CRITICAL ANALYSIS OF PH AND PG+ ASPHALT BINDER TEST METHODS

By

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A thesis submitted in partial fulfillment
of the requirements for the degree of

Master of Science

(Civil and Environmental Engineering)

At the

UNIVERSITY OF WISCONSIN-MADISON

2015

Acknowledgements

I would like to express my gratitude to my advisor, Professor Hussain Bahia for his support and guidance in completing this Master Thesis research study, and participation from my committee members, Professors Teresa Adams and William Likos. I would like to thank my current and former colleagues for creating an open and enjoyable work environment; specifically Dr. Andrew Hanz for his technical expertise and guidance. Thank you to all Western Cooperative Testing Group members for partnering with the University of Wisconsin and providing all of the materials necessary for completing this study. Lastly, I would like to thank my family and friends for their support and encouragement that made it possible to accomplish and finish my degree.

Abstract

One of the principle products from the Stratigic Highway Research Program (SHRP) was the Superpave asphalt binder performance grading (PG) specification system. This system was based on fundamental engineering principles and has been adopted by almost all local and state agencies. Recently, different types of asphalt binder modification have increased in popularity. All the potential implications of asphalt binder modification cannot be captured by the Superpave grading system. In response to the apparent deficiencies of Superpave, state agencies have implemented PG plus (PG+) test methods. Specification limits for current PG+ tests have largely been based on engineering judgement and little research has been conducted to correlate PG+ test methods to field performance. In addition, new PG+ test methods, not currently included in any state specification, have been developed to characterize an asphalt binder's resistance to each type of asphalt pavement distress in the field. Materials provided by the Western Cooperative Testing group (WCTG) were used in this study to investigate all available PG and PG+ methods. Each test was evaluated for adherence to fundamental engineering principles and correlated with select mixture performance tests. Mixture performance tests can measure the bulk properties of asphalt pavements and those binder test methods that have the highest correlation with mixture testing results merit consideration for state specifications. Results of this study were successfully able to identify PG+ test methods that correlate with mixture testing performance and recommendations were provided for future research and development of a new performance based specification.

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1. Introduction

1.2 Background

In the early 1990s, the Strategic Highway Research Program (SHRP) published several reports explaining each mode of asphalt pavement distress as it relates to a performance based characterization system. In the reports, every aspect of asphalt binder and mixture performance was addressed: mixing/compacting temperature selection, high temperature rutting resistance, intermediate temperature fatigue cracking, low temperature thermal cracking and more [1, 2]. Each SHRP report was dedicated to characterizing the material properties of asphalt and/or one mode of pavement distress. The reports were so comprehensive, that a nation-wide standard specification was produced and labeled the Superior Performing Paving or Superpave system.

After development of the Superpave system, asphalt binder modification with polymers, rubbers and oils have become increasing popular. Modification of asphalt binders has been shown to increase the performance of asphalt binders, but the potential benefits cannot be comprehensively measured or identified by the current Superpave grading system. State and local agencies have responded by including new PG plus (PG+) test methods to capture the presence of polymers. Each state requires different PG+ test methods to identify polymer modification. However, implications of polymer modification on binder rheology may not be captured by current PG+ tests. In addition, a survey conducted by University of Wisconsin researchers indicated that specification limits have largely been selected by engineering judgement rather than asphalt rheology and performance; unlike development of the Superpave system.

New asphalt binder tests such as the MSCR, LAS, BYET, etc… have been developed to directly address mechanisms that cause pavement distress. To date, asphalt producers, state agencies and research institutes have not been able to determine which of these new test methods provide a means to more accurately characterize the performance of asphalt materials. Pavements can take years to show signs of damage and direct comparisons between new

asphalt binder characterization methods with pavement distress in service is not practical. In addition, comparisons between field and laboratory test methods can give misleading results. Performance of pavements is largely dependent on the proficiency of contractors to construct pavements. Are subgrades properly stabilized? Is roller compactor equipment being properly utilized? Are quality control measures organized to monitor the consistency of aggregate and asphalt products? Uncontrollable climatic, in situ soil conditions, traffic loading and differences in construction practice can create "false" correlations between lab and field. Although these factors cannot be included in the lab, new asphalt mixture test methods may provide a more direct comparison between asphalt binder tests with pavement performance. The following Master's thesis organized a comprehensive mixture-binder performance testing framework targeting each type of field distress in the laboratory. Newly developed asphalt binder test methods were compared with mixture tests to validate which methods correlate with pavement performance.

1.3 Objectives

1. Compare and contrast each available asphalt binder test using rheology and engineering mechanics.
2. Validate potential of available asphalt binder tests to characterize pavement performance using laboratory mixture testing methods.
3. Understand the mechanism(s) that influences any binder-binder or binder-mixture performance correlations and propose a future mixture testing framework to create new comprehensive performance based binder characterization system.

1.4 Hypothesis

Asphalt binder testing procedures that have the highest correlation with mixture performance tests will be fundamentally related to the rheological and mechanical properties of asphalt materials. Asphalt binder rheology and mixture performance validation will identify which asphalt binder test methods should be considered for implementation into state and local agency specifications.

1.5 Thesis Outline

I. *Introduction:* This chapter includes a brief background of the current asphalt binder grading systems and problems with implementation of new test methods.

II. *Literature Review:* This section is separated into two sub sections. The first describes the nature of asphalt materials and the current asphalt binder specification system. The second section describes all available tests and discusses justification for implementation in state DOT specifications for each test.

III. *Materials and Methodology*: Research methods were made possible by the Western Cooperative Testing Group (WCTG) and University of Wisconsin-Madison Pooled Fund. This section describes the structure of the WCTG and UW-Madison Pooled Fund research.

IV. *Mixing and Compacting Temperatures*: Investigation of alternative methods for selection of Hot-Mix Asphalt mixing and compaction temperatures. Viscosity measurements were compared with a newly developed modified boiling test to characterize the quality of aggregate-binder coating.

V. *High Temperature Rutting Resistance:* Provides a comparative analysis for all available high temperature rutting resistance asphalt binder tests. Each test was analyzed for correspondence to the mechanisms that cause accumulation of permanent deformation at high in-service temperatures. Each test was then validated with a Flow Number mixture performance testing procedure.

VI. *Intermediate Temperature Fatigue Cracking*: Provides a comparative analysis for all available asphalt binder intermediate temperature cracking tests. Fundamental mechanical properties of asphalt binders were applied to available tests in order to recommend methods that that best characterize fatigue cracking. Fatigue performance of each available binder test was validated using the Indirect Tension Fatigue mixture testing.

VII. *Low Temperature Thermal Cracking:* Provides a comparative analysis for all available low temperature thermal cracking asphalt binder tests. Thermal and fracture properties that cause low temperature cracks were summarized and compared with

the available testing procedures. The Asphalt Thermal Cracking Analyzer was used to measure low temperature material properties of Hot Mix Asphalt.

VIII. *Conclusion and Recommendations*: Summary of all meaningful results and recommendations for state agency implementation/future research.

2. Literature Review

In order to meet the goals of the aforementioned research objectives, a literature review was conducted on the implementation of the Superpave Performance Grading system and all available PG+ testing methods. This section was broken down into two sections: Superpave testing procedures and available PG+ testing procedures. History of testing implementation and any literature citing a correlation with field and or mixture performance will be covered for each asphalt binder test.

2.1 Strategic Highway Research Program (SHRP)

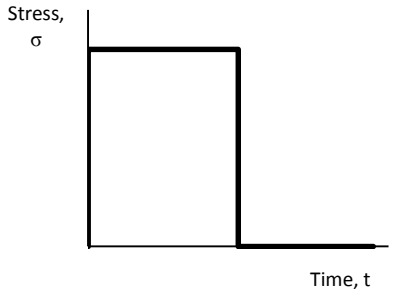
Development of the Superpave asphalt binder grading system was a result of the Strategic Highway Research Program (SHRP) in the late 1980s to early 1990s. Three primary objectives of the study were: to develop mathematical models that explain the rheological behavior of asphalt binders, create performance based asphalt binder specification and validate asphalt binder research with mixture and field measurements. Through field observation and review of literature, asphalt mixture performance was categorized in the SHRP reports as follows: high temperature permanent deformation, intermediate fatigue cracking, low temperature thermal cracking, moisture damage and aging susceptibility. Each category was thoroughly investigated in order to create a purchase performance based specifications, called Superpave, that help local and state agencies ensure high quality asphalt roadway materials at typical in service climactic conditions. Relevant results and discussion were reviewed and summarized in the following sections.

2.1.1 Linear Viscoelasticy and Rheological Measurements

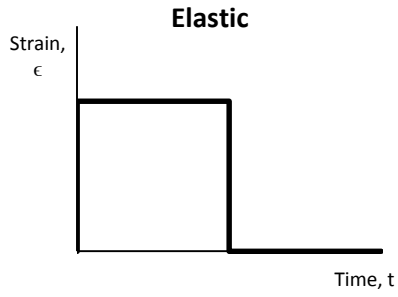
In order to accurately characterize material behavior, a fundamental understanding of asphalt binder's mechanical properties was required. Asphalt binder is regarded as a viscoelastic material. A viscoelastic material's behavior changes relative to the stress, strain, time and temperature when loaded. For short loading times and low temperatures asphalt binders resemble elastic solids and can have a stiffness on the order of 10000 kPa. For long loading times and high temperatures, asphalt binders' are more viscous and can have a stiffness on the order of 1 kPa. When tested over a wide range of temperatures and loading times an

asphalt binder's stiffness can change by over three orders of magnitude. Through review of literature and asphalt binder testing, SHRP researchers identified a finite number of parameters that closely relate to in-service failure mechanisms. In this way, a reduced number of quality control tests could be identified to reduce the extensive amount of time required to fully characterize the behavior of asphalt binders. A description and justification for selecting each parameter was reviewed and summarized in Superpave section.

Simple creep tests provide a way to characterize materials as elastic, viscous or viscoelastic. When a constant load is applied to a material for a finite amount of time an elastic material instantaneously responds to a creep load and maintains the deformation; once the creep load is removed, perfectly elastic materials recover all accumulated strain. Viscous materials do not deform instantly under a creep load and do not recover any deformation after loading. Viscoelastic materials exhibit three stages during a creep load: 1) instantaneous deformation, 2) creep deformation and 3) deformation recovery [1]. A schematic for each type of material behavior is shown in **Figure 1**.



(a)



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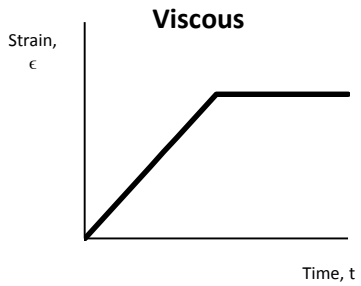

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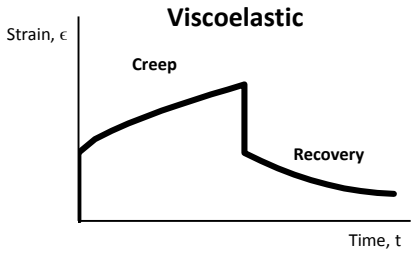

(d)

Figure 1- Typical material response to creep load. (a) is the creep load applied with time. (b) Elastic material strain response to creep load. (c) Viscous material strain response to creep load. (d) Viscoelastic material strain response to creep load.

Creep stiffness modulus or creep compliance are often reported as the viscoelastic material properties from simple creep tests. Creep stiffness is defined as the creep stress divided by the resulting creep strain with time. Creep compliance is defined as the inverse relationship of the creep stiffness. Another material property, called relaxation modulus, can be obtained by conducting a constant strain test with time and measuring a materials stress response. Relaxation modulus is defined as the applied strain divided by the resulting relaxation stress. See mathematical definitions below:

$$\boldsymbol{S(t) = \sigma_0/\varepsilon(t)} \quad \textbf{(1)}$$

$$\boldsymbol{J(t) = \epsilon(t)/\sigma_0} \quad \textbf{(2)}$$

$$\boldsymbol{E(t) = \sigma(t)/\epsilon_0} \quad \textbf{(3)}$$

Where, S(t) is the creep stiffness, J(t) is the creep compliance, E(t) is the relaxation modulus, σ_0 is the creep stress, $\epsilon(t)$ is the creep strain or deformation, $\sigma(t)$ is the relaxation stress and ϵ_0 is the constant relaxation strain.

For very short times of loading, asphalt binder material properties approach a limiting stiffness referred to as the glassy modulus [1]. For long periods of creep loading asphalt binders behave as a Newtonian fluid rather than a viscoelastic solid [1]. In addition to time dependence, asphalt binders are temperature dependent. At low temperatures asphalt binder can be characterized as a glassy viscoelastic material and as the temperature increases asphalt binder decreases in stiffness until it becomes a Newtonian fluid. Change in temperature is analogous to changing loading times in the case of asphalt binders; this specific type of behavior is indicative of a thermorheologically simple material. Over the service life, varying climatic conditions and loading times create a system where measurement of all loading conditions is necessary to fully characterize the behavior of asphalt binders. Due to the amount of time and resources required to conduct a series of simple creep and relaxation tests to fully characterize an asphalt binder, engineers used dynamic mechanical analysis equipment to efficiently measure viscoelastic properties.

Dynamic mechanical analysis or DMA is used to characterize the mechanical properties of viscoelastic materials, like asphalt binder, using an oscillatory applied load. Oscillatory loads, stresses or strains, are commonly applied in the form of a sinusoidal wave function to measure the complex dynamic shear modulus. The mathematical definition of complex dynamic modulus is shown below:

$$\boldsymbol{|G^*(\omega)| = |\tau(\omega)|/|\gamma(\omega)|} \quad \textbf{(4)}$$

Where, $|G^*(\omega)|$ is the complex dynamic shear modulus at frequency ω, Pa, $|\tau(\omega)|$ is the absolute magnitude of the dynamic shear stress response, Pa, and $|\gamma(\omega)|$ is the absolute magnitude of the applied dynamic shear strain, m/m.

Dynamic modulus can also be defined as a complex number consisting of a storage modulus G'(ω) and loss modulus G''(ω) which are related by a phase angle, δ. Dynamic complex modulus is the vector sum of the storage and loss modulus and can be best understood through visualization shown in **Figure 2.**

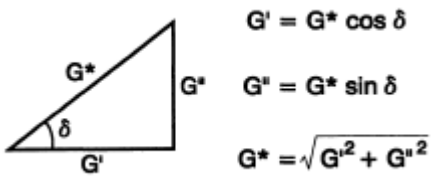

Figure 2- Relationship between complex modulus, storage modulus, loss modulus and phase angle [1].

Storage modulus, G'(ω), represents the in-phase component of the complex modulus and the loss modulus, G''(ω), represents the out-of-phase component of the complex modulus. Phase angle, δ, relates the in-phase and out-of-phase components of the complex modulus by the following definition:

$$\boldsymbol{tan\delta} = \frac{G''(\omega)}{G'(\omega)} \qquad \textbf{(5)}$$

Complex modulus and phase angle can represent both the stiffness and relative in-phase and out- of- phase components of a viscoelastic material. Materials with a lower phase angle are generally considered more elastic and materials with a higher phase angle are more viscous. Depending on the mode of pavement distress, specific values of complex modulus and phase angles are desirable. At high temperatures, a high complex modulus is desirable and at low temperatures a lower complex modulus is desirable. In general, the opposite relationship is desirable for phase angles. To characterize all potential mechanical properties of asphalt binders, complex modulus and phase angle are plotted against loading frequency to create a mastercurve. A mastercurve shows all the potential complex moduli and phase angle measurements for a given loading frequency on a log-log scale. One example of mastercurve was taken from the SHRP report A-369 and is shown in **Figure 3**.

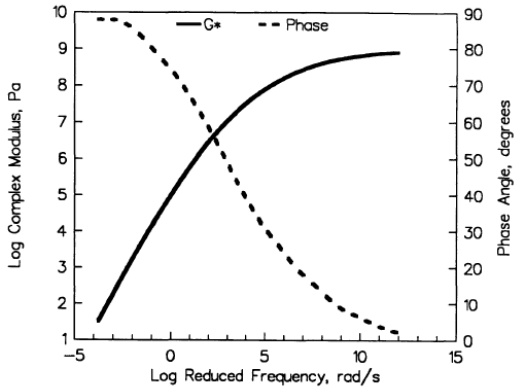

Figure 3- Typical mastercurve for an asphalt binders [1].

As the frequency is reduced, so does the complex modulus; the opposite relationship exists for phase angle and frequency. In order to successfully model the frequency dependence of asphalt binders, all measurements must be conducted in the linear viscoelastic region. A linear viscoelastic response is one that proportionally increases strain in response to stress and vice versa. For high levels of stress and strain, an asphalt binder's response to loading becomes non-linear and requires complex mathematical modeling that was deemed impractical for industrial applications. A comprehensive study was conducted to validate use of linear viscoelasticity to model the behavior of asphalt binders by SHRP researchers. Currently, there are three material properties used to fit G*-loading frequency linear viscoelastic mastercurves: glassy modulus (G_g), cross over frequency (ω_c), and rheological index (R). Glassy modulus is the complex modulus that an asphalt binder approaches, but does not exceed, at low temperatures and high frequencies. Cross over frequency represents the frequency at which the storage modulus and loss modulus are equivalent or the tanδ equals 1; refer to **equation (5).** Physically, the cross over frequency can be interpreted as an asphalt binder's hardness [3]. An asphalt binder with a higher ω_c is indicative of a harder asphalt binder. Rheological index is the difference between the glassy modulus and the complex modulus at the cross over frequency, $G^*(\omega_c)$. The larger the rheological index the more gradual the change from glassy to viscous behavior. Two different mathematical models are commonly used to construct a mastercurve: Cristian-Anderson (CA) and Cristian-Anderson-Marasteanu (CAM) models. CA and CAM models take the following form:

$$G^*(\omega) = G_g(1 + (\omega_c/\omega)^{(log2)/R})^{-R/(log2)} \quad (6)$$

$$\boldsymbol{G^*(\omega) = G_g(1 + (\omega_c/\omega)^{\nu})^{-\frac{w}{\nu}}} \quad \textbf{(7)}$$

Where, G*(ω) is the dynamic complex shear modulus, ω is the frequency of interest and k/w are fitting parameters.

In the CAM model, the parameter w represents how fast the complex modulus approaches the glassy and viscous asymptotes of the mastercurve. Steady-state viscosity, η_o, is another parameter often used to describe the high temperature-low frequency asymptote of the master curve. Steady-state viscosity represents the coefficient of viscosity under linear flow conditions. **Figure 4** provides a visual representation of each parameter.

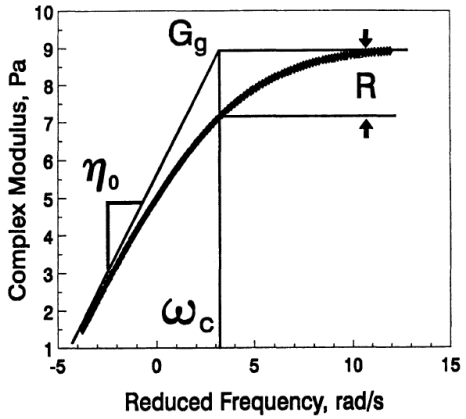

Figure 4- Visual representation of each rheological parameter used to model the dynamic complex modulus frequency dependence [1].

Rheology is defined as the study of materials that flow. Properties of rheological materials are dependent on time and temperature, like asphalt binders. Two rheological measurement devices were identified to measure the viscoelastic material properties of asphalt binders: Dynamic Shear Rheometer (DSR) and the Bending Beam Rheometer (BBR). The DSR provides a means for measuring material properties at high/intermediate temperature ranges and the BBR provides a way to measure the low temperature properties of asphalt binders.

DSR testing procedures were used to conduct DMA analysis necessary to characterize asphalt binders. The word dynamic refers to the manner in which the stresses or strains are applied to a specimen. In the DSR, stress or strains are applied from positive to negative values in a sinusoidal load pattern. Various frequencies can be applied in a dynamic system form 0.1 Hz to 100 Hz. As previously mentioned, these types of measurements are necessary to measure the time dependent nature of asphalt binders. Strains and stresses were applied in the DSR using a steel parallel plate system. Where, a lower steel plate was fixed and an upper steel plate was attached to a spindle driven by an electric motor. To make a rheological measurement, an asphalt binder specimen is placed between parallel plates and bonded to the steel surfaces. As the top plate oscillates, data acquisition software monitors the torque felt by the electric motor and converts this information into material properties. An experimental testing plan was conducted to determine what frequencies, strains, stresses and parallel plate geometry were necessary to obtain accurate material properties [1]. Based on the experimental plan, it was determined that 8 mm plates and should be used when the complex modulus ranges from 1 x 10^5 Pa to 1 x 10^7 Pa; 25 mm plates should be used when the complex modulus ranges form 1 x 10^3 to 1 x 10^5 mm [1]. When the complex modulus is greater than 1 X 10^7 Pa, the bending beam rheometer (BBR) should be used. To determine the appropriate stress and strains to characterize asphalt binders, strain sweeps were conducted. Development of the Superpave binder grading system was based on the asphalt binder properties in the linear viscoelastic region. This region can be determined by gradually increasing the strain applied to an asphalt binder specimen over a range of frequencies. After an asphalt binder reaches the non-linear region of behavior, the complex modulus will begin to decrease. For the purposes of the SHRP study, the strain that indicated the onset of non-linear behavior was defined as the strain corresponding to a decrease in the complex modulus by 5% [1]. SHRP researchers recommended strains ranging from 1 to 12% depending on the stiffness of the binder being tested. Lastly, frequency sweeps were conducted in the linear viscoelastic region to validate that time-temperature superposition mathematical modeling can be applied to asphalt binders. As previously mention, asphalt binders are considered to be a thermo-rheologically simple material. This implies that a shift factor exists for each asphalt binder that can substitute the effects of frequency with the effects of temperature. In this way, asphalt binders can be tested

over a range of frequencies and temperatures to estimate the behavior of asphalt binders for extremely short and long loading times. To do this using the DSR, frequency sweeps were conducted from 0.1 to 100 rad/s at nine temperatures ranging from -25 ˚C to 60 ˚C. Results of the study validated the use of time temperature superposition. An example was taken form SHRP report A-369 and is shown in **Figure 5** [1]**.**

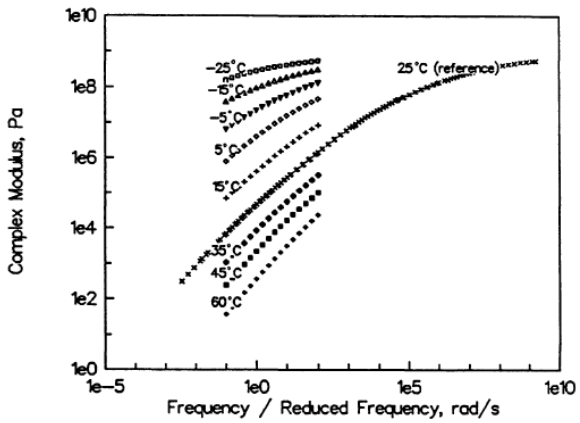

Figure 5- Example of mastercurve calculation using time-temperature superposition taken from SHRP report A-369 [1].

At low service temperatures, the DSR was not capable of measuring stiffness properties of asphalt binders. For this reason, the bending beam rheometer was developed. The BBR is a three point flexural creep testing system. Prismatic asphalt binder samples were created to have a span of 125 mm, a depth of 12.5 mm and a width of 6.25 mm. To conduct a test, each asphalt binder sample is placed in an alcohol bath typically ranging from -5 to -35 ˚C. Specimens are loaded at the center of the span with two equidistant vertical supports on either end. At low temperatures, asphalt binders adhere to the elastic-viscoelastic correspondence principle. The correspondence principle states that the stress distribution in a viscoelastic beam is identical to that of an elastic beam under the same load with time. Therefore, the BBR force-deflection data during a constant creep load can be used to measure the time dependent stiffness modulus of asphalt binders using the following simple beam equation:

$$S(t) = PL^3/4bh^3\delta(t) \quad \textbf{(8)}$$

Where, S(t) is the time-dependent flexural stiffness, P is the constant point load, L is the specimen span, b is the specimen cross-sectional width, h is the cross-section depth and δ(t) is the time dependent deformation.

Several tests were conducted using the BBR and showed a very high repeatability. **Equation 8** was used to define parameters to indicate an asphalt binder's stiffness at service temperatures in the field.

2.1.2 Oxidative Aging

For a given asphalt binder chemistry, linear viscoelastic concepts can be applied to characterize behavior. In service, exposure to oxygen and high temperatures alters the chemistry of asphalt. This phenomenon was referred to as aging. Once asphalt is sufficiently aged, it becomes prone to fatigue or durability cracking. The primary mechanism that causes aging is irreversible oxidation. Oxidation is a chemical process in which oxygen diffuses into and reacts with asphalt, changing its chemistry [1]. Rate of oxidation is asphalt source dependent, but causes all asphalt binders to increase in stiffness and become more brittle. Change in the chemical make-up of asphalt binders with aging prompt SHRP researchers to understand aging's effect on rheology.

SHRP researchers used two laboratory aging simulation methods: thin film oven (TFOT) and pressure aging vessel (PAV). TFOT simulates aging that occurs during construction and placement of asphalt binders; short term aging. This is done by placing thin films of asphalt binder in an oven at elevated temperatures. PAV simulates long term aging of asphalt binders in service. The PAV forces oxygen into a thin layer of asphalt binder at 60 ˚C for 144 hours (for this study). Mastercurves were then calculated and compared in the un-aged, TFOT aged and PAV aged condition. For a given asphalt binder, the Rheological index, R, cross over frequency, ω_c, and temperature shift factors were estimated from mastercurves with increasing aging levels to indicate time, hardness and temperature sensitivity with aging, respectively. **Figure 6** depicts a typical mastercurve change with aging. As the asphalt binder ages, the mastercurve "flattens". For all asphalt binders, the rheological index increases, cross

over frequency decreases and temperature shift factors decrease [1]. This implies that the embrittlement of asphalt binders with aging can be captured by rheological measurements.

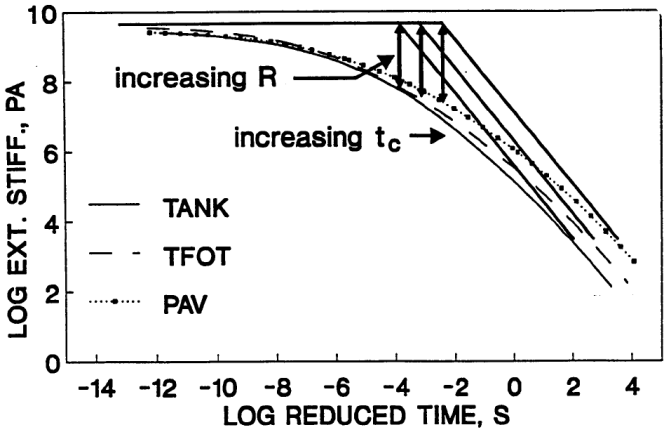

Figure 6- Creep mastercurve for asphalt binder with different levels of aging. Where tank represents the un-aged condition [1].

SHRP researchers successfully validated the use of linear viscoelastic theory to mathematically model the stiffness characteristics of asphalt binders. Extensive testing demonstrated how to determine the linear viscoelastic region, measure the temperature dependence, and aging susceptibility of asphalt binders. At high and intermediate temperatures, the DSR accurately measures the complex modulus, G*, and phase angle, δ, of asphalt binders. Through flexural three-point bending creep testing, the BBR measured the time-dependent flexural stiffness. Each testing system was therefore used to develop the Superpave specifications in use today.

2.1.3 The Superpave Performance Grading Specification

Superpave, short for Superior Performing Pavements, performance grading was a principal specification product of the SHRP research program [2]. The specification system was based on aged asphalt binder stiffness for a specific combination of traffic loading and climactic conditions [2]. Each asphalt binder can be designated based on a high and low environmental temperature. Environmental conditions are delineated by values of 6˚C and are

based on the average 7-day maximum pavement design temperature and minimum pavement design temperature. Performance Grades or PGs were labeled as such:

$$PG\ X - Y \quad (9)$$

Where, PG stands for Performance Graded, X average 7-day high temperature, and Y is the minimum pavement design temperature.

Different potential performance grades were summarized into tables for simplicity. An example of a PG table was take from SHRP report A-410 and is shown below in **Table 1** [2].

Table 1- Performance Graded asphalt binder specification table.

PERFORMANCE GRADE	PG 46-			PG 52-							PG 58-					PG 64-					
	34	40	46	10	16	22	28	34	40	46	16	22	28	34	40	10	16	22	28	34	40
Average 7-day Maximum Pavement Design Temperature, °C[a]	<46			<52							<58					<64					
Minimum Pavement Design Temperature, °C[a]	>-34	>-40	>-46	>-10	>-16	>-22	>-28	>-34	>-40	>-46	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34	>-40
ORIGINAL BINDER																					
Flash Point Temp, T48: Minimum °C	230																				
Viscosity, ASTM D4402:[b] Maximum, 3 Pa•s, Test Temp, °C	135																				
Dynamic Shear, TP5:[c] $G^*/\sin\delta$, Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
ROLLING THIN FILM OVEN (T240) OR THIN FILM OVEN RESIDUE (T179)																					
Mass Loss, Maximum, percent	1.00																				
Dynamic Shear, TP5: $G^*/\sin\delta$, Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
PRESSURE AGING VESSEL RESIDUE (PP1)																					
PAV Aging Temperature, °C[d]	90			90							100					100					
Dynamic Shear, TP5: $G^*\sin\delta$, Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	10	7	4	25	22	19	16	13	10	7	25	22	19	16	13	31	28	25	22	19	16
Physical Hardening[e]	Report																				
Creep Stiffness, TP1:[f] S, Maximum, 300 MPa, *m* - value, Minimum, 0.300 Test Temp @ 60s, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30
Direct Tension, TP3:[f] Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30

Each test in the far left hand column of **Table 1** indicates a required engineering property corresponding to a particular temperature and aging level of interest. At high service temperatures asphalt binders are susceptible to rutting, at intermediate temperatures asphalt binders are susceptible fatigue cracking and at low temperatures asphalt binders are susceptible to thermal cracking. Aging levels are specified for each engineering property to simulate the most critical condition for performance. Unaged asphalt binders represent the binder properties

in the least stiff condition. A minimum G*/sinδ of 1.0 kPa was specified to guard against mixture tenderness. During placement of asphalt mixtures, the binder begins to age due to the high temperatures necessary to mix and compact asphalt pavements. The Rolling Thin Film oven (RTFO) was developed to simulate the aging process during placement of hot mix asphalt. A minimum G*/sinδ of 2.2 kPa was specified after RTFO aging to guard against permanent deformation at high temperatures. During the aging process in service, asphalt binders continue to increase in stiffness. If an asphalt binder becomes too stiff, it may become susceptible to fatigue cracking. The pressure aging vessel (PAV) was developed to simulate long term aging in service; each binder may not exceed a G*sinδ values of 5000 kPa at intermediate temperatures to guard against fatigue cracking. Again, for extended periods aging in the field, asphalt binders may become susceptible to thermal cracking at low temperatures; a minimum BBR creep stiffness and maximum m-value of 300 MPa and 0.300 were specified, respectively. Three different testing devices must be used to measure all of the properties necessary to produce a PG asphalt binder: bending beam rheometer, dynamic shear rheometer and direct tension apparatus.

In addition to the BBR, another low temperature thermal cracking fracture test was developed called the direct tension test (DTT). To conduct this test a dogbone-shaped specimen is elongated at a constant displacement rate of 1.0 mm/min at the minimum pavement temperature plus 10 ˚C until failure [2]. A minimum failure strain of 1% was specified to ensure resistance to thermal cracking. Direct tension was only included into the PG grading system when binders have a BBR stiffness greater than 300 MPa, but have a passing m-value. If this is the case, a binder must have a direct tension failure strain greater than 1%.

Performance grading was based on a combination of fundamental engineering properties such as linear viscoelasticity, time temperature superposition and rheology. This system provided a framework for characterizing the performance of asphalt binders in service and provided a performance related purchase specification for practitioners. However, an increase in use of different asphalt binder modifiers and lack of binder-field correlations have prompted additional testing requirements. Specifically, state agencies are using “PG+” specifications which include tests outside the Superpave system aimed to verify that sufficient

polymer is present in binder. These tests are required because the information used to verify elastic response is not available in the tests used in the current Superpave system.

2.2 PG Plus(+) Binder Procedures

Shortcomings of the Superpave system have been addressed by including PG+ (pronounced PG plus) testing procedures to target specific asphalt binder performance characteristics related to asphalt binder modification. Examples include elasticity, failure toughness, failure energy, fatigue life, ductility and fracture energy at targeted performance temperatures. Several of these tests have already been implemented into current state DOT specifications. The following section will describe how each test procedure is conducted, how it relates to asphalt binder performance and describe any mixture or field performance validation that has been conducted.

2.2.1 PG Plus tests implemented by state DOTs

There are four asphalt binder tests that are currently included in some state DOT specifications: phase angle, elastic recovery, ductility and toughness and tenacity. Phase angle and elastic recovery test methods measure the presence of an elastomeric polymer at high and intermediate temperatures, respectively. Two ductility procedures are available to measure the elongation of asphalt binder at intermediate and low temperatures. This test is used to indicate the presence of a modifier and resistance of asphalt binder to cracking. Toughness and tenacity (T&T) is specified to measure an asphalt binder's resistance to deformation before failure in a similar manner to ductility. The following section will describe each test in detail and cite literature correlating each test to performance.

2.2.1.1 Elastic Recovery and Phase Angle

Both the phase angle and elastic recovery procedures are currently being specified by some state DOTs to indicate the presence of elastomeric modifiers. An increase in asphalt binder elasticity may also correlate with the overall rutting and fatigue cracking resistance. Phase angle is defined as the time lag between the in-phase loading and out-of-phase response of a material loaded in a dynamic system. **Equation 5** provides the mathematical definition of phase angle. The out-of-phase component is often considered the viscous part of the complex

modulus and in the in-phase component is often considered the elastic part of the complex modulus. As the phase angle decreases the asphalt binder elasticity increases. Typically, a maximum value of 70 is specified at the high PG temperatures. D'Angelo and Mehta have showed the phase angle is sensitive to elastomeric modification type, but no definitive correlations to field performance could be made [4, 5].

Elastic recovery test procedure AAHSTO T 301 is one of the most widely used PG+ methods to measure the presence of elastomeric modifiers. The test is conducted by pouring dogbone shaped samples with asphalt binder, extending the asphalt out to 10 or 20 cm, cutting the samples in the center span and then measuring the length each binder sample has recovered. **Figure 7** pictures two different asphalt binders being tested in an ER ductilometer bath at 25 ˚C. Typically states specify minimum %recovery values from 50 to 65%.

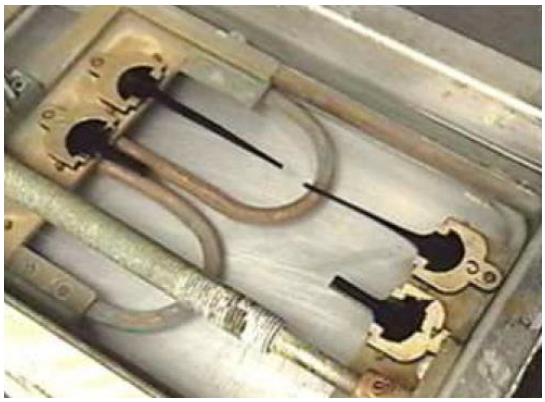

Figure 7-Elastic recovery testing apparatus. Binder on the right was unmodified. Binder on the left was modified with an elastomer.

Although elastic recovery has been shown to indicate the presence of elastomers, it cannot measure the extent to which a binder is modified. Mehta et al. conducted elasticity tests on several different types of modified binders and found that other tests could delineate between asphalt modification types where the ER T301 could not [5].

Phase angle and elastic recovery have been shown to indicate the presence of a polymer, but elastomeric modification does not necessarily correlate directly with actual asphalt binder performance. Clopotel et. al. conducted a study comparing results from elastic

recovery testing with binder fatigue. He found that that there is little to no correlation between elastic recovery and fatigue life [6]. Clopotel and Bahia further investigated this relationship by conducting a similar experimental plan on asphalt binder mixed with mineral filler. Results validated the results found from asphalt binder testing in the previous study [7]. In fact, some correlations suggested that a reduction binder fatigue life corresponded with a higher elastic recovery [8]. There are conflicting publications that discuss the relationship between elasticity and high temperature rutting resistance. Golalipour successfully correlated an increased elastic response of asphalt binders to an improved rutting resistance of mixtures [9]. Conversely, Arshadi modeled the creep response of asphalt binders at high temperatures and found that the viscous component of asphalt binders has the largest influence on mixture performance and the elastic component was relatively minor by comparison [10]. Conflicting results may be related to the type of analysis that was conducted. Golalipour correlated elastic binder testing with mixture testing; while Arshadi modeled the high temperature behavior of asphalt mixtures using computer simulations.

2.2.1.2 Ductility

Ductility testing, ASTM D113 or AASHTO T51, involves a similarly shaped test specimen as in the ER T301 procedure, but instead of cutting the binder specimen, the asphalt is elongated until failure. Typically, the test is run on the un-aged asphalt binder at 25 ˚C and the RTFO aged binder at 4 ˚C. At the completion of the test, the length at which the specimen fails is recorded as the ductility length. Ductility tests can be used to identify certain types of asphalt binder modifications and has been related to field cracking [11]. **Figure 8** shows a comparison between two binders being tested for ductility. Only 6 of the 50 states have implemented ductility testing into their specification. Of the six states, only one has required ductility testing to be conducted on every type of asphalt binder sample. The other five states only specify ductility measurements for polymer modified asphalt binders. Lack of nation-wide implementation may be related to the relatively complex testing configuration. Like the AASHTO ER T 301 procedure, the ductility tests requires a large amount of asphalt material, a water bath longer than 4 feet and the capability to maintain a constant temperature of 4 to 25˚C. Once the test has started, the rate of change in cross sectional area varies by binder type and formulation, as a result the materials are being loaded at different effective strain rates.

Asphalt binder is sensitive to loading rate, so this testing flaw results in comparing the performance of materials under different loading conditions. A more detailed explanation of the strain sensitivity is provided in the BYET section.

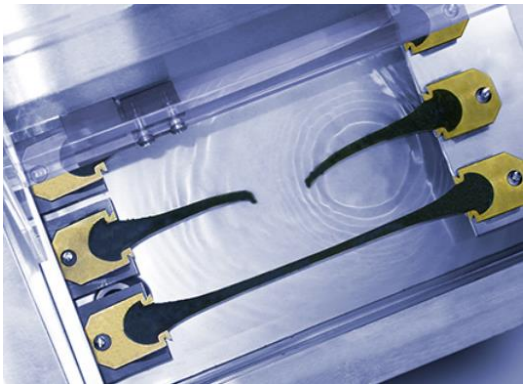

Figure 8- Ductility specimens elongating until failure at 4 C.

Despite the practical concerns and lack of any relationship to fundamental engineering properties, field experiments have been conducted that were able to determine the level of aging of asphalt binders in the field and laboratory by measuring ductility [12, 13, 14]. Kandhal attempted to correlate conventional binder tests with field cracking using Pennsylvania test sections [14]. This study showed that between viscosity, penetration and ductility testing, only ductility was able to give the correct ranking for cracking of roadways after 10 years of in-service conditions. Goodrich studied the fatigue performance of conventional asphalt and modified binders using bending beam fatigue mixture testing [15]. Ductility tests conducted at low temperatures correlated relatively well with beam fatigue at 25˚C. Results also showed that polymer modification did not significantly alter the fatigue mixture performance. This implies that ductility testing may indicate a larger change in performance than what is actually measured in the mixture. Ductility has shown promise as an indicator of mixture performance, but drawbacks related to configuration of the apparatus make it less desirable compared to the simple test methods using the DSR.

2.2.1.3 Toughness and Tenacity

The last test that is currently being specified by some state DOTs is called toughness and tenacity (T&T), ASTM D5801-12 designation. T&T measures an asphalt binder's ability

to withstand a tensile force under a constant deformation rate. Again, this test is often used to characterize the elastomeric properties of asphalt binders. This test was first introduced by Benson to better characterize rubberized asphalt binders [16]. As the test gained more popularity in the 1970s, and began to be used as an indication of polymer modification [17]. The test apparatus consists of a loading probe that is submerged into cup of asphalt binder at 25 ˚C. After submerging the steel probe, the probe is drawn out of the binder and an analog or digital measurement system records the force and displacement until the sample fails. **Figure 9** pictures the T&T steel probe drawing asphalt binder from a cup at 25 ˚C.

Figure 9-Picture of Toughness and Tenacity apparatus pulling asphalt binder form cup at 25 ˚C [18].

Two parameters are derived from the force displacement output: toughness and tenacity. Each parameter can be obtained by calculating different areas under the force and displacement curve. Each calculation can be best described by looking at a typical force deflection curve shown in **Figure 10.**

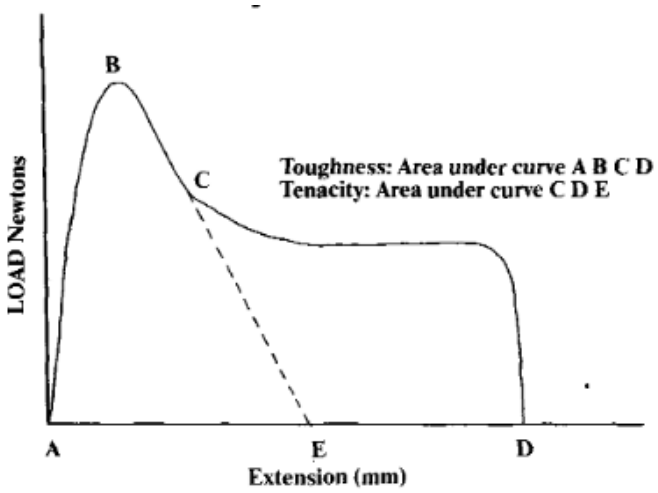

Figure 10-force-deflection curve showing toughness and tenacity area calculations.

Toughness represents the total area under the force deflection curve until failure and tenacity represents the area under the failure portion of the curve. Tenacity is used to measure an asphalt binders elastomeric properties and its ability to delay failure [17]. Goodrich and Isacsson showed limited correlations between mixture performance and field performance, respectively [15, 13]. Primary concerns for the test are related to nature of the testing and repeatability. Asphalt binders experience extremely large strains when tested in the T&T apparatus; these strains may not be indicative of the strains a mixture experiences in the field. In combination with complex steel probe loading system, derivation of any fundamental engineering properties is extremely difficult. Lastly, there can be high variability associated with the test due to subjective data analysis. **Figure 11** shows an ambiguous load-displacement curve for calculating tenacity.

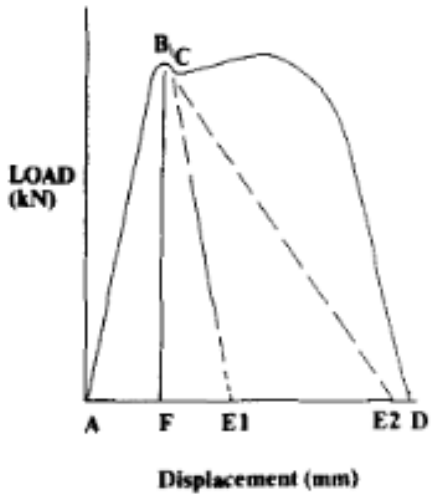

Figure 11- Difficult tenacity force-displacement analysis that may lead to high variability.

Currently implemented PG+ test methods have been shown to indicate changes in asphalt binder chemistry, but there are little correlations to field performance. Advancements in measuring techniques using the DSR and mixture performance testing equipment have provided a more direct way to validate PG and PG+ tests of interest. Newly developed binder test methods that show promise when correlated with mixture performance will be covered in the following section.

2.2.2 Newly Developed PG+ testing procedures

Recently, new PG+ tests have been developed to provide a more comprehensive assessment of asphalt binder performance. Each of these new test methods have been developed to either provide an alternative method to available PG+ tests or more directly target failure mechanisms in the field. Two tests have been proposed as alternatives to the current elastic recovery, ductility and T&T test methods: elastic recovery in the DSR and the binder yield energy test (BYET). Three procedures have been developed to target the failure mechanisms associated with rutting, fatigue cracking and thermal cracking: multiple stress creep and recovery (MSCR), linear amplitude sweep (LAS) and single-edged notch bending (SENB) test, respectively. The following section will describe how each test is conducted, reason for development and any field or mixture performance validation for each new test.

2.2.2.1 ER-DSR and Binder Yield Energy (BYET)

Two tests methods have been developed as alternatives for PG+ tests currently being implemented by state DOTs. As mentioned in the previous sections, one of the major concerns with the current PG+ tests is complex test method configurations. For the elastic recovery and ductility test methods, the "dog-bone" shape and elongation technique used to load asphalt specimens does not address any fundamental engineering properties of the asphalt binder, further explanation is provided below. Similar concerns exist for the T&T's steel probe apparatus. ER-DSR and BYET were developed to be tested in a DSR rather than an alternative testing apparatus. Currently, both ER-DSR and BYET are being considered as a provisional standard by AASHTO.

In order to conduct the elastic recovery procedure in the DSR, 8 mm samples should be poured and placed in the DSR's parallel plate geometry at 25 ˚C. In shear, the 8 mm samples should be loaded at the same shear rate as in the AASHTO T 301 procedure, 2.30% s^{-1} until a strain of 278%. After which the load is removed from the sample for 30 min. Elastic recovery in the DSR should be calculated in the DSR using the following equation and **Figure 12**:

$$\gamma_2/\gamma_1\, x100 \quad \textbf{(10)}$$

Where, γ_2 is the recovered strain at 1800 seconds after removal of load and γ_1 is the peak strain.

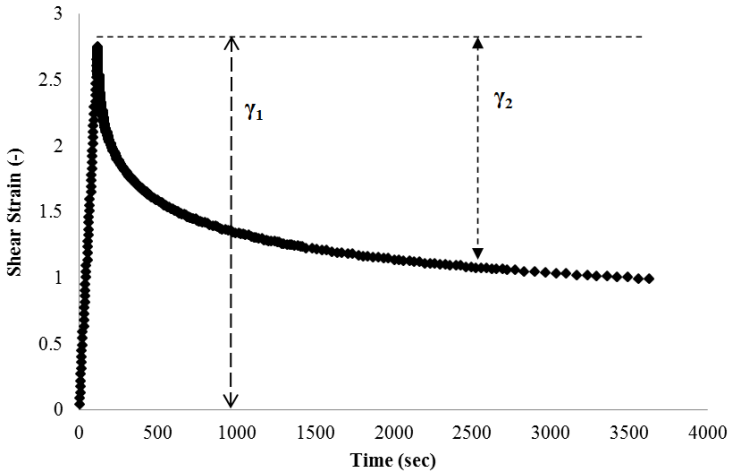

Figure 12-Typical strain curve for elastic recovery test in the DSR [19].

BYET is conducted by preparing an 8 mm sample and placing it in the DSR 8 mm geometry. In shear, the sample is loaded at rate of 2.30% s^{-1} until 4140% strain is reached. Stress and strain is recorded during loading of the specimen. Two parameters are reported for the ductility test: strain at peak stress and yield energy. Strain at peak stress represents an asphalt binder's ductility and yield energy or area under the curve until the sample yields represents an asphalt binder's toughness. **Figure 13** provides a visual representation for how each parameter is calculated.

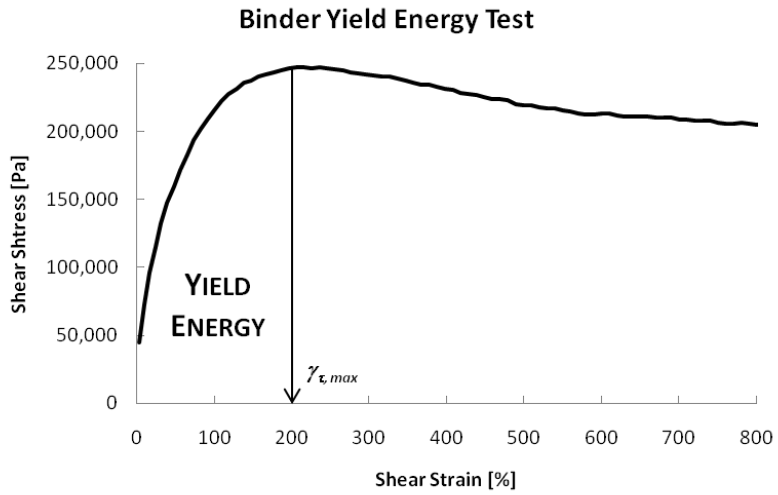

Figure 13-Typical stress-strain curve and parameter calculation for BYET test. Where, $\gamma_{\tau,max}$ represents the strain at peak stress and yield energy is the area under the curve up to the strain at peak stress.

Laboratory studies have been conducted that correlate elastic recovery AASHTO T 301 with elastic recovery in the DSR. Clopotel et al. showed that for two different base binders with five different modification types, there is a linear correlation between the two elastic recovery procedures with an R^2 value of 0.97 [6]. Results from the same study, however, indicated poor correlations between elastic recovery and binder fatigue. Tabatabaee et al. showed a relationship between BYET strain at peak stress and binder ductility based on modifications type [20]. Binders with the same modification type showed a linear correlation between BYET strain at peak stress and ductility. However, no correlation existed for elastomerically modified binders and unmodified binders. Modification type and ductility influence the strain accumulation rate during the ductility test. Different modification types cause different polymer networks to form within an asphalt binder. As a sample elongates, the orientation and length of polymer chains alter the rate of accumulated strain; which is reflected in the change of the ductility specimen's cross sectional area. Because the rate of elongation is held constant throughout the ductility test, a highly modified asphalt binder may experience higher levels of strain for a given elongation. To control the accumulation of strain, the geometry of the specimen being loaded must be held constant. In this way, the rate of strain accumulation remains identical for all modification types. This concept was validated by

Tabatabaee et al using the BYET testing procedure, as shown in **Figure 14**. Elastomeric modification decreased the ductility in some instances. However, the BYET results showed an increase in ductility for all elastomeric modification-binder combinations.

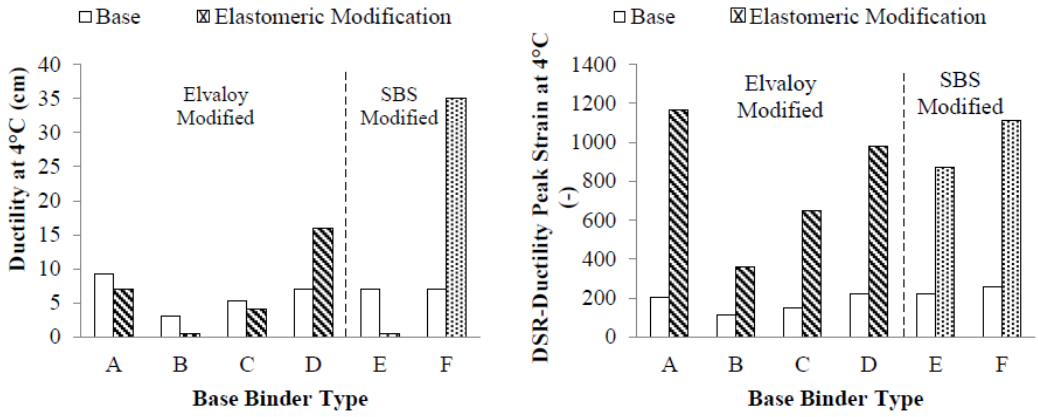

Figure 14-Ductility comparison with BYET strain at peak stress for two elastomeric modification types on six base binders.

No literature has indicated a relationship for BYET yield energy and toughness and tenacity. Conceptually, these parameters are expected to describe similar binder properties because they measure the area under a load versus deflection curve.

2.2.2.2 Multiple Stress Creep and Recovery (MSCR)

The MSCR asphalt binder testing procedure was developed to provide a more accurate indication of rutting resistance. In general, there are three different mechanisms that cause rutting in asphalt pavements: accumulation of permanent deformation in the asphalt surface layer, permanent deformation of subgrade, and wear of pavements caused by studded tires [21]. Obviously, the MSCR procedure can only address the accumulation of permanent deformation in the asphalt surface layer. This type of rutting is related to two different types of deformation: low shear strength and densification. Densification occurs first in asphalt pavements; the pavement volumetrically reduces in size proportional to the reduction in air voids within the pavement structure. All pavements densify with traffic to a certain extent and may not cause deformation that results in noticeable rutting. Weak asphalt surface layers deform in shear with no change in volume and can show high levels of permanent deformation.

At sufficiently high temperatures, pavements can also dilate or increase in volume with traffic. Mixtures that dilate show debonding between asphalt binder and aggregates caused by asphalt binder plastic flow. Garba visually described the different types of rutting as shown in **Figure 15** [21]**.**

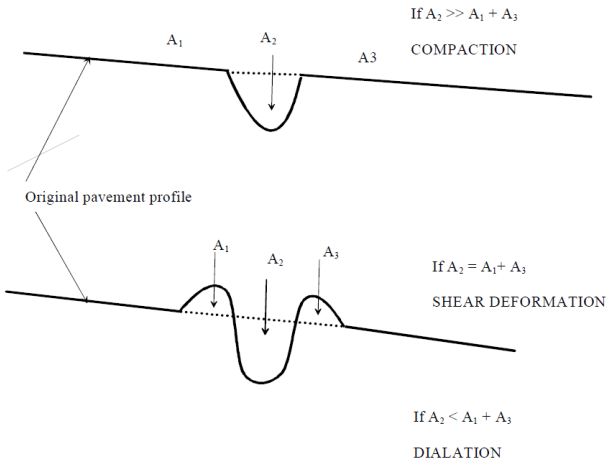

Figure 15-Illistration for different types of rutting in an asphalt surface layer [21].

In the Superpave system, the G*/sinδ is measured at a frequency of 10 rad/s and 12% strain. 10 rad/s approximately corresponds to a traffic speed 62 mph. Accumulation of permanent deformation is primarily caused by heavy truck traffic at low traffic speeds (<45 mph). The MSCR procedure was intended to simulate the critical conditions, high stress at low speeds, causing permanent deformation. To conduct the MSCR test procedures, AASHTO M 332, 25 mm DSR samples are poured and placed in the DSR parallel plate testing apparatus. Ten creep and recovery cycles are applied to the binder specimen at two stress levels: 0.1 kPa and 3.2 kPa. Each creep cycle consists of 1 seconds of loading and 9 seconds of recovery. Two parameters are reported for each test: average non-recoverable creep compliance, Jnr kPa^{-1}, and average percent recovery. Jnr represents the average viscous or unrecovered strain and %recovery represents the average recovered or elastic strain at each creep stress level. The following equations and illustrations best describe calculation of the Jnr and %recovery:

$$\textbf{Jnr} = \sum_{1}^{10} \frac{Un-recovered\ strain}{Applied\ stress} \quad (11)$$

$$\textbf{Average Percent Recovery} = \sum_{1}^{10} \frac{Recovered\ Strain}{Peak\ Strain} \quad (12)$$

Where each parameter is represented in **Figure 16**.

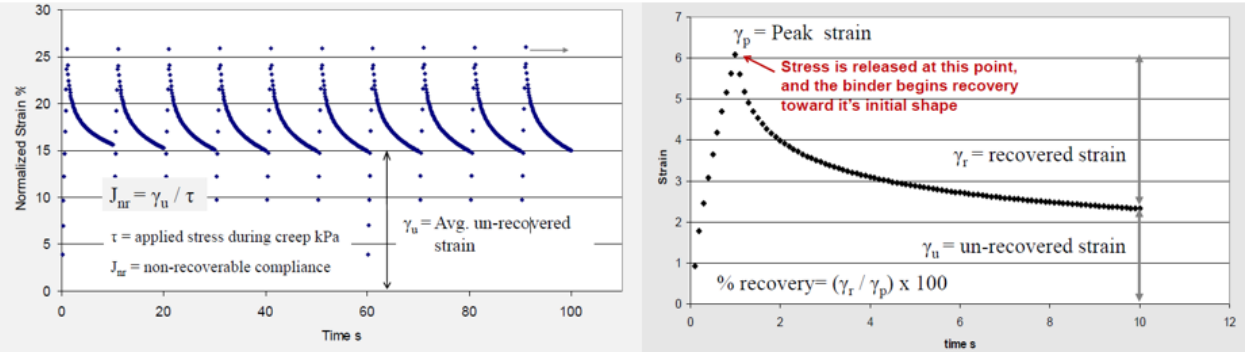

Figure 16- Illustrations to help explain calculation of MSCR Jnr and %recovery parameters.

By measuring the accumulated strain of asphalt binders at relatively high temperatures, this procedure directly measures the same mechanism that causes rutting in the field. A study sponsored by FHWA's accelerated loading facility (ALF) observed a linear correlation between Jnr and rutting with a R^2 value of 0.82 [22]. With the same binders, the G*/sinδ was measured and resulted in a linear correlation R^2 value of 0.13 with rutting. Golalipour verified this correlation using the flow number testing procedure for four different mixtures designs [9].

AASHTO M 332 standard was written to describe the implementation process for the MSCR test procedure into the Superpave grading system. In M 332, Jnr values are specified as a substitute for the grade bumping system. Currently, state agencies specify that the high temperature PG grade should be "bumped" to account for slow traffic and high traffic volumes. **Table 2** shows the required PG grade "bumps" necessary for traffic speed and volume, taken from Hanz [23].

Table 2- Current PG grade "bumping" to account for traffic volume and speed. Where, ESALS are Equivalent Single Axel Loads [23]. Where +1 is one 6 ˚C PG grade bump.

Traffic Volume ESALs	Traffic Speed		
	Standing	Slow	Standard
<0.3	(~ +1)	-	-
0.3 to < 3	+2	+1	-
3 to <10	+2	+1	-
10 to < 30	+2	+1	(~ +1)
> 30	+2	+1	+1

In the MSCR grading system, lower Jnr values are specified at the climatic temperature rather than increase the high temperature PG grade. **Table 3** shows the required Jnr values based on traffic volume and speed. By specifying stricter Jnr values at the climactic temperature, rheological measurements will be done at expect climactic temperatures; rather than G*/sinδ measurements at temperatures not indicative of the climatic of the pavement in service.

Table 3- Jnr traffic specifications in the AASHTO M332 grading system, taken from Hanz [23].

Traffic Rating	Maximum Jnr (kPa^{-1})*	Traffic Guidelines (ESALS and Traffic)
S Standard	4.5	<10 mil ESALs and standard traffic loading
H Heavy	2.0	10-30 mil ESALs or slow moving traffic.
V Very Heavy	1.0	>30 mil ESALs or standing traffic.
E Extreme	0.5	>30 mil ESALs and standing traffic.

2.2.2.3 Linear Amplitude Sweep (LAS)

The linear amplitude sweep test (LAS) is a DSR test that was developed at the University of Wisconsin-Madison by Johnson et al. and modified by Hintz to indicate asphalt binder fatigue [24, 25]. In asphalt pavements, there are two types of fatigue cracking: top-down and bottom-up. Pavements become most susceptible to fatigue damage during the spring months. As snow melts in the spring, the base and subgrade layers underneath the pavement structure become saturated with water. Saturation of aggregate base layers with water leads to a decrease in load bearing capacity. When pavement layers experience high volumes of repetitive traffic loading, with reduced support from the base layers, the pavement experiences higher amounts of

deflection or strain per vehicle. Bottom up cracks are caused by high flexural stresses and first appear at the base of the asphalt pavement layer. For extended periods of traffic, bottom cracks propagate to the surface; often referred to as diamond or alligator cracking. Top down cracking appears longitudinally in the wheel path of motor vehicles. High surface tensile stresses due to wide-based tires with high inflation pressures and age hardening of asphalt binders are regarded as the primary causes for top down cracking, but this topic is still up for debate [26].

Linear amplitude sweep testing has been proposed as an accelerated method to measure asphalt binder fatigue. To conduct a LAS test, an 8 mm sample is placed in the DSR at intermediate temperature ranges. First, a frequency sweep measures the asphalt binder's linear viscoelastic properties. Next, a strain amplitude sweep is run from 1% strain to 30% strain at constant frequency of 10Hz [25], during the test the strain amplitude is increased linearly. After completion of the test, viscoelastic continuum damage mechanics are used to analyze the DSR data; resulting in a fatigue life equation. Derivation of a fatigue life model gives an equation that directly relates the expected traffic load to the expected fatigue life of an asphalt binder or pavement. A typical fatigue law equation is described by the following equations and **Figure 17** [27, 25]:

$$\boldsymbol{N_f = A * (Load\ Amplitude)^B} \quad \textbf{(13)}$$

Where, A and B are derived material properties. A visual representation of the parameters is shown in **Figure 17.**

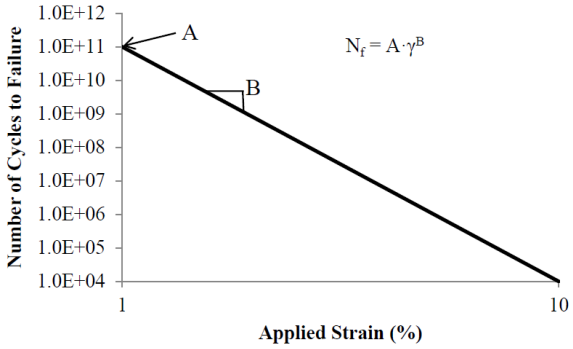

Figure 17- Fatigue law on log-scale taken from Hintz [25].

In order to derive the fatigue law from the LAS test data, the accumulated damage at each strain amplitude must be monitored. In general, the amount of energy dissipated after each strain amplitude in the DSR relates to damage accumulation. Hintz et al used viscoelastic continuum damage theory (VECD) to develop an analysis template that converts material properties measured in the DSR to the fatigue law parameters A and B. Based on the predicted strain experienced by an asphalt binder in service, the resulting fatigue life can be estimated; this is reflected in on the x-axis of **Figure 17**.

An LAS procedure has been drafted and accepted as provisional standard with the designation AASHTO TP 101 and the aforementioned analysis template can be found at the following website: http://uwmarc.wisc.edu/. The LAS test is relatively new, but a collaboration between in the University of Wisconsin and the WisDOT conducted a field evaluation study which correlated LAS parameters to cracked field pavement segments. Results of the study showed that LAS testing has a strong correlation with field cracking relative to the Superpave G*sinδ parameter [28].

2.2.2.4 Single-Edge Notch Bending (SENB)

The Single-Edge Notch Bending (SENB) test has been developed by researchers at the University of Wisconsin-Madison to help provide a more robust low temperature specification system. Low temperature failures, commonly referred to as thermal cracks, appear as transverse cracks relative to the direction of traffic. As asphalt pavements cool down, from day to night or summer to winter, contraction strains lead to thermal stress build-up in asphalt pavement layers. Because the asphalt layer becomes more brittle at the surface and is restrained

to the aggregate layers beneath it, the asphalt layer cracks to relieve stress build-up. Cracks initiate at the surface of the asphalt layer and propagate downward. Asphalt binders that can maintain low stiffness and high relaxation properties at low service temperatures have a higher resistance to thermal cracking; this was the logic behind the BBR Superpave specification. However, linear viscoelastic stiffness and relaxation do not indicate any fracture properties of an asphalt binder. Thermal cracking is directly related to both viscoelastic and fracture mechanics of asphalt binders [29]. The SENB test measures fracture properties of asphalt binders in a simple three point bending apparatus at low PG temperatures.

To conduct a SENB test, samples similar in dimension to the BBR, PAV aged asphalt binder is poured with an induced notch at the center of the span. **Figure 18** provides a schematic of a typical SENB specimen [30].

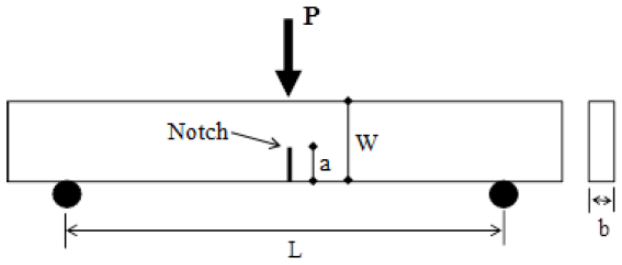

Figure 18- Schematic of SENB testing apparatus. Where P is the point load on the opposite side of the notch [30].

A point load is then applied to the opposite side of the notch at a constant displacement rate in a bath at typical Superpave PG low temperatures (PG LT +10 ˚C). LVDTs and a load cell are used to measure the force and deflection until the sample fails. Three fracture parameters can be calculated from the force displacement data: fracture toughness, K_{IC}, fracture energy, G_f, and displacement at the maximum load. Each parameter can be calculated using the following equations:

$$K_{IC} = \frac{PL}{bW^{3/2}} * f(\frac{a}{W}) \quad \textbf{(14)}$$

$$G_f = \frac{W_f}{A_{lig}} \quad \textbf{(15)}$$

Where, K_{IC} is the fracture toughness, G_f is the fracture energy, W_f is the work or area under the SENB force displacement curve, A_{lig} is the area of the ligament and all others are geometric constants shown in **Figure 18.**

Marasteanu et al. conducted a study that attempted to correlate BBR, m-value, SENB parameters and low temperature mixture tests [30]. Results showed that there is not clear correlation between SENB and BBR. Both SENB and BBR stiffness showed a relatively high correlation with the glass transition temperature of asphalt mixtures. The study validated the idea that both viscoelastic stiffness and fracture properties are necessary to characterize thermal cracking resistance of asphalt binders.

3. Materials and Methodology

3.1 Materials

In order to meet the aforementioned objectives, data was accumulated and organized from the Western Cooperative Testing Group (WCTG). Members of the WCTG receive asphalt binder specimens once a month from different asphalt binder procedures in the western United States. Each member conducts all available testing procedures and reports the results to an online database. The Modified Asphalt Research Center (MARC) partnered with WCTG to provide an independent source to analyze the inter-lab variability for each WCTG member. As the WCTG has progressed, additional test methods and Hot-Mix asphalt material that corresponds with each binder have been provided to the University of Wisconsin. In addition to ensuring lab competence, the WCTG provides the necessary materials to conduct an experimental work plan designed to identify asphalt binder tests that:

1) Adhere to fundamental engineering properties for a wide range of binder chemistry and modification types.
2) Are redundant measurements of the same asphalt binder properties.

3) Can be readily implemented into current state agency specifications
4) Correlate with mixture performance test methods.

A seconded research project at the University of Wisconsin, called the Pooled Fund, has similar objectives to the WCTG. The wide range of sources in WCTG and Pooled provided the asphalt binders and mixtures required to conduct the following Master's Thesis research plan. **Figure 19** is a map showing all of the states involved in the WCTG and Pooled Fund. WCTG states are highlighted in blue, pooled fund states are highlighted in green and yellow states are involved in both research projects to date.

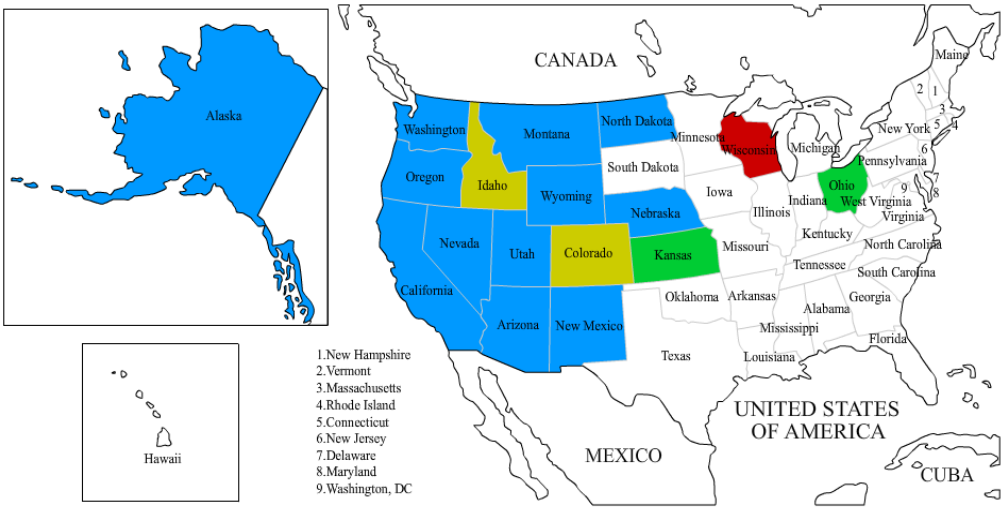

Figure 19-Map of states involved in the WCTG and pooled fund. States in blue are WCTG states, states in green are pooled fund states and states in yellow are involved in both.

3.2 Methodology

Materials provided by WCTG producers were tested at mixture and binder levels to target each mode of distress in the field. **Table 4** summarizes each test method by the temperature range and targeted in-service distress. Within each temperature range, select asphalt binder tests were used to determine those tests that adhere to fundamental engineering principles. Once established, selected binder tests was used in mixture performance tests that

target in-service distress modes. Binder tests that demonstrate the highest correlation to mixture performance tests were recommended for state DOT implementation. A description of each binder test method in **Table 4** are provided in the literature review section. Descriptions of each mixture performance test are explained in each respective results and analysis section.

Table 4-Experimental testing plan for each target temperature range and distress.

Temperature Range	Mixture Performance Test	Binder Testing Method: Parameter	Targeted In Service Parameter/Distress
Mixing (>100 ˚C)	**Modified Boiling Test:** Quality of coating index	**-Viscosity:** 1 and 20 RPM in Pa*s	**Coating**-aggregate binder adhesive bond
High Temperautres (46 to 82 ˚C)	**Flow Number:** Number of loading cycles to tertiary flow	-**MSCR**: Jnr & % Recovery	**Rutting** Potential
		-**G*/sinδ:** high temperature PG	
		-**Phase Angle:** at high PG temperatures	
Intermediate Temperatures (4 to 40 ˚C)	**IDT Fatigue:** Number of cycles required to fail disc shaped specimen at 25°C	-**LAS:** Number of cycles to failure for in-service asphalt binder strain	**Fatigue** resistance
		-**G*sinδ**: intermediate temperature PG	
		-**Elastic Recovery**: AASHTO T 301 and DSR procedure	
		-**Ductility at 4°C:** strain at rupture	
Low temperatures (-40 to 0 ˚C)	**ATCA Testing:** Glass transition temperature, coefficients of thermal contraction, fracture stress, and fracture temperature	-**BBR: PG** Stiffness and m-value for 1 and 24 Hours conditioning, physical hardening index	**Thermal Cracking** resistance
		-**SENB:** Energy required to fracture beam with induced notch at low temperature PG +10 ˚C	

4. Mixing and Compaction Temperatures

Cuciniello defined coating performance in two categories: extent of coating and quality of coating [31]. Extent of coating indicates the degree at which asphalt binder coats the aggregate's surface after mixing. Quality of coating is an indication of the bond strength, moisture resistance and overall performance of the asphalt binder-aggregate interaction after mixing. Viscosity may be used to specify the extent of coating, but not the quality of coating. A lower viscosity binder will coat aggregates more thoroughly, but the bond created after coating cannot be captured by solely measuring the asphalt binder viscosity. Different asphalt binders require different amounts of thermal energy to achieve the same bond strength. Cuciniello modified the ASTM D3625 boiling test that describes a procedure to measure quality of coating by boiling aggregates coated with asphalt binder. First, an aggregate source is washed and sieved over the 3/8" sieve to capture the coarse aggregates. Next, the coarse aggregates are mixed with asphalt binder until 100% extent of coating is observed; in this case an asphalt binder content of 3% was used for all tests. The mixture is put in boiling water for 20 minutes and then visually inspected for coating. Each aggregate is observed and categorized based on the percent of asphalt that was removed after boiling. The end result of the modified boiling test is a coating index (C.I.); a higher C.I. indicates that less asphalt binder debonded after boiling. In the Superpave system, a maximum viscosity of 3 Pa*s at 135˚C is specified to ensure the appropriate extent of coating and workability for Hot Mix Asphalt. To determine if adherence to the specification results in a higher coating index, 8 different binders were used to conduct the modified boiling test. **Figure 20** shows the results for both viscosity and coating index measurements taken at a temperature of 135 ˚C. Viscosity measurements were made at a shear rate of 1 rotations per minute (RPM). 1 RPM was recommended by Cuciniello to represent the shear forces experienced by asphalt binder during the aggregate-asphalt binder mixture process [31].

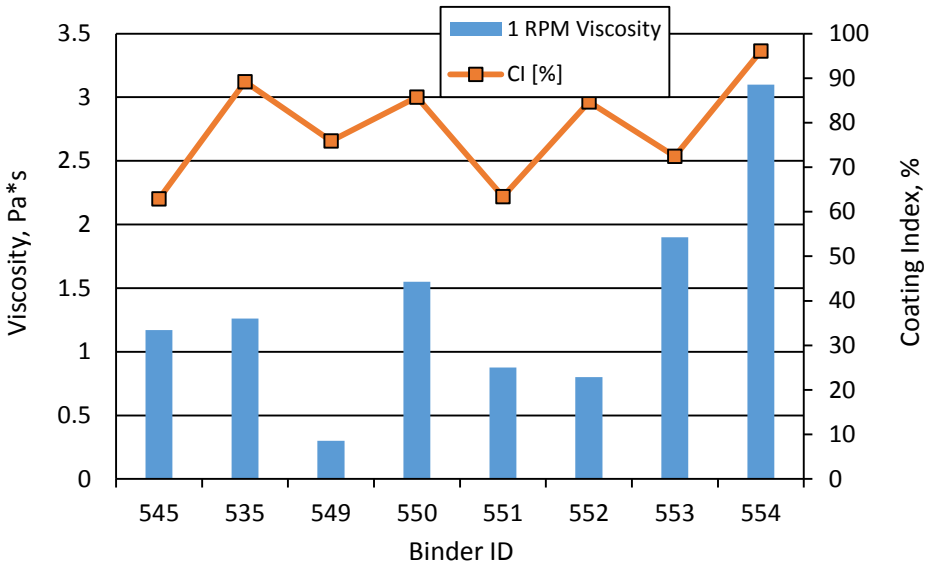

Figure 20- Coating index-viscosity measurements for 8 different binders at 135 ˚C.

Superpave specifications did not indicate the relative quality of coating for the asphalt binders tested. In fact, the asphalt binder that exceeded the Superpave specification limit gave the highest coating index. To further investigate the relationship between viscosity and coating index, two binders with similar viscosities at 135 ˚C were tested for viscosity and coating index at 115 and 155˚C. **Figure 21** shows the relationship between viscosity and coating index for two different binders.

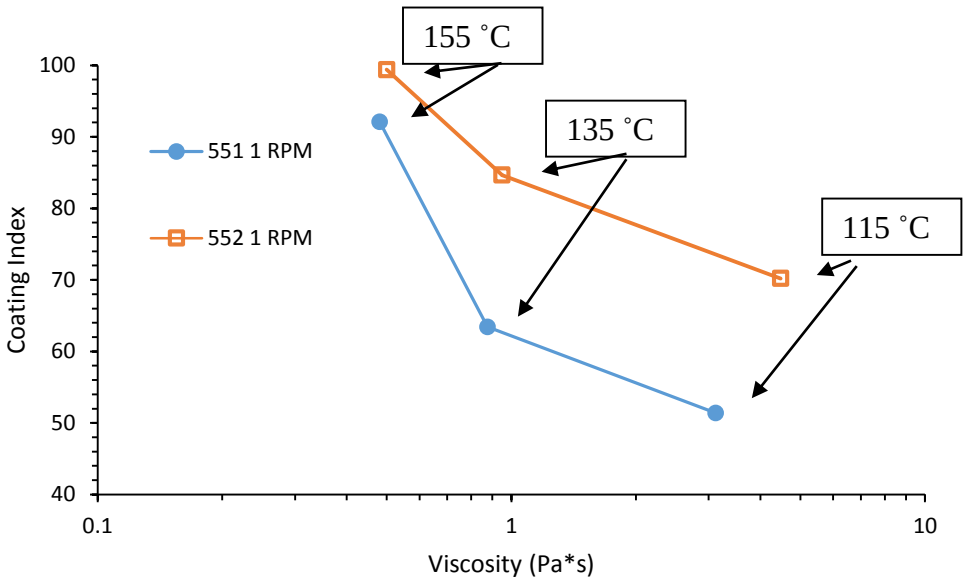

Figure 21-Coating index versus viscosity for two binders, 551 and 552.

As the viscosity decreases the coating index increases for each binder, but each binder had an independent relationship between coating index and viscosity. Changes in asphalt chemistry require different viscosities to obtain the same coating index. Instead of specifying a minimum binder viscosity, specify a minimum coating index. The quality of coating is an interactive property related to both the binder and aggregates selected for a given mixture design. After establishing a mixing temperature for production, an HMA supplier can run the modified boiling test to determine if they are above the specified coating index.

A compaction temperature can be selected after measuring the coating index. For unmodified binders, the equiviscous method can be used to determine the compaction temperature. For modified asphalt binders, the equiviscous method often leads to excessively high temperatures and will not be practical. Other common methods for determining compaction temperatures include: high shear rate, steady shear flow, zero shear viscosity, mixture workability and compaction test methods. A literature review, as part of the NCHRP report 648, indicated that there is no clear relationship between compaction temperature and performance of hot mix asphalt [32]. Manufacturer instructions and field experience are the best recommendations for selecting compaction temperature until a more direct relationship between compaction temperatures and mixture performance can be identified. In most states, in-place asphalt pavement is accepted based on density, coating tests and manufacturer

guidelines can be used to define mixing/compaction temperature minimums, but practical considerations including weather, haul distance, and other factors should be considered in final selection of mixing and compaction temperatures. The mix needs to be produced at a temperature range that will allow for delivery of workable mix at the jobsite so that in-place density requirements can be met.

5. Evaluation of High Temperature Rutting Resistance Test Procedures

G*/sinδ is the current Superpave PG parameter used for indicating rutting resistance. G* represents the stiffness and the phase angle, δ, represents the elasticity of an asphalt binder. To increase the rutting resistance of an asphalt binder G*/sinδ should be maximized. More recently, the MSCR test method has gained popularity as a potential replacement or addition to the G*/sinδ specification, these changes are detailed in the AASHTO M332 standard. After PG grading an asphalt binder, M332 specifies the MSCR Jnr based on the expected level of traffic. High traffic level Jnr values must be less than 0.5 kPa^{-1} and standard levels of traffic Jnr values must be less than 4.0 kPa^{-1}; intermediate levels of traffic are accounted for in M332 as well [22]. A critical analysis of how PG and MSCR relate to one another and mixture performance testing were analyzed in the following sections.

5.1 Comparison of G*/sinδ and MSCR parameters

PG and MSCR parameters were plotted against one another to identify any correlations. **Figure 22** shows a plot comparing G*/sinδ and Jnr at three stress levels.

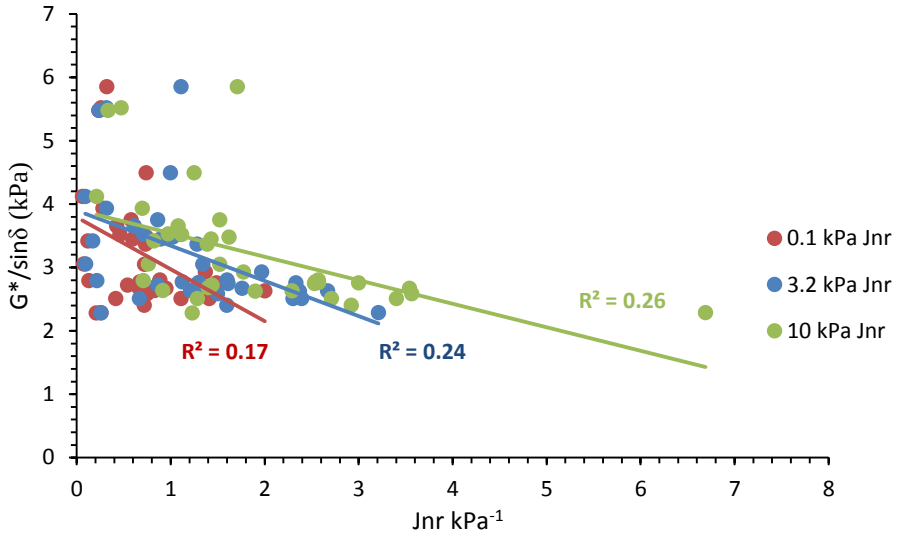

Figure 22- Correlation between G*/sinδ at three different stress levels. All tests were conducted at the high temperature PG and RTFO binder.

Logical trends were observed for each stress level, as the Jnr increases the G*/sinδ decreases. However, the correlation between both parameters was low at every stress level. To better understand the low correlations, the PG parameters (G* and phase angle) were plotted against Jnr independently. Phase angle had the highest correlation at low stress levels, shown in **Figure 23**. G* shows the same trend as the G*/sinδ with an even lower R^2 values.

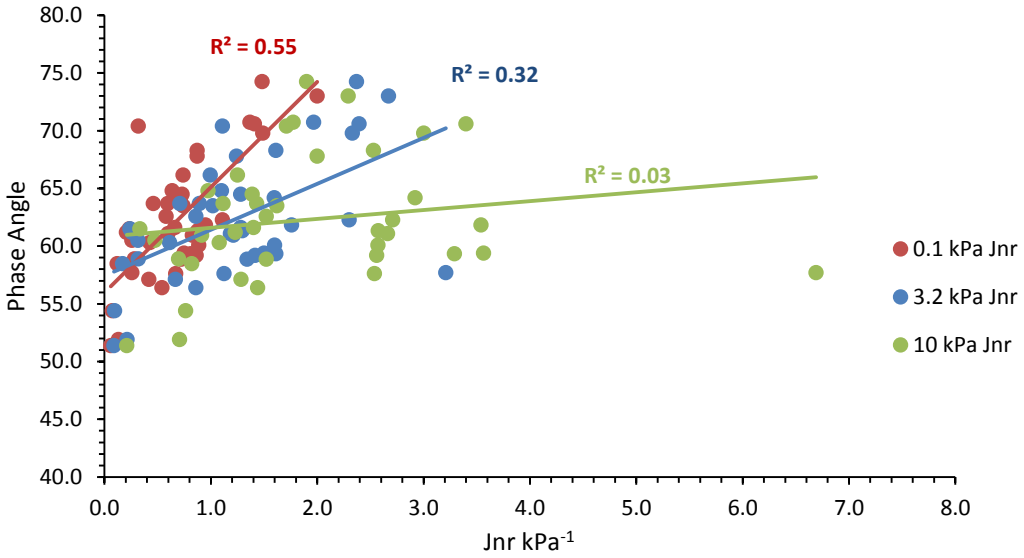

Figure 23-Correlation between phase angle and Jnr at three MSCR Stress levels.

PG procedures require testing the asphalt binder at 10% strain, most binders experience over 20% strain during the 0.1 kPa testing interval of the MSCR. After a certain amount of accumulated strain, asphalt no longer exhibits linear viscoelastic behavior and thus the correlation to material properties decreases.

Poor correlations between G* and MSCR Jnr may be best explained by understanding the definitions of G* and the non-recoverable creep compliance (Jnr). G* is defined as a complex number consisting of a storage modulus (real part) and loss modulus (imaginary part). The storage modulus represents an asphalt binder's ability to store strain energy; in other words, the asphalt binder's elastic component. Loss modulus represents the strain energy dissipated after one loading cycle in the DSR; the asphalt binder's viscous component. After data analysis of the MSCR testing results, two parameters are obtained: non-recoverable creep compliance and the percent recovery. Non-recoverable creep compliance is another measure for the viscous component of an asphalt binder; % recovery is a measure of an asphalt binder's elastic component. Separation of elastic and viscous components in the MSCR test and inherent differences between dynamic and creep testing may explain the poor correlations between G* and Jnr.

5.2 Correlation of MSCR and PG to Repeated Creep mixture testing

A repeated creep mixture testing procedure was used to correlate G*/sinδ and Jnr to mixture rutting resistance. AASHTO TP 79-09 standard was followed to test mixture samples at 46 ˚C in an environmental chamber to simulate high temperature climactic conditions. Samples were then loaded in creep cycles until failure. One creep cycle consisted of a 150 psi load being applied for 0.1 s and then unloaded for 0.9 s. The flow number is the cycle at which the sample changes from secondary to tertiary failure, shown in **Figure 24**. The Francken model was used to calculate the flow number (FN) from the testing data:

$$\varepsilon_p = An^B + C[e^{Dn} - 1] \quad \textbf{(16)}$$

$$\frac{d^2\varepsilon_p}{dn^2} = AB(B-1)^{B-2} + CD^2e^{Dn} \quad \textbf{(17)}$$

Where, ε_p= the accumulated permanent strain, $\frac{d^2\varepsilon_p}{dn^2}$= the second derivative of the accumulated strain, and n is equal to the cycle. A, B, C, and D are fitting parameters used to model the experimental data

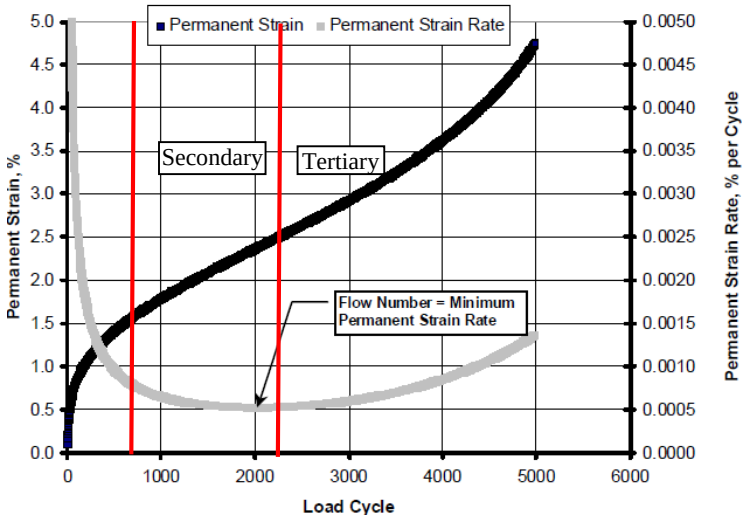

Figure 24- Graph illustrating cycle that represents the flow number.

Jnr and G*/sinδ were measured at the PG temperature and correlated with the FN data. **Figure 25** and **Figure 26** are correlations for Jnr and G*/sinδ with FN, respectively.

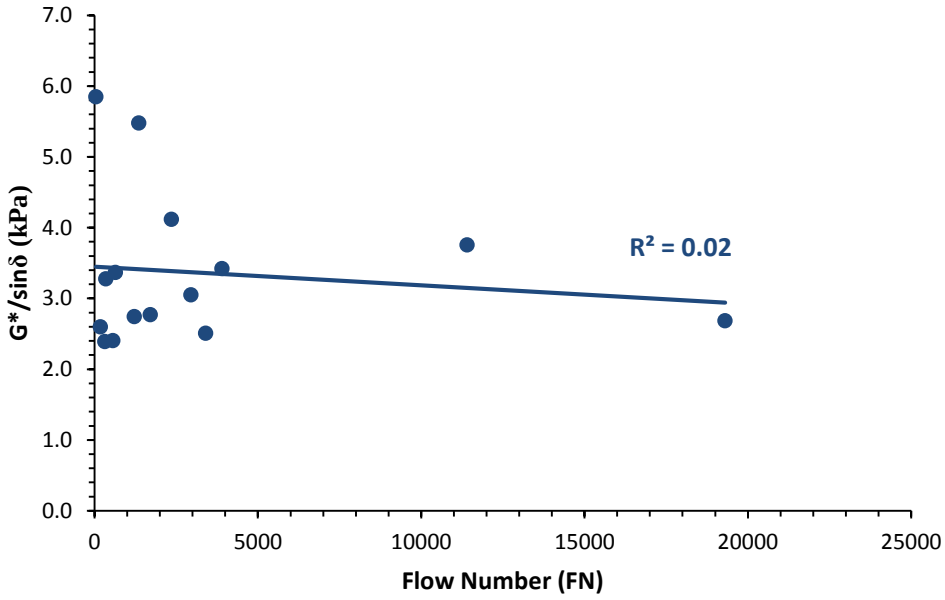

Figure 25- Correlation between G*/sinδ measure at the PG temperature and FN.

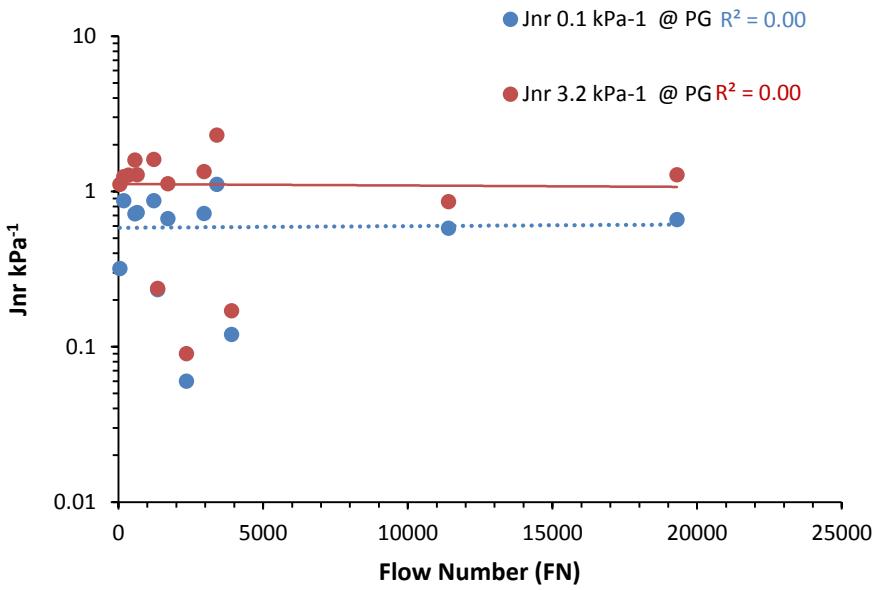

Figure 26- Correlation between Jnr at PG temperatures and Flow Number.

Trends for each correlation were not logical and no linear correlations between FN could be made. This result was expected because PG temperatures range from 52 to 76 ˚C compared

with the constant 46 ˚C repeated creep test temperature. Jnr and G*/sinδ measurements were taken at 46 ˚C to remove the effect of temperature. **Figure 27** and **Figure 28** are correlations for G*/sinδ and Jnr measured at 46 ˚C with FN, respectively.

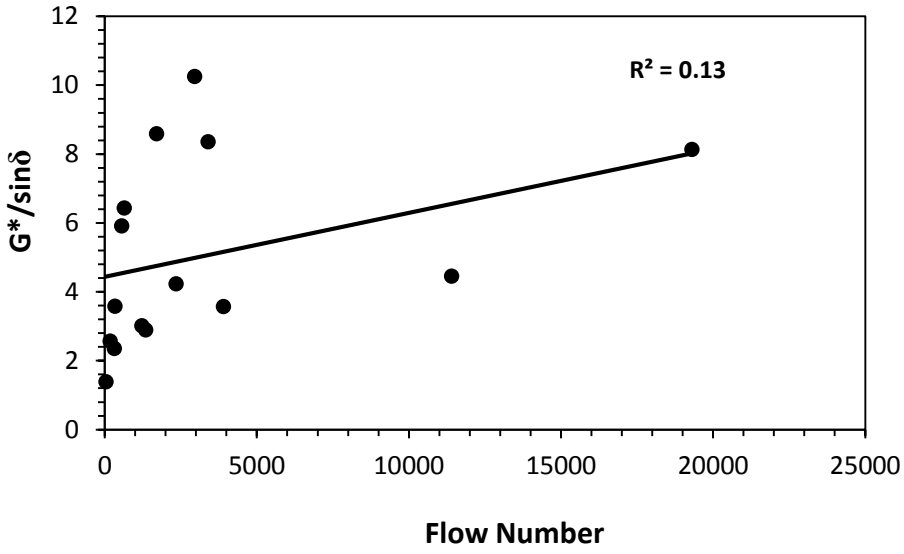

Figure 27- Correlation of G*/sinδ and flow number measured at 46 ˚C.

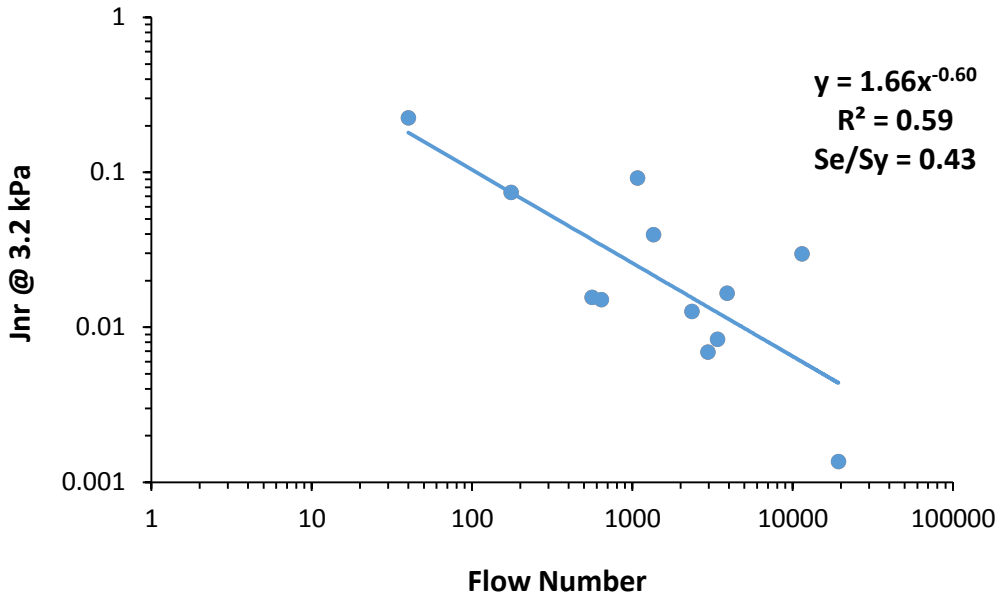

Figure 28-Correlation between Jnr an FN at 46 ˚C.

Logical correlations could be observed in both cases. Jnr showed a highest correlation with FN at a 3.2 kPa stress level. A power law model was fit to the Jnr-FN correlations; goodness of fit value, Se/Sy, of 0.43 indicates a fair model accuracy. Given that the Jnr measurements cannot account for the aggregate properties of a mixture design, the power law model was deemed acceptable. Although stronger correlations were observed, the Jnr and G*/sinδ values were out of the typical range used in standard specifications. Performance grading requires that asphalt binders have G*/sinδ values greater than 2.2 kPa for the RTFO asphalt binder; Jnr values typically range from 0.1-4 kPa^{-1}. In both cases, asphalt binders over performed by at least an order of magnitude. To make specification recommendations, Jnr and G*/sinδ values should be compared at typical climactic conditions. A testing temperature of 64 ˚C was selected, a typical value for the climate for most of the PG graded asphalt binders used in this study.

All mixture tests were conducted at a lower temperature, 46 ˚C. Each flow number result needs to be adjusted to reflect the rheological response of asphalt binders at 64 ˚C in the mixture. NCHRP projects 9-25 and 9-31 developed a model used to adjust flow number testing results to a desired reference temperature [33]:

$$FN_R = FN_T(e^{(0.1155(T-R))})^{1.373} \quad \textbf{(18)}$$

Where, FN_R is the flow number at the desired reference temperature, FN_T is the flow number at the test temperature, T is the test temperature and R is the desired reference temperature.

Equation 18 was used to estimate the temperature adjusted flow number measurements from 46 ˚C to 64 ˚C. Jnr measurements were linearly interpolated to obtain an estimated Jnr at 64 ˚C. **Figure 29** and **Figure 30** show the correlation between G*/sinδ and Jnr with flow number for the adjusted temperature.

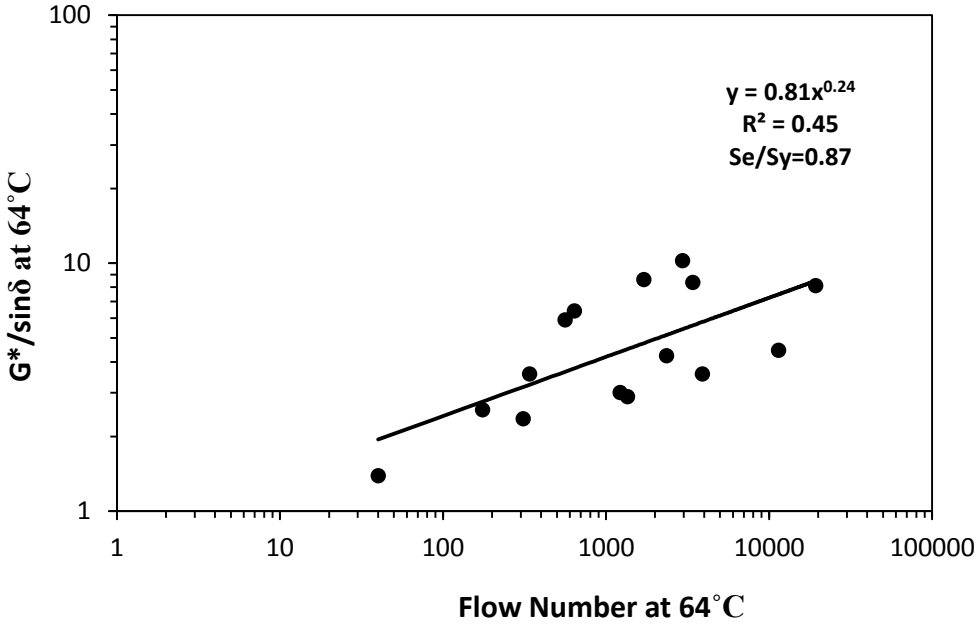

Figure 29-Correlation between G*/sinδ and flow number at 64˚C.

There is a logical trend between G*/sinδ and flow number at 64 ˚C using the power law model. However, the correlation was lower than the 3.2 kPa Jnr measurement and the goodness of fit Se/Sy parameter is 0.87, which is too high to be considered an accurate model.

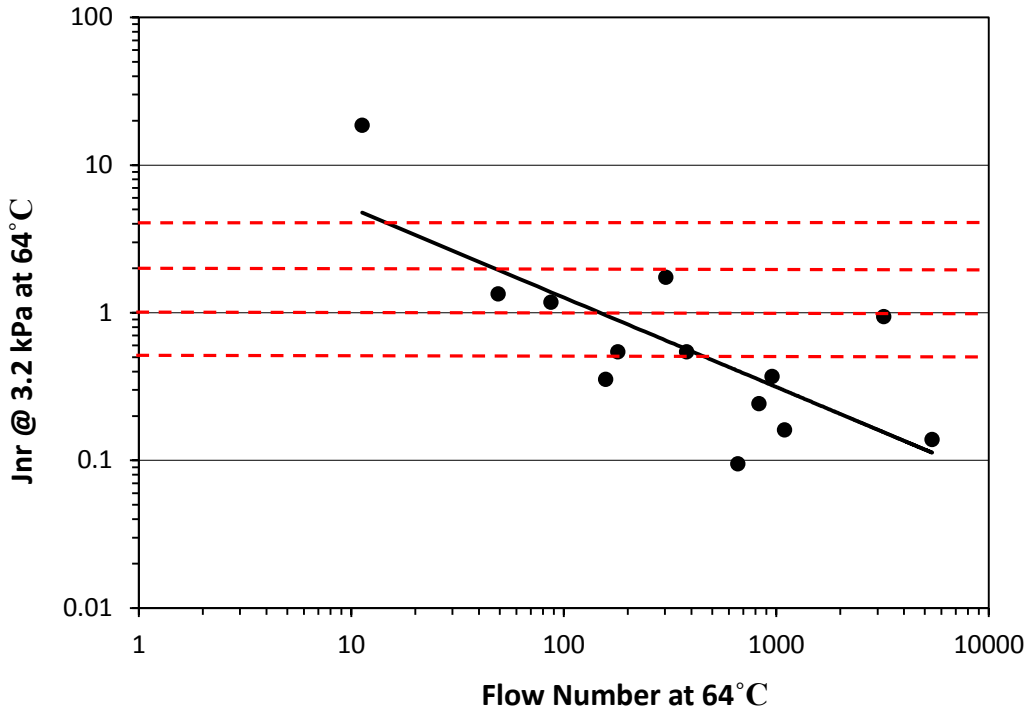

Figure 30- Correlation between Jnr and flow number. Red lines indicate the recommended Jnr limits according to AASHTO M332.

Red lines in **Figure 30** represent the AASHTO M332 recommended specifications. This concept is logical, but does not encompass the potential performance of an asphalt mixture within each Jnr specification range. An asphalt binder between 0.5 kPa^{-1} and 1.0 kPa^{-1} can yield mixtures that have flow numbers spanning an order of magnitude. Therefore, specifications should be set based on the decade Jnr values are measured. This way, adjustments to the mixture design can be made to increase high temperature rutting performance using repeated creep testing or performance testing measures. **Table 5** shows a suggested Jnr specification. A specification value of, 2 kPa^{-1} was recommended for intermediate levels of traffic to ensure polymer modification at climactic grades.

Table 5- Suggested Jnr specification that allows for high performing mixture designs for a given binder

MSCR measured at PG temperature	
Traffic Level M ESALS	**Jnr, kPa^{-1}**
<3	5, max
3 to 10	2, max
>10	0.5, max

Table 5 requires asphalt producers to provide binders with higher rut resistance and accounts for the changes in rut resistance associated with aggregate and mixture design properties. Data gathered from this study was conducted at one stress level and temperature for a wide variety of mixture designs. A more comprehensive study should be conducted to measure each mixture's sensitivity to stress and temperature. Each mixture should be tested in conditions that simulate the in-service climatic temperatures and stress levels. Sensitivity of Jnr with flow number could retroactively validate the results from this study.

5.3 Critical Analysis of PG+ tests to measure elasticity

Currently, there are two testing procedures already implemented by different state DOTs to measure the elastic response of asphalt binders: phase angle and the AASHTO T301 elastic recovery. MSCR %recovery parameter has been cited as a viable alternative to the elastic recovery AASHTO T301 standard [22, 34, 35]. States typically specify a minimum elastic recovery of 50% to ensure binders have some type elastomeric polymer modification. Although ER T301, MSCR and phase angle are popular in the asphalt industry, there are inherent problems with AASHTO T301 procedure and implementation of the MSCR and phase angle testing procedures:

1. ER T301 is conducted at 25 ˚C. One of the main advantages from the Performance Graded SHRP project was development of system that accounts for the temperature dependence of asphalt binders [35]. ER T301 ignores this fundamental property of asphalt binders. In addition, MSCR and phase angle are measured at high temperature

PG grades; it is not logical to compare asphalt binder tests measured at 25 ˚C with asphalt binder tests conducted at temperatures of 58-82 ˚C.

2. ER T301 testing apparatus is large with a relatively complicated testing configuration. A ductility bath, almost 6 feet long, and the "dog bone" test specimen make testing inefficient compared with 25 and 8 mm binder specimens tested in a Dynamic Shear Rheometer (DSR) for PG grading.

The second problem associated with the ER T301 has been solved by Clopotel et. al by applying a shear load to an 8 mm sample in the DSR at the same rate as in ER T301 [34]. Results of the study were verified, shown in **Figure 31**.

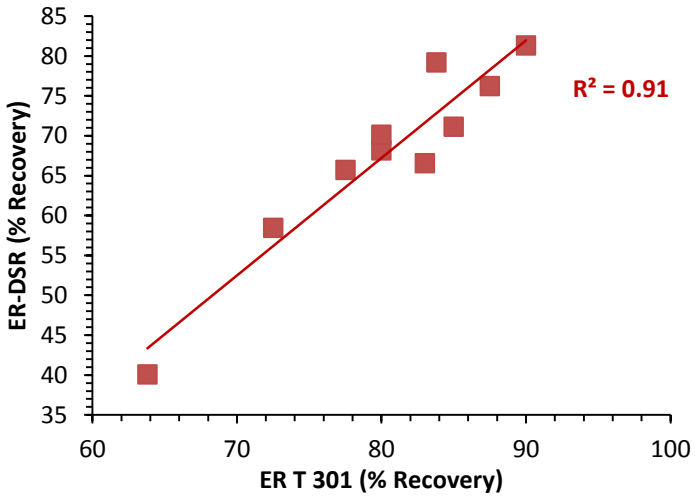

Figure 31-Correlation between the ER-DSR testing procedure and DER T301 testing procedure.

A R^2 value of 0.91 indicates a high correlation; ER T301 procedure can be substituted for the ER-DSR. ER-DSR test procedure can also be conducted at high temperatures, but an investigation into the required shear rate and testing geometry must be conducted in order to write a specification.

In the appendix of the MSCR M332 specification is a %recovery-Jnr curve at 3.2 kPa to indicate the presence of an elastomeric polymer, shown in **Figure 32**. The curve was derived from a data set with asphalt binder modified at different concentrations of elastomeric polymer [22]**. Figure 32** also includes %recovery-Jnr results for several binders as part of the WCTG. The red curve represents the recommended limits for passing and failing the AASHTO M 332

specification. Implementation of the AASHTO M 332 MSCR-PG system will be conducted at typical climactic conditions, so the curve in **Figure 32** would be expected to shift left horizontally and up vertically if implemented into state specifications.

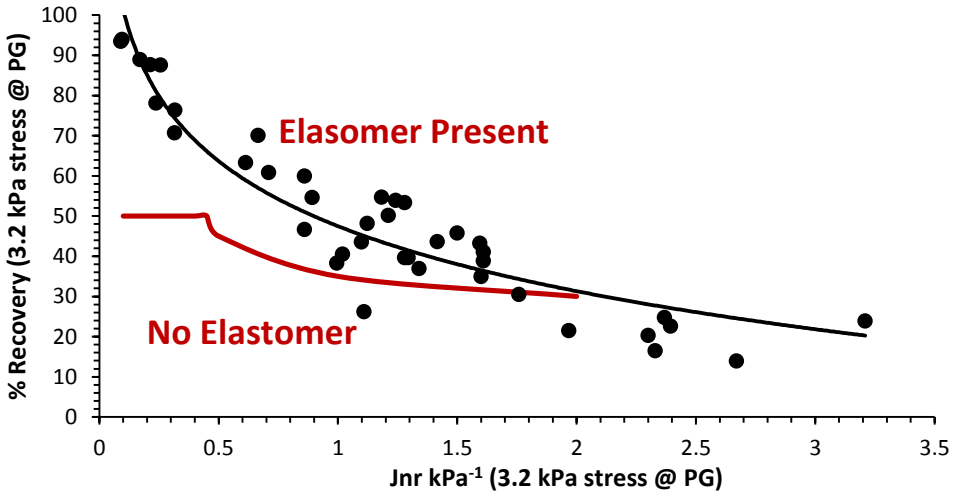

Figure 32- MSCR %recovery-Jnr curve used to determine presence of elastomeric modification.

Based on the %recovery-Jnr curve, all binders tested in this study have some type of elastomeric modification. Although the use of the specification in **Figure 32** has been shown to indicate polymer modification, it may not provide additional information regarding the high temperature performance of asphalt binders. To further investigate the %recovery-Jnr relationship, MSCR testing was conducted at the high temperature PG grade for 40 binders**. Figure 33** shows the relationship between % recovery and Jnr measured at the high temperature PG for three different stress levels.

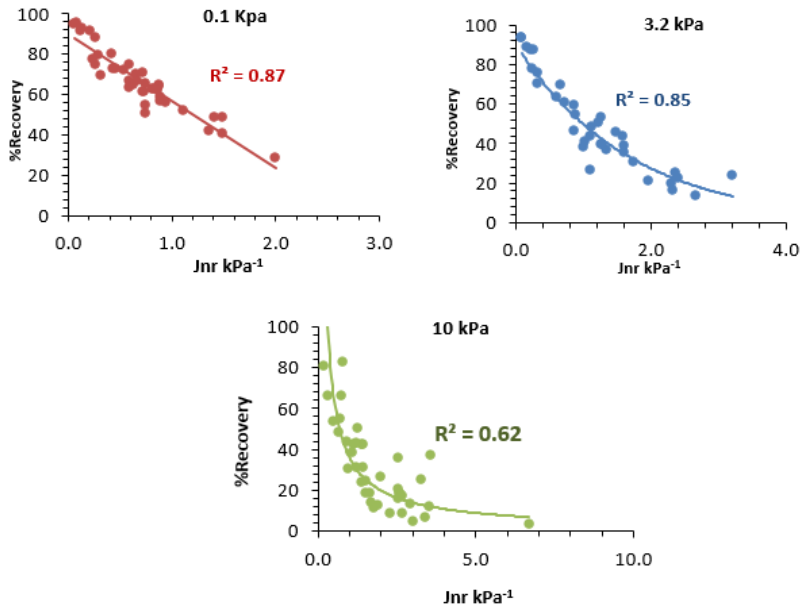

Figure 33- MSCR relationship measured at PG and three different stress levels.

Asphalt binders are stress dependent materials. Once the stress level is increased outside the linear-viscoelastic region, the strain response at an applied stress will increase disproportionately. Jnr will continue to increase with stress level outside of the linear-viscoelastic region. This concept is validated by the increase in Jnr from 0.1kPa to 10 kPa. In **Figure 33**, a linear, logarithmic and power model gave the highest correlations for 0.1 kPa, 3.2 kPa and 10 kPa applied stress, respectively. Differences in the curve fit were expected; as the stress level increases the Jnr increases while the %recovery approaches a constant value. However, non-linearity for the high stress level curves may not be related to the stress dependence of the material. Change in curve fit is related to mathematical definitions of Jnr and %recovery, shown by in **Equations 11** and **12**. %recovery is normalized to the peak strain at each creep cycle and the Jnr is divided by a constant stress. % recovery values cannot decrease below 0, and the Jnr values can continue in to increase with stress level. Future work should be conducted to validate that the %recovery approaches a constant value unique to the modification type. If true, binders that are targeting lower Jnr values with higher elastic recovery do not need to include both the % recovery and Jnr parameter. From a performance perspective, Jnr at 3.2 kPa correlates with high temperature rutting resistance, as shown in

Figure 28, and not % recovery. Elasticity is an important property of asphalt binders and has been shown to increase the quality of asphalt mixtures. Binder and mixture tests should be developed at the respective temperature and aging regime to capture the potential benefits of elastomeric modification in a performance based system.

Some states also specify maximum phase angles at high temperature PGs as an indicator of binder elasticity and/or elastomeric modification. MSCR % recovery and high temperature phase angle were correlated to determine if the parameters are interchangeable. **Figure 34** shows a correlation for MSCR %recovery and phase angle measured at the high temperature PGs. %Recovery measurements were found to have the highest correlation at a stress level of 0.1 kPa; 3.2 kPa and 10 kPa correlations were therefore excluded.

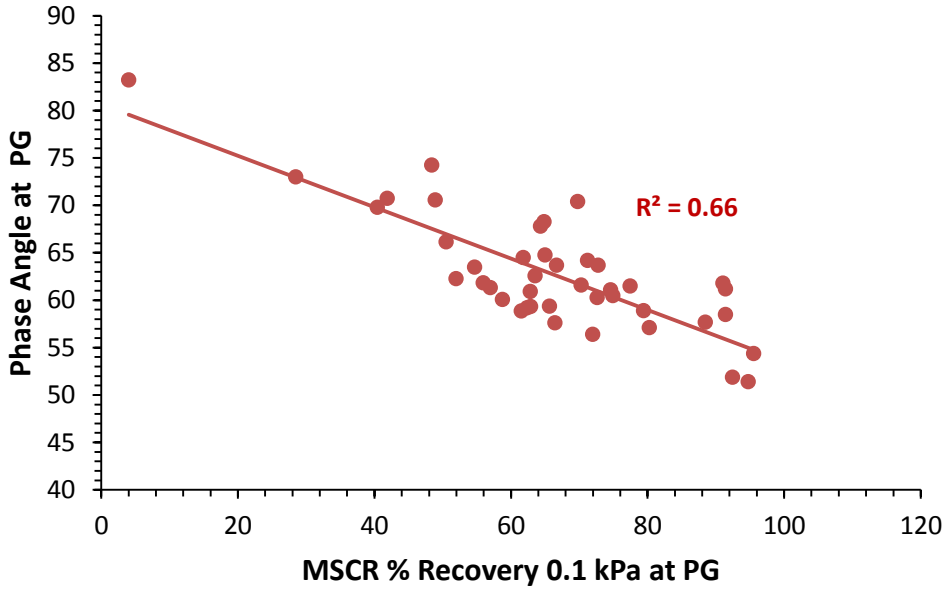

Figure 34-Correlation between MSCR 0.1 kPa % Recovery and phase angle at high temperature PG.

As expected, there are is a logical correlation between MSCR %Recovery and phase angle at high temperatures. However, the range of potential %recovery values, 7-99%, was larger than the range of phase angle values (50-80); there are high amounts of variability for binders with similar phase angle measurements. For example, there are six binders with a phase angle of 60 that have a % recovery ranging from 50 to 80% recovery. %Recovery and phase angle cannot be used interchangeably to measure high temperature elasticity. Further research should be

conducted to identify the effects of modification type on phase angle and %recovery measurements.

6. Fatigue Cracking at Intermediate Temperatures

Fatigue or durability cracking is caused by repetitive traffic loading at intermediate service temperatures. When asphalt pavements are subjected to repeated loading, micro-cracks begin to develop in the pavement structure. Extended periods of loading cause larger macro cracks to propagate from the micro cracks which results in fatigue cracking [36]. Fatigue cracking typically occurs late in a pavement's service life and is a common type of distress in cold climatic conditions. The current Superpave system specifies a maximum G*sinδ parameter of 5000 kPa after aging in the rolling thin film oven RTFO and pressure aging vessel PAV. Extended periods of aging or oxidation cause asphalt binders to become brittle relative to newly refined asphalt binder. Embrittlement of asphalt binder is linked with an increase in G*sinδ; therefore, a maximum limit was placed. Although there is a relationship between asphalt rheology and fatigue cracking, G*sinδ does not directly measure fracture properties associated with fatigue cracking. In this section, intermediate PG+ plus tests were evaluated to determine each tests potential for characterizing fatigue resistance. Four PG+ intermediate temperature testing procedures have gained popularity and were analyzed in the following sections: ductility, toughness and tenacity, linear amplitude sweep (LAS) and binder yield energy test (BYET). Recommendations for inclusion in to state DOT standard specifications were made based on correspondence with fundamental engineering principles and mixture performance validation.

6.1 Comparison of available PG+ Testing Procedures.

Current state DOT PG+ binder tests used to specify intermediate temperature properties of asphalt binder include ductility and toughness and tenacity. Both testing procedures require use of a unique testing apparatus. Two additional DSR testing procedures have been proposed to replace or more accurately characterize the performance of asphalt binder: Binder Yield Energy Test (BYET) and the Linear Amplitude Sweep test (LAS). In this section, each new binder tests will be correlated with current PG and PG+ test methods.

6.1.1 Binder Yield Energy Test (BYET) correlations with available PG plus test methods

In the BYET test procedure, a shear load is applied to an 8 mm binder sample, in the DSR, at the same displacement rate as the ductility bath test method. After completion of the test, the stress strain response of the asphalt binder sample is used for analysis. Two parameters are reported: strain at peak stress and yield energy to peak stress (refer to **Figure 13**). Strain at the peak stress provides an indication of an asphalt binder's ductility. Binders with high ductility experience high amounts of strain prior to reaching peak stress. Yield energy was measured by calculating the area under the BYET stress-strain curve up to the peak stress. Binders with a high yield energy can experience high levels of stress and strain before failure. Both parameters were correlated to ductility length, as shown in **Figure 35** and **Figure 36**.

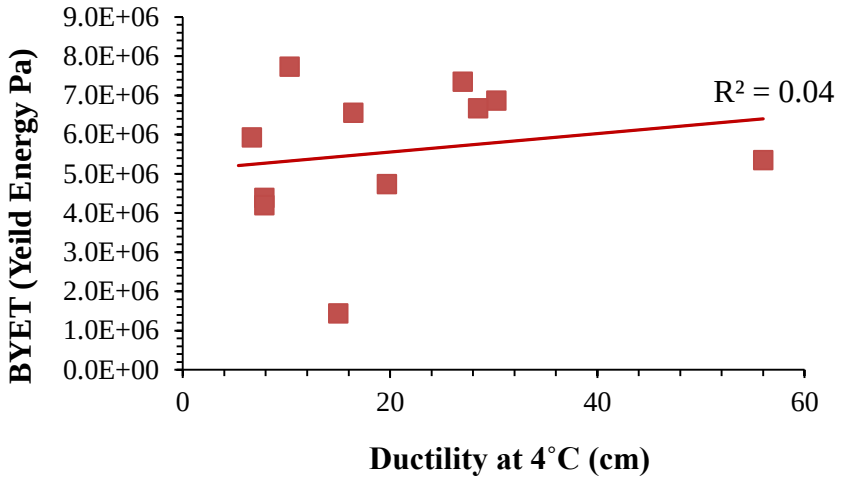

Figure 35-Binder yield energy (BYET) correlated against ductility for the RTFO binder at 4˚C.

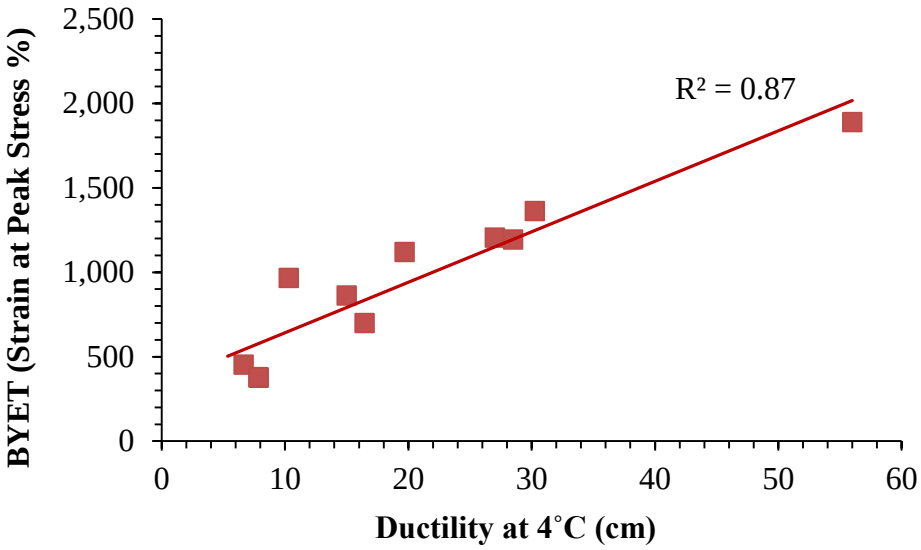

Figure 36-Binder strain at peak stress correlated with ductility on the RTFO binder at 4˚C.

On the RTFO aged asphalt binder there was a moderate correlation for strain at peak stress and no correlation for yield energy. Strain at peak stress trends are logical; as the ductility increases so does strain at peak stress. An explanation for this trend can be understood by analyzing the stress -strain curves from BYET test results. Three binders with different ductility values were selected and their stress-strain curves from the BYET testing results are shown in **Figure 37**. Binders with the highest peak stress corresponded to those binders with the highest yield

energy. For the binders produced by WCTG members, the strain at peak stress from the BYET can be an indicator for the stress and energy required to yield an asphalt binder.

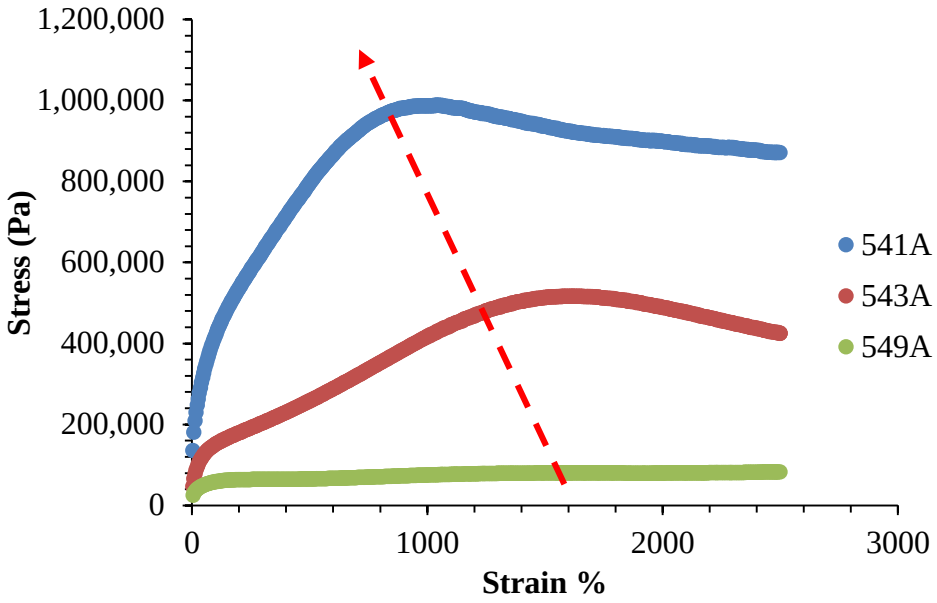

Figure 37-BYET stress strain curves for three different binders with varying ductility.

However, the yield energy parameter from BYET testing was introduced as a means to consider both the stress the material can withstand and its strain tolerance. Using the yield energy approach can better differentiate between modified asphalts as materials ductile with high yield energies and vice versa. One example of a binder having a higher yield energy and similar ductility is shown in **Figure 38**.

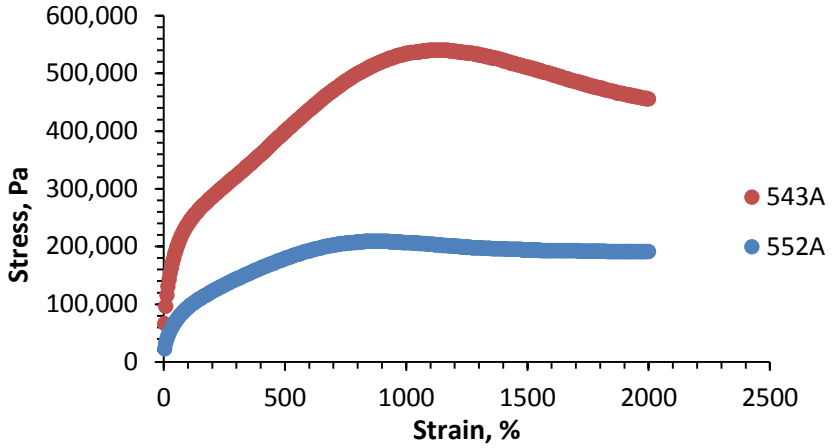

Figure 38-Stress strain curve for two binders with similar ductility, but different yield energies.

Therefore, to best quantify the effectiveness of modification, development of specifications using the BYET test method must consider yield energy and strain at peak stress in unison. Depending on the traffic level; different binders are expected to withstand different amounts of stress. Considering both parameters allows for flexibility in materials selection. For example, if the sacrifice in strength leads to an increase in ductility, selection of asphalt binders can be based on the expected stress level. If an asphalt binder is expected to experience low levels of stress, the strength of an asphalt binder can be sacrificed to provide a more ductile binder.

Toughness and tenacity was expected to have a high correlation with binder yield energy. In both testing procedures, the area under a load versus deflection curve are reported. **Figure 39** shows that there is no correlation between toughness and tenacity and BYET yield energy.

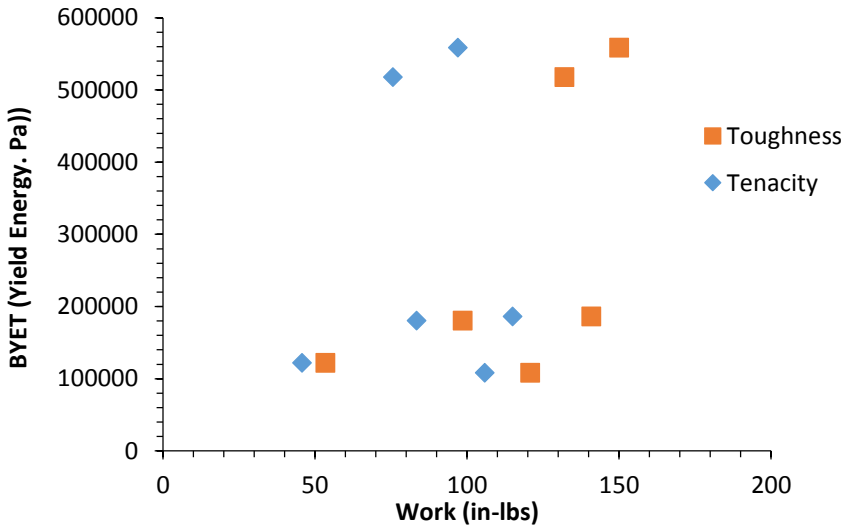

Figure 39-Toughness and tenacity correlated with BYET yield energy.

Although no correlation could be obtained, two distinct "groups" were observed; one group showed a relatively high binder yield energies for a given T&T and the other was low yield energies for a given T&T. To investigate this potential relationship further, ductility, the "dogbone" procedure, was correlated with the toughness and tenacity parameters to gather more data for comparison, shown in **Figure 40.**

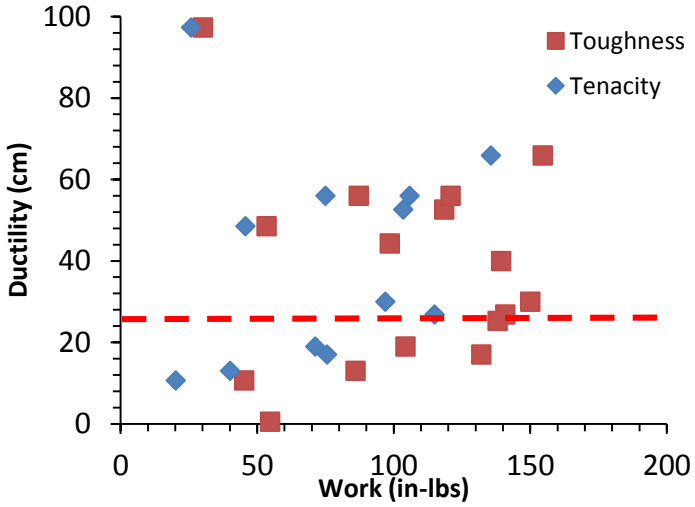

Figure 40-Correlation between ductility and T&T. Where the red dotted line represents the typical minimum ductility specification.

Again, the comparison could be divided into categories: T&T correlations that fail the ductility specification and T&T correlations that pass the ductility correlations, shown by red line. Each binder sample tested was categorized by T&T and ductility specification to identify any differences in the BYET stress-strain response due to specification type. BYET stress-strain results may distinguish between the different modification types required to meet the intermediate ductility and T&T specifications**. Figure 41** shows a typical BYET stress-strain curves for binders categorized by the specification they were intended to meet.

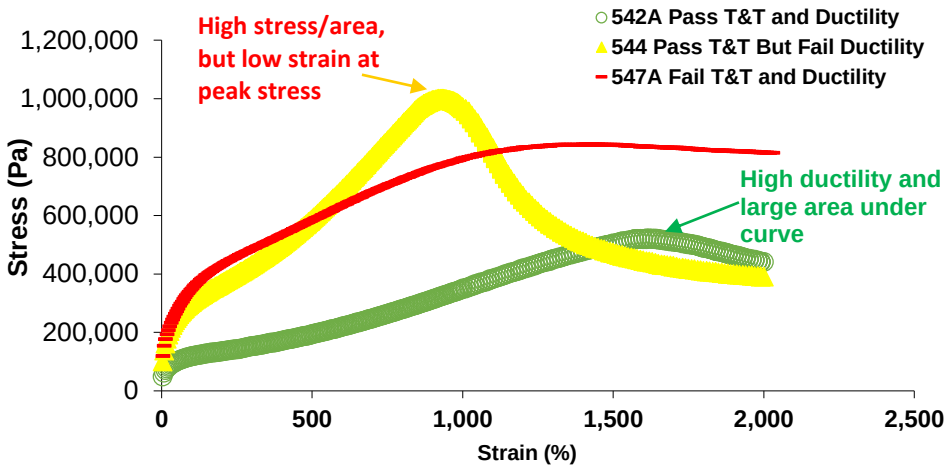

Figure 41- Typical BYET stress-strain curves categorized by T&T and ductility specification.

If a T&T value is specified without ductility, the BYET stress strain curve has a stress peak at a low strain level. If T&T and ductility are specified, a relatively smaller stress peak was observed with an extended strain at peak stress. If neither T&T nor ductility are specified, a well-defined peak stress was not observed. Certain modifiers can be distinguished by a well-defined peak and others can be distinguished by the strain at peak stress. Specifications can be developed for both to target the same properties as in T&T and ductility with one BYET test in the DSR based on the stress strain response. PG+ results for the binders from **Figure 41** are shown in **Table 6**; in this comparison all of the binders have similar yield energies and the strain at peak stress distinguishes between ductility.

Table 6-Comparison of intermediate PG+ tests for three different binders.

Binder	Strain at Peak Stress,%	Yield Energy (Pa)	Ductility (in)	Toughness (in-lbs)
542A	1510	6.86×10^6	30.3	155
544A	701	6.56×10^6	16.5	150
547A	1200	6.67×10^6	28.5	105

6.1.2 Linear Amplitude Sweep (LAS) correlations with PG and PG plus test methods

To conduct a LAS test, an 8mm binder sample is placed in the DSR and loaded in the following steps: a frequency sweep from 1 to 30 Hz at 1% strain followed by a strain amplitude sweep from 0.1% to 30% at a constant frequency of 10 Hz. Viscoelastic continuum damage theory (VECD) is then applied to measured material properties model the number of loading cycles required to fail an asphalt binder at in-service strain levels (refer to **Figure 17**).

LAS could not be correlated to either T&T or ductility. LAS is a unique testing method in the sense that it uses constitutive equations based on the testing results to generate fatigue law curve; all of the current PG or PG + testing procedures are related to the basic stress-strain curves at constant rate of loading or undamaged binder properties. In service, asphalt binder becomes damaged due to repeated loading of traffic which is not represented by current PG+ parameters. Therefore, PG+ parameters were not expected to correlate with the LAS test. Other PG and PG+ tests offer other ways to indicate elasticity or toughness, but the LAS provides a more logical way to connect binder to mixture fatigue performance. **Figure 42** shows a fatigue law comparison for three different binders from LAS testing. Two different pieces of information can bet taken from fatigue law curves: fatigue life and strain sensitivity. Some binders may have a high fatigue life at low strain levels and a relatively low fatigue life at high strain levels.

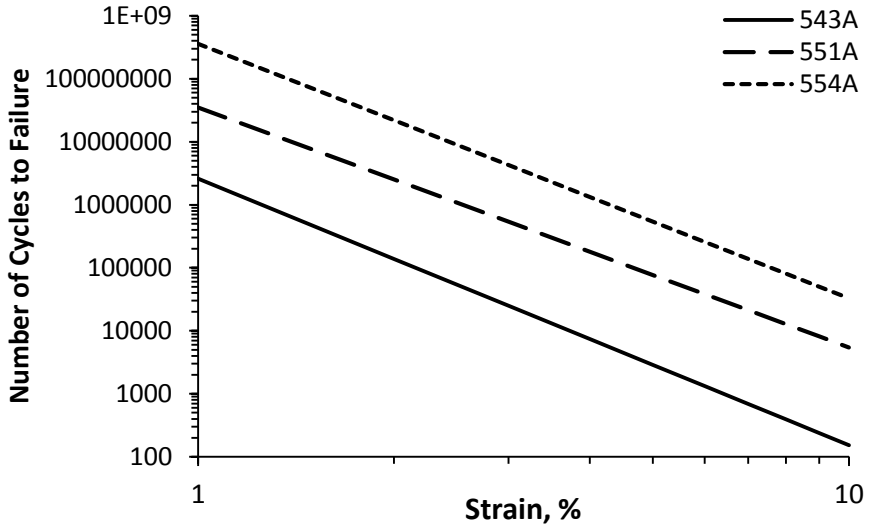

Figure 42-Fatigue law curves for three different asphalt binders.

To make a correlation between G*sinδ and LAS results, all binders were tested in the PAV aged condition at intermediate temperature grades. Correlations with G*/sinδ were made at two strain levels: 2.5% and 5% to address fatigue life at low and high strain levels. Figure 43 shows each correlation, respectively. Low correlations were measured at both strain levels. This was expected, given that G*sinδ was measured in the linear-viscoelastic range of behavior and in an undamaged state.

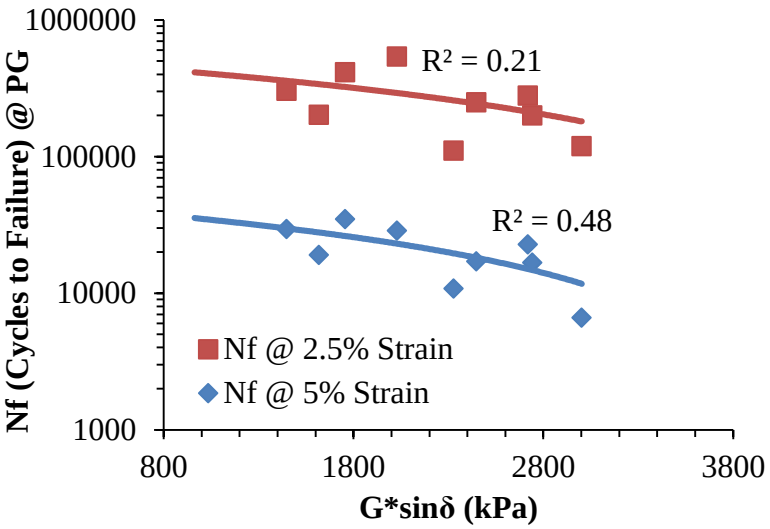

Figure 43- LAS correlated with G*sinδ parameter at the PG temperature.

6.2 Indirect Tension Fatigue Validation

Indirect tension fatigue testing was conducted, with Arshadi [37], on four binders and two different mixture designs, one fine and one coarse, to determine which binder parameters best correlated with mixture performance. **Figure 44** shows the aggregate gradations selected for testing.

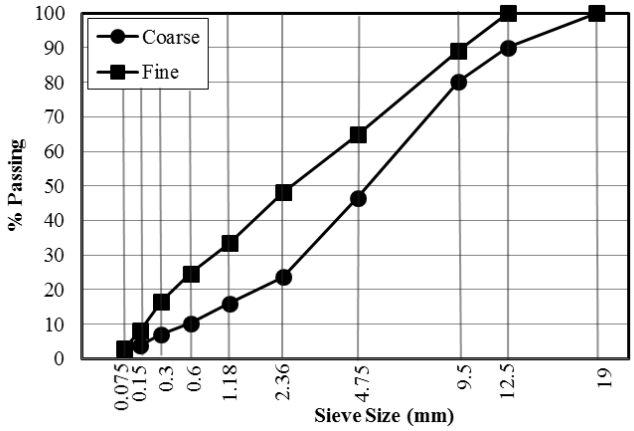

Figure 44-Aggregate gradations selected for IDT fatigue testing [37].

Four levels of asphalt binder modification include: unmodified, plastomer, ground tire rubber and elastomer modification. Disk shaped specimens were prepared and dynamically loaded at frequency of 10 Hz and a load of 730 lbs until failure [37]. All specimens were compacted to

an air voids content of 4%. 4% was selected to reflect the air void level expected later in the pavements service life. LVDTs were attached to each specimen parallel and perpendicular to the direction of loading which is shown in **Figure 45**.

Figure 45-Disc shaped IDT fatigue sample with LVDTs perpendicular and parallel to servo hydraulic loading apparatus.

Each sample was loaded with the same configuration until failure. Number of cycles to failure was defined as the cycle at which the constant rate of horizontal deformation increases to rapid deformation, shown in **Figure 46** [37]**.**

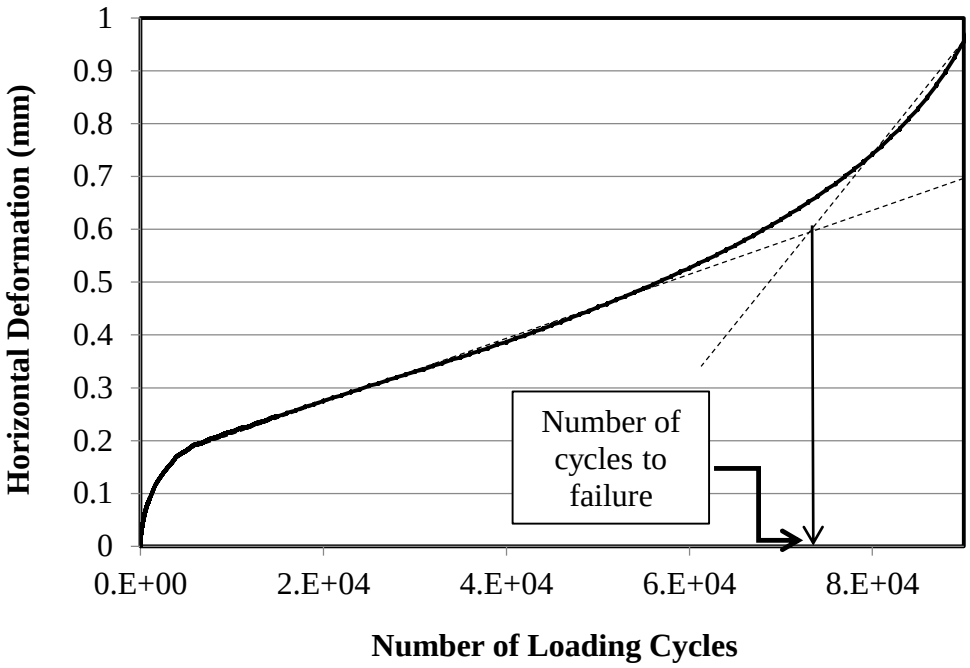

Figure 46-Failure criteria for IDT fatigue testing.

The number of loading cycles required to fail the specimen was recorded and correlated with results from LAS, BYET and ER-DSR testing procedures.

Kose et al indicated that the average binder strain level is 7.8% higher than the asphalt mixture bulk strain [38]. In the IDT fatigue test, the bulk strain is approximately 0.3% at each loading cycle, which corresponds to a binder strain of 2.5%. **Figure 47** shows the correlation between number of cycles to failure for the LAS at 2.5% strain and IDT mixture fatigue tests.

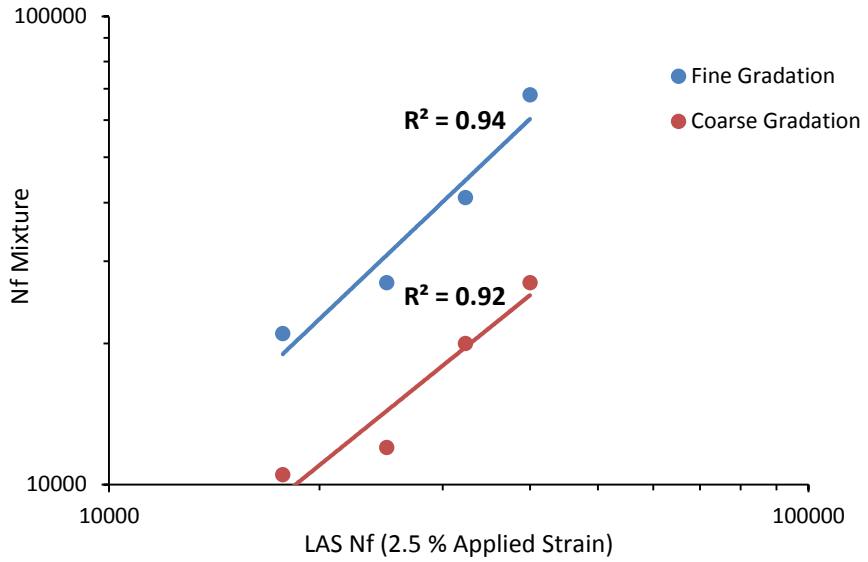

Figure 47-LAS comparison between IDT fatigue testing and LAS fatigue results.

A linear log-log relationship was calculated using the power law with an R^2 value of 0.94 and 0.92 for the fine and coarse HMA mixture designs, respectively. High correlations were measured at different LAS strain levels, but 2.5% applied strain provided a high correlation on the same logarithmic decade as the mixture tests. Results validate that LAS can be used to predict the fatigue performance of asphalt mixtures at the estimated strain levels in the asphalt binder cited by Kose et al. Similair tests should be conducted at multiple mixture strain levels to determine if the 7.8x rule applies in every case.

IDT fatigue did not have direct correlation with BYET, G*sinδ, and elastic recovery measurements. However, elastic recovery and complex modulus correlations provided insight on the fatigue behavior of HMA mixtures. **Figure 48** and **Figure 49** show the correlation between elastic recovery and complex modulus with IDT fatigue, respectively. Plastomer modified binder has a higher IDT fatigue life than the ground tire rubber binder, but has a lower elastic recovery. This was expected given that plastomers do not add elasticity to asphalt binders, but increase stiffness. In **Figure 49,** there is no direct correlation between stiffness and fatigue life. Elastomers and tire rubber do not change the stiffness of the asphalt binder, but the elastomer and tire rubber increase the fatigue life of the mixture. An identical correlation was measured when comparing the PG G*sinδ with fatigue life of the mixture. This suggests that, the G*sinδ and elasticity measurements cannot be used to estimate the fatigue

life of asphalt materials. Fatigue life is a function multiple viscoelastic properties and the LAS provides one alternative to do so with fatigue law modeling.

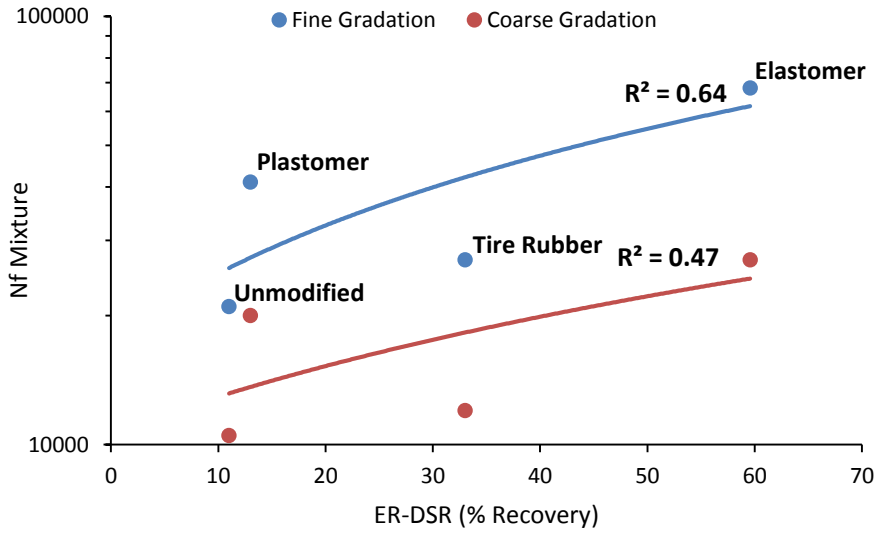

Figure 48- Correlation between elastic recovery and IDT fatigue cycles to failure.

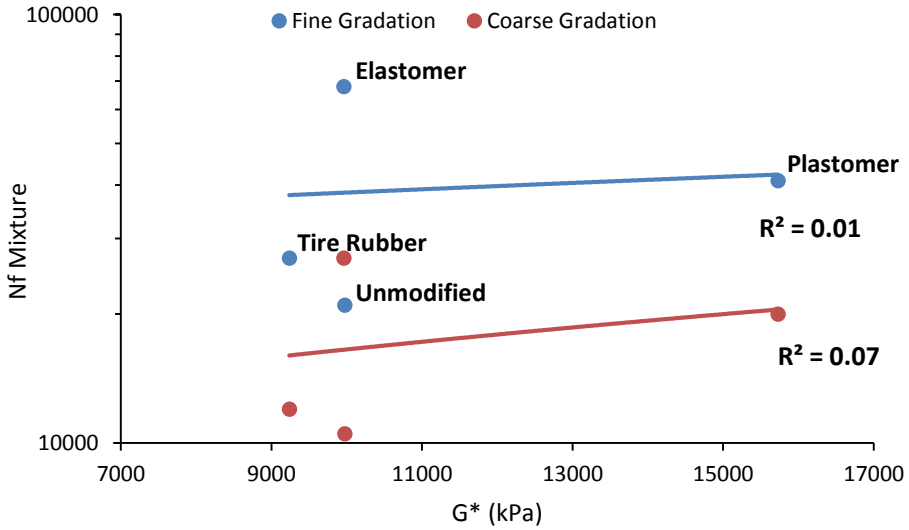

Figure 49-Correlation between asphalt binder stiffness and fatigue life.

7. Thermal Cracking at Low Temperatures

Thermal cracking is caused by tensile stress buildup in restrained asphalt pavement layers. In order to release stress build-up, pavements will crack transversely across the pavement

surface. Currently, the Superpave system specifies low temperature flexural stiffness and stress relaxation parameters to guard against thermal cracking. This information is valuable, but does not address all the material properties that lead to thermal cracking; physical hardening and failure properties. By changing the asphalt binder chemistry, the rate at which an asphalt binder stiffens may change. Thermal cracking is caused by stresses in the asphalt binder; this implies that understanding how different binders fail may provide a more robust method for low temperature characterization of asphalt binders. BBR physical hardening, direct tension and SENB tests were conducted to understand any correlations between stiffness and failure properties at typical Superpave performance grades. After correlations between binder tests were made, mixture performance validation was conducted to identify parameters most closely related to the glass transition and fracture temperature using the Asphalt Thermal Cracking Analyzer (ATCA).

7.1 Physical Hardening- BBR Stiffness

Forty different asphalt binders were tested in the BBR after 1 hour and then again after 24 hours at the low temperature asphalt PGs. It has been hypothesized that the rate of change in the stiffness of an asphalt binder may be related to an asphalt binder's susceptibility to thermal cracking. **Figure 50** shows the relationship between the BBR stiffness at 24 hours compared with the BBR stiffness at 1 hour.

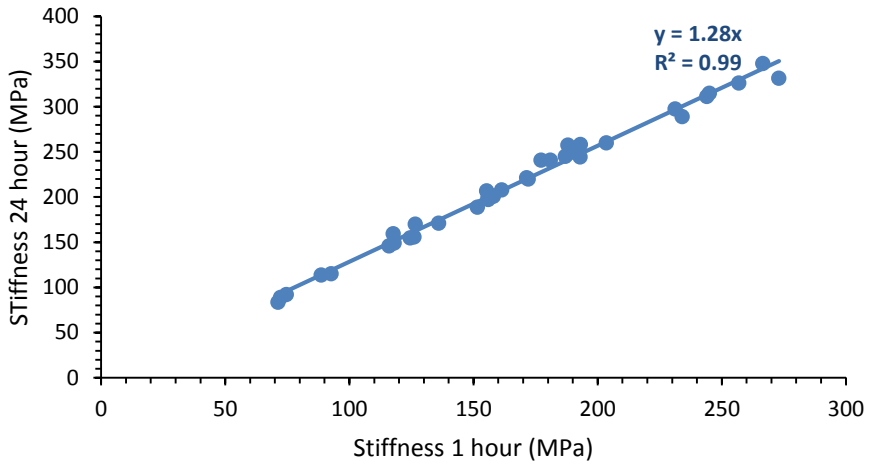

Figure 50-Asphalt binder stiffness at one hour correlated with asphalt binder stiffness at 24 hours.

There is a direct linear relationship between stiffness at 24 hours and stiffness at 1 hour with a R^2 value of 0.99. For all of the asphalt binders tested in this study, the BBR stiffness at 1 hour may be multiplied by 1.3 to obtain the stiffness at 24 hours. Although the results seem conclusive, additional testing is required that may show differences in the physical hardening. Tabatabaee has shown, for a given binder, that the rate of physical hardening changes relative to the glass transition temperature [39]. In addition, differences in binder chemistry may show different rates of hardening for extended time periods or conditioning temperatures.

7.2 Available Low Temperature Fracture Tests

SENB and Direct Tension test methods were compared with BBR stiffness and m-value parameters to determine: 1) is there are correlation between the BBR and fracture tests? 2) If yes, is it necessary for binder specifications? All testing was conducted on the PAV aged condition to measure thermal properties most susceptible to thermal cracking. The direct tension test (DTT) was included into the original Superpave specification system for those binders that failed the BBR stiffness parameter. If an asphalt binder passed the m-value but failed the stiffness value, the DTT test could be run as a substitute with a minimum strain at failure of 1%. **Figure 51** shows the correlation between direct tension failure strain and the BBR parameters stiffness and m-value. There is a logical trend between the DTT failure strain and BBR parameters. As the stiffness of an asphalt binder increases the failure strain decreases.

Those binders that are able to maintain a high m-value or ability to relax stress can experience higher levels of strain prior to failing.

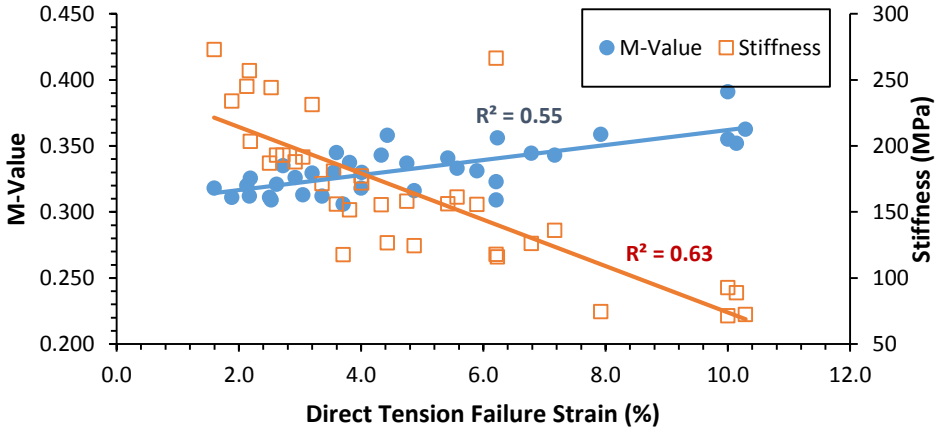

Figure 51- Correlation between BBR parameters and direct tension failure strain.

Equations 14 and **15** were used to calculate the SENB parameters: displacement at max load, fracture toughness, and fracture energy. Displacement at max load represents the amount of deformation an asphalt binder can withstand at typical low temperature service temperatures. Fracture toughness is related to maximum load an asphalt binder experiences at failure. Fracture energy represents the area under the force deflection curve until failure. **Figure 52** shows a typical SENB data output with an illustration of each parameter provided.

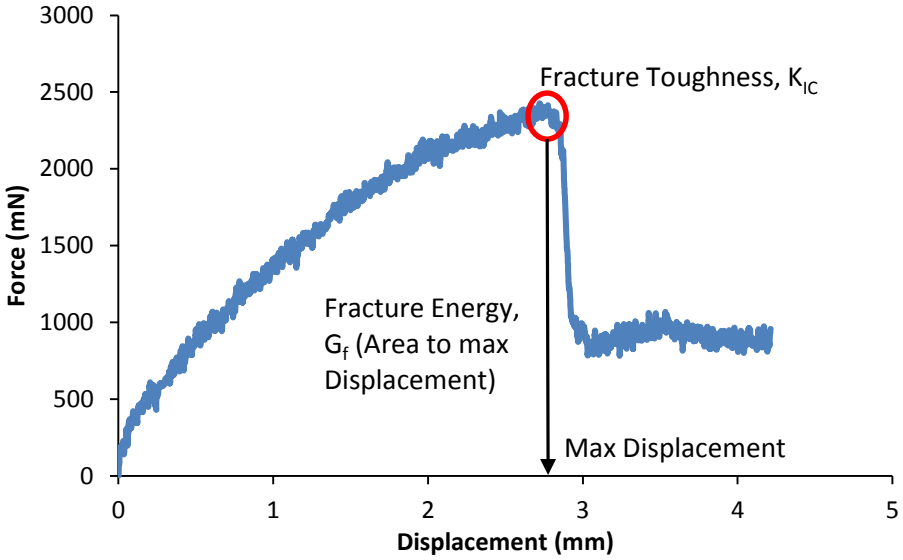

Figure 52-Typical SENB force versus displacement data.

Each SENB parameter was correlated with the BBR stiffness and m-value true grades, shown in **Figure 53**, **Figure 54** and **Figure 55**. The true grade is defined as the temperature that corresponds with a BBR stiffness and m-value of 300 MPa and 0.300, respectively. Binders with a lower true grade will have a lower binder stiffness and higher ability to relax stresses at low temperatures. For all SENB parameters there are no correlations with BBR stiffness or m-value. In fact, asphalt binders with a similar true grades can have a SENB parameters ranging from the highest to lowest measured values. The SENB test procedure is run at typical low temperature performance grades and could be implemented to measure fracture properties in addition to the linear viscoelastic stiffness and compliance. To maximize the low temperature asphalt binder cracking resistance, the displacement should be maximized, fracture toughness minimized and fracture energy maximized. Fracture energy, G_f, combines both for force and displacement properties and should be given the highest consideration for implementation into the current specification.

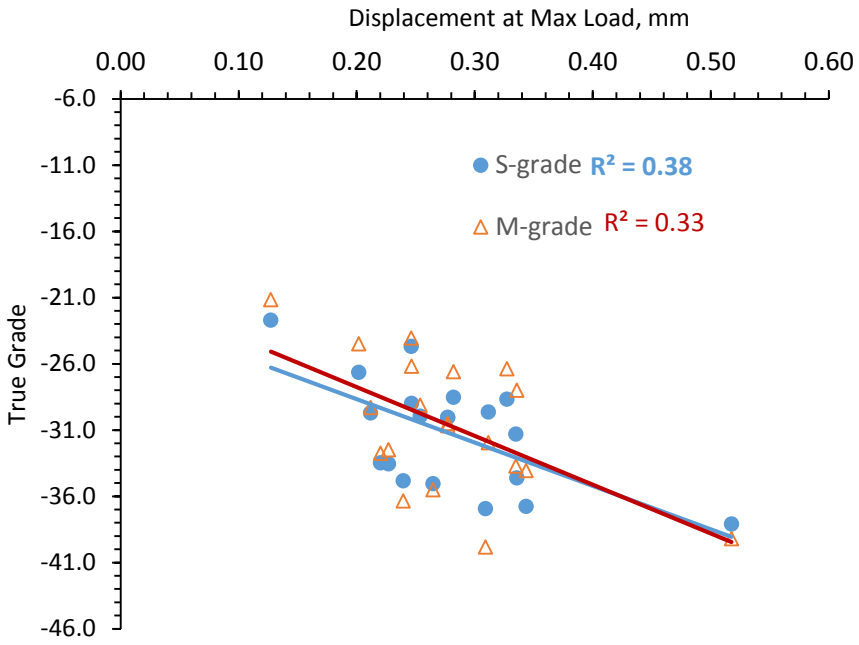

Figure 53-Correlation between stiffness and m-value true grade with SENB maximum displacement.

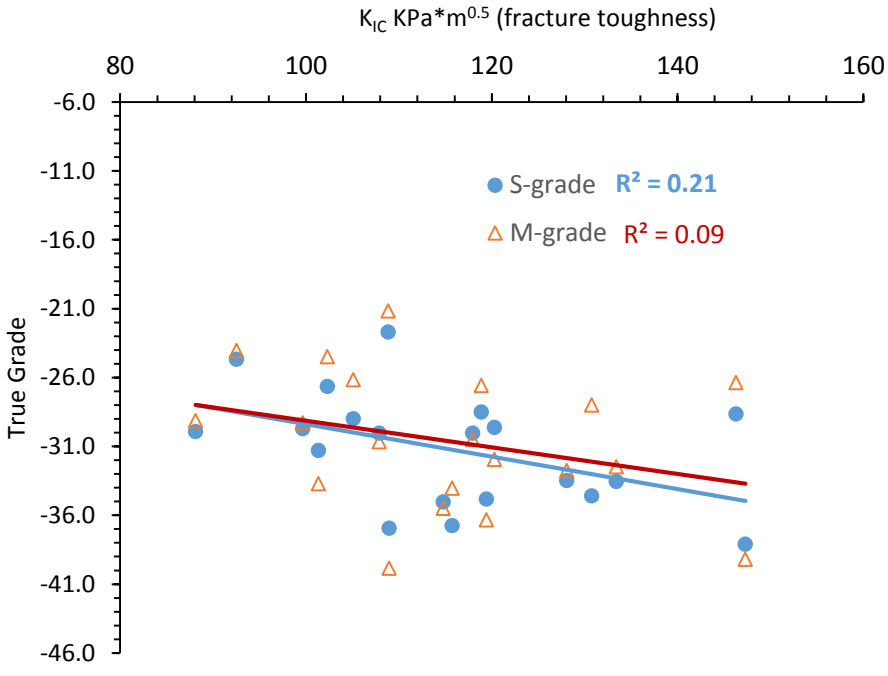

Figure 54- Correlation between BBR stiffness and m-value true grade with SENB fracture toughness.

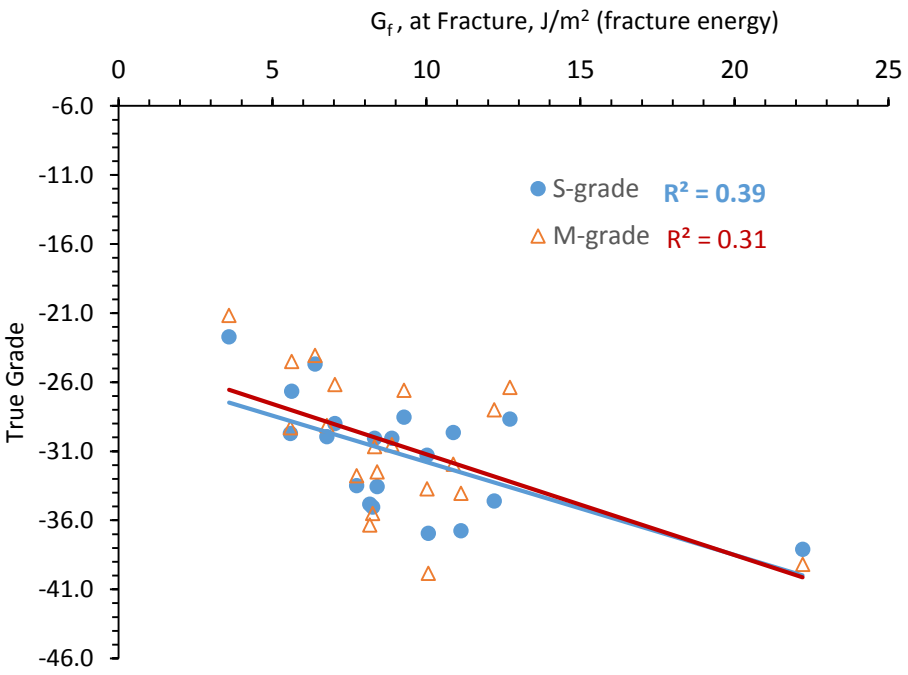

Figure 55- Correlation between BBR stiffness and m-value and SENB fracture energy.

7.3 Mixture Performance Validation-Asphalt Thermal Cracking Analyzer

The Asphalt Thermal Cracking Analyzer (ATCA) was used to validate which asphalt binder testing parameters, between the BBR and SENB methods, have the highest correlations with mixture performance. Binders in the mixtures tested had three different low temperature PG grades: two with -28, one with -34, and one with -22. Four different mixtures were tested to obtain four mixture performance parameters: fracture temperature, glass transition temperature, liquid coefficient of thermal contraction and glassy coefficient of thermal contraction. Two tests are run in the ATCA apparatus to obtain each parameter: restrain and unrestrained. In the unrestrained test method, prismatic asphalt mixture samples are placed in an insulated box and cooled at 1 ˚C/min from room temperature to -70 ˚C. LVDTs on either side of the sample measure contraction of the beam as it cools. The rate of contraction decreases as it approaches the glass transition temperature. The temperature at which the sample changes contraction rates is the glass transition temperature. Liquid coefficient of contraction is the rate above the glass transition temperature and glassy coefficient of

contraction is the rate below the glass transition temperature. In the restrained test, the asphalt specimen is prevented from contracting. As the specimen is cooled, the sample builds up stress until it cracks. The temperature at which the specimen cracks is defined as the fracture temperature. **Figure 56** and **Figure** 57 show a schematic of the ATCA apparatus and typical ATCA data outputs used to measure each parameter, respectively.

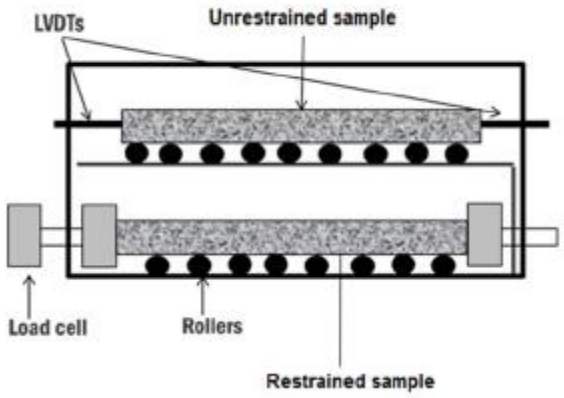

Figure 56-Schematic of ATCA apparatus, taken from Marasteanu et al [30].

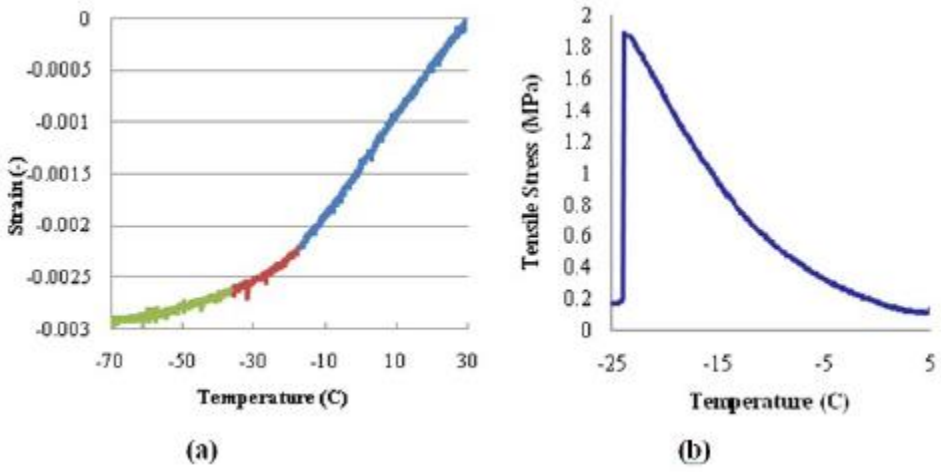

Figure 57-Typical data outputs for (a) unrestrained and (b) restrained ATCA testing, taken from Marasteanu et al [30].

Each asphalt binder and mixture combination was correlated with the ATCA mixture testing results. ATCA fracture temperature was the only mixture testing parameter that could be correlated to binder testing results. Strong correlations were measured for BBR stiffness, SENB fracture toughness and SENB displacement at maximum load. R^2 values of 0.99, 0.99

and 0.74 were measured for stiffness, fracture toughness and displacement at maximum load, respectively. **Figure 58** shows the strong linear correlations form BBR and SENB parameters.

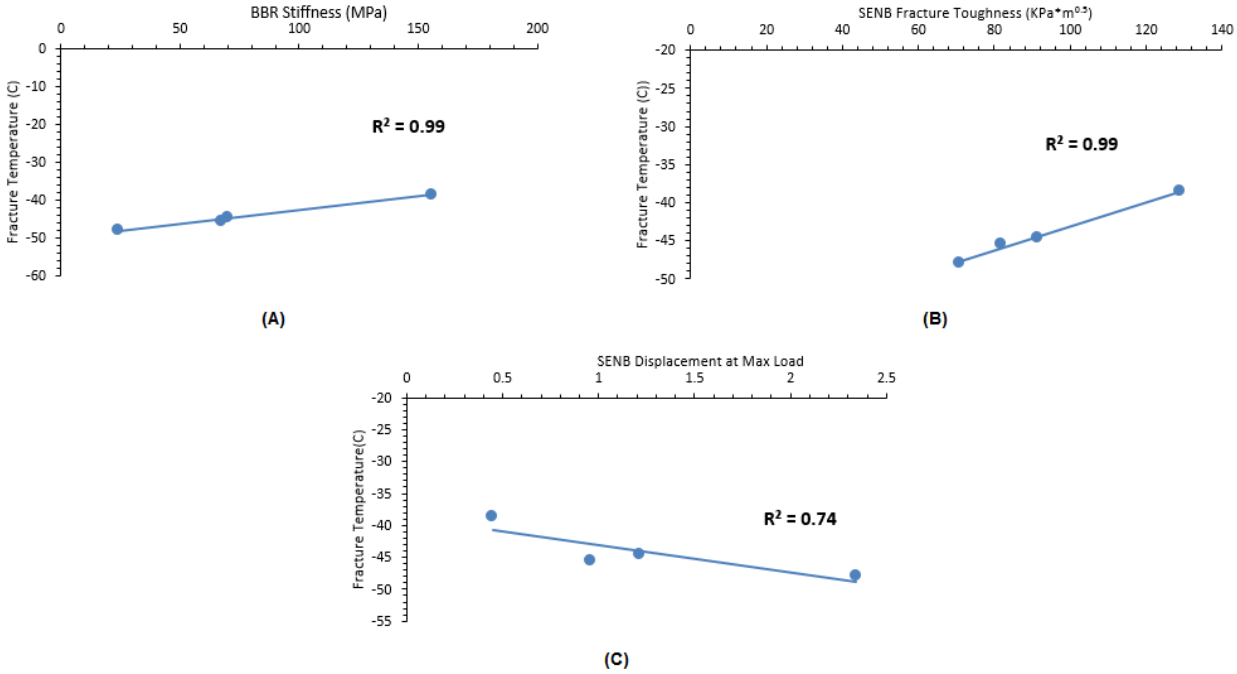

Figure 58-Correlations between mixture and binder parameters. (A) BBR stiffness and fracture temperature. (B) SENB fracture toughness and fracture temperature (C) Displacement at max load and fracture temperature.

Although binder testing results showed promise for measurement of asphalt mixture thermal cracking resistance, Teymourpour conducted a study clearly showing that aggregate structure is one of the most important factors related to low temperature properties [40]. In addition, the limited set of mixture data does not provide conclusive information regarding the relationship between binder and mixture thermal properties. For future specification development the following should be considered: 1) additional mixtures should be tested to ensure a strong binder-mixture fracture property correlations are maintained and 2) mixture design factors should be developed in parallel with binder tests to predict thermo-volumetric properties.

8. Conclusions and Recommendations

8.1 Conclusions

Analysis of the current PG and PG+ specifications was successfully able to identify those binder tests that correspond with fundamental engineering properties using viscoelasticity concepts and mixture performance validation testing. The following conclusions were made based on binder-binder and mixture-mixture correlations:

- ***Mixing Temperatures:*** Selection of mixing temperatures should be related to the quality of coating. Quality of coating is an interactive property based on asphalt binder and aggregate surface chemistry. The modified boiling test, resulting in a coating index, provides an alternative means of measuring the quality of coating. For a given aggregate-binder combination, a minimum coating index can be specified and correlated to a viscosity measurement at 1 RPM. The temperature that corresponds to that viscosity is selected as the minimum mixing temperatures.
- ***High Temperature Rutting Resistance:*** There is no direct relationship between the high temperature PG grading system and the newly proposed MSCR Jnr test method. Of all the high temperature binder tests, Jnr at a stress level of 3.2 kPa showed the highest correlation with flow number (FN) performance testing. A power-law model was fit to the Jnr-FN correlation with relatively high model accuracy. Given the power law relationship between Jnr and flow number, the current AASHTO specifications may not accurately encompass the potential performance of an asphalt pavement layer within each MSCR Jnr spec range.
- ***High Temperature Elasticity:*** Phase angle and MSCR %recovery were shown to have a logical correlation with high variability. Phase angle and %recovery measure different elasticity parameters due to inherent differences between dynamic and creep testing methods. Although each parameter may be used to identify the presence of an elastomeric modifier, there was no correlation between elasticity parameters and flow number testing. Elasticity should not be considered for high temperature specifications in conjunction with $G^*/\sin\delta$ and MSCR Jnr until a logical relationship between

elastomeric modification and improved rutting resistance can be established. Attempts to infer fatigue cracking resistance using high temperature test methods is not technically sound. High temperature elasticity measurements restrict use polymers to a specific class of materials and may limit use of different modifiers that may or may not correlate with resistance to fatigue cracking.

- ***Intermediate Temperature Cracking Resistance:*** An elastic recovery procedure in the DSR has a direct relationship with the elastic recovery AASHTO T 301 procedure. BYET strain at peak stress and yield energy can be used to identify polymer modification types used to meet T&T and ductility state specifications. Although BYET and ER-DSR can be used as alternatives for the current PG+ test methods, there was no direct correlation to IDT fatigue testing. LAS testing showed the highest correlation with IDT fatigue. In fact, the LAS fatigue law model at 2.5% strain was on the same order of magnitude when comparing the number of cycles to failure for the mixture and binder. Additional durability related test methods should be considered in the future to address different modes of fatigue distress; such as crack propagation or reflective cracking.
- ***Low Temperature Thermal Cracking Resistance:*** For the binders tested in this experiment, the physical hardening from 1 to 24 hours occurred at the same rate. Additional conditioning times with a wide variety of binder chemistries and temperatures should be tested to validate this conclusion. DTT and BBR had a direct linear correlation; which validated the conclusions of the SHRP researchers. No correlations existed for BBR and SENB parameters. SENB provides a means to measure an asphalt binder's viscoelastic fracture properties that cannot be captured by the BBR linear viscoelastic stiffness and relaxation m-value. Both SENB and BBR parameters had a high correlation with ATCA fracture temperature. No relationships could observed for the glass transition temperature and coefficients of contraction. Additional mixtures should be tested to validate the fracture temperature correlations. Aggregate properties should be combined with asphalt binder properties to correlate the thermo-volumetric measurements (glass transition temperature and coefficients of thermal contraction).

8.2 Recommendations

Results of this thesis clearly show that current local and state agency PG+ specifications do not necessarily indicate performance of asphalt mixtures. The primary results of current PG+ tests are indication of polymer modification and specifically indication of elastomeric modification. Although polymer modification has been shown to increase performance of asphalt pavements, quantification of increased performance is not reflected by "modifier indication" test methods. At high temperatures, Jnr at 3.2 kPa stress level directly addresses non-recoverable creep deformation that has been shown to indicate rutting resistance. Fatigue cracking damage was found to be a function of both stiffness and elasticity that could be measured by LAS fatigue law analysis. Implementation of a new PG+ performance based specifications allows state and local agencies to directly address implications of polymer modification rather than simply requiring polymer modification.

The following will recommend binder tests should be considered for developing a new performance based specification for each type of pavement distress:

- ***Mixing Temperature for Effective Coating:*** Viscosity is an inadequate measurement of coating. In order to measure the quality of coating, aggregate-binder test methods are necessary. Cuciniello successfully correlated the modified boiling test to the current mixture test method commonly used to measure moisture damage resistance: Tensile Strength Ratio (TSR) [31]. Although the modified boiling test provides a less labor intensive measurement of moisture resistance than the TSR, the TSR is already well established quality control measure. Therefore, implementation of the modified boiling test may not be necessary if HMA suppliers are satisfied controlling moisture resistance with the TSR. Further research is recommended to understand the mixture performance implications of increased asphalt-aggregate coating quality.
- ***Rutting Resistance:*** At high climatic temperatures, the Jnr at 3.2 kPa showed the highest correlation with flow number testing. Jnr specifications can be adjusted based on the anticipated traffic volume during the service life of the pavement, shown in **Table 5**. As outlined in AASHTO M 332, by testing each binder at the climatic temperature rather than specifying PG grade "bumps", an increase in performance can

be reflected in the Jnr corresponding to the temperature actually experienced by the pavement in service.

- ***Fatigue Cracking Resistance:*** LAS testing provides a fundamental way to measure the fatigue life of asphalt binders. Hintz et al found that the most critical aging conditions for onset binder fatigue failure are strain dependent [41]. At low strain levels, aging increases the fatigue life of asphalt binders. At high strain levels, aging decreases the fatigue life. Strain levels experienced in service are most closely related to pavement thickness. Bahia et al recommended specifying LAS N_f at 2.5% strain for pavements thicker than 4.5 inches and a strain level of 5% strain for pavements thinner than 4.5 inches. Both strain level results can be obtained using one LAS test, but the aforementioned strain levels should be given highest consideration depending on the pavement thickness. Based on the work conducted by Hintz, further research is needed to select aging levels to target the most critical condition in service. Aging studies should also be conducted for mixture and binder tests to identify if the implications of aging are comparable.
- ***Thermal Cracking Resistance:*** BBR and SENB tests both showed correlations to ATCA fracture temperature. Although SENB testing can add fracture properties to the linear viscoelastic BBR specifications, it may not be necessary. Tabatabaee, Bahia, Teymourpour and others have showed that qualification of low temperature thermal properties related to field performance is very complex [39, 40]. Factors such as cooling rate, glass transition temperature, coefficients of contraction, binder chemistry, aging and physical hardening of only the asphalt binder can impact thermal cracking resistance. Any attempts to consider all of these factors prior to the mixture and pavement design may prove to be too labor intensive. Instead, mixture performance test methods may provide a more direct way to ensure temperature cracking resistance.

Based on the mixture performance testing, the MSCR and LAS merit the highest consideration for implementation into state agency specifications. Both tests were directly correlated with Flow number and IDT fatigue performance, respectively. Additional mixture-binder testing is recommended; taking into consideration a wide range of mixture

design factors and testing conditions in order to set well defined performance limits for each test.

9. References

[1] D. A. Andrerson, D. W. Christensen, H. U. Bahia and R. Dongre, *Binder Characterization and Evaluation,* Stratigic Highway Research Program, 1994.

[2] T. Kennedy, G. Huber, E. Harrigan, R. Cominsky, C. Hughes, H. Von Quintus and J. Moulthrop, *Superior Performing Asphalt Pavements (Superpave): The Product of the SHRP Asphalt Research Program,* Washington, DC: Strategic Highway Research Program, 1994.

[3] D. W. Christensen and D. A. Anderson, *Chemical-Physical Property Relationships for Asphalt Cements and the Dispersed Polar Fluid Model,* University Park, PA: The Pennsylvania Transportation Institute.

[4] J. D'Angelo, *Superpave Plus Binder Specifications Why do we have them.,* 2nd International Symposium on Binder Rheologly, 2001.

[5] M. Yusuf, A. Nolan, D. Eric, S. Zorn, E. Batten and P. Shirodkar, *Correlation between Multiple Sttress Creep and Recovery (MSCR) Results and Polymer Modification of Binder,* New Jersey Department of Transportation Bureau of Research, 2013.

[6] C. Clopotel, E. Mahmoud and H. Bahia, *Modification of the Elastic Binder Recovery Test and its Relationship to Performance Related Properties of Modified Asphalt Binders,* Washington, D.C.: Transportation Research Board, 2010.

[7] C. Clopotel and H. Bahia, *Importance of Elastic Recovery in the DSR for Binders and Mastics,* Bangkok: International Conference on Road and Airfield Pavement Technology, 2012.

[8] A. Faheem and H. Bahia, *Evaluation of PG Plus Testing Methods by the Asphalt Research Consortium,* Presenation as part of the Asphalt Research Consortium, 2009.

[9] A. Golalipour, *Modification of Multiple Stress Creep and Recovery Test Procedure and Usage in Specification,* Madison: University of Wisconsin Master's Thesis, 2011.

[10] A. Arshadi, *Importance of Asphalt Binder Properties on Rut Resistance of Asphalt Mixtures,* Madison: University of Wisconsin-Madison Master's Thesis, 2013.

[11] Y. Ruan, R. Davison and C. Glover, *An investigation of Asphalt Durability: Relationships Between Ductility and Rheological Properties for Unmodified Asphalts,* London: Petroleum Science and Technology, 2007.

[12] M. Elseifi, *Evaluation of the Ductility Specifications for Strain Asphalt Binder in Louisiana,* Presentation from Louisiana State University, 2009.

[13] U. Isacsson and X. Lu, *Testing and appraisal of polymer modified road bitumens - state of the art,* Stockholm: Materials and Structures, 1995.

[14] P. Kandhal and W. Koehler, *Significant Studies on Asphatl Durability: Pennsylvania Experience,* Washington, D.C: Transportation Research Board, 1985.

[15] J. Goodrich, *Asphalt and Polymer Modified Asphalt Properties Related to teh Performance of Asphalt Concrete Mixes,* Williamsburg: Asphalt Paving Technology, 1988.

[16] J. R. Bension, *New Concepts for Rubberized Asphalts,* Roads and Streets, 1955.

[17] D. J. Tosh and R. Morin, *Building Seals and Sealants,* American Society for Testing Materials, 1976.

[18] Instron, *Toughness and Tenacity Asphalt Test System,* Norwood, 2005.

[19] Modified Asphalt Research Center, *Measuring Asphalt Binder Yield Energy and Elastic Recovery Using the Dynamic Shear Rheometer,* Madison: Not currently published.

[20] H. Tabatabaee, C. Clopotel, A. Arshadi and H. Bahia, *Critical Problems with Using the Asphalt Ductility Test as a Performance Index for Modified Binders,* Washington, D.C.: Transportation Research Board, 2013.

[21] R. Garba, *Permanent Deformation Properties of Asphalt Concrete Mixtures,* Oslo: Norwegian University of Science and Technology, 2002.

[22] J. Bukowski, J. Youtcheff and T. Harman, "The Multiple Stress Creep Recovery (MSCR) Procedure," in *Federal Highway Administration,* 2011.

[23] A. Hanz, *MSCR Implementation and Impacts on Asphalt Binder Grading,* Minneapolis: Presentation at MAPA Contractors' workshop, 2015.

[24] C. Johnson, *Estimating Asphalt Binder Fatigue Resistance using an Accelerated Test Method,* Madison: University of Wisconsin, 2010.

[25] C. Hintz, *Understanding Mechanisms Leading to Asphalt Binder Fatigue,* Madison: University of Wisconsin-Madison Phd dissertation, 2012.

[26] Pavement Interactive, *Top-Down Cracking,* http://www.pavementinteractive.org/article/top-down-cracking/, 2015.

[27] C. M. Monismith, J. A. Epps, D. A. Kasianchuk and D. B. Mclean, *Asphalt Mixture Behavior in Repeated Flexure,* Institute of Transportation and Traffic Engineering, 1970.

[28] H. Bahia, H. Tabatabaee and A. Faheem, *Field Evaluation of Wisconsin Modified Binder Selection Guidelines-Phase II,* Madison: Wisconsin Highway Research Program, 2013.

[29] W. Buttlar, R. Roque and D. Hiltunen, *Prediction of Thermal Cracking with TCMDOEL,* ASCE Press, 2009.

[30] M. Marasteanu, W. Buttlar, H. Bahia and C. Williams, *Investigation of Low Temperature Cracking in Asphalt Pavements,* Minneapolis: Minnesota Department of Transportation Research Services, 2012.

[31] G. Cuchiniello, "Developing a Boiling Test to Quantify Effective Bonding of Aggregates with Binder," University of Wisconsin Madison Masters Thesis, Madison.

[32] R. C. West , D. E. Watson, P. A. Turner and J. R. Casola, "NCHRP Report 648 Mxing and Compaction Temperatures of ASphalt Binders in Hot Mix Asphalt," Transportation Research Board, Washington, D.C., 2010.

[33] R. Bonaquist, *Evaluation of Flow Number (Fn) as a Discriminating HMA Mixture Property,* Wisconsin Highway Research Program, 2012.

[34] C. S. Clopotel, E. Mahmoud and H. Bahia, "Modification of the Elastic Recovery Test and its Relationship to Performance Related Properties of Modified Asphalt Binders," in *Transportation Research Board,* Washington D.C..

[35] J. D'Angelo, "New High-Temperature Binder Specification Using Mulistress Creep and Recovery," in *Transportation Research Board,* Washington D.C., 2010.

[36] R. Y. Kim, *Modeling of Asphalt Concrete,* American Society of Civil Engineers , 2009.

[37] A. Arshadi, *Development of an Image-based Multi-Scale Finite Element Approach to Predict Fatigue Damage in Asphalt Mixtures,* Madison: Phd Disertation at the University of Wisconsin, 2015.

[38] S. Kose, M. Guler, H. Bahia and E. Masad, *Distribution of Strains Within Hot-Mix Asphalt Binders,* Washington DC: Transportation Research Board, 2007.

[39] H. Tabatabaee, *Critical Behavior of Asphalt Mixtures Undergoing Glass Transition and Physical Hardening,* Madison: University of Wisconsin-Madison Phd Dissertation, 2012.

[40] P. Teymourpour and H. Bahia, *Effects of Binder Modification on Aggregate Structure and Thermo Volumetric Properties of Asphalt mixtures,* Madison: Transportation Research Board, 2014.

[41] C. Hintz, R. Velasquez, Z. Li and H. Bahia, *Effect of Oxidative Aging on Binder Fatigue Performannce,* Pennsylvania: Asphalt Paving Technology, 2011.

[42] J. Zhang and J. Pei, "Research on Flow Number of Asphalt Mixture under High Temperature and Heavy Load," in *ASCE*, 2011.

[43] D. W. Christensen, "Airfield Asphalt Pavement Technology Program Procject 04-02: PG Binder Grade Selection for Airfield Pavements," *Advanced Asphalt Technologies,* 2008.

[44] L. Advanced Asphalt Technologies, "A Manual for Design of Hot Mix Asphalt with Commentary, NCHRP Preport 673," National Cooperative Highway Research Program, Washington D.C., 2011.