Report No. UT-10.08

DEVELOPMENT OF METHODS TO CONTROL COLD TEMPERATURE AND FATIGUE CRACKING FOR ASPHALT MIXTURES

FINAL REPORT

Prepared For:

Utah Department of Transportation
Research Division

Submitted By:

University of Utah
Department of Civil & Environmental Engineering

Authored By:

Pedro Romero, Ph.D., P.E.
Chun Hsing Ho, Ph.D.
Kevin VanFrank, P.E.

May 2011

RESEARCH

UDOT CONNECTING COMMUNITIES

Utah Department of Transportation-Research Division
4501 South 2700 West, Box 148410 - Salt Lake City, Utah 84114-8410

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Pavement distresses caused by low and intermediate temperatures are a significant source of problems for highway agencies. While many tests have been developed to address this type of distress, few of them are considered practical for day to day operations. This report presents a methodology for controlling low temperature properties of asphalt mixtures by using the Bending Beam Rheometer (BBR). Through a rigorous statistical analysis, the number of replicate samples needed for valid conclusions was determined so that the test can be used for quality control/quality assurance during asphalt construction. Viscoelastic modeling was employed to evaluate the effect of aggregate size. The prediction of thermal stress is also presented. It was concluded that the BBR is a viable tool to determine low temperature properties of asphalt mixtures. Finally, a specification was drafted and an example is shown where the proposed methodology was used to control the quality of a field project.</td></tr>
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Conversion Factors

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	millimeters squared	mm^2
ft^2	square feet	0.093	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometers squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3
	NOTE: Volumes greater than 1000 L shall be shown in m^3.			
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit	(F-32)/1.8	Celsius	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
m^2	meters squared	1.196	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	kilometers squared	0.386	square miles	mi^2
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celsius	1.8C+32	Fahrenheit	°F

*SI is the symbol for the International System of Measurement

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Executive Summary

For several years, the Utah Department of Transportation (UDOT) has been implementing the use of mechanical tests such as the Hamburg Wheel Tracking Device (WTD) and the Asphalt Mixture Performance Tests (AMPT) to screen asphalt mixtures that might not have adequate high temperature performance due to either mixture instability (i.e., not able to carry its loads) or incompatibility between components. The implementation of the Hamburg WTD as a screening test and the AMPT as a performance test has significantly benefited the high temperature performance of asphalt pavements; however, it has not addressed the low and intermediate temperature performance reflected as thermal and fatigue cracking. Throughout the state of Utah there are cases of premature pavement failures caused by inadequate intermediate and low temperature properties of the asphalt mixture.

Pavement distresses caused by low and intermediate temperatures are a significant source of problems for highway agencies. While there are several tests that have been developed to address this type of distress, few of them are considered practical for day to day operations. In fact no low temperature test has been adopted by any highway agency. This research was initiated to address this issue. A methodology was developed for controlling low temperature failures of asphalt mixtures using the Bending Beam Rheometer (BBR), a device currently used to characterize asphalt binders. The familiarity and availability of this equipment at many construction materials laboratories make implementation and use of the proposed tests more likely.

A series of experiments using the BBR were undertaken to evaluate the low temperature properties of asphalt mixture beams. Through statistical analysis, the number of sample replicates was determined to give UDOT and other highway agencies an informative guidance of how to prepare samples for quality control/quality assurance (QC/QA) during asphalt construction. Viscoelastic modeling was employed to evaluate the effect of aggregate size on the thermal properties of asphalt mixtures. The prediction of pavement temperatures using numerical analysis methods was performed to calculate temperature gradients in an asphalt pavement system.

The conclusion of this work is that the BBR is a viable tool that can be used to control pavement performance at low temperatures. Finally, a draft specification is presented along with examples to demonstrate how this work can be easily adopted to facilitate QC/QA operations in asphalt construction.

While the validity of the proposed approach was demonstrated, it is recommended that data be obtained from field projects so that actual limits can be placed on the low temperature properties of asphalt mixtures. Such an approach can help establish a balance between the high and low temperature properties of asphalt mixtures thus ensuring longer lasting pavements.

1.0 Introduction

In 1992 the Strategic Highway Research Program (SHRP) introduced a performance related grading for asphalt binders called the Performance Grading (PG) system. The PG system was developed in order to control the inconsistency in quality of asphalt binder supply in the US. Unfortunately, a similar performance-related grading system was not implemented for hot-mix asphalt due to lack of easy-to-use testing methods. Thus, hot-mix asphalt continues to be evaluated today based only on volumetric limits.

For several years, the Utah Department of Transportation (UDOT) has been implementing the use of mechanical tests such as the Hamburg Wheel Tracking Device (WTD) and the Asphalt Mixture Performance Tests (AMPT) to screen asphalt mixtures that might not have adequate high-temperature performance due to either mixture instability (i.e., not able to carry its loads) or incompatibility between components. An informal investigation done by the University of Utah in 2006 concluded that, as a result of Hamburg WTD specifications along with recent efforts in adopting the AMPT, materials being placed on the roads in Utah today have low tendency for rutting (i.e., stable mixes) or moisture damage (i.e., compatible components).

While the implementation of the Hamburg WTD as a screening test and the AMPT as a performance test has significantly benefited the high temperature performance of asphalt pavements, it has not addressed the low and intermediate temperature performance reflected as thermal and fatigue cracking. Furthermore, as will be shown in Section 1.1, given the accelerated load applications of mechanical testing used during design, brittle mixes are being placed on the roads.

If such problems were not enough, the asphalt binder supply is shrinking and hence becoming more expensive. This has encouraged the use of additives such as recycled asphalt pavements (RAP), roof shingles, etc., and different modifiers (e.g., Aggcoat) into the hot-mix. Unfortunately, to date, no practical test is available to characterize the impact of these additives on hot-mix asphalt low and intermediate temperature performance. For example, even if a good quality PG binder is selected for a given pavement, it does not necessarily mean that the hot-mix produced using that asphalt binder will perform well because RAP or other additives are also being added and change the behavior of the material. What is needed is an easy-to-use test that can characterize hot-mix as produced in the lab or the field to control distresses such as fatigue and low temperature thermal cracking which are directly affected by hot-mix additives.

1.1 Premature Pavement Failures

As discussed in the previous section, the current practice of using accelerated mechanical tests for mixture design has resulted in the unintended consequence of producing asphalt mixtures that are brittle or have questionable performance. The result of this practice has been road surfaces that show no signs of permanent deformation but are susceptible to severe cracking. The cracking leads to premature failure from water intrusion and structural deterioration resulting in costly maintenance.

As an example of this condition, the off ramp from Route 201 westbound to 3200 West in Salt Lake City, Utah was investigated as part of this work (Figure 1). This ramp is a recent alignment constructed less than 5 years ago and expected to last many years. However, as shown in Figure 2, it shows a significant amount of cracking that will soon require maintenance.

Figure 1: Location of Distressed Pavement

It is evident from a survey of the road performed by the researchers, as well as the pictures presented in Figure 2, that the cracks are not the result of a pavement structural deficiency. The cracks do not occur under the wheel path as would be expected in fatigue cracking and they do not occur at the joint to suggest faulty construction. Instead, the pattern suggests that the cracks are the result of a brittle mixture being placed on this road. Obviously, such failure was not the intent of the mixture designers when this road was built.

(a) Ramp facing north

(b) Close up of road showing cracks

(c) Ramp facing north-west

(d) Cracks on the road

Figure 2: Pictures from SR 201 and 3200 West

Pavement failure from cracking is becoming more common not only in Utah but across North America. Numerous studies have found that the main form of deterioration in asphalt pavements within freeze areas of the US and Canada is cracking (Marasteanu et al., 2007). Highway agencies are struggling to seek an effective way to facilitate improvements of low temperature performance in asphalt pavements. As will be discussed in Section 2.1, while there are some materials testing methods that have been used to predict the low temperature properties of asphalt mixtures, none of these methods are practical for the quality control/quality assurance activities in the field. At present, no highway agency in the world has adopted any mixture test to control the quality of asphalt mixtures based on their potential for thermal-induced cracking.

A research work, therefore, is needed to develop a methodology to determine the cracking resistance of asphalt mixtures. This methodology should be accurate yet simple enough that it can be adopted by state highway agencies and producers. Such methodology will aid material engineers to find the balance between high and low temperature mix properties, thus reducing the possibility of premature failures such as the one shown in this section.

1.2 Proposed Solution

During the Strategic Highway Research Program (SHRP) both intermediate and low temperature cracking properties of asphalt mixtures were investigated. The researchers concluded that "*performance is mixture dependent and cannot be controlled from binder properties alone*" [Lytton et al., 1993] and that "*lower stiffness materials take longer to crack and exhibited a lower rate of cracking than higher stiffness material*" [Tayebali, et al., 1994]. A bending beam device was used by the University of California at Berkeley to develop performance predictions for fatigue cracking [Tayebali, et al., 1994]. A diametral indirect tensile test (IDT) was used by Penn State University to develop performance predictions for low temperature cracking [Lytton et al, 1993]. While both of these tests have been successfully used to predict, and thus control, intermediate and low temperature distresses [Roque et al., 1995, Roque and Hiltunen, 1994, Romero et al.

2000, Epps, 1999], they are time consuming, require large quantities of materials, take extensive preparation, and need a sophisticated analysis to interpret the results. These issues make them impractical for day-to-day use and quality control.

Work at the University of Minnesota by Zofka [2007] showed that the modulus or stiffness of asphalt mixture can be obtained from thin mixture beams tested using a slightly modified BBR commonly used for asphalt testing. The researchers at Minnesota were able to relate the mixture properties to the low temperature properties of the asphalt binder contained in the asphalt mixture without the need of a recovery and extraction process. Based on the wealth of research available as of today, it is clear that using a BBR to measure modulus or stiffness of thin-mixture beams might be a good method to determine some of the intermediate and low temperature properties and perhaps provide a practical method to control the production of hard brittle mixes. Furthermore, given the fact that many laboratories already own and operate a BBR and that only a small amount of material needs to be tested to obtain mixture properties, it should be evident that this test has many advantages and can overcome the adoption issues listed earlier.

However, some questions still exist regarding the applicability of testing these small beams to the global properties of asphalt mixtures in general. This report seeks to address such concerns by evaluating the BBR in a controlled environment and demonstrate its ability to predict low temperature properties of asphalt mixtures.

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2.0 Literature Review

The evaluation of asphalt mixtures at low temperature has been the subject of research for many years. As computer-controlled systems have advanced, so has the ability to apply forces and test asphalt mixture to determine its low temperature properties. This section provides some background on the testing of asphalt mixtures at low temperatures.

2.1 Methods to Predict Low Temperature Properties of Asphalt Materials

Currently there are several established testing methods used for the prediction of thermal stresses of asphalt materials at low temperatures. Among the most common ones are the Indirect Tensile Test (IDT) specified in the **American Association of State Highway and Transportation Officials (**AASHTO) standard T322 [AASHTO, 2009], or the American Society of Testing and Materials (ASTM) D6931-07 Standard Test Method for Indirect Tensile (IDT) Strength of Bituminous Mixtures [ASTM, 2008], the Thermal Stress Restraint Specimen Test (TSRST), specified in the AASHTO standard TP10 [AASHTO, 2009], and the Bending Beam Rheometer (BBR), specified in the AASHTO standard T313 [AASHTO, 2009], or ASTM D6648-08 Standard Test Method for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR) [ASTM, 2008].

2.1.1 Indirect Tensile Test

The IDT (AASHTO T322/ASTM D6931-07) is perhaps the most commonly used test to characterize thermal cracking properties of asphalt mixtures (Figure 3). In this test, a cylindrical sample is placed on its side in a temperature-controlled chamber where a vertical compressive load is applied along the top edge of a sample creating a zone of tension along the horizontal axis. The horizontal deformations are measured by linear variable differential transformers (LVDT) mounted on the specimen and used to determine the material's creep compliance. A number of papers have evaluated the tensile behavior of asphalt mixtures at low temperatures using IDT tests, this includes work by Buttlar and Roque [1994], Christensen and Bonaquist [2004], Christensen and Mehta [1998], Christensen [2004], and Kim [2002]; just to name a few. The creep compliance data obtained from the IDT are converted to relaxation modulus using a Laplace Transformation. The relaxation modulus is then used to derive the thermal stresses of asphalt concrete from which the critical cracking temperature can be obtained. However, the procedures of sample preparations and material testing are time-consuming, making the IDT impractical for routine day-to-day quality control applications.

Figure 3: Indirect Tensile Test Setup

2.1.2 Temperature Stress Restrained Specimen Test

The TSRST method (AASHTO TP10) has also been used to evaluate thermal cracking properties of asphalt pavements (Figure 4). In this test, a beam-shaped asphalt mix specimen is cooled to a temperature of 5°C and held for one hour to establish thermal equilibrium. After an hour, the test is performed by cooling the specimen at a specified cooling rate while restraining it from contracting. As the material is cooled, the thermal-induced tensile stresses will develop in the specimen. When the tensile stresses exceed the tensile strength of the mixture, a crack will develop. The cooling rate can be selected to represent typical cooling rates seen in the field. However, practical limitations usually require a much faster rate. Jung and Vinson [1993] stated that the cooling rate significantly affected the experimental measurements of TSRST and the determination of fracture temperature. Just like the previous test, the operational procedures of TSRST are time-consuming. For example, the specimen needs to be glued to the stand for at least 24 hours until the epoxy is cured, then the test itself takes over 5 hours. The results are valid assuming failure does not occur at the edges where the epoxy creates large stress risers. Generally, sample preparations along with the processes of cooling rates with the corresponding fracture temperatures are too time-consuming and impractical for highway agencies to execute day-to-day QC/QA operations.

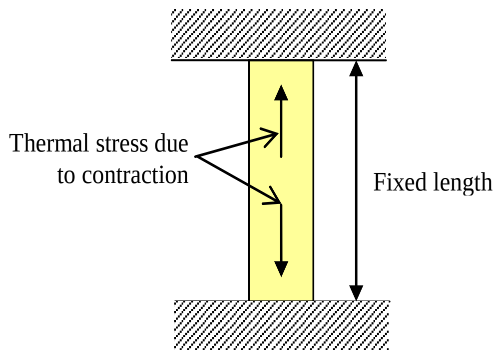

Figure 4: Tensile Stress Restrained Specimen Test Setup

2.1.3 Bending Beam Rheometer

The BBR as described in the AASHTO T313/ASTM D6648 is used to perform tests on beams of asphalt binder conditioned at a desired temperature (Figure 5). Based on the elastic solution for a simply-supported beam and the creep compliance behavior, the time-dependent deflection is measured from which the flexural creep stiffness S(t) and stress relaxation capacity "*m*" of asphalt binders are determined. Both S(t) and *m* values are directly used to control the thermal cracking resistance of asphalt binders (Bahia and Anderson [1995]). The BBR is a creep load test, like the IDT, and has been adopted by many highway agencies to evaluate the thermal cracking resistance of asphalt pavements. As currently used, the data measured from the BBR tests are based on asphalt binders and do not consider the effect of aggregates, the differences between mixtures, or the additions of recycled asphalt pavements (RAP), all factors known to significantly affect the performance of asphalt pavements.

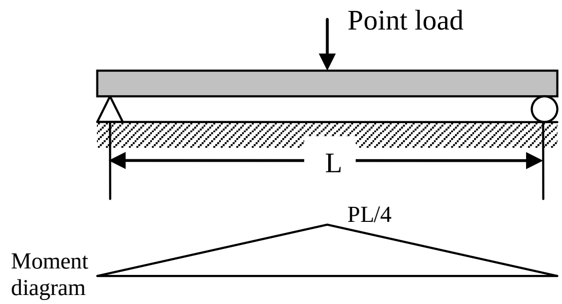

Figure 5: Bending Beam Rheometer Setup

Research by Zofka et al. [2005, 2007, 2008a, 2008b] replaced asphalt binder beams with small asphalt mixture beams (12.7 mm x 6.35 mm x 127 mm, width x thickness x length) (Figure 6) to evaluate low temperature cracking properties of asphalt mixtures using the BBR instrument. The test results presented in those documents showed that the compliance curve predictions derived from the BBR had good correlation with the ones obtained from the IDT. Their work which has resulted in a draft AASHTO testing specification [Marasteanu et al. (2009)] has shown that it is reasonable to consider the BBR for practical estimation of mixture creep compliance and low temperature properties of asphalt mixtures.

Figure 6: Picture of asphalt mixture beams for the BBR tests

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3.0 Research Objective

The need for a practical yet accurate test that can measure the low temperature properties of asphalt concrete is well established. While many different tests procedures have been proposed in the past, the potential benefits of testing smaller samples (cheaper equipment, less material, faster conditioning, easier availability for QC, etc.) are well recognized, so it is of primary importance to highway agencies to consider an experiment that would develop the ranges for which such testing is valid. Zofka et al. [2005, 2007, 2008a] focused on using BBR tests to theoretically evaluate the tensile properties of asphalt mixtures. However, using such small asphalt mixture beams as a material testing specification is still being evaluated and has raised some concerns. For example, how many asphalt mixture beams are considered sufficient to give a valid result for the BBR test? To address the issue, this research will characterize material properties using small asphalt mixture beams and evaluate the corresponding statistical factors such that the number of replicates needed for an unbiased result can be determined. Based on the intended use of the data (e.g., quality control, spot checks, pay factors, etc.) agreed by both highway agency and contractor, this given number of replicates will be used to evaluate whether the quality of asphalt mixtures placed in the roads comes from the same population as laboratory results, thus ensuring that the requirements to produce a durable material are met.

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4.0 Procedures

Mechanical testing of asphalt mixtures is done 1- to characterize the material, 2- to control its manufacture, and 3- to determine the properties which are decisive for its application [Nijboer, 1948]. The quest for a simple, yet accurate mechanical test has been the subject of discussion since asphalt mixtures have been used in roads. While the equipment might have changed, the objective remains the same: simple, accurate, quick, and cheap. In following with those objectives, the issue of testing smaller and smaller samples has been discussed over the years.

As far back as 1997, Ron Reese from the California Department of Transportation (CalTrans) presented his work in which small beam samples of asphalt mixtures (3 mm x 12.5 mm x 44 mm) were tested in torsion using the dynamic shear rheometer (DSR) to evaluate the potential for fatigue failure. He compared the results to four-point beam fatigue testing. The conclusion from this work was that: "*DSR testing of mix slices yields fatigue data comparable to other mix fatigue tests*" [Reese, 1997]. A similar approach was presented by Reinke, who show good correlations ($R^2 > 90\%$) between the results from small samples (50-mm x 12-mm x 10-mm) tested using the DSR and field performance in rutting at MinnRoad and the Federal Highway Administration's Accelerated Loading Facility (ALF). His conclusion was that: "*…DSR dynamic creep tests correlated extremely well to rutting behavior of the mixes in the field*." [Reinke and Glidden, 2004]. As far as low temperature testing is concerned, Zofka and Marasteanu used small asphalt mixture beams (12.7 mm x 6.35 mm x 127 mm) and tested them in the BBR. In one project they tested 20 different asphalt mixtures and used the creep compliance curves predicted from the BBR as input to the TCModel. Their conclusion was that: "*the proposed relations led to prediction of cracking similar to the cracking measured by IDT*." [Zofka, et al., 2008a].

These studies, and possibly others, have shown that small asphalt mixture samples can be used to obtain mechanical properties that are representative of mixture behavior. However, the asphalt mixture community, as a whole, has been dismissive of using such small samples to obtain global mixture properties. The two main criticisms are: 1- the thickness of the beam is smaller than the maximum aggregate size, thus a single aggregate particle can affect the results of the test (see Figure 6), and 2- such small samples cannot represent the overall property of the mix. However, to date, there is no evidence in the literature of any control experiment designed to evaluate such criticism and determine the effect of aggregate size on test results for such specimens.

It is the intention of the researchers to demonstrate, through a methodical and scientific-based approach, the effect of aggregate size on the results of small asphalt mixture beam samples (12.7 mm x 6.35 mm x 127 mm) tested using the BBR. While the approach presented is conceptually valid at any temperature, tests at low temperatures have a better chance for success. The mismatch between binder and aggregate stiffness diminishes as the temperature decreases so that the bulk properties of the composite asphalt mixtures become less dependent on the size and spatial distribution of the aggregate particles [Weissman et al., 1999; Romero and Masad, 2001].

4.1 Representative Volume Element

It is well established that in composite materials having spatial disorder with no microstructural periodicity (e.g., asphalt mixtures) the determination of any stress, strain, or energy field is actually an average value over the given domain [Du and Ostoja-Starzewski, 2006; Ostoja-Starewski, 2006]. This means that the stress or strain recorded as part of any test is not the actual value experienced by the components at the microscale but rather an average or bulk property. The question then becomes whether this ‘averaging’ is carried out over a mesoscale that includes all heterogeneities or whether it is affected by localized phenomenon. The answer to the question depends both on the function being evaluated (i.e., the specific property) and the morphology (i.e., shape and geometry) of the specimen. The size of the domain that satisfies the averaging requirements is known as the representative volume element (RVE) [Hashin, 1963].

Most of the work to determine the RVE has been done by increasing the size of the analysis volume until some statistical stability is reached. As an example, consider Figure 7 which shows an X-ray tomography picture of an asphalt mixture specimen. The concentric circles with varying radius represent different size areas. Depending on the location of the circle, the function that represents the percent aggregate within the area can vary between 0% (no aggregate) and 100% (all aggregate); thus, there might be large fluctuations in the function indicating that some areas contain mostly aggregates while others contain mostly asphalt mastic and voids. However, as the radius increases, the fluctuations gradually attenuate. The radius at which the function stabilizes is the minimum size needed to overcome the domain of small scale heterogeneity. Such domain is the RVE (representative area element in the example). In general, the RVE ensures a given accuracy of the estimated property obtained by spatial averaging of the stress, strain, or energy fields in the given domain; alternatively, the use of smaller volumes can be compensated for by averaging over several realizations of the microstructure to get the same accuracy, provided no bias is introduced in the estimation by edge effects generated by the boundary conditions [Kanit et al., 2003].

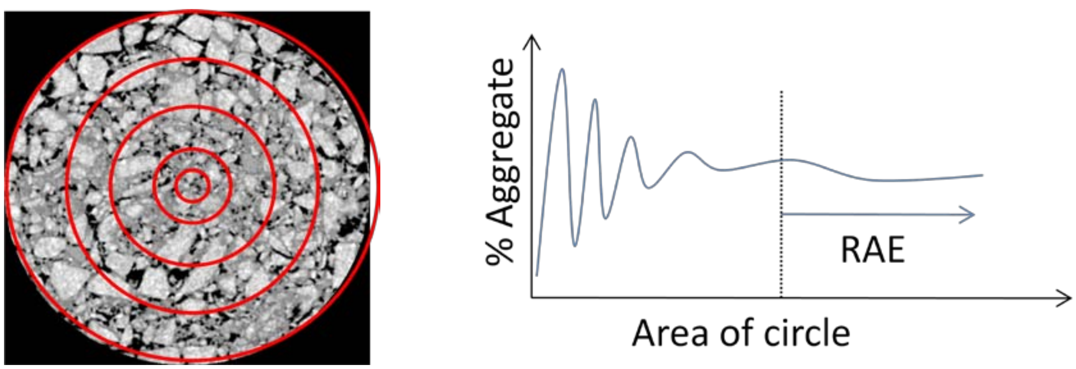


Figure 7: Schematic representation of the RVE Concept. Representative Area Element (RAE) used in 2-D example [after Romero and Masad, 2001]

In 2001, Romero and Masad published their work on the determination of the representative volume element of asphalt mixtures tested using the Super Shear Tester (SST). Their work used a slightly different approach as the one described in Figure 5. In their work, the volume was kept the same (50-mm height, 150-mm diameter) while the size of the large particles was decreased by reducing the nominal maximum aggregate size of the mix (NMAS) from 37.5-mm to 12.5-mm [Romero and Masad, 2001]. The work proposed in this document intends to follow a similar approach but carried out to even smaller sizes.

4.2 Aggregate Gradation

In the approach followed by Romero and Masad, the decrease in NMAS was obtained by eliminating the large aggregate sizes and keeping the same aggregate gradation for sizes below the 4.74-mm sieve. While this resulted in an asphalt mixture with no large particles, it did not consider the structure of the material. As the NMAS decreased, there was a net increase in particle specific surface area that required an increase in binder content to maintain the same air void content. Such changes created a confounding effect since the behavior of smaller NMAS mixtures might have been influenced by the increase in binder content.

In recognizing the role that binder content and mixture volumetrics play in the response of the material, this work intends to consider these volumetric parameters when determining the RVE of asphalt mixtures.

This work intends to ‘scale down’ the gradation by proportionally changing the percent of material retained on each sieve so that the shape of each gradation curve with respect to the maximum density line is maintained. Conceptually similar shape implies similar aggregate structure. At the same time, the approach intends to match a prescribed set of volumetric parameters. As the NMAS is reduced, the amount of fines increases. This leads to an increase in specific surface area, a higher need for binder, and a corresponding increase in the voids in the mineral aggregate (VMA) [The Asphalt Institute, 1995]. The volumetric requirement is selected such that increases in VMA and increases in binder content result in constant voids filled with asphalt (VFA).

As shown in work by Campen et al. [1965] and later by Radovskiy [2003], there is some relation between the VFA and the effective film thickness (generally speaking, VFA requirements are placed to ensure enough asphalt is used to coat the particles). Thus, similar VFA ensures equivalent binder-aggregate ratios. Furthermore, it has been shown that, when properly calculated, film thickness can relate to the mechanical behavior of asphalt mixtures [Nukunya, et al., 2001]. Based on all of these concepts, mixtures with similar shape in their gradation curves and the same VFA should have equivalent response, in terms of binder-aggregate interaction, when subjected to load.

A 12.5-mm NMAS mixture was considered the ‘standard’ mix in this work. This mixture was selected because: 1- it is an actual mix used on SR-210 in Salt Lake City, UT; 2- it was designed using a single source, low absorption aggregate, making it easier to modify; 3- at 75 gyrations it ensures a sufficient amount of binder; and 4- it uses a relatively soft, PG 64-34 binder that will ensure large movements when tested at relatively high temperatures.

Once the 12.5-mm NMAS mixture was verified, two other mixtures were developed, a 9.5-mm NMAS and a 4.75-mm NMAS. While the general shape of the gradation was kept, the individual sieves were adjusted to match the VFA. The final properties of all mixtures are shown in Table 1. The gradations and the maximum density lines for each of the mixtures are shown in Figure 8.

Table 1: Mix Properties

NMAS		**12.5-mm**	**9.5-mm**	**4.75-mm**
Gradation	**Sieve Size, mm**			
	19	100		
	12.5	93	100	
	9.5	83	97.8	100
	4.75	60	67.5	93.4
	2.36	38	45.2	60.4
	1.18	26	32.2	37.3
	0.300	14	16.8	19.8
	0.075	6.2	6.6	9.0
Binder Grade	**PG**	64-34		
Design gyrations	**Ndes**	75		
Binder Content, %	**Pb**	5.2	6.2	6.5
Air Voids, %	**Va**	3.6	3.6	3.6
VMA, %		14.6	15.9	17.1
VFA, %		78.3	78.7	78.8
Dust Proportion		1.3	1.2	1.5
Aggregate Absorption		0.43		
Max. Specific gravity	**Gmm**	2.431	2.402	2.391

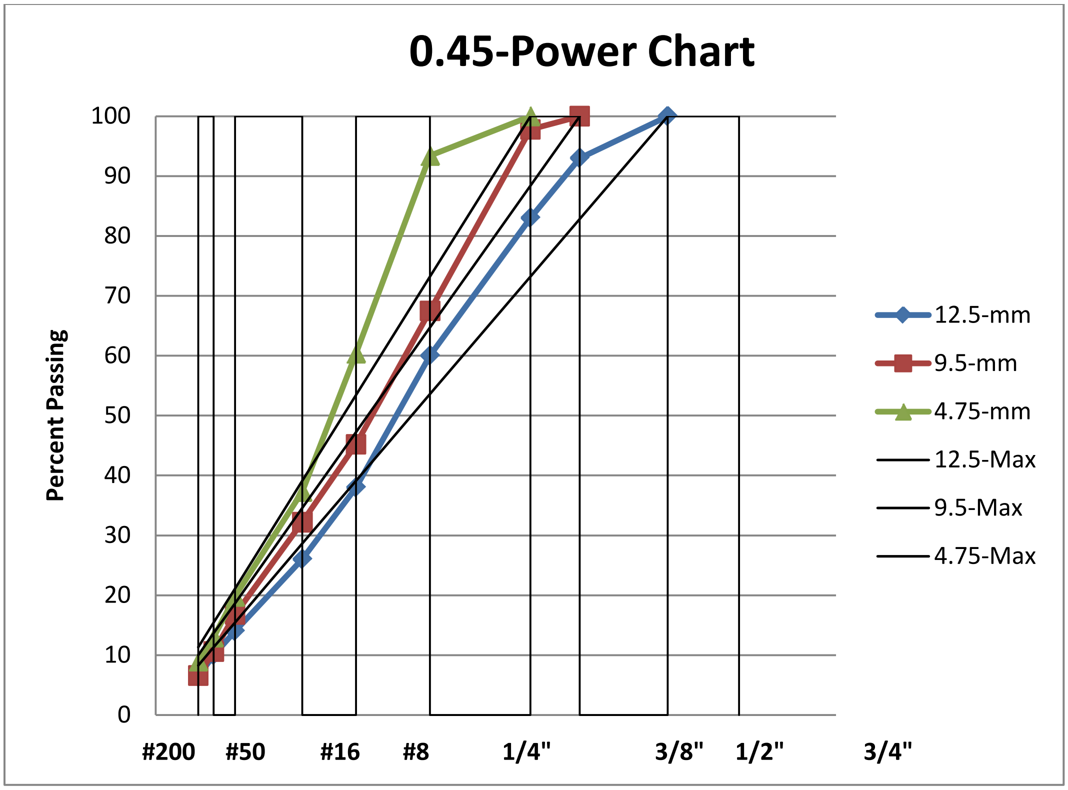


Figure 8: Gradation and Maximum Density line

4.3 Air Void Content and Distribution

The air void content has been identified as a critical parameter in the response of asphalt mixtures. However, there is contradictory information regarding the effect of air void content and distribution on the low temperature properties of asphalt mixtures. Theoretical work by Zofka using the Hirsch model showed that the low temperature modulus is not affected by air voids between the ranges of 0% and 8% [Zofka et al., 2008b]. Unfortunately, this work was only theoretical. Further work is presented in Section 6.2 of this report to validate these results experimentally.

However, from a practical point of view, the determination of air voids is not easily done in the small samples used for this research. AASHTO T-166, which is the common method used to determine the density of compacted bituminous mixtures, specifically requires that the thickness of the specimen be at least one-and-one-half times the maximum size of the aggregate. Further requirements include a weighing device readable to 0.1 percent of the sample mass. With beams weighing less than 25-grams, this means that the balance used should be readable to 0.02 grams; a few drops of water weigh more than 0.2 grams, so results using AASHTO T-166 methods would be highly questionable.

To overcome the limitations of AASHTO T-166, the relative density of the prismatic beams was calculated by measuring the mass and dividing it by the volume. This approach neglects the surface voids; however, work by Masad et al. has shown that the air void distribution in Superpave gyratory compacted specimens exhibits a bathtub shape, whereby larger voids are present in the top and bottom parts of the specimen [Masad et al., 2002]. Based on Masad's work, it can be shown that the error incurred by neglecting the surface voids is constant, so that comparisons of density based on mass over volume are reasonable.

4.4 Statistical Analysis

The determination of the RVE is based on the statistical analysis of variability. If the volume evaluated represents an RVE, then the variability should be constant. In other words, the analysis is based on the variance and not on the mean. Those gradations with NMAS smaller than the point where the variance starts to increase contain an RVE. This is shown conceptually on Figure 9.

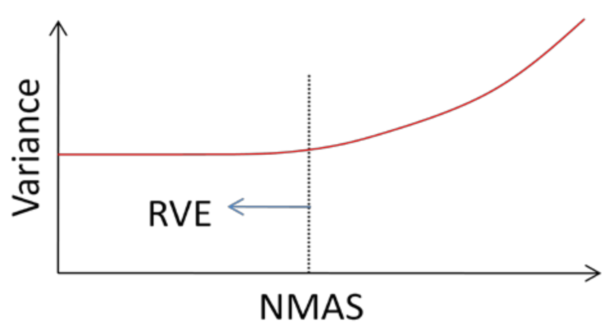


Figure 9: Schematic Representation of the RVE determination

Such an approach brings the issue of how many replicates are needed to obtain a reliable value. An approach was used based on the central limit theorem. The central limit theorem states that as the number of replicate testing approaches infinity, the sampling mean approaches the population mean with normal distribution. In other words, as the number of tests replicates increases, the mean value of the mixture modulus obtained from these ‘n’ observations using BBR testing becomes the ‘actual’ value of the actual mixture modulus. Once such value is determined, it can be compared to the modulus from another mixture. Such comparison leads to the hypothesis that both means are the same. The ‘reliability’ and ‘power’ of the hypothesis testing can be quantified. This is the approach used by Romero and Masad [2001]. Such will be explored in detail in Section 5.3.

An alternative approach consists in evaluating the variance calculated from an increased number of observations. Based on the central limit theory, as the number of replicates increases, the distribution of results approaches a constant. For example, tests are run on 15 replicates. From these data, the variance based on samples 1 through 5 (S_{1-5}) is calculated and compared to the variance based on samples 1 through 10 (S_{1-10}) and the variance based on samples 1 through 15 (S_{1-15}). Using a Bartlett Distribution we test for equal variances; if $S_{1-5} \neq S_{1-10}$ then we know we should tests more than 5 samples. If $S_{1-10} = S_{1-15}$ then we know we should test at least 10 samples.

4.5 Testing

The testing of asphalt mixture beams was done in the following way:

- An asphalt concrete sample (150 mm in diameter and 110 mm in height) was compacted using the Superpave Gyratory Compactor (SGC)
- Both sides of the compacted sample were cut using a lapidary saw
- The sample was cut using a tile saw and trimmed into a block
- The block was cut into several flat slabs using a tile saw
- The flat slabs were cut into thin beams suitable for the BBR test.

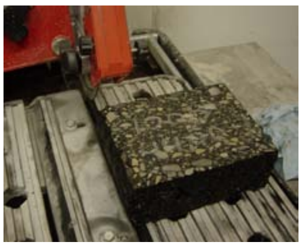
The entire process of sample preparation is illustrated in Figure 10.

1. Sample was made using Superpave Gyratory Compactor

2. Sample was placed on a lapidary saw

3. Sample was cut on one side

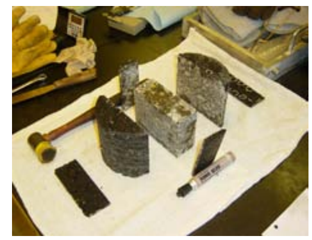
4. Sample was turned and cut on the other side



5. Place the sample on a tile saw

6. All sides are cut

7. A six-faces block



8. Original components

9. Block was further cut to several flat beams using a tile saw

10. Flat beams were trimmed to thin beams suitable for the BBR tests

11. Condition beams in the bath of the BBR instrument at a desired temperature for 1 hour

12. Run the BBR test

Figure 10: Sample preparations for the BBR test

It was observed that when cutting flat beams to thin beams, the narrowing effect on the dimensions of thin beams is very significant due to the heat created between the blade and the edge of the sample. Thus, it was found that better results are obtained when the flat beams are placed in a freezer for at least 4 hours so that the thermal impact on cutting of the sample is minimized. After mixture beams are prepared in a lab, the next step is to run the BBR test as specified by AASHTO T-313.

The initial load (35 mN, milliNewton, ± 10 mN) applied by the BBR was the same as what is described in AASHTO T313. The applied load for the standard binder beam is 980 mN ± 50 mN.; however, such loading condition cannot create measurable deflections for asphalt mixture beams. Previous work by Zofka et al. [2005] recommended several loading options (e.g., 450 g, 730 g, and loading superposition) to be used in the BBR tests. In addition, the BBR manufacturer, Canon Instrument Company, stated that the BBR can apply up to 450-gram force without further change in air bearing system. After preliminary BBR tests in the lab, it was found that the 450 gram of applied loading for the BBR test can produce significant deflections of asphalt mixture beams at the recommended test temperatures (PG +10°C), so the 450-gram applied force (4413 mN ± 50 mN) was selected for the BBR tests in the study.

4.6 Evaluation of Air Voids Consistency

The air void measurement in the compacted samples and the cut blocks is different. This difference between compacted samples and blocks may create inconsistent results that can impact the validity of the BBR test. Thus, measurements of air void content in the blocks were performed and the results were compared with the SGC samples to ensure consistent values between SGC samples and the trimmed blocks. Twelve SGC samples with 2 different numbers of gyrations (100 and 75) were produced based on the mix design formula with a range of air voids from 2.5 % to 8 % as measured using the procedures specified in AASHTO T166- Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens [AASHTO, 2009].

Figure 11 shows the comparison result between the air voids of the compacted samples and the air voids of the blocks for different gyrations and different initial sample mass. The result indicates that the change in air voids between compacted samples and blocks is constant at about 1 percent. This is relevant since for practical reasons the air voids are normally measured on the original gyratory sample and not on the individual beams. The same difference between samples with cut and uncut surfaces has been observed in other mixture tests.

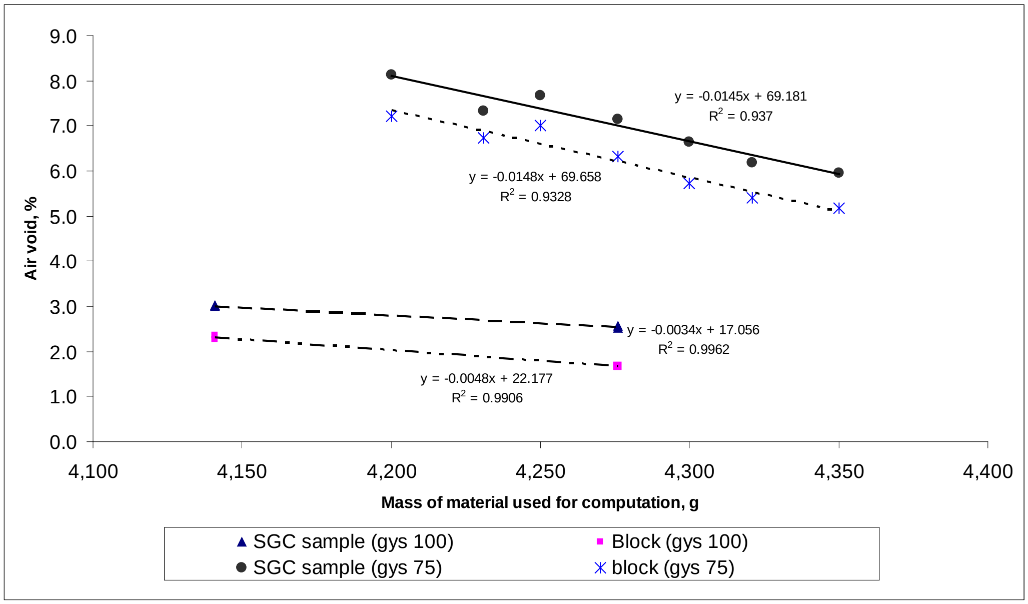


Figure 11: Air void change between SGC samples and blocks

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5.0 Evaluation of Statistical Factors

Inherent in this work is hypothesized that the stiffness data measured from the BBR tests has a normal distribution. Thus, a statistical analysis was performed to validate the assumption of normality. The stiffness at -24 ºC used to evaluate the flexural property of the asphalt mixture beam was measured at 60 seconds during a BBR test, and used to validate the assumption of normality.

5.1 Validation of Normality of Sample Response

Asphalt concrete can be seen as a composite viscoelastic material having spatial disorder with no micro-structural periodicity. In other words, asphalt mixture samples are never exactly alike; thus results from the BBR tests between samples will have some inherent variability that needs to be taken into consideration when performing tests. A concern regarding testing of asphalt mixture samples is the effect of normality. As previously mentioned, responses measured from BBR tests are assumed to be normally distributed. Such assumption, of course, needs to be validated experimentally by testing multiple replicates and evaluating the distribution of the results. Once the normality of the samples has been statistically validated, the number of replicates needed for a valid test can be determined using statistical analyses.

There are many statistical methods that can be used to evaluate variations of the normal distribution. An easy way to find whether samples are normally distributed is to develop a histogram. In doing so, 71 stiffness data obtained from the BBR tests were grouped at 0.5 GPa segments. Frequencies against stiffness values were plotted in a histogram. The sample distribution and the histogram are shown in Figure 12. As can be observed, sample distribution shows minor skewness. Whether this skewness is severe enough to reject the assumption of normality or simply lightly tailed to accept the assumption will be evaluated.

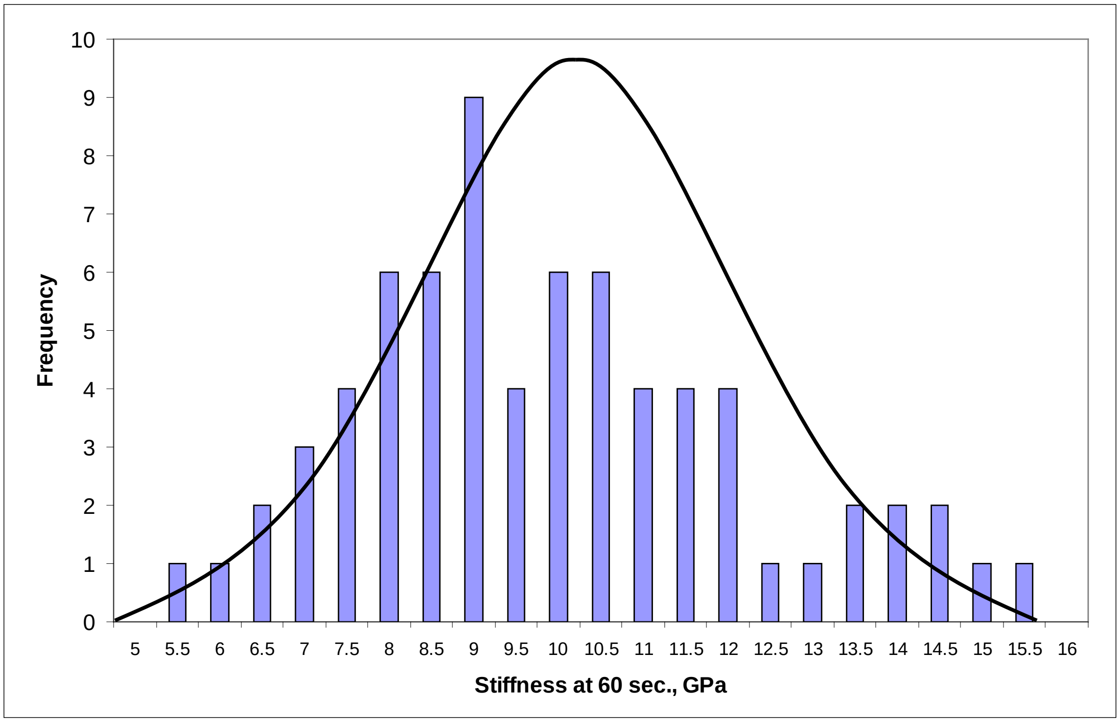


Figure 12: Plot of stiffness versus frequency

(Curve was drawn for illustration purposes.)

A commonly used index for skewness is the Pearson skewness coefficient (SK_p) [Salkind, 2007].

$$\text{Pearson skewness coefficient} = SK_p = \frac{3\left(\overline{X} - M\right)}{s} \qquad (1)$$

Where:

$\overline{X}$, M, and s denote the mean, median, and standard deviation of the samples

For a standard normal distribution, SK_p is equal to 0. If the index is less than -1 then it is skewed to the left and if it is more than 1, then it is skewed to the right. Lehman [1991] suggested that values of SK_p between -0.5 and +0.5 indicate general acceptance levels of skewness. Substituting 71 data points into Equation 1 results in a SK_p =0.38. The value of 0.38 is within the acceptance level (-0.5, 0.5). This indicates that the sample distribution shown in Figure 12 is not significantly skewed, only slightly skewed to the left.

In addition, there is a more specialized graphical display called the Normal Probability Plot that can quantify deviations from Normality more clearly than the corresponding histogram (Figure 12) [De Veaux et al. ,2005]. The Normal Probability Plot is developed by calculating the value of a standard normal quantile (z-score) of each data and plotting a scatter graphic of z-scores versus sample stiffness data. If scatter points appear to roughly describe a line, then it is reasonable to think the data are fairly normal-distributed.

Using the algorithm of the Normal Probability Plot, the values of z-scores from 71 data points were determined and the relation of z-scores against stiffnesses of samples is shown in Figure 13. Clearly, the points follow a line. Applying the linear regression technique to the scatter points results in a R^2=0.972.

In combination with these three statistical methods (i.e., a histogram Figure 12, the Pearson skewness coefficient, and the Normal Probability Plot), we can conclude that the data obtained in this research is normally distributed.

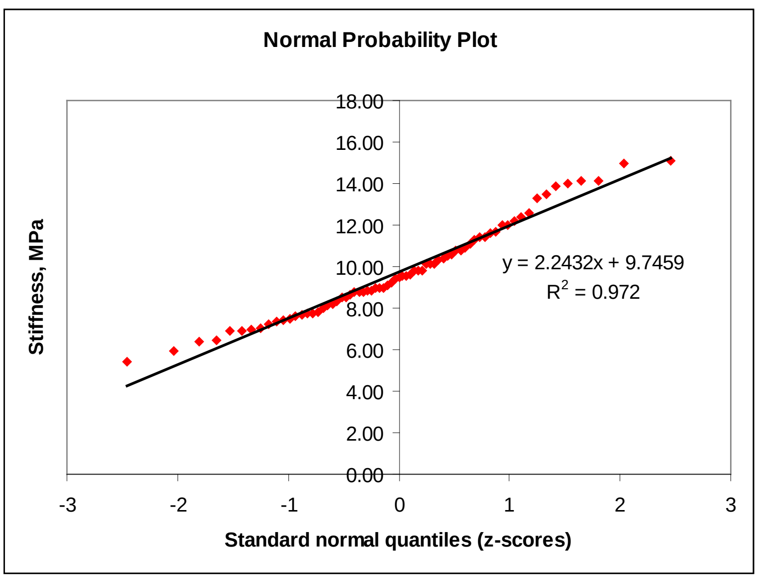


Figure 13: Normal Probability Plot

5.2 Determination of Replicate Samples

Having validated the normality of the acquired data, one of the important questions to be addressed as part of this research is: how many replicate samples are needed to reach a valid conclusion. The answer to this is not a single number since it depends on the assumptions made, the risk tolerated, and the number of samples available. In most fields, multiple replicates and statistical analysis are often used to filter through variability and reach unbiased conclusions. In asphalt mixture testing, the number of replicate samples is sometimes limited and might not always meet rigorous statistical requirements. Two factors should be taken into consideration to arrive at a given number of replicates in the final conclusion. The first one is the range of the coefficient of variation (or standard deviation) in the sample distribution. Another factor is the range of sample mean selected prior to statistical analyses. Combination of these two variables provides better perspectives in the determination of the number of replicates needed for a desired result.

This section first discusses the impact of coefficient of variation in the selection of the number of replicates. Once the relationship between the number of replicates and its corresponding coefficient of variation is recognized, the effect of sample mean on the final results will be presented in the following section.

To determine the number of replicate samples the variance and coefficient of variation (CV) are analyzed.

The variance and the coefficient of variation have the following relation.

Variance (σ^2) is expressed as:

$$\sigma^2 = \frac{\sum_{i=1}^{N}(x_i - \bar{x})^2}{N-1} \qquad (2)$$

Where:

x_i refers to the stiffness value at i^{th} sample,

$\overline{x}$ is the mean of a sample group,

N is the degrees of freedom.

Coefficient of variation (CV) is defined as:

$$CV = \frac{\sqrt{\sum_{i=1}^{N}(x_i - \overline{x})^2 \Big/ N-1}}{\overline{x}} = \frac{\sqrt{\sigma^2}}{\overline{x}} \quad (3)$$

The criterion of determining sample replicates is based on initially randomizing the order of the samples and calculating the corresponding variance and CV to observe the dispersed situations of variables as a function of number of replicates. Ten rounds of sample randomizations were carried out. The variance and CV were calculated after each round. By using such statistical approaches, it can be observed that CV stabilized as a function of increasing the number of replicates. Therefore, the minimum number of samples needed for a statistically valid BBR test can be determined. The plot of the scatter of variables against the number of replicates for 10 rounds is shown in Figure 14.

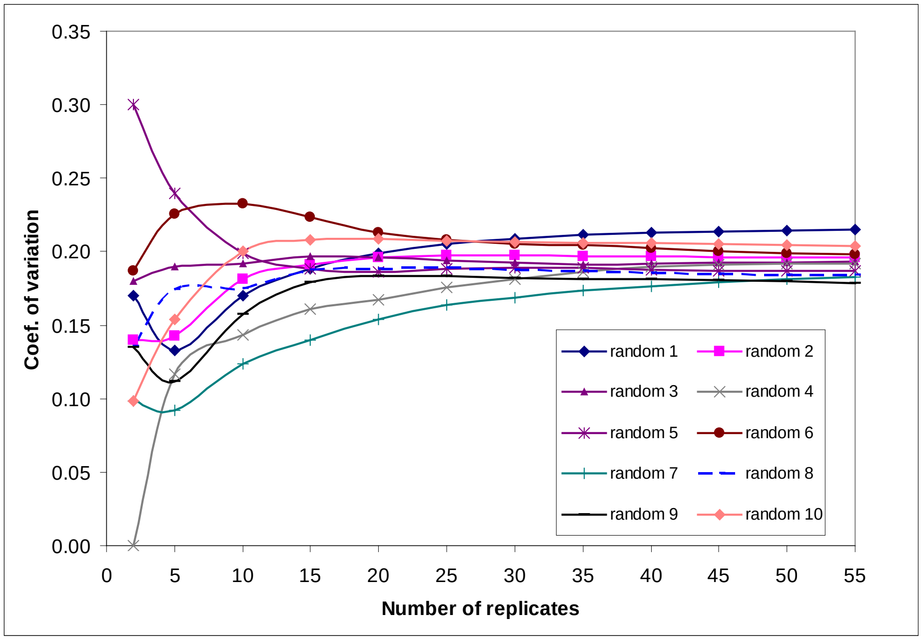


Figure 14: Scatter of variables

It is noticed that, among 10 randomized processes, each scatter of CV is approximately stabilized as the number of replicate approaches 15. In addition, the range of corresponding CV varies from 0.18 to 0.22. Such value might seem extremely high, yet it is typical in asphalt mixture testing [Romero and Anderson, 2001]. This finding leads to two statements:

1. To evaluate the variance, the minimum replicate of asphalt mixture beams should be at least 15.

2. A value of CV at 0.2 can be used. This implies that highway agencies can select 5 (for example) mixture beams to run the BBR tests. If the CV of stiffness data measured from the BBR tests is less than 0.2, the results used for QC/QA should be considered acceptable.

5.3 Evaluation of Type I and II Errors in Sample Means

The previous section recommended the number of replicates and CV values needed for a valid test. Such results only give a range of standard deviation of samples and do not take into consideration the sample mean in the final conclusion. It is vital to understand the implications of using a given number of replicates in the final decision, even if the final number of samples tested falls short of the desired number. Thus, it is desired to incorporate the values of CV and sample mean to provide an understanding of using a given number of replicates in the final conclusion.

A common use of data is to compare two conditions. Examples of such comparisons include issues such as: is the material produced in the field the same as the material previously tested in the laboratory? Or is the material delivered by the contractor the same as the material specified in some contract documents (e.g., specifications)? In these cases we are comparing the mean of two groups; such comparisons lead to hypothesis testing:

H_0: $u_1 = u_2$, the means are the same, or

H_a: $u_1 \neq u_2$, the means are different.

Hypothesis testing was mentioned in Section 4.4 and is concerned with two types of errors:

Type I error: the null hypothesis is true, but we mistakenly reject it. In other words the means are the same but we conclude they are not. This is the so-called false positive.

Type II error: the null hypothesis is false, but we fail to reject it. In other words the means are different but we conclude they are the same. This is the so-called false negative.

The probability of these errors is referred to as α and β, respectively. Thus, for a given value of α and β the required number of replicates can be determined using a statistical approach. When performing hypothesis tests, it is intended to decrease Type I error as well as Type II error simultaneously. De Veaux et al. [2005] mentioned that the only way to minimize both Type I and II errors is to reduce the standard deviation by increasing the sample size. Thus, it is of interest to determine the number of replicates needed to provide unbiased results (e.g., the minimum replicate samples) as well as to simultaneously minimize type I and II errors.

As the number of replicates increases, we are interested in knowing about the degree to which the results among these samples are “true” or “representative of the population”. A concern has been raised that there is no way to avoid arbitrariness in the final decision as to what level of significance or the values of α and β that will be treated as really significant. That is, the selection of some level of significance, up to which the results will be rejected as invalid, is arbitrary. A state agency and a contractor must first decide the degree of difference between the sample mean prior to material testing. This agreement will lead to the determination of certain significance level to which the outcome is accepted or rejected based on analyses and comparisons performed on the entire data set. The following will demonstrate procedures to determine a number of replicates to reduce variances as well as to simultaneously minimize Type I and Type II errors.

As described earlier, the probability of Type I and II errors is referred to as α and β respectively. Typically, values of α are 0.05 or 0.01. The value of β is a function of the differences in mean, δ, and sample sizes, n, which is also specific to a given value of α. Note that δ is the actual difference between means of samples, a value not always known. As the sample size increases, the probability of type II error decreases for a given difference and variance. Ferris et al. [1946] developed a graph of β versus δ/s (s=standard deviation) for a given α that can be used to determine the relationship between α, β, and δ. This graph can be found in this paper [Ferris et al., 1946] and it is referred to as the operating characteristic (OC) curve. In practice, OC curves are used to display the mean of competing choices of sample sizes n, type I error probability α, and type II error probability β [Heiberger and Holland, 2004]. To reduce type I and II error parameters, Ferris's OC curve is suited in the study to achieve control over the two errors. Table 2 provides 3 scenarios that describe the statistical relations between type I error (α), type II error (β), and difference (δ/s) using Ferris's OC curve.

Table 2: Operating characteristics of different number of replicates

	Difference (δ/s) =1.0			Difference (δ/s) =1.2			Difference (δ/s) =1.5		
Number of replicates	5	10	15	5	10	15	5	10	15
Type II, β	60%	20%	6%	50%	10%	2%	30%	3%	1%
Type I, α (given)	5%	5%	5%	5%	5%	5%	5%	5%	5%

As explained previously the value of difference (δ/s) must be determined by the contractor and the state agency prior to material testing. The number of replicates can be decided by selecting δ/s associated with the value of β or vice verse. In Table 2, for a given α =5% if both contractor and state agency have agreed to determine the difference (δ/s) at 1.0, the probability of type II error (β) is about 60% when only 5 samples are tested. The value of β decreases to 20% as the number of replicates increases to 10. Statistical results in Figure 14 recommend the minimum number of replicates at 15. Thus, by increasing the number of replicates to 15, the type II error is significantly reduced to 6%.

Similarly, if δ/s is decided at 1.5 by both the contractor and state agency, the probability of type II error (β) is about 30% when only 5 samples are tested. The value of β reduces to less than 5% if the number of samples tested increases to 10 and the selection of 15 replicates would minimize type II error to only 1%. Clearly, 15 replicates determined by Figure 14 and Table 2 not only reduce the value of CV below 20%, but also minimize type II error to less than 6%.

By applying both decision criteria in Figure 14 and the OC curve, the number of replicate samples in the final conclusion can be better understood. Generally, a better solution to answer how many replicates are needed to reach a valid test depends on the intended use of data and what degrees to which the comparisons and statistical results are accepted by both state agency and contractor (i.e., the level of risk that each is willing to take). These decisions must be made prior to any statistical analysis.

5.4 Summary

1. The normality of sample distribution is confirmed through the combination of a histogram, the Pearson skewness coefficient, and the Normality Probability Plot. Thus, the use of statistical methods in the study to evaluate the variance and determine the number of replicates for the BBR tests is appropriate.

2. Coefficient of Variation and sample mean are taken into consideration for the determination of the number of replicates to reach a desired and valid test. Three scenarios were analyzed to evaluate the operating characteristics between type I error, type II error, and the difference. Based on statistical analyses associated with the OC curve, 15 replicates corresponds to the probability of making a type II error (β) less than 5% for a given type I error $\alpha = 5\%$ while the value of CV is maintained below 20%. Thus, the selection of 15 replicates in the BBR testing can reduce CV and type II error simultaneously.

3. In practice, both state agency and contractor can reach an agreement of what degrees to which the comparisons and statistical results from the BBR tests are accepted or rejected. This is based on the level of risk that each is willing to take. However, it

should be noted that, in contrasts to other tests, the simplicity and sample size used in these tests allow for multiple replicates from one gyratory sample.

6.0 Evaluation of Sources of Variability

Since the objective of this research is to validate the work of using asphalt mixture beams to run the BBR test for QC/QA, the sources of variability during BBR testing were carefully evaluated. Based on preliminary experience, the processes of cutting compacted specimens to small asphalt mixture beams have the following sources of variability that need to be addressed.

6.1 Dimensions

The standard BBR test on asphalt binders uses aluminum molds to produce the test samples. Thus, the dimensions of the samples are fairly constant. However, preparations of asphalt mixture beams were carried out using a tile saw, so the dimensions of the beams can vary and their effect on the BBR results need to be evaluated prior to the experiments. The dimensional measurement of a mixture beam is illustrated in Figure 15. The middle points in the width and thickness directions are measured by averaging the two readings at W2 and W3 in the width direction, and T2 and T3 in the thickness direction. The precision and bias requirements for multiple samples are specified in AASHTO procedure T313. However, such requirements apply only for binder beams, not the mixture beams used in the study.

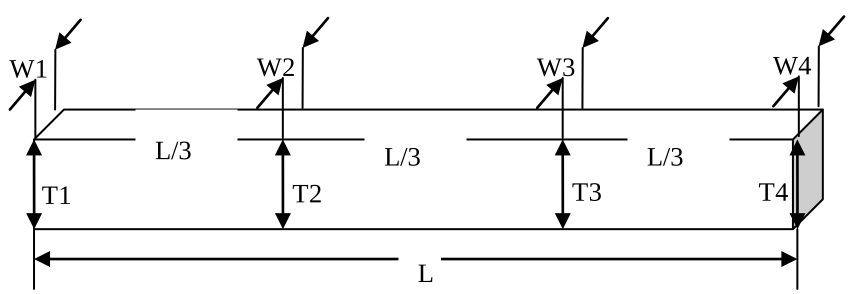


Figure 15: Measurement of Asphalt Mixture Beam

The statistical analysis discussed on the previous section showed that when multiple samples are used to test thermal properties of asphalt mixtures, the coefficient of variation (CV) tends to converge at 20%. This value is typical of mixture testing in the asphalt industry. Thus, the value of 20% was used in this study to determine the acceptance range of deflection difference between asphalt mixture beams and the standard beam.

To better understand the dimensional issue in the sample preparation, a numerical analysis was performed to evaluate the effect of dimensional difference in the accuracy of deflections in the asphalt mixture beams.

As shown in Figure 15, the dimensional range is defined as:

In the width direction: $|W2 - W3|$ (4)

In the thickness direction: $|T2 - T3|$ (5)

Dimensional ranges were selected from 0.3 mm to 1.0 mm in both width and thickness directions. A finite element method (FEM) was employed to calculate the middle deflection of a sample beam. The difference in deflection between a beam with exact dimensions (standard beam) and beams with varying widths and thicknesses was compared based on FEM results. The comparisons of deflection differences were used to assess the effect of dimensional variations on the testing accuracy. Once dimensional issues were evaluated, the tolerance range of asphalt mixture beams was determined. Figure 16 shows examples of the analysis for a mixture beam with 0.5 mm dimensional range and a standard beam using 2-D FEM analysis. It is noticed that the CV values resulting from the dimensional changes from 0.3 mm to 1.0 mm in both width and thickness directions vary from 5.7 % to 18.45%. These results are less than 20%. Note that AASHTO T313 requires the difference between two tested BBR binders by multiple laboratories to be below 17.8 %. Thus, the deflection differences influenced by dimensional ranges are within a reasonable accuracy, balancing accuracy with practicality.

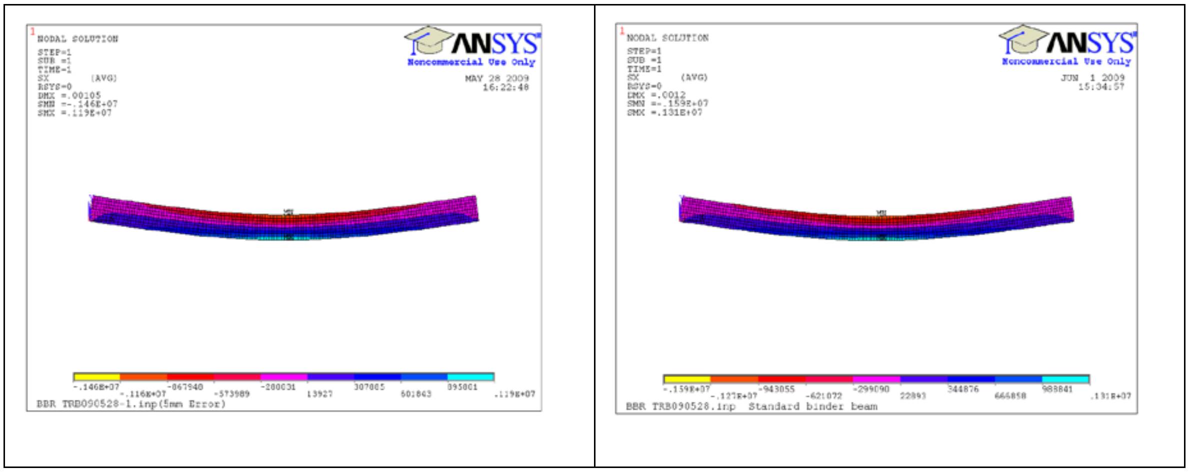


Figure 16: FEM results for asphalt mixtures beams with 0.5 mm dimensional range

After some practice in the laboratory and using a commercially available tile saw, it was determined that most dimensional ranges can be controlled within 0.5 mm without much effort. In Table 3, the dimensional range of 0.5 mm corresponds with a CV of 9.43% that is significantly lower than the desired 20%. Since a lower tolerance range will minimize the dimensional error and provide more accurate test results, it is recommended to set 0.5 mm as a tolerance range to run the BBR tests.

Table 3: Comparison of Calculated Deflections

Dimensional Range, mm	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1
Measured width, mm	12.85	12.9	12.95	13	13.05	13.1	13.15	13.2
Measured thickness, mm	6.5	6.55	6.6	6.65	6.7	6.75	6.8	6.85
Calculated deflection of standard beam, mm	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
Calculated deflection of mixture beam, mm	1.107	1.078	1.05	1.023	0.997	0.972	0.947	0.923
Deflection difference, mm	0.093	0.122	0.15	0.177	0.203	0.228	0.253	0.277
Standard Deviation, mm	0.066	0.086	0.106	0.125	0.144	0.161	0.179	0.196
CV,%	5.70	7.57	9.43	11.26	13.07	14.85	16.66	18.45

6.2 Air Voids

The air void content has been identified as a critical parameter in the response of asphalt mixtures. However, as discussed in Section 4.3, there is contradictory information regarding the effect of air void content and distribution on the low temperature properties of asphalt mixtures. Thus, this section presents the results of experiments intended to settle the issues related to air voids.

As previously mentioned, the relative density of the prismatic beams was calculated by measuring the mass and dividing it by its volume. This approach neglects the surface voids; however, work by Masad et al. [2002] has shown that the air voids in a gyratory cylinder are normally distributed except for the top and bottom parts of a specimen. Since the beams are cut from the middle of specimens, any error incurred by neglecting the surface voids is constant, as shown on Figure 11. This assumption was validated experimentally using a wide range of relative density values from which the relationship between air voids (or relative densities) and the stiffness was evaluated. Each mixture beam was measured at one-third span with width and thickness readings. The relative density was therefore determined by measuring the mass of the sample and dividing by its volume (average width x average thickness x length) as shown in Figure 15. The relative density was then divided by the maximum theoretical density of asphalt mixtures, G_{mm}, so its unit is shown as percent.

A series of stiffness data from the three NMAS samples were obtained from BBR testing. Fifty five asphalt mixture beams were produced from 6 compacted asphalt samples. The stiffness data at 60 seconds and the corresponding relative density were measured after the BBR testing. Figure 17 shows the plot of stiffness values of 12.5mm NMAS mixture beams against their corresponding relative density measurements. Clearly, the effect of relative density (i.e., air voids) on the stiffness of asphalt mixtures is not significant at low temperatures. To further study the role that aggregate size played in the stiffness of asphalt mixtures, 15 stiffness data from each NMAS mixture samples of 9.5mm and 4.75 mm were obtained from the BBR tests. The relative density measurements were carried out following the same procedures as implemented in the 12.5 mm NMAS samples. As shown in Figure 18, analysis results coincided with Figure 17 confirming that, regardless of aggregate size, the stiffness of asphalt mixtures is not affected by air voids at low temperatures. This result not only validates the theoretical work by Zofka et al. but also provides scientific evidence relating to the less effect of air voids on the thermal properties of asphalt mixtures.

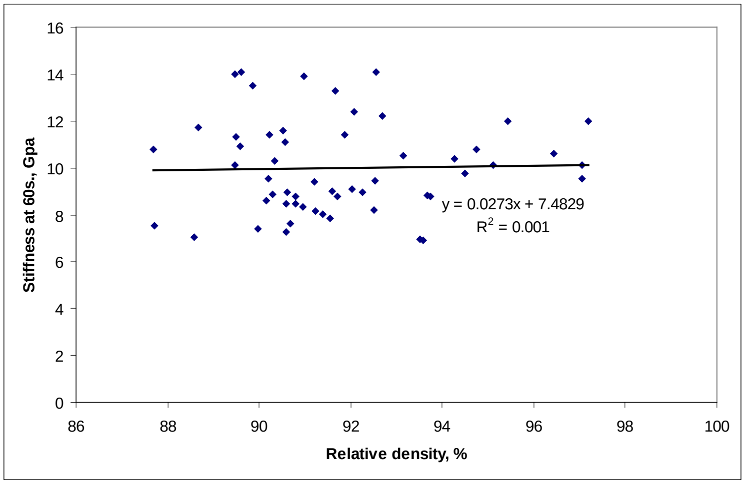
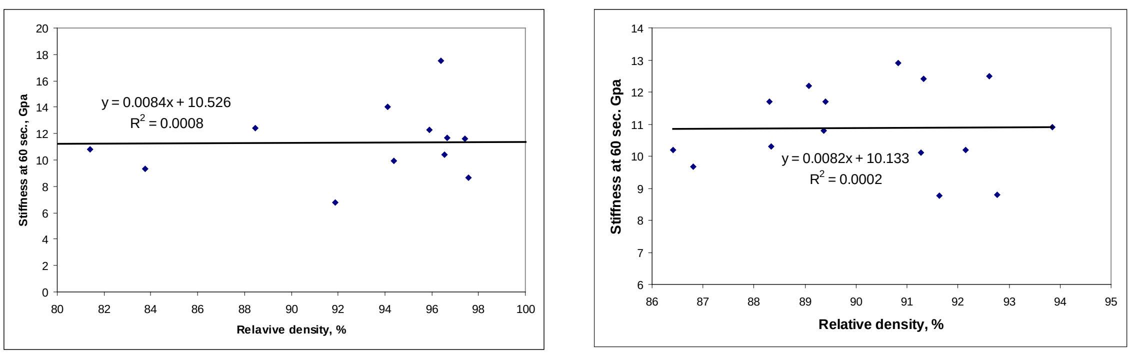


Figure 17: Relationship between stiffness and relative density for 12.5 NMAS



(a) 9.5 mm Mix **(b) 4.75 mm Mix**

Figure 18: Relationship between stiffness and relative density

6.3 Gauge Length and Larger Specimens

The effect of aggregate sizes on the stiffness of asphalt mixtures has been a concern in the asphalt community. As mentioned earlier, the major criticism is that the aggregate particles occupy most of the volume in a beam, thus any single aggregate particle can play an important role in the results of the test. Without a doubt, the ratio of the aggregate particle size to the gauge length is an important value to ensure creep compliance measurements within a reasonable accuracy. Research work by Binns and Mygind [1949], Marshall [1973], and Window [1992] suggested that a gauge length should be at least four times the nominal maximum aggregate size for errors less than 5 percent with a greater degree of confidence, so for the largest 12.5mm NMAS used in the study the minimum gauge length should be 50 mm. The span of an asphalt mixture beam in the BBR equipment is 101.6 mm, thus the requirement of the minimum gauge length was met.

Asphalt mixture is not a homogenous material; it is made up of aggregate particles randomly dispersed in a bituminous matrix. The measurements from BBR tests are based on averaging creep compliance responses along bending asphalt mixture beams. Work by NijBoer [1948] has shown that any stiffness difference between asphalt mixture samples is caused by aggregate quantities not aggregate particle size (spatial distributions). Furthermore, previous research by Weissman et al. [1999] and Romero and Masad [2001] revealed that, as temperature drops below an asphalt binder's glass transition temperature, the properties of both asphalt binder and aggregate tend to converge. Thus, the bulk properties of both constituent materials become less dependent on the aggregate size and spatial distributions. A numerical analysis will be used in the following section to evaluate the effect of spatial distributions in the low temperature properties of asphalt mixtures.

Velasques et al. (2009) performed some very detailed work to address the issue of RVE when testing asphalt mixtures using the BBR. They determined the creep stiffness of ten asphalt mixtures using beams of three different sizes: 6.25 x 12.5 x 100 mm (standard beam size used in the BBR), 12.5 x 25 x 200 mm (twice the size), and 18.75 x 37.5 x 300 mm (three times the size). They compared them to values obtained from larger samples tested using the IDT. They ran a total of 360 tests at three temperatures.

Their conclusion was that the size of the aggregate does not have an effect on the creep stiffness obtained from the beams at a temperature of 10 ºC higher than the low limit performance grade of the asphalt binder (i.e., PG + 10 ºC). Strangely, at a lower test temperature closer to the binder low limit performance grade (i.e., PG – 2 ºC) they found that the beam size does have a significant effect on the creep stiffness of the asphalt mixtures. They attributed this result to equipment issues including the difficulties associated with measuring the small deflections at such low temperatures and the formation of layers of ice on the supports during their test of the larger beams. Furthermore, the tests were done using a servo-hydraulic machine and not the BBR.

6.4 Spatial Distributions in Thermal Properties of Asphalt Mixtures

In this study, a linear viscoelastic (LVE) modeling analysis was implemented to quantify previous research results. Raw deflection data from the three NMAS (i.e., 12.5 mm, 9.5 mm, and 4.75 mm) samples tested at three different temperatures were exported from the BBR computer program and converted to creep compliance. A power law function was used to represent the responses of creep compliance:

$$D(t) = D_0 + D_1 \cdot t^n \qquad \text{(Power law)} \qquad (6)$$

Where D(t) =creep compliance at reduced time, t, D_0, D_1 and, n = power function parameters.

The relationship between creep compliance and relaxation modulus can be correlated to the following interconversion equation:

$$\hat{D}(s)\hat{E}(s) = \frac{1}{s^2} \qquad (7)$$

Where a caret (^) over the symbols shows that the quantity is now a function of Laplace transform and s is a Laplace transform parameter.

Taking Laplace transform of Equation 6 and substituting to Equation 7 obtain:

$$\hat{E}(s) = \frac{1}{\hat{D}(s)s^2} = \frac{1}{sD_0 + D_1\Gamma(n+1)s^{1-n}} \qquad (8)$$

Where Γ is defined as a gamma function.

The direct inverse of the Laplace transform of Equation 8 cannot be solved. However, an approximate method proposed by Schapery [1962] can be applied to overcome this dilemma. The details of Schapery's algorithm are discussed later in this report. The relaxation modulus is then given as:

$$E(t) = \frac{1}{D_0 + D_1\Gamma(n+1)(1.786t)^n} \tag{9}$$

Based on calculated relaxation modulus, the thermal stresses for each NMAS sample were predicted by the following equation:

$$\sigma(T) = \int_0^T E(T - T')\frac{\partial\varepsilon(T)}{\partial T'}dT' \tag{10}$$

E(T-T') is the relaxation modulus that has been previously determined (T' refers to the parameter of integration). The only parameter needed is ε(T) (strain at temperature T) and it is equal to α (coefficient of thermal contraction) multiplied by dT/dt (temperature increment).

Power law parameters were obtained from nonlinear regression methods and α ($1.7\text{x}10^{-4}$ mm/mm/°C) and dT/dt (1°C per hour) were obtained from Bouldin et al. [2000].

Applying Equation 6 through Equation 10 determines the thermal stresses of the NMAS asphalt mixture beam. Figure 19 shows the result of the prediction of cracking temperature (actually, temperature at which the thermal stress reaches 3 MPa since strength was not measured; see Section 7.5 for details) for the three NMAS mixture samples. The cracking temperature for three NMAS samples exhibit close agreement with each other with a difference of less than 3 degrees. Generally, the overall effect that aggregate size played in controlling the thermal resistance of asphalt mixtures has no engineering significance when predicting cracking temperature for these mixtures.

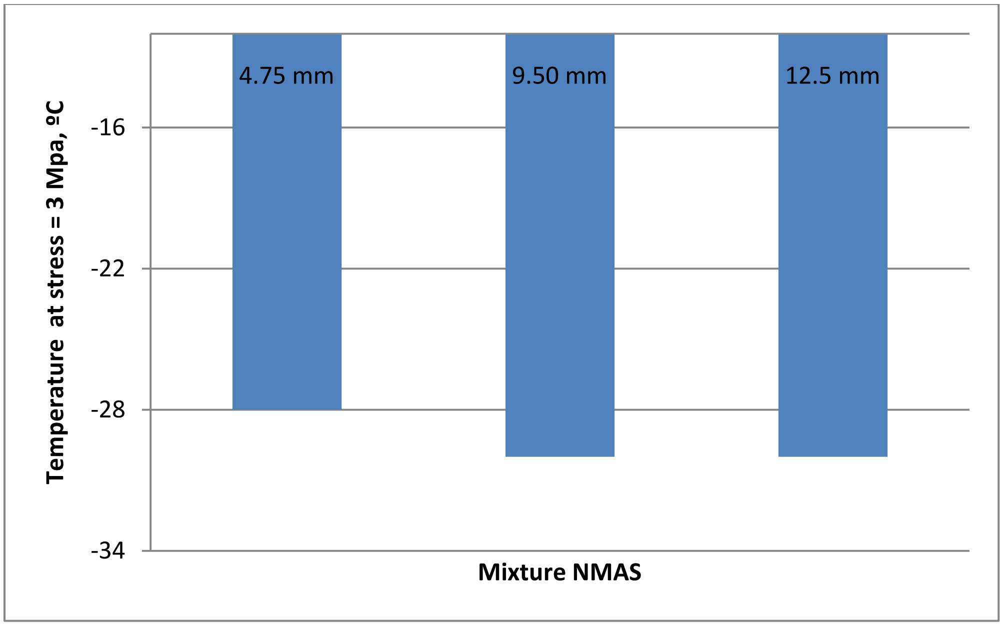


Figure 19: Predicted cracking temperature of mixtures with different NMAS and the same binder (Temperature where stress equals 3.0 MPa)

In the nature of linear viscoelasticity, creep compliances are measured based on the average responses along an entire beam. This means that each component along with its adjunct materials in a beam contribute together to the global properties of asphalt mixtures when subjected to loading; the flexural stiffness of asphalt mixture beams at low temperatures does not primarily come from any single large aggregate particle in a beam. Instead, if a beam is seen as the sum of segments, the thermal properties of asphalt mixtures are resulted from the mechanical responses of each segment where binder and aggregates are both related to the responses. This is why the linear viscoelastic behavior of asphalt mixtures is based on the "average" responses received along the gauge length of the beam during a BBR test, not the width or the height. The responses shown in Figure 19 confirm that the cracking temperatures are a function of binder grade and perhaps aggregate type but independent of the NMAS used for the mix design. Aggregate sizes do not significantly affect the results of the tests. This experimental work and numerical analysis reflect the nature of linear viscoelasticity of asphalt mixtures at low temperatures and provides evidence to address the issue of spatial distributions in the global properties of asphalt mixtures.

6.5 Homoscedasticity of Variances

Considering the composite nature of asphalt concrete, it is possible that the response discussed in Section 6.4 is the result of binder grade and not the aggregate NMAS. It is possible though that the aggregate size might influence the variance of the results. In this study, 3 NMAS sample groups are presented to evaluate the sources of variability. All of the experimental and statistical work is based on the assumption that the samples came from populations with identical variances and that larger aggregate do not cause more variability (i.e., increased variance). However, this hypothesis must be validated to ensure the homogeneity along the samples. The most common method used to evaluate the homoscedasticity of variances is the Bartlett's test [Zar, 1999]. The Bartlett test is defined as:

Hypothesis: $\sigma_a^2 = \sigma_b^2 = \sigma_c^2$

σ_a^2, σ_b^2, and σ_c^2stand for variance based on the a, b, and c replicates from three NMAS groups, respectively.

Test statistic:

$$B = (\ln s_p^2)(\sum_{i=1}^{k} v_i) - \sum v_i \cdot \ln s_i^2 \tag{11}$$

Where $v_i = N_i - 1$, N_i is the size of sample i, k is the number of groups, $\boldsymbol{s_i^2}$ is the variance of the group with i replicates expressed as

$$s_i^2 = \sum_{i=1}^{k} SS_i / v_i \tag{12}$$

Where SS_i is the sum of the squares of the deviations from the mean.

$\boldsymbol{s_p^2}$ is the pooled variance defined as:

$$s_p^2 = \sum_{i=1}^{k} (N_i - 1) s_i^2 / v_i \tag{13}$$

Correction factor C is defined as:

$$C = 1 + \frac{1}{3(k-1)}\left(\sum_{i=1}^{k}\frac{1}{v_i} - \frac{1}{\sum_{i=1}^{k} v_i}\right) \tag{14}$$

The corrected test static T is obtained from Equation 11 through Equation 14 given:

$$T = \frac{B}{C} \tag{15}$$

The variances have a chi-squared probability distribution; therefore the following comparison can be made.

If $T < X^2_{(\alpha,\, k-1)}$, we fail to reject the hypothesis and conclude that the variance based on a number of samples is the same as the variance based on b and c number of samples.

Note that $X^2_{(\alpha,\, k-1)}$, is the upper critical value of the chi-square distribution with k - 1 degrees of freedom and a significance level of α.

Forty-eight samples from the 12.5mm NMAS group were randomly selected from the samples prepared in the laboratory. Fifteen samples from the 9.5mm NMAS group and 15 samples from the 4.75mm NMAS group were also chosen. Applying Equation 11 through Equation 15, the result of the test will test the hypothesis that all samples from the three NMAS groups came from populations with identical variances such that the statistical comparisons among these three NMAS groups are reasonable in the study.

Table 4 shows the results of evaluating the homoscedasticity of variances among the three groups using the Bartlett's test. As shown in Table 4, T(2.2) < $X^2_{(a,\, k-1)}$ (96.2) is determined. Thus, it is concluded that the variance among the three NMAS groups is equal so that the statistical comparisons and numerical analyses presented in this study are valid.

Table 4: Bartlett's test

H_0:	$\sigma^2_{12.5NMAS} = \sigma^2_{9.5NMAS} = \sigma^2_{4.75NMAS}$							
Group	**12.5 NMAS**				**9.5 NMAS**	**4.75 NMAS**		
	stiffness at 60 sec., MPa							
	12.4	**9.52**	**12.2**	**10.5**	**11.6**	**10.2**		
	11.4	**8.79**	**6.96**	**11.1**	**12.3**	**8.77**		
	12	**7.83**	**6.9**	**9.78**	**13.4**	**10.8**		
	10.9	**8.94**	**10.1**	**10.1**	**14.0**	**10.1**		
	10.4	**8.62**	**7.38**		**9.27**	**11.7**		
	11.6	**7.03**	**8.22**		**9.91**	**9.67**		
	11.4	**8.77**	**8.14**		**12.4**	**8.79**		
	10.8	**9.46**	**8.49**		**10.4**	**10.3**		
	7.64	**8.94**	**8.86**		**10.7**	**10.2**		
	10.3	**9.09**	**9**		**9.35**	**11.7**		
	8.79	**8.49**	**7.25**		**10.8**	**12.4**		
	9.4	**8.81**	**11.7**		**6.77**	**12.9**		
	8.03	**9.55**	**10.8**		**11.7**	**10.9**		
	7.51	**10.6**	**10.1**		**12.4**	**12.5**		
	8.35	**12**	**11.3**		**8.64**	**12.2**		
SS_i	**112.968**				**52.347**	**24.263**	$\sum SS_i$	**189.577**
v_i	**48**				**14**	**14**	$\sum v_i$	**76**
s_i^2	**2.353**				**3.739**	**1.733**		
$\log s_i^2$	**0.372**				**0.573**	**0.239**		
$v_i \log s_i^2$	**17.842**				**8.019**	**3.343**	$\sum v_i \log s_i^2$	**29.204**
$\frac{1}{v_i}$	**0.021**				**0.071**	**0.071**	$\sum \frac{1}{v_i}$	**0.164**
$s_p^2 = \frac{\sum SS_i}{\sum v_i}$					**2.49444**			
$\log s_p^2$					**0.39697**			
$B = (\ln s_p^2)(\sum_{i=1}^{k} v_i) - \sum v_i \cdot \ln s_i^2$			**2.2235**		$C = 1 + \frac{1}{3(k-1)}\left(\sum_{i=1}^{k}\frac{1}{v_i} - \frac{1}{\sum_{i=1}^{k} v_i}\right)$		**1.0251**	
$T = \frac{B}{C}$			**2.1691**					
$X^2_{(\alpha=0.05, 75)}$			**96.2050**					

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7.0 Low Temperature Properties of Asphalt Mixtures

The primary task in the study is to evaluate the effect of aggregate particle size on the low temperature properties of asphalt mixtures. Thus, the study will pursue an analytical algorithm that presents the entire processes of calculations to determine the thermal stresses using creep compliance data from the BBR tests.

7.1 Approach

The essential features of the full calculation are:

1. The creep compliance data of asphalt mixture beams for each NMAS are obtained from deflection data at different low temperatures based on the binder grade.
2. Based on linear viscoelastic theory (LVE), one of the major response functions, such as power law shown in Equation 6, is used in this study to model the creep compliance of asphalt mixture beams. The reason for the selection is based on the previous research recommendations by Christensen and Bonaquist [2004]. Additionally, the purpose of the study is to evaluate the role played by aggregate sizes (spatial distributions) in controlling the thermal properties of asphalt mixtures; it is not intended to compare the analysis differences between linear viscoelastic responses (e.g., Prony series versus power law).
3. The creep compliance data is shifted to form a master compliance curve using the time-temperature superposition principle (TTSP). The TTSP translates viscoelastic functions (i.e. creep compliances) at different temperatures into a master curve within an expanded domain on a log stiffness versus log reduced time scale.
4. Prior to the power law analyses, a pre-smoothing technique associated with nonlinear regression methods was employed to fit the experimental data and generate power law parameters. Following the pre-smoothing of the data, a LVE representation function named generalized power law function was established.
5. In the nature of linear viscoelasticity, relaxation modulus of asphalt mixtures was obtained from creep compliance and determined through a Laplace transformation.

6. Calculated relaxation modulus is a function of time but needs to be converted to a function of temperature through a mathematical relationship between shift factors and temperatures obtained from the TTSP.
7. The thermal stresses were predicted using equation 10. The low temperature properties of asphalt mixtures among the three NMAS samples were compared based on the calculations of thermal stresses.

7.2 Time-Temperature Superposition Principle

Time-Temperature Superposition Principle (TTSP) has been widely used to shift creep compliance curves at different temperatures to form a curve known as a master creep compliance curve [Marasteanu, et al., 2008, Schwartz et al., 2002, Andriessou and Hesp, 2008, Zhao and Kim, 2003, Brostow et al., 1999, and Hiltunen and Roque, 1995].

Asphalt mixture beams from three NMAS samples were tested as described in Section 4.5 of this report. The creep compliance data was obtained at each temperature after testing and plotted against the time scale shown in Figure 20 a-c. A reference temperature (T_R) of -24°C was selected for each NMAS group.

Schwarzl and Staverman [1952] stated that the effect of temperature change on the viscoelastic properties of materials is equivalent to a shift on the log time scale expressed below:

$$\xi = \frac{T}{a_T(T)} \tag{16}$$

Where: ξ=reduced time,

a_T (T) =shift factor, and

T = temperature

An Arhennious function was used to relate the shift factors and temperatures under a reference temperature [Christensen and Anderson, 1992]:

$$\log[a_T(T)] = 2.303\frac{E_a}{R}\cdot\left(\frac{1}{T_R}-\frac{1}{T}\right) \quad (17)$$

Where, Ea= the activation energy for flow below T_R, 261 kJ/mol.

R= the ideal gas constant, 8.34J/mol-°K

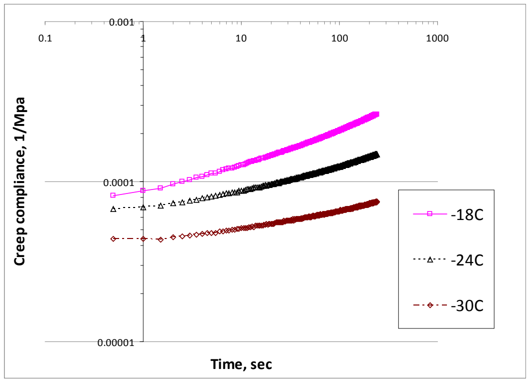
T_R = reference temperature, °C or °K

T= selected temperature, °C or °K

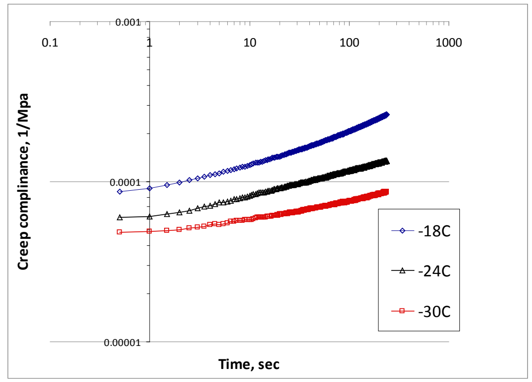
The selection of Arhennious equation in this study was based on work reported by Franck [2008] indicating that Arhennious function appeared to be more appropriate than the Williams-Landel-Ferry (WLF) equation in the prediction of shift factors at low temperatures.

Individual creep compliance curves with corresponding temperatures from the three NMAS samples were shifted along a log time scale to superimpose to a master creep compliance curve (Figure 20 d-f). The typical relationship between the shift factors and temperature is shown in Figure 21.

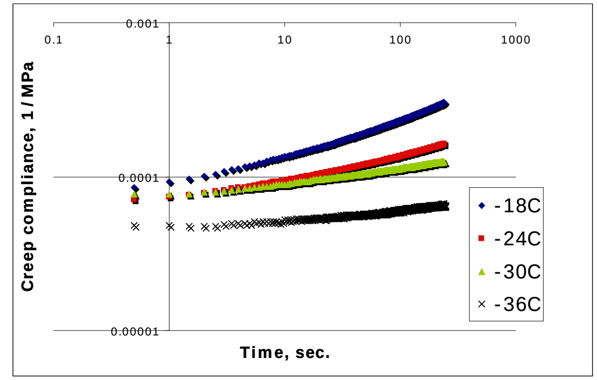
After a master curve was developed, the next step was to employ a power law function to fit the experimental data using a pre-smoothing technique associated with nonlinear regression methods.



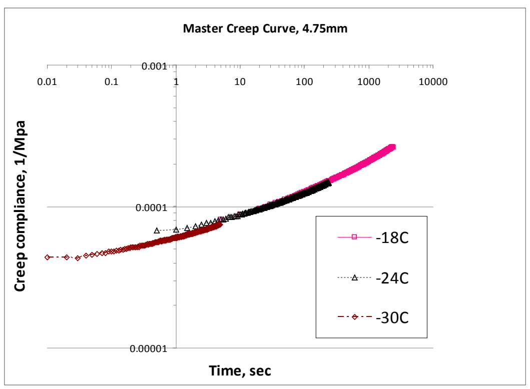
(a) Creep curves for 4.75 mm NMAS



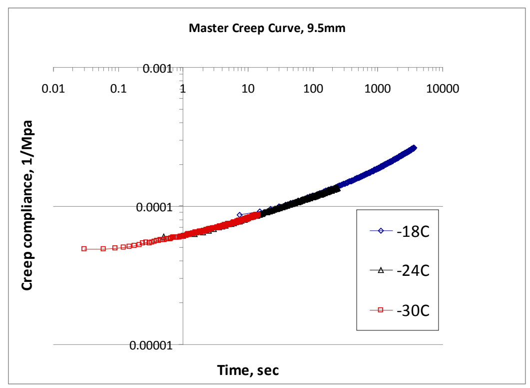
(b) Creep curves for 9.5 mm NMAS



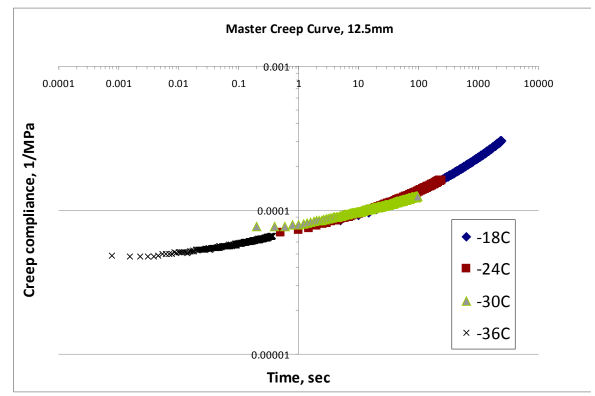
(c) Creep curves for 12.5 mm NMAS



(d) Master Creep curve for 4.75 mm NMAS



(e) Master Creep curve for 9.5 mm NMAS



(f) Master Creep curve for 12.5 mm NMAS

Figure 20: Creep curves for mixtures with different NMAS

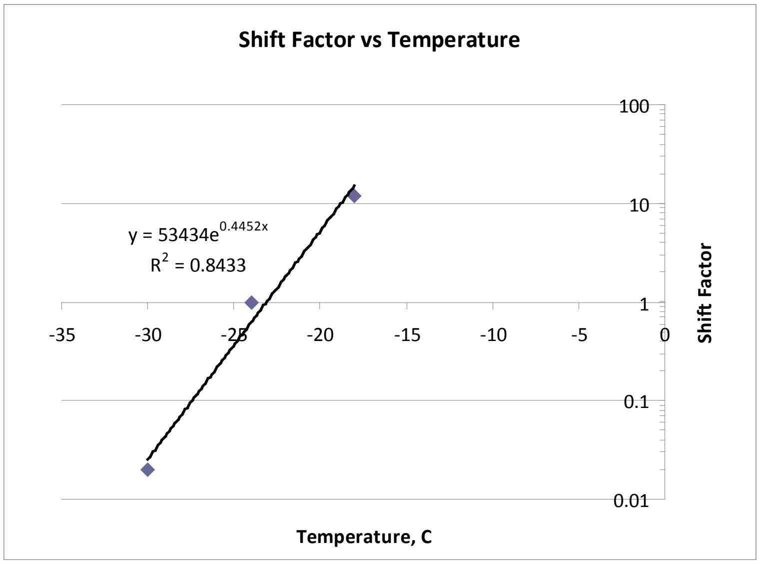


Figure 21: Shift factors versus temperatures

7.3 Pre-smoothing Experimental Data

Due to the fact that a master curve is created by shifting three (or four) creep compliance curves to partly overlap with each other, there needs to be a fitted curve to represent the linear viscoelastic responses of these scattered experimental data. In the 60's and 70's, different methods of fitting Prony series to available data were proposed by Schapery [1962] and Cost and Becker [1970] using a collocation method and multi-data methods. Recent studies by Park and Kim [2001] and Chehab and Kim [2009] provided an algorithm called a pre-smoothing method to fit the given experimental data. Kim et al. [2008] showed a process of fitting creep and complex compliance data using power law function and Prony series function. The fundamental of the pre-smoothing technique presented in this report is the use of nonlinear regression by minimizing the sum of squared errors between the raw data and fitted values.

This approach can be expressed as:

$$\text{Minimize } \sum \left| D_p(\xi) - D(\xi) \right|^2 \tag{18}$$

Where $D_p(\xi)$ = fitted power law response at reduced time, ξ

$D(\xi)$ = raw experimental data at reduced time, ξ

Substituting Equation 6 to scattered experimental data (Figures 20 a-c), the fitted curves are shown in Figure 22 and power law parameters, D_0, D_1, and n were determined in Table 5.

Table 5: Power law function parameters

NMAS	D_0 (1/MPa)	D_1 (1/MPa)	n
4.75mm	3.9899E-05	2.1326E-05	0.29995616
9.5mm	4.3708E-05	1.835E-05	0.2999514
12.5mm	4.04809E-05	2.37741E-05	0.299800535

As can be seen in Figure 22, the generalized power law functions work well in fitting the experimental data at three NMAS samples except for minor scatters at long loading times for the 12.5mm NMAS curve.

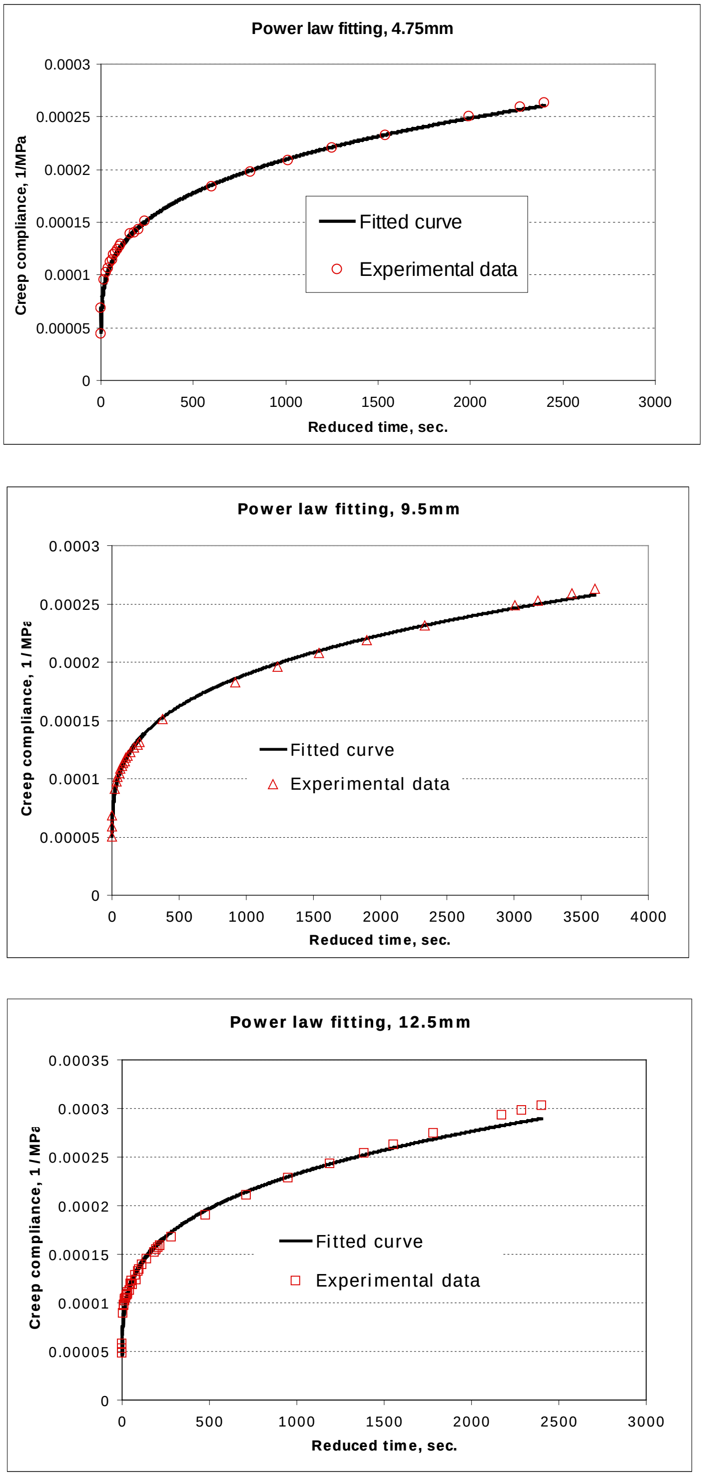


Figure 22: Power law fitting approach for all NMAS

7.4 Linear Viscoelastic Response Modeling

The three fitted curves with determined parameters from Section 7.3 will be used in the following section to derive a linear viscoelastic model and then determine the thermal stresses of three NMAS asphalt mixtures.

7.4.1 Establishment of Analytical Response Function

The creep compliance data were obtained by means of averaging responses received from a bending mixture beam when subjected to a constant load (or stress) during BBR testing. The time-dependent response (or strain) obtained exhibits linear viscoelastic behavior. The deflections (or strains) as a function of time can vary significantly under a constant applied load. In order to predict the time-dependent ratio of strain under a constant stress, linear viscoelastic theory is used. A number of research efforts have been studied in the past decades using linear viscoelasticity to predict mechanical behaviors of asphalt concrete [Secor and Monismith, 1964; Christensen, 1982; Findley et al., 1982; Lytton et al., 1993; Anderson et al., 1994; Christensen and Mehta, 1998; Christensen, 1998; Shields et al., 1998; Kim, J, 2002; and Elseifi et al., 2006]. Particularly, several viscoelastic response equations have been developed to characterize the linear viscoelastic behavior of asphalt concrete. Among these equations, the commonly-used models are the relaxation modulus E(t) and creep compliance D(t), and complex modulus E^* [Chehab and Kim, 2009]. Creep compliance is defined as a time-dependent ratio of strain ε(t) when subjected to a constant stress, and relaxation modulus can be referred to as a time-dependent ratio of stress σ(t) when subjected to a constant strain. Creep compliance D(t) and ε(t) have the following relations:

$$\varepsilon(t) = \int_0^t D(t-\xi)\frac{\partial \sigma(\xi)}{\partial \xi} d\xi \tag{19}$$

Where ξ denotes reduced time.

Similarly, relaxation modulus E(t) and σ(t) can be related as:

$$\sigma(t) = \int_0^t E(t-\xi)\frac{\partial \varepsilon(\xi)}{\partial \xi}d\xi \tag{20}$$

Taking Laplace transform of Equation 19 associated with a convolution theorem obtains:

$$\hat{\varepsilon}(s) = \hat{D}(s)\left[s\hat{\sigma}(s) - \sigma(0)\right] \tag{21}$$

As before, a caret (^) over the symbols shows that the quantity is now a function of Laplace transform and s is a Laplace transform parameter.

Simplifying Equation 21 derives a response function of strain in the Laplace transform domain:

$$\hat{\varepsilon}(s) = s\hat{D}(s)\hat{\sigma}(s) \tag{22}$$

Similarly, the Laplace transform of Equation 20 gives a response function of stress in the Laplace transform domain:

$$\hat{\sigma}(s) = s\hat{E}(s)\hat{\varepsilon}(s) \tag{23}$$

In the nature of linear viscoelasticity, creep compliance and relaxation modulus are interrelated; one function can be solved as long as another one is known. Thus, Equation 22 and Equation 23 can be reformulated to the interconversion equation (Equation 7) along with Equation 6.

The Laplace transform of D(t) becomes:

$$\hat{D}(s) = \frac{D_0}{s} + D_1\frac{n!}{s^{n+1}} = \frac{D_0}{s} + D_1\frac{\Gamma(n+1)}{s^{n+1}} \tag{24}$$

Where Γ is a gamma function and can be described as $\Gamma(n) = \int_0^\infty t^{n-1}e^{-t}dt$

Based on the interconversion principle (Equation 7), relaxation modulus in the Laplace domain can be obtained as shown in Equation 8, repeated here for convenience.

$$\hat{E}(s) = \frac{1}{\hat{D}(s)s^2} = \frac{1}{sD_0 + D_1\Gamma(n+1)s^{1-n}}$$

To solve relaxation modulus *E(t)*, the Laplace transform given in Equation 8 needs to be inversed. Unfortunately, the direct inverse of the Laplace transform of Equation 8 cannot be solved. However, an approximate method proposed by Schapery [1962] can be applied to overcome this dilemma:

$$E(t) = s\hat{E}(s)\bigg|_{s=\frac{e^{w_0 \ln 10}}{t}} \tag{25}$$

As described in Schapery's algorithm, the analogous relationship between *E(t)* and $\hat{E}(s)$ is given by multiplying a Laplace transform parameter, s, to Equation 8 and replacing s with a constant of $e^{\omega_0 ln10}/t$. Note that the term, $e^{\omega_0 ln10}$, is defined as Euler's constant

Where ω_0 is given as:

$$w_0 = \frac{-0.58}{\ln 10} \tag{26}$$

Inserting Equation 26 to Equation 25 gives:

$$E(t) = s\hat{E}(s)\bigg|_{s=\frac{0.56}{t}} \tag{27}$$

Applying Equation 27 to Equation 8 yields Equation 9, shown here for convenience.

$$E(t) = \frac{1}{D_0 + D_1\Gamma(n+1)(1.786t)^n}$$

It is desired to validate Equation 9 prior to the process of inverting Laplace transform of Equation 8 to obtain relaxation modulus. An approximate method by Christensen, D. [1998] can be used to achieve this goal of validation. In a paper, Christensen, D. cited an analytical approach by Christensen, R. [1982] to propose a solution for inverting the Laplace transform of Equation 8 shown below:

$$E(t) = \frac{1}{D_0 + D_1 \Gamma(n+1)(1.73t)^n} \tag{28}$$

This approximate equation (Equation 28) shows a good correlation to Equation 9. Thus, the derivation of Equation 9 is validated and can be properly used in the computation of relaxation modulus.

7.5 Determination of Thermal Properties of Asphalt Mixture Beams for 3 NMAS Samples

As previously mentioned, the argument of using small mixture beams to represent the global properties of asphalt mixtures is directly related to the role played by the aggregate particle size in small asphalt mixture beams. The research presented here addresses these issues by pursuing numerical analyses to evaluate the low temperature properties of three NMAS asphalt mixture beams.

The linear viscoelastic analytical function (Equation 9) for the determination of relaxation modulus of asphalt mixtures is well established and validated in the previous section. The power law parameters needed in the equation are also determined by using nonlinear regression methods (Table 5). Using Equation 9 and Table 5, relaxation moduli for the three NMAS mixture samples were calculated. The comparison of relaxation modulus curves between 4.75mm, 9.5mm, and 12.5mm NMAS samples is shown in Figure 23. Clearly, the three relaxation curves show agreement with each NMAS mixture sample, such that any thermal stress prediction would lead to the same conclusion.

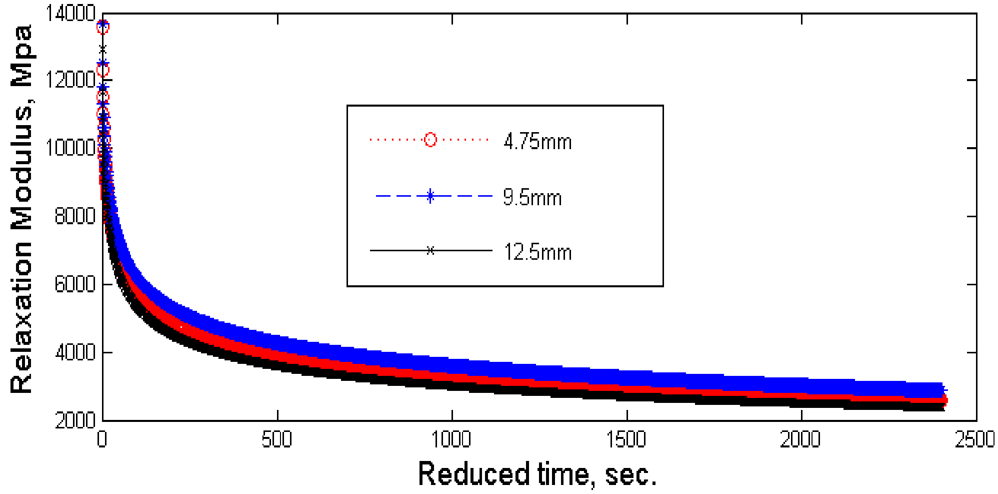


Figure 23: Comparison of relaxation modulus curves

Following the determination of relaxation moduli from three NMAS mixture samples, it is of interest to predict the thermal stresses in order to evaluate the effect of aggregate particle size on the thermal properties of asphalt mixtures. The calculated relaxation modulus is a function of time, so it needs to be converted to a temperature domain. The relationship between shift factors and temperature is given in Equation 16. Through linear regression techniques associated with the TTSP, the shift factors $a_T(T)$ and temperatures T are shown in Figure 21. Using such mathematical relations, relaxation moduli can be converted to as a function of temperature.

Recall the equation to determine thermal stresses of asphalt mixtures, Equation 10:

$$\sigma(T) = \int_0^T E(T - T')\frac{\partial \varepsilon(T)}{\partial T'}dT'$$

E(T-T') is the relaxation modulus (a function of temperature) that has been previously determined. The only parameter needed is ε(T) (strain at temperature T) and it is equal to α (coefficient of thermal contraction) multiplied by dT/dt (temperature increment).

Research by Bouldin et al. [2000] recommended that the coefficient of thermal contraction (α) for asphalt concrete can be selected as 1.7×10^{-4} mm/mm/°C, and 1°C per hour of the temperature increment (dT/dt) appeared to be appropriate in freeze regions.

Substituting the recommended values of α (coefficient of thermal contraction) and dT/dt (temperature increment) along with the determined relaxation modulus (Equation 9) into Equation 10, the thermal stresses of asphalt mixtures for three NMAS samples were predicted as shown in Figure 24.

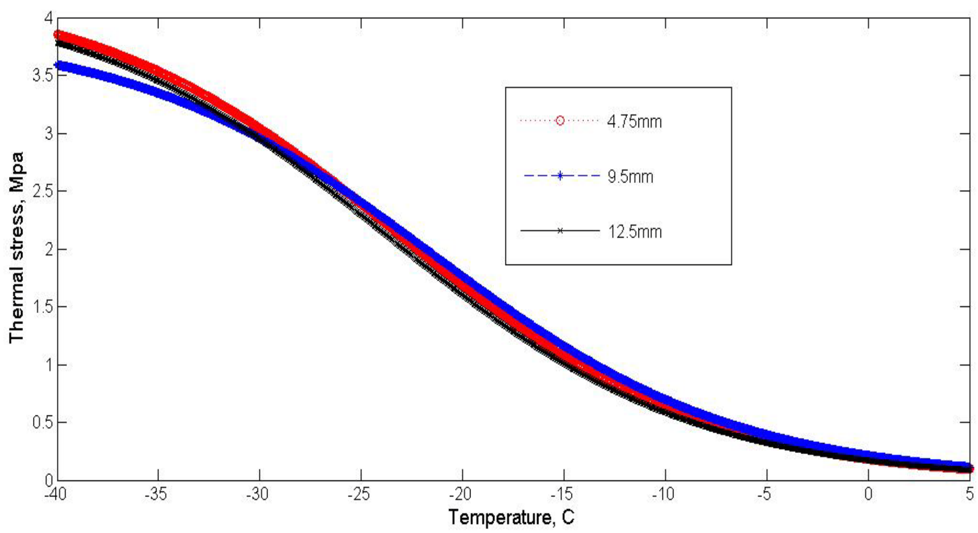


Figure 24: Thermal stresses of three NMAS samples

The strength of asphalt mixtures of 3.0 MPa was reported based on test done using an IDT by Bouldin et al. [2000], although higher values have also been measured. Using their results, the critical thermal cracking temperature can be determined to be approximately -30°C for the three NMAS samples. The thermal properties of asphalt mixtures among the three NMAS samples show reasonable agreement except for minor differences after the temperature drops below -34°C. In all cases, these differences are less than the 6 °C increments used to grade asphalt binder. Generally, the overall effect that aggregate size played in controlling the thermal resistance of asphalt mixtures has no engineering significance since the binder selected only works above -34°C. The thermal cracking property of asphalt mixtures is a function of temperature with no direct relation with a NMAS designed for the mixes. This addresses the issues of spatial distributions in representing the global properties of asphalt mixtures and validates the previous work of using small asphalt mixture beams to determine thermal properties of asphalt mixtures in the BBR tests. However, these comparisons are based on the assumption that the strength of the three mixtures is the same. Such assumption needs to be further validated.

7.6 Comparison between BBR and IDT Data

Zofka et. al (2005, 2008a, 2008b) showed that the results obtained from testing asphalt mixtures using the BBR were similar to those obtained using the IDT. In an effort to verify their results, asphalt mixture was obtained from a field project (US 6, MP 218.7 to Emma Park) during the summer of 2009. As will be discussed in Section 8.3, some field samples were shipped to the University of Utah Materials Laboratory for testing using the BBR. Material from the same project was shipped to the Asphalt Institute where it was tested using the IDT in accordance with AASHTO T-322. This provided an opportunity to compare the results of the mixture properties as determined from these two different tests. Three different conditions were tested by the Asphalt Institute: laboratory material, field material sampled at 320 ºF during August 3 (the first day of paving), and field material sampled at 373 ºF during August 7. The field material for August 3 and the lab material are compared in this section; the field materials from August 3 and 7 are discussed in Section 8.

Unfortunately, due to differences in testing protocols, the temperatures used in both sets of tests were not the same; tests using the BBR were run at -12 ºC, -18 ºC and -24 ºC while tests using the IDT were run at -20 ºC, -30 ºC, and -40 ºC. Furthermore, the actual IDT data was not provided so the numbers were read off a graph. Nevertheless, even though both tests use different temperatures and different loading modes, they both result in the determination of the creep compliance of the material as a function of time, thus at least a relative comparison is feasible.

Figure 25 shows the creep compliance as a function of time obtained from both tests at different temperatures for the field material sampled on August 3 and from the IDT for the laboratory material. It is noted that the creep compliance of the laboratory material at -20 ºC from the IDT is higher than the creep compliance from the field material. Such differences are not unusual between lab and field material since the actual preparation and conditioning of the materials differs. Of interest is the comparison of field material. Figure 25 shows that the creep compliance obtained from the IDT at -20 ºC is lower than the creep compliance from the BBR at -18 ºC and slightly higher than the value from the BBR at -24 ºC; this is a very favorable comparison. While many factors can contribute to differences in results observed, it is clear that the values obtained from both tests, while not identical, are comparable to each other. Obviously, separate calibration would be needed for each test if the data were to be used in performance predictions models (i.e., Mechanistic-Empirical Pavement Design input).

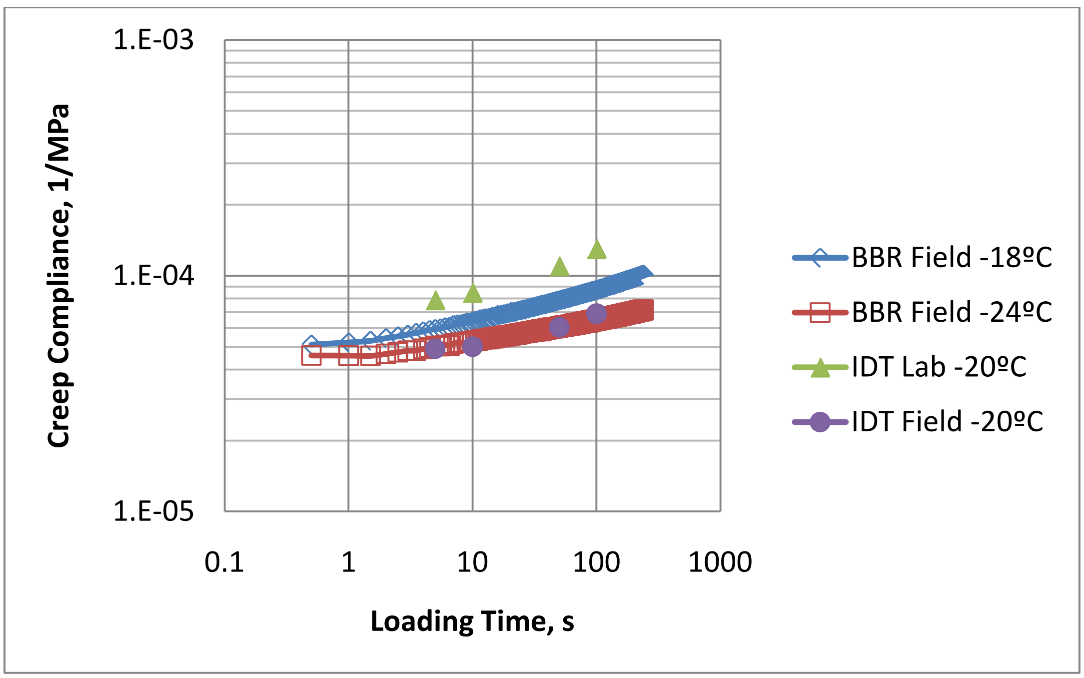


Figure 25: Comparison of creep compliance obtained from BBR and IDT (IDT tests were performed by the Asphalt Institute)

It is of interest to note that using the BBR allows for many replicate samples to be tested from one cylindrical specimen (see Section 4.5) while the IDT allows for a maximum of two samples from one cylindrical specimen (top and bottom). The BBR curves shown in Figure 25 are the average of 5 replicates with a coefficient of variation of 10%, while the reported IDT points are an average of three tests. The ability to have increased replication is an advantage of the proposed BBR testing protocols.

7.7 Summary

Asphalt mixture beams were created from three NMAS mixture samples and tested at different temperatures using the BBR. Time-Temperature Superposition Principle was employed to develop a master creep compliance curve. Nonlinear regression methods were used to fit the scattered experimental data and yield power law parameters. Relaxation moduli were derived through the Laplace transform processes and incorporated to determine the thermal stresses of asphalt mixtures.

Through the analysis work using the theory of linear viscoelasticity, the following conclusions are summarized:

1. As previous studies indicated, at low temperatures, the bulk properties of both aggregates and binder tend to converge. Based on the linear viscoelastic analysis using three NMAS samples, the role played by aggregate particle size in controlling thermal properties of asphalt mixtures is not significant for a given binder. The study confirms previous research's statements.
2. The viscoelastic analysis results indicate that the thermal properties of asphalt mixture beams are not affected by spatial distributions of aggregates within an asphalt mixture beam. The claim that the volume of a mixture beam is substantially occupied by the aggregates so that the results from BBR tests cannot represent the global properties of asphalt mixtures is not supported by this, and others', work.
3. Comparisons between creep compliance obtained using the BBR and creep compliance obtained using the IDT on the same asphalt mixture resulted in values that, while not identical, were comparable to each other.

The authors recognize that while aggregate nominal size was not a significant factor in the prediction of thermal stresses on a particular mix, it might affect its fracture properties; thus research on fracture properties is recommended.

8.0 Application of Using Asphalt Mixture Beams for QC/QA

As previously mentioned, thermal cracking has been a critical issue in the cold regions. One of the reasons that contribute to the decrease of the service life in highway pavements is the inconsistencies between design and construction. As discussed in Section 2, even though there are several tests that can evaluate thermal cracking of asphalt mixtures, highway agencies have not adopted any of them and thus cannot ensure that the material placed in the field has the same low temperature properties as the one that was originally submitted to the highway agencies for approval. Consequently, as was shown in Section 1.1, constructed pavements may not meet the desired requirements for serviceability and durability. The lack of practical material testing protocols for QC/QA operations at low temperatures is a major issue in the existing highway pavement construction and maintenance activities. However, as described in this report, the use of asphalt mixture beams in the BBR testing provides a promising material testing protocol that can address this issue.

This section presents examples on how to use asphalt mixture beams for the application of Quality Control/Quality Assurance (QC/QA) of asphalt mixtures. The quality validation of asphalt mixtures produced in the laboratory and in the roads is based on the comparisons of numerical and statistical analyses from data collected after the BBR testing. It is important to realize that when performing a statistical analysis for QC/QA, an acceptable range where data from field sampling are considered satisfactory must be defined based on both statistics and contractual agreements.

8.1 Determination of Confidence Intervals

The confidence interval is used in this report to establish a band where tested data should fall to be considered valid. The confidence interval (CI) is defined as:

$$CI = \bar{x} \pm t_{n-1}^{*} \times SE(\bar{x}) = \bar{x} \pm t_{n-1}^{*} \frac{s}{\sqrt{n}} \qquad (29)$$

Where:

$\bar{x}$ is the mean of samples

t^*_{n-1} is a critical value depending on the particular significance level and the number of degrees of freedom, n-1.

SE($\bar{x}$) is the standard error of the mean defined as

$$SE(\bar{x}) = \frac{s}{\sqrt{n}} \tag{30}$$

Where s and n denote the standard deviation and degrees of freedom, respectively.

The coefficient of variation (CV) was defined in Equation 3. Previous statistical analyses from Section 5.2 determined that the maximum CV was less than 0.2, so Equation 3 can be rewritten as:

$$S = 0.2 \cdot \bar{x} \tag{31}$$

Combining Equation 29 and Equation 31, we obtain:

$$CI = \bar{x} \pm t^*_{n-1} \times \frac{s}{\sqrt{n}} = \bar{x} \pm t^*_{n-1} \frac{0.2 \cdot \bar{x}}{\sqrt{n}} \tag{32}$$

The right term of Equation 32 indicates the information of confidence interval parameters for a given sample size. Their mathematical relation is depicted in Table 6. The CI upper and lower limits are as follows:

$$\text{Upper limit (UL)} = \bar{x} \cdot \left(1 + t^*_{n-1} \times \frac{0.2}{\sqrt{n}}\right) \tag{33}$$

$$\text{Lower limit (LL)} = \bar{x} \cdot \left(1 - t^*_{n-1} \times \frac{0.2}{\sqrt{n}}\right) \tag{34}$$

Applying Equations 33 and 34, the confidence intervals can be determined with two significance levels of 95% and 99% (Table 6). Using the UL and LL values, a band of confidence interval can be developed. This band can be used when running QC/QA operations.

Table 6: Number of samples selected versus CI

Significance level=0.95			Significance level=0.99		
n (samples)	t_{n-1}^{*}	CI, % of mean	n (samples)	t_{n-1}^{*}	CI, % of mean
5	2.132	19.1	5	3.747	33.5
10	1.833	11.6	10	2.281	14.4
15	1.753	9.1	15	2.624	13.6
20	1.725	7.7	20	2.539	11.4

Table 6 shows the maximum confidence interval, expressed as a percent of the mean, that can be expected based on the number of samples. Knowing what the actual values are, then the maximum difference between two samples can be determined.

8.2 Using BBR to Compare Samples from Two Laboratories

To illustrate a potential QC/QA process, two sample groups having 12.5mm NMAS and the same asphalt binder and aggregate were produced at two different laboratories to simulate a control and a field sample. The preparation of mixture beams followed the same procedures outlined earlier. Ten mixture beams were selected from each group to run BBR tests. Raw deflection data were obtained from the BBR computer program and then converted to relaxation modulus and thermal stresses using Equations 9 and 10.

Applying Equation 33 and 34, the values of the average relaxation modulus/thermal stresses, the upper limit (UL), and the lower limit (LL) for the control group were determined. The UL and LL were subsequently used to create a band of confidence interval. This band shows a confidence interval with the corresponding significance level (99%) that can be used to run the QC operations. The relaxation modulus and thermal stresses of 10 samples from the compared group were determined and plotted into the developed band of confidence interval as shown in Figures 26 and 27. It is noted that both the averaged relaxation modulus and thermal stress curves of the compared (field) group are located within the bands of the confidence interval created from the laboratory sample so that the quality of thermal cracking properties of the compared (field) group would be considered acceptable as compared with the control (lab) group. The evaluation in Figure 27 has been done in terms of thermal stress as a function of temperature, thus the probability of reaching a given stress can be evaluated given known weather data and used to predict the expected decrease in performance.

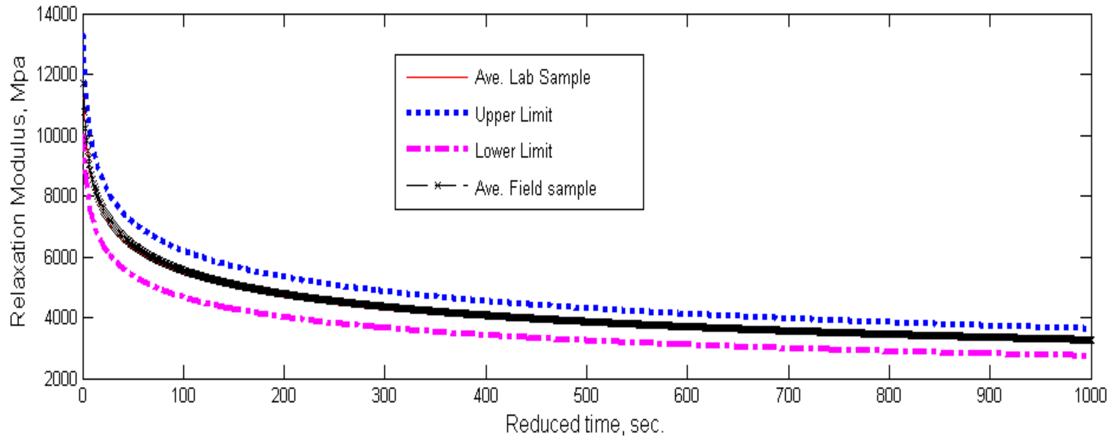


Figure 26: Application of confidence interval in relaxation modulus

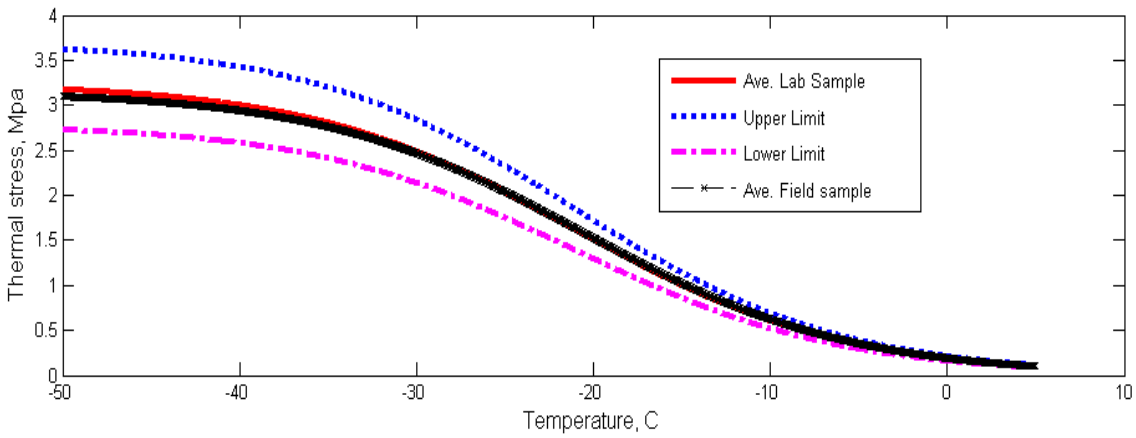


Figure 27: Application of a confidence interval in thermal stresses

The control group sample can be seen as an original mix design that was approved by a highway agency and tested in a laboratory prior to construction. The compared group sample is the asphalt mix placed in the roads during construction. Both highway agency and contractor can collect asphalt mixtures at the end of the paver and make SGC samples in their laboratories. By following the procedures described in this report, both UDOT and contractor can compare the results of in-field samples with the original mixtures carried out in laboratories. As shown in Figures 26 and 27, if the BBR results show that field samples (black line) are within the band of the confidence interval in terms of relaxation modulus or thermal stresses, then the quality asphalt mixture placed during construction comes from the same population as the laboratory results, thus ensuring that the requirements are met; otherwise the thermal properties of asphalt mixtures placed in the roads may have potential problems that need to be further evaluated. As previously mentioned, since the exact thermal properties of the material are determined, a penalty based on its specific performance can be assessed instead of having fixed values. This allows for innovation yet ensures that the material placed in the field has the same characteristics as the material approved by the agency. This is one of the methods from this study that can be used by highway agencies for the QC/QA.

8.3 Using the BBR as a QC Tool During Daily Paving Operation

Throughout this research, the BBR is used to determine, through numerical analysis, the low temperature properties of asphalt mixtures. As shown in the previous section, the performance of the mixture can be predicted and a penalty can be assessed based on the decrease in performance due to non compliance. Such approach, while powerful, requires significant effort and time. A simpler approach with minimal numerical analysis is desired for most situations. However, this approach must be consistent with UDOT’s MOI Part 8 Section 1011: Materials Acceptance Program. The following section provides an example of such approach by using BBR testing to construct control charts.

8.3.1 Testing of a Field Project

During the month of August of 2009, a paving project located on US-6 (MP 218.7 to Emma Park) was evaluated using the BBR as a way to demonstrate its applicability in day-to-day operations. As it is typical for UDOT projects, the asphalt mixture was sampled behind the paver and taken to the regional laboratory where it was reheated and compacted into the standard 150-mm diameter samples using the Superpave gyratory compactor. The compacted samples were then tested for density as a means to ensure all volumetric properties were met. Based on the laboratory reports, the mixes were satisfactory in meeting the volumetric requirements.

Once the project was completed, the compacted samples were obtained and transported to the Materials Laboratory at the University of Utah. No information was given to the researchers other than the mixture met all volumetric requirements. For each paving day, one sample was cut based on the protocols shown on Figure 10. Given the size of the beams tested in the BBR, over 15 valid samples (as defined in Section 5) were easily obtained. All samples were tested at a temperature of -24 °C, which corresponds to the low temperature PG of the binder plus 10 degrees. The BBR software automatically reported stiffness and the m-value (slope of the curve) at 60 seconds. These values were plotted in a control chart without further data manipulation. The results of the tests run at the University of Utah are shown on Figure 28.

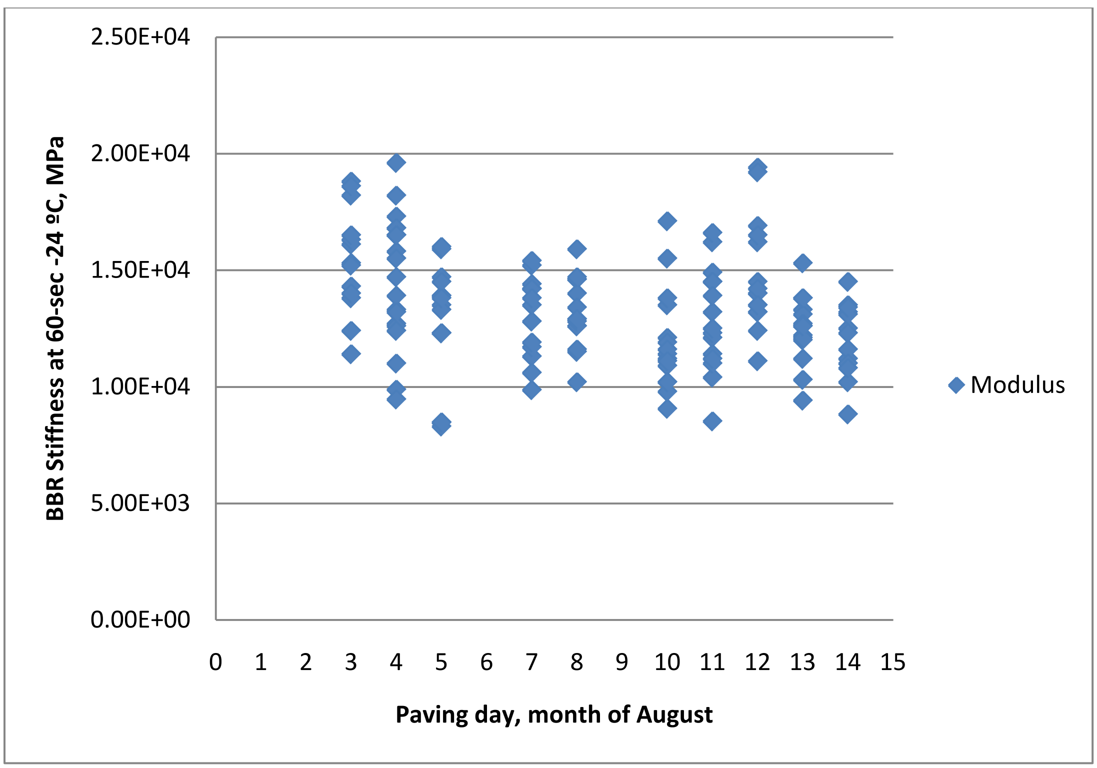


Figure 28: Control chart for Stiffness at 60 seconds

As can be seen on the figure, the average modulus (or stiffness) of the material varies roughly between 12,000 MPa and 15,000 MPa, resulting in range of values of approximately 20%. This variation is consistent with what was shown in Section 5 of this report (note that this is raw data obtained from the BBR output). By looking at the chart, there is a decrease in average stiffness after August 5. It can be argued that such a decrease in average stiffness would lead to further testing; however, such determination can only be made after many more projects are evaluated and trends from multiple production days are understood.

A second parameter that is directly obtained from the test is the slope of the stiffness curve at 60 seconds. This is shown on Figure 29. As can be seen in the figure, there seems to be an anomaly on the 7th and 8th of August. The m-value during those two days seems higher than during the rest of the days. This anomaly could be easily verified with the appropriate statistical analysis; however, recall that the idea behind this section is to do as little analysis as possible.

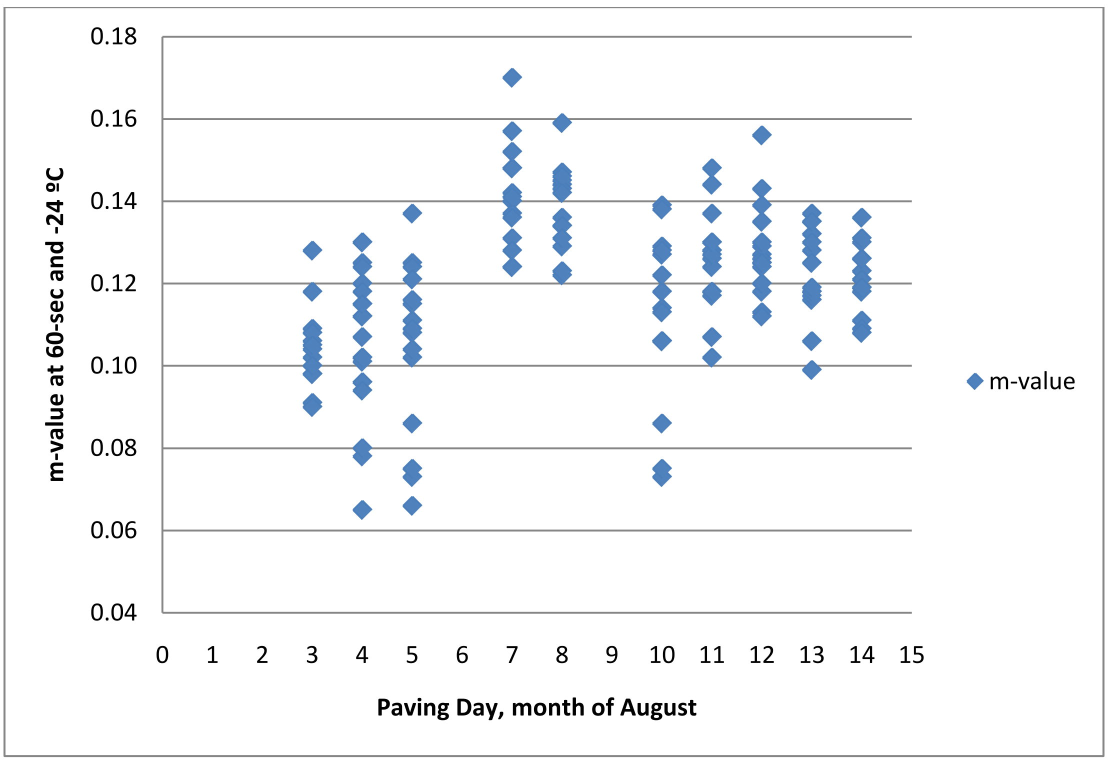


Figure 29: Control chart for m-value at 60-sec

The results from August 7 and 8 were enough to raise suspicion on the data so the researchers went back to the records, talked to the field engineer, and talked to the plant operator. Through these discussions, it was learned that during those two days, the mixture arrived at the project at a temperature 50 °F (27 °C) higher than the rest of the days. Apparently, there were some operational problems at the plant that were not discovered on time. Table 7 shows the mix temperature during the duration of the project based on field records.

Table 7: Arriving Mix Temperature

Paving Day August	**Mix Temperature °F**
4	320
5	326
7	373
8	320*
10	337
11	320
12	332
13	330
14	321

* field personnel observed temperature at 370 ºF

It is well known that changes in temperature during asphalt concrete production is detrimental to its properties and that it might lead to a decrease in low temperature performance. Such effect is not always evident at high temperatures where current tests are run but, as shown on Figures 28 and 29, it can be detected using the BBR. Had BBR testing of mixtures been in place during this project, it would have raised a flag during August 7 resulting in more detailed inspection of the operation and further testing to determine the decrease in pavement performance as a result of overheating the material.

However, while the data from the BBR shows an anomaly during two paving days for which higher arriving mix temperature were later confirmed with field personnel; the actual trend is contrary to what is expected. When asphalt materials are heated, they tend to harden; thus an increase in modulus (or stiffness) and a corresponding decrease in m-value are expected. This was not the case in this set of data; thus more testing results had to be analyzed. As was discussed in Section 7.6, samples from the same project were sent to the Asphalt Institute for testing in the IDT. One set of samples corresponded to a day where the reported temperature was within normal parameters (August 3), while the other set corresponds to the August 7 paving materials. Figure 30 shows a comparison between both days.

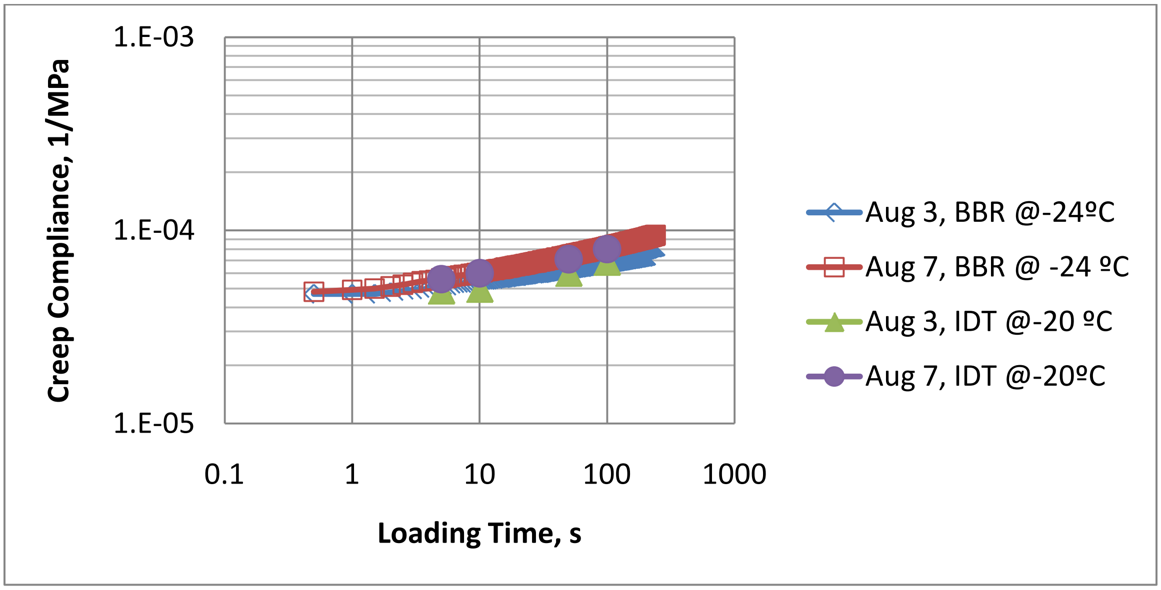


Figure 30: Comparison of Creep Compliance from 2 different production days

Figure 30 shows that the compliance of the material is higher on August 7 than it is on August 3. In other words the material got softer. The increase in compliance was captured by the tests done at the University of Utah using the BBR as well as the tests done by the Asphalt Institute using the IDT. The authors do not have an explanation for this behavior; perhaps field records are not accurate, perhaps the material was mislabeled, or perhaps it is simply a statistical anomaly.

Nevertheless, regardless of situations that lead to the materials becoming more compliant, it is clear that the BBR is capable of measuring changes in material properties just as the IDT would. The only difference is that the BBR can do it quicker, easier, and less expensive.

8.4 Percent within Limits

The Percent within Limits (PWL) method uses the mean and the standard deviation to quantify the amount of material that is within given specification limits (target values ± tolerances) based on limited sampling [AASHTO, 1996]. Pay factors, which include incentives (bonuses) and disincentives (penalties) are assigned for different PWL values and serve as a basis for payment. The test proposed as part of this work is ideal for PWL applications since, out of one field core or gyratory sample, several beams can be obtained allowing for statistically robust results. Stroup-Gardner et al. [1994] suggest that limits be based on known test variability; thus for the proposed test the limit should be 20%.

The method for calculating PWL is as follows:

- Obtain a gyratory sample already compacted and used in the verification of mixture volumetrics.
- Cut the sample according to the procedure shown in Figure 10.
- Test at least 5 valid beams using the BBR at a predetermined temperature (Binder PG +10 ºC has been suggesting to maintain consistency with binder grading). The BBR software automatically reports stiffness and m-value at 60-seconds so use this value for control.
- Find the sample average, $\bar{x}$.
- Find the sample standard deviation, s.
- Given that the test variability has been determined as 20%, the limits will be $\pm 0.2\bar{x}$. The quality index is calculated as:

  $$QI = 0.2\bar{x} / s \tag{35}$$
- Estimate the PWL based on existing statistical tables.

See Appendix A for a proposed specification incorporating the PWL method.

8.5 Summary

This section shows three approaches that can be easily implemented to evaluate pavement performance at low temperatures and sets the format for a new specification.

In the first approach, the BBR can be used to develop master curves and predict the critical cracking temperature of two materials. These two materials can be the same material but prepared in two separate laboratories or can be material from two potential sources (typically field and lab).

In the second approach, the BBR can be used to test the samples that are already being fabricated and develop a control chart to ensure the consistency of day-to-day paving operations. Any deviation from the control chart should lead to further inquiries regarding the paving operation. If needed, an inconsistency in the control chart should trigger further testing as described in the first approach.

In the third approach, Percent within Limits values are determined based on daily production. This is an extension of the second approach in which tests are run based on sampling.

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9.0 Conclusions and Recommendations

The work presented in this report shows a methodology for controlling the low temperature properties of asphalt mixtures using the BBR.

9.1 Summary

Based on the research work, the following findings are summarized:

1. The existing material testing methods for predicting low temperature properties of asphalt mixtures are not practical in construction QC/QA. Using asphalt mixture beams in the BBR test was shown to be an effective tool to evaluate thermal-induced cracking performance of asphalt concrete in a simple yet accurate manner.

2. Statistical factors were taken into consideration for the determination of the number of replicates to reach a valid test. In practice, both state agency and contractor can reach an agreement as to what degrees to which the comparisons and statistical results from the BBR tests are accepted or rejected. This will depend on the level of risk that each entity is willing to take. However, it should be noted that, in contrast to other tests, the simplicity and sample size used in the BBR tests allow for multiple replicates from one gyratory sample.

3. The sources of variability that correspond to the experiments including homoscedasticity of variances, dimensional issues, gauge length requirements, etc. were addressed. The application of the band of the confidence interval presented in Section 8 provides a methodology that is capable of evaluating the thermal properties of asphalt mixtures shortly after construction so that QC can be performed during construction and QA can be done based on existing programs.

4. As previous studies indicated, at low temperatures the bulk properties of both aggregates and binder tend to converge. Based on the linear viscoelastic analysis using three NMAS samples, the role played by aggregate particle size in

controlling thermal properties of asphalt mixtures is not significant for a given binder.

5. Comparisons were made between asphalt mixture compliance obtained from BBR testing and IDT. The results show that, while the results are not identical, they are comparable to each other. Both pieces of equipment result in the same trend.

6. The study confirms previous research's statements and validates the experimental results used in the BBR tests.

9.2 Conclusions

Based on the findings of this research, it is concluded that the Bending Beam Rheometer is a viable test to determine the low temperature properties of asphalt mixtures. It is concluded that the size of the specimen used for testing is not an issue regarding the validity of the test. Finally, it is concluded that the BBR can be used for day-to-day QC applications.

9.3 Recommendations

1. It is recommended that the BBR be adopted as a method to control low temperature properties of asphalt concrete. A draft specification is provided but it will require monitoring of field projects to develop actual limits.

2. While aggregate nominal size was shown to be not a significant factor in the prediction of thermal stresses on a particular mix, it might affect its fracture properties; thus research on fracture properties is recommended to further evaluate the thermal-induced cracking behavior of asphalt materials.

3. This study includes one field section to demonstrate the capabilities of BBR testing. It is recommended that a field study be conducted where samples are tested using the protocols described in this document so that specification limits can be obtained. Long term monitoring of field projects is also desirable to validate the performance predictions of the measurements obtained using the BBR.

10.0 References

American Association of State Highway and Transportation Officials. *Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens*. Standard Specifications for Transportation Materials and Methods of Sampling and Testing T 166. AASHTO 29th edition, 2009.

American Association of State Highway and Transportation Officials. *Standard Test method for Indirect Tensile (IDT) Strength of Bituminous Mixtures*. Standard Specifications for Transportation Materials and Methods of Sampling and Testing T 322. AASHTO 29th edition, 2009.

American Association of State Highway and Transportation Officials. *Standard Method for Testing Asphalt Mixtures Using the Thermal Stress Restrained Specimen Test (TSRST).* Standard Specifications for Transportation Materials and Methods of Sampling and Testing TP10. AASHTO 29th edition, 2009.

American Association of State Highway and Transportation Officials. *Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)*. Standard Specifications for Transportation Materials and Methods of Sampling and Testing T 313. AASHTO 29th edition, 2009.

American Society of Testing and Materials. *Standard Test method for Indirect Tensile (IDT) Strength of Bituminous Mixtures*, D6931-07 ASTM, 2008.

American Society of Testing and Materials. *Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)*. D6648-08, ASTM, 2008.

Anderson, D., Christensen, D.W., Bahia, H.U., Dongre R., Sharma, M.G., Antle, C.E., and Button, J. *Binder Characterization and Evaluation*. SHRP Report: SHRP-A-39, Vol. 3, Washington, D.C., 1994.

Andriessou, A. and Hesp, S.A. M. *Time-Temperature Superposition in Rheology and Ductile Failure of Asphalt Binders*. In the Journal of International Pavement Engineering, July 2008, pp.1-12.

Bahia, H.U. and Anderson, D.A. *The Development of the Bending Beam Rheometer: Basic and Critical Evaluation of the Rheometer*. Physical Properties of Asphalt Cement Binders, American Society of Testing and Materials, 1995, STP 1241, pp. 28-50.

Binns, R.D. and Mygind, H.S. *The Use of Electrical Resistance Strain Gauges and the Effect of Aggregate Size on Gauge Length in Connection with the Testing of Concrete*. Magazine of Concrete Research. January 1949, No. 1, pp.35-39.

Bouldin, M.G., Dongre, R., Rowe, G.M., Sharrock, M.J., and Anderson, D.A. *Predicting Thermal Cracking of Pavements from Binder Properties: Theoretical Basis and Field Validation*. In the Journal of the Association of Asphalt Paving Technologists, Asphalt Paving Technology, Vol. 71, 2000, pp. 455-495.

Brostow, W., D'Souza, N.A., Kubat, J., and Maksimov, R. *Creep and Stress Relaxation in a Longitudinal Polymer Liquid Crystal: Prediction of the Temperature Shift Factor*. In the Journal of Chemical Physics, Vol. 110, No. 19, 1999, pp.9706-9712.

Bouldin, M.G., Dongre, R., Rowe, G.M., Sharrock, M.J., and Anderson, D.A. *Predicting Thermal Cracking of Pavements from Binder Properties: Theoretical Basis and Field Validation*. In the Journal of the Association of Asphalt Paving Technologists, Asphalt Paving Technology, Vol. 71, 2000, pp. 455-495.

Buttlar, W.G. and Roque, R. *Determination and Evaluation of the Strategic Highway Research Program Measurement and Analysis System for Indirect Tensile Testing at Low Temperature*. In the Journal of Transportation Research Board No. 1454, Transportation Research Record of the National Academies, Washington D.C., 1994, pp.163-171.

Campen, W.H., Smith, J.R., Erickson, L.G., and Mertz, L.R. *The Relationship Between Voids, Surface Area, Film Thickness, and Stability in Bituminous Paving Mixtures.* Proceedings, Association of Asphalt Paving Technologist, Volume 34, 1965

Chehab, G.R. and Kim, Y.R. *Interrelationships among Asphalt Concrete Stiffnesses*. Modeling of Asphalt Concrete, ASCE Press, McGraw-Hill Inc, 2009.

Christensen, R.M. *Theory of Viscoelasticity*. 2nd edition. Dover Inc., New York, 1982.

Christensen, D.W. and Anderson, D.A. *Interpretation of Dynamic Mechanical Test Data for Paving Grade Asphalt Cements*. In the Journal of the Association of Asphalt Paving Technologists, 1992, Vol. 61, pp. 67-116.

Christensen, D.W. and Bonaquist, R. F. *Evaluation of Indirect Tensile Test (IDT) Procedures for Low-Temperature Performance of Hot Mix Asphalt*. NCHRP Report No. 530, Transportation Research Board of the National Academies, Washington, D.C., 2004.

Christensen, D. W, and Mehta, Y. A. *Reference Standards for Use with Indirect Tension Test*. In the Journal of Transportation Research Board No. 1630, Transportation Research Record of the National Academies, Washington D.C., 1998a, pp.37-41.

Christensen, D.W. Analysis *of Creep Data from Indirect Tension Test on Asphalt Concrete*. In the Journal of the Association of Asphalt Paving Technologists, Asphalt Paving Technology, Vol. 69, 1998, pp. 458-492.

Cost, T. L., and Becker, E. B. *A Multidata Method of Approximate Laplace Transform Inversion*. In the International Journal of Numerical. Methods in Energy, Vol. 2, 1970, pp. 207–219.

De Veaux, R. D., Velleman, P. F., and Bock, D. E. *Stats: Data and Models*. Pearson Education Inc., 2005.

Du, X, and Ostoja-Starzewski, M: *On the scaling from statistical to representative volume element in thermoelasticity of random materials*. Networks and Heterogeneous media, Volume 1, No2, Pp 259-274. June 2006.

Elseifi, M.A., Dessouky, S.H., Al-Qadi, I.L., and Yang, S.H. Viscoelastic Model to Describe Mechanical Response of Bituminous Sealants at Low Temperature. In the Journal of Transportation Research Board No. 1598, Transportation Research Record of the National Academies, Washington D.C., 2006, pp.82-89.

Epps, A.: "An Approach to Examine Thermal Fatigue in Asphalt Concrete. Association of Asphalt Paving Technologist, Volume 68, 1999.

Ferris, C.L., Grubbs, F.E., and Weaver, C.L. *Operating Characteristics for the Common Statistical Tests of Significance*. Annuals of Mathematical Statistics, June 1946, pp. 178-197

Findley, W.N., Lai, J.S., and Onaran, K. *Creep and Relaxation of Nonlinear Viscoelastic Materials*. 2nd edition. Dover Inc., New York, 1982.

Franck, A.J. *Guidelines for Using TTS*. Technical Report PN001, Texas Instruments. www.tainstruments.com/library_download.aspx. Accessed in July 2008.

Hashin, Z.: A *Variational Approach to the Theory of Elastic Behavior of Multiphase Materials*. Journal of the Mechanics and Physics of Solids. Vol 11, Issue 2. Pp. 127-140. April 1963.

Heiberger, R. M., and Holland, B. *Statistical Analysis and Data Display*. Springer, 2004

Hiltunen, D.R, and Roque, R. The Use of *Time-Temperature Superposition to Fundamentally Characterize Asphalt Concrete Mixtures at Low Temperatures*. Engineering Properties of Asphalt Mixtures and the Relationship to Their Performance. The American Society of Testing and Materials, STP 1265, 1995, pp.74-96.

Jung, D., and Vinson, T.S. *Low Temperature Resistance of Asphalt Concrete Mixtures*. Asphalt Paving Technology, Journal of the Association of Asphalt Paving Technologists, 1993, Vol. 62, pp. 54-92.

Kanit, T, Forest, S, Galliet, I., Mounoury, V, and Jeulin, D: *Determination of the Size of the Representative Volume Element for Random Composites: Statistical and Numerical Approach*. International Journal of Solids and Structures. Volume 40. Pp 3647-3679. June-July 2003

Kim, J. S. *Complex Modulus from Indirect Tension Testing*. Master's Thesis, University of Florida, Gainesville, FL, 2002.

Kim, J., Scolar, G.A., and Kim, S. *Determination of Accurate Creep Compliance and Relaxation Modulus at a single Temperature for Viscoelastic Solids*. In the Journal of Materials in Civil Engineering, ASCE, February, 2008, pp. 147-156.

Lehman, E. *Theory of Point Estimation.* Wadsworth and Brooks Cole, CA, 1991

Lytton, R., Uzan, J., Fernando, E. G., Roque, R., Hiltunen, D., and Stoffels, S. *Development and Validation of Performance Prediction Models and Specifications for Asphalt Binders and Paving Mixes*. SHRP Report: SHRP-A-357, Washington, D.C., 1993.

Marasteanu, M., Zofka, A., Turos, M., Li, X., Velasques, R., Li, Xue., Buttlar, W., Paulino, G., Braham, A., Dave, E., Ojo, J., Bahia, H., Williams, R.C., Bausano, J., Gallistell, A., and McGraw, J. *Investigation of Low Temperature Cracking in Asphalt Pavements.* National Pooled Fund Study 776. Minnesota Department of Transportation Report MN/RC 2007-43. 2007

Marasteanu, M., Basu, A., Hesp, S.A.M., and Voller, V. *Time-Temperature Superposition and AASHTO MP1a Critical Temperature for Low-temperature Cracking*. Journal of International Pavement Engineering, Vol. 5, March 2008, pp.31-38.

Marasteanu, M., Velasquez, R., A. Cannone Falchetto, and Zofka, A. Development of a Simple Test to Determine the Low Temperature Creep Compliance of Asphalt Mixtures. NCHRP Idea 133, 2009.

Marshall, A.L. *A New Strain Meter for Concrete*. Journal of Strain Analysis for Engineering Design, Vol. 8, No. 1, 1973, pp.19-23.

Masad, E., Jandhyala, K., Dasgupta, N., Somadevan, N., and Shashidhar, N.: *Characterization of Air Voids Distribution in Asphalt Mixtures using X-ray Computed*

Tomography. Journal of Materials in Civil Engineering. Vol 14, Issue 2. Pp 122-129. March/April 2002

Masad, E., Jandhyala, V. K., Dasgupta, N., and Somadevan, N. *Characterization of Air Void Distribution in Asphalt Mixes using X-ray Computed Tomography*. Journal of Materials in Civil Engineering, ASCE, 2002, Vol. 14, No. 2, pp. 122-129

NijBoer, L.W. Plasticity as a Factor in the Design of Dense Bituminous Road Carpets. Elsevier Publishing Company, The Netherlands, 1948.

Nukunya, B., Roque, R., Tia, M., and Birgisson, B.: Evaluation of VMA and other Volumetric Properties as Criteria for the Design and Acceptance of Superpave Mixtures. Journal of the Association of Asphalt Paving Technologist. Volume 70. Pp 38-69. 2001

Ostoja-Starewski, M: *Materials Spatial Randomness: From Statistical to Representative Volume Element*. Probabilistic Engineering Mechanics, ISSN 0266-8920, Vol 21, No 2. Pp 112-132, 2006

Park, S.W., and Kim, Y.R. *Fitting Prony-Series Viscoelastic Models with Power-Law Presmoothing*. In the Journal of Materials in Civil Engineering, January/February, 2001, pp. 26-32.

Radovskiy, B.: *Analytical Formulas for Film Thickness in Compacted Asphalt Mixture.* Journal of the Transportation Research Board, Volume 1829. Pp 26-32. 2003

Reese, R. : *Properties of Aged Asphalt Binder Related to Asphalt Concrete Fatigue Life*. Journal of the Association of Asphalt Paving Technologist, Volume 66. Pp. 604-632. 1997.

Reinke, G. and Glidden, S.: *Development of Mixture Creep Performance Tests Using Dynamic Shear Rheometer.* Transportation Research Board Circular E-C068: New Simple Performance Tests for Asphalt Mixtures. Pp 1- 43. Sept. 2004.

Romero, P, and Anderson, R.M. *Variability of Asphalt Mixtures Tests using the Superpave Shear Tester Repeated Shear at Constant Height Test*. CD-ROM. Paper: 01-2098, the Annual Meeting of the Transportation Research Board, Washington, D.C., 2001.

Romero, P. and Masad, E.: *Relationship Between the Representative Volume Element and Mechanical Properties of Asphalt Concrete*. Journal of Materials in Civil Engineering. Vol 13, No 1. Pp. 77-84. 2001

Romero, P., Stuart, K., and Mogawer, W.: “Fatigue Response of Asphalt Mixtures Tested by FHWA’s Accelerated Loading Facility.” Journal of the Association of Asphalt Paving Technologist, Volume 69. 2000. Pp 212-235

Roque, R., Hiltunen, D.R., and Buttlar, W.G. “Thermal Cracking Performance and Design of Mixtures Using Superpave.” Journal of the Association of Asphalt Paving Technologist, Volume 64. 1995. Pp 718-735

Roque, R. and Hiltunen, D.R.: “ Use of Canadian SHRP Test Sections to Evaluate the SHRP Indirect Tensile Creep and Failure Test for Control of Thermal Cracking.” Proceedings of the Canadian Technical Asphalt Association, Vol XXXIX. 1994. Pp 441-464

Salkind, N.J. *Statistics:for people who think they hate statistics*. 2nd Edition, SAGE Publications Inc. UK, 2007

Schapery, R.A. *Approximate Methods of Transform Inversion for Viscoelastic Stress Analysis*. In the Proceeding of 4th U.S. national Congress of Applied Mechanics, 1962

Schapery, R. A. *A Simple Collocation Method for Fitting Viscoelastic Models to Experimental Data.* Report GALCIT SM 61-23A, California Institute of Technology, Pasadena, CA, 1961.

Schwartz, C.W., Gibson, N. and Schapery, R. A. *Time-Temperature Superposition for Asphalt Concrete at Large Compressive Strains.* In the Journal of the Transportation Research Board, No. 1789, Transportation Research Board of the National Academies, Washington, D.C., 2002, pp. 101–112.

Schwarzl, F., and Staverman, A. *Time-Temperature Dependence of Linear Viscoelastic Behavior*. In the Journal of Applied Physics, Vol. 23, No.8, 1952, pp.838-843.

Secor, K., and Monismith, C.L. *Analysis and Interrelation of Stress-Strain-Time Data for asphalt Concrete*. In the Journal of Transaction of the Society of Rheology, Vol. 3, 1964, pp. 19-32.

Shields, D., Zeng, M., and Kwok, R. *Nonlinear Viscoelastic Behavior of asphalt Concrete in Stress Relaxation*. In the Journal of the Association of Asphalt Paving Technologists, Asphalt Paving Technology, Vol. 69, 1998, pp. 359-400.

Stroup-Gardiner, M., Newcomb, D., and Savage, D.: Defining Limits with Respect to Testing Variability. In the Journal of the Association of Asphalt Paving Technologists, Asphalt Paving Technology, Vol. 63, 1994, pp. 359-400.

Tayebali, A.A. et al. Fatigue Response of Asphalt-Aggregate Mixes. Final Report, SHRP-A-404. National Research Council, 1994

The Asphalt Institute. Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types. MS-2. 6th ed. 1995.

Velasques, R., Marasteanu, M., Turos, <., and Labuz, J.: *Effect of Beam Size on the Creep Stiffness of Asphalt Mixtures and Low Temperatures*. Advanced Testing and Characterization of Bituminous Material (ATCBM09), Loizos, Parti, Scarpas, and Al-Qadi, Eds. ISBN 978-0-415-55854-9 Pp. 313-322 (May 2009).

Weissman, S.L., Harvey, J., Sackman, L., and Long, F.: *Selection of Laboratory Test Specimen Dimension for Permanent Deformation of Asphalt Concrete Pavements*. Journal of the Transportation Research Board. No 1681. Pp. 113-120. 1999

Window, A.L. *Strain Gauge Technology*. Elsevier Applied Science, 2nd edition, New York, 1992

Zar, J.H. *Biostatistical Analysis*. Fourth edition. Prentice Hall. 1999.

Zhao, Y. and Kim, Y. R. *Time-Temperature Superposition for Asphalt Mixtures with Growing Damage and Permanent Deformation in Compression*. In the Journal of the Transportation Research Board, No. 1832, Transportation Research Board of the National Academies, Washington, D.C., 2003, pp. 161–172.

Zofka, A., *Investigation of Asphalt Concrete Creep Behavior Using 3-Point Bending Test*. PhD thesis, University of Minnesota, Minneapolis, July 2007

Zofka, A., Marasteanu, M. O., Li, Xinjun, Clyne, T. R., McGraw, J. *Simple Method to Obtain Asphalt Binders Low Temperature Properties from Asphalt Mixtures Properties*. Journal of the Association of Asphalt Paving Technologists, 2005, Vol. 74, pp. 255-282.

Zofka, A., Marasteanu, M.O., and Turos, M.: *Determination of Asphalt Mixture Creep Compliance at Low Temperatures by Using Thin Beam Specimens*. Journal of the Transportation Research Board. No 2057. Pp 134-139. 2008a

Zofka, A., Marasteanu, M.O., and Turos, M.: *Investigation of Asphalt Mixture Creep Compliance at Low Temperatures*. Road Materials and Pavement Design, Volume 9, Pp. 269-286. 2008b

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Appendix A: Draft Specification for Controlling Low Temperature Properties of Asphalt Mixtures Using the Bending Beam Rheometer

This appendix shows a draft specification to demonstrate how this work can be implemented as part of asphalt mixtures design and quality control. Questions regarding the validity of the tests have been thoroughly evaluated in this report; however, specification limits need to be developed. Such limits can only be obtained experimentally by monitoring field projects.

In this appendix, some values are highlighted and shown as examples only. They will need to be determined experimentally.

XXX Low Temperature Properties of Asphalt Mixtures

XXX.01 Scope

This standard specifies requirements and procedures to determine the low temperature properties of asphalt mixtures using the bending beam rheometer.

XXX.02 Referenced Documents

AASHTO R30, Standard Practice for Mixture Conditioning of Hot Mix Asphalt (HMA)

AASHTO T313, Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR).

XXX.03 Apparatus

- See AASHTO T313 requirements for a BBR
- Masonry saw capable of cutting through 6 inch diameter cylindrical asphalt concrete samples prepared using the Superpave Gyratory Compactor (SGC) or cored from the road.
- Quality tile saw capable of cutting asphalt concrete beams 12.7 x 6.35 x 127 mm

XXX.04 Sample Preparation

- Obtain an asphalt concrete sample (150 mm in diameter and 110 mm in height if compacted using the SGC) that is representative of the material to be placed on the road. Alternatively, cored samples are acceptable.
- Cut the sides of the sample using a masonry or lapidary saw. This is a rough cut and is made to reduce the size of the sample so it can be cut more precisely with a tile saw.
- Cut the resulting block into several flat slabs using a tile saw. The slab thickness should be 12.7 ± 0.25 mm; the length should be greater than 115 mm to accommodate testing; the width will vary depending on the location where the flat slab is cut from.
- Cut the flat slabs into thin beams with a width of 6.35± 0.25 mm.

- Measure the beam dimensions (width and thickness) at the third points; the difference in the dimensions of the beam (width and thickness) from one end to the other shall not exceed 0.5 mm. Discard those beams that do not meet these tolerances.

It has been found that better results are obtained when the samples are placed in a freezer for at least 4 hours prior to cutting.

XXX.05 Testing Procedures

Follow the procedures described in AASHTO T313 (ASTM D6648) for testing asphalt binders. A load of 450 grams can be applied without need to modify the BBR bearing system.

XXX.05.01 Test Temperatures For quality control purposes the single test temperature shall be 10 ºC above the specified binder grade used in the mixture. For performance prediction at least 3 temperatures shall be used at 6 ºC intervals. The test temperatures of 4 ºC, 10 ºC, and 16 ºC above the specified binder grade used in the mixtures have been successfully used. Other temperatures can also be used depending on the project requirements.

XXX.05.02 Sample Replicates A minimum of five sample replicates shall be tested at each condition. Fifteen to twenty samples can be obtained from a single gyratory specimen.

XXX.06 Measurements

Input the necessary parameters for BBR testing. Once the test is started, the BBR software automatically records the load and deformation of the sample beam as a function of time. Export the data related to one mixture for performance analysis. At the end of the test the software automatically reports the modulus or stiffness and the m-value at 60-seconds. Record this value and use it for quality control.

XXX.07 Analysis

XXX.07.01 For quality control purposes, obtain mixture representative of the project and follow the instructions for sample preparation and testing. Perform the analysis on the stiffness and m-value at 60-seconds.

- Find the sample average, x, and standard deviation, s. If the standard deviation of 5 samples is greater than 15% of the mean, determine if there is an outlier. An outlier is defined as a measurement that is 2 standard deviations from the mean. Remove the outlier value from the calculation. If the standard deviation of the 4 remaining values is still greater than 15% of the mean, you must repeat the test using a new set of samples.
- Compare the mean and standard deviation of the tests to the values obtained during mixture design using a paired t-test as shown on Appendix C of UDOT's MOI Part 8.
- To determine the percent within limits (PWL), consider the overall variability as 20%, thus the limits will be ±0.2x and the quality index is calculated as: QI = 0.2x /s.
- Estimate the PWL based on existing statistical tables.

XXX.07.02 For mixture design purposes, prepare samples that are representative of the mixture being evaluated. Perform the analysis on the stiffness and m-value at 60-seconds.

- Find the sample average, x, and standard deviation, s. If the standard deviation of 5 samples is greater than 15% of the mean, determine if there is an outlier. An outlier is defined as a measurement that is 2 standard deviations from the mean. Remove the outlier value from the calculation. If the standard deviation of the 4 remaining values is still greater than 15% of the mean, you must repeat the test using a new set of samples.
- The average stiffness of the mixture at 60-seconds and at a temperature of 10 ºC above the performance grade of the binder shall not exceed 15,000 MPa; the average m-value at the same loading time and temperature shall not exceed 0.12.
- If the mixture exceeds the limits, it is too stiff at low temperatures and will likely fail prematurely. It should be re-designed.

XXX.07.03 For performance testing, the complete set of data containing load-deformation as a function of time at three or more different temperatures is needed. This data is converted to creep compliance which is used to predict thermal stresses as a function of temperature. The point where the thermal stress reaches the strength of the material, estimated at 3 MPa, is considered the cracking temperature.

Note: at present there is no method to determine the strength of the asphalt mixture using the BBR test.

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