highly depends on the number of aggregate fractions, fraction sizes as well as aggregate processing methods (sources), and that introducing one optimized gradation is not applicable for all mixes. Therefore, CPM optimization is an approach that can help to obtain precise blending ratios and to achieve the highest packing degree.



(a)



(b)

Figure 3 - 4: Aggregate gradation limits, (a): 12.5 mm NMAS; (b):19 mm NMAS

3.5.2 Compaction Ability

Workability and volumetrics of AC are basic mix design parameters. In general, the asphalt mix should be workable enough to facilitate placement and compaction without being very tender. According to specifications, for conventional mixes, the air voids should be equal or greater than 11% at initial number of gyration (N_{ini}) to ensure the mix is not too workable and has enough stability under heavy traffic loading. Additionally, the minimum air voids of 2% is required at the maximum number of gyrations (N_{max}) to ensure no bleeding would occur at the end of pavement service life. In this study, asphalt mixes were fabricated using mean of the upper and lower gradation limits which is named as used gradation, Table 3 - 4. Asphalt mixes were fabricated using PG 82-28 binder and PG 58-28 binder + 10% elastomer additives. Different percentages of asphalt binder by weight of total mix were used to reach the richness factor (K) of 3.6 and 3.5 for PG 88-28 and PG 58-28 + 10% Modifier accordingly using French method as described in Equation 7:

$$K = \frac{\frac{100B}{100 - B}}{a\sqrt[5]{\sum}} \qquad \text{Equation 3.7}$$

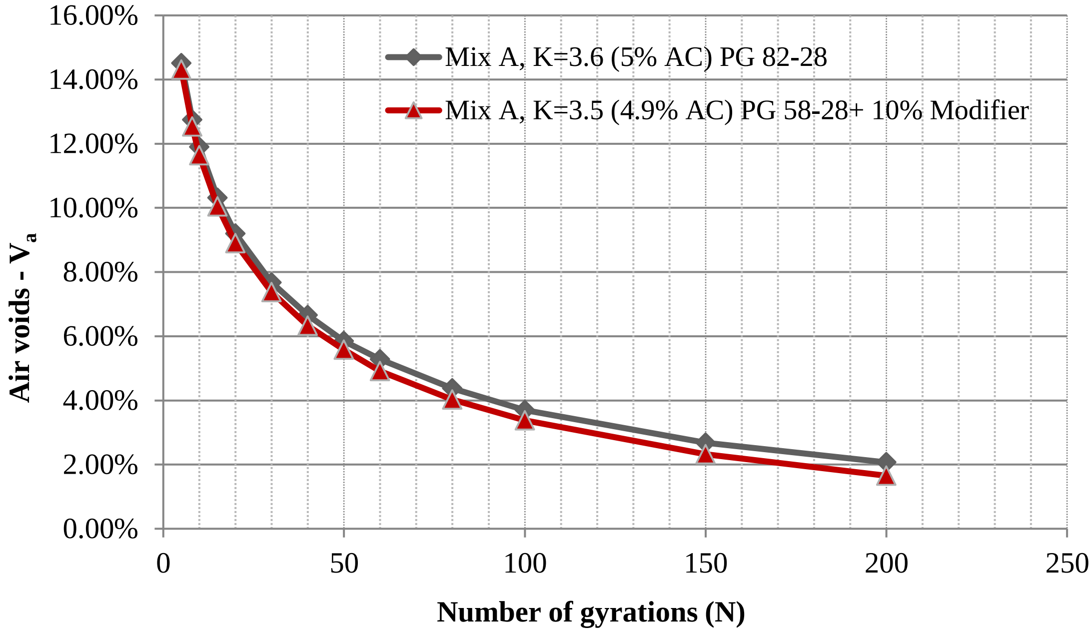
where;

B is internal percentage of binder content which is the ratio of binder mass to the sum of binder mass and mass of dried aggregate. *a* is the correction coefficient relative to the aggregate density; and $\sum$ is the specific surface area of aggregates.

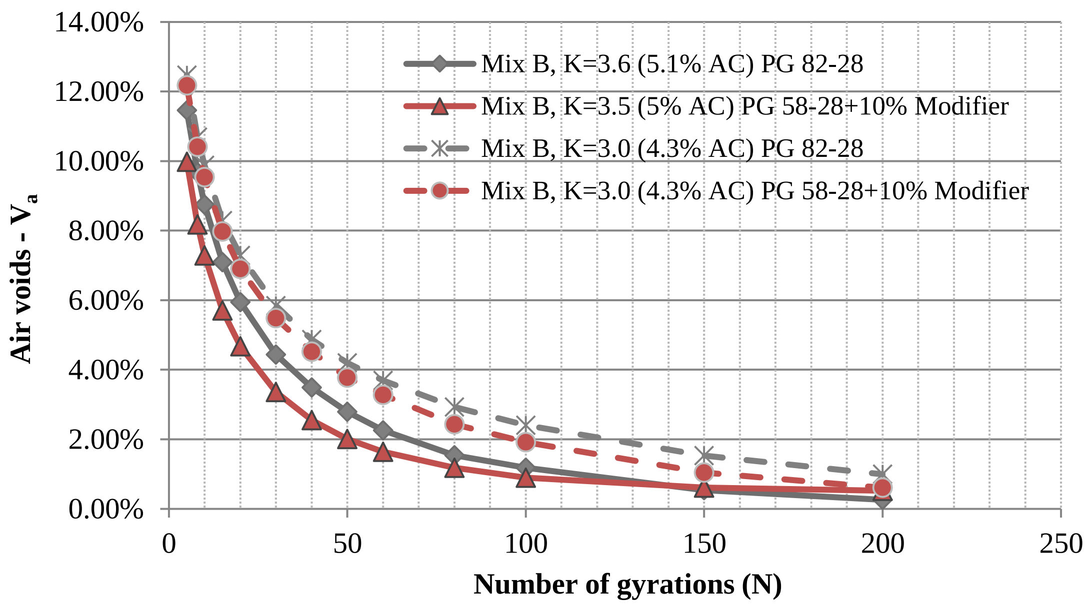
It is good mentioning that the binder contents were chosen to meet the requirement for High Modulus Asphalt Mix - Class 2 ($K \geq 3.4$).

Volumetrics and compactibility of both mixes (Mix A and Mix B) are provided in Table 3 - 4 and Figure 3 - 5 respectively. It can be seen from the results that the air void (V_a) contents of both mixes are lower than the required value for conventional mixes (Va = 4%) (Ramirez Cardona et al., 2015, Gouvernement du Québec, 2003). As shown in Table 3 - 4, the air void content at design number of gyrations are around 3.5% for Mix A and 1.0% for Mix B. In addition, as shown in Figure 3 - 5, Mix B has higher compactibility and PD (less porosity) than Mix A. This is compatible with the virtual packing density results of dry blends as was previously described in Table 3 - 3. It is worth mentioning that according to mix design criteria for high modulus asphalt mixes (EME mixes) there is no minimum air void content requirement since hard asphalt binder is used, and, therefore, both mixes can satisfactorily pass the compactibility requirements for EME mixes (NF EN 12697-31, 2007).

Since Mix B was very compactible compared to Mix A, additional mixes were fabricated using smaller richness factor, K=3.0 and the results are depicted in Figure 3 - 5 (b). It is clear from the results that the compactability of mixes was reduced after reducing the binder content; however, the compactibility of the new mixes was still higher than Mix A. This represents aggregate gradation has great impact on compactibility of asphalt mix.



(a)



(b)

Figure 3 - 5: Compactibility of asphalt mixes

Table 3 - 4: Volumetrics of asphalt mixes

<table>
<tr><th colspan="3">Used gradation</th><th rowspan="3">Mix properties</th><th rowspan="3">Criteria (Conventional Mix) (Gouvernement du Québec, 2003)</th><th colspan="4">Obtained values</th></tr>
<tr><th rowspan="2">Sieve size (mm)</th><th colspan="2">Passing (%)</th><th colspan="2">Mix A</th><th colspan="2">Mix B</th></tr>
<tr><th>Mix A</th><th>Mix B</th><th>PG 82-28</th><th>Modified PG 58-28</th><th>PG82-28</th><th>Modified PG 58-28</th></tr>
<tr><td>25</td><td>100</td><td>100</td><td>BRD @ N_{max}</td><td>-</td><td>2.624</td><td>2.633</td><td>2.670</td><td>2.664</td></tr>
<tr><td>19</td><td>100</td><td>95</td><td>V_a @ N_{ini} (%)</td><td>≥ 11.0</td><td>11.90</td><td>11.64</td><td>9.20</td><td>7.70</td></tr>
<tr><td>12.5</td><td>94.5</td><td>77</td><td>V_a @ N_{des} (%)</td><td>4.0-7.0</td><td>3.70</td><td>3.38</td><td>1.20</td><td>0.90</td></tr>
<tr><td>9.5</td><td>82.5</td><td>65</td><td>V_a @ N_{max} (%)</td><td>≥ 2</td><td>2.07</td><td>1.65</td><td>0.30</td><td>0.52</td></tr>
<tr><td>4.75</td><td>51.5</td><td>47</td><td>VMA (%)</td><td>≥14</td><td>15.05</td><td>14.80</td><td>13.46</td><td>13.12</td></tr>
<tr><td>2.36</td><td>31.4</td><td>29</td><td>VFA (%)</td><td>65-75</td><td>75.40</td><td>77.16</td><td>91.23</td><td>93.14</td></tr>
<tr><td>1.18</td><td>21.3</td><td>20</td><td rowspan="2">G_{mm}</td><td rowspan="2">-</td><td rowspan="2">2.677</td><td rowspan="2">2.676</td><td rowspan="2">2.678</td><td rowspan="2">2.678</td></tr>
<tr><td>0.60</td><td>15.1</td><td>14</td></tr>
<tr><td>0.30</td><td>10.7</td><td>10</td><td rowspan="3">Dust to binder ratio</td><td rowspan="3">0.6-1.2</td><td rowspan="3">1.08</td><td rowspan="3">1.08</td><td rowspan="3">1.03</td><td rowspan="3">1.03</td></tr>
<tr><td>0.15</td><td>7.6</td><td>7</td></tr>
<tr><td>0.075</td><td>4.9</td><td>4</td></tr>
</table>

G_{mm}= Maximum relative density of loose asphalt mix,
BRD: Bulk relative density of compacted mix,
VMA= Voids in mineral aggregate, **VFA=** Voids filled with asphalt, **Va=**Air voids,
N_{ini}=Initial number of gyrations= 10 gyr, **N_{des}=** Design number of gyrations= 100 gyr,
N_{max}= Maximum number of gyrations= 200 gyr.

3.5.3 Dynamic Modulus Test Results

Dynamic modulus, $|E^*|$, determines stress and strain responses in asphalt mix, and correlates the time-temperature dependant properties of the mix to field performance. It is a key input parameter of flexible pavements design according to the mechanistic-empirical pavement design guide (MEPDG). Dynamic modulus value depends on the properties of individual constituents of asphalt mix including asphalt binder, aggregate particles as well as physicochemical interactions between them. Since aggregate particles are the major component of asphalt mix, the aggregate properties and their packing can significantly affect dynamic modulus of the mix (Yu and Shen, 2012).

Dynamic modulus test was conducted to evaluate the effect of aggregate packing on rheological behavior of asphalt mixes. Figure 3 - 6 illustrates the dynamic modulus master curve at reference temperature of 15˚C constructed by RHEA rheological software (RHEA, 2012). As can be seen in this figure, Mix A has higher dynamic modulus values compared to Mix B with the same richness factor values although the PD of Mix A was lower. As also shown in Figure 3 - 6, at lower frequencies, mixes fabricated with binder PG 82-28 had higher dynamic modulus (stiffness). On the other hand, at higher frequencies, mixes fabricated with PG 58-28 + 10% modifiers were stiffer. Among the mixes, the highest modulus value belongs to Mix B with lower binder content (K=3.0). Moreover, it can be seen in Figure 3 – 6 (b) that the binder content plays an important role in rheological properties of asphalt mixes specifically at lower frequencies.

According to specification, high modulus/performance asphalt mixes or EME should meet the minimum stiffness requirement of 14,000 MPa at 15˚C and under 10 Hz loading (NF EN 13108-1, 2007, Delorme et al., 2007). Based on the obtained results, EME 12.5 mixes and EME 19 (K=3.0) could meet this requirement although the stiffness of other mixes were close to the required value. It should be noted that dynamic modulus value of asphalt mixes depends not only on the aggregate structure, but also asphalt binder type. According to European specification, very hard asphalt binders (10/20, 15/25 penetration grade) are used in fabrication of EME mixes which are much harder than the binders used in this study.

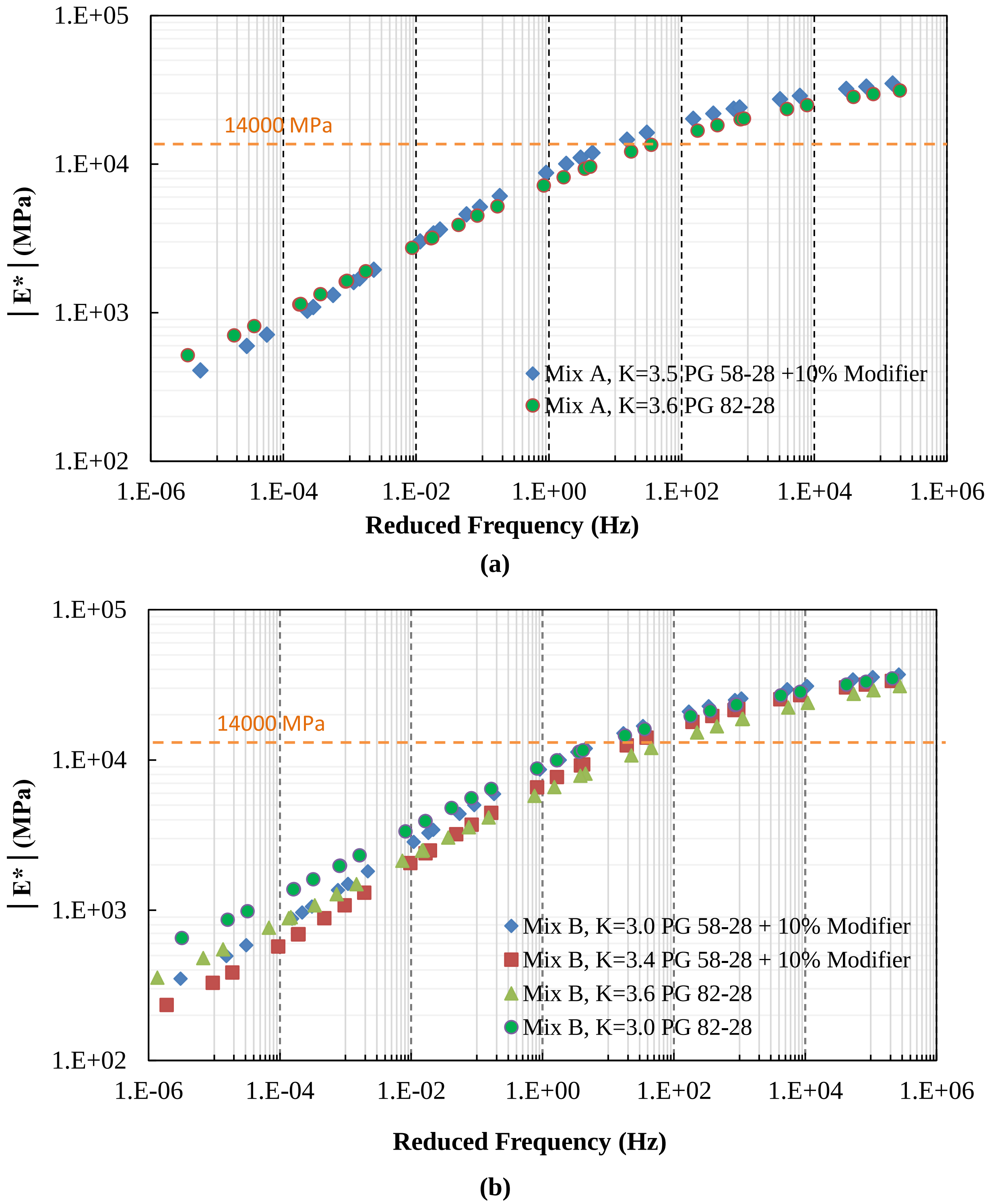
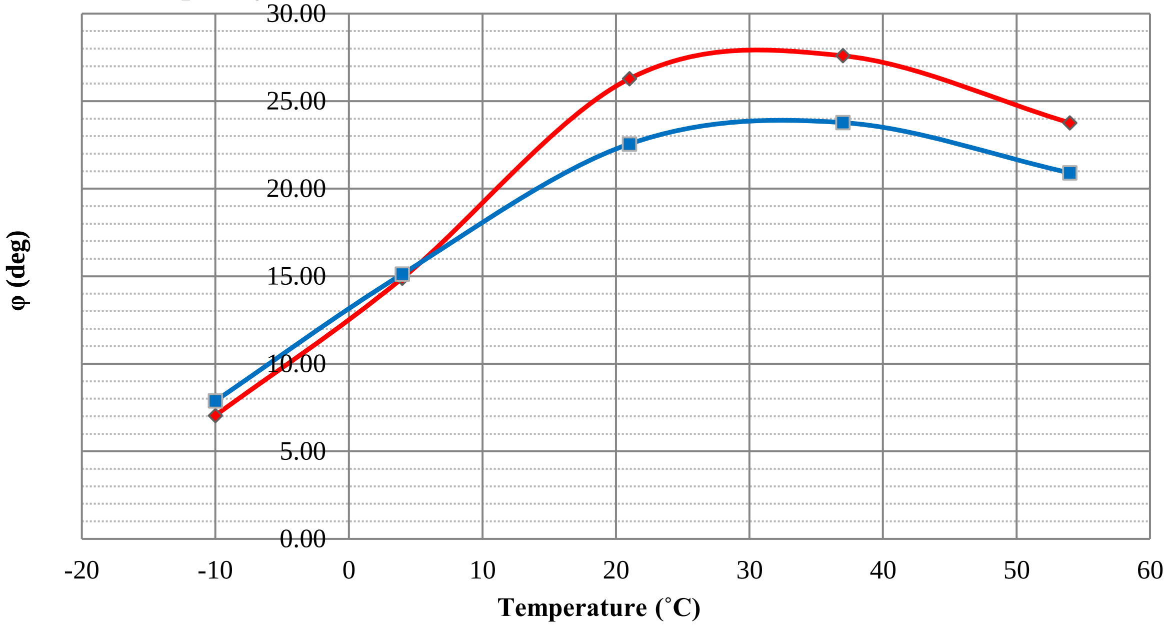


Figure 3 - 6: Dynamic modulus master curves at 15˚C reference temperature

Figure 3 - 7 and Figure 3 - 8 illustrate the phase angle (φ) of asphalt mixes versus temperature. As can be seen from these Figs, phase angles were increased by rising the temperature; however, phase angles are relatively low at all temperatures ($\varphi < 45^\circ$). The low phase angle represents elastic component of modulus is bigger than the viscous component and thus the mix behaved more elastically. It is observed that the mixes fabricated with PG 82-28 binder had lower phase angle compared to the mixes with PG 58-28 + 10% modifier. It is also clear from the results that at higher temperatures (e.g. 54˚C) and lower frequency (1 Hz) when the asphalt binder becomes soft, the optimized aggregate packing takes over the role to maintain the modulus. This is more evident in Mix B which had higher packing degree. Further, as can be seen in Figure 3 - 8, the binder content had an important effect on the behavior of asphalt mixes, and elastic tendency was increased when less amount of binder was used in the mix. This is more evident at higher temperatures when aggregate packing plays a dominant role on the mix properties.

a) 1 Hz Frequency



b) 25 Hz Frequency

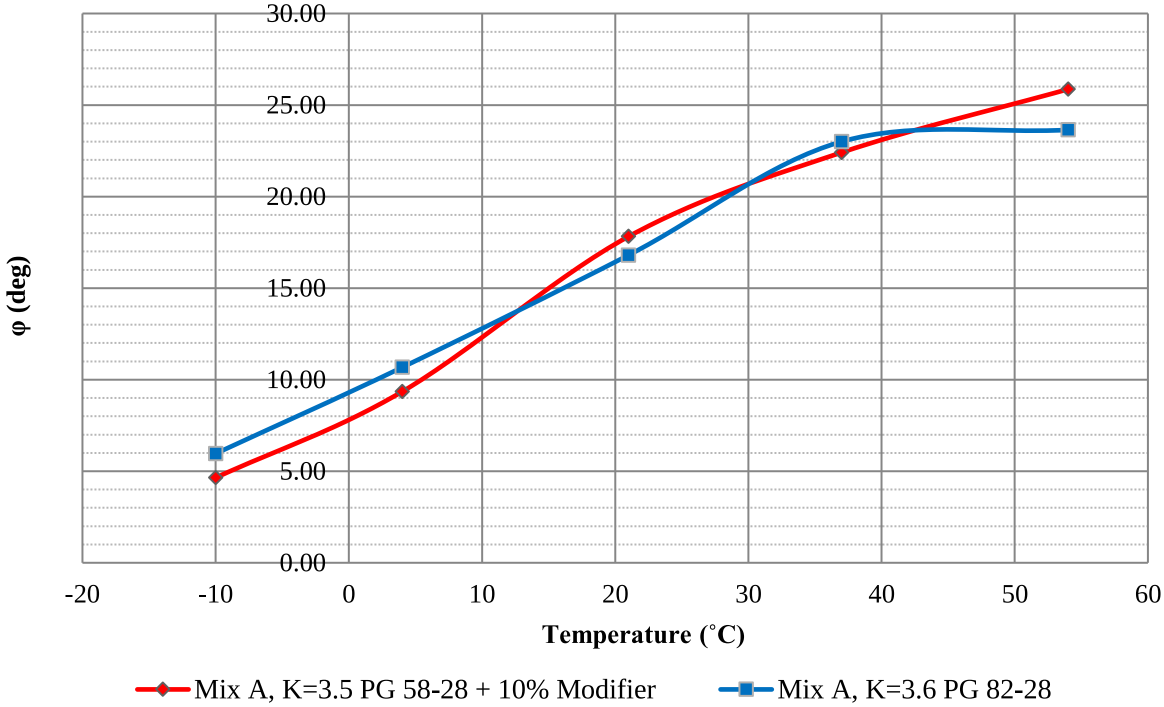


Figure 3 - 7: Mix A: Phase angle vs Temperature

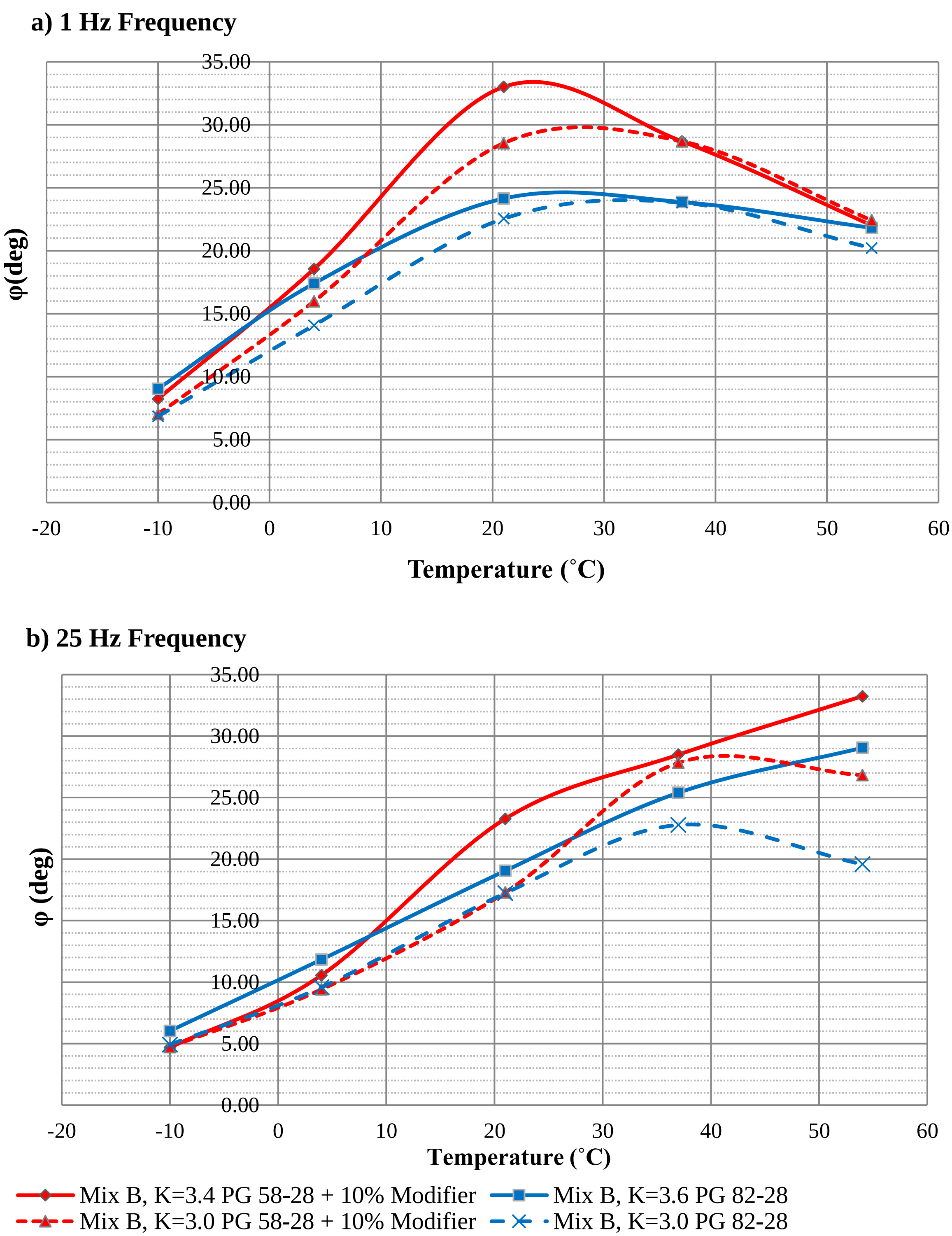


Figure 3 - 8: Mix B: Phase angle vs Temperature

3.6 Conclusions

This study investigated the use of CPM in optimization of aggregate packing for high performance asphalt mix. The parameters affecting aggregate packing were discussed in the paper. Five classes of aggregates were used in this study and morphological parameters of fine and midsize aggregates were determined using an aggregate image analyser. CPM model was used to obtain the gradation limits of particles as discussed in this paper and the results were compared with the theoretical maximum density curve used in the Superpave mix design methodology. Based on the results achieved in this study the following conclusions can be derived:

(1) The comparison showed for 12.5 mm NMAS the gradation curves obtained from CPM and the maximum density curve were very close at some points. However, the two curves were considerably different which could be attributed to the other involved factors such as aggregate shape parameters and limitation of aggregate size distribution in the blend.

(2) There was a significantly better correlation between the CPM-obtained gradation and the maximum density curve for 19 mm NMAS.

(3) The CPM-obtained gradation limits were compared with EME 14 and EME 20 limits. It was concluded that the obtained limits were close the values recommended by European specifications for EME mixes.

(4) Asphalt mixes were fabricated using the design gradations (mean of upper and lower gradation limits). Compactibility and volumetrics of asphalt mixes were also assessed. It was observed that both mixes had higher workability than the conventional mix, further, Mix B was more workable and had smaller void content compared to mix A which could be associated with higher packing degree of the mix.

(5) Dynamic modulus test results showed that Mix A had relatively higher stiffness than Mix B when both mixes had the same richness factor.

(6) Both mixes could meet the minimum requirement of stiffness value, 14,000 MPa at 15˚C and 10 Hz loading, by adjusting the binder content.

(7) Both mixes had relatively low phase angle and behaved more elastically. Effect of optimized aggregate structure was more evident on the behavior of asphalt mixes at lower frequency and higher temperatures.

For further research, asphalt mixes will be fabricated using CPM-obtained gradations and the thermo-mechanical performance tests will be conducted to evaluate asphalt mix performance in terms of fatigue and permanent deformation resistance.

CHAPTER 4
THE USE OF COMPRESSIBLE PACKING MODEL AND MODIFIED ASPHALT BINDERS IN HIGH-MODULUS ASPHALT MIX DESIGN

This chapter is based on the following published article in Journal of Road Materials and Pavement Design. Baghaee Moghaddam T, Baaj H. (2018). The Use of Compressible Packing Model and Modified Asphalt Binders in High-Modulus Asphalt Mix Design. Road Materials and Pavement Design. DOI: 10.1080/14680629.2018.1536611. Some minor modifications may have been applied to satisfy the examiners' comments.

Summary

High-modulus asphalt mix, Enrobé à Module Élevé (EME) in French, is a type of HMA representing high modulus or stiffness. Traditionally, EME mixes are fabricated with straight-run hard grade asphalt cement which has poor performance at lower temperatures and is very susceptible to low-temperature cracking in cold regions. The main objective of this study is therefore developing a new approach to EME mix design that contributes to good performance at high, medium and low temperatures. EME mixes have dense structure. In this regard, Compressible Packing Model (CPM) was used to optimize the packing degree of EME mixes for two different mix types based on nominal maximum aggregate size (EME 12.5 and EME 19). In addition, three types of modified asphalt binders, namely: PG 88-28, PG 82-28 and PG 58-28 plus 10% Elastomer additives were used in this study. Thermo-mechanical tests were conducted to evaluate performance of EME mixes in terms of stiffness, rutting, fatigue and low-temperature cracking. Obtained results showed that the developed mixes had acceptable performance at all levels, and that the mixes could satisfactorily perform at lower temperatures.

4.1 Introduction

Hot Mix Asphalt (HMA) is a composite material consisting of aggregate particles and asphalt binder. Aggregates play an important role in HMA properties since they form almost 95% of asphalt mix by weight. Aside from aggregate mineralogy, aggregate packing is a key parameter in asphalt mix performance. In this regard, different factors, including: aggregate gradation, aggregate shape and texture, aggregate strength, compaction type and effort along with thickness of the asphalt layer are influential (Olard, 2012, Corté and Di Benedetto, 2004, De Larrard and Sedran, 2002). In addition, in HMA, the aggregate particles are hold together by use of asphalt binder. Asphalt binder is a thick, heavy residue remaining obtained from refining crude oil, and consists mainly of carbon and hydrogen. It is viscoelastic thermoplastic material. Physical and mechanical properties of asphalt binder considerably changes by temperature variations. At higher temperatures, it behaves more like a fluid, but at room temperature it is more likely to behave like a soft rubber. Asphalt binder becomes very brittle when temperature drops below zero (Jenks et al., 2011).

Over the last decades, number of vehicles on roads has been increased considerably. Especially number of heavy trucks with higher axle loads. By rising in number and weight of vehicles, intensity of induced stresses on road pavements has increased significantly which has resulted in premature distresses in asphalt pavements.

High-modulus asphalt mix, or Enrobé à Module Élevé (EME) in French, is a type of HMA which has been developed in 1980's (Sybilski et al., 2010). EME mixture has high stiffness modulus and is a very good option to be used in binder course and base course of pavement layers which are subjected to the highest amount of stress in the pavement structure (Backer et al., 2008), see Figure 4 - 1. EME was firstly designated to reinforce old pavement structure and reconstruct thinner layers in urban areas due to having underground facilities such as pipes and curbs which restricts the pavement thickness to a specific value (Corte, 2001). Additionally, it has the advantage of avoiding the complete removal of old asphalt layer as it contributes to lower pavement thickness (Capitão and Picado-Santos, 2006). Subsequently, it has been also used to reduce the pavement construction cost by reducing the thickness of road pavement especially when the aggregates had low crushing index value or in case the traffic was intense, slow and channeled (Caroff and Corté, 1994).

EME has several advantages over conventional asphalt mix (Distin and Vos, 2015, Bitume Québec, 2014):

i. Increase the service life of the pavement without increasing the thickness of bound layers;
ii. Reduce the thicknesses of pavement layers for the same service life;
iii. Reduce the cost of the pavement as a result of the two above-mentioned advantages;
iv. Promote environmental gains by savings in raw materials and reduction of greenhouse gas emissions.

High modulus/stiffness of EME is derived from two essential components: 1) using hard grade (with typical penetration grade between 10×10^{-1} mm and 30×10^{-1} mm) asphalt, and 2) structure of EME. The second component refers to closed structure of EME mixes which contributes to higher stiffness. In other words, in order to achieve optimum result in EME performance using proper asphalt binder with optimum amount along with suitable aggregate particle gradation are key components.

There has been always an issue regarding using EME in cold regions which experience intense weather conditions (Judycki et al., 2017, Bertaux et al., 1996). In cold climate conditions, traditional straight-run hard grade asphalt binders cannot be used since these types of binders are not adapted for such conditions and would not meet the low temperature criteria of the Superpave performance grading (PG) system.

This current study therefore focuses on developing a new mix design method for EME mixes that can be used in regions with colder climate conditions. This was achieved by maximizing packing density of mixes as well as utilizing high-modulus modified asphalt binder with acceptable performance at both high and low temperatures.

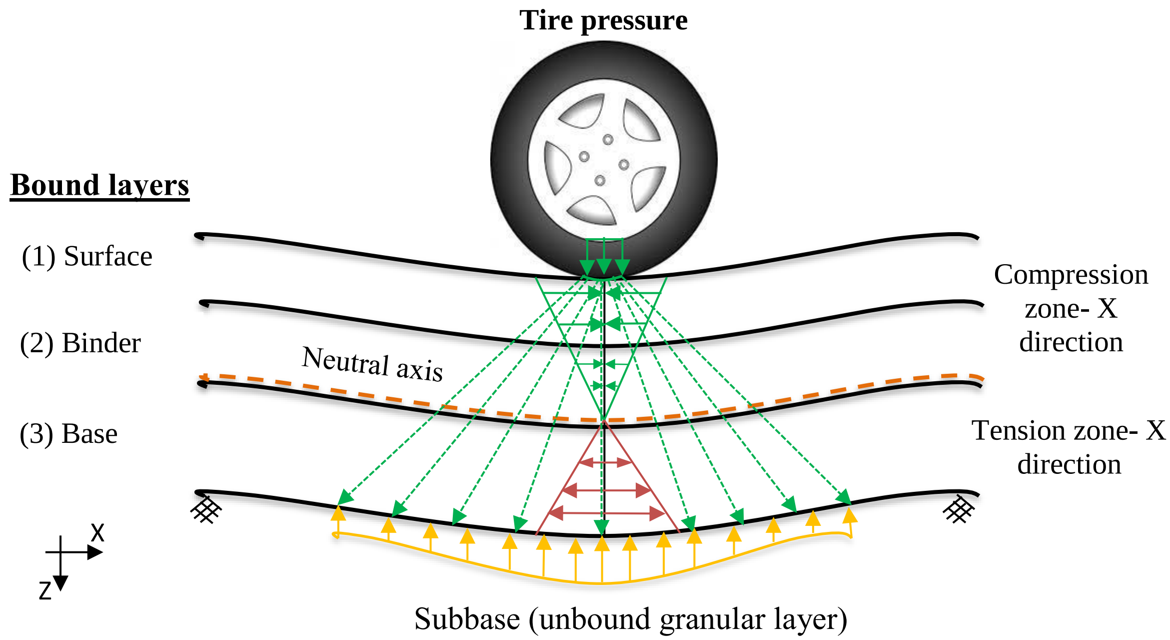


Figure 4 - 1: Schematic representation of stress/strain distribution in a typical flexible pavement structure

4.2 Experimental Procedures and Methods

In this study, Compressible Packing Model (CPM) was used as a new method to optimize the packing of aggregate blends for two different mix categories based on nominal maximum aggregate size (EME 12.5 and EME 19). Further, three types of modified asphalt binders were developed to be used in this study. Thermo-mechanical performance tests were conducted to evaluate EME performance in terms of stiffness, rutting, fatigue and low-temperature cracking.

Figure 4 - 2 provides a graphical illustration of the research methodology.

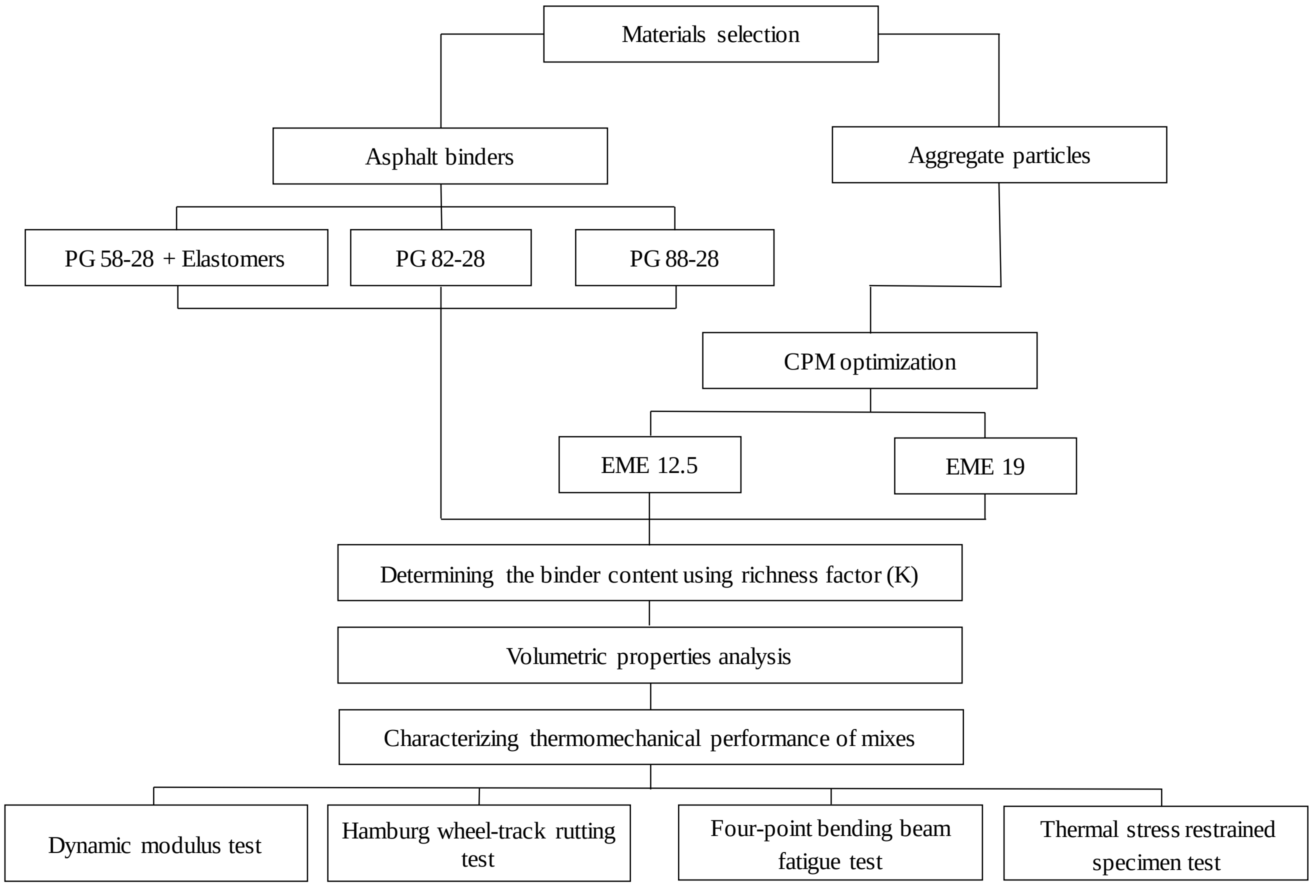


Figure 4 - 2: Graphical illustration of research methodology

4.2.1 Compressible Packing Model (CPM) Optimization Method

EME represents a category of asphalt mix with a very dense and closed structure. Therefore, to maximize aggregate packing degree with respect to available aggregate fractions, Compressible Packing Model (CPM) was used. CPM calculates virtual packing density of the blend. It is defined as the maximum packing density that can be achieved by placing the grains one by one while keeping their original shape (De Larrard, 1999). Actual packing can be determined using the virtual value with respect to the compaction method. CPM formulation is shown in Equation 4.1:

$$\Upsilon_i = \frac{\beta_i}{1-\sum_{j=1}^{i-1}\left[1-\beta_i+b_{ij}\beta_i\left(1-1/\beta_j\right)\right]y_j-\sum_{j=i+1}^{n}\left[1-a_{ij}\beta_i/\beta_j\right]y_j} \qquad \text{Equation 4.1}$$

In this equation, βi is virtual packing of the i-class compacted alone; yi is volume of each fraction in the mix, or the ratio of the volume of size class i to the total solid volume; ϒi is the virtual packing of the blend when class i is dominant and a_{ij} and b_{ij} parameters are interaction coefficients describing the “loosening effect” and “wall effect” of the particles respectively. These effects correspond to the proportions of aggregate fractions or sizes in the mix. The “loosening effect” occurs when the ratio of fine to coarse aggregates increases and the coarse aggregates are pushed away by the excessive amounts of fines in the blend. However, the “wall affect” is referred to as the interaction of aggregates with any sort of wall such as mold and pipe. The interaction exists between coarse and fine aggregates may also be considered as the “wall effect”. Figure 4 - 3 illustrates the wall and loosening effects in the aggregate blends. The a_{ij} and b_{ij} interaction coefficients can be calculated using the following equations:

$$a_{ij} = \sqrt{1-(1-\frac{d_j}{d_i})^{1.02}} \quad \text{when} \quad d_j \leq d_i \qquad \text{Equation 4.2}$$

$$b_{ji} = 1-\left(1-\frac{d_i}{d_j}\right)^{1.50} \quad \text{when} \quad d_i \leq d_j \qquad \text{Equation 4.3}$$

In these equations, d_i and d_j are diameters of the fractions i and j respectively as defined by sieve sizes.

To make use of CPM, density of aggregates, mean size and packing density of the aggregate fractions after compaction need to be determined initially. In-depth analysis and method of calculations are fully explained in a study published earlier (Baghaee Moghaddam and Baaj, 2018).

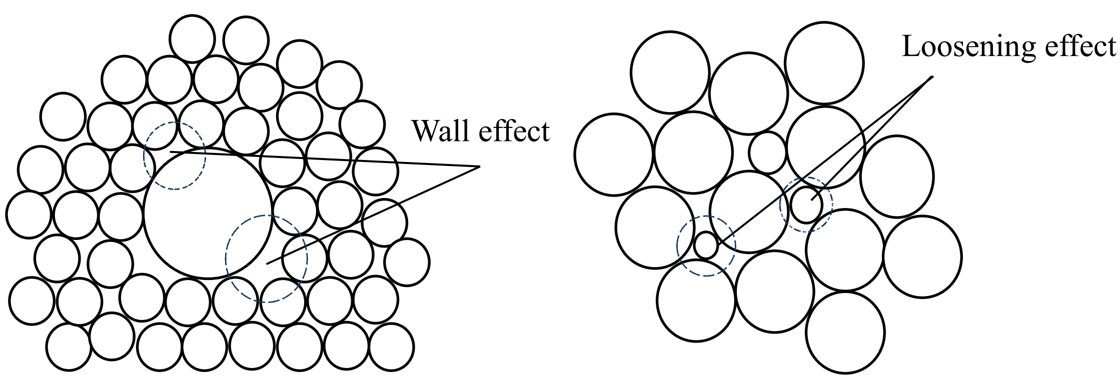


Figure 4 - 3: Wall effect and loosening effect in aggregate blends

4.2.2 Used Materials

Different fractions of aggregate materials were provided from Havelock Quarry located in Northern Ontario, Canada. Properties of coarse and fine aggregates are determined and listed in Table 4 - 1.

The gradation curves of EME mixes are shown in Figure 4 - 4. In addition, three types of modified asphalt binders namely: PG 58-28 + 10% Elastomer additives, modified PG 82-28 and PG 88-28 were used in this research. The binder properties are provided in Table 4 - 2.

Table 4 - 1: Properties of coarse and fine aggregates

Property	**Value**	**Unit**	**Specification**
Coarse aggregate			
Micro-Deval	8.8	%	LS-618
Fractured faces (more than one)	100	%	LS-607
Flat/Elongated (4:1)	8.3	%	LS-608
Specific gravity			
Dry	2.873		LS-604
SSD	2.882		
Apparent	2.901		
Absorption	0.3	%	LS-604
Fine aggregate			
Micro-Deval	11.1	%	LS-619
Specific gravity			
Dry	2.889		LS-605
SSD	2.900		
Apparent	2.921		
Absorption	0.4	%	LS-605
Wash loss	8.5	%	LS-601

Table 4 - 2: Asphalt binder properties

Properties	PG 58-28 + Elastomers (PG 70-22)	PG 82-28	PG 88-28	Specification
Brookfield viscosity, 135°C, Pa·s	1.03	1.95	5.63	AASHTO T 316
Brookfield viscosity, 165°C, Pa·s	0.32	0.60	0.79	AASHTO T 316
$G^*/\sin(\delta)$, 88°C, kPa (Original binder)	0.57	1.62	2.34	AASHTO T 315
$G^*/\sin(\delta)$, 88°C, kPa (RTFOT aged binder)	2.00	1.68	2.30	AASHTO T 315
Creep stiffness, -18°C MPa (PAV & RTFOT aged binder)	283.0	124.0	137.7	AASHTO T 313
m-value, -18°C (PAV & RTFOT aged binder)	0.282	0.361	0.322	AASHTO T 313

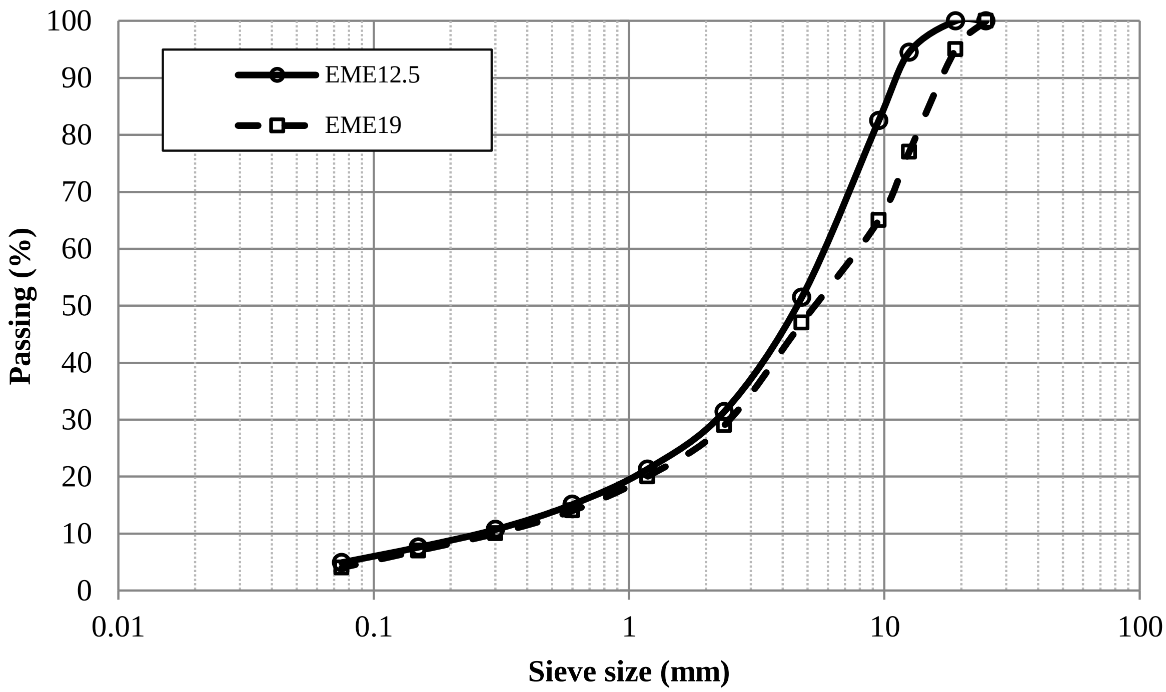


Figure 4 - 4: Particle size distribution of EME mixes

4.2.3 Sample Preparation and Volumetrics

In order to fabricate EME mixes, asphalt binders and blended aggregates were heated in an oven to reach their mixing temperatures. Mixing temperatures were determined based on the asphalt binders' viscosities. With this regard, the mixing temperatures of 155˚C, 165˚C and 175˚C were used for PG 58-28 + 10% elastomer additives, PG 82-28 and PG 88-28 respectively. After mixing the aggregates and asphalt binders, the loose mixes were covered and conditioned at 135˚C for four hours prior to compaction. This was done to simulate the short-term aging of asphalt mixes in the field according to AASHTO R 30-02, (2015). Thereafter, the loose mixes were compacted using Superpave Gyratory Compactor (SGC) and Asphalt Vibratory Compactor (AVC) to fabricate cylindrical shaped specimens and slab specimens respectively. Compaction temperatures of 145˚C, 155˚C and 160˚C were used for PG 58-28 + 10% Elastomer additives, PG 82-28 and PG 88-28 respectively according to the recommendations provided by the binders' manufacturers. It is worth mentioning that the binder contents were chosen to get the richness factor (K) of 3.5 and 3.0 for EME 12.5 and EME 19 respectively using Equation 4.4 (NF EN 13108-1, 2007):

$$K = \frac{\frac{100B}{100-B}}{a\sqrt[5]{\sum}} \qquad \text{Equation 4.4}$$

In this equation, B is internal percentage of binder content which is the ratio of binder mass to the sum of binder mass and mass of dried aggregate; a is the correction coefficient relative to the aggregate density; $\sum$ is the specific surface area. The volumetrics of the designed mixes are provided in Table 4 - 3.

Table 4 - 3: Volumetric properties of developed EME mixes

Properties	**EME 12.5**			**EME 19**		
	PG 58-28 + Elastomers	**PG 82-28**	**PG 88-28**	**PG 58-28 + Elastomers**	**PG 82-28**	**PG 88-28**
B	5.0	5.0	5.0	4.3	4.3	4.3
G_{mm}	2.676	2.677	2.678	2.691	2.714	2.709
BRD @ N_{max}	2.633	2.624	2.632	2.677	2.687	2.682
VMA (%)	14.30	14.60	14.66	12.50	12.18	12.44
VFA (%)	80.35	78.15	81.92	88.80	84.57	84.01
V_a @ N_{ini} (%)	12.1	12.3	11.4	10.0	10.3	10.3
V_a @ N_{des} (%)	2.8	3.2	2.7	1.4	1.9	2.0
V_a @ N_{max} (%)	1.6	2.0	1.7	0.5	1.0	1.0
P_{be}	4.5	4.5	4.5	4.2	4.0	4.1

B: Binder content; G_{mm}: Maximum relative density of loose asphalt mix; BRD: Bulk relative density of compacted mix; VMA: Voids in mineral aggregate; VFA: Voids filled with asphalt; V_a: Air voids; N_{ini}: Initial number of gyrations: 9 gyr; N_{des}: Design number of gyrations: 125 gyr; N_{max}: Maximum number of gyrations: 205 gyr; P_{be}: Effective binder content.

4.3 Methodology

The EME mix design approach is combination of empirical and performance-based tests and is more costly and time consuming than the conventional mix design (NF EN 13108-1, 2007). In this study, Thermo-mechanical performance tests were conducted to evaluate the performance of the mixes in terms of rutting, fatigue and low-temperature cracking. It is worth mentioning that the test procedures used in Ontario are not the same as the French methods. In this regard, the available and currently used performance testing procedures were selected for this study as explained in the following sections.

4.3.1 Dynamic Modulus Test

Modulus (stiffness) is a fundamental mix design parameter of flexible pavements (Baghaee Moghaddam et al., 2015a). Asphalt mix is a viscoelastic material and the modulus value is affected by time of loading (loading frequency) and the ambient temperature. In pavement laboratories, dynamic modulus ($|E^*|$) test is conducted to determine stress and strain responses in asphalt mix and correlates the time-temperature dependant properties of the mix to field performance. In this study, dynamic modulus test was conducted according to (AASHTO T 342-11, 2011) using Material Testing System (MTS-810) equipment. Dynamic modulus samples were fabricated using Superpave gyratory compactor. The gyratory compacted samples were then cored and cut to produce Ø100×150H mm cylindrical specimens. During the test, a sinusoidal axial compressive stress with different loading frequencies (0.1, 0.5, 1, 5, 10 and 25 Hz) is applied on the specimen at specific temperatures (-10, 4, 21, 37 and 54˚C). The applied stress and the corresponding strain response of the specimen were measured continuously during the test using a data acquisition system, and the dynamic modulus values were calculated by dividing stress magnitudes by average strain magnitudes.

4.3.2 Permanent Deformation (Rutting) Test

Asphalt layer can be deformed when a load is applied to the surface of asphalt pavement. Since asphalt mix is a viscoelastic material, portion of deformation recovers once the load is removed (elastic behavior); however, a portion of deformation would remain (plastic behavior). The amount of deformation is greatly influenced by amount of load, time of loading or loading frequency, pavement temperature as well as type of asphalt mix (Baghaee Moghaddam et al., 2014). In this study, rutting performance of the mixes was evaluated using the Hamburg Wheel-Track Tester (HWTT) according to (AASHTO T 324, 2016). In this regard, duplicate cylinder specimens (150 mm diameter × 63 mm height) were fabricated using SGC for each run. The specimens were tested in the wet condition (submerged under water) using solid rubber wheels. The test was carried out at 50˚C. During the test two linear variable displacement transducers (LVDTs) were used to measure the depths of the impression of the wheel (rut-depth).

4.3.3 Four-Point Bending (4PB) Beam Fatigue Test

Fatigue failure is a common distress mode of asphalt pavement structures which is caused by successive tensile strain induced by repeated traffic loadings (Di Benedetto et al., 2004a). This type of cracking is initiated at the bottom of asphalt layer that subject to the highest tensile stress (or strain) in the pavement structure. Fatigue distress appears in the form of cracking (alligator cracking) on the surface of asphalt pavement. Previous studies showed that the fatigue performance of asphalt mix has correlation with the mode and amount of applied loads as well as environmental temperature (Soltani et al., 2015, Al-Khateeb and Ghuzlan, 2014). Further, fatigue cracking resistance of asphalt mixes is highly impact by the mix properties and in particular the binder properties (Baaj et al., 2005). Several tests are available to evaluate the fatigue resistance of asphalt mixes (Di Benedetto et al., 2004a). Four-point bending beam fatigue test and Tension-Compression test are among the popular testing methods to evaluate the fatigue performance of asphalt mixes. The common test used in Ontario is the four-point bending beam fatigue test. The test has been designated for this study to determine the fatigue life of EME mixes.

The test setup is entirely computer-controlled and consists of a load frame, a closed-loop control, and data acquisition system. The test was carried out in accordance with the AASHTO T 321-14 (2014) procedure. 380L×63W×50H mm beam specimens were used. In a four-point bending frame, the test beams were subject to repeating flexural (sinusoidal) loading under 10 Hz loading frequency. The deflection level (strain level) was selected to allow the specimen undergo a minimum of 10,000 loading cycles before stiffness of asphalt mix was reduced to 50% of its initial value. The initial flexural stiffness was estimated by applying 50 load cycles at a constant peak-to-peak strain level between 250–750 μm/m (microstrain). Prior to the test, each beam was conditioned at testing temperature of 20˚C for two hours to reach to a uniform temperature in the specimen. The test was conducted at four different displacement (strain) levels.

4.3.4 Thermal Stress Restrained Specimen Test

Low-temperature cracking is another major mode of failure of asphalt pavements in cold regions (Tan et al., 2012). To evaluate the low-temperature performance of EME mixes, Thermal Stress Restrained Specimen Test (TSRST) was designated to be used in this study. TSRST is well-known to simulate the actual material properties with the temperature variations during service life. It is an automated closed loop system which measures the tensile stress in an asphalt specimen as it is cooled down at a defined constant rate. During the test, the temperature drops and as the specimen is restrained from contraction, tensile stress starts building up in the specimen. The tensile stress and the corresponding temperature are measured as part of the test. The test was performed according to AASHTO TP 10 (1993 (Reapproved 1996)). The rectangular test specimen (50 mm × 50 mm × 250 mm) was glued to two aluminum end platens. Cooling rate was kept constant at 10 ºC/hour. The contraction of the specimen was measured during the cooling process using two extensometers which were placed on the specimen by springs. The test was terminated when the thermally induced tensile stress in the specimen exceeded its tensile strength as shown in Figure 4 - 5.

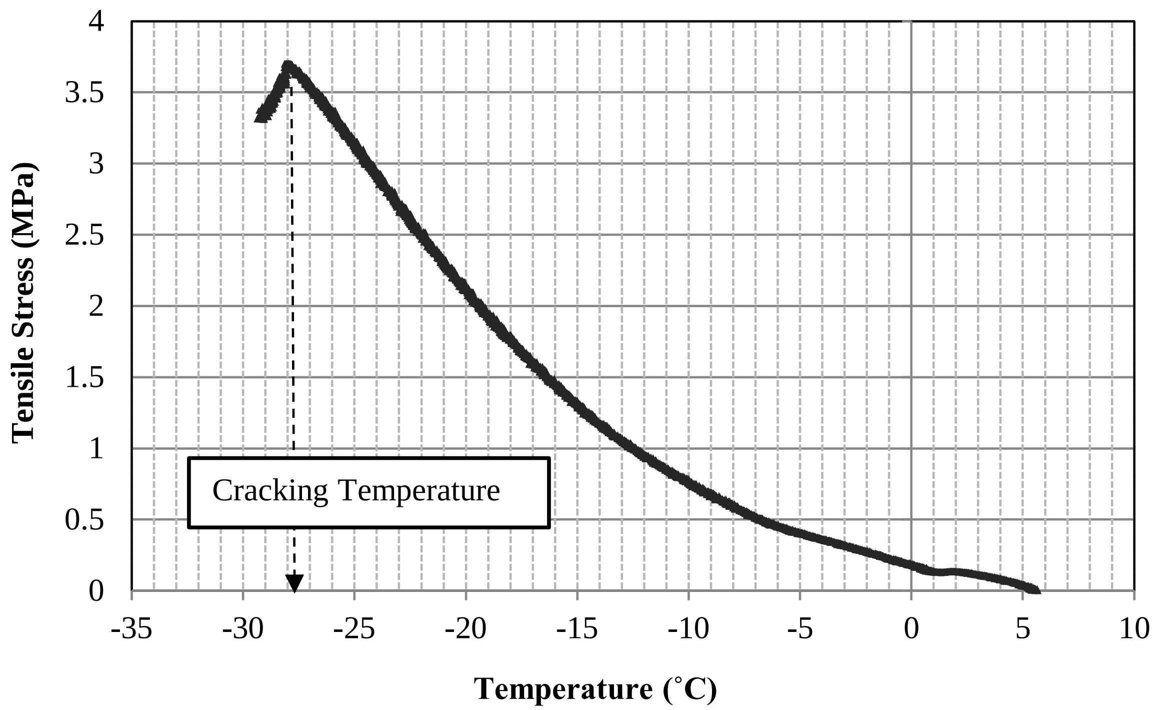


Figure 4 - 5: Determining TSRST cracking temperature- cooling rate: 10°C/hour (test principle)

4.4 Experimental Results and Discussion

4.4.1 Dynamic Modulus Test Results

Dynamic modulus test was conducted at different temperatures as well as loading frequencies. Figure 4 - 6 and Figure 4 - 7 show the dynamic modulus test results for EME 12.5 and EME 19 respectively. As can be seen in these figures, the dynamic modulus values were reduced significantly by rising temperature. Further, mixes showed higher modulus under higher loading frequencies. The obtained results are compatible with the previous studies (Nivedya et al., 2017, Cho et al., 2010). Among mixes, the mix fabricated with PG 88-28 had the highest modulus values. In addition, EME 19 showed higher modulus than EME 12.5 regardless of the binder used. According to European specification modulus of EME mixes should meet 14,000 MPa at 15˚C temperature and under 10 Hz loading (NF EN 13108-1, 2007; Delorme et al., 2007). The obtained results showed stiffness of the developed mixes could meet or are very close to the minimum requirement. It is worth mentioning that this requirement might not be same for cold regions where the average annual air temperatures are less than 15˚C. For instance, in Ontario, Canada, the average annual temperature is less than 10˚C (Mills et al., 2009), and therefore the minimum stiffness should be selected with respect to this temperature.

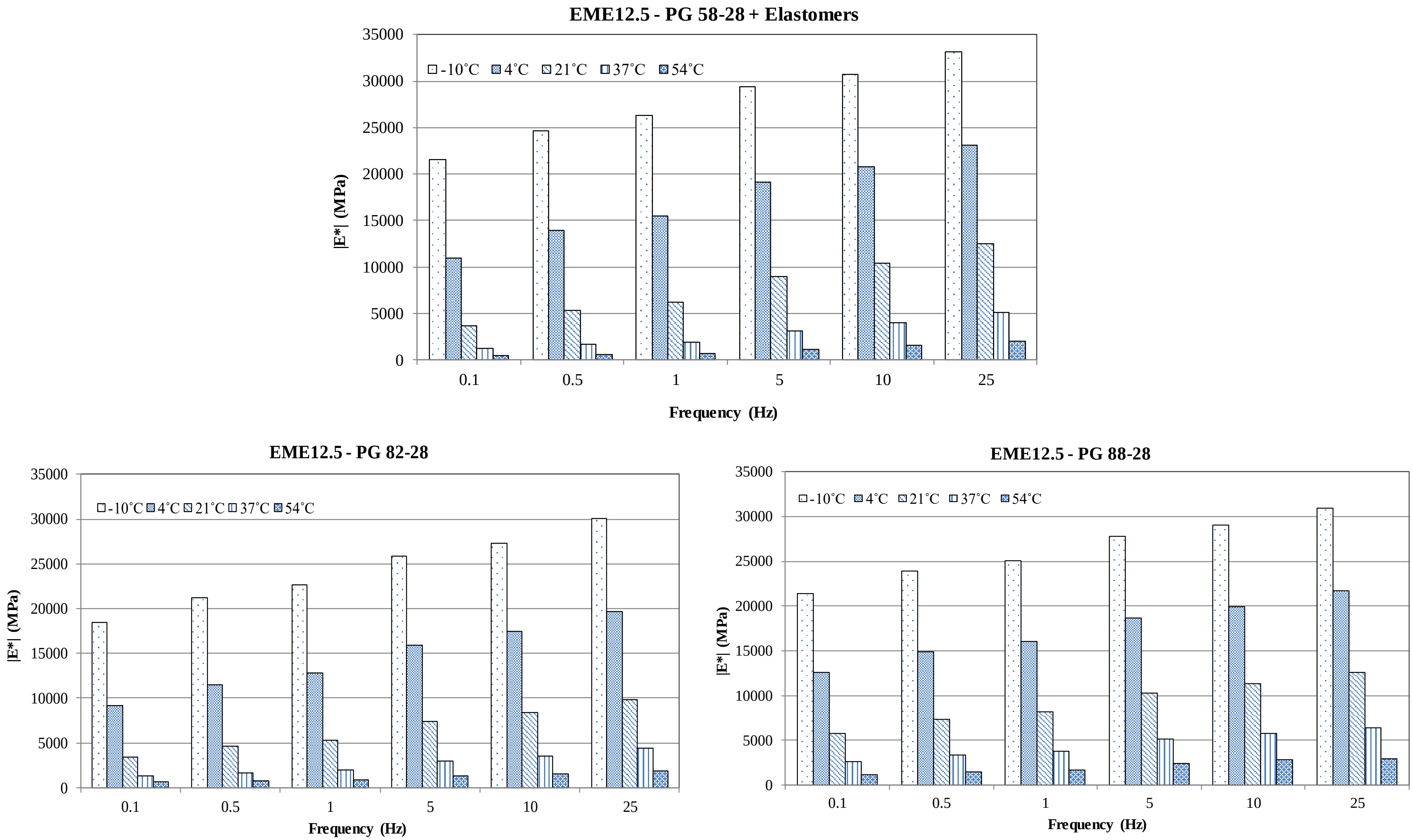


Figure 4 - 6: Dynamic modulus (|E*|) test results for EME 12.5

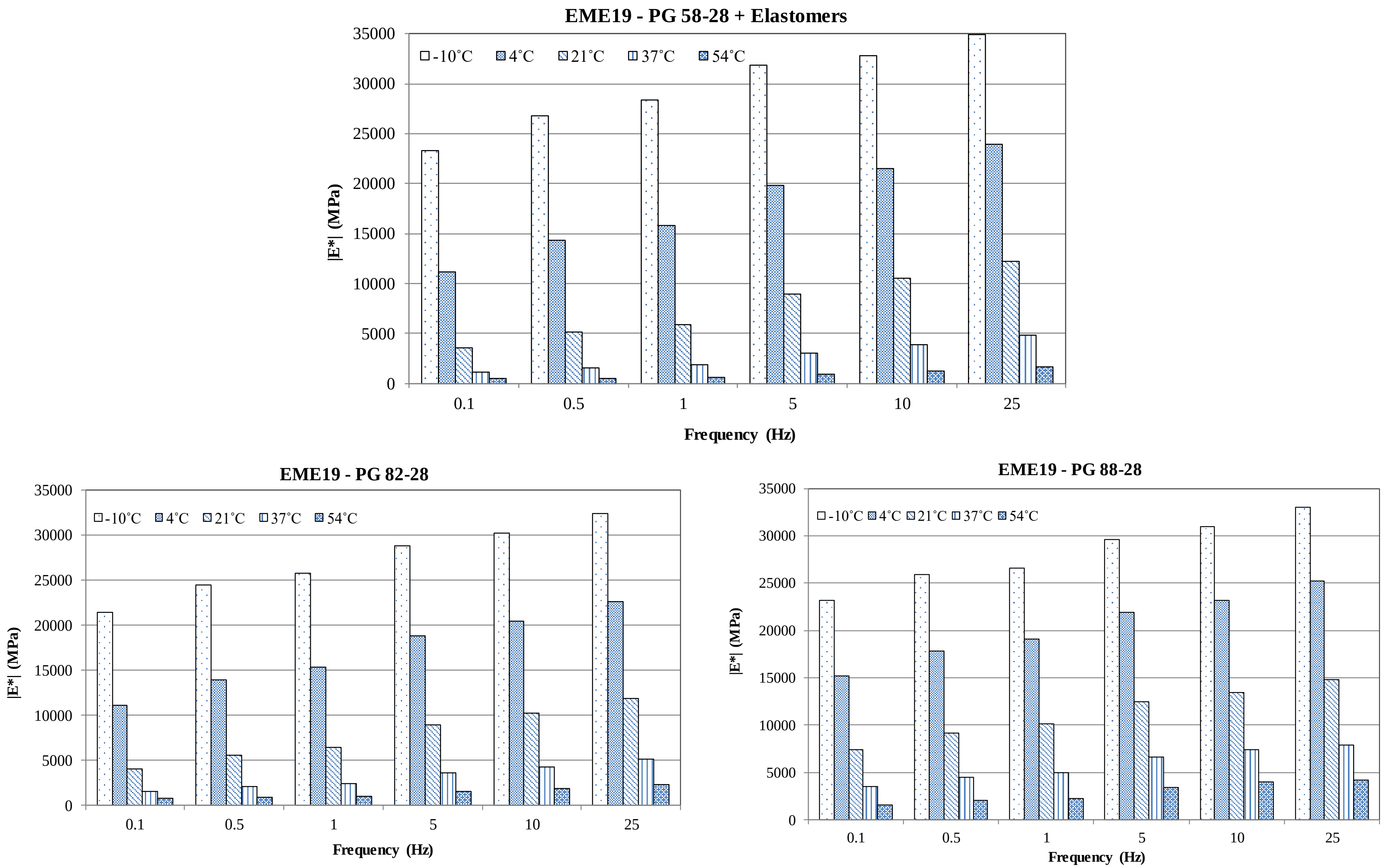
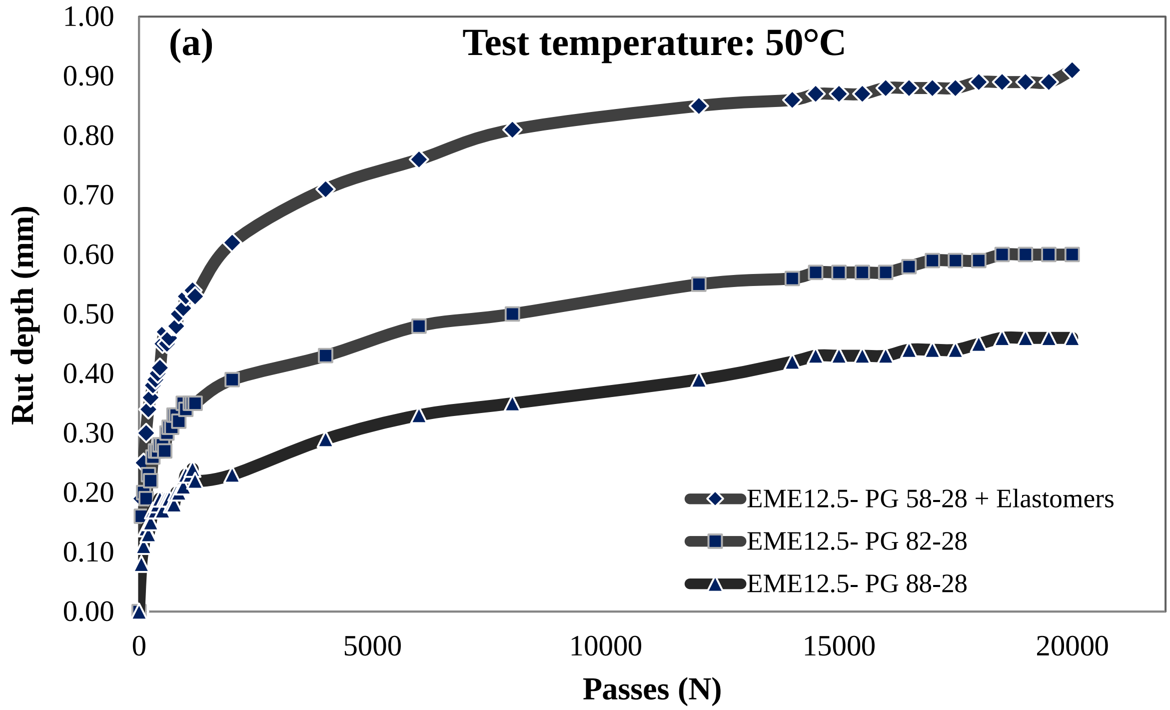


Figure 4 - 7: Dynamic modulus (|E*|) test results for EME 19

4.4.2 Permanent Deformation Test Results

Permanent deformation (rutting) is a major pavement distress mode of flexible pavements at higher temperature. Therefore, HWTT was conducted in this study to evaluate the rutting performance of EME mixes. The test results are depicted in Figure 4 - 8. As can be seen in this figure, all of the mixes performed well in terms of rutting. Maximum deformations or rut-depth of the mixes are less than 1 mm after 20,000 passes. Among which, those fabricated with PG 88-28 asphalt binder showed to have the lowest deformation which could be attributed to the fact that it has the highest stiffness among the binders. In addition, it can be obtained from the results that EME 19 performed slightly better than EME 12.5 which could be associated to higher aggregate packing of the mix (Baghaee Moghaddam and Baaj, 2018).



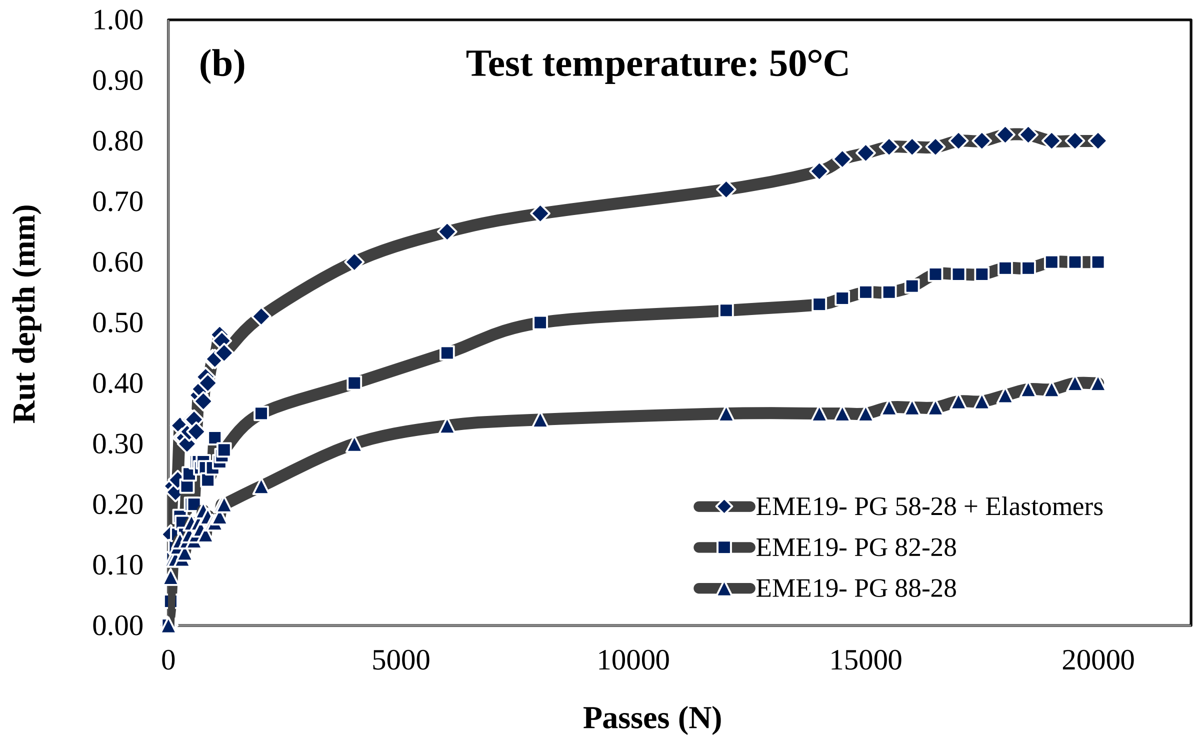


Figure 4 - 8: Rutting test results- wet condition: (a) EME12.5; (b) EME19

4.4.3 Fatigue Test Results

Fatigue is a common pavement distress mode of flexible pavements which occurs at intermediate temperatures. In this study, four-point bending beam fatigue test was conducted to evaluate fatigue performance of EME mixes. The fatigue test was conducted at controlled displacement mode; and four different strain levels were designated to develop the fatigue curves (Wöhler curves) using the following equation (Di Benedetto et al., 2004a):

$$N_{f/50} = \alpha . \varepsilon^{-\beta} \qquad \text{Equation 4.5}$$

where $N_{f/50}$ is the number of load cycles to failure (50% reduction in initial stiffness), ε is the strain value; and α and β are the regression coefficients or fatigue parameters related to mix properties.

The fatigue equations of EME mixes are presented in Table 4 - 4 and fatigue curves are plotted in Figure 4 - 9 in logarithmic scale. As can be observed from Figure 4 - 9, EME 12.5 fabricated with modified PG 82-28 had the best fatigue performance at all strain levels. On the other hand, mixes fabricated with PG 58-28 + 10% Elastomer additives had the lowest fatigue performance among all binder types. It is also clear form the results that, regardless of the binder type, the fatigue lives at 20˚C temperature of EME 12.5 were longer than those of EME 19 which could be due to the higher binder content of EME 12.5 (Nejad et al., 2010).

According to European specification (NF EN 13108-1, 2007), the minimum strain levels to give 1,000,000 loading cycles fatigue life (ε_6) are 130 μm/m and 100 μm/m for EME Class 1 and EME Class 2 respectively. Based on the results achieved in this study, ε_6 values of EME mixes are larger than the minimum requirements. It should be noted that the minimum requirements in the specification are based on the results of two-point bending test. According to Di Benedetto et al. (2004a) and Poulikakos et al. (2015) there is a difference in the results obtained from the two methods.

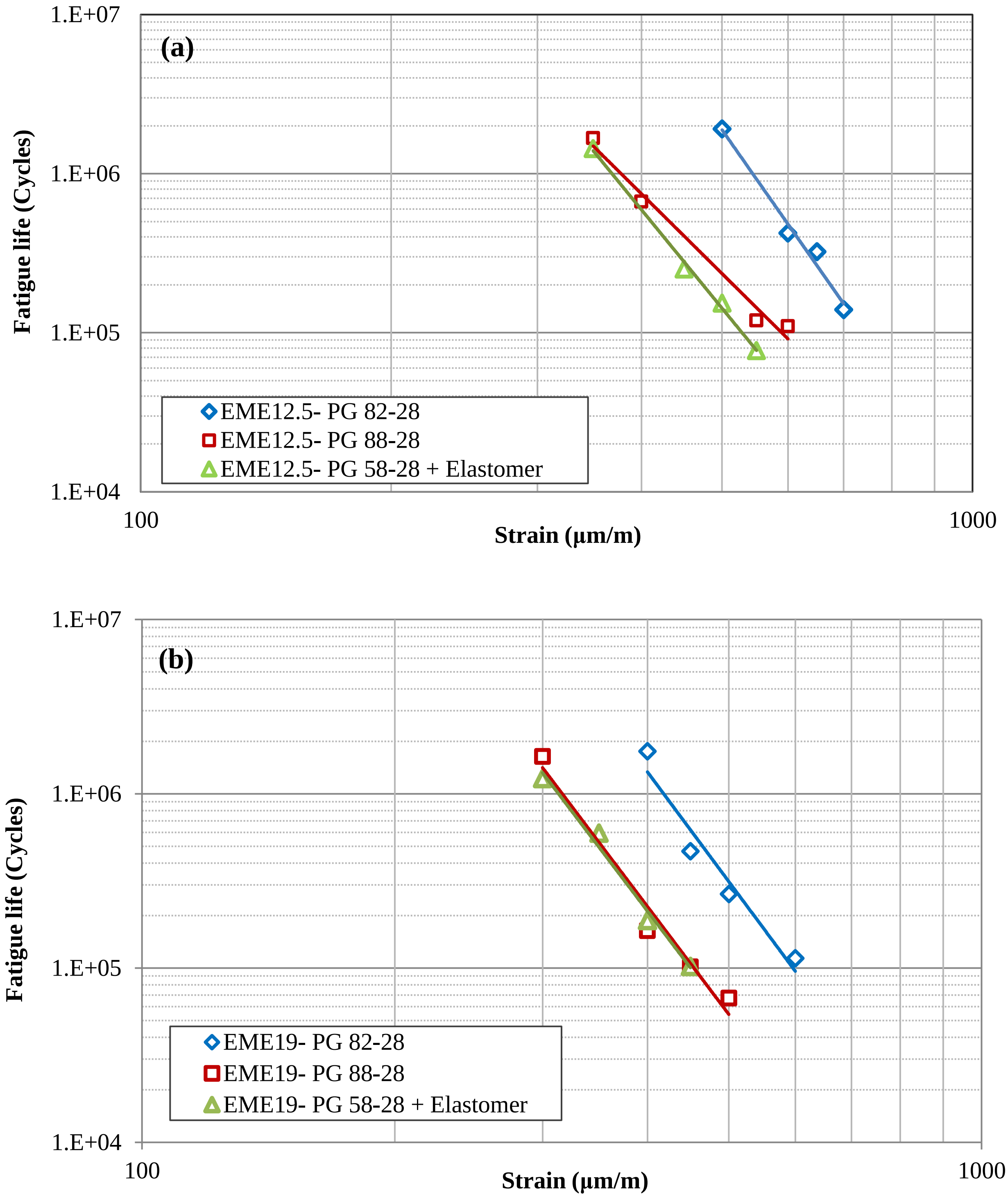


Figure 4 - 9: Fatigue lines: (a) EME12.5; (b) EME19 (Peak-to-Peak strain)

Table 4 - 4: Fatigue models for EME mixes

Mix type	Binder type	ε (μm/m) Peak-to-Peak	$N_{f/50}$ (Cycles)	Fatigue equation	Correlation coefficient
EME 12.5	PG 82-28	500	1,912,332	$N_f = 3.2 \times 10^{26}\, \varepsilon^{-7.5}$	0.98
		600	423,195		
		650	323,796		
		700	139,749		
	PG 88-28	350	1,671,975	$N_f = 2.4 \times 10^{19}\, \varepsilon^{-5.2}$	0.98
		400	666,590		
		550	119,548		
		600	109,798		
	PG 58-28 + Elastomers	350	1,436,875	$N_f = 2.9 \times 10^{22}\, \varepsilon^{-6.4}$	0.99
		450	251,647		
		500	153,685		
		550	77,349		
EME 19	PG 82-28	400	1,754,377	$N_f = 1.2 \times 10^{23}\, \varepsilon^{-6.5}$	0.94
		450	467,794		
		500	266,496		
		600	113,848		
	PG 88-28	300	1,636,179	$N_f = 8.8 \times 10^{21}\, \varepsilon^{-6.4}$	0.97
		400	163,898		
		450	101,449		
		500	67,323		
	PG 58-28 + Elastomers	300	1,223,181	$N_f = 7.1 \times 10^{21}\, \varepsilon^{-6.4}$	0.98
		350	594,393		
		400	187,898		
		450	102,362		

4.4.4 Thermal Stress Restrained Specimen Test (TSRST) Results

The TSRST cracking temperatures are illustrated in Figure 4 - 10. As shown in this figure, in general, EME 12.5 had better performance than EME 19. EME 12.5 with PG 82-28 and PG 88-28 binders failed when the temperature dropped to -28˚C although the cracking temperature of the mix fabricated with the elastomer additives was around -26˚C. Additionally, cracking temperatures of EME 19 were around 1˚C higher than EME 12.5 when the same binder type is used. This could be attributed to the fact that less amount of binder was used in EME 19 since asphalt binder has more contribution in low-temperature performance of HMA than aggregate gradation (Gao et al., 2018).

According to the results, EME 12.5 with PG 82-28 binder had relatively better low-temperature cracking resistance than the other mixes. It should be noted that since EME mixes are typically used in binder course and base course of asphalt pavement structure, these layers may not be as susceptible as the surface layer to the low-temperature cracking. Therefore, -28˚C should be a reasonable temperature for low-temperature performance of EME mixes.

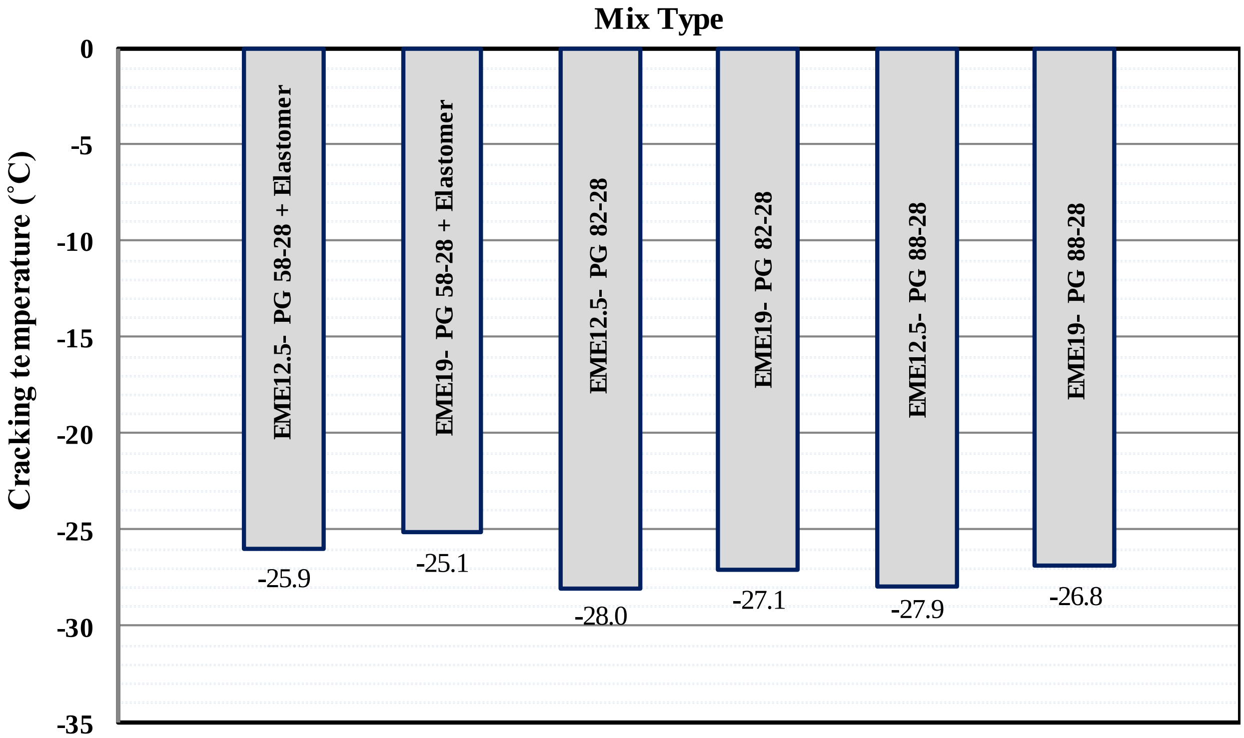


Figure 4 - 10: TSRST cracking temperatures of EME mixes

4.5 Summary of the Design Parameters

This section provides comparisons among EME mixes based on the above performance testing results. Summary of test results are listed in Table 4 - 5. As can be derived from this table, all of the mixes performed well. The mixes with higher modulus/stiffness had better rutting performance. On the other hand, the results describe that higher stiffness of the mix would adversely impact the fatigue resistance; however, such a correlation may not be very precise. That is to say, having higher stiffness, magnitude of induced tensile strains at the bottom of the bound layer will be reduced which can prolong fatigue life of pavement structure. Therefore, In the future, Tension-Compression fatigue test together with an intrinsic fatigue approach developed at the Département Génie Civil et Bâtiment (DGCB) will be conducted to fully characterize the fatigue behaviour of the mixes (Perraton et al., 2003, Baaj et al., 2003).

Low-temperature fracture (cracking) temperatures of the mixes were less than -25˚C. EME 12.5 with both PG 88-28 and PG 82-28 binders performed slightly better than the other mixes. In general, based on the achieved test results and by taking everything into consideration, EME 12.5 with PG 82-28 performed the best among all mix types; conversely, EME 19 with elastomer additives, regardless of having higher stiffness, had the lowest overall performance.

Table 4 - 5: Summary of performance testing results of EME mixes

EME	Binder Type	|E*| @ 10 Hz (MPa)		Rut-depth 20,000 passes (mm)	Fatigue (20˚C; 10 Hz) ε_6 (µm/m)	TSRST cracking temperature (˚C)
		10˚C	15˚C			
12.5	PG 82-28	13,852	11,132	0.60	549.3	-28.02
	PG 88-28	16,518	13,939	0.46	371.3	-27.96
	PG 58-28 +10% Elastomer	16,886	13,689	0.91	377.5	-25.93
19	PG 82-28	16,366	13,335	0.60	415.2	-27.05
	PG 88-28	19,133	15,640	0.40	317.3	-26.86
	PG 58-28 +10% Elastomer	17,206	13,756	0.80	312.2	-25.14

4.6 Conclusions

This study aims to develop high-modulus asphalt (EME) mixes for cold climates. To achieve this aim, Compressible Packing Model (CPM) was used as new packing method along with developing three types of modified asphalt binders with good performance at lower temperatures. Series of laboratory tests were designated in this study to evaluate thermo-mechanical performance of EME mixes in terms of stiffness, rutting, fatigue and low-temperature cracking. Based on the result obtained in this study the following conclusions can be derived:

(1) Optimization of aggregate packing using CPM showed to be promising.

(2) Dynamic modulus test results showed that the mixes had high stiffness values, and that EME mixes fabricated with PG 88-28 binder had the highest modulus values compared to the other two binder types. In addition, EME 19 represented higher stiffness than EME 12.5.

(3) All mixes had superior permanent deformation performance. Maximum deformations of all mixes were less than 1 mm after 20,000 wheel passes.

(4) Four-point bending beam fatigue test results showed that the mixes could meet the minimum ε_6 requirement of EME mixes as described in the paper. In addition, mixes fabricated with PG 82-28 binder performed the best among all binder types.

(5) TSRST results showed that the fracture temperatures of EME mixes were less than -25˚C. EME 12.5 performed relatively better compared with EME 19. EME 12.5 with PG 82-28 and PG 88-28 had the lowest temperature cracking resistance.

(6) Based on the achieved results it could be concluded that the developed mixes had acceptable performance at all levels.

CHAPTER 5
RHEOLOGICAL CHARACTERIZATION OF HIGH-MODULUS ASPHALT MIX WITH MODIFIED ASPHALT BINDERS

This chapter is based on the following published article in the Journal of Construction and Building Materials. Baghaee Moghaddam T, Baaj H. (2018). Rheological Characterization of High-Modulus Asphalt Mix with Modified Asphalt Binders. Construction and Building Materials. DOI: 10.1016/j.conbuildmat.2018.10.194. Some minor modifications may have been applied to satisfy the examiners' comments.

Summary

Complex Modulus (E^*) or stiffness is a fundamental design parameter of flexible pavements which can be used to determine the response of asphalt mix under traffic loading and thermal conditions. The modulus value heavily depends on environmental temperatures as well as loading frequencies. High-modulus asphalt mix, or Enrobé à Module Élevé (EME) in French, is a type of Hot Mix Asphalt (HMA) that has high stiffness at intermediate temperature. EME has several advantages over conventional asphalt mix including reduction in layer thickness and improved structural life. The purpose of this study is to evaluate rheological behavior of EME mixes fabricated with modified asphalt binders. In this study, two types of EME mixes based on nominal maximum aggregate size (NMAS) were developed (EME 12.5 and EME 19) using three different polymer modified asphalt binders (PG 58-28 +10% Elastomer additives, modified PG 82-28 and PG 88-28). Complex modulus test was conducted at five different temperatures (-10, 4, 21, 37, 54˚C) and six loading frequencies (0.1, 0.5, 1, 5, 10 and 25 Hz). 2S2P1D rheological model was used to characterize the rheological properties of EME mixes. Environmental Scanning Electron Microscope (ESEM) was used to investigate the binders' microstructures in terms of density, fibril size and structural network along with time required for fibril formation. The obtained results showed that there was a considerable difference between the rheological properties of the mixes, and that EME 19 with PG 88-28 performed more elastically. On the other hand, complex modulus of EME mixes with PG 58-28 + 10% Elastomer additives had larger viscous component. In addition, a good correlation between EME rheological properties and microstructure of the modified binders was observed as explained in this paper.

5.1 Introduction

Modulus or stiffness is a fundamental design parameter of flexible pavements which can be used to determine the response of asphalt mix under traffic loading and thermal conditions. High-modulus asphalt mix, Enrobé à Module Élevé (EME) in French, is a type of asphalt mix representing high modulus or stiffness at intermediate temperature (14,000 MPa at 15˚C and 10 Hz loading frequency). EME was developed in the 1980's at Laboratoire Central des Ponts et Chaussées (LCPC) in France, in co-operation with road agencies (Sybilsky et al., 2010). It has high durability, superior rutting performance, and good fatigue resistance. The first type of EME was patented in 1980 and about five years after implementation, a significant number of applications of this type of mix were reported. Since the first application of EME, it has been included in several manuals, including: SCETAUROUTE's Manual of Pavement Design for Motorways, 1994 and Road Directorate's catalogue of new pavements, 1998. Over the years and by development and diversification of EME, it was decided to be included and codified in the AFNOR standard, published in October 1992, under reference number NF P 98-140.

EME is a very good option to be used in lower and upper binder courses in the pavement structure which are subjected to the highest levels of tensile and compressive stresses (Backer et al., 2008). EME was initially designated to reinforce old pavement structure and reconstruct thinner layers in urban areas due to having underground facilities such as pipes and curbs which restricts the pavement thickness to a specific value (Corte, 2001). Additionally, it has the advantage of avoiding the complete removal of old asphalt layer as it contributes to lower pavement thickness (Capitão and Picado-Santos, 2006). It also has been used to reduce the pavement construction cost by reducing the thickness of road pavement especially when the aggregates had low crushing index value or in case the traffic was intense, slow and channeled (Caroff and Corté, 1994). EME offers other advantages over conventional base course material used in pavement structure including: improved structural life, increase in axle loading, and environmental benefits by saving in raw materials (Distin and Vos, 2015).

To develop impermeable high modulus (or stiffness) asphalt mix with excellent performance, utilizing hard grade straight-run asphalt binder (e.g. 15/25, 10/20 or even 5/15 penetration grade asphalt at 25˚C) is required. In this regard, compactible and dense aggregate structure is also a very important element. Therefore, to achieve the optimum result in EME performance, using optimum amount of a hard grade asphalt binder along with a suitable aggregate particle gradation are key factors. Using hard grade asphalt, however, would considerably increase the risk of occurrence of low-temperature (thermal) cracking in cold climatic regions (Rys et al., 2017, Judycki et al., 2015). A solution to this problem can be developing polymer modified asphalt binders with enhanced mechanical performance (Wang et al., 2017, Ranieri and Celauro, 2018, Wang et al., 2018).

The main purpose of this study is to characterize the viscoelastic behavior of EME mixes fabricated with different modified asphalt binders at elevated temperatures and loading frequencies, and to find correlations between the rheological parameters and binders' microstructures.

5.2 Linear Viscoelastic Behavior of Asphalt Mix

Asphalt mix is a viscoelastic material and its properties highly depend on ambient temperature as well as loading conditions (loading type and frequency). Modulus (E^*) which presents the linear viscoelastic (LVE) behavior of asphalt mix within small range of strains (ε < 100 μm/m) and loading cycles is an important asphalt mix property (Airey and Rahimzadeh, 2004, Di Benedetto et al., 2001) It is also used as a key input parameter of structural pavement design according to Mechanistic-Empirical Pavement Design Guide (MEPDG) (NCHRP, 2004). When a viscoelastic material is subjected to sinusoidal type of loading, due to the viscous property of the material, a phase lag (φ) between stress and strain exists. This lag is between zero and ninety degrees. Equations 5.1 and 5.2 present this type of behavior under such condition:

$$\sigma(t) = \sigma_0 \mathrm{Sin}(\omega t) \qquad \text{Equation 5.1}$$

$$\varepsilon(t) = \varepsilon_0 \mathrm{Sin}(\omega t - \varphi) \qquad \text{Equation 5.2}$$

σ_0 and ε_0 are stress and strain amplitudes under specific conditions. t is time. ω is called pulsation which is equal to $2\pi f$ (f is the loading frequency). Introducing a complex number $i^2 = -1$, Equations 5.1 and 5.2 can be re-written in the following forms:

$$\sigma^* = \sigma_0 e^{i\omega t} \qquad \text{Equation 5.3}$$

$$\varepsilon^* = \varepsilon_0 e^{i(\omega t - \varphi)} \qquad \text{Equation 5.4}$$

The axial modulus can then be calculated using Equation 5.5 (Kim, 2008):

$$\frac{\sigma^*}{\varepsilon^*} = E^*(i\omega) = \left(\frac{\sigma_0}{\varepsilon_0}\right) e^{i\varphi} = E_1 + iE_2 \qquad \text{Equation 5.5}$$

According to Equation 5.5, the modulus consists of two components. The real part of the complex modulus (E_1) is called elastic or storage modulus and the imaginary part is referred to as viscous or loss modulus (E_2). Elastic modulus is related to the amount of energy stored in sample during each loading cycle which will be recovered once the load is removed. Viscous modulus, on the other hand, is attributed to amount of energy that is lost (Venudharan et al., 2016). The ratio of stress to strain amplitudes is defined as dynamic (sometimes referred to as cyclic) modulus, Pa:

$$|E^*(\omega)| = \frac{\sigma_0}{\varepsilon_0} = \sqrt{E_1^2 + E_2^2} \qquad \text{Equation 5.6}$$

5.3 Objectives and Procedures

This study aims to characterize the viscoelastic behavior of high-modulus asphalt mixes with three types of modified asphalt binders. To achieve this aim the following steps are designated:

(a) Two types of EME mixes based on nominal maximum aggregate size (NMAS), EME 12.5 and EME 19, are fabricated using different modified asphalt binders.
(b) Binder morphology and rheological properties are assessed.
(c) Complex (dynamic) modulus test is conducted in a wide range of temperatures and loading frequencies.
(d) 2S2P1D rheological model is used to model and compare the viscoelastic behavior of the binders and the mixes.

5.3.1 Used Aggregates and Asphalt Binders

Aggregate particles were provided from Havelock Quarry located in Northern Ontario, Canada. Physical and mechanical properties of used aggregates are provided in Table 5 - 1. The gradation charts of EME mixes are depicted in Figure 5 - 1. Modified PG 88-28 and PG 82-28 as well as unmodified PG 58-28 asphalt binders were used in this study. 10% Elastomer additive by weight of total binder was used in order to enhance the thermo-mechanical properties of PG 58-28 binder (Tang et al., 2018).

Table 5 - 1: Properties of used aggregates

Property	Value	Standard method
Coarse aggregate		
Micro-Deval (%)	8.80	LS-618
Fractured faces (more than one) (%)	100	LS-607
Flat/Elongated (4:1) (%)	8.30	LS-608
Specific gravity		LS-604
Dry	2.87	
SSD	2.88	
Apparent	2.90	
Absorption (%)	0.34	LS-604
Fine aggregate		
Micro-Deval (%)	11.05	LS-619
Specific gravity		LS-605
Dry	2.89	
SSD	2.90	
Apparent	2.92	
Absorption (%)	0.38	LS-605
Wash loss (%)	8.50	LS-601

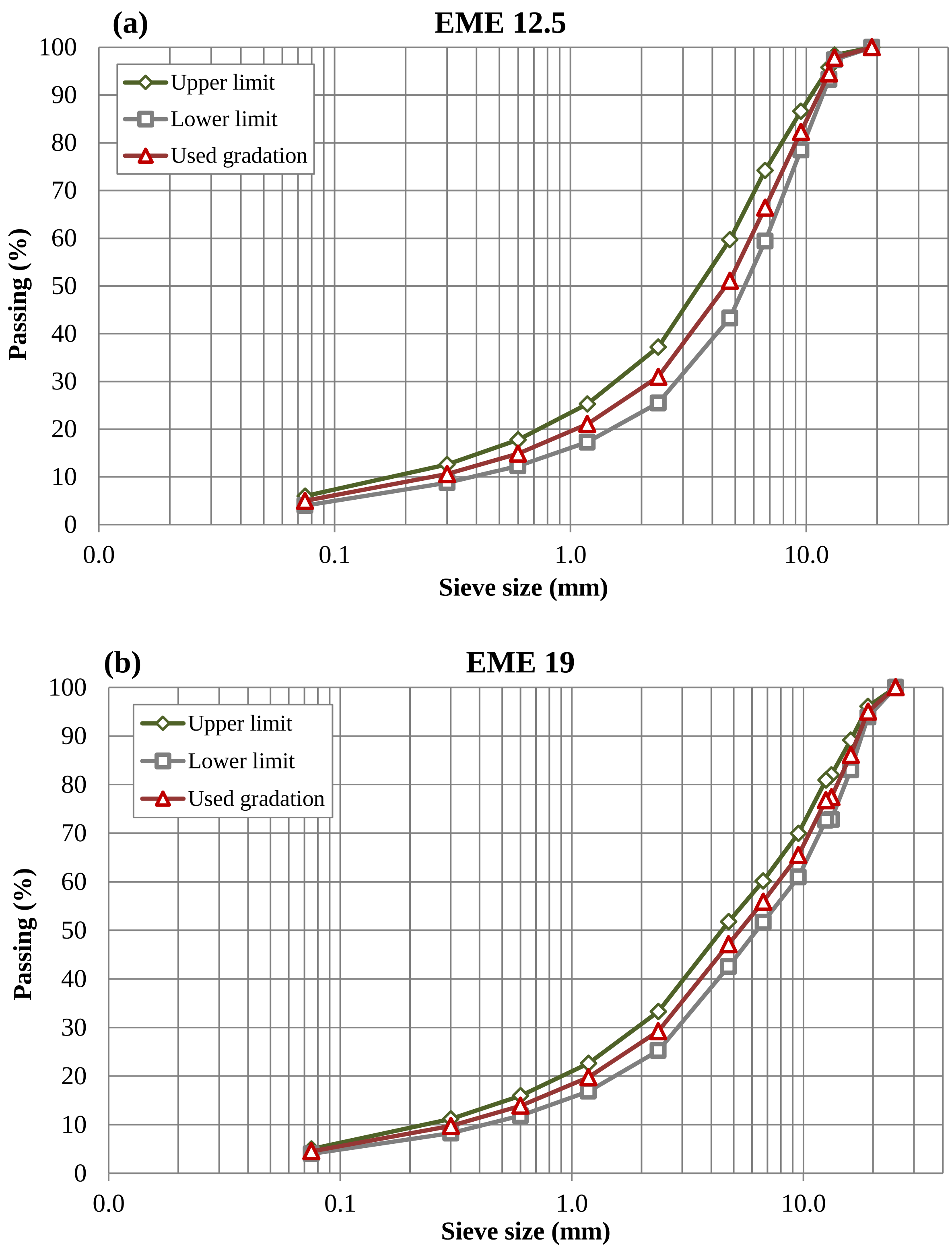


Figure 5 - 1: Gradation charts: (a) EME 12.5; (b) EME 19

5.3.2 Binder Morphology Analysis Using Environmental Scanning Electron Microscope (ESEM)

To have an idea about microstructure of the modified binders, images were taken from the binders' microstructures using FEI Quanta 250 FEG environmental scanning electron microscope (ESEM). The scanning was performed at room temperature. The observation parameters were selected as below to get a clear view of binders' microstructures (Mikhailenko et al., 2017):

i. acceleration voltage: 20 keV;
ii. chamber pressure: 0.8 mbar in low vacuum mode;
iii. magnification: 1000x in secondary electron (SE) mode.

5.3.3 Dynamic Shear Rheometer Test

Dynamic shear rheometer (DSR) sweep test is very useful method to evaluate the viscoelastic properties of asphalt binders. The test was conducted using Anton Paar SmartPave 102 Asphalt Rheometer. The test was carried out eleven temperatures; and sixteen frequencies were designated. Two different temperature rages (2 to 35˚C and 40 to 90˚C) were used. For the lower temperature range, an 8-mm plate with a gap of 2 mm was used as for higher temperature range an 25-mm plate with smaller gap (1 mm) was utilized. In addition, a frequency range of 1 to 100 rad/s was designated for each temperature. It must be mentioned that the linear viscoelastic checks were performed before starting the test to make sure the material's behavior would stay within the linear domain.

5.3.4 Asphalt Mix Preparation

To fabricate asphalt mixes, asphalt binders and blended aggregates were heated in an oven to reach to their mixing temperatures. Mixing temperatures were determined based on the asphalt binders' viscosities. The mixing temperatures of 155˚C, 165˚C, and 175˚C were used for PG 58-28 + 10% Elastomer additives, PG 82-28 and PG 88-28 respectively. After mixing the aggregates and the asphalt binders, loose mixes were conditioned at 135˚C for four hours prior to compaction. This was done to simulate the short-term aging of asphalt mixes in the field according to AASHTO R 30-02 (2015). Thereafter, the loose mixes were compacted at their compaction temperatures (145˚C for PG 58-28 + Elastomer additives, 155˚C for PG 82-28 and 165˚C for PG 88-28). It is worth mentioning that the binder contents were determined according to the richness factor (K) criterion which correlates with binder film thickness in the mix. The richness factor of 3.5 and 3.0 were used for EME 12.5 and EME 19 respectively according to Equation 5.7 (NF EN 13108-1, 2007):

$$K = \frac{\frac{100B}{100 - B}}{a\sqrt[5]{\sum}} \qquad \text{Equation 5.7}$$

In this equation, B is the ratio of binder mass to the mass of mixture; a is the correction coefficient relative to the aggregate density (2.65/aggregate density); $\sum$ is the specific surface area.

5.3.5 Asphalt Mix Complex Modulus Rheological Test

Complex modulus (E^*) test is used to determine stress and strain responses in asphalt mix and correlates the time-temperature dependant properties of the mix to field performance. Complex (dynamic) modulus test was conducted according to AASHTO T 342 (2011). Cylindrical samples were prepared using SGC. The samples were then cored and cut to produce specimens with 100 mm diameter and 150 mm height. In this test, a sinusoidal axial compressive load with 0.1-25 Hz loading frequencies is applied on each specimen at five different temperatures ranges between -10˚C and 54˚C. The applied load and the corresponding displacement response of the specimen were measured continuously using a data acquisition system. The modulus values were calculated for each loading frequency and temperature.

5.3.6 2S2P1D Rheological Model

2S2P1D rheological model is a powerful tool to characterize behavior of asphalt binders as well as mixes (Di Benedetto et al., 2004b, Olard and Di Benedetto, 2003). 2S2P1D which is combination of physical elements (two springs, two parabolic elements and one dashpot) is generalization of the Huet-Sayegh model (Sayegh, 1965). Figure 5 - 2 provides a representation of 2S2P1D model. Equation 5.8 is expression of 2S2P1D model at a given temperature which consists of seven parameters as described below (Olard and Di Benedetto, 2003):

$$E^*_{2S2P1D}(j\omega\tau) = E_\infty + \frac{E_0 - E_\infty}{1 + \delta(j\omega\tau)^{-k} + (j\omega\tau)^{-h} + (j\omega\beta\tau)^{-1}} \qquad \text{Equation 5.8}$$

where;

E_0: the glassy modulus when $\omega \rightarrow \infty$;

E_∞: the static modulus when $\omega \rightarrow 0$;

ω: the solicitation pulsation = $2\pi f$ (f= frequency);

h, k: exponents such as $0 < k < h < 1$, related to the ratio $E_{imaginary}/E_{real}$ when ω tends to 0;

δ: dimensionless constant which works as a shape factor;

τ: characteristic of time, which varies with temperature variation;

j: complex number defined ($j^2 = -1$);

β: constant value related to the dashpot's viscosity, $\eta = (E_0 - E_\infty)\beta\tau$.

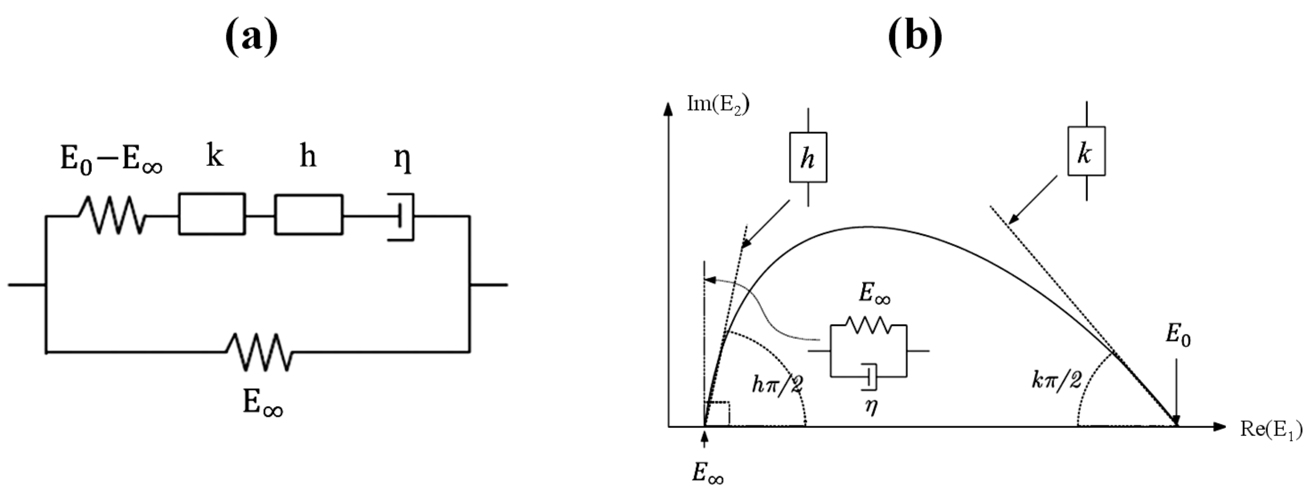


Figure 5 - 2: (a) Analogue representation of the 2S2P1D Model; (b) representation of the model parameters on a Cole-Cole diagram (Mangiafico et al., 2016, Ramirez Cardona et al., 2015)

5.4 Results and Discussion

This section provides the laboratory test results. 2S2P1D rheological model parameters are also determined for the modified asphalt binders as well as EME mixes using Excel Solver. The information and discussions on the test results are provided in the following subsections.

5.4.1 ESEM Test Results

ESEM apparatus was used to evaluate microstructures of the modified binders. As can be seen in Figure 5 - 3, structures of the binders are dense. The microstructure of PG 88-28 modified binder is more connected and larger in diameter compared to other binder types. PG 58-28 + Elastomers on the other hand has the thinnest fibril size. In addition, the fibril formation took the longest time for PG 88-28 binder (218 s); followed by148 s and 46 s for PG 82-28 and PG 58-28 + 10% Elastomer respectively. This represents that higher energy is required to push the lighter components of PG 88-28 binder compared to other binder types.

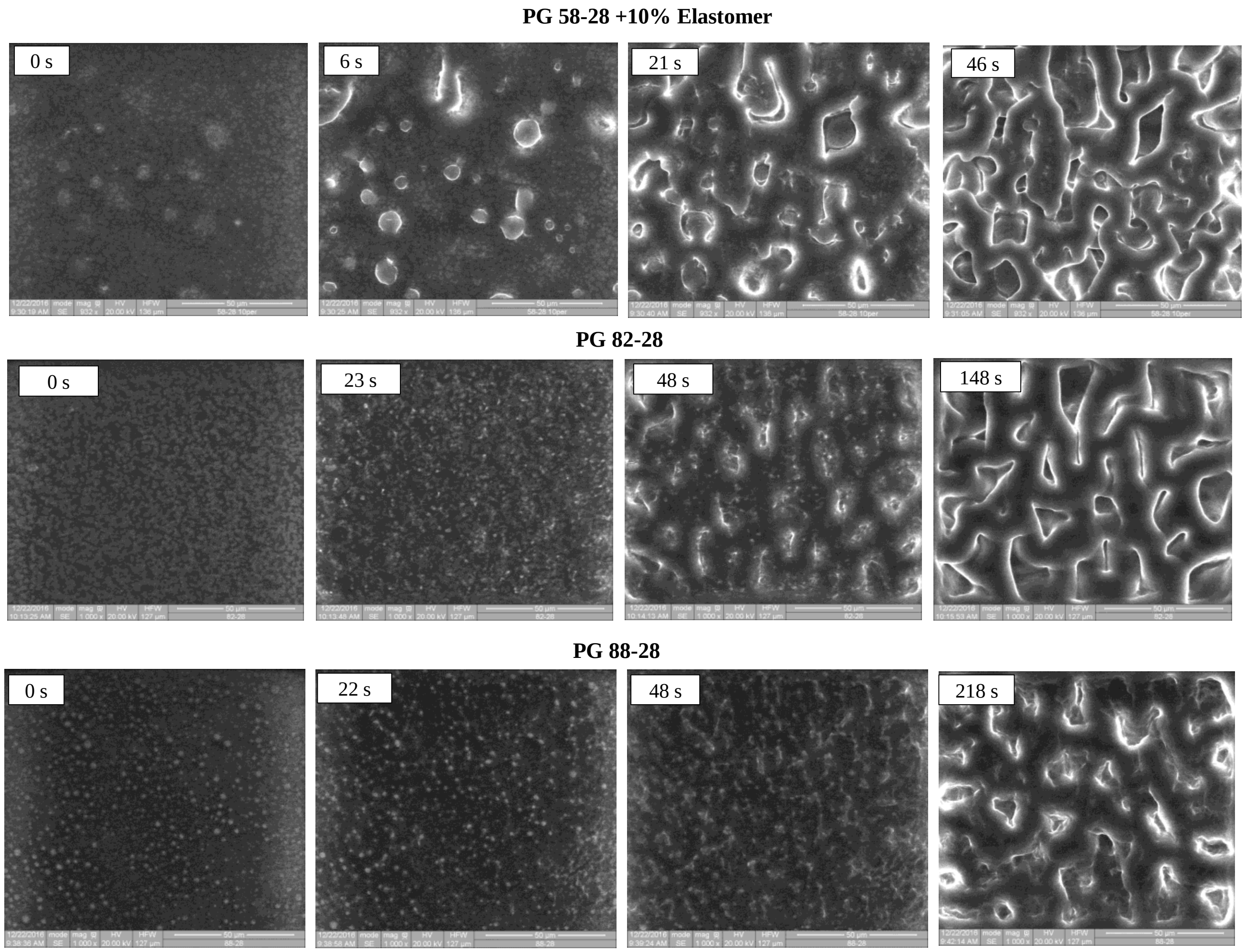


Figure 5 - 3: Images of PG 58-28 +10 % Elastomer, PG 82-28 and PG 88-28 at different observation time

5.4.2 Rheological Analysis

It is common to interpret and compare the modulus (or phase angle) values at a wide range of frequencies with respect to one reference temperature. This can be done by developing master curve using time-temperature superposition principles of viscoelastic material as shown in Figure 5 - 4. In the construction of master curves, isotherms of modulus tested at multiple temperatures are shifted by applying a multiplier (shift factor - a_t) to the frequency (or time) at which the measurement is taken so that the individual isotherms of stiffness are combined to form a single smooth curve of frequency or time versus stiffness (the master curve) (Rowe and Sharrock, 2011).

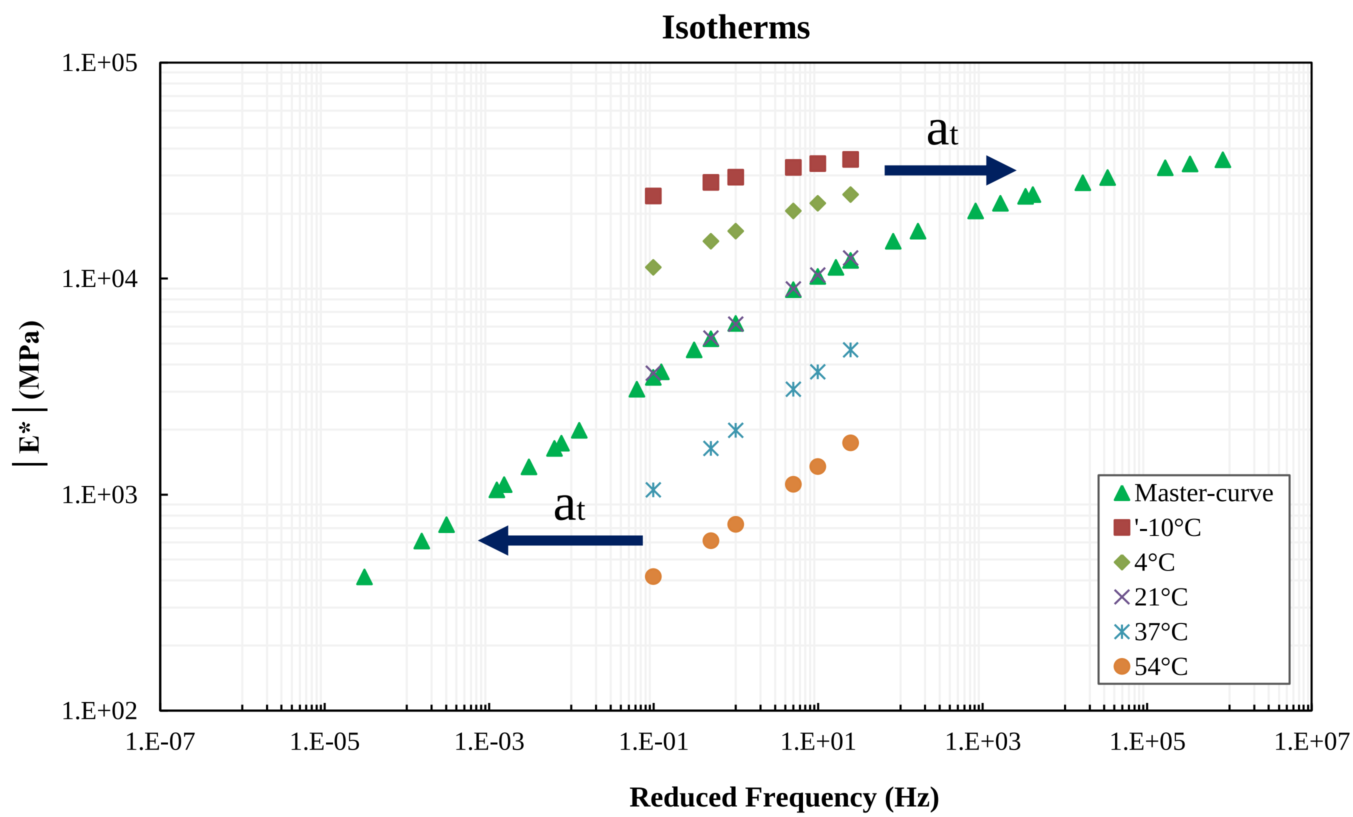


Figure 5 - 4: Developing Master Curve based on time-temperature superposition principle of viscoelastic material: Case of Dynamic Modulus |E*|; t_{ref} = 21˚C

Shift factors can be calculated using Williams–Landel–Ferry (WLF) equation:

$$\log a_t = -\frac{C_1(t - t_{ref})}{C_2 + (t - t_{ref})} \qquad \text{Equation 5.9}$$

In this equation, C_1 and C_2 are constants; t_{ref} is reference temperature which is sometimes referred to as glass transition temperature (Ramirez Cardona et al., 2015, Rowe and Sharrock, 2011).

5.4.2.1 Asphalt Binders

Rheological parameters (shear modulus and phase angle) of modified asphalt binders were measured using DSR test. Modulus and phase angle master curves were developed at reference temperature of 21˚C and are shown in Figure 5 - 5. Table 5 - 2 also lists binder shift factors for each test temperature. As can be illustrated in Figure 5 - 5 (a), PG 88-28 has the highest shear modulus, although at a very high frequency (more than 10^4 rad/s) PG 58-28 + 10% Elastomer was comparably high. It is worth noting that unlike the other binder types, PG 88-28 modulus master curve is linear, and no inflection point could be observed. This would indicate that higher temperature (more than 90˚C) is required to reach to that point for PG 88-28 binder. Comparing the results of both DSR test and ESEM test, it can be concluded that the binders with dense microstructure corresponds to high stiffness (Mikhailenko et al., 2019).

Figure 5 - 5 (b) depicts there is a considerable difference in binders` phase angles. The results illustrated that PG 58-28 + 10% Elastomer had the highest phase angle at frequencies lower than 10^2 rad/s. Conversely, PG 88-28 showed to have the lowest phase angle. In addition, the phase angle for PG 88-28 binder behaved very differently at lower frequencies (higher temperatures) compared to the other binder types. That is to say, PG 88-28 has performed more elastically at higher temperatures which might be due to high polymerization and bridging effect in the binder. The results are compatible with the ESEM test results where PG 88-28 showed to have thick fibril chains with more complex network.

The 2S2P1D model is used to model the binders' viscoelastic behaviors. The model parameters are provided in Table 5 - 3. Fitted model to the experimental data are also provided in Figure 5 - 5. As shown in this figure, the models could precisely fit to the modulus values in all cases. Although the model may not precisely reflect the effect of polymer modification on the phase angle particularly for PG 58-28 + Elastomer binder.

Table 5 - 2: Log shift factors (a_t) for the tested asphalt binders at t_{ref} = 21 °C

Asphalt binder type	**Temperature (˚C)**											**WLF parameters**	
	2	**5**	**15**	**25**	**35**	**40**	**50**	**60**	**70**	**80**	**90**	**C_1**	**C_2**
PG 58-28 +10% Elastomer additives	2.62	2.19	0.76	-0.54	-1.53	-2.02	-2.79	-3.47	-4.04	-4.51	-4.94	13.1	111.2
PG 82-28	2.50	2.05	0.70	-0.46	-1.45	-1.90	-2.61	-3.31	-3.96	-4.56	-5.10	14.6	130.3
PG 88-28	2.61	2.16	0.84	-0.38	-1.47	-2.81	-3.62	-4.30	-4.88	-5.41	-5.93	19.8	151.3

Table 5 - 3: 2S2P1D model parameters for the modified asphalt binders (t_{ref} = 21˚C)

Asphalt binder type	**G_∞ (Pa)**	**G_0 (Pa)**	**k**	**h**	**δ**	**τ_0 (s)**	**β**
PG 58-28 + Elastomers	0	10^9	0.55	0.98	77.42	0.00025	2.45×10^{12}
PG 82-28	0	10^9	0.48	0.98	800.71	0.00412	2.45×10^{12}
PG 88-28	0	10^9	0.17	0.53	4.16	0.00000028	2.45×10^{12}

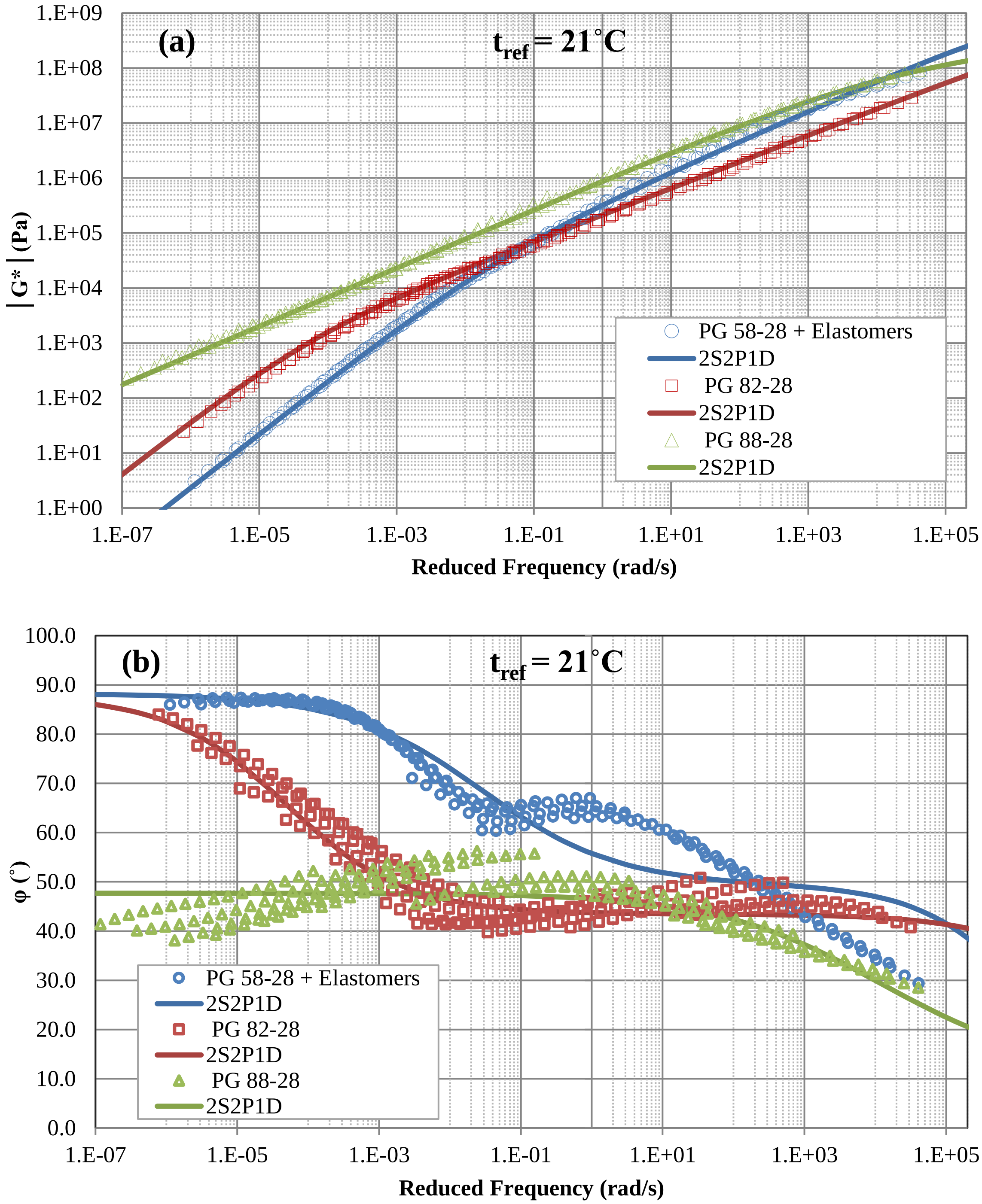


Figure 5 - 5: Master curves of modified binders: (a) shear modulus |G*|; (b) phase angle (φ)

5.4.2.2 EME Mixes

Complex modulus test was conducted on EME mixes at elevated temperatures and loading frequencies using Material Testing System (MTS) 810 testing frame. Master curves were developed by shifting the isotherms. Log shift factors (a_t) of the mixes are listed in Table 5 - 4. It could be observed that the mixes with PG 88-28 binder have the highest shift values compared to those fabricated with the other binder types which could be associated with the higher stiffness of this binder. The developed master curves for EME 12.5 and EME 19 are plotted in Figure 5 - 6 and Figure 5 - 7 respectively. In addition, the 2S2P1D model parameters are calculated and provided in Table 5 - 5.

As can be seen in Figure 5 - 6 and Figure 5 - 7, all the mixes had high stiffness. Mixes with PG 82-28 binder had the lowest moduli values under 10 Hz loading which were around 9,000 MPa for EME 12.5 and more than 10,000 MPa for EME 19 which are quite high. Both EME 12.5 and EME 19 with PG 88-28 represented higher stiffness at medium and lower frequencies. In addition, at a very high frequency (e.g. 10^4 Hz) the mix with PG 58-28 + Elastomers had the highest modulus, although the difference in moduli among the mixes is less at higher loading frequencies.

The mixes had low phase angles ($\varphi < 30^\circ$). The lowest phase angles belonged to PG 88-28 which represents the highest elastic behavior of the mixes. Further, elastic return, which is defined as the point where the phase angle reaches to its maximum, could be clearly observed for mixes with PG 58-28 + Elastomers and PG 82-28 binder types. However, this point was not clearly observed for EME mixes fabricated with PG 88-28.

It is also worth mentioning that based on the achieved test results; a clear correlation was observed between the binders' microstructure (Figure 5 - 3) and phase angles of the mixes. The binders with denser structure and stronger bonds resulted in lower phase angles. For the modulus; however, it was shown that binder with higher intertwined network caused higher mix moduli particularly at higher frequencies (lower temperatures). For instance, mixes with PG 58-28 + Elastomers binder had higher moduli than the mixes with PG 82-28 although PG 82-28 binder was denser, and its fibril size was thicker.

Table 5 - 4: Log shift factors (a_t) for the EME mixes; t_{ref} = 21 °C

EME	**Asphalt binder type**	**Temperature (˚C)**					**WLF parameters**	
		-10	**4**	**21**	**37**	**54**	**C_1**	**C_2**
12.5	PG 58-28 + Elastomers	3.83	1.85	0.00	-1.78	-3.13	18.2	178.7
	PG 82-28	4.15	1.97	0.00	-1.64	-3.02	19.8	178.5
	PG 88-28	4.36	2.17	0.00	-1.81	-3.45	26.4	218.8
19	PG 58-28 + Elastomers	4.23	1.94	0.00	-1.80	-3.36	20.1	178.4
	PG 82-28	4.09	1.90	0.00	-1.70	-2.93	19.5	178.5
	PG 88-28	4.53	2.68	0.00	-1.76	-2.77	21.5	178.3

Table 5 - 5: 2S2P1D model parameter for EME mixes (t_{ref} = 21˚C)

EME	**Asphalt binder type**	**E_∞ (MPa)**	**E_0 (MPa)**	**k**	**h**	**δ**	**τ_0 (s)**	**β**
12.5	PG 58-28 + Elastomers	495.2	39422.6	0.233	0.568	3.831	0.076	9×10^9
	PG 82-28	560.7	48268.2	0.171	0.442	2.954	0.005	9×10^9
	PG 88-28	289.7	51823.2	0.111	0.287	1.872	0.003	9×10^9
19	PG 58-28 + Elastomers	313.3	57403.6	0.139	0.435	2.222	0.004	9×10^9
	PG 82-28	837.3	44290.0	0.191	0.483	2.730	0.015	9×10^9
	PG 88-28	361.4	53119.5	0.131	0.414	3.491	0.088	9×10^9

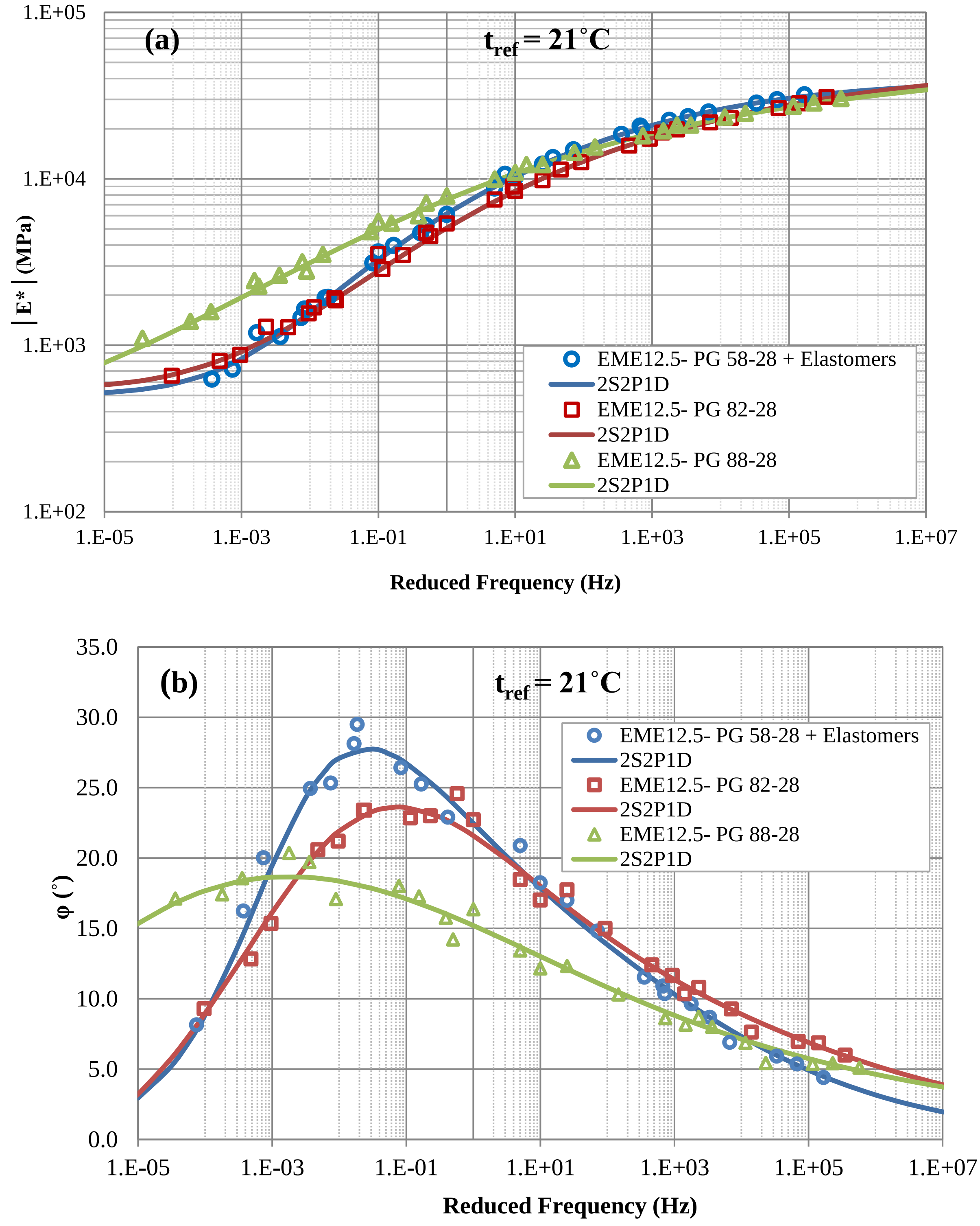


Figure 5 - 6: Master curves for EME 12.5: (a) dynamic modulus $|E^*|$; (b) phase angle (φ)

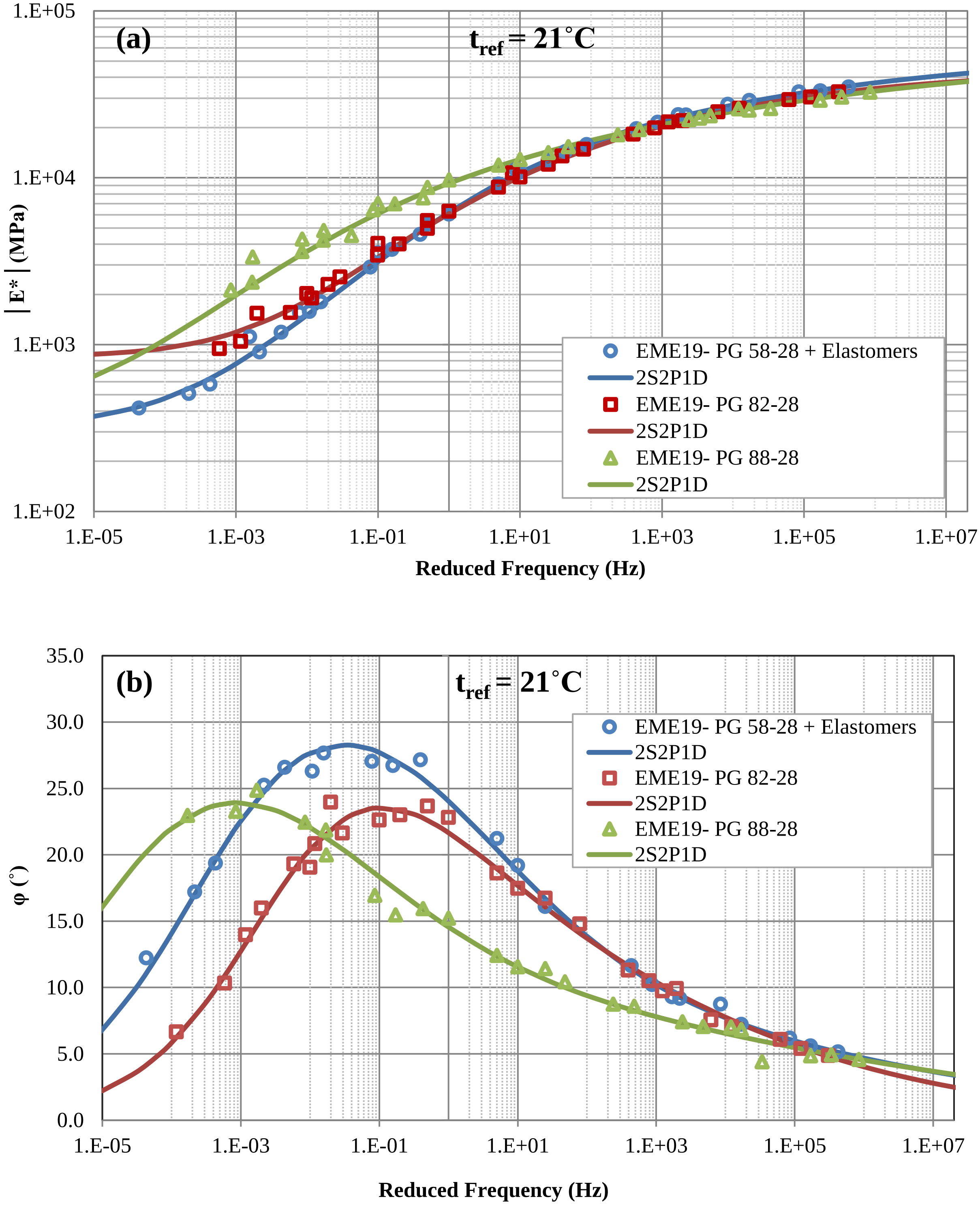
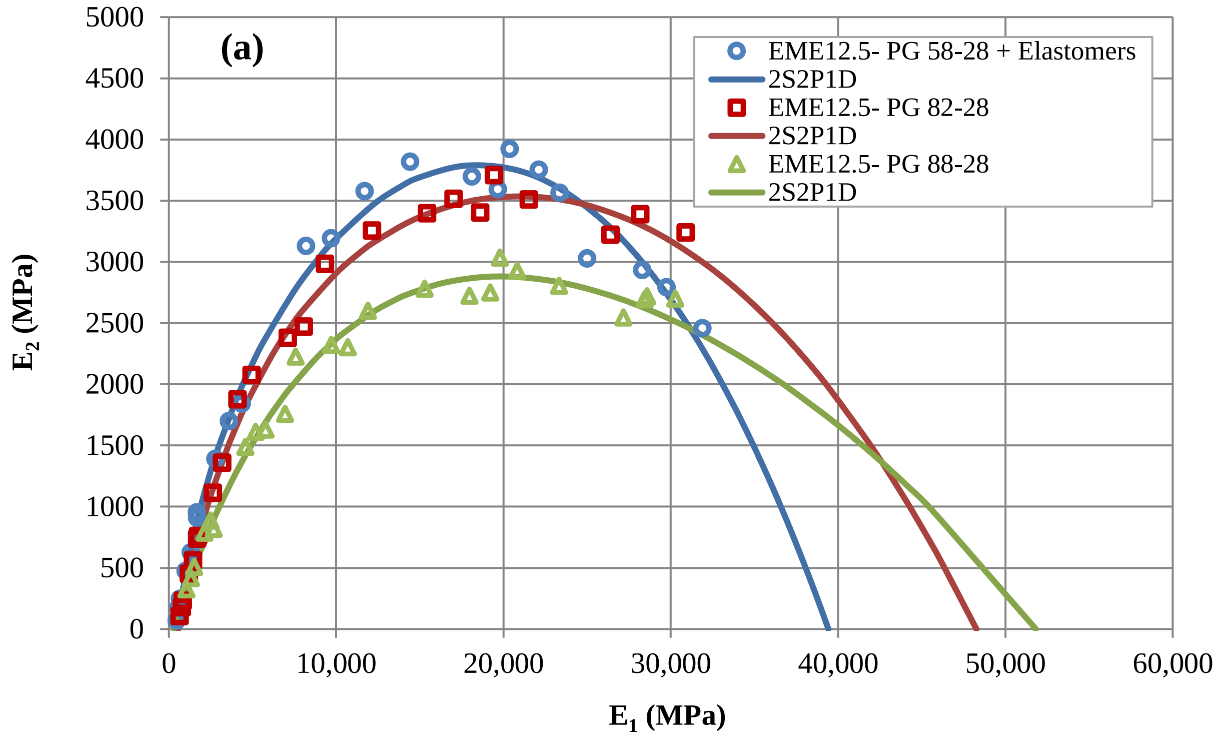


Figure 5 - 7: Master curves for EME 19: (a) dynamic modulus $|E^*|$; (b) phase angle (φ)

5.4.2.2.1 Cole-Cole and Black Space Diagrams

Cole-Cole plot can be used to express the relation between elastic component (E_1=|E*|cos (φ)) and viscous component ((E_2=|E*|sin (φ)) of complex modulus. Additionally, the relation between the norm of the complex modulus (|E*|) and phase angle (φ) can be shown using black space diagram. Cole-Cole and black space diagrams of EME mixes are plotted in Figure 5 - 8 and Figure 5 - 9 respectively. Simulated curves are also plotted for each of the tested materials using 2S2P1D model (Equation 5.8). Figure 5 - 8 illustrates the viscous component (E_2) of the mixes with PG 58-28 + Elastomers is the highest among all mix types particularly at higher temperatures. This would indicate that higher amount of energy was lost in the mixes with PG 58-28 + Elastomers under loading cycle due to permanent flow or deformation (also known as dissipated energy (Specht et al., 2017). On the other hand, the mixes with PG 88-28 binder behaved more elastically which would mean higher amount of energy was recovered once the load was removed (Xiao et al., 2012).

The viscous components of the moduli were not significantly influenced by aggregate gradation; however, the elastic components were considerably affected. The elastic component (real part) of EME 19 modulus was high compared to EME 12.5 which might be associated with higher packing degree of aggregate blends in this mix (Baghaee Moghaddam and Baaj, 2018). Further, as can be seen in Figure 5 - 9, phase angles of the mixes were not affected by change in aggregate gradations except for EME 19 with PG 88-28 binder.



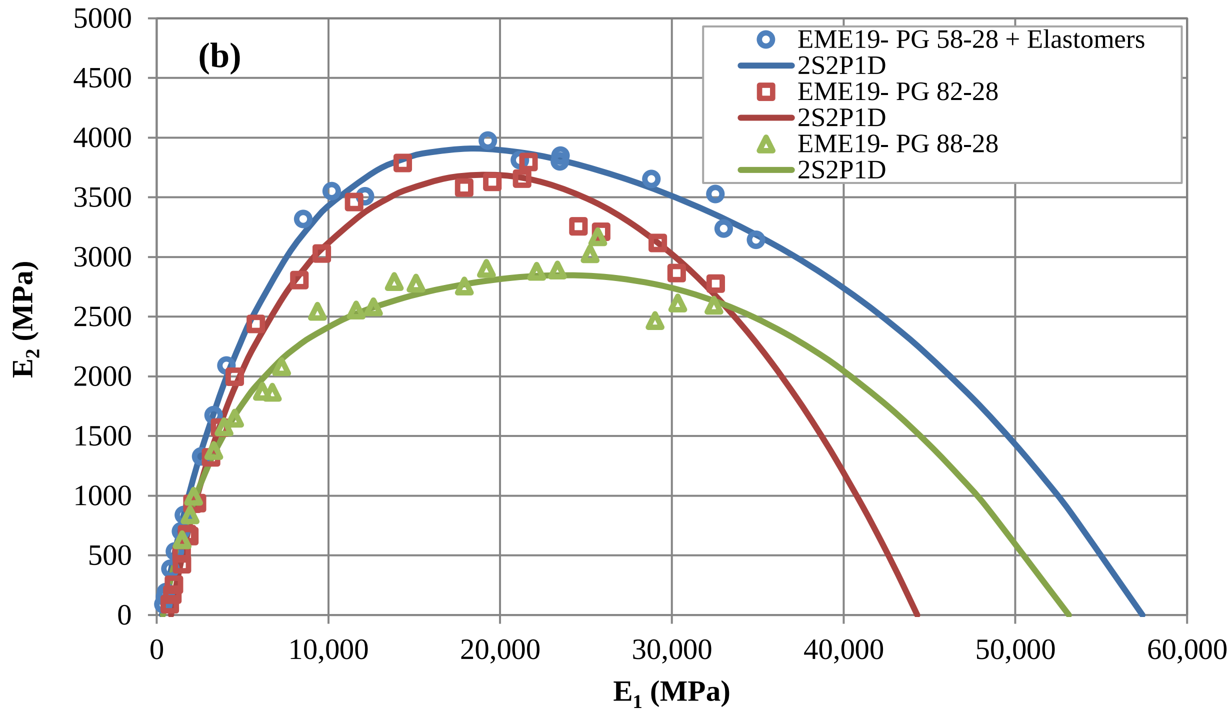


Figure 5 - 8: Cole-Cole diagrams: (a) EME 12.5; (b) EME 19

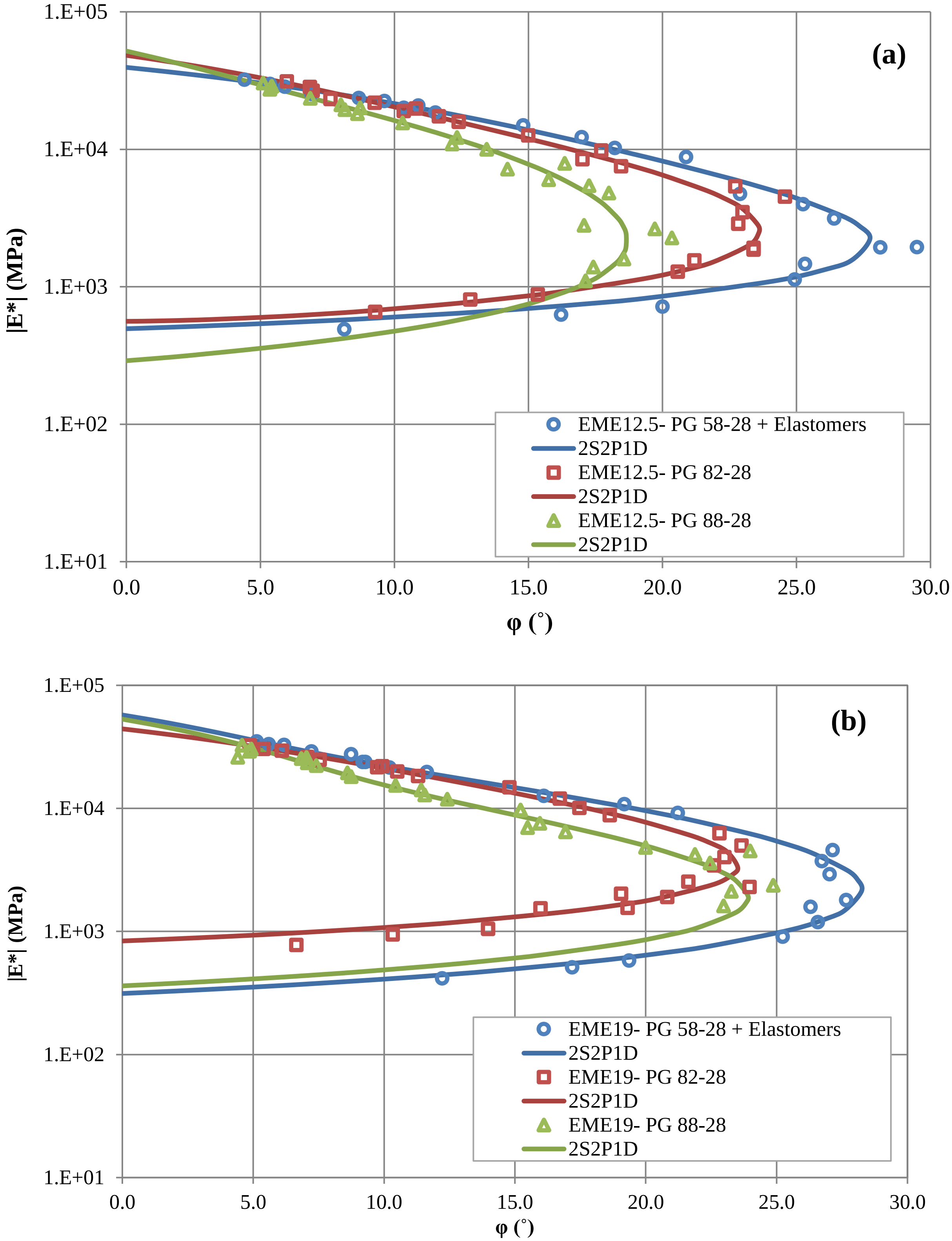


Figure 5 - 9: Black space diagrams: (a) EME 12.5; (b) EME 19

5.5 Conclusions

This paper investigated the rheological properties of different types of highly modified asphalt binders (PG 58-28 + Elastomers, PG 82-28 and PG 88-28) and EME mixes (EME 12.5 and EME 19). For this purpose, dynamic shear rheometer (DSR) test and complex modulus test were conducted at a wide range of environmental temperatures and loading frequencies. In addition, binder morphology analysis was performed at room temperature using ESEM. 2S2P1D rheological model was used to characterize the rheological behavior of the modified binders and EME mixes. The obtained results are summarized as follows:

(1) ESEM test results showed a significant difference in binders' microstructures. According to the pictures, microstructure of PG 88-28 binder was denser and connected with thicker fibril size compared to the other binder types. PG 58-28 + Elastomers had highly intertwined structural network with the thinnest fibril size.

(2) DSR test results showed that PG 88-28 had the highest shear modulus values due to its higher stiffness. Although at a very high loading frequency, PG 58-28 + Elastomers showed similar stiffness values to the PG 88-28 binder. Phase angle of PG 88-28 binder was low and behaved differently compared to the other binder types which might be due to higher polymer modification rates in this binder.

(3) Dynamic modulus test results of EME mixes showed that among the mixes those with PG 82-28 binder had the lowest modulus values at 10 Hz loading speed.

(4) All of the mixes had relatively low phase angles. Mixes with PG 88-28 binder generally had lower phase angle values which represented this mix behaved more elastically compared to others.

(5) A clear correlation was observed between the binder's microstructure and the phase angles. Binders with denser structure and stronger bonds resulted in lower mix phase angles. For the modulus; however, it was observed that binders with intertwined network caused higher mix moduli particularly at higher loading frequencies.

(6) Cole-Cole diagrams depicted that at higher temperatures the viscous component (E_2) of the EME mixes with PG 58-28 + Elastomers was the highest among all mix types. Consequently, higher amount of energy was lost in this mix (higher dissipated energy).

(7) In general, EME 19 showed higher elastic modulus values than EME 12.5 which might be associated with the higher packing degree of aggregate blends in that mix.

CHAPTER 6
EME MIX DESIGN WITH THE SECOND SOURCE OF AGGREGATE MATERIALS

6.1 Introduction

As discussed in Chapter 3, source of used aggregates can considerably impact the packing degree in the aggregate blends and asphalt mix performance. Therefore, to validate the developed mix design approach and to see how different the results would be if the source of aggregate is changed, it is necessary to investigate the effects of second aggregate source on EME mix design. This chapter provides the laboratory test results and performance evaluation of the mixes for EME 12.5 and EME 19 using another source of aggerate materials.

6.2 Materials

Four fractions of aggregate materials namely: HL4, HL1, unwashed and washed fines were provided from Bark Lake Quarry, Ontario. The gradation curve of each aggregate fraction is illustrated in Figure 6 - 1. The PG 82-28 binder was used for this phase of the project because it had better overall performance according to the test results obtained in Chapter 4.

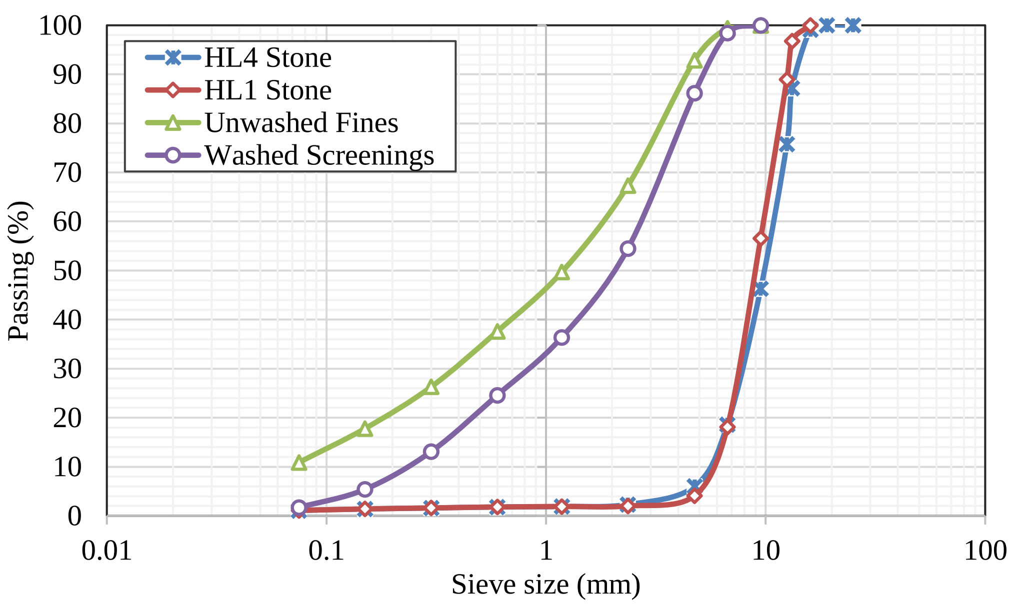
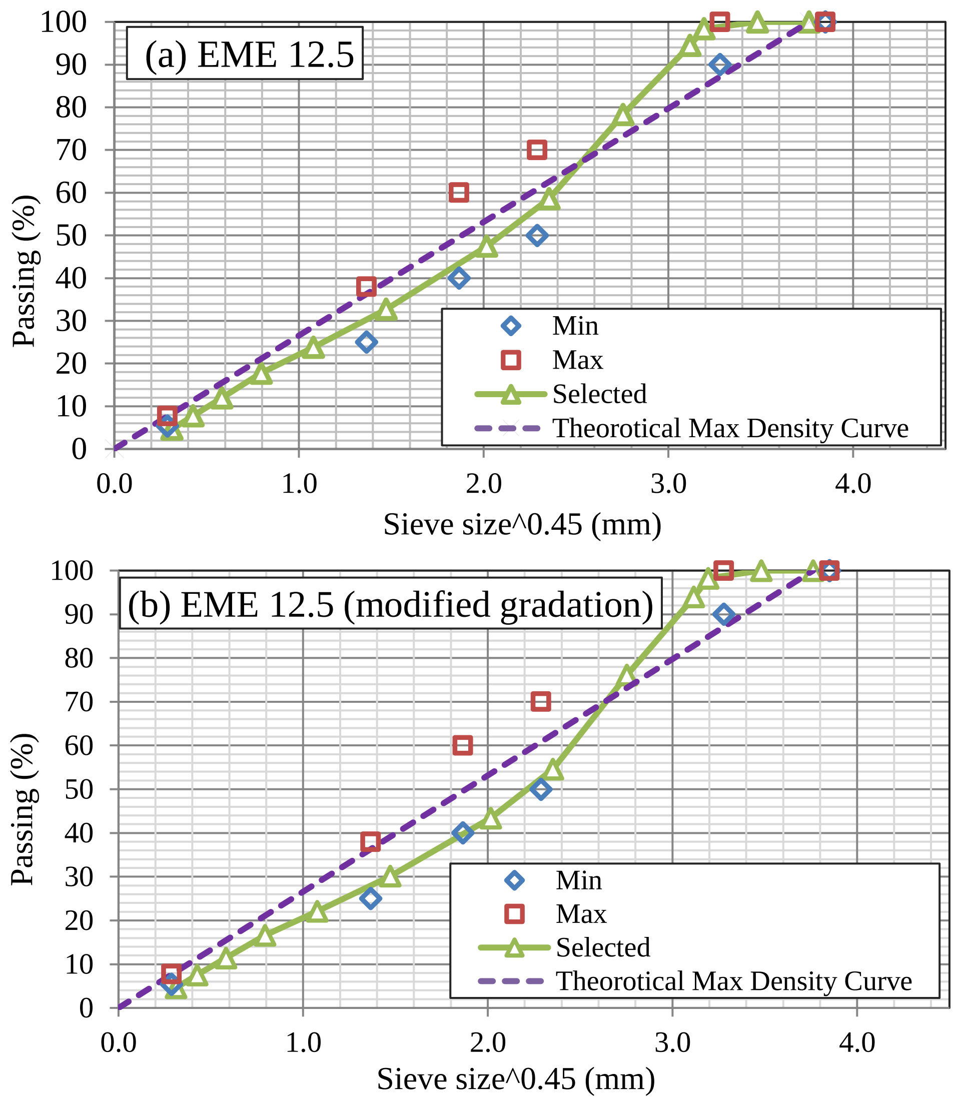


Figure 6 - 1: Gradation curve of each class of aggregate

The particle size distribution of EME 12.5 and EME 19 were selected according to the French gradation limits for EME mixes (NF EN 13108-1, 2007). The gradation curves of the mixes are provided in Figure 6 - 2 with respect to the maximum gradation curves. The gradation curve of EME 12.5 was modified because the minimum stiffness requirement of modulus was not satisfied initially.



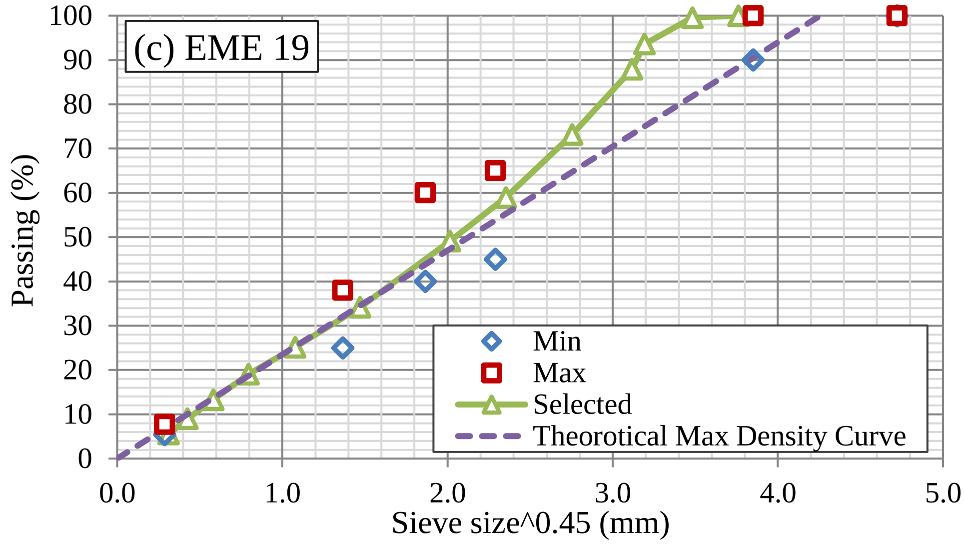


Figure 6 - 2: Aggregate particle size distributions with respect to the maximum density curve; (a) EME 12.5, (b) EME 12.5 (modified gradation), (C) EME 19

6.3 Laboratory Test Procedures

Different laboratory test methods were designated for EME mix design with the second source of aggregates including: compactibility analysis using SGC, dynamic modulus test, Hamburg wheel track rutting test, four-point bending beam fatigue test and TSRST. It must be mentioned that the same laboratory testing procedures were used as explained in the previous chapters.

6.4 Results and Discussions

This section provides the laboratory testing results and discussions of EME mixes with the second source of aggregates.

6.4.1 Dynamic Modulus Test Results

Master curves of the dynamic modulus test results of the mixes are developed at reference temperature of 10°C and illustrated in Figure 6 - 3. EME 12.5 with modified gradation had the highest modulus values at loading frequencies more than 10 Hz although its modulus was the lowest at lower frequencies (higher temperatures). According to the results EME 12.5 with modified aggregate gradation and EME 19 could meet the suggested modulus requirement of EME mixes for cold climatic regions ($|E^*| \geq 14{,}000$ MPa at

10°C and 10 Hz loading). Therefore, these two mixes were selected for the rest of testing and performance evaluation. It is also worth mentioning that for the rest of testing and evaluation, EME 12.5 with modified gradation is simply referred to as EME 12.5.

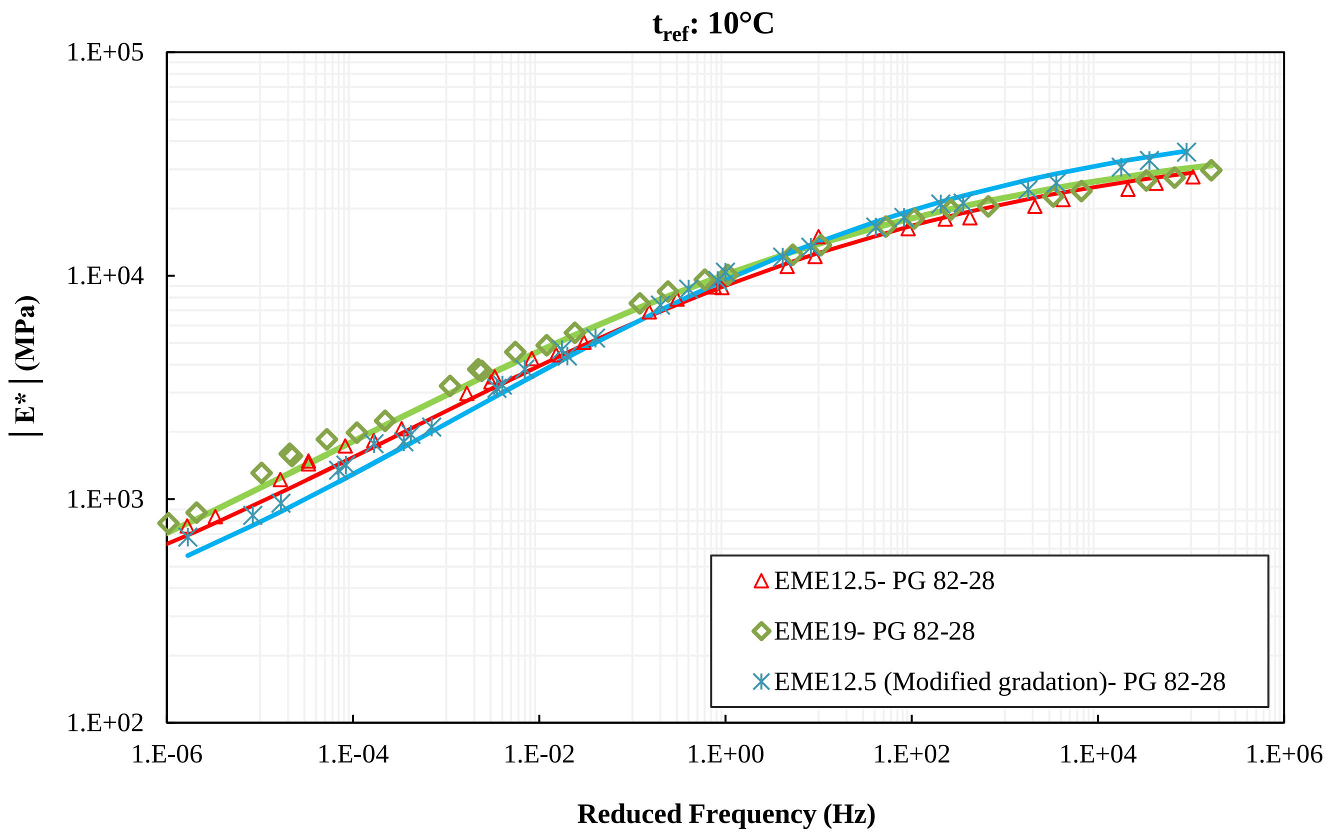


Figure 6 - 3: |E*| master curves at 10˚C reference temperature

6.4.2 Compactibility and Volumetric Properties

Compaction ability of the mixes were evaluated for both mixes using SGC, and the results are shown in Figure 6 - 4. As can be observed form this figure, both mixes have high compactibility. The mixes had around 4-5% air voids after 50 gyrations and it reduced to around 2% at the design number of gyrations (N_{des}=125).

Table 6 - 1 also provides the volumetrics of the mixes. It is clear from the results that EME 12.5 had slightly lower air voids compared to EME 19.

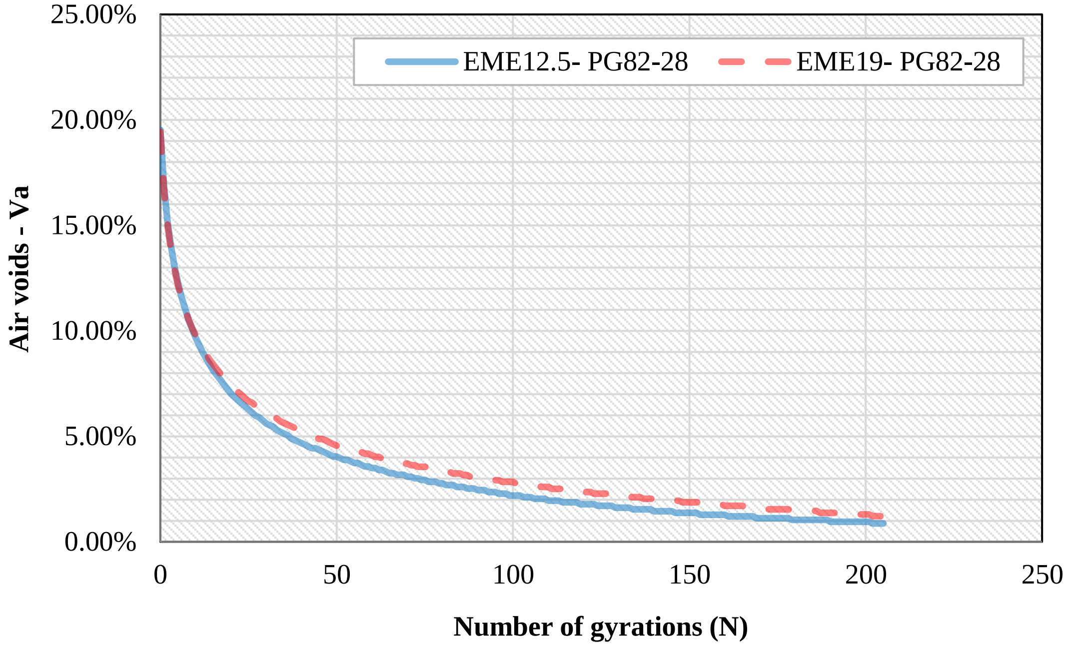


Figure 6 - 4: Compactibility of EME mixes with second aggregate source

Table 6 - 1: Volumetrics of the developed mixes (second aggregate source)

Properties	**EME 12.5**	**EME 19**
	PG 82-28	**PG 82-28**
B	4.9	4.5
G_{mm}	2.530	2.550
BRD @ N_{max}	2.510	2.519
VMA (%)	12.4	12.21
VFA (%)	86.29	81.16
V_a @ N_{ini} (%)	10.1	10.1
V_a @ N_{des} (%)	1.7	2.3
V_a @ N_{max} (%)	1.0	1.2
P_{be}	4.4	4.1

B: Binder content; G_{mm}: Maximum relative density of loose asphalt mix; BRD: Bulk relative density of compacted mix; VMA: Voids in mineral aggregate; VFA: Voids filled with asphalt; V_a: Air voids; N_{ini}: Initial number of gyrations: 9 gyr; N_{des}: Design number of gyrations: 125 gyr; N_{max}: Maximum number of gyrations: 205 gyr; P_{be}: Effective binder content.

6.4.3 Rutting Test Results

The rutting test results are depicted in Figure 6 - 5. It is clear from the results that both mixes have superior rutting resistance where the rut-depth of both mixes were less than 1 mm after 20,000 of wheel passes (almost no rut). EME 12.5 had slightly lower rut-depth than EME 19 although it had higher binder content. This represents the importance of aggregate structure on the rutting performance of the mix.

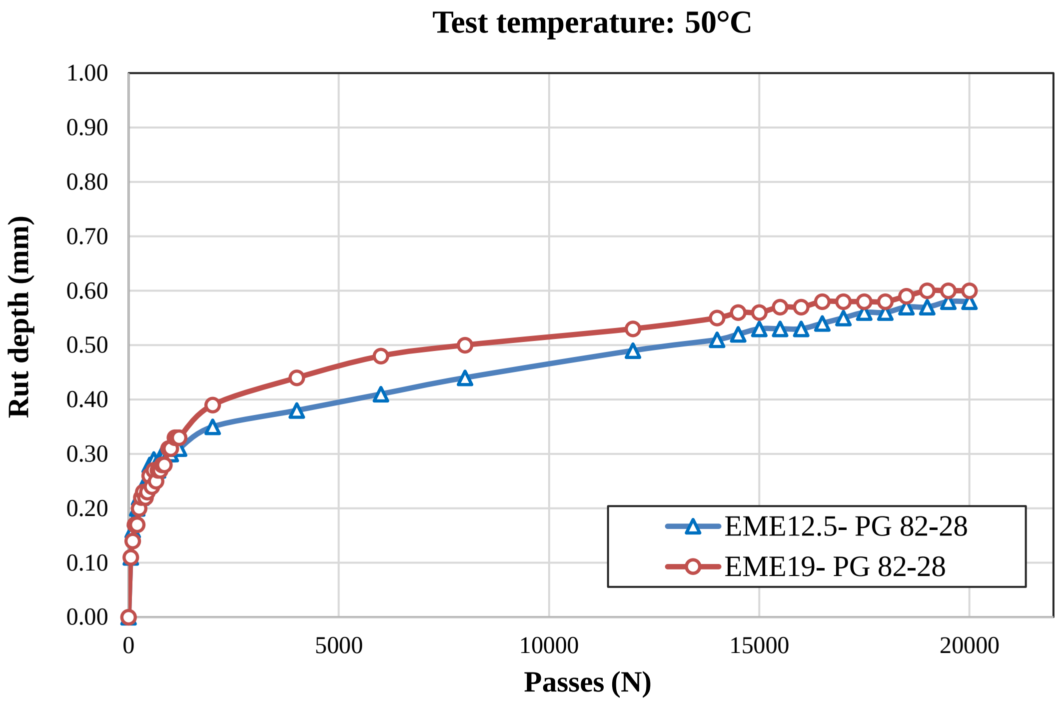


Figure 6 - 5: HWTT results (second aggregate source)

6.4.4 Fatigue Test Results

Fatigue test was performed at three different strain levels, and the results are shown in Figure 6 - 6. As illustrated in this figure, EME 12.5 had higher fatigue life at lower strain (displacement) level compared to EME 19. Although, the curve was steeper for EME 12.5, and, consequently, the fatigue lives of this mix were shorter at higher strain level. Both mixes had high ε_6 values (more than 550 μm/m).

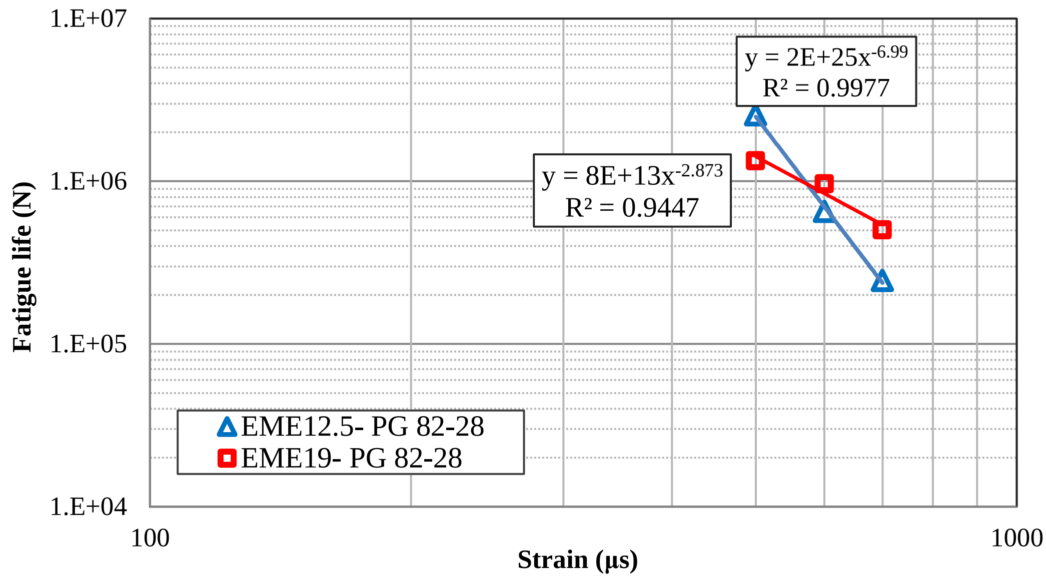


Figure 6 - 6: Fatigue curves for EME mixes (second aggregate source)

6.4.5 TSRST Results

Figure 6 - 7 illustrates the TSRST cracking temperature of EME mixes. According to the results, both mixes had acceptable low-temperature cracking resistance when the cracking temperature of both mixes were lower than -30°C. Although EME 19 had lower binder content than EME 12.5, its cracking temperature was slightly lower (-34.15°C).

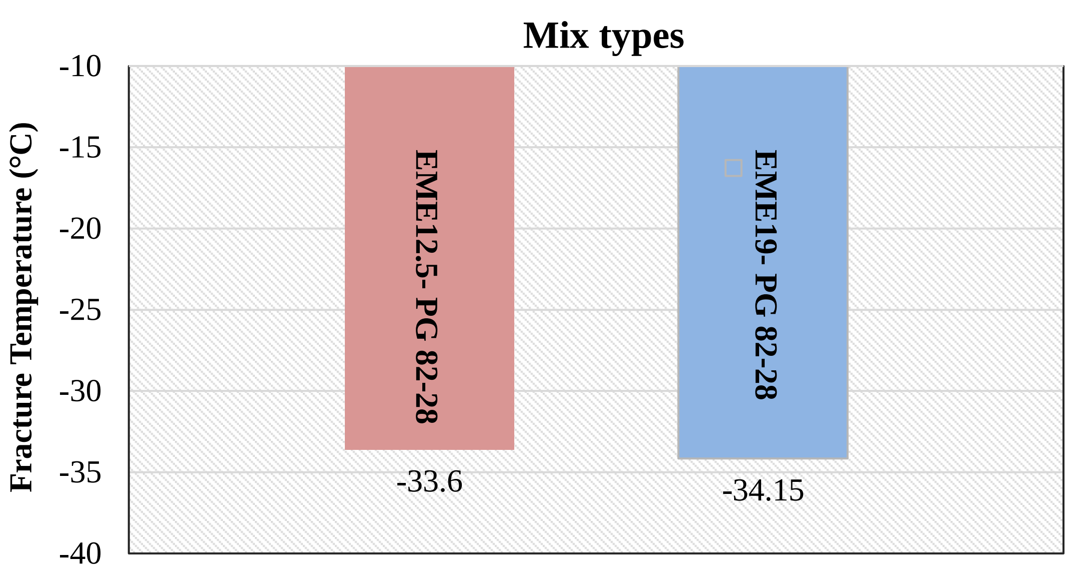


Figure 6 - 7: Thermal stress restrained specimen test fracture temperatures

6.5 Conclusions

This chapter investigates the effects of second aggregate source on EME mix design. The granite aggregates were provided from Bark Lake Quarry, Ontario. Two types of EME were fabricated, EME 12.5 and EME 19. Compactibility and volumetric properties of the mixes were assessed. Thermo-mechanical tests were conducted to evaluate the mixes in terms of modulus, rutting, low-temperature properties and fatigue. The mixes had high compatibility and the mixes had around 2% air voids at N_{des}. Summary of thermo-mechanical testing results is also provided in Table 6 - 2. As can be concluded from this table, in overall, the mixes performed well.

Table 6 - 2: Performance testing results of the mixes (second aggregate source)

EME	**Binder Type**	**|E*| @ 10 Hz (MPa)**	**Rut-depth 20,000 passes (mm)**	**Fatigue (10˚C; 10 Hz) ε_6 (μm/m)**	**TSRST cracking temperature (˚C)**
		10˚C			
12.5	PG 82-28	14,173	0.58	577.1	-33.60
19	PG 82-28	13,876	0.60	563.4	-34.15

CHAPTER 7
SUMMARY, CONCLUSIONS AND FUTURE WORKS

7.1 Overall Summary

Along with a background and literature study on "Enrobé à Module Élevé" (EME) application and mix design, a new approach was developed for EME mix design that could be adopted and used for Ontario's highways. The methodology proposed in this study was based on performance-based mix design approach. Compressible Packing Model (CPM) was utilized to optimize the packing densities of blended aggregates for EME 12.5 and EME 19. Three types of modified binders were used namely: PG 88-28, PG 82-28 and PG 58-28 modified with Elastomer additives. Different laboratory testing methods were designated to assess performances of the developed mixes. Apart from gyratory compactibility test, series of thermo-mechanical testing were conducted to evaluate the mix performance at different levels. Hamburg wheel track rutting test was used to assess the performance of developed mixes in terms of permanent deformation. Fatigue resistance of the mixes were evaluated at four different strain levels using four-point bending beam fatigue test. TSRST was performed to investigate the low-temperature cracking resistance of the mixes. In addition, viscoelastic properties of the used binders as well as EME mixes were analysed using 2S2P1D rheological models and correlation between the rheological properties of the mixes and binders' morphologies were investigated using ESEM testing. The following paragraphs describe the findings of each chapter.

Chapter 3: Different classes of aggregate blends were used for optimization of aggregate blends for EME 19 and EME 12.5. OCCHIO belt aggregates image analyser was utilized to determine the morphological parameters of fine and midsize aggregates. The obtained results were then compared to the maximum density curve. The obtained results showed that there was a considerable difference between EME 12.5 gradation curve and theoretical maximum density curve although there were very close at some points. This could be attributed to the other affecting factors such as aggregate shape parameters or limitation in aggregate size distribution. The results also showed that there was a better correlation between the CPM-obtained gradation for EME 19 and theoretical maximum density curve. In addition, the gradation curves of both mixes were within the limits recommended by European specification for EME mixes. Compactibility test results of the mixes showed that both mix types had higher compactibility than the Superpave conventional asphalt mix; and that EME 19 was more compactible than EME 12.5. It was also observed that EME 19 with the same richness factor as EME 12.5 had relatively lower stiffness. As a result, to get higher stiffness in EME 19, lower richness factor was adopted for this mix (K=3.0).

Chapter 4: According to the complex modulus test results, the developed mixes had relatively high modulus values. Mixes with PG 88-28 binder showed the highest modulus values compared to the other binder types. Hamburg wheel tack rutting test results showed that all of the mixes had superior rutting resistance when the permanent deformation (rut-depth) of the mixes were less than 1 mm after 20,000 passes of the wheel on the submerged specimens. TSRST results depicted that the cracking temperatures of the mixes were lower than -25˚C. EME 12.5 had slightly better low-temperature cracking resistance than EME 19. Additionally, for each mix type, cracking resistance of the mixes with PG 82-28 and PG 88-28 modified binders were almost 2˚C lower than those with elastomer additives. It was also observed from the fatigue test results that EME 12.5 had longer fatigue lives than EME 19 which could be due to the fact that EME 12.5 had higher binder content than EME 19. The best fatigue performance belonged to the mixes with PG 82-28 binder. It was also noted that all of the mixes had ε_6 values more than 300 μm/m which is higher than the minimum requirement for EME mixes Class 2 ($\varepsilon_6 \geq 130$ μm/m).

Chapter 5: ESEM test results of the modified binders showed that among the binders, modified PG 88-28 was the densest and was connected with thicker fibrils, and that microstructure of PG 58-28 + 10 % Elastomer additives consisted of thinner fibrils with highly intertwined structure. PG 88-28 binder had the highest shear modulus ($|G^*|$) at a wide range of frequency although at a high frequency PG 58-25 + Elastomer additives represented a comparable stiffness. Phase angle (φ) of PG 88-28 was the lowest and behaved differently compared to the other binder types. Similarly, mixes fabricated with PG 88-28 binder represented lower phase angles which could contribute to the improved elastic behavior among the mixes. Correlation existed between the binder's microstructure and phase angles of the mixes. It was also observed that 2S2P1D rheological model could successfully model the rheological properties of the modified asphalt binders as well as the mixes. According to the test results, mixes fabricated with structurally denser binder and stronger bonds tended to have lower phase angles. It could be also observed that using binders with more intertwined structural network could contribute to higher modulus in the mix particularly at higher frequencies. Additionally, dissipated energy in the mixes fabricated with PG 58-28 + 10% Elastomer additives was the highest compared to the other mixes which was due to higher viscous component (E_2) of the modulus in these mixes. Among the mix types, except for the mix with PG 82-28, EME 19 represented higher elastic behavior than EME 12.5. Phase angles were not influenced by aggregate gradation except for EME 19 with PG 88-28 binder which showed to have improved viscous behavior than EME 12.5 with the same binder type.

Chapter 6: The effects of another source of aggregates were investigated on the EME mix design. Based on the results presented in this chapter, the mixes had shown higher compactibility than conventional Superpave mixes when the air void contents of the mixes were around 2% at the design number of gyrations.

The mixes performed well in terms of rutting and the rut-depth of the mixes were less than 1 mm for both mixes. The fatigue test results showed that the mixes had relatively high fatigue resistance (ε_6>550 µm/m). In addition, the fatigue curve for EME 12.5 was steeper compared to EME 19 which resulted lower fatigue life at higher levels of strain. Low-temperature performance of the mixes were assessed, and according to the obtained result both mixes had cracking temperatures below -30°C.

7.2 Recommendations and Future Research Directions

In Chapter 3, Compressible Packing Model (CPM) was adopted as a new technique for optimization of packing density of blended aggregates in high performance asphalt mix. In this research, five classes of aggregates were used for the optimization process. It will be of interest if CPM optimization will be performed on a larger number of aggregates fractions to investigate the influence of aggregate size distribution in the packing optimization. Further, in order to have a deeper understanding on aggregate packing optimization, a Computed Tomography (CT) – Scanner can be utilized to see the impact of CPM optimization on the structure of coarse aggregates, voids distribution and mastic volume in the mix. The obtained information can also be cross-linked with the thermo-mechanical testing results.

In Chapter 4, series of laboratory testing procedures were used to evaluate the thermo-mechanical properties of the developed mixes. However, these testing procedures were not the same as the French methods traditionally used for development of EME mixes. Therefore, it will be useful to evaluate the performances of the developed mixes using French testing procedures and to see at what level the mixes would perform if French test methods are used.

Since EME mix designs are quite new to Ontario, it was recommended that a test section be constructed to determine the feasibility of producing the EME mixes. Constructing a test section in a heavily trafficked area will provide an opportunity to evaluate the impact of conventional production and construction practices or any changes that may be required for EME mixes. It will also allow the performance of the mixes to be evaluated under field loading conditions to further assess the potential for EME mixes on future highway applications.

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Appendix A: Specimen Preparation and Testing

(a)

(b)

(c)

Figure A - 1: (a) Aggregate batch; (b) Mixing aggregates and asphalt binder; (c) Quartering

(a)

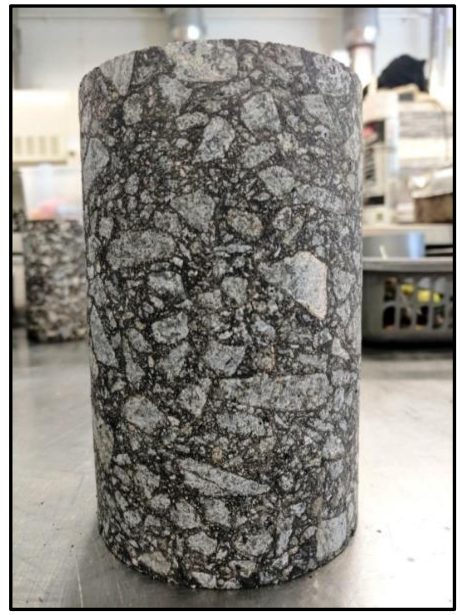
(b)

(c)

(d)

Figure A - 2: (a) Superpave gyratory compactor (SGC); (b) Asphalt vibratory compactor (AVC); (c) SGC compacted samples; (d) AVC compacted slab

(a)



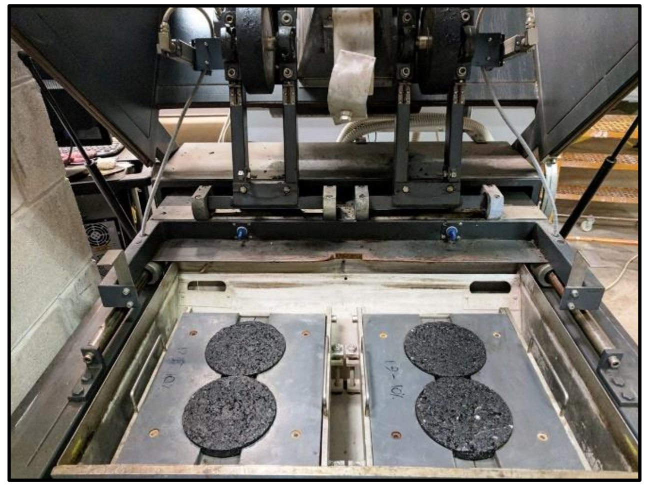
(b)

(c)

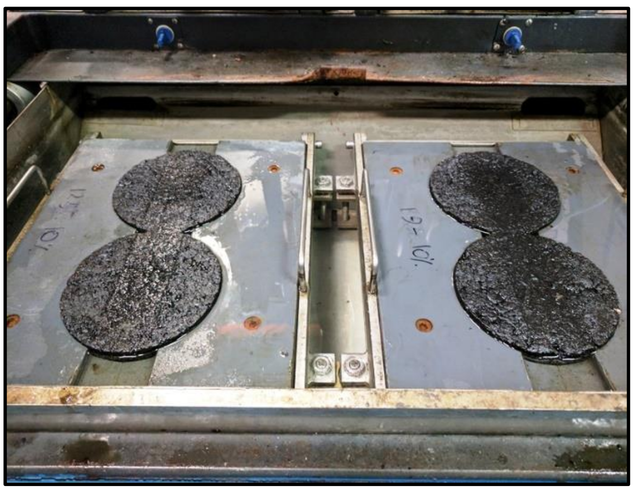
(d)

Figure A - 3: (a) Sample coring; (b) Cylindrical specimen after coring and cutting; (c) Dynamic modulus sample preparation; (d) Dynamic modulus test set up

(a)



(b)



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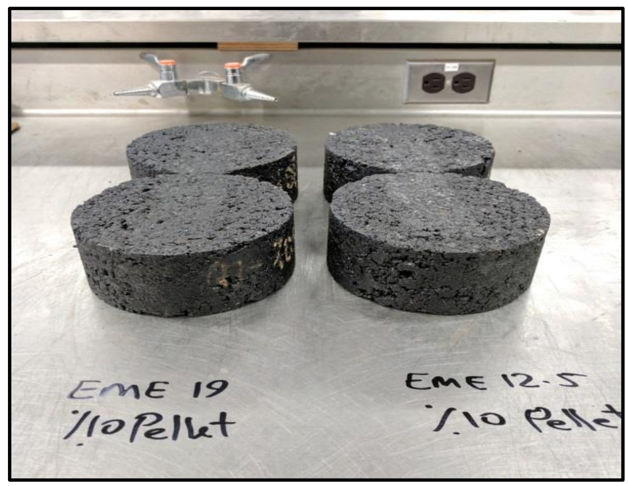


(d)

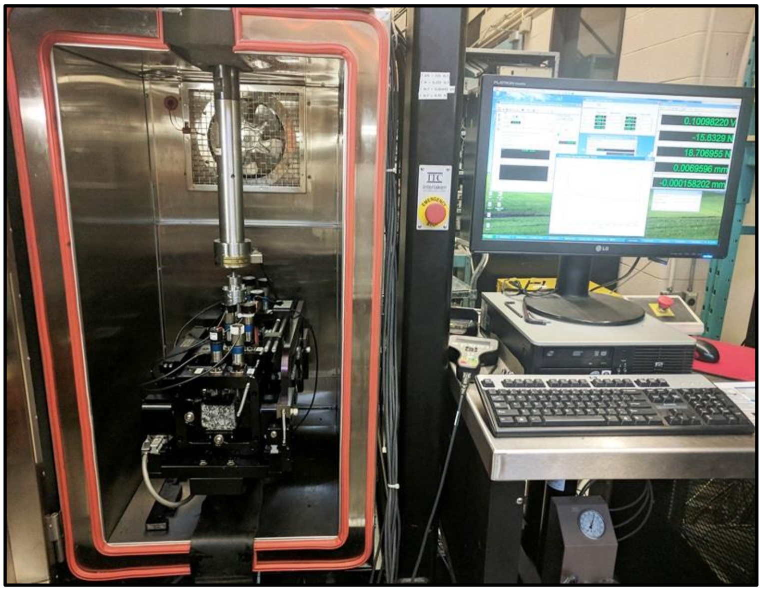
Figure A - 4: (a) HWTT rutting specimens; (b) HWTT rutting test set up; (c) HWTT tested specimens (wet); (d) HWTT tested specimens (dry)

(a)

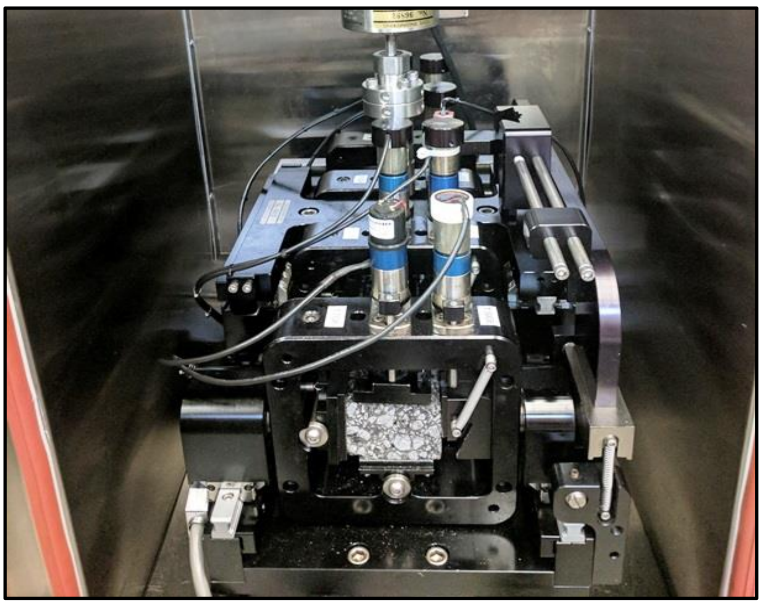
(b)

(c)

Figure A - 5: (a) Saw cutting; (b) Fatigue beams; (c) TSRST beams



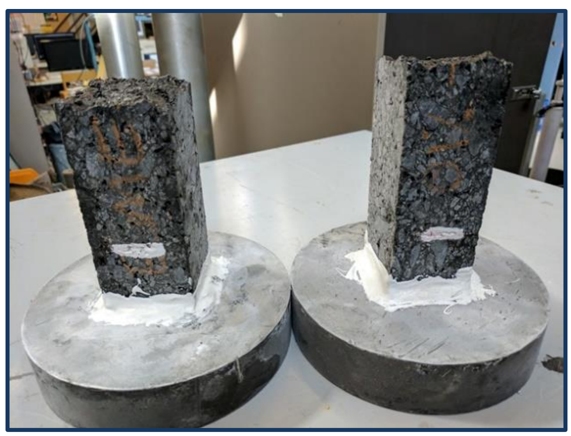
(a)



(b)

Figure A - 6: (a) Fatigue apparatus; (b) 4PB fatigue frame

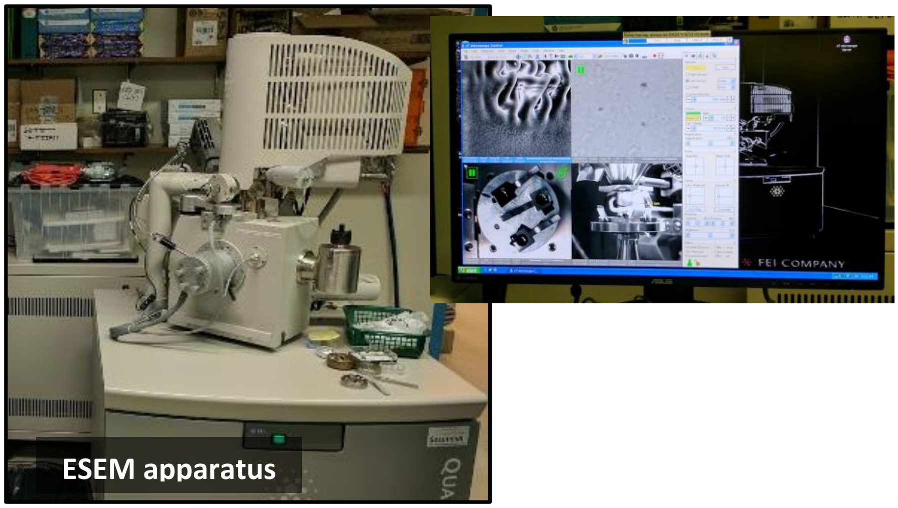
(a)



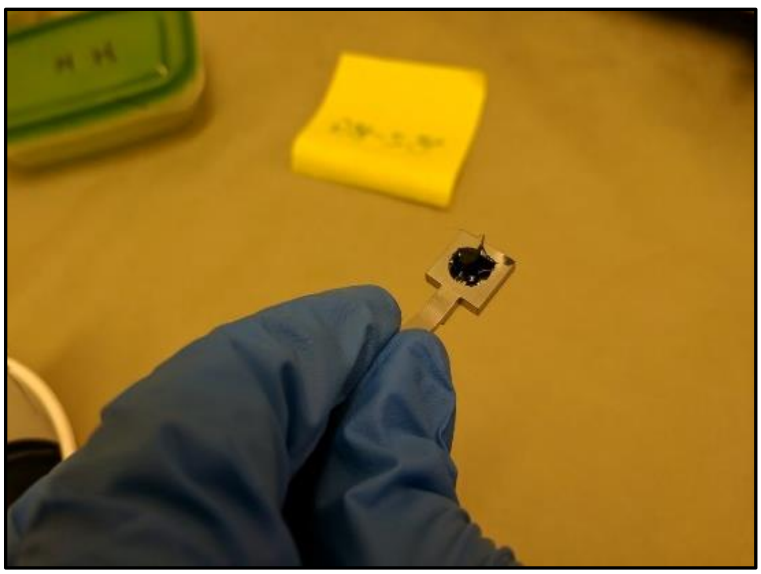
(b)

Figure A - 7: (a) TSRST test set up; (b) TSRST specimen after failure

Figure A - 8: Dynamic shear rheometer (DSR) equipment



(a)



(b)

Figure A - 9: (a) ESEM apparatus; (b) ESEM sample

(a)

(b)

(c)

Figure A - 10: (a) Elastomer additives (pellets); (b) Adding Elastomers to the binder; (c) Mixing Elastomers with the binder using high shear mixer