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Evaluation of Structural Contribution of Asphalt Mixtures Through Improved Performance Indices

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Evaluation of Structural Contribution of Asphalt Mixtures through Improved Performance Parameters

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DEDICATION

To Moloud, my lovely wife, for all of her support, patience and kindness during this journey and to Liana, my daughter who is going to bring a lot of joy and happiness to our life when she is born.

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TABLE OF CONTENTS

List of Figures

List of Tables

ABSTRACT

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A variety of approaches are available to design the pavement structures. These approaches are generally divided into two main categories as empirical and mechanistic-empirical (M-E) methods. The most widely used empirical method is the AASHTO 1993 design approach which uses material specific coefficients (layer coefficients) to quantify the structural capacity provided by each pavement layer. These coefficients are experimentally developed values from the AASHO road test which was conducted in early 1960s and are based on statistical regressions. Almost no fundamental or engineering mixture properties or explicit failure criterion were used in their original development. On the other hand, the M-E approaches use fundamental mixture properties such as complex modulus (E* and phase angle) to determine the pavement's structural response. However, M-E approaches require extensive data for local calibration and as a results many state agencies are still using the empirical approach.

One of the major modifications in the AASHTO 1993 design approach has been to update the layer coefficients (a-value) of the asphalt mixtures using different mechanistic and performance based measures. The layer coefficients have significant influence in determining the layer thickness which translates into the structural contribution of the layers as well as the long term performance of the pavement and consequently the construction and maintenance costs. Therefore, it is critical to determine reliable a-values that are most relevant to the regional conditions and locally used materials.

A set of 18 commonly used mixtures in New Hampshire were selected for performance testing and evaluation of structural contribution in terms of layer coefficients. In order to develop the layer coefficients, comprehensive research was performed on the performance and properties of the mixtures through different mechanistic based laboratory testing methods. In addition, mixtures from all over the New England region were used to develop and validate three novel performance index parameters for rutting, fatigue and transverse cracking. The developed parameters were incorporated with the field distress data such as International Roughness Index (IRI) in order to develop mechanistically informed layer coefficients for New Hampshire flexible pavement design approach.

1. Introduction

1.1 Background and Motivation

Asphalt mixture as the top most layer in the pavement structure is prone to different types of structural distresses such as rutting, fatigue and thermal cracking. Depending on the loading and climatic conditions, the asphalt layer thickness in flexible pavements can vary from 5 cm to over 30 cm. To enhance reliability of pavement designs and to ensure high return on investment for agencies, it is paramount to use performance driven design philosophies. This dissertation explores use of laboratory performance testing informed asphalt mixture structural coefficients to improve reliability of AASHTO 1993 empirical pavement design system.

It is well known that the type and magnitude of structural responses (deformations, stresses and strains) vary throughout the pavement so there is need for different types of asphalt mixtures, each designed to handle specific types and magnitude of responses within the structure. This not only increases the design reliability but can also result in considerable savings in financial resources by optimizing material properties in each layer.

Within the pavement structure, and depending on the design thickness, the asphalt course is generally divided into three sublayers namely the base, intermediate (binder) and surface (wearing) layers. The wearing course is usually made of smaller aggregate size and higher binder content to prevent both functional and structural distresses. The intermediate and base courses contain relatively coarser aggregates and lower binder content. The intermediate layer is placed directly under the wearing course to facilitate the construction of the wearing course and to distribute the traffic loads onto a larger area. This layer increases the overall pavement structural capacity and helps prevent the wearing course from different types of premature distresses. The asphalt base layer is very similar to the intermediate course in terms of the performance expectations. Base layers are used in addition to the intermediate layer in cases where the load magnitudes and repetitions call for a relatively thicker pavement. In this case the base layer provides a strong foundation for the overlaying lifts to prevent or reduce the risk of rutting and fatigue related distresses.

A variety of approaches are available to design the pavement structures. These approaches are generally divided into two main categories as empirical and mechanistic-empirical (M-E) methods. The most widely used empirical method is the AASHTO 1993 design approach which uses material specific coefficients (layer coefficients) to quantify the structural capacity provided by each pavement layer. These coefficients are experimentally developed values from the AASHO road test which was conducted in early 1960s and are based on statistical regressions. Almost no fundamental or engineering mixture properties or explicit failure criterion were used in their original development. On the other hand, the M-E approaches use fundamental mixture properties such as complex modulus (E* and phase angle) to determine the pavement's structural response. Using different transfer functions, the pavement response is used to determine the damage accumulation over the pavement design life and the pavement failure is determined when a predefined distress threshold is reached. Although the M-E methods have the superiority of using the fundamental mix properties, the calibration of transfer functions for different climatic and loading conditions requires substantial amounts of field and laboratory data for different types of mixtures. In many instances, these data may not be readily available and could require extensive amount of time and financial investments to generate them. As a result, many state highway agencies still prefer to use the empirical design approaches with some enhancements in the design equations to address their specific traffic and climatic circumstances. One of the major modifications in the AASHTO 1993 design approach has been to update the layer coefficients (a-value) of the asphalt mixtures using different mechanistic and performance based measures. The layer coefficients have significant influence in determining the layer thickness which translates into the structural contribution of the layers as well as the long term performance of the pavement and consequently the construction and maintenance costs. Therefore, it is critical to determine reliable a-values that are most relevant to the regional conditions and locally used materials.

Motivated by need for development of performance incorporated layer coefficients for asphalt mixtures used in New Hampshire, the primary objectives of this research are set up as:

1. Evaluation of the effect of mixture type and nominal design properties on various lab measured pavement performance indices.
2. Assessment of the current lab based performance indices in their suitability for use with New Hampshire flexible pavements. If needed, develop improved indices to better distinguish the mixtures performances.
3. Development of a methodology to update the layer coefficients for AASHTO 1993 design approach on the basis of laboratory performance tests and field performance data.

In order to accomplish the objectives of this dissertation, a number of research efforts have been undertaken and several of these have culminated into peer-review journal articles. This doctorate dissertation is organized in 11 chapters. A short summary of each chapter is provided in this section and full manuscripts are either attached as appendices to this document or have been discussed within the thesis under designated chapter number and title.

Chapter 1 is dedicated to the introduction and motivation for this research, as well as the study objectives. Chapter 2, includes the literature review on the topic of AASHTO 1993 pavement design approach and the origination of layer coefficients along with different methods in the literature that have been used to update the layer coefficients. Chapter 3 discusses the breadth of materials that have been examined in this study, followed by a description of mechanistic and performance-based laboratory tests that have been used to characterize the study mixtures. This chapter also includes an overview of the research approach that has been used in this dissertation to fulfill the study objectives. Chapters 4 evaluates the effect of asphalt mixture constituents on the predicted performance through use of various statistical approaches. Also, the correlation of different performance index parameters is investigated to determine the indices that can be used interchangeably. As one of the important influential parameters that can significantly affect the mixtures' mechanical properties and performance, the production methods (hot versus cold mixtures) have been investigated in the presented research, this is discussed in Chapter 5. Generation of large data-sets from current and previous laboratory testing campaigns, led to development of nominal property based predictive models for asphalt mixtures' complex modulus (E* and Phase angle), this effort is described in chapter 6. Since many state highway agencies such as New Hampshire Department of Transportation do not employ mechanical testing to evaluate rutting susceptibility of mixtures, a novel complex modulus based rutting index parameter is developed and validated in chapter 7. The time dependency of the asphalt mixture fracture process is evaluated and incorporated with the use of Illinois semi-circular bend test in chapter 8. These investigations resulted in development of a rate dependent cracking index (RDCI) parameter that is able to lower the coefficient of variation of the test results and can reliably differentiate different aging levels with a higher reliability as compared to the flexibility index. Chapter 9 evaluates different fatigue failure criterion derived from the simplified viscoelastic continuum damage (S-VECD) theory and direct tension cyclic fatigue testing methods. The evaluations indicated that neither of the existing criterion are able to reliably rank the field fatigue cracking performance. A damage growth rate based fatigue failure criterion is proposed in this chapter which is shown to reliably rank the field fatigue performance. Chapter 10 combines the findings from previous chapters where the layer coefficients are back-calculated from incorporation of the rutting, fatigue and transverse cracking index parameters with the field measured International Roughness Index (IRI). In order to back-calculate the layer coefficients, the Present Serviceability Index (PSI) is determined from the field measured IRI, which leads to determination of the structural number and consequently the IRI based layer coefficients. Using the statistical analysis the lab measured performance index parameters are combined with the IRI based layer coefficients and result in the performance incorporated layer coefficients which indicate the structural contribution of the mixtures in the pavement design. Chapter 11 summarizes the findings of the research and contribution of the thesis to the asphalt mixture and pavement design while pointing out the future extension of the research. Details of research efforts and corresponding results and discussion for Chapters 4 thru 8 of this dissertation are in form of peer-reviewed journal manuscripts. Full manuscripts are attached as appendices to this thesis, a brief synopsis of the work from these manuscripts are provided as body of these chapters.

2. Literature Review

2.1 AASHTO 1993 Design Guide

Currently, New Hampshire Department of Transportation (NHDOT) uses the AASHTO 1993 guide to design the structure of pavements. This design guide is based on the American Association of State Highway Officials (AASHO) road test that was performed in the late 1950s and early 1960s in Ottawa, Illinois.

The primary goal in this test was to come up with a relationship between the number of axle load repetitions and the pavement performance during the pavement service life. The AASHO test was conducted on six different loops and the loading started in October 1958 and ended in November 1960 [1]. The main variables in this road test were the hot mix asphalt, base and subbase thicknesses as well as the different axle configurations that were applied on different test loops. The observations and data obtained from this test were converted into different parts of a design equation which relates the number of applied axle loads to the required thickness of the pavement. AASHO road test resulted in major findings in the pavement engineering science such as the relationship between the load and the damage called as Fourth Power Law. It also introduced important factors such as Serviceability, Equivalent Single axle load (ESAL) and the Structural Number (SN). The first AASHO design guide was published in 1961 as the "AASHO Interim Guide for the Design of Rigid and Flexible Pavements". Since the equations derived from AASHO test were based on limited data from two years of loading and only one climatic condition (Ottawa, Illinois), the design guide was significantly updated in 1972 and 1993 to meet different nationwide requirements and climatic conditions. The latter update is called as the AASHTO 1993 design guide and has not changed since then. Although major steps have been taken to switch from this empirical guide to a mechanistic-empirical pavement design guide (MEPDG) in the past three decades, the high costs and lack of available database for regionally calibrating such design guide has become an issue for many of the state DOTs. Hence, AASHTO 1993 design equation (Equation.1) is still being used as a reliable pavement structure design tool in many states in US as well as many other countries around the world [2].

$$log_{w18} = Z_R S_0 + 9.36\, log(SN+1) - 0.2 + \frac{log\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.4+\frac{1094}{(SN+1)^{5.19}}} + 2.32 log M_r - 8.07 \qquad \text{Equation (1)}$$

Where:

log_{w18}: Logarithm of number of the allowable equivalent single axle loads (8.2kN) in design period

Z_R: Z-statistic, determined based on reliability level of the design

S_0: Standard deviation of the design

ΔPSI: Allowable serviceability loss at end of design life

M_r: Resilient Modulus of subgrade soil

2.2 AASHTO 1993 Design Factors

2.2.1 Equivalent Single Axle Load (ESAL)

AASHTO 1993 design procedure for determining the thickness of different layers is based on the total number of applied wheel load over the design life of the pavement. Since the axle loads and configurations vary among different vehicle types, the damage induced by them would be different. One of the important achievements by the AASHO road test was the concept of Equivalent Axle Load Factor (EALF) which basically converts the amount of induced damage from any type of vehicle to an equivalent damage caused by an 18kip (80kN) single axle load. Then the summation of equivalent damage over the pavement design life is considered as the Equivalent Single Axle Load (ESAL) which is the only traffic factor in the design [1].

2.2.2 Reliability

This parameter is defined as the probability that the design will perform its intended function over the pavement design life and changes based on the type and importance of the road. Reliability is indeed the factor of safety of the pavement design that is implemented in the AASHTO 1993 design guide. In other words, reliability of the design is used to ensure that the actual ESALs over the design life will not exceed the estimated ESALs. For instance, a 50% reliability means that the actual ESALs will be equal to the estimated ESALs at the end of the design period.

2.2.3 Present Serviceability Index (PSI)

The serviceability of a pavement is essentially evaluated by the ride quality experienced with the road users. The serviceability index is ranked from 5 to 1 as the best and worst ride quality respectively. This factor is mainly used as a tool to determine the proper time for the correct type of maintenance, rehabilitation or even reconstruction of the pavement by the pavement management system (PMS). The initial serviceability of the pavement is a function of pavement type and construction quality and the typical value for the flexible pavements is considered as 4.2 whereas the adopted value for the terminal serviceability for this type of pavement is typically 2 for new designs.

2.2.4 Structural Number (SN)

Structural number of pavement is defined as a criteria to measure the ability of pavement to withstand the applied load. The primary purpose of any pavement design is to protect the subgrade soil from the stresses due to the loading, as well as penetration of surface water into the subgrade soil which can significantly decrease its modulus and result in different types of damages. Therefore, the structural number of a pavement is a function of type, thickness, and drainage capability of different materials used in the pavement structure. The weaker the subgrade soil the higher the required structural number will be for the same loading and climatic conditions. Equation. 2 defines the structural number for a pavement structure with “n” layers.

$$SN = \sum_{i=1}^{n} a_i \times D_i \times m_i \qquad \text{Equation (2)}$$

Where:

SN: Structural number
a_i: Layer coefficient of the i^{th} layer
D_i: Thickness of the i^{th} layer
m_i: Drainage coefficient of the i^{th} layer

2.2.5 Soil Resilient Modulus (M_r)

The resilient modulus of the soil is an important factor in AASHTO 1993 design guide. The resilient modulus of the subgrade soil is subjected to significant changes due to weather conditions during the year. Therefore, the effective resilient modulus which is the representative modulus value for different weather conditions is calculated based on the damage that could occur to the pavement during different seasons with different subgrade soil modulus. This value is the only subgrade soil property that is considered in AASHTO 1993 and because of that is highly influential in determining the structural number (SN) of the overall design.

2.3 Layer Coefficient

As it was mentioned earlier one of the important factors that contributes to the Structural Number is the type of material that is covered in the form of the layer coefficient (a-value). As the definition provided by AASHTO design guide, layer Coefficient is the empirical relationship between the structural number of a pavement structure and layer thickness, which indicates the relative ability of a material to function as a structural component of a pavement [2]. Layer coefficients (a-value) were originally derived as the regression coefficients in relating the SN to the thickness of different layers in AASHTO road test. In other words, for a given pavement structure the SN value was first determined through using equation 1 and then based on the configuration of the pavement the calculated SN value was correlated to different layer thicknesses through equation 2 and finally the regression coefficients (a-values) were determined. The main factors affecting the a-value are:

- Material type and properties
- Layer thickness and location
- Failure criterion
- Loading level

In terms of asphalt material, the layer coefficient is not only based on the asphalt material properties and thickness but it is highly affected by the underlying material's properties.

2.4 Layer Coefficient Calibration

Research conducted in National Center for Asphalt Technology (NCAT) revealed that the layer coefficient followed by traffic level and resilient modulus are the most influential factors in determination of the pavement thickness considering the AASHTO 1993 design equation[3]. Although the original layer coefficients from the AASHTO road test are reliable, they are applicable only to the types of material, traffic and the environmental conditions under which they have been generated. Since layer coefficient has a significant influence in determining the layer thickness and consequently on the construction expenses as well as the long term performance of the pavement, it is essential to determine the calibrated and reliable a-values for different regions and materials. For this purpose, many of the states that implement AASHTO 1993 design procedure or use layer coefficient as part of their specific design methodology have tried to evaluate their own commonly used materials and assign new a-values to them. Old studies which are mainly experimental based, have shown that a-values are correlated with gradation, thickness, abrasion of aggregate and more important the strength or stability of asphalt mixtures [4]. Layer coefficient recalibration is conducted based on different methodologies which will be discussed briefly in this subsection.

2.4.1 Pavement Structural Response

Some mechanistic based studies have tried to use the concept of equivalent deflection by assigning a reference mixture with a defined thickness and compare other mixtures to that by determining the required thickness to result in the identical deflection to that of the reference mixture under same loading magnitude. Similarly, the identical maximum vertical stress on top of subgrade soil for different types of hot mix asphalt mixtures has widely been used to recalculate the a-values. Maximum tensile strain at the bottom of asphalt layer has also been used to determine the layer coefficient of recycled mixes. The thickness of the recycled layer to result in the equivalent number of load repetitions to failure (N_f) of the standard reference hot mix asphalt on the same subgrade soil is used to determine the layer coefficient since the SN is equal for both cases [4].

2.4.2 Pavement Performance

AASHTO pavement performance analysis has also been used as another practical method for layer coefficient calibration. This method monitors the serviceability indicators (rut depth, cracks, patching, IRI and etc.) and calculates the PSI. The rate of change in serviceability for a given pavement structure with known thicknesses for different layers is then converted to SN value. Running the regression analysis on the SN (Equation 2) results in the new layer coefficients. This method has been successfully used by NCAT to recalibrate the a-value of the asphalt layer used by Alabama Department of Transportation (ALDOT). Using the IRI value and converting that to PSI for the known cross sections, researchers in NCAT suggested a new layer coefficient of 0.54 instead of o.44 for the hot mixed asphalt which can reduce the construction costs by approximately 18% [3, 5].

2.4.3 Mechanistic-Empirical Design Approach

A more sophisticated way to calibrate the a-value is to use the mechanistic-empirical method (MEPDG). This method which has been used by Washington State is highly data intensive and is recommended to be used by the agencies that are in the process of transforming from the empirical to the mechanistic-empirical design approach and have enough database available for the calibration. Once the database is available, the calibration can be simply done by designing the required thickness through MEPDG approach and then calibrate the a-value in the AASHTO design method to obtain the same thickness for the structure. Using this method by WSDOT the a-value of hot mixed asphalt increased from 0.44 to 0.50 which significantly reduces the construction costs [5].

2.4.4 Material Properties Characterization

Among all the factors that influence the layer coefficients, the material type and properties have the highest impact, and to account for these factors, AASHTO 1993 design guide proposes the resilient modulus (M_r) of the material [2] since it is not only a measure of stiffness but also can be an indicator of strength of the material.

The relationship between the asphalt mixture’s layer coefficient and the elastic resilient modulus at 21°C was established in 1972. This relationship (Equation 3) which is shown in Figure 1 is valid for a dense graded asphalt mixture and can only be used if the elastic modulus is between 750 MPa and 3100 MPa. AASHTO 1993 design guide proposes the value of 0.44 as the layer coefficient for M_r corresponding to 3100 MPa [1].

$a_i = 0.4 \log(M_r) - 0.951$ Equation (3)

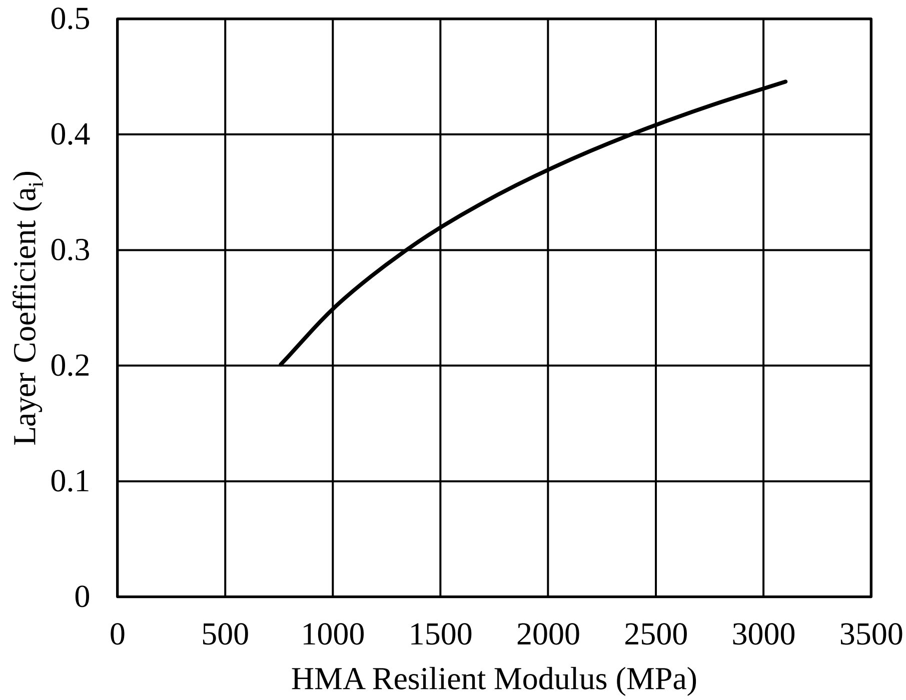


Figure 1. Estimating layer coefficient of dense-graded asphalt concrete based on resilient modulus.

Research conducted at University of Wisconsin for recalibrating the a-values of commonly asphalt mixes used by Wisconsin Department of Transportation (WisDOT) implemented the M_r measurement in lab. The test was performed in accordance to the AASHTO T-294-94 standard at 20 °C. Using equation 3 new layer coefficients were derived. The main concern stated by researchers was that the resilient modulus measurements for different types of mixtures at the aforementioned temperature were so close and as a result the a-values were turned out to be nearly the same. As a solution and for better differentiating the mixtures, a triaxial testing apparatus was used to measure the rutting at 52 °C and 64 °C. The researchers proposed the correlation of the a-value with the combination of resilient modulus, rutting performance and any other available damage factor to calculate the new a-values [6].

Among many state agency DOTs that use empirical design methods, South Carolina is using the AASHTO 1972 design guide and is trying to switch into the mechanistic-empirical (MEPDG) design method. Research was performed to enhance the precision of the a-values used for the asphalt base mixtures in South Carolina. The procedure included running the dynamic modulus test on the mixtures and prediction of the M_r value from the (E*) master curve at the frequency of 1.59 Hz at 20°C which is indeed equal to 0.1 second of loading on the specimen (same loading time for M_r test in accordance to ASTM D7369). Once the M_r was predicted, equation 3 was utilized to calculate the new a-values that were increased from the initial value of 0.34 to above 0.44 [7].

2.4.5 Falling Weight Deflectometer

According to the AASHTO 1993 design guide a more reliable way to determine the layer coefficients is to back calculate the moduli from Falling Weight Deflectometer (FWD) test on the road in lieu of lab testing since there might be a variation between the lab made samples and the mixture placed in the field [2] and also FWD is considered as a way to simulate the dynamic loading of a moving wheel in a wide range of loading level which is a more realistic way of loading.

Perhaps, New Hampshire Department of Transportation (NHDOT) has been one of the leading State DOTs in the nation to use FWD for recalibration of layer coefficient values for pavement materials. Research conducted in 1994 by Janoo on a segment of I-93 between exit 18 and 19 through construction of ten test sections with different combination of materials for the same structural number. The primary purpose of testing was to evaluate the a-value used for the Reclaimed Stabilized Base (RSB) that had been used during the construction, since the sections constructed with this type of material revealed higher surface deflections compared to other sections of the road. The results from FWD and back calculated moduli confirmed the hypotheses of using the incorrect a-value for this type of material as well as some other material in the design and the new a-value for RSB was assigned which decreased from 0.17 to 0.14. The layer coefficient of the asphalt material used by NHDOT ranges between 0.34-0.38 and back calculations from FWD in this research resulted in a-value of 0.37 for the wearing course [8].

2.5 Review of Layer Coefficients used by Other Agencies

As part of the literature review in this research, a survey was conducted from 21 State DOTs that currently use any of the AASHTO based empirical design methods to determine the typical a-values that are used for the surface and non-wearing course asphalt mix materials. The survey was not limited to any specific region or climatic condition but the main aim was to evaluate and compare the current NHDOT's layer coefficients with the values used by other state DOTs.

Table 1 shows the result of the survey and it can be seen that New Hampshire is using one of the lowest a-values compared to other states even in the New England area that the environmental and perhaps the traffic loading doesn't seem to be significantly different. The result of the survey confirmed the potential possibility to obtain new layer coefficients for the asphalt materials in New Hampshire as the asphalt mix design method, production and construction techniques have changed quite extensively since the last evaluation done in 1994.

Table 1. Layer coefficients used by other agencies

Layer type	Layer coefficient(a_i)	DOTs
wearing Course	0.54	ALDOT
	0.5	WSDOT
	0.44	ARDOT, FDOT, SCDOT, CTDOT, MaineDOT, MassDOT, IADOT, PADOT, WisDOT, NJDOT, MDOT, GDOT, ConnDOT
	0.43	ODOT
	0.42	NYCDOT
	0.4	DelDOT, IDOT
	0.38	**NHDOT**
	0.35	NDOT, VTDOT
Non-wearing Course	0.44	FDOT, PADOT, SCDOT
	0.42	NYCDOT
	0.4	DelDOT,ConnDOT
	0.36	ODOT
	0.35	NDOT
	0.34	**NHDOT**, MassDOT, MaineDOT, MDOT
	0.33	VTDOT
	0.31	WisDOT
	0.3	GDOT, IDOT

3. Materials, Methods and Research Approach

3.1 Materials

In order to evaluate the structural contribution of the asphalt mixtures in this dissertation, a variety of 18 asphalt mixtures were investigated. The mixtures include two asphalt rubber gap graded (ARGG), four cold central plant recycled (CCPR) mixtures as well as other different types of conventional and polymer modified hot mixed asphalt (HMA) mixtures as wearing, intermediate, and base course layers. The study mixtures are selected to represent a wide range of aggregate size and gradation, binder type, recycled asphalt pavement (RAP) amount and recycled binder ratio (RBR), and gyration level commonly used on New Hampshire highways. Information on the study mixtures is summarized in Table 2. It should be mentioned that the RBR and RAP percentages are used for HMA and CCPR mixtures respectively and all of the HMA mixtures are designed at 4% air void level.

Table 2. Study Mixtures Design Characteristics

Mixture	Course	NMAS (mm)	BinderGrade/ Emulsion type	AC%	VMA %	V_{be}%	RBR/RAP%	Gyration
ARGG-1	**Wearing**	12.5	PG58-28	7.8	19.1	15.1	0.0	75
ARGG-2			PG58-28	7.6	18.4	14.4	6.6	75
W-6428H-12.5			PG64-28	5.4	16.1	12.1	18.5	75
W-5828L			PG58-28	5.8	15	11	16.2	50
W-5834L			PG58-34	5.4	15.3	11.3	18.5	50
W-7628H-12.5			PG76-28	5.4	16.1	12.1	18.5	75
W-7034PH			PG70-34	5.8	16	12	0.0	75
W-7628H-9.5		9.5	PG76-28	6.1	16.3	12.3	14.8	75
W-5828H			PG58-28	5.9	16.6	12.6	16.9	75
W-6428H-9.5			PG64-28	6.4	17.1	13.1	0.0	75
B-6428H	**Intermediate**	19	PG64-28	4.8	14.3	10.3	20.8	75
B-5834L			PG58-34	4.6	14.1	10.1	21.7	50
B-5828H			PG58-28	4.8	14.9	10.9	20.8	75
BB-6428L	**Base**	25	PG64-28	4.8	14.8	10.8	20.8	50
CCPR-1	**Cold Mix Interlayer**	19	MS-4*	4	–	–	100	–
CCPR-1-a				4	–	–	100	–
CCPR-2		12.5		4	–	–	100	–
CCPR-2-a				4	–	–	100	–

* MS-4 is a special type of emulsion that is specified by the NHDOT bureau of material which is explained in Paper2-Chapter 5 of the dissertation.

3.2 Laboratory Evaluation Methods

3.2.1 Resilient Modulus (M_r)

In order to determine the preliminary layer coefficients for the mixtures the resilient modulus (M_r) test was conducted at 25°C on three disk-shaped replicates in accordance to ASTM D7369-11 standard test method [9]. The test method involves a haversine loading protocol with 0.1s of loading and 0.9s of rest period in 105 cycles where the last five cycles are used to calculate the resilient modulus. The preliminary a-values are then calculated using Equation 3 which is recommend by AASHTO 1993 design guide. It should be mentioned that all replicates have been compacted at 6±0.5% air void level. This air-void level was chosen for all experimental evaluation in this dissertation. The air void level choice was informed by NHDOT's typical air-void level for in-place pavements after initial consolidation from traffic loading. In other words, 6% air void level represents a large amount of asphalt pavements in New Hampshire.

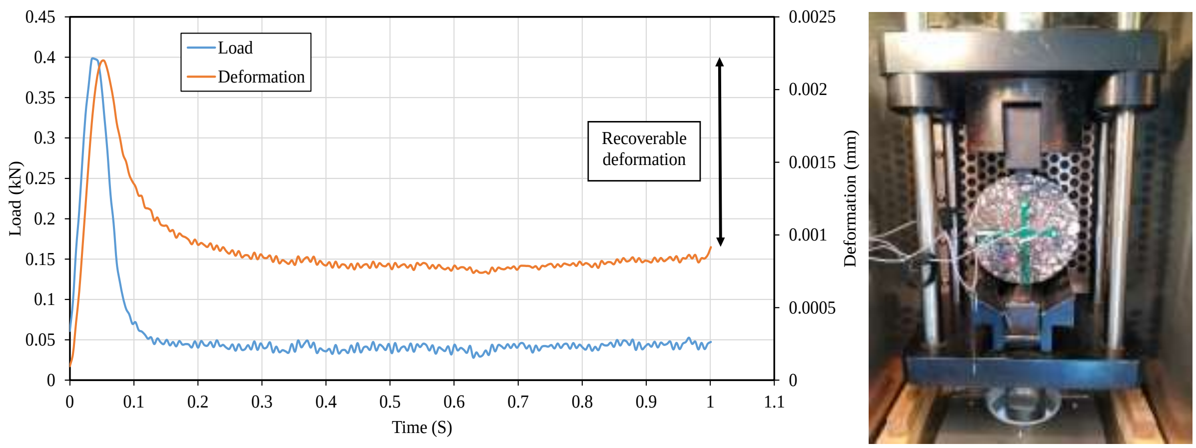


Figure 2. Resilient modulus test result and setup

3.2.2 Complex Modulus (E)*

Complex modulus test is performed in accordance to AASHTO T342 standard [10] using an Asphalt Mixture Performance Tester (AMPT) machine on 150×100 mm cylindrical specimens with a target air-void of 6±0.5%. The test is conducted on three replicate specimens at different temperatures (4.4°C, 21.1°C and 37.8°C) and loading frequencies (25, 10, 5, 1, 0.5, 0.1 Hz) to characterize the linear viscoelastic properties of the asphalt mixtures; dynamic modulus |E*| and phase angle (δ). The dynamic modulus and phase angle master-curves are constructed at a reference temperature of 21.1°C using appropriate shift factors. The |E*| master-curve indicates the stiffness of the mix in a broad range of frequencies at the reference temperature. The master-curve is an excellent tool to compare different mixtures in terms of the stiffness and the rutting susceptibility of the mix. Also, the mixtures can be assessed and ranked in terms of their potential for fatigue and low temperature cracking based on their stiffness. Usually the mixtures with higher stiffness and relatively flatter master-curve are more prone to crack. The phase angle master-curve on the other hand reflects the extent of viscous and elastic properties of the mix at a given temperature and frequency with higher phase angle indicating a better crack resistance of the mix and vice versa.

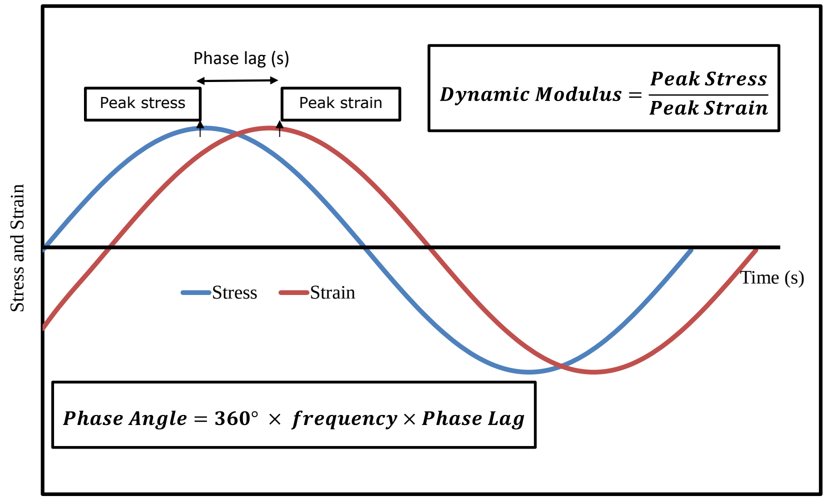


Figure 3. Complex modulus test result and setup

3.2.3 Semi-Circular Bend (SCB)

Semi-Circular Bend test is performed to determine the medium temperature fracture properties of asphalt mixtures in accordance to AASHTO TP 124 standard [11]. The test is conducted in a line-load displacement rate of 50mm/min at 25°C for 4 replicates at a target air void level of 6±0.5%. Fracture energy (G_f) defined as the amount of energy required to create unit fracture surface is determined from area under the load-displacement curve through Equation 4 and the Illinois flexibility index (FI) which normalizes the fracture energy by the post peak slope at inflection point can be determined through equation (5) [12]. While the fracture energy of different mixtures could be the same, the post peak slope is shown to be a good indicator of fracture propagation rate throughout the mixture, hence it is a good discriminating factor for mixture fracture resistance ranking.

$$G_f = \frac{Area\ under\ load-displacement\ curve\ (Farcture\ work)}{Fracture\ Area} \qquad \text{Equation (4)}$$

$$FI = \frac{G_f}{Slope\ at\ post\ peak\ inflection\ point} \qquad \text{Equation (5)}$$

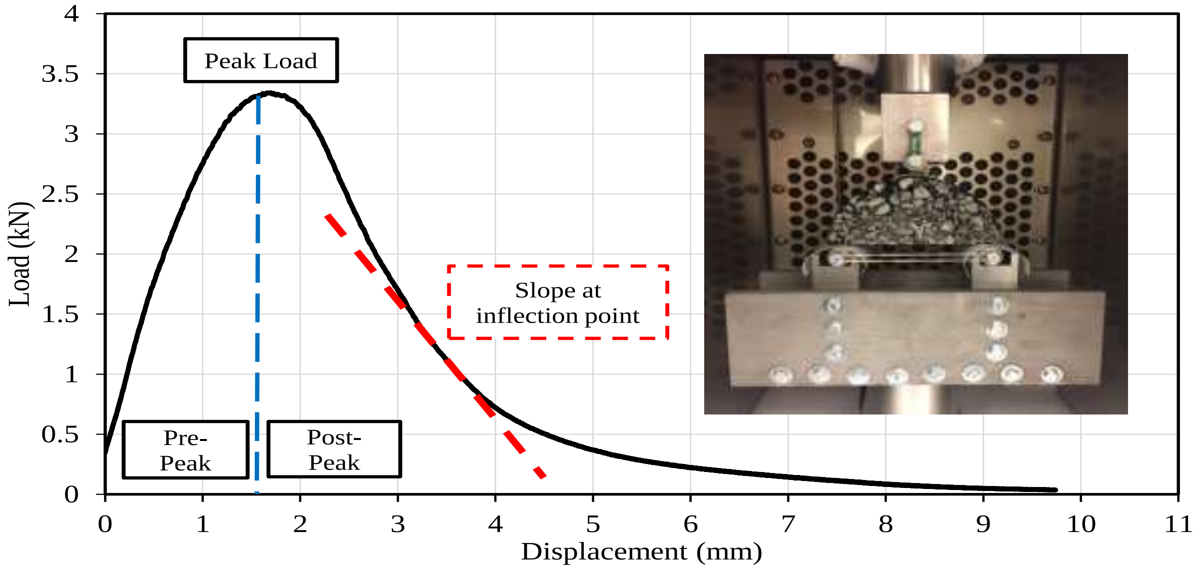


Figure 4. Semi-Circular Bend test result and setup

3.2.4 Direct Tension Cyclic Fatigue (S-VECD)

The uniaxial fatigue test is performed in accordance with AASHTO TP 107 [13] using Simplified Viscoelastic Continuum Damage (S-VECD) theory. The test is conducted on four replicates each at a different strain level under cyclic tension and constant crosshead testing mode. The main outcome of this test is the damage characteristic curve (DCC) which is a mixture property that is independent of loading mode and temperature and indicates the trend of reduction of pseudo stiffness (C) as damage grows.

Currently, there are three accepted fatigue criteria based on the S-VECD approach: G^R and D^R and S_{app}. G^R is the rate of averaged dissipated pseudo strain energy which indicates the decrease in the mixture's energy storage capacity due to each loading cycle [14]. The number of cycles to failure at G^R equal to 100 (N_f @ G^R=100) is usually used to rank mixtures. D^R is the average reduction in pseudo stiffness per loading cycle and indicates the decrease in material integrity in terms of stiffness as the load is applied. D^R values usually range from 0.3 to 0.7 with higher values indicating better fatigue resistance [15]. Sapp is the accumulated damage when C is equal to 1-D^R [16].

$$G^R = \frac{\int_0^{N_f} w_c^R}{N^2{}_f} \qquad \text{Equation (6)}$$

$$D^R = \frac{\int_0^{N_f} (1-C)}{N_f} \qquad \text{Equation (7)}$$

$$S_{app} = \frac{1}{10000} \times \left(\frac{1}{C_{11}} \times D^R\right)^{\frac{1}{C_{12}}} \qquad \text{Equation (8)}$$

Where: w_c^R: total released pseudo strain energy, C: Pseudo stiffness, N_f: number of loading cycles to failure, C_{11} and C_{12}: Model coefficients of the damage characteristic curve

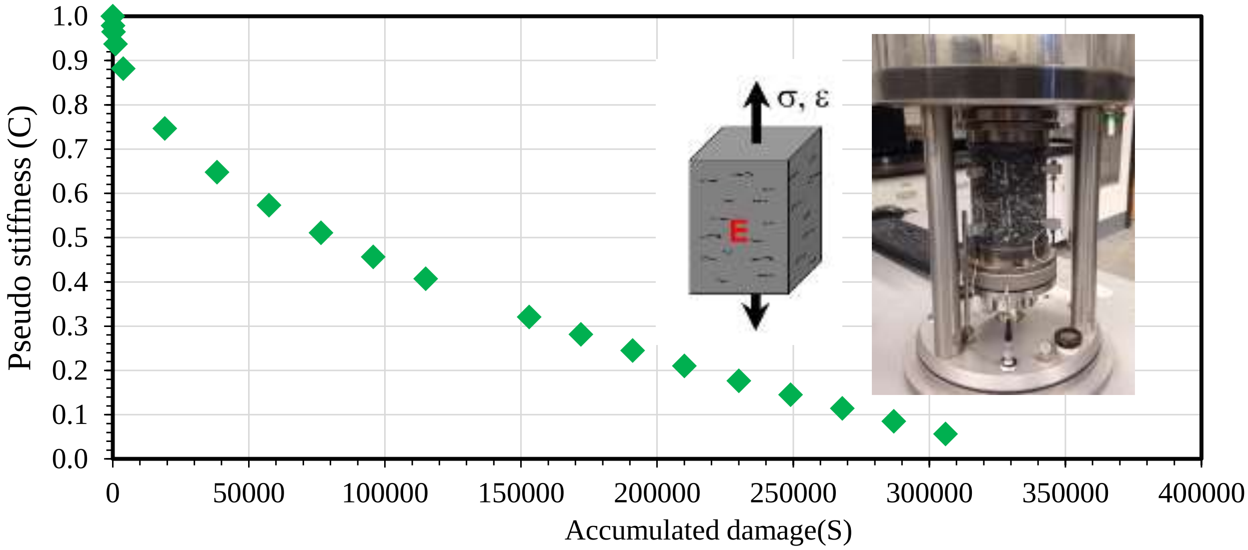


Figure 5. Damage characteristic curve and direct tension cyclic fatigue test setup

3.2.5 Disk-Shaped Compact Tension Test (DCT)

The disk-shaped compact tension (DCT) test was performed in accordance to the ASTM D7313 standard testing method [17] on three replicates. The test is developed to determine the low temperature fracture properties of the asphalt mixtures and is conducted at a crack mouth opening displacement (CMOD) rate of 1 mm/min. In general, the testing temperature is determined by 10°C+PGLT. The two index parameters that are used to analyze the DCT test results are the fracture energy (G_f) and the fracture strain tolerance (FST) [18]. The FST is determined by dividing G_f by the fracture strength (S_f).

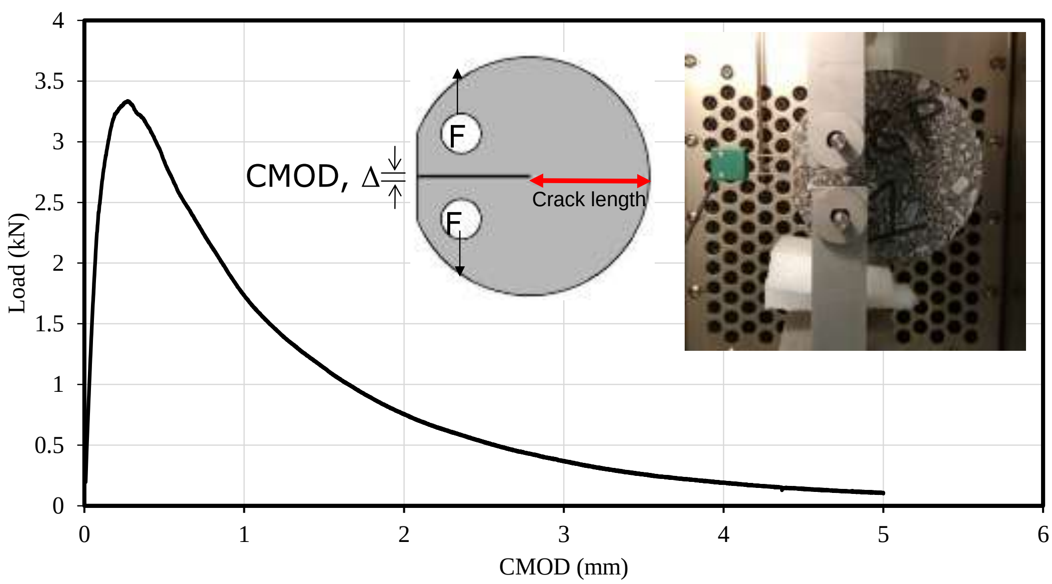


Figure 6. Disk-shaped compact tension test result and setup

3.3 Research Approach

In order to fulfill the dissertation objectives a number of research efforts were undertaken to determine the asphalt mixture performance characteristics and their correlations with the structural contribution of asphalt mixtures commonly used in New Hampshire. These studies are divided into two major categories, where each category includes three efforts that form different chapters of this dissertation (Ch. 4, 5, 6, 7, 8 and 9). Finally the ultimate goal of evaluation of the structural contribution of the asphalt mixtures in form of layer-coefficients for the AASHTO 1993 design approach is realized on basis of inputs from all previous chapters. The two major categories and their respective sub-studies that are indicated in Figure 7 and will be briefly discussed next:

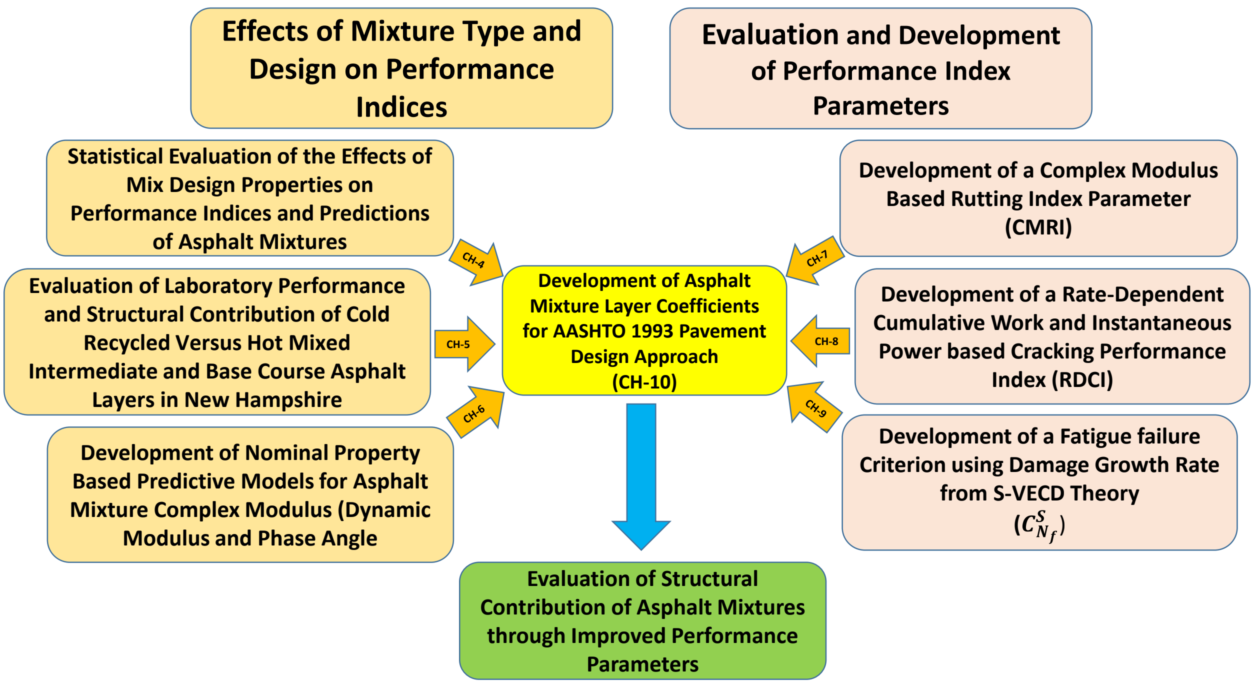


Figure 7. The dissertation objectives and overall research approach

3.3.1 Effects of Mixture Type and Design on Performance Indices

As a first step in determining the structural contribution of the asphalt mixtures in the pavement, it is necessary to understand the effect of individual mixture design parameters such as aggregate size and gradation, and mixture volumetrics such as air void, binder content, voids in the mineral aggregate, etc. Also, due to the advancements in asphalt mixture materials production and placement, different types of mixtures such as wearing, intermediate and base course may be used within the structure of the pavement where each of these courses have their own specific functionality in the pavement. For example, with respect to base courses, both hot and cold mixtures are often used while each of these mixture types may exhibit significantly different performance. Therefore, it is important to investigate and evaluate the implications of using different mixture types in the pavement. Because of these reasons, a comprehensive evaluation of the study mixtures was conducted through performing mechanistic and performance tests to characterize the mixtures and determine the significant factors in terms of nominal properties of the mixtures through advanced statistical techniques. Also, the effect of mixture types and production methods (hot versus cold mixtures) were investigated through modeling the pavement structure and implementing advanced pavement mechanistic analysis software tools such as FlexPAVE^{TM}. The results from the testing and analysis are provided in chapters 4 (Appendix paper 1) and 5 (Appendix paper 2) of the dissertation and are submitted as manuscripts in peer reviewed journals.

As one of the most important properties of asphalt mixtures, the complex modulus has been widely used in order to characterize the mixtures linear viscoelastic (LVE) properties as these properties are directly used in the M-E pavement design approaches. For this reason, many regression based models have been developed to predict the asphalt mixture dynamic modulus. However, most of these models are highly dependent on variables such as binder dynamic shear modulus, aggregate gradation etc. which still need significant lab effort and expensive equipment which may not always be readily available in the lab. Also, limited work has been conducted to predict the mixture's phase angle. Based on the findings from chapters 4 and 5, predictive models for asphalt mixture complex modulus (E* and phase angle) were developed in chapter 6 (Appendix paper 3). These models use only nominal properties of the mixtures such as nominal maximum aggregate size, air void, binder type and content as well as percentage of recycled material in the mixture. Usage of these properties that are readily available during the mixture design procedure, eliminates the need for even simple laboratory based variables such as gradation in the predictive models. Also, these locally calibrated models can be used in pre-evaluating the mixtures performance and their contribution to the overall capacity of the pavement structure. The results from this study is published in a peer review journal.

3.3.2 Evaluation and Development of Performance Index Parameters

Different testing methods and index parameters are available to evaluate and rank the mixtures performance with respect to particular distress types. In order to evaluate and determine the structural contribution of the asphalt mixtures, it is important to investigate the discriminability of these tests and indices and their correlation with the field distress.

There are different empirical and mechanistic-empirical tests such as Hamburg wheel tracking test, asphalt pavement analyzer and flow number that are currently used to determine the mixtures rutting susceptibility. However, some State highway agencies such as NHDOT do not use any type of specific performance tests to determine the rutting performance of the mixtures. Nevertheless, it is well-known that rutting susceptibility can significantly affect the structural contribution of the mixtures. Therefore, it was necessary to determine the rutting susceptibility of the study mixtures and its effect on pavements structural capacity. For this reason, a set of 7 mixtures (outside of the study mixtures) for which the Hamburg wheel track test data is available, were selected to investigate the possibility of development a complex modulus based rutting index parameter. The analysis indicated that there is high correlation between the behavior of pre-peak portion of phase angle master-curve with rutting. Based on the observations, a complex modulus based rutting index parameter was developed in chapter 7 (Appendix paper 4) which is able to rank the mixtures performance with high reliability while indicating a high correlation with the Hamburg wheel test track as well as field rut depth for different mixtures. In order to evaluate the mixtures rutting susceptibility during the mixture design phase, the complex modulus predictive models (developed in chapter 6) can be used to construct the master-curves and determine the value of the rutting index parameter.

There are a variety of cracking performance index parameters such as and G_f, FI, G^R D^R and S_{app} that are used to rank and evaluate the mixtures cracking performance in the lab. However, these indices may not always be able to distinguish the mixtures performance with respect to field distress conditions or different levels of aging. In chapter 8 of the dissertation, a study was conducted to evaluate the ability of FI in discriminating the asphalt mixtures' performance and their sensitivity to different aging levels. The statistical analysis revealed that FI may not be able to distinguish the long term aging levels of mixtures while indicating a relatively high coefficient of variation (COV) between the replicates. Therefore, in this dissertation a rate-dependent cracking index (RDCI) parameter which is based on cumulative work and instantaneous power from the SCB test results was developed. This index revealed to be capable of categorizing the mixtures in a broader differentiated groups with respect to cracking with nearly 10% lowered COV in average for the studied mixtures. This index also revealed to be able to well distinguish different long term ageing levels. The full description of this study is included in Appendix paper 5.

The capability of S-VECD based index parameters (G^R,D^R and Sapp) were investigated in chapter 9 of the dissertation. A set of 6 different mixtures (outside of study mixtures) that are placed on same cross sections of I-93 highway were analyzed through S-VECD approach to determine if the indices are able to rank the mixtures with respect to available field fatigue distress data. The analysis revealed that neither of the indices can reliability rank the mixtures. Therefore, a new fatigue failure criterion based on damage growth rate ($C^S_{N_f}$) is proposed and investigated. The new criterion indicated to not only be able to rank the mixtures but it also is highly correlated with the magnitude of fatigue distress in 5 years after construction. The results of this study are presented in Chapter 9 of the dissertation

In order to develop the layer coefficients, the findings from previous chapters in terms of index parameters were combined with the layer coefficients back-calculated from the field distress data such as International Roughness Index (IRI). As a result, three different types of layer coefficients called as a_{IRI}-value, a_{ave}-value, a_{min}-value were determined in Chapter 10. The a_{IRI}-value is determined based on back-calculation from field IRI data whereas the other two types of layer coefficients are developed based on statistically incorporating the performance index parameters with the a_{IRI}-value at different levels of reliability.

4. Statistical Evaluation of the Effects of Mix Design Properties on Performance Indices of Asphalt Mixtures (Appendix: Paper1)

The content of this chapter of dissertation is in form of a peer-reviewed journal article. Manuscript for the article is provided in the appendix to this dissertation. Abstract and significance of this article within the overall scope of this dissertation as described next.

4.1 Abstract

A variety of testing and performance index parameters are available to assess the asphalt mixture performance with respect to different structural distresses. However, due to continuous improvements in asphalt material production and construction techniques, it is necessary to regularly evaluate the correlation of the performance index parameters with mixture design properties. It is also important to determine the correlation between index parameters from different tests to help save time and financial resources by making engineering based adjustments to the mixture design before conducting multiple tests. This study explores the statistical correlation between mixture design properties and performance index parameters as well as the correlations among the performance index parameters from different tests. A total of 14 commonly used asphalt mixtures in New Hampshire were evaluated using the complex modulus (E*), resilient modulus (M_r), direct tension cyclic fatigue (S-VECD), Illinois semi-circular bend (SCB-IFIT), and disk-shaped compact tension (DCT) tests to assess the correlations between various performance indices and mix design properties. The results indicate that the aggregate fractions that pass 4.75 mm and 75 µm sieve sizes, the binder useful temperature interval, and recycled asphalt content significantly affect most of the index parameters. Medium to high correlations were observed between S-VECD, DCT and SCB with respect to different index parameters.

4.2 Significance of the Study

The structural contribution of asphalt mixtures in the pavement structure is a direct function of mixture design properties such as aggregate size and gradation, binder type and content, air void percentage and any additives in the mixture. For that reason, a thorough understanding of the effect of each of the mixture's components with respect to a specific type of distress is of high interest to pavement engineers. Moreover, the increasing traffic demand along with use of higher amounts of recycled material in the mixtures have led to need for performance based asphalt mixture design procedures such as performance engineered mix design. However, the time and costs associated with conducting multiple performance tests during mix design iterations may be one of the biggest challenge in routine use of such approaches. Therefore, it is necessary to determine the correlations between different performance index parameters and make engineering based adjustments to the mixture design prior to conducting multiple time consuming and expensive tests. The research conducted in this portion of dissertation allows to improve the understanding of correlations between asphalt mix properties (such as, aggregate size and binder grade) to lab measured performance properties. A mapping of the work in this portion of dissertation to overall thesis objectives in provided in Table 3.

Table 3. Summary of chapter 4 contributions to dissertation objectives.

Chapter	Paper	Contribution to the objectives
4	1	Direct contributions to objective 1
		No indirect contributions

5. Evaluation of Laboratory Performance and Structural Contribution of Cold Recycled Versus Hot Mixed Intermediate and Base Course Asphalt Layers in New Hampshire (Appendix: Paper2)

The content of this chapter of dissertation is in form of a peer-reviewed journal article. Manuscript for the article is provided in the appendix to this dissertation. Abstract and significance of this article within the overall scope of this dissertation as described next.

5.1 Abstract

Depending on the local conditions and structural design of the pavement, multiple asphalt concrete layers including base, intermediate, and wearing courses are used. Typically, the base and intermediate layers have larger aggregate sizes and lower total asphalt binder contents as compared to the wearing course. Recently, cold recycled (CR) asphalt mixtures have gained attention as an alternative to the typical base, and to some extent intermediate courses, because of economic and environmental advantages. Challenges with CR include the potential high variability of recycled asphalt pavement (RAP) and lack of knowledge in terms of structural contribution and long term performance of such layers. This study investigates 4 different types of CR and 4 hot mixed plant produced asphalt mixtures (3 intermediate courses and 1 base course) that are typical mixtures used in New Hampshire. The laboratory performance evaluation is conducted through the resilient modulus (M_r), complex modulus (E*), semi-circular bend (SCB) and direct tension cyclic fatigue (S-VECD) tests. Pavement performance prediction is carried out using the results from S-VECD approach in the FlexPAVE^{TM} software. The test results indicate that the performance of CR is highly affected by the amount of oil distillate percentage in the emulsion as well as the amount of recovered binder in the RAP. While having a relatively lower rutting resistance capability, the CR mixtures maintained an acceptable fatigue performance. As compared to CR mixtures, hot-mixed intermediate and base course mixtures indicated better rutting performance while having lower resistance to cracking.

5.2 Significance of the Study

Due to the advancements in technology, different asphalt mixture production methods such as warm and cold mixtures have been produced and used in different layers. However, due to the temperature sensitivity of asphalt binders, the production method can significantly influence the mixtures properties and performance. Therefore, the structural contribution of mixtures within the pavement will be a direct function of mixture production method which is investigated in the research contribution that is presented in Paper 2. The research conducted in this portion of dissertation allows to improve the understanding of the effect of mixture production methods (cold versus hot) in the performance of asphalt mixtures. A mapping of the work in this portion of dissertation to overall thesis objectives is provided in Table 4.

Table 4. Summary of chapter 5 contributions to dissertation objectives.

Chapter	Paper	Contribution to the objectives
5	2	Direct contributions to objective 1
		No indirect contributions

6. Nominal Property Based Predictive Models for Asphalt Mixture Complex Modulus (Dynamic Modulus and Phase Angle) (Appendix: Paper3)

The content of this chapter of dissertation is in form of a peer-reviewed journal article. Manuscript for the article is provided in the appendix to this dissertation. Abstract and significance of this article within the overall scope of this dissertation as described next.

6.1 Abstract

Dynamic modulus (|E*|) and phase angle (δ) are necessary for determining the response of asphalt mixtures to in-service traffic and thermal loadings. While a number of |E*| and δ predictive models have been developed, many of them require lab measured properties (e.g. binder complex modulus). The majority of previous work has focused only on prediction of |E*|, limited models exist for prediction of δ. This research utilized generalized regression modelling of lab measurements (from 81 asphalt mixtures) to develop and verify prediction models for |E*| and δ using only nominal asphalt mix properties that are readily available during the initial mixture design and specification process.

6.2 Significance of the Study

Although |E*| and δ can be effectively used to predict the long term performance of asphalt mixtures using mechanistic analysis, there are limitations related to equipment requirements, specimen fabrication complexity, data analysis and other expenses in terms of human resources and time requirements. These limitations have severely restricted wide-spread usage of mechanistic-empirical and mechanistic pavement analysis and design. Although there are different regression based predictive models for dynamic modulus, a limited work has been conducted to predict the phase angle. Moreover, most of the available models use variables that still need significant lab testing such as the ones that are used in the Pavement ME software. A distinguishing factor for research and the prediction models presented in this study as compared to previous research is that here only nominal properties of asphalt mixtures, such as nominal maximum aggregate size, air void content, asphalt content, the percentage of recycled asphalt pavement (RAP) and recycled asphalt shingles (RAS) and asphalt binder performance grade (PG) are used in model development. Therefore, development of complex modulus predictive models based on the aforementioned properties was deemed important as the performance indices can be estimated during materials selection and pavement design process. Also, such models can be used to assess how each mix constituent and volumetric property impacts complex modulus. A mapping of the work in this portion of dissertation to overall thesis objectives in provided in Table 5.

Table 5. Summary of chapter 6 contributions to dissertation objectives.

<table>
<tr><th>Chapter</th><th>Paper</th><th>Contribution to the objectives</th></tr>
<tr><td rowspan="2">6</td><td rowspan="2">3</td><td>Direct contribution to objective 1</td></tr>
<tr><td>Indirect contribution to objectives 2 and 3</td></tr>
</table>

7. Development of a Complex Modulus Based Rutting Index Parameter for Asphalt Mixtures) (Appendix-Paper4)

The content of this chapter of dissertation is in form of a peer-reviewed journal article. Manuscript for the article is provided in the appendix to this dissertation. Abstract and significance of this article within the overall scope of this dissertation as described next.

7.1 Abstract

Different testing methods have been used to evaluate the rutting susceptibility of asphalt mixtures. Among them, loaded wheel testers, such as the Hamburg Wheel Tracking Test (HWTT), has shown to have promising correlation with the field rutting. Moreover, since rutting distress within pavement structure has a direct correlation with mixtures' structural response to loading, the complex modulus (|E*| and phase angle) master-curves can be potentially used to estimate the mixtures rutting performance. This research introduces and investigates 5 different complex modulus based parameters to evaluate the rutting performance of asphalt mixtures. These parameters are developed based on two critical points on the |E*| and phase-angle master-curves. The first point is related to the frequency corresponding to peak phase angle and the second point is related to the reduced frequency on the master-curve which reflects the HWTT testing conditions. The results from investigating 22 asphalt mixtures indicate that there is a strong correlation between the rutting and the rate of drop in |E*| with respect to changes in frequency between the two selected critical points.

7.2 Significance of the Study

Many state highway agencies such as New Hampshire Department of Transportation do not have any specific requirements for evaluating the rutting susceptibility of the mixtures other than the general ones put forth by the AASHTO R30 standard Superpave mixture design which specifies the minimum number of gyrations for a given traffic level. However, in general, the mixtures that are designed to have an acceptable cracking performance are softer and might be prone to rutting in warmer climatic conditions which can significantly affect the ride quality, highway safety and maintenance and rehabilitation costs. Considering the limitations of the equipment as well as the state agency requirements, there is need for a reliable index parameter to provide a preliminary evaluation of the mixtures rutting susceptibility. The significance of this study is to be able to identify a complex modulus based rutting parameter that can then be used in layer coefficient prediction for purposes of mechanistically informed empirical design approach. The research conducted in this portion of dissertation allows for usage of viscoelastic properties of mixtures in predicting viscoplastic based distress modes such as rutting without need to perform destructive testing which can result in major savings in time and cost. A mapping of the work in this portion of dissertation to overall thesis objectives in provided in Table 6.

Table 6. Summary of chapter 7 contributions to dissertation objectives.

Chapter	Paper	Contribution to the objectives
7	4	Direct contribution to objective 3
		No Indirect contributions

8. Development of a Rate-Dependent Cumulative Work and Instantaneous Power Based Asphalt Cracking Performance Index (Appendix-Paper 5)

The content of this chapter of dissertation is in form of a peer-reviewed journal article. Manuscript for the article is provided in the appendix to this dissertation. Abstract and significance of this article within the overall scope of this dissertation as described next.

8.1 Abstract

Use of the semi-circular bending (SCB) test has gained popularity for evaluating cracking performance of asphalt mixtures. An Illinois Flexibility Index Test (I-FIT) variant of SCB has shown the ability to distinguish mixtures through use of the flexibility index (FI) parameter. While this index has been able to rank the mixtures with respect to performance, a high coefficient of variation (COV) among the replicates has often been observed. Furthermore, parameters such as total fracture energy and FI do not incorporate rate-dependency of fracture processes which are very important for viscoelastic materials such as asphalt mixtures at low and intermediate temperatures. In light of these observations, a rate dependent cracking index (RDCI) is proposed that utilizes cumulative fracture work potential and instantaneous power calculated from the I-FIT test to assess impulse of the mixture. A total of 18 wearing course mixtures were analysed using the RDCI and resulted in an average overall reduction of 10.6% in COV as compared to FI while maintaining similar ranking of mixtures. In general, RDCI was able to better discriminate the 18 mixtures as compared to FI. Evaluation of five mixtures at three aging levels showed robustness of RDCI in capturing effects of aging on fracture behaviour of asphalt mixtures.

8.2 Significance of the Study

One of the main goals of this dissertation is to evaluate the current commonly used performance indices to determine the repeatability and reliability of such indices in discriminating the mixtures performance. This will allow the state highway agencies to improve their specifications by selecting the appropriate test, index parameter and the thresholds associated with that parameter to improve the quality of the mixtures. For this reason the flexibility index as a widely used index parameter by many of the state transportation department agencies, was investigated. The research conducted in this portion of thesis allows for direct application of the rate dependency of asphalt mixtures in characterizing the mixtures' fracture properties which can result in further improvements in designing dynamic based loading tests as opposed to monotonic based ones to better simulate the realistic traffic conditions. A mapping of the work in this portion of dissertation to overall thesis objectives in provided in Table 7.

Table 7. Summary of chapter 8 contributions to dissertation objectives.

Chapter	Paper	Contribution to the objectives
8	5	Direct contributions to objective 2
		Indirect contributions to objective 3

9. Development of a Damage Growth Rate-Based Fatigue Failure Criterion for Asphalt Mixtures Using Simplified-Viscoelastic Continuum Damage Model

9.1 Introduction

The simplified viscoelastic continuum damage (S-VECD) theory has gained wide-spread attention among researchers as a promising asphalt fatigue cracking characterization tool [19-20]. The S-VECD theory uses a damage evolution law to determine the reduction in the pseudo stiffness (C) of material as a function of damage accumulation (S) due to loading cycle (N) [21]. The damage characteristic curve (DCC) reveals the disintegration of a mixture (decrease in pseudo stiffness) as the damage grows. However, the determination of the crack localization point from lab test results has been a major challenge in mixture characterization using the S-VECD approach. The rate of averaged dissipated pseudo strain energy (G^R) [14] and average reduction in pseudo stiffness (D^R) [15] are among the most recent failure criterions that have been proposed to capture the crack localization. The pseudo-stiffness based criteria (D^R criterion) has originally been proposed to mitigate the extrapolation problems associated with the logarithmic scale of G^R [22]. However, mixtures with significantly different DCC curves could have similar D^R values as it only considers the accumulated decrease in pseudo stiffness ($\int_0^{N_f}(1-C)\,dN$) and the number of loading cycles to failure (N_f) regardless of the total damage (S_f) prior to localization. In order to overcome this deficiency, another parameter called as S_{app}has been recently proposed to combine the effects of modulus and toughness in determining cracking susceptibility of asphalt mixtures. This parameter is defined as the damage accumulation (S) when pseudo-stiffness (C) is equal to $1-D^R$ [16]. While these parameters have tried to differentiate the asphalt mixtures fatigue cracking performance in the lab, their strength in discriminating the mixtures' field performance through actual distress data needs to be assessed.

The research presented in this chapter contributes to overall thesis objectives as shown in Table 8. The objectives of this portion of dissertation research are as follows:

1- Evaluate the applicability of the exiting S-VECD based fatigue failure criteria such as G^R, D^R and S_{app} in differentiating the fatigue performance of the asphalt mixtures studied in this dissertation to determine their applicability as fatigue performance index parameter; and,
2- Develop a new S-VECD based fatigue failure criterion that is better correlated with the field performance of asphalt mixtures in New Hampshire.

Table 8. Summary of chapter 9 contributions to dissertation objectives.

Chapter	Paper	Contribution to the objectives
9	N.A.	Direct contributions to objective 2
		Indirect contributions to objective 3

9.2 Material and Testing

A set of 6 mixtures for which the field performance data is available were used to assess various S-VECD theory based fatigue failure criterions. The direct tension cyclic fatigue test in accordance to AASHTO TP 107 standard on the field cores taken after 1 year of construction have been conducted by asphalt research group at University of New Hampshire [23]. The mixture design and properties of the material used to investigate the fatigue failure criterions are summarized in Table 9. The mixtures are part of the North-East High RAP Pooled Fund Study that were placed on I-93 in 2011 and yearly field distress data is available for them [24]. Since the pavement structure, traffic and climatic conditions are same for each of the six mixtures, it would be possible to compare and rank the mixtures independent from other variables that can affect the overall pavement response and performance. As the basis of comparison of the mix fatigue performance indices will be with respect to the field conditions, the testing and evaluation is conducted on the field cores taken after 1 year of construction. Moreover, testing the field cores will eliminate the difference between the production air voids which will lead to a more realistic assessment of the failure criterions.

Table 9. Mixtures Characteristics

Mix		AC (%)	Recycled Binder Ratio (%)	V_a (%)	VMA (%)	VFA (%)
Mixture Design	Virgin 58-28	5.9	0	4.4	16.8	74
	15% RAP 58-28	5.8	13.9	4.3	16.9	74.2
	25% RAP 58-28	5.8	23.1	4.1	16.7	75.3
	25% RAP 52-34	5.8	23.1	3.5	16.5	79
	30% RAP 52-34	5.8	27.7	3.6	16.4	78.1
	40% RAP 52-34	5.8	37	4.2	17	75.2
Production	Virgin 58-28	5.96	0	3.5	16.9	79.5
	15% RAP 58-28	6.11	13.2	2.5	15.6	84.2
	25% RAP 58-28	5.98	22.4	2.2	15.2	85.9
	25% RAP 52-34	5.91	22.7	2.5	15.8	84.1
	30% RAP 52-34	6.23	25.8	3.7	16.4	77.7
	40% RAP 52-34	6.19	34.6	3.4	16.7	79.7

9.3 Field Conditions

Field performance of the sections has been monitored yearly since construction using an automated pavement distress data collection van by New Hampshire DOT. The fatigue cracking is tracked at three severity levels. The weighted crack length for each section is calculated using the following equation (more details on use of this approach to characterize field performance can be found in Daniel et al., 2018 [24]):

$$Weighted\ crack\ length\ \left(\frac{m}{km}\right) = (Severity1\ crack\ length) + 2(Severity2\ crack\ length) + 3(Severity3\ crack\ length) + (Sealed\ crack\ length)$$

Equation (8)

The amount of fatigue cracking in each section is shown in Figure 8. In general, the mixtures with a lower RAP content indicates less cracking and the PG 58-28 binder appears to be performing better than the PG 52-34 binder (as seen from the two 25% RAP mixes). The 25% RAP PG52-34 section appears to have the worst performance overall, whereas the 15% RAP PG58-28 indicates the best fatigue performance among all other mixtures. While the virgin mixture reveals a relatively good and steady performance until the 4th year after construction, it indicates a relatively high rate of damage during the 5th year of service.

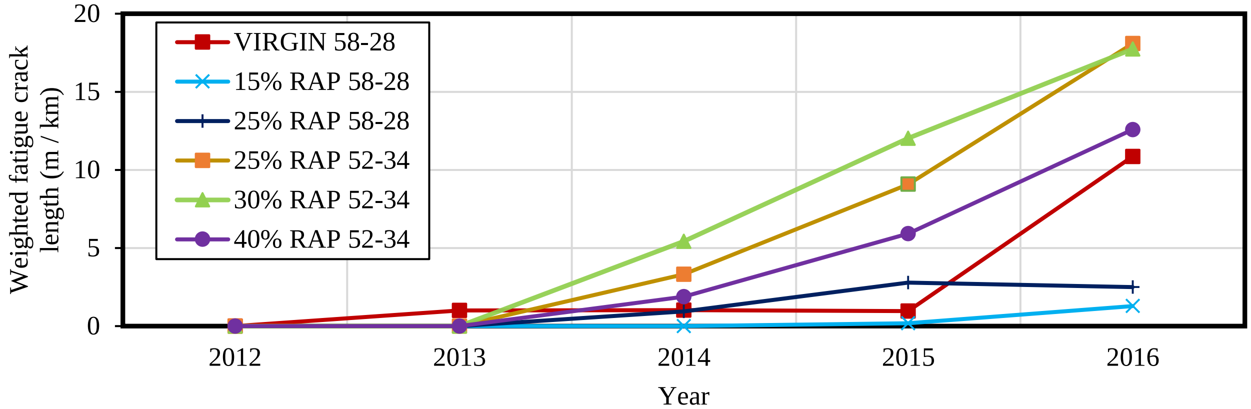


Figure 8. Normalized Field Fatigue Cracking

9.4 Results of the Direct Tension Cyclic Fatigue Test

As mentioned before, the direct tension cyclic fatigue test was conducted in accordance to the AASHTO TP 107 standard method in order to determine the decrease in materials load bearing capacity through the averaged damage characteristic curves (DCC) plots tested for each mixture (Figure 9). Each curve is averaged from the test results conducted on 4 different replicates. The curves indicate the disintegration of the mixtures (decrease in pseudo stiffness) as the damage (S) grows [25]. However, with respect to DCC, a direct comparison between the mixtures may not be appropriate since the number of cycles to failure is missing between curves [26]. Therefore, the mixtures are ranked and evaluated with respect to different available failure criterion: (i) $N_f@G^R = 100$; (ii) D^R; and, (iii) S_{app}, which are plotted in Figure 10, Figure 11 and Figure 12 respectively. The plots indicate that the ranking of mixtures from the three indices are quite different such that the 40% RAP 52-34 is shown to have best fatigue performance with respect to $N_f@G^R = 100$ criteria whereas it holds one of the lowest D^R values and at the same time it is ranked as the third best mixture in accordance to S_{app}. Similar observations can be made for other mixtures such as 25% RAP 52-34 indicating that none of the indices have been able to reliably predict this mixture's field performance. It can be seen from the results that the current failure parameters have not been able to rank the mixtures as compared to the actual field fatigue cracking as a standalone parameter. One main reason for this observation could be that with respect to continuum damage mechanics, it is the evolution and localization of micro-cracks that results in macro-cracks to form fatigue cracking. The magnitude and number of micro-cracks in the S-VECD analysis is quantified by the (S) value where neither G^R and D^R parameters explicitly take the amount of damage (S) into account. Although the S_{app} parameter tries to incorporate the magnitude of the damage in determination of fatigue resistance of the mixtures, it considers the damage at the average mixture's integrity (C at 1- D^R). However, since the accumulation of damage as well as decrease in capacity is a nonlinear phenomenon, the use of average C and corresponding (S) value may not be an appropriate indicator of fatigue failure. These results motivated the need to explore development of a new fatigue failure criterion based on S-VECD theory that can improve the reliability of predicting the field fatigue cracking performance.

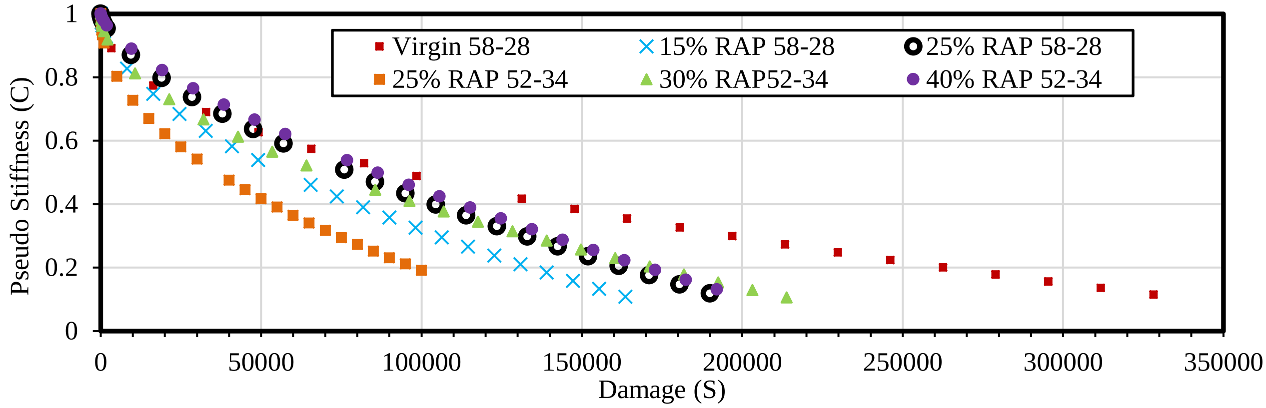


Figure 9. Damage Characteristic Curves (DCC)

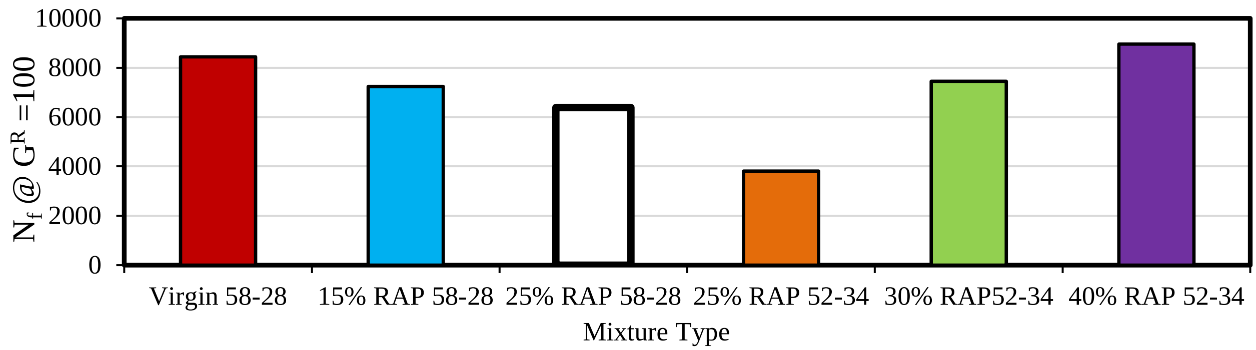


Figure 10. N_f @ G^R =100 Fatigue Failure Criteria

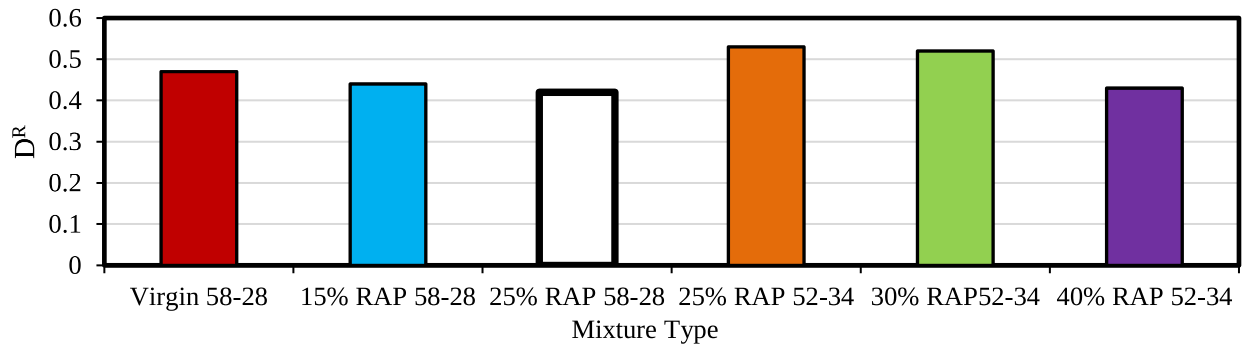


Figure 11. D^R Fatigue Failure Criterion

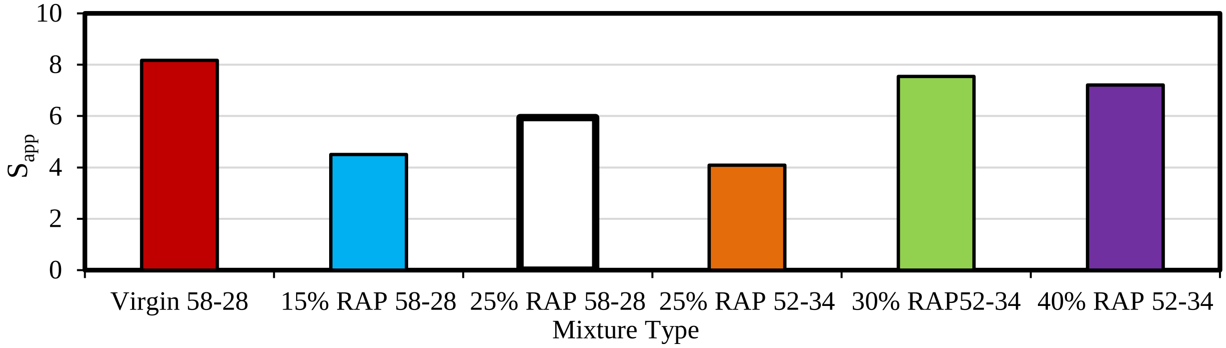


Figure 12. S_{app} Fatigue Failure Criterion

9.5 Development of the Damage-Growth Rate based Fatigue Failure Criterion

As discussed in the previous section, neither of the currently available S-VECD theory based failure criterion were able to rank the mixtures on the I-93 test sections with respect to the actual field fatigue cracking performance which necessitates exploration of new failure criterion.

For a given test specimen in the direct tension cyclic fatigue test, the decrease in pseudo stiffness (C) can be explained through two separate graphs indicted in Figure 13. Figure 13(a) shows the damage characteristic curve where decrease in pseudo stiffness is associated with the accumulation of the damage (C vs S), and Figure 13(b) indicates the decrease in pseudo stiffness for the same test due to loading cycles (C vs N). In the cyclic fatigue test, the failure point of the test is determined through the peak phase angle [27]. This point in the test corresponds to the loading cycle at failure (N_f) and the accumulated damage at failure (S_f). The area above the C vs N curve indicates the accumulated decrease in material's capacity [15] and can be calculated through the following equation:

Accumulated decrease in material's capacity = $\int_0^{N_f} (1 - C)\, dN$ Equation (9)

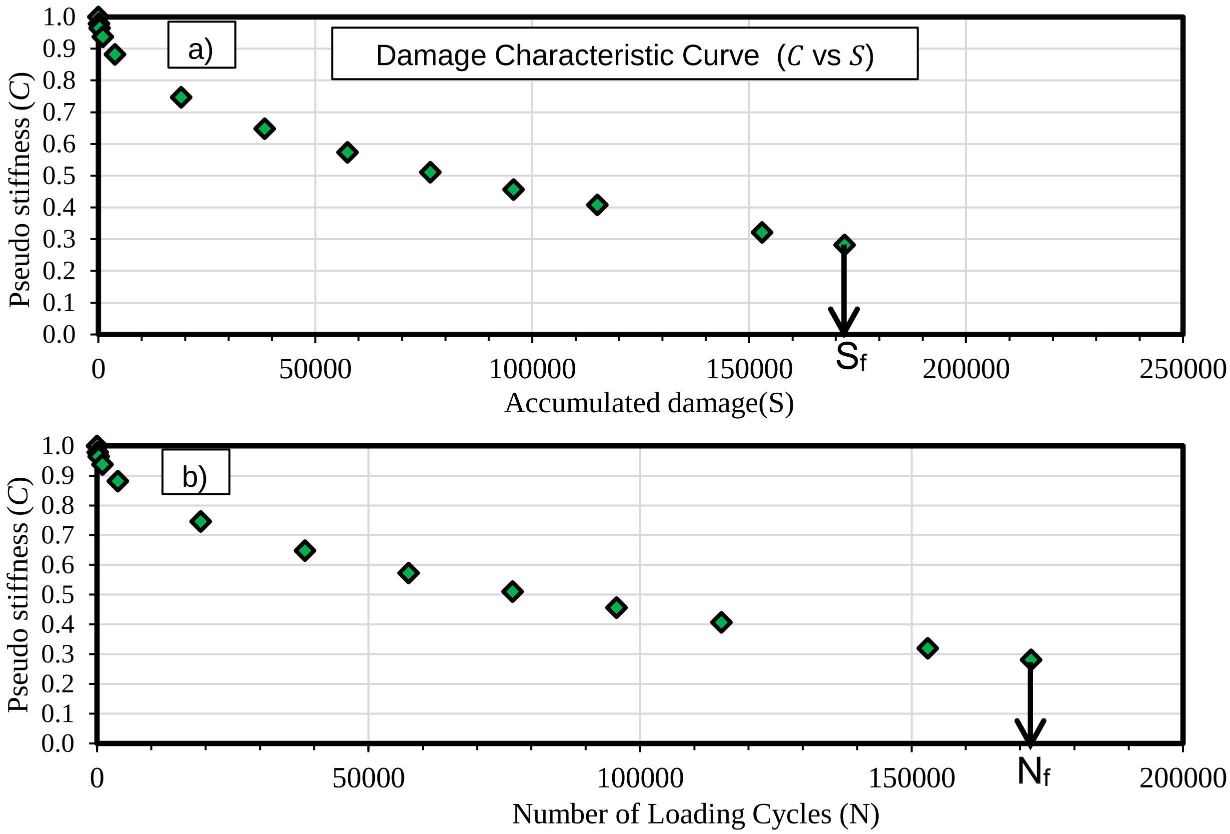


Figure 13. a) Pseudo stiffness versus damage accumulation (*C* vs *S*), b) Pseudo stiffness versus loading cycle (*C* vs *N*)

In order to develop a new fatigue failure criterion in this study, the correlations between these three components of S-VECD theory and analysis as the accumulated decrease in material's capacity ($\int_0^{N_f}(1-C)\,dN$), number of loading cycles to failure (N_f) and accumulated damage at failure(S_f) were investigated and incorporated to result in a damage growth rate based fatigue failure criterion. Using the test results for the 6 mixture from I-93 test section, Pearson's correlation coefficients were determined to investigate the relationships between the aforementioned parameters. These are shown in Table 10. The results indicate that as the number of cycles to failure increases, the magnitude of the accumulated damage at failure decreases. This correlation indicates that for different replicates of a same mixture, a higher level strain in the test may result in a lower amount of accumulated damage at the peak phase angle. In other words, at higher cyclic strains the rate of development of micro-cracks and their localization to form a macro-crack is high enough that the mixture is not able to use its full capacity to evenly disperse damage throughout the continuum to withstand the failure. This phenomenon is similar to a thermal shock occurrence for many other types of materials including asphalt mixtures where a sudden change in temperature results in premature cracks in the material before the material is able to reorganize its microscopic or even molecular structure to accommodate the temperature gradient. For the same reason and similarly, as the accumulated reduction in material's capacity ($\int_0^{N_f}(1-C)\,dN$) increases due to higher strain levels the amount the accumulated damage at failure decreases. These observations indicate that not only the magnitude of the damage at failure is important but also the rate of increase in damage growth (governed by the strain levels at cyclic loading) is an important parameter that should be taken into consideration in development of a mixture fatigue performance index using the S-VECD theory.

Table 10. Pearson's correlation coefficients for the S-VECD based parameters.

S-VECD based parameters	N_f	S_f	$\int_0^{N_f}(1-C)\,dN$
N_f	1.00	-	-
S_f	-0.53	1.00	-
$\int_0^{N_f}(1-C)\,dN$	1.00	-0.53	1.00

With regards to the aforementioned discussion on the correlations between the investigated parameters, new fatigue failure criterion which is based on damage growth rate is proposed and indicated in Equation 10.

$$C_{N_f}^S=\frac{\int_0^{N_f}(1-C)\,dN}{S_f}\times m \qquad \text{Equation (10)}$$

Where:

$C_{N_f}^S$: Damage growth rate based fatigue failure criterion,

$\int_0^{N_f}(1-C)\,dN$: Accumulated decrease in pseudo stiffness,

S_f: accumulated damage at failure

m : Unit correction factor set to 10^3 to increase the order of magnitude of the $C_{N_f}^S$ and for simplicity of comparisons between different mixtures

With respect to $C_{N_f}^S$, the higher S_f and lower $\int_0^{N_f}(1-C)\,dN$ are more desirable for fatigue performance as they indicate that material is able to withstand higher amounts of damage with less disintegration.

9.6 Comparison of the Proposed Damage Growth Rate Fatigue Criteria (CSNf) with Currently Available Criteria (Nf @ GR =100, DR and Sapp)

In order to evaluate the reliability of the proposed failure criterion, the $C_{N_f}^S$ versus number of cycles to failure (N_f) graphs were plotted for each replicate tested for the six mixtures in Figure 14. The direct tension cyclic fatigue is usually conducted on 4 specimens each tested at a different strain level. The accumulation of damage and decrease in material's capacity due to different strain levels is significantly non-linear which can result in wide ranges N_f value as can be seen in the figure. The results indicate a linear relationship in arithmetic scale between the proposed failure criterion and number of cycles to failure at each level of strain which eliminates the possible errors of the extrapolation that may occur in a logarithmic based relationship. In general, With respect to the graphs and definition of $C_{N_f}^S$ a lower slope of the fitted trend line between different mixtures is more desirable. For the purposes of simplicity in applying the $C_{N_f}^S$ for comparing the mixtures' performance, a threshold parameter as $N_f@C_{N_f}^S = 100$ is suggested to be used in this study. This threshold parameter has been able to differentiate the mixtures performance with respect to field data. However, more investigations is required to confirm that this threshold is applicable to all types of mixtures and traffic levels. The ranking order from all the available failure criterions with respect to field performance is presented in Table 11. According to the rankings the $N_f@C^S_{N_f} = 100$ parameter has been able to rank all 6 mixtures, while other parameters such as $N_f @ G^R = 100$ and S_{app} have only predicted the worst mixture's performance among others. The results from the comparisons indicate the robustness of the newly proposed fatigue failure criteria in discriminating the mixtures performance with respect to normalized field crack length in 5 years after construction. It is worth mentioning that although a parameter such as D^R has not been able rank a mixture's performance, this parameter has been previously indicated to be a useful tool in discriminating the general properties of the mixtures with respect to production method and overall performance [26].

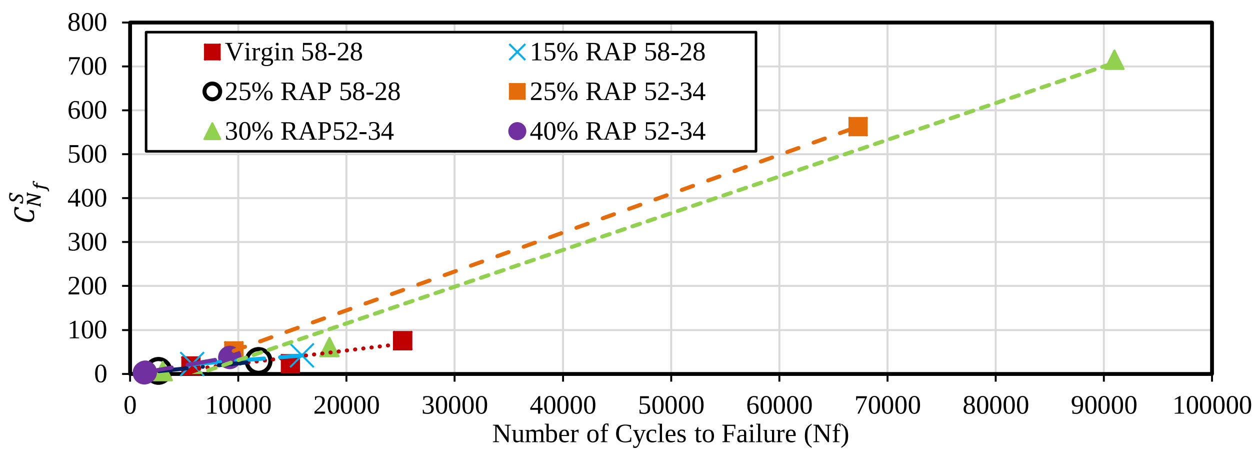


Figure 14. $C^s_{N_f}$ versus N_f plots

Table 11. Mixture ranking order in accordance to different failure criterion

Ranking with respect to different parameters (1;best , 6 worst)					
Mixture	Field Rank (5years after construction)	$N_f@C^S_{N_f} = 100$	$N_f@G^R = 100$	D^R	S_{app}
Virgin 58-28	3	3	2	3	1
15% RAP 58-28	1	1	4	4	5
25% RAP 58-28	2	2	5	6	4
25% RAP 52-34	6	6	6	1	6
30% RAP52-34	5	5	3	2	2
40% RAP 52-34	4	4	1	5	3

Although the mixture ranking is an important tool to discriminate the mixtures' performance, the statistical and mathematical correlation between the developed index parameter and the field data should be evaluated to examine if the parameter is capable of determining the order of magnitude, if different, between mixtures. For this reason, the normalized fatigue crack lengths were plotted versus the $N_f@C^S_{N_f} = 100$ parameter to determine the statistical correlation in terms of the R^2 goodness of fit parameter for the data. As it is shown in Figure 15, a power function fit with an R^2=0.73 was fitted to the data. The power function fitting is presumed to be suitable for fatigue as it can more realistically describe the boundary conditions of the crack length while it also can help in determining the appropriate threshold N_f value for a specific project during a performance based mixture design phase. Also, The power function has similar format as fatigue endurance limit where very poor mixtures have a very low N_f at the proposed threshold and similarly there is an asymptotic form for very good performers having infinite fatigue life.

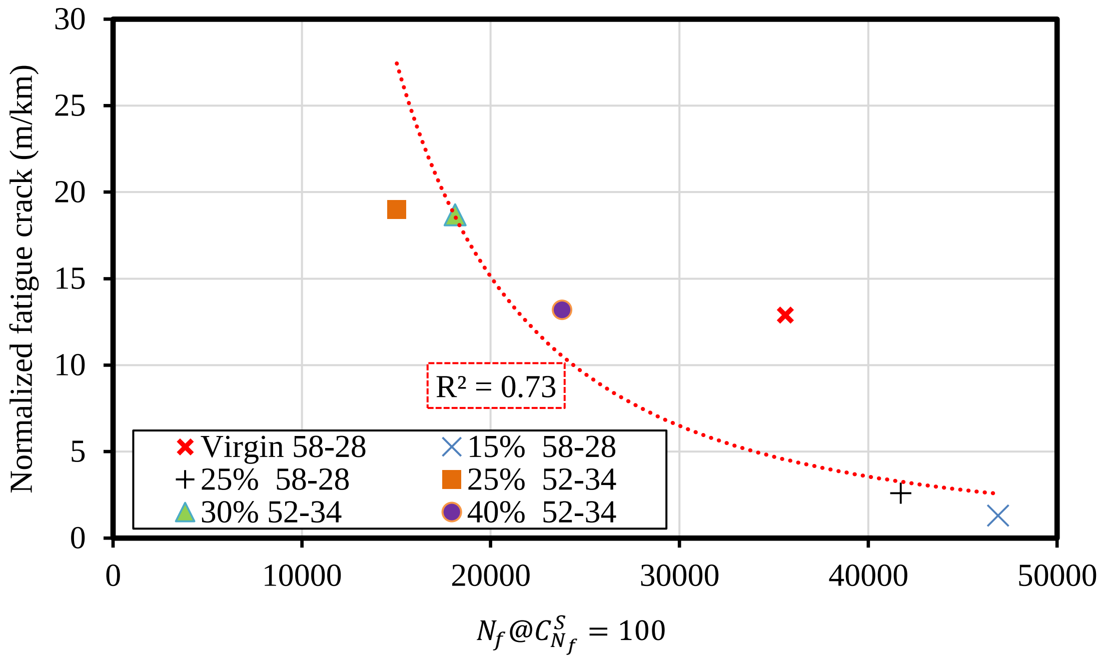


Figure 15. Statistical correlation between the field cracking length and the proposed fatigue criterion

9.7 Evaluation of the Study Mixtures through the Proposed Failure Criterion

Figure 16 indicates the results from evaluation of the study mixtures (c.f. Table 2) through the proposed fatigue failure criterion. Similar to the results from D^R and G^R criterions as indicated in paper1-chapter4, ARGG-1 has a better performance compared to ARGG-2. However, with respect to other wearing course mixtures, the ranking is different such that W-7628H-12.5 is shown to have the best performance followed by W-5834L and W-6428H-9.5. With respect to intermediate and base course mixtures, $C^S_{N_f}$ ranks B-6428H as a better mixture compared to BB-6428L while the inverse ranking had been observed from both D^R and G^R criterions.

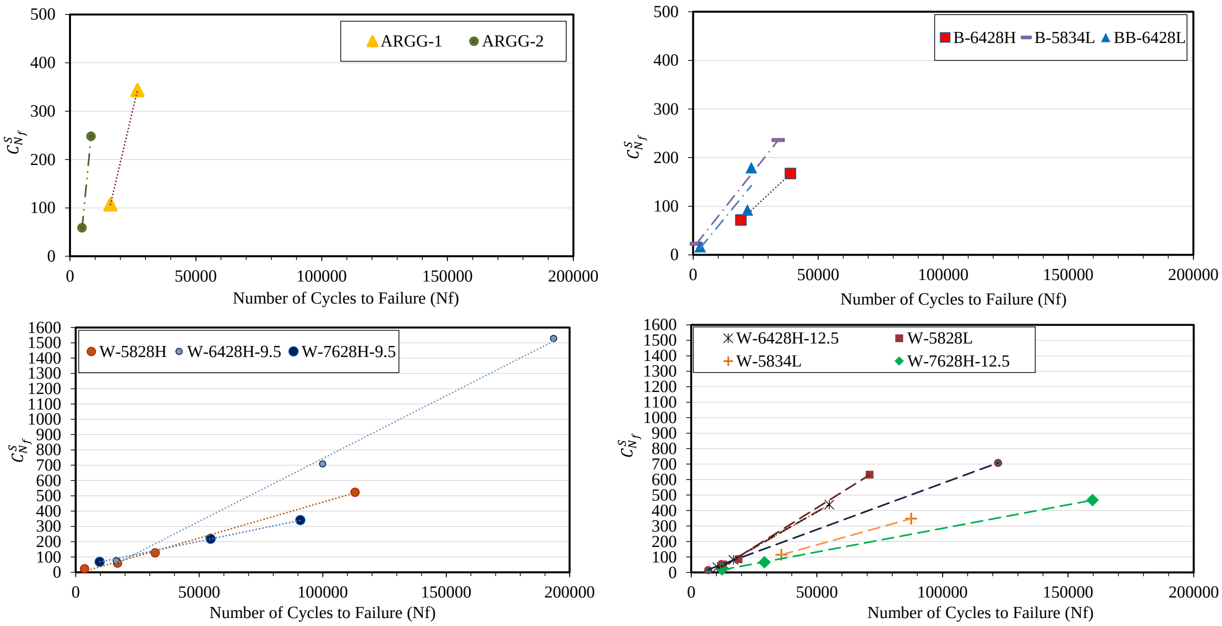


Figure 16. Proposed ($C^S_{N_f}$) fatigue failure criterion against number of load repetitions to failure (N_f) plots for the study mixtures.

9.7.1 Evaluating the Field Performance of the Study Mixtures through New Fatigue Cracking Performance Criteria $C^S_{N_f}$

In order to further evaluate the applicability of $C^S_{N_f}$ to different types of cross sections with different amount of traffic levels, a set of 6 wearing course mixtures from the study mixtures were selected for more detailed evaluation. The mixtures have been used in different construction projects with different levels of traffic in New Hampshire and the field distress data for several years after construction are available for them. It needs to be mentioned that selection of these mixtures and cross sections has been based on availability of data for analysis at the time of performing this research and further analysis will on other cross sections will be conducted as a future work. Using equation 8, the normalized field crack lengths have been plotted in Figure 17. Since the cross sections, traffic volume and weather situations have been different for these projects, the area under the curves normalized by the squared time after construction was used to unify the cracking performances for different mixtures as shown in Figure 18. This method of normalizing field cracking performance form different pavement sections has been developed and validated by previous work by Dave et al. 2016 [28]. As it is demonstrated in Figure 18, a power function fit has resulted in a very good correlation (R^2=0.80) between the fatigue index parameter and the amount of field cracking for different mixtures. These observations reaffirm the reliability and usefulness of the proposed fatigue failure criterion to be used as an indicator for the structural contribution of the mixtures in the pavement design.

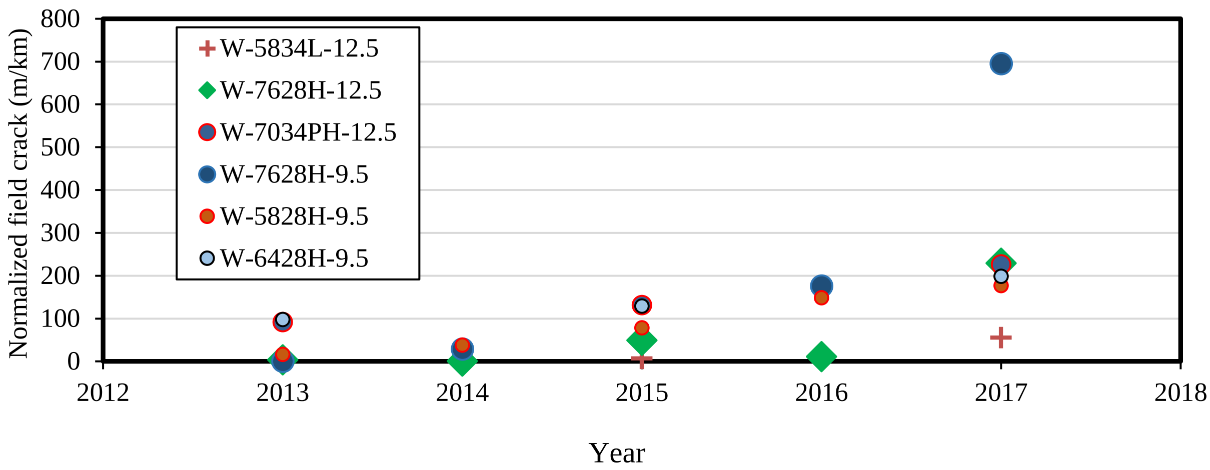


Figure 17. Normalized field crack length for different mixtures in terms of meter per kilometer

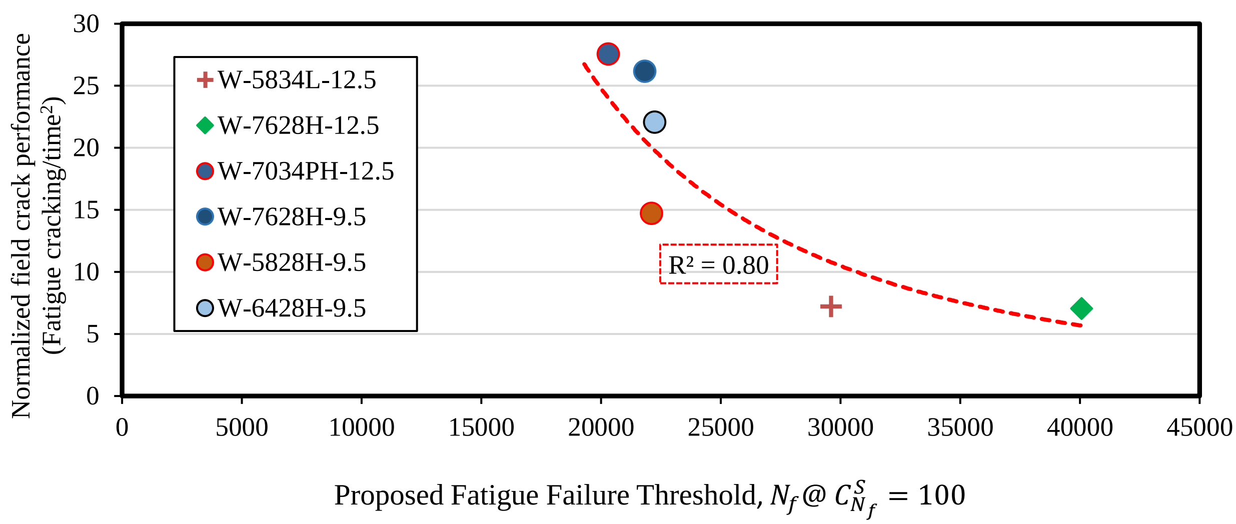


Figure 18. Normalized field fatigue cracking performance versus proposed fatigue failure threshold ($N_f@\ C^S_{N_f} = 100$).

9.7.2 Evaluating the Laboratory Performance of the Study Mixtures through Different Failure Criteria

In order to evaluate and rank the mixtures' laboratory performance, the analysis was performed using the existing and newly developed fatigue failure criteria to compare the results between different parameters. Figure 19 indicates the normalized results determined through different parameters. As it can be seen from the figure, each parameter has resulted in a different order of ranking compared to others as well as the ranking with respected to the field conditions which was previously shown in Figure 18. For example, the W-7034PH-12.5 mixture is shown to be best by by $N_f@G^R = 100$ and D^Rparameters while it is indicated to be worst by $N_f@C^S_{N_f} = 100$ parameter. Table 12 indicates the mixture ranking with respect to different failure criteria for the evaluated mixtures. The results from comparisons indicate that $N_f@C^S_{N_f}=100$ is a better discriminating among other parameters.

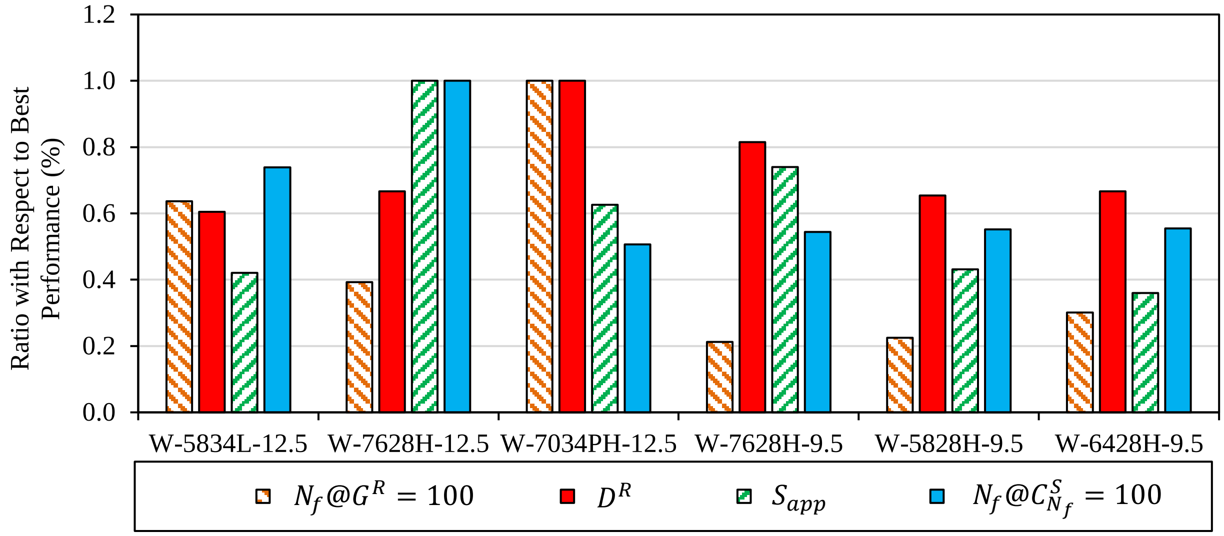


Figure 19. Comparison of asphalt mixture laboratory performance using different failure criteria (for each criteria best performing mixture is used as normalizing factor).

Table 12. Mixture ranking order in accordance to different failure criterion

Ranking with respect to different parameters (1;best , 6 worst)					
Mixture	Field Rank (5years after construction)	$N_f@C^S_{N_f} = 100$	$N_f@G^R=100$	D^R	S_{app}
W-5834L-12.5	2	2	2	5	5
W-7628-12.5	1	1	3	3	1
W-7034PH-12.5	6	6	1	1	3
W-7628H-9.5	5	5	6	2	2
W-5828H-9.5	3	4	5	4	4
2-6428H-9.5	4	3	4	3	6

9.8 Summary and Conclusion

There are currently three fatigue failure criterion that are commonly used to evaluate fatigue performance of asphalt mixtures that are tested through direct tension cyclic fatigue testing method and the S-VECD theory. However, because of the challenge of using logarithmic scale in defining G^R, insensitivity of D^R to the amount of damage growth prior to crack localization, and lack of S_{app} parameter in appropriately ranking mixtures as per field performance (as shown in this chapter), there is a need for a stand-alone fatigue threshold that can be reliably used to rank field performance. Therefore, a new failure criterion called as $C^{s}_{N_f}$ was developed and investigated. This criterion incorporates three components of the S-VECD theory ($\int_0^{N_f}(1-C)\,dN$, N_f and S_f) to capture the mixture's disintegration with respect to damage growth rate. In order to use this parameter for a given mixture, the $C^{s}_{N_f}$ is calculated for each tested replicate (minimum of two strain levels are required) and the results are plotted versus the number of cycles to failure (N_f). An index parameter called as $N_f@C^{S}_{N_f} = 100$ is determined for ranking purposes of different mixtures. The evaluations of the new parameter were conducted through investigations of two different set of mixtures (each set combined of 6 mixtures) for which the field distress are available and $C^{s}_{N_f}$ indicated to be able to reliably rank the mixtures. The parameter indicated that it is not only able to rank the mixtures but it also has a high correlation with the magnitude of cracking in the field. Therefore, as a future step in this research this index can be used to determine the appropriate threshold value to be used in a performance engineered based mixture design approaches.

10. Methodology to Develop the Layer Coefficients for AASHTO 1993 Design Approach

10.1 Introduction

Although a mechanistic pavement design approach which can precisely predict the evolution of distresses during the service life is considered as the ultimate goal in the pavement design system, the transition from a purely empirical based design approach to a mechanistic based design may take decades required for generating reliable database and transfer functions. While some state highway agencies have been pioneer in accepting a mechanistic-empirical pavement design approach, many others are still using the AASHTO 1993 empirical design approach as the available pavement design and performance database and engineering experience gained from using this approach keeps it as a simple yet reliable tool for the pavement design.

As discussed in chapter 2, one of the main inputs of the AASHOT 1993 design equation is the layer coefficients (a-values) that are used to quantify the structural contribution of the material in the pavement structure. The original layer coefficients within the current AASHTO 1993 design equation is based on statistical regression analysis from AASHO road tests and the layer coefficients of various pavement layers are functions of traffic level, weather conditions, subgrade soil modulus and level of reliability at the time of the AASHO road test in early 1960s. However, due to the improvements in material properties and production, quality control as well as construction techniques, many state agencies have tried to reevaluate and update their layer coefficients to accommodate these improvements.

Different approaches have been investigated and implemented by researchers and practitioners to update the layer coefficients (also known as, structural coefficients) for the asphalt concrete pavement layers within AASHTO 1993 empirical pavement design system. In this chapter of the thesis, the main aim would be to develop a generalized methodology to develop mechanistic performance incorporated layer coefficients for asphalt mixtures and for this reason, the performance index parameters for rutting (paper 4 chapter 7), transverse cracking (paper 5 chapter 8) and fatigue cracking (chapter 9) that were previously developed in this research will be used as the primary material inputs in development of layer coefficients. It should be mentioned that in the original AASHO road test and layer coefficients the pavement rutting was only considered to be due to plastic deformation of the subgrade soil, however, it is well-known that part of the overall rutting could result from the asphalt mixtures and for this reason, the mixtures' rutting performance will be incorporated in development of layer coefficients in this research.

The layer coefficient, as an indicator of structural contribution of each layer, may not be a constant value for a pavement structure during its design life as material properties and climatic conditions are ever changing, resulting in different overall pavement response, possible even in relatively small time interval of a day. As a result, determining a continuously evolving layer coefficient, might be a significantly challenging task. Moreover, the establishment of a direct correlation between the performance index parameters such as the ones reviewed and developed in this research and layer coefficients, may not be appropriate since the resulting layer coefficients will be solely dependent on the material properties which ignores other effective variables in determining a realistic layer coefficient. As a result, it is necessary to incorporate the field distress conditions in development of layer coefficients to account for other types of variables such as traffic level and climatic circumstances. Therefore, among different types of distress index parameters such as PSI, PCI and etc. in this research the International Roughness Index (IRI) as a standardized distress index parameter was selected to be used as the primary tool in evaluating the field distress data. Although the pavement functionality measures such as IRI include different types of non-structural degradation such as raveling, potholes etc. a significant portion of them is related to structural distresses such as rutting and cracking. In addition, among the available functional distress index parameters, the IRI is more popular since it is measured by a standard vehicle's accumulated suspension motion and therefore it is less affected by external variables such as visual observations that can reduce the reliability of a functional distress index parameter. Moreover, there are different regression based equations that relate IRI to PSI which is one of the variables in the AASHTO 1993 equation. Besides, many state highway agencies such as NHDOT gather yearly IRI data for different highways as part of their pavement management system.

To fulfill the third objective of this dissertation, this chapter will mainly focus on development of layer coefficient for wearing course mixtures introduced in Table 2 as the distress data is only measured on the wearing course. Using the New Hampshire Pavement Management System data base, the field distress data for a set of 17 cross sections which have been constructed by similar mixtures to the ones in this research were investigated and utilized to develop the layer coefficients. As a final product in this chapter, through incorporating the performance index parameters with the field distress based back-calculated layer coefficients, a set of new layer coefficients called as performance incorporated a-values will be developed and proposed to be used at different levels of reliability.

10.2 Resilient Modulus Based Layer Coefficients

A series of laboratory testing including resilient modulus (ASTM D7369), complex modulus (AASHTO T 342), direct tension cyclic fatigue (AASHTO TP 107), semi-circular bend (AASHTO TP 124) and disk-shaped compact tension (ASTM D7313) were conducted to characterize the mixtures in the lab. The detailed information of the testing condition and results are provided in chapter 3 as well as appendix 2 of this thesis. As mentioned earlier in section 2.4.4, the resilient modulus has been conventionally used to back-calculate the layer coefficients of asphalt mixtures through Equation 11. For this reason, it was decided to first explore use of this equation to determine layer coefficients for the study mixture (Table 2) prior to incorporating the distress data and other laboratory test results in development of layer coefficients. The results are plotted in Figure 20. As expected from the resilient modulus based a-value equation, the stiffer mixtures with higher resilient modulus have higher layer coefficient values whereas most of the wearing course mixtures such as W-7034PH, W7628H-9.5 and W-5834L as well as the cold mixtures are indicated to have a relatively lower a-values. The results reveal the fact that resilient modulus alone may not be an appropriate tool to determine the layer coefficients of asphalt mixtures as some polymer modified mixtures such as W-7034PH with a comparable fatigue performance and relatively better transverse cracking performance compared to many other wearing courses in this study has the lowest resilient modulus based a-value.

$$a_i = 0.4 \log(M_r) - 0.951 \qquad \text{Equation (11)}$$

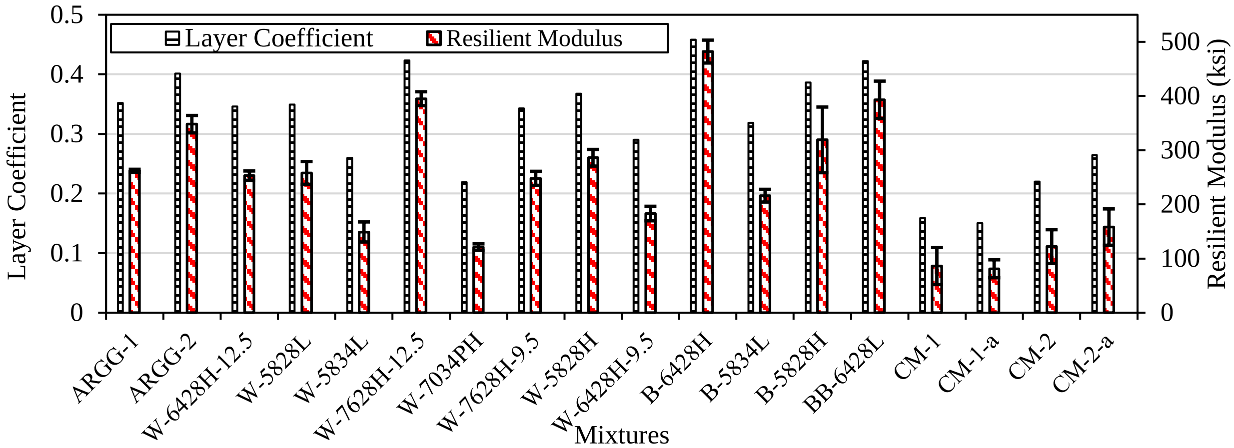


Figure 20. Resilient modulus based layer coefficients

10.3 Field Distress Data Analysis

As one of the important steps in evaluating the structural contribution of the mixtures in form of layer coefficients, it is necessary to assess the field distress data of the study mixtures (Table 2) after construction. However, at this point of time in this research, there is no considerable field data available for the study mixtures as they have been placed during 2016, 2017 and 2018 construction season. For this reason, a set of yearly measured distress data including IRI, rutting, fatigue and transverse cracking for similar mixtures to those tested and analyzed in this dissertation were provided by the New Hampshire Department of Transportation from their pavement management system. Similar mixtures in this work are defined as those that have same NHDOT mixture designations, that is, same application (wear, binder or base course), NMAS, gyration level, recycled binder amount and binder PG grade.

Due to the aforementioned reasons, among different types of distress data, the International Roughness Index (IRI) was selected for further evaluations and development of layer coefficients. A total of 17 cross sections were investigated and the average yearly IRI values were plotted versus the pavement service time. The distress measurements are available only for a maximum of 5 years after construction for different mixtures and cross sections. However, the investigations indicated that at least for the first 5 years after construction, the yearly increase of the IRI has been following a linear trend for almost all of the mixtures and cross sections. While it is acknowledged that there is likelihood that the life-time IRI performance trends will not be linear in shape and will most likely follow an "S" shaped response, due to limitations of data availability in NHDT's PMS, this dissertation used linear shape. Furthermore, use of linear shape is expected to have a more severe deterioration rate and thus can provide some added reliability in the analysis. With use of linear fitting of IRI with time, the field IRI values after 20 years in service were determined for each cross section separately. It is well known that the initial IRI values immediately after construction can vary significantly among different cross sections with similar traffic and climatic situations due to differences in the construction quality as well as the conditions of the underlying layers. Also the increasing trend of the IRI right after construction up to the first year may not necessarily follow the trend after the first year and beyond. Nonetheless, determination of initial IRI immediately after construction is important as it translates into the Initial Serviceability (P_i) value for use in the AASHTO 1993 design process. The initial serviceability is deducted from the

Terminal Serviceability (P_t) to obtain the ΔPSI value as the allowable serviceability loss at the end of design life. As described in section 2.2.3, ΔPSI is a key input in the AASHTO 1993 design equation which can significantly affect the layer coefficient back calculation using the field data.

Research performed by Al-Omari [29] investigated the correlation between IRI and PSI using distress data of over 370 cross sections including flexible, rigid and composite pavements from 6 different states such as Indiana, Louisiana, Michigan, New Mexico, New Jersey and Ohio. The results indicated that IRI and PSI were found to be highly correlated (R^2 =0.81) and their relationship can be described using a nonlinear model. Equations (12) and (13) indicate the relationship between IRI and PSI for flexible pavements.

$$PSI = 5e^{(-0.0038*IRI)} \qquad \text{Equation (12)}$$

Where IRI is in inches per mile

$$PSI = 5e^{(-0.24*IRI)} \qquad \text{Equation (13)}$$

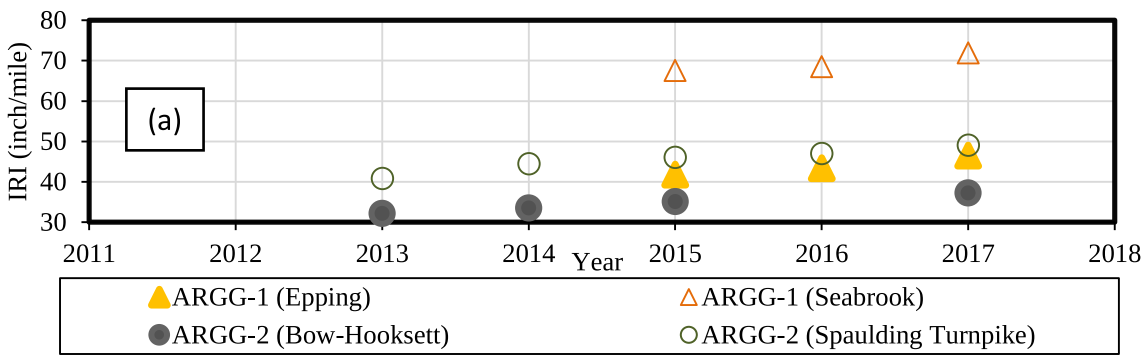
Where IRI is in meters per kilometer

Considering the nature of the power function type of equation, the initial serviceability can significantly vary based on the initial IRI value. However, an initial value of 52 inches per mile is considered to be acceptable [30]. Base on this value, it was decided to divide the quality of the construction and initial IRI values into three categories. These categories are defined based on the construction quality and measured field IRI values at one year after construction. Table 13 indicates these categories and their criteria. As it can be seen from the table when the first year IRI is above 55 inch/mile the extrapolated IRI from the linear fit will be used to determine the initial IRI and consequently Equation 9 will be used to determine the P_i. However, when IRI is below 45 a fix P_i value of 5 and when IRI is between 45 and 55 inch/mile a fixed P_i value of 4.5 will be used for the analysis. With respect to AASHO road test and AASHTO 1993 pavement design approach for flexible pavements, the P_i value is usually considered to be 4.2 as an average initial serviceability. However, this is a generic value and may not be appropriate to be used when actual project data are available. In addition, with respect to Equations 12 and 13 when IRI is equal to zero the PSI will be equal to 5 and for small changes in IRI the PSI will have major reductions due to the power nature of the functions. For this reason different ranges of the construction quality were taken into account to mitigate the art effect of the power function in the analysis.

Table 13. Defining the initial serviceability value based on construction quality and IRI values one year after construction.

Construction quality	Range of field IRI, one year after construction (inch/mile)	Assumption of Initial Serviceability (P_i)	Remarks
High	IRI < 45	5	-
Medium	45 ≤ IRI ≤ 55	4.5	-
Low	IRI > 55	Varied	Use the extrapolated linear fit from the measured IRI values to determine the initial IRI and back-calculate PSI using Equation 12

Figure 21 indicates the increase of IRI with time after rehabilitation for different mixtures that are placed on different projects in New Hampshire. The first point for each mixture and project is related to the IRI after one year of construction. As it can be seen from the figure, the mixtures with smaller aggregate size (indicated in Figure 21 (c)) generally have a higher IRI in the first year whereas the ARGG mixtures (Figure 21(a)) which are relatively stiffer compared to rest of the mixtures have lower IRI values. It should be noted that in general the IRI measurements are better correlated with rutting rather than cracking since even minor rutting results in deflections along the roadway while minor or medium cracking may not indicate high deflections. Therefore, it might be necessary to directly incorporate the cracking performance in the back-calculated layer coefficients from the field data.



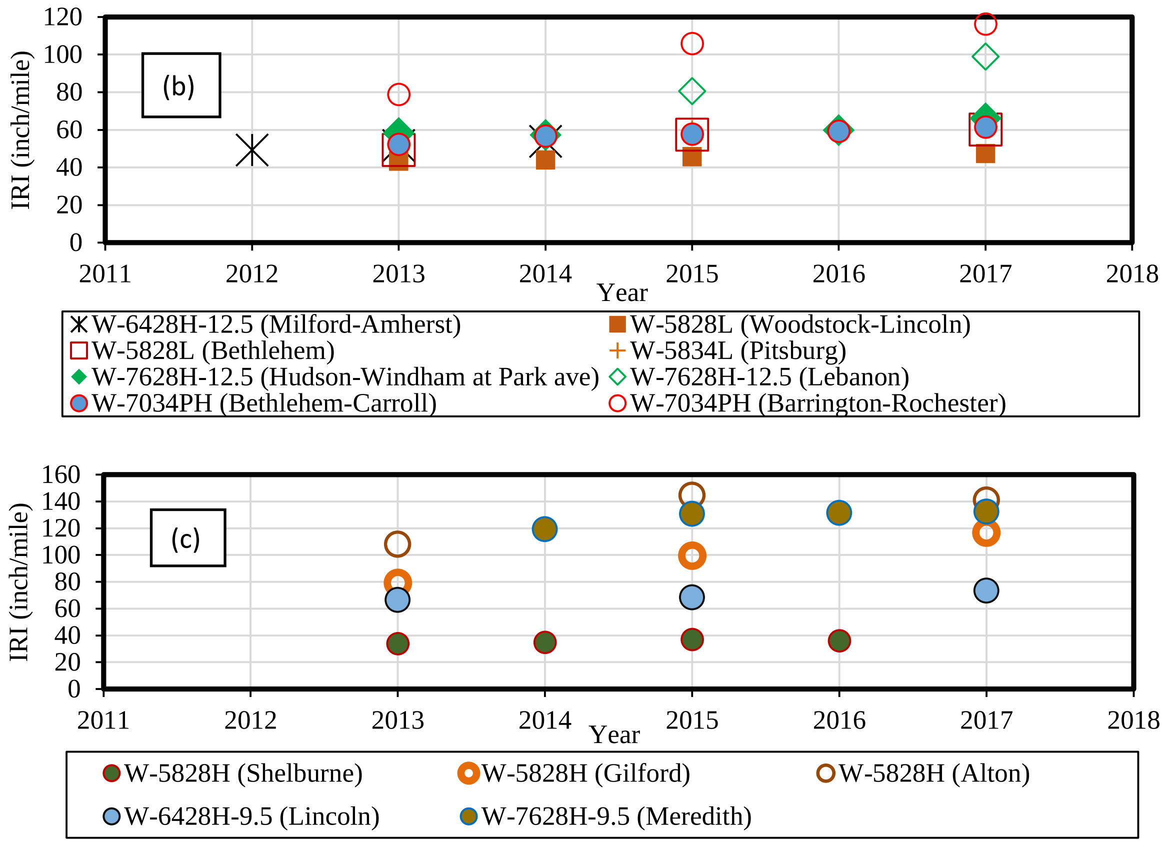


Figure 21. IRI versus time for different mixtures and projects: a) ARGG mixtures, b) 12.5mm NMAS mixtures, c) 9.5 mm NMAS mixtures.

10.4 Back-Calculation of Layer Coefficients from Field IRI measurements

In order to back-calculate the layer coefficients from the field IRI measurements, it is essential to have the cross section and traffic information of the road sections where mixtures similar to ones in this research are placed. The pavement management data provided by NHDOT included the original pavement cross sectional information as well as the method and thickness of the overlays using mixtures with similar characteristic to the study mixtures. The traffic information were gathered using the NHDOT online transportation data management system [31]. This GIS based online tool provides a comprehensive traffic information including the annual average daily traffic as well as truck percentage for different roadways within the state. After analyzing the traffic data, the total design traffic in terms of equivalent single axle load (ESALs) was calculated for each cross section for 20 years after reconstruction or major rehabilitation. Since not all the structural design information of the pavements were available, some general assumptions were made and applied to all the cross sections regardless of the type of road, to facilitate the back-calculation of layer coefficients through the AASHTO 1993 design equation. These assumption are summarized in Table 14. It should be pointed out that IRI provides the amount and severity of distresses of the surface layers only and there is no direct information about the type and magnitude of distress originating from the underlying layers, if any. However, with respect to general functionality of binder and base course asphalt mixtures and considering that these type of mixtures are usually used to improve the load bearing capacity of the pavement structure through improving the rutting susceptibility due to their relatively higher stiffness, it can be reasonably concluded that a stiffness based layer coefficient such as the ones determined through resilient modulus in Figure 20 can be used to indicate the structural contribution of such mixtures in the pavement. Therefore, in back-calculating the layer coefficients of wearing course mixtures from the field data, the layer coefficients of binder and base course mixtures will be based on resilient modulus only. However, the performance based a-values for these types of mixtures should be explored using the approach developed in this research. Through the process of back-calculations, the layer coefficients of the granular material are based on the values that are conventionally used by the NHDOT pavement design approach [32]. Also, the subgrade soil resilient modulus is a typical average modulus value in New Hampshire which is determined based on the research conducted by Janoo in 1994 [32]. The level of reliability is selected such that it includes almost all the roadway categories with respect to their functionality.

Table 14. General design assumptions to back-calculate a-values from field data

Design Reliability	Standard deviation	z-statistic	Actual ΔPSI	Resilient Modulus of the subgrade soil (psi)	Traffic (ESALs)	Layer coefficients for granular material	Layer coefficient for binder and base course asphalt mixtures
95%	0.45	-1.645	Varied among the sections based on back-calculations from Equation (10)	8000	Varied among the sections based on the location	Cold recycled mix=0.22 Crushed stone=0.14 Crushed gravel=0.10 Gravel=0.07 Sand=0.05	Back-calculated from resilient modulus from Equation (3)

Table 15 summarize the layer coefficients of the hot mixed binder and base course mixtures. However, since there are multiple types of binder course mixtures used in New Hampshire, each with a varying layer coefficient, it is important to determine what layer coefficient value should be used for these mixtures within a given pavement structure. For this reason, a binder performance grade categorizing based map (Figure 22) provided by the NHDOT was utilized to determine the proper binder course mixture for a specific project with respect to its location in the state of New Hampshire.

Table 15. Resilient modulus based layer coefficients of the hot mixed binder and base course mixtures

Mixture	B-6428H	B-5834L	B-5828H	BB-6428L
Layer coefficient	0.46	0.32	0.39	0.42

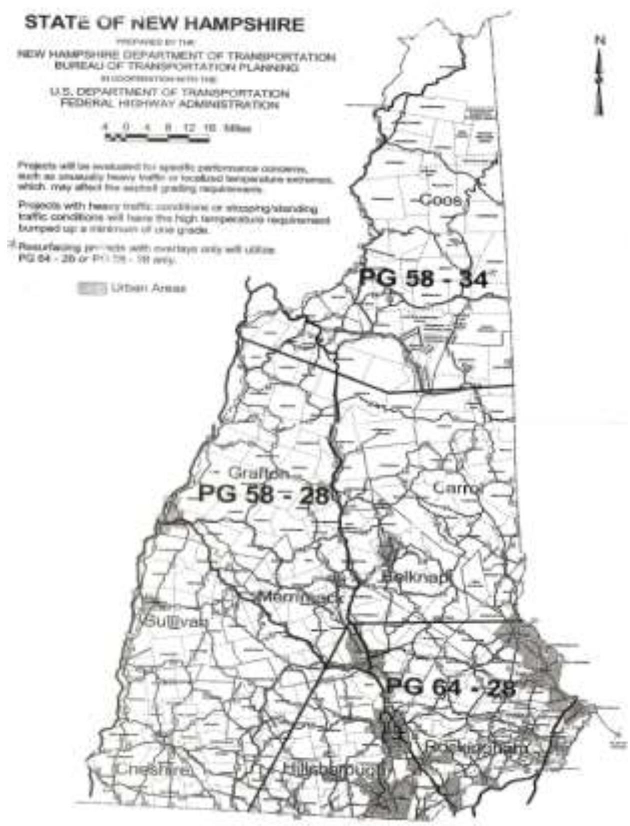


Figure 22. Binder performance grade specification map for New Hampshire

For a given cross section and a wearing course mixture, once all the required data in Table 14 is provided, the overall structural number ($SN_{overall}$) based on the back-calculated ΔPSI value can be determined using the AASHTO 1993 design equation. Then, the structural number of the granular material as well as other base and binder course asphalt mixtures can be simply determined through Equation 2 (Chapter 2) by using their thickness and layer coefficients. This structural number is related to all the non-wearing course material and is called $SN_{non\text{-}wearing}$. The structural number of the wearing course asphalt mixtures ($SN_{wearing}$) can be determined through Equation (14) and the layer coefficient of the wearing course mixtures can be determined using Equation (15).

$$SN_{wearing} = SN_{overall} - SN_{non\text{-}wearing} \qquad \text{Equation (14)}$$

$$a - value = \frac{SN_{wearing}}{Wearing\ course\ thickness} \qquad \text{Equation (15)}$$

The following flowchart (Figure 23) is used to summarize the procedure of back-calculation of wearing course layer coefficients. As it is shown in the flowchart, the first step is to fit a linear function to the distress measurements which will result in determining the initial (right after construction) and terminal (20 years after construction) IRI values. The conditions to determine the initial IRI value will be based on the magnitude of IRI in the first year after construction as described in Table 13. Once the initial and terminal IRI values are determined the initial and terminal serviceability values (P_i and P_t respectively) can be determined using either equation (12) or (13) depending on the units of IRI. Once these values are determined the ΔPSI as one of the inputs in AASHTO 1993 design equation can be determined. The other input variables are assumed based on the values provided in Table 14. The back-calculations from this equation will result in the overall structural number ($SN_{overall}$) value which includes all types of materials in the cross section. In order to determine the structural number of the non-wearing course materials, the layer coefficients of the granular and non-wearing course asphalt mixtures determined from Table 14 and Table 15 respectively. Ultimately, the layer coefficient of the wearing course is determined through dividing the $SN_{wearing}$ by the thickness of the layer.

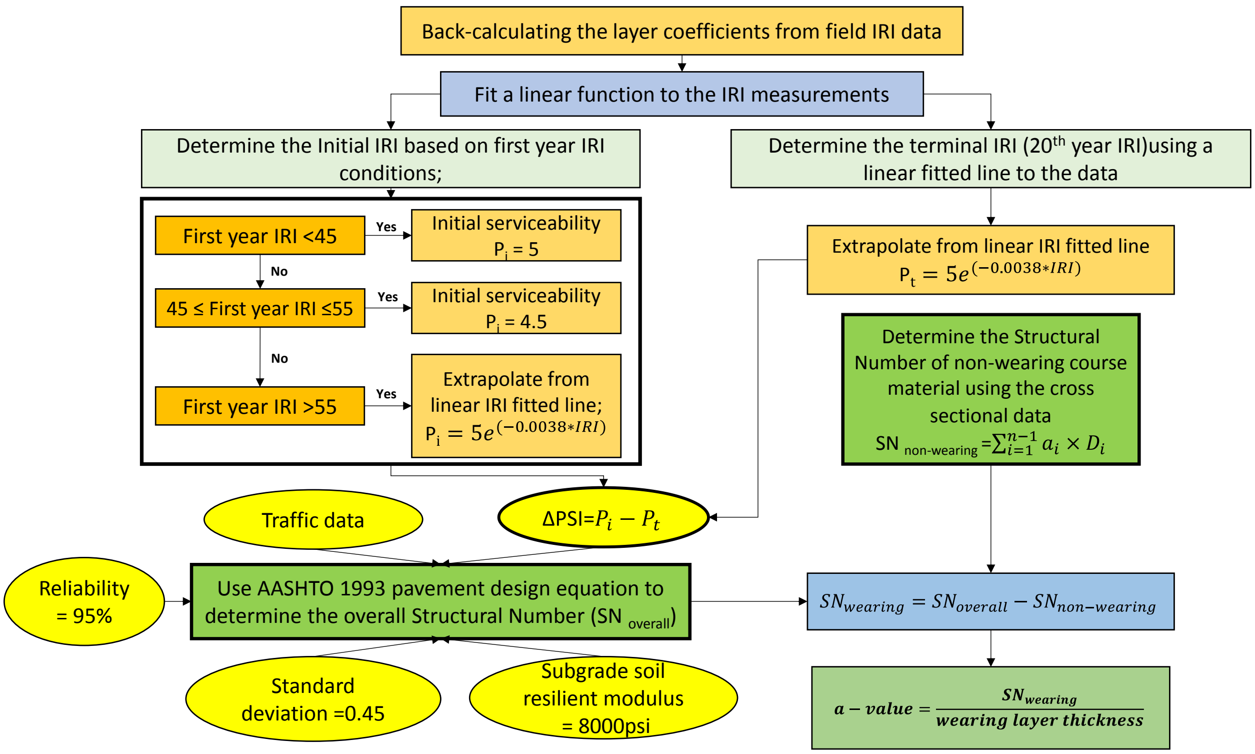


Figure 23. Flowchart to back-calculate the layer coefficients from field IRI data

Table 16 indicates the back-calculated wearing course layer coefficients from the field IRI data (a_{IRI}-value) using the aforementioned approach. According to the table, ARGG-2 has an extraordinary high a-value compared to rest of the mixtures. This mixture has previously been indicated (Appendix 1- Paper 1- Chapter 4) to have a higher modulus value compared to rest of the wearing course mixtures except for W-7628H-12.5 mixture. However, both ARGG mixtures in this study have much lower phase angles values compared to all others including the binder and base course mixtures. The high stiffness and flexibility (and simultaneously a low creep deformation potential) of these mixtures which is associated to the crumb rubber could have resulted in their outstanding performance considering the high traffic volume of the projects where they have been placed. In general, based on field IRI data, all of the mixtures with an average layer coefficient value of 0.58 are indicated to have a considerably higher layer coefficient compared to the original value of 0.38 that is currently being used by the NHDOT pavement design manual. Although in most cases, a similar mixture from different project sites (if applicable) have a relatively close a-values, there are some cases such as ARGG-2, W-5828L, and W7628H-12.5

where the a-values from different projects are significantly different. Closer inspection of the pavement sections in these cases indicated that this could be a result of an overly designed granular base or difference between the assumed subgrade soil resilient modulus and actual modulus value. Since the actual design data is not available for every single project, it is not possible to definitively conclude the effects of subgrade modulus. Due to presence of instances where a-values for a mix type varied significantly, a one tailed t-test was conducted and based on the average and standard deviation of the whole dataset (all mixtures and all pavement sections), layer coefficients at two levels of reliability for all wearing courses (85% and 90%) used by NHDOT are suggested for the pavement design purposes based on IRI data. Based on the statistical analysis the suggested layer coefficients are 0.43 and 0.39 for 85% and 90% reliability levels respectively. It is important to note that based on the laboratory as well as field performance of ARGG mixtures and considering their significantly different production methods, they can be reasonably separated from other wearing course mixtures. Table 17 indicates the average layer coefficients, as well as layer coefficients at different levels of reliability for ARGG and non-ARGG wearing course asphalt mixtures. The results from separating ARGG mixtures from the rest of the wearing course mixtures indicate that an a-value of 0.43 at 90% reliability level can be used for non-ARGG mixtures whereas this value would be 0.41 for ARGG mixtures due to the higher standard deviation in these types of mixtures.

Table 16. Back-calculated layer coefficients from the field IRI data

Mix	Project	Traffic (ESALs)	P_i	P_t	ΔPSI	$SN_{overall}$	$SN_{non\text{-}surface}$	$SN_{surface}$	a_{IRI}-value
ARGG-1	Epping	5,344,563	5.0	3.6	1.4	4.919	3.789	1.130	**0.56**
	Seabrook	3,741,194	5.0	3.3	1.7	4.431	3.610	0.821	**0.55**
ARGG-2	Bow-Hooksett	3,741,194	5.0	4.0	1.0	5.096	3.525	1.571	**1.05**
	Spaulding turnpike	3,343,302	5.0	3.7	1.3	4.764	3.692	1.072	**0.71**
W-6428H-12.5	Milford-Amherst (NH101)	4,489,433	5.0	3.5	1.5	4.701	4.014	0.687	**0.46**
W-5828L	Woodstock-Lincoln (I-93 NB)	823,063	5.0	3.9	1.1	3.903	2.713	1.190	**0.40**
	Bethlehem	411,531	4.5	3.4	1.1	3.350	2.490	0.860	**0.57**
W-5834L	Pittsburg (US3)	328,500	4.5	3.2	1.3	3.130	2.560	0.570	**0.46**
W-7628H-12.5	Hudson-Windham - (Park Ave)	1,624,854	4.5	3.5	1.0	4.570	2.547	2.023	**0.67**
	Lebanon NH120	2,338,169	3.7	1.9	1.8	4.080	3.202	0.878	**0.44**
W-7034PH	Barrington	213,783	3.8	1.9	1.9	2.758	2.288	0.470	**0.63**
	Bethlehem-Carroll (US 302)	619,969	4.5	3.5	1.0	3.756	3.288	0.468	**0.62**
W-7628H-9.5	Meredith (US 3)	1,389,586	3.9	1.9	2.0	3.681	3.081	0.600	**0.60**
W-5828H	Shelburne (US 2)	383,846	5.0	4.1	0.9	3.502	2.778	0.724	**0.48**
	Gilford	1,192,296	3.8	1.8	2.0	3.584	3.054	0.530	**0.53**
	Alton	794,049	3.5	1.7	1.8	3.443	2.913	0.530	**0.53**
W-6428H-9.5	Lincoln (NH 112)	300,471	4.1	3.4	0.7	3.526	2.913	0.613	**0.63**
General layer coefficient for all types of wearing course mixture	**Average a-value for all wearing course mixtures (50% reliability)**								**0.58**
	Standard deviation of the layer coefficients for all wearing course mixtures								**0.15**
	a_{IRI}-value at 85% reliability								**0.43**
	a_{IRI}-value at 90% reliability								**0.39**

Table 17. Layer coefficients at different reliability levels for ARGG and non-ARGG wearing course asphalt mixtures based on field IRI data

a_{IRI}-value at different reliability levels	ARGG wearing course mixtures	Non-ARGG wearing course mixtures
50% reliability	0.72	0.54
85% reliability	0.48	0.45
90% reliability	0.41	0.43

10.5 Incorporating the Laboratory Performance Test Results in Development of Layer Coefficients

Although a layer coefficient developed on the basis of field performance data (such as IRI) is expected to have greater reliability, it may not explicitly represent the structural contribution of the mixtures with respect to individual structural distress types such as rutting, fatigue and transverse cracking due to following reasons:

1- IRI includes functional distresses such as raveling, stripping, patching, potholes etc. which may not necessarily be related to structural deficiency of the asphalt mixtures.
2- With respect to structural distresses, IRI is more correlated to rutting than fatigue and transverse cracking, however actual pavement service lives are controlled by first dominant failure, which could be fatigue or transverse cracking.

Due to these reasons, it is necessary to incorporate the laboratory performance results in development and refining the field IRI based layer coefficients (a_{IRI}-value). For this purpose, the previously developed index parameters for rutting (Paper 4-chapter 7), transverse cracking (Paper 5-chapter 8) and fatigue cracking (chapter 9) will be statistically incorporated with the field IRI based a-values to develop mechanistically informed layer coefficients. To develop such layer coefficients, this research work undertook the following steps:

1- Determine the individual performance index parameter value for each mixture on basis of laboratory performance tests.
2- Determine the average and standard deviation of layer coefficients for all mixtures (including ARGG and non-ARGG wearing course mixtures) for each performance index parameter.
3- Using a normal distribution function, determine the z-statistic (number of standard deviations from average) and level of reliability of each mixture under each performance index category.
4- Using the average a-value and standard deviation of IRI based layer coefficients indicated in Table 16, calculate the layer coefficient for each mixture under each performance index category at the specific level of reliability that was determined in step 3.

5- Determine the performance based a-value by averaging the layer coefficients determined under individual performance index category for each mixture. It is acknowledged that in present work it is an assumption to use average of layer coefficients from each of the three primary structural distresses, future studies should further explore a weighted average approach that is based on the performance data of New Hampshire's highways that can provide details on the distress modes that control the service lives. For this reason, another set of layer coefficients combined of minimum performance based a-values for each mixture is generated for investigations.
6- Determine the average and standard deviation of the new sets of layer coefficients (both average and minimum layer coefficients).
7- Select the a-value at 90% reliability as the finalized layer coefficient.

Following the aforementioned steps, the levels of reliability and specific distress based layer coefficients as well as the averaged performance based incorporated a-values are determined and summarized in Table 18.

As expected, the ARGG mixtures have the highest reliability and consequently highest a-values with respect to rutting index while they are indicated to have the lowest reliabilities as well as a-values with respect to fatigue criterion. However, these mixtures reveal a medium range of reliability (between 30% and 50% reliability) with respect to transverse cracking. Among other mixtures, W-7628H-12.5 indicates a good rutting (between 50% to 70% reliability) and excellent fatigue (above 70% reliability) performance while having a medium performance with respect to transverse cracking. In general, the non-ARGG wearing course mixtures are shown to have medium to weak reliability (below 50% reliability) with respect to rutting and transverse cracking while having a good performance with respect to fatigue cracking. An averaged a-values (a_{ave}-value) from each performance index was determined for each mixture without assigning any weight factor to individual a-values driven from specific distresses since at the present time it is assumed that all three distresses are equally important in the overall performance of a given mixture within the pavement structure. According to the analysis, a layer coefficient equal to 0.50 at the reliability level of 90% can be used for all wearing course mixtures. The table also includes a minimum individual performance incorporated a-value (a_{min}-value) which essentially uses the minimum of the three a-values determined from each index parameter (bolded in each individual performance based a-value column). Based on the minimum layer coefficients an a-value of 0.39 at 90% reliability level is determined. This conservative a-value results in a thickness design that covers all the three distresses at the same time. It should be noted that due to the limited number of ARGG mixtures, separation of these mixtures from the rest of the wearing course mixtures may not be statistically appropriate at this point in the analysis. In other words, determination of a separate average index value for the two ARGG mixtures will result in constant reliability levels of 24% and 76% for every single index parameter which may not be realistic and appropriate.

Table 18. Development of averaged performance base incorporated a-values for the study mixtures

Mixture	Rutting			Fatigue cracking			Transverse Cracking			(a_{ave}-value)	(a_{min}-value)
	Index value (CMRI)	Reliability (%)	a-value	Index value ($N_f@C_{N_f}^s \times 1000 = 100$)	Reliability (%)	a-value	Index value (RDCI)	Reliability (%)	a-value		
ARGG-1	407.2	70	0.66	15764	29	**0.50**	33.64	49	0.58	0.58	0.50
ARGG-2	905.9	99	0.97	5523	5	**0.34**	29.52	36	0.53	0.61	0.34
W-6428H-12.5	194.2	37	0.53	12563	19	**0.45**	28.08	32	0.51	0.50	0.45
W-5828L	190.6	36	0.53	18985	42	0.55	24.05	20	**0.46**	0.51	0.46
W-5834L	138.1	29	**0.50**	29611	82	0.72	32.71	46	0.57	0.59	0.50
W-7628H-12.5	353.6	62	0.63	40069	98	0.89	30.97	41	**0.55**	0.69	0.55
W-7034PH	121.2	26	**0.49**	20301	48	0.57	65.72	99	0.97	0.68	0.49
W-7628H-9.5	180.0	36	**0.52**	21815	53	0.60	36.03	57	0.61	0.58	0.52
W-5828H	171.9	33	0.52	22104	55	0.60	22.36	18	**0.44**	0.52	0.44
W-6428H-9.5	118.5	26	**0.48**	22237	56	0.60	35.10	53	0.60	0.56	0.48
Average										**0.58**	**0.47**
Standard deviation										**0.06**	**0.06**
Layer coefficients at 90% reliability										**0.50**	**0.39**

10.6 Correlation of the Layer Coefficients with Mixture Properties

As a summary up to the current point of this chapter three different types of layer coefficients are developed:

1- a_{IRI}-value: determined through the back-calculations of the filed IRI measurements
2- a_{ave}-value: determined through averaging the performance incorporated a-values
3- a_{min}-value: determined through the minimum performance incorporated a-value for each mixture

In order to determine which a-value is more reasonable for the purpose of application in the pavement design, it is necessary to evaluate the strength of correlations between the a-values, mixture properties and performance index parameters. For this reason, the Pearson's correlation coefficient is selected to be used to assess the correlations between these three components. In general the final selected a-value needs to indicate a positive correlation with the performance index parameters while it should maintain a reasonable correlation (either positive or negative) with the mixture properties.

Table 19 and Table 20 indicate the Pearson's correlation matrix between the mixture properties and performance indices along with the various a-values. As mentioned before, ARGG mixtures have significantly different mix properties and performance and for that reason and in order to better discriminate and evaluate the effect of mixture properties on a-values, separate tables were generated. Table 19 includes all the wearing course mixtures, and the mixture properties include the performance grade high temperature (PGHT), performance grade low temperature (PGLT), performance grade useful temperature interval (UTI), nominal maximum aggregate size (NMAS), asphalt content (AC), level of gyration and recycled asphalt pavement (RAP). The main noteworthy observations with respect to the highlighted cells of Table 19 are as follows:

1- While a_{IRI}-value does not reveal any significant correlations with the binder grade properties including PGHT, PGLT and UTI or the aggregate size, the a_{ave}-value indicates much higher sensitivity to these properties. However, this observation is reverse for a property such as AC as a_{IRI}-value indicates a relatively high positive correlation with this property. On the other hand, a_{min}-value indicates similar correlations to that of the a_{ave}-value with respect to binder grade properties while unlike the a_{IRI}-value, it is negatively correlated with the AC. Another important observation is the insensitivity of a_{ave}-value to the AC which may be a result of existence of the ARGG mixtures with relatively higher binder contents in the dataset while their individual a_{ave}-value is notably close to the overall average a_{ave}-value which neutralizes the effect of binder, considering that the rest of the mixtures have close asphalt contents in general.
2- With regards to correlation of different types of a-values with the developed index parameters in this research it is shown that the a_{IRI}-value has an expected high positive correlation to the rutting index due to the relatively high correlation between the IRI and rutting distresses in general. However, a_{IRI}-value is negatively proportional to the fatigue index. This observation confirms that a_{IRI}-value needs to be refined through incorporating the lab performance tests in the analysis and as a result the a_{ave}-value is positively correlated to all three of the performance index parameters. On the other hand, a_{min}-value reveals a reverse trend as compared to a_{IRI}-value when considering the correlations to rutting and fatigue indices. Since there is negative correlation between fatigue and rutting index parameters, the a_{IRI}-value and a_{min}-value indicate negative correlations with respect to

fatigue and rutting indices. This is primarily associated to the ARGG mixtures' performance with respect to these types of distresses in the lab.

3- As a general conclusion with respect to Table 18 and Table 19 due to the aforementioned results and discussions on the correlations, the a_{ave}-value is probably a better tool to indicate the structural contribution of the mixtures when all types of mixtures (ARGG and non-ARGG) are included in the analysis. However, a_{min}-value is a more conservative selection which may also increase the initial construction costs while it will probably result in less pavement maintenance and rehabilitation costs.

Table 19. Correlation matrix including ARGG mixtures

Variables	PGHT	PGLT	UTI	NMAS	AC	Gyration	RAP	Rutting index	Fatigue index	Transverse Cracking index	a_{IRI}-value	a_{ave}-value	a_{min}-value
PGHT	1.0	-	-	-	-	-	-	-	-	-	-	-	-
PGLT	0.00	1.0	-	-	-	-	-	-	-	-	-	-	-
UTI	0.95	-0.32	1.0	-	-	-	-	-	-	-	-	-	-
NMAS	-0.18	-0.33	-0.07	1.0	-	-	-	-	-	-	-	-	-
AC	-0.38	0.34	-0.47	0.02	1.0	-	-	-	-	-	-	-	-
Gyration	0.42	0.38	0.28	-0.33	0.34	1.0	-	-	-	-	-	-	-
RAP	0.09	0.11	0.05	0.04	-0.67	-0.40	1.0	-	-	-	-	-	-
Rutting index	-0.23	0.32	-0.32	0.35	0.69	0.25	-0.18	1.0	-	-	-	-	-
Fatigue index	0.50	-0.23	0.54	-0.09	-0.61	-0.19	0.38	-0.47	1.0	-	-	-	-
Transverse Cracking index	0.42	-0.67	0.62	0.15	-0.04	0.24	-0.58	-0.22	0.04	1.0	-	-	-
a_{IRI}-value	**0.03**	**0.14**	**-0.02**	**-0.03**	**0.65**	**0.44**	**-0.52**	**0.77**	**-0.47**	**0.21**	1.0	-	-
a_{ave}-value	**0.56**	**-0.43**	**0.67**	**0.30**	**0.02**	**0.26**	**-0.29**	**0.23**	**0.43**	**0.64**	0.39	1.0	
a_{min}-value	**0.61**	**-0.20**	**0.64**	**-0.08**	**-0.45**	**-0.06**	**0.13**	**-0.64**	**0.80**	**0.26**	-0.59	0.33	1.0

Table 20. Correlation matrix including only non-ARGG mixtures

Variables	PGHT	PGLT	UTI	NMAS	AC	Gyration	RAP	Rutting index	Fatigue index	Transverse Cracking index	a_{IRI}-value	a_{ave}-value	a_{min}-value
PGHT	1.0												
PGLT	0.12	1.0											
UTI	0.94	-0.24	1.0										
NMAS	-0.05	-0.45	0.11	1.0									
AC	0.05	0.30	-0.06	-0.81	1.0								
Gyration	0.60	0.33	0.47	-0.45	0.30	1.0							
RAP	-0.15	0.28	-0.25	0.24	-0.65	-0.34	1.0						
Rutting index	0.48	0.44	0.31	0.29	-0.50	0.16	0.56	1.0					
Fatigue index	0.36	-0.11	0.39	0.14	-0.29	-0.06	0.22	0.63	1.0				
Transverse Cracking index	0.43	-0.67	0.66	0.20	0.12	0.27	-0.72	-0.35	-0.05	1.0			
a_{IRI}-value	**0.61**	**-0.01**	**0.61**	**-0.48**	**0.71**	**0.60**	**-0.80**	**-0.18**	**0.10**	**0.63**	1.0		
a_{ave}-value	**0.67**	**-0.47**	**0.83**	**0.29**	**-0.18**	**0.24**	**-0.31**	**0.33**	**0.68**	**0.67**	0.52	1.0	
a_{min}-value	**0.78**	**-0.15**	**0.81**	**0.14**	**-0.16**	**0.10**	**0.03**	**0.53**	**0.78**	**0.28**	0.39	0.8	1.0

In context of correlations for wearing course mixture without ARGG in the population (shown in Table 20), following key observations can be made:

1- All three a-value types (a_{IRI}, a_{ave} and a_{min}) indicate similar correlations regarding the binder grading properties with exception of a_{min}-value which is nearly insensitive to the PGLT. The table indicates that the a_{ave}-value and a_{min}-value have uncommon correlation directions with NMAS and AC such that mixtures with larger aggregate size and lower asphalt content have higher a-values. However, the prevailing effect of binder type and properties which results in a relatively better of these mixture should be taken into consideration.
2- The elimination of the ARGG mixtures from the database resulted in improved correlations with respect to direction and magnitude of the correlations between all three types of a-values with fatigue and cracking index. Also, both of the a_{ave}-value and a_{min}-value indicate higher positive correlations with the rutting index for non-ARGG mixtures. However, the a_{IRI}-value indicates a negative correlation with the rutting index which reaffirms the necessity of incorporating performance based indices in development of a reliable a-value.
3- As a general conclusion with respect to Table 18 and Table 20, considering the previous discussions, both of the a_{ave}-value and a_{min}-value based layer coefficients can be reasonably used for the pavement structural design and the selection of the proper a-value should be based on the level of importance and reliability of a specific project.

Based on the discussions in this section a summary for the a_{ave}-value and a_{min}-value for non-ARGG mixtures is provided in Table 21.

Table 21. a_{ave}-value and a_{min}-value for non-ARGG mixtures

a-value	Level of reliability	Non-ARGG wearing course mixtures
a_{ave}-value	50% reliability	0.58
	90% reliability	0.48
a_{min}-value	50% reliability	0.49
	90% reliability	0.44

10.7 Mixture Property Based Predictive Model for Layer Coefficients

In many instances and due to the limitations in time and laboratory equipment as well as the unavailability of field distress data, it may not be possible to determine an accurate performance incorporated layer coefficient for the pavement design. Therefore, a predictive model based on nominal properties of the mixtures can help mixture and pavement design engineers to have an acceptable level of estimation of the structural contribution of the mixtures within the pavement structure. For this reason, the database created in this thesis in terms of mixture properties and layer coefficients are used to develop a simple model to predict the performance incorporated layer coefficients (a_{ave}-value) for all the wearing course mixtures including ARGG and non-ARGG mixtures used in New Hampshire.

In order to develop the model, the nominal mixture properties including PGHT, PGLT, NMAS, AC, gyration level, and RAP content were selected as the initial variables in the model. These parameters are readily available before the mixture design and can help the design engineers with an initial estimation of the structural contribution of the mixture in the pavement system. A second degree factorial variable selection was performed to determine the significance of possible two-way interactions in the model. To build the model, the stepwise regression analysis was utilized to determine the influential mixture properties and two-way interactions on the a_{ave}-value at 0.25 significance level which is equal to 75% confidence interval. This level of significance was determined through performing trial and error efforts to determine what level can result in the best possible fit for the available data. However, it should be noted that this level of significance can change based on the level of accuracy and importance of the project. Table 21 indicates the terms and resultant statistics of the developed model. The lower p value and higher t-ratio indicate the significance of the variable in the prediction equation. As it can be seen the PGHT with the lowest p-value and the NMAS with the highest p-value are the most and least significant variables in the models. Although the p-value for NMAS is higher than the threshold value of 0.25, its interaction with the PGHT is a significant and for that reason NMAS is kept in the model. The results from the prediction are indicated in Figure 24. A strong correlation close to the line of equality is achieved through the developed model. In using any predictive equation including the one provided in this section, it is important to consider the constraints of the variables in terms of the input value as not every single value may result in a reasonable prediction. Also, as this predictive equation results in the a_{ave}-value (50% reliability level) it would be necessary to increase the level of reliability by calculating the layer coefficient at an increased reliability level by using a standard deviation of 0.06 (determined from Table 22) depending on the importance of a given design project.

Table 22. Prediction model for a_{ave}-value

Term	Statistics			
	Estimate	Standard Error	t-Ratio	Prob>|t|
Intercept	-1.09659	0.25611	-4.28	0.0505
PGHT	0.011663	0.00177	6.61	0.0222
PGLT	-0.017115	0.00415	-4.13	0.054
NMAS (mm)	0.000691	0.00618	0.11	0.9212
AC%	0.100468	0.02156	4.66	0.0431
Gyration level	-0.003087	0.00133	-2.33	0.1453
RAP%	0.002695	0.00158	1.71	0.2294
(PGHT-64)*(NMAS-11.6)	0.004462	0.00103	4.33	0.0494

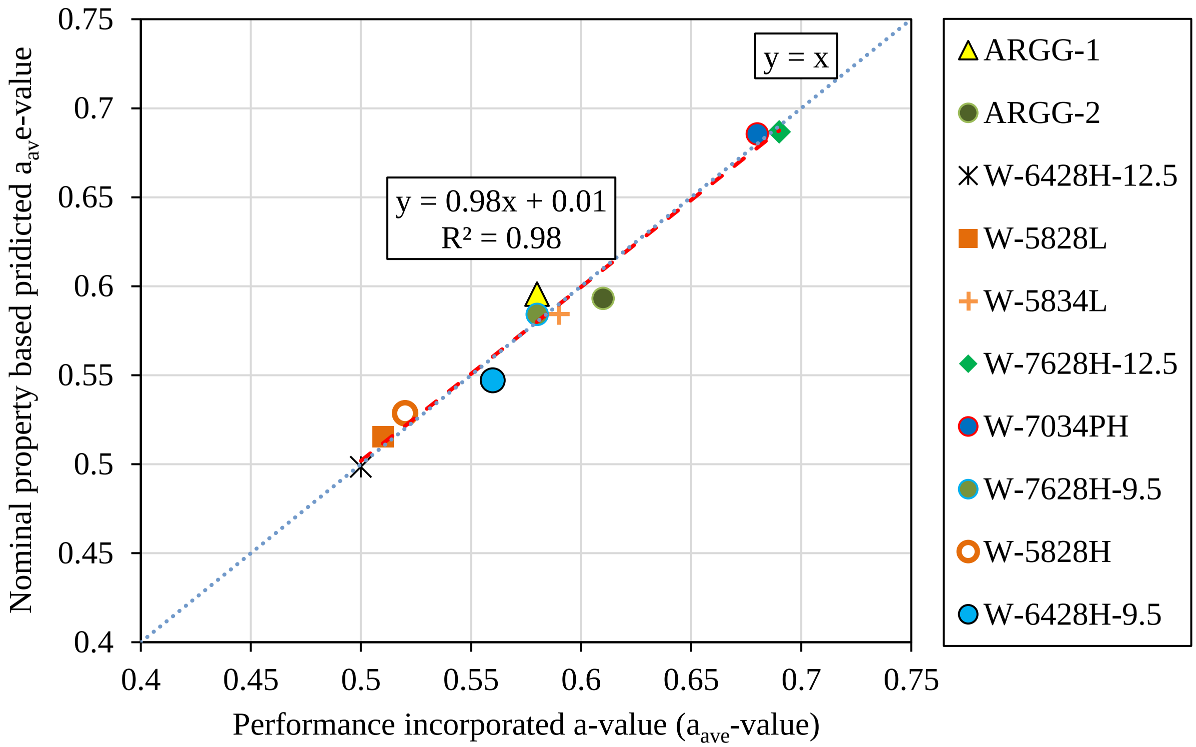


Figure 24. Nominal property based predicted a_{ave}-value for all the study surface mixtures

10.8 Summary, Conclusion and Future Work

This chapter of the thesis aimed on combining the developed rutting, fatigue and transverse cracking index parameters (chapters 7, 8 and 9 respectively) to evaluate the structural contribution of wearing course mixtures (Table 2) in New Hampshire highways. Since many state highway agencies including New Hampshire are still using the AASHTO 1993 empirical equation as their primary pavement design approach, this method was selected to evaluate the mixtures in this research. The AAHTO 1993 design equation uses the structural layer coefficients (a-values) to quantify the material's properties and their contribution to the pavement load bearing capacity. Therefore, a methodology was developed to update the layer coefficient values for the wearing course mixtures used in New Hampshire. The procedure of development and evaluation of layer coefficients included investigating the field distress data of 17 cross sections that have been constructed with the study mixtures that are shown in Table 2. The main evaluated distress data was selected to be the International Roughness Index (IRI) as it is a standardized distress index parameter and can be an indicator of the combined effect of multiple distress types such as rutting and cracking. Also through the available equations in the literature, it would be feasible to convert IRI to present serviceability index (PSI) that is one of the primary inputs in the AASHTO 1993 design equation. For each cross section, a back-calculation analysis was conducted to determine the required structural number ($SN_{overall}$) of the pavement in accordance to traffic level and IRI measurements. On the other hand, another structural number ($SN_{non\text{-}wearing}$) based on the available cross sections was determined. The difference between these two types of structural numbers results in the structural number of the wearing course mixtures which when divided by the thickness of the wearing course results in the back-calculated layer coefficients of the mixtures based on IRI data which is denoted by the a_{IRI}-value. Using a standard normal distribution function the aver gage and standard deviation of the a_{IRI}-value was calculated for the whole dataset. Since the a_{IRI}-value is primarily based on IRI analysis it is necessary to incorporate other types of performance based test results in the back-calculated layer coefficients. For this reason, the three performance index parameters including rutting (paper 4 chapter 7), transverse cracking (paper 5 chapter 8) and fatigue (chapter 9) were calculated for each mixture separately and their level of reliability was determined through a standard normal distribution for each index parameter. Using these levels of reliabilities along with the average and standard deviation of the a_{IRI}-value, three individual performance index incorporated a-values were determined for each mixture. The average of the three layer coefficients resulted in the a_{ave}-value for each mixture. Also using the minimum of three performance incorporated layer coefficients for each mixture a set of a_{min}-values were determined for the study mixtures.

In order to determine which type of a-value (a_{IRI}-value, a_{ave}-value and a_{min}-value) can best represent the structural contribution of the mixtures, the Pearson's correlation matrix was used to examine the strength and reasonability of the correlations between the mixture properties and index parameters with the three types of layer coefficients. The analysis indicated that a_{ave}-value for all types of mixtures can be a better tool for the pavement design purposes. However, the a_{min}-value can cover all distress types as it will result in the highest thickness value. The choice of selection will be primarily based on the importance level of a given project. Finally, a predictive model based on the nominal properties of the mixtures was developed to facilitate determination of a relatively accurate a_{ave}-value for the study mixtures.

As a future work in this study, using the proposed methodology in this research the layer coefficients of the binder and base mixtures including the cold recycled mixtures should be developed and evaluated on basis of the field distress data.

11. Summary, Conclusion, Recommendations and Future Extensions

11.1 Summary

With advancements in laboratory testing equipment and scientific theories to capture constitutive behavior of asphalt mixtures, the performance based material characterization and pavement design is becoming reality. These approaches as preferred as they can result in major savings through prolonging of pavement service lives. Therefore, it is necessary to develop appropriate testing and performance index parameters to determine the mixtures' distress susceptibility. The performance index parameters should not only be able to differentiate the mixtures in lab, but they also need to provide acceptable correlations with the field distress data.

Perhaps, one major issue in pavement industry is the gap between the mixture and pavement design. Since different asphalt mixtures have a wide range of variety with respect to their nominal properties and production methods, they can perform very differently under comparable loading and climatic conditions. For this reason, in many instances the pavements' failures are primarily associated with the improper design due to lack of knowledge about the structural contribution and performance of different mixtures within the pavement system, this is especially true when the design is based on empirical approaches. In this doctoral thesis the aim has been to address part of the stated knowledge gaps through conducting research on different aspects associated with pavement structure contributions and performance of asphalt mixtures.

In order to fulfill this aim, 18 different types of asphalt mixtures including asphalt rubber gap graded, cold recycled as well as other types of conventional and polymer modified mixtures that are commonly used in New Hampshire were selected for investigation. The laboratory testing plan included resilient modulus (M_r), complex modulus (E*), direct tension cyclic fatigue (S-VECD), semi-circular bend (SCB) and disk-shaped compact tension (DCT) tests. The results of the tests and analysis were utilized to evaluate the effect of mixture design properties on performance prediction indices through statistical analysis, correlations between different performance index parameters from each test were also conducted. Furthermore, the effect of production methods (cold versus hot) was evaluated through predicted performances from mechanistic approaches. The correlations of the existing performance index parameters were compared to available field distress data for the study mixtures to evaluate the strength of correlations between the field and laboratory predicted performances. On the basis of the observations from these correlations, it was determined that there is need to develop new index parameters that can better reflect the field performance for New Hampshire roadways. Therefore, three index parameters for rutting (based on complex modulus, transverse cracking (based on SCB) and fatigue cracking (based on S-VECD) were developed. The rutting and fatigue index parameters were shown to be highly correlated with the field distress data while the transverse cracking index was shown to be able to reduce the variation among results from different replicates. Finally, the new index parameters and field performance data were utilized in development of performance incorporated layer coefficients that are proposed to be used in the AASHTO 1993 pavement design approach.

11.2 Conclusions

Over the course of this doctorate research a number of significant findings were inferred. Specific conclusions and findings from each component of research are discussed in individual chapters. A high-level summary of key conclusions from the research efforts are as following:

- A generalized regression based approach is well suited to predict mechanistic properties of asphalt mixtures using nominal mix attributes. A generalized regression based model has been developed for complex modulus in this research and it has shown to have better prediction capability than existing models, including those that require some lab measured inputs.
- The performance of cold recycled (CR) mixtures as per NHDOT specifications appears to be more influenced by the RAP source, RAP binder properties and, emulsion type and content as compared to the gradation of RAP. The S-VECD fatigue results using the D^R failure criterion indicate that the CR mixtures would be expected to have better fatigue resistance than the HMA mixtures. However, the lower stiffness of CR mixtures, as measured by the resilient modulus, indicates that they may have more susceptibility to rutting as compared to the HMA mixtures.
- Since rutting is a viscoplastic distress, the phase angle plays an important role in mechanism of rutting formation. Therefore, in this research a complex modulus based rutting index parameter is developed which takes into account the effect of phase angle and stiffness at the same time. This index is indicated to be highly correlated with the field data as well as Hamburg wheel tracking test (HWTT) results for different mixtures in New Hampshire. This index parameter can also help reducing the specimen fabrication and testing time required to conduct a separate destructive testing such as HWTT, flow number etc.
- The development of a rate dependent cracking index (RDCI) in this research is supported by fundamental fracture mechanics, it is free from any type of empirical or undefinable variable within the parameter. The use of continuous cumulative work at various times can help with describing and evaluating the crack formation and propagation mechanisms at any given time during the test. Due to inherent presence of time in all work and power terms of RDCI; it is expected to better capture the rate dependency of fracture in asphalt mixture.
- The current fatigue failure criteria within S-VECD analysis framework did not indicate to be able to reliably rank the field performance of the mixtures as a stand-alone parameter. Therefore, a new failure criterion called as $C_{N_f}^{s}$was developed and investigated. This criterion incorporates three components of the S-VECD theory (cumulative pseudo-strain, number of cycles to failure at strain level and amount of continuum damage in material at failure) to capture the mixture's disintegration with respect to damage growth rate. The parameter indicated that it is not only able to rank the mixtures but it also well correlated with the magnitude of fatigue cracking in the field.
- The application of performance index parameters has mostly been limited to differentiating the mixtures' lab performance as well as performance based mixture design approaches. However, the use of these indices to influence the pavement design has been limited. The new layer coefficients developed in this research incorporate the lab measured performance indices and reflect the expected performance of asphalt mixtures in the field. Thus, pavement structures designed with the proposed layer coefficients are expected to have

higher reliability in terms of performance and service life. This will result in reduced initial construction and routine maintenance costs since the pavements structure is designed based on reliable performance parameters that the pavement deterioration time can be estimated on basis of them.

11.3 Recommendations

Asphalt mixtures are complicated viscoelastic materials that exhibit significantly different performance under varying climatic and loading circumstances. Due to these attributes, the mixtures may undergo different types of distresses such as rutting, fatigue and transverse cracking each with a specific mechanism of initiation and evolution. Therefore, the mixture's performance need to be evaluated through appropriate testing method and performance index parameters. The performance index parameters when correlated to the mixture design properties and volumetrics, can provide useful information for the mix design engineers which can ultimately lead to mixtures that are able to withstand competing distresses such as rutting and cracking at the same time. In addition, implementing performance based testing and index parameters in mixture design can result in prolonged pavement service life, reduced costs of maintenance and rehabilitation, safer highways to accommodate more traffic and finally environmental friendly pavements with smaller greenhouse gas emission footprint.

In order to reduce the construction costs and air pollutions, different types of materials, mixture production and construction techniques such as rubberized binders, cold recycling, warm mix technology etc. have been used in the last few decades which need to be evaluated through performance based testing methods before extensive productions of those mixtures. However, in many instances, due to the limitations in time, cost, laboratory equipment as well as experienced technicians, it may not be possible to determine the mixtures' performance properties through lab testing. For this reason, predictive models that are developed based on large datasets can be used as a tool to help in preliminary mixture screening procedure. These models, if developed to predict the mixtures viscoelastic properties such as, dynamic modulus and phase angle (complex modulus), can also be utilized in mechanistic-empirical pavement design approach to predict the magnitude and evolution of distress over time. In addition, complex modulus master-curves provide useful information about different mixture properties and can be used as an initial tool to predict the mixtures distress susceptibility potential. With respect to thermal cracking the high frequency portion, with respect to fatigue cracking the middle portion of the master-curves and for rutting the lower portion of the master-curves can be used to screen the mixtures.

Ultimately, any performance index parameter or failure criterion should have an acceptable correlation with the field distress data. A well correlated performance index parameter not only can be used in mixture screening phase, but it can also be utilized in pavement structural design system to help calibrating the design inputs in an empirical design approach such as the layer coefficients in AASHTO 1993 approach or in a mechanistic-empirical design approach by calibrating the coefficients of the transfer functions. Therefore, more reliable pavement structures can be designed which result in longevity of transportation infrastructures. Thus, it is necessary to evaluate the structural contribution of asphalt mixtures through reliable performance index parameters.

11.4 Future Extensions

The research study conducted in this doctoral thesis can be further extended in different areas of asphalt mixtures science. Following are some examples of the future works that are categorized in different areas of application in this thesis and need to be conducted as a future expansion to different portions of this research;

- **Additional material characterization**

The mechanistic performance of cold recycled mixtures can be further examined using different types of stabilizers such as lime and cement. In addition, the effect of curing time needs to be evaluated in the cold recycled mixtures' performance for different types of emulsions. Development of a mechanistic testing incorporated mixture design approach for the cold recycled mixtures can improve the properties of mixtures and facilitate development of predictions models for such mixtures.

The complex modulus predictive models can be further improved by addition of more mixtures to the dataset with even more varying types of binders and aggregate geologies. With expanding datasets, the predictive models can be categorized with respect to different types of binders or different types of courses to increase the reliability of the models.

- **Index development**

In this research the correlations between the developed rutting index parameter and Hamburg wheel tracking (HWTT) test was evaluated for mixtures used in New England area. Since the temperature conditions of HWTT is varying among different climatic conditions, it would be necessary to adjust the selection of HWTT based on local conditions.

The rate dependent cracking index parameter is applicable for any type of monotonic fracture testing including low temperature cracking tests such as disk-shaped compact tension test, however, the results need to be further investigated through correlations to the field distress data.

The fatigue failure criterion although indicated reliable results as a standalone parameter with respect to the available field distress data, it needs to be further evaluated for other types of mixtures and varying cross sections. An appropriate damage factor for this criterion needs to be developed so that this parameter can be utilized as an alternative to other available fatigue failure criteria in software such as FlexPAVE^{TM}. As another improvement to this failure criteria, a level of strain obtained from pavement structural analysis can be added as coefficient to the parameter so that it can better discriminate the field performance of different cross sections.

- **Layer coefficient development**

The methodology of development of layer coefficients can be further improved by figuring out the appropriate weighing system to different types of distresses as some distresses are more correlated to the rutting and this may induce bias in the average and minimum layer coefficients.

Although resilient modulus based layer coefficients for binder and base course mixtures were calculated and determined in this study, the framework developed in this research can be applied to develop performance incorporated layer coefficients for these types of mixtures. In order to achieve this goal, the developed mixture specific layer coefficients for the wearing course mixtures can be used to back-calculate the layer coefficients of the base and binder mixtures. Then individual performance based a-values for these mixtures can be calculated and compared to the back-calculated a-values to determine the prevailing parameter on different mixtures. As a result, the performance incorporated a-values can be determined by using either only one or more parameters that contribute to the distresses in these type of mixtures.

12. References

[1]. Huang, Yang H. Pavement design and analysis. Pearson/Prentice Hall, 2004.

[2]. Transportation Officials. AASHTO guide for design of pavement structures, 1993. Vol. 1. AASHTO, 1993.

[3]. Davis, Kendra, and David Timm. "Recalibration of the asphalt layer coefficient." Journal of Transportation Engineering 137.1 (2010): 22-27.

[4]. Van, Wijk Adriaan. "Determination of the Structural Coefficients of a Foamed Asphalt Recycled Layer." (1984).

[5]. Timm, David H., et al. Recalibration Procedures for the Structural Asphalt Layer Coefficient in the 1993 AASHTO Pavement Design Guide. No. NCAT Report 14-08. 2014.

[6]. Bahia, Hussain U. Layer coefficients for new and reprocessed asphaltic mixes. Wisconsin Department of Transportation, Division of Transportation Infrastructure Development, Bureau of Highway Construction, Technology Advancement Unit, 2000.

[7]. Prowell, Brian D., et al. "Evaluation of Moduli and Structural Coefficients of South Carolina's Asphalt Base Mixtures." Transportation Research Record: Journal of the Transportation Research Board 2641 (2017): 21-28.

[8]. Janoo, Vincent C. Layer Coefficients for NHDOT Pavement Materials. No. CRREL-SP-94-30. Cold Regions Research and Engineering Lab Hanover NH, 1994.

[9]. ASTM D7369-11: Standard Test Method for Determining the Resilient Modulus of Bituminous Mixtures by Indirect Tension Test, ASTM International, West Conshohocken, PA, 2011.

[10]. AASHTO T 342, Standard method of test for determining dynamic modulus of hot-mix asphalt concrete mixtures, Standard Specifications for Transportation Materials and Methods of Sampling and Testing, (2011).

[11]. AASHTO TP 124-16. (2016). Standard method of test for determining the fracture potential of asphalt mixtures using semicircular bend geometry (SCB) at intermediate temperature. Washington, DC: American Association of State Highway and Transportation Officials.

[12]. Ozer, Hasan, et al. "Development of the fracture-based flexibility index for asphalt concrete cracking potential using modified semi-circle bending test parameters." Construction and Building Materials 115 (2016): 390-401.

[13]. AASHTO TP 107: Determining the Damage Characteristic Curve of Asphalt Concrete from Direct Tension Cyclic Fatigue Tests, American Association of State and Highway Transportation Officials, Washington, D.C., 2004.

[14]. Zhang, Jun, et al. "Development of a failure criterion for asphalt mixtures under fatigue loading." Road Materials and Pavement Design 14.sup2 (2013): 1-15.

[15]. Wang, Yizhuang, and Y. Richard Kim. "Development of a pseudo strain energy-based fatigue failure criterion for asphalt mixtures." International Journal of Pavement Engineering (2017): 1-11.

[16]. Wang, Y. D., Keshavarzi, B., & Kim, Y. R. (2018). Fatigue Performance Analysis of Pavements with RAP Using Viscoelastic Continuum Damage Theory. *KSCE Journal of Civil Engineering*, *22*(6), 2118-2125.

[17]. ASTM D7313–06. "Standard test method for determining fracture energy of asphalt-aggregate mixtures using the disk-shaped compact tension geometry." ASTM International, April (2007).

[18]. Zhu, Yuefeng, et al. "Comprehensive evaluation of low-temperature fracture indices for asphalt mixtures." Road Materials and Pavement Design 18.sup4 (2017): 467-490.

[19]. Babadopulos, Lucas Feitosa de Albuquerque Lima, Jorge Barbosa Soares, and Verônica Teixeira Franco Castelo Branco. "Interpreting fatigue tests in hot mix asphalt (HMA) using concepts from viscoelasticity and damage mechanics." Transportes 23.2 (2015): 85-94.

[20]. Daniel, J. S. and Y. R. Kim. (2002). “Development of a Simplified Fatigue Test and Analysis Procedure Using a Viscoelastic Continuum Damage Model.” Journal of the Association of Asphalt Paving Technologists, Vol. 71, pp. 619-650.

[21]. Underwood, B. S., Kim, Y. R., & Guddati, M. N. (2010). Improved calculation method of damage parameter in viscoelastic continuum damage model. *International Journal of Pavement Engineering*, *11*(6), 459-476.

[22]. Wang, Y. D., Keshavarzi, B., & Kim, Y. R. (2018). Fatigue Performance Prediction of Asphalt Pavements with FlexPAVE^{TM}, the S-VECD Model, and D^{R} Failure Criterion. *Transportation Research Record*, 0361198118756873.

[23]. Jacques, C. D. (2016). Impact of Silo Storage Time and Specimen Fabrication Methods on Hot Mix Asphalt Mixtures.

[24]. Daniel, Jo Sias, et al. "Comparison of asphalt mixture specimen fabrication methods and binder tests for cracking evaluation of field mixtures." Road Materials and Pavement Design (2018): 1-17.

[25]. Park, S. W., Kim, Y. R., & Schapery, R. A. (1996). A viscoelastic continuum damage model and its application to uniaxial behavior of asphalt concrete. *Mechanics of Materials*, *24*(4), 241-255.

[26]. Nemati, R. Dave, E. V., Daniel, J. S. (2019) Evaluation of Laboratory Performance and Structural Contribution of Cold Recycled Versus Hot Mixed Intermediate and Base Course Asphalt Layers in New Hampshire. Under publication. Transportation Research Board, Journal of Transportation Research Record.

[27]. Wang, C., Castorena, C., Zhang, J., & Richard Kim, Y. (2015). Unified failure criterion for asphalt binder under cyclic fatigue loading. *Road Materials and Pavement Design*, *16*(sup2), 125-148.

[28]. Dave, E.V., C. Hoplin, B. Helmer, J. Dailey, D. Van Deusen, J. Geib, S. Dai, and L. Johanneck (2016). Effects of Mix Design and Fracture Energy On Transverse Cracking Performance of Asphalt Pavements in Minnesota. *Transportation Research Record*, 2576, pp. 40-50.

[29]. Al-Omari, Bashar, and Michael I. Darter. "Relationships between international roughness index and present serviceability rating." Transportation Research Record 1435 (1994).

[30]. Smith, K. D., & Ram, P. (2016). *Measuring and Specifying Pavement Smoothness: [techbrief]* (No.FHWA-HIF-16-032). United States. Federal Highway Administration.

[31]. https://nhdot.ms2soft.com/tcds/tsearch.asp?loc=Nhdot&mod=

[32]. Janoo, Vincent C. Layer Coefficients for NHDOT Pavement Materials. No. CRREL-SP-94-30. Cold Regions Research and Engineering Lab Hanover NH, 1994.

Appendices

Appendix: Paper 1 (Chapter 4)

Title: Statistical Evaluation of the Effects of Mix Design Properties on Performance Indices of Asphalt Mixtures

Journal: ASTM Journal of Testing and Evaluation

STATISTICAL EVALUATION OF THE EFFECTS OF MIX DESIGN PROPERTIES ON PERFORMANCE INDICES OF ASPHALT MIXTURES

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ABSTRACT

A variety of testing and performance index parameters are available to assess the asphalt mixture performance with respect to different structural distresses. However, due to continuous improvements in asphalt material production and construction techniques, it is necessary to regularly evaluate the correlation of the performance index parameters with mixture design properties. It is also important to determine the correlation between index parameters from different tests to help save time and financial resources by making engineering based adjustments to the mixture design before conducting multiple tests. This study explores the statistical correlation between mixture design properties and performance index parameters as well as the correlations among the performance index parameters from different tests. A total of 14 commonly used asphalt mixtures in New Hampshire were evaluated using the complex modulus (E*), resilient modulus (M_r), direct tension cyclic fatigue (S-VECD), Illinois semi-circular bend (SCB-IFIT), and disk-shaped compact tension (DCT) tests to assess the correlations between various performance indices and mix design properties. The results indicate that the aggregate fractions that pass 4.75 mm and 75 µm sieve sizes, the binder useful temperature interval, and recycled asphalt content

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significantly affect most of the index parameters. Medium to high correlations were observed between S-VECD, DCT and SCB with respect to different index parameters.

Keywords: Statistical Correlation, Performance Index, S-VECD, Flexibility Index, Fracture Energy

INTRODUCTION

The performance of asphalt mixtures is a direct function of mixture design properties such as aggregate size and gradation, binder type and content, air void percentage and any additives in the mixture. Due to the viscoelastic nature of asphalt mixtures, the loading and climatic conditions will also significantly affect mixture performance. Therefore, it is important to examine the mixture performance through laboratory testing and apply the necessary adjustments to the mixture design to ensure the best possible field performance. For that reason, a thorough understanding of the effect of each of the mixture's components with respect to a specific type of distress is of high interest to pavement engineers. In general, asphalt mixtures may encounter three types of structural distresses: rutting, fatigue, and thermal cracking, each with specific failure mechanisms. Many studies have been conducted to determine the relationship between mixture design parameters and different types of distresses. The paragraphs below provide recent examples that evaluate similar parameters to those included in this study.

Work performed by Zhao [1] investigated the correlation of the aggregate gradation, voids of mineral aggregate (VMA), voids filled with asphalt (VFA) and asphalt film thickness (FT) with rutting. The results indicated that the effects of these parameters are considerably lower for a mixture with 4% air void content compared to one with 7% air voids.

Diab et al. [2] indicated the importance of the effect of fine aggregate source and dust to effective binder (d/b_e) ratio by means of conducting the indirect tensile strength and moisture susceptibility tests, where a combination of hydrated lime and 0.95 (d/b_e) resulted in improved mechanistic properties of the mixtures. In another study to demonstrate the effect of ageing level, Rahbar showed that there may not be a strong correlation between binder and mixtures cracking properties [3]. Through statistical analysis during development of a balanced mix design for overlay mixtures in Texas, it was indicated that the binder performance grade (PG), effective binder volume (V_{be}), FT and aggregate surface area (SA) have a significant effect on the mixture cracking performance, while air void content was shown to have minimal effect in the results of the Texas Overlay (OT) test [4]. Using a large dataset of more than 170 mixtures from New England and Minnesota, Oshone et al. showed high correlation of binder related properties such as binder content, film thickness and performance grade useful temperature interval (UTI) with fracture energy (G_f) obtained from the disk shaped compact tension (DCT) test [5]. A research study conducted by the National Center for Asphalt Technology (NCAT) on the refinement and validation of 4.75 mm Superpave mixtures examined the statistical correlation between the mixture design volumetric properties such as V_{be}, VMA, VFA, d/b_e and FT to the rut depth from the Asphalt Pavement Analyzer (APA) and G_f from the indirect tensile creep (IDT) test [6]. The Pearson correlation coefficients indicated that d/b_e is the most significant factor followed by FT. The results also revealed the importance of V_{be} and its two-way interaction with the amount of natural sand in the mixture with regards to the rut depth. In a research study to develop predictive models for dynamic modulus ($|E^*|$) and phase angle of asphalt mixtures, Nemati and Dave implemented only the nominal mixture design properties such as recycled binder ratio (RBR), nominal maximum aggregate size (NMAS), air void percentage (AV%) and asphalt content (AC%) in construction of the models and indicated the significance of two-way interactions of these parameters in the linear viscoelastic behavior of asphalt mixtures [7]. There are many more examples of the research studies conducted on correlations between mixtures properties and performance conducted by other researchers [8-12]. However, due to continuous improvements in asphalt material production and construction techniques, it is necessary to regularly evaluate the correlation of the distress index parameters with mixture nominal properties to identify the gaps between production and performance evaluation tools.

A variety of laboratory performance based testing and analysis methods are available to characterize mixture performance with respect to individual types of distresses. In order to simplify the application of these tests to rank and correlate the performance of the mixtures in the lab to that of the field, different failure criteria and performance indicator parameters have been proposed and evaluated. Many of the existing criteria are based on different mechanics of failure (i.e. fracture mechanics, continuum damage mechanics) and are designed to evaluate only one particular type of performance. Optimally, the mixture should be designed in a manner to tolerate multiple competing distresses such as rutting and cracking at the same time. In many instances laboratories are not equipped with all the required testing equipment and if so, the exhausting amount of time required for sample fabrication and testing in addition to the required number of trained technicians in the lab may not be feasible to conduct all tests. As a result, there is a need to investigate the correlation between laboratory test results and index parameters of different tests to determine if any of them can be used interchangeably or at least provide some preliminary estimate of the mixture's performance in laboratory tests that have not been performed. For instance, Na Chiangmai found a high correlation between the plateau value (PV) from the bending beam fatigue test and pre-peak fracture energy from the DCT test [13]. Also, Tang investigated the correlation between fracture and fatigue resistance of high RAP continued HMA mixtures [14]. However, there is generally very little available in the literature correlating the indices from different tests.

Among the asphalt mixture characterization tests, complex modulus has been widely used as the main input in development of rutting and cracking transfer functions in many mechanistic-empirical pavement modeling software (e.g., AASHTOWare Pavement ME and FlexPAVE™). The dynamic modulus (|E*|) master-curve indicates the stiffness of the mix over a broad range of loading frequencies at a reference temperature, and can be potentially used to provide preliminary performance predictions with respect to different types of distress. For example, the lower frequency tail of the master-curve which is associated with the stiffness in higher temperatures has been used to discriminate the rutting performance [15], [16]. Nonetheless, the mid and high range frequencies which correspond to medium and low temperatures have not been explicitly evaluated in terms on correlation with other types of distresses such as fatigue and thermal cracking.

This study pursues the following three objectives:

- Identify key mixture design factors with respect to different asphalt mixture performance indices with aim of improving pavement performance to specific distresses.
- Determine the correlations between indices from different performance prediction tests in order to estimate other types of distress.
- Introduce and explore three complex modulus based indices and their correlation to the specific distress index parameters from other tests in order to assess the applicability of master-curves for evaluating mixture performance.

STUDY MATERIALS AND EXPERIMENTAL PLAN

This study investigates 14 hot mixed asphalt (HMA) mixtures, including two asphalt rubber gap graded (ARGG) and different types of dense graded mixtures with unmodified and polymer modified asphalt binders used in construction of wearing, intermediate, and base courses. The study mixtures are selected to represent the range of aggregate size and gradation, binder type, RBR, and gyration level commonly used on New Hampshire roadways. Information on the study mixtures is summarized in Table 1. RBR is defined as the percentage of recycled binder with respect to total binder content. All mixtures studied herein are designed at the 4% air void level. The useful temperature interval (UTI) is defined as the difference between the PG high temperature and the PG low temperature. To fabricate the specimens, the plant-produced mixtures were reheated at 135°C and specimens were compacted to 6±0.5% air void level using a Superpave gyratory compactor to represent typical in-place density.

Table 1. Study Mixtures Design Characteristics

Mixture	Course	NMAS (mm)	Binder Performance Grade	Useful Temperature Interval (UTI) (°C)	AC%	VMA	V_{be}%	RBR %	%Passing 4.75 mm	%Passing 0.075 mm	Gyration
ARGG-1	Wearing	12.5	58-28	86	7.8	19.1	15.1	0.0	40.0	3.5	75
ARGG-2			58-28	86	7.6	18.4	14.4	6.6	37.0	3.5	75
W-6428H-12.5			64-28	92	5.4	16.1	12.1	18.5	57.0	3.8	75
W-5828L			58-28	86	5.8	15	11.0	16.2	57.0	3.6	50
W-5834L			58-34	92	5.4	15.3	11.3	18.5	60.0	3.7	50
W-7628H-12.5			76-28	104	5.4	16.1	12.1	18.5	57.0	4.0	75
W-7034PH			70-34	104	5.8	16	12.0	0.0	59.0	3.7	75
W-7628H-9.5		9.5	76-28	104	6.1	16.3	12.3	14.8	68.0	4.9	75
W-5828H			58-28	86	5.9	16.6	12.6	16.9	70.0	4.5	75
W-6428H-9.5			64-28	92	6.4	17.1	13.1	0.0	69.0	5.2	75
B-6428H	Intermediate	19.0	64-28	92	4.8	14.3	10.3	20.8	46.0	3.5	75
B-5834L			58-34	92	4.6	14.1	10.1	21.7	43.0	3.2	50
B-5828H			58-28	86	4.8	14.2	10.2	20.8	47.0	3.2	75
BB-6428L	Base	25.0	64-28	92	4.8	14.2	10.2	20.8	36.0	3.5	50

PERFORMANCE TEST RESULTS AND STATISTICAL ANALYSIS

This section presents the results of the mechanistic and performance prediction tests including complex modulus (E*), resilient modulus (M_r), direct tension cyclic fatigue (S-VECD), Illinois semi-circular bend (SCB-IFIT), and disk-shaped compact tension (DCT). Each subsection will briefly compare the mixtures for an individual test using the specific performance index parameter related to that test. Along with the performance tests, a forward direction step-wise linear regression statistical analysis is conducted between the mixture design properties and performance index parameters to determine the most significant design variables affecting the index parameter. All the statistical analysis are conducted by means of JMP PRO statistical software. The evaluated design properties used in this paper are those presented in Table 1, and separately evaluating the high and low performance PG grades (PGHT, PGLT).

The statistical analysis includes the following notable parameters:

- The coefficient of estimate of linear regression where a positive estimate means that the variable and response are directly proportional and a negative estimate value indicates they are inversely proportional.
- The t-ratio which is the estimate divided by the standard error. Since the degree of freedom (DF) for the t-test analysis in this study is 10 (DF=number of variables-1), the t-ratios over 1.812 (in absolute value) at 90% confidence level, suggesting that the coefficient is significantly different from the mean.
- The probability value (p-value) of a two tailed t-test analysis which reveals the influence of each mix design factor on the performance test where a lower p-value means a higher effectiveness of the factor. A significance level (α) of 0.1 equal to 90 percent confidence level was considered as the set p-value to discriminate the influential parameters.

Complex modulus

The complex modulus test was performed in accordance with AASHTO T342 test standard [17] using an Asphalt Mixture Performance Tester (AMPT) and three replicate specimens. The test is conducted at three different temperatures (4.4°C, 21.1°C and 37.8°C) and six frequencies (25Hz, 10Hz, 5Hz, 1Hz, 0.5Hz and 0.1Hz). The master-curves were constructed at a reference temperature of 21.1°C using the time-temperature superposition principle. The dynamic modulus master-curves for all the mixtures are depicted in Figure 1. The results indicate that ARGG-2 is stiffer than ARGG-1 over the range of frequencies because of the RBR content in the ARGG-2. In terms of other wearing courses, W-7628H-12.5 is shown to be the stiffest mixture while W-7034PH which contains a polymer modified binder with 4% styrene butadiene styrene (SBS) and 4%

aromatic oil is the softest. With respect to the intermediate and base course mixtures, B-6428H is the stiffest and B-5828L is the softest.

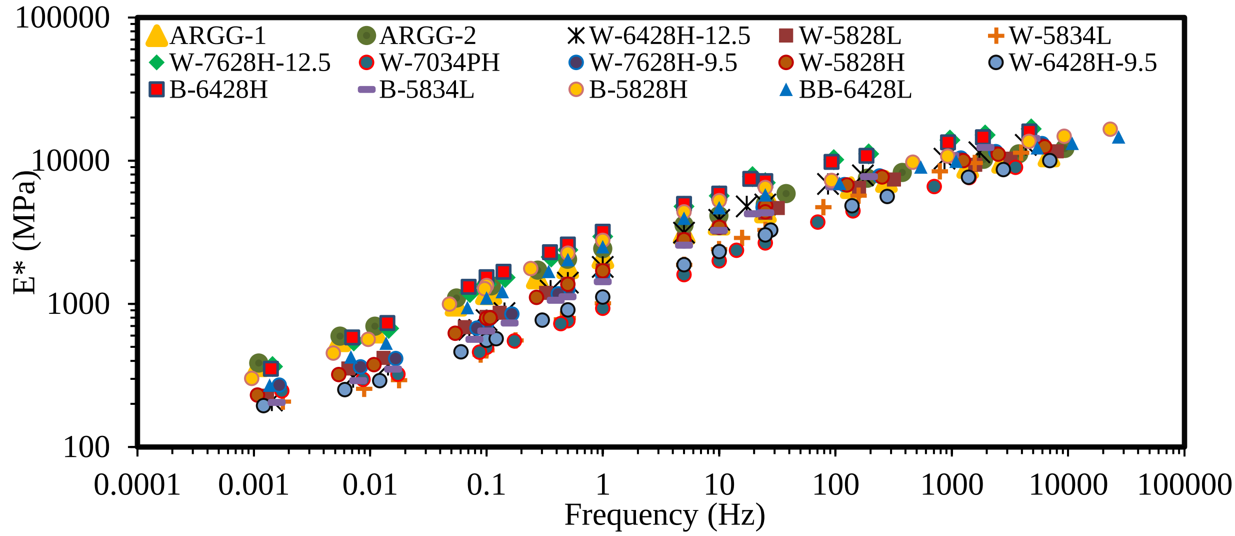

Figure 1. |E*| Master-curve (Reference Temperature = 21.1°C)

Three different dynamic modulus based index parameters for rutting, fatigue, and thermal cracking are proposed to help evaluate the performance prior to conducting other distress specific performance tests during the iterative mix design procedure. The aim of these parameters is to not replace the use of laboratory performance tests, rather help lower the amount of testing that might be necessary. The │E*│based distress parameter measurements are shown in Figure 2. These index parameters are selected and described based on the following assumptions:

- The rutting parameter as │E*│@ 1.59Hz & 40°C was selected to represent a worst case scenario for rutting at which the material has still a linear viscoelastic response. The 1.59 Hz is selected as an equivalent frequency to the Superpave binder rutting criteria measured in dynamic shear rheometer at 10 rad/s. A higher value of this parameter is more desirable as mixtures with higher stiffness are generally more rut resistant.
- The fatigue parameter as │E*│@ 15Hz & 12°C was selected based on the recommended S-VECD fatigue test temperature selection by in the AASHTO TP 107 specification. This

temperature is determined as 3˚C lower than average of PGHT and PGLT. Considering New Hampshire climatic conditions the 12°C is most applicable to mixtures studied herein. The selected frequency of 15 Hz for fatigue and thermal cracking is representative of nearly 90 km/h traffic speed. A lower value for this parameter is preferred for better fatigue cracking performance as it would indicate a less stiff material that typically has greater ductility.

- Finally the thermal cracking parameter is chosen as │E*│@ 15Hz & -18°C to comply with the binder bending beam rheometer (BBR) test temperature selection for majority of mixtures in this study. Similar to fatigue cracking parameter a lower value is preferred here as well.

It should be mentioned that the selection of the temperatures for the dynamic modulus based index parameters has been based on the local climatic conditions in New Hampshire and is associated with the common binder grades that is used in this area. Therefore, these values can change appropriately with respect to the climatic conditions of other regions.

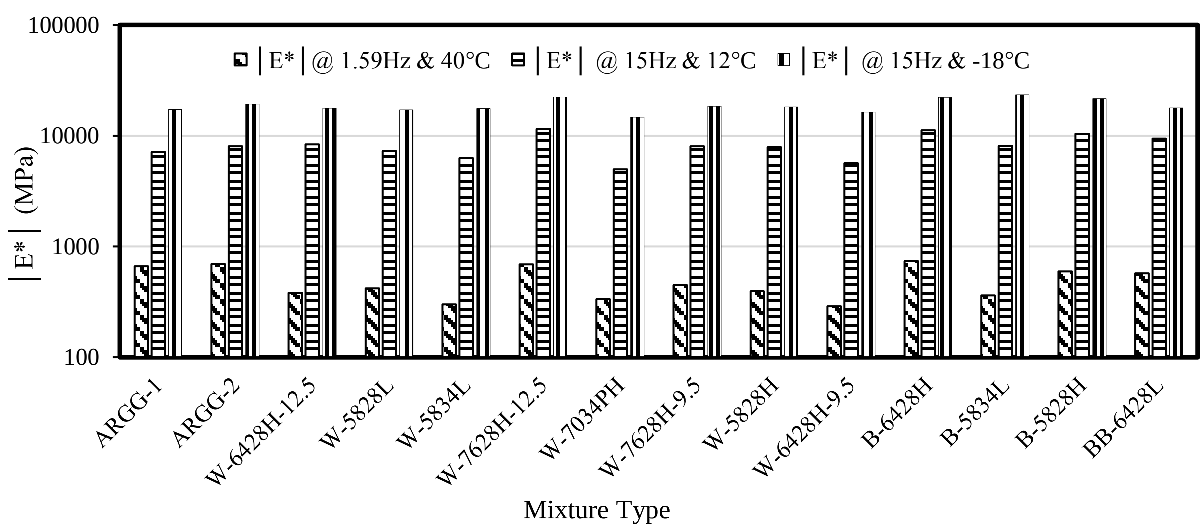

Figure 2. Dynamic Modulus Based Distress Criteria

The results from the statistical analysis regarding the significant mix design properties affecting the dynamic modulus based distress index parameters is depicted in Table 2. The results can be summarized as follows:

- Binder PGHT and UTI have significant effect on the midrange stiffness of the mixtures and the selected fatigue parameter. In comparing the mixtures containing conventional binder types with similar design properties such as B-6428H and B-5828H, the one with warmer PGHT is stiffer. However, comparing mixtures with same PGHT such as W-5828 and W-5834, the one with higher UTI is less stiff. Although polymer modified binders usually have warmer PGHT, because of the influence of polymer, they usually have much higher UTI and therefore better stress relaxation and creep recovery properties compared to conventional binder and consequently are less fatigue susceptible.
- As expected, the higher level of gyration increases the stiffness, which is desirable for rutting, but it may also increase the fatigue susceptibility as higher gyrations usually result in reduced VMA and relatively lower binder contents.
- The cumulative percent passing the 4.75 mm sieve is shown to be an important factor for all three parameters. The negative estimate coefficient of this variable means that increasing the amount of fine aggregate in the mixture will reduce the stiffness which can help in cracking resistance while increasing the rutting susceptibility.
- The significance of RBR in increasing the mixtures stiffness can be seen for both rutting and fatigue cracking.
- With respect to thermal cracking, the effect of volumetrics such as VMA and AC% are significant to the performance. A high amount of VMA if not filled with sufficient amount of AC% would result in thin asphalt films around the aggregate resulting in a mixture with lower stress relaxation capabilities which will increase the thermal cracking susceptibility.

Table 2. Significant mix design properties for dynamic modulus based distress index parameters

Distress	Index Parameter	Mix Property	Estimate	Std Error	t-Ratio	Prob>|t|
Rutting	**|E*|@ 1.59Hz & 40°C**	**%Passing 4.75mm**	-10.52	2.30	-4.57	0.0018
		Gyration	6.78	2.61	2.59	0.0321
		RBR	11.94	4.93	2.42	0.0418
Fatigue	**|E*|@ 15Hz & 12°C**	**RBR**	183.98	28.22	6.52	0.0002
		%Passing 4.75mm	-74.64	18.94	-3.94	0.0043
		Gyration	65.75	24.41	2.69	0.0273
		PGHT	70.21	35.08	2.00	0.0803
		UTI	-183.30	93.63	-1.96	0.0859
Thermal Cracking	**|E*|@ 15Hz & -18°C**	**%Passing 4.75mm**	-264.84	113.91	-2.32	0.053
		AC%	-4901.74	2110.69	-2.32	0.0532
		VMA	1784.16	916.02	1.95	0.0925

Resilient modulus

The resilient modulus test was conducted at 25°C in accordance with ASTM D7369-11 standard test method [18] with three replicate specimens. The results from this test are shown in Figure 3. The error bars on the graph show one standard deviation from the mean. It has been indicated that the asphalt mixtures' resilient moduli are correlated to the rutting susceptibility [19], therefore the M_r measurements are used as the primary rutting performance indicator in this study. The results indicate that ARGG-2 has a higher M_r value compared to ARGG-1 which is mainly related to the RBR percentage difference. The other wearing courses show expected results such that W-7628H-12.5 with a stiffer binder and high gyration level resulted in a higher modulus whereas mixtures such as W-5834L because of lower number of gyration and W-7034PH as a virgin mixture with lower PGLT are the softest among all mixtures. According to mix properties W6428H-12.5 is expected to have a higher modulus than W5828L, while the M_r values for both mixtures are very similar. One possible explanation is that the single testing temperature and single loading frequency is not able to capture the viscoelastic properties of the mixtures. Therefore, the future validation of the test results with respect to more specific rutting tests such as Hamburg wheel track test, asphalt pavement analyzer, flow number etc. will be necessary [20,21].

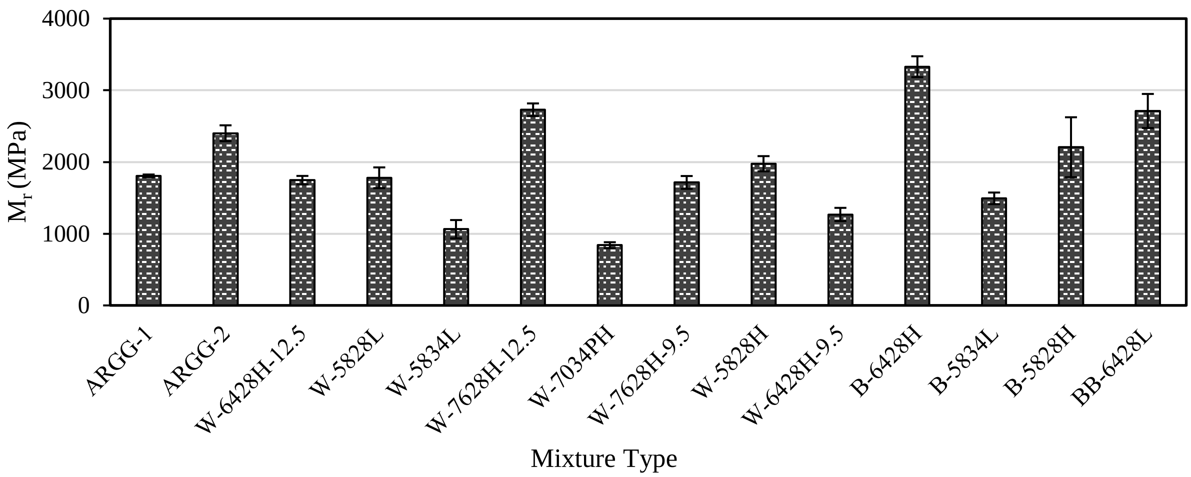

Figure 3. M_r Test Results

The results from the statistical analysis on M_r test is depicted in Table 3. Similar to the rutting criteria from the |E*| master-curves the % passing 4.75 mm sieve size and RBR are among the significantly effective factors on the resilient modulus. However, while performing the statistical analysis it was observed that almost all volumetric parameters as well as gyration levels have less than 10% significance level when rutting is a concern. However, the NMAS with a positive estimate coefficient was seen to be close to the 0.10 significance level. Considering the results from statistical analysis on M_r, with an intention to mitigate the rutting issues, designers can simply change the binder type as an efficient adjustment to the mix design.

Table 3. Significant Mix Design Properties Affecting Resilient Modulus (M_r)

Index Parameter	Mix Property	Estimate	Std Error	t-Ratio	Prob>|t|
Resilient Modulus, M_r	**PGHT**	181.20	42.42	4.27	0.0021
	UTI	-156.22	40.84	-3.83	0.0041
	%Passing 4.75mm	-29.15	9.57	-3.05	0.0139
	RBR	32.15	12.38	2.6	0.0289

Direct Tension Cyclic Fatigue (S-VECD)

The uniaxial fatigue test was performed in accordance with AASHTO TP 107 standard [22] on four replicates each at a different strain level. The test is conducted with respect to the binder performance grade at $(\frac{PGHT-PGLT}{2} - 3°C)$. Currently, there are two widely accepted fatigue criteria based on the S-VECD approach: N_f @ G^R=100 and D^R [23]. G^R is the rate of averaged dissipated pseudo strain energy which indicates the decrease in the mixture's energy storage capacity due to each loading cycle. The number of cycles to failure at G^R equal to 100 (N_f @ G^R=100) is usually used to rank mixtures. D^R is the average reduction in pseudo stiffness per loading cycle and indicates the decrease in material integrity in terms of stiffness as the load is applied. D^R values usually range from 0.3 to 0.7 with higher values indicating better fatigue resistance [24]. The results from N_f @ G^R=100 and D^R are shown in Figure 4 and Figure 5, respectively. The effect of addition of RBR is evident when comparing the fatigue performance of ARGG-1 to ARGG-2. The W-7034PH, which is a virgin polymer modified mixture, shows very good fatigue performance. As expected, the intermediate and base course mixtures have lower fatigue resistance because of larger aggregate size, lower binder content, and higher RBR percentages. The two indices have minor differences in ranking, especially for mixtures such as W-6428H-12.5, W7628H-12.5, W7628H-9.5, and B6428H. Also, the discrimination of the magnitude of difference between performance is shown to be quite different. For example, with respect to N_f @ G^R =100, W-7034PH is indicated to have 4 times higher N_f value compared to W-7628H-9.5, however, their D^R values is relatively close. The discussion on the difference of these two indices is out of the scope of this paper, however, a hierarchical clustering analysis indicated that, with the exception of ARGG-2, both indices are able to separate the intermediate and based courses from wearing courses. Similar observations with respect to capability of clustering the type of mixtures with respect to production method (hot mixed versus cold recycled) has been seen by applying the D^R criterion in other studies conducted by authors [25].

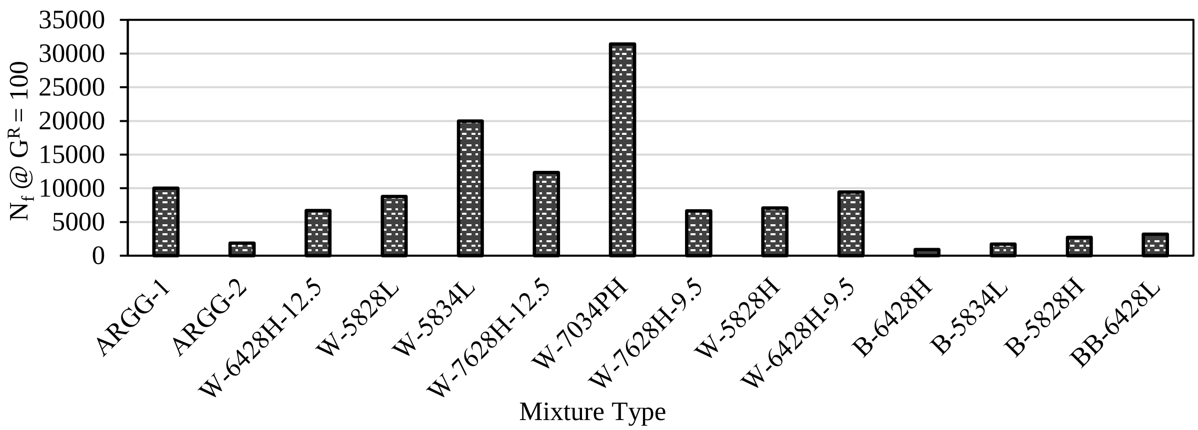

Figure 4 . Energy Based Viscoelastic Continuum Damage Fatigue Index Parameter (N_F @ G^R = 100)

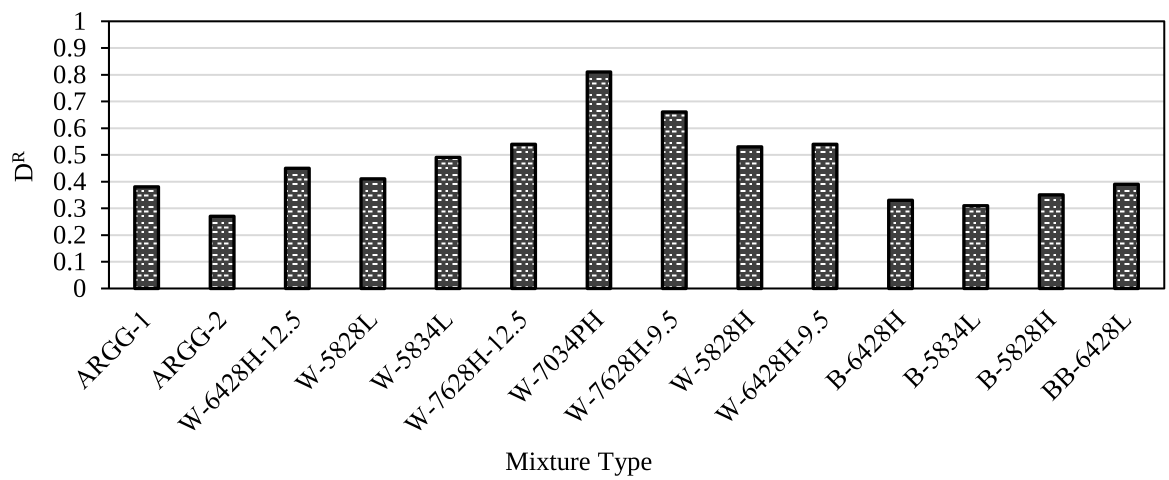

Figure 5. Pseudo-Stiffness Based Viscoelastic Continuum Damage Fatigue Index Parameter (D^R)

The effect of mix design parameters on S-VECD based index parameters in shown in Table 4. The analysis confirms the high importance of fine aggregate and dust in the mixtures as both indices are highly dependent on the cumulative percent passing amounts on 4.75 mm and 75 μm sieves, where an increase in the dust amount causes a lower fatigue life. On the other hand, the increase in cumulative percent passing 4.75 mm results in a lowering of non-uniform air void dispersion in the aggregate matrix and results in a better fatigue resistant mixture. The effect of UTI is apparent in both indices, with a larger value being better for fatigue resistance. Both indices show sensitivity to the changes in the amount of RBR with G^R being more influenced by the RBR content. The effect of gyration level could be the main noteworthy difference between the two parameters where a higher level of gyration is shown to have a negative effect on the N_f @ G^R=100 parameter. Similar observations were seen from the effect of gyration on the dynamic modulus based fatigue parameter (Table 2). as higher gyrations can increase the modulus and lower the fatigue life.

From an engineering perspective, higher VMA is required for a durable mixture to retain a satisfactory amount of binder, however the statistical analysis indicated a negative effective of VMA. It should be noted that throughout the paper, wherever VMA is indicated as a negative effective factor, it is accompanied by the positive effect of either V_{be} or AC% as they are tied to each other. This means that a higher VMA in the mixture design is not a guarantee for a crack resistant mixture and the effective binder content to fill the void spaces in aggregate skeleton is critical.

Table 4. Significant Mix Design Properties Affecting Simplified Viscoelastic Continuum Damage Based Fatigue Performance Indices (N_f @ G^R = 100 and D^R)

Index Parameter	Mix Property	Estimate	Std Error	t-Ratio	Prob>|t|
N_f @ G^R=100	**RBR**	-730.84	162.26	-4.5	0.0041
	% Passing 4.75 mm	469.03	187.52	2.5	0.0465
	%Passing 0.075mm	-9011.72	3748.20	-2.4	0.053
	UTI	372.52	170.75	2.18	0.0719
	VMA	-190.285	90.28	-2.11	0.0796
	Gyration	-5262.82	2511.85	-2.1	0.081
	V_{be}	4089.5	2004.01	2.04	0.0874
D^R	**UTI**	0.012	0.002	5.69	0.0007
	% Passing4.75 mm	0.021	0.005	4.29	0.0036
	%Passing 0.075mm	-0.156	0.051	-3.03	0.0191
	RBR	-0.006	0.002	-2.62	0.0343
	V_{be}	0.061	0.031	2.00	0.0850

Illinois Semi-Circular Bend (SCB-IFIT)

The Illinois semi-circular bend (SCB-IFIT) test was conducted to determine the fracture properties of asphalt mixtures at intermediate temperature in accordance to AASHTO TP 124 standard [26] at 25°C using four replicates. The main outcomes of this are the fracture energy (G_f) and flexibility index (FI). The results from the G_f and FI are shown in Figure 6 and Figure 7, respectively. The results indicate that fracture energy is not able to fully differentiate the cracking resistance of different mixtures. For example, according to the results from S-VECD, ARGG-2 was shown to be a less crack resistant mixture compared to ARGG-1 whereas both mixtures have similar fracture energies. Apart from this discrepancy, the results from flexibility index agrees well with the other test results and crack resistance expectations. For instance, the W-7034PH was previously indicated to be a superior fatigue cracking resistance through S-VECD analysis which is also differentiated in the same manner by FI.

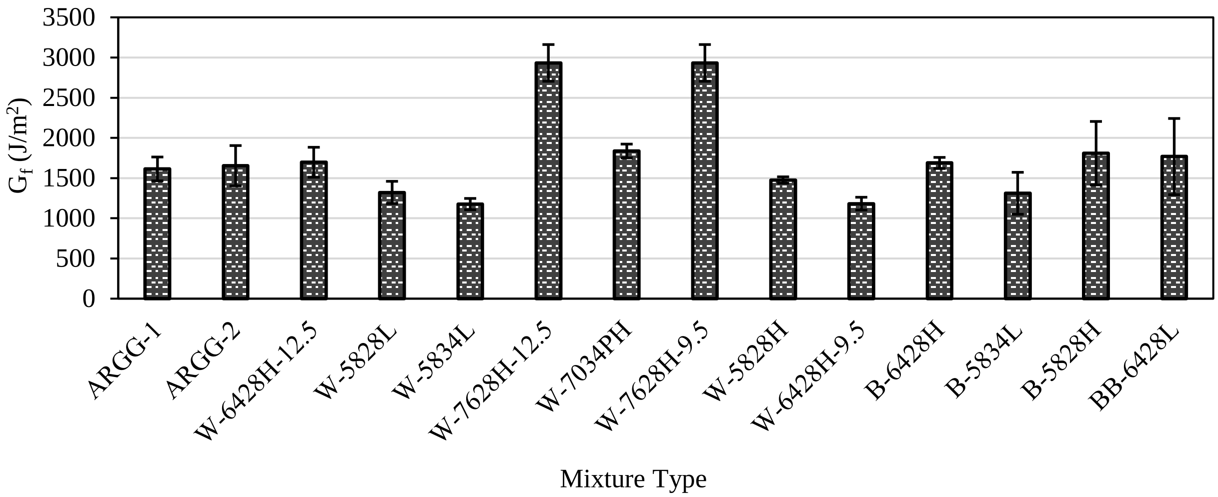

Figure 6. SCB-IFIT Fracture Energy (G_f) plots

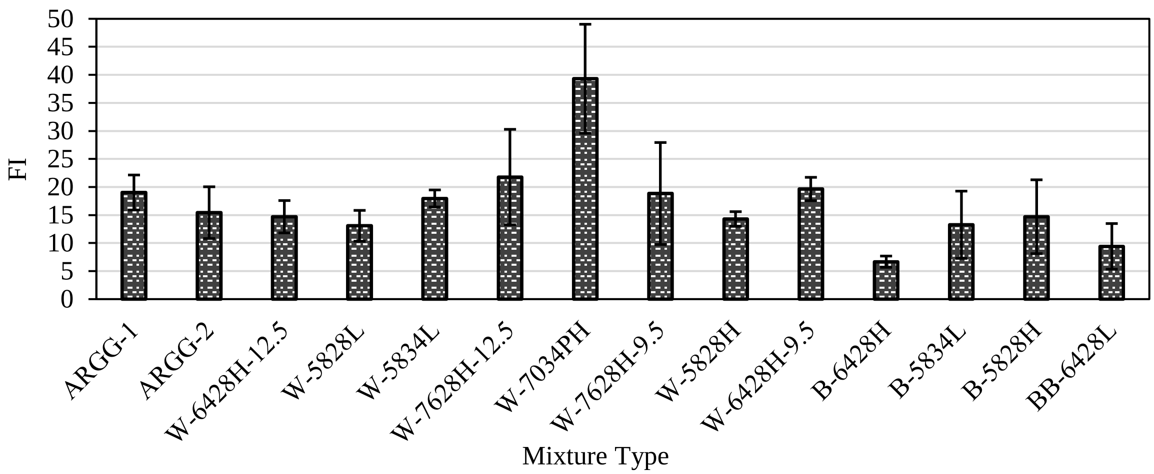

Figure 7. Flexibility Index plots

The effect of mix design properties on the SCB fracture energy and FI are shown in Table 5. With respect to G_f, the higher PGHT and RBR result in a stiffer mixture and higher G_f values. However, it should be considered that higher fracture energy is not always about the higher stiffness and depending on the binder type and content a ductile mixture could also have a high fracture energy. This can be seen through a more in-depth investigation on a mixture such as W-7628H-9.5 which maintains high G_f and FI values at the same time, while it retains one of the highest binder contents among the study mixtures.

One of the main intents of developing the FI has been to capture the effect of RBR on mixtures cracking [27] which is clearly seen in table 5. In general, the significant mixture design parameters on FI are similar to those for S-VECD performance indices, indicating that there could potentially be a high correlation between the two indices. Therefore, if a mixture is optimized for one of these tests then it is likely to meet the requirements for other one as well.

Table 5. Significant Mix Design Properties Affecting SCB G_f and FI

Index Parameter	Mix Property	Estimate	Std Error	t-Ratio	Prob>|t|
G_f	**PGHT**	79.64	9.81	8.12	0.0001
	RBR	41.09	12.26	3.35	0.0074
	AC%	321.92	105.53	3.05	0.0122
FI	**RBR**	-0.79	0.15	-5.11	0.0006
	VMA	-5.69	1.67	-3.39	0.0080
	UTI	0.27	0.13	1.94	0.0843
	V_{be}	2.90	1.58	1.83	0.1008

Disk-Shaped Compact Tension (DCT) Test

The disk-shaped compact tension (DCT) test was performed in accordance to the ASTM D7313 standard testing method [28] on three replicates. The test is developed to determine the low temperature fracture properties of the asphalt mixtures. In general, the testing temperature is determined by 10°C+PGLT. However, in this study the LTTPBind software was used to determine the testing temperature as 10°C warmer than the 98% reliability pavement low temperature without rounding to nearest 6°C increment. In other words, continuous PGLT value on basis of the pavement location was used in the test temperature calculation. The two index parameters that are used to analyze the DCT test results are the G_f and the fracture strain tolerance (FST) [29]. The FST is determined by dividing G_f by the fracture strength (S_f). The results from G_f (including the test temperatures in degrees Celsius) and FST are shown in Figure 8 and Figure 9, respectively. It should be noted that the results of this test are not available for B-5828H at this time and the analysis is conducted on 13 mixtures. The ARGG mixtures have better performance compared to other wearing courses such as W-6428H-12.5, W-5834L and W-7628H-9.5 that are tested in similar temperature range, that is, these mixtures may have better thermal cracking performance in similar climatic conditions. Identical to the S-VECD and SCB-IFIT tests results, W-7034PH mixture indicates an outstanding performance with respect to low temperature cracking. The ranking from both G_f and FST are similar except for three mixtures (ARGG-2, W-5828L and, BB-6428L).

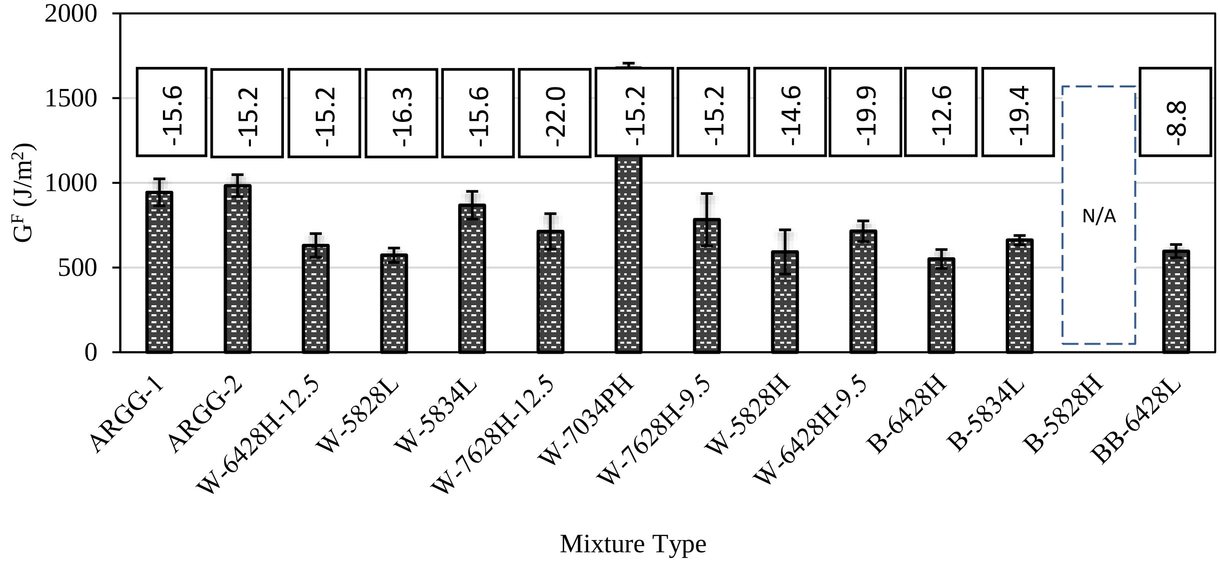

Figure 8 . DCT Fracture Energy (G_f) plots (number in box indicate test temperature in ˚C)

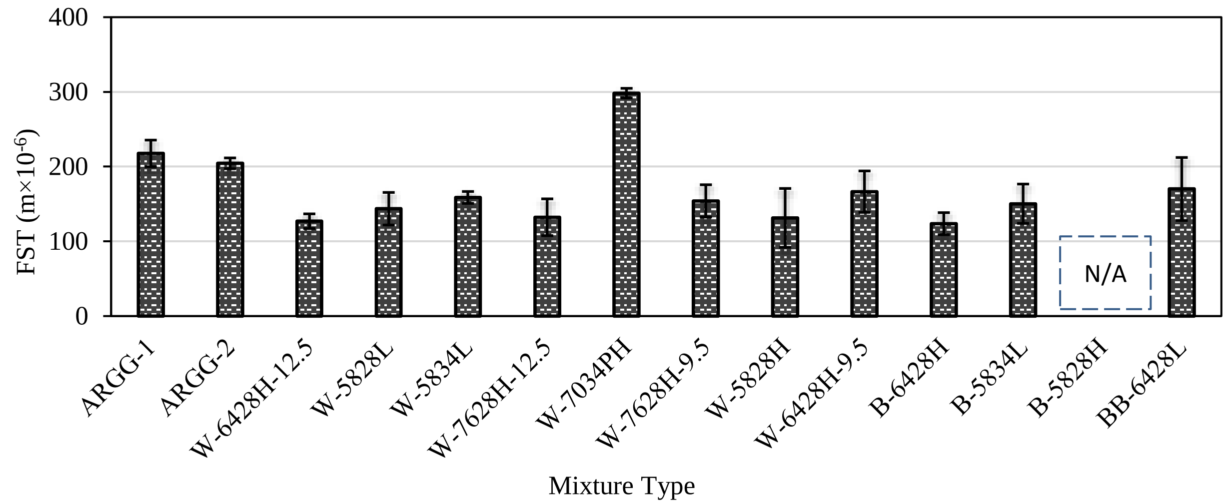

Figure 9 Fracture Strain Tolerance (FST) plotsTable 6. Significant Mix Design Properties Affecting Disk-Shaped Compact Tension Test Performance Indices (GF and FST)

Index Parameter	Mix Property	Estimate	Std Error	t-Ratio	Prob>|t|
Gf	**RBR**	-21.70	5.28	-4.11	0.0063
	VMA	-175.10	45.61	-3.84	0.0086
	UTI	47.19	12.50	3.77	0.0092
	%Passing 0.075mm	-157.88	45.69	-3.45	0.0136
	AC%	227.63	68.16	3.34	0.0156

	PGLT	-36.21	13.13	-2.76	0.033
	Gyration	6.54	2.62	2.49	0.0473
FST	**RBR**	-4.96558	0.78	-6.38	<0.0001
	%Passing 0.075mm	-44.16	11.55	-3.82	0.0034
	UTI	2.17	0.96	2.26	0.0474

Table 6 summarizes the significant mixture design properties on DCT index parameters. Similar to previous observations for FI and N_f @ G^R =100, higher VMA has a negative effect on G_f while accompanied with the positive effect of AC%. The statistical analysis confirms that a lower PGLT will result in higher fracture energy values, indicating the importance of selecting an appropriate binder type for mixtures to withstand low temperature cracking.

Similar to results from S-VECD fatigue indices, the % passing 75µm is considered as a significant mix design property for both G_f and FST. Higher amounts of dust can over-stiffen the mastic phase of mix and lower the asphalt film thickness which can lead to cracking. While the higher amount of RBR is shown to increase the cracking susceptibility, increase of UTI which, in general, can be translated into use of polymer-modified binders, mitigates the cracking. The positive effect of gyration level in increasing the fracture energy is an interesting point in the statistical analysis. For both the low temperature fracture energy and N_f @ G^R =100, the four significant parameters are common and they have similar type of effects on the performance (increasing cracking resistance); this indicates the potential for a high correlation between these two parameters.

Table 6. Significant Mix Design Properties Affecting Disk-Shaped Compact Tension Test Performance Indices (G_F and FST)

Index Parameter	Mix Property	Estimate	Std Error	t-Ratio	Prob>|t|
G_f	**RBR**	-21.70	5.28	-4.11	0.0063
	VMA	-175.10	45.61	-3.84	0.0086
	UTI	47.19	12.50	3.77	0.0092

	%Passing 0.075mm	-157.88	45.69	-3.45	0.0136
	AC%	227.63	68.16	3.34	0.0156
	PGLT	-36.21	13.13	-2.76	0.033
	Gyration	6.54	2.62	2.49	0.0473
FST	**RBR**	-4.96558	0.78	-6.38	<0.0001
	%Passing 0.075mm	-44.16	11.55	-3.82	0.0034
	UTI	2.17	0.96	2.26	0.0474

STATISTICAL CORRELATION OF THE PERFORMANCE INDEX PARAMETERS

With respect to conventional volumetric mixture design, it is important for the designers to understand the significance of each mix design variable on the performance of the product. However, with changes in materials and loadings, asphalt mixture design is moving towards performance based design approaches such as the performance engineered mix design (PEMD). One of the main intent of the PEMD approach is to embed the mixture performance prediction tests within the structure of the volumetric based mixture design. The final product of the PEMD is a mixture that can withstand competing distresses such as rutting and cracking at the same time. However, such approaches need intensive testing to evaluate both rutting and cracking performance which may be time consuming and unaffordable in many settings. As a result, identifying the correlation among different distress index parameters could be a key factor in reducing the costs of performance based mixture design procedures.

Before determination of any correlation among the indices, it is important to verify the data and identify any outliers within the dataset which could cause biased or unrealistic correlations. Therefore, the Mahalanobis distance (MD) [30] analysis was performed to determine the outlier mixtures in the study. This analysis essentially identifies an overall mean as the centroid for multivariate data. In a multivariate space, this point is where all the averages from all variables intersect. Therefore, as a data point gets further away from the centroid, the MD for that data point becomes larger. In a dataset with different mixtures such as this study, the MD value is first calculated for the whole dataset and is called the MD threshold value. Then, for an each individual mixture, a separate MD value is calculated; if it exceeds the threshold MD value then that mixture is an outlier. As it can be seen from Figure 10, the W-7034PH and W-7628H-9.5 mixtures are very close to the calculated threshold value, however, they are below this point. Therefore, all the mixtures in the study can be used to establish the correlations.

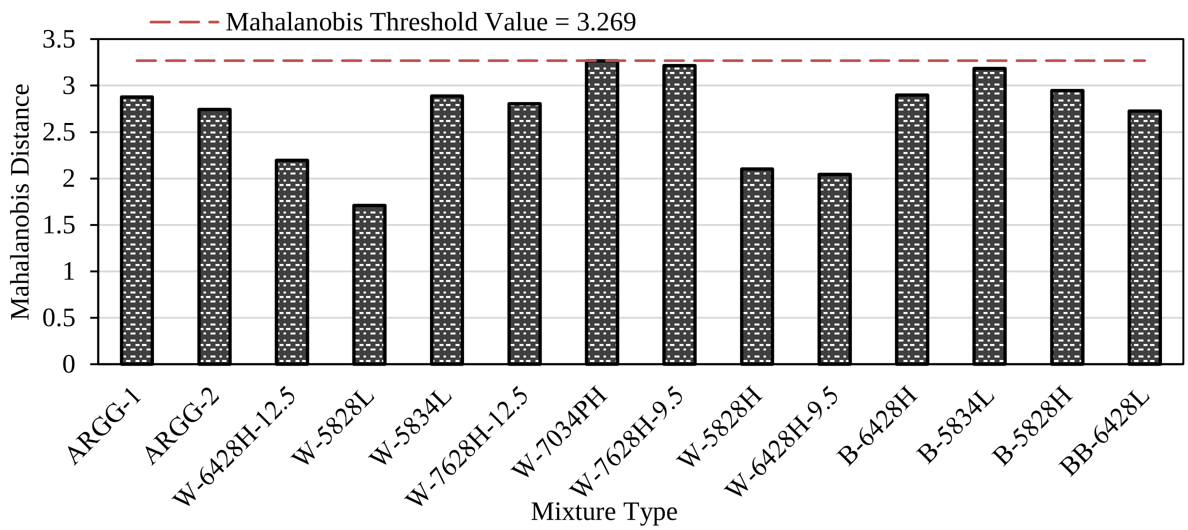

Figure 10. Mahalanobis distance of the study mixtures

To determine the correlations among the parameters, the Pearson's linear correlation coefficients (r) were calculated and depicted in Table 7. In general, the Pearson's coefficients range from -1 to 1 indicating a perfect negative and positive correlations, respectively. The strength of the correlations in this research are categorized into three categories defined as follows:

- Weak correlation *(W)*, *$|r| < 0.3$* (*indicated by italic font*)
- Medium correlation (M), $0.3 \leq |r| < 0.7$
- Strong correlation (**S**), **$0.7 \leq |r| \leq 1$ (indicated by bold font)**

M_r has a medium to high inverse correlation with N_f at G^R=100 and FI, indicating increased stiffness can increase the cracking susceptibility. As expected, D^R and N_f @ G^R=100 are highly tied to each other and both of them are strongly correlated with the FI. The strong correlations between the N_f at G^R=100 and FI with DCT (G_f) are among the most important observations in the table indicating that the proper adjustments to the mixture design properties can improve both fatigue and thermal cracking performance at the same time. However, it should be noted that the distress mechanism is substantially different between fatigue and thermal cracking and more in-depth analysis is required to discriminate the capability of the mixtures to withstand either of these distresses. A strong correlation between the M_r and dynamic modulus based rutting parameter (|E*| at 1.59Hz &40°C) is observed. The dynamic modulus based parameters for thermal and fatigue cracking are on the average to higher end of the medium correlation range with respect to the performance indices such as FST, FI and N_f at G^R=100. These correlations indicate that these dynamic modulus based parameters can potentially be used as preliminary mixture performance evaluation checkpoints throughout the mixture design procedure.

Table 7. Pearson's Correlations among Performance Index Parameters

Distress										
Criterion	**Rutting**		**Fatigue**					**Thermal cracking**		
	M_r	**|E*| @ 1.59Hz & 40°C**	**D^R**	**N_f @ G^R=100**	**SCB (G_f)**	**FI**	**|E*| @ 15Hz & 12°C**	**DCT (G_f)**	**FST**	**|E*| @ 15Hz & -18°C**
M_r	1.00	**-**	-	-	-	-	-	-	-	-
|E*| @ 1.59Hz & 40°C	**0.85 (S)**	1.00	-	-	-	-	-	-	-	-
D^R	-0.54 (M)	-0.50 (M)	1.00	-	-	-	-	-	-	-
N_f @ G^R=100	-0.65 (M)	-0.46 (M)	**0.78 (S)**	1.00	-	-	-	-	-	-
SCB (Gf)	0.33 (M)	0.40 (M)	0.38 (M)	*0.02 (W)*	1.00	-	-	-	-	-
FI	-0.64 (M)	-0.37 (M)	**0.80 (S)**	**0.87 (S)**	*0.17 (W)*	1.00	-	-	-	-

│E*│ @ 15Hz & 12°C	**0.89 (S)**	**0.74 (S)**	-0.46 (M)	-0.61 (M)	0.48 (M)	-0.61 (M)	1.00	-	-	-
DCT (G_f)	-0.53 (M)	*-0.19 (W)*	0.60 (M)	**0.80 (S)**	*0.07 (W)*	**0.91 (S)**	-0.60 (M)	1.00	-	-
FST	-0.47 (M)	*-0.11 (W)*	0.43 (M)	0.66 (M)	*-0.06*	**0.84 (S)**	-0.62 (M)	**0.94 (S)**	1.00	-
│E*│ @ 15Hz & -18°C	0.60 (M)	0.50 (M)	-0.56 (M)	-0.61 (M)	*0.25 (W)*	-0.60 (M)	**0.80 (S)**	-0.52 (M)	-0.57 (M)	1.00

SUMMARY, CONCLUSIONS AND FUTURE WORK

The time and costs associated with conducting multiple performance tests during mix design iterations may be one of the biggest challenge in routine use of performance engineered mix design approach. Therefore, it is necessary to determine the correlations between different performance index parameters and make engineering based adjustments to the mixture design prior to conducting multiple time consuming and expensive tests. In this study, a total of 14 commonly used asphalt mixtures in New Hampshire were evaluated using the complex modulus (E*), resilient modulus (M_r), direct tension cyclic fatigue (S-VECD), Illinois semi-circular bend (SCB-IFIT), and disk-shaped compact tension (DCT) tests to evaluate the correlations between various performance indices and mix design properties using advances statistical analysis techniques. In addition, three different dynamic modulus based performance index parameters were proposed and evaluated in terms of their correlations to other destructive performance tests. The important results and observations of the study are summarized as follows:

- The cumulative percent passing 4.75 mm sieve size, RBR and gyration level have significant effects on all three dynamic modulus based performance index parameters.
- The cumulative percent passing 4.75 mm sieve size, binder type and RBR can significantly affect the resilient modulus.

- While an increase in cumulative percent passing 4.75 mm sieve size can improve the fatigue life the percent passing 0.075 mm sieve size has an adverse effect on fatigue performance.
- The increase in binder useful temperature interval (UTI) can improve the fatigue performance.
- Increase in the effective binder volume (V_{be}) and UTI can improve the SCB flexibility index.
- The low temperature fracture energy is highly affected by mixture volumterics, binder type and cumulative percent passing 0.075 mm sieve.
- Although the statistical analysis did not indicate the significance of the NMAS, in general, this parameter should still be considered as one of the most important factors in determining the mixtures performance as the total gradation of the mixture is a function of this value. The significance of this parameter was determined when cumulative percent passing 4.75 and cumulative percent passing 0.075 sieve sizes were removed from the regression analysis.
- Resilient modulus has a medium negative correlation with all other index parameters of performance based destructive tests.
- There is a strong positive correlation between N_f @ G^R =100, FI, and low temperature fracture energy.
- The three dynamic modulus based index parameters have medium to high correlations with their respective performance index parameters and can be potentially used as a preliminary checkpoint to adjust the mixture design before conducting other tests.

It is well-known that asphalt mixtures are heterogeneous composites and the interaction among the mix design variables can significantly affect their performance. Therefore future work in evaluating the mix design parameters should include the two and three way interactions of the variables to determine the significant factors. With respect to the three proposed dynamic modulus based performance index parameters, it is necessary to incorporate the phase angle of the response to gain stronger correlations with other performance index parameters and the appropriate threshold values need to be determined with respect to field distress data analysis. Also, as a significant future step, the temperatures at which the modulus values are determined should be adjusted with respect to the specific binder type, modifier and RAP content if applicable.

The use of resilient modulus as the only rutting performance index in this paper may not be able to directly evaluate the mixtures rutting susceptibility as it is conducted in one loading frequency and considers the recoverable deformation, whereas rutting is a measure of permanent deformation, therefore it is necessary to use a more direct measure of rutting through conducting tests such as Hamburg wheel tracking test, asphalt pavement analyzer, flow number, etc.

REFERENCES

[1]. Zhao, Wenbin. "The effects of fundamental mixture parameters on hot-mix asphalt performance properties." (2011).

[2]. Diab, Aboelkasim, and Mahmoud Enieb. "Investigating influence of mineral filler at asphalt mixture and mastic scales." *International Journal of Pavement Research and Technology* 11.3 (2018): 213-224.

[3]. Rahbar-Rastegar, Reyhaneh, and Jo Sias Daniel. "Mixture and production parameters affecting cracking performance of mixtures with RAP and RAS." *8th RILEM International Conference on Mechanisms of Cracking and Debonding in Pavements*. Springer, Dordrecht, 2016.

[4]. Zhou, Fujie, Sheng Hu, and Tom Scullion. *Integrated asphalt (overlay) mixture design, balancing rutting and cracking requirements*. No. FHWA/TX-06/0-5123-1. Texas Transportation Institute, Texas A & M University System, 2006.

[5]. Oshone, Mirkat, et al. "Effect of Mix Design Variables on Thermal Cracki.ng Performance Parameters of Asphalt Mixtures." *Transportation Research Record* (2018): 0361198118797826.

[6]. West, Randy, et al. "Laboratory Refinement and Field Validation of 4.75 mm Superpave Designed Asphalt Mixtures." *NCAT report* (2011): 11-01.

[7]. Nemati, Rasool, and Eshan V. Dave. "Nominal property based predictive models for asphalt mixture complex modulus (dynamic modulus and phase angle)." *Construction and Building Materials* 158 (2018): 308-319.

[8]. Abu Abdo, Ahmad M., and S. J. Jung. "Effects of asphalt mix design properties on pavement performance: a mechanistic approach." *Advances in Civil Engineering* 2016 (2016).

[9]. Bonaquist, Ramon. "Impact of mix design on asphalt pavement durability." *Enhancing the Durability of Asphalt Pavements* 1 (2013).

[10]. Dong, Yuming, and Yiqiu Tan. "Mix design and performance of crumb rubber modified asphalt SMA." *Pavements and Materials: Recent Advances in Design, Testing and Construction*. 2011. 78-86.

[11]. Esfandiarpour, Saman, et al. "Sensitivity of pavement ME design predicted distresses to asphalt materials inputs." *Proceedings of the 2013 Conference and Exhibition of the Transportation Association of Canada-Transportation: Better-Faster-Safer (TAC/ATC'13)*. 2013.

[12]. Bazzaz, Mohammad, et al. "A straightforward procedure to characterize nonlinear viscoelastic response of asphalt concrete at high temperatures." *Transportation Research Record* (2018): 0361198118782033.

[13]. Na Chiangmai, Chaiwat. "Fatigue-fracture relation on asphalt concrete mixtures." (2010).

[14]. Tang, S. (2014). Evaluate the fracture and fatigue resistances of hot mix asphalt containing high percentage reclaimed asphalt pavement (RAP) materials at low and intermediate temperatures.

[15]. Apeagyei, Alex K. "Rutting as a function of dynamic modulus and gradation." *Journal of Materials in Civil Engineering* 23.9 (2011): 1302-1310.

[16]. Ekingen, Erkan Ruhi. *Determining gradation and creep effects in mixtures using the complex modulus test*. Diss. University of Florida, 2004.

[17]. AASHTO T 342, “Standard method of test for determining dynamic modulus of hot-mix asphalt concrete mixtures”, Standard Specifications for Transportation Materials and Methods of Sampling and Testing, (2011).

[18]. ASTM D7369-11: Standard Test Method for Determining the Resilient Modulus of Bituminous Mixtures by Indirect Tension Test, ASTM International, West Conshohocken, PA, 2011.

[19]. Brown, E. Ray, Prithvi S. Kandhal, and Jingna Zhang. "Performance testing for hot mix asphalt." *NCAT report* 1.05 (2001).

[20]. Darabi, Masoud K., et al. "Predicting Rutting Performance of Flexible Airfield Pavements Using a Coupled Viscoelastic-Viscoplastic-Cap Constitutive Relationship." *Journal of Engineering Mechanics* 145.2 (2018): 04018129.

[21]. Bazzaz, Mohammad., “Experimental and Analytical Procedures to Characterize Mechanical Properties of Asphalt Concrete Materials for Airfield Pavement Applications”. University of Kansas, 2018.

[22]. AASHTO, TP. 107-14.(2014). Standard method of test for determining the damage characteristic curve of asphalt mixtures from direct tension cyclic fatigue tests.

[23]. Wang, Yizhuang David, Behrooz Keshavarzi, and Y. Richard Kim. "Fatigue Performance Prediction of Asphalt Pavements with FlexPAVETM, the S-VECD Model, and DR Failure Criterion." *Transportation Research Record* (2018): 0361198118756873.

[24]Wang, Yizhuang, and Y. Richard Kim. "Development of a pseudo strain energy-based fatigue failure criterion for asphalt mixtures." *International Journal of Pavement Engineering* (2017): 1-11.

[25]. Nemati, Rasool, Dave, E. V., Daniel, J. S., Thibodeau, E. S., Worsman, R. K., “Evaluation of Laboratory Performance and Structural Contribution of Cold Recycled Versus Hot Mixed Intermediate and Base Course Asphalt Layers in New Hampshire” *Transportation Research Record (in press).*

[26]. AASHTO TP 124-16. (2016). Standard method of test for determining the fracture potential of asphalt mixtures using semicircular bend geometry (SCB) at intermediate temperature. Washington, DC: American Association of State Highway and Transportation Officials.

[27]. Al-Qadi, Imad L., et al. *Testing protocols to ensure performance of high asphalt binder replacement mixes using RAP and RAS*. Illinois Center for Transportation/Illinois Department of Transportation, 2015.

[28]. ASTM D7313–06. "Standard test method for determining fracture energy of asphalt-aggregate mixtures using the disk-shaped compact tension geometry." ASTM International, April (2007).

[29]. Zhu, Yuefeng, et al. "Comprehensive evaluation of low-temperature fracture indices for asphalt mixtures." *Road Materials and Pavement Design* 18.sup4 (2017): 467-490.

[30]. Mahalanobis, Prasanta Chandra. "On the generalized distance in statistics." National Institute of Science of India, 1936.

Appendix: Paper 2 (Chapter 5)

Title: Evaluation of Laboratory Performance and Structural Contribution of Cold Recycled Versus Hot Mixed Intermediate and Base Course Asphalt Layers in New Hampshire

Journal: Transportation Research Record

Evaluation of Laboratory Performance and Structural Contribution of Cold Recycled Versus Hot Mixed Intermediate and Base Course Asphalt Layers In New Hampshire

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ABSTRACT

Depending on the local conditions and structural design of the pavement, multiple asphalt concrete layers including base, intermediate, and wearing courses are used. Typically, the base and intermediate layers have larger aggregate sizes and lower total asphalt binder contents as compared to the wearing course. Recently, cold recycled (CR) asphalt mixtures have gained attention as an alternative to the typical base, and to some extent intermediate courses, because of economic and environmental advantages. Challenges with CR include the potential high variability of recycled asphalt pavement (RAP) and lack of knowledge in terms of structural contribution and long term performance of such layers. This study investigates 4 different types of CR and 4 hot mixed plant produced asphalt mixtures (3 intermediate courses and 1 base course) that are typical mixtures used in New Hampshire. The laboratory performance evaluation is conducted through the resilient modulus (M_r), complex modulus (E*), semi-circular bend (SCB) and direct tension cyclic fatigue (S-VECD) tests. Pavement performance prediction is carried out using the results from S-VECD approach in the FlexPAVE™ software. The test results indicate that the performance of CR is highly affected by the amount of oil distillate percentage in the emulsion as well as the amount of

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recovered binder in the RAP. While having a relatively lower rutting resistance capability, the CR mixtures maintained an acceptable fatigue performance. As compared to CR mixtures, hot-mixed intermediate and base course mixtures indicated better rutting performance while having lower resistance to cracking.

Keywords: Hot mixed asphalt, cold recycling, intermediate mixture, base mixture, performance prediction

INTRODUCTION AND BACKGROUND

Asphalt pavements are constructed from different types of materials with significantly different behavior which makes them complex structures for analysis and performance prediction. This complexity is not only because of the combination of these materials but also the diversities within the properties of one particular type of material which is a problem that needs to be considered in the design.

Amongst all the materials in the pavement structure, asphalt mixtures are one of the most complicated materials for characterization purposes as they are composite materials with viscoelastic behavior. Depending on the loading and climatic conditions, the asphalt layer thickness can vary from 5 cm to over 30 cm. It is well known that the type and magnitude of stress varies within the depth of the pavement so there is need for different types of asphalt mixtures, each designed to handle specific types and magnitude of stress within the structure. This not only increases the design reliability but can also result in considerable savings in financial resources by optimizing material properties in each layer.

Within the pavement structure, and depending on the design thickness, the asphalt course is generally divided into three sublayers namely the base, intermediate (binder) and top (wearing) layers. The wearing course is usually made of smaller aggregate size and higher binder content to prevent both functional and structural distresses.

The intermediate and base courses contain relatively coarser aggregates and lower binder content. The intermediate layer is placed directly under the wearing course to facilitate the construction of the wearing course and to distribute the traffic loads onto a larger area. This layer increases the overall pavement structural capacity and helps prevent the wearing course from different types of premature distresses (*1,2*).

The asphalt base layer is very similar to the intermediate course in terms of the performance expectations. Base layers are used in addition to the intermediate layer in cases where the load magnitudes and repetitions call for a relatively thicker pavement. In this case the base layer provides a strong foundation for the overlaying lifts to prevent or reduce the risk of rutting and fatigue related distresses.

At the end of the pavement service life, and depending on the overall pavement conditions, it is necessary to take the proper maintenance or rehabilitation action to upgrade the pavement serviceability. In many instances a hot mixed asphalt (HMA) overlay is used to cover the underlying distresses and enhance the pavement performance quality. A major issue with overlaying an aged or cracked pavement is the reflective cracking on the new overlay course. The reflective cracks initiate and propagate because of lateral horizontal movements of the overlay on the cracks as well as the concentrated vertical stress on the existing cracks which eventually result in premature cracks in the overlay (*3, 4*).

To eliminate the risk of reflective cracking, pavement cold recycling (CR) has been used successfully over the last three decades as an alternative to the typical base material and to some extent intermediate layers. The cold recycling can result in up to 100% reuse of RAP as well as reduction in fuel consumption in the mix production process which can significantly reduce the construction costs (*5*).

Significant work has been conducted with different researchers to develop mixture design procedures for different types of emulsified and foamed CR mixtures (6). However most of these procedures are based on Marshal stability, indirect tensile strength or resilient modulus of the mixtures (7) which may not be considered as performance tests. Recent studies have shown an improved pavement life cycle when rehabilitation is conducted using cold-recycling and reclaiming as opposed to use of mill and overlay treatments (*8*). Some important issues are the selection of the appropriate emulsion type, the high variability in the RAP in terms of age and binder chemistry, and the selection of the proper laboratory curing method to simulate the field condition; these all make the mix design and laboratory performance testing a challenging task. Nevertheless, the main structural concern with CR is generally reported to be rutting susceptibility and the required curing time after compaction (*9, 10*). Research studies have been conducted to investigate the fatigue performance of CR mixtures, including use of digital image correlation (*11*). However, it is more common to use the indirect tensile stress test to evaluate the CR fatigue performance (12). The semi-circular bend (SCB) test as an indicator of mixture fracture properties has also been implemented to evaluate the effect of emulsion content on the CR cracking properties (*13*).

The main objective of this study is to evaluate and compare the performance properties and structural contribution of conventional HMA intermediate and base course mixtures with the CR mixtures. This comparison is conducted through laboratory based mechanistic and performance prediction tests such as resilient modulus (M_r), complex modulus (E*), semi-circular bend (SCB), and direct tension cyclic fatigue (S-VECD). In addition, mechanistic pavement modeling is conducted through the finite element based software FlexPAVE ™ to evaluate the structural contribution and the predicted fatigue life of different mixtures in this study. The relative performance of the various CR mixtures is also evaluated with respect to the mixture composition and emulsion quality.

MATERIALS AND METHODOLOGY

In order to evaluate and compare mixture performance, 2 CR mixtures and 4 plant produced HMA mixtures (3 intermediate courses and 1 base course) that are commonly used in New Hampshire were selected. The CR mixtures (denoted as CM-1 and CM-2) are produced by mixing recycled asphalt pavement (RAP) with MS-4 emulsion for 5 minutes using a Wirtgen twin-shaft pugmill mixer (model WLM30) in the laboratory. The MS-4 emulsion is an anionic medium setting emulsion which is specified by New Hampshire Department of Transportation (NHDOT) (*14*). In addition, two emulsions that did not meet the AASHTO T 59 minimum requirement of 2% oil distillate were included in the study to investigate the effect of the oil distillate percentage on performance. These two mixtures are denoted as CM-1-a (1% oil distillate emulsion) and CM-2-a (1.25% oil distillate emulsion). Two different sources of cold central plant recycled (CCPR) material with different nominal maximum aggregate size (NMAS) (19 mm for CM-1 and 12.5 mm for CM-2) were used in fabricating the CR mixtures in the lab. The RAP used in lab testing was sampled from the field and is representative of the material used in actual pavement construction at two sites. The age of the RAP is unknown. Based on the QA documentation, the binder content for RAP used in CM-1 and CM-1-a ranges from 4.8% to 5.45%, while the RAP source used for CM-2 and CM-2-a contains 6.3% binder.

The CR mixtures were fabricated to replicate the mixtures produced for actual construction. The mix designs were conducted by pavement contractors for CM-1 and CM-2 using Marshall stability criteria to determine the optimum binder content. In the CR mixture design procedure the moisture content of the RAP was not altered to achieve the maximum density, rather the design was conducted at the existing moisture level of about 1.75% for both mixtures. The logic for this is that many construction projects are utilizing the cold in-place recycling (CIR) technique where addition of water to the RAP is impractical.

In order to eliminate the effect of compaction method as a variable in comparing the performance of CR and HMA mixtures to each other, the CR specimens were fabricated using a Superpave gyratory compactor to achieve 10±0.5% air void level to replicate typical air void content in the field for these materials. The specimens were cured for 7 days at room temperature and subsequently cured 3 days in an oven at 40°C. The 7-day room temperature curing assures that the specimen has gained enough strength before it is placed in the oven so that it will not fall apart during the second phase of the curing. The oven curing process follows the Wirtgen method that ensures specimens reach a constant mass before testing (*15*).

The selection of the intermediate and base course HMA mixtures is to include typical binder type and NMAS used in the region. The New Hampshire Department of Transportation (NHDOT) divides the state into three different climatic zones where PG 58-34, PG 58-28 and PG 64-28 are generally used for northern, middle and southern parts of the state respectively. Therefore, for this study the intermediate mixtures with relatively similar aggregate gradation and mixture design properties with different binder PG grades were selected from different areas of New Hampshire. This selection facilitates the comparison between the mixtures with respect to the binder type and aggregate size and their effect on the mixture performance separately.

The HMA intermediate and base course materials were sampled in the form of loose mix from plants. The mixtures were reheated and compacted specimens were fabricated using Superpave gyratory compactor at 6±0.5% air void level, which is the typical field value in New Hampshire. The mixture design properties and gradations are shown in Table 1 and Figure 1 respectively. The amount of RAP in the intermediate and base mixtures is reported as the percentage of recycled binder in relation to the total binder content.

The intermediate mixtures have similar mix design and overall aggregate gradation properties. The 25 mm NMAS base mixture has a coarser gradation compared to the intermediate mixtures. There are two curves associated with each of the CR mixtures. The RAP curves are the black rock gradation values before the binder extraction and the aggregate curves are the gradation of RAP aggregate after binder extraction.

TABLE 1 . Mixtures Design Properties

Mix	**B-64-28**	**B-58-34**	**B-58-28**	**BB-64-28**	**CM-1**(1)	**CM-1-a**(2)	**CM-2**(1)	**CM-2-a**(2)
Course	Intermediate	Intermediate	Intermediate	Base	Base	Base	Base	Base
Asphalt Binder/ Emulsion	PG 64-28	PG 58-34	PG 58-28	PG 64-28	MS-4	MS-4	MS-4	MS-4
NMAS (mm)	19	19	19	25	19	19	12.5	12.5
Asphalt (%)	4.8	4.6	4.4	4.8	4.0(3)	4.0(3)	4.0(3)	4.0(3)
V_a(%)	4.0	4.0	4.0	4.0	10.0	10.0	10.0	10.0
VMA (%)	15.0	15.0	14.9	14.8	----	----	----	----
VFA (%)	68.6	67.3	73.1	72.9	----	----	----	----
V_{be}(%)	10.3	10.1	10.9	10.8	----	----	----	----
Gyration	75	50	75	50	35	32	30	35
RAP (%)	20.8	21.7	19.6	20.8	100.0	100.0	100.0	100.0
Emulsion Residue (4) (%)	----	----	----	----	> 65	67.4	> 65	65.6
Penetration at 25°C(5), 100g, 5s	----	----	----	----	> 200	>200	> 200	>200
Oil Distillate (6) (%)	----	----	----	----	2.0 – 7.0	1.00	2.0 – 7.0	1.25

(1) Percentage of oil distillate in the emulsion equal to the minimum requirements of 2% as per NHDOT MS-4 requirements.
(2) Percentage of oil distillate in the emulsion below the minimum requirements of 2% as per NHDOT MS-4 requirements.
(3) Undiluted emulsion amount by weight of total mix
(4) Minimum = 65%
(5) Minimum =200
(6) 2.0 ≤ oil distillate% ≤7.0

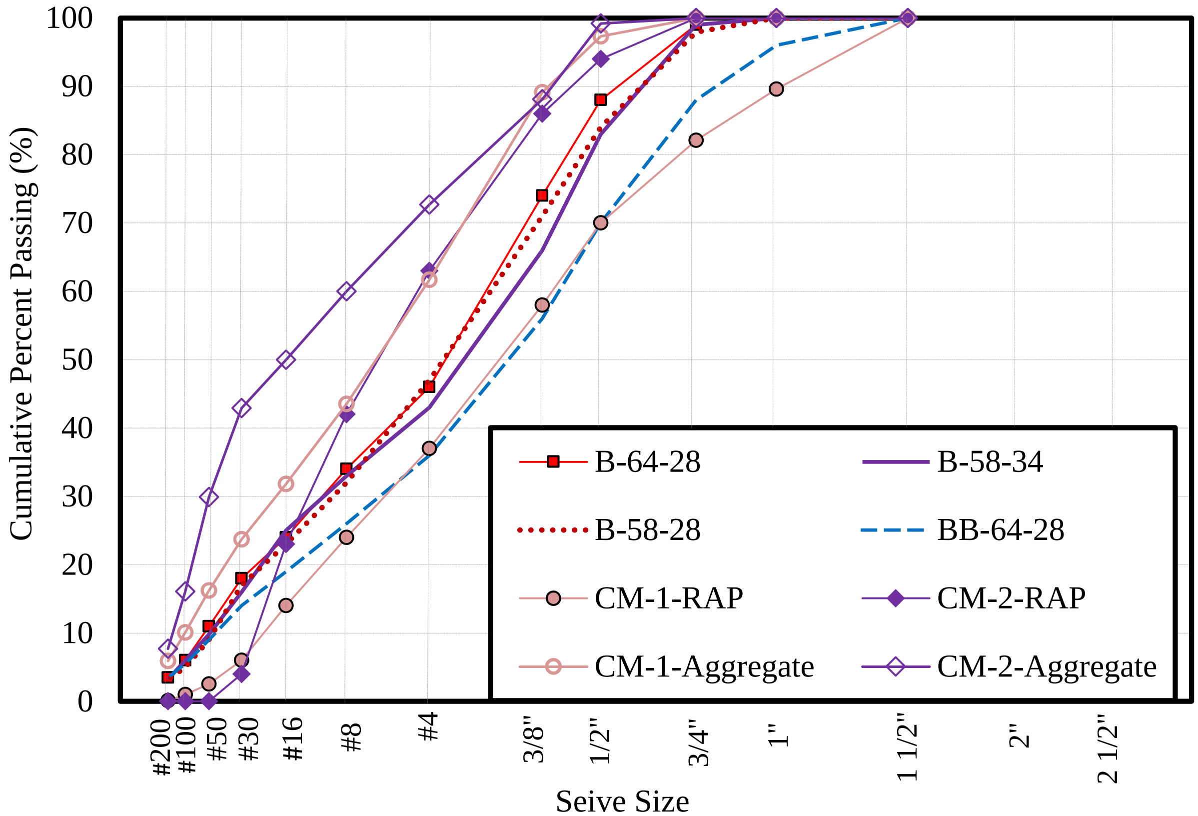

FIGURE 1. Mixture Gradation Properties

TEST RESULTS AND DISCUSSION

This section discusses the results from laboratory based performance prediction tests. The mixture performance evaluation is conducted through resilient modulus (M_r), complex modulus (|E*| and phase angle), semi-circular bend (SCB) and direct tension cyclic fatigue (S-VECD) tests.

Resilient Modulus

The resilient modulus (M_r) test was conducted at 25°C in accordance with ASTM D7369-11 standard test method with three replicate specimens. The results from this test are shown in Figure 2. The three intermediate course mixtures show expected rankings; the B-64-28 with stiffer binder has the highest M_r value and the B-58-34 with softer binder, lower gyrations, and finer gradation is the softest amongst the intermediate mixtures. The BB-64-28 mixture, the only HMA base course mixture, ranks as the second stiffest mixture with respect to M_r test results. Even though this mixture has a coarser gradation and similar binder content compared to B-64-28, the effect of compaction level (50 gyrations for BB-64-28 and 75 gyrations for B-64-28) is evident.

The CM-1 and CM-1-a mixtures have similar modulus values while the CM-2 mixture is softer than the CM-2-a mixture. The effect of the distillate amount changes for the two mixtures. Although CM-2 has a finer gradation and the RAP has more binder compared to CM-1, it has a higher M_r value. These results are likely due to differences in the RAP sources; other studies have shown that the chemical interaction between the emulsion and the RAP binder will be significantly different depending on the age and chemistry of the binder in the RAP (*16*). This illustrates that the RAP age and binder chemistry can have a greater effect than other factors such as the emulsion's oil distillate percentage or even the maximum aggregate size on the CR mixture properties. M_r is correlated with rutting susceptibility (*17*), so the results indicate that the CR mixtures can be expected to have lower rutting resistance than HMA mixtures.

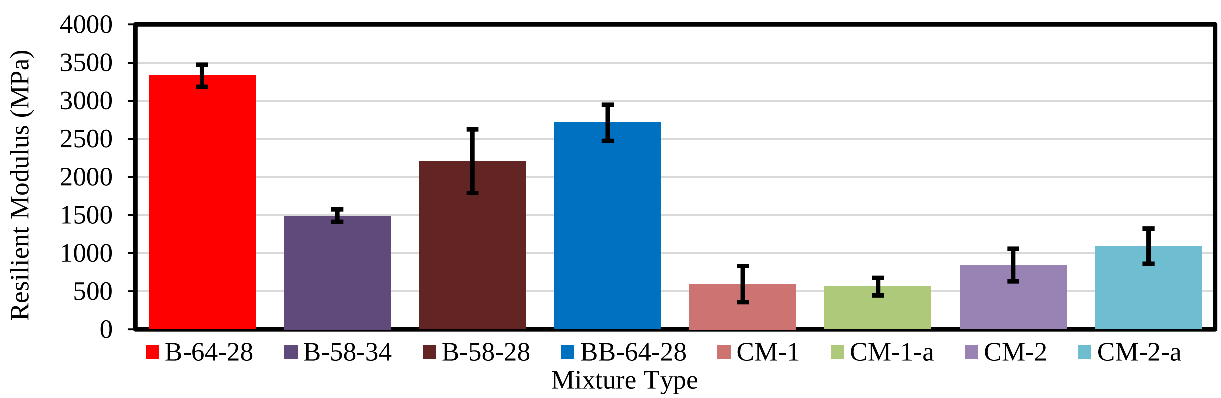

FIGURE 2. M_r Test Results (Error bars represent 1 standard deviation interval)

Complex Modulus (|E*| And Phase Angle)

Complex modulus testing was performed in accordance with the AASHTO T342 standard using an Asphalt Mixture Performance Tester (AMPT). The two components of complex modulus are the dynamic modulus, |E*|, and phase angle, δ. The test was run on 150×100 mm cylindrical specimens at three different temperatures (4.4°C, 21.1°C and 37.8°C) and 6 different loading frequencies (25, 10, 5, 1, 0.5, 0.1Hz). All CR mixtures except CR-1 exhibited substantial permanent deformation (creep) at the 0.1Hz and 37.8°C test condition and this data point was removed from the analysis.

The |E*| and δ master-curves are depicted in Figure 3 and Figure 4, respectively. The master-curves were constructed at a reference temperature of 21.1°C to compare and evaluate the rheological properties. In Figure 3a, the B-64-28 (75 gyration) mixture is the stiffest along all the frequencies whereas the B-58-34 (50 gyration) mixture is the softest through the middle frequencies. The BB-64-28 (50 gyration) mixture is softer than B-64-28 (75 gyration) and B-58-28 (50 gyration) mixtures even though it has a coarser gradation and larger NMAS with similar mixture asphalt content and effective binder volume. This illustrates the effect of design gyration level on the HMA mixture properties.

Similar trends exist with the HMA phase angle master curves. The B-58-28 (75 gyration) and BB-64-28 (50 gyration) mixtures are very similar. The B-58-28 (75 gyration) has a softer binder, finer gradation and smaller NMAS compared to BB-64-28 (50 gyration) mixture (Table 1) While all these factors should result in a softer mix, the higher design compaction level led to a lower binder content, resulting in a decreased phase angle and increased modulus. This shows that binder content (and compaction level) can supersede the effects of larger aggregate size and coarser gradation when considering the mix stiffness.

The CR mixtures have comparable |E*| master-curves with CM-2-a being stiffest of four. The amount of oil distillate percentage has consistent impact on changing mixture stiffness. The CR mixtures with higher oil distillate percent emulsions have lower stiffness, for example, at 21.1°C and 10 Hz, the |E*| of CM-1 mixtures is 20.3% lower than CM-1-a, whereas at same temperature and frequency, CM-2 has a 40% lower |E*| than CM-2-a. The oil distillate percent has a greater effect on the CM-2 complex modulus; this agrees with the M_r results. The difference in the response of the two mixtures is likely due to the effect of interaction between the RAP binder and emulsion.

The modulus values of CR mixtures are approximately half of HMA mixtures and the phase angles are higher in the mid to high frequency ranges. The higher relative viscous component of the response can be considered as a positive feature for CR mixtures which will be discussed in the next sections. On the other hand, and as indicated from the M_r results, the low stiffness of the CR mixtures at high temperatures and low frequencies indicates a greater susceptibility to rutting.

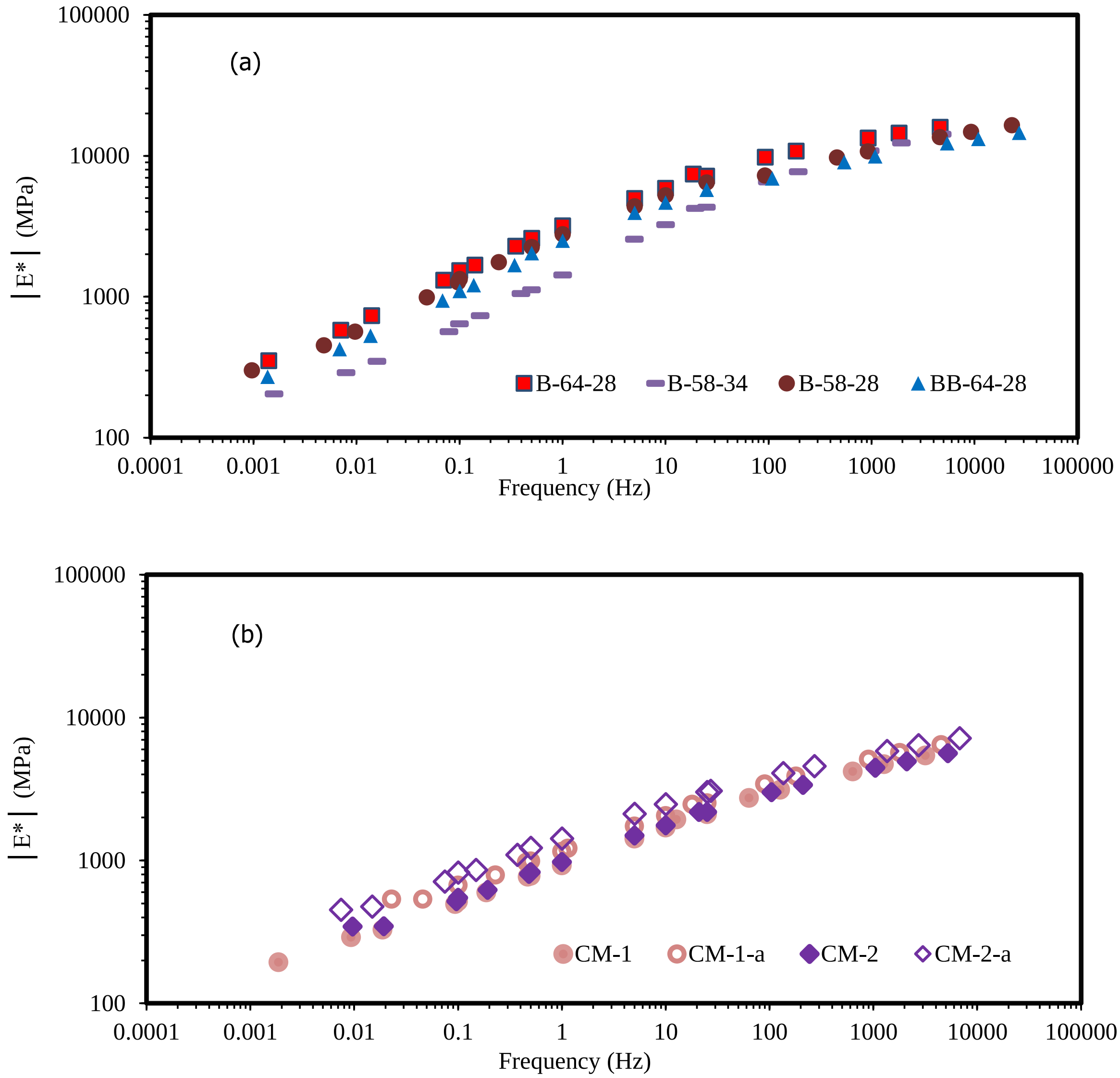

FIGURE 3. Dynamic Modulus |E*| Master-Curves at Reference Temperature of 21.1°C: a) HMA Mixtures, b) CR Mixtures

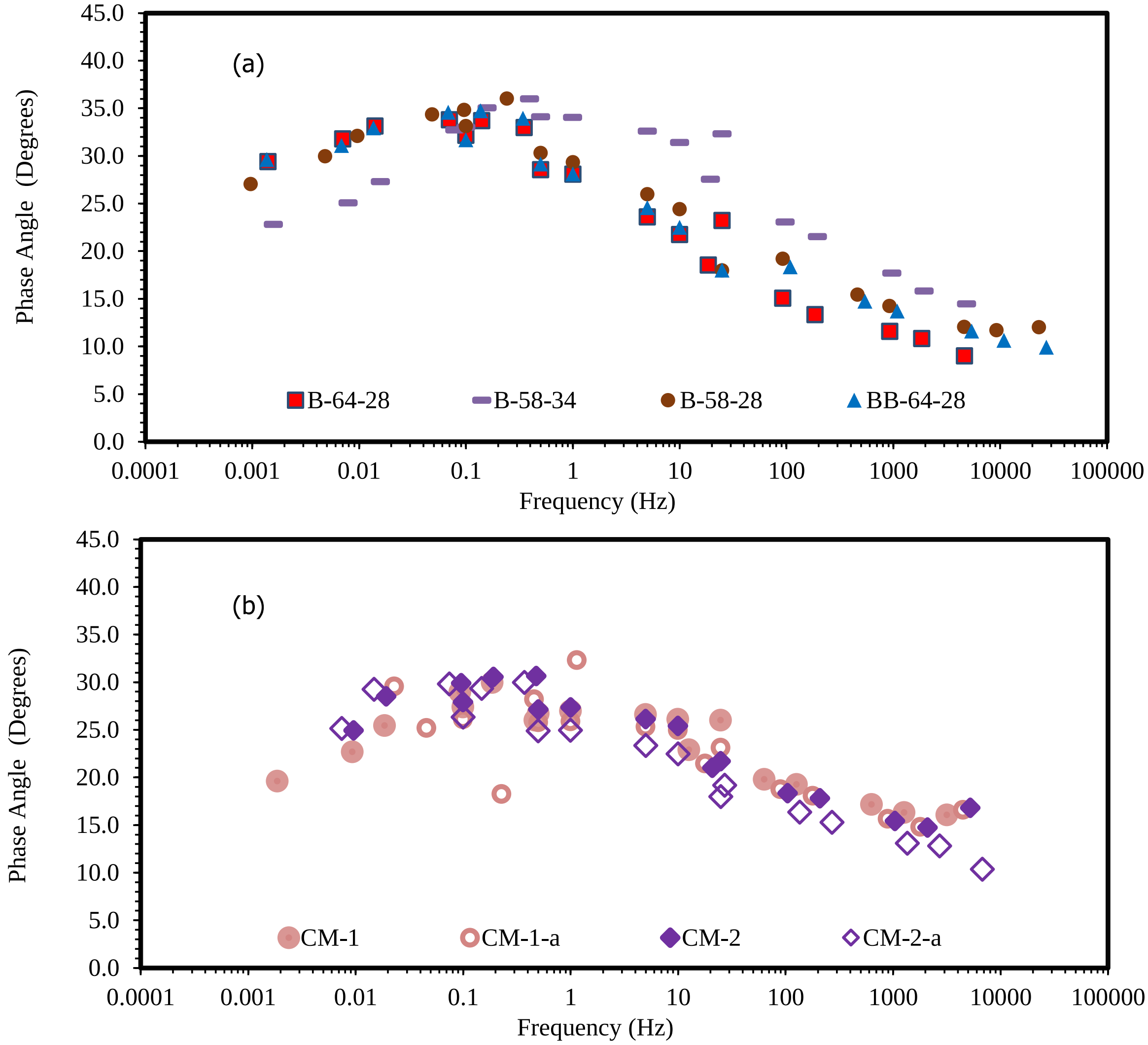

FIGURE 4. Phase Angle Master-Curves at Reference Temperature of 21.1°C: a) HMA Mixtures, b) CR Mixtures

Semi-Circular Bend Cracking Test

Semi-Circular Bend (SCB) testing was conducted to determine the fracture properties of asphalt mixtures in accordance with the AASHTO TP 124 standard. The test was performed using the Illinois method at a line-load displacement rate of 50mm/min at 25°C on 4 replicates (*18*). Figure 5a and 5b show the fracture energy and flexibility index values, respectively. The error bars on the plots indicate one standard deviation from mean. This test is primarily designed to address the fracture properties of mixtures with NMAS less than 19 mm, therefore a relatively high coefficient of variations (COV) for HMA mixtures with 19 and 25 mm NMAS in this study is expected. In general, the HMA mixtures have much larger fracture energy compared to the CR mixtures due to their stiffness and much higher peak load before the crack initiation during the test.

The flexibility index (FI) values show that the stiffer B-64-28 mixture (75 gyration) has the lowest FI and the B-58-28 (75 gyration) and B-58-34 (50 gyration) mixtures have the highest FI values, following the same trend as dynamic modulus. The CR mixtures have FI values that are similar to those for the HMA materials; the CM-1 did not show an inflection point on the post peak side of the load-displacement curve and therefore the FI cannot be calculated for this mixture. However, it should be mentioned that the combination of SCB testing temperature (25°C) and relative softness of the CR mixtures resulted in noticeable creep in specimens, surface indentation was observed in tested specimens under loading head and at supports. This could invalidate the flexibility index calculation. Nevertheless, the higher RAP asphalt content of CM-2 mixtures likely explains higher FI values. The effect of oil distillate percentage is evident in the fracture energies for CM-1 mixtures and in the FI values for the CM-2 mixtures.

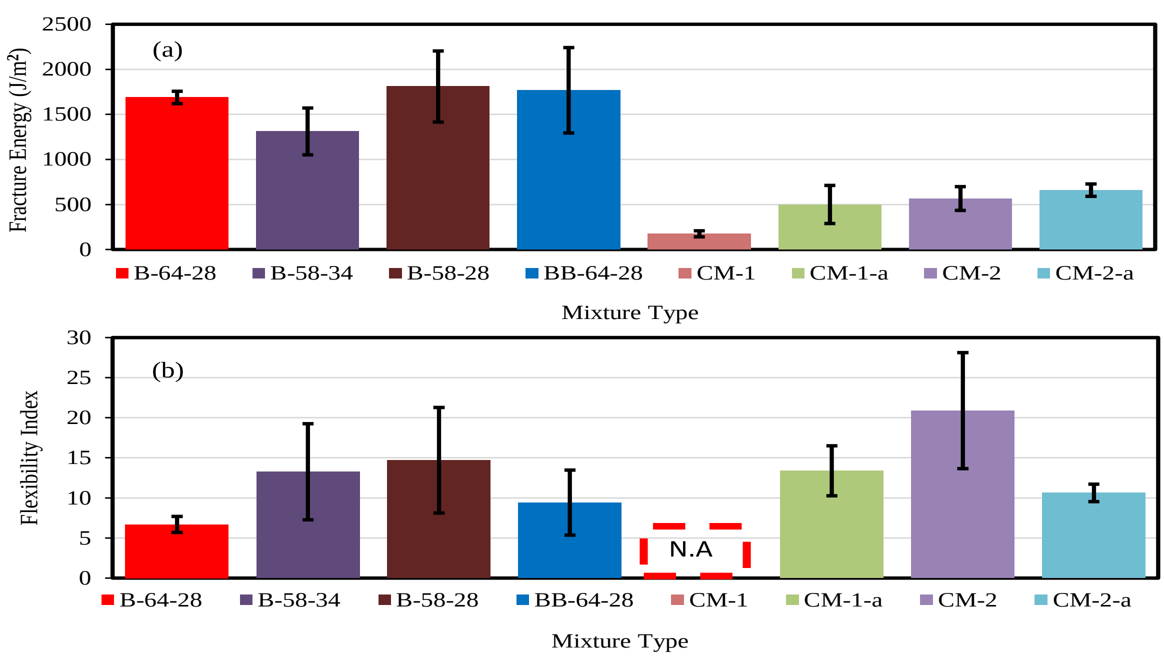

FIGURE 5. (a) Fracture Energy, (b) Flexibility Index (Error bars represent 1 standard

deviation interval)

Direct Tension Cyclic Fatigue (S-VECD)

The simplified viscoelastic continuum damage (S-VECD) approach was utilized in accordance with the AASHTO TP 107 testing method to evaluate fatigue characteristics. Four specimens were tested at different strain levels under direct tension cyclic test in constant crosshead displacement mode. The testing temperature for the HMA mixtures is determined based on the binder PG grade. Since the binders in the CR mixtures do not have a known PG grade, the test temperature was selected to be 21°C for all the CR mixtures.

The damage characteristic curve (DCC) is the main output of S-VECD testing and is a fundamental mix property that is independent of testing temperature and loading mode. The DCC represents the relationship between the asphalt mixture's material integrity (called the Pseudo stiffness, C) and the level of damage over time, S (*19)*. The DCC curve provides important information such as the rate and amount of accumulated damage and the mixture terminal integrity before the crack localization, but cannot reliably be used by itself to rank mixtures (*20*). A direct comparison with respect to these curves not appropriate since the number of cycles to failure (N_f) is missing in between curves and more detailed information is required for a better interpretation of the results. In other words, two mixtures could have very similar DCC curves with similar terminal C and S values, while the rate of damage accumulation due to each loading cycle could be different. Therefore, different S-VECD based fatigue failure criteria have been proposed by researchers to rank mixtures. The two widely accepted failure criteria with the S-VECD approach are the rate of averaged dissipated pseudo strain energy (G^R) and the average reduction in pseudo stiffness (D^R) (*21, 22*). It has been shown that the D^R criterion may better and more reliably explain the mixture fatigue properties as it eliminates the extrapolation problems caused by using the G^R parameter in a logarithmic scale (*23*). As a result, this study uses the D^R criterion to compare the mixtures fatigue performance. D^R is the slope of the accumulated reduction in pseudo stiffness versus number of loading cycles to failure in arithmetic scale and it generally varies from 0.3 to 0.8 where the higher slope value indicates better fatigue resistance (*22*).

The DCCs and D^R failure criterion are shown in Figure 6(a) and 6(b), respectively. The DCCs are clustered by mixture type with the HMA mixture curves further towards the upper right. As mentioned earlier, a direct comparison between the mixtures with respect to DCC curves is almost impossible, however, the fitting coefficients of the DCC curves are used as the primary input in the mixtures structural modeling through S-VECD approach which will be discussed in the next section.

The D^R values are also clustered by mixture type. The HMA mixtures have values ranging from 0.31 to 0.39, which is typically considered poor for fatigue performance (*21*). Although the B-58-34 has a softer binder and is expected to have better fatigue performance than the other HMA mixtures, the lower amount of binder content, slightly higher recycled binder replacement and the stiffening behavior on the mid to high ranges of frequencies on the |E*| master-curve likely contributed to the overall ranking. The relative D^R ranking for the other three HMA mixtures can be directly correlated with their |E*| master-curves. The stiffer mixtures that maintain higher phase angles, such as B-64-28 (75 gyration) and BB-64-28 (50 gyration), are more fatigue resistant mixtures.

The CR mixtures have D^R values ranging from 0.54 to 0.67, which are values typically observed for wearing courses with good to excellent fatigue performance in the lab (*21*). This indicates that it may be beneficial to use CR mixtures closer to the surface course which could result in substantial cost savings. The D^R values show a nearly inverse ranking trend as compared to resilient modulus (Figure 2) with softer mixtures having higher D^R values. Lower distillate oil percentage results in a higher D^R value for CM-1-a mixture while resulting in a slightly lower D^R

value for the CM-2-a mixture.

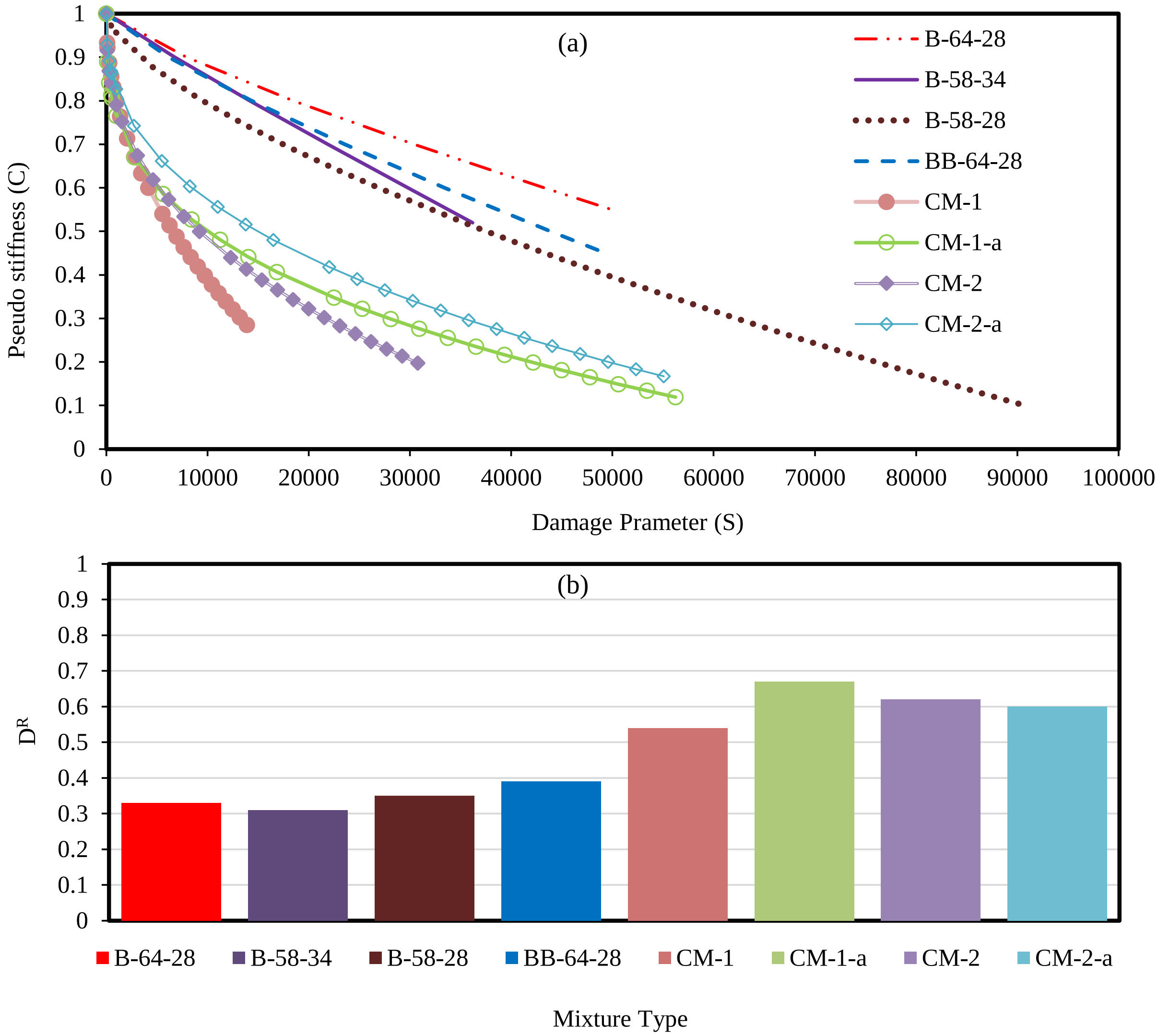

Figure 6. (a)Damage Characteristic Curve C vs S (b) D^R Fatigue Failure Criterion

Structural Modeling

Although failure criteria such as D^R are useful tools to compare and rank the mixtures with respect to the predicted performance, pavement structural modeling is needed to project field performance because the loading and climatic conditions also dictate the amount of damage that occurs. A finite element based pavement structural modeling software, FlexPAVE™ (formerly known as LVECD) is used to model and predict the fatigue performance using the S-VECD model (*24, 25, 26*). The asphalt layer is divided into a mesh of 1000 elements (100×10) to determine the critical response and damage calculations (*27, 28*). The DCC and the D^R value are used by the software to calculate the damage factor (D_F) and determine the number of failed elements (defined as D_F=1). Equation 1 indicates the damage factor calculation for a D^R based analysis in FlexPAVE ™.

$$\boldsymbol{Damage\ Factor, D_F} = \frac{\boldsymbol{1 - C_{ave}}}{\boldsymbol{D^R}} \qquad \text{Equation 5}$$

Where:

C_{ave} = Average pseudo stiffness per cycle up to the current number of loading cycles

D^R= Average reduction in pseudo stiffness up to failure

Simulation Results and Discussion

A typical cross section was selected for the modeling purposes; a commonly used wearing course in New Hampshire is used for all runs while the study mixtures are used as the intermediate/base asphalt layer for comparison purposes. The design parameters, cross section details and the results from FlexPAVE™ analysis are shown in Figure 7. The graph shows the evolution of cracking in terms of number of failed elements (D_F=1) during the 20 year design life. As part of the outputs of the software, the damage level in each finite elements is also provided to track the extent and location of damage at any time during the simulated pavement life. All simulations showed that the predicted distress was a bottom-up type of fatigue cracking and no failure points were predicted in the wearing course.

The analysis shows the two CR mixtures with lower oil distillate percentage (CM-1-a and CM-2-a) to have the best performance with the B-64-28 (75 gyration) and BB-64-28 (50 gyration) mixtures ranked next. As expected from the D^R results, the B-58-34 (50 gyration) mixture is one of the worst fatigue performing mixtures among the study mixtures. This mixture has a slightly higher binder content and higher effective binder volume with a softer binder PG grade as compared to the B-58-28 (75 gyrations) mixture. Intuitively, all these factors should result in a better fatigue resistant mixture. However, compared to other intermediate courses, the B-58-34 (50 gyration) mix has a slightly coarser gradation on the larger aggregate fraction side and a lower design level of gyration which may cause the observed poorer fatigue performance. The CM-1 mixture has the worst performance of all of the mixtures even though it has a relatively high D^R value. However, this mixture also has significantly low fracture energy and the FI could not be calculated from the SCB test results.

In general, the overall ranking of the mixtures does not completely follow the ranking from D^R criterion, but the rankings within each mixture type (HMA vs CR) does. This observation reinforces that although mixture property based failure criteria can be helpful in determining the overall mixture performance, they should not be used for comparison purposes by themselves, especially in cases where the production method is substantially different.

Comparing the FlexPAVE™ simulation to the complex modulus master-curves, the most fatigue resistant mixtures within each category have higher modulus values or phase angles. Considering the induced stress and strains in the pavement structure, the stiffness has a more significant effect on fatigue performance regardless of the production method. This confirms the general mixture design requirements that result in intermediate and base course asphalt mixtures that are stiffer than wearing courses. For instance, the B-64-28 (75 gyration) and BB-64-28 (50 gyration) and CM-2-a and CM-1-a mixtures have the highest modulus among the HMA and CR mixtures, respectively. Intermediate and base course mixtures with higher stiffness have lower tensile strains at the bottom of asphalt layer, resulting in better fatigue performance. However, to make conclusion in a concrete manner, it is important to also consider other factors, such as properties of all pavement sub-layers (granular base, subgrade soil etc.), location of water table and critical distress type, since different circumstances might need a specific type of mixture design for an intermediate and base course layer.

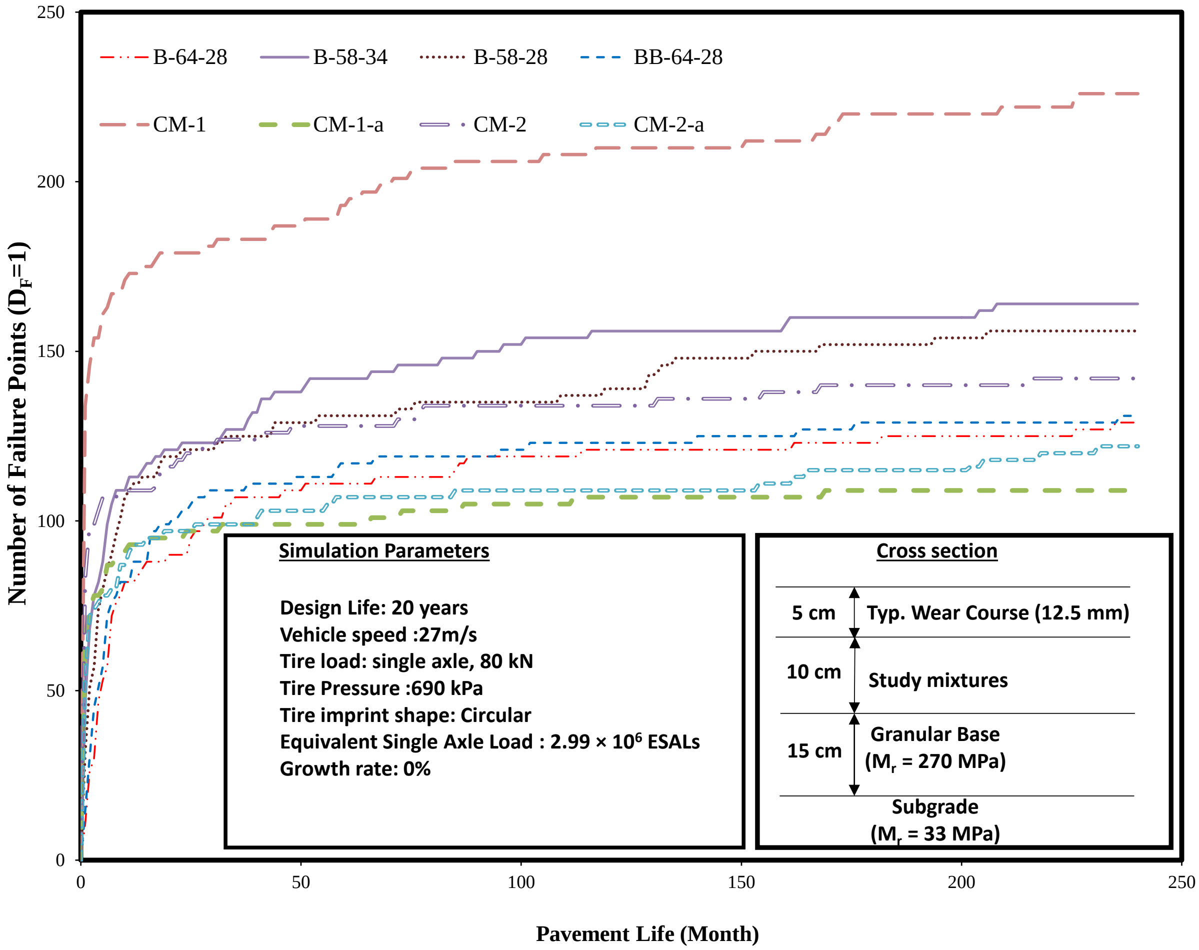

FIGURE 7. FlexPAVE ™ Analysis, Simulation Parameters and Cross Section Details

SUMMARY AND CONCLUSIONS

This study evaluates the lab performance and structural contribution of typically used HMA and cold recycled (CR) base and intermediate course mixtures in New Hampshire. Four HMA mixtures with different binder PG grades (3 intermediate and 1 base) along with four CR mixtures were selected for evaluation. The four CR mixtures were made with two different sources of RAP and emulsion of MS-4 grade. Mechanistic and performance prediction tests such as resilient modulus (M_r), complex modulus (E*), semi-circular bend (SCB), and direct tension cyclic fatigue (S-VECD) tests were conducted in order to compare the mixtures properties with respect to various distress types. Predicted performance of mixtures was compared to the general specification and mixture design properties such as binder type and content, aggregate size and gradation as well as design gyration level for HMA mixtures. The CR mixtures were compared with respect to the RAP binder content and the composition of the MS-4 emulsion in terms of the percentage of oil distillate. To determine the structural contribution of the study mixtures, mechanistic pavement simulations were conducted using the FlexPAVE^{TM} software. A typical cross section was modeled using a conventional HMA wearing course on top and study mixtures as intermediate/base layers in the cross section.

The following conclusion are drawn on the basis of test results and performance evaluations:

- The low stiffness of CR mixtures, as measured by the resilient modulus, indicates that they will be more susceptible to rutting than the HMA mixtures. The complex modulus test results indicated that the CR mixtures have relatively lower stiffness (nearly 50% lower) compared to HMA mixture along all measurement temperatures and frequencies.
- Based on the SCB test results, no consistent trends were observed with respect to the effect of mix properties and compaction level for HMA and CR mixtures. This could be because of relatively higher variability of the test results for CR mixtures with larger NMAS as compared to wearing course mixtures with smaller NMAS.
- The S-VECD fatigue results using the D^R failure criterion indicate that the CR mixtures would be expected to have better fatigue resistance than the HMA mixtures.
- The pavement simulation results agree with the overall expectations of the mixtures with respect to the lab performance test results and mixture design properties (such as better fatigue performance for mixtures with higher binder contents, greater stiffness or stiffer binder types and mixes with higher phase angles). Although the overall ranking from the FlexPAVE^{TM} analysis is slightly different from that of the fatigue failure criterion (D^R),

the ranking within each mix type category (i.e. only HMA or only CR) closely follows the D^R ranking.

- In general, performance of CR mixtures appears to be more influenced by the RAP source, RAP binder properties and emulsion type and content rather than the RAP gradation.
- Lower oil distillate percentage in the emulsion does not appear to have a significantly negative effect on the measured properties of the CR mixtures and in limited cases it might even improve the performance. More data and a specific study of the RAP binder chemistry and emulsion properties is essential to confirm this observation.

Although the lab performance tests and FlexPAVE™ performance predictions can be considered as reliable tools in evaluating and ranking mixtures with respect to the specific distress types, they may not be able to completely simulate the actual field conditions because of the wide variations in the loading and climatic circumstances. As a result, some mixtures may perform significantly different from what is observed through the laboratory testing. Therefore, as a future step in this study, the results from the lab tests and mechanistic simulations should be compared to the field distress data to confirm the conclusions drawn in this research. The mixtures used in this study have been placed in the field as part of pavement projects that were constructed in 2016 and 2017. Continued monitoring of these projects is planned and in future field performance will be compared with the results presented herein. The field data is also essential to adjust the mix design properties with respect to the desired performance of intermediate and base course mixtures. In order to make comparisons between the mixtures, a single identical pavement cross-section was utilized in this study, in future it is recommended that different typical cross-sections, traffic levels and locations be evaluated to develop recommendations for use of intermediate/base courses that are tailored to specific pavement cross-section types, location and traffic levels. There are different types of additives such as cement and lime that are used to stabilize the CR mixtures and improve their rutting performance, therefore, in future it is of utmost importance to evaluate the effect of such additives in fatigue performance of CR mixtures.

Due to the increasing traffic loads along with higher demands for greater amounts of RAP in the mixtures, the asphalt mixture design processes are moving towards use of performance tests in determining the optimum binder content to satisfy multiple competing distresses such as rutting and cracking at the same time. The results from this research can help pavement engineers to gain a better perspective of the influence of mix design parameters in the mixtures performance while it can facilitate the development and implementation of performance-engineered mix design for intermediate and base course layers.

ACKNOWLEDGEMENTS

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AUTHOR CONTRIBUTION

The authors confirm contribution to the paper as follows: study conception and design: all authors; data collection: R. Nemati, E. Thibodeau; analysis and interpretation of results: all authors; draft manuscript preparation: R. Nemati. All authors reviewed the results and approved the final version of the manuscript.

REFERENCES

1. Wang, D., C. Zhou and D. Feng. Calibration on MEPDG Low Temperature Cracking Model and Recommendation on Asphalt Pavement Structures in Seasonal Frozen Region of China. Advances in Materials and Engineering, 2015 (3). 1-11.
2. Garcia, J. and K. Hansen. HMA Mix Type Selection Guide. Information Series 128. National Asphalt Pavement Association, Lanham, MD., 2001.
3. Babaei, K., J. P. Walter. Evaluation of the Performance of Cold-Mix Recycled Asphalt Concrete Pavement in Washington 1998. Final Report WA-RD 201.1
4. R. Rogers, M. Plei, N. Aguirre, A. Taghavi Ghalesari. Optimizing Structural Design for Overlaying HMA Pavements with a Concrete Pavement Using Critical Stresses/Strains. TRB 11th University Transportation Center (UTC) Spotlight Conference: Rebuilding and Retrofitting the Transportation Infrastructure, Washington D.C. 2017.
5. Chen, D., C. Jahern. Evaluation of Long Term Performance of Cold In-Place Recycled Roads: Field and Laboratory Testing, 2007. Final Report IHRB Poject TR-502
6. Recycling, C. (2010). Wirtgen cold recycling technology.
7. Kim, Y., Lee, H. D., & Heitzman, M. (2007). Validation of new mix design procedure for cold in-place recycling with foamed asphalt. *Journal of Materials in Civil Engineering*, *19*(11), 1000-1010.
8. Dave, E.V., C. Hoplin, B. Helmer, J. Dailey, D. Van Deusen, J. Geib, S. Dai, and L. Johanneck. Effects of Mix Design and Fracture Energy On Transverse Cracking Performance of Asphalt Pavements in Minnesota. Transportation Research Record, 2576, pp. 40-50, 2016. http://dx.doi.org/10.3141/2576-05
9. Babagoli, R., A. Ameli and H. Shahriari. Laboratory Evaluation of Rutting Performance of Cold Recycling Asphalt Mixtures Containing SBS Modifies Asphalt Emulsion. Petroleum Science and Technology, 2016. 34:4, 309-313
10. Fathi, A., Tirado, C., Gholamy, A., Lemus, L., Mazari, M., and Nazarian, S. Consideration of Depth of Influence in Implementation of Intelligent Compaction in Earthwork Quality Management. Transportation Research Board, 2018, Washington, DC, United States, 18-02100
11. Gao, L., F. Ni, C. Ling and J. Yan. Evaluation of Fatigue Behavior in Cold Recycled Mixture Using Digital Image Correlation Method. Construction and Building Materials, 2015. 102 (1), 393-402.
12. Yan. J., F. Ni, M. Yang and J. Li. An Experimental Study on Fatigue Properties of Emulsion and Foam, 2010. 24 (11), 2151-2156.
13. S. Charmot., W. Wenlong. X. Xu. Feasibility of Cold Recycled Asphalt Mixture Design Optimization on the Basis of Fracture Properties Using the Semi Circular Bending (SCB) at Intermediate Temperature. Transportation Research Record, 2017. No. 17-06335.
14. New Hampshire Department of Transportation. Standard specifications for road and bridge construction. Section 702, Division 700. Edition 2010.
15. Valentin, J., P. Mondschein., J. Fiedler., K. Mollenhauer., F. Batista and A. C. Freire. Report on Incorporation of Cold Recycled Pavement Layers in Empirical and Mechanistic pavement

Design Procedure. CEDR Transnational Research Programme, 2014.
16. Du, S. Performance Characteristic of Cold Recycled Mixture with Asphalt Emulsion and Chemical Additives. Advances in Materials Science and Engineering, 2015.
17. Brown, E.R, P.S. Kandhal and J. Zhang. Performance Testing for Hot Mix Asphalt, 2001. NCAT Report No. 01-05. National Center for Asphalt Technology. Auburn, AL.
18. Haslett, K. E. Evaluation of Cracking Indices for Asphalt Mixtures Using SCB Tests at Different Temperatures and Loading Rates. 2018
19. Park, S. W., Y. R. Kim., R. A. Schapery. A Viscoelastic Continuum Damage Model and its Application to Uniaxial Behavior of Asphalt Concrete. Mechanics of Material, 1996. 24, 241-255.
20. Gibson, N., and X. Li. Characterizing cracking of asphalt mixtures with fiber reinforcement: use of cyclic fatigue and direct tension strength tests. Transportation Research Record: Journal of the Transportation Research Board 2507 (2015): 57-66.
21. Zhang, J., M. Sabouri., M. Guddati. Development of a Failure Criterion for Asphalt Mixtures under Fatigue Loading. Road Materials and Pavement Design, 2013. 14, 1-15.
22. Wang, Y., Y. R. Kim., Development of a Pseudo Strain Energy-Based Fatigue Criterion for Asphalt Mixtures, 2017. International Journal of Pavement Engineering, 2017.
23. Wang, Y. D., Keshavarzi, B., & Kim, Y. R. (2018). Fatigue Performance Prediction of Asphalt Pavements with FlexPAVE™, the S-VECD Model, and DR Failure Criterion. Transportation Research Record, 0361198118756873.
24. Oshone, M., Mensching, D. J., Daniel, J. S., & McCarthy, L. M. Comparative evaluation of mechanistic–empirical performance models as a tool for establishing pavement performance specifications. Road Materials and Pavement Design, 2018, 1-19.
25. Daniel, J. S., Corrigan, M., Jacques, C., Nemati, R., Dave, E. V., & Congalton, A. Comparison of asphalt mixture specimen fabrication methods and binder tests for cracking evaluation of field mixtures. Road Materials and Pavement Design, 1-17, 2018.
26. Nemati, R., & Dave, E. V. Nominal property based predictive models for asphalt mixture complex modulus (dynamic modulus and phase angle). Construction and Building Materials, 158, 308-319, 2018.
27. Lucas F. de A. L. Babadopulos, Jorge B. Soares, Jorge Luis S. Ferreira & Luis Alberto H. do Nascimento. Fatigue cracking simulation of aged asphalt pavements using a viscoelastic continuum damage model, Road Materials and Pavement Design, 2018, 19:3, 546-560.
28. Rahbar-Rastegar, R., Sias Daniel, J., & Reinke, G. Comparison of asphalt binder and mixture cracking parameters. Road Materials and Pavement Design. 2017 Design, 18(sup4), 211-233.

Appendix: Paper 3 (Chapter 6)

Title: Nominal Property Based Predictive Models for Asphalt Mixtures Complex Modulus (Dynamic Modulus and Phase Angle)

Journal: Construction and Building Materials

NOMINAL PROPERTY BASED PREDICTIVE MODELS FOR ASPHALT MIXTURE COMPLEX MODULUS (DYNAMIC MODULUS AND PHASE ANGLE)

[1]Rasool Nemati, [2]Eshan V. Dave

ABSTRACT

Dynamic modulus (|E*|) and phase angle (δ) are necessary for determining the response of asphalt mixtures to in-[3]service traffic and thermal loadings. While a number of |E*| and δ predictive models have been developed, many of them require lab measured properties (e.g. binder complex modulus). The majority of previous work has focused only on prediction of |E*|, limited models exist for prediction of δ. This research utilized generalized regression modelling of lab measurements (from 81 asphalt mixtures) to develop and verify prediction models for |E*| and δ using only nominal asphalt mix properties that are readily available during the initial mixture design and specification process.

Paper Highlights

- Provides models that use only nominal inputs to make reliable property estimates during design phase
- Presents generalized regression framework for developing asphalt property prediction models
- Model is verified through statistical comparisons and comparisons with other predictive models
- Application of proposed model for pavement performance prediction is demonstrated

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Key Words: Dynamic modulus, phase angle, prediction models, asphalt mixture, generalized regression.

INTRODUCTION AND BACKGROUND

Complex modulus (E*) is one of the most commonly used property of asphalt mixtures for conducting pavement analysis and modelling. Two components of complex modulus are, dynamic modulus (|E*|), which describes materials stiffness at given temperature and frequency, and phase angle (δ), which describes the extent of viscous and elastic behavior of the material at a given temperature and loading frequency. Laboratory measurements of |E*| and δ are commonly done at different temperature and frequency combinations using AASHTO T342 procedure. An |E*| master curve is the primary asphalt mixture input in the current AASHTO PavementME design procedure.

Although |E*| and δ can be effectively used to predict the long term performance of asphalt mixtures using mechanistic analysis, there are limitations related to equipment requirements, specimen fabrication complexity, data analysis and other expenses in terms of man-power and time requirements. These limitations have severely restricted wide-spread usage of mechanistic empirical and mechanistic pavement analysis and design. In order to alleviate expensive and time-consuming laboratory testing requirements, a number of predictive equations for |E*| have evolved during the last three decades. Two of the most popular predictive equations for dynamic modulus are the Witczak model (*1*) and the Hirsh model (*2*). Most of these predictive equations are based on regression analysis of large datasets and use the volumetric properties of mixtures along with the binder dynamic shear modulus (G*) as their primary input. While there are several models to predict |E*|, there have been far fewer attempts to predict δ.

A distinguishing factor the for research and the prediction model presented herein as compared to previous research is that here only nominal properties of asphalt mixtures, such as nominal maximum aggregate size, air void content, asphalt content, the percentage of recycled asphalt pavement (RAP) and recycled asphalt shingles (RAS) and asphalt binder performance grade (PG) are used in model development. These parameters are often readily available during the initial phases of asphalt specification and mix design process. The use of PG grades in lieu of other rheological properties of binder like viscosity and complex shear modulus (G*) as a continuous factor in the model is logical, since binder PG grade (if not modified) has its own definite rheological characteristics that will impact mixture stiffness. For example, a PG 64-28 in the same temperature and loading condition for a given mix is expected to result in a stiffer (higher |E*| and lower δ) mixture compared to a PG 58-34. This simply means that actual rheological performance of a binder is expressed by the binder's PG grade. Therefore, the information based on PG can be utilized in a predictive model to capture the viscoelastic behavior of the mix. The use of NMAS instead of gradation of the aggregate relies on the fact that any dense graded aggregate with a given NMAS has to be in a specified gradation band to be adopted for construction purposes. Thus, the NMAS itself could be an indicator of the general gradation and can be used as a predictor in the model. Using these simple properties as effective factors in the model, the outcome not only eliminates the need for even simple lab tests, but also provides the pavement design engineers with a tool for specifying asphalt mixture that would yield the best performance and the lowest cost-benefit ratios. The development of phase angle prediction model used the same nominal mix properties as described, with the exception of |E*|. This additional variable in prediction of δ was deemed necessary to be included during the initial model development trials. The existence of this variable is inevitable since δ is related to |E*| as discussed by Rowe et al. (*3*) and Oshone et al. (*4*).

In the initial development of the model, this research used lab measured |E*| values for the prediction of δ, however, the proposed model can effectively use predicted |E*| values

As one of the most comprehensive equations for prediction of |E*|, the Witczak 2006 model (*1*) shown below in Equation 1 is applicable over a wide range of temperatures and frequencies. This model is a revised version of the Witczak 1999 model in which the viscosity-temperature susceptibility (VTS) method which assumes a linear relationship between temperature and log of viscosity is implemented to characterize the behavior of mixture. This assumption is generally valid for unmodified binders. However, for modified binders it may not be applicable. Thus, this approach could suffer from lack of accuracy when used for characterization of viscoelastic behavior of modified binders (*5*). Several studies have been conducted to calibrate these predictive models based on local mixtures and binder types (6). The Hirsch model alleviates some of these short-comings by using binder G* which is applicable for both modified and conventional binders

$$Log|E^*| = -0.349 + 0.754(|G_b^*|^{-0.0052})\left(\begin{array}{l} 6.65 - 0.032P_{200} + 0.0027{P_{200}}^2 + 0.011P_4 - 0.0001{P_4}^2 \\ +0.006P_{38} - 0.00014{P_{38}}^2 - 0.08\, V_a - 1.06\left(\frac{V_{beff}}{V_{beff}+V_a}\right) \end{array}\right)$$

$$+\frac{2.558+0.032\, V_a+0.713\left(\frac{V_{beff}}{V_{beff}+V_a}\right)+0.0124P_{38}-0.0001{P_{38}}^2-0.0098P_{34}}{1+e^{(-0.7814-0.5785 log\left|G_b^*\right|+0.8834\, log\, \delta_b}} \quad (1)$$

Where:

$|E^*|$ = Asphalt mix dynamic modulus (psi),

V_a = Air voids in the mix (% by volume),

V_{beff} = Effective binder content (% by volume),

P_{200} = % Passing # 200 (0.075 mm) sieve,

P_4 = Cumulative % retained on # 4 (4.75 mm) sieve,

P_{38}= Cumulative % retained on 3/8 inch (9.5 mm) sieve,

P_{34} = Cumulative % retained on 3/4 inch (19 mm) sieve.

$|G_b^*|$ = Dynamic shear modulus of asphalt binder, (psi)

δ_b = Phase angle of binder associated with $|G_b^*|$, (degree)

The Hirsch model (*2*) is based on the Paul's law of mixtures, which combines series, and parallel elements of the material phases. According to this law, asphalt concrete tends to behave like a series composite at high temperatures and as a parallel composite at lower temperatures. Equation (2) denotes the Hirsch model for predicting |E*|**.**

$$|E^*| = P_C\left[4200000\left(1-\frac{VMA}{100}\right)+3|G^*|_{binder}\left(\frac{VFA.VMA}{10000}\right)\right]+(1-P_C)\left[\frac{1-\frac{VMA}{100}}{4200000}+\frac{VMA}{VFA.3|G^*|_{binder}}\right]^{-1} \quad (2)$$

And,

$$P_c = \frac{\left[20+\frac{VFA\,.\,3|G^*|_{binder}}{VMA}\right]^{0.58}}{650+\left[20+\frac{VFA\,.\,3|G^*|_{binder}}{VMA}\right]^{0.58}} \quad (3)$$

Where:

$|E^*|$ = dynamic modulus, (psi)

$|G^*|_{binder}$ = binder dynamic modulus, (psi)

VMA= voids in the mineral aggregate, (%)

VFA = voids filled with asphalt, (%)

P_C = aggregate contact factor

Recently some new approaches have been developed to predict |E*| using artificial intelligence tools and one of them is the Artificial Neural Networks (ANN) method (*7*). While this method has shown promising results with a high accuracy of prediction, there are some shortcomings such as, low convergence speed as well as lack of generalizing performance. In other words, even small changes in the input of the model could cause major effects in the model response. Furthermore, they might encounter an overfitting problem (*8*). Dynamic modulus has also effectively been predicted using the rheological models like Burger's and Huet-Sayegh model (*9*). Other models have been constructed based on viscoelastic and time–temperature superposition concepts (*10*). Finite element based predictive models have been developed to predict dynamic modulus through modeling the effect of random aggregate arrangement during the compaction (*11*).

A well-known predictive equation for δ is based on nonlinear regression analysis (*12,13*). There are two major limitations to this model, the first being that it uses 16 variables to build up the model that could be decreased. Secondly, this model uses two different regression equations to construct the δ master curve resulting in a break point at the peak value of the master curve which causes non-continuity at that point (*14*).

This study presents a practical and simple approach to developing |E*| and δ prediction models using generalized regression analysis. Using this approach, |E*| and δ models that utilize only nominal properties of HMA mixtures have been developed for New England region of the United States. These properties are readily available during any preliminary mixture design procedure, which means that there would be no requirements for any type of lab tests. The attributes of the mixtures used in this study as well as a brief description of generalized regression platform and model development is discussed next in the paper. The predictive models are evaluated statistically and their actual field applicability was assessed using a case study, these are presented later in the paper.

RESEARCH APPROACH AND MATERIALS

This study utilized 81 asphalt mixtures with diverse volumetric and constituent properties. All mixtures were designed according to the Superpave procedure (*15*) and tested following the AASHTO T342 (*16*) procedures in unconfined condition using an Asphalt Mixture Performance Tester. The mixtures represent materials from the New England region of the United States. Each test was conducted with three replicate specimens tested at three temperatures and six loading frequencies. Among the whole dataset, there are 27 mixtures that have been manufactured with the usage of modified binder and implementing the warm mix asphalt (WMA) production technology. The mixture attributes and the test parameters are shown in Table 1.

Along with the variable selection for the predictive models, principal component analysis (PCA) was performed on the correlations of variables and lab measured values of |E*| and δ. Correlation values from this analysis are presented in Table 2. The values shown in the table indicate the dependence of one factor on another in a numerical manner. Negative correlation coefficients indicate that the two variables are inversely dependent on each other. This type of analysis allowed for better understanding the relationship between the selected factors and the responses. Initially, it might appear that the correlations of individual independent variables with the |E*| and δ is low. It should be noted that in asphalt mixtures, which are composite materials, it is the interaction of the individual variables that has a significant effect on both |E*| and δ. In other words, |E*| and δ are direct functions of a mix design which is a combination of all the variables (Va, AC, NMAS, RAP, RAS etc.). Therefore, a general expectation of a high correlation between the response and individual mix related variables might not be true. For example, it was observed that the PGHT did not have a significant effect on the model whereas its interaction with the logarithm of temperature is an effective term. Additionally, using these uncorrelated factors helps avoid the multi-collinearity problems, which might cause erratic p-values for the independent variables as well as incorrect relationship between the predicted response and the predictors. On the other hand, the temperature and loading frequency (referred to as the test related variables) can individually affect the viscoelastic behavior of the mix (*17*) and as Table 2 shows these two variables have high correlations with the response. The significance of the variable interactions is presented in the model development section.

Table 1- Scope of mixture attributes and test parameters in the dataset

Predictive Model	Type of Factor	Name of Factor	Abbreviation	Number of Levels	Level (Number of mixes)	Min	Max
E* and δ	Mixture Related	Nominal Maximum Aggregate Size (mm)	NMAS	3	9.5 (37), 12.5 (35), 19 (9)	----	----
		Air Void of Total Mixture (%)	Va%	23	----	3%	9.63%
		Total Asphalt Content (%)	AC%	14	----	4.70%	6.80%
		Recycled Asphalt Pavement (%)	RAP	13	----	0%	40%
		Recycled Asphalt Shingle (%)	RAS	2	12.2 (4), 11.1 (2)	----	----
		Binder High Temperature PG grade (°C)	PGHT	3	64 (49), 58 (16), 52 (16)	----	----
		Binder Low Temperature PG grade (°C)	PGLT	3	-34 (16), -28 (61), -22 (4)	----	----
	Test Related	Logarithm of Loading Frequency (Hz)	Log(Frequency)	6	0.1, 0.5, 1, 5, 10, 25 (81 at each)	----	----
		Logarithm of Test Temperature (°C)	Log(Temperature)	3	3.9, 20, 35 (29); 4.4, 21.1, 37.8 (52)	----	----
δ	Material Mechanical Property	Logarithm of Dynamic Modulus	Log(E*)	----	----	----	----

Table 2- Correlation matrix of variables

Variables	Log(E* (lab measured))	Phase Angle	Log (Temperature)	Log (Frequency)	AC%	NMAS	Va%	RAP%	RAS%	PGHT	PGLT
Log(E* (lab measured))	1.00	----	----	----	----	----	----	----	----	----	----
Phase Angle	-0.67	1.00	----	----	----	----	----	----	----	----	----
Log (Temperature)	-0.79	0.75	1.00	----	----	----	----	----	----	----	----
Log (Frequency)	0.40	-0.12	0.07	1.00	----	----	----	----	----	----	----
AC%	-0.06	-0.09	-0.06	0.01	1.00	----	----	----	----	----	----
NMAS	0.04	0.13	0.04	-0.01	-0.85	1.00	----	----	----	----	----
Va%	-0.07	-0.01	-0.02	0.02	0.07	-0.14	1.00	----	----	----	----
RAP%	0.06	0.06	0.02	-0.02	-0.21	0.23	-0.08	1.00	----	----	----
RAS%	0.003	0.05	0.01	0.01	-0.38	0.49	-0.08	-0.12	1.00	----	----
PGHT	0.06	-0.09	0.02	-0.01	0.27	-0.36	0.11	-0.36	-0.32	1.00	----
PGLT	0.11	-0.06	0.06	-0.02	-0.03	-0.13	0.10	-0.14	-0.21	0.83	1.00

The analysis to build predictive models presented in this study was conducted using JMP PRO® software which is a statistics analysis tool. Generalized regression platform allows for fitting of penalized generalized linear models to data sets. The models are penalized in the sense that a penalty is added to the likelihood of the model. The penalizing equivalently constrains the sum of absolute values of the estimates and causes some of them to turn out to be zero, which helps in eliminating the redundant variables. Depending on the form of the penalty, this allows variable selection as well as shrinkage of estimators. Generalizing the models eliminates the need of normal distribution of response. This is useful, since in many instances there are responses which are not normally distributed. Generalized regression approach uses relationships between the dependent and independent variables using coefficients that can vary with respect to one or more grouping variables for non-normally distributed situations (*18*).

Use of penalized regression also helps in lowering the number of effective terms in a model. In standard linear regression, having higher numbers of effective terms can easily cause an issue that is commonly referred to as overfitting. This means that the model will fit the observed data very well, but it will perform poorly on new observations. If a model is optimized by penalization, there would be certain benefits such as the better prediction of data by avoiding overfitting, as well as easier interpretation of the resultant model. The two main penalization methods are the “Lasso” and “Elastic-net”; both of these methods shrink some predictors to a nil or zero value. The Lasso method will tend to give a more parsimonious model than the elastic-net, while the Elastic-net can better handle collinearity than the lasso. Simulation studies and real data examples show that the Elastic-net method often outperforms the Lasso method in terms of prediction accuracy. In addition, the Elastic-net encourages a grouping effect, where strongly correlated predictors tend to be in or out of the model together (*18,19*).

The Elastic-net procedure was selected in this study as a penalization method due to its superiority in terms of variable selection and prediction accuracy. Optimal penalty values can be determined using different validation methods. In the present work, Bayesian Information Criterion was used due to its computational efficiency and ability to result in a parsimonious model (*19*).

DEVELOPMENT OF DYNAMIC MODULUS (|E*|) AND PHASE ANGLE (δ) PREDICTION MODELS

Prior to determination of the predictive models, the Mahanalobis distance analysis (*20*) was conducted on the whole dataset to assess data quality and to statistically identify outliers. Using this analysis, 237 data-points were excluded from the analysis resulting in a substantially unified dataset and an unbiased predictive model. Since |E*| is considered to be one of the variables for δ prediction model, the same outliers were also omitted in δ model development. The procedure to develop the model requires response distribution data to be provided to the statistical software. Figure 1 represents the distribution shapes for the logarithm of |E*|, |E*| and δ. Although the shapes of logarithm of |E*| and δ can be considered as gamma distributions, further trial and error attempts proved that using the normal distribution slightly improved the RMSE (Root Mean Squared Error) and goodness of fit (R^2).

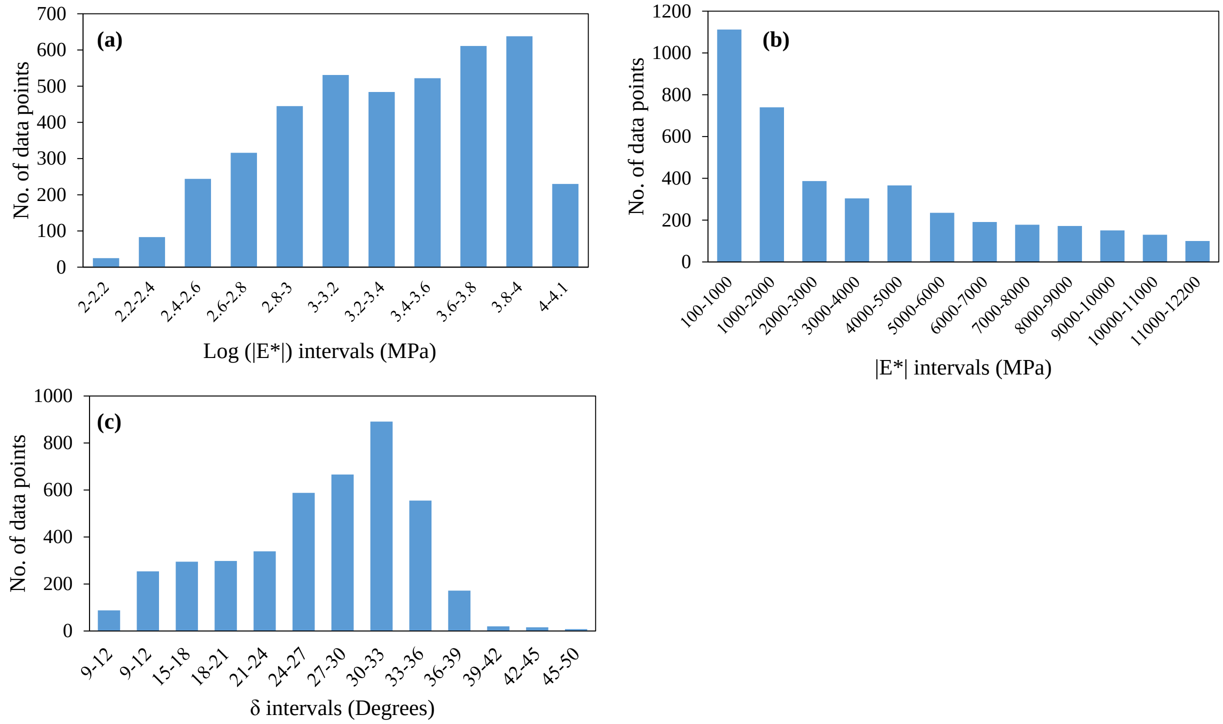

FIGURE 1. Distributions of (a) Logarithm of |E*|, (b) |E*| and (c) δ data used in model development.

The next step in developing the model was to examine diverse types of combinations of factors to evaluate their significance as well as practical interpretability. Using this iterative process, the prediction models were developed. The prediction models for |E*| and δ are depicted in Table 3 and Table 4 respectively. It can be inferred from the |E*| model that the test related factors and their interaction along with the quadratic effect of the logarithm of temperature have the highest impact on |E*|. Consequently, the mixture related factors are observed to have significant effects on |E*| except for the binder high PG temperature (PGHT). The |E*| and δ distribution plots indicate that more than 25% of the observations on |E*| range from 100-1000MPa and for δ from 30 to 50^{o}. These values normally indicate the results from the higher test temperatures. The main load-bearing component at higher test temperature (low loading frequency) is the aggregate. Usually aggregates used in HMA mixtures have similar mechanical properties that result in less variations in the observed values of |E*| and δ at higher temperatures. This is hypothesized to be the cause for lower significant effect related to asphalt PGHT. However, both predictive models show that the interaction of PGHT with logarithm of temperature and logarithm of loading frequency are effective and that is another reason to keep the PGHT in the model. The reverse state happens for asphalt binder PG low temperature (PGLT) where there is a large variation in observed |E*| and δ values at lower testing temperatures. The analysis also shows that this factor has a high impact on both models. The predictive values of |E*| and δ can be calculated through the coefficients and equations shown in Table 3. Since the predictive models are based on the generalized regression, which is different from multiple regression analysis, the interaction of variables along with the independent variables can be effectively used. The |E*| predictive model has been built upon the logarithm of measured dynamic modulus which has resulted in small coefficient values in the model.

Measurements of δ in the lab are usually challenging and there is a higher variability associated to this mixture property, which makes the construction of a reliable and accurate predictive model more challenging. This resulted in the usage of a much higher number of effective terms in the model to increase the level of accuracy, which is still lower than the accuracy of the |E*| model.

TABLE 3- Predictive model for Dynamic Modulus (|E*|)

(|E*|) Predictive Model				
Active Factors (a_i)		**Coefficient (b_i)**	**Standard Error**	**Prob > ChiSquare**
1	Intercept	6.7176428	0.0976212	<0.0001
2	Log (Temperature)	-1.390417	0.007481	<0.0001
3	Log(Frequency)	0.2716079	0.0021966	<0.0001
4	(Log (Temperature)-1.20037)*(Log (Temperature)-1.20037)	-1.395977	0.0207529	<0.0001
5	(Log (Temperature)-1.20037)*(Log (Frequency)-0.26115)	0.1726025	0.0054005	<0.0001
6	Va%	-0.034862	0.0011471	<0.0001
7	PGLT	0.0308918	0.0013407	<0.0001
8	RAP%	0.0029715	0.0001347	<0.0001
9	AC%	-0.067239	0.0047671	<0.0001
10	(Log (Temperature)-1.20037)*(PGHT-60.3887)	-0.012624	0.001892	<0.0001
11	(Log (Temperature)-1.20037)*(PGLT+28.9976)	0.0222484	0.0034946	<0.0001
12	(Log (Temperature)-1.20037)*(RAS%-0.88064)	0.0081275	0.001892	<0.0001
13	NMAS	-0.004575	0.001164	<0.0001
14	RAS%	0.0025448	0.0007382	0.0006
15	PGHT	-0.000955	0.0008396	0.2555
$Log(E^*) = \sum_{i=1}^{15} a_i b_i$ where: a_i= Coefficient and b_i = values of active factors (3)				

TABLE 4. Predictive model for Phase Angle (δ)

(δ) Predictive Model				
Active Factors (c_i)		**Coefficient (d_i)**	**Standard Error**	**Prob > ChiSquare**
1	Intercept	74.252783	4.7805364	<0.0001
2	Log(E*)	-14.91529	0.5985993	<0.0001
3	NMAS	-104.083363	0.0738808	<0.0001
4	(Log(E*)-3.33159)*(AC%-5.85508)	-193.251436	0.0207529	<0.0001
5	(Log(E*)-3.33159)*(PGLT+28.9966)	-282.419509	0.3225955	<0.0001
6	(NMAS-11.8035)*(AC%-5.85508)	-371.587582	0.8088849	<0.0001
7	Log(Frequency)	-460.755655	1.6404459	<0.0001
8	(Log(Frequency)-0.26128)*(PGHT-60.3898)	-549.923728	0.0188966	<0.0001
9	(Va%-6.52872)*(Va%-6.52872)	-639.091801	0.0196971	<0.0001
10	(Log(Frequency)-0.26128)*(Log(Frequency)-0.26128)	-728.259874	0.1650034	<0.0001
11	(Log(E*)-3.33159)*(Log(Frequency)-0.26128)	-817.427947	0.7823135	<0.0001
12	(AC%-5.85508)*(Va%-6.52872)	-906.59602	0.0629724	<0.0001
13	Va%	-995.764093	0.0433031	<0.0001
14	(Log(E*)-3.33159)*(RAP%-13.9445)	-1084.932166	0.0188966	<0.0001
15	Log (Temperature)	-1174.100239	0.8280957	<0.0001
16	AC%	-1263.268312	0.2598378	<0.0001
17	RAP%	-1352.436385	0.0046111	<0.0001
18	(RAP%-13.9445)*(AC%-5.85508)	-1441.604458	0.0063719	<0.0001
19	(Log(Temperature)-1.20042)*(PGHT-60.3898)	-1530.772531	0.0441622	<0.0001
20	(AC%-5.85508)*(AC%-5.85508)	-1619.940604	0.2955319	<0.0001
21	PGLT	-1709.108677	0.0422435	<0.0001
22	(Log(E*)-3.33159)*(Log(Temperature)-1.20042)	-1798.27675	3.2845529	<0.0001
23	(Log(Temperature)-1.20042)*(AC%-5.85508)	-1887.444823	0.3712081	<0.0001
24	(Log(Temperature)-1.20042)*(RAS%-0.88107)	-1976.612896	0.0589393	<0.0001
25	(Log(E*)-3.33159)*(Log(E*)-3.33159)	-2065.780969	1.3394092	0.0002
26	RAS%	-2154.949042	0.0245643	0.0023
27	(Log(Frequency)-0.26128)*(RAP%-13.9445)	-2244.117115	0.0074356	0.0028
28	(Log(Frequency)-0.26128)*(Log(Temperature)-1.20042)	-2333.285188	0.8936574	0.0035
29	(Log(Temperature)-1.20042)*(Log(Temperature)-1.20042)	-2422.453261	2.3767651	0.0109
30	(Log(Temperature)-1.20042)*(RAP%-13.9445)	-2511.621334	0.0216374	0.0267
31	PGHT	-2600.789407	0.023707	0.3452

$$\delta = \sum_{i=1}^{31} c_i d_i$$

where: c_i= Coefficient and d_i = values of active factors (4)

EVALUATION OF PREDICTION MODEL CAPABILITIES

To verify the |E*| and δ prediction models, a set of analyses were conducted to compare predicted values with lab measured values. The comparisons are made on unity plots where predicted and measured values are plotted against each other for the whole dataset. The "goodness of fit" statistics parameters such as the correlation coefficient (R^2) and RMSE were calculated. The correlation coefficient indicates how well the regression line approximates the measured data points. RMSE is a way to measure the difference between predicted and measured values in a prediction model. Figure 2 shows the goodness of fit statistics for predicted |E*| (log and arithmetic) and δ with measured values respectively. A very high R^2 and low RMSE for linear fit between predicted and measured E* values on log-log plot and the equation of linear fit to be very close to unity slope line (X = Y) reveals that model is fitted very well to the measured data. A small deviation in the linear fitting line for the predicted and measured E* (for both log-log and arithmetic scales) comes from the software number rounding when the log scale is changed into the arithmetic one. The actual values of |E*| range from 100-12200MPa, a RMSE of 833 indicates that only an average of 7% difference is anticipated between the predicted and actual |E*| values, this is well below typical laboratory variability.

A high R^2 of 0.83 for δ predictive model ranks it among the good correlations between the actual observations and the predicted values. The lab measured values of δ range from 9° to 45º and a RMSE of 2.94º expresses an average of 8% difference between the predicted and lab measured δ values in the dataset.

In addition to comparing the complete datasets between measured and predicted values of |E*| and δ, further investigations were conducted for 6 mixtures. The mixtures were chosen to be significantly different from each other in terms of constituents and the lab measured |E*| and δ. The properties of these mixtures are shown in Figure 3.

Master curves of |E*| and δ were constructed at 21.1°C for Mixtures A-D and at 20°C for Mixtures E-F. All the shift factors were obtained by using a second order polynomial shift factor equation using rheological and viscoelastic analysis software RHEA (21).

The sample standard deviation for each replicate at each test temperature and frequency was also calculated to obtain the high and low range of measured |E*| and δ. For each set of measured data (average, average + 1 standard deviation and average - 1 standard deviation), independent time-temperature shifting was conducted and this yielded three master curves for |E*| and δ from lab data. These will be referred to as “Measured”, “Measured High Range” and “Measured Low Range” throughout the remainder of this report.

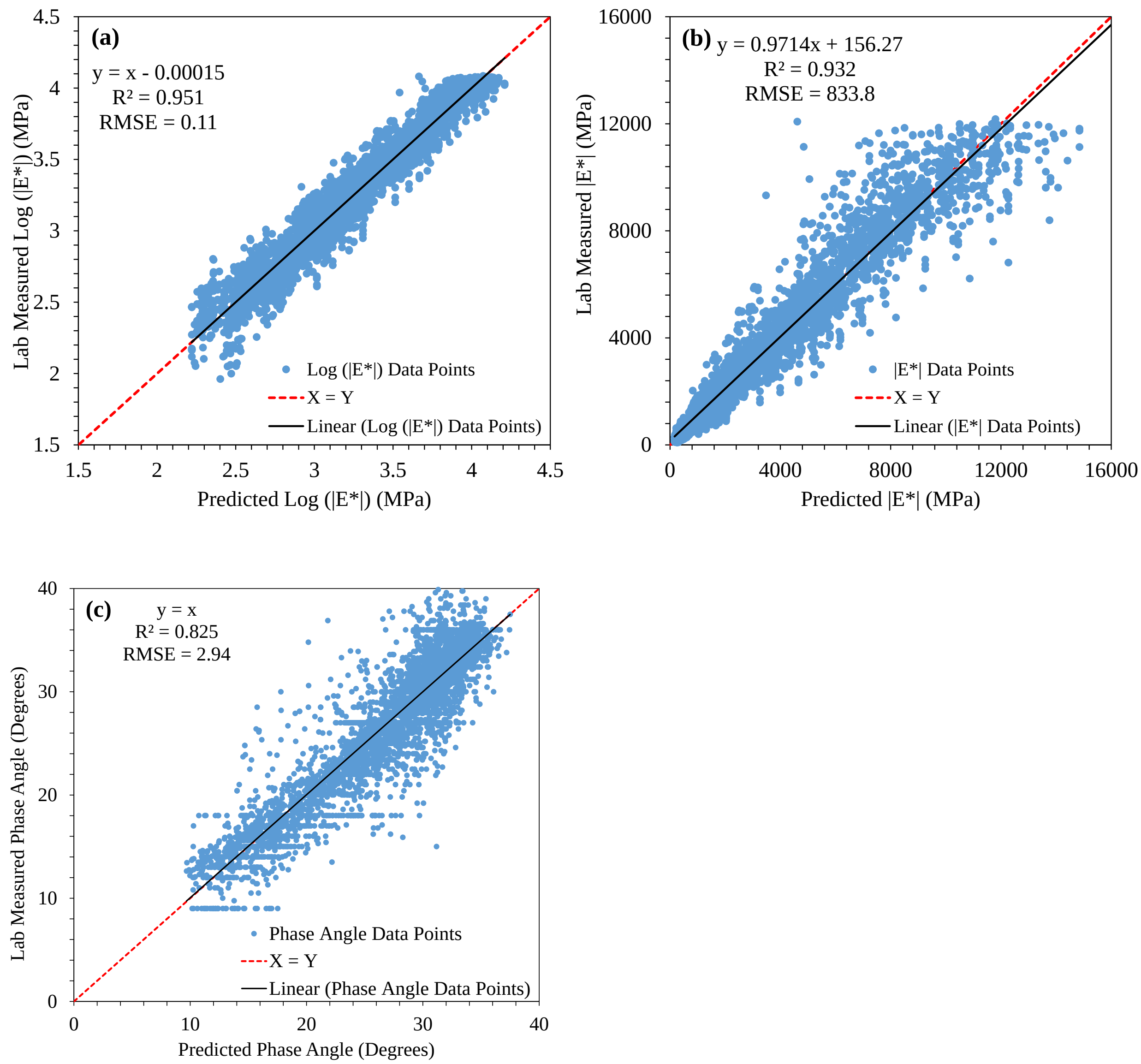

FIGURE 2. Measured and predicted (a) Log |E*|, (b) |E*| and (c) δ.

A four parameter logistic regression sigmoidal equation was used to fit shifted data for constructing |E*| master curves. The fitting equation is shown below.

$$Log\ (E^{*}) = c + \frac{[d-c]}{\left[1+e^{[-a[Log(f)-b]]}\right]} \tag{5}$$

Where:

f = Load Frequency

a = Growth Rate

b = Inflection Point

c = Lower Asymptote

d = Upper Asymptote

A Lorentzian peak equation was used to fit the shifted phase angle results to construct the δ master curves.

$$\delta = \frac{[a \times b^2]}{\left[[(Log(f)) - c]^2 + b^2\right]} \quad (6)$$

Where:

f = Load Frequency

a = Peak Value

b = Growth Rate

c = Critical Point

In order to calculate quantifiable differences between master curves from lab measurements and model predictions, sum of squared errors (SSE) were calculated for each of the six mixtures for both |E*| and δ. Eleven frequencies (0.001, 0.01, 0.1, 0.5, 1, 5, 10, 25, 100, 1000 and 10000Hz) were selected for SSE calculations. Also, for the purpose of visual comparisons of |E*| and δ on single plots, Black space diagrams have been prepared. The comparison plots for |E*|, δ and Black space are presented in Figures 3, 4 and 5 respectively.

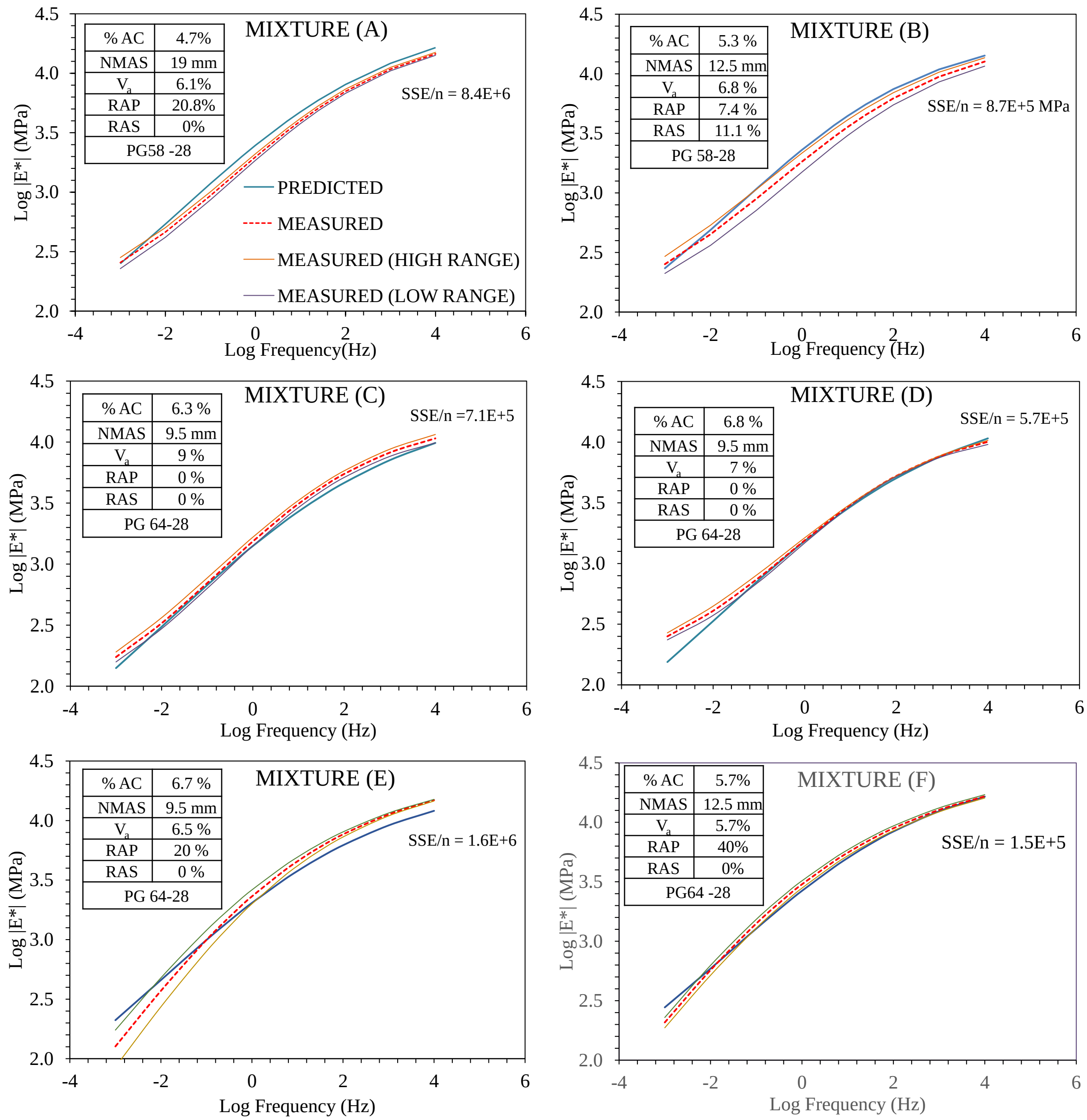

FIGURE 3. Dynamic modulus (|E*|) master curves for six mixtures (master curves from average lab measurements; average + 1 standard deviation; average – 1 standard deviation; and model predictions are shown)

Using the values of SSE/n statistics, the comparison plots of |E*|for the six mixtures show that the prediction equation for majority of mixtures yields values that are close to lab measured values and often within lab measurement variability. A majority of the deviation between measured and predicted |E*| is observed at very low frequencies. At these frequencies, the |E*| response of asphalt mixture is often dominated by aggregate skeleton; the model presented here does not take into account aggregate size distribution and thus a small discrepancy is expected in this region. Overall, the SSE/n values are quite low indicating that the model predictions are quite close to |E*| from lab results.

Considering the SSE/n values in case of δ, the model predictions at a majority of frequency ranges for all mixtures is close to the master curve from lab measurements. As with |E*|, there is some variation observed between predicted phase angle master curves and those generated using lab data in the lower frequency ranges. The differences are typically in the range of 2 to 5º, while typical lab variability of this measure is also about 5º. The average SSE/n values for all six mixtures are also relatively low with the highest being 107.6, which indicates average distributed prediction error of 10.4º. As described before, one major advancement in the current research over previous research is the δ prediction model. The majority of current |E*| prediction equations do not provide phase angle prediction and the ones that do provide it require viscoelastic characterization of binder for accurate prediction. In order to fully describe viscoelastic behavior of asphalt materials and to accurately calculate stress and strain response at different service temperatures and at different loading frequencies, it is critical to have δ master-curve.

In recent years, Black space diagrams have been used for comparison of asphalt mixture performances, for example, work by Mensching et al. (*22*). In order to compare the model predictions with lab measurements for both |E*| and δ, Black space diagrams have been generated for the six mixtures discussed here (see Figure 5). The plots show very similar Black space response for model predictions and lab measured data. Thus, if Black space based performance prediction parameters are used for performance based specifications, the models proposed herein can be easily utilized for determining these parameters during the mix design stage.

Among the evaluated mixtures, C and D reveal a larger difference between the measured and predicted values of phase angle and this could be due to the usage of modified binder as well as implementing the warm mix asphalt (WMA) technology in manufacturing process of these mixes. Even so, with the use of WMA and modifiers, the predicted |E*| for these mixtures is close to the actual lab measurements.

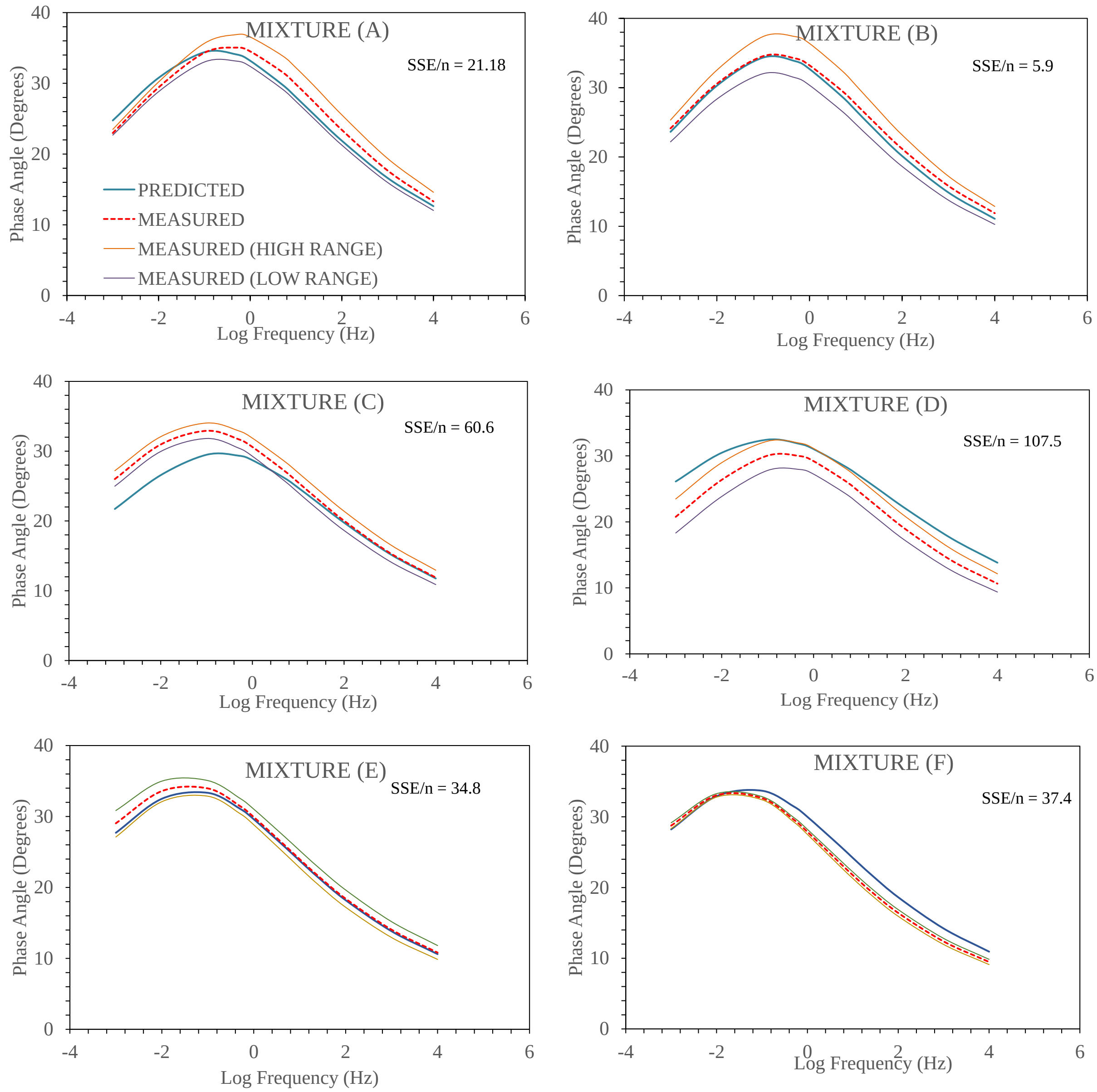

FIGURE 4. Phase angle (δ) master curves for six mixtures (master curves from average lab measurements; average + 1 standard deviation; average – 1 standard deviation; and model predictions are shown)

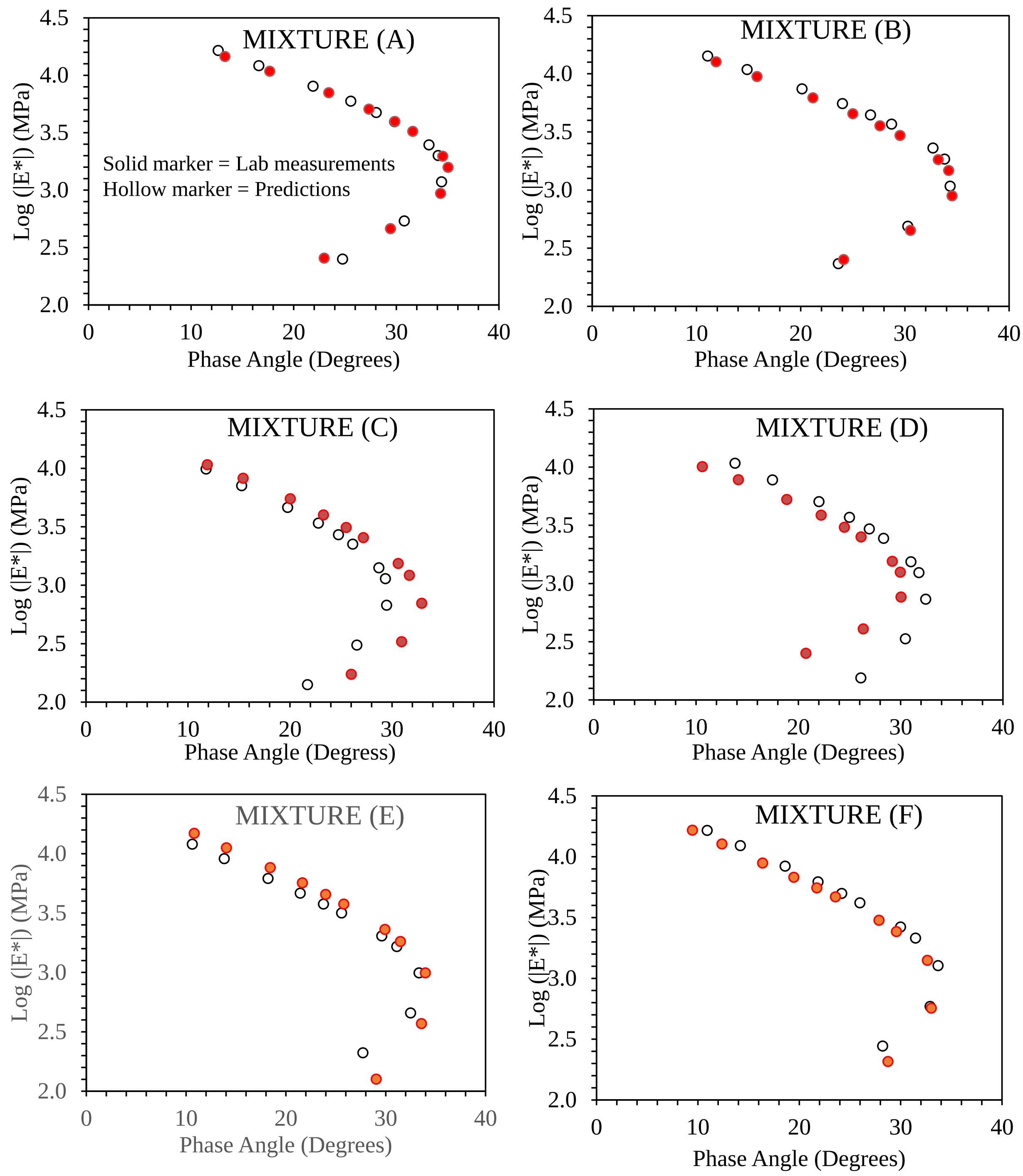

FIGURE 5. Black space diagrams for six mixtures (lab measurements and model predictions)

FATIGUE PERFORMANCE ANALYSIS USING PREDICTED PROPERTIES

To demonstrate the ability of the prediction models for purposes of pavement performance evaluation, a brief case study was conducted. This was also driven by the underlying intent of this research, which is to implement |E*| and δ prediction models for determination of the pavement performance as a combined asphalt mixture and pavement design tool. The case study used lab measured and predicted |E*| and δ values within simplified viscoelastic continuum damage (S-VECD) framework for fatigue cracking performance evaluation. While the research presented here is very useful for conducting PavementME designs and analysis during mix design and selection phase, this research predicts more comprehensive mixture characterizations vis-a-vis |E*| and δ than what is needed for PavementME. For brevity only one mixture (Mixture A) from the previous section was selected for the fatigue performance analysis.

The lab measured results of uniaxial fatigue testing from the selected mixture along with the |E*| and δ (lab measured and predicted) were used as the principal inputs for SVECD analysis. SVECD analysis resulted in damage characteristic curves (DCC) for mixture, DCC indicates the relationship between the asphalt mixture's material integrity (called the Pseudo stiffness (C)) and the level of damage over time (S) (*23*). DCC were calculated using both measured and predicted |E*| and δ.

While DCC is an indicator of how well the mixture can bear the applied loads and how damage progresses with repeated loading, the actual performance of a mixture also depends substantially on the pavement cross section, climatic conditions and material constitutive properties. In order to determine the pavement performance, an investigation was conducted using the layered viscoelastic pavement analysis for critical distresses (LVECD). This program utilizes the SVECD model to calculate the rate of strains and stresses over the life of pavement to make the performance predictions (*24*). During recent years this software has been widely used by many researchers in predicting the fatigue performance of asphalt mixtures as well as determining how mix parameters affect its performance in actual field situations, for example recent work by Rastegar et al. (*25*). One of the main results from LVECD analysis is damage factor, this factor simply reveals that how much of a cross section has been damaged due to loading and other factors leading to pavement deterioration over time. Using Miner's law, the number of points (evenly distributed regions of asphalt concrete over the simulation domain) where the damage factor is equal to one, or where asphalt concrete has fully damaged, is calculated over life of pavement.

Two types of cross sections were analyzed using LVECD. Only the thickness of the asphalt layer was changed in these cross sections. Figure 6 presents the DCC and LVECD analysis results. As it can be seen from the figures, both measured and predicted $|E^*|$ and δ led to very comparable DCC. Additionally, in the context of pavement performance evaluation, the predicted results are not identical between measured $|E^*|$ and δ and predicted ones. However, the results are very comparable with each other. The pavement performances using the predictive $|E^*|$ and δ values at 20 years are very comparable to the performance predicted using lab measured values for both thin and thick asphalt pavements. Thus, fatigue performance calculations from the predicted $|E^*|$ and δ are comparable to the measured ones for both cross sections, which is a good indicator of the applicability of the predictive models presented in this paper.

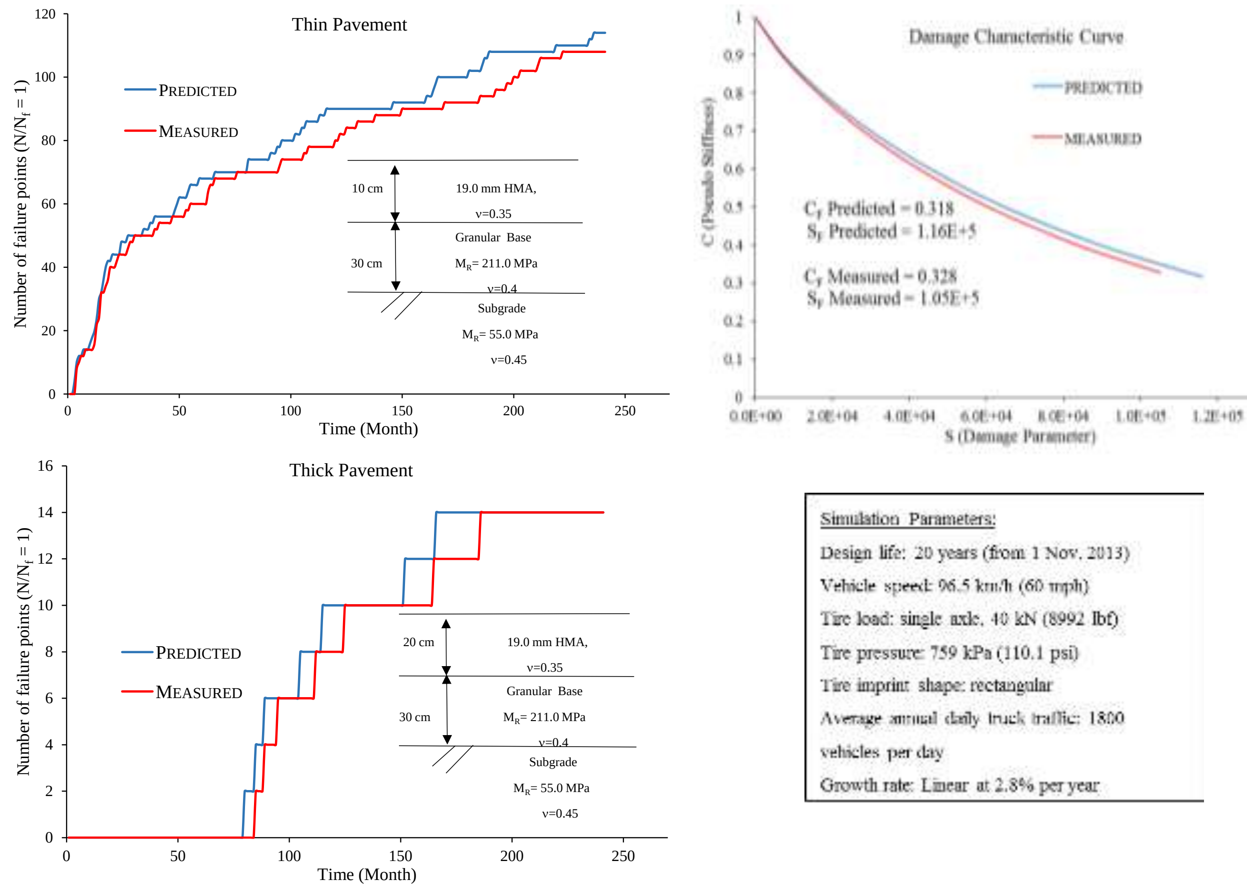

FIGURE 6. LVECD and Damage Characteristic results

COMPARISON OF THE |E*| PREDICTIVE MODEL TO THE WITCZAK MODEL AND HIRSCH MODEL

As the final step in the evaluation of the accuracy of the predictive models presented in this study, the |E*| predictions are compared to the Witczak model and Hirsch models. The comparison is made for Mixture B. As indicated in previous discussion, both Witczak model and Hirsch model require asphalt mix properties that may not be readily available during the mix design stage. Please note that there are different versions of Witczak model for dynamic modulus prediction, the newest published version of the model, Witczak 2006 (*1*), is used in this work. In this study, comparisons are made to For example, the results shown in Figure 7 required asphalt binder complex shear modulus (G*) at different temperatures for both Witczak model and Hirsch model predictions. Measured values for the binder were used in this study to make the predictions. Figure 7 indicates that even with lab measured binder properties both Witczak model and Hirsch model substantially over-predicted |E*| at lower load frequencies. The generalized regression based model from the present study yielded |E*| master-curve to be very close to that generated from the lab measurements.

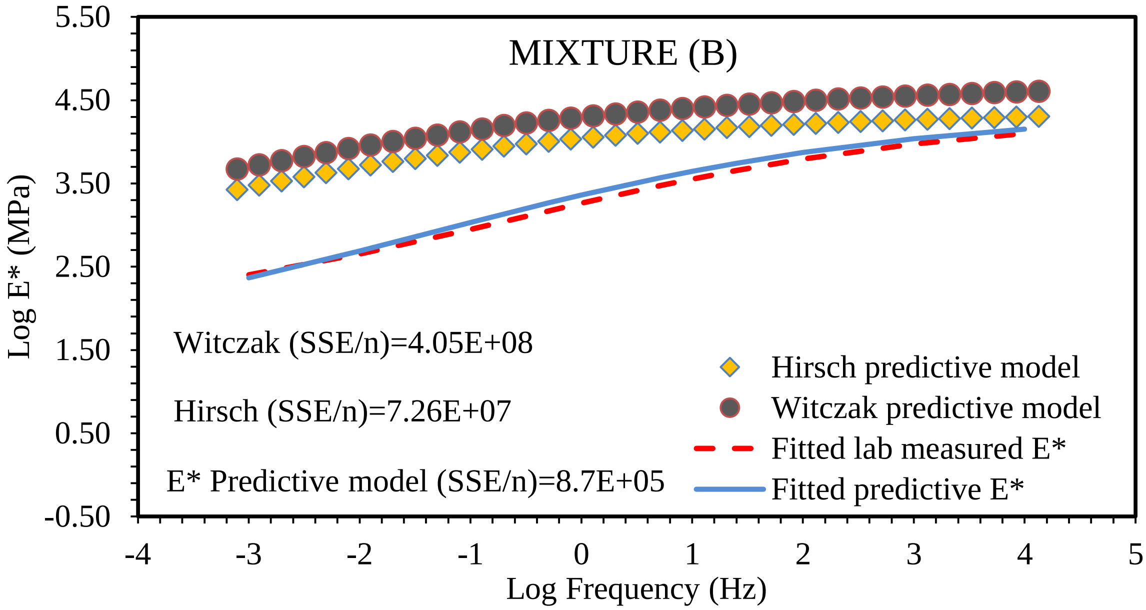

FIGURE 7. Comparison of |E*| Predictive models

SUMMARY, CONCLUSION AND FUTURE EXTENSIONS

This paper describes generalized regression based prediction models for dynamic modulus (|E*|) and phase angle (δ) of asphalt mixtures. The models were developed using over 4300 laboratory test results for asphalt mixtures in the New England region of the United States. A unique feature of the model development approach, as well as the models presented here are that they utilize nominal asphalt mixture properties. This is different than currently adopted prediction models, and thus make proposed models very useful for pavement designers. For example, using the models proposed here, a pavement designer can conduct mechanistic pavement analysis and make recommendations for modifying asphalt mixture specifications for a given project and/or optimize the pavement structure and material to lower life cycle costs. As previously mentioned, the models proposed here have been developed using the dataset gathered in the New England region, therefore the applicability of the regression coefficients presented herein may be limited to this region due to similarities in aggregate geological sources, binder grades and recycled asphalt material characteristics. However, the methodology behind the development of these models can be applied to other regions for development of regional prediction models. Although the dataset has been gathered for complex moduli of asphalt mixtures, it does not necessarily mean that the development of such models is associated to only complex moduli and only measurements made using AMPT device. In fact, the framework presented in this study can be applied to develop similar models for other material properties. It is also important to note that the use of prediction models does not necessarily result in full omission of conducting $|E^*|$ and δ lab tests, rather predictive models aid in lowering the amount of testing requirements as well as provide reliable estimate of properties when lab testing is impractical or not possible due to time or economic constraints.

A practical application of the proposed model is for developing asphalt specifications and for conducting pavement structural design. At present, a major hurdle in developing asphalt mix specifications on basis of mechanistic properties and conducting pavement structural designs using properties that reliably represent the actual mixture that will be produced and placed in the field, is unavailability of reliable prediction models that only use nominal properties to predict $|E^*|$ and δ. While Hirsch model and Witczak model have been adopted, these require binder viscoelastic characterization for prediction. Furthermore, as shown in this paper, even with binder viscoelastic characterization these models can fail to make reliable predictions.

Comparisons were made between lab measured data and the model predictions. While the same data was used for developing the model, this comparison provides verification of the model development process. Apart from visual comparisons, statistical comparisons were also conducted. To further ensure veracity of the models, six distinctly different mixtures were chosen and comparisons were made between model predicted and lab measured $|E^*|$ and δ. The predictions were mostly within one standard deviation lab variability of measured quantities. Finally, to demonstrate the application of the prediction model and to make further comparisons between model predictions and lab measurements, a case study is presented for two asphalt pavement cross-sections and their predicted fatigue cracking performances. The results from this analysis demonstrates that the model predictions presented herein are capable of use in pavement mechanistic analysis tools and yield comparable results to those from lab measured properties.

On the basis of the research results presented in this paper, the following conclusions can be drawn:

- Generalized regression based methodology can be employed for developing dynamic modulus and phase angle prediction models that require only nominal asphalt mixture parameters as inputs;
- The predictions from generalized regression based models match the lab measurements within typical lab variability;
- Rheological indices for pavement performance can be easily calculated using the prediction models presented here, these indices can be used for performance based specifications; and,
- Using prediction models presented herein, pavement designers can optimize asphalt mix specifications to increase reliability of pavement designs and to lower life cycle costs.

While this paper presented prediction models, their comparisons with lab measurements and a case study to demonstrate applicability of the models, there were several areas identified during this research that will further improve the applicability of this research and extend the findings further. Some of the future extension of the present research are the following:

- The current models are developed for New England region; similar regional models can be developed for other part of United States and other countries. Notice that it is important to try to limit these type of models to a region, that way only nominal asphalt properties would be necessary as model inputs, otherwise the required number of inputs might become overwhelming.
- In this work, the generalized regression based model development was applied to linear viscoelastic asphalt properties; future efforts should undertake similar model development for non-linear properties such as asphalt fatigue and fracture parameters.
- Validation of the predictions models should be conducted using additional mixtures that are not part of the model development data set. Furthermore, field performance validation should also be conducted.
- The other aspect of the future work is the improvement of the accuracy of the proposed models and especially the δ by using additional number of mixtures and calibrating the models in accordance to the newly added data.
- Future predictive models can be developed within framework of analytical/physical models so that such model can thereafter be incorporated within mechanistic calculation algorithms.

REFERENCES

1. Bari, J., and Witczak, M.W. Development of a New Revised Version of the Witczak |E*| Predictive Model for Hot Mix Asphalt Mixtures. Journal of the Association of Asphalt Paving Technologists, Vol. 75, 2006, pp. 381-423
2. Christensen, D.W., Pellinen, T.K., and Bonaquist, R.F.: Hirsch Model for Estimating the Modulus of Asphalt Concrete. *Journal of the Association of Asphalt Paving Technologists,* 72, 2003, pp 97-121, Lexington, KY.
3. Rowe, G.M., Baumgardner, G. Sharrock, M.J.: Functional forms for master curve analysis of bituminous materials. *7th International Symposium on Advanced Testing and Characterization of Bituminous Materials Organized by RILEM TC 206 ATB*, Rhodes, Greece. 2009.
4. Oshone M., Dave E., Daniel J. and Rowe G., "Prediction of Phase Angles from Dynamic Modulus Data and Implications on Cracking Performance Evaluation," Journal of the Association of Asphalt Paving Technologists, In press
5. Garcia, G., Thompson, M.: *HMA Dynamic Modulus Predictive Models- A Review*. Publication FHWA-ICT-07-005. FHWA, U.S. Department of Transportation, 2007.
6. Georgouli, K., Loizos, A., Plati, C., "Calibration of dynamic modulus predictive model", Construction and Building Materials, Volume 102, Part 1, 15 January 2016, Pages 65-75
7. Ceylan, H., Gopalakrishnan, K., and Kim, S.: Advanced Approaches to Hot-Mix Asphalt Dynamic Modulus Prediction, *Canadian Journal of Civil Engineering*, 35(7), 2008, pp. 699-707.
8. Jun L., Yan, K., You, L., Liu, P. & Yan, K.: Prediction models of mixtures' dynamic modulus using gene expression programming, *International Journal of Pavement Engineering,* 2016, DOI: 10.1080/10298436.2016.1138113.
9. Komba, J., Maina, J.W, Anochie-Boating, J. K, O'Connel, J., Analytical Modeling of Visco-Elastic Behaviour of Hot-Mix Asphalt, 31st Southern African Transport Conference, July, Pretoria, South Africa, 2012
10. Sakhaeifar, M. S., Kim, R. Y., Kabir, P., "New predictive models for the dynamic modulus of hot mix asphalt", Construction and Building Materials, Volume 76, 1 February 2015, Pages 221-231

11. Cao, P., Jin, F., Feng, D., Zhou, C., Hu, W., "Prediction on dynamic modulus of asphalt concrete with random aggregate modeling methods and virtual physics engine" Construction and Building Materials, Volume 125, 30 October 2016, Pages 987-997
12. Biligiri, K. P.: Asphalt mixtures' properties indicative of tire/pavement noise. *Ph.D. dissertation,* Arizona State Univ., Phoenix, AZ. 2008
13. Biligiri, K. P., Kaloush, K. E., and Uzan, J.: Evaluation of asphalt mixtures' viscoelastic properties using phase angle relationships, *International Journal of Pavement Engineering*., 2010.
14. Naik, A., Biligiri, K.: Predictive Models to Estimate Phase Angle of Asphalt Mixtures. *Journal of Materials in Civil Engineering,* Vol. 27, 2015
15. American Association of State Highway Transportation Officials (AASHTO): *Standard Practice for Superpave Volumetric Design for Asphalt Mixtures,* AASHTO R 35, 2015
16. American Association of State Highway Transportation Officials (AASHTO): *Standard Method of Test for Determining Dynamic Modulus of Hot Mix Asphalt (HMA),* AASHTO T 342, 2015
17. Yoon-Hoa, C., Dae-Wook, P., Sung-Do, H. A predictive equation for dynamic modulus of asphalt mixtures used in Korea, *Construction and Building Materials*, 24(4), 2010, pp. 513–519
18. Crotty, Michael, Clay, B.: Penalizing Your Models: An Overview of the Generalized Regression Platform, SAS Institute, JMP Division, 2014.
19. Zou, H., Hastie, T.: Regularization and Variable Selection via the Elastic Net, *Journal of the Royal Statistical Society*, Series B, 67, 2005 ,pp.301–320.
20. .Mahalanobis, P.C.: On the generalized distance in statistics. In: *Proceedings of the National Institute of Sciences of India*, 1936, pp. 49–55.
21. *Rheology Analysis Software User's Manual*, Abatech Inc., Blooming Glen, PA, 2011.
22. .Mensching, D. J., G. M. Rowe, J. S. Daniel.: A Mixture-Based Black Space Parameter for Low Temperature Performance of Hot Mix Asphalt, *Road Materials and Pavement Design*, 2016.
23. Prowell, B.D, E.R. Brown, R.M. Anderson, J. S. Daniel, et al.: Validating the Fatigue Endurance Limit for Hot Mix Asphalt. Publication NCHRP Report 646, Washington DC: National Cooperative Highway Research Program, Transportation Research Board
24. Sabouri, M., Bennert, T., Daniel, J., Kim, Y.R.: Fatigue and Rutting Evaluation of Laboratory-Produced Asphalt Mixtures Containing Reclaimed Asphalt Pavement, In *Transportation*

Research Record: Journal of the Transportation Research Board, No. 2506, Transportation Research Board of the National Academies, Washington, D.C., 2015, pp.32-44.

25. Rahbar-Rastegar, R., Dave, E.V., Daniel, J.S.: Fatigue and Thermal Cracking Analysis of Asphalt Mixtures Using Viscoelastic Continuum Damage and Cohesive Zone Models, submitted to ASCE Journal of Transportation Engineering, 2017

Appendix: Paper 4 (Chapter 7)

Title: Development of a Complex Modulus Based Rutting Index Parameter for Asphalt Mixtures

Journal: ASCE Journal of Transportation, Part B: Pavements

DEVELOPMENT OF A COMPLEX MODULUS BASED RUTTING INDEX PARAMETER FOR ASPHALT MIXTURES

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ABSTRACT

Different testing methods have been used to evaluate the rutting susceptibility of asphalt mixtures. Among them, loaded wheel testers, such as the Hamburg Wheel Tracking Test (HWTT), has shown to have promising correlation with the field rutting. Moreover, since rutting distress within pavement structure has a direct correlation with mixtures' structural response to loading, the complex modulus (|E*| and phase angle) master-curves can be potentially used to estimate the mixtures rutting performance. This research introduces and investigates 5 different complex modulus based parameters to evaluate the rutting performance of asphalt mixtures. These parameters are developed based on two critical points on the |E*| and phase-angle master-curves. The first point is related to the frequency at which the peak phase angle happens and the second point is related to the reduced frequency on the master-curve which reflects the Hamburg Wheel Tracking Test (HWTT) testing conditions. The results from investigating 22 asphalt mixtures indicate that there is a strong correlation (R^2=0.89) between the rutting and the rate of drop in |E*| with respect to changes in frequency between the two selected critical points which is called as Index III in this study.

Keywords: Complex modulus, Rutting, Permanent Deformation, Hamburg Wheel tracking Test

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INTRODUCTION

Rutting as one of the major asphalt pavement structural distresses has long been investigated by researchers. This phenomenon is the permanent plastic strain due to repeated loads, rutting rate increases significantly due to overweight vehicles and traffic consistently moving below design speed. At warmer conditions, the asphalt mixtures tend to behave in a more viscous manner because of the softened binder which consequently results in creep and permanent deformations. There are different testing and analysis methods such as Hamburg wheel tracking test (HWTT), asphalt pavement analyzer (APA), flow number (FN), stress sweep rutting (SSR), Superpave shear tester (SST) etc. to evaluate the rut susceptibility of the asphalt mixtures. The majority of these tests need equipment specifically designed to evaluate only the rut performance and involve specimen fabrication and analysis methods which often time is relatively time consuming. Others such as SST, although provide fundamental mixture properties, are substantially complicated to conduct the test and analyze the result while there is no acceptable model associated with the results from this test to predict performance (Brown, 2001).

One of the most widely accepted tests to determine the asphalt mixtures rutting and stripping susceptibility is the HWTT. The test is run in accordance to the AASHTO T-324 standard testing method in 52±2 cycles per minute at a temperature usually set in accordance with the PG high temperature. Based on the test track length (22.7 cm), number of loading cycles in time domain and the sinusoidal loading function of the wheel on the specimen, the applied loading frequency in the middle of the track length were the rut depth is measured can be calculated (Mohammad, 2015). This frequency would be equal to 0.866 Hz. As the test is run in wet condition, the results not only indicate the rut susceptibility but are also used to determine the mixture stripping potential. The deformation versus loading cycles graph is generally divided into three distinguished parts. The initial part of the graph is related to the densification of the asphalt mixture due to loading which usually happens after about 1000 cycles (Yildirim, 2006). The second portion of the curve with a constant creep slope can be used to evaluate the mixture rutting susceptibility. The third portion of the curve which can be distinguished through an inflection point and a steeper slope is related to mixture degradation due to stripping (Brown, 2001). Although different destructive testing methods are available to specifically determine the mixtures rutting susceptibility, the specimen fabrication and conditioning as well as testing different replicates might be time consuming. In addition, many of these tests, require expensive testing equipment which may not be available in many labs.

It has been shown that rutting is directly correlated with the mixtures' stiffness (Sivasubramaniam, 2005), and as a result stiffness based measures can be used to evaluate the mixtures' rutting susceptibility. There are different tests such as resilient modulus (M_r) and complex modulus (E^* and phase angle) to determine the mixtures stiffness. Although resilient modulus has been shown to have some correlations with the rutting (Brown, 2001), it measures the recoverable strain of the mixture due to repeated loading whereas rutting is related to the mixtures plastic deformation (unrecoverable strain) which is a substantially different aspect of mixtures performance. In other words, it is possible that a mixture with a higher resilient modulus could have a higher plastic deformation compared to a mix with a lower M_r value. Also, resilient modulus measures the stiffness at only one temperature and frequency (25°C at 10Hz), however depending on the loading and climatic conditions, rutting can happen at different circumstances and not only in one specific situation.

The complex modulus test evaluates the mixtures performance at multiple frequency and temperature combinations. Using the dynamic modulus master-curves, the mixtures' stiffness can be measured at different temperatures and frequencies. Therefore, it can be used to estimate the mixtures rutting performance in different conditions. Despite the fact that rutting is a plastic deformation with non-recoverable strains (Bazzaz, Mohammad, et al. 2018), the dynamic modulus as a linear viscoelastic property has been widely used in distress prediction models in many performance prediction tools such as Pavement ME because of its simplicity in conducing the test and analysis (Mohammad, 2006; El, 2013). Also, because of the correlation between rutting and stiffness, the dynamic modulus can be used as an additional preliminary rutting performance screening tool. The NCHRP project 19-9 introduces dynamic modulus as one of the three main tests to discriminate the mixtures rutting performance where the other two are flow time and repeated loading permanent performance tests (Witczak 1997). Therefore, a dynamic modulus based index parameter can facilitate the mixture performance ranking and reduce the extensive specimen fabrication and lab work. Many studies have been conducted to relate the dynamic modulus to any one of the specific rutting tests. Research conducted by Apeagyei tried to develop a mathematical model between the FN as function of dynamic modulus at 38°C and 10, 1, 0.1Hz plus the mixture gradation (Apeagyei, 2011). Dynamic modulus at 38°C and 54°C and 5Hz has also been examined to investigate the relationship between the modulus and field rutting (Witczak, 2002), but no consistent results with respect to different mix types were observed with consideration of only $|E^*|$ at this specific temperature and frequency. Nemati (2017) used the combination of $|E^*|$ at 37.8° and 1.59Hz to investigate the relationship between this parameter and resilient modulus for typically used mixtures in New Hampshire. This combination revealed promising correlation for majority of mixtures in the study (Nemati, 2017). However, since rutting is the measure of plastic deformation, it is important to consider the mixture's viscous properties along with the stiffness through incorporating the phase angle. The prevailing pre-peak behavior of the phase angle master-curve towards the lower frequencies is generally considered to be controlled with the aggregate (Figure 1). Theoretically, in this frequency range, the binder-aggregate interaction phase weakens and as the mixture overall load bearing capacity decreases, rutting happens.

A comprehensive study was conducted by Bhasin (2003) to evaluate the rut susceptibility of some of the commonly used asphalt mixtures in southern United States. Mixtures were tested using the APA, flow test and dynamic complex modulus. The mixtures were ranked and compared to APA as the baseline (Bhasin, 2003). The results indicated that $|E^*|/\sin\phi$ at 1Hz was able to rank the mixtures similar to APA whereas the same criteria at 10 Hz revealed poor correlation to APA results. In general there has not been much success in implementing the dynamic modulus at single temperature and frequency to relate to the rutting performance as mixtures with different gradation and characteristics behave differently in a predefined loading conditions (Birgisson, 2004). Another problem with considering only one point on the master-curve is that there is a possibility that the master-curves of different mixtures would intersect at that specific point or may hold close stiffness and phase angle values which can make it difficult to distinguish between different mixtures. Therefore, there is need to investigate the complex modulus master-curves at more than one single point to evaluate the effect of temperature and frequency on the rutting mechanism.

The main objective of this study is to explore the possibility of development of a rutting index parameter using the complex modulus master-curves (dynamic modulus and phase angle). Such index parameter can help reducing the required lab work during the mixture design and evaluation procedure by narrowing down the number of mixtures that should be tested through aforementioned destructive testing methods. The approach in this study uses two points on the master-curves to develop various index parameters. The first point (marked by subscript "A" in this paper) is associated with the reduced frequency on the master-curves at which the peak phase angle happens. This point is selected due to it indicative of the highest potential for non-recoverable deformation within LVE response range of the asphalt mixtures. The selection of the second point (marked by subscript "B" in this paper) is directly correlated to the HWTT testing temperature (regionally assigned temperature) and loading frequency (52 cycles per minute equal to 0.866 Hz) and its projected reduced frequency on the master-curves. The choice for this point is driven by need to capture asphalt mixture behavior in a low stiffness range, where greater compressive strains will be experienced by asphalt under traffic loading. As HWTT has shown to distinguish asphalt mixtures' rutting performance and correlate well to field performance, the loading frequency associated with that test was chosen. Please note that the frequencies used in this research are shifted as per the time-temperature superposition principle. Once the points are assigned, 5 different parameters are investigated to determine their correlations with the rutting. Figure 1 and Table 1 indicate and describe the selected points on the master-curves used to develop the index parameters.

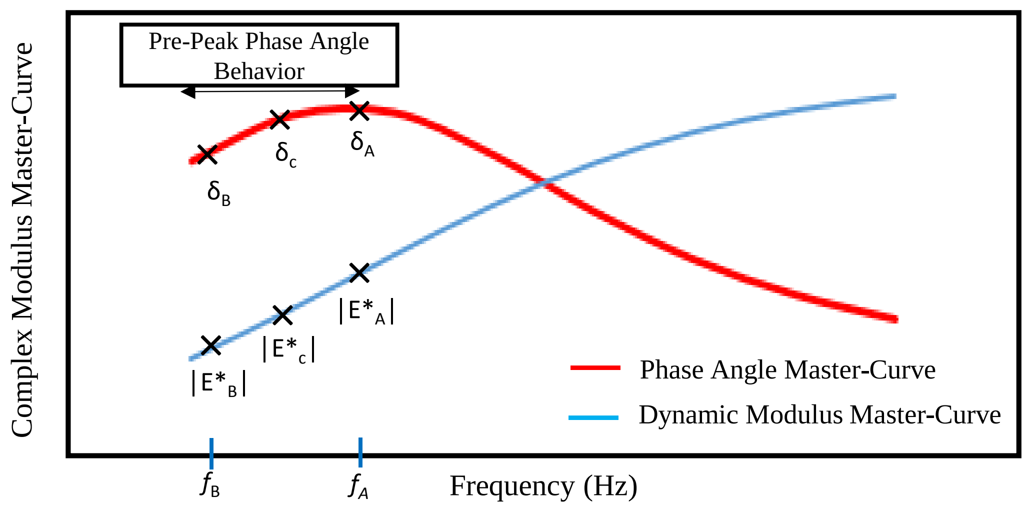

Fig.1. Typical dynamic modulus and phase angle master-curves for asphalt mixtures. Selected points on the master-curves to develop the rutting index parameters

Table 1. Description of the selected points on the master-curves

Complex Modulus Master-Curve	Point	Description
Phase Angle (δ)	δ_A	Peak Phase Angle (δ)
	δ_B	(δ) corresponding to HWTT testing condition (45°C at 0.866Hz)
	δ_C	Estimated average phase angle between points A and B; $\delta_C = [\delta_A + \delta_B]/2$
Dynamic Modulus $\lvert E^* \rvert$	$\lvert E^*_A \rvert$	$\lvert E^* \rvert$ corresponding to peak phase angle
	$\lvert E^*_B \rvert$	$\lvert E^* \rvert$ corresponding to the HWTT testing condition (45°C at 0.866Hz)
	$\lvert E^*_c \rvert$	Estimated average dynamic modulus between points A and B; $\lvert E^*_c \rvert = [\lvert E^*_A \rvert + \lvert E^*_B \rvert]/2$
(δ) and $\lvert E^* \rvert$	f_A	Logarithm of frequency corresponding to Peak Phase Angle
	f_B	Logarithm of frequency corresponding to HWTT testing condition (45°C at 0.866Hz)

In order to develop, verify and validate a complex modulus based rutting index parameter, three different sets of mixtures are evaluated. The first set is combined of 7 mixtures for which the HWTT results are available and 5 different complex modulus based indices will be investigated to determine the best statistically correlating index parameter with the rut depth measurements. The second set of the mixtures includes 6 hot mixed asphalt mixtures for which the field rut measurements and available and the index parameters will able evaluated in terms of capability of mixture ranking as well as their statistical correlations with the field performance. The third set is combined of 9 mixtures with same gradation but varying air void and binder content. This set is used to verify the capability of the indices in determining the expected rutting performance of a mixture with respect to the mixture design properties. The material and the results of analysis will be discussed in the next sections of the paper.

RESEARCH APPROACH AND MATERIAL

As a preliminary step to explore the capability of master-curves in determining the mixtures rut susceptibility, a set of 7 different mixtures with diverse rheological properties were selected. These mixtures have been evaluated through the HWTT at 45°C as per the AASHTO T324 test specifications. All testing for these mixtures was conducted in submerged conditions. The rut depth and creep slope are primarily used to evaluate the mixture rutting susceptibility. In this study the rutting depth at 7000th pass in the test was selected to compare the mixtures with respect to rutting performance. The creep slope was not used for comparison purposes, since some of these mixtures start to reveal stripping related behavior after about 7000 test cycles. The design properties of this set of mixtures is summarized in Table 2. It should be mentioned that The MEP, MEM-1, MEW and VTG-1 are polymer modified mixtures. Also, an antistripping agent has been used in VTP-2 mixture production.

Table 2. Properties of the first set of mixtures used to develop the rutting index parameters

Mixture	**Binder Grade**	**NMAS (mm)**	**RAP%**	**AC (%)**	**(VMA, %)**	**(VFA, %)**	**%Passing #200 (%)**	**(N_{des} gyrations)**
MEP	64-28	12.5	10	5.9	Not Available	Not Available	5.1	50
MEM-1	64-28	12.5	20	5.6	Not Available	Not Available	5.0	50
MEW	64-28	12.5	20	5.8	Not Available	Not Available	4.5	50
VTP-1	58-28	9.5	20	6.0	16.5	76	4.8	50
VTP-2	58-28	9.5	20	6.0	16.5	76	4.8	50
VTG-1	70-28	12.5	15	4.9	15.5	74	4.4	80
CTG-1	64-22	12.5	0	5.0	15.5	72	2.5	50

The results from HWTT test is depicted in Fig. The plots and the rut depths are averaged measured values from two specimens in the HWTT test. The test results confirm the reasonability of selecting the 7000th pass of the HWTT test as the comparison reference point, where all the slopes remain steady before the stripping initiates. Also this point on the curve is significantly away from the post-compaction consolidation point which is happening after about 1000 cycles. On the other hand, this point is selected as such that mixtures like MEP and VTP-2 could be evaluated in terms of rutting performance only and the response is not overwhelmed by moisture induced stripping damage.

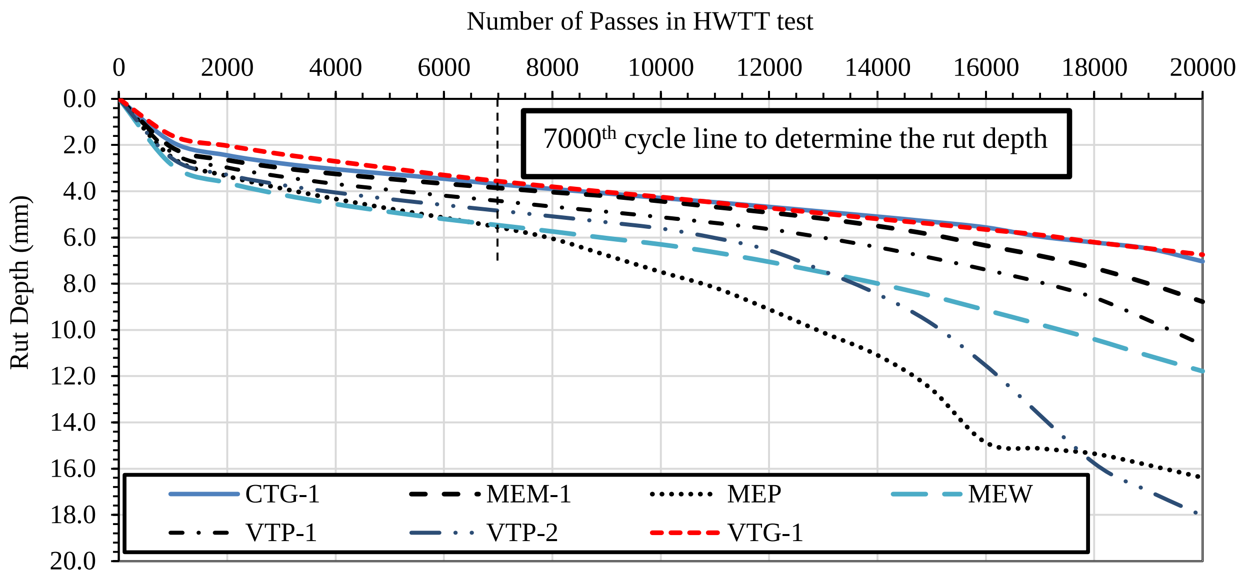

Fig 2. HWTT test results for the first set of study mixtures

Complex modulus master-curves

Complex modulus specimens were cored, cut and tested with respect to AASHTO T342 standard procedure in three temperatures 4.4°C, 21.1°C and 37.8°C and 6 loading frequencies as 0.1Hz, 0.5Hz, 1Hz, 5Hz, 10Hz and 25Hz. Then using the time temperature superposition principle (TTSP) master-curves were constructed at a reference temperature of 21.1°C.

In order to analyze the master-curves to develop the index parameters, Equations 1 and 2 were used to fit the dynamic modulus and phase angle graphs respectively (Nemati, 2018).

$$\boldsymbol{Log\ (E^*) = c + \frac{[d-c]}{\left[1+e^{[-a[Log(f)-b]]}\right]}} \qquad \text{Equation (6)}$$

Where:

f = Load Frequency

a = Growth Rate

b = Inflection Point

c = Lower Asymptote

d = Upper Asymptote

$$\boldsymbol{\delta} = \frac{[\boldsymbol{a} \times \boldsymbol{b}^2]}{\left[[(\boldsymbol{Log}(\boldsymbol{f})) - \boldsymbol{c}]^2 + \boldsymbol{b}^2\right]} \qquad \text{Equation (7)}$$

Where:

f = Load Frequency

a = Peak Value

b = Growth Rate

c = Critical Point

The fitted dynamic modulus and phase angle master-curves are depicted in Figure 3 and Figure 4 respectively. As discussed earlier the aim of this study is to implement the pre-peak behavior of phase angle master-curve (Figure 1) as a starting point for developing a rutting index parameter. The behavior of this portion of the phase angle master-curve is controlled with the aggregate stiffness and binder flow which is reflected as a drop in phase angle master-curve. The rate, magnitude and coordination of the phase angle and stiffness drop on the master-curves is deemed to be unique for different mixtures as mixtures have different rheological properties. The peak point on the phase angle is considered as the maximum extent of the viscous behavior of a mixture and depending on the aggregate size and gradation, binder type and content as well as other mixture properties the coordination of this peak point can change significantly. It is hypothesized that after this peak point towards the lower frequencies, the combination of the low loading frequency and high temperature will cause the binder to flow into the mixture air voids and thinner asphalt film will be left around the aggregate resulting in a more elastic response. This phenomena causes the aggregate to be the prevailing load bearing element in the mixture and because of that the phase angle starts to drop (Zhao 2003). For the same aggregate gradation and different binder type, the faster the binder flows (softer binder) the faster the phase angle will drop and probably the higher amount of rutting will be observed.

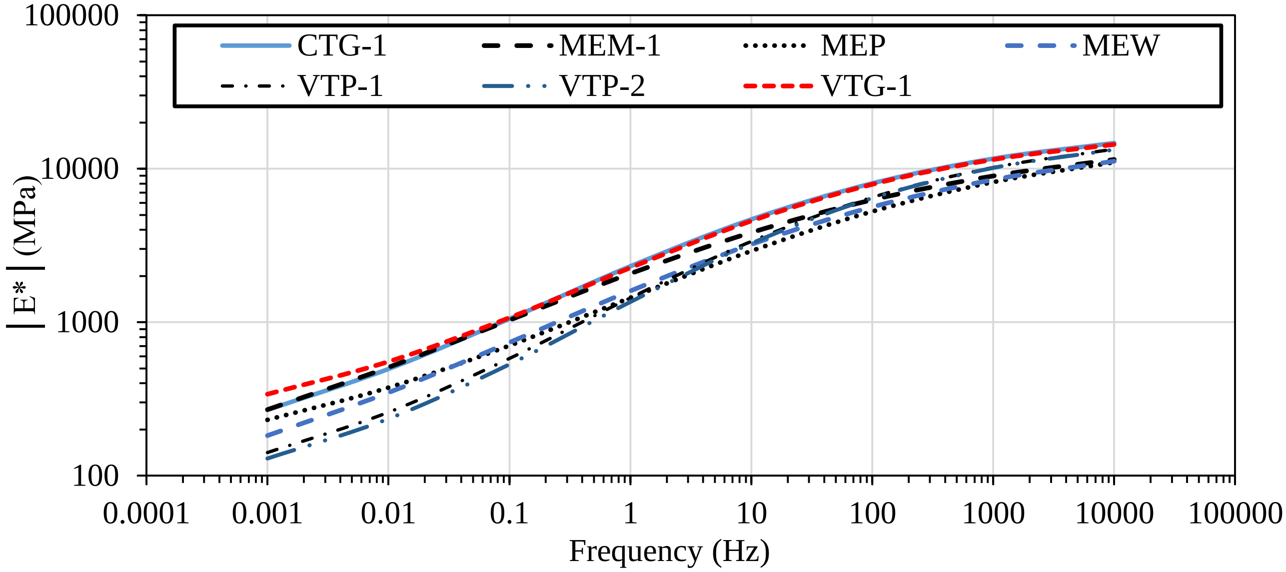

Fig. 3. **Fitted dynamic modulus master-curves of the first set of mixtures**

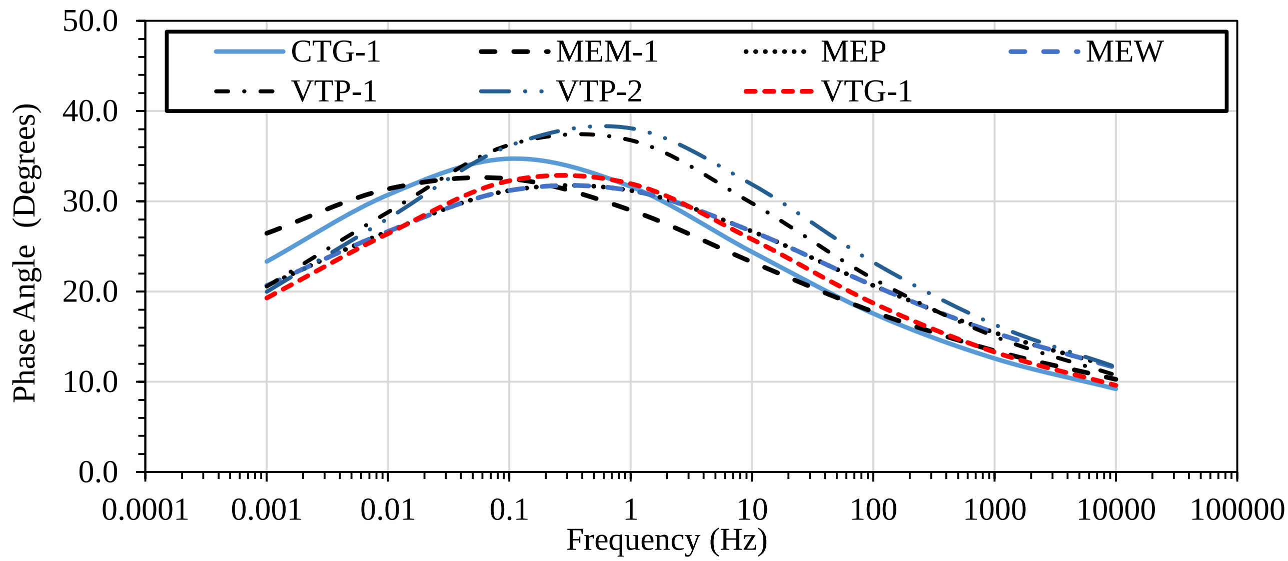

Fig. 4. **Fitted phase angle master-curves of the first set of mixtures**

In order to establish any type of correlation between rutting and the observed performance, it is necessary to define and select the critical rutting related points on the master-curves and resume the analysis on basis of them. As mentioned earlier the peak point on the phase angle master-curve can be physically interpreted and correlated to the mixture rut susceptibility but this point alone may not be enough to explain the mixture performance. For this reason, a second point on the pre-peak side of the phase angle master-curve should be selected for evaluation purposes. In this study, this second point on the master-curve is selected based on the HWTT loading and temperature conditions.

The HWTT test temperature is usually selected either with respect to binder high temperature performance grade (PGHT) or the regional climatic conditions needs. In New England area, this temperature is usually set at 45°C. Also, the frequency of the rolling wheel in HWTT is 52 passes per minute which is equal to 0.866 Hz. Thus, the second point on the master-curves is selected as such to be equivalent to 45°C at 0.866 Hz. The corresponding reduced frequency on the master-curves is calculated through using the appropriate shift factors for this temperature and frequency combination. The investigated indices and their calculation are described in Table 3. The subsequent text discusses each index individually.

Table 3. Proposed complex modulus based index parameters to evaluate the rutting performance

Index Parameter	Calculation Method
I	$[\delta_A - \delta_B] / \lvert f_A - f_B \rvert$
II	$[\lvert E^*_A \rvert - \lvert E^*_B \rvert] / \lvert f_A - f_B \rvert$
III	$[\lvert E^*_A \rvert - \lvert E^*_B \rvert] / \lvert f_A - f_B \rvert^2$
IV	$\lvert E^*_c \rvert / [\delta_c \times \lvert f_A - f_B \rvert^2]$
V	$[\lvert E^*_A \rvert - \lvert E^*_B \rvert] / [[\delta_A - \delta_B] \times \lvert f_A - f_B \rvert^2]$

The first index (I) investigates the rheological properties of the mixtures by determining the rate of drop in phase angle with respect to changes in frequency without considering the effect of stiffness in the analysis. The second index (II), evaluates the dynamic modulus drop rate with respect to changes in frequency to investigate the effect of stiffness. The third index (III) is similar to the second index (II) in the context of using the rate of changes in dynamic modulus over the loading frequency. The main difference here is that Index (III) uses the squared effect of logarithm of frequency to increase the emphasis on the effect of loading rate on the rutting. Further analysis confirmed that the squared logarithm of frequency can improve the correlation between the rutting and the indices. Although it might seem that indices (II) and (III) do not consider the effect of phase angle in the parameters, the selection of the reduced loading frequencies in these parameters is a direct function of where the peak phase angle happens. Indices IV and V try to incorporate the effect of modulus, phase angle and frequency at the same time. Generally, from the Superpave binder PG grading system, it is well known that shear modulus divided by the phase angle ($|G^*|/\sin\delta$) has been a good indicator of binder rutting properties and indices IV and V are developed based on this logic. It should be mentioned that in order to develop a reliable index parameter, many different indices were investigated, however on basis of the observations and strength of correlations between the HWTT results and indices, only five were selected for discussion in the paper.

RESULTS AND DISCUSSION

Evaluation Of The Rutting Indices Through HWTT Data

The calculated values and the ranking from each of individual introduced rutting index parameters are shown in Table 4. With respect to how the indices have been mathematically written and physically described, a lower calculated value for Index (I) is more desirable indicating less decrease in phase angle due to loading/temperature whereas for indices (II), (III), (IV) and (V), a larger calculated value indicates a better rut resistance mixture.

Mixture Ranking

As it can be seen in Table 4, Index (I) is not capable of determining the mixtures performance because of merely focusing on the phase angle drop rate. Although Index (II) is showing some success in predicting the ranking of 3 mixtures, in general it over/underestimates the mixtures rutting performance. The ranking order indicates the high capability of Index (III) in terms of its determining the mixtures rutting performance order. With regards to this index, the only difference is between the order of VTG1 and CTG1 mixtures. With respect to HWTT results, the difference between the measured rut for these two mixtures is less than 0.13mm or 3.5% which can be reasonably considered as negligible. The usefulness of Index (III) is clearer when considering mixtures like MEM1 and MEW. The two mixtures have similar properties in terms of binder type, RAP percentage, aggregate size and binder content. Considering these properties one may expect very similar rutting performance for the two mixtures while one is significantly more rut-resistant than the other. Similar to HWTT results, Index (III) has been able to well predict this difference. Also considering the dynamic modulus master-curves for VTP-1 and MEP, the latter has a relatively higher stiffness in the lower end of the master-curve and a single point type index would indicate it to have a higher rutting resistance. However, the HWTT results show MEP to be a poor performer as compared to VTP-1 and this is correctly captured by Index (III). Indices (IV) and (V) resulted in same type of ranking compared to each other, but with respect to HWTT ranking, the results are not promising.

In summary, it seems that Index (III) has a better capability in ranking the mixtures with respect to HWTT. However, it is necessary to evaluate these parameters through the field rutting performance. The next subsection will investigate the introduced parameters through ranking a set of 6 other mixtures for which the field rutting data is available after 5 years of in service.

Table 4. Calculated value and ranking of individual rutting index parameters for the first set of mixtures

Mixture	HWTT Results		Index and Rank									
	HWTT Rut Depth (mm)	Rank	I	Rank	II	Rank	III	Rank	IV	Rank	V	Rank
MEP	5.5	**7**	4.1	**3**	387.3	**6**	173.5	**7**	5.6	**5**	20.6	**5**
MEM-1	3.8	**3**	2.9	**1**	353.5	**5**	302.0	**3**	11.9	**1**	80.7	**1**
MEW	5.5	**6**	4.0	**2**	314.1	**7**	194.3	**6**	5.9	**4**	30.0	**4**
VTP-1	4.4	**4**	6.3	**6**	396.6	**4**	207.6	**4**	5.0	**6**	17.4	**6**
VTP-2	4.8	**5**	6.5	**7**	417.0	**3**	205.5	**5**	4.7	**7**	15.6	**7**
VTG-1	3.6	**1**	5.0	**5**	590.9	**1**	339.2	**2**	11.3	**3**	38.8	**3**
CTG-1	3.7	**2**	4.6	**4**	505.7	**2**	353.4	**1**	11.7	**2**	53.7	**2**

Evaluation Of The Rutting Indices Through Field Rutting Data

As mentioned in the previous subsection, the introduced rutting index parameters will be further evaluated through comparing the index parameter based ranking to that of the field performance. The 6 investigated mixtures in this subsection are placed on I-93 as part of the North-East High RAP Pooled Fund Study. The field performance of this set of mixtures has been monitored yearly through construction of six test sections in 2011 on the southbound lanes on I-93 between exits 30 and 32 in Woodstock and Lincoln, New Hampshire. It is worth mentioning that the distresses have been measured through an automated pavement distress data collection van by New Hampshire DOT. Since the pavement structure, traffic and climatic condition is the same for these mixtures, it would be possible to compare and rank the mixtures independent from other variables that can affect the pavement overall response and performance. The mix design properties for this second set of mixtures are summarized in Table 5.

Table 5. Mixture design and properties of the second set of mixtures to verify the index parameters

Mixture	Binder Grade	NMAS (mm)	RAP (%)	AC (%)	VMA (%)	VFA (%)	Field Air Void (%)	Pavement Rut Depth (mm)
Virgin-58-28	58-28	12.5	0	5.9	16.8	74	5.4	4.15
15%RAP-58-28	58-28	12.5	15	5.6	16.9	74.2	5.3	5.24
25%RAP-58-28	58-28	12.5	25	5.8	16.7	75.3	5.9	3.96
25%RAP-52-34	52-34	12.5	25	6.0	16.5	79	5.3	3.73
30%RAP-52-34	52-34	12.5	30	6.0	16.4	78.1	6.2	3.84
40%RAP-52-34	42-34	12.5	40	4.9	17	75.2	4.5	2.85

As it can be seen from the table, the 15%RAP-58-28 mixture has the highest rut depth whereas the 40%-52-34 has the lowest. Unlike these two, the rest of the mixtures although possess different values of rut depth, their magnitude is generally very close to each other and one can reasonably consider them as similar rut performing mixtures at least in the first five years after construction. It should be noted that there is inconsistency among the mixtures' measured field air void which might seem to affect the rutting ranking and performance. However, the air voids of the first four mixtures in the table are close to each other and within one standard deviation of the mean for all the mixtures. The air void level of the last two mixtures in the table (30%RAP-52-34 and 40%RAP-52-34) are only marginally away from one standard deviation from the mean.

The other noteworthy point in the table is the higher rut depth of the mixture with stiffer binder PG grade with the same RAP content in the case of 25%-58-28 and 25%-52-34 mixtures. A possible reason for this observation is related to the binder properties and the binder's ΔT_{cr} parameter. A comprehensive study has been conducted on different aspects of properties of these mixtures and further discussion about these properties is beyond the scope of this paper (Daniel et al. 2018). The dynamic modulus and phase angle master-curves were constructed and fitted through the same procedure explained in section 2.1. The fitted dynamic modulus and phase angle master-curves are depicted in Figure 5 and Figure 6 respectively.

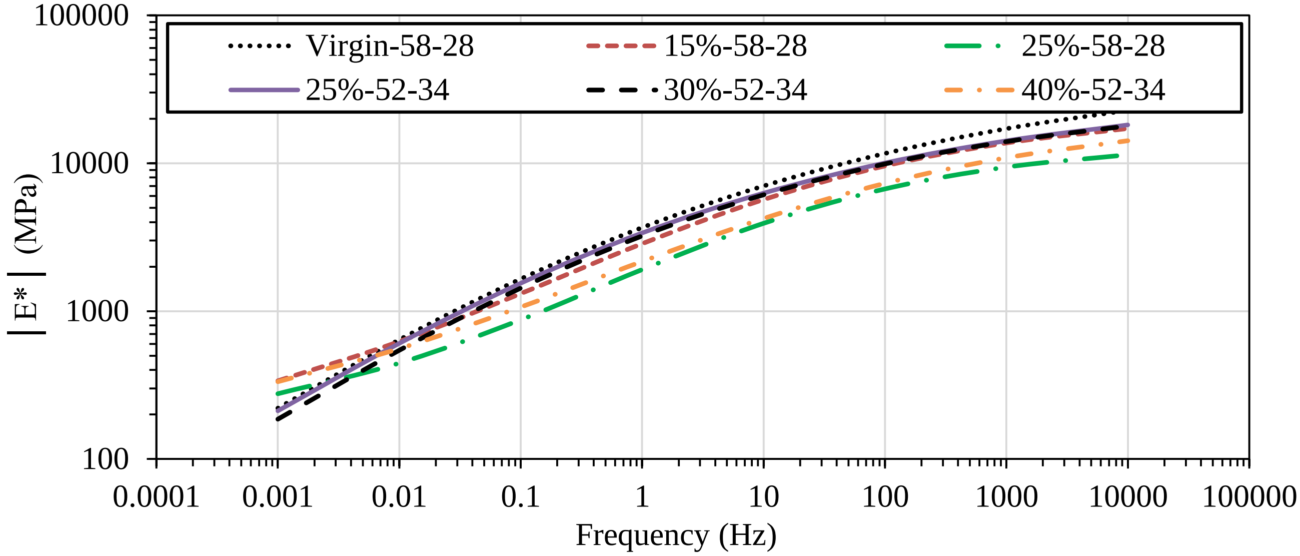

Fig. 5. Fitted |E*| master-curves of the second set of mixtures

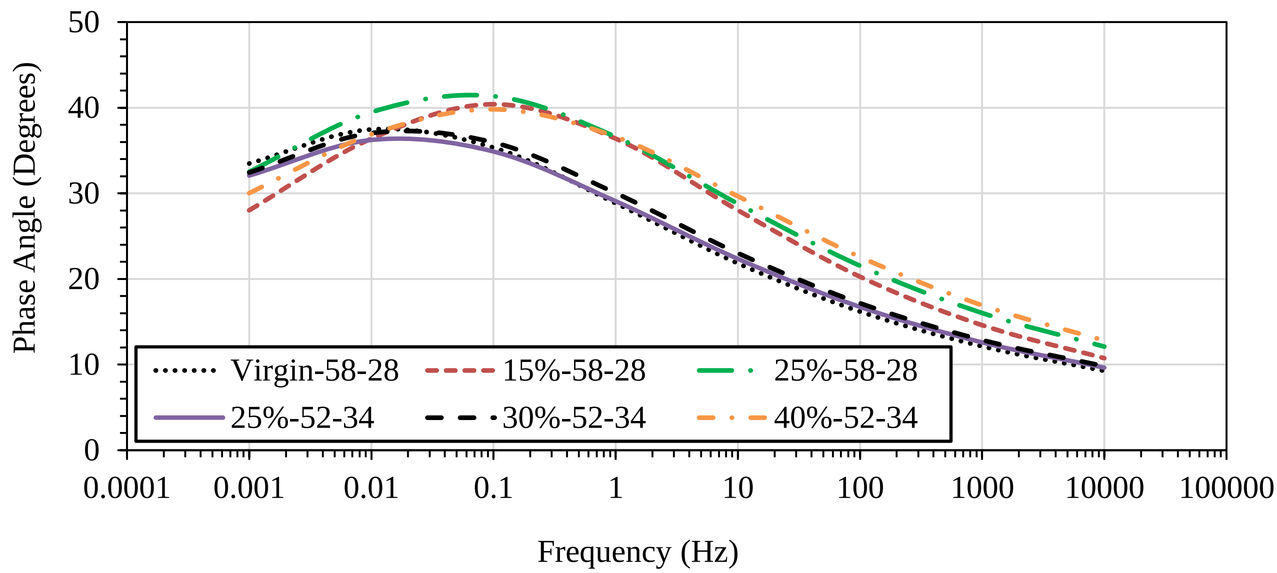

Fig. 6. Fitted phase angle master-curves of the second set of mixtures

Mixture Ranking

The five different indices were calculated for the 6 mixtures in order to make comparisons to the field distress measurements. The calculated indices and the rankings have been depicted in Table 6. As it can be seen from the table, except for Index (II) all other indices have been able to capture the worst (15%-58-28) rut resistant mixture. Based on the results, Index (I) seems not be a good enough tool to indicate the rutting susceptibility. Also, Index (II) has not been able to predict the ranking of majority of the mixtures. With respect to Index (III), the ranking comparison indicates that this index has identical ranking to the field conditions. Also, a comparison between the calculated index parameters and the rut depths reveals that Index (III) is able to better distinguish the relative difference between the mixtures in addition to ranking them. Index (IV) indicates to be incapable of ranking the mixtures which was also seen previously for the first set of the study mixtures dataset. On the other hand, Index (V) is showing promising results in terms of discriminating the mixtures rutting ranking with respect to field data.

Table 6. Values of Rutting Index Parameters for High RAP Pooled Fund Study Mixtures

Mixture	Field Results		Index and Rank									
	Field Rut Depth (mm)	Rank	I	Rank	II	Rank	III	Rank	IV	Rank	V	Rank
Virgin-58-28	4.1	**5**	3.1	**2**	351.8	**6**	355.6	**5**	12.5	**4**	117.3	**4**
15%-58-28	5.2	**6**	6.0	**6**	505.4	**1**	267.1	**6**	6.7	**6**	23.3	**6**
25%-58-28	4.0	**4**	3.1	**4**	358.6	**5**	397.1	**4**	17.5	**2**	141.4	**5**
25%-52-34	3.7	**2**	2.9	**1**	498.6	**2**	478.8	**2**	14.0	**3**	156.1	**2**
30%-52-34	3.8	**3**	3.2	**5**	469.3	**4**	425.4	**3**	11.4	**5**	117.7	**3**
40%-52-34	2.9	**1**	3.1	**3**	497.6	**3**	496.3	**1**	20.7	**1**	160.9	**1**

Evaluation Of The Index Parameters Through Mixture Design Properties Variations

As one of the main goals for exploration of a complex modulus based rutting performance index parameter in this study is to estimate the mixtures performance and screen the mixtures based on the complex modulus results ahead of conducting the specific rutting test such as the HWTT, it is necessary to examine the performance index parameters through a sensitivity analysis with respect to mixture design properties. In order to accomplish this goal a third set of mixtures with the same gradation and varying design air void and asphalt content were selected to determine how each of the index parameter can capture these variations in the mix design and consequently the rutting performance. The mixtures were designed and compacted at three different levels of air void and binder content (resulting in 9 different combinations). The mixture design and properties are summarized in Table 7. The complex modulus specimens were fabricated and tested in accordance to AASHTO TP342 test method and the master-curves were constructed and fitted at 21.1°C. The dynamic modulus and phase angle master-curve plots are depicted in Figure 7 and Figure 8 respectively. It can be seen from the plots that for each set of binder content the dynamic modulus master-curves become relatively softer (indicate lower modulus values) with increasing air void but no specific trend is observed for phase angle master-curves which makes the overall performance prediction challenging through comparing only master-curves.

Table 7. Properties of the third set of mixtures used to examine the index parameters based on altering the mix design properties

Mixture	Binder Grade	NMAS (mm)	RAP (%)	AC (%)	AV (%)
5.9AC-4AV	64-28	9.5	0	5.9	4
5.9AC-7AV	64-28	9.5	0	5.9	7
5.9AC-9AV	64-28	9.5	0	5.9	9
6.3AC-4AV	64-28	9.5	0	6.3	4
6.3AC-7AV	64-28	9.5	0	6.3	7
6.3AC-9AV	64-28	9.5	0	6.3	9
6.8AC-4AV	64-28	9.5	0	6.8	4
6.8AC-7AV	64-28	9.5	0	6.8	7
6.8AC-9AV	64-28	9.5	0	6.8	9

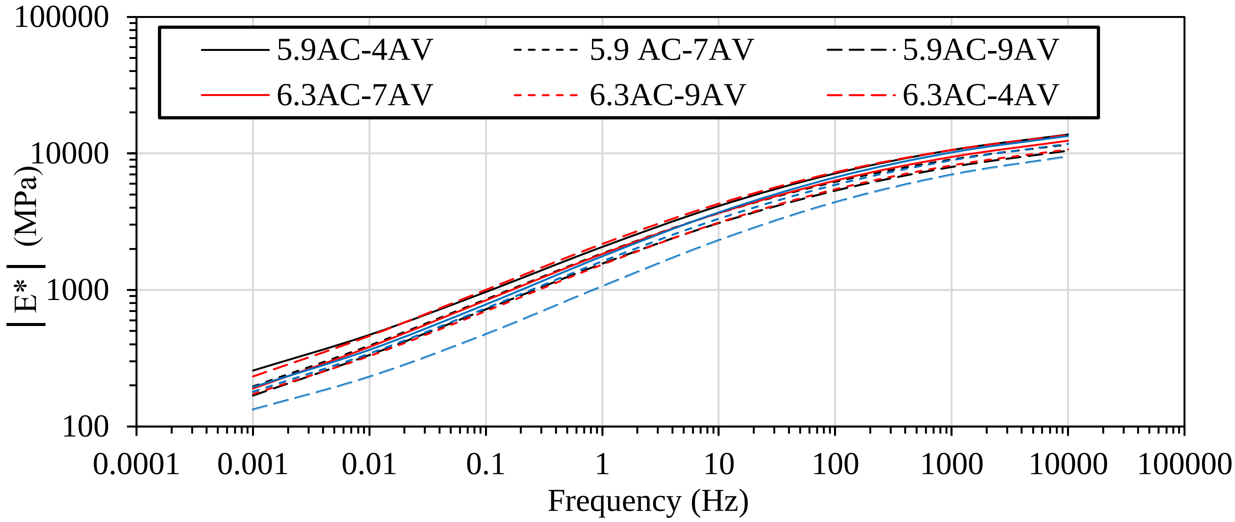

Fig.7. Fitted |E*| master-curves of the third set of mixtures

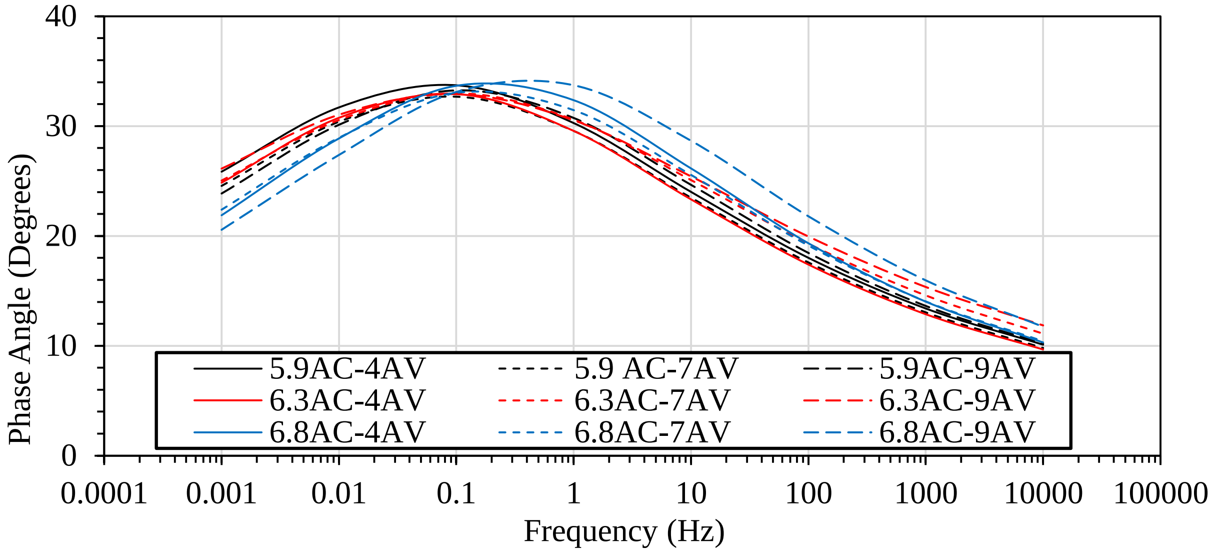

Fig.8. Fitted phase angle master-curves of the third set of mixtures

Mixture Ranking

In order to evaluate the capability of the index parameters to estimate the mixtures rutting performance, the ranking was conducted in three different categories for each variable separately. Based on the general expectations from volumetric point of view, a mix with lower binder content would be more rut resistant compared to one with higher binder content. In general, mixes with excessively high air void levels have potential for rutting due to lower stiffness and mixtures with a low air voids (typically below 4%) also have high propensity for rutting due to lack of sufficient air voids to allow expansion of binder during high temperatures. However, in the data-set used in this paper, the air void levels are within 4 to 9% range and not sufficiently varied to draw conclusions regarding air void associated rutting performance prediction without performing lab test, such as HWTT. Nonetheless to make full comparisons for effects of both air-voids and asphalt binder contents, the mixtures are ranked in two ways; first, a constant air void level and varying binder content (Table 8) and second, a constant binder content and varying air void level (Table 9)

Table 8. Ranking is based on the varying binder content at constant air void level

Mixture	Volumetric based Expected Rank	Index and Rank									
		I	Rank	II	Rank	III	Rank	IV	Rank	V	Rank
5.9AC-4AV	1	3.1	1	366.5	3	239.5	1	8.1	1	41.2	1
6.3AC-4AV	2	4.0	2	385.7	2	229.5	2	7.0	2	33.7	2
6.8AC-4AV	3	4.8	3	409.1	1	224.7	3	6.4	3	25.7	3
5.9AC-7AV	1	3.4	3	379.6	1	281.7	1	9.8	1	61.2	1
6.3AC-7AV	2	3.6	2	359.3	3	212.4	2	6.3	2	34.9	2
6.8AC-7AV	3	4.4	1	360.0	2	197.4	3	5.7	3	24.4	3
5.9AC-9AV	1	3.9	2	336.5	1	218.1	1	6.9	2	36.2	2
6.3AC-9AV	2	2.8	1	304.5	2	217.7	2	7.5	1	55.4	1
6.8AC-9AV	3	5.1	3	266.9	3	117.0	3	3.1	3	10.0	3

Table 9. Ranking is based on the varying air void level at constant binder content

Mixture	Volumetric based Expected Rank	Index and Rank									
		I	Rank	II	Rank	III	Rank	IV	Rank	V	Rank
5.9AC-4AV	1	3.8	2	366.5	2	239.5	2	8.1	2	41.2	2
5.9AC-7AV	2	3.4	1	379.6	1	281.7	1	9.8	1	61.2	1
5.9AC-9AV	3	3.9	3	336.5	3	218.1	3	6.9	3	36.2	3
6.3AC-4AV	1	4.0	3	385.7	1	229.5	1	7.0	2	33.7	3
6.3AC-7AV	2	3.6	2	359.3	2	212.4	3	6.3	3	34.9	2
6.3AC-9AV	3	2.8	1	304.5	3	217.7	2	7.5	1	55.4	1
6.8AC-4AV	1	4.8	2	409.1	1	224.7	1	6.4	1	25.7	1
6.8AC-7AV	2	4.4	1	360.0	2	197.4	2	5.7	2	24.4	2
6.8AC-9AV	3	5.1	3	266.9	3	117.0	3	3.1	3	10.0	3

It can be observed from the tables that with respect to variations in the binder content (Table 8) while the rankings from parameters (I) and (II) are not promising, Index (III) has been able to fully predict the mixtures ranking and indices (IV) and (V) show partial success in ranking the mixtures. With respect to varying air void (Table 9) Index (II) followed by Index (III) best rank the mixtures. In general, and considering all three sets of investigated mixtures, Index (III) shows to be a reliable tool in ranking the mixtures rutting resistant capability. However, in many instances, it is not only the ranking that is important, but also the correlation between the index parameter an the test/field measurements is of high interest in order to determine the relative difference in the performance to design and select the most cost effective mixture. The correlation between the index parameters and the test/field measurements is discussed in the next subsection.

Correlation Of The Index Parameters With The HWTT/Field Rut Measurements

As it was seen in the previous section, some of the indices were able to rank the mixtures in a similar way to the ranking from the volumetric based expectations. However, the ability of the indices to differentiate the mixtures performance would be different. Therefore, it is necessary to further investigate the correlation between the index values with the actual measurements from field and the HWTT test results. Figure 9 reveals the correlation and goodness of fit in terms of (R^2) between different index parameters and the measured rut depths for the first and second set of the study mixtures. Using the standard least squared method to determine the line of best fit the R^2 value was determined for both sets of data for individual index parameters. It can be seen that fairly good correlation exists between the indices and the rutting measurements except for Index (I) where there is no correlation between the index and the HWTT measurements. Amongst all the indices, Index (III) indicates a clearly strong linear correlation with both the field and HWTT rut depth measurements. Considering the ranking results from the previous sections, this index can reasonably determine the mixtures relative rutting performance difference in addition to ranking them. Also, with respect to Index (III), for most of the mixtures, the ratio of the two measured rut values and their calculated index parameter counterpart is closely comparable indicating the capability of this parameter in determining the relative difference in performance between mixtures.

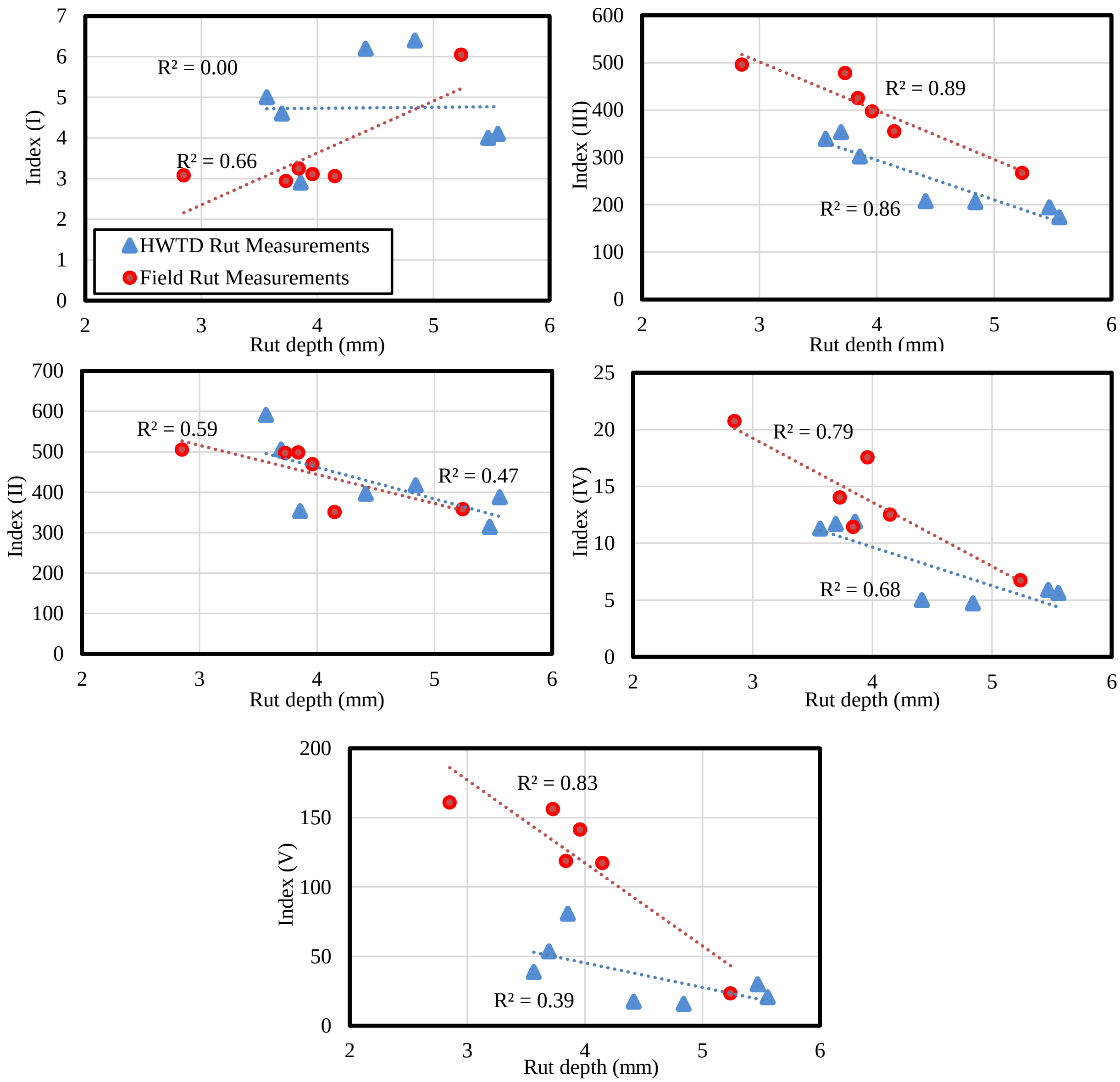

Fig.9. Correlation between the index parameters and the measured rut depths

SUMMARY, CONCLUSION AND FUTURE WORK

Dynamic modulus $|E^*|$ as a measure of stiffness is generally considered as an indicator for asphalt mixtures rutting resistance where a mixture with higher modulus value frequency is generally considered to be less rut susceptible. However, this hypothesis ignores the viscoelastic behavior of the asphalt mixtures and the phase angle as the viscous part of the response. This research introduces and investigates 5 different complex modulus based index parameters to evaluate the asphalt mixtures rutting susceptibility. These parameters implement the full linear viscoelastic properties of the asphalt mixtures (dynamic modulus and phase angle) at two specific points on the master–curves. The first point is associated to the frequency at which the peak phase angle takes place. The second point is selected based on the Hamburg Wheel Tracking Test (HWTT) test loading frequency and temperature (52 passes/min equivalent to 0.866Hz at 45°C) and its equivalent reduced frequency on the master-curves.

The investigations were conducted on three different sets of plant produced lab compacted mixtures (total of 22 mixtures). The first set includes 7 mixtures for which the HWTT test results are available to develop the parameters. The second set comprises from 6 mixtures for which the field rut depths in the 5th year after construction are used for verification of the introduced rutting index parameters. The third set of the mixtures is used to evaluate the index parameters with respect to volumetric variations in the mixture design where no rut measurements would be available. In addition to mixture ranking, the strength of correlation between the rut depths and the index parameters was evaluated through statistical analysis and the goodness of fit (R^2).

Amongst the 5 introduced parameters, Index (III) revealed high capability in mixture ranking while maintaining high correlation with both the HWTT and field measured rut. The results indicated that this index can be used as a preliminary tool in evaluating and screening the mixtures rutting performance.

In order to implement the index parameters in this study, it is important to consider the regional mixture design properties (especially in southern United States) in selecting the HWTT related point on the master-curves. For stiffer mixtures, there is a possibility for the second critical point (f_b) to be projected on the post-peak side of the phase angle master-curve. In this situation, it is recommended to use the high binder PG grade temperature as opposed to the conventional HWTT test temperature to determine the reduced frequency on the master-curve.

As a future step in this study, more field data should be used to determine the rutting threshold values for different types of pavement structures and traffic levels so that the parameters can be utilized in development of performance space diagrams as part of balanced mixed design approach to optimize the binder content in the mixture through a mechanistic based mixtures design.

REFERENCES

AASHTO, T. "Standard method of test for Hamburg wheel-track testing of compacted hot mix asphalt (HMA)." (2011).

Apeagyei, Alex K. "Rutting as a function of dynamic modulus and gradation." *Journal of Materials in Civil Engineering* 23.9 (2011): 1302-1310.

Bazzaz, Mohammad, et al. "A straightforward procedure to characterize nonlinear viscoelastic response of asphalt concrete at high temperatures." *Transportation Research Record* (2018): 0361198118782033.

Birgisson, Bjorn, et al. "The use of complex modulus to characterize the performance of asphalt mixtures and pavements in Florida." *Final report* (2004): 4910-4501.

Bhasin, Amit, Joe W. Button, and Arif Chowdhury. *Evaluation of simple performance tests on HMA mixtures from the south central United States*. USA: Texas Transportation Institute, Texas A & M University System, 2003.

Brown, E. Ray, Prithvi S. Kandhal, and Jingna Zhang. "Performance testing for hot mix asphalt." *NCAT report* 1.05 (2001).

Daniel, Jo Sias, et al. "Comparison of asphalt mixture specimen fabrication methods and binder tests for cracking evaluation of field mixtures." *Road Materials and Pavement Design* (2018): 1-17.

EI, Nur Hossain, and Dharamveer Singh. "Dynamic modulus-based field rut prediction model from an instrumented pavement section." *Procedia-Social and Behavioral Sciences*104 (2013): 129-138.

Mohammad, Louay N., et al. "Permanent Deformation Analysis of Hot-Mix Asphalt Mixtures with Simple Performance Tests and 2002 Mechanistic–Empirical Pavement Design Software." *Transportation Research Record* 1970.1 (2006): 133-142.

Mohammad, Louay N., et al. *Hamburg Wheel-Track Test Equipment Requirements and Improvements to AASHTO T 324*. No. NCHRP Project 20-07/Task 361. 2015.

Nemati, Rasool, Eshan V. Dave and Jo S. Daniel, “Comparative Evaluation of New Hampshire Mixture Based on Lab performance Tests” Proceedings of *International Society of Asphalt Pavements, 2018.*

Nemati, Rasool, and Eshan V. Dave. "Nominal property based predictive models for asphalt mixture complex modulus (dynamic modulus and phase angle)." *Construction and Building Materials* 158 (2018): 308-319.

Sivasubramaniam, Sivaranjan, and John E. Haddock. "Validation of Superpave mixture design and analysis procedures using the NCAT test track." *Joint Transportation Research Program* (2006): 252.

Witczak, M. W., H. L. Von Quintus, and C. W. Schwartz. "Superpave support and performance models management: Evaluation of the SHRP performance models system." *Eighth International Conference on Asphalt Pavements". Federal Highway Administration*. No. Volume III. 1997.

Yildirim, Yetkin, and K. Stokoe. "Analysis of Hamburg wheel tracking device results in relation to field performance." *Analysis* (2006).

Witczak, Matthew W. *Simple performance test for superpave mix design*. Vol. 465. Transportation Research Board, 2002.

Zhao, Yanqing, and Y. Kim. "Time-temperature superposition for asphalt mixtures with growing damage and permanent deformation in compression." *Transportation Research Record: Journal of the Transportation Research Board* 1832 (2003): 161-17

Appendix: Paper 5 (Chapter 8)

Title: Development of a Rate-Dependent Cumulative Work and Instantaneous Power based Asphalt Cracking Performance Index

Journal: Road Materials and Pavement Design

Development of a Rate-Dependent Cumulative Work and Instantaneous Power based Asphalt Cracking Performance Index

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ABSTRACT

Use of the semi-circular bending (SCB) test has gained popularity for evaluating cracking performance of asphalt mixtures. An Illinois Flexibility Index Test (I-FIT) variant of SCB has shown the ability to distinguish mixtures through use of the flexibility index (FI) parameter. While this index has been able to rank the mixtures with respect to performance, a high coefficient of variation (COV) among the replicates has often been observed. Furthermore, parameters such as total fracture energy and FI do not incorporate rate-dependency of fracture processes which are very important for viscoelastic materials such as asphalt mixtures at low and intermediate temperatures. In light of these observations, a rate dependent cracking index (RDCI) is proposed that utilizes cumulative fracture work potential and instantaneous power calculated from the I-FIT test to assess impulse of the mixture. Thus, in spirit, this parameter captures the fracture energy and crack velocity of the material; however, these are calculated in a rate-dependent manner. A total of 18 surface course mixtures were analysed using the RDCI and resulted in an average overall reduction of 10.6% in COV as compared to FI while maintaining similar ranking of mixtures. In general, RDCI was able to better discriminate the 18 mixtures as compared to FI.

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Evaluation of five mixtures at three aging levels showed robustness of RDCI in capturing effects of aging on fracture behaviour of asphalt mixtures.

Keywords: Semi-Circular Bend (SCB), Cracking, Flexibility Index, Cumulative Energy, Instantaneous Power

INTRODUCTION

Cracking is one of the major structural distresses in asphalt mixtures and has been widely investigated by researchers. Based on different mechanistic theories, numerous laboratory testing methods have been proposed to characterize the cracking performance of asphalt mixtures. Fracture mechanics has extensively informed development of laboratory tests and as they relate to the formation and growth of cracks with respect to material's microstructure, loading rate and environmental circumstances. The application of fracture mechanics in characterizing the cracking performance of asphalt mixtures has been documented as early as the 1970s. Using a simple beam test under cyclic loading, a study by Majidzadeh et al. aimed to relate the stress intensity factor to the crack growth rate with respect to Paris' law for asphalt mixtures' fatigue performance (Majidzadeh, Kauffmann, & Ramsamooj, 1971; Paris & Sih, 1965).

With respect to the fracture mechanics for heterogeneous composites such as asphalt mixtures, materials are assumed have a uniform distribution of pre-existing flaws. In order to conduct a laboratory test with stable crack growth and to hone in on the energy needed to propagate that crack, a specimen with a pre-existing crack or notch is needed. Using these concepts, Wagoner et al. explored the use of a single-edge notched beam (SENB) test to quantify the fracture properties of asphalt mixtures under repetitive loading (Wagoner, Buttlar, & Paulino, 2005). The geometry of the SENB test is challenging in terms of obtaining field samples from existing pavements, so alternative testing methods such as disk-shaped compact tension (DCT) and semi-circular bend (SCB) tests have been proposed (Molenaar, Scarpas, Liu, & Erkens, 2002; Molenaar, J. &

Molenaar, M. 2000; Wagoner, Buttlar, Paulino, & Blankenship, 2005). Both of these geometries can be prepared using cored specimens from pavements or from cylindrical gyratory samples following the standard Superpave mix design and compaction approaches. While the DCT test is generally used to characterize low temperature fracture properties of asphalt mixtures, the SCB test has been used to determine both low and intermediate temperature cracking performance (Elseifi, Mohammad, Ying, & Cooper III, 2012; Li & Marasteanu, 2010; Mohammad, Kim, & Elseifi, 2012). Since its first implementation in rock mechanics (Chong & Kuruppu, 1984) and later application in asphalt performance testing, the SCB test has been shown to have acceptable sensitivity to mix design variables (Al-Qadi et al., 2015) as well as loading rate and testing temperatures (Haslett, Dave, & Daniel, 2017). Moreover, the SCB has also shown potential to be used for characterizing the mixed-mode fracture properties of asphalt mixtures (Im, Ban, & Kim, 2014).

In order to determine the fracture properties of asphalt materials using the SCB geometry, the force versus load-line displacement (LLD) curve under a monotonic loading protocol has been investigated by different researchers (Li & Marasteanu, 2010; Saha & Biligiri, 2016). Figure 1 indicates a typical SCB force-LLD curve.

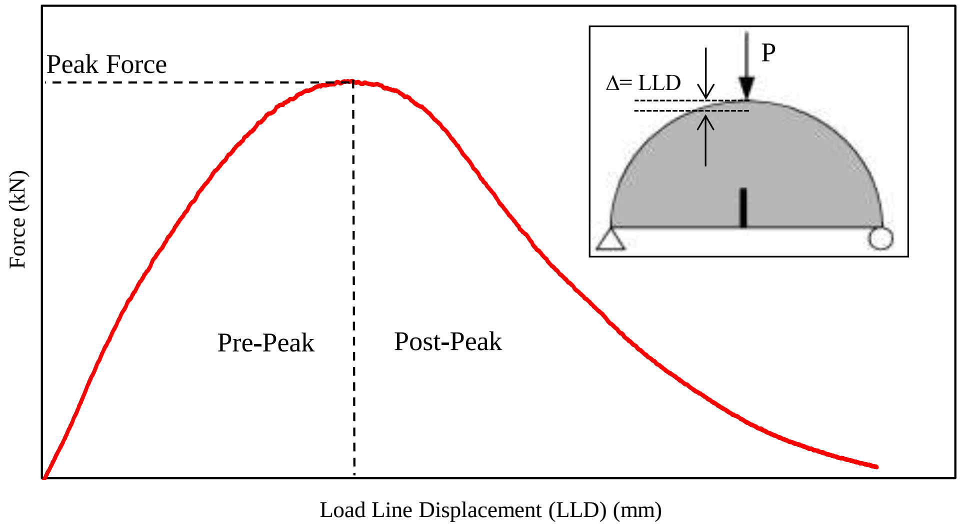

Figure 1: Typical Load-LLD curve.

The curve can be divided into two distinct portions with respect to the required force for the crack to initiate (pre-peak) which is followed by a decrease in the force when the crack propagates along the specimen (post-peak). As the force is applied, the portion of the specimen below the neutral axis undergoes tensile strains, which result in the accumulation of tensile stresses in the notch tip vicinity. These stresses bring about the creation of a region of micro-cracks, namely the fracture process zone (FPZ), in front of the crack tip. Although the length of the fracture process zone can be considered as a material specific property (Bažant & Kazemi, 1990), its determination requires either reliance on inverse analysis based modelling approaches or use of advanced laboratory characterization techniques, such as acoustic emissions (Li, Marasteanu, Iverson, & Labuz, 2006). Based on principles of fracture mechanics, the energy required for generation of a unit fracture surface area in a material is called the fracture energy (G_f) (Anderson, 2005). This energy is the sum of the positive surface energy (S) and the negative released strain energy (U). The surface energy is the energy absorbed during the crack growth because of the creation of newly made free surfaces as the atomic bonds break and the specimen's bulk energy is converted into the surface energy. The surface energy increases linearly with respect to crack length. On the other hand, the released strain energy is related to the unloaded region of material adjacent to the free surfaces as the crack is growing. The strain energy is proportional to the squared length of the crack (Anderson, 2005). Considering the order of correlation of these energies to the crack length, the energy required for the crack to propagate decreases at a critical crack length where the peak resistive force occurs in the force vs LLD curve (Figure 1).

In order to use the SCB geometry to evaluate the cracking properties of asphalt mixtures in a relatively simple manner, different testing protocols and analysis methods have been proposed and investigated. For instance, in the work conducted by Louisiana Transportation Research Centre (LTRC) (Mohammad et al., 2012), the SCB test is conducted at a loading rate of 0.5 mm/minute at 25°C using three different notch depths on specimens 75 mm diameter and 57 mm thick. The analysis of the test results is performed through determination of the critical strain energy rate (*J*-integral) (AASHTO TP105). The low temperature SCB fracture test, developed by (Li & Marasteanu, 2005), utilizes a 25 mm thick specimen that is tested using the crack mouth opening displacement (CMOD) rate of 0.015 mm/minute at low temperatures (typically in range of -12 to -40°C) and determines asphalt mixture's fracture energy and stress-intensity factor (AASHTO TP107). The third commonly used SCB testing method, which is of main interest in this paper, is the protocol developed by the Illinois Centre for Transportation, commonly referred to as the Illinois Flexibility Index Test (I-FIT) (Ozer et al., 2016). The test is conducted using a 50 mm/minute LLD rate at 25°C on specimens 50 mm thick and 75 mm diameter. The notch depth is constant among all the replicates and is equal to 15 mm (AASHTO TP124). The I-FIT test was originally developed with the purpose of discriminating the cracking performance of mixtures with varying amounts of recycled asphalt pavement/shingles (RAP/RAS) (Al-Qadi et al., 2015). In order to rank the mixtures through I-FIT results, the fracture energies (area under the force-LLD curve divided by the ligament area) of different mixtures were compared. The comparisons indicated the insufficiency of this parameter as mixtures with different rheological properties could result in similar fracture energy values. It should be noted that fracture energy here is a global fracture energy (not a material scale property) that is a function of the material's intrinsic fracture energy but dependent on specimen geometry and other testing factors (such as loading rate and test temperature). Due to poor discrimination between mixtures from fracture energy alone, other possible influential parameters from the force-LLD curve on the FPZ such as the peak load, the slope at the inflection point, and critical displacement were investigated (Al-Qadi et al., 2015). As a result, the flexibility index (FI), which is an engineering parameter, was developed to correlate the crack growth velocity and the brittleness of the mixtures.

$$FI = A \times {}^{G_f}\!/_{abs\,(m)} \qquad \text{Equation 1}$$

Where:

A= Unit correction coefficient taken as 0.01,

G_f = Fracture energy (J/m^2)

m = Slope at the inflection point

Although the flexibility index has generally been shown to be a good indicator of cracking performance, in many instances it results in relatively high coefficient of variation (COV) among the replicates, which can significantly reduce the practicality of using this parameter for routine use. The high COV results from the fact that the m-value is derived from the shape of the post-peak segment of the force-LLD curve and is highly sensitive to the gradation, density and air void distribution within the specimen, as well as other random variables such as operator variability etc. (Al-Qadi et al., 2015). For the same reasons, the FI may not be able to discriminate the performance of brittle or long-term aged mixture, as these mixtures may exhibit steep post peak curves resulting in indeterminate or quite low FI values (as low as 1) (Kaseer et al., 2018). Other studies have also indicated that the FI may not be sensitive to variations in asphalt content (Zhou et al., 2017).

As an alternate to FI, (Zhu et al., 2017) proposed the use of P_{max} to determine the fracture strength (S_f) from the DCT test. The fracture energy normalized by S_f resulted in an index parameter called Fracture Strain Tolerance (FST). FST was shown to lower the COV, while maintaining high discriminability among the mixtures. Researchers at the Texas A&M University also used the P_{max} as a normalizing factor for fracture energy and introduced the Crack Resistance Index (CRI) as an alternative to FI (Kaseer et al., 2018). It should be noted that CRI does not account for specimen to specimen geometric differences, whereas FST does account for specimen geometry in the index calculation. The comparisons for CRI indicated a decrease in COV for the short-term oven aged (STOA) mixtures compared to FI. However, the study indicated a higher variability of CRI compared to FI for long-term oven aged (LTOA) mixtures with both indices indicating similar trends for different mixtures. Moreover, CRI may need further evaluation since the peak load as a normalization factor may not be physically interpretable in terms of fracture process. There could also be examples of polymer modified mixtures with high fracture energy and high peak load where the CRI and FST may not be capable of discriminating among them.

A study conducted at the University of Arkansas used the concept of Resistance Curve (R-Curve) to determine mixture fracture properties. The R-Curve indicates the cumulative fracture energy as a function of crack extension. In general, if the slope of R-Curve is zero then the material is brittle and if the slope maintains a gradual increment then the behaviour is ductile (Anderson, 2005). The benefit of using the R-Curve is that it provides a dynamic trace of the strain energy with respect to crack growth and it can better explain the crack initiation and propagation mechanism. Therefore, the application of R-Curve can be supported by the fracture mechanics. In addition, The R-curve indicated a high potential for determining the effect of mixture properties as well as environmental factors on the cracking performance (Yang & Braham, 2018).

Most of the current approaches and parameters used in analysing the SCB test focus merely on the characteristics of the force-LLD curve whether it is the slope, peak force, or the crack extension. However, the factor of loading rate is an equally important influential parameter in characterizing viscoelastic material; this has been neglected in the development of existing index parameters for discriminating the fracture properties of asphalt mixtures. Based on the rate dependency of the viscoelastic material, it can be hypothesized that the development and growth of the FPZ and consequently the crack propagation, could be significantly different for mixtures with similar G_f, P_{max} or even post-peak slopes at the inflection point. However, the displacement measured by means of the extensometers or clip-on gauges in most of the fracture tests is an average deformation value that could be far from the actual FPZ and may not be appropriate to characterize the true fracture properties of viscoelastic materials at intermediate temperatures. A study conducted at the University of Nebraska indicated the importance of rate dependency of asphalt mixtures in capturing the local fracture processes and FPZ through analysing the SCB test results at different loading rates (1, 5, 10 and 50 mm/minute) using a digital image correlation system and finite element modelling (Im, Kim, & Ban, 2013).

The objective of this study is to explore use of a rate dependent cracking index parameter based on the I-FIT testing method which can be used to describe the crack initiation and propagation process with respect to the fracture processes in viscoelastic materials. The ability of this index to discriminate cracking resistance of asphalt mixtures as well as for a mixtures at different aging levels is evaluated and compared to the FI parameter. Furthermore, the resultant coefficient of variation for the new parameter (rate dependent cracking index, RDCI) is compared to FI.

DEVELOPMENT OF THE RATE DEPENDENT CRACKING INDEX (RDCI)

Perhaps, the rate dependency and hereditary behavior are the main distinguishing characteristics for viscoelastic materials that delineate them from elastic solids. Multiple studies (Bažant & Li, 1997; D'Amico et al., 2013) have indicated the importance of time dependency in the fracture mechanics of viscoelastic materials and indicate that the crack growth in viscoelastic materials originates from viscoelastic deformation in the process zone. This deformation provides the required energy for gradual propagation of the crack with respect to time as opposed to brittle materials such as metals (Bradley, Cantwell, & Kausch, 1997). For example, a study conducted by Chung and Williams (Chung & Williams, 1991) used a three-point bend notched specimen to evaluate the effect of time along with the load line displacement measurements in the cracking process. The results indicated that before crack growth initiates, the displacement as a function of time is caused by the viscoelastic deformation. As the crack growth commences, the viscoelastic constitutive relationships combined with compliance can be applied to calculate the crack size with respect to time and consequently the crack growth rate and stress intensity factor using only simple LLD versus time measurements.

In order to develop a simple, useful rate dependent cracking index in this study, three main parameters in the I-FIT test and the force-LLD curve were considered:

1- The cumulative work (W_c) as a function of time (t)
2- The peak force (P_{max})
3- Times to reach the peak force and 10% of the peak force (post-peak) on the force-LLD curve (t_c). (The use of these specific times is discussed in the following portion of the paper.)

The average fracture energy has been used as the main parameter in evaluating the mix crack resistance in the I-FIT test. However, it was indicated that a single average value may not be able to differentiate the behaviour of the mixture during crack growth. Therefore, the cumulative work (accumulated area under the force-LLD curve, Figure 2) as a function of time is used to lower the challenge faced by use of the fracture energy value in terms of its inability to capture the crack velocity.

The cumulative work over time not only exhibits the history of the dissipated work during the crack growth, but it can also be used to indicate the crack resistance rate at any time during the loading period. From a mechanistic perspective, the rate of the work over time ($\frac{\Delta W}{\Delta t}$) is power ($P$), which simply indicates the amount of energy transferred per unit time. Thus, when evaluating a material's fracture resistance potential, it can be hypothesized that for a certain duration from the start of loading application (indicated by t_0), a material requiring more power will exhibit more brittle behaviour due to a larger amount of stored potential strain energy. In the case of viscoelastic materials such as asphalt mixtures, while part of the energy is stored as a potential strain energy, a portion of the energy is spent towards the creep dissipation prior to the crack initiation. A coupled experimental and numerical simulation based analysis can provide the decomposition of the potential strain energy and creep dissipation (Song, Paulino, & Buttlar, 2006). However, for routine usage of a cracking index parameter such analysis is not feasible. Over a smaller range of time, such that when Δt approaches 0, it can be reasonably assumed that power is the rate of the work with respect to time, i.e. $\frac{\Delta W}{\Delta t} \approx \frac{\mathrm{d}W}{\mathrm{d}t}$. Typically, this slope is referred to as the instantaneous power (P_t), which is a scalar quantity that indicates the instantaneous energy dissipation rate and can be rewritten as follows:

$$P_t = \frac{\mathrm{d}W}{\mathrm{d}t} = \boldsymbol{F}\cdot\frac{\mathrm{d}\boldsymbol{x}}{\mathrm{d}t}; \frac{\mathrm{d}\boldsymbol{x}}{\mathrm{d}t} = \boldsymbol{V} \qquad \text{Equation 2}$$

Therefore:

$$P_t = \boldsymbol{F} \cdot \boldsymbol{V} \qquad \text{Equation 3}$$

Where:

P_t = instantaneous power; $\boldsymbol{F}$= force; $\boldsymbol{V}$= instantaneous velocity

The instantaneous power is the scalar product of force and velocity at any time (t) during the test. During the crack initiation process, the rate of instantaneous power will change drastically; this is due to transition of the energy state of the material from predominantly controlled by potential strain and creep dissipation modes to fracture dissipation dominant mode. The crack initiation usually occurs in quasi-brittle materials such as asphalt mixtures when the internal stresses approach the tensile strength of the material (Dave & Behnia, 2018). This instance can be reasonably assumed to happen near the occurrence of the peak force in a fracture test. Therefore, the time required for the peak load to occur will be used as the first time point in this study; this is indicated by symbol t_{peak}. Defining a second critical point of time is necessary to consistently define the ending point of the I-FIT test where it is assumed that full crack propagation has happened and the test specimen has no more load carrying capacity. For test practicality purposes and to prevent damage to test equipment, tests are typically stopped at 10% peak force. Therefore, the time at which 10% of peak force occurs, in the post peak segment of the load-LLD curve, is chosen as the second time point of interest.

As a summary, in order to explore a rate dependent cracking index parameter the following variables have been introduced and described:

- Cumulative work with time, W_c
- Instantaneous power at the peak force, $P_{t\,=\,peak}$
- Two times of interest on the force-LLD curve;
    - Time at peak load, t_{peak}
    - Time at 10% peak load at post-peak, $t_{0.1peak}$

In order to calculate various parameters from the fracture test results, the cumulative work (W_c) and time data were fitted using a polynomial equation and the area under the curve from the t_{peak} to $t_{0.1peak}$ were calculated. In order to focus on the fracture work associated with propagation of the crack in the specimen, the area under the cumulative work and time from start of test to t_{peak} was excluded from the integration. The resulting area is then normalized by the product of the instantaneous power at peak force ($P_{t\,=\,tpeak}$) and the fractured ligament area (product of fracture width and length) to calculate an index referred to as the rate dependent cracking index (RDCI):

$$\boldsymbol{RDCI} = \frac{\int_{t_{peak}}^{t_{0.1peak}} \boldsymbol{W_c}\ .\boldsymbol{dt}}{\boldsymbol{P_{tpeak}} \times \boldsymbol{ligament\ area}} \times C \qquad \text{Equation 4}$$

Where:

RDCI = rate dependent cracking index ($\frac{s^2}{m^2} \times 10^4$)

$\int_{t_{peak}}^{t_{0.1peak}} W_c\ .dt$ = area under the cumulative work vs time

P_t = instantaneous power at peak force

C= Unit correction factor set to 0.01 to lower the order of magnitude of the RDCI and for simplicity of plotting

Ligament area = specimen thickness times the ligament length

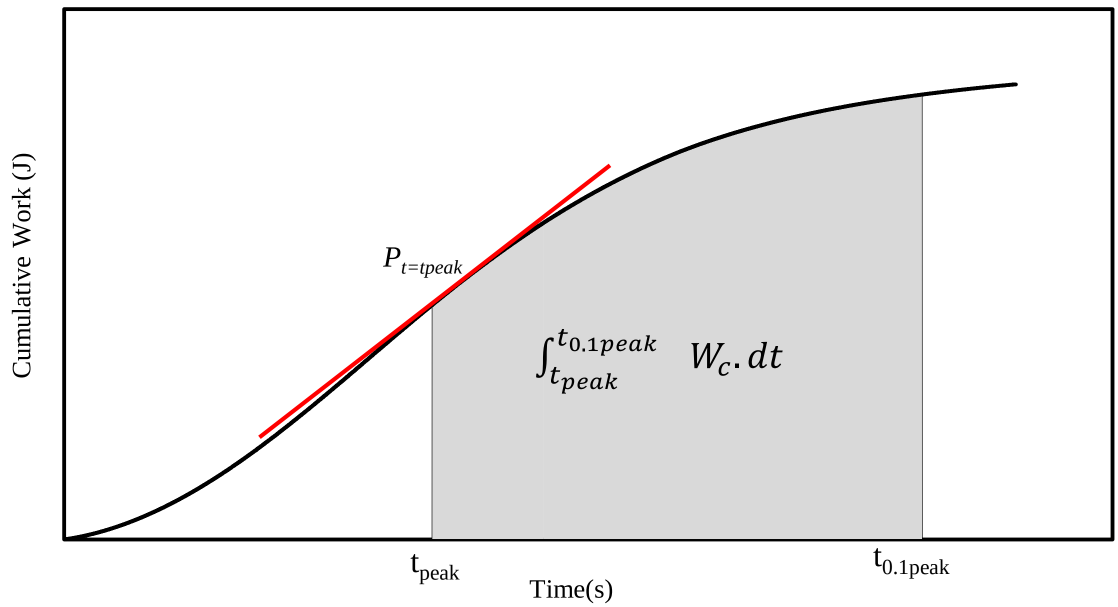

Figure 2 : Determination of cumulative work between time at peak load and 0.1 of peak load.

Comparing the RDCI to the FI, the area under the W_c curve and the $P_{t\,=\,tpeak}$ in RDCI replaced the fracture energy (G_f) and the post peak slope (m) at the inflection point on the force-LLD curve in FI respectively. Although it may appear that the area under the W_c curve is still a single average value, the expansion of the integral results in the product of impulse (J) and displacement as such:

$$\frac{\Delta W}{\Delta t} = \boldsymbol{F} \cdot \frac{\Delta \boldsymbol{x}}{\Delta t} \qquad \text{Equation 5}$$

Multiplying both sides of above equation by $(\Delta t)^2$

$$\boldsymbol{\Delta W} \cdot \boldsymbol{\Delta t} = (\boldsymbol{F} \cdot \Delta t) \cdot \boldsymbol{\Delta x} \text{ ; such that } \Delta \boldsymbol{x} = \boldsymbol{x}_{0.1peak} - \boldsymbol{x}_{peak} \qquad \text{Equation 6}$$

Where:

$\boldsymbol{F} \cdot \Delta t$ = J or impulse (N.s)

$\boldsymbol{x}_{0.1\mathrm{peak}}$ = displacement at 0.1 peak load (post-peak segment)

$\boldsymbol{x}_{\mathrm{peak}}$ = displacement at peak load

Impulse in a fracture test can be interpreted as the capability of the material to tolerate force over a duration of time (similar in concept to momentum, however here in context of the formation of new fractured surfaces within the specimen). A material with higher impulse during the course of a fracture test will have more fracture resistance capacity and typically a more ductile response (due to ability of having greater fracture work potential during crack propagation). On the other hand, the instantaneous power at the peak load as a normalizing parameter indicates the rate at which the total work accumulation occurred until the point of crack initiation. A smaller rate is more desirable as it reveals a shallower transition between pre-peak and post-peak energies, that is, a balance between potential strain energy accumulation, viscous creep dissipation and fracture dissipation. A major advantage of the above described parameters is that they inherently account for the rate dependency of the material.

One may argue that for a constant rate of displacement (in case of I-FIT 50 mm/min test), the instantaneous power will have a similar normalizing effect of peak load in parameters such as CRI. While this holds true for a test with a constant crack velocity, for tests controlled with constant LLD rate, often the crack velocity is not constant (Yang & Braham, 2018). Furthermore, the field distress investigations often indicate a non-uniform crack growth rate (Daniel et al., 2018), thus for these reasons, a rate dependent parameter such as instantaneous power can better describe the fracture processes in viscoelastic materials.

MATERIALS AND METHODS

To evaluate the proposed RDCI parameter in terms of distinguishing cracking resistance of different types of asphalt mixtures, 18 different plant-produced hot mixed asphalt (HMA) mixtures with varied designs, fabrication processes, and aging levels were selected from Minnesota, New Hampshire, and Virginia. This resulted in a total of 28 different mix types and conditions that are assessed herein. The mixtures include two asphalt rubber gap graded (ARGG) and other types of conventional and polymer modified mixtures which were tested using the I-FIT test in accordance with the AASHTO TP 124 test method at a test temperature of 25°C. Minnesota and New Hampshire mixtures were reheated and compacted in lab (referred to as plant-mixed lab compacted or PMLC) while the Virginia mixtures were sampled and compacted in the plant (plant mixed plant compacted, PMPC) without reheating. Using a Superpave gyratory compactor, the specimens were compacted to the typical target in-field construction air void level of 6±0.5% for New Hampshire and 7±0.5% for Minnesota and Virginia. The number of replicates from each source is 4, 24 and 3 for New Hampshire, Minnesota and Virginia respectively.

Table 1 summarizes the mixture design and specimen fabrication methods. The RBR in the table is the percent replacement binder ratio for New Hampshire and Minnesota mixtures, however for the Virginia mixtures this is actually the amount of RAP in the mixture by weight of total mix. However, since the scope of the paper is to compare the indices rather than the mixtures performance with respect to the design properties, this discrepancy may not affect the conclusions. There are five highlighted mixtures in the table which are selected for further evaluation of the cracking index parameter at different aging levels which will be discussed in the following sections of the paper.

Table 1: Mixture characteristic summary

Mixture	Mix Sampling Location	Specimen Fabrication Method	Nominal Maximum Aggregate Size (mm)	Air Void (%)	Gyration	Binder (PG)	Asphalt Content (%)	RBR/ RAP (%)
NH_ARGG-1	New Hampshire	Plant Mixed-Lab Compacted (PMPC)	12.5	4.0	75	58-28	7.8	0.0
NH_ARGG-2							7.6	6.6
NH_64-28						64-28	5.4	18.5
NH_70-34						70-34	5.8	0.0
NH_76-28_1						76-28	5.4	18.5
NH_58-28_1					50	58-28	5.8	16.2
NH_58-34						58-34	5.4	18.5
NH_76-28_2			9.5		75	76-28	6.1	14.8
NH_58-28_2						58-28	5.9	16.9
NH_64-28_1						64-28	6.4	0.0
NH_64-28_2							6.3	18.5
MN_58-34	Minnesota	Plant Mixed-Lab Compacted (PMPC)	9.5	3.0	90	58-34	5.1	15.8
MN_58-28_1				4.0		58-28	5.8	17.2
MN_58-28_2							5.4	16.7
MN_58-28_3				5.0	50		5.8	12.1
VA_76-22	Virginia	Plant Mixed-Plant Compacted (PMPC)	9.5	4.0	75	76-22	5.6	0.0
VA_70-22						70-22	5.2	20.0
VA_64-22						64-22	5.4	40.0

RESULTS AND DISCUSSION

Statistical analysis of Means

The RDCI parameter was calculated and compared to the FI. A graphical comparison between the RDCI and FI is depicted in Figure 3. The error bars on the graph indicate one standard deviation from the mean. Although similar trends may be observed from the graphs, there are differences between the rankings from the two indices such as NH_58-28_2, NH_76-28_1 and MN_58-28_2 that have different orders of ranking from the two indices. A Spearman's rank-order statistical analysis was conducted to determine the significance of difference in the ranking of mixtures using

FI and RDCI. The correlation coefficient was determined to be 0.98, which indicates that there is only negligible difference between the ranks yielded by these two parameters.

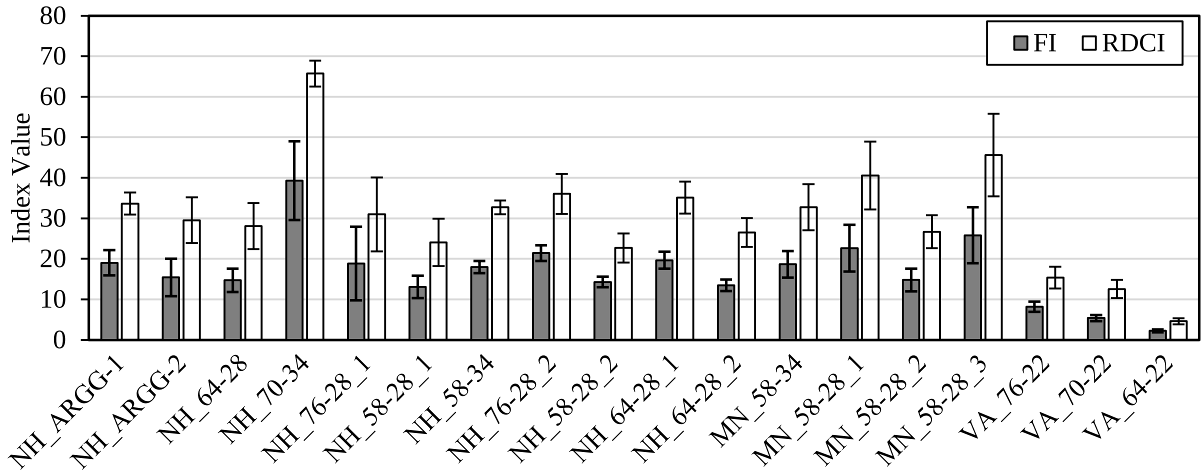

Figure 3. Comparison between RDCI and FI

In comparison of the two indices, the ability to discriminate mixture performance based on magnitude is also important. A student's t-test using 0.05 significance level was conducted to determine the statistical difference between the means for each mixture for each index. The results from this test are presented in Table 2. The mixtures that have the same letter in each column indicate that those mixtures are statistically similar in terms of mean and standard deviation, whereas the mixtures that are not connected by the same letter are significantly different. For example, considering the RDCI, MN_58-28_3 is not grouped with any of the other mixtures, while the analysis indicates that the FI for this mixture is similar to MN_58-28_1 and NH_76-28_2. It should be noted that the MN_58-28_3 mixture has same binder type and comparable aggregate gradation as the MN_58-28_1 mixture, however the volumetric properties and RBR between the two are substantially different, specifically the MN_58-28_3 mixture is designed using Superpave5 concept and has a significantly lower RBR. Also, with respect to RDCI the mixtures are categorized in slightly broader differentiated groups (A to K) as compared to that by FI (A to

H). The t-test results and corresponding grouping demonstrate that the RDCI is either equal or better at discriminating different asphalt mixtures as compared to FI.

Table 2. Results from Each Pair Student's t-test at significance level of 0.05

FI										RDCI												
Mixture	Connecting Letters								Mean	Mixture	Connecting Letters											Mean
NH_70-34	A								39.3	NH_70-34	A											65.7
MN_58-28_3		B							25.8	MN_58-28_3		B										45.6
MN_58-28_1		B	C						22.6	MN_58-28_1			C									40.5
NH_76-28_2		B	C	D					21.4	NH_76-28_2			C	D								36.0
NH_64-28_1			C	D	E	F			19.6	NH_64-28_1			C	D	E							35.1
NH_ARGG-1			C	D	E	F			19.0	NH_ARGG-1			C	D	E	F						33.6
NH_76-28_1			C	D	E	F			18.8	MN_58-34				D	E							32.8
MN_58-34				D		F			18.6	NH_58-34				D	E	F	G					32.7
NH_58-34			C	D	E	F			18.0	NH_76-28_1				D	E	F	G	H				31.0
NH_58-28_2				D	E	F			15.7	NH_ARGG-2				D	E	F	G	H				29.5
NH_ARGG-2				D	E	F			15.4	NH_64-28				D	E	F	G	H				28.1
MN_58-28_2					E				14.8	MN_58-28_2						F	G	H				26.7
NH_64-28					E	F	G		14.7	NH_64-28_2					E	F	G	H				26.5
NH_64-28_2					E		G		13.5	NH_58-28_1							G	H	I			24.0
NH_58-28_1					E		G		13.1	NH_58-28_2								H	I	J		22.4
VA_76-22							G	H	8.2	VA_76-22									I	J	K	15.3
VA_70-22								H	5.4	VA_70-22										J	K	12.6
VA_64-22								H	2.2	VA_64-22											K	4.6

Analysis of Replicate Variations

The coefficient of variation (COV) of the test replicates demonstrates the extent of the variation between the replicates with respect to the mean and a lower COV is more desirable as it indicates higher repeatability of the test results as well as higher reliability in selecting the proper mixture to withstand cracking. Figure 4 indicates the COVs for FI and RDCI. It can be observed that except for a few cases, the COVs associated with RDCI are lower as compared to ones determined for FI. The results for Minnesota mixtures (MN_##-##_# mixtures) are of particular interest. As opposed to other mixtures that have been tested with three or four replicate specimens, these have been tested with 24 replicate specimens. Due to such high number of replication, the COVs of both indices for these mixtures are expected to be resulting from the material scale variability associated with the index itself and not necessarily the variability in testing. For the four Minnesota mixtures, RDCI consistently showed a lower COV than FI.

Figure 5 indicates the relative percent difference in COV between FI and RDCI with respect to COV of FI. The range of relative differences is from 31% higher COV (VA_70-22) to 80.3% lower COV (NH_70-34) for RDCI as compared to FI. An overall average of 10.6% lowering of COV is observed as a combined average of all mixtures and tests.

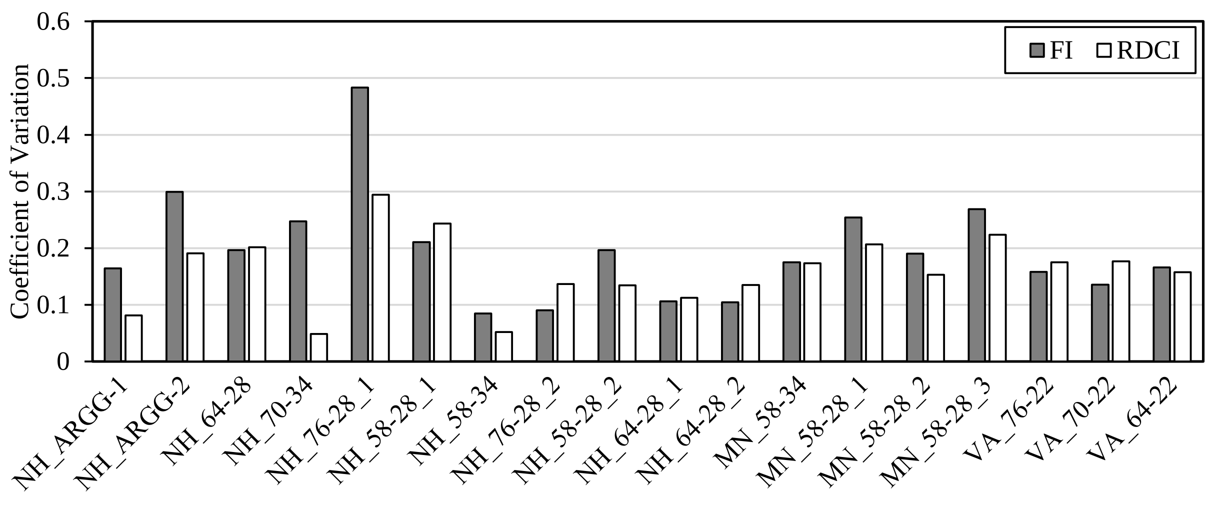

Figure 4. Comparison of Coefficient of variations (COV) determined by cracking index parameters

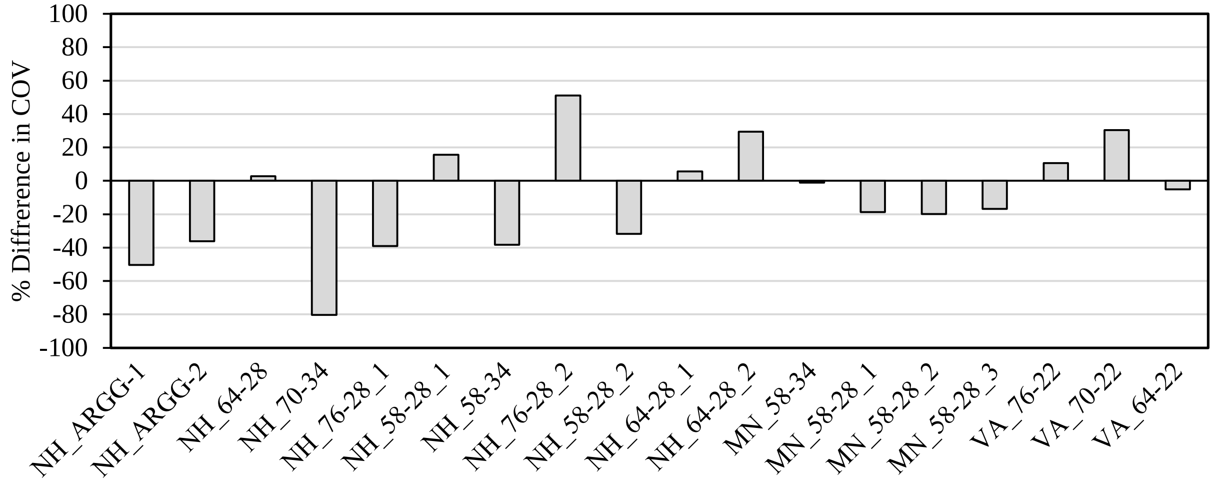

Figure 5. Percent difference in COV for RDCI and FI with respect to COV for FI

Analysis of effect of aging

Mechanical properties of the asphalt mixtures can substantially change during production and in-service due to the volatilization and oxidation of asphalt binder. This typically results in a reduced relaxation capability and increased brittleness and therefore greater susceptibility to cracking. Different laboratory aging methods have been proposed and investigated to simulate the short and long-term aging of the mixtures. However, proper analysis tools should also be available to distinguish the mixture performance at different levels of aging. A set of five plant-produced mixtures (previously indicted in Table 1) were selected to investigate the effect of aging at three different levels as following:

- STA: Short-term aged (aged during production and reheated for compaction)
- LTOA 5D: Long-term oven aged (loose mixture aged for 5 days at 95°C)
- LTOA 12D: Long-term oven aged (loose mixture aged for 12 days at 95°C)

The comparative FI and RDCI results from the analysis of the five mixtures at three aging levels is presented in Figure 6. In general, it can be seen that both indices follow a similar trend with respect to aging levels and the difference between the STA and 5 days at 95°C is relatively pronounced. However, there is not as perceptible a difference between the two LTOA levels. Therefore, each pair student's t-test at a 0.05 significance level (95% confidence interval) was used to determine if the two indices have been able to discriminate between different aging levels. The results from the statistical evaluation of cracking indices with respect to different aging levels is summarized in Table 3. Both indices have been able to easily determine the difference between STA and 5 days at 95°C aging levels for all five mixtures. For the LTOA conditions, the capability of the indices in discriminating the mixtures performance is different. Both indices have been able to differentiate the aging levels of NH_76-28_2, but considering the NH_70-34 mixture, only RDCI with a p-value of 0.0328 has been able to discriminate the two LTOA conditions. Although for the rest of mixtures in the table none of the indices have been able to statistically discriminate the LTOA levels at a reliability level of 95%, the p-values determined for RDCI are lower as compared to FI. Thus for the cases where neither index was able to confidently distinguish 5 and 12 days oven aging, RDCI has a greater probability of discrimination than FI. The results and discussions of test results for the five mixtures at different aging levels indicate that RDCI can distinguish effects of aging on cracking properties of asphalt mixtures as well or better than FI.

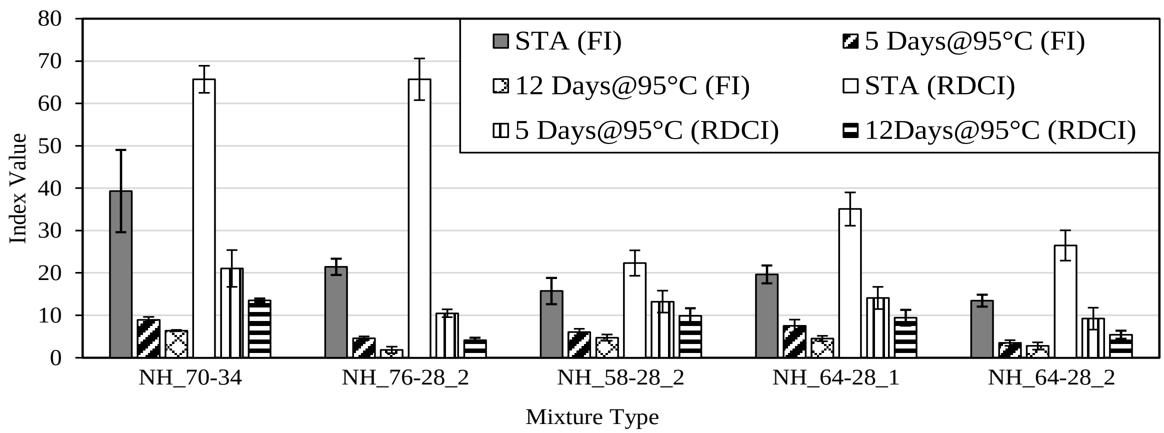

Figure 6. Evaluating the sensitivity of FI and RDCI to aging

Table 3. Statistical evaluation of cracking indices to distinguish the aging levels

Mixture	Compared Aging Levels		p-value (FI)	p-value (RDCI)
NH_70-34	NH_70-34_STA	NH_70-34_12D	0.0012	<0.0001
	NH_70-34_STA	NH_70-34_5D	0.0029	<0.0001
	NH_70-34_5D	NH_70-34_12D	0.6752	0.0328
NH_76-28_2	NH_76-28_2_STA	NH_76-28_2_12D	<0.0001	<0.0001
	NH_76-28_2_STA	NH_76-28_2_5D	<0.0001	<0.0001
	NH_76-28_2_5D	NH_76-28_2_12D	0.0409	0.0485
NH_58-28_2	NH_58-28_2_STA	NH_58-28_2_12D	<.0001	0.0001
	NH_58-28_2_STA	NH_58-28_2_5D	0.0002	0.0014
	NH_58-28_2_5D	NH_58-28_2_12D	0.406	0.1189
NH_64-28_1	NH_64-28_1_STA	NH_64-28_1_12D	<0.0001	<.0001
	NH_64-28_1_STA	NH_64-28_1_5D	<0.0001	<.0001
	NH_64-28_1_5D	NH_64-28_1_12D	0.1225	0.1092
NH_64-28_2	NH_64-28_2_STA	NH_64-28_2_12D	<0.0001	<0.0001
	NH_64-28_2_STA	NH_64-28_2_5D	<0.0001	<0.0001
	NH_64-28_2_5D	NH_64-28_2_12D	0.4545	0.1338

The COVs of the replicates at the two LTOA levels were evaluated and plotted in Figure 7. At the 5-day aging level, the COVs determined through the FI are indicated to be lower for most of the instances, whereas this trend is essentially reversed for the 12-day aging level with lower COVs determined through RDCI. In general, with the assumption of a 20% COV as an acceptable range of variation of results in one standard deviation from the mean (Ozer et al., 2017), there is only one case (NH_64-28_2 at 5D) that RDCI exceeds the threshold while there are three cases (NH_76-28_2 at 12D, NH_64-28_1 at 12D and NH_64-28_2 at 5D) where FI exceeds the threshold.

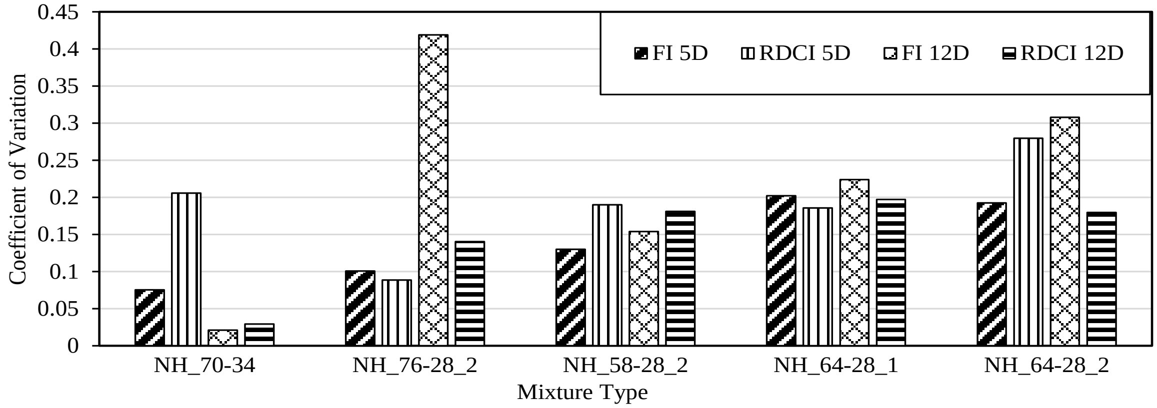

Figure 7. Comparison of coefficient of variations of the index parameters at different long-term oven aged levels

SUMMARY, CONCLUSION AND FUTURE WORK

A number of asphalt laboratory cracking performance tests have been proposed and investigated at different loading and temperature conditions. The semi-circular bending test is one of the simplest geometries that has shown promising results with respect to field cracking. However, the Flexibility Index (FI) parameter associated with this test may not always be able to discriminate the performance between different mixtures, as it is highly dependent on the post-peak slope at the inflection point of the load-displacement curve. This can be a more significant problem for brittle or aged mixtures where the post-peak slope might be too steep resulting in significantly low or even undetermined FI values, which are sometimes followed by relatively high variations in FI values.

Based on the viscoelastic nature of asphalt mixtures, this study proposed a rate-dependent cracking index (RDCI) parameter to analyse the SCB-IFIT results. In order to develop this parameter, the time evolution of cumulative work curve (W_C) is used. Two critical time points on this curve are determined; the time associated with the peak force (t_{peak}) and the time associated with the 10% of the peak force on the post-peak portion ($t_{0.1peak}$). The instantaneous power at t_{peak} is used to normalize the area under W_c curve between t_{peak} and $t_{0.1peak}$. Through use of cumulative fracture work at different times and through use of instantaneous power, the RDCI parameter provides an estimate of the impulse associated with crack propagation in the specimen. Because rate is inherently included in the calculations, the RDCI parameter allows for better characterization for rate dependent fracture processes.

With respect to method of development of RDCI the following general observations can be made:

- The development of RDCI is supported by the combination of physics and fracture mechanics and is free from any type of empirical or undefinable variable within the parameter.
- The use of continuous cumulative work at various times can help with describing and evaluating the crack formation and propagation mechanisms at any given time during the test and due to inherent presence of time in all work and power terms of RDCI; it is expected to better capture the rate dependency of fracture in asphalt mixtures.

In order to evaluate the capability of this parameter to discriminate different asphalt mixtures and aging conditions, a total of 18 mixtures gathered from Minnesota, New Hampshire and Virginia with different mix design and rheological properties were selected and tested by means of SCB-IFIT test at 25°C. The statistical analysis of RDCI and FI for each mixture was used to assess the capability of each index in differentiating the cracking performance of mixtures. Further investigations were conducted on five select mixtures by evaluating the index parameters at three different aging levels. The findings of the analysis presented in this paper are summarized as follows:

- Based on Spearman's rank-order statistical analysis no significant difference was determined in the ranking of mixtures using FI and RDCI.

- The student's t-test analysis indicates RDCI has better discrimination between the mixtures at short and long-term oven aged conditions.

- In this study RDCI resulted in an overall average reduction of 10.6% in the COV.

- Results from statistical comparisons of the aging levels for 5 mixtures indicated that the RDCI is able to differentiate the effect of different aging levels at a reliability level of 86% whereas the FI is able to do so at a reliability level of only 32%.

As opposed to laboratory testing conditions, the crack growth in the field may not follow a constant displacement rate, especially after the crack reaches the critical length. Therefore, a comparative evaluation between FI and RDCI should be conducted to investigate the capability of parameters in discriminating the rate dependency of the fracture using different displacement rates and testing temperatures. Also, the RDCI as a flexible cracking index and independent of the post-peak slope, can help improve the testing protocol repeatability. Moreover, further investigations of this index can be performed in light of other monotonic tests such as disk-shaped compact tension to improve the results and repeatability of such tests.

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REFERENCES

AASHTO TP105.105-13. 2013. *Determining the Fracture Energy of Asphalt Mixtures using the Semicircular Bend Geometry (SCB),*

AASHTO TP107.107-14.(2014). *Standard Method of Test for Determining the Damage Characteristic Curve of Asphalt Mixtures from Direct Tension Cyclic Fatigue Tests,*

AASHTO TP124.124, 2016. *AASHTO TP 124: Standard Method of Test for Determining the Fracture Potential of Asphalt Mixtures using Semicircular Bend Geometry (SCB) at Intermediate Temperature,*

Al-Qadi, I. L., Ozer, H., Lambros, J., El Khatib, A., Singhvi, P., Khan, T., . . . Doll, B. (2015). *Testing Protocols to Ensure Performance of High Asphalt Binder Replacement Mixes using RAP and RAS,*

Anderson. (2005). *Fracture mechanics fundamentals and application*. New York: Taylor and Francis.

Bažant, Z. P., & Kazemi, M. (1990). Determination of fracture energy, process zone length and brittleness number from size effect, with application to rock and conerete. *International Journal of Fracture, 44*(2), 111-131.

Bažant, Z. P., & Li, Y. (1997). Cohesive crack with rate-dependent opening and viscoelasticity: I. mathematical model and scaling. *International Journal of Fracture, 86*(3), 247-265.

Bradley, W., Cantwell, W., & Kausch, H. H. (1997). Viscoelastic creep crack growth: A review of fracture mechanical analyses. *Mechanics of Time-Dependent Materials, 1*(3), 241-268.

Chong, K., & Kuruppu, M. (1984). New specimen for fracture toughness determination for rock and other materials. *International Journal of Fracture, 26*(2), R59-R62.

Chung, W. N., & Williams, J. G. (1991). Determination of J IC for polymers using the single specimen method. *Elastic-plastic fracture test methods: The user's experience (second volume)* () ASTM International.

D’Amico, F., Carbone, G., Foglia, M., & Galietti, U. (2013). Moving cracks in viscoelastic materials: Temperature and energy-release-rate measurements. *Engineering Fracture Mechanics, 98*, 315-325.

Daniel, J. S., Corrigan, M., Jacques, C., Nemati, R., Dave, E. V., & Congalton, A. (2018). Comparison of asphalt mixture specimen fabrication methods and binder tests for cracking evaluation of field mixtures. *Road Materials and Pavement Design, ,* 1-17.

Dave, E. V., & Behnia, B. (2018). Cohesive zone fracture modelling of asphalt pavements with applications to design of high-performance asphalt overlays. *International Journal of Pavement Engineering*, *19*(3), 319-337.

Elseifi, M. A., Mohammad, L. N., Ying, H., & Cooper III, S. (2012). Modeling and evaluation of the cracking resistance of asphalt mixtures using the semi-circular bending test at intermediate temperatures. *Road Materials and Pavement Design, 13*(sup1), 124-139.

Haslett, K. E., Dave, E. V., & Daniel, J. S. (2017). Exploration of temperature and loading rate interdependency for fracture properties of asphalt mixtures.

Im, S., Ban, H., & Kim, Y. (2014). Characterization of mode-I and mode-II fracture properties of fine aggregate matrix using a semicircular specimen geometry. *Construction and Building Materials, 52,* 413-421.

Im, S., Kim, Y., & Ban, H. (2013). Rate-and temperature-dependent fracture characteristics of asphaltic paving mixtures. *Journal of Testing and Evaluation, 41*(2), 257-268.

Kaseer, F., Yin, F., Arámbula-Mercado, E., Martin, A. E., Daniel, J. S., & Salari, S. (2018). Development of an index to evaluate the cracking potential of asphalt mixtures using the semi-circular bending test. *Construction and Building Materials, 167*, 286-298.

Li, X., & Marasteanu, M. (2010). The fracture process zone in asphalt mixture at low temperature. *Engineering Fracture Mechanics, 77*(7), 1175-1190.

Li, X., & Marasteanu, M. O. (2005). Cohesive modeling of fracture in asphalt mixtures at low temperatures. *International Journal of Fracture, 136*(1-4), 285-308.

Li, X., Marasteanu, M. O., Iverson, N., & Labuz, J. F. (2006). Observation of crack propagation in asphalt mixtures with acoustic emission. *Transportation Research Record*, *1970*(1), 171-177.

Majidzadeh, K., Kauffmann, E., & Ramsamooj, D. (1971). Application of fracture mechanics in the analysis of pavement fatigue. Paper presented at the *Association of Asphalt Paving Technologists Proc, , 40*

Mohammad, L. N., Kim, M., & Elseifi, M. (2012). Characterization of asphalt mixture's fracture resistance using the semi-circular bending (SCB) test. Paper presented at the *7th RILEM International Conference on Cracking in Pavements,* 1-10.

Molenaar, A., Scarpas, A., Liu, X., & Erkens, S. (2002). Semi-circular bending test; simple but useful? *Journal of the Association of Asphalt Paving Technologists, 71*

Molenaar, J., & Molenaar, A. (2000). Fracture toughness of asphalt in the semi-circular bend test. Paper presented at the *Proceedings of the Papers Submitted for Review at 2nd Eurasphalt and Eurobitume Congress, Held 20-22 September 2000, Barcelona, Spain. Book 1-Session 1.*

Ozer, H., Al-Qadi, I. L., Lambros, J., El-Khatib, A., Singhvi, P., & Doll, B. (2016). Development of the fracture-based flexibility index for asphalt concrete cracking potential using modified semi-circle bending test parameters. *Construction and Building Materials, 115*, 390-401.

Paris, P. C., & Sih, G. C. (1965). Stress analysis of cracks. *Fracture toughness testing and its applications* () ASTM International.

Saha, G., & Biligiri, K. P. (2016). Fracture properties of asphalt mixtures using semi-circular bending test: A state-of-the-art review and future research. *Construction and Building Materials, 105*, 103-112.

Song, S. H., Paulino, G. H., & Buttlar, W. G. (2006). A bilinear cohesive zone model tailored for fracture of asphalt concrete considering viscoelastic bulk material. *Engineering Fracture Mechanics, 73*(18), 2829-2848.

Wagoner, Buttlar, & Paulino. (2005). Development of a single-edge notched beam test for asphalt concrete mixtures. *Journal of Testing and Evaluation, 33*(6), 452-460.

Wagoner, Buttlar, Paulino, & Blankenship. (2005). Investigation of the fracture resistance of hot-mix asphalt concrete using a disk-shaped compact tension test. *Transportation Research Record: Journal of the Transportation Research Board*, (1929), 183-192.

Yang, S., & Braham, A. (2018). R-curves characterisation analysis for asphalt concrete. *International Journal of Pavement Engineering, 19*(2), 99-108.

Zhou, F., Im, S., Hu, S., Newcomb, D., & Scullion, T. (2017). Selection and preliminary evaluation of laboratory cracking tests for routine asphalt mix designs. *Road Materials and Pavement Design, 18*(sup1), 62-86.

Zhu, Y., Dave, E. V., Rahbar-Rastegar, R., Daniel, J. S., & Zofka, A. (2017). Comprehensive evaluation of low-temperature fracture indices for asphalt mixtures. *Road Materials and Pavement Design*, *18*(sup4), 467-490.