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USING THE BENDING BEAM RHEOMETER FOR LOW TEMPERATURE TESTING OF ASPHALT MIXTURES

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Submitted By:

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TECHNICAL REPORT ABSTRACT

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16. Abstract

This work showed that the bending beam rheometer is a viable test to determine the low temperature performance of asphalt mixtures; it balances the rigor required of any mechanical test and the relation to field performance with the practicality of a procedure that can be easily adopted. The protocols developed parallel existing asphalt binder protocols thus allowing for easy implementation since they are based on robust and commercially available equipment that requires no modification.
It was shown that the size of the beams used for this test are a representative volume element. Mixtures with nominal maximum aggregate size of ½" or even larger can be tested without large aggregates introducing any further variability. The size of the beams also allows for multiple specimens obtained from either gyratory compacted samples or field cores; agencies and contractors can test mixtures and compared results as the difference between two labs was determined to be less than 10 percent for the modulus and about 20 percent for the m-value. Finally, based on field performance measurements, it was determined that both creep modulus and m-value are needed to determine the low temperature performance of asphalt pavements. These values can be visually presented in a Black Space diagram. Mixtures with high modulus and low m-values are expected to show poor performance but mixtures with high modulus AND high m-value should have better performance, perhaps allowing for better performing asphalt mixtures at both low and high temperatures.
The benefits found from using the BBR to test small beams of asphalt mixtures indicate that this is an ideal test to evaluate the low temperature performance of asphalt mixtures

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

UNIT CONVERSION FACTORS

Units used in this report and not conforming to the UDOT standard unit of measurement (U.S. Customary system) are given below with their U.S. Customary equivalents:

- 25.4 millimeters (mm) = 1 inch (in)
- 1 megapascal (MPa) = 145.04 pounds per square inch (psi)

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	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	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 (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa
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	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. (Adapted from FHWA report template, Revised March 2003)

LIST OF ACRONYMS

AASHTO	American Association of State Highway Transportation Officials
AFT	Apparent Film Thickness
BBR	Bending Beam Rheometer
EDF	Empirical Distribution Function
FHWA	Federal Highway Administration
HMA	Hot Mix Asphalt
IDT	Indirect Tensile Test
NMAS	Nominal Maximum Aggregate Size
PG	Performance Grade
RAP	Recycled Asphalt Pavement
RVE	Representative Volume Element
SR	State Route
SGC	Superpave Gyratory Compactor
TSRST	Thermal Stress Restrained Specimen Test
UDOT	Utah Department of Transportation
UofU	University of Utah
VFA	Voids Filled with Asphalt
VMA	Voids in the Mineral Aggregate
VTM	Voids in Total Mix

EXECUTIVE SUMMARY

This work showed that the bending beam rheometer is a viable test to determine the low temperature performance of asphalt mixtures. While there are many tests that have been proposed to evaluate the low temperature performance of asphalt mixtures, none of them have been adopted for routine testing. The BBR balances the rigor required of any mechanical test and the relation to field performance with the practicality of a procedure that can be easily incorporated into materials specifications. The protocols developed parallel existing asphalt binder protocols thus allowing for easy implementation since they are based on robust and commercially available equipment that requires no modification.

The BBR test requires asphalt mixture specimens to be cut out of laboratory compacted samples or field cores into beams that measure 12.7 mm x 6.35 mm x 127 mm (width x thickness x length) with tolerances in the width and thickness of ±.0.25 mm. More than 20 beams can be obtained from one gyratory-compacted sample. Once cut, the specimens are conditioned for 1 hour in the BBR bath at a temperature 10 ºC higher than the expected performance grade for the location where the asphalt mixture will be placed. After one hour, a load of 4500 mN ± 50 mN is applied to the midspan of the beam and the deflection is measured as a function of time for 240 seconds. The BBR software calculates and reports the creep modulus and m-value of the beam and highlights the results at 60 seconds. For consistency and convenience, these two value were selected for evaluation although values at any other loading time or even multiple values or temperatures could be used for performance modeling.

It was shown that the size of the beams used for this test are a representative volume element. Mixtures with nominal maximum aggregate size of ½” or even larger can be tested without large aggregates introducing any further variability. The size of the beams also allows for multiple specimens obtained from either gyratory compacted samples or field cores; agencies and contractors can test mixtures and compared results. Comparison done between tests run at two different laboratories by two different technicians using two different BBR machines showed that the difference in the modulus between two labs was less than 10 percent. The m-value had differences of over 20 percent indicating inconclusive results for this parameter.

Measurements of mixture modulus and m-value taken between 2 and 14 days since fabrication of the gyratory sample and cutting of the small beams showed no difference in results. Steric hardening or any time-dependent change not related to oxidative aging has no effect on BBR results after 48 hours. If needed, the same beam can be re-tested without compromising the results as long as testing is done within the right temperature conditions.

Finally, based on field performance measurements, a Black Space diagram where creep modulus is plotted against m-value was used to characterize the asphalt mixtures in terms of both stress accumulation (modulus) and stress rate of relaxation (m-value). Out of seven mixtures evaluated in the field, the three that had high modulus AND low m-value showed thermal cracking during the first two winters. One of the mixtures with high modulus AND high m-value did not cracked, perhaps allowing for better performing asphalt mixtures at both low and high temperatures. All seven mixtures had the same asphalt binder grade specified but different mixture components (aggregate source, RAP content, etc.) indicating that mixture testing is needed to characterize performance.

The benefits found from using the BBR to test small beams of asphalt mixtures indicate that this is an ideal test to evaluate the low temperature performance of asphalt mixtures

This work resulted in an AASHTO Provisional Standard (TP125-16). Based on the results, a specification that incorporates both modulus and m-value is proposed.

1.0 INTRODUCTION

1.1 Background

Thermal cracking due to stresses at low temperature is a major factor in roadway degradation. Many studies have found that in areas which routinely experience freezing temperatures thermal cracking is the principal form of deterioration of asphalt pavements (Marasteanu et al., 2007). In some pavements, transverse cracks caused by thermal stresses can appear after only one or two winters even if the proper low temperature binder grade is used. These cracks lead to further deterioration of the pavement structure resulting in poor performance. While there are many reasons for poor low temperature performance of mixtures; they all, in one way or another, are the result of brittle mixtures that lack the ability to relax thermal stresses. It is believed that these hard, brittle mixtures come from two primary sources: 1) the emphasis on rut resistance mixtures, and 2) the addition of reclaimed asphalt pavements (RAP). To address this issue, it is recognized that mixture testing at low temperatures should be a priority.

Superpave Performance Grade (PG) specifications are currently used to control low temperature cracking. However, these specifications apply to the asphalt binder only and do not take into account aggregate-binder interaction or the effects of reclaimed asphalt. In order to increase sustainability and decrease cost, the vast majority of HMA mixtures today are produced with the addition of reclaimed asphalt product (RAP). While adjusting the low end performance range of the binder is done to counteract the stiffening that occurs of the HMA mixture with the addition of RAP; studies have shown that it is not sufficient (Marasteanu et al., 2007; Marasteanu et al., 2012). Methods such as extraction and recovery of asphalt binders result in misleading results as it assumes perfect blending between the binder contribution from the RAP and the virgin asphalt binder. Clearly, testing of the actual asphalt mixture is preferable and it can be argued that it is in fact the only way to determine mixture performance. However, while there are several mixture tests that have been developed to address this issue, none of them have been adopted. Cost, time, and practicality have been cited for the lack of adoption.

Work done at the University of Utah and at the University of Minnesota have shown that a good alternative for mixture testing at low temperature is to use the Bending Beam Rheometer (BBR) developed during the Strategic Highway Research Program (SHRP) at Penn State University for asphalt binder testing. Using the BBR to test mixtures has several advantages over other proposed mixture tests, including:

1. Equipment availability – Since this device was developed for binders during the 1990s, it is already in many asphalt labs. This resolves issues of equipment development, equipment reliability/ruggedness, commercial availability of equipment and technical support, reasonable costs, and existing staff training.

2. Reduced specimen size – Beams used in the BBR are only 12.5 x 6.75 x 127 mm (0.5 x 0.25 x 5 in), this makes them ideal to evaluate overlays or thin layers and allows for multiple samples obtained from one core or one gyratory-prepared specimen. Furthermore, the small samples expedite conditioning time, reduce equipment load requirements, and facilitate handling.

3. Theoretically valid – The BBR is based on theoretical considerations of the mechanics of materials (beam theory and elastic-viscoelastic correspondence principle). The results can be used in theoretical analysis as well as index-type classifications.

Based on these advantages, a research program was initiated with the objective of developing a standard method to incorporate BBR testing of mixtures in addition to the regular testing requirements currently in place.

1.2 Objectives

The objectives of this research are:

1. To develop a testing procedure to evaluate the low temperature performance of asphalt mixtures using the Bending Beam Rheometer.
2. To establish consistent testing protocols that can be adopted as a standard specification.
3. To determine the within and between lab variability and study any other factors that might affect the reproducibility of the results.

4. To demonstrate the relation between results obtained using the BBR and field performance.

1.3 Scope

This research is based on material and conditions available in the state of Utah. Both laboratory-prepared specimens and field cores were used. Theoretical and practical applications where considered. Utah Department of Transportation (UDOT) standards were followed when appropriate.

1.4 Outline of Report

This report is a continuation of the work previously described on the research report titled *Development of Methods to Control Cold Temperature and Fatigue Cracking for Asphalt Mixtures* (UDOT 10-08) by Romero et al. (2011). While some information is repeated in this report for clarity and eased of reading, most of the theoretical background has been omitted as it has already been presented on that report. Readers are encouraged to read the previous report available at Utah Department of Transportation website: (https://www.udot.utah.gov/public/ucon/uconowner.gf?n=4493029359845211).

This report is divided into the following chapters

1. Introduction
2. Low Temperature Testing of Asphalt Mixtures
3. Evaluation of Beam Size
4. Test Repeatability
5. Relation to Field Performance
6. Conclusions, Recommendations, and Implementation

2.0 LOW TEMPERATURE TESTING OF ASPHALT MIXTURES

2.1 Overview

Thermal cracking of asphalt concrete is the resulting distress from exposure to low-temperature conditions. Like most materials, asphalt concrete contracts when exposed to low temperatures. This contraction is countered by the frictional force of the underlying layers inducing thermal stresses on the pavement. As temperatures decrease, contraction of the pavement subsequently increases and results in an increase in thermal stress experienced by the pavement. Once the stress reaches the strength of the material, a crack will develop. Different materials will accumulate and relax stresses at a different rate depending on their properties; specifically, their relaxation modulus. Thus, relaxation modulus is one of the most important material property used to predict thermal cracking.

2.2 Testing Modes

Determination of the relaxation modulus of asphalt mixtures is done through mechanical testing. Mechanical testing of any material can be done in one of two ways: stress controlled or strain controlled. In a stress controlled test, the stress function is known while the corresponding response of strain is measured. For the case of time-dependent materials, such as asphalt concrete, the stress is known and the strain is time dependent. A specific example of a stress controlled tests is the creep test. In a creep test a constant load is applied resulting in a constant stress (σ_c) and the time dependent strain (ε_t) is measured. The ratio of these two values is called the creep compliance, D(t), of the material as shown in Equation 1.

$$\mathbf{D(t)} = \frac{\varepsilon(t)}{\sigma_c} \qquad \textbf{Equation 1}$$

Strain controlled tests, also known as relaxation tests, are just the opposite. They involve applying a known strain while the response of stress is measured. Again, for asphalt concrete and other time dependent materials the strain is known while the responding stress is time dependent. These type of tests are not as common due to testing difficulties. A specific example of a strain controlled test is the relaxation test in which a material is subject to an instantaneous strain (ε_c). The strain is held constant while the decreasing stress (σ_t) is measured. The ratio between these two values is referred to as the relaxation modulus, E(t), shown in Equation 2.

$$\mathbf{E(t)} = \mathbf{e}\frac{\sigma(t)}{\varepsilon_c} \qquad \textbf{Equation 2}$$

Creep compliance and relaxation modulus are representations of the same viscoelastic behavior. However, they are not reciprocals of each other due to the fact that in creep compliance there is constant stress while strain is time dependent, but the opposite is true for relaxation modulus. Although they are not reciprocals of each other, if one is known the other one can be determined by transforming the time relationship to a different domain through the use of the LaPlace Transform (Christensen, R.M., 1982). It is also recognized, however, that for most engineering applications, using creep compliance can be confusing, instead creep modulus (the inverse of creep compliance) has been found to be a good approximation for relaxation modulus. Therefore, most of work presented on this report is based on the creep modulus, sometimes call creep stiffness.

2.3 Test Methods

Currently there are several tests that can be conducted to determine low-temperature performance of asphalt mixtures based on their relaxation modulus. Three of the most common are the Temperature Specimen Restraint Specimen Test (TSRST), the Superpave Indirect Tensile Test (IDT), and the Bending Beam Rheometer (BBR).

2.3.1 Thermal Stress Restraint Specimen Test

The Thermal Stress Restraint Specimen Test, TSRST, is a strain and temperature controlled test used to determine if an asphalt pavement is susceptible to low-temperature thermal cracking by simulating a thermal event that may be experienced in the field. In this test the temperature is lowered at a constant rate while the sample is restrained. This restraint keeps the sample from contracting which results in tensile stress. Load cells and LVDTs are used to take measurements throughout the test allowing for both the load and the temperature to adjust simultaneously while determining tensile strength (Velázquez et. al., 2009; Jung et al., 1994).

2.3.2 Superpave Indirect Tensile Test

The Indirect Tensile Tests, IDT, is a stress controlled test that can be used to determine creep compliance and indirect tensile strengths of asphalt mixtures. The IDT is normally conducted at low temperatures for thermal cracking predictions. In this test, a cylindrical specimen undergoes a compressive creep load along its radius. Over the loading period the deformation is measured and the creep compliance is calculated and used for performance predictions through different models (Christensen and Bonaquist, 2004).

2.3.3 Bending Beam Rheometer

Like the IDT, the Bending Beam Rheometer, BBR, is a stress controlled test. AASHTO T313/ASTM D6648 describes the BBR, pictured in Figure 2-1, as being used to perform tests on beams of asphalt binder after being conditioned at the desired test temperature. The test produces the creep modulus and the stress relaxation capacity (slope of the modulus versus time curve in a log-log scale), also called m-value, by way of applying the elastic solution to a simply supported beam. These values have been used to calculate thermal stresses in pavements (Bahia and Anderson, 1995; Marasteanu, 2004). Using the BBR to test asphalt mixtures in place of binder was originally proposed by Marasteanu et al. (2009). They found that the compliance curves resulting from their tests showed good correlation with curves generated by the IDT. This research, which was further advanced by Ho (2010), Romero et al. (2011), and Ho and Romero (2011) who determined that BBR testing of small amounts of material can produce behavioral results that are representative of the entire mixture.

Both the TSRST and the IDT have been successfully used for the prediction of low-temperature thermal cracking of asphalt pavements, but they both require significantly more material and are a more involved testing process than the BBR. For this reason, as well as other previously discussed issues, BBR testing is considered more practical and was chosen to be used in this study.

Figure 2-1 Picture of Cannon bending beam rheometer

2.4 BBR Testing and Data Interpretation

2.4.1 Sample Preparation

The BBR test requires minimal amounts of material. Because of this, it is possible to test field cores as well as gyratory prepared samples that are constructed in the laboratory. To make samples, a lapidary saw is used to cut the field cores or gyratory prepared samples into a small block; from the small block, small beams are cut using a commercial tile saw; preferably with an asphalt cutting blade. The BBR test requires each sample to be cut into beams that measure 12.7 mm x 6.35 mm x 127 mm (width x thickness x length) with tolerances in the width and thickness of ± 0.25 mm.

Details of sample preparation from a 150-mm diameter gyratory compacted specimen are shown graphically on Figure 2-2. A similar procedure would be used for field cores. While cutting the beams, their dimensions can be easily verified by using a template, as shown in pictures s, t, v, w, and x. For testing, the exact dimensions of each beam are needed as input for the BBR software and can be measured using digital calipers. The dimensions included total length, thickness at one third of the total length from each end, width at one third of the total length from each end, and mass. Once the samples are cut to proper dimensions, they are stored together on a flat tray until ready for testing. This prevents any deformation.

With practice, more than 20 test samples can be obtained from a single gyratory specimen. The number of beams that can be obtained from field cores will depend on the actual layer thickness but at least five beams have been cut from a 50-mm thick layer. The time it takes to prepare the samples is relatively small. Once a gyratory sample has been made, or a field core has been obtained, the cutting process takes only a few hours. After cutting, conditioning of the samples can start almost immediately, as no effect has been found from any residual water accumulated during cutting. As will be shown in Section 4.0, if properly stored, the beams can be tested after more than a week with no change in results.

a. Tools for BBR beam fabrication

b. Lapidary saw

c. Tile saw

d. Superpave Gyratory Compactor (SGC) Sample

e. Marking the central line on sample

f. Using lapidary saw to cut the sample in half

g. Using a wood block to mark on sample

h. Cutting one side of the half sample

i. Turning the sample and cutting the other side of the half sample

Figure 2-2 (a-i) Initial cutting of asphalt samples

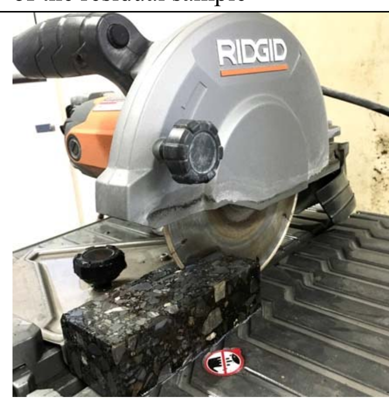
		
j. Original components of the SGC sample	k. Marking the central line on the residual of the half sample	l. Cutting the sample along the marked central line
		
m. Original components of the half sample	n. Using the tile saw to cut both end of the residual sample	o. Using the tile saw to cut one side of the residual sample
		
p. The half sample is cut into two blocks	q. Original components of the half sample	r. Using the tile saw to further cut each block into several flat beams with a thickness of 12.7mm

Figure 2-2 (j-r) Trimming asphalt concrete samples

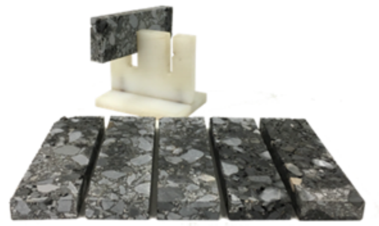
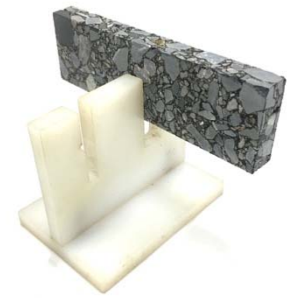
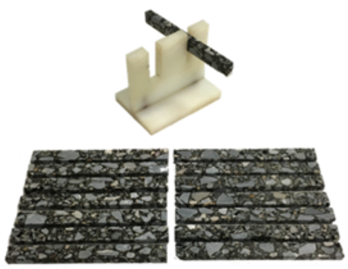
		
s. Flat beams with the dimension checking template	t. Using the wide slot of dimension checking template to check the thickness of each flat beam (thickness of flat beam = width of BBR testing beam)	u. Using the tile saw to trim each flat beam to several thin beams with a thickness of 6.35mm (These thin beams are the beams that can be used for BBR test)
		
v. BBR testing beams with dimension checking template	w. Using narrow slot of the dimension checking template to check the thickness of each BBR testing beams	x. Using wide slot of the dimension checking template to check the width of each BBR testing beams
		
y. Condition beams into BBR testing bath for 1 hour	z. Placing each beam on the testing platform to run BBR test	

Figure 2-2 Making asphalt mixture beams for BBR testing

2.4.2 Testing Procedure

To maintain consistency between asphalt binder testing and mixture testing, beams are tested at three temperatures: low binder grade +16°C, low binder grade +10°C, and low binder grade +4°C. This allows for the development of a master curve. However, if a master curve is not desired, only one temperature is needed and it should be the target low binder grade for the expected location +10°C.

Before each testing session the BBR must be calibrated for both temperature and force/deflection as recommended by the manufacturer. Prior to testing, each sample is soaked in the temperature controlled bath for 60 minutes to ensure that the entire beam is brought to test temperature. Testing of each sample requires approximately 8 minutes. Every 10 minutes a beam can be added to the bath. After an hour the first beam placed in the bath is ready to test. Every 10 minutes the beam that has been in the bath for 1 hour is ready to be tested, the previously tested sample is removed, and a new beam is placed in the bath to begin soaking. This allows for a quick and effective way to test materials. All testing procedures follow AASHTO T313 *Standard Test Method for Determining the Flexural Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)* with minor modifications as described next.

The initial load (35 mN, milliNewton, ± 10 mN) applied by the BBR is the same as what is described in AASHTO T313. The testing protocol of the BBR manufacturer states that the BBR can apply up to 450-gram force without further change in the air bearing system. Previous research has determined that the 450 grams of applied loading for the BBR test can produce significant deflections of asphalt mixture beams at the recommended test temperatures of PG +10°C (Ho, 2010 and Romero et al. 2011). This led to the applied load of 450 grams (4413 mN ± 50 mN) being selected for the BBR tests in this research. Each test produces a series of data that includes force and deflection as a function of time. These values are then used to calculate creep modulus and the m-value (slope). Figure 2-3 shows the BBR with a beam in testing position.

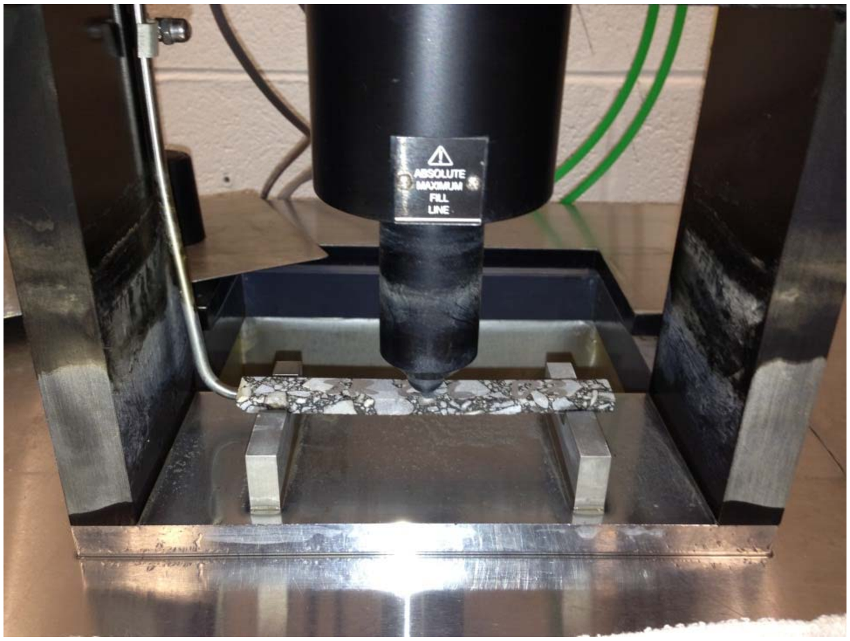


Figure 2-3 Sample beam in the BBR testing position (pictured out of bath for clarity)

2.4.3 Data Analysis

The BBR automatically records the load and the deformation of the beam. Knowing the beam dimensions and using beam elastic solutions along with elastic-viscoelastic correspondence principle, the creep modulus as a function of time of the material is determined. Standard software provided with the BBR automatically calculates the creep modulus and m-value at the end of the test and highlights these values at 60-seconds. These two values at this specific time have been used to evaluate expected mixture performance. While other times can also be used, 60 seconds allows for consistency between asphalt binder and mixture testing. Most of the analysis in this work has been done using the creep modulus and m-value at 60 seconds. The BBR screen with the highlighted values is shown in Figure 2-4.

The complete set of data from the BBR can be plotted and further analyzed. Creep modulus curves, as shown in Figure 2-5, can be used for performance modeling if desired.

CANNON® Instrument Company, USA 3.21 03/04/2013 02:40:52 PM

Project :	BBR	Target Temp (°C) :	-12.0	Conf Test (GPa) :	219
Operator :	ZLJ	Min. Temp (°C) :	-12.0	Conf Date :	12/12/2012
Specimen :	SR111-D1.12	Max. Temp (°C) :	-12.0	Force Const (mN/bit) :	0.15
Test Time :	04:32:37 PM	Temp Cal Date :	12/12/2012	Defl Const (μm/bit) :	0.156
Test Date :	12/12/2012	Soak Time (min) :	60.0	Cmpl (μm/N) :	6.91
File Name :	SR111-D1.12	Beam Width (mm)	12.63	Cal Date :	12/12/2012
BBR ID :	U of U BBR	Thickness (mm) :	5.91	Software Version :	BBRw 3.21

t Time (s)	P Force (mN)	d Deflection (mm)	Measured Stiffness (MPa)	Estimated Stiffness (MPa)	Difference (%)	m-value
8.0	4450	0.04188	1.07e+004	1.07e+004	0.000	0.144
15.0	4451	0.04590	9.75e+003	9.72e+003	-0.308	0.164
30.0	4451	0.05209	8.59e+003	8.6e+003	0.116	0.187
60.0	**4452**	**0.05982**	**7.48e+003**	**7.5e+003**	**0.267**	**0.210**
120.0	4452	0.06956	6.44e+003	6.43e+003	-0.155	0.232
240.0	4453	0.08240	5.43e+003	5.43e+003	0.000	0.255

A = 4.13 B = -0.0762 C = -0.0375 R^2 = 0.999935

Force (t=0.0s) = 30 mN Deflection (t=0.0s) = 0.00000 mm
Force (t=0.5s) = 4158 mN Deflection (t=0.5s) = 0.02904 mm

Max Force Deviation (t=0.5 - 5.0s) = -293, +0 mN
Max Force Deviation (t=5.0 - 240.0s) = -2, +2 mN

Average Force (t=0.5 - 240.0s) = 4451 mN
Maximum Force (t=0.5 - 240.0s) = 4453 mN
Minimum Force (t=0.5 - 240.0s) = 4158 mN

Figure 2-4 Screen capture of the BBR with values at 60 s highlighted

Figure 2-5 Creep modulus versus time curve

2.5 Summary

This section describes the test setup for BBR testing of asphalt mixture beams. Specimens can be obtained from gyratory compacted samples or from field cores. A tile saw is used to trim the beams with final dimensions of 12.7 mm x 6.35 mm x 127 mm (width x thickness x length) with tolerances in the width and thickness of ±.0.25 mm. After an hour of temperature conditioning the beams are testing in the BBR. The software automatically calculates the creep modulus and the relaxation capacity (m-value) then highlights these values at 60 s. While values at other times are available, 60 s has been used standard time in binder specifications. The similarities between mixture testing and binder testing allow for eased of staff training and equipment support.

Asphalt mixture testing at low temperature can be easily accomplished using the procedures described on this section.

2.5.1 AASHTO Specifications

At the time of this writing, the procedures described on this section have been adopted as an AASTHO Temporary Procedure, AASHTO TP 125-16: Standard Method of Test for Determining the Flexural Creep Stiffness of Asphalt Mixtures Using the Bending Beam Rheometer (BBR). The procedure is available at the AASHTO website.

3.0 EVALUATION OF BEAM SIZE

3.1 Introduction

The approach of testing small asphalt concrete beams was originally met with some skepticism. Critics were concerned with the beam size in relation to the maximum aggregate. In other words, the concern is that a large aggregate, like the one highlighted on Figure 3-1 would dominate and affect the results. These critics were hesitant to accept that the gauge length (distance over which the measurement is made) is actually the length between the supports of the beam (101 mm) and that the measurements obtain are in fact the representative volume element (RVE) of the material.

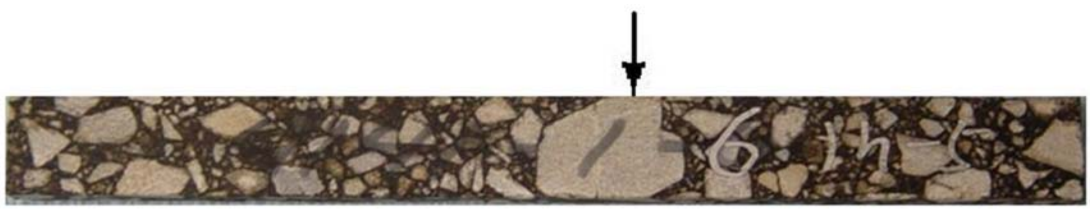


Figure 3-1 Beam with large aggregate

This section addresses the concern of small beam size to aggregate size ratio. In this work three equivalent mixtures with decreasing nominal maximum aggregate sizes (NMAS) were analyzed. This resulted in three different NMAS-to-beam dimension ratios. The three different mixtures were evaluated to ensure that the mixtures were in fact equivalent by comparing the volumetric parameters and the gradations. An alternative validation was done by visually analyzing scanned images of the three mixes at the optimum asphalt content. The BBR measurements were collected from the three groups and the variances of the data groups were proven to be equal using Bartlett's Test. Bartlett's Test relies on the data being normally distributed and tests based on empirical distribution function (EDF) statistic confirmed this.

3.2 Background

The Representative Volume Element or RVE is a certain volume of a composite material that has been determined through calculation and laboratory testing to represent the global properties of the material. Traditionally an RVE is selected by starting with a sample whose smallest dimension is of nominal maximum aggregate size (NMAS); the samples are tested and the sample size is increased accordingly to obtain a normalized variability. The intent of determining if the RVE is met, is to ensure that all individual materials of a composite are present for testing. For example, if we have a composite material consisting of aggregate, air, and binder, we want an RVE that contains all three components in enough quantities so that when tested, the response is dependent on all components. Take the two-dimensional example of the X-ray tomography image in Figure 3-2, When the representative area element (RAE) is very small the % Aggregate can vary from 0% to 100% depending on the location of the circle. As the area is increased the fluctuations in % Aggregate abate and when the fluctuations stabilize this area is the RAE of the sample (Romero and Masad, 2001).

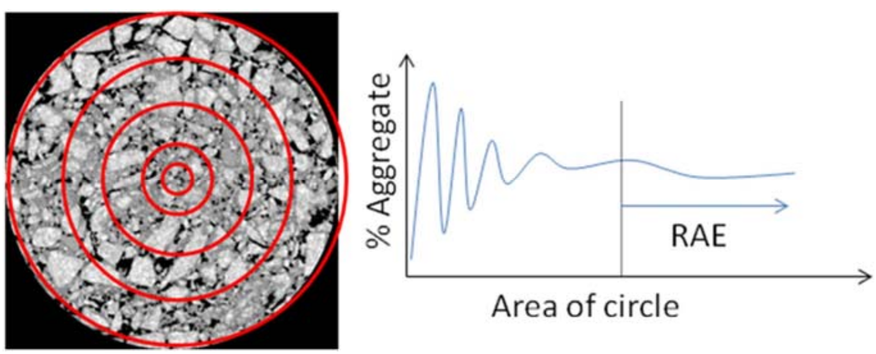


Figure 3-2 Representation of a representative area element

It would be meaningless to have an RAE (or RVE) that has only binder and air, or binder and aggregate when there are three components to the composite. Having a volume that is not representative of the whole composite material would results in extreme fluctuations in variability of results obtained through any testing. A sample volume consisting of only air and binder would result in a very different creep modulus than, for example, a volume consisting of only binder and aggregate. If the variability of measured results from sample to sample is stable, then the minimum requirement of the RVE has been met.

Composite theory states that in composite materials having spatial disorder with no microstructural periodicity (like asphalt concrete mixtures) the determination of any stress, strain, or energy field can be measured as an average over the given domain (Du and Ostoja-Starzewski, 2006; Osteja-Starzewski, 2006). Therefore, the stress or strain recorded as part of any analysis is not the actual value experienced by a specific component but rather an average or bulk property over the given section. The question asked is whether this averaging is done over the entire domain that includes all heterogeneities or whether it is influenced by localized phenomenon. This answer depends on the property being measured and the shape of the sample. The size of the domain that satisfies these averaging requirements is the RVE (Hashin, 1963). Velásquez et al. (2009b) conducted a study to determine the effect of beam size on the creep modulus of asphalt mixtures at high, intermediate, and low temperatures. In that work, the beam sizes were increased to see if the size of the beams had an effect on the creep compliance. For the smallest size, the BBR was used but for the subsequent larger sizes equipment designed for the purpose of the research was developed. Velásquez concluded that the creep compliance was affected by the beam size at high and low temperatures, although Velásquez also concluded that the cold temperature fluctuation could be due to ice buildup on the measurement devices and the beams themselves. This rationalizes concerns of not only beam size to maximum aggregate size ratio but also the call for proper equipment and standardization of testing.

3.3 Approach

It is customary in developing the RVE to increase the size of the analyzed volume until a statistical stability is reached, as was done by the Velásquez study. As an alternative, you can keep the volume constant and change the aggregate size to beam dimension ratio. As an example, consider a chain with three randomized components: a large steel link, a medium copper link, and a small iron link. Consider the case where a small sample of this composite material is tested in the laboratory for its elongation, as shown in Figure 3-3. Depending on the location of the measurement at the given gauge length, the percent steel in the sample can vary between 0% (no steel) and 100% (all steel). The resulting measured elongation of this sample might result in large fluctuations depending on whether some lengths contain steel while others do not. This sample length does not represent all elements of the chain; the length in Figure 3-3 is not the RVE of this composite material. However, as the gauge length is increased, the fluctuations in measured elongation due to variations in steel content greatly abate. In this case, the gauge length at which the elongation function stabilizes is the minimum size needed to overcome the domain of small scale heterogeneity. This domain is the RVE. 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 (Kanit et al., 2003). The gauge length in Figure 3-4 contains all three components no matter where on the composite structure this length is obtained. This length is the RVE (representative length element in this example). Now consider for a moment if the gauge length must remain constant and the exact same chain were then reduced in size as in Figure 3-5. Now the gauge length that was rejected in Figure 3-3 is acceptable as the RVE of the composite material. All elements of the composite material are present and the properties obtained (elongation) are representative of the composite material. If the reduction process is carried a step farther then more individual elements of the original composite fit within the constant sampling domain. This is the approach used as proof of this work (Clendennen and Romero, 2013).

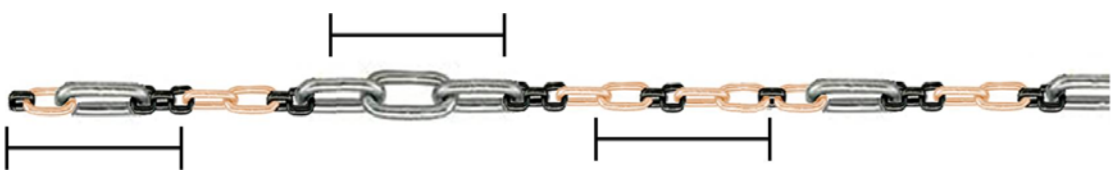


Figure 3-3 Composite chain with non-representative length

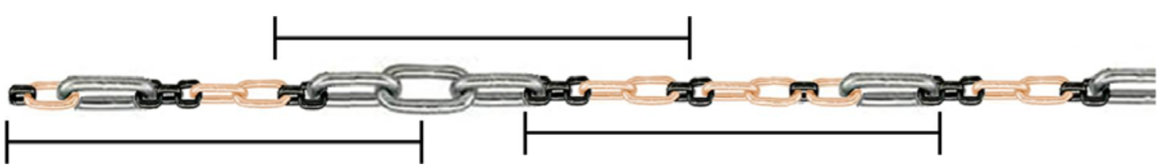


Figure 3-4 Composite chain with representative length

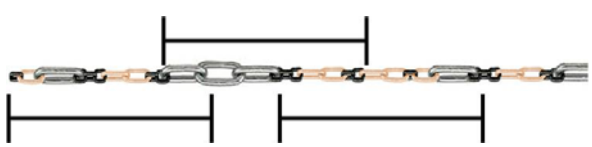


Figure 3-5 Composite chain scaled to 45% of original size

3.4 Materials and Procedures

The hypothesis of this work is that the large aggregate size (12.5-mm) in the mixture does not cause an increase in variability of the creep modulus results from the BBR testing. This is proven by demonstrating that the variability of the larger mixture is equivalent to the variability of the smaller NMAS mixture. If the variability is consistent then the hypothesis is supported, but if the variability is greatly affected then the hypothesis must be rejected.

In this research hot mix asphalt was used with a 12.5-mm NMAS mix as the standard mix. The performance grade (PG) 64-34 binder was selected for use because it has low temperature tolerance and is a relatively soft binder that will allow large movements when tested. The aggregate is of high quality quartz with very low absorption, this allows for easier modification of the mix design. Asphalt concrete was chosen as a good composite material to illustrate the hypothesis because it is a material with spatial disorder and no microstructural periodicity.

3.4.1 Volumetric Parameters

A voids analysis is very important while developing a gradation and mix design. It is important to understand the volumetric parameters of compacted asphalt concrete mixtures for both mix design and construction control. Many mix design methods including the Superpave mix design method, preferred by many department of transportations (DOTs) and government agencies, require the volumetric parameters to be within certain ranges. These parameters help standardize asphalt concrete mix designs and achieve certain desirable properties. The percent voids in the mineral aggregate (VMA), is a measure of the space available in the aggregates for the addition of the asphalt cement. The voids filled with asphalt are called VFA and the total voids of the mix are called the VTM. Figure 3-6 depicts the components of the compacted asphalt mixture and the corresponding equations to calculate the volumetric parameters of the mixture.

As the NMAS of a mixture decreases, the specific surface area of the mix increases therefore, more asphalt is required to maintain a constant apparent film thickness (AFT) (Christensen and Bonaquist, 2006). The film thickness is referred to as apparent because within the mixture a film thickness cannot truly be measured, it can only be estimated. Although the AFT can only be estimated it can be used to generalize comparisons between mixtures. For example, an aggregate with a much smaller AFT will not have the same response as aggregate with a larger AFT.

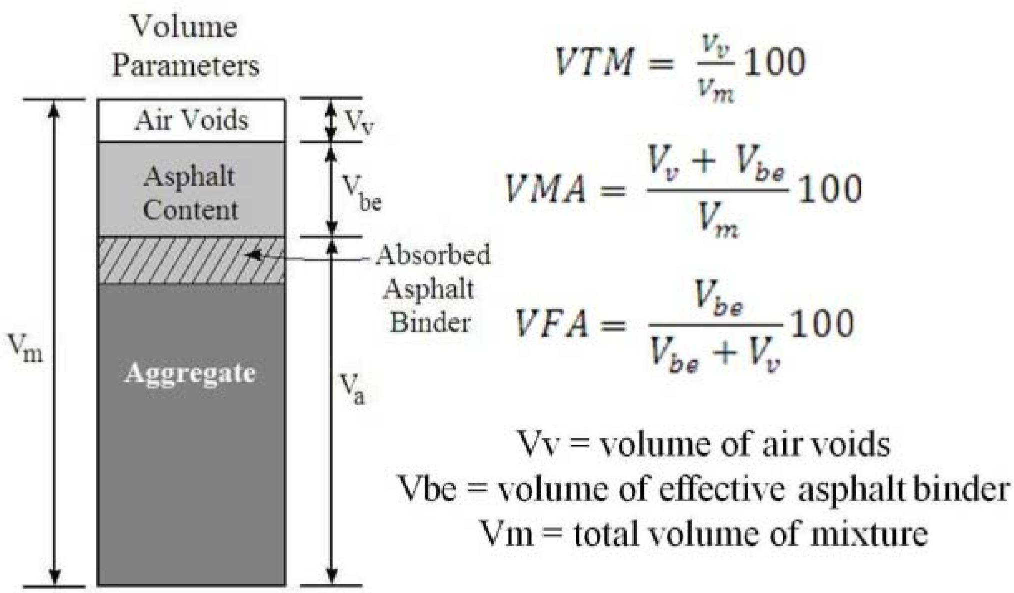


Figure 3-6 Components of compacted mixture with equations

3.4.2 0.45-Power Chart Theory and Gradation

The 0.45 power chart is a common tool for mixture gradation design. The maximum density line on the 0.45 power chart represents a gradation of spheres of different sizes that create a mixture that has maximum theoretical density. This maximum density line is a straight line from the origin to the NMAS of the mixtures (Goode and Lufsey, 1962). This line represents the gradation or particle size distribution that results in the highest level of packing, thus the highest bulk density. For example, Figure 3-7 shows the space between large aggregate represented by theoretical spheres. The space between these large spheres is then filled with the next largest sphere size, then the next size of spheres fill the resulting spaces, then smaller spheres in the space around those spheres, continuing into infinity into the theoretical maximum density, leaving no space or voids. Decreasing the NMAS would merely shift the maximum density line to the left of the chart but still result in the same level of packing as the larger NMAS max density line that is to the right.

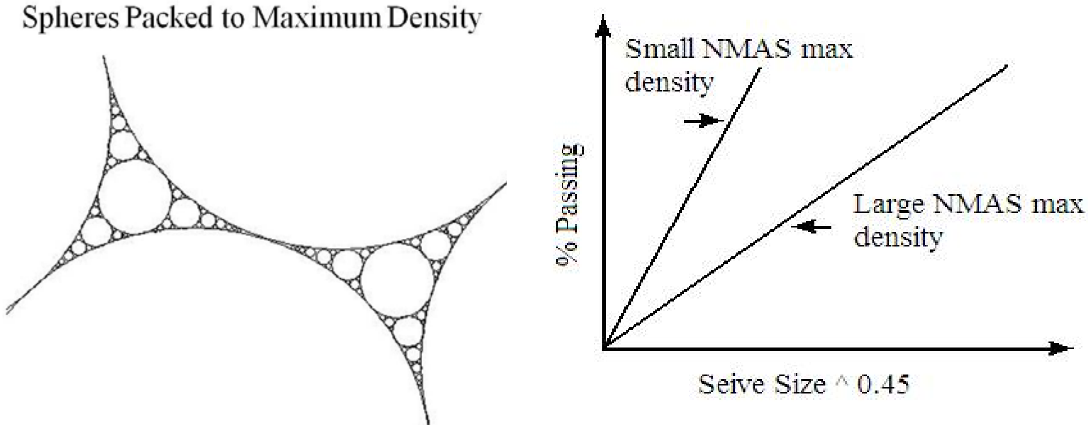


Figure 3-7 Theoretical maximum density example

Voids play an important role in asphalt mixture performance. There has to be enough voids in the mix to support the asphalt binder which is the glue, essentially, to keep the aggregate together. Too many voids and the mix will rut and too few voids and the mix will crack when placed in the field. An S shaped curve with respect to the maximum density line has been the preferred shape for Superpave gradation mix designs. This shape has shown good performance when used in conjunction with certain volumetric parameters.

The mixtures used in this work had NMAS of 12.5-mm, 9.5-mm, and 4.75-mm. The goal was to use the 12.5-mm aggregate gradation as the model and scale the gradation curve to the other two respective NMAS sizes. The goal was not to merely eliminate the larger particles but have actual mixes with equivalent volumetric parameters and similar gradation shapes. The shape of each gradation curve was kept similar with respect to the maximum density line as seen in Figure 3-8. The gradations and the volumetric properties of the mixtures used in this work are shown in Table 3-1.

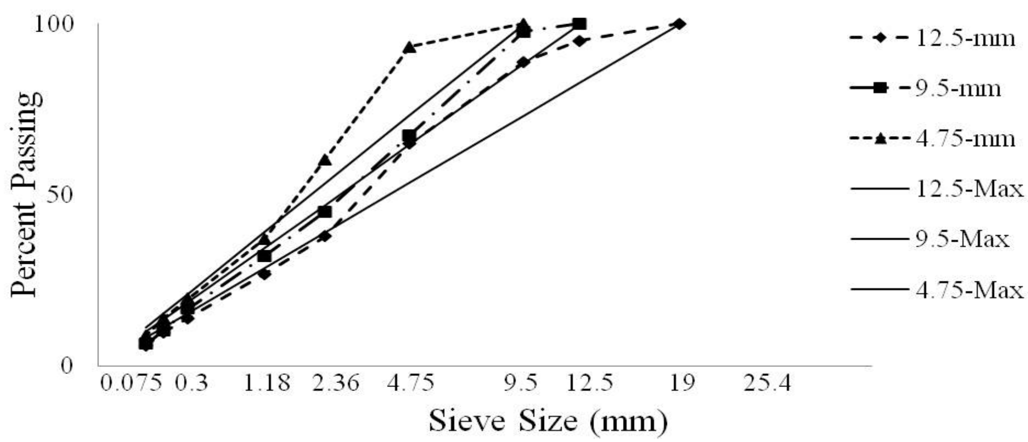


Figure 3-8 0.45-Power chart with gradations used

Table 3-1 Mixture design and volumetric properties

Mixture Design			
Seive Size (mm)	12.5-mm	9.5-mm	4.75-mm
	Percent Passing (%)		
19	100	100	100
12.5	95	100	100
9.5	89	97.8	100
4.75	65	67.5	84.9
2.36	38	45.2	60.4
1.18	27	32.2	37.3
0.3	14	16.8	19.8
0.149	10	10.6	14
0.075	6.2	6.6	9

Volumetric Properties				
NMAS (mm)	Asphalt Content	VTM	VMA	VFA
4.75	6.5	4	17.19	79.05
9.5	6.2	4	16.70	78.44
12.5	6	4	16.38	78.03

In theory similar shaped 0.45 gradations imply similar aggregate structure within the composite. As the NMAS is reduced the amount of fines and therefore aggregate surface area increases. To match the VFA of the two developed mixes to the standard 12.5-mm mix, the binder content had to increase. This resulted in an increase in the VMA, but held the VFA constant. The goal was to keep the VFA and VTM constant, ensuring that proportionally the same amount of binder was present resulting in the same apparent film thickness on the particles. The gradation and binder content was adjusted several times through trial and error to accomplish similar gradation shape and volumetric parameters. A similar 0.45 gradation shape, VMA, VTM, and VFA ensured that the mixes were essentially scaled equivalents of each other.

3.5 Homoscedasticity

Homoscedasticity, or homogeneity of variances, is where there are equal variances across samples. The hypothesis throughout this work is that all three mixtures resulted in the same variability in creep modulus; in other words, large aggregates do not induce more variability. If the mixtures of these three NMAS have the same variability and volumetric properties, as discussed earlier, then it is true that for these NMAS the small beams of size 12.7-mm x 6.35-mm x 127-mm are the RVE and the data collected from the BBR can be used as descriptive properties of these mixtures. Before a test on the homogeneity of variances could be performed, it was desirable to determine the distribution of the data sets. It was assumed that the sample creep moduli of the asphalt mixtures were drawn from a normal distribution. Graphical analysis showed that the creep moduli fit the normal distribution well, shown in Figure 3-9; tests based on empirical distribution function (EDF) statistics also confirmed this (D'Agostino and Stephens, 1986). These tests were the Kolmogorov-Smirnov D test, Kuiper V test, Cramer-von Mises W^2 test, Watson U^2 test, and the Anderson-Darling A^2 test. With a confidence level of 99%, all sample sets were found to be of a normal distribution. Outliers were not evaluated because the data sets were a good fit to the normal distribution when all the data points were included for each sample set as well as when the data sets were trimmed. Therefore, no data was excluded from any statistical test for the purposes of this research.

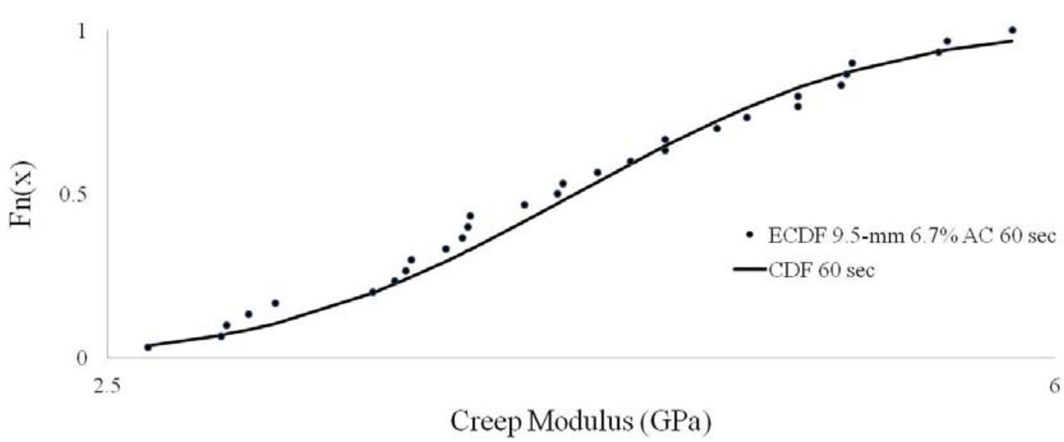


Figure 3-9 Modulus cumulative distribution function for 9.5 mm mix at 6.7% asphalt content and 60s of loading

The three NMAS groups were developed for this research to explore possible sources of variability with respect to aggregate size. If the variances of the three NMAS are equal then we know that the size of the aggregate does not affect the variability of the data and we can conclude that for the 12.5-mm NMAS the small beam of size 12.7-mm x 6.35-mm x 127-mm is in fact a representative volume element (RVE). The most robust method to evaluate for homoscedasticity is Levene's test. But if there is strong evidence that the data do in fact come from a normal distribution, then Bartlett's test has better performance (Zar, 1999). Because the tests based on EDF statistics confirmed that all our data sets are of normal distribution, the Bartlett's test was selected to prove homogeneity of variances. The variable used in Bartlett's test is defined as:

$$\boldsymbol{T = \frac{(N-k)\ln(s_p^2) - \sum_{i=1}^{k}(N_i-1)\ln(s_i^2)}{1+\left(\frac{1}{3(k-1)}\right)\sum_{i=1}^{k}\left(\frac{1}{N_i-1}\right)-\frac{1}{N-k}}} \qquad \textbf{Equation 3}$$

Where N is the total sample size, s_i^2 is the variance of the ith group, N_i is the sample size of the ith group, k is the number of groups, and s_p^2 is the pooled variance. The pooled variance is a weighted average of the group variances.

With significance level α the variances are judged to be unequal if:

$$\boldsymbol{T > X^2_{(1-\alpha,k-1)}} \qquad \textbf{Equation 4}$$

Where $X^2_{(1-\alpha,k-1)}$ is the upper critical value of the chi-square distribution, with $k-1$ degrees of freedom. If $T < X^2_{(1-\alpha,k-1)}$ we fail to reject the null hypothesis and the mixtures have statistically equal variances. Through Bartlett's test, the three sample groups fail to reject H_0. This supports the hypothesis that with the NMAS of 12.5-mm and small beam of dimensions 12.7-mm x 6.35 mm x 127-mm no more variability is introduce than for the 9.5-mm or 4.75-mm NMAS mixtures with the same beam size. Since the mixture with NMAS of 12.5-mm has aggregate that is larger than the smaller dimensions of the beam, it can be extended that when measuring the global properties of the mixture, such as stress, strain, or deformation the aggregate size does not have an effect on the variance of the sample group. Because the variability of the sample groups remains constant the test is not adversely affected by the large aggregate. It can be extended further that the smaller NMAS with increased number of aggregate in the small size beams do not increase the variability either.

Six other groups were developed by mixing binder sweeps of the optimum asphalt content +0.5% and -0.5% for each mixture. The null hypothesis was tested utilizing the Bartlett's test for all sample groups. This included the three optimum mixtures with asphalt content +0.5% and -0.5% with each of those 9 mixtures having creep modulus readings taken at 60 seconds and 120 seconds recorded as separate data sets. This resulted in 18 total sample groups. Bartlett's test fails to reject H_0; therefore, all 18 sample groups have statistically equal variances. A summary of all data can be found in Table 3-2.

Table 3-2 Bartlett's tests on creep modulus for mixtures at optimum AC

H_0:	$\sigma^2_{12.5NMAS} = \sigma^2_{9.5NMAS} = \sigma^2_{4.75NMAS}$							
H_A:	The three samples variances are heterogeneous							
Group	12.5 NMAS Creep Modulus at 60 sec. (GPa)		9.5 NMAS Creep Modulus at 60 sec. (GPa)		4.75 NMAS Creep Modulus at 60 sec. (GPa)		k=3	
	2.94	4.38	5.08	7.1	4.64	6.35		
	3.17	4.43	5.12	7.11	4.79	6.47		
	3.27	4.57	5.28	7.16	5.03	6.55		
	3.82	4.76	5.7	7.18	5.2	6.62		
	3.94	4.78	5.77	7.2	5.22	6.67		
	4.03	4.97	6.08	7.44	5.24	6.72		
	4.04	4.97	6.27	7.56	5.29	6.77		
	4.06	4.99	6.61	7.73	5.43	6.81		
	4.09	5.23	6.73	7.75	5.56	6.84		
	4.1	5.31	6.74	8.31	5.6	6.88		
	4.13	5.79	6.77	8.57	5.66	6.9		
	4.16	5.81	6.82	8.78	5.76	6.98		
	4.22		6.85	8.83	6.07	7.06		
			7.1		6.31	7.64		
					6.31			
N_i	25		27		29		$N = \sum N_i$	81
s_i^2	0.5392		1.0586		0.6269			
$s_p^2 = \frac{\sum_{i=1}^{k}(N_i - 1)\, s_i^2}{N - k}$							0.7438	
$B = (N-k)\ln(s_p^2) - \sum_{i=1}^{k}(N_i - 1)\ln(s_i^2)$							3.3346	
$C = 1 + \left(\frac{1}{3(k-1)}\right)\sum_{i=1}^{k}\left(\frac{1}{N_i - 1}\right) - \frac{1}{N-k}$							1.0258	
$T = B/C$		3.2508			$X^2_{(\alpha=0.05,2)}$		5.991	
Conclusion: $T < X^2_{(\alpha=0.05,2)}$, we fail to reject H_0								

3.6 Scaled Mixtures

For this work to be valid the three mixtures must be 'scaled equivalents' of each other. While volumetric measurements and 0.45 power chart shapes show this to be true, an additional visual examination was done. For this visual approach, the asphalt pucks were cut into blocks exposing the aggregate of the mixtures. These blocks were then scanned into images to digitally analyze the skeletal make-up of the mixes. The images were digitally scaled according to the physical ratios of the mixtures' NMAS with respect to each other. After digitally scaling, the images of the mixtures were visually indistinguishable. To be sure that these scaled mixtures were equivalent, the images were randomly divided into thirteen equally sized sections and the aggregate in each section was counted. If the number of aggregate in each scaled image is statistically equivalent, then this further supports the assumption that the mixtures are scaled equivalents of each other.

Most of the literature can agree that the aggregate or fines passing the 0.075-mm sieve is filler and does not interact directly with the larger aggregate. Although this filler does affect some properties of the mix, for the purposes of this visual analysis, it was not be considered to affect the skeleton of the mixture. This mineral filler has an upper limit requirement of all passing the 0.6-mm sieve. Consequently, any aggregate larger than the 0.6-mm sieve could be contributing to the structural integrity of the asphalt mixture. Greene theorized that the dominant aggregate size range (DASR) is the interactive range of particle sizes that form the primary structural network of aggregates, and for the 12.5-mm mix only particle sizes greater than 1.18-mm can be considered coarse enough to provide the particle interlock necessary to resist permanent deformation (Kim et al., 2006; Green and Choubane, 2010). For the purposes of this visual analysis the concerned will solely be with the aggregate sizes that directly contribute to the skeleton of the mixture. This includes all aggregate larger than the 0.6-1.18 mm range. Conveniently this range of aggregate size is the smallest range that the eye can clearly see in a scanned image.

At a mid-point in the beam cutting process, each block of the optimum mixtures was scanned. The scanned images of the optimum asphalt content mixtures for the 12.5-mm, 9.5-mm, and 4.25-mm were scaled to 50%, 66%, and 100% original size, respectively; this can be seen in

Figure 3-10. This scaling is based on the US customary measurements where the 4.75-mm = 0.25 inch NMAS, the 9.5-mm = 0.375 inch NMAS, and the 12.5-mm = 0.5 inch NMAS. The 0.5 inch NMAS is two times the size of the 0.25 inch NMAS and the 0.375 inch NMAS is one and one-half times the size of the 0.25 inch mix. Using the 0.25 inch mix at 100% to scale from, we scale $\frac{0.25\ inch\ mix}{0.50\ inch\ mix} = {}^{1}/_{2}$ x $100\% = 50\%$ and $\frac{0.25\ inch\ mix}{0.375\ inch\ mix} = {}^{1}/_{1.5}$ x $100\% = 66\%$. An alternative scaling scheme could be to increase the 4.75-mm mixture x 200% and the 9.5-mm mixture x 150% and keep the 12.5-mm mixture x 100% but this scheme resulted in issues with the image clarity. The resolution of the scanned images was not high enough to support this kind of magnification. Thirteen areas of equal size and random location were selected from each scaled image and the aggregate was counted for each area. Any aggregate visually identifiable within a given area was tallied. If the number of aggregate counted within each area of the three mixtures was the same, we can further confirm that the two smaller NMAS mixtures are in fact scaled equivalents of the 12.5-mm NMAS mix because the solids in the volume fraction images are roughly the same. The tallied aggregates were, in fact, statistically equivalent.

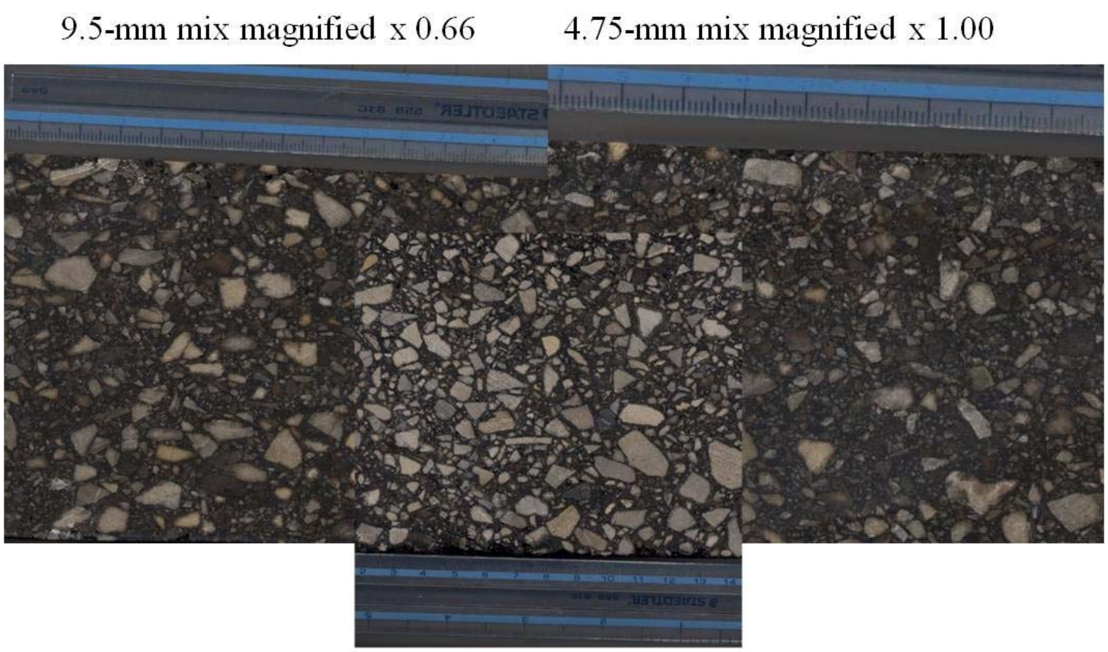


Figure 3-10 Scanned and scaled images of mixtures for visual equivalence evaluation

3.7 Summary

In this section it has been shown that three mixtures of descending NMAS can be created to evaluate if the size of the large particles affect the variability of the results obtained from BBR measurements. The results show that in utilizing the BBR to test asphalt concrete mixtures, NMAS as large as 12.5-mm do not introduce any excess variability than a smaller NMAS of 9.5-mm or 4.75-mm at the sample dimension recommended for use in the BBR (12.7 mm x 6.35 mm x 127-mm). Therefore, the 12.5-mm NMAS can be used with confidence in the BBR at the specified beam size.

3.7.1 List of Observations

- The sample groups in this work were found to be of normal distributions with statistically equal variances.
- Within a constant area, the 4.75-mm mixture was found to have twice the amount of solids as the 12.5-mm mixture and the 9.5-mm mixture was found to have one and a half times as many solids as the 12.5-mm mixture.
- Large aggregate taking up the entire width and/or thickness dimensions of the beams do not create outliers within the data sets because the gauge length is the *length* between supports of the beam when determining the flexural creep modulus.
- It is simple to obtain flexural creep modulus of small asphalt mixture beams using the BBR.

Based on this work, it is concluded that the BBR can successfully be used to test asphalt mixtures. The beam dimensions of 12.7 mm x 6.35 mm x 127 mm meet the RVE requirement for testing and characterization of material.

4.0 Test Repeatability

4.1 Overview

As previously discussed, the BBR was originally developed to test asphalt binders and since the current modified BBR test for asphalt mixtures lacks standardized specification, it is necessary to research if this modified BBR test has adequate repeatability within a single set of lab tests and reproducibility across multiple labs.

4.1.1 Objectives

In order to investigate the BBR test's repeatability and reproducibility, several specific objectives must be addressed:

- *The Reproducibility of the BBR Test Across Labs*
  Different labs with different test operators may arrive at different results. The objective here is to ensure that the BBR test can be performed in multiple labs for the same asphalt mixture, and still arrive at consistent results.
- *The Effect of Time Interval on Materials' Low-Temperature Properties*
  The testing time interval is the time between the sample's creation and when it is tested. It will be examined if varying this time interval for a given sample results in different low-temperature properties that perhaps are caused by steric hardening.
- *The Effect of Repeated Testing on a Single Specimen*
  This objective requires the verification of whether a single specimen can be reused across multiple tests without compromising the consistency of the test's results.

4.2 Procedures

Three asphalt mixture pucks were made from the same mix design, resulting in three identical pucks. Each puck was then cut into 20 beams of standard dimensions on the same day. The beams, after cutting, were then immediately stored in a sealed container in order to prevent any moisture changes to the beam that would result from exposure to the air. Of the resultant 60 beams, 40 were chosen at random to be used in this study. These 40 beams were then randomly divided in half: 20 of the beams were used in the University of Utah (UofU) lab, and 20 were used in the Utah Department of Transportation (UDOT) Central lab. Based on the time at which the beams were cut, the BBR tests were performed at specific intervals of time past the time of cutting. These intervals were: two days (48 hours), three days (72 hours), one week (7 days), and two weeks (14 days); plus or minus 10% of that interval's total duration to allow for slight variation in time available for testing. As there were four intervals to be tested, each lab's set of 20 specimens was divided into four groups of five specimens. Each group of five was tested at each interval (Li 2015).

In addition to these tests run by both laboratories, the UofU lab ran tests not only of the group to be tested at each interval, but also the groups that were tested at previous intervals again at each new interval. The BBR test results were compared for each relevant group between the labs and, additionally, the extra tests run at the UofU lab were compared to the main series of tests for each interval.

4.2.1 Materials

All three asphalt mixture pucks were originally prepared by the UDOT Region 2 lab for volumetric verification and quality control of State Road 89. These pucks were made from one mix design based on that used for State Road 89, with a design binder grade of PG 64-28. The aggregates used for this design were locally sourced. The mixing temperature was 333°F-342°F, and compaction temperature was 312°F-322°F. These samples were taken from the field mix, but compacted in the lab. The specific mix properties of this design can be found in Table 4-1.

All beams of each group and each experiment were tested at one temperature (-18°C). This temperature was 10°C higher than the low temperature performance grade (PG) of the binder used. For each BBR test, measurements of modulus and m-value were recorded at two loading times, 60s and 120s; by using the time-temperature superposition principle this corresponds to test results at two temperatures or testing of mixtures made with softer binders. These two loading times are also the typical loading times used in studying field data.

Table 4-1 Mix design properties

Nominal Maximum Aggregate Size	3/4"	Material Description	Percent
Gradation		Washed Sand	7%
Sieve Size	Passing	1/8'' Unwashed Fines	12%
1''	100%	1/4'' Unwashed Chip	15%
3/4''	100%	1/2'' Unwashed Rock	20%
1/2''	87%	3/4'' Unwashed Rock	25%
3/8''	74%	3/4'' Milled Reclaimed Asphalt Pavement (RAP)	5%
No.4	50%	1/2'' Crushed RAP	15%
No.8	32%	Lime	1%
No.16	22%	RAP Content, %	20%
No.50	12%	Binder Content, %	4.30%
No.200	6%	RAP Binder Content, %	1.10%
		Virgin Binder Content, %	3.20%
Air Voids, %	3.70%	Binder Grade	PG64-28
VMA, %	13.80%	Design Gyratory	100

4.3 Results and Discussion

This study's focus is on analysis of creep modulus and m-values of the asphalt mixture. As previously explained, the BBR will automatically record the applied load and deflection of each specimen. The BBR output files include the calculated modulus and m-value. With the outcome of the BBR test, the data was collected and analyzed for each experiment.

4.3.1 Multi-Lab Comparison

The first experiment was to compare the test results of creep modulus and m-value of the asphalt mixture specimens between the UofU lab and the UDOT lab. There were 4 BBR tests performed by each lab, one at each interval (2-day, 3-day, 1-week, 2-week). Each BBR test included 5 replicated specimens. The stiffness and m-value at 60s and 120s were analyzed. As each test involves five samples, there are five m-values and five stiffness values. In order to use these values in comparisons between tests, they must be simplified into single values for each test. This was done by first averaging the results then determining the Coefficient of Variance (CV), which is the ratio between the standard deviation and the mean of the five values. If this CV is too high, then the specimens with the highest and lowest values are removed as outliers. Once this has been done, or if the CV is within 10% and no such outliers require deletion, then the mean of the values is recorded to be used in comparisons with other tests. If, after the deletion of outliers, the CV of a set of values is still above 10%, the mean is still used without any further deletion. The modulus and m-value percent differences between the UofU lab and the UDOT lab are illustrated in Figure 4-1.

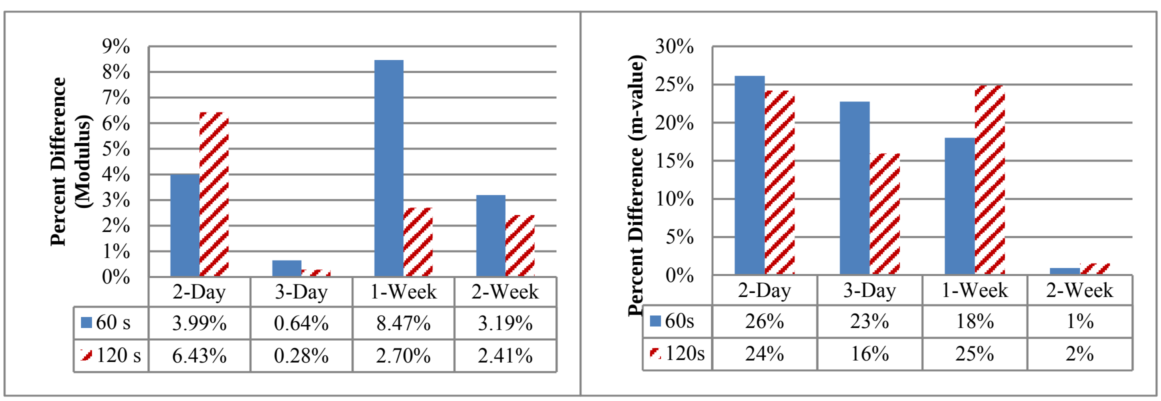


Figure 4-1 Difference in results between UofU and UDOT labs

As shown in Figure 4-1, the percent difference of the modulus at 60s and 120s between the UofU lab samples and the UDOT lab samples for all BBR tests at each of the 4 intervals are below 10%. In order to ensure the BBR test has reproducibility across labs, AASHTO T313 suggested that for multi-laboratory experiments, variation between modulus should be at or below 17.8%. This indicates that the two test results for creep modulus from the two labs are within the acceptable range for multi-laboratory precision. In addition, the highest percent difference in modulus between the two labs is around 8.5% for the 1-week interval test at 60s; this is half of the multi-laboratory precision acceptable range. This finding indicates the modulus measurements using the BBR test between two labs are consistent.

The m-value percent difference for the 2-day interval test, the 3- day interval test, and the 1-week interval test between both labs in Figure 4-1 have large differences at 60s and 120s. The m-value percent difference for the 2-week interval is within 2% at 60s and 120s. The allowable range of m-values for multi-laboratory precision suggested by AASHTO T313 is 6.8%. Only, the m-value percent difference for the 2-week interval test is within the multi-laboratory precision acceptable range. This indicates that the results may not be entirely conclusive for m-value.

Figure 4-2 shows the modulus values for both labs for different interval tests and at 60s and 120s. For both graphs, there is no obvious difference in the modulus measurements across both labs. Furthermore, there is no obvious trend of modulus as a function of time since cutting. The error bar was plotted on both figures based on the standard deviation that was calculated according to lowest percentage of CV from each group of samples. Considering this error bar, both figures show very consistent results for creep modulus measurements for both labs when testing anytime between 2 and 14 days.

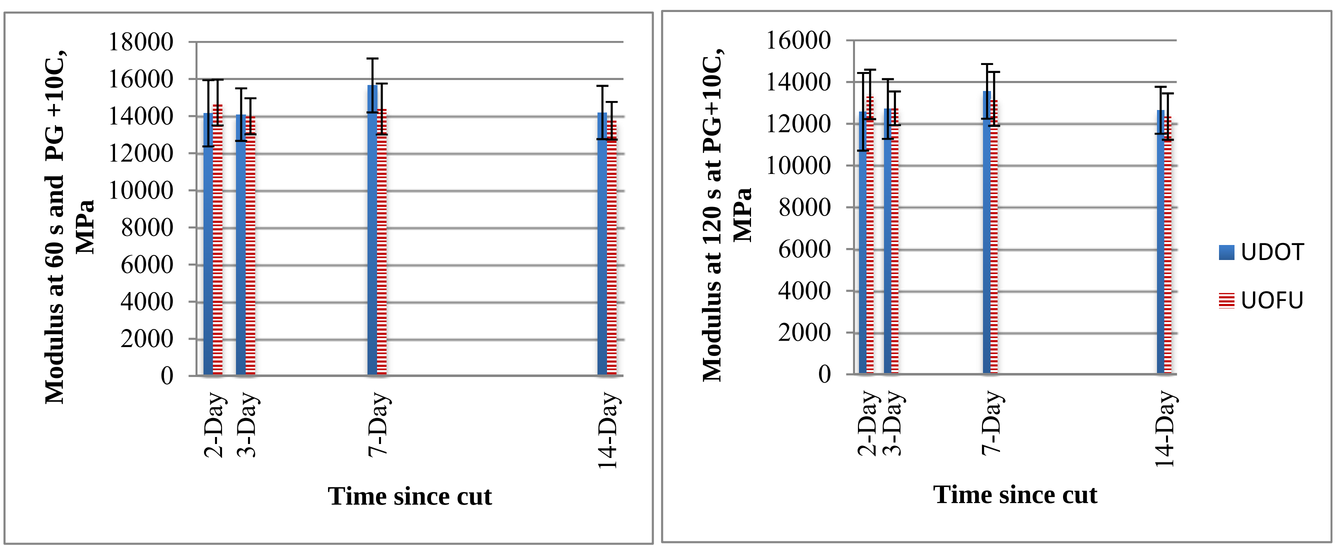


Figure 4-2 Modulus changes as a function of time for both labs at 60s and 120s

4.3.2 Testing Intervals Comparison

The BBR test results were further analyzed between multiple intervals to investigate the influence of different amounts of time from the creation of the gyratory-produced sample to the measurement of modulus and m-value. Figure 4-3 and Figure 4-4 demonstrate the trend of modulus and m-value varying with time of testing. When the data from both labs (UofU and UDOT) is combined, there is a slight downward trend in the modulus as a function of time since cutting. However, the R-squared for each trend line was very small and did not provide evidence that the trend line fit the data set particularly well. Furthermore, at 60 seconds the trend will result in a decrease in modulus of less than 300 MPa at 14 days or about 2 percent from the 2-day value; such a difference is within the accepted margin of error.

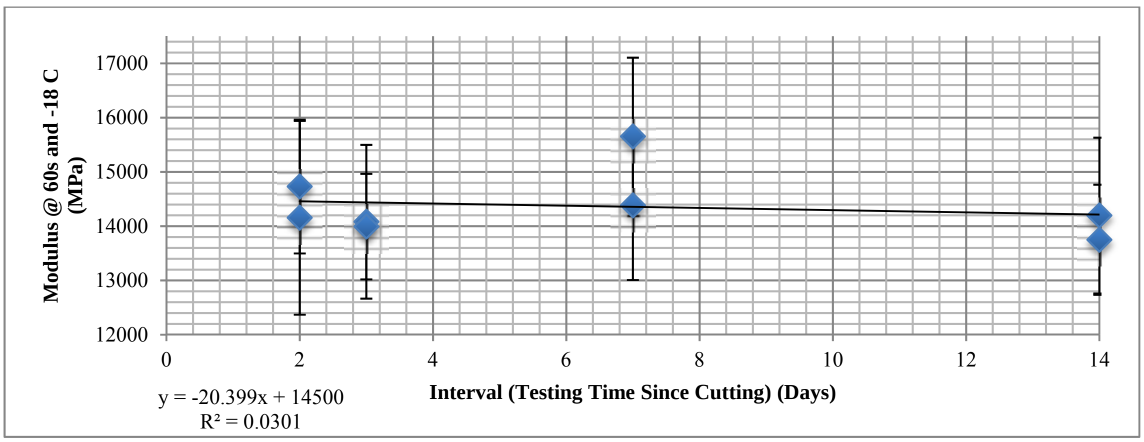


Figure 4-3 Trend in modulus at 60s and -18 C with testing time interval

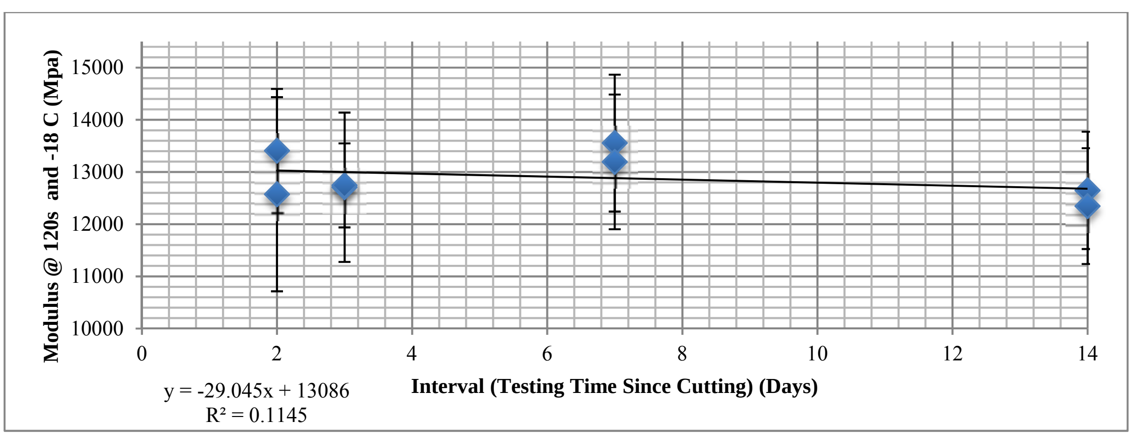


Figure 4-4 Trend in modulus at 120s and -18C with testing time interval

Additional comparisons were made between the m-value between each test interval and 2-day test interval to again analyze the effect of test interval on the m-value. When the results from both labs are combined there seems to be an increasing trend with time. As shown in Figure 4-5 and Figure 4-6, the trend seems to be more pronounce than the one seen in the modulus resulting in an 18 percent increase in m-value after 14 days. However, just like the case for modulus, the low R-square would indicate that, statistically speaking, this is not a significant trend and given the large variability in m-value, the results are considered inconclusive at best.

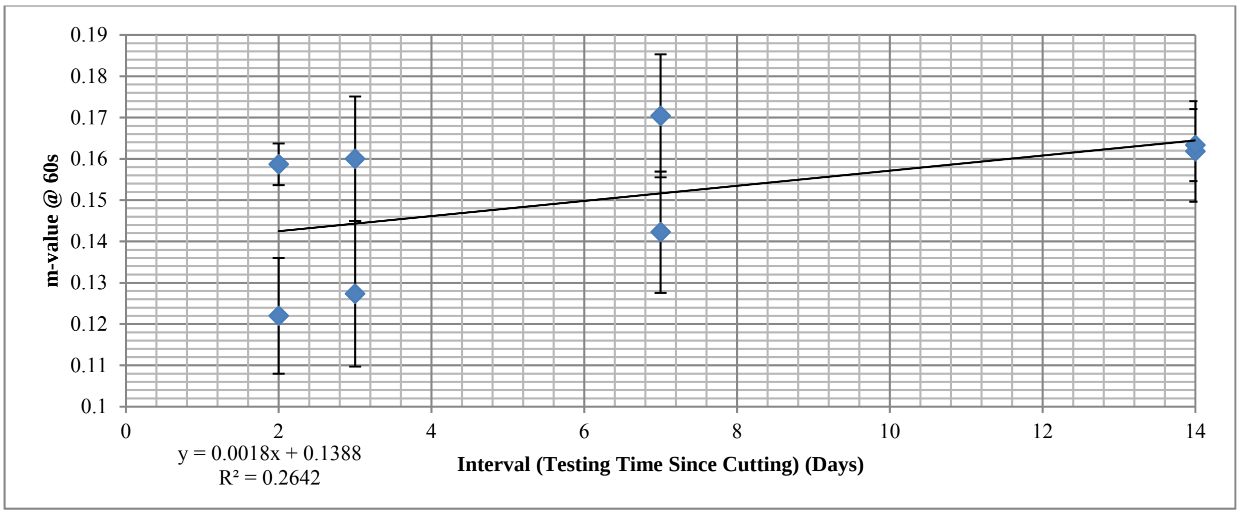


Figure 4-5 Trend in m-value at 60 s with testing time interval

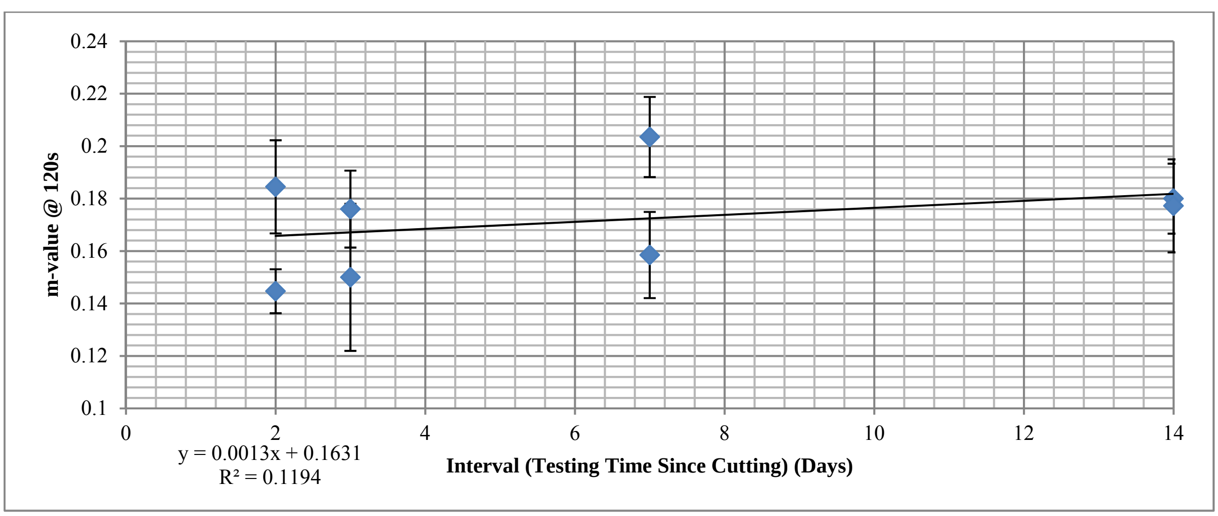
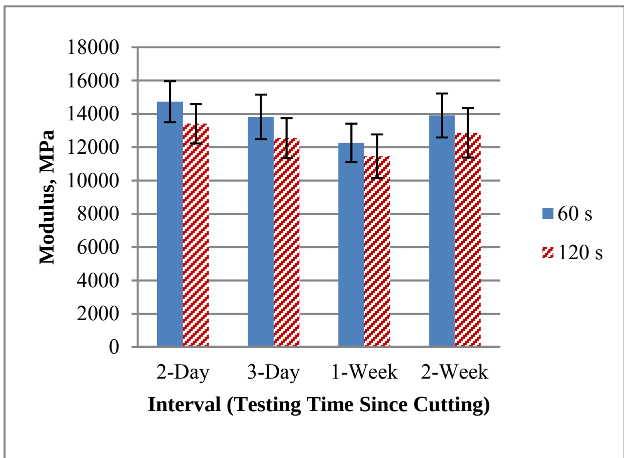


Figure 4-6 Trend in m-value at 120 s with testing time interval

Both modulus measurements and m-value measurements did not show a clear relationship with the testing interval indicating that no steric or age hardening is occurring during that time interval. While, it has been documented that for asphalt binders, steric hardening occurs rapidly at first but appears to approach a limiting degree of hardness on prolonged standing (Barth 1962, Grant 2001); it is likely that, since all samples were prepared to be tested 48 hours after the gyratory sample was made, the mechanical properties for the asphalt mixtures were more stable. This leads to the conclusion that, after 48 hours from being made, the testing interval has very little effect on the measurements of creep modulus and m-value.

4.3.3 Repeated Testing on UofU Samples

The same 2-day interval test samples were repeatedly tested at the 3-day interval, 1-week interval and 2-week interval, while the 3-day interval test samples were again tested at the 1-week interval and the 2-week interval. Figure 4-7 shows the modulus and m-value for the repeat test of 2-day interval test samples and Figure 4-8 shows the same for the 3-day interval test samples. Not only were the results of each actual testing interval examined (i.e. the specimen first examined at that interval), but the results of groups from prior interval tests that were again run at the new interval were also compared to the subsequent actual test. Table 4-2 shows the percent difference of modulus and m-value between the results of each original test at the given interval and the repeated test at that corresponding interval.



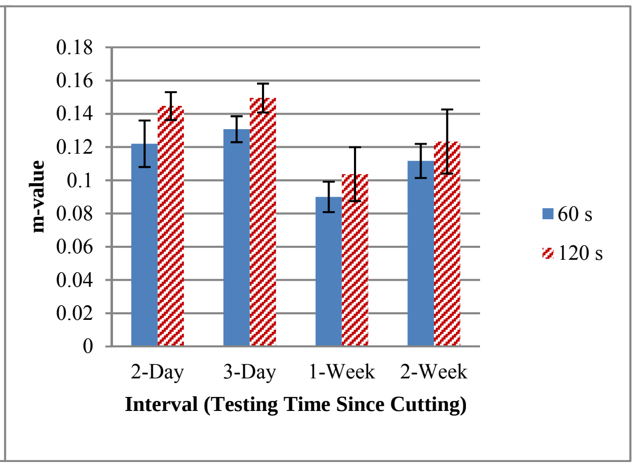


Figure 4-7 Modulus and m-value of repeated 2-day interval samples

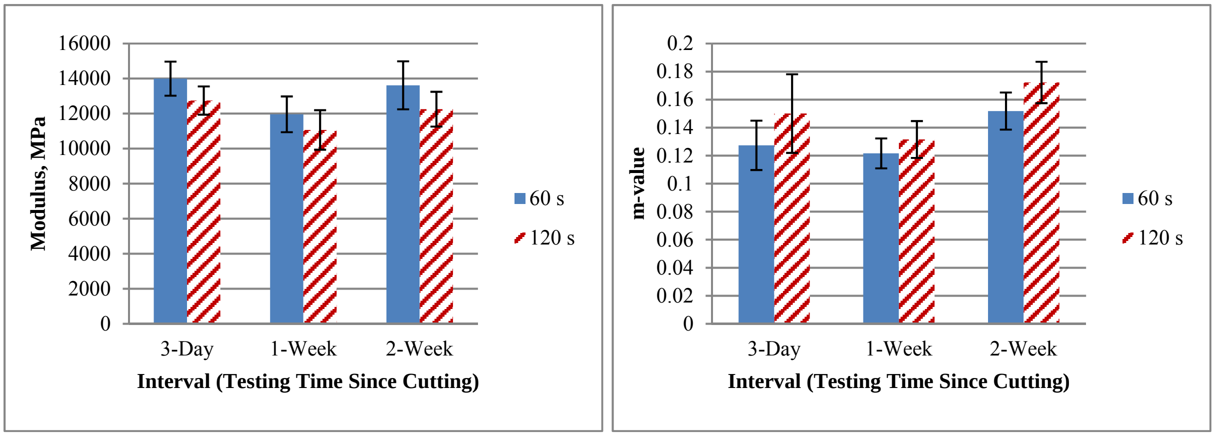


Figure 4-8 Modulus and m-value of repeated 3-day interval samples

Table 4-2. Summary of percent difference of results for repeated test samples

	60s		**120s**	
	Modulus (MPa)	**m-value**	**Modulus (MPa)**	**m-value**
Original 3-Day Interval Test	13992.18	0.13	12743.61	0.15
2-Day Interval Samples Tested at 3-Day Interval	13814.24	0.13	12545.29	0.15
Percent Difference	1%	3%	2%	0%
Original 1-Week Interval Test	14381.85	0.14	13193.46	0.16
2-Day Interval Samples Tested at 1-Week Interval	12259.10	0.09	11449.96	0.10
Percent Difference Refer to 2-Day Interval Samples	16%	45%	14%	42%
3-Day Interval Samples Tested at 1-Week Interval	11959.49	0.12	11064.88	0.13
Percent Difference Refer to 3-Day Interval Samples	18%	16%	18%	19%
Original 2-Week Interval Test	12346.34	0.16	13749.79	0.18
2-Day Interval Samples Tested at 2-Week Interval	13901.28	0.11	12867.57	0.12
Percent Difference Refer to 2-Day Interval Samples	12%	38%	7%	36%
3-Day Interval Samples Tested at 2-Week Interval	13614.40	0.15	12250.74	0.17
Percent Difference Refer to 3-Day Interval Samples	10%	7%	10%	3%
1-Week Interval Samples Tested at 2-Week Interval	13597.89	0.13	12632.59	0.14
Percent Difference Refer to 1-Week Interval Samples	10%	22%	8%	25%

In Table 3-1Table 4-2 the comparisons between each test run at a given interval are shown, i.e. the 3-day interval compares the results from the original 3-day specimens and the 2-day specimens that were re-tested at the 3-day interval. The data shows that there is almost no difference between the first time test at 3-day interval and the 2-day interval beam re-tested at 3 days. As the same 2-day interval beam is re-tested a third time at 1 week (i.e., four days later), the percent difference grows to about 15 percent. The same beam then sat at room temperature for a seven days and was re-tested a fourth time at two weeks. The percent difference went down to 12 and 7 percent for 60s and 120s, respectively. Similar results are observed for the 3-day specimen when re-tested at 1 week and 2 weeks.

These results indicate that the modulus measurements for a single beam are repeatable even after previously being tested in the BBR with only about a 15 percent error. The percent difference decreases if the beam is allowed to relax at room temperature. However, the m-values have large variation for the repeated 2-day and 3-day interval specimens. For m-values, the 3-day interval had a percent difference of 5% for 60s and 120s between the original 3-day specimen and the 2-day specimen. The other intervals, 1-week and 2-week, have large percent differences of over 35 percent between repeated specimens and the original interval specimen. Once again given the variability in m-value, the results are inconclusive.

4.4 Summary

Based upon the results of this investigation into the repeatability and reproducibility of the modified BBR test on the asphalt mixtures, the following conclusions can be drawn:

1. The BBR test has good reproducibility across multiple laboratories for quantifying the low temperature performance of asphalt concrete (less than 10% difference).
2. Steric hardening or any time-dependent change has no effect on BBR test results after 48 hours, since measurements of modulus and m-value did not vary with time interval. This does not apply to oxidative aging.
3. The BBR test can be repeated on the same beam without compromising its consistency, as long as testing is done at the proper low temperature.
4. Modulus has less variation than m-value in all of the comparisons.

5.0 Relation to Field Performance

5.1 Introduction

The final evaluation of the effectives of BBR testing on asphalt mixtures at low temperature is to determine if the results relate to field performance. Testing was completed for seven field sections as well as six laboratory mixes to evaluate: (1) the test method, in terms of practicality and precision, to determine reliability of laboratory samples as a representative of field performance, and (2) the possibility of using a single point measurements such as creep modulus or m-value at 60 seconds for quality check of the in-place material. This section details the testing methods employed in the study, resulting data, field surveys, laboratory comparisons, and conclusions formed from the results of each.

5.2 Field Samples

5.2.1 Site Selection

Field cores were taken from 7 State roads around the Salt Lake Valley, each of which were constructed based upon UDOT design specifications. The selection of the sections was based upon the following criteria:

1. Constructed within the past year.
2. Thick pavement layers to ensure any visible distress was not reflective of the underlying layers.
3. All were built using the same low-temperature binder grade (-28°C).
4. There are materials available to recreate laboratory samples.
5. Had ability to obtain cores.

In order to obtain cores, the road or lane must be closed following UDOT safety protocols. Without express permission from UDOT this cannot be done, thus certain roadways were not available for use in this study (e.g., interstate highways). The locations of these cores can be seen in Figure 5-1. Two to four six inch cores were taken from each section. The cores were taken in close proximity to one another; thus it can be assumed that cores taken from the same road are of the same mixture and should have very similar properties. The cores were numbered in order and were grouped according to the road from which they were taken. For example, cores 590 and 591 both were taken from SR 266. All core numbers and the roads they came from can be seen in Table 5-1. The exact location is shown in Appendix I.

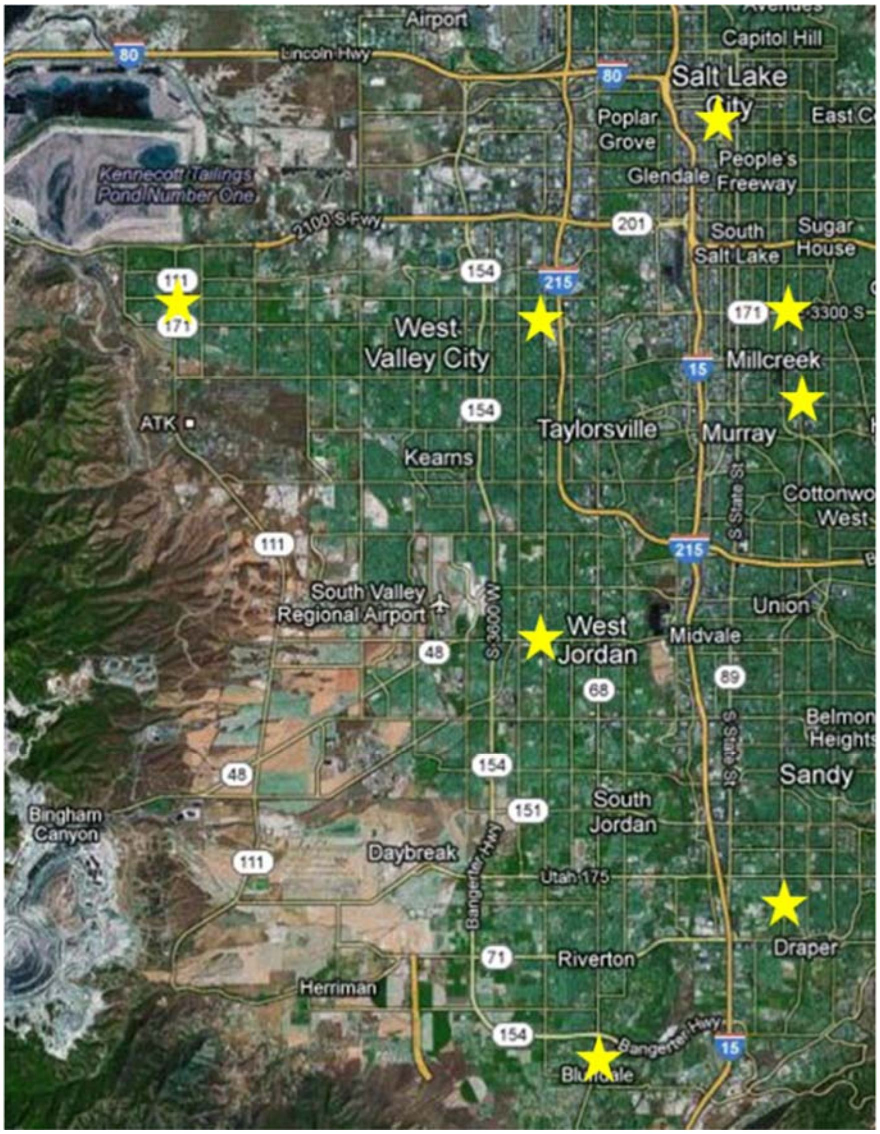


Figure 5-1 Map of the Salt Lake Valley with stars indicating core locations

5.3 Mix Design Information

All road surfaces evaluated were designed based on UDOT specifications (UDOT, 2015). They were all Superpave, densely graded mixtures designed based on an N-design of 100 gyrations (except for SR 68). The VMA was in the range of 13-14% and the air voids were between 2.5-3.7%; the percent RAP was not measured but typically varied between 20% and 25%. As shown in Table 5-1, the low-temperature binder grade of all sections was -28°C except for SR 48 (Jones et al., 2014). The specific mixture properties are shown in Appendix II.

Table 5-1 Summary of field sample results

Project	Core ID	Binder Grade	Creep Modulus @ 60s -18 ºC (MPa)	Coefficient of Variation Creep Modulus (%)	m-Value @ 60s -18ºC	Beams Tested
SR 171	576	PG64-28	2 938	8.5	0.233	8
	577		2 715	10.9	0.211	4
	578		2 626	15.1	0.280	8
	579		2 550	12.2	0.285	6
SR 111	580	PG64-28	9 081	15.7	0.103	10
	581		11 386	10.9	0.124	10
SR 269	586	PG64-28	5 726	15.4	0.159	5
	587		5 186	15.5	0.179	10
SR 266	590	PG64-28	6 523	6.0	0.084	4
	591		7 388	12.7	0.130	4
SR 71	592	PG64-28	9 533	10.2	0.126	13
	593		8 931	13.8	0.127	11
SR 68	594	PG64-28	4 284	7.1	0.185	5
	595		4 547	10.4	0.181	7
SR 48	596	PG64-34	10 437	13.3	0.160	12
	597		10 774	14.1	0.151	16

5.3.1 Quality Control of Data

A quality check of the data was conducted for each core by using the creep modulus at 60 seconds during each test. The coefficient of variation (CV) was determined by dividing the standard deviation by the mean. Previous work has shown that a CV of 15% or less is reasonable when testing asphalt mixtures (Romero and Anderson, 2001). These works also show that when conducting analysis of many beams, such as 50 or more, the results are similar to results from far less beams so long as the CV is 15% or less. In cases where the CV was greater than 15%, a trimmed mean method was used as described in Section 4.3. The trimmed mean method is particularly useful for this study because it removes the samples with results lying furthest from the mean in both the positive and negative direction. This allows for the data to take the form of a normal distribution, as any group of samples from the same mixture should be.

Once the variability of the test was verified, the modulus of each sample beam was used to calculate the average modulus of each core at the selected temperature. The point of evaluation was selected to be 60 seconds. From a mechanical point of view, it is important to have the point of evaluation be at least 10 seconds after the initial load to allow for stabilized readings. After this, the time which is taken for the point of evaluation is irrelevant as long as it is consistent throughout each test. The point of evaluation was taken at 60 seconds for two reasons: (1) 60 seconds is the default output for the BBR testing program (refer to Figure 2-4) and (2) it is also the same for the BBR binder testing protocol AASHTO T313/ASTM D6648.

5.4 Field Sample Test Results

5.4.1 Variability

As can be seen in Table 5-1, the coefficient of variation for each core was 15% or less. The difference in creep modulus between cores was less than 10% for all but one section as shown in Figure 5-2.

During preparation, precautions were taken to ensure that the layer each beam came from was documented. This allowed for evaluation of the modulus at different depths within each core. No correlations were observed between the depth of the sample and modulus.

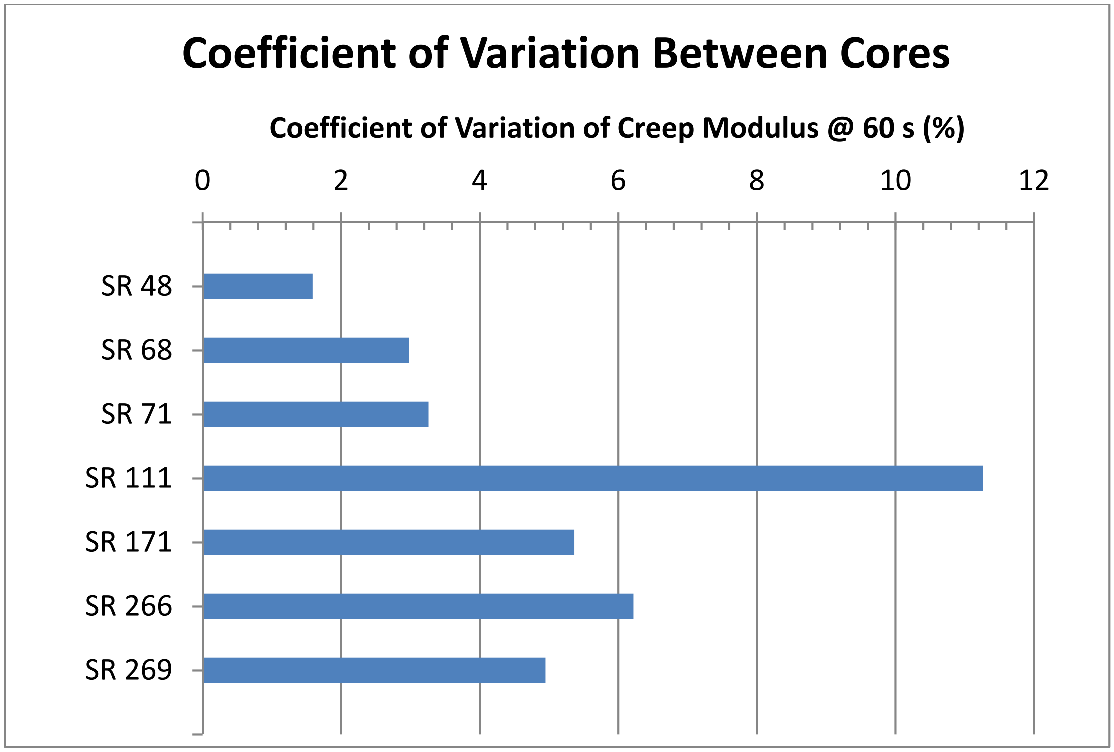


Figure 5-2 Comparison of variation between cores from each section

<u>5.4.2 Creep Modulus and m-value</u>

As can be seen in Table 5-1, the values of the creep modulus varied widely even though all asphalt binders used had the same low-temperature grade. For example, SR 171 had an average creep modulus of 2 700 MPa while SR 48 had an average creep modulus of 10 600 MPa despite the fact that both of these sections specified PG64-28 binder. The m-values for these two sections were 0.252 and 0.156, respectively.

This difference in modulus indicates that both binder and mixture properties influence performance characteristics of pavements. Other research has shown similar results and has tried to bridge the gap by modeling the different components (Christensen et al., 2003). Rather than model the different components, BBR testing allows for direct measurement of mixture properties.

As previously mentioned, the results had a wide range of creep moduli and m-values. However, two roads stood out: SR 111 and SR 48 both had relatively high creep modulus when compared to the other roads even though SR 48 used a PG 64-34 binder. Material with a high modulus has been shown to be prone to thermal cracking (Deme and Young, 1987). A very simple explanation for this is a drop in temperature that causes thermal strain based on the material's coefficient of thermal contraction ($\varepsilon = \alpha\Delta T$), for a strain controlled condition the stress will depend on the modulus ($\sigma = \varepsilon E$), higher modulus will mean higher stress. Because of this, it was predicted that these two roads had the highest potential to show low-temperature thermal distress.

5.5 Black Space Diagram

Asphalt concrete mixtures are viscoelastic materials; because of this, it is important to evaluate not only the structural reaction which takes the form of stress, but also the energy component of the reaction. When a viscoelastic material is loaded, the work done by the external load is either stored as potential energy by the material or lost through heat, flow, etc. At low temperatures the flow the material, asphalt concrete, is limited; thus when the materials rate of relaxation fails to keep up with the rate of deformation, the energy balance is maintained by the creation of a new surface in the form of a crack.

Rheological plots of asphalt materials which relate the structural reaction in the form of dynamic modulus, like the shear modulus (G^*), and the energy component in the form of phase angle (δ) are known as Black Space diagrams (King et al., 2012). These diagrams are typically created from results of Dynamic Shear Rheometer testing but, since at low temperatures asphalt mixtures have very low phase angles, it is reasonable to substitute modulus and m-value from BBR results for G^* and δ, respectively, in the Black Space diagram. Mathematically, the phase angle is approximately equal to the derivative of the logarithm of stiffness, much like the m-value (Booij and Thoone, 1982) and related to the energy dissipated.

Using a Black Space diagram allows for the evaluation of the relationship of creep modulus and m-value when assessing BBR test results of asphalt mixtures. As discussed on the previous paragraph, both the creep modulus and the m-value are needed to characterize the materials and predict low-temperature cracking. Finally, although Black Space diagrams typically create a master curve from multiple data points, a variation of this method could compare multiple mixtures by way of a single point of evaluation. In the case of BBR testing, it is logical to choose 60 seconds since it is the default output of the test. Figure 5-3 shows the Black Space diagram of the field samples.

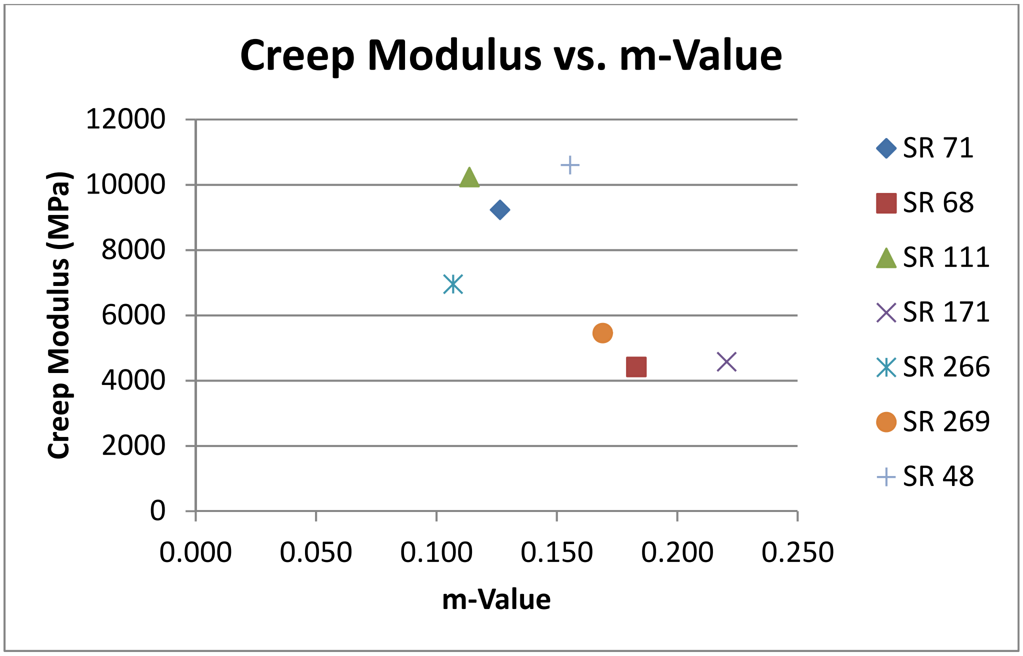


Figure 5-3 Black Space diagram of field samples

<u>5.5.1 Data Analysis</u>

As can be seen in Figure 5-3, when the data is plotted on a Black Space diagram two distinct groups are formed, four sections: SR 111, SR 48, SR 71, and SR 266 are grouped in the upper left of the plot while the rest of the sections SR 269, SR 68, and SR 171 are grouped in the lower right. Based on our knowledge of material behavior, it is expected that the four sections in the upper right would show poor low temperature performance while the sections in the lower right would be more suited to resist cracking. This is an expansion of what was mentioned in Section 5.4.2 that includes both modulus and m-value.

5.6 Field Surveys

In order to validate the predictions from the previous section, it was necessary to survey the roads from which the cores came from and determine if the pavements cracked or not.

The location of the core removal was found in every road to ensure the accuracy of the survey. Each road was surveyed and photographed to document signs of thermal cracking and degradation or the lack thereof. Initial surveys were conducted on three separate occasions:

1. June 13th, 2012
2. January 9th, 2013
3. January 23rd, 2013

The surveys that took place on June 13th, 2012 resulted in no visual thermal distresses on any of the sections. Surveys on January 9th, 2013 also showed no thermal distresses. In the days following January 9th, 2013 the Salt Lake Valley experienced extremely cold weather, as shown in Figure 5-4 (NOAA, 2013). In the days following these extremely low temperatures, it was determined that one more round of visual surveys would be necessary. On January 23rd, 2013 each section was surveyed once more. As predicted, SR 111 showed signs of thermal distress in the form of thermal cracking. This can be seen in Figure 5-5. SR 111 cracked during the first winter even though the temperature was higher than the low temperature binder grade of -28 ºC.

Although both SR 111 and SR 48 have high creep moduli, SR 111 has a significantly lower m-value, or a lesser ability to relax stress. This observation confirms the idea that energy absorption and loss, must be considered along with modulus when evaluating asphalt concrete mixtures.

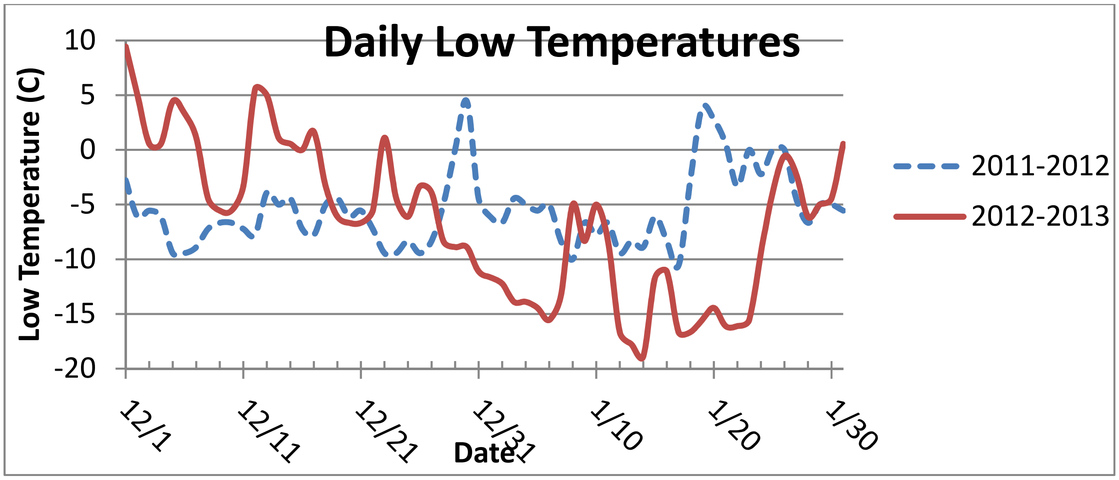


Figure 5-4 Daily low temperatures for Salt Lake City

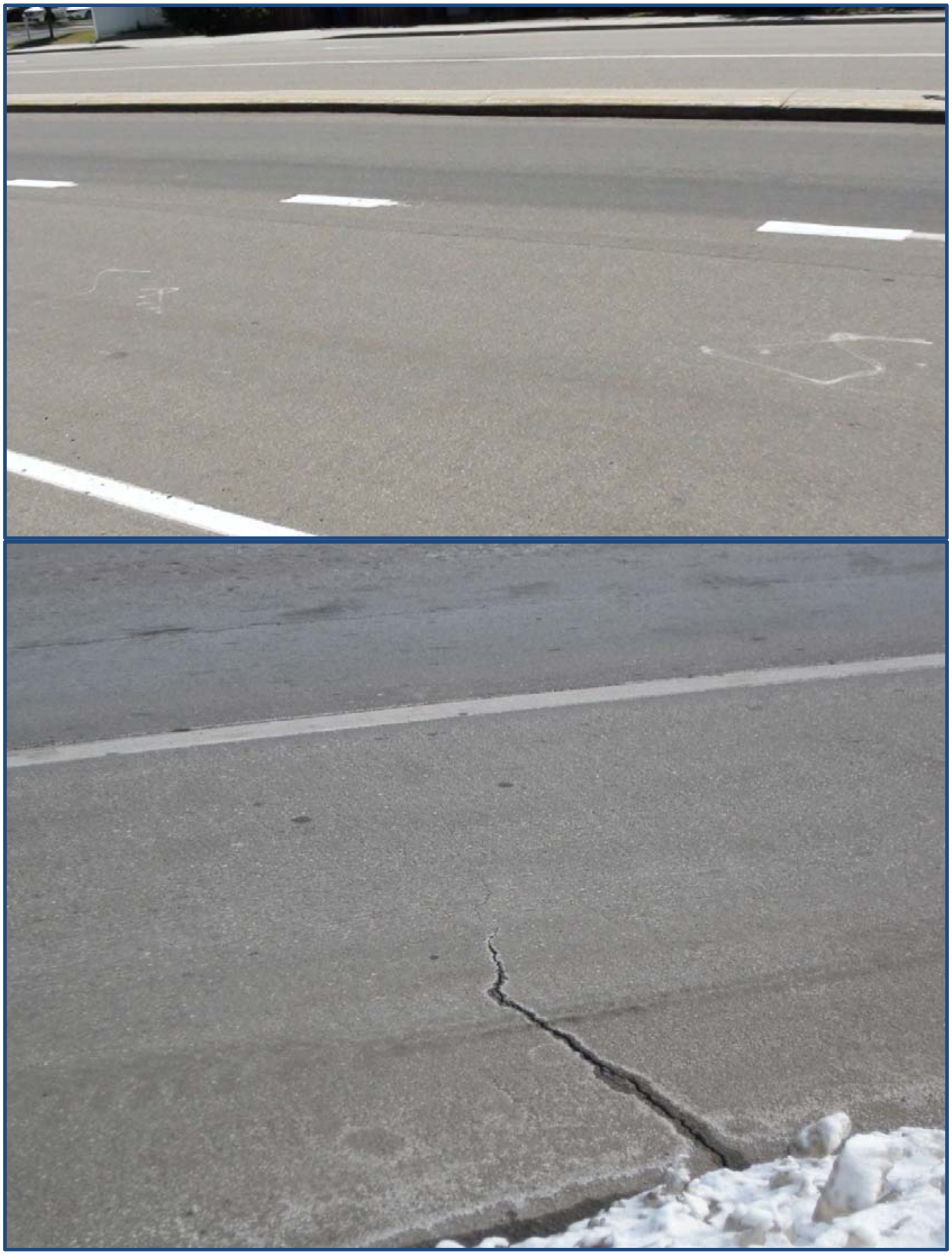


Figure 5-5 SR 111 on June 13, 2012, no visible thermal distress (Top) and January 23, 2013, showing a thermal crack (Bottom)

The roads were surveyed again during the following winter in the month of February 2014. Two more roads showed visible cracks: SR 71 and SR 266. These two other roads were part of the original four sections that were predicted, based on the Black Space diagram, to crack. Interestingly enough, SR 48 which has the highest creep modulus of all the sections did not crack. This road had the highest m-value of all section in that group. As previously explained, high m-value relates to an ability to relax stresses.

Based on this information, a failure envelope was proposed. This envelope is presented graphically in Figure 5-6.

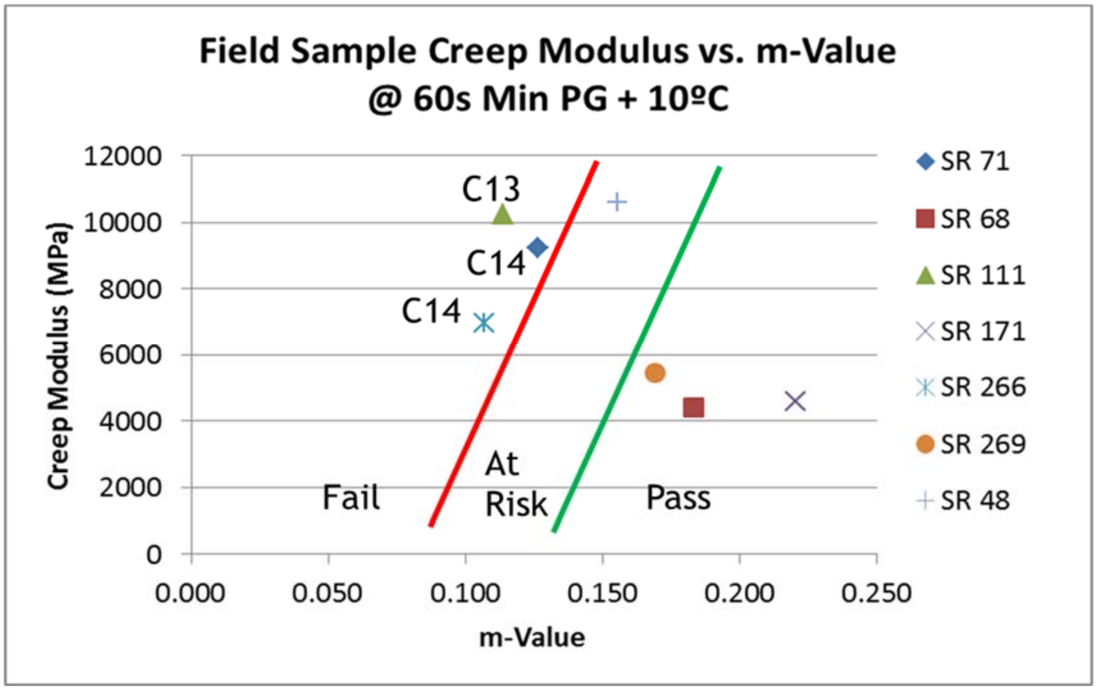


Figure 5-6 Black Space Diagram showing possible performance envelope

The C followed by the numbers next to the sections indicate the winter in which the pavement first cracked (i.e., 2013 and 2014)

5.7 Laboratory Samples

The next step in the study was to reproduce laboratory samples of each section for which the materials were available. This is important at the mix design phase as results will help determine if the samples are representative of how the mixture performs in the field. If the test results from the laboratory samples correlate well with the test results of the field cores then, theoretically, samples could be created and tested to determine the low-temperature performance of the mix prior to construction to avoid costly failures.

The samples were constructed following the original mix designs and by using the same raw materials, even going so far as to collect aggregates and RAP from the same pits and using binder of the same year from the same plant. Once the laboratory samples were created they were tested and analyzed following the same protocol as previously described.

5.7.1 Results from Laboratory Prepare Mixtures

A summary of laboratory sample test results can be seen in Table 5-2. Laboratory sample results displayed a wide range of creep moduli and m-values. All samples also had a satisfactory coefficient of variation.

Table 5-2 Summary of lab results

Project	Binder Grade	Creep Modulus @ 60s, -18ºC (MPa)	Coefficient of Variation of Creep Modulus (%)	m-Value @ 60s, -18 ºC
SR 68	PG64-28	14 842	12.7	0.156
SR 71	PG64-28	8 367	15.5	0.162
SR 111	PG64-28	9 578	12.2	0.161
SR 171	PG64-28	11 403	15.4	0.150
SR 266	PG64-28	14 900	15.4	0.141
SR 269	PG64-28	13 141	15.7	0.132

5.7.2 Comparison of Lab and Field Results

The test results of laboratory prepared samples and field cored samples were compared. Figure 5-7 shows the comparison of creep moduli for each available section. Figure 5-8 shows the comparison of m-values for the same sections. A line of equality is present in both figures. It can be seen that, except for two sections, the creep modulus for laboratory samples is considerably greater than that of the field samples of the same mix design. They also do not show a linear correlation as would be expected. It is also apparent that the m-value for laboratory samples and field samples do not demonstrate any correlation with each other.

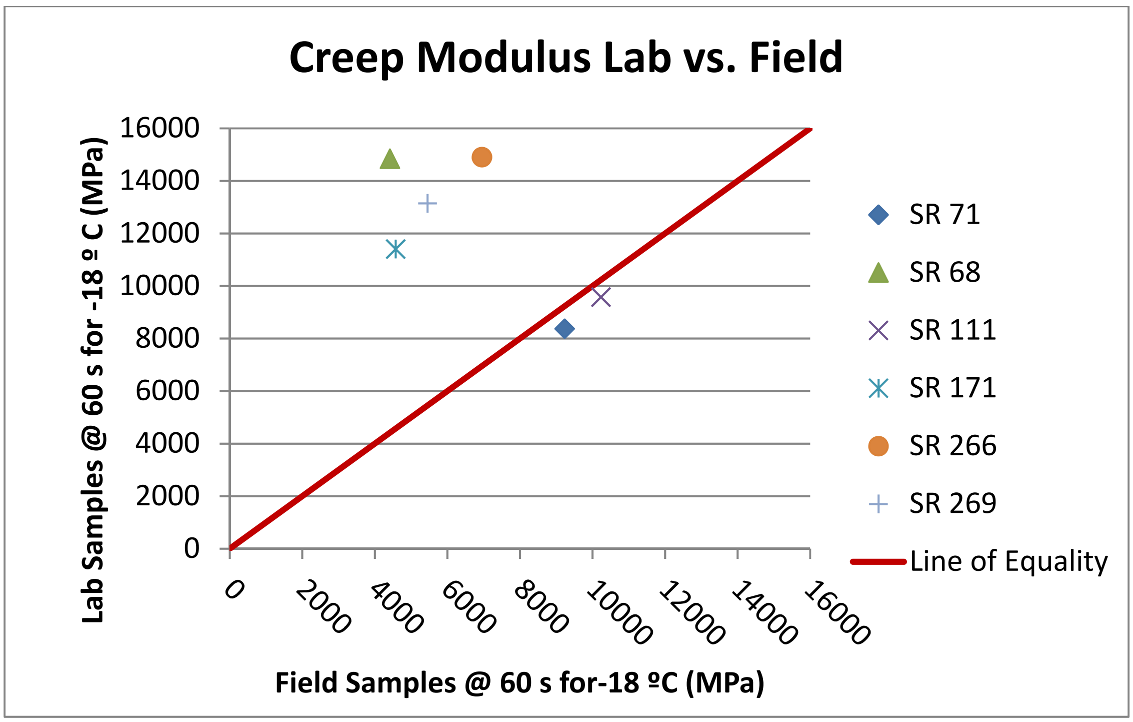


Figure 5-7 Comparison of laboratory and field sample creep moduli

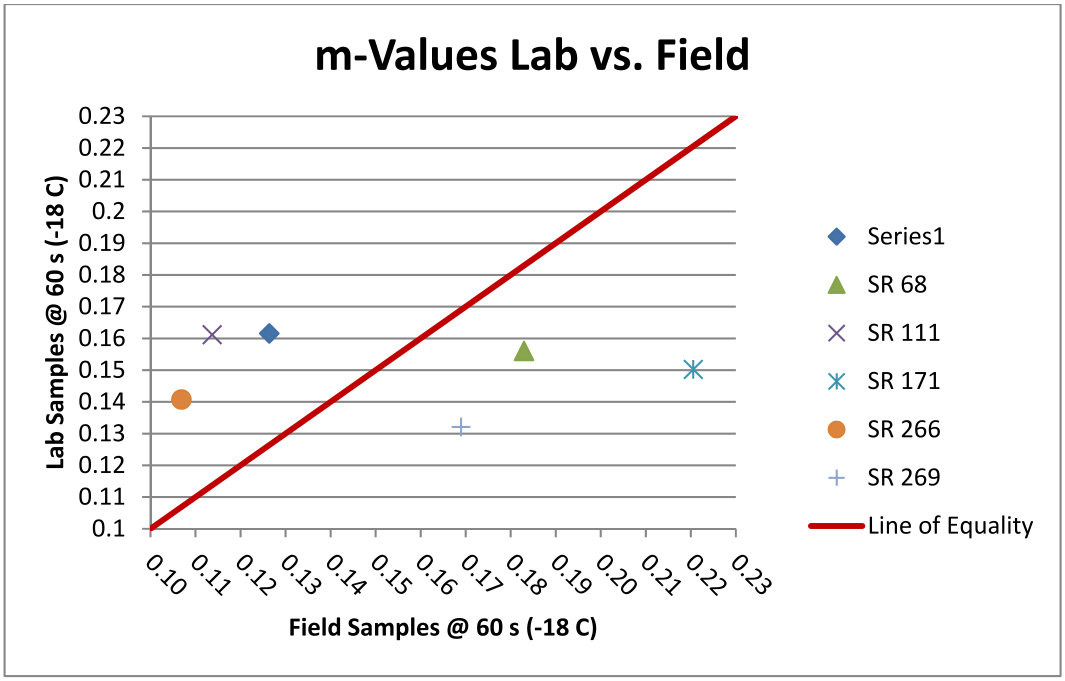


Figure 5-8 Comparison between laboratory and field sample m-values

Possible explanations for the reasons the laboratory sample results do not match that of the field samples include variation regarding RAP and aggregate sources and mixing methods. Although care was taken to obtain RAP and aggregates from the same source as the original mixture it is possible, and likely in the case of RAP, that the material obtained was not identical to the material used in the original mix. It is possible that, after more than one year, the source of the RAP used to create the laboratory samples has aged differently resulting in different absorption and in different binder grades than the RAP used in the field mix. If this is true, then the response of the laboratory mix will be different than that of the field mix even though the both mixes had similar RAP content and source. Another possible source of variation comes from mixing methods. In the lab it is possible to precisely control the lending and mixing procedure. Such control is much more difficult to achieve in the construction process with such large batches. Any deviation in the mixing procedure could produce varied results.

Given the difficulties in reproducing field properties in the lab, it became clear that more research is needed to achieve satisfactory results. Such research was outside the scope of this work thus no further analysis was performed.

5.8 Summary

The response of field cores and subsequent viscoelastic analysis showed that even though the same binder grade is used in the region, the resulting asphalt mixtures have significant differences in creep moduli and m-values. This leads to the undeniable conclusion that binder testing alone is not enough to determine the expected material behavior and eventual performance. Mixture testing is necessary to properly characterized asphalt mixtures and predict performance.

Any specification used to predict low-temperature performance will need to include the accumulation of thermal stresses and the relaxation ability of the material which are represented through the creep modulus and the m-value, respectively. These values are easily obtained from the BBR tests.

Based on the proposed failure envelope, it is theorized that a mixture with high modulus can be used in pavement construction as long as it has a high m-value. Knowing that a high modulus mix with a high m-value can successfully perform at low temperatures can be beneficial in that it can also resist permanent deformation. These results could help optimize mixtures for both high and low temperature conditions.

Finally, although every attempt was made to reproduce field mixture properties in the lab; it became evident that the same material sources would results in different mechanical properties. Thus lab mixtures are not considered representative of field mixes for this project. The reasons for these differences are not clear and should be further investigated.

5.8.1 Recommendations

While a failure envelope was proposed based on a total of seven field sections, further research should focus on taking field cores of thick layer pavements with known mix designs that show thermal distress to verify the conclusion which states that pavements with a combination of high creep moduli and low m-values are more prone to thermal distress. Analysis of more mixtures that are prone to thermal stress will allow for a more accurate definition of the proposed thermal stress failure envelope. Field testing of pavements that do not show thermal distress will also be beneficial in defining the thermal stress failure envelope.

Sources of these pavements should not be limited to state roads; they should also include city, county, and federal sections.

More research is needed in order to reproduce the response of field samples with lab samples and thus predict performance. It is recommended that for future new construction or full-depth reconstruction projects, a sample of field mix be stored in a sealed can in order to prevent aging. This will allow for the mix to be compacted in the lab and tested. Results from these tests would help indicate whether the relationship between lab and field samples is strictly influenced by material variance or construction differences.

6.0 SUMMARY, CONCLUSONS, RECOMMENDATIONS AND IMPLEMENTATION

6.1 Summary of Results

The following results were obtained as a results of this multi-year study:

1. Low temperature performance of asphalt concrete pavements is a concern for highway agencies in cold regions. While there are many tests that have been proposed to evaluate the low temperature performance of asphalt mixtures, none of them have been adopted for routine testing. Testing asphalt mixtures using the bending beam rheometer (BBR), a device originally designed to test asphalt binders, was shown to be a promising test due to its practicality while still being theoretically valid.
2. The BBR test requires asphalt mixture specimens to be cut out of laboratory compacted samples or field cores into beams that measure 12.7 mm x 6.35 mm x 127 mm (width x thickness x length) with tolerances in the width and thickness of ±.0.25 mm. More than 20 beams can be obtained from one gyratory-compacted sample.
3. Once cut, the specimens are conditioned for 1 hour in the BBR bath at a temperature 10 ºC higher than the expected performance grade for the location where the asphalt mixture will be placed. After one hour, a load of 4500 mN ± 50 mN is applied to the midspan of the beam and the deflection is measured as a function of time for 240 seconds. The BBR software calculates and reports the creep modulus and m-value of the beam and highlights the results at 60 seconds. For consistency and convenience, these two value were selected for evaluation although values at any other loading time or even multiple values or temperatures could be used for performance modeling.
4. Based on experimentation and statistical analysis of mixtures with different aggregate sizes, it was shown that any large aggregate taking up the entire width and/or thickness dimensions of the beams do not create outliers within the data sets because the gauge length is the length between supports of the beam when determining the flexural creep modulus. It was concluded that the beam

dimensions meet the representative volume element requirements for testing and characterization of asphalt mixtures.

5. Comparison done between tests run at two different laboratories by two different technicians using two different BBR machines showed that the difference in the modulus between two labs was less than 10 percent. The m-value had differences of over 20 percent indicating inconclusive results for this parameter.
6. Measurements of mixture modulus and m-value taken between 2 and 14 days since fabrication of the gyratory sample and cutting of the small beams showed no difference in results. Steric hardening or any time-dependent change not related to oxidative aging has no effect on BBR results after 48 hours.
7. If needed, the same beam can be re-tested without compromising the results as long as testing is done within the right temperature conditions.
8. A Black Space diagram where creep modulus is plotted against m-value was used to characterize the asphalt mixtures in terms of both stress accumulation (modulus) and stress rate of relaxation (m-value). Out of seven mixtures evaluated in the field, the three that had high modulus AND low m-value showed thermal cracking during the first two winters. One of the mixtures with high modulus AND high m-value did not cracked. All seven mixtures had the same asphalt binder grade specified but different mixture components (aggregate source, RAP content, etc.) indicating that mixture testing is needed to characterize performance.

A specification that incorporates both modulus and m-value is proposed.

6.2 Conclusions

This work showed that the bending beam rheometer is a viable test to determine the low temperature performance of asphalt mixtures; it balances the rigor required of any mechanical test and the relation to field performance with the practicality of a procedure that can be easily implemented by the Utah Department of Transportation.

The protocols developed parallel existing asphalt binder protocols thus allowing for easy implementation since they are based on robust and commercially available equipment that requires no modification. Furthermore, many of the staff and technicians are familiar with the procedures and operation of the equipment. The values for data analysis are calculated and reported by the existing equipment software requiring no further data manipulation unless detailed modeling is desired.

The size of the beams used for this test are a representative volume element. Mixtures with nominal maximum aggregate size of ½” or even larger can be tested without large aggregates introducing any further variability. The size of the beams also allows for multiple specimens obtained from either gyratory compacted samples or field cores; agencies and contractors can test mixtures and compared results as the difference between two labs was determined to be less than 10 percent for the modulus and about 20 percent for the m-value.

Both creep modulus and m-value are used to determine the low temperature performance of asphalt pavements. These values can be visually presented in a Black Space diagram. Mixtures with high modulus and low m-values are expected to show poor performance but mixtures with high modulus AND high m-value should have better performance, perhaps allowing for better performing asphalt mixtures at both low and high temperatures.

The benefits found from using the BBR to test small beams of asphalt mixtures indicate that this is an ideal test to evaluate the low temperature performance of asphalt mixtures

6.3 Recommendations and Implementation Plan

Based on the information presented in this work, it is recommended that UDOT start implementing the BBR as a mixture test to evaluate the low temperature performance of asphalt pavements. During the next paving seasons, asphalt mixtures should be collected from projects across the state. The mixtures should be tested following the procedures outlined in this report and the pavement performance should be monitored. Using this updated information, a failure envelope similar to the one shown on Figure 5-6 should be developed and eventually used as a performance based specification.

6.4 Suggestion for Future Work

While a significant amount of work has been done to develop a procedure for mixture testing using the BBR, there are still many unknowns. The following experiments are suggested to increase the knowledge of the low temperature behavior of asphalt mixtures.

1. Develop a relation between lab-mixed lab-compacted (LMLC) material and plant-mixed field-compacted (PMFC) materials. Section 5.7 showed that there are still many unknowns in when relating the properties of the same materials prepared through different processes. Variations in volumetric properties and short-term aging procedures should be evaluated using the BBR. Establishing a relation between LMLC and PMFC as well as the effect of varying volumetric parameters will allow for evaluation of the material during the design phase as well as using the BBR test for quality control process.
2. Develop an understanding of the effects of Recycled Asphalt Pavement (RAP) on the low temperature performance of asphalt mixtures. Control asphalt mixtures with different amounts of RAP should be tested using the BBR. Once there is a clear understanding of the effect of RAP, different amounts and types of RAP can be evaluated; in cases where the use of RAP type or amount results in poor performance, preventive measures such as used of rejuvenators can be taken.
3. The rate of oxidative aging of asphalt mixtures along with the combined effects of aging with RAP remains unknown. Oxidative aging results in mixtures with higher creep modulus and lower m-value thus it is possible that a non-aged asphalt mixture might pass any specification but fail once placed in the field. An experiment in which asphalt mixtures are tested at different aging intervals, perhaps as proposed by ongoing NCHRP 9-54 research, should be done. Section 4.3.3 demonstrated that the same beam can be tested multiple times.
4. The relation between stress relaxation and energy dissipation at intermediate temperature should be explored.

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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., and Turos, M. "*Determination of Asphalt Mixture Creep Compliance at Low Temperatures Using Thin Beam Specimens*." Journal of the Transportation Research Board, No. 2057, Transportation Research Board of the National Academies, Washington, D.C., 2008, pp. 134–139.

Zofka, A., Marasteanu, M., and Turos, M. "*Investigation of Asphalt Mixture Creep Compliance at Low Temperatures*." Journal of Road Materials and Pavement Design, 2008, Vol. 9, pp. 269-286.

APPENDIX I LOCATION OF PAVEMENT SECTIONS

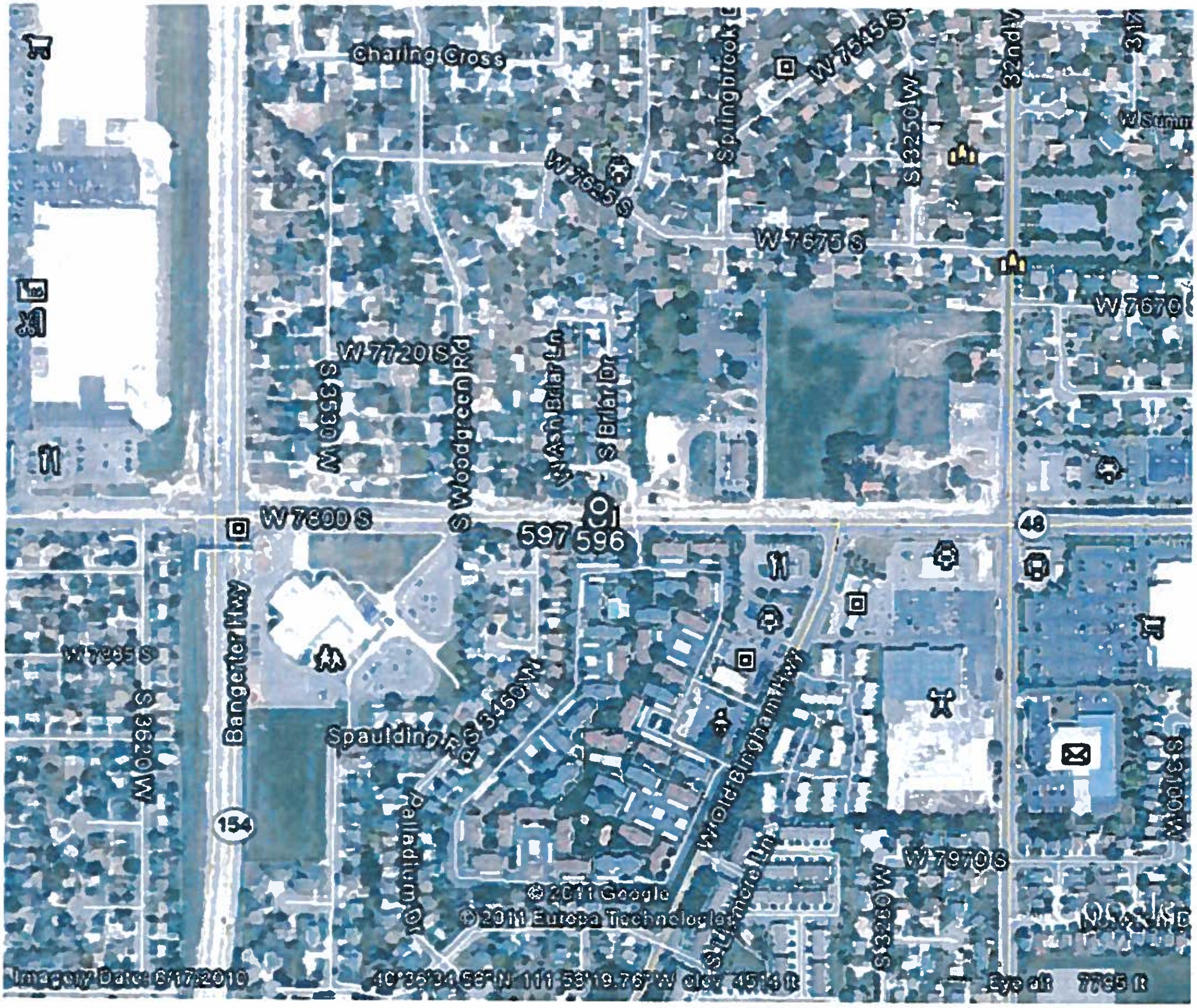
UTAH DEPARTMENT OF TRANSPORTATION
CORE THICKNESS

PROJECT NAME 3E SR 48 2700 W TO BANGERTER "1' PIN NUMBER: CTR

PAVEMENT TYPE HMA DATE 11/9/2011

CORE/ GPS Number	RELATION TO ROAD (ex:NB,SB..) (OSL,ISL,Shoulder)	GPS COORDINATES	HOLE THICK-NESS (in)	CORE CONDITION			COMMENTS
				TOP	MIDDLE	BOTTOM	
596	OSSWB	N40 36.576 W111 58.329	4 3/4				6 IN CORE
597	OSSWB	N40 36.569 W111 58.330	4 7/8				6 IN CORE

H:\Core Log Data\Maps\GE SR 48 2700 W TO BANGERTER "11.pdf


Charing Cross
W 7645 S
Springbrook
32nd W
W 7635 S
S 3250 W
W 7675 S
W 7670 S
W 7720 S
S Woodgreen Rd
W Ash Briar Ln
S Briar Dr
S 3530 W
W 7800 S
597 596
48
Bangerter Hwy
W 7985 S
S 3620 W
S 3450 W
Spaulding Rd
W Old Bingham Hwy
154
Palladium Dr
W 7970 S
S 3250 W
S 3100 W
© 2011 Google
© 2011 Europa Technologies
Sycamore Ln
Imagery Date: 8/17/2010
40°36'34.56" N 111°58'19.76" W elev 4514 ft
Eye alt 7765 ft

UTAH DEPARTMENT OF TRANSPORTATION
CORE THICKNESS

PROJECT NAME: SR 68 BANGERTER TO SARATOGA '   PIN NUMBER: CTR

PAVEMENT TYPE: HMA   DATE: 11/9/2011

CORE/ GPS Number	RELATION TO ROAD (ex:NB,SB..) (OSL,ISL,Shoulder)	GPS COORDINATES	HOLE THICK-NESS (in)	CORE CONDITION			COMMENTS
				TOP	MIDDLE	BOTTOM	
594	OSSSB	N40 29.814 W111 56.329	7 1/4				6 IN CORE
595	OSSSB	N40 29.806 W111 56.326	6 7/8				6 IN CORE

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154
68
W 13600 S
S 1735 W
S 1950 W
W 13930 S
S 1835 W
S 1700 W
W 13970 S
594
595
W 14020 S
S 1850 W
S 1800 W
W 14165 S
© 2011 Google
W 14260 S
Google
Imagery Date: 6/17/2010
Eye alt 7737 ft

UTAH DEPARTMENT OF TRANSPORTATION
CORE THICKNESS

PROJECT NAME SR 71 11400 S TO 12300 S ON 700 E　　PIN NUMBER: CTR

PAVEMENT TYPE HMA　　DATE 11/9/2011

CORE/ GPS Number	RELATION TO ROAD (ex:NB,SB..) (OSL,ISL,Shoulder)	GPS COORDINATES	HOLE THICK-NESS (in)	CORE CONDITION			COMMENTS
				TOP	MIDDLE	BOTTOM	
592	OSSHOULDER	N40 32.414 W111 52.293	7 1/8				6 IN CORE
593	OSSHOULDER	N40 32.414 W111 52.292	6 7/8				6 IN CORE

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Normandy Loop Ln
E 11620 S
Normandy Loop Ln
Wyngate Pointe Ln
593
592
71
S Auburn Fields Way
S Shadow View Ln
Porter Rockwell Trail (Sandy Rail Trail)
Auburn Fields Way
© 2011 Google
E Harvest Bend Way
Google
Imagery Date: 6/17/2010
Eye alt 5730 ft

UTAH DEPARTMENT OF TRANSPORTATION
CORE THICKNESS

PROJECT NAME GE SR 266 4500S 700 E TO I-215"11 PIN NUMBER: CTR

PAVEMENT TYPE HMA DATE 11/9/2011

CORE/ GPS Number	RELATION TO ROAD (ex:NB,SB..) (OSL,ISL,Shoulder)	GPS COORDINATES	HOLE THICK-NESS (in)	CORE CONDITION			COMMENTS
				TOP	MIDDLE	BOTTOM	
590	OSLWB	N40 40.464 W111 52.052	7 7/8				6 IN CORE
591	OSLWB	N40 40.462 W111 52.046	7 3/4				6 IN CORE

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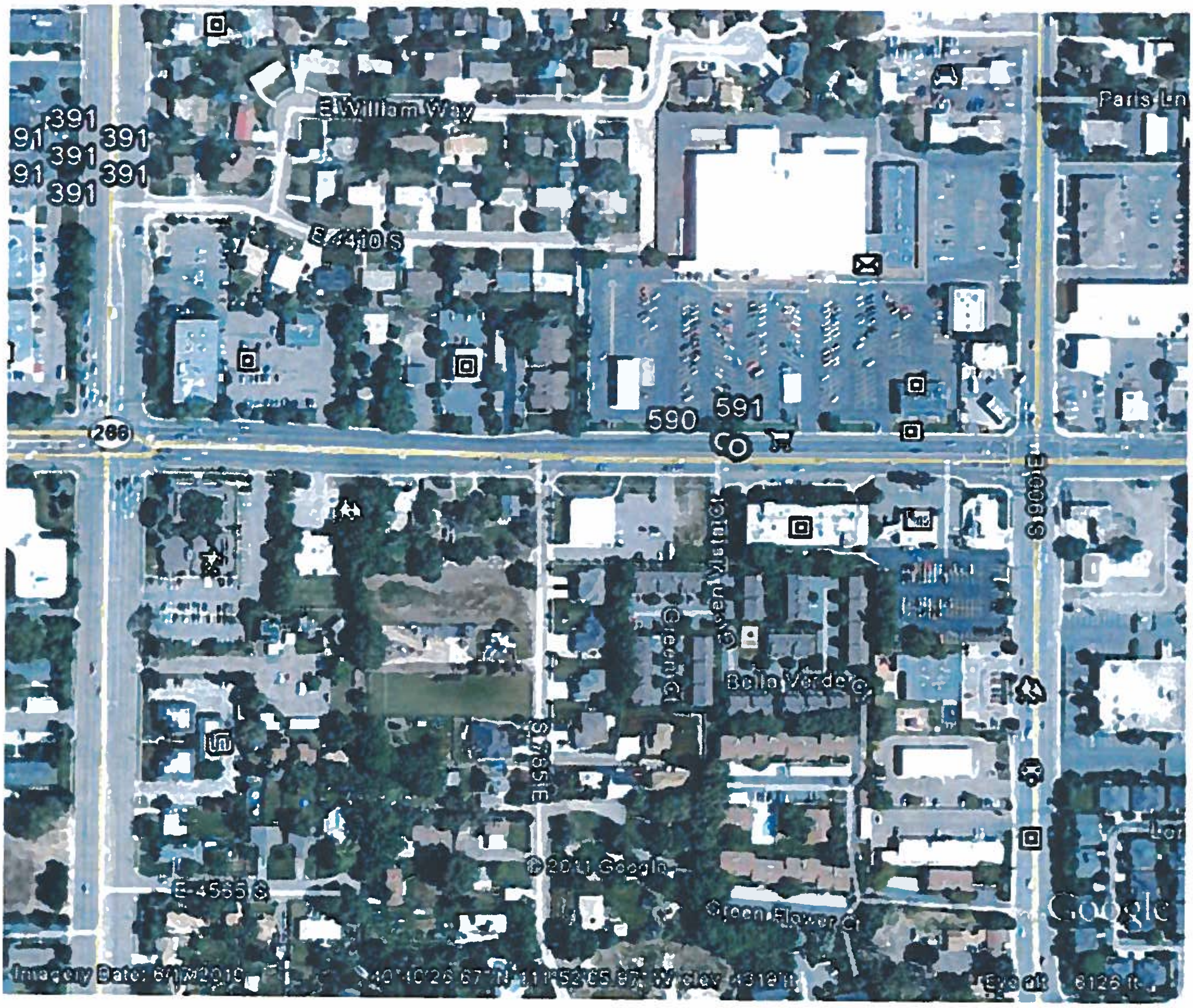
E William Way
Paris Ln
E 4410 S
266
590
591
S 900 E
Green Vista Ct
Green Ct
Bella Verde Ct
S 765 E
© 2011 Google
E 4555 S
Green Flower Ct
Google

UTAH DEPARTMENT OF TRANSPORTATION
CORE THICKNESS

PROJECT NAME SR 269 500S TO 600 S WESTTEMPLE   PIN NUMBER: CTR

PAVEMENT TYPE HMA   DATE 11/9/2011

CORE/ GPS Number	RELATION TO ROAD (ex:NB,SB..) (OSL,ISL,Shoulder)	GPS COORDINATES	HOLE THICK-NESS (in)	CORE CONDITION			COMMENTS
				TOP	MIDDLE	BOTTOM	
586	OSLNB	N40 45.422 W111 53.636	6 1/2	4 1/4	1		6 IN CORES
587	OSLNB	N40 45.422 W111 53.632	7 1/4				6 IN CORES

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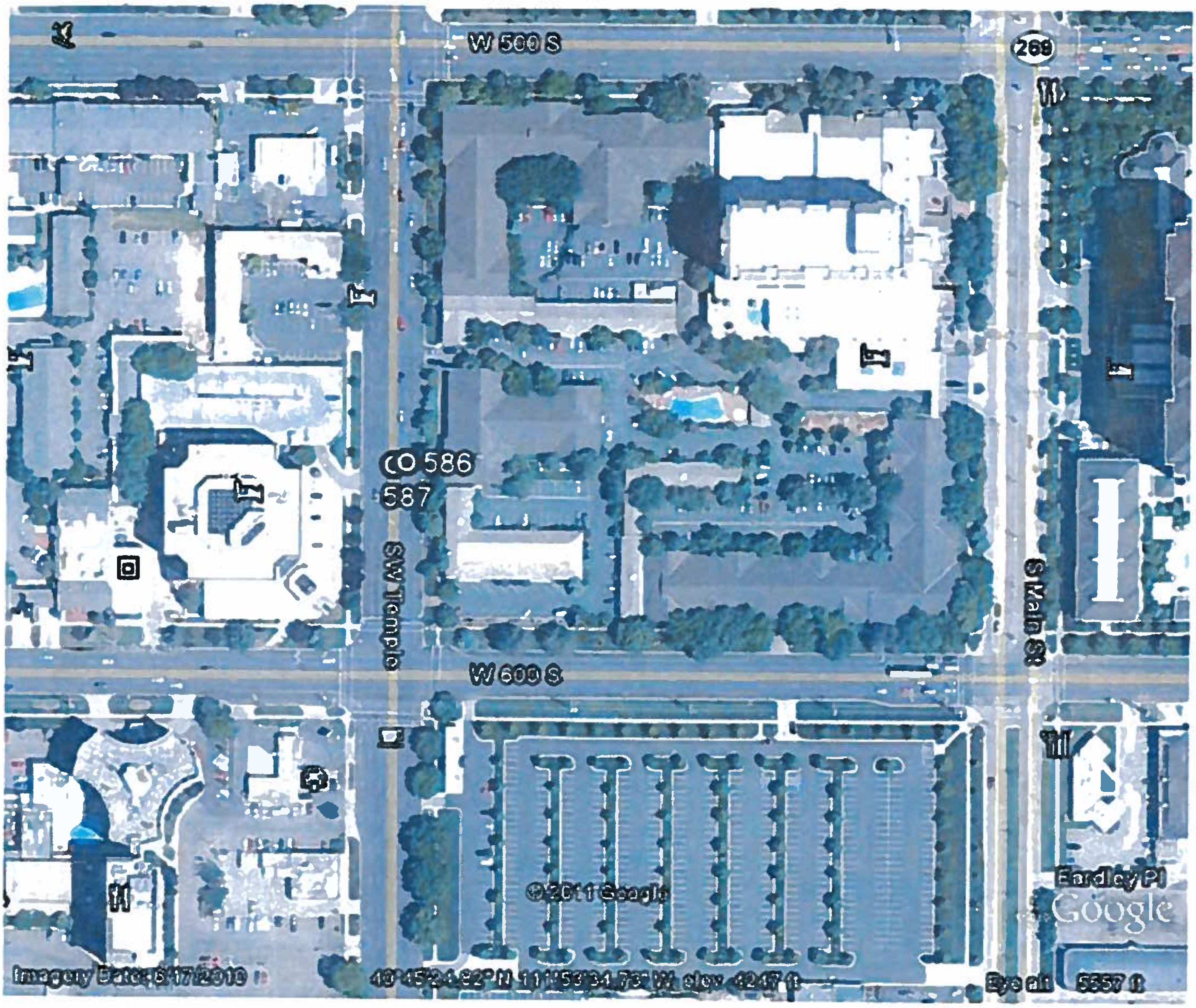
W 500 S
269
CO 586
587
SW Temple
S Main St
W 600 S
Eardley Pl
Google
© 2011 Google
Imagery Date: 6/17/2010
Eye alt 5557 ft

UTAH DEPARTMENT OF TRANSPORTATION
CORE THICKNESS

PROJECT NAME GE SR U-111 5600 S TO 201 "11 **PIN NUMBER:** CTR

PAVEMENT TYPE HMA **DATE** 11/3/2011

CORE/ GPS Number	RELATION TO ROAD (ex.NB,SB..) (OSL,ISL,Shoulder)	GPS COORDINATES	HOLE THICK-NESS (in)	CORE CONDITION			COMMENTS
				TOP	MIDDLE	BOTTOM	
580	OSLNB	N40 41.831 W112 05.500	10	4 1/4	5 5/8		6 IN CORES
581	OSLNB	N40 41.832 W112 05.502	10 1/2				6 IN CORES
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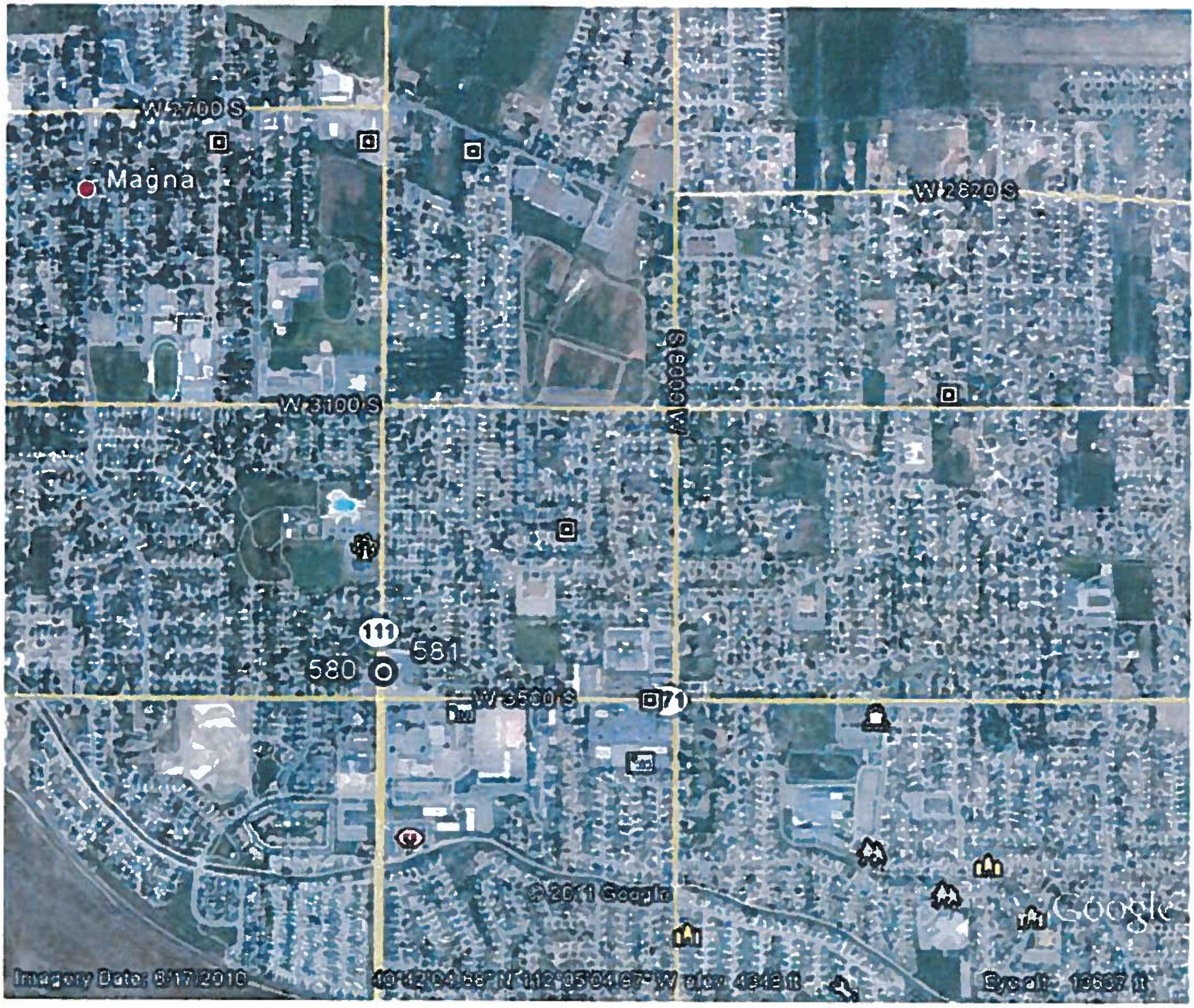
W 2700 S
Magna
W 2820 S
W 3100 S
8000 S
111
581
580
W 3500 S
71
© 2011 Google
Google
Imagery Date: 8/17/2010

UTAH DEPARTMENT OF TRANSPORTATION
CORE THICKNESS

PROJECT NAME GE SR 171 REDWOOD TO 5600 W "11 **PIN NUMBER:** CTR

PAVEMENT TYPE HMA **DATE** 11/3/2011

CORE/ GPS Number	RELATION TO ROAD (ex:NB,SB..) (OSL,ISL,Shoulder)	GPS COORDINATES	HOLE THICK-NESS (in)	CORE CONDITION			COMMENTS
				TOP	MIDDLE	BOTTOM	
576	OSL WB	N40 41.802 W111 56.505	9				6 IN CORES
577	OSL WB	N40 41.803 W111 56.511	11				6 IN CORES
578	OSL WB	N40 41.803 W111 59.047	9	6	3		6 IN CORES
579	OSL WB	N40 41.802 W111 59.044	8 1/2	4 1/2	3		6 IN CORES

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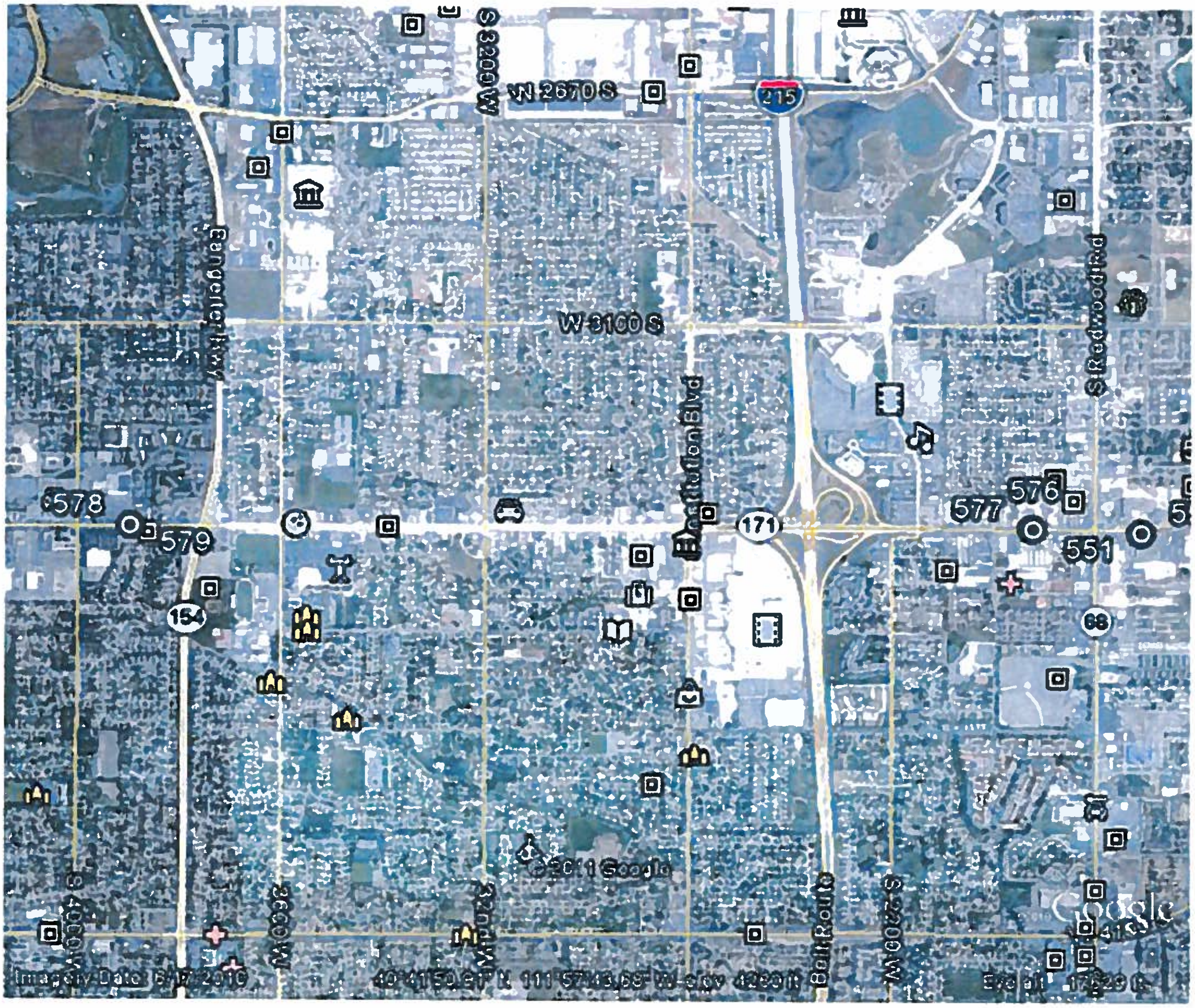
W 2670 S
S 3200 W
215
Bangerter Hwy
W 3100 S
Constitution Blvd
S Redwood Rd
578
579
171
577
576
551
154
68
S 2200 W
Belt Route
S 4000 W
S 3600 W
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APPENDIX II MIX DESIGNS FOR PAVEMENT SECTIONS

State of Utah

GARY R. HERBERT
Governor

GREG BELL
Lieutenant Governor

DEPARTMENT OF TRANSPORTATION

JOHN R. NJORD, P.E.
Executive Director

CARLOS M. BRACERAS, P.E.
Deputy Director

Memorandum

Jun 28, 2010 (Aug 28, 2009)

TO: Todd Gunderman, P.E.
Resident Engineer

CID#5103303C

Lonnie Marchant

FROM: Lonnie Marchant, P.E.
Region Two Materials Engineer

SUBJECT: Superpave Level I Mix Design Review Report (1/2" HMA Volumetric Mix Design)
Project No.: **STP-0048(18)8**
Project Name: **SR-48; 4800 West to 2700 West**
Contractor: CONDIE CONSTRUCTION COMPANY, INC.
Design Verification

The contractor has indicated that their source of aggregate, Size **1/2"** will be from **Hansen Pit**. The contractor has further indicated that **Peak Asphalt** will supply a **PG64-34** asphalt cement binder. The asphalt concrete pavement mix will be produced at the **Geneva Rock Point of Mnt Plant**. The following recommendations are based on tests of the aggregate and bituminous mix.

The optimum AC recommendation is based on the Gyratory Mix Design Method. **Ninitial = 8**, **Ndesign = 100** and **Nmax = 160**. The gyratory specimen compaction temperature is **284-302 °F**. The combined specific gravity(Gsb) of the aggregates is 2.592.

Asphalt Cement Grade: **PG64-34**
Percentage Asphalt Cement: **5.2%**
Mixing Temperatures: **320-325 °F**
Minimum Compaction Temperature: **302 °F**

Stockpile Blends:

Source	Description	Percent	Gsb
Hansen Pit	CHPA	32%	2.615
Hansen Pit	CHPE	17%	2.548
Hansen Pit	FINE	26%	2.592
Hansen Pit	ROCD	10%	2.625
Hansen Pit	Sand	14%	2.579

CONTRACTOR'S DESIGN RESULTS:

Hydrated Lime **1%** (Dry Wt. Agg.)

VMA: **14.5%**

Combined Gsb: **2.592**

Max. Specific Gravity(Rice): **2.421**

Voids at Ndesign: **3.6%**

Hamburg Results: **PASSED** (**Left 1.84, Right 4.18, Average 3.01**)

Burn-off Correction Factor:

Approximate Gyratory Weight: **4700 g**

Job Mix Gradation

Sieve	% Passing	Spec. % Passing
1.5 inch		
1 inch	100	
3/4 inch	100	100.0
1/2 inch	91	90.0 - 100.0
3/8 inch	81	<90
1/4 inch		
No. 4	57	
No. 8	38	28.0 - 58.0
No. 16	25	
No. 50	13	
No. 200	6.4	2.0 - 10.0

Contractor's Super pave Mix Design Was: (See Checked Below)

Approved As Submitted	Approved With Conditions	Not Approved for Following Reasons
X		

Copy of Region Two Mix Design 09-2H-055; 10400 South Bangerter Hwy to Redwood Road

2010 South 2760 West, Salt Lake City, Utah 84104
telephone 801-975-4926 * facsimile 801-975-4933 * lmarchant@utah.gov

State of Utah

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Governor

GARY R. HERBERT
Lieutenant Governor

DEPARTMENT OF TRANSPORTATION

JOHN R. NJORD, P.E.
Executive Director

CARLOS M. BRACERAS, P.E.
Deputy Director

Memorandum

Apr 30, 2009

TO: Josh Vanjura, P.E.
Resident Engineer

CID#5270103C

FROM: John Butterfield, P.E.
Region Two Materials Engineer

SUBJECT: Superpave Level I Mix Design Review Report (3/4" HMA Volumetric Mix Design)
Project No.: **F-0068(45)33**
Project Name: **SR-68; Bangerter Highway through Saratoga Springs**
Contractor: GENEVA ROCK PRODUCTS, INC.
Design Verification

The contractor has indicated that their source of aggregate, Size **3/4"** w/RAP will be from **Hansen Pit**. The contractor has further indicated that **Peak Asphalt** will supply a **PG64-28** asphalt cement binder. The asphalt concrete pavement mix will be produced at the **Geneva Rock Point of Mnt Plant**. The following recommendations are based on tests of the aggregate and bituminous mix.

The optimum AC recommendation is based on the Gyratory Mix Design Method. **Ninitial = 9**, **Ndesign = 125** and **Nmax = 205**. The gyratory specimen compaction temperature is **279-297 °F**. The combined specific gravity(Gsb) of the aggregates is 2.612.

Asphalt Cement Grade: **PG64-28**
Percentage Asphalt Cement: **4.6%**
Mixing Temperatures: **320-325 °F**
Minimum Compaction Temperature: **295 °F**

Stockpile Blends:

Source	Description	Percent	Gsb
Hansen Pit	CHIP	9%	2.61
Hansen Pit	CHPA	19%	2.615
Hansen Pit	CHPE	8%	2.609
Hansen Pit	FINE	15%	2.595
Hansen Pit	RAP	15%	2.623
Hansen Pit	ROCD	24%	2.618
Hansen Pit	Sand	9%	2.618

CONTRACTOR'S DESIGN RESULTS:

Hydrated Lime **1%** (Dry Wt. Agg.)

VMA: **13.6%**

Combined Gsb: **2.612**

Max. Specific Gravity(Rice): **2.454**

Voids at Ndesign: **3.6%**

Hamburg Results: **PASSED** (**Left 3.34, Right 2.97, Average 3.16**)

Burn-off Correction Factor:

Approximate Gyratory Weight: **4700 g**

Job Mix Gradation

Sieve	% Passing	Spec. % Passing
1.5 inch		
1 inch	100	100.0
3/4 inch	100	90.0 - 100.0
1/2 inch	87	<90
3/8 inch	75	
1/4 inch		
No. 4	51	
No. 8	33	23.0 - 49.0
No. 16	22	
No. 50	12	
No. 200	5.8	2.0 - 8.0

Contractor's Super pave Mix Design Was: (See Checked Below)

Approved As Submitted	Approved With Conditions	Not Approved for Following Reasons
X		

2010 South 2760 West, Salt Lake City, Utah 84104
telephone 801-975-4926 * facsimile 801-975-4933 * lmarchant@utah.gov

State of Utah

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Governor

GREG BELL
Lieutenant Governor

DEPARTMENT OF TRANSPORTATION

JOHN R. NJORD, P.E.
Executive Director

CARLOS M. BRACERAS, P.E.
Deputy Director

Memorandum

Apr 07, 2010

TO: John Montoya, P.E.
Resident Engineer

CID#5242603C

FROM: Joe Kammerer, P.E.
Region Two Materials Engineer

SUBJECT: Superpave Level I Mix Design Review Report (3/4" HMA Volumetric Mix Design)
Project No.: **F-0071(23)9**
Project Name: **700 East; 11400 S. to Carnation & 12300 S. Intersection**
Contractor: HARPER CONTRACTING INC
Design Verification

The contractor has indicated that their source of aggregate, Size **3/4"** w/RAP will be from **Harper Pit 16**. The contractor has further indicated that **Ergon** will supply a **PG64-28** asphalt cement binder. The asphalt concrete pavement mix will be produced at the **Kilgore Plant**. The following recommendations are based on tests of the aggregate and bituminous mix.

The optimum AC recommendation is based on the Gyratory Mix Design Method. **Ninitial = 8**, **Ndesign = 100** and **Nmax = 160**. The gyratory specimen compaction temperature is **295-305 °F**. The combined specific gravity(Gsb) of the aggregates is 2.657.

Asphalt Cement Grade: **PG64-28**
Percentage Asphalt Cement: **4.4%**
Mixing Temperatures: **301-309 °F**
Minimum Compaction Temperature: **295 °F**

Stockpile Blends:

Source	Description	Percent	Gsb
Harper Pit 16	1/8" Unwshd Sand	23%	2.64
Harper Pit 16	1/4" Unwshd Chip	12%	2.652
Harper Pit 16	1/2" Unwshd Chip	18%	2.68
Harper Pit 16	3/4" Unwshd Chip	20%	2.652
Harper Pit 32	Wshd Conc. Sand	11%	2.641
Harper Pit 10	1/2" Crushed RAP	15%	2.659

CONTRACTOR'S DESIGN RESULTS:

Hydrated Lime **1%** (Dry Wt. Agg.)

VMA: **13.7%**

Combined Gsb: **2.657**

Max. Specific Gravity(Rice): **2.484**

Voids at Ndesign: **3.4%**

Hamburg Results: **PASSED** (**Left 3.78, Right 4.96, Average 4.37**)

Burn-off Correction Factor:

Approximate Gyratory Weight: **4850 g**

Job Mix Gradation

Sieve	% Passing	Spec. % Passing
1.5 inch		
1 inch	100	100.0
3/4 inch	100	90.0 - 100.0
1/2 inch	87	<90
3/8 inch	77	
1/4 inch		
No. 4	56	
No. 8	36	23.0 - 49.0
No. 16	25	
No. 50	12	
No. 200	5.4	2.0 - 8.0

Contractor's Super pave Mix Design Was: (See Checked Below)

Approved As Submitted	Approved With Conditions	Not Approved for Following Reasons
X		

2010 South 2760 West, Salt Lake City, Utah 84104
telephone 801-975-4926 * facsimile 801-975-4933 * lmarchant@utah.gov

Memorandum

UTAH DEPARTMENT OF TRANSPORTATION

DATE: May 8, 2007

TO: Dallas Linford P.E.
Project Engineer

FROM: John C Butterfield P.E.
Region Materials Engineer

SUBJECT: Superpave Level I Mix Design Review Report
Project No.: **NH-0266(4)4**
Project Name: **SR266; 4500 South 700 East to I-215(East)**
Contractor: **Geneva Rock Products**
Supplier: **Geneva Rock Products**

The contractor has indicated that their source of aggregate, **Size ¾"** will be **Hansen Pit** and the brand of PG asphalt cement will be**. Peak PG 64-28.** The asphalt concrete pavement mix will be produced at the **Point of the Mountain and Orem plant.** The following recommendations are based on tests of the aggregate and bituminous mix.

The optimum AC recommendation is based on the Gyratory Mix Design Method**. N initial = 8, N design =100, and N max =160** The field specimen compaction temperature is **290° F TO 300° F** and the combined specific gravity (Gsb) of aggregates is **2.597.** It is recommended that field Gyratory tests be done at production start, upon any mix adjustment, and at least once a week.

Asphalt Cement Grade**: Peak PG 64-28**
Percentage Asphalt Cement**: 4.8%**
Mixing Temperatures:
Minimum 146°C (195°F)
Maximum 163°C (325°F)
Minimum Compaction Temperature 110°C (230°F)

Stockpile Blends:

22 %	CHPA
14 %	CHPE
26 %	FINE
26 %	ROCD
11 %	SAND

CONTRACTOR'S DESIGN RESULTS:

Hydrated Lime **1.0%** (Dry Wt. Agg.):

VMA: **13.9**

Combined Gsb: **2.597**

Max. Specific Gravity (Rice): **2.436**

Voids at Ndesign: **3.6%**

Pavement Analyzer Results: **PASS**

Approximate Gyratory Weight: **4700g**

Job Mix Gradation

American Sieve	Metric Sieve	% Passing	Spec. % Passing
1"	25.4 mm	**100**	**100**
3/4"	19 mm	**100**	**90-100**
1/2"	12.5 mm	**87**	**<90**
3/8"	9.5 mm	**76**	
#4	4.75 mm	**49**	
#8	2.36 mm	**35**	**23-49**
#16	1.18 mm	**23**	
#30	600 um		
#50	300 um	**14**	
#100	150 um		
#200	75 um	**6.0**	**2-8**

Contractor's Super pave Mix Design Was: (See Box Checked Below)

Verified As Submitted **Verified With Conditions** **Not Verified for Following Reasons**
X (Verified With Conditions)

Comments/Conditions/Reasons: **CID# 5137103C**

(Contractor must demonstrate ability to meet design values and coating requirements in the field.)

Mix Design results can be seen at ..\..\Mix Designs\07-2H-019;SR266 4500 South 700 East to I-215 (East).xls

Aggregate Blend Physical Properties

Test Method	Results	UDOT Specification
Soundness (Coarse) ASTM C-88	0.40	16% Max
Soundness (Fine) ASTM C-88	1.25	16% Max
Fracture Face Count-One Face	100.0	95% Min
Fracture Face Count - Face's	98.4	90% Min
Los Angeles Wear AASHTO T-96	17.6	35% Max
Sand Equivalent AASHTO T-176	70.00	60% Min
Uncompacted Voids AASHTO T-304	47.20	45% Min
Flat and Elongated (1:3) ASTM D-4791	2.10	20% Max
Dust Ratio SP-2	1.3	06-1.4
Plastic Index ASTM D-4318	0.0	0% Max

State of Utah

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Lieutenant Governor

DEPARTMENT OF TRANSPORTATION

JOHN R. NJORD, P.E.
Executive Director

CARLOS M. BRACERAS, P.E.
Deputy Director

Memorandum

Jun 08, 2009 (May 19, 2009)

TO: Lee Nitchman, P.E.
Resident Engineer

CID#5290303C

FROM: Joe Kammerer, P.E.
Region Two Materials Engineer

SUBJECT: Superpave Level I Mix Design Review Report (3/4" HMA Volumetric Mix Design)
Project No.: **F-R299(93)**
Project Name: **SR-269 AND SR-270; VARIOUS LOCATIONS**
Contractor: STAKER & PARSON COS DBA STAKER PAVING &
Design Verification

The contractor has indicated that their source of aggregate, Size **3/4"** w/RAP will be from **Staker Parson Beck Street Plant**. The contractor has further indicated that **Peak Asphalt** will supply a **PG64-28** asphalt cement binder. The asphalt concrete pavement mix will be produced at the **Staker Parson Beck Street Plant**. The following recommendations are based on tests of the aggregate and bituminous mix.

The optimum AC recommendation is based on the Gyratory Mix Design Method. **Ninitial = 8**, **Ndesign = 100** and **Nmax = 160**. The gyratory specimen compaction temperature is **279-297 °F**. The combined specific gravity(Gsb) of the aggregates is 2.667.

Asphalt Cement Grade: **PG64-28**
Percentage Asphalt Cement: **4.5%**
Mixing Temperatures: **320-325 °F**
Minimum Compaction Temperature: **295 °F**

Stockpile Blends:

Source	Description	Percent	Gsb
Beck Street Pit	3/4" Agg.	15%	2.69
Beck Street Pit	1/2" Agg.	25%	2.707
Beck Street Pit	Pep Fines	22%	2.68
Brigham Pit	Brigham Fines	14%	2.672
Staker Parson Point East	P.E. Squeegee	8%	2.528
Beck Street Pit	RAP	15%	2.659

CONTRACTOR'S DESIGN RESULTS:

Hydrated Lime **1%** (Dry Wt. Agg.)

VMA: **13.5%**

Combined Gsb: **2.667**

Max. Specific Gravity(Rice): **2.504**

Voids at Ndesign: **3.5%**

Hamburg Results: **PASSED** (**Left 4.23, Right , Average**)

Burn-off Correction Factor:

Approximate Gyratory Weight: **4800 g**

Job Mix Gradation

Sieve	% Passing	Spec. % Passing
1.5 inch		
1 inch	100	100.0
3/4 inch	100	90.0 - 100.0
1/2 inch	88	<90
3/8 inch	73	
1/4 inch	55	
No. 4	47	
No. 8	29	23.0 - 49.0
No. 16	19	
No. 50	11	
No. 200	6	2.0 - 8.0

Contractor's Super pave Mix Design Was: (See Checked Below)

Approved As Submitted	Approved With Conditions	Not Approved for Following Reasons
X		

Copy of Region Two mix Design 09-2H-021; SR-201 - Bangerter Hwy to 5600 West

2010 South 2760 West, Salt Lake City, Utah 84104
telephone 801-975-4926 * facsimile 801-975-4933 * lmarchant@utah.gov

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DEPARTMENT OF TRANSPORTATION

JOHN R. NJORD, P.E.
Executive Director

CARLOS M. BRACERAS, P.E.
Deputy Director

Memorandum

Jun 24, 2009

TO: John Montoya, P.E.
Resident Engineer

CID#5289203C

FROM: Joe Kammerer, P.E.
Region Two Materials Engineer

SUBJECT: Superpave Level I Mix Design Review Report (3/4" HMA Volumetric Mix Design)
Project No.: **F-0111(15)9**
Project Name: **SR-111 (8400 WEST); 3500 SOUTH TO SR-201**
Contractor: KILGORE PAVEMENT MAINTENANCE LLC
Design Verification

The contractor has indicated that their source of aggregate, Size **3/4"** w/RAP will be from **Harper Pit 16**. The contractor has further indicated that **Paramount Fernley** will supply a **PG64-28** asphalt cement binder. The asphalt concrete pavement mix will be produced at the **Kilgore Plant**. The following recommendations are based on tests of the aggregate and bituminous mix.

The optimum AC recommendation is based on the Gyratory Mix Design Method. **Ninitial = 8**, **Ndesign = 100** and **Nmax = 160**. The gyratory specimen compaction temperature is **266-294 °F**. The combined specific gravity(Gsb) of the aggregates is 2.664.

Asphalt Cement Grade: **PG64-28**
Percentage Asphalt Cement: **4.4%**
Mixing Temperatures: **320-325 °F**
Minimum Compaction Temperature: **290 °F**

Stockpile Blends:

Source	Description	Percent
Harper Pit 16	1/8" Crushed Fines	21%
Harper Pit 16	1/8" Crushed Fines	19%
Harper Pit 16	1/2" Unwshd Rock	12%
Harper Pit 16	3/4" Unwshd Rock	21%
Harper Pit 34	Wshd Conc. Sand	11%
Harper Pit 10	1/2" Crushed RAP	15%

CONTRACTOR'S DESIGN RESULTS:

Hydrated Lime **1%** (Dry Wt. Agg.)

VMA: **13.9%**

Combined Gsb: **2.664**

Max. Specific Gravity(Rice): **2.486**

Voids at Ndesign: **3.5%**

Hamburg Results: **PASSED** (**Left 3.65, Right 3.55, Average 3.6**)

Burn-off Correction Factor:

Approximate Gyratory Weight: **4800 g**

Job Mix Gradation

Sieve	% Passing	Spec. % Passing
1.5 inch		
1 inch	100	100.0
3/4 inch	100	90.0 - 100.0
1/2 inch	87	<90
3/8 inch	78	
1/4 inch		
No. 4	55	
No. 8	33	23.0 - 49.0
No. 16	23	
No. 50	12	
No. 200	5.5	2.0 - 8.0

Contractor's Super pave Mix Design Was: (See Checked Below)

Approved As Submitted	Approved With Conditions	Not Approved for Following Reasons
X		

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State of Utah

GARY R. HERBERT
Governor

GREG BELL
Lieutenant Governor

DEPARTMENT OF TRANSPORTATION

JOHN R. NJORD, P.E.
Executive Director

CARLOS M. BRACERAS, P.E.
Deputy Director

Memorandum

Jun 30, 2011 (Apr 07, 2010)

TO: Russ Brown, P.E.
Resident Engineer

CID#5329003C

Lonnie Marchant

FROM: Lonnie Marchant, P.E.
Region Two Materials Engineer

SUBJECT: Superpave Level I Mix Design Review Report (3/4" HMA Volumetric Mix Design)
Project No.: **F-0171(32)4**
Project Name: **SR-171, 3500 SOUTH; 5600 WEST TO BANGERTER**
Contractor: KILGORE COMPANIES, A DELAWARE LLC DBA KI
Design Verification

The contractor has indicated that their source of aggregate, Size **3/4"** w/RAP will be from **Harper Pit 16**. The contractor has further indicated that **Ergon** will supply a PG64-28 asphalt cement binder. The asphalt concrete pavement mix will be produced at the **Kilgore Plant**. The following recommendations are based on tests of the aggregate and bituminous mix.

The optimum AC recommendation is based on the Gyratory Mix Design Method. **Ninitial = 8, Ndesign = 100 and Nmax = 160.** The gyratory specimen compaction temperature is **295-305 °F**. The combined specific gravity(Gsb) of the aggregates is 2.657.

Asphalt Cement Grade: **PG64-28**
Percentage Asphalt Cement: **4.4%**
Mixing Temperatures: **301-309 °F**
Minimum Compaction Temperature: **305 °F**

Stockpile Blends:

Source	Description	Percent	Gsb
Harper Pit 16	1/8" Unwshd Sand	23%	2.64
Harper Pit 16	1/4" Unwshd Chip	12%	2.652
Harper Pit 16	1/2" Unwshd Chip	18%	2.68
Harper Pit 16	3/4" Unwshd Chip	20%	2.652
Harper Pit 32	Wshd Conc. Sand	11%	2.641
Harper Pit 10	1/2" Crushed RAP	15%	2.659

CONTRACTOR'S DESIGN RESULTS:
Hydrated Lime **1%** (Dry Wt. Agg.)

VMA: **13.7%**

Combined Gsb: **2.657**

Max. Specific Gravity(Rice): **2.484**

Voids at Ndesign: **3.4%**

Hamburg Results: **PASSED** (**Left 3.78, Right 4.96, Average 4.37**)

Burn-off Correction Factor:

Approximate Gyratory Weight: **4850 g**

Job Mix Gradation

Sieve	% Passing	Spec. % Passing
1.5 inch		
1 inch	100	100.0
3/4 inch	100	90.0 - 100.0
1/2 inch	87	<90
3/8 inch	77	
1/4 inch		
No. 4	56	
No. 8	36	23.0 - 49.0
No. 16	25	
No. 50	12	
No. 200	5.4	2.0 - 8.0

Contractor's Super pave Mix Design Was: (See Checked Below)

Approved As Submitted	Approved With Conditions	Not Approved for Following Reasons
X		

Copy of Region Two mix design 10-2H-002; 700 East; 11400 South to Carnation 123000 South Int

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