Final Report

EFFECTS OF LABORATORY HEATING, CYCLIC PORE PRESSURE, AND CYCLIC LOADING ON FRACTURE PROPERTIES OF ASPHALT MIXTURE

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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 yards	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	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	l
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
psi	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	35.71	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	megagrams	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°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	psi

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

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16. Abstract

This study involved the identification and evaluation of laboratory conditioning methods and testing protocols considering heat oxidation, moisture, and load that more effectively simulate asphalt mixture aging in the field, and thereby help to properly assess asphalt mixture property changes over time. In this study, aging was defined as any detrimental effect on asphalt mixture properties during pavement life. Three laboratory conditioning procedures were identified and further developed to evaluate the effects of heat oxidation, moisture, and repeated load: heat oxidation conditioning (HOC), cyclic pore pressure conditioning (CPPC), and repeated load conditioning (RLC) respectively. Results indicated that the most effective approach to effectively induce changes in damage and fracture-related properties involved a combination of HOC and CPPC. HOC was accomplished using the Superpave Long-Term Oven Aging (LTOA) procedure. The combination of LTOA and CPPC was found to effectively induce reduction in failure strain and fracture energy (FE) limit of mixtures to levels consistent with those observed in the field. Although RLC was able to effectively induce damage, the method was found to be impractical because of issues regarding the identification of proper load level to achieve damage without fracture during RLC. The energy ratio (ER) parameter was used for relative comparison of cracking performance of mixtures subjected to similar levels of HOC and CPPC. Results showed that addition of hydrated lime improved cracking performance of granite mixtures, which exhibited higher ER than the limestone mixture tested. In addition, the limestone mixture tested exhibited better cracking performance (higher ER) than the granite mixture without lime. Recommendations were made for additional field and laboratory studies using the conditioning procedures developed in this study.

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EXECUTIVE SUMMARY

This study involved the identification and evaluation of laboratory conditioning methods and testing protocols considering heat oxidation, moisture, and load that more effectively simulate asphalt mixture aging in the field and thereby help to properly assess asphalt mixture property changes over time. In this study, aging was defined as any detrimental effect on asphalt mixture properties during pavement life. Three laboratory conditioning procedures were identified and further developed to evaluate the effects of heat oxidation, moisture, and repeated load: heat oxidation conditioning (HOC), cyclic pore pressure conditioning (CPPC), and repeated load conditioning (RLC) respectively.

Researchers have determined that the most effective approach to induce changes in damage and fracture-related properties involved a combination of HOC and CPPC. HOC was accomplished using the Superpave Long-Term Oven Aging (LTOA) procedure. The combination of HOC and CPPC was found to effectively induce reduction in failure strain and fracture energy (FE) limit of mixtures to levels consistent with those observed in the field. Experimental results indicated that heat oxidation only could not reduce the fracture energy (FE) limit to levels observed in the field, even though it was able to stiffen and embrittle mixtures.

The CPPC includes effects of loading in addition to moisture effects because the CPPC induces internal pressure (stress) in a tensile mode that is similar to the effect of repeated load experienced by mixtures in the field. Furthermore, it was determined that the CPPC was a more suitable way than RLC to induce microdamage in the specimen without fracture. This was probably due to the CPPC inducing a more uniformly distributed stress state than RLC.

Although RLC was able to effectively induce damage, the method was found to be impractical because of issues regarding the identification of proper load level to achieve damage without fracture. In addition, the RLC procedure used in this project exposed further issues concerning its application in terms of controlling testing conditions. Specifically with the CPPC followed by the RLC, it was difficult to maintain constant moisture content during 30-minute-long test without significantly affecting the test results.

The energy ratio (ER) parameter was used for relative comparison of cracking performance of mixtures subjected to similar levels of HOC and CPPC. As expected, results showed that addition of hydrated lime improved cracking performance of granite mixtures, which exhibited higher ER than the limestone mixture tested. However, the beneficial effect of hydrated lime appeared to diminish with the higher level of oxidative aging. In addition, the limestone mixture tested exhibited better cracking performance (higher ER) than the granite mixture without lime. This was consistent with the fact that limestone is highly resistant to moisture damage.

Recommendations were made for further efforts based on extensive evaluations performed in this study. The recommendations highlighted the need to further evaluate the effect of healing properties and different types of mixture in order to improve the conditioning procedures used in the project. In addition, the need to further develop improved mixture property aging models to enhance existing pavement performance prediction models based on additional field and laboratory studies using the conditioning procedures developed in this study was emphasized.

TABLE OF CONTENTS

page

DISCLAIMER ii

SI (MODERN METRIC) CONVERSION FACTORS…………………………………..iii

TECHNICAL REPORT DOCUMENTATION PAGE…………………………………..iv

ACKNOWLEDGMENTS v

EXECUTIVE SUMMARY vi

LIST OF TABLES xi

LIST OF FIGURES xii

CHAPTERS

1 INTRODUCTION 1

1.1 Background 1
1.2 Objectives 2
1.3 Scope 4
1.4 Research Approach 4

2 LITERATURE REVIEW 6

2.1 Introduction 6
2.2 Aging Procedure and Evaluation Method for Asphalt Mixture.............................6
2.3 Aging Effects on Mixture Property Changes...12
2.4 Closure 18

3 IDENTIFICATION OF LABORATORY CONDITIONING AND TESTING PROTOCOLS TO EFFECTIVELY CHARACTERIZE THE AGING IN THE FIELD 19

3.1 Introduction 19
3.2 Heat Oxidation Conditioning (HOC)...21
3.2.1 Introduction 21
3.2.2 Short-Term Oven Aging (STOA)..22
3.2.3 Long-Term Oven Aging (LTOA)...22
3.2.4 Ten-Day-Term Oven Aging (TTOA)..23
3.2.5 Closure 24
3.3 Cyclic Pore Pressure Conditioning (CPPC) 24
3.3.1 Introduction 24
3.3.2 Description of CPPC system 25

3.3.3 CPPC procedure 28
3.3.3.1 Sample Saturation 29
3.3.3.2 Cyclic Pore Pressure..30
3.3.3.3 Additional Conditioning (Wet or Dry)..32
3.3.4 Closure..34
3.4 Repeated Load Conditioning (RLC)..34
3.4.1 Introduction 34
3.4.2 Load Level and Rest Period 35
3.4.3 Modified Brittleness Index (I_{MB})..38
3.4.4 Determination of Load Level 40
3.4.5 RLC Test Procedure 42
3.4.6 Closure on Conditioning Methods..44
3.5 Closure...44

4 MATERIALS AND LABORATORY MIXTURE TESTS 45

4.1 Introduction...45
4.2 Materials 46
4.3 Test Specimen Preparation 49
4.3.1 Batching and Mixing 51
4.3.2 Compaction..52
4.3.3 Measuring Air Voids 53
4.3.4 Cutting 53
4.3.5 Gauge Points Attachment 54
4.4 Superpave IDT Test Procedure...55
4.4.1 Resilient Modulus Test..56
4.4.2 Creep Test...58
4.4.3 Tensile Strength Test 60
4.5 Closure..63

5 EFFECTS OF LABORATORY CONDITIONING ON FRACTURE PROPERTIES OF ASPHALT MIXTURE 64

5.1 Introduction...64
5.2 Issues Associated with RLC Tests...64
5.2.1 Introduction 64
5.2.2 Repeated Load Conditioning after Heat Oxidation 65
5.2.2.1 Load Level Threshold Affected by Heat Oxidation.........................68
5.2.3 Repeated Load Conditioning after Heat Oxidation plus Cyclic Pore Pressure Conditioning 74
5.2.3.1 Loss of Moisture during RLC after CPPC 75
5.2.4 Closure...78
5.3 Effect of Laboratory Conditioning on Change in HMA Fracture Properties 79
5.3.1 Effect of Heat Oxidation Conditioning (HOC Only) 79
5.3.2 Effect of Drying after Cyclic Pore Pressure Conditioning (CPPC + Dry) .84
5.3.3 Effect of Cyclic Pore Pressure Conditioning (CPPC + Wet) 90
5.4 Closure..99

6 CLOSURE 100

6.1 Summary and Findings 100
6.2 Conclusions 103
6.3 Recommendations and Future Work 104

LIST OF REFERENCES 106

APPENDICES

A LABORATORY MIXTURES INFORMATION 111

B MIXTURE TEST RESULTS 114

LIST OF TABLES

Table page

Table 2-1 Summary of asphalt mixture aging tests (Airey et al., 2005) 7

Table 3-1 I_{MB} values of the three mixtures tested (TTOA and 10 ℃) 39

Table 3-2 Loading level range of each group 41

Table 4-1 Batching sheet for granite mixture without and with lime 47

Table 4-2 Batching sheet for limestone mixture 47

Table 4-3 Aggregate sources 48

Table 4-4 Number of specimens for mixture testing 50

Table 5-1 Reduction in moisture content 75

Table A-1 Aggregate sources (Granite) 112

Table A-2 Gradation information and blend proportions (Granite) 112

Table A-3 Aggregate sources (Limestone) 113

Table A-4 Gradation information and blend proportions (Limestone) 113

Table B-1 Superpave IDT test results 115

LIST OF FIGURES

Figure page

Figure 1-1 Research concept 3

Figure 2-1 Change in fracture energy over time 15

Figure 2-2 Change in resilient modulus over time 15

Figure 2-3 Proposed AC stiffness model 16

Figure 2-4 Proposed FE limit model 17

Figure 3-1 Expected trend of asphalt mixture property (fracture energy) with aging 20

Figure 3-2 Loose mixture for STOA 22

Figure 3-3 Specimens setup for LTOA conditioning 23

Figure 3-4 Tabletop triaxial chamber 25

Figure 3-5 Triaxial cell and temperature control units 26

Figure 3-6 Change in fracture energy due to different CPPC temperature 27

Figure 3-7 Change in resilient modulus due to different CPPC temperature 28

Figure 3-8 Overall CPPC process flowchart 29

Figure 3-9 Vacuum saturation chamber 30

Figure 3-10 Deairator 31

Figure 3-11 Applied pressure waveform for the CPPC 32

Figure 3-12 Wet conditioning after the CPPC 33

Figure 3-13 Dry conditioning after the CPPC 33

Figure 3-14 Asphalt mixture fatigue curve due to cyclic loading 35

Figure 3-15 Normalized resilient deformation at different rest periods and constant load 37

Figure 3-16 Normalized resilient deformation with different loading level at 0.9 second rest period 37

Figure 3-17 Relationship between loading level and I_{MB} 42

Figure 3-18 Applied load for RLC test and creep test after RLC test 43

Figure 3-19 Testing protocol to determine fracture properties before and after RLC test 44

Figure 4-1 Overall testing plan 46

Figure 4-2 Asphalt mixture gradations 49

Figure 4-3 Mixer used for mixing...51

Figure 4-4 Superpave Servopac Gyratory Compactor 52

Figure 4-5 Cutting machine used in this research..54

Figure 4-6 Gauge points attachment 55

Figure 4-7 Marking the loading axis..55

Figure 4-8 Haversine load applied and typical deformation response for resilient modulus test 57

Figure 4-9 Power model of creep compliance 59

Figure 4-10 Determination of FE and $DCSE_f$...62

Figure 5-1 RLC test result (TTOA, granite w/o lime) 66

Figure 5-2 Approach used to determine the starting point of the regression line.............67

Figure 5-3 Threshold for determination of the starting point of the linear regression 67

Figure 5-4 Normalized effective modulus reduction for STOA granite w/o lime with 45 % loading level 69

Figure 5-5 Normalized effective modulus reduction for LTOA granite w/o lime with 40 % loading level 69

Figure 5-6 Normalized effective modulus reduction for TTOA granite w/o lime with 35 % loading level 70

Figure 5-7 Sensitivity to loading level for STOA granite w/o lime (AGS).......................71

Figure 5-8 Sensitivity to loading level for LTOA granite w/o lime (AGL) 72

Figure 5-9 Effect of heat oxidation on load level threshold 74

Figure 5-10 Effect of moisture content reduction on RLC test results for STOA condition77

Figure 5-11 Effect of HOC on change in FE 79

Figure 5-12 Effect of HOC on change in creep rate 80
Figure 5-13 Effect of HOC on change in tensile strength 81
Figure 5-14 Effect of HOC on change in failure strain 82
Figure 5-15 Effect of HOC on change in resilient modulus 83
Figure 5-16 Effect of HOC on change in energy ratio..84
Figure 5-17 Effect of drying after CPPC on change in FE limit 86
Figure 5-18 Effect of drying after CPPC on change in creep rate 87
Figure 5-19 Effect of drying after CPPC on change in stiffness 89
Figure 5-20 Effect of CPPC and HOC on change in FE...92
Figure 5-21 Fracture energy reduction by HOC and HOC + CPPC.................................92
Figure 5-22 Effect of CPPC and HOC on change in creep rate..94
Figure 5-23 Effect of CPPC and HOC on change in tensile strength................................96
Figure 5-24 Effect of CPPC and HOC on change in energy ratio 99
Figure 6-1 Recommended conditioning procedure...103

CHAPTER 1
INTRODUCTION

1.1 Background

Asphalt pavement undergoes aging during production, construction, and service life, which negatively affects properties of the asphalt mixture. Appropriate consideration of aging effects in terms of Hot-Mix Asphalt (HMA) property changes is critical to evaluating cracking performance of mixtures and pavement system. However, most research related to aging of asphalt mixtures has focused on the asphalt binder alone. According to SHRP A-003A (Strategic Highway Research Project) aging study (Bell and Sosnovske, 1994), aging of asphalt-aggregate mixes is influenced by both asphalt and aggregate. They also found that aging of certain asphalts is strongly mitigated by some aggregates and less so by others. This indicates that there is a need to evaluate aging by mixture-level assessment.

The conventional Long-Term Oven Aging (LTOA) procedure (AASHTO PP2, 2001) provides a method to simulate oxidative aging that would occur over years of service in situ. However, it is well known that oxidation caused by heating alone (e.g., LTOA) may not simulate all of the effects of long-term aging. Thus, it is possible that many of the detrimental effects involved in long-term aging that actually occur in the field, including effects of moisture and load, may not be simulated by the current LTOA method. In fact, this is in agreement with the findings by Harvey and Tsai (1997) and with recently completed research using the heavy vehicle simulator (HVS) conducted for FDOT (Roque et al., 2007).

There is no generally agreed upon definition for aging. For purposes of this research, aging includes any effects that detrimentally affect properties of mixture, including reduction in fracture energy (FE), reduced resistance to accumulation of damage, reduction in effective stiffness, and reduction in healing potential. Factors that can cause aging as defined in this manner include oxidation, moisture effects, thermal effects, and load effects. There may also be interactions between these effects.

In any case, it is clear that LTOA, which involves only oxidation, does not introduce these other effects. Also, excessive oxidation is not a substitute for moisture and loading effects. To properly evaluate these other effects, it is necessary to stop heating after expected field oxidation levels are reached, then continue aging with moisture conditioning and loading at lower temperature to minimize artificial healing that would occur if these processes were carried out at elevated temperatures.

Therefore, there is strong need to identify and/or enhance appropriate laboratory aging procedures considering heat, moisture, and load to properly assess asphalt mixture property changes over time, which may help to effectively simulate asphalt mixture aging that actually occur in the field and to identify approaches for use in full-scale pavements in APT (Accelerated Pavement Testing). This can also provide the ability to appropriately evaluate the effect of aging on fracture properties of asphalt mixtures.

1.2 Objectives

The primary objective of this study was to identify and evaluate laboratory conditioning procedures, including heating, moisture, and loading, that may be capable of aging mixtures (i.e., detrimentally affecting mixture properties, including reduction in fracture energy (FE), reduced resistance to accumulation of damage, reduction in effective stiffness, and reduction in healing potential) to levels consistent with those observed in the field. The overall research concept is illustrated in Figure 1-1.

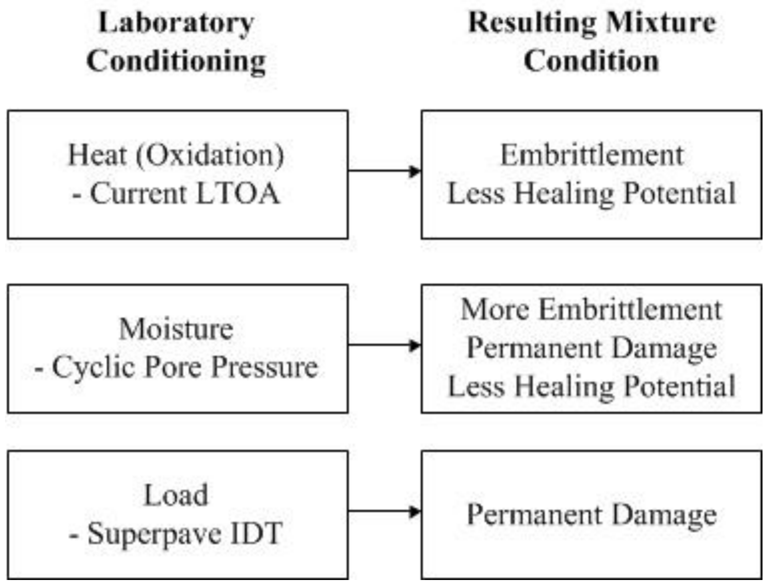


Figure 1-1 Research concept

Note: IDT denotes Indirect Tension Test.

The detailed goals of this research are as follows.

- Identify the factors that affect changes in fracture properties of asphalt mixture subjected to aging in field pavement.
- Identify and develop appropriate laboratory conditioning procedures and testing protocols considering heat oxidation, moisture, and load, to properly assess asphalt mixture property change over time in the field.
- Evaluate the effect of laboratory conditioning process identified on change in fracture properties of asphalt mixtures.
- Identify and evaluate the interaction between each conditioning procedure and their combined effects on changes in fracture properties of asphalt mixtures.

1.3 Scope

This study was initiated because appropriate laboratory aging procedures and evaluation methods were not available that considered all detrimental effects known to affect asphalt mixture aging in the field. In order to meet the objectives of this research, it was necessary to identify and develop laboratory conditioning processes and to reliably assess the changes in fracture properties induced by individual conditioning procedures, as well as by their combined effects.

For laboratory conditioning procedures, three factors known to affect aging were involved: heat oxidation, moisture and load-induced damage. Current standard method using forced- draft oven (AASHTO PP2, 2001) were involved to simulate the effect of heat oxidation. Based on previous research effort for FDOT (Birgisson et al., 2005), the cyclic pore pressure conditioning (CPPC) system was employed to effectively induce moisture damage in a more realistic manner. In addition, Superpave Indirect Tension Test (IDT) system was used for cyclic load conditioning to simulate effects of loading on mixture degradation and to obtain fracture properties of conditioned mixture specimens.

Three types of mixtures were evaluated: granite w/o lime, granite w/ lime, and limestone mixtures. All mixtures were fine dense-graded mixtures. The same type of binder (PG 67-22) was used for all mixtures to reduce asphalt binder effect. All tests were performed at 10 ℃.

1.4 Research Approach

This study primarily focused on identifying the effects of laboratory conditioning procedures for heat oxidation, cyclic pore pressure, and load conditioning on fracture properties of asphalt mixtures. The overall approach used to meet all the objectives of this project is involved the following steps:

- Review previous research regarding aging procedure and evaluation method for asphalt mixture and aging effects on mixture property changes to obtain the understanding necessary to help identify the most appropriate approaches for asphalt mixture aging.
- Perform preliminary tests to establish laboratory conditioning and testing protocols. The main goal of this task involved identification of appropriate ranges of conditioning:
  - For moisture conditioning: applied pressure level and number of cycles
  - For load conditioning: applied load level, rest period, and load cycles
- Design experiment and perform tests based on conditioning procedures identified to evaluate the effects of each conditioning procedure and their combined effects on changes in fracture properties of asphalt mixtures.
- Analyze mixture test results to determine the relative effects of heat oxidation, moisture, and load conditioning on mixture aging.
- Determine the most effective conditioning procedure in a controlled manner that simulates the field aging mechanisms in the laboratory.

CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

It is well documented that during its service life, asphalt pavements are subjected to change in its properties that affect the pavement performance. Several factors such as environment, moisture, changes in volumetric properties and traffic load have considerable detrimental effects on cracking performance of asphalt mixtures. Significant amount of researches have been focused on the effects of each one of these factors on change in mixture properties and performance. Nevertheless, there is no generally agreed upon definition for aging. Previously, aging has been considered mostly associated with oxidative aging and binder hardening and no other factors are included in the conventional definition of aging. However, it is now well known that other detrimental effects play a fundamental role on the deterioration of asphalt mixture properties.

2.2 Aging Procedure and Evaluation Method for Asphalt Mixture

Among the factors which lead to the detrimental effects on mixture performance, environment has been conventionally defined as the major cause of aging. If we define aging as the detrimental effect due to environment, there are a number of existing laboratory aging methods to evaluate the effect of aging on mixture properties and performance. A summary of asphalt mixture aging tests to simulate field aging is presented in Table 2-1. From Table 2-1, it was indicated that all tests listed can be divided into two categories. Tests in the first category involve the application of elevated temperatures and atmospheric pressure over varying periods of time up to 25 days, while tests in the second category apply both elevated temperatures and pressures and consequently reducing the duration of the test.

Table 2-1 Summary of asphalt mixture aging tests (Airey et al., 2005)

Test Method	Test Procedure
Ottawa sand mixtures Pauls et al. (1952)	Various time periods @ 163 ℃ on 50 mm x 50 mm cylinder
Plancher et al. (1976)	150℃ for 5 hours on 25 mm x 40 mm Φ specimens
Ottawa sand mixtures Kemp et al. (1981)	60℃ for up to 1200 hours
Hugo and Kennedy (1985)	4 or 7 days @ 100 ℃ in a dry or 80 % relative humidity environment
Long-term aging Von Quintas et al. (1988)	60 ℃ for 2 days followed by 107 ℃ for 3 days on compacted specimens
SHRP long-term oven ageing (LTOA) Harrigan et al. (1994)	5 days @ 85 ℃ on compacted specimens
Bitutest protocol Brown et al. (2000) & Scholz (1995)	5 days @ 85 ℃ on compacted specimens
Kumar and Goetz (1977)	1, 2, 4, 6 and 10 days @ 60 ℃ on compacted specimens while pulling air through specimen at a constant head of 0.5 mm of water
SHRP low pressure oxidation (LPO) Bell (1989)	Passing oxygen through a confined triaxial specimen at 1.9 l/min at either 60 ℃ or 85 ℃ for 5 days
Khalid and Walsh (2000)	Feeding compressed air at 3 l/min at 60 ℃ for up to 25 days through compacted specimens
Long-term aging Von Quintas et al. (1988)	5 to 10 days @ 60 ℃ at a pressure of 0.7 MPa on compacted specimens
Oregon mixtures Kim et al. (1986)	0, 1, 2, 3 and 5 days @ 60 ℃ at a pressure of 0.7 MPa on compacted specimens
PAV mixtures Korsgaard (1996)	100 ℃ for 72 hours on compacted specimens in a 2.07 MPa air pressure vessel

A comprehensive study regarding laboratory simulation of aging has been provided by Bell and associates as part of their efforts on the SHRP project (Bell and Sosnovske, 1994).

Three published SHRP reports (SHRP-A-383, SHRP-A-384 and SHRP-A-390) present a good overview of the experience and practice regarding aging procedures for asphalt binder and mixture. In these studies, aging is associated with the phenomenon of hardening due to environment effects. The six major factors that contribute to the age hardening can be summarized as follow (Brown et al., 2009):

- Oxidation: reaction of oxygen with asphalt cement that leads to changes in composition.
- Volatilization: loss of lighter constituents (oils) from asphalt cement. Usually it occurs primarily during construction.
- Polymerization: like molecules combine together to form larger molecules, causing progressive hardening.
- Thixotropy: formation of a structure within the asphalt cement with a consequent progressive hardening over a period of time which can be destroyed by reheating and working the material. This effect is commonly called steric hardening.
- Syneresis: loss of the thin oily liquids which are exuded to the surface of the asphalt cement, causing hardening.
- Separation: loss of oily constituents, resins and asphaltenes by absorption of porous aggregates.

However, it is universally accepted that heat oxidation, volatile loss, and steric hardening are the three dominant factors affecting age hardening of asphalt mixtures. Hardening by itself may be beneficial since stiffer mixture are more resistant to permanent deformation but combined with embrittlement, aged asphalt mixtures are generally less durable in terms of wear resistance and moisture susceptibility. In fact, asphalt mixture hardening increases the load bearing capacity and permanent deformation resistance of the pavement by stiffening the material but reduces pavement flexibility (i.e., embrittlement).

In reality, asphalt pavements undergo two distinct aging-hardening stages: short term, which occurs during the construction phase and is primarily due to oxidation and volatilization and long term, due to the progressive oxidation during pavement life. For this reason, Bell and associates (SHRP-A-383) defined the following aging procedures:

- Short-Term Aging (for loose mixtures)
  - Forced-draft oven aging
  - Extended mixing
- Long-Term Aging (for compacted samples)
  - Forced-draft oven aging
  - Pressure oxidation
  - Triaxial cell aging

In terms of evaluation method to quantify the effects of aging on change in asphalt mixture properties, Bell and associates defined three different tests: resilient modulus test, dynamic modulus test, and tensile test. Rheology test was used to evaluate the aging effects on change in mixture properties. Also, they validated their extended work with different types of asphalt binder (SHRP-A-384) and in the field (SHRP-A-390). Based on the results of their study, Bell and associates recommended two procedures for dense mixture:

- Short-term aging: oven aging of loose mixture at 135 °C (275 °F) for 4 hours.
- Long-term aging: oven aging of compacted samples at 85 °C (185 °F) for 5 days.

Based on these works carried out under SHRP A-003-A, laboratory procedures were developed to simulate the field hardening of asphalt binders and mixes by the American

Association of State Highway and Transportation Officials (AASHTO R28 and AASHTO R30 respectively). These protocols require asphalt mixes to be tested at the LTOA condition which provides a method to simulate field-hardening due to oxidative aging that would occur over years of service in situ.

However, the approach used by Bell and associates (Bell et al., 1994) was based on the assumption that aging is due to only asphalt oxidation causing embrittlement. In addition, they assumed stiffness of asphalt mixture increases with aging. They made use of stiffness ratio, which is the ratio of the resilient modulus before and after a period of oxidation. Even though they were successful in suggesting procedures for short-term and long-term aging for mixture based on stiffness measurements on field cores, their basic assumption that stiffness of mixture increases with aging is questionable because additional damage effects caused by moisture and repeated loads that actually decrease stiffness in the field should be considered for evaluation. They explained that the difference between laboratory aging and field core results were caused by differences in traffic volume and/or moisture damage in the field.

A more recent work by Witczak and associates (Raghavendra et al., 2005) indicated that the AASHTO R28 protocol is not sufficient to simulate the field aging behavior of asphalt binder in the laboratory. They summarized the major limitations of this protocol: (1) only two PAV aging temperatures were used to represent a wide range in Mean Annual Air Temperature (MAAT), and (2) binder type (binder properties) and volumetric properties are not taken in account.

Witczak and associates (Raghavendra et al., 2006) also indicated similar limitations in the LTOA standard protocol (AASHTO R30). They concluded that the conventional LTOA procedure should account for the asphalt layer thickness, air-voids and asphalt mixture properties in order to accurately predict the aging characteristics of the asphalt mixes. In order to evaluate aging effects on asphalt mixture, complex modulus test was carried out at six loading frequencies and temperatures. However, both works presented by Witczak and associates are built on the assumption that oven aging temperature would account for most of the variables that affect aging in the field.

Harvey et al. (Harvey and Tsai, 1997) in their study concluded that the standard procedure of long-term aging does not simulate all the effects of long-term aging that occur in the field. Other detrimental effects conventionally assumed to be caused by long-term aging, may be caused by other detrimental time-dependent processes, which interact each other, such as water damage. These detrimental effects may occur in the asphalt concrete at the same time with oxidative aging. These conclusions were based on fatigue test results that showed for all cases how predictions of pavement life increased with LTOA. Based on their test results, they then concluded that increase in stiffness caused by oxidative aging are not always detrimental to pavement fatigue performance.

A successful attempt to reproduce in the laboratory the loss of stiffness modulus observed in the field has been performed by Airey and associates (Airey et al., 2005 and Airey et al., 2007) developing a new test known as Saturation Aging Tensile Stiffness (SATS). This test combines aging with moisture conditioning by aging compacted asphalt mixture cylindrical specimens at an elevated temperature and pressure in the presence of moisture. Specimen are saturated and then placed into the cylindrical vessel partially filled with water. The conditioning procedure is performed at a pressure of 2.1 MPa (304.5 psi) and a temperature of 85 °C (185 °F) for 65 hours. Then the "retained stiffness modulus" is calculated as the ratio of the final stiffness modulus over the unconditioned (i.e., initial) stiffness modulus and is a function of the final (i.e., retained) saturation level. Although their definition of aging is the same as previous researchers (aging due to environment), Airey and associates indicated that moisture conditioning plays a major role on the “aging” of asphalt mixtures.

2.3 Aging Effects on Mixture Property Changes

One of the main objectives of this project is to identify the factors affecting fracture properties of asphalt mixture subjected to aging process on in-place condition. Several studies had similar objective, trying to identify the effects of aging on change in asphalt mixture properties.

Harvey and Tsai (1997) evaluated the effect of long-term aging, air void content and asphalt content on mix fatigue life, initial stiffness and pavement fatigue life. As previously mentioned, their work was based on the assumption that LTOA procedure provides a means to quickly simulate aging that would occur over years of service in situ (i.e., no other aging effects were taken into consideration). SHRP A-003A recommendations were used for STOA and LTOA; three LTOA periods were used (i.e., 0, 3, and 6 days). Controlled strain fatigue beam test was used to determine flexural stiffness and fatigue life. Based on their test results, they concluded that pavement fatigue life for thick and thin pavement structures was increased with LTOA. On the other hand, they demonstrated that the effect of LTOA on pavement fatigue life highly depends on asphalt type, aggregate type, air voids and pavement structure.

Opposite results were obtained by Raad et al. (2001). They investigated the effects of field aging on fatigue of asphalt concrete based on stiffness and fatigue from controlled-strain fatigue beam tests at 22 ℃ (71.6 ℉) and -2 ℃ (28.4 ℉). Specimens were obtained from a 10-year field section. Results showed an increase in stiffness of 30 percent for the dense graded mixture at 22 ℃ and a decrease of 12 percent at -2 ℃ for aged specimens. Furthermore, aging on dense graded mixture reduced the beam fatigue life, showing a detrimental effect to pavement fatigue.

Roque et al. (2007) investigated the cracking potential of asphalt mixtures to top-down cracking using Accelerated Pavement Testing (APT) loaded with and Heavy Vehicle Simulator (HVS). In order to simulate age-hardening of in-service pavement, an Accelerated Pavement Aging System (APAS) was used to artificially aging asphalt mixture and pavement in the APT. As binder oxidation is conventionally considered the major cause of asphalt mixture aging, the APAS equipment developed at FDOT uses heat as the driving force for accelerating aging, creating a stiffness gradient through the asphalt concrete layer. Therefore, other effects such traffic load and moisture effects are not taken into account. Asphalt mixture properties were determined from Falling Weight Deflectometer (FWD) tests on the pavement section and from Superpave IDT tests performed on asphalt concrete cores. Even if the APAS equipment was able to create a substantial oxidative aging, however, mixture properties of aged pavements did not result in reduction in tensile failure limits consistent with the field observation. Roque and associates concluded that factors others that oxidative aging and binder stiffening not replicated by the APAS equipment affect the failure of the field pavement.

As mentioned before, several researchers indicated moisture damage as one of the major detrimental effects during pavement life. A number of tests exist in the literature to determine asphalt mixtures moisture sensitivity; the current Superpave specification uses the AASHTO T-283 moisture susceptibility test for determining moisture sensitive mixtures. Although most state agencies use the AASHTO T-283, several papers questioned its accuracy leading to the initiation of the National Cooperative Highway Research Program (NCHRP) entitled “NCHRP Project 9-34: Improved Conditioning Procedure for Predicting the Moisture Susceptibility of HMA

Pavements" ended in 2007. A laboratory procedure based on the ECS (Environmental Conditioning System) and dynamic modulus test was developed under this research and presented in the final report (NCHRP report 589, Solaimanian et al., 2007). It demonstrated the ability to discriminate between mixes with good and poor performance in resisting moisture damage. However, this procedure still needs further work to be simplified and shortened. Although moisture damage is recognized as a major problem in asphalt pavements, the researchers didn't find any work to date on an appropriate aging procedure that includes heat, moisture and loading, as well as their combined effect.

More recently, Roque et al. (2011) indicated that the conventionally accepted trend of asphalt concrete stiffness that continuously increases with time does not coincide with the results of their field monitoring project (BDK75 977-06). Roque et al. evaluated 11 field sections throughout the State of Florida. It was found that the stiffness generally reduces with time after a certain age. Figures 2-1 and 2-2 show changes in fracture energy and resilient modulus over time (for 6 to 12 years of age). Based on the project results, the percent reduction in fracture energy was approximately 25 to 56 % for sections without moisture damage (Projects 1, 2, 3, 4, 6, 7 and 8). On the other hand, moisture-damaged sections (Projects 9, 10 11 and 12) exhibited a range of fracture energy reduction of 46 to 76 %.

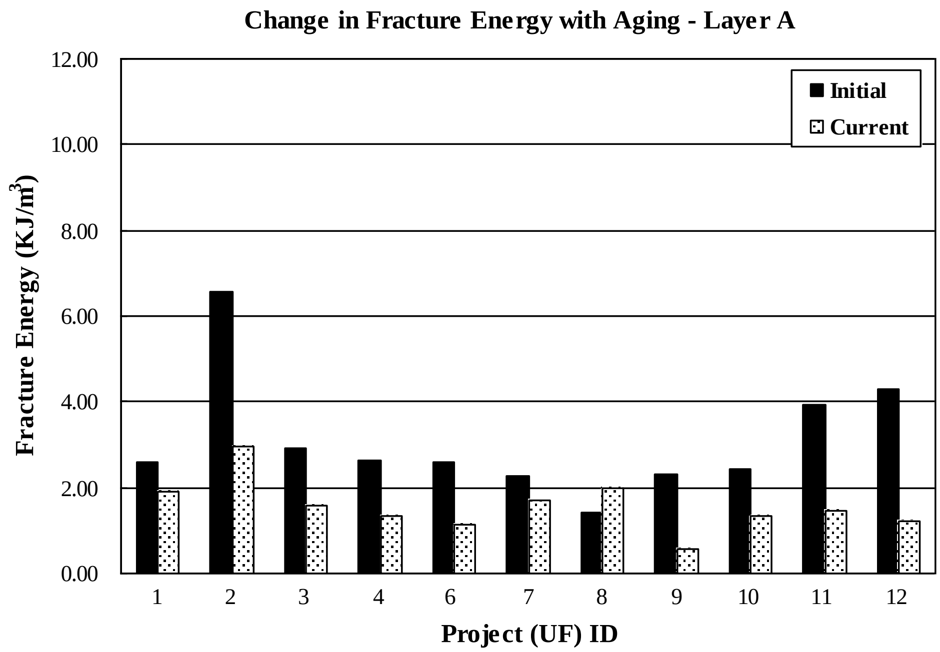


Figure 2-1 Change in fracture energy over time

Note: Layer A denotes top Superpave asphalt concrete layer.

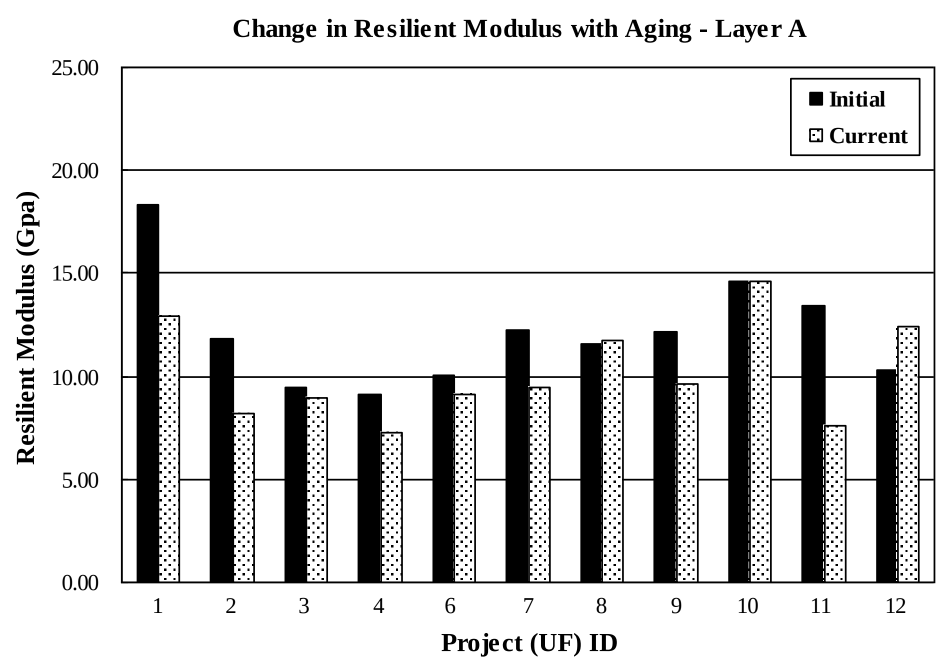


Figure 2-2 Change in resilient modulus over time

Note: Layer A denotes top Superpave asphalt concrete layer.

Therefore, a new concept was proposed and modifications of the existing aging model were conceived in order to include the effect of non-healable permanent damage related to load and moisture damage on the change in AC stiffness (S) with time. Figure 2-3 describes the proposed modifications.

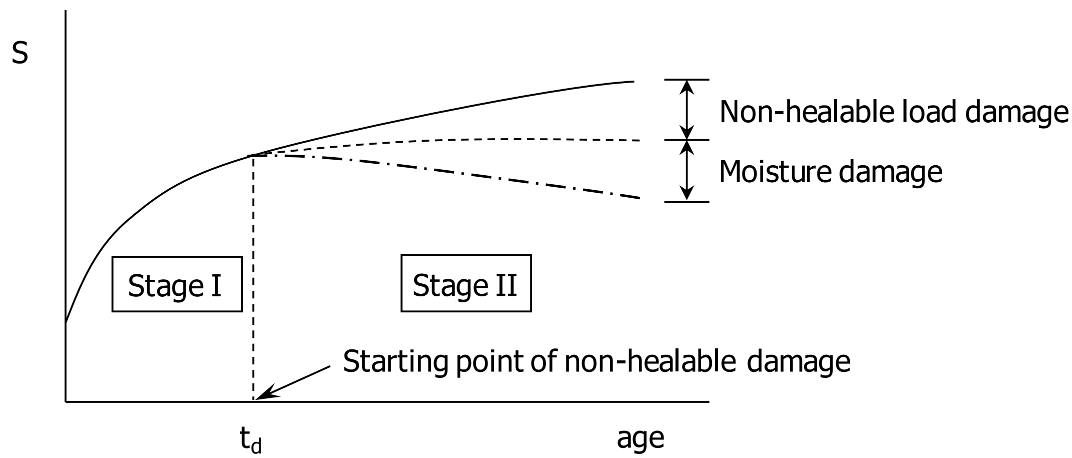
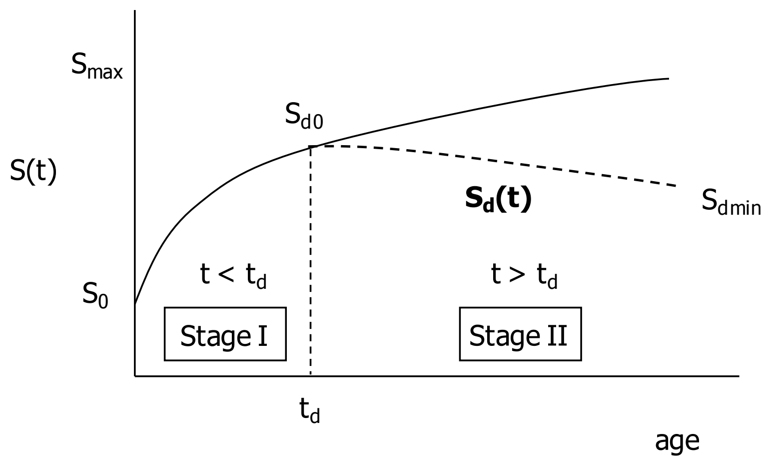


Figure 2-3 Proposed AC stiffness model

As shown in Figure 2-3, AC stiffness trend can be separated into two stages by a time denoted t_d. During Stage I, the trend of change in AC stiffness is governed by a healing process more than a damage process and is mainly controlled by oxidative aging. On the other hand, the non-healable permanent damage process takes control during Stage II, which includes load-induced damage and moisture-related damage.

Based on the same principles, Roque and associates proposed a modification to the existing fracture energy limit aging model developed as part of the NCHRP Project 01-42A in order to include the effect of the non-healable permanent damage related to load and moisture on change in fracture energy limit with time. As in the modified AC stiffness model, a critical time t_d separates the trend in two stages. During Stage I, it was assumed that there is no permanent damage induced by load and moisture and only oxidative aging controls the trend. Again, in Stage II a modified relationship for FE limit age function was proposed to consider the permanent damage effect on AC stiffness and therefore on fracture energy. Figure 2-4 (a) and Figure 2-4 (b) describes the proposed modifications for the FE limit aging model.



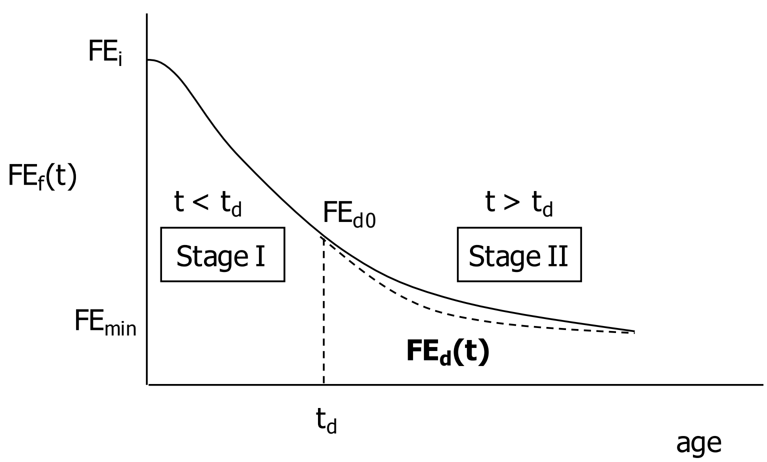
1) For t < t_d (Stage I)

$$S_n(t) = \frac{S(t) - S_0}{S_{\max} - S_0}$$

2) For t > t_d (Stage II)

$$S_{dn}(t) = \frac{S_{d0} - S_d(t)}{S_{d0} - S_{d\min}}$$

(2-4a) Change in AC stiffness over time



1) For t < t_d (Stage I)

$$\frac{FE_i - FE_f(t)}{FE_i - FE_{\min}} = \left[S_n(t)\right]^{k_1}$$

2) For t > t_d (Stage II)

$$\frac{FE_{d0} - FE_d(t)}{FE_{d0} - FE_{\min}} = \left[S_{dn}(t)\right]^{k_2}$$

(2-4b) Change in FE limit over time

Figure 2-4 Proposed FE limit model

2.4 Closure

Based on the literature review conducted, the following conclusions can be reached:

- SHRP A-003A recommendations which define the conventional Long-Term Oven Aging (LTOA) procedure (AASHTO PP2, 2001) provides a method to simulate oxidative aging that occurs over 5 to 10 years of service in situ.
- Raghavendra et al. (2006) indicated that the AASHTO R30 protocol is not sufficient to simulate the field aging behavior of asphalt binder in the laboratory: asphalt layer thickness, air-voids, and asphalt mixture properties should be taken in account.
- Airey et al. (2005) indicated that moisture conditioning plays a major role on the "aging" of asphalt mixtures. By developing a new aging system based on a combination of moisture and heat, they successfully reproduced in the laboratory with respect to reduction in stiffness observed in the field.
- In accordance with the findings of previous researchers (Airey et al., 2007 and Harvey and Tsai, 1997), Roque et al. (2007) indicated that other detrimental factors such as traffic load and moisture effects should be taken in account to simulate the field aging.
- Roque et al. (2011) indicated that the conventionally accepted trend of asphalt mixture stiffness that continuously increases with time does not coincide with the results of their field monitoring project.
- Therefore, there is a need to develop a new aging procedure that accounts for other parameters such as moisture and load to effectively simulate asphalt mixture aging that actually occur in the field.

CHAPTER 3
IDENTIFICATION OF LABORATORY CONDITIONING AND TESTING PROTOCOLS TO EFFECTIVELY CHARACTERIZE THE AGING IN THE FIELD

3.1 Introduction

As reviewed in Chapter 2, it is now well known that oxidation caused by heating alone (e.g., current LTOA) may not simulate all of the effects of long-term aging in the field condition. In other words, it is possible that many of the detrimental effects involved in long-term aging that actually occur in the field, including effects of moisture and load, may not be simulated by the current LTOA method. In addition, there may be very little work to date on an appropriate aging procedure and evaluation method considering all these detrimental effects to simulate asphalt mixture aging for in-place condition. Therefore, there was a strong need to identify and develop laboratory conditioning and testing protocols to effectively characterize aging in the field condition.

For purposes of this research, aging was defined as all detrimental effects during pavement life on fracture properties of asphalt mixture, including reduction in fracture energy (FE), reduction in resistance to accumulation of damage, reduction in effective stiffness, and reduction in healing potential. Factors that can cause aging as defined in this manner include oxidation, moisture effects, and load effects. There may also be interactions between these effects. One of the primary objectives of this study is to identify and evaluate laboratory conditioning procedures, including heating, moisture, and loading, as well as their combined effects that may be capable of aging mixtures to levels consistent with those observed in the field (see Figure 2-1 and 2-2). Based on experience and observation, basic concepts for developing appropriate and relevant conditioning approaches were defined as shown in Figure 3-1.

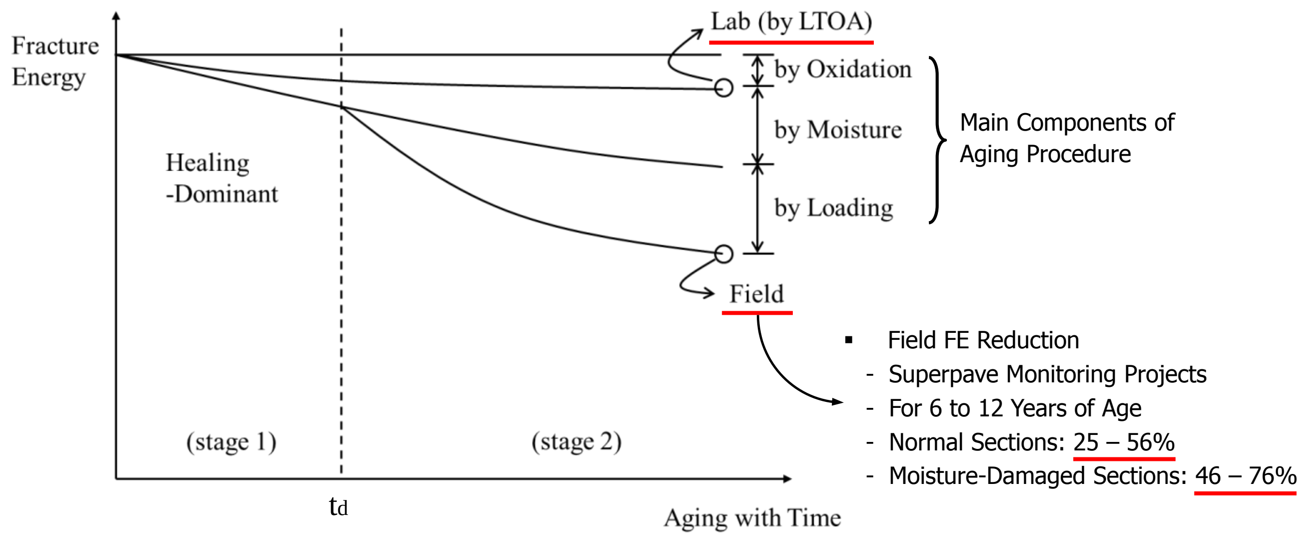


Figure 3-1 Expected trend of asphalt mixture property (fracture energy) with aging

As shown in Figure 3-1, there is a critical time t_d which separates pavement life into two stages. In stage 1 which is early years of aging, there is no effect of load –induced permanent damage on change in fracture energy with time. For stage 1, change in fracture energy is mainly controlled by oxidative aging and moisture damage rather than load-induced permanent damage. Stage 1 is named as healing-dominant stage because healing process is a dominant factor as compared to damage process to govern the trend of change in fracture energy in this stage.

However, as also indicated in Figure 3-1, load-induced permanent damage process starts taking control in stage 2. The permanent damage induced by load can accelerate the aging process which will result in the trend of change in fracture energy with time in this stage. Based on previous research (Roque et al., 2011), field fracture energy reductions observed from Superpave monitoring sections (6 to 12 years of aging) varied approximately from 25 to 56 % for normal sections (without moisture damage) and from 46 to 76 % for moisture-damaged sections. Approximately 20 % further reduction in fracture energy was observed in moisture-damaged sections.

3.2 Heat Oxidation Conditioning (HOC)

3.2.1 Introduction

Oxidation is the reaction of oxygen molecules with asphalt binder and the rate of oxidation depends on the characteristics of the asphalt binder and temperature. Asphalt pavement is continuously affected by oxidative aging during its service life and the rheological properties of asphalt binder are highly affected by oxidative aging process. Heat oxidation is generally considered as a primary factor contributing to hardening or embrittlement of asphalt mixtures. The major contributions to asphalt hardening were summarized by Petersen (1984) as follows.

- Loss of oily components by volatility or absorption.
- Change in composition by reaction with atmospheric oxygen.
- Molecular structuring that produces thixotropic effects (steric hardening).

Heat oxidation occurs in the construction phase including the period between mixing and placement in the field (short-term aging). Also, continuous process of oxidation occurs for in-place mixtures in the field over time (long-term aging). To simulate heat oxidation including short-term and long-term aging, standard short-term oven aging (STOA) and long-term oven aging (LTOA) procedure was introduced under the Strategic Highway Research Program (SHRP) (AASHTO PP2, 2001). The STOA simulates the aging effects during mixing and construction process of asphalt mixtures that indicates the condition immediately after construction. The LTOA simulates the additional aging effect on mixtures subjected to in-place condition for 5 to 10 years. It is obvious that current STOA and LTOA methods to age samples involve only heat oxidation of asphalt mixtures. In this study, the existing STOA and LTOA procedure was used for initial conditioning of asphalt mixtures to simulate the effect of heat oxidation.

3.2.2 Short-Term Oven Aging (STOA)

The current STOA process proposed by SHRP is basically related to heating a loose mixture in a forced-draft oven for 2 hours at a temperature of 300-315 °F. During the heating process, the loose mixture was spread in a pan and then stirred after 1 hour to ensure uniform aging throughout material. Figure 3-2 shows the loose mixture used for STOA procedure.

Figure 3-2 Loose mixture for STOA

3.2.3 Long-Term Oven Aging (LTOA)

The LTOA process involves aging of compacted mixtures after the STOA procedure. The LTOA requires a compacted sample (after STOA) to be placed in a forced-draft oven at 185 ± 5 °F for 5 days. A wire mesh with openings of 0.125 inch and steel band clamps were used to contain the specimens. The mesh size was selected to certify good air circulation within the samples and to prevent the falling apart of smaller aggregate particles during the LTOA process as well. The samples were wrapped twice using mesh cloth and held by two clamps without applying excessive pressure. The specimens were then placed on porous plates in forced-draft oven for LTOA. The specimens were turned over twice during the 5-day LTOA period. The samples with the wire mesh and porous plate for LTOA are shown in Figure 3-3.

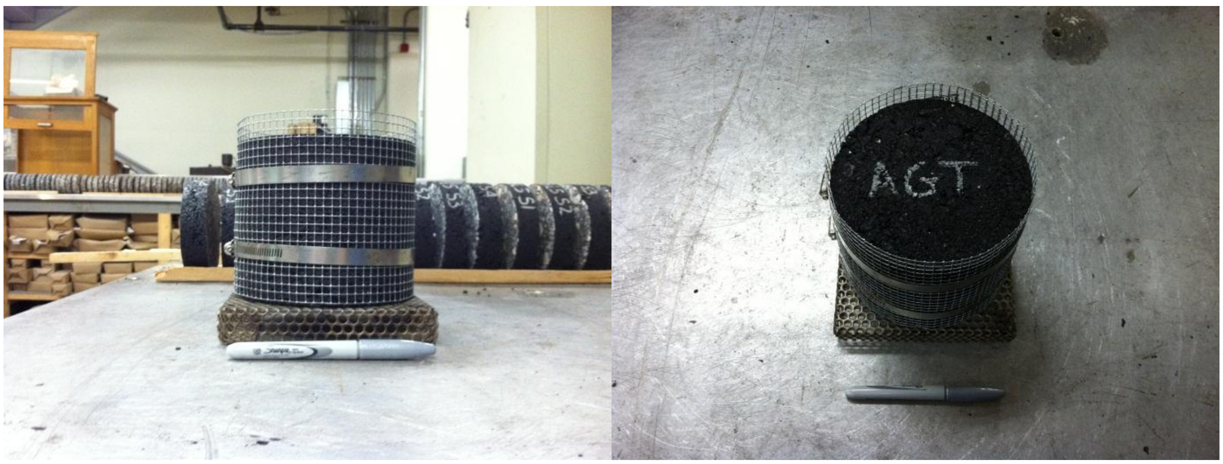


Figure 3-3 Specimens setup for LTOA conditioning

3.2.4 Ten-Day-Term Oven Aging (TTOA)

In this study, a longer LTOA process than the current 5-day LTOA, i.e., a 10-day LTOA termed as Ten-Day-Term Oven Aging (TTOA), was added to evaluate change in tensile properties of asphalt mixtures beyond current LTOA. It was expected that this additional conditioning could possibly help to evaluate the effect of oxidative aging on fracture properties of asphalt mixtures at an excessive level of heat oxidation.

3.2.5 Closure

Laboratory conditioning procedure to simulate the effect of heat oxidation was identified and selected. All test specimens were subjected to different levels of heat oxidation conditioning (HOC) during the preparation of test specimens based on procedures introduced in this chapter.

3.3 Cyclic Pore Pressure Conditioning (CPPC)

3.3.1 Introduction

Moisture damage of asphalt mixture is a complicated mode of deterioration that leads to reduction in stiffness and possibly disintegration. Asphalt pavement is continuously affected by water during its service life. However, whereas oxidation increases stiffness, moisture effects decrease stiffness. Therefore, it is necessary to evaluate their effects simultaneously. There are several methods to evaluate moisture sensitivity of asphalt mixtures including standard AASHTO T283 procedure which involves freeze-thaw process (AASHTO, 2001). However, none of these methods has been found to accurately predict the magnitude of moisture damage of different asphalt mixtures in the field (Airey et al., 2007). In addition, none of them simulate repeated generation of pore pressure under wheel load which is believed to be a major cause of moisture damage that is similar and realistic as possible as that experienced in the field. For this research, only pore water pressure induced moisture damage was considered. Based on previous FDOT research effort (Birgisson et al., 2005), the CPPC system was determined to effectively induce moisture damage in mixtures. This system was thought to provide the advantage of inducing moisture damage in a more realistic manner than other systems that involve either boiling or freeze-thaw cycles.

3.3.2 Description of CPPC system

Cyclic pore water pressure has been identified as a likely major mechanism of premature moisture damage in asphalt mixtures. In this study, a new CPPC system using triaxial chamber was identified and proposed based on previous FDOT research effort (Birgisson et al., 2005). The concept of the CPPC system was prompted by the need to better analyze the effect of moisture-induced damage on asphalt mixtures. The use of the triaxial chamber for conditioning allows for precise application of stress in three different directions. The overall appearance of the triaxial cell used is shown in Figure 3-4.

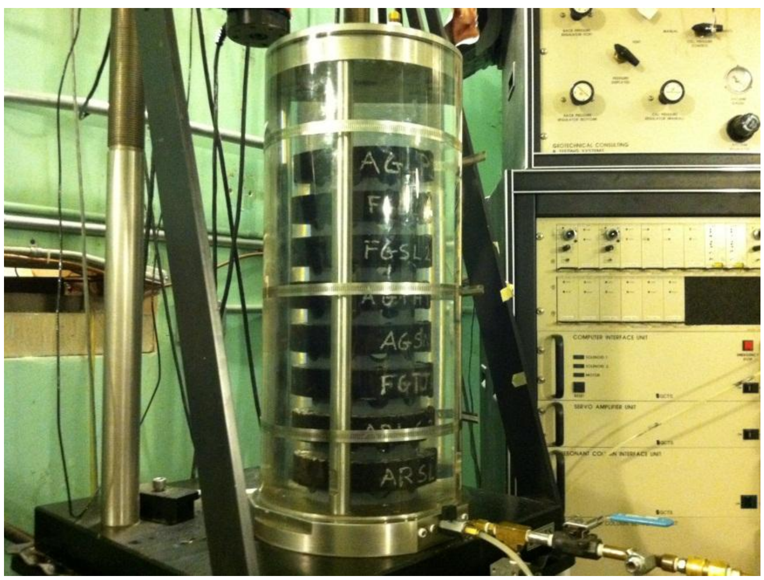


Figure 3-4 Tabletop triaxial chamber

The structural core of the cell consists of two round plates separated by posts or struts. The structural core is encased with a cylinder and the entire package is sealed which creates an enclosed cavity capable of being pressurized. More detailed information can be found in the reference (Birgisson et al., 2005). In addition, since this triaxial system was used for moisture conditioning only (no need for external load), the previous triaxial system was greatly simplified and become a self-contained tabletop system that would not require an external loading frame. That is, moisture conditioning could be achieved without axial loading. To achieve desired and consistent conditioning levels, there might be a need to develop a saturation system and procedure. This system would be capable of applying a vacuum as well as forcing the permeant through the specimen from the influent end.

The CPPC system also allows for temperature control. The water delivery system can be connected to either a heater or chiller unit (see Figure 3-5). The combination of the heating and chiller units allows the test specimen to be controlled within ± 2 ℃, up to 75 ℃. It was expected that relatively high temperature might be used in this study since high temperature could not only accelerate moisture-induced damage but also be more representative of Florida's climate. However, excessively high temperature should be avoided because additional deterioration in asphalt mixture due to heating or permanent deformation could be a problem.

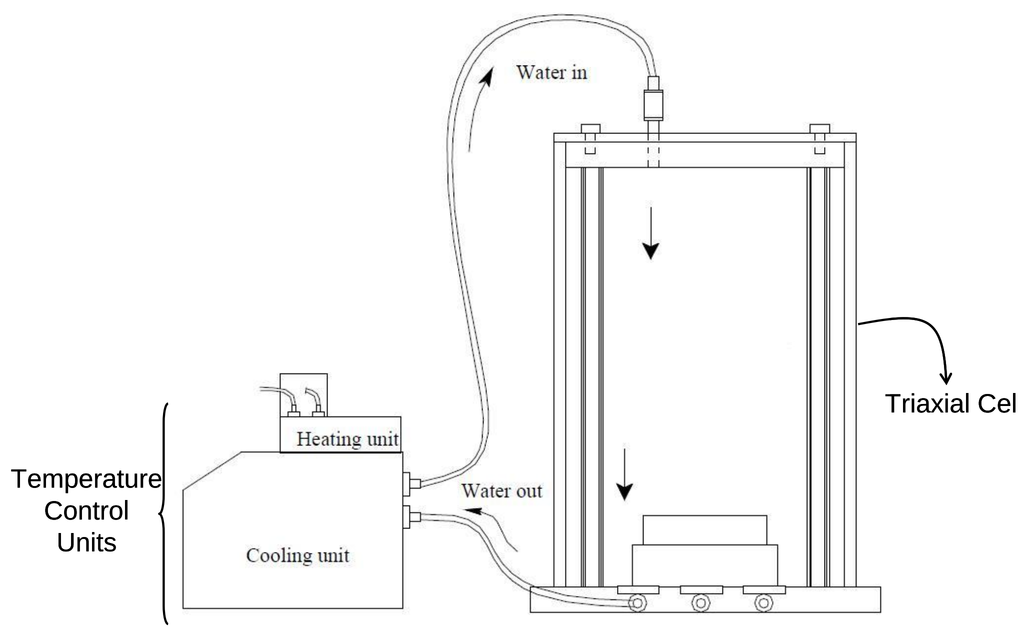


Figure 3-5 Triaxial cell and temperature control units

In this study, a careful review of past research conducted regarding the moisture conditioning of asphalt specimens was undertaken to determine CPPC temperature. Two conditioning temperatures, 25 ℃ and 40 ℃, were used for this evaluation. Several mixtures were subjected to both of these conditioning temperatures and the testing results are compared in Figure 3-6 (fracture energy) and Figure 3-7 (resilient modulus). The mixtures tested were all granite mixtures, since it is well known that granite mixtures generally perform poorly in moisture conditioning tests. Conditioning temperature had a significant effect on the coarse mixture fracture energy, but had almost no effect for the fine graded mixtures. Changes in conditioning temperature had no effect on resilient modulus. It was hypothesized that reduction in FE observed for the coarse mixture at the higher conditioning temperature (40 ℃) were caused not only by moisture effect, but also by volumetric and/or structural changes, resulted in coarse mixtures at the higher conditioning temperature (40 ℃). Therefore, 25 ℃ was selected as the conditioning temperature to isolate moisture effects using CPPC.

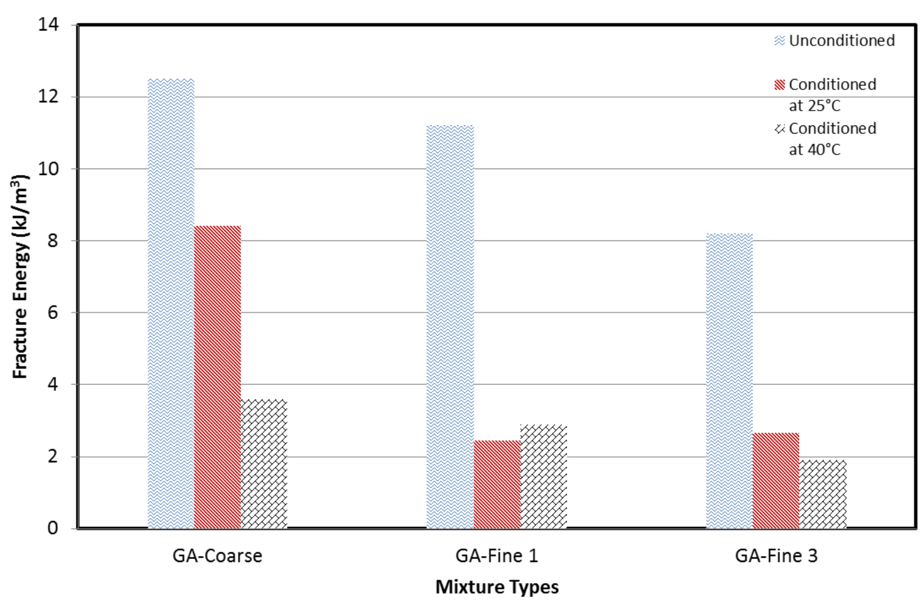


Figure 3-6 Change in fracture energy due to different CPPC temperature

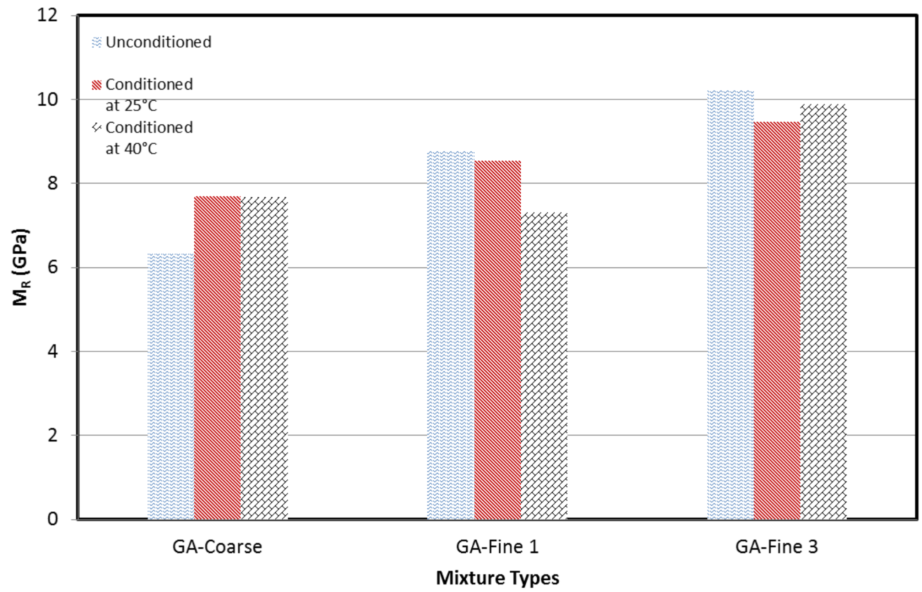


Figure 3-7 Change in resilient modulus due to different CPPC temperature

3.3.3 CPPC procedure

The conditioning of the asphalt specimens takes place by exerting cyclic pore pressure on all water accessible voids of the sample. This is accompanied by saturating the sample and then placing it into an airtight, water-filled chamber. Water is forced into the chamber to build up pressure. This pressure is transferred to every surface that the water is in contact with. When this pressure is cycled in the chamber at a constant rate by a sine wave, it can simulate a portion of the pumping action that can occur in the field when vehicles travel over wet pavement. Figure 3-8 exhibits overall CPPC procedure flowchart.

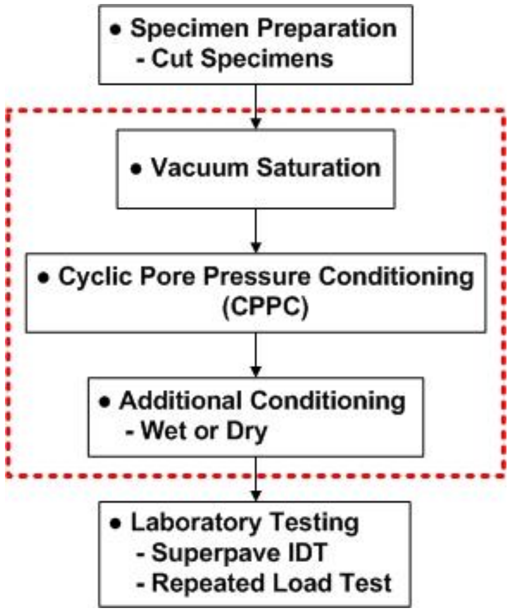


Figure 3-8 Overall CPPC process flowchart

3.3.3.1 Sample Saturation

The test specimens were vacuum saturated at a pressure of 25 + 2 in·Hg (12.28 + 0.98 psi) for 15 minutes. The samples were slightly agitated to remove some of the air bubbles clinging to the surface of the specimens before releasing the vacuum. The specimens were then allowed to sit submerged for 20 minutes at the normal pressure. This allowed the water to infiltrate the deeper voids in the specimens that were previously filled with air. The samples were considered saturated after this process was completed for a second time (i.e., two cycles). No specific saturation levels were targeted since each mix has a unique void structure that may enhance or reduce the saturation capacity of the mixture. It was felt that forcing a target saturation level might cloud the effective differences between mixtures in resisting moisture ingress and therefore may possibly cause inadvertent moisture damage. Appropriate pressure levels and repetitions were determined through preliminary testing. Figure 3-9 shows the triaxial chamber used for vacuum saturation.

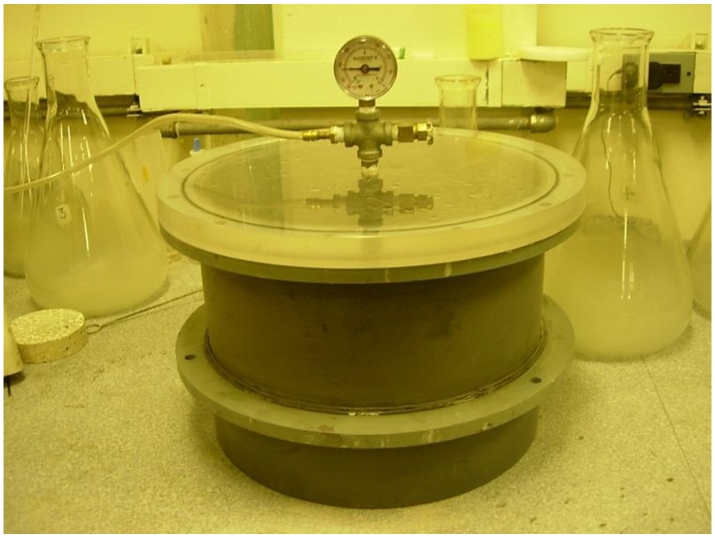


Figure 3-9 Vacuum saturation chamber

3.3.3.2 Cyclic Pore Pressure

As indicated, cyclic pore water pressures have been identified as a primary mechanism of moisture damage in asphalt mixtures. Based on previous FDOT research effort (Birgisson et al., 2005), the cyclic CPPC system was determined to effectively induce moisture damage in asphalt mixtures. In this study, a novel CPPC system using triaxial chamber was conceived and proposed. Detailed description of the CPPC system was introduced in section 3.3.2.

After vacuum saturation procedure, specimens were moved to tabletop triaxial chamber and piled up in the chamber (Figure 3-4). The spacers between specimens were used to facilitate water infiltration and to protect the gauge points attached on both faces of specimens. Triaxial chamber was then connected with deairated water supplier, pressure sensor, and pressurizer. The three clamps help to prevent the cell from expanding by pressure. Deairator was used to remove the air from the water being supplied for the CPPC system. Figure 3-10 shows deairator used. It was necessary to make sure that no or little air was remained in the chamber as possible, as well as whole the CPPC system.

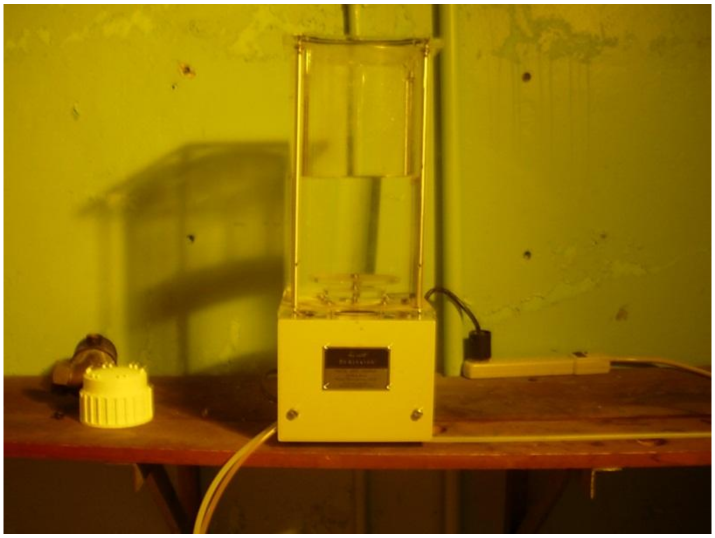
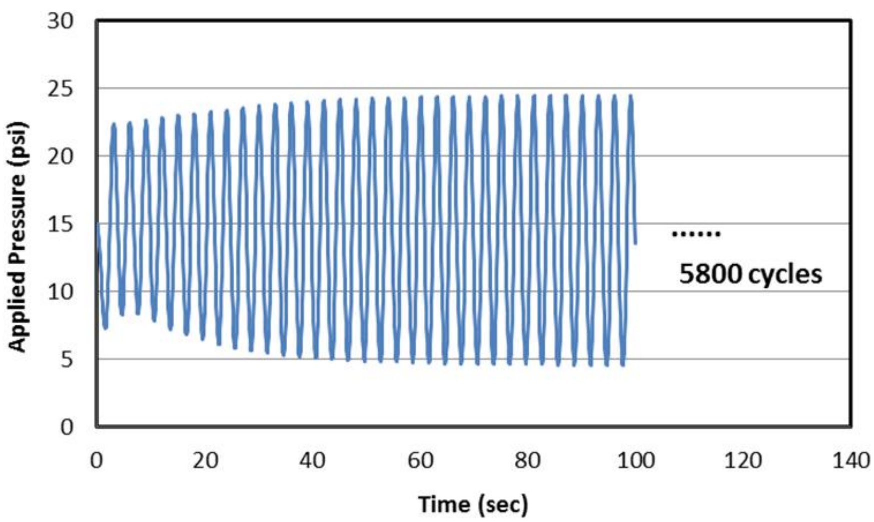
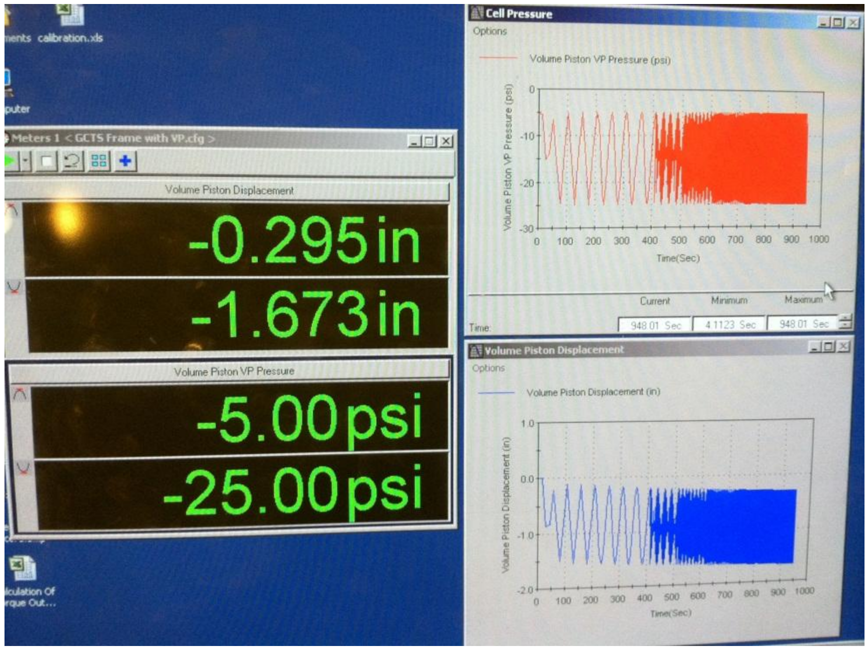


Figure 3-10 Deairator

Based on the previous research for FDOT (Birgisson et al., 2005), 5 psi to 25 psi (10.2 in·Hg to 50.9 in·Hg) cyclic pore pressure was applied at 25 °C (room temperature). A sine waveform with a 0.33 Hz frequency and a total of 5800 cycles (4.8 hours) were used. Figure 3-11 represents the applied pressure waveform for the CPPC.



(3-11a) Applied pressure



(3-11b) Picture of the computer screen

Figure 3-11 Applied pressure waveform for the CPPC

3.3.3.3 Additional Conditioning (Wet or Dry)

According to the testing plan finalized, additional conditioning procedures were involved for obtaining wet or dry specimens following the CPPC. The purpose of this extra conditioning was mainly to evaluate recovery effect of drying on moisture damage. Two conditionings, including wet and dry, were employed after the CPPC. To obtain wet-condition specimens, samples were placed in the water bath immediately after the CPPC. The temperature of water was maintained at 10 °C, which is the same as the test temperature. Figure 3-12 shows specimens placed in the water bath after the CPPC.

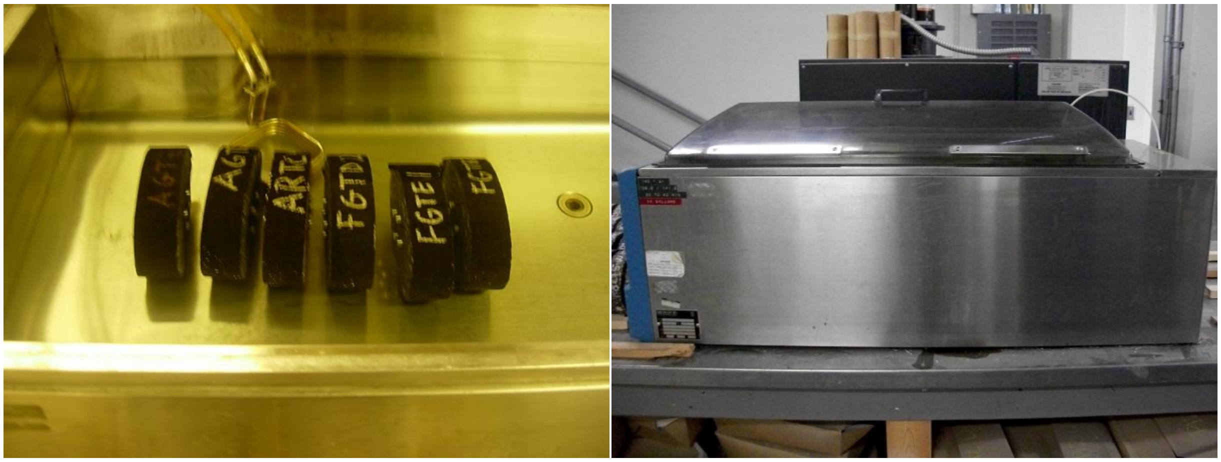


Figure 3-12 Wet conditioning after the CPPC

To obtain dry-condition specimens, samples were placed in the dehumidifier for at least two days (i.e., 48 hours) to eliminate moisture in the specimens. After dry conditioning, specimens were placed in the temperature chamber prior to testing. Figure 3-13 shows the dehumidifier used for dry conditioning.

Figure 3-13 Dry conditioning after the CPPC

3.3.4 Closure

Laboratory conditioning process to simulate the effect of moisture damage in a more realistic manner was identified and introduced (the CPPC system). During specimen preparation, some of the test specimens were subjected to the CPPC based on procedures introduced in this chapter.

3.4 Repeated Load Conditioning (RLC)

3.4.1 Introduction

Repeated load conditioning (RLC) test system was developed based on the Superpave indirect tension test (IDT) in order to be suitable for testing laboratory-compacted specimens and field cores. Because the main intent of this test was to induce damage in asphalt mixture, the same assumptions and procedures associated with the damage phase in healing evaluations (concurrent FDOT research project BDK-75-977-26) have been applied here as well. These are summarized as follows.

Static and repeated loading approaches were investigated since these two methods have widely been used to evaluate damage and fracture of asphalt mixture. However, even though static loading methods shorten testing time, delayed elasticity may cause error during healing phase if static load is employed. Roque et al. (1997) showed and explained the effects of delayed elastic response due to static loading even for very short loading times. On the other hand, resilient modulus test using Superpave IDT system was identified as an effective test to induce and measure damage. The rest periods incorporated between repeated loading applications minimize delayed elasticity. Thus, repeated loading approach was selected to damage asphalt mixture.

Furthermore, resilient modulus test is a convenient way to measure effective stiffness of asphalt mixture, which could indicate damage and damage recovery. Therefore, in this study, damage is characterized by reduction in effective stiffness, which can be measured by reduction in resilient modulus. If measured effective stiffness is decreased (i.e., increase in resilient horizontal deformation), it would indicate damage in asphalt mixture. Previous research by Honeycutt (2000) has showed that normalized horizontal deformation could be directly related to the theoretical crack length of the specimen, indicating that measure of change in horizontal deformation is a way of monitoring damage.

3.4.2 Load Level and Rest Period

As mentioned earlier, since the main intent of the RLC test is to induce damage in asphalt mixture, it is necessary to determine load level and rest period. An appropriate load level and rest period should result in development of a reasonable amount of damage within the steady state damage range (i.e., constant rate of damage) as shown in Figure 3-14.

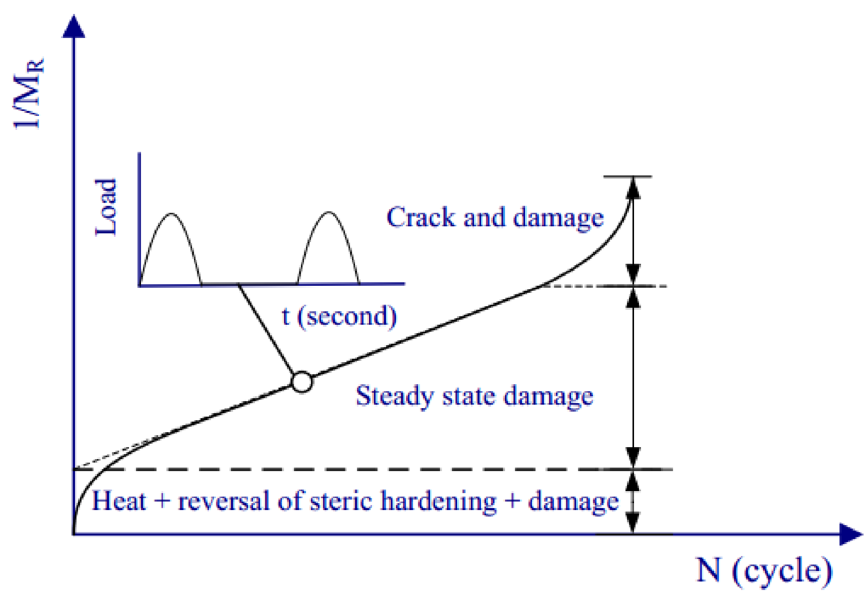


Figure 3-14 Asphalt mixture fatigue curve due to cyclic loading

Longer rest periods might allow for the time to fully heal from microdamage but take longer testing time. On the other hand, if shorter rest periods are introduced during damage phase, delayed elasticity would likely be a problem, leading to large permanent deformation in a relatively short time. Therefore, rest period should be long enough to allow most of delayed elasticity to recover, yet short enough to minimize healing while inducing damage. Four different rest periods were tested: 0.1, 0.2, 0.4 and 0.9 second.

Figure 3-15 shows the effect of different rest periods at a constant loading level on horizontal resilient deformation. Moreover, Figure 3-15 discloses another piece of important information: resilient deformation almost remains the same during the whole damage phase for 0.9 second rest period loading. This result implies that 0.9 second rest period allows enough time not only for delayed elasticity recovery, but also for complete healing in the material. Results indicated that the 0.4 second rest period resulted in more controllable damage development and a longer steady state damage range. A further proof of this conclusion is shown in Figure 3-16. From Figure 3-16, it can be seen that even if the loading amplitude has been almost doubled (1100 lbs to 1974 lbs), the reduction in resilient deformation did not show any significant difference.

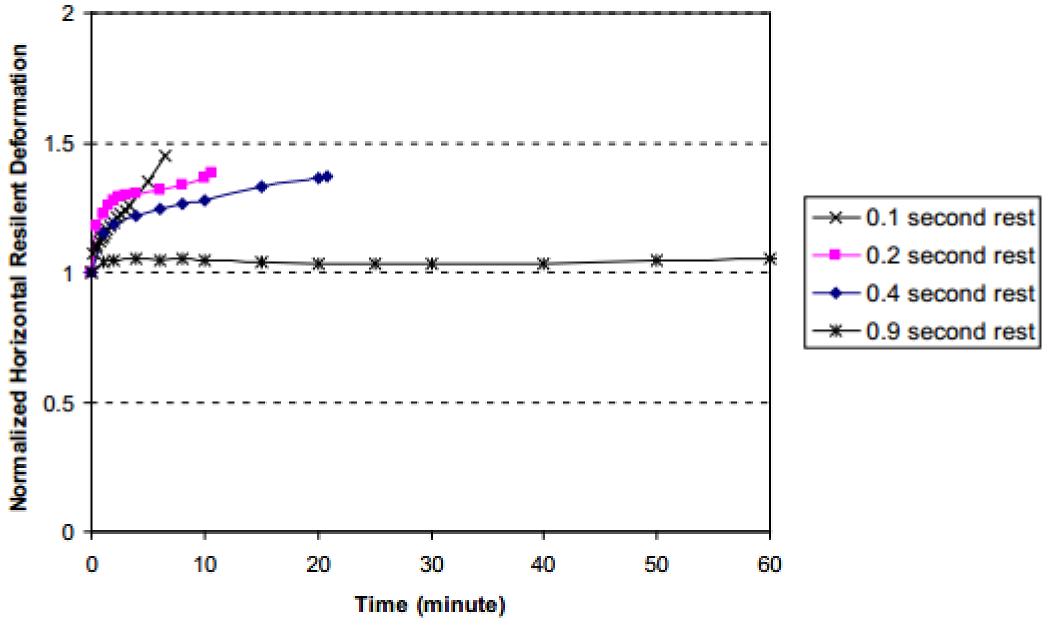


Figure 3-15 Normalized resilient deformation at different rest periods and constant load

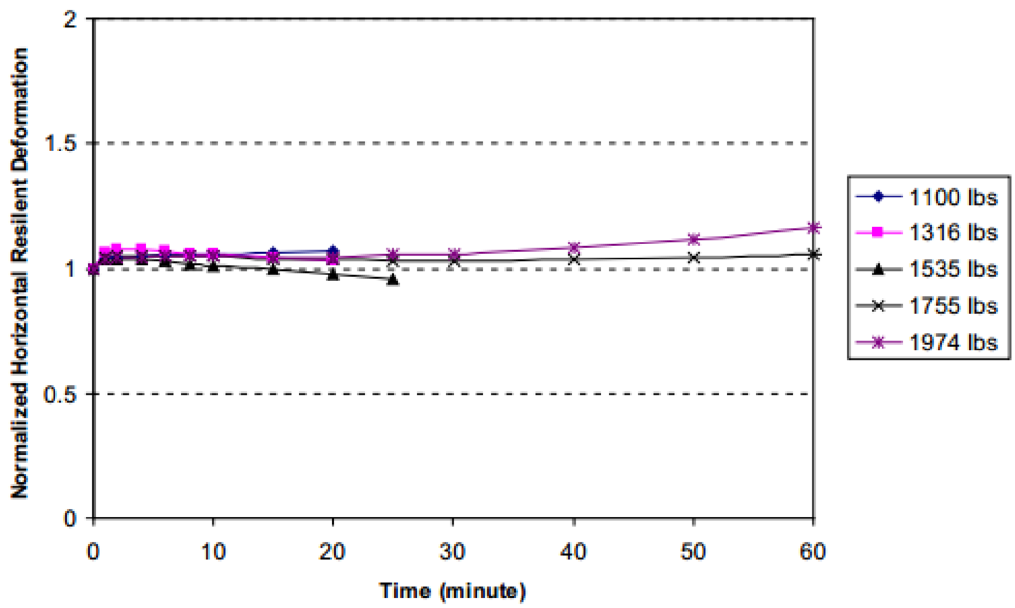


Figure 3-16 Normalized resilient deformation with different loading level at 0.9 second rest period

In addition, several potential parameters to help determine appropriate load levels for damage and healing tests were considered including failure strain (ε_f), fracture energy (FE), dissipated creep strain energy (DCSE) and strength (S_t). The percent of strength approach was selected as the most practical approach because of the consistency and simplicity of interpretation of strength test results. High loading amplitude might lead to not only nonlinear response but also excessive stress concentration at the loading strip on the IDT specimens. On the other hand, low loading amplitude might result in excessive testing time to damage asphalt mixture.

3.4.3 Modified Brittleness Index (I_{MB})

An important issue regarding RLC test is that damage criteria is mixture dependent. It means that a brittle mixture accumulates less damage before fracture compared to a ductile mixture. Thus it can be hypothesized that the appropriate load level to induce steady state damage in an asphalt mixture might depend on the brittleness of the mixture. The traditional approach used in rock mechanics is a reversible energy based approach. It defines a brittleness index (I_B) as the ratio of the specific elastic energy (S) accumulated in the material up to the point of fracture over the total specific energy (W) consumed for its deformations up to that point.

$$I_B = \frac{\text{reversible energy}}{\text{total energy}} = \frac{S}{W} \tag{3-1}$$

In the HMA fracture mechanics model developed at University of Florida, these two terms can be replaced respectively by the Elastic Energy (EE) and the Fracture Energy (FE), which have been introduced in section 4.4.

$$I_{B_{HMA}} = \frac{\text{Elastic Energy (EE)}}{\text{Fracture Energy (FE)}} \tag{3-2}$$

However, this energy based approach was not able to properly rank asphalt mixture brittleness at different temperatures. Consequently, a new approach was developed to determine brittleness index based on the observation that strength increases while failure strain decreases as the test temperature is reduced. A modified brittleness index (I_{MB}) was developed and is shown below.

$$I_{MB} = \frac{\text{Strength (kPa)}}{\text{Failure Strain } (\mu\varepsilon)} \tag{3-3}$$

Based on this definition, the asphalt mixtures can be categorized into one of the following three distinct groups (from concurrent FDOT research project BDK75 977-26). Table 3-1 shows the I_{MB} values of the three mixtures (conditioned by TTOA and tested at 10℃).

- Brittle, $I_{MB} > 2.5$
- Medium, $1.0 < I_{MB} < 2.5$
- Ductile, $I_{MB} < 1.0$

Table 3-1 I_{MB} values of the three mixtures tested (TTOA and 10 ℃)

Mixture	I_{MB}
Granite w/o lime	1.69
Granite w lime	2.39
Limestone	2.41

3.4.4 Determination of Load Level

As mentioned earlier, the percent of the ultimate load from the strength test was selected as a practical starting point to determine the appropriate loading levels. Since the specimens used in the RLC tests were dimensionally different, the strength values were corrected using the following equations.

$$P_{applied} = \frac{S_t \times \pi \times d \times t}{2 \times C_{SX}} \times A \qquad (3\text{-}4)$$

Where,

$$C_{SX} = 0.948 - 0.01114\left(\frac{t}{d}\right) - 0.2693(\nu) + 1.436\left(\frac{t}{d}\right)(\nu)$$

S_t = strength from strength test (psi)

$P_{applied}$ = load to be applied (lbf)

A = percent of strength

d = specimen diameter to be tested (in)

t = specimen thickness to be tested (in)

ν = Poisson's ratio from the strength test

It is noted that Poisson's ratio of the asphalt mixture is unknown before damage phase testing. According to previous research done by Li (2009), Poisson's ratio from strength test is similar with that obtained from the repeated loading mode. Therefore, this value was used for RLC test in this study.

In order to establish a standard RLC procedure based on the modified brittleness index (I_{MB}), a series of preliminary RLC tests were performed using different combinations of I_{MB}, load level ranges and rest periods. Based on the results of these tests, it was determined that the following conditions would likely be appropriate to induce damage in a controlled manner for RLC.

- Testing time: 30 min
- Rest period: 0.4 sec
- Loading level: Table 3-2

Table 3-2 Loading level range of each group

Group	Mixture Condition	Modified Brittleness Index (I_{MB})	Initial Loading Level
1	Ductile	$I_{MB} < 1.0$	20 % to 40 % of P_{fail}
2	Medium	$1.0 < I_{MB} < 2.5$	25 % to 45 % of P_{fail}
3	Brittle	$I_{MB} > 2.5$	30 % to 50 % of P_{fail}

Based on the preliminary damage phase test results the linear relationship showed in Figure 3-17 between loading level and I_{MB} was created to determine a starting loading level for each mixture. Then, using on a trial and error method a final load level for each mixture and aging condition was determined in order to achieve a reasonable damage level or modulus reduction (between 10 and 20 % for STOA mixtures). The relationship between loading level and I_{MB} may be further updated with the completed set of results from this study and from the concurrent FDOT project BDK75 977-26.

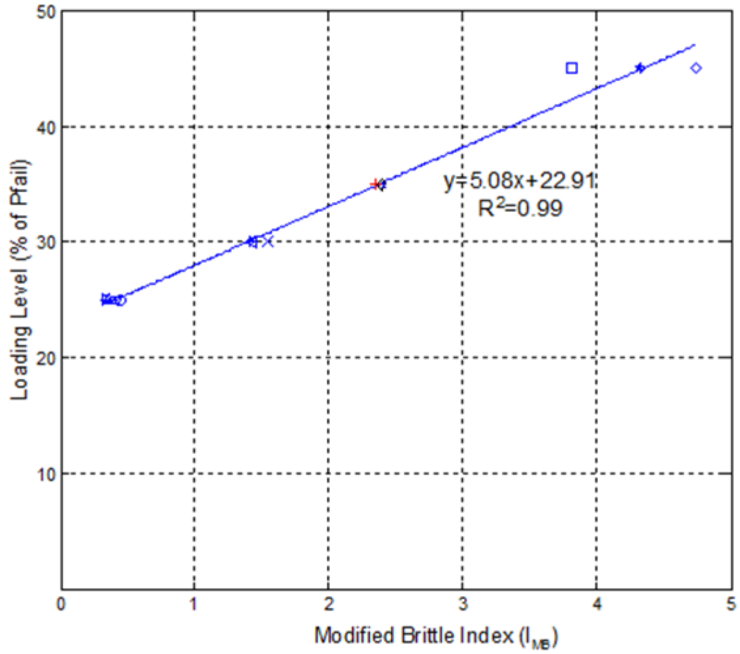


Figure 3-17 Relationship between loading level and I_{MB}

3.4.5 RLC Test Procedure

The goal of this research was to evaluate the effect of aging conditioning on change in fracture properties of asphalt mixture. Therefore, it was necessary to determine the change in fracture properties of mixtures due to load conditioning using repeated load (i.e., before and after RLC test). During the preliminary testing efforts, some issues associated with testing protocols and data interpretation method were exposed. First, the effects of the reversal of steric hardening and heat on specimens introduced during the RLC test will affect the results of creep and strength tests performed afterwards. Secondly, it was expected that healing may occur during the creep test performed after RLC test because the creep test employs a relatively low loading level (approximately 10 % of RLC loading level) and long loading time (17 minutes) compared to the RLC test. This will greatly affect test results. For this reason, it was decided not to perform creep test after RLC test. Figure 3-18 describes applied loads for RLC test and creep test after RLC test.

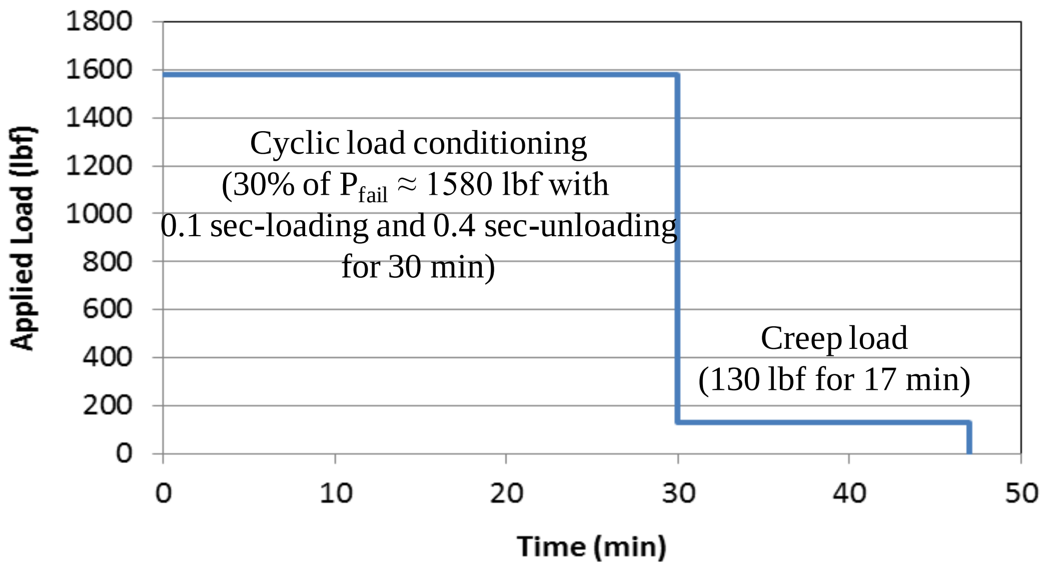


Figure 3-18 Applied load for RLC test and creep test after RLC test

In addition, it was possible to determine an effective modulus from RLC test data since RLC test was developed based on the resilient modulus test using Superpave IDT test system. Therefore, only strength test was performed after RLC test. Figure 3-19 summarizes the RLC testing protocol finalized in this study. A starting loading level of 35 % of P_{fail} was determined based on the linear relationship between loading level and I_{MB} shown in Figure 3-17. Reduction in effective modulus was used to estimate the damage induced during the RLC test. In order to develop the effective modulus reduction curve, data was collected 19 times (i.e., time at 0, 6, 13, 26, 40, 60, 80, 120, 160, 240, 320, 400, 600, 800, 1000, 1200, 1400, 1600, 1800 seconds) during the 30-minute RLC test at a frequency of 500 Hz for 5 seconds.

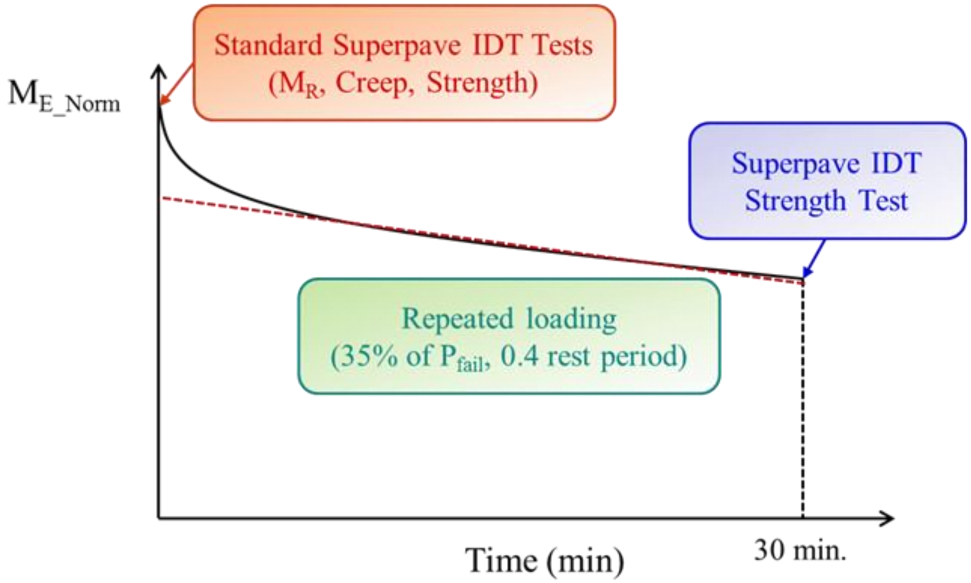


Figure 3-19 Testing protocol to determine fracture properties before and after RLC test

3.4.6 Closure on Conditioning Methods

The laboratory conditioning process (RLC test system) to appropriately simulate the effect of load damage was identified and developed in this study. The repeated loading approach was selected to induce damage in asphalt mixtures using Superpave IDT system. RLC tests were performed using the testing protocols introduced in this chapter.

3.5 Closure

Laboratory conditioning and testing protocols to effectively characterize the aging procedure in the field condition were established by including the effect of heat oxidation, moisture, and load that may be capable of aging mixtures to levels consistent with those observed in the field. The current LTOA method, including TTOA, CPPC, and RLC systems, were employed in this study. All specimens were conditioned and tested using these conditioning procedures for evaluation.

CHAPTER 4
MATERIALS AND LABORATORY MIXTURE TESTS

4.1 Introduction

An experimental test plan for different types of asphalt mixture was designed to evaluate the effects of laboratory conditioning including heat oxidation, moisture conditioning, and load conditioning. The conditioning and testing protocols established in Chapter 3 was used to evaluate these effects on fracture properties for three types of asphalt mixtures: granite w/o lime, granite w/ lime, and limestone mixtures. All mixtures were fine, dense-graded mixtures and tests were performed at 10 ℃.

Superpave IDT test was used throughout this research. The following conditioning sequence was considered for evaluation and Figure 4-1 shows the flowchart for the overall testing plan. In total, five combinations of conditioning methods were used as listed below.

- Heat oxidation only
- Heat oxidation + load conditioning
- Heat oxidation + moisture conditioning (CPPC + dry)
- Heat oxidation + moisture conditioning (CPPC + wet)
- Heat oxidation + moisture conditioning (CPPC + wet) + load conditioning

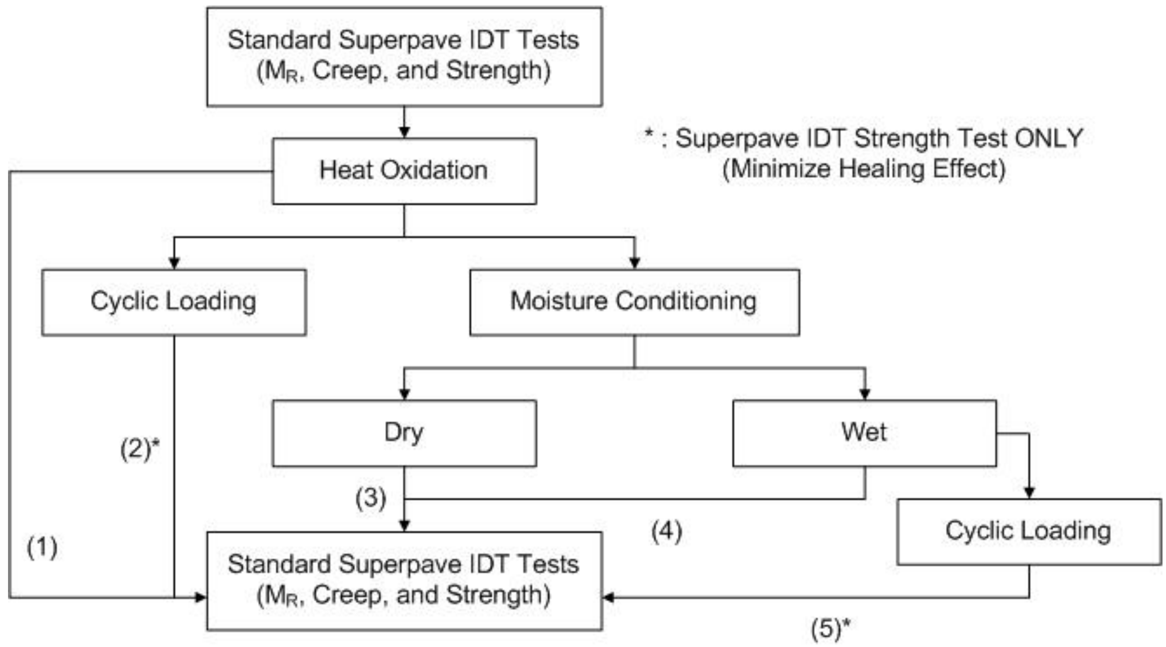


Figure 4-1 Overall testing plan

Note: (1) Heat oxidation effect, (2) Heat oxidation and cyclic load effect, (3) Recovery effect from moisture conditioning, (4) Moisture conditioning effect, and (5) Heat oxidation, moisture conditioning, and cyclic load effect

4.2 Materials

This section includes information regarding the materials used to produce laboratory asphalt mixture specimens for this study. Two aggregate types were used in this study: Georgia granite and Florida Rinker limestone. These aggregates have been widely used in the state of Florida and are approved by the FDOT for road construction and rehabilitation projects. To reduce asphalt binder effect, the same type of binder (PG 67-22) was used for all mixtures.

In this study, one additional granite mixture (granite w/ lime) has been added to mixture testing plan. This mixture incorporates hydrated lime into granite aggregate to evaluate effects of the lime on HMA fracture performance. For this new mixture, one of the granite mixtures was modified by replacing 1 % of the -200 fraction with dry hydrated lime. This incorporates the lime into the mixture without changing the original aggregate gradation. In addition, no change was made in the binder content, which is 4.8 % as determined in the original mix design for the granite mixture. A detailed batching sheet for the granite (with and without lime) and limestone mixtures are included in Tables 4-1 and 4-2, respectively.

Table 4-1 Batching sheet for granite mixture without and with lime

Sieve Size	Retained Weight, g			
	#78 Stone	#89 Stone	W-10 Screenings	Local Sand
3/4”	0.0	0.0	0.0	0.0
1/2”	44.6	0.0	0.0	0.0
3/8”	564.3	0.9	0.0	0.0
#4	742.5	219.6	0.0	0.0
#8	74.3	81.9	675.0	0.0
#16	29.7	6.3	630.0	0.0
#30	0.0	3.1	382.5	27.0
#50	14.8	0.0	202.5	184.5
#100	0.0	0.0	135.0	189.0
#200	0.0	0.0	67.5	36.0
Pan	14.9	3.2	157.5 (W-10)[1)] 112.5 (W-10) + 45.0 (lime) = 157.5[2)]	13.5
Sum	1485.0	315.0	2250.0	450.0

Note: 1) case of granite without lime, 2) case of granite with lime.

Table 4-2 Batching sheet for limestone mixture

Sieve Size	Retained Weight, g			
	#67 Stone	S-1-B	Med. Screening	Local Sand
3/4”	0.0	0.0	0.0	0.0
1/2”	144.0	0.0	0.0	0.0
3/8”	135.0	182.3	0.0	0.0
#4	144.0	1012.5	0.0	0.0
#8	9.0	668.3	207.9	0.0
#16	4.5	81.0	415.8	0.0
#30	0.0	20.3	340.2	8.1
#50	0.0	0.0	264.6	55.3
#100	0.0	0.0	396.9	56.7
#200	2.3	10.1	192.8	10.8
Pan	11.3	50.6	71.8	4.1
Sum	450.0	2025.0	1890.0	135.0

Detailed information regarding aggregate sources for both granite and limestone used in this study are shown in Table 4-3.

Table 4-3 Aggregate sources

Source	Type of Material	FDOT Code	Pit No.	Producer
Georgia Granite	# 78 Stone	43	GA-553	Junction City Mining
	# 89 Stone	51	GA-553	Junction City Mining
	W-10 Screenings	20	GA-553	Junction City Mining
Florida Rinker Limestone	# 67 Stone	42	87-090	Rinker Materials Corp.
	S-1-B	55	87-090	Rinker Materials Corp.
	Med. Screening	21	87-090	Rinker Materials Corp.
Local Sand	Local Sand	-	Starvation Hill	V. E. Whitehurst & Sons

All mixtures were designed with 12.5 mm nominal maximum aggregate size gradations and a traffic level C, which corresponds to 3 to 10 million Equivalent Single Axle Loads (ESALs) over 20 years, were employed based on Superpave system. Figure 4-2 represents the gradation chart of granite and limestone mixture including restricted zone and control points. Detailed gradation information and blend proportions are included in Appendix A.

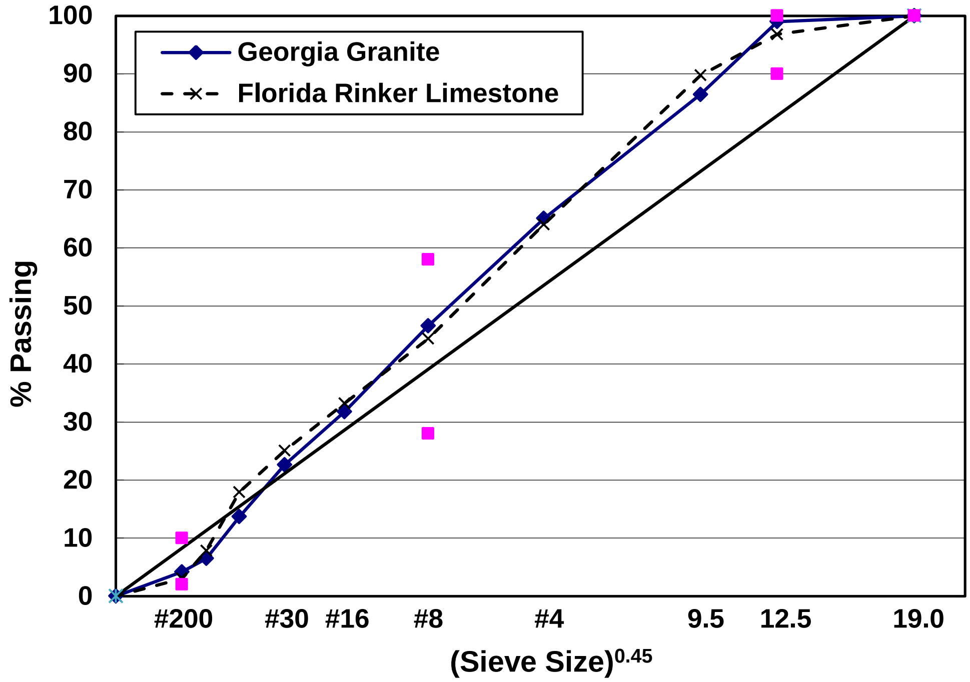


Figure 4-2 Asphalt mixture gradations

4.3 Test Specimen Preparation

Specimens were prepared for laboratory testing using the materials described in section 4.2. A total of 135 specimens for Superpave IDT tests were produced according to testing plan. Table 4-4 summarizes the number of samples for mixture testing. As introduced, three types of asphalt mixtures were produced for both standard Superpave IDT test and cyclic load test. All mixtures produced for testing were designed with the Superpave mix design procedure which is based upon the selection of design asphalt content for a set of criteria regarding the volumetric properties of the mixture at 4 % air voids. In this study, design asphalt contents were determined as 4.8 % for granite mixture and 6.6 % for limestone mixture, respectively.

Table 4-4 Number of specimens for mixture testing

Test Temperature	Condition	Mixture Type	Aging	Number of Specimen
10 ℃	Heat Oxidation	Granite w/o Lime	STOA	3
			LTOA	3
			TTOA	3
		Granite w/ Lime	STOA	3
			LTOA	3
			TTOA	3
		Limestone	STOA	3
			LTOA	3
			TTOA	3
	Heat Oxidation + Load Conditioning	Granite w/o Lime	STOA	3
			LTOA	3
			TTOA	3
		Granite w/ Lime	STOA	3
			LTOA	3
			TTOA	3
		Limestone	STOA	3
			LTOA	3
			TTOA	3
	Heat Oxidation + Moisture Conditioning (Wet Condition)	Granite w/o Lime	STOA	3
			LTOA	3
			TTOA	3
		Granite w/ Lime	STOA	3
			LTOA	3
			TTOA	3
		Limestone	STOA	3
			LTOA	3
			TTOA	3
	Heat Oxidation + Moisture Conditioning (Dry Condition)	Granite w/o Lime	STOA	3
			LTOA	3
			TTOA	3
		Granite w/ Lime	STOA	3
			LTOA	3
			TTOA	3
		Limestone	STOA	3
			LTOA	3
			TTOA	3
	Heat Oxidation + Moisture Conditioning + Load Conditioning	Granite w/o Lime	STOA	3
			LTOA	3
			TTOA	3
		Granite w/ Lime	STOA	3
			LTOA	3
			TTOA	3
		Limestone	STOA	3
			LTOA	3
			TTOA	3
Total Number of Specimen				135

4.3.1 Batching and Mixing

The 4500 g batched samples were heated in the oven at the mixing temperature (300-315 ° F) for approximately 3 hours. The tools and asphalt binder used for mixing were also heated at the mixing temperature. The aggregates and asphalt binder were then removed from the oven and mixed until all aggregates were well coated with asphalt binder (approximately 3-5 minutes). Figure 4-3 shows a laboratory mixer used for mixing.

Figure 4-3 Mixer used for mixing

The mixed samples were then spread out in pans and heated in an oven for 2 hours at a temperature of 300-315 °F which is the same temperature as mixing for short-term oven aging (STOA). Each of the mixtures was stirred after 1 hour to obtain uniformly aged samples. In addition to the STOA condition, two more oxidative aging levels, i.e., a 5-day long-term oven aging (LTOA) and a 10-day long-term oven aging (TTOA), were used for evaluation.

4.3.2 Compaction

After short-term oven aging, the 4500 g samples were then removed from the oven and quickly compacted using the Superpave Gyratory Compactor. Figure 4-4 shows the Servopac Model of Superpave Gyratory Compactor used in this study.

Figure 4-4 Superpave Servopac Gyratory Compactor

The samples were compacted with a compaction stress of 600 kPa at a gyratory external angle of 1.25°. All mixtures were compacted at 310 ± 5 °F. Although mixtures were designed to have 4 % air voids at N_{design}, they were compacted to the number of gyrations required for obtaining 7 % air voids for cut specimens. Seven % air voids is recommended to more accurately simulate air void conditions of asphalt mixture immediately after construction. In addition, it is conservative to use 7% air voids as this will result in the worst case scenario in terms of age hardening. Higher air voids lead to more air circulation inside of the specimen and thus, more age hardening. The number of gyrations attained based on mix design to get 7 % air voids were 22 for limestone and 27 for granite mixtures. The 150 mm diameter pills were compacted using the Superpave gyratory compactor and they were then allowed to fully cool down before being extruded from the mold.

4.3.3 Measuring Air Voids

After pills were completely cooled down, the bulk specific gravity (G_{mb}) of each compacted pill was measured and the percent air voids for each pill was calculated. The target air voids of the gyratory pill was 7.5 % since the air voids of the cut specimen obtained from the middle of the pill is generally 0.5 % less than the air voids of the compacted pill. After cutting procedure (see section 4.3.4), the bulk specific gravity test was conducted on each cut specimen and air voids were calculated using the G_{mb} and maximum specific gravity (G_{mm}). Air voids of cut specimens had to be in the range of 7 ± 0.5 % to be considered for mixture testing. Specimens were then placed in the dehumidifier for at least two days (24 hours) to prevent moisture effect on testing. All cut specimens were then grouped based on air voids measured for each test including standard Superpave IDT test and cyclic load test.

4.3.4 Cutting

Once air voids of compacted pills were properly checked and logged, all pills were sliced to obtain test specimens. A cutting device, which has a cutting saw and a special attachment to hold the pills, was used to slice the pills into test specimens of desired thickness (1.5 inch). Since the saw uses water to keep the blade wet, the specimens were dried at least one day (24 hours) at room temperature prior to further testing. Figure 4-5 represents the cutting machine used in this study.

Figure 4-5 Cutting machine used in this research

4.3.5 Gauge Points Attachment

Gauge points were attached to both faces of the specimens using a steel template, a vacuum pump setup, and a strong adhesive. Four gauge points (with a 5/16 inch diameter and a 1/8 inch thickness) were placed with epoxy on each side of the specimens at a distance of 19 mm (0.75 inch) from the center, along the vertical and horizontal axes. Next, the loading axis was marked on the specimens. This procedure helped for assisted in the placement of specimen in the testing chamber for proper loading of the specimen. Figures 4-6 and 4-7 show the marking of the loading axis using a steel plate and attaching gauge points, respectively.

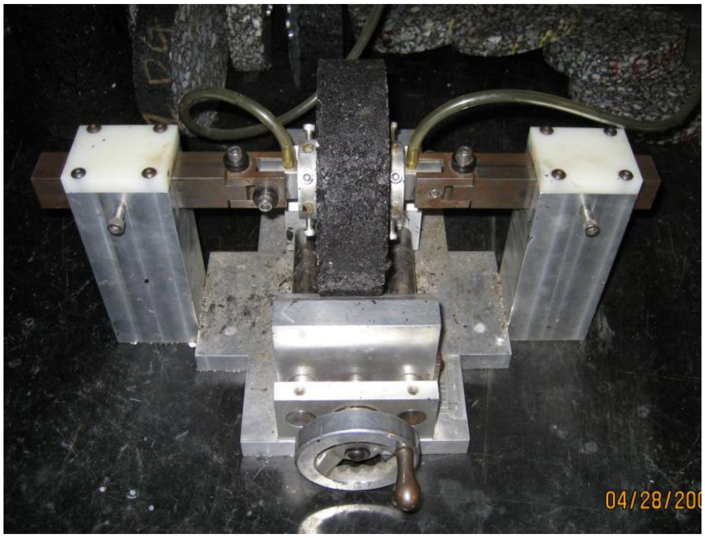


Figure 4-6 Gauge points attachment

Figure 4-7 Marking the loading axis

4.4 Superpave IDT Test Procedure

Two types of mixture test were included in the testing plan, i.e., the standard Superpave IDT test to determine asphalt mixture properties and the repeated load conditioning (RLC) test to induce and measure microdamage in asphalt mixtures. Test procedure for Superpave IDT test is introduced in this section, and RLC test procedure is included as part of Chapter 3 (see Section 3.4). One complete set of Superpave IDT tests consists of resilient modulus, creep compliance and strength tests. This series of tests provides the fracture properties necessary to identify the effects of laboratory heating, cyclic pore pressure and cyclic loading.

4.4.1 Resilient Modulus Test

Resilient modulus test is a nondestructive test used to determine the resilient modulus (Mr) of asphalt mixtures. Resilient modulus is defined as the ratio of the applied stress over the recoverable strain when a repeated load is applied. The test was performed according to the system developed by Roque et al. (1997) to determine the resilient modulus and Poisson's ratio. A haversine waveform load is applied to the specimen for 0.1 second followed by a rest period of 0.9 seconds for a total of 5 cycles. The load was appropriately selected in order to keep the horizontal resilient deformations within the linear viscoelastic range (a typical range for horizontal deformations is 100 to 180 micro-inches). Figure 4-8 describes the haversine load applied and typical deformation response for resilient modulus test.

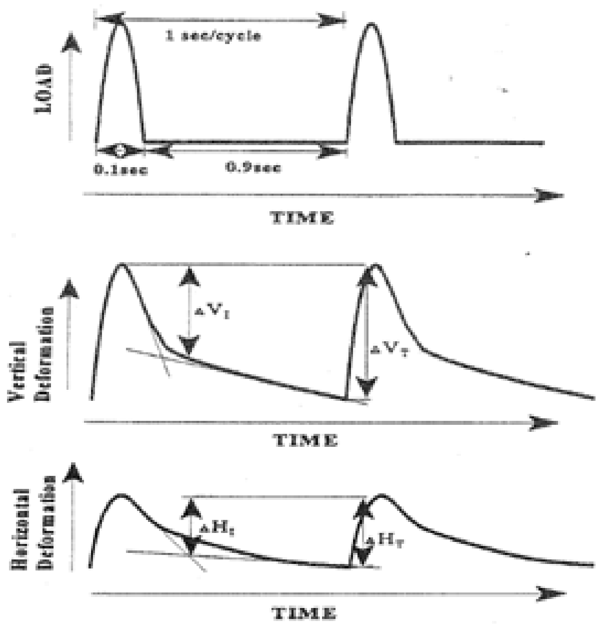


Figure 4-8 Haversine load applied and typical deformation response for resilient modulus test

Roque and Buttlar (1992) developed the following equations to calculate the resilient modulus and the Poisson's ratio. These equations are incorporated into the Superpave Indirect Tension Test at Low Temperatures (ITLT) computer program developed by Roque et al. (1997).

$$M_R = \frac{P \times GL}{\Delta H \times t \times D \times C_{cmpl}} \tag{4-1}$$

$$\upsilon = -0.1 + 1.480 \times (X/Y)^2 - 0.778 \times (t/D)^2 \times (X/Y)^2 \tag{4-2}$$

Where,

M_R = resilient modulus

P = maximum load

GL = gauge length

ΔH = horizontal deformation

t = thickness

D = diameter

$C_{cmpl} = 0.6354 \times (X/Y)^{-1} - 0.332$

ν = Poisson's ratio

(X/Y) = ratio of horizontal to vertical deformation

4.4.2 Creep Test

Creep test is a nondestructive test used to determine the creep compliance and associated parameters. Creep compliance is defined as the ratio of the time-dependent strain over stress. Since it well represents the time-dependent behavior of asphalt concrete, it has been usually used to evaluate the rate of damage accumulation of asphalt mixture. Creep test was performed in a load controlled mode by applying a static load in the form of a step function to the specimen and then holding it for 1000 seconds. The magnitude of the load is appropriately selected in order to maintain the accumulated horizontal deformations in the linear viscoelastic range, which is below the total horizontal deformation of 750 micro-inches. During the first 100 seconds of test, a horizontal deformation of no greater than 100 to 130 micro-inches is generally considered to be acceptable to keep the maximum horizontal deformation below 750 micro-inches.

As shown in Figure 4-9, D_0, D_1, and m-value are creep parameters obtained from creep test. Although D_1 and m-value are related to each other, D_1 is more related to the early portion of creep compliance curve, while m-value is more associated with the later portion of the creep compliance curve.

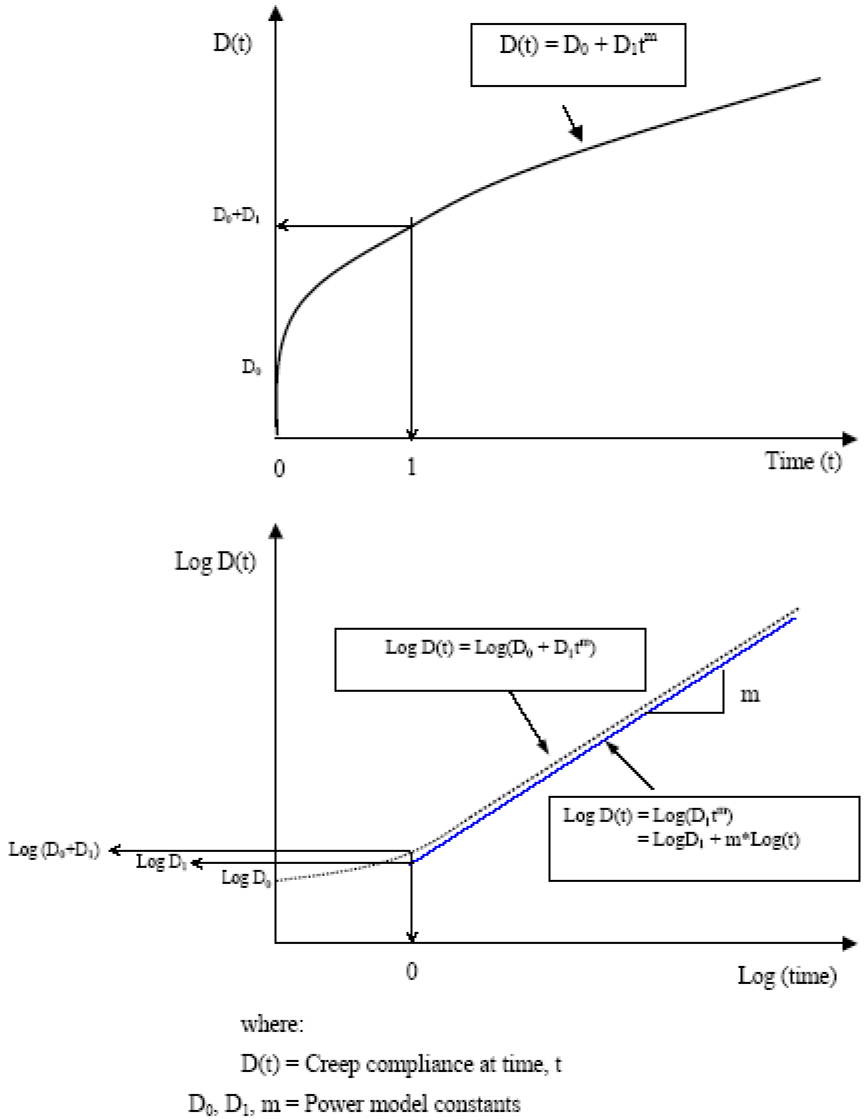


Figure 4-9 Power model of creep compliance

Creep properties of the mixture were determined using the Superpave Indirect Tension test at Low temperatures (ITLT) computer program by analyzing the load and deformation data. The program uses the following equations to compute creep compliance and Poisson's ratio.

$$D(t) = \frac{\Delta H \times t \times D \times C_{cmpl}}{P \times GL} \tag{4-3}$$

$$\upsilon = -0.1 + 1.480 \times (X/Y)^2 - 0.778 \times (t/D)^2 \times (X/Y)^2 \qquad (4\text{-}4)$$

Where,

D(t) = creep compliance at time t (1/psi)

ΔH, t, D, C_{cmpl}, GL, ν, P and (X/Y) are the same as described previously.

4.4.3 Tensile Strength Test

Strength test is a destructive test used to determine the failure limits of the asphalt mixture. These properties, including tensile strength, failure strain and fracture energy can be used to estimate the cracking resistance of the asphalt mixture. The test is performed in a displacement controlled mode by applying a constant rate of displacement of 50 mm/min until failure. Tensile strength can be calculated using the following equation.

$$S_t = \frac{2 \times P \times C_{sx}}{\pi \times t \times D} \qquad (4\text{-}5)$$

Where,

S_t = maximum indirect tensile strength

P = failure load at first crack

$C_{sx} = 0.948 - 0.01114\left(\frac{t}{D}\right) - 0.2693(\nu) + 1.436\left(\frac{t}{D}\right)(\nu)$

t = thickness

D = diameter

ν = Poisson’s ratio

From strength test and resilient modulus test, following relationship can be developed.

$$M_R = \frac{S_t}{\varepsilon_f - \varepsilon_0} \quad \rightarrow \quad \varepsilon_0 = \frac{M_R \varepsilon_f - S_t}{M_R} \tag{4-6}$$

Fracture energy (FE), which is the total energy applied to the specimen until it fractures, is determined as the area underneath the stress-strain curve until failure. Dissipated creep strain energy (DCSE) is the absorbed energy that damages the specimen, and dissipated creep strain energy to failure is the absorbed energy to fracture ($DCSE_f$). As shown in Figure 4-10, FE and $DCSE_f$ can be determined as shown below. The ITLT program calculates FE and $DCSE_f$ automatically.

$$\text{Fracture Energy (FE)} = \int_0^{\varepsilon_f} S(\varepsilon)d\varepsilon \tag{4-7}$$

$$\text{Elastic Energy (EE)} = \frac{1}{2} S_t(\varepsilon_f - \varepsilon_0) \tag{4-8}$$

$$\text{Dissipated Creep Strain Energy } (DCSE_f) = FE - EE \tag{4-9}$$

Where,

S_t = Tensile strength

ε_f = Failure strain

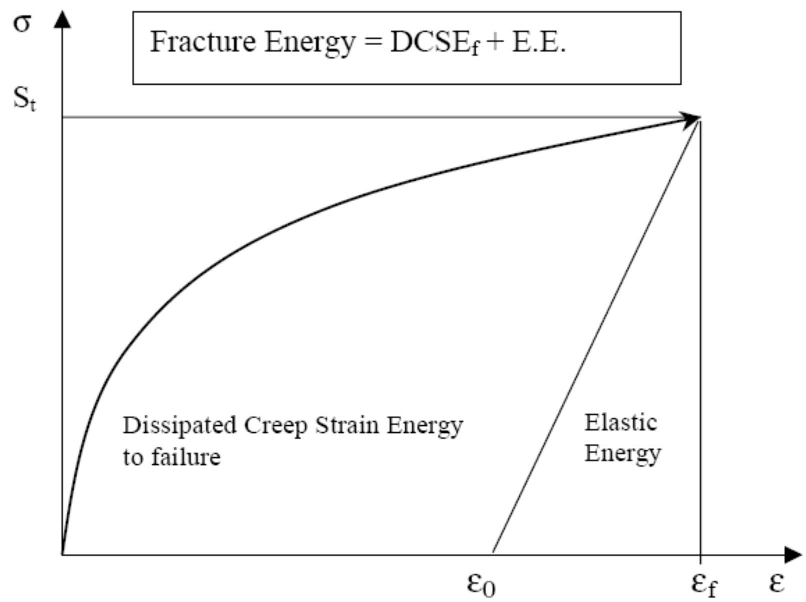


Figure 4-10 Determination of FE and $DCSE_f$

The ITLT program includes also the calculation of the Energy Ratio (ER). This parameter developed by Roque et al. (2004) represents the asphalt mixture's potential for top-down cracking. ER allows the evaluation of cracking performance on different pavement structures by incorporating the effects of mixture properties and pavement structural characteristics. Energy ratio is computed using the following formula.

$$\mathrm{ER} = \frac{\mathrm{DCSE_f}}{\mathrm{DCSE_{min}}} = \frac{\mathrm{a} \times \mathrm{DCSE_f}}{\mathrm{m}^{2.98} \times \mathrm{D_1}} \tag{4-10}$$

Where,

$DCSE_f$ = dissipated creep strain energy to failure (kJ/m^3)

$DCSE_{min}$ = minimum dissipated creep strain energy for adequate cracking performance (kJ/m^3)

D_1 and m = creep parameters

$$a = 0.0299\sigma^{-3.1} \times (6.36 - \mathrm{S_t}) + 2.46 \times 10^{-8}$$

σ = tensile stress of asphalt layer (psi)

S_t = tensile strength (MPa)

4.5 Closure

Mixture tests including standard Superpave IDT and RLC test were performed using specimens conditioned based on testing plan for different types of asphalt mixtures. Test results were analyzed to evaluate the effects of laboratory conditioning on change in fracture properties. This information was also used to evaluate and validate conditioning procedures established in this study, including heat oxidation, CPPC, and RLC, which were expected to effectively simulate asphalt mixture aging that actually occur in the field.

CHAPTER 5
EFFECTS OF LABORATORY CONDITIONING ON FRACTURE PROPERTIES OF ASPHALT MIXTURE

5.1 Introduction

Asphalt mixture test results for different conditioning were evaluated to determine the relative effects of oxidation, CPPC, and loading on mixture aging. An attempt was made to assess the implications of the results on the probable mechanisms of field aging, as well as on the potentially most effective laboratory aging conditioning system. Careful analyses were performed on test results with respect to different conditioning procedures and types of asphalt mixture designed to evaluate their effects on change in fracture properties. It is noted that “HOC” denotes “Heat Oxidation,” “CPPC” denotes “Cyclic Pore Pressure Conditioning,” and “RLC” denotes “Repeated Load Conditioning.”

5.2 Issues Associated with RLC Tests

5.2.1 Introduction

The purpose of the repeated load conditioning (RLC) test was to induce damage in order to simulate the effects of traffic load. This method was developed based on the Superpave indirect tension test (IDT) which is suitable for testing laboratory-compacted specimens and field cores. Therefore, cylindrical specimens with a diameter of 6 inches and a thickness of 1.5 inches were used for both untreated (Granite w/o lime) and lime-treated (Granite w/ lime) mixtures.

Based on extensive research explained in Chapter 3, test procedures can be summarized as follow:

- Haversine load with a 0.1-second load period and a 0.4-second rest period
- Testing time: 30 minutes
- Test temperature: 10 °C
- Sitting load: 10 lb
- Load level determined by a trial and error approach using a linear relationship between the percentage of P_{max} and I_{MB} as a starting point
- Tensile strength test run immediately after the RLC

5.2.2 Repeated Load Conditioning after Heat Oxidation

As mentioned earlier in this report, for the purpose of this research, accumulation of damage is related to the reduction in effective modulus (M_E). Therefore, during the RLC test, nineteen resilient modulus tests were conducted. For each of the tests, the resulting data was collected at a frequency of 500 Hz for 5 seconds. In total, nineteen sets of recorded data points were used to determine the reduction in effective modulus over time.

The resilient modulus test data from RLC test was analyzed using the Superpave Indirect Tension Test at Low Temperatures (ITLT) computer program developed by Roque et al. (1997) and a Matlab code specifically developed by the researchers for this project to obtain the effective modulus at each test time. Both programs led to comparable results.

Each load was appropriately selected in order to keep the horizontal deformations within the linear viscoelastic range (steady state damage) and to achieve a reasonable amount of damage (between 10 and 30 percent reduction in normalized resilient modulus). The reason was to avoid the tertiary stage where the permanent deformation begins to accelerate leading towards crack formation and failure.

Figure 5-1 presents test results of granite w/o lime specimen after TTOA with a 35 % loading level. As can be seen, the linear portion of the reduction curve of normalized effective modulus (with respect to its initial value) was fitted by linear regression to determine the damage rate as its slope. The starting point of the regression line was determined by comparing the absolute differences "d_i" between consecutive slopes "m_i" of the test data (see Figure 5-2). When three consecutive "d" values are less than 0.0025 (the threshold), then the regression line starts. This threshold was determined based on observations from all RLC test results. A typical set of results for calculated "d" values over time is presented in Figure 5-3. As shown, after certain time when all "d" values are beneath the absolute value 0.0025, the starting point of the regression line is identified, indicating the beginning of steady state.

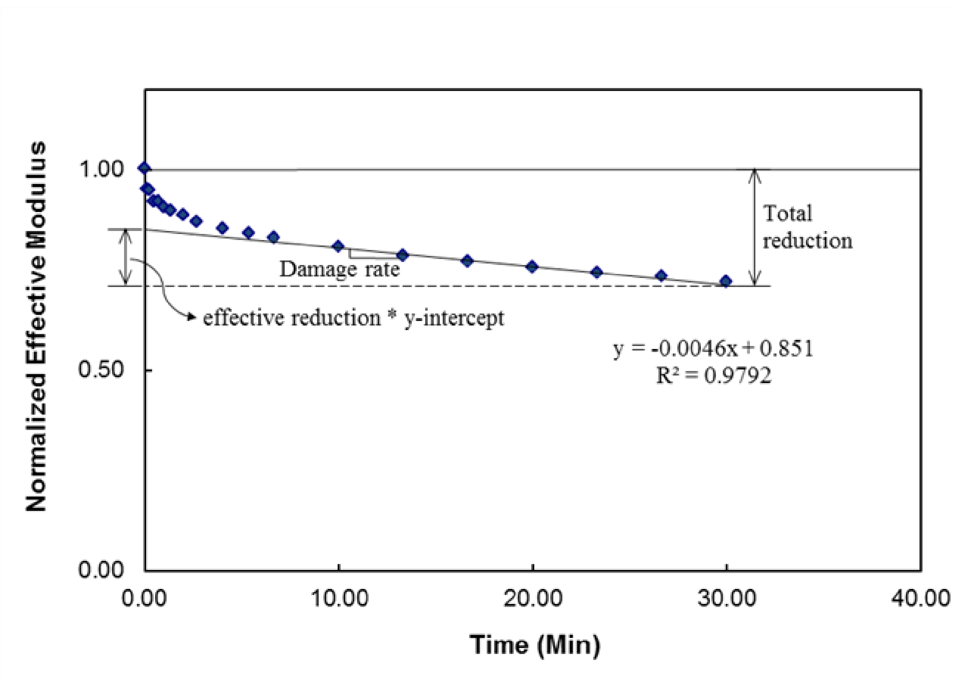


Figure 5-1 RLC test result (TTOA, granite w/o lime)

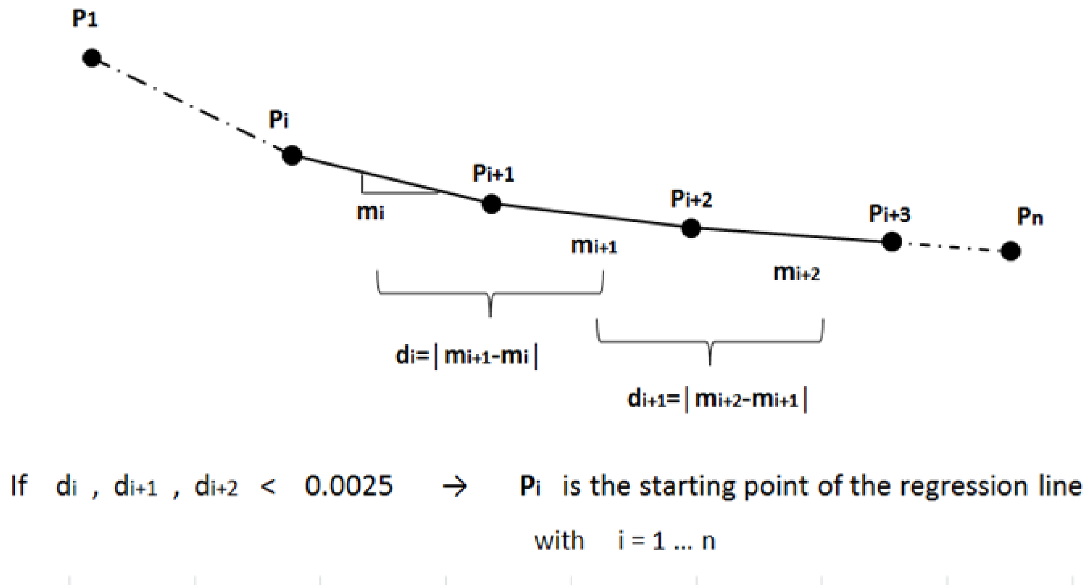


Figure 5-2 Approach used to determine the starting point of the regression line

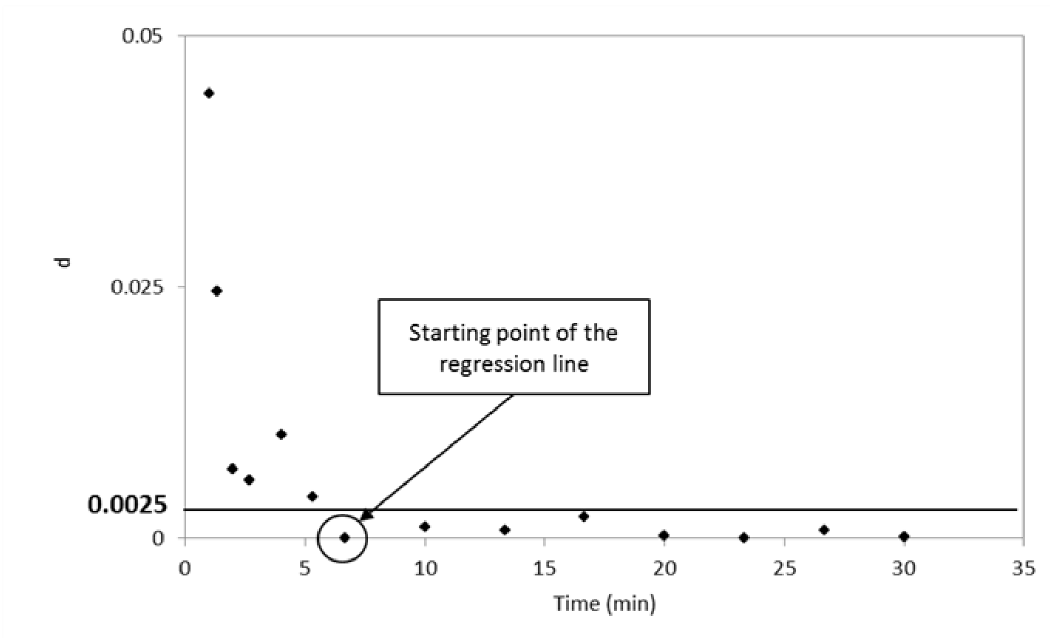


Figure 5-3 Threshold for determination of the starting point of the linear regression

As mentioned earlier, each effective modulus curve was normalized using the initial effective modulus. Effective reduction in normalized resilient modulus was determined as the ratio of the difference between the initial effective modulus (y intercept) and the effective modulus at the end of the test (@ t = 30 min) over the initial effective modulus.

Three parameters were used to characterize the curve (see Figure 5-1):

- Total reduction = (1 – normalized effective modulus @ t = 30 min) $\times$ 100 %
- $\text{Effective reduction} = \dfrac{\text{y intercept} - M_E\ (@\ t = 30\text{min})}{\text{y intercept}} \times 100$

  where, y intercept = y intercept of the regression line, M_E = effective modulus

- Damage rate = absolute value of the slope of the regression line

5.2.2.1 Load Level Threshold Affected by Heat Oxidation

The effects of repeated load without moisture damage at three oxidative aging conditions (STOA, LTOA and TTOA) were evaluated for both mixtures through a total of eighteen specimens (three replicates per oxidative aging condition). The RLC successfully induced damage in a controlled manner. Figures 5-4 through 5-6 show the normalized effective modulus reduction of granite w/o lime for three oxidative aging conditions, respectively.

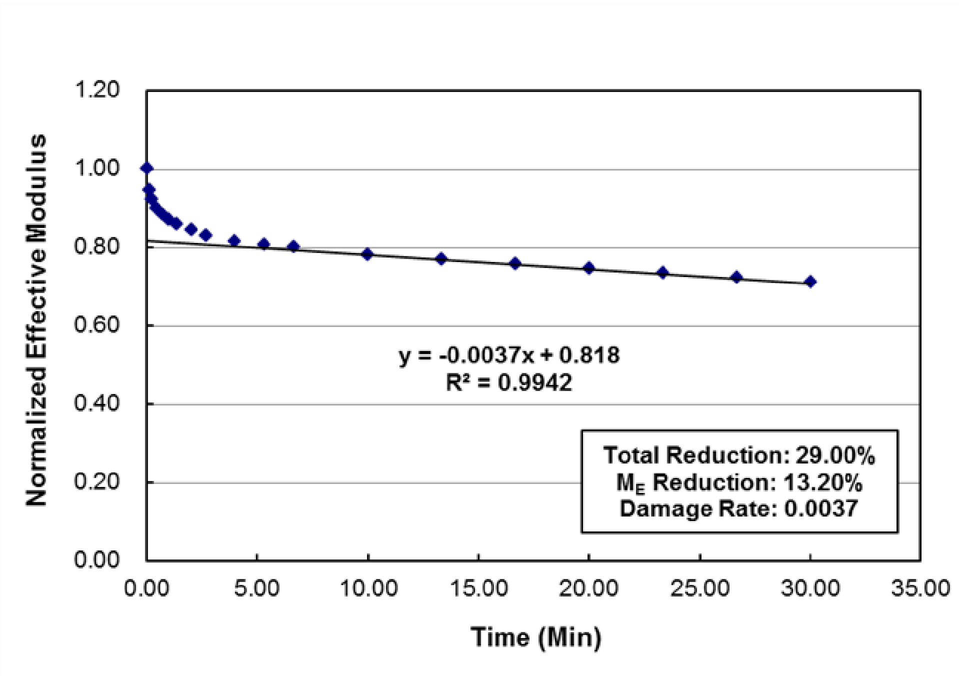


Figure 5-4 Normalized effective modulus reduction for STOA granite w/o lime with 45 % loading level

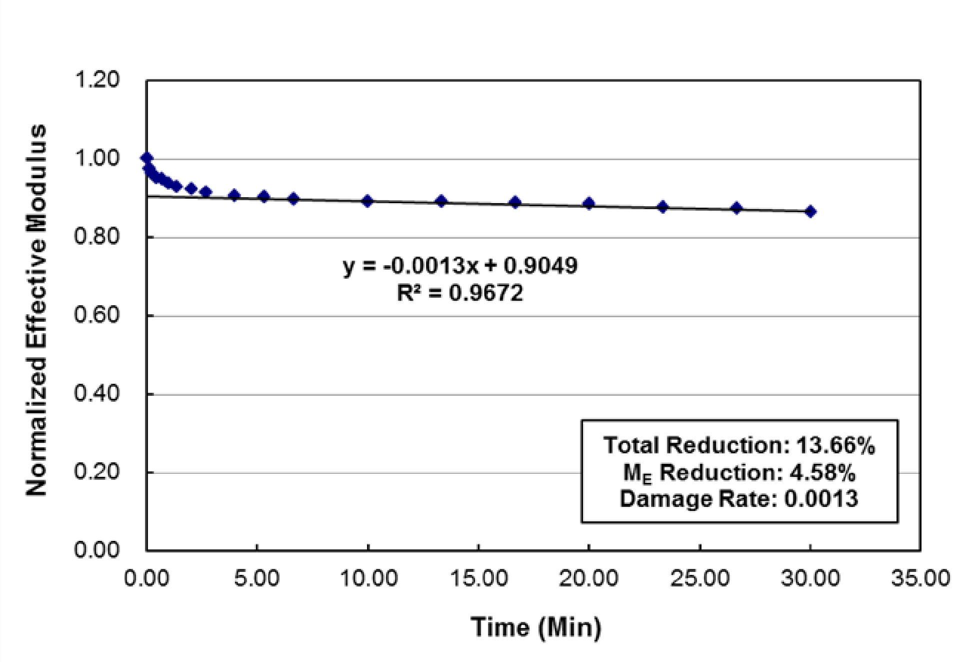


Figure 5-5 Normalized effective modulus reduction for LTOA granite w/o lime with 40 % loading level

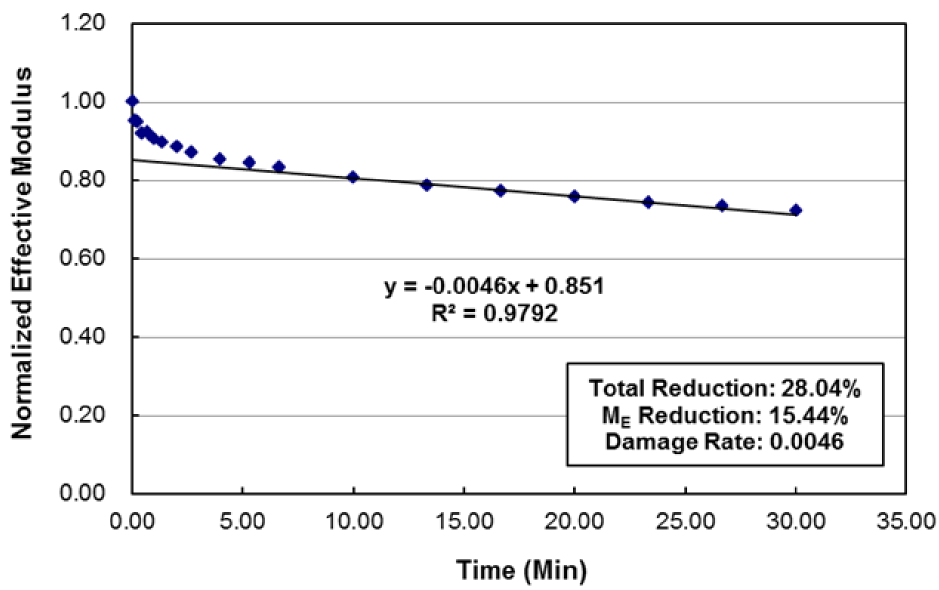
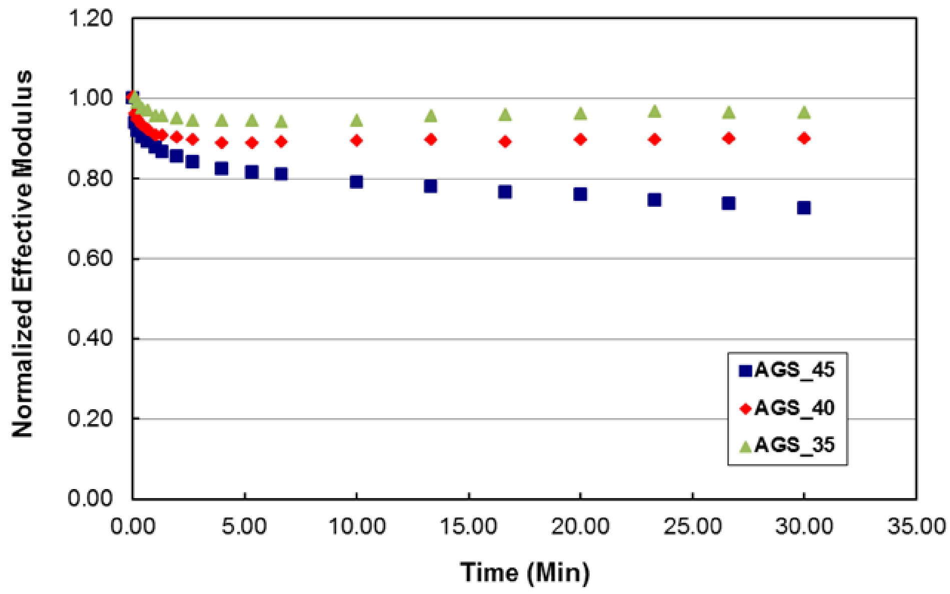
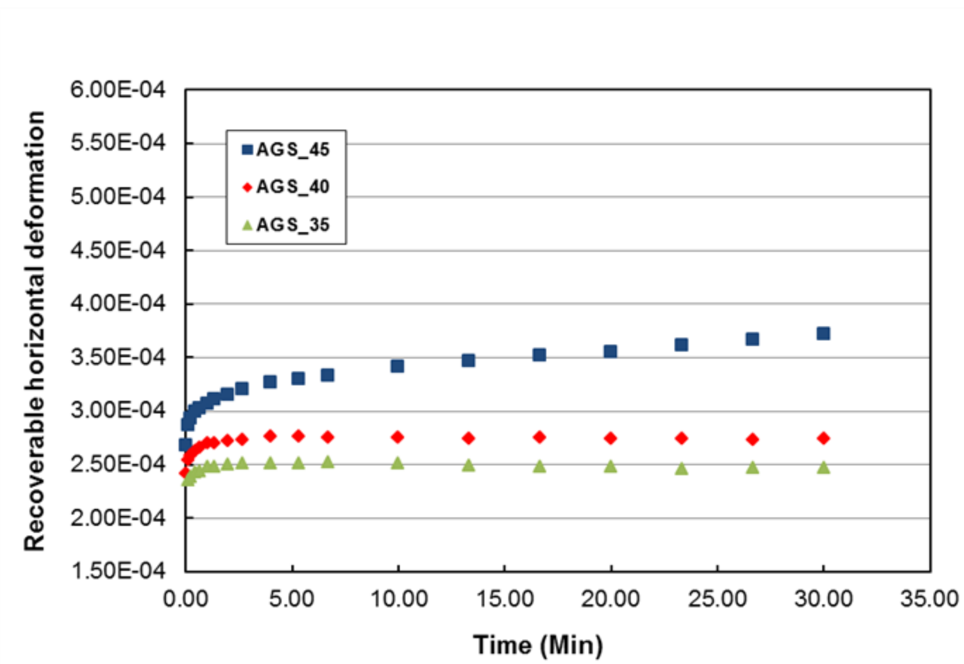


Figure 5-6 Normalized effective modulus reduction for TTOA granite w/o lime with 35 % loading level

However, the test results revealed some issues with respect to the testing procedure, in particular with the trial and error process for the determination of the proper load level. Results showed that material properties were excessively sensitive to load level, especially as the oxidation level increases, e.g., from STOA (Figure 5-7) to LTOA (Figure 5-8). It was impossible to induce damage in a controlled manner and concurrently maintain the same loading level as the oxidation level increases. Therefore, the load level needed to achieve a reasonable amount of damage was reduced from 45 % of failure load (from strength test) for STOA, to 40 % for LTOA and 35 % for TTOA. In other words, the more the asphalt mixture is oxidized, the lower the load level required to induce microdamage in a controlled fashion.

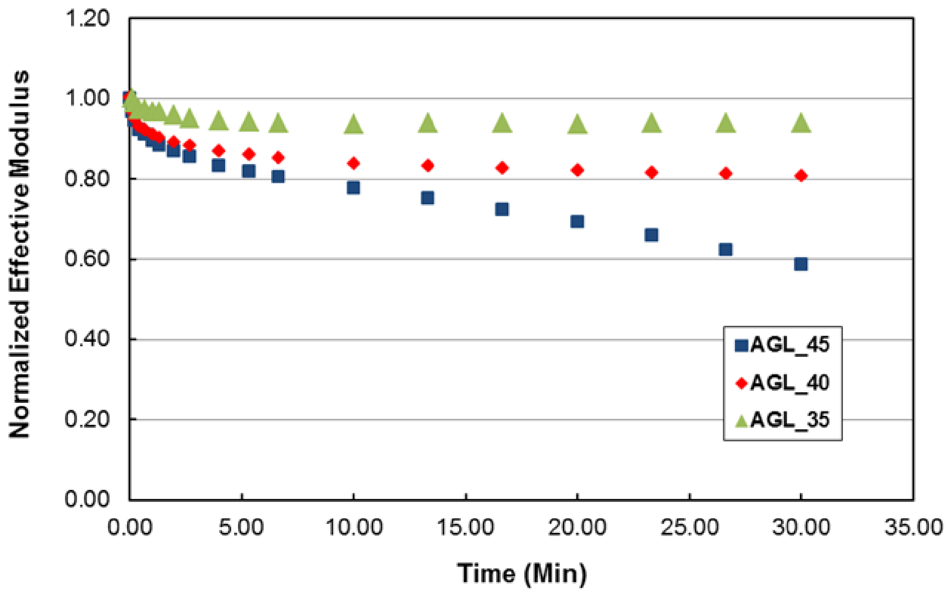


(5-7a) Sensitivity of effective modulus to loading level

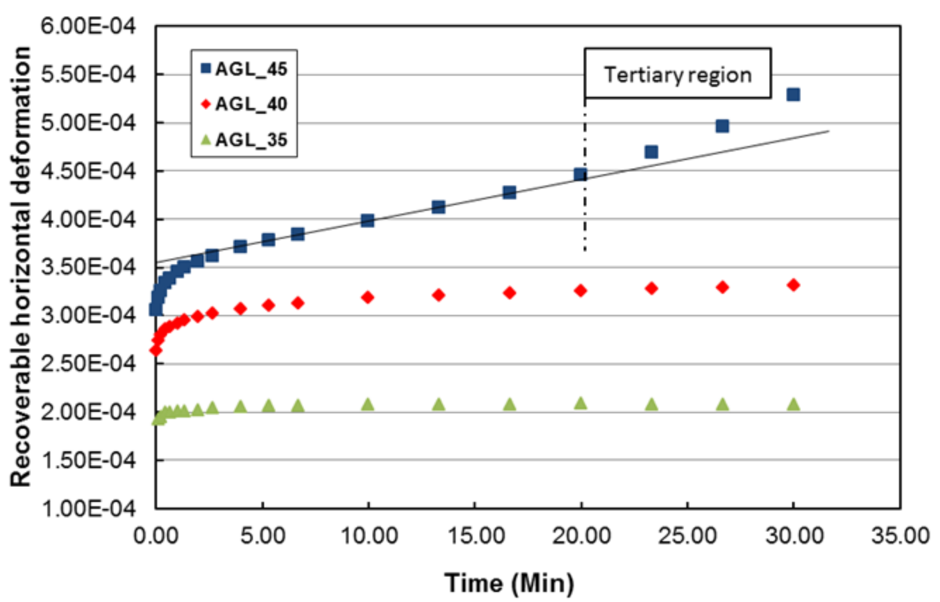


(5-7b) Sensitivity of recoverable horizontal deformation to loading level

Figure 5-7 Sensitivity to loading level for STOA granite w/o lime (AGS)



(5-8a) Sensitivity of effective modulus to loading level



(5-8b) Sensitivity of recoverable horizontal deformation to loading level

Figure 5-8 Sensitivity to loading level for LTOA granite w/o lime (AGL)

Figures 5-7 and 5-8 also show that an increase of load by as small as 5 percent of the strength of the material greatly affected the effective modulus reduction curve for both cases (i.e., granite w/o lime at STOA and LTOA). For instance, for STOA condition an appreciable effective reduction in M_E was induced by increasing the load level from 40 to 45 percent (Figure 5-7 (a)). On the other hand, for LTOA condition, it was the increase of load level from 35 to 40 percent that led to a reasonable amount of steady state damage while the 45 percent load level induced horizontal deformations no longer within the linear viscoelastic range (Figure 5-8 (b)). Furthermore, at TTOA the 35 percent load level was enough to induce a reasonable amount of damage (Figure 5-6). The same load levels identified for granite w/o lime specimens were used for granite w/ lime specimens, and the results showed the same trend. However, the resulted damage rate was highly sensitive to the applied load level and the heat oxidation level.

In summary, a load level threshold was identified above which any load starts to induce macrodamage and break up specimens. This threshold seemed to be dependent upon heat oxidation level. In particular, the more the asphalt mixture was heat oxidized, the lower the threshold as shown in Figure 5-9. Moreover, as heat oxidation increases, the material's response became more sensitive to the change in load level. In other words, the more the mixture was oxidized, the lower the difference between the load level that induced no microdamage and the load level threshold.

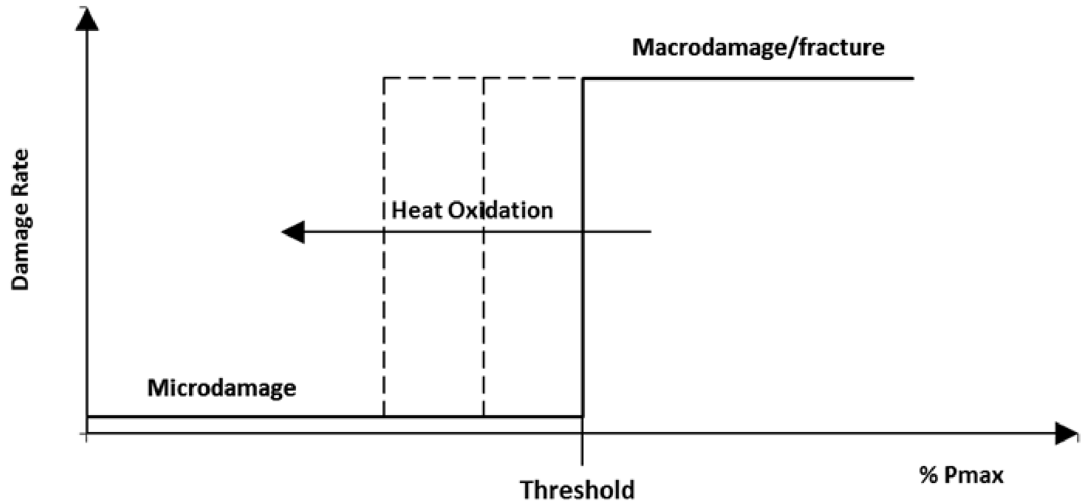


Figure 5-9 Effect of heat oxidation on load level threshold

Based on the results analyzed, the same load levels identified for granite w/o lime mixtures were also used for granite w/ lime mixtures. However, for limestone mixtures, loading levels for STOA and LTOA were reduced by 5 percent, respectively.

5.2.3 Repeated Load Conditioning after Heat Oxidation plus Cyclic Pore Pressure Conditioning

The effects of repeated load applied after cyclic pore pressure at three oxidative aging conditions (STOA, LTOA and TTOA) were evaluated for both mixtures through a total of eighteen specimens (three replicates per oxidative aging condition). The sequence of testing consists of the following:

- Heat oxidation: STOA, LTOA, or TTOA
- Vacuum saturation (procedure explained in Chapter 3)
- CPPC (procedure explained in Chapter 3)

- Two days in water bath at 10 °C
- RLC (test procedure explained in Chapter 3)
- Tensile strength test at 10 °C

5.2.3.1 Loss of Moisture during RLC after CPPC

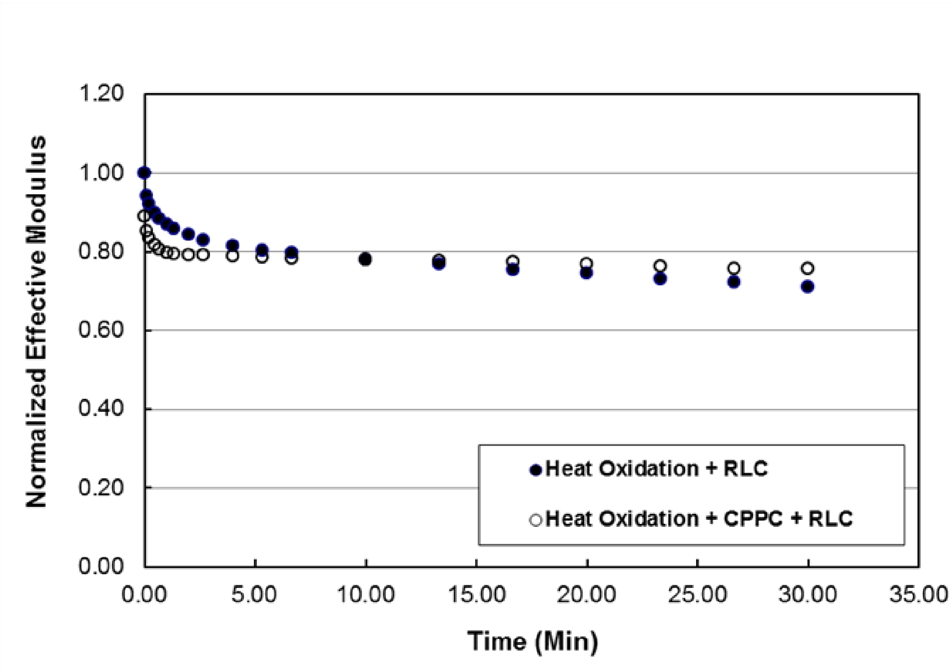
The same load levels identified through running the RLC test without the CPPC (Section 5.2.2) were used in order to obtain comparable results. However, it should be noted that the determination of a proper load level remained an issue for this test procedure.

Another issue was exposed after completion of all conditioning sets. Due to the dry air of the environmental chamber, the moisture content of the specimens underwent a significant reduction during the 30-minute RLC test. The moisture content reduction was estimated by weighing each specimen before and after the test and by assuming initial moisture content of 80 percent. Table 5-1 shows the reduction in moisture content for each material and heat oxidation level.

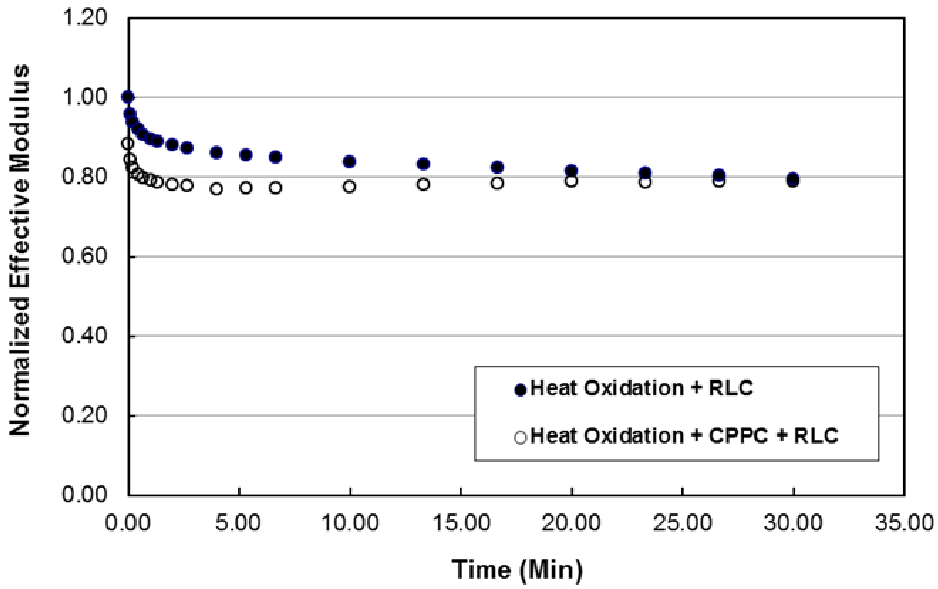
Table 5-1 Reduction in moisture content

Mixture Type	Aging Condition	Initial Moisture Content (%)	Final Moisture Content (%)	Reduction in Moisture Content (%)
Granite w/o lime	STOA	80	64.7	15.3
	LTOA	80	63.3	16.7
	TTOA	80	68.0	12.0
Granite w/ lime	STOA	80	71.5	8.5
	LTOA	80	69.3	10.7
	TTOA	80	66.7	13.3
Limestone	STOA	80	65.6	14.4
	LTOA	80	64.3	15.7
	TTOA	80	59.3	20.7

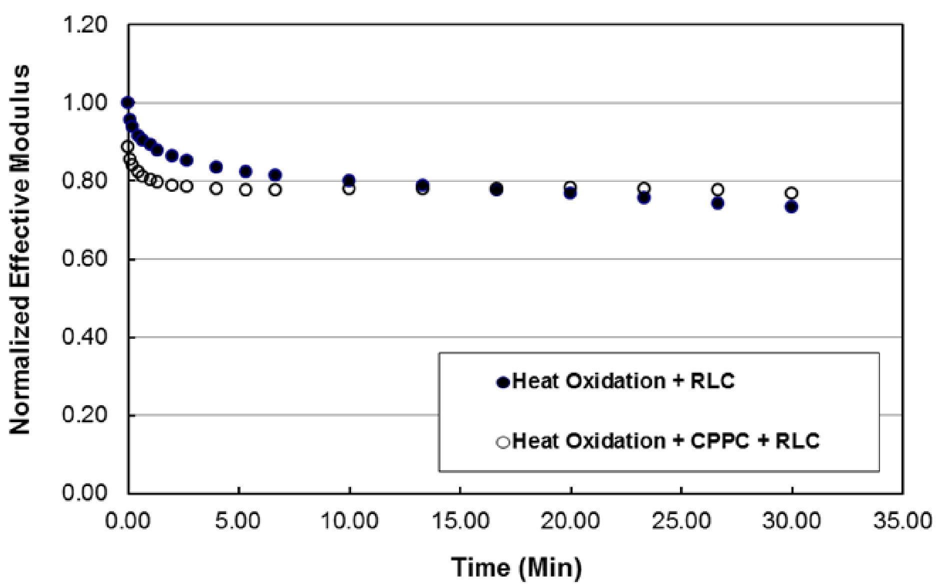
The effects of the reduction in moisture content on both mixtures at STOA are shown in Figure 5-10. The reductions in effective modulus obtained from the RLC tests without and with the CPPC, as indicated by "HOC + RLC" and "HOC + CPPC + RLC" respectively, were plotted together in order to identify the effects of CPPC on mixture properties. The M_E reduction curve for both cases were normalized by the initial M_E from the RLC test without the CPPC.



(5-10a) Effect of moisture content reduction on STOA granite w/o lime specimens



(5-10b) Effect of moisture content reduction on STOA granite w/ lime specimens



(5-10c) Effect of moisture content reduction on STOA limestone specimens

Figure 5-10 Effect of moisture content reduction on RLC test results for STOA condition

As also shown in Figure 5-10, the expected reduction in effective stiffness by repeated load did not occur for the RLC test with the CPPC. Although the CPPC induced additional damage to the mixture at the beginning of the test, the loss of water during the RLC test possibly caused negative pore pressure. The stiffening effect due to negative pore pressure appeared to have counterbalanced the repeated load effect to reduce stiffness. Therefore, the reliability of all three parameters used to characterize the reduction curve was compromised. Results of similar trends were obtained at LTOA and TTOA.

5.2.4 Closure

In summary, repeated load conditioning (RLC) test procedure was developed to simulate the effects of traffic load combined with oxidative aging and/or cyclic pore pressure. However, test results revealed the following issues:

- Material properties are excessively sensitive to load level, especially as the oxidative aging level increases. Therefore, it was difficult to determine the proper load level to induce a reasonable amount of damage in a controlled manner. The approach based on modified brittleness index I_{MB} provided a general guideline, but further work is needed to refine the procedures (see Final Report for FDOT project BDK-75-977-26).
- Saturated specimens during the RLC test performed after the CPPC underwent a decrease in moisture content, which significantly affected the test results.

It appeared that at this stage, the RLC procedure cannot provide data that are reliable enough for further evaluations such as the effects of additives. Therefore, these data were not included in the sections that followed.

5.3 Effect of Laboratory Conditioning on Change in HMA Fracture Properties

5.3.1 Effect of Heat Oxidation Conditioning (HOC Only)

It is generally agreed that hardening and embrittlement of asphalt mixtures are primarily affected by heat oxidation (i.e., oxidative aging). Three different levels of oxidative aging, including STOA, LTOA, and TTOA, were used for conditioning of test specimens to evaluate the effect of heat oxidation on change in fracture properties of asphalt mixtures.

Fracture energy (FE) reflects the resistance of mixture to damage without fracture. It was identified that FE could be a good indicator for cracking performance of asphalt pavements (Zhang et al., 2001). FE results for three different level of heat oxidation are presented in Figure 5-11. As expected, FE has generally decreased with the increase of oxidative aging level. It was indicated that heat oxidation conditioning (HOC) was capable of inducing the reduction in FE limit to certain level mainly associated with stiffening and embrittlement of mixtures. However, heat oxidation only could not reduce FE limit to the level observed in the field (Roque et al., 2011).

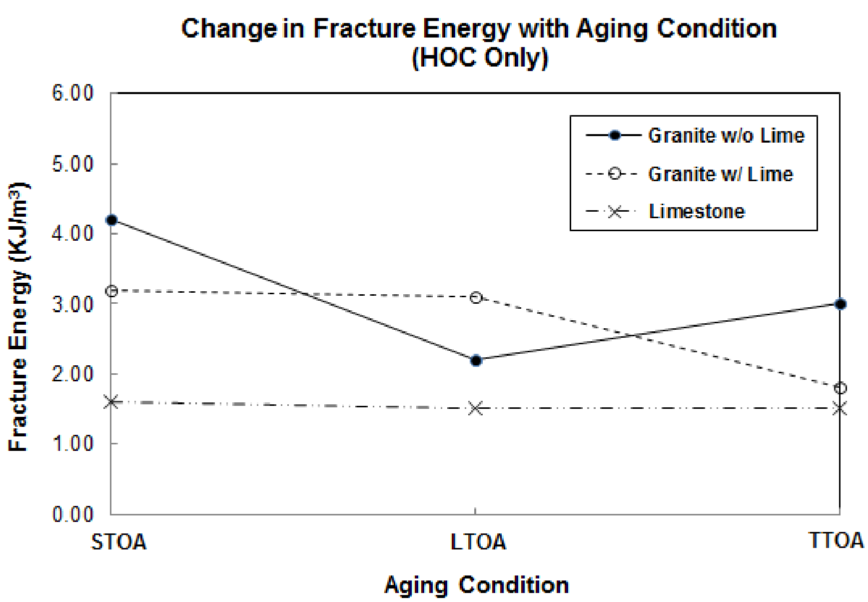


Figure 5-11 Effect of HOC on change in FE

Further analyses with respect to the effects of different type of mixture were also available using Figure 5-11. Based on the results shown in Figure 5-11, it was clearly indicated that granite w/o lime mixture appears more susceptible on heat oxidation in early stage of aging (i.e., up to LTOA condition) than granite w/ lime and limestone mixtures. However, it is noted that limestone mixture exhibited relatively low FE limit for STOA condition that affect the trend of change in FE with aging (Roque et al., 2010). It was expected that hydrated lime enhanced FE retention of mixtures subjected to oxidative aging and reduces its effect on change in FE with aging. However, the beneficial effect of hydrated lime seems to diminish with the higher level of oxidative aging (i.e., TTOA condition).

Creep rate, or the rate of change in creep compliance at times, is associated with the rate of damage accumulation in the mixture. Figure 5-12 shows the creep rate results.

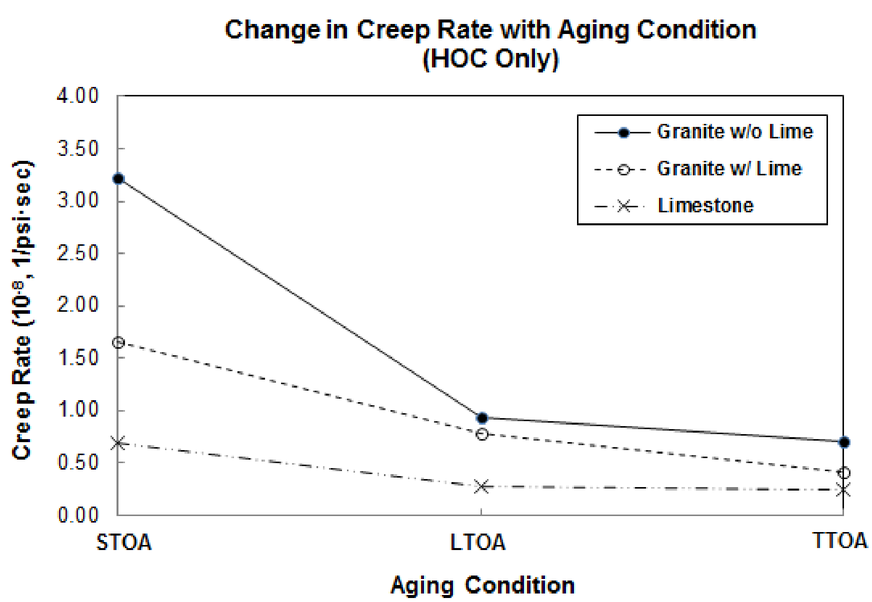


Figure 5-12 Effect of HOC on change in creep rate

Figure 5-12 shows that the creep rate decreases with oxidative aging increases. The trend was also consistent with the fact that the rate of reduction in creep rate generally decreases with oxidative aging increases. Based on the results from Figure 5-12, it was identified that granite w/o lime mixture exhibits higher rate of reduction in creep rate with higher initial creep rate magnitude than granite w/ lime and limestone mixtures. However, the rate of reduction between LTOA and TTOA condition was not prominent for all mixtures evaluated. This appeared to indicate that TTOA condition did not significantly induce further effect of oxidative aging than LTOA condition.

Figure 5-13 represents the tensile strength results. Tensile strength indicates the maximum tensile stress that the mixtures are able to tolerate before fracturing. As shown in Figure 5-13, tensile strength generally increases with increasing the level of oxidative aging. A lime-treated granite mixture overall exhibits higher range of tensile strength than granite w/o lime and limestone mixtures.

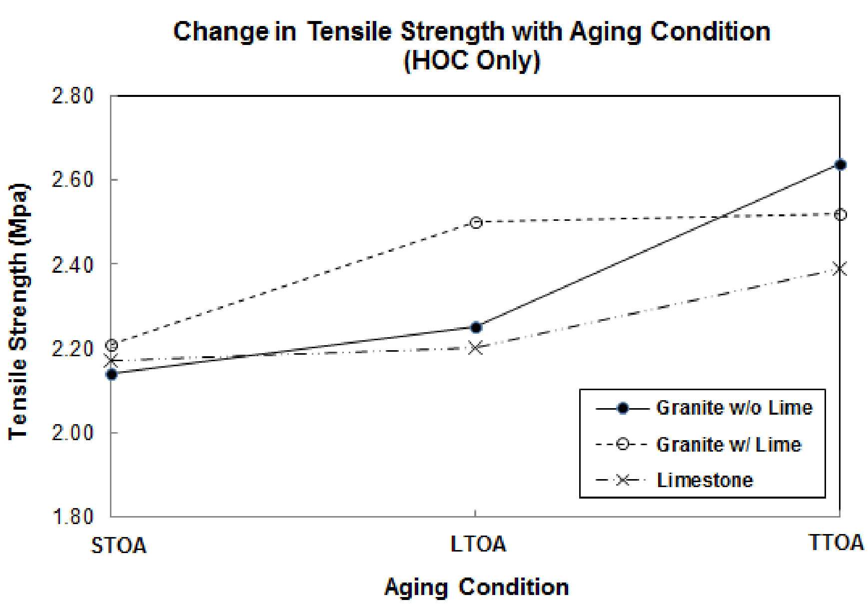


Figure 5-13 Effect of HOC on change in tensile strength

Figure 5-14 exhibits the results of failure strain. Failure strain, which characterizes the brittleness of a mixture, is associated with the severity and mixture susceptibility to oxidative aging. As expected, failure strain has overall decreased with the increase of oxidative aging level. It was clearly identified that mixtures were embrittled by oxidative aging process (i.e., heat oxidation). Also, results show that granite w/o lime mixture seems more susceptible on embrittlement process by oxidative aging than lime-treated granite and limestone mixtures, specifically in early stage of aging (i.e., up to LTOA condition).

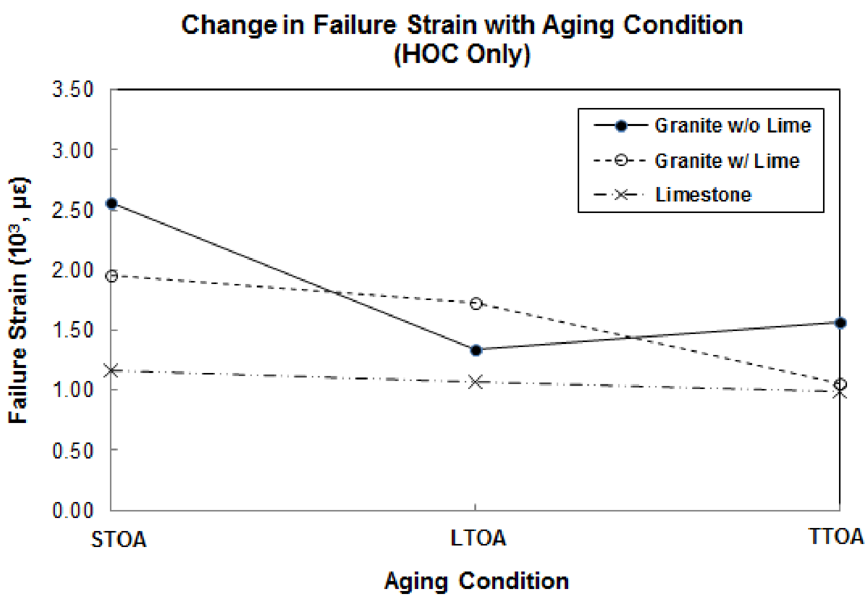


Figure 5-14 Effect of HOC on change in failure strain

Results of resilient modulus (M_R) which indicates a measure of elastic stiffness are shown in Figure 5-15. As expected, resilient modulus has generally increased with the increase of oxidative aging. Lime-treated granite and limestone mixtures exhibit greater resilient modulus values than granite w/o lime mixture throughout the oxidative aging process.

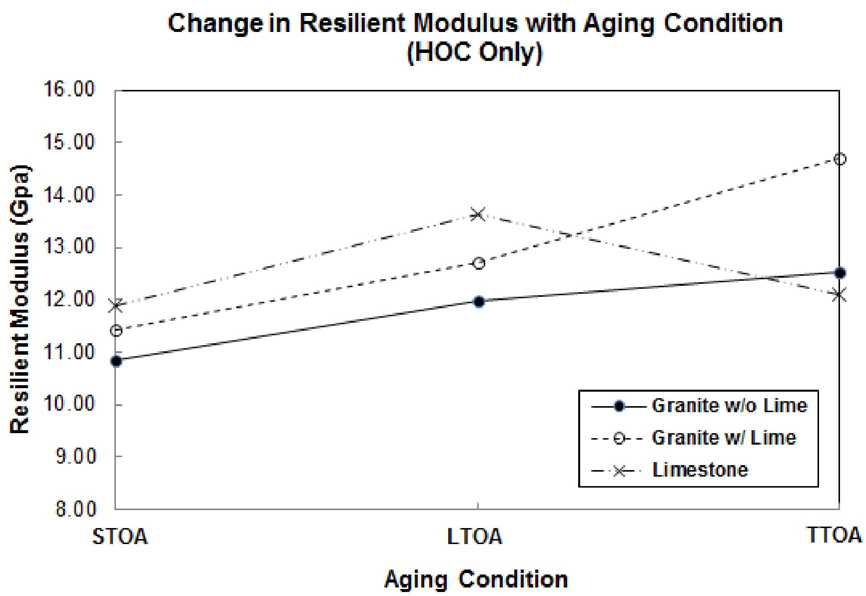


Figure 5-15 Effect of HOC on change in resilient modulus

The energy ratio (ER), which was developed by Roque et al. (2004), represents the fracture resistance of asphalt mixtures and it can be used to evaluate cracking performance. The ER results are shown in Figure 5-16. For lime-treated granite and limestone mixture, ER was maintained higher throughout the oxidative aging process than granite w/o lime mixture that indicates better cracking resistance of mixtures. Typically, higher ER values are related to higher FE limit and lower creep rate. Based on results from Figures 5-11 and 5-12, the trends of ER result are consistent with the results of FE limit and creep rate.

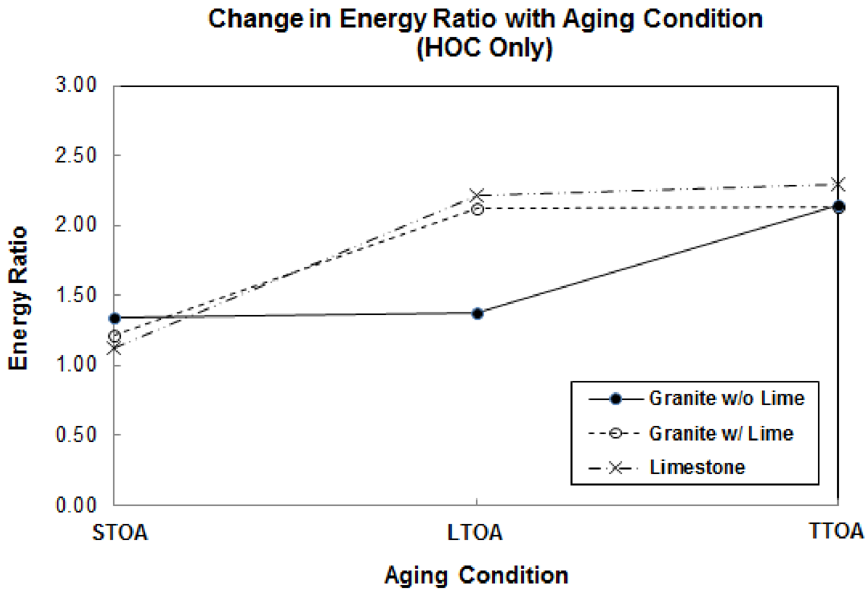
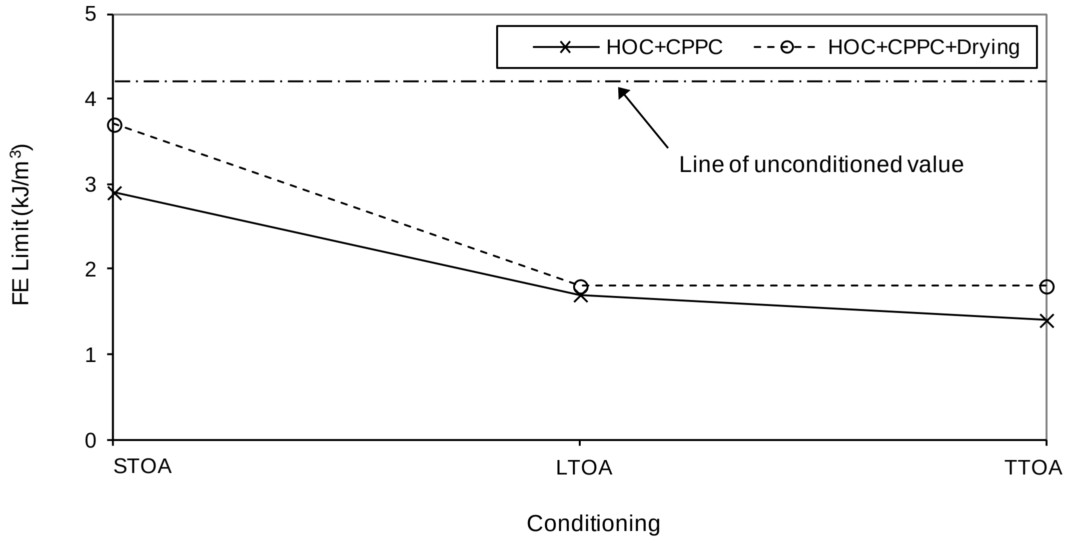


Figure 5-16 Effect of HOC on change in energy ratio

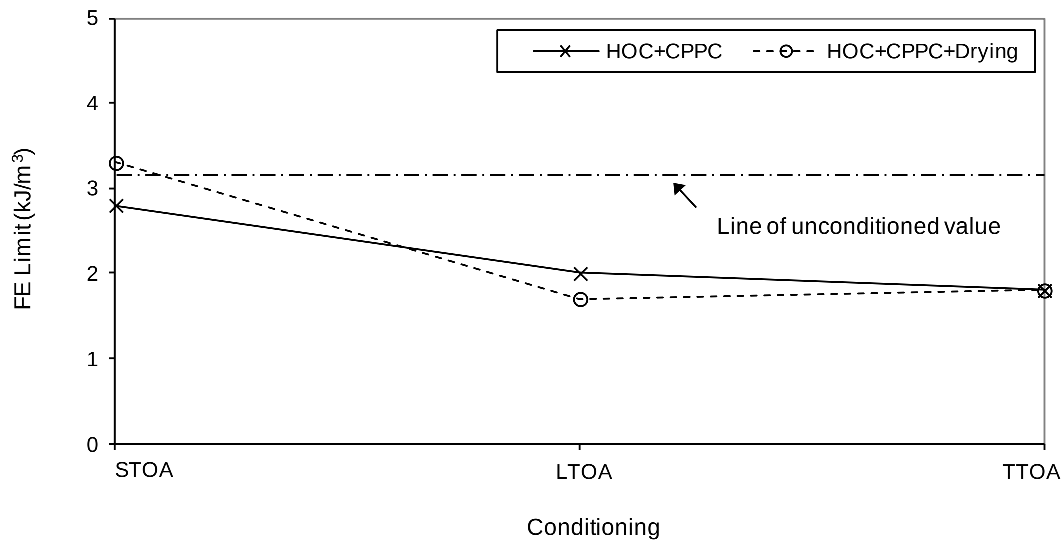
5.3.2 Effect of Drying after Cyclic Pore Pressure Conditioning (CPPC + Dry)

The effect of drying was evaluated by comparing the properties of specimen tested after the drying process (HOC + CPPC + drying) with those determined after being soaked in water bath as described earlier (HOC + CPPC). The results for all three mixtures are presented in Figures 5-17 and 5-18. As can be seen, when allowed to dry after the CPPC, both granite mixtures generally exhibited an increase in FE limit (except for the lime-treated granite mixture after LTOA), which is consistent with our expectation that drying helps recover the damage tolerance of the mixtures.

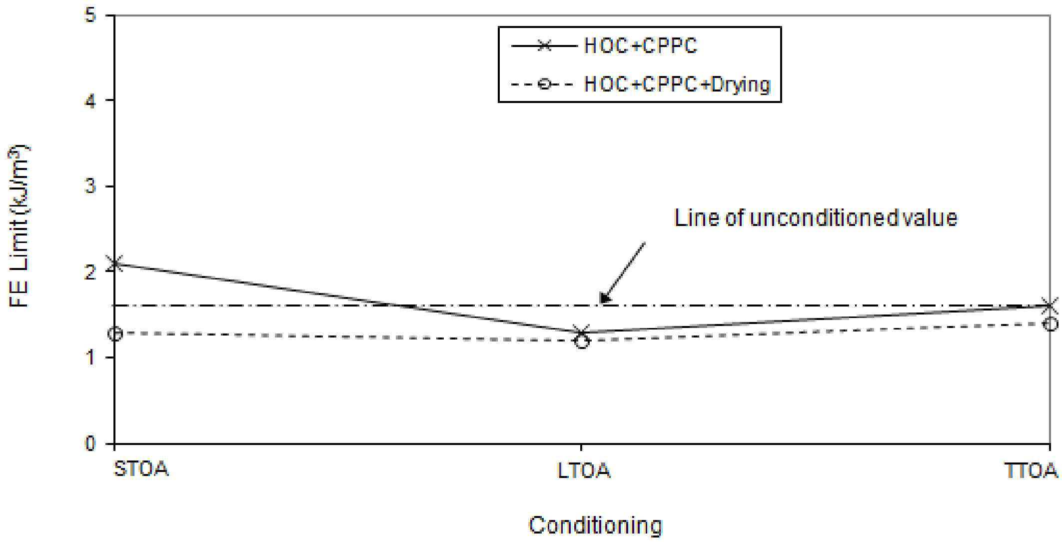
However, limestone mixture exhibited an opposite trend. On the other hand, all three mixtures also exhibited an increase in creep rate (except for the lime-treated mixture after STOA), which is not consistent with our expectation that drying helps reduce the rate of damage accumulation. It appeared that the effect of the drying process was more involved than simply recovery of fracture and damage properties.



(5-17a) Granite w/o lime

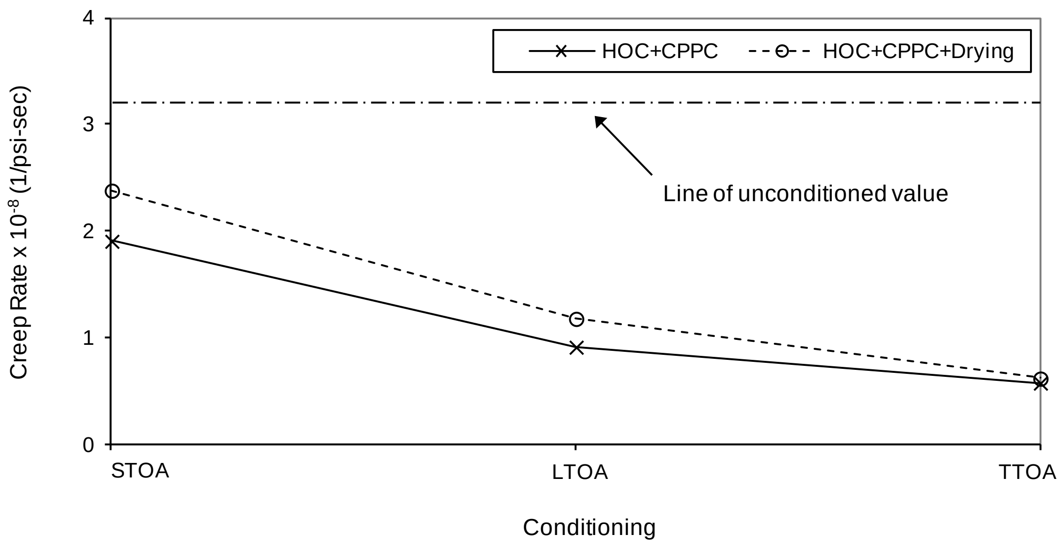


(5-17b) Granite w/ lime

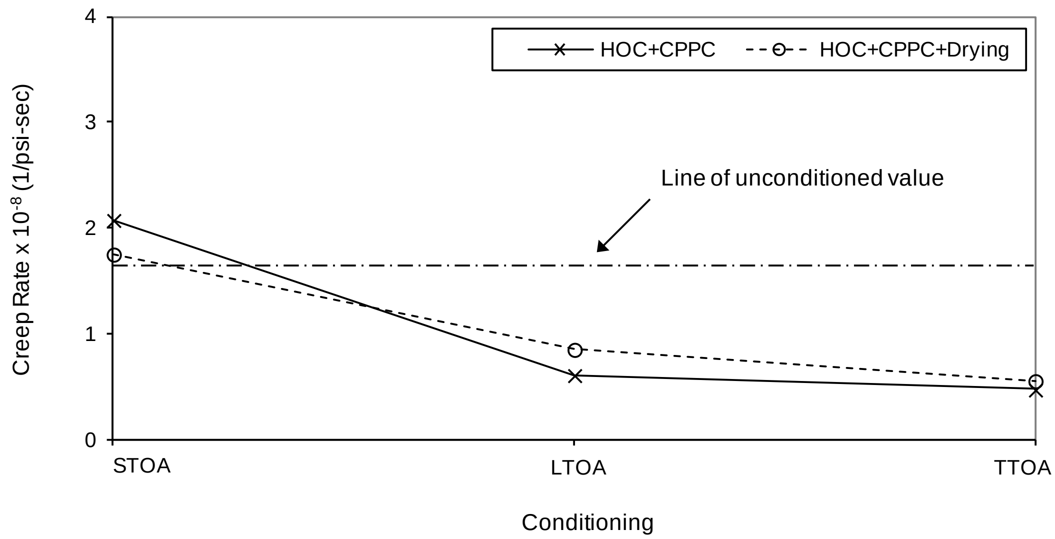


(5-17c) Limestone

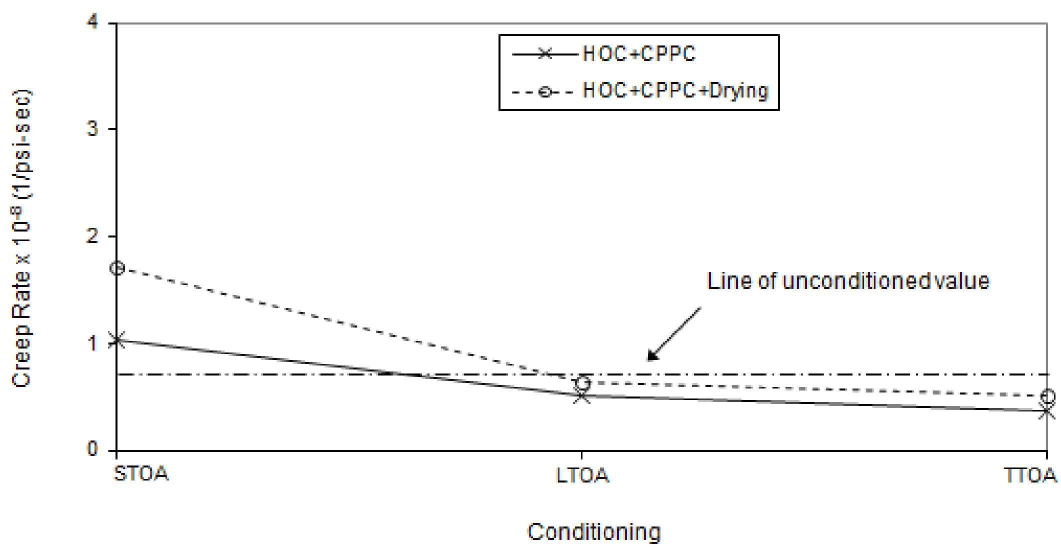
Figure 5-17 Effect of drying after CPPC on change in FE limit



(5-18a) Granite w/o lime



(5-18b) Granite w/ lime

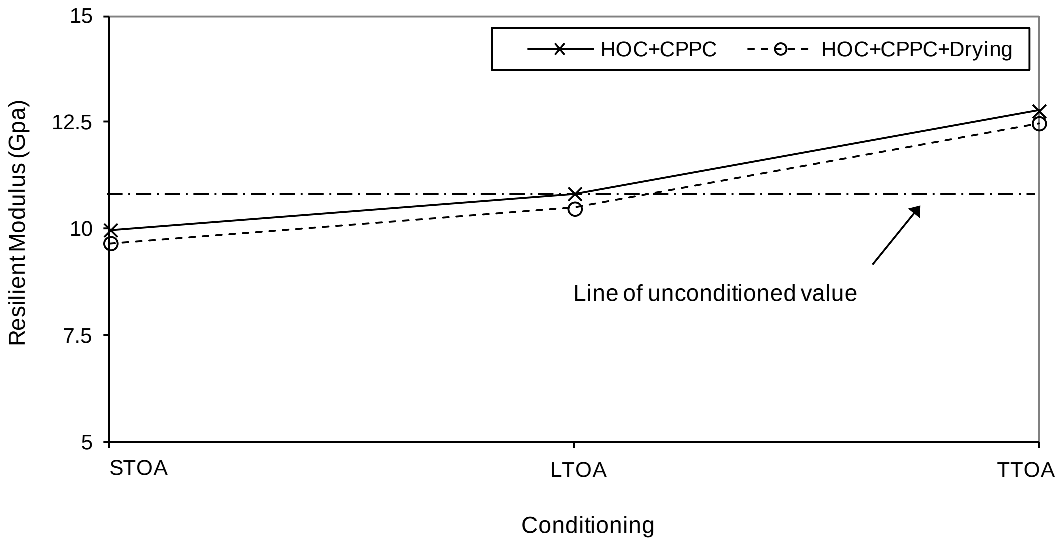


(5-18c) Limestone

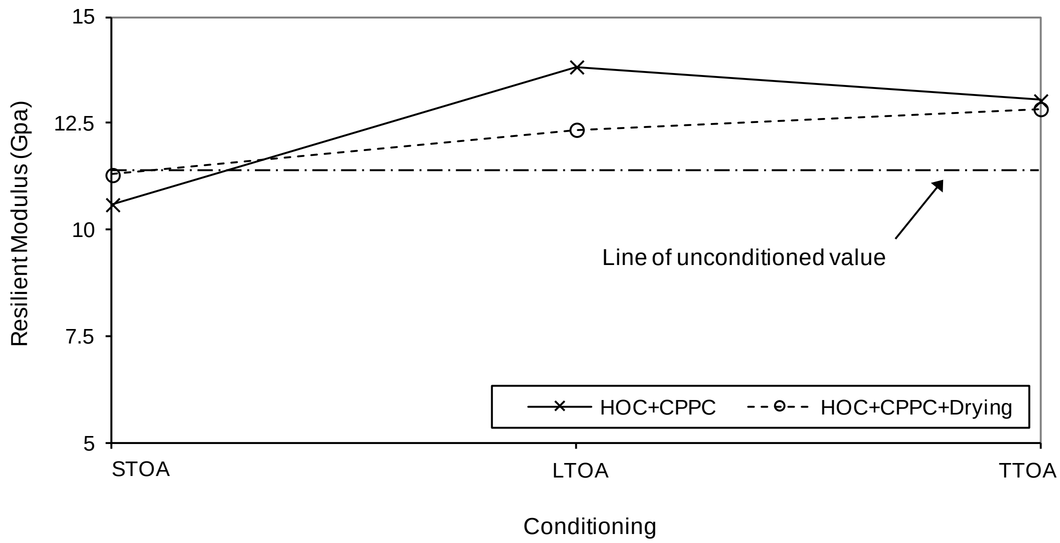
Figure 5-18 Effect of drying after CPPC on change in creep rate

In addition, Figure 5-19 shows the results of resilient modulus for all three mixtures. After the process of drying, the stiffness of the untreated granite mixture was decreased for all levels of oxidation. A similar trend was observed for the lime-treated granite and limestone mixtures after long-term oxidative aging (i.e., LTOA and TTOA). It appeared that the complete drying eliminated the negative pore pressure, which helps stiffen the mixture when tested at wet status (i.e., HOC + CPPC).

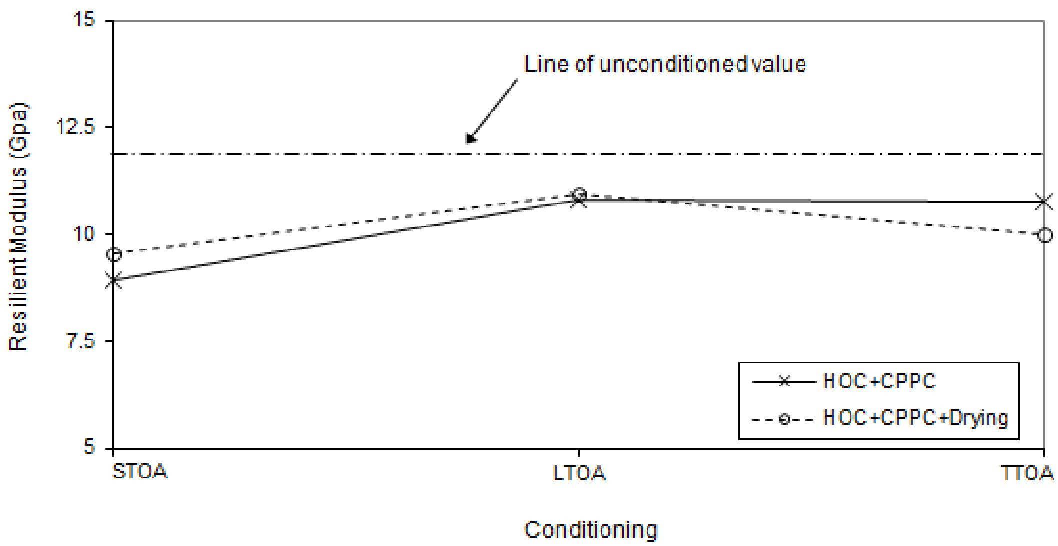
In summary, the drying process after the CPPC was not able to recovery all of the key fracture properties. Also, it added complexity to interpretation of test results. Therefore, it was not recommended for inclusion as part of the laboratory conditioning procedure.



(5-19a) Granite w/o lime



(5-19b) Granite w/ lime



(5-19c) Limestone

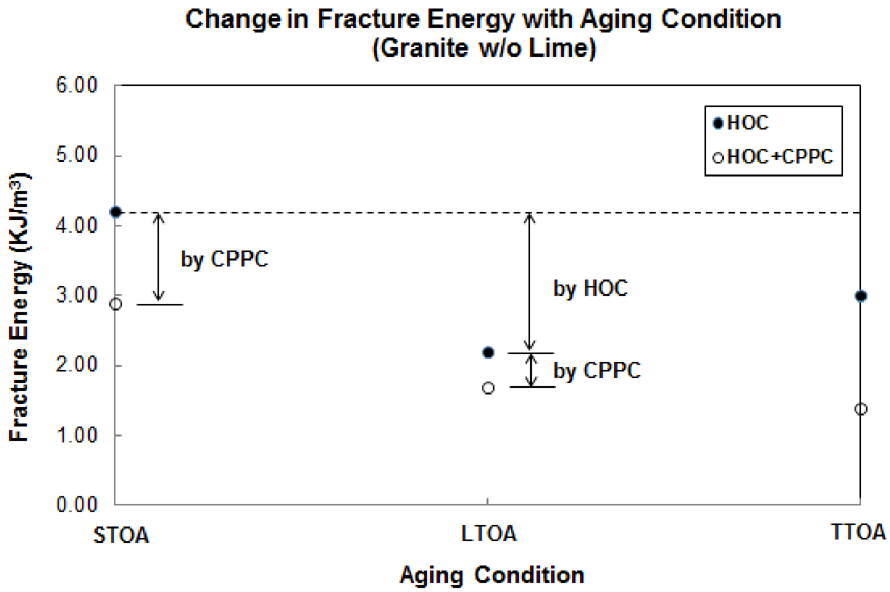
Figure 5-19 Effect of drying after CPPC on change in stiffness

5.3.3 Effect of Cyclic Pore Pressure Conditioning (CPPC + Wet)

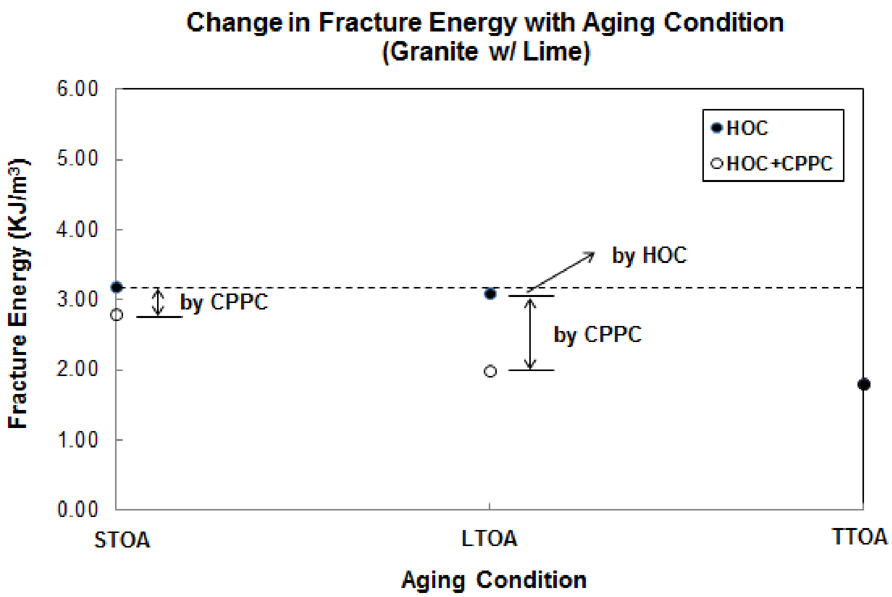
Moisture damage leads to reduction in stiffness and possibly disintegration of asphalt mixtures. Asphalt pavement is continuously affected by moisture throughout the service life. However, even though heat oxidation increases stiffness of mixture, effects of moisture decrease stiffness. Therefore, it is necessary to consider the effects of heat oxidation and moisture simultaneously on change in fracture properties of asphalt mixtures. In this study, the CPPC system was selected to effectively induce moisture damage in mixtures. It was expected that this system can provide the advantage of inducing moisture damage in a more realistic fashion that actually occurs in the field.

The FE results, including the effects of the CPPC in addition to heat oxidation, are shown in Figure 5-20. It was identified that further reduction in fracture energy (FE) limit was induced by the CPPC throughout the oxidative aging process. In other words, the CPPC system devised effectively generated damage in mixtures. Based on the results in Figure 5-20, granite w/o lime mixture appears more susceptible on the CPPC than lime-treated granite and limestone mixtures. It was expected that hydrated lime reduced the effect of the CPPC on change in FE in mixtures.

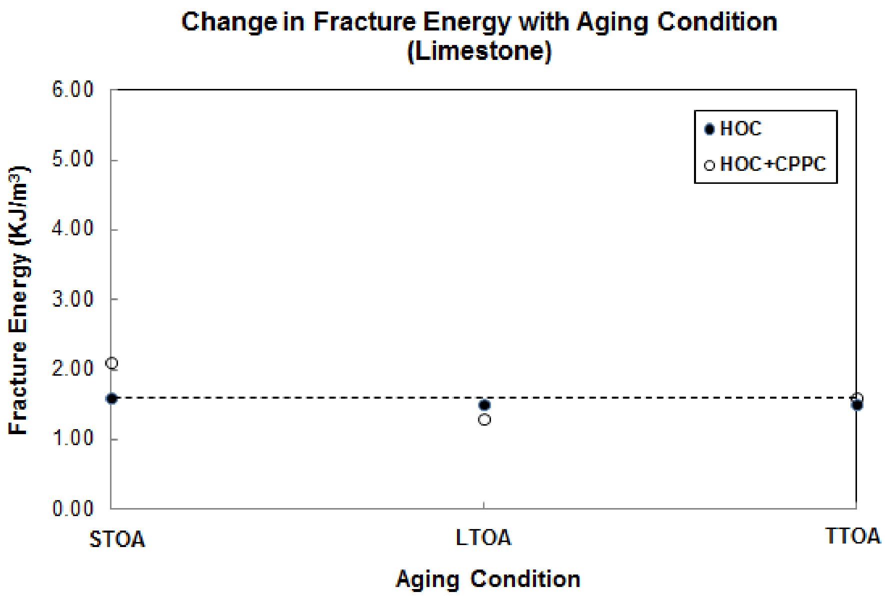
Therefore, the CPPC appeared to be a suitable way to effectively induce microdamage due to moisture, specifically for early stage of aging process associated with adhesive failure mechanism (i.e., premature moisture damage) in asphalt mixtures. In addition, the CPPC can also be a good tool to evaluate the effect of cyclic load for later stage of aging process related to cohesive failure mechanism because the CPPC can also effectively simulate the effect of cyclic loading by inducing the internal pressure (stress) in a tensile mode on mixtures. For higher level of oxidative aging, material’s behavior becomes more sensitive to the effect of load due to higher brittleness and lower strain tolerance.



(5-20a) Granite w/o lime



(5-20b) Granite w/ lime



(5-20c) Limestone

Figure 5-20 Effect of CPPC and HOC on change in FE

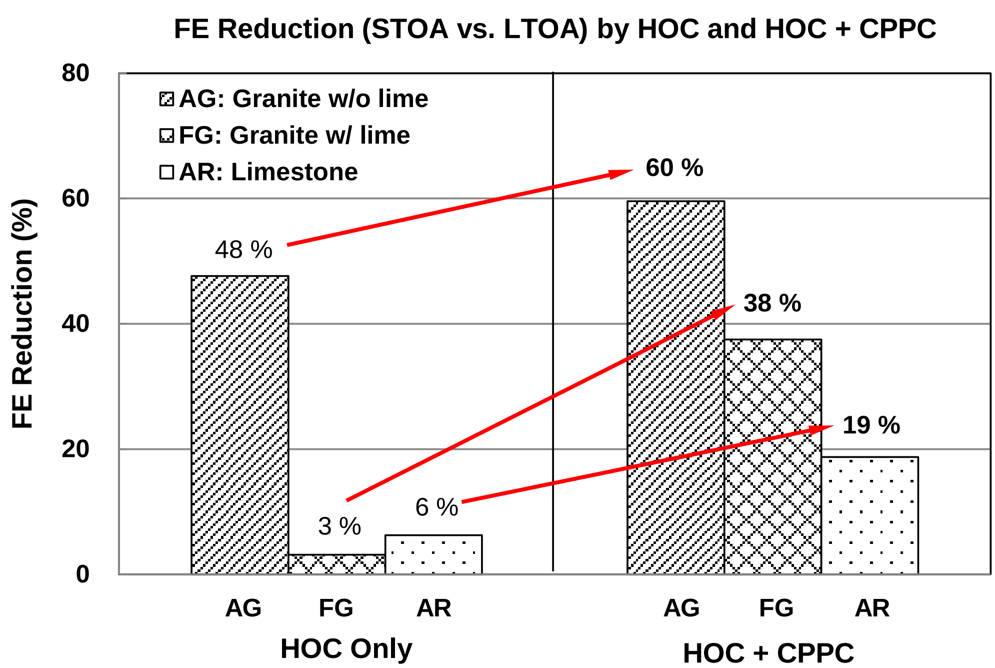
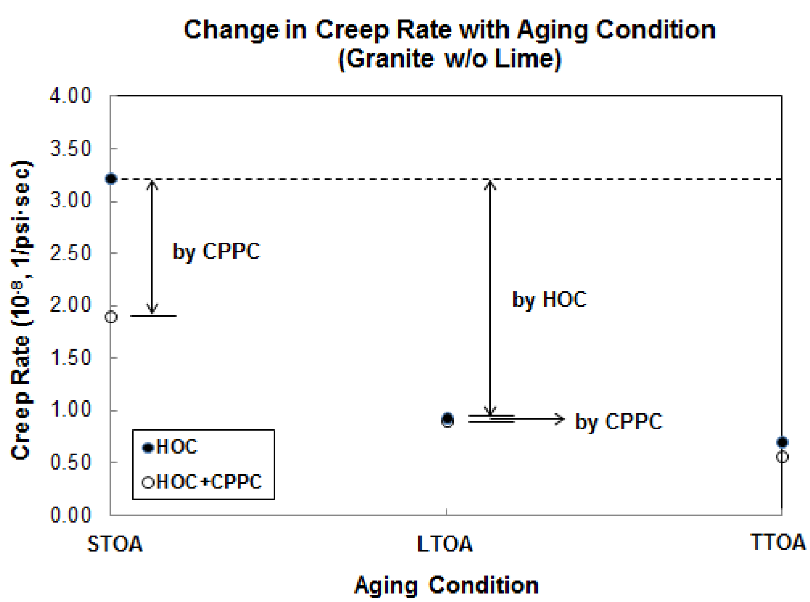


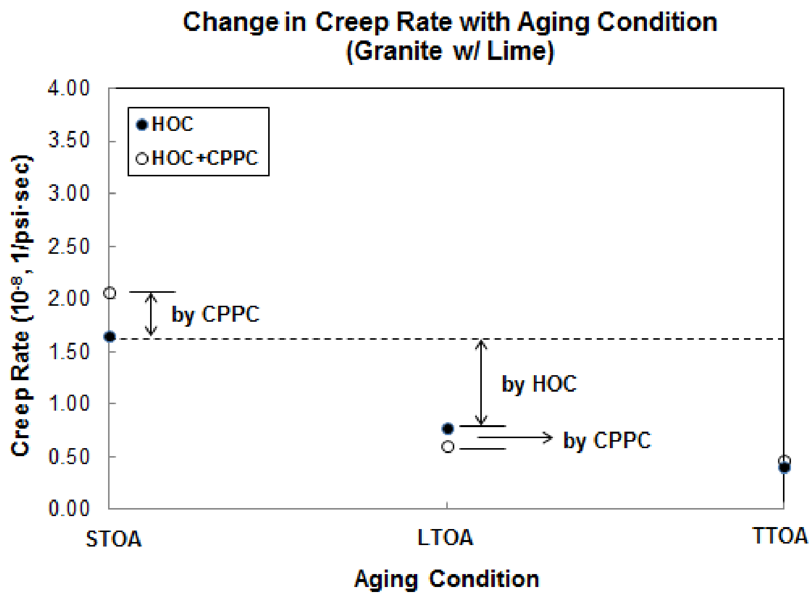
Figure 5-21 Fracture energy reduction by HOC and HOC + CPPC

Figure 5-21 shows the comparison between fracture energy reduction induced by HOC only and HOC + CPPC. The CPPC was found to effectively induce further reduction in FE limit of mixtures to levels consistent with those observed in the field (see Figure 3-1).

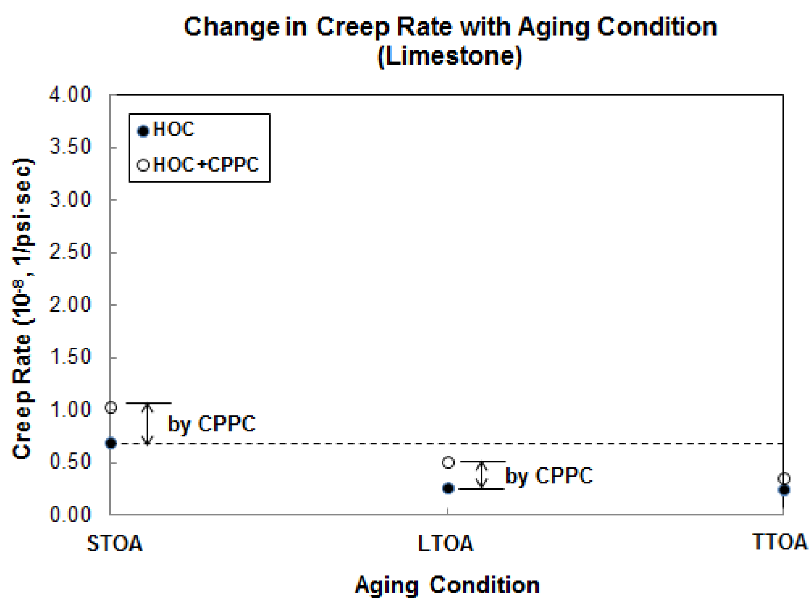
Figure 5-22 shows the creep rate results. Results indicated that the reduction in creep rate was mainly controlled by the effect of oxidative aging. It was found that lime-treated granite and limestone mixtures exhibited an increase in creep rate due to the CPPC for STOA condition (except for the granite w/o lime mixture), which is consistent with our expectation that moisture damage led to increase in the rate of damage accumulation. In addition, granite w/o lime mixture seems more sensitive to the effects of CPPC and heat oxidation on change in creep rate than limestone mixtures. As expected, creep rate values were generally varied in relatively lower range for lime-treated granite and limestone mixtures than granite w/o mixture.



(5-22a) Granite w/o lime



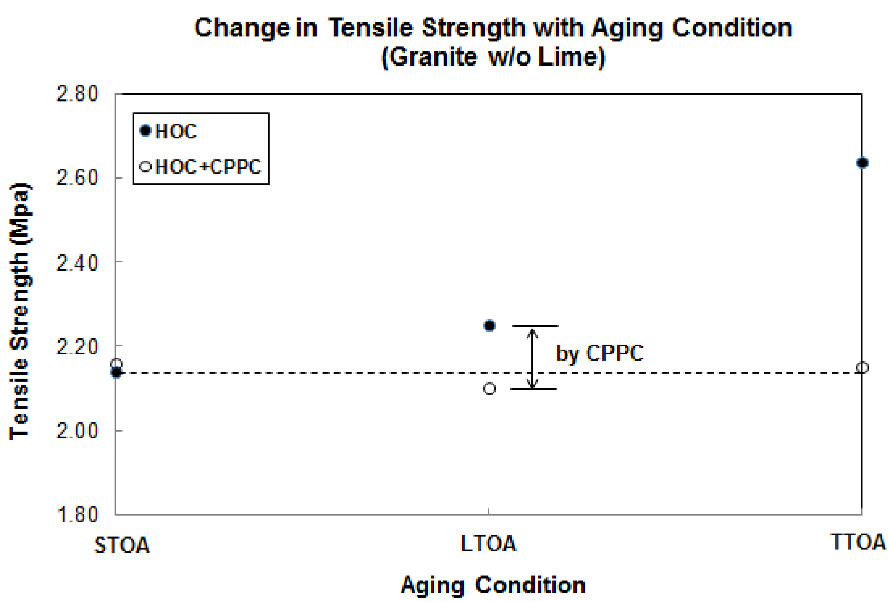
(5-22b) Granite w/ lime



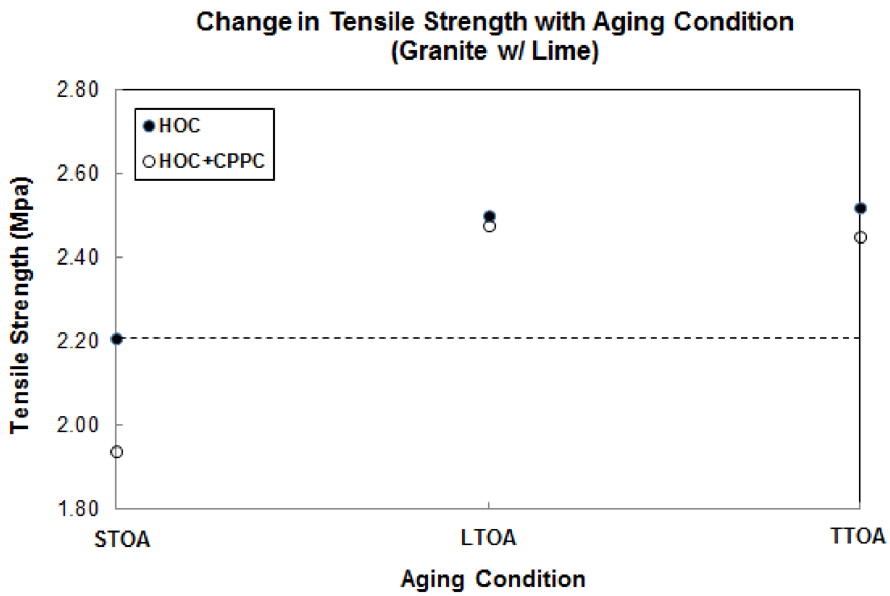
(5-22c) Limestone

Figure 5-22 Effect of CPPC and HOC on change in creep rate

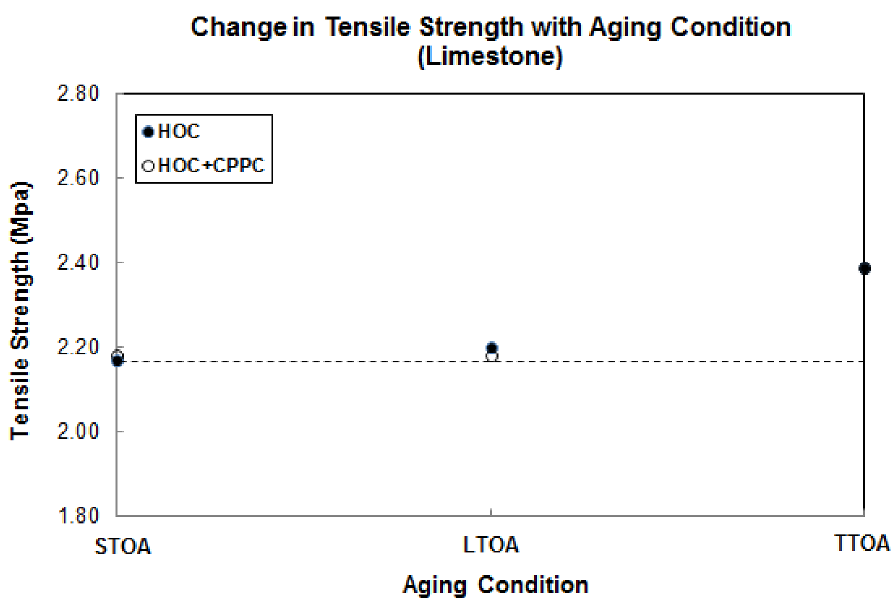
Figure 5-23 shows the tensile strength results. Based on the results in Figure 5-23, it was indicated that the granite w/o lime mixture showed a decrease in tensile strength due to the CPPC for LTOA and TTOA conditions. This indicates that the CPPC induced permanent damage in mixture and it also appears to be associated with the existence of incomplete healing after a certain level of aging. However, almost no reduction in tensile strength by the CPPC was identified for lime-treated granite and limestone mixtures. In particular, for lime-treated granite mixture, tensile strength was maintained in relatively higher range than granite w/o lime and limestone mixtures for LTOA and TTOA condition even though applying the CPPC.



(5-23a) Granite w/o lime



(5-23b) Granite w/ lime



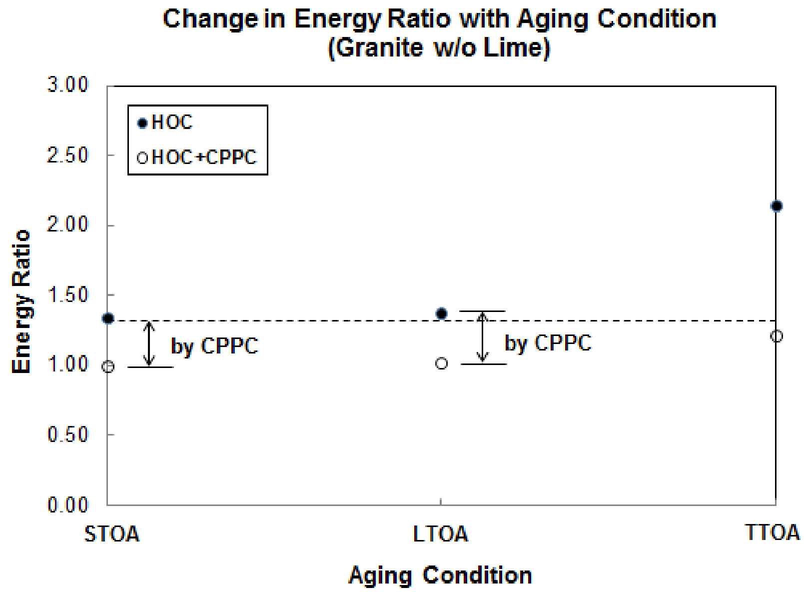
(5-23c) Limestone

Figure 5-23 Effect of CPPC and HOC on change in tensile strength

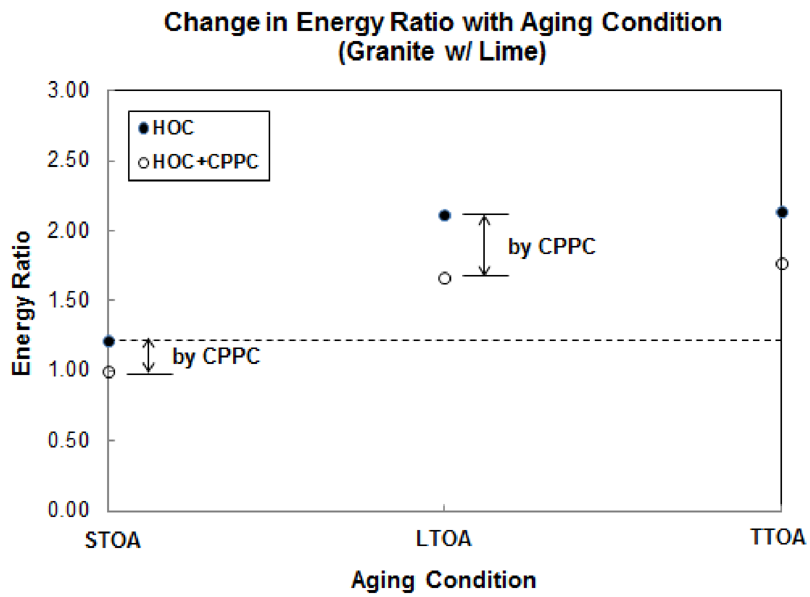
The Energy Ratio (ER) results are shown in Figure 5-24. In this study, the ER parameter was evaluated for relative comparison of fracture performance of mixture subjected to a similar level of oxidative aging. Results clearly indicated that the CPPC led to worse fracture performance for all types of mixtures. This observation was consistent with findings from previous research that the ER can be used as an indicator to evaluate the effect of moisture damage on changes in fracture resistance of asphalt mixtures (Birgisson et al., 2005).

As expected, it was found that the ER for lime-treated granite mixture was greater than the ER for granite w/o lime mixture for LTOA and TTOA followed by the CPPC. This indicated that the addition of hydrated lime helped resist heat oxidation and the CPPC. In sum, hydrated lime reduced the effect of oxidative aging and moisture damage on change in fracture properties of asphalt mixtures. However, the beneficial effect of lime appeared to diminish with the higher level of oxidative aging.

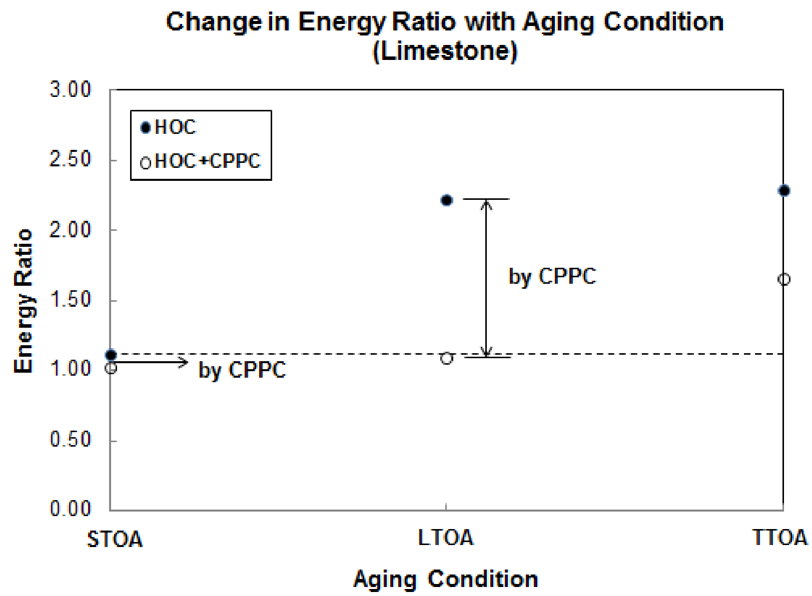
In addition, limestone mixture exhibited higher ER than granite w/o lime mixture for LTOA and TTOA followed by the CPPC which is consistent with the fact that limestone mixture is more resistant to moisture damage than granite mixtures.



(5-24a) Granite w/o lime



(5-24b) Granite w/ lime



(5-24c) Limestone

Figure 5-24 Effect of CPPC and HOC on change in energy ratio

5.4 Closure

Mixture test results were carefully analyzed to evaluate the effectiveness of laboratory conditioning procedure identified, including heat oxidation, CPPC, and RLC, as well as their effects on change in fracture properties of different types of asphalt mixtures evaluated. An attempt was made to evaluate and validate conditioning procedures as well as to establish the potentially most effective laboratory aging conditioning system to properly simulate asphalt mixture aging in the field.

CHAPTER 6
CLOSURE

6.1 Summary and Findings

This study was conducted to identify and evaluate appropriate laboratory aging (i.e., conditioning) procedures and testing protocols considering heat oxidation, moisture, and load that more effectively simulate asphalt mixture aging in the field, and thereby help to properly assess asphalt mixture property changes over time. For purpose of this study, aging was defined as all detrimental effects during pavement life on asphalt mixture properties. Factors that can cause aging as defined in this manner include heat oxidation, moisture, and load effects. Findings associated with the objectives of this study are summarized as follows:

- The combination of HOC and CPPC effectively induced:
  - Reasonable and expected trend of changes in damage and fracture related properties.
  - Changes in damage and fracture related properties to levels consistent with field observation.
- Even though heat oxidation was able to stiffen and embrittle mixtures by increasing stiffness, and by reducing failure strain and fracture energy (FE) limit to certain level, it could not reduce FE limit to the level observed in the field.
  - Similar to APAS in APT/HVS.
- The CPPC was found to effectively induce further reduction in FE limit of mixtures to levels consistent with those observed in the field.

- The drying process after the CPPC was not able to recovery all of the key fracture properties. Also, it added complexity to interpretation of test results. Therefore, it was not recommended for inclusion as part of the laboratory conditioning procedure.
- The Energy Ratio (ER) parameter was evaluated in this study for relative comparison of fracture performance of mixtures subjected to a similar level of oxidative aging. As expected, the results showed that the CPPC led to worse fracture performance for given oxidative aging levels.
- The RLC was capable of inducing a reasonable amount of damage (i.e., 10 % to 20 % of reduction in effective stiffness) while maintaining the rate of damage in steady state (i.e., a constant rate of damage), once an appropriate load level was determined.
- An approach was developed based on the modified brittleness index (I_{MB}) (i.e., strength divided by failure strain from Superpave IDT test) to estimate the load level for use in the RLC test. In general, the approach provided a reasonable starting value, but a trial and error approach is still required to determine the final load level. Further work was needed to develop a more robust approach (see final report for FDOT project BDK-75-977-26).
- Results of RLC tests revealed the following:
  - There is a load level threshold above which RLC induces macrodamage in the specimen within very few load repetitions. The load level threshold was found to be highly dependent upon the oxidative aging level. The more the mixture was

oxidized, the lower the threshold, and the smaller the difference between the load level that induced no microdamage and the load level threshold.

- It was impossible to maintain the same load level for mixtures subjected to different levels of oxidative aging and concurrently induce a reasonable amount of steady state damage.
- The RLC was not able to induce further reduction in effective stiffness (i.e., microdamage) after cyclic pore pressure conditioning. The reduction in moisture content (approximately 10 % to 15 %) during the 30-minute-long RLC process possibly caused a negative pore pressure in the specimens. The stiffening effect due to negative pore pressure appeared to have counterbalanced the effect on reduction in stiffness caused by the repeated load.

- As expected, it was found that ER for lime-treated granite mixture was greater than ER for granite w/o lime mixture for LTOA and TTOA followed by the CPPC. This indicates that the addition of hydrated lime helped resist heat oxidation and the CPPC.
- Limestone mixture exhibited higher ER that indicates better fracture resistance than granite w/o lime mixture for LTOA and TTOA followed by the CPPC. This is consistent with the fact that limestone is highly resistant to moisture damage.

In summary, Figure 6-1 shows the recommended conditioning procedure.

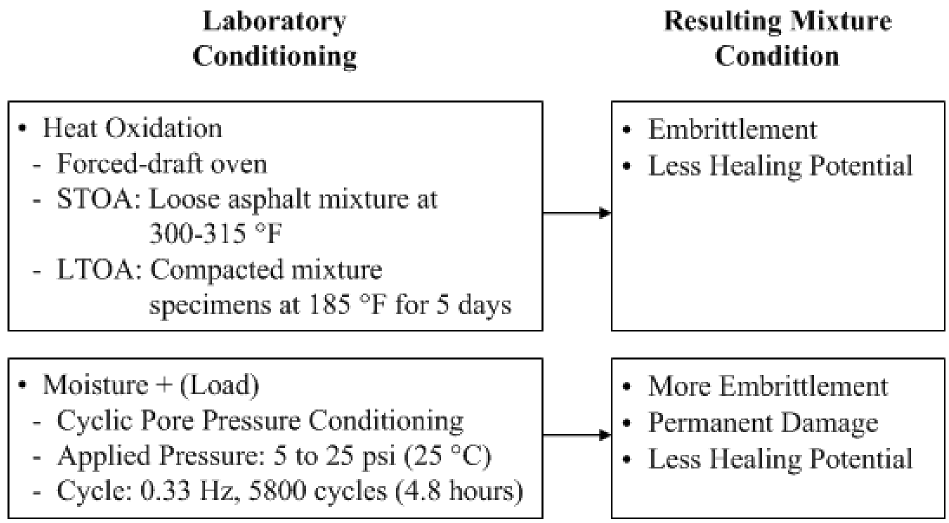


Figure 6-1 Recommended conditioning procedure

6.2 Conclusions

The following key conclusions were drawn based on the findings of this study.

- The combination of HOC and CPPC can effectively induce the change on damage and fracture related properties of asphalt mixtures consistent with those observed in the field.
- The CPPC appears to combine effects of moisture and load since the CPPC induces internal pressure (stress) in a tensile mode that is similar to the effect of repeated load actually experienced by mixtures in the field.
- The best approach to effectively characterize the aging process in the field, including the effect of heat oxidation, moisture, and loading, seems to be the HOC in conjunction with the CPPC (i.e., LTOA followed by CPPC).
- The CPPC appears a more realistic way than the traditional freeze-thaw or boiling methods to simulate moisture damage in the field.

- The CPPC appears a more suitable way to induce microdamage in the specimen without fracture than the RLC, since the CPPC provide uniformly distributed stress state whereas the RLC does not (i.e., nonuniform stress state).
- The RLC is able to induce a reasonable amount of damage and maintain the rate of damage in a steady state for a proper load. However, no proper way has been identified to evaluate the effect of RLC on mixtures with different levels of oxidative aging due to the dependence of load level (for steady state damage) on brittleness of mixtures.
- The current RLC procedure exposed some issues on its application in terms of controlling testing condition. Particularly with the CPPC followed by the RLC, it is difficult to maintain constant moisture content during 30-minute-long test that significantly affect the test results.
- The addition of hydrated lime helps mixtures to reduce the effects of oxidative aging and CPPC, and leads to improved fracture performance. However, the beneficial effect of hydrated lime appears to diminish with the higher level of oxidative aging.

6.3 Recommendations and Future Work

Based on extensive evaluations performed in this study, recommendations for further investigations are summarized below.

- Further investigations need to be done with respect to the effect of healing properties which have been identified to be a fundamental factor that affects changes in fracture properties of asphalt mixtures.

- Further evaluations need to be done using additional conditioning regarding the effect of cooling without moisture to induce permanent damage on mixtures.
- Further evaluations using different types of mixtures, including Reclaimed Asphalt Pavement (RAP), Reclaimed Asphalt Shingles (RAS), granite mixtures with liquid anti-stripping agent compared with hydrated lime and no anti-stripping additives, are recommended for implementation of the conditioning procedures identified (i.e., HOC + CPPC) to eventually establish design and specification criteria.
- Further investigations using mixtures with modified binders (i.e. SBS or GTR) are recommended for implementation of the conditioning procedures identified since these are typically what are on the surface of the pavement and exposed to the greatest amount of the environmental aging.
- Further development and evaluation of improved mixture property aging models is needed to account for the effect of non-healable permanent damage related to moisture and load, based on additional field and laboratory studies using the conditioning procedures developed in this study.
- Further development of improved pavement performance prediction model based on the enhanced HMA fracture mechanics model (HMA-FM-E) is needed by developing and incorporating relationships to predict changes of mixture properties subjected to HOC and the CPPC.
- The implementation of moisture conditioning procedure for full-scale pavements in APT facility using APAS and HVS loading will lead to more realistic evaluation of the effect of aging on pavement performance.

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APPENDIX A
LABORATORY MIXTURES INFORMATION

Table A-1 Aggregate sources (Granite)

No.	Type of Material	FDOT Code	Producer	Pit	Terminal	Date Sampled
1	# 78 Stone	43	Junction City Mining	GA-553	TM-561	7/28/2005
2	# 89 Stone	51	Junction City Mining	GA-553	TM-561	7/28/2005
3	W-10 Screenings	20	Junction City Mining	GA-553	TM-561	7/28/2005
4	Local Sand	-	V. E. Whitehurst & Sons	Starvation Hill		7/28/2005

Table A-2 Gradation information and blend proportions (Granite)

Sieve (mm)	Sieve Size	Retained Weight (g)				
		1	2	3	4	JMF
19.0	3/4"	0.0	1485.0	1800.0	4050.0	
12.5	1/2"	44.6	1485.0	1800.0	4050.0	45
9.5	3/8"	608.9	1485.9	1800.0	4050.0	565
4.75	# 4	1351.4	1705.5	1800.0	4050.0	962
2.36	# 8	1425.6	1787.4	2475.0	4050.0	831
1.18	# 16	1455.3	1793.7	3105.0	4050.0	666
0.600	# 30	1455.3	1796.9	3487.5	4077.0	413
0.300	# 50	1470.2	1796.9	3690.0	4261.5	402
0.150	# 100	1470.2	1796.9	3825.0	4450.5	324
0.075	# 200	1470.2	1796.9	3892.5	4486.5	104
0	Pan	1485.0	1800.0	4050.0	4500.0	189
G_{sb}		2.809	2.799	2.770	2.626	2.770

Table A-3 Aggregate sources (Limestone)

No.	Type of Material	FDOT Code	Producer	Pit	Terminal	Date Sampled
1	# 67 Stone	42	Rinker Materials Corp.	87-090	TM-447	8/2/2005
2	S-1-B	55	Rinker Materials Corp.	87-090	TM-447	7/29/2005
3	Med. Screenings	21	Rinker Materials Corp.	87-090	TM-447	7/29/2005
4	Local Sand	-	V. E. Whitehurst & Sons	Starvation Hill		7/28/2005

Table A-4 Gradation information and blend proportions (Limestone)

Sieve (mm)	Sieve Size	Retained Weight (g)				
		1	2	3	4	JMF
19.0	3/4"	0	450	2475	4365	
12.5	1/2"	144	450	2475	4365	144
9.5	3/8"	279	632	2475	4365	317
4.75	# 4	423	1645	2475	4365	1157
2.36	# 8	432	2313	2683	4365	885
1.18	# 16	437	2394	3099	4365	501
0.600	# 30	437	2414	3439	4373	369
0.300	# 50	437	2414	3704	4428	320
0.150	# 100	437	2414	4100	4485	454
0.075	# 200	439	2424	4293	4496	216
0	Pan	450	2475	4365	4500	138
G_{sb}		2.335	2.339	2.471	2.626	2.400

APPENDIX B
MIXTURE TEST RESULTS

Table B-1 Superpave IDT test results

Mixture Type	Laboratory Conditioning	Oxidative Aging days	Oxidative Aging hours	m-value	D_1 (1/psi)	S_t (MPa)	M_R (GPa)	FE (kJ/m^3)	$DCSE_f$ (kJ/m^3)	Creep Rate (1/psi-sec)	D(t) (1/GPa)	ε_f ($\mu\varepsilon$)	ER
Granite w/o lime	Heat Oxidation	0	2	0.668	4.77E-07	2.14	10.85	4.20	4.00	3.22E-08	7.055	2566.05	1.34
		5	122	0.532	4.48E-07	2.25	11.99	2.20	2.00	9.40E-09	2.619	1336.78	1.38
		10	242	0.495	4.66E-07	2.64	12.53	3.00	2.72	7.06E-09	2.121	1564.96	2.15
	CPPC (Wet)	0	2	0.556	7.37E-07	2.16	9.97	2.90	2.67	1.90E-08	5.015	1943.23	1.00
		5	122	0.508	5.36E-07	2.10	10.83	1.70	1.50	9.09E-09	2.662	1170.91	1.02
		10	242	0.484	4.18E-07	2.15	12.77	1.40	1.22	5.71E-09	1.777	989.45	1.22
	CPPC (Dry)	0	2	0.592	6.70E-07	2.05	9.66	3.70	3.48	2.38E-08	5.901	2450.31	1.20
		5	122	0.538	5.29E-07	1.99	10.47	1.80	1.61	1.17E-08	3.248	1259.17	0.94
		10	242	0.464	5.38E-07	2.24	12.48	1.80	1.60	6.13E-09	1.995	1184.19	1.40
Granite w/ lime	Heat Oxidation	0	2	0.540	7.35E-07	2.21	11.42	3.20	2.99	1.65E-08	4.487	1963.91	1.22
		5	122	0.502	4.86E-07	2.50	12.72	3.10	2.85	7.80E-09	2.301	1727.23	2.12
		10	242	0.488	2.90E-07	2.52	14.71	1.80	1.58	4.10E-09	1.262	1055.26	2.14
	CPPC (Wet)	0	2	0.579	6.55E-07	1.94	10.59	2.80	2.62	2.07E-08	5.262	1983.17	1.00
		5	122	0.495	4.01E-07	2.48	13.83	2.00	1.78	6.03E-09	1.825	1146.81	1.67
		10	242	0.466	4.00E-07	2.45	13.03	1.80	1.57	4.67E-09	1.506	1052.75	1.77
	CPPC (Dry)	0	2	0.545	7.41E-07	1.96	11.29	3.30	3.13	1.75E-08	4.702	2139.90	1.26
		5	122	0.495	5.60E-07	2.18	12.35	1.70	1.51	8.47E-09	2.562	1103.83	1.05
		10	242	0.478	4.24E-07	2.54	12.84	1.80	1.55	5.50E-09	1.729	1064.43	1.51
Limestone	Heat Oxidation	0	2	0.477	5.42E-07	2.17	11.88	1.60	1.40	6.97E-09	2.176	1167.65	1.12
		5	122	0.385	4.89E-07	2.20	13.62	1.50	1.30	2.69E-09	1.062	1066.45	2.22
		10	242	0.400	3.94E-07	2.39	12.12	1.50	1.26	2.50E-09	0.964	990.86	2.30
	CPPC (Wet)	0	2	0.489	7.26E-07	2.18	8.94	2.10	1.83	1.04E-08	3.149	1373.84	1.02
		5	122	0.447	5.19E-07	2.18	10.82	1.30	1.08	5.10E-09	1.707	943.10	1.10
		10	242	0.400	5.77E-07	2.39	10.78	1.60	1.33	3.65E-09	1.407	997.86	1.66
	CPPC (Dry)	0	2	0.565	6.08E-07	1.77	9.55	1.30	1.14	1.71E-08	4.436	1117.59	0.51
		5	122	0.461	5.76E-07	1.89	10.95	1.20	1.04	6.39E-09	2.068	993.28	0.90
		10	242	0.423	6.57E-07	2.01	10.01	1.40	1.20	5.18E-09	1.836	1040.72	1.15