STATE OF FLORIDA

Evaluating the Use of Lower VMA Requirements for Superpave Mixtures

Research Report
FL/DOT/SMO/04-479

Gregory A. Sholar
Gale C. Page
James A. Musselman
Howard L. Moseley

November 2004

STATE MATERIALS OFFICE

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ABSTRACT

The Florida Department of Transportation specifies coarse graded asphalt mixtures for high traffic roadways with the rationale that coarse graded mixtures will offer better rutting performance compared to fine graded mixtures. Contractors struggle to meet minimum voids in the mineral aggregate (VMA) specification requirements, especially when using aggregates native to Florida. Contractors often gap-grade asphalt mixture gradations to obtain enough void space to meet VMA requirements. It is generally believed that gap-grading an asphalt mixture will be detrimental to the mixture's rutting performance. This study examines the effects on laboratory measured rutting, cracking, moisture sensitivity and permeability of asphalt mixtures that have been designed with gap-graded and continuous gradations with the thought that should the continuous gradation provide better performance, then perhaps the VMA specification requirements should be lowered to allow for this type of gradation.

The Asphalt Pavement Analyzer and Servopac gyratory compactor were used to determine the mixtures' rutting performance. The Superpave indirect tensile tests (IDT) and calculated parameters (energy ratio, dissipated creep strain energy and fracture energy) were used to determine the mixtures' cracking and moisture sensitivity performance. Additional standard laboratory tests were used to evaluate permeability and moisture sensitivity.

Test results indicate that the addition of coarse aggregate on the 12.5 and 9.5 mm sieves of 12.5 mm coarse graded mixtures improved the rutting performance of the

mixtures. However, cracking performance was adversely affected by the addition of coarse aggregate. Moisture sensitivity results varied depending on the test method used. Permeability results were unaffected by the gradation change.

Since cracking is the predominant form of distress for Florida pavements, it is recommended that no change be made to the Department's specifications at this time. Performance test results indicate that not all mixtures perform at their optimum when designed volumetrically. The Department should continue to conduct research and move towards implementation of one or more performance tests to augment or replace volumetric mix design.

CHAPTER 1
INTRODUCTION

1.1 Problem Statement

The Florida Department of Transportation, herein referred to as the Department, adopted the Superpave mix design system in 1996 as a replacement for the Marshall mix design system, which the Department had used since the 1970's. One major difference between the two mix design methodologies is the recommendation in the Superpave system to use coarse graded mixtures for pavements subject to high traffic levels. The Department defines a high traffic level as any pavement that will be subjected to ten million equivalent single axle loads (ESALs) or greater over the pavement's 20-year design period. This is in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Standard Practice for Superpave Volumetric Design of Hot-Mix Asphalt PP 28-03.

The rationale for using coarse graded mixtures on high traffic pavements is for the prevention of rutting. Coarse graded mixtures are typically those in which the gradation curve initially starts above the maximum density line for the larger sieve sizes and then curves below the maximum density line for the smaller sieve sizes. This results in a mixture with more coarse aggregate and more stone-on-stone contact. A coarse aggregate skeleton is created in which the voids are filled with fine aggregate and asphalt binder. Because of this, coarse graded mixtures are thought to provide equal or better resistance to rutting than fine graded mixtures, which have a gradation curve entirely above the maximum density line.

The Superpave mix design system also sets minimum requirements for the mixture property voids in the mineral aggregate (VMA). The VMA is the percent by volume of the air voids plus asphalt binder that has not been absorbed into the aggregate. Commonly, to meet the minimum VMA requirements for coarse graded mixtures, mix designers have to gap grade the mixture by removing a portion of the coarse aggregate from the mix design. This problem is exacerbated for limestone aggregates from Florida, which are less angular, softer, and breakdown more easily than imported granite aggregates or limestones from other states. In general, for a given gradation, an angular aggregate will result in a higher VMA than a less angular aggregate. Additionally, as the aggregate breaks down during the production process, it becomes more rounded and less angular, which results in a reduction in VMA. Since the Department has implemented the Superpave mix design system, asphalt contractors have struggled to meet minimum VMA requirements at the mix design stage and more so during production. The gap grading of the aggregate gradation is necessary to meet the minimum VMA requirements. However, the removal of a portion of the coarse aggregate from the mix design may nullify the benefits of the strong rut resistant coarse aggregate skeleton.

1.2 Objectives

The objectives of the study are as follows:

- Determine the effects on laboratory performance of adding additional coarse aggregate to a mixture’s gradation resulting in a reduction in VMA, which may violate Superpave specifications.
- Based on the results, make recommendations regarding specification changes, further research, or no changes to the current specifications.

1.3 Scope of Work

This research focuses on identifying the laboratory performance difference between mixtures which have been designed to meet Superpave specifications and then subsequently modified by adding more coarse aggregate to the mixtures gradations. The scope of work is as follows:

- Construct four Superpave mix designs using aggregates from different geological sources that are commonly used in Florida. The mixtures will all be 12.5 mm coarse graded mixtures since this is the most common coarse mixture type used by Contractors performing work for the Department. The mixtures will be gap graded to match common practice by mix designers.

- Determine the laboratory performance of the four mixtures by using tests that give an indication of a mixtures resistance to rutting, cracking, moisture sensitivity and permeability.

- Modify the gradations of the four mixtures evaluated in the first objective to provide more coarse aggregate on the 12.5 mm and 9.5 mm sieve sizes. This will result in a reduction of VMA, which may be less than the minimum specified value. These four mixtures will then be evaluated using the same laboratory performance tests used to evaluate the unmodified mixtures.

- Compare the performance between the unmodified and modified mixtures to ascertain the effects of the addition of coarse aggregate on a mixture's performance.

- Evaluate the results and make recommendations.

1.4 Research Plan

The following items constitute the research plan for this study:

- A literature review was conducted.

- Four aggregate types were selected for study: Alabama limestone, Florida limestone from the Brooksville area, Nova Scotia granite, and Florida limestone from the Miami area (Tarmac mine). These aggregate types are commonly used in Florida and represent a wide range of softness and angularity.

- Four mixtures were designed to meet Superpave mix design criteria. All of the mixtures were 12.5 mm coarse graded mixtures and were gap graded. Each mixture contained only one aggregate type.

- The Brooksville limestone mixture met all Superpave mix design criteria except for the minimum VMA requirement. Brooksville limestone is a soft Florida limestone that cannot be used solely to construct a 12.5 mm coarse graded Superpave mixture and meet minimum VMA requirements. This aggregate type was chosen intentionally so that a mixture not able to meet VMA criteria could be evaluated in terms of performance.

- The following laboratory tests were used to ascertain rutting performance: the asphalt pavement analyzer with the conventional and modified analysis approach and the ServoPac gyratory compactor to measure shear stress, gyratory shear slope and strain.

- The following laboratory test was used to ascertain cracking performance: the Superpave indirect tension test.

- The following laboratory tests were used to ascertain moisture sensitivity performance: tensile strength, the Superpave indirect tension test and a falling head permeability test.

- Each of the four mixtures was then modified by adding more 12.5 mm and 9.5 mm coarse aggregate. The resulting gradations were more continuously graded and less gap graded than the unmodified mixtures. A reduction in VMA occurred for each mixture.

- The modified mixtures were then evaluated with the same laboratory tests used to evaluate the unmodified mixtures.

- The data was analyzed and conclusions and recommendations were made.

CHAPTER 2
BACKGROUND

2.1 Definition of VMA

Voids in the mineral aggregate (VMA) is a volumetric property and is the sum of the air voids in the mixture plus the amount of asphalt binder that has not been absorbed into the aggregates. This unabsorbed binder is termed the "effective binder." The concept of VMA is illustrated in Figure 2-1.

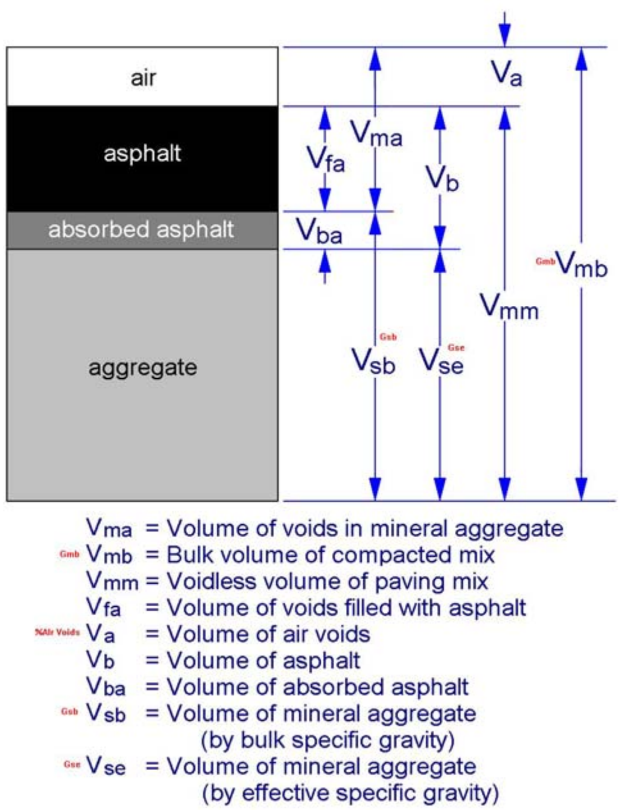

Figure 2-1. Volumetric diagram

2.2 Coarse and Fine Gradations

The Superpave mixture design system designates mixtures as either coarse or fine. As mentioned previously, this study focuses only on coarse graded mixtures, which are thought to have equal or better rutting resistance compared to fine graded mixtures. Coarse graded mixtures have gradation curves that start above the maximum density line and curve downward below the restricted zone, whereas fine graded mixtures have gradation curves which lie solely above the maximum density line. The maximum density line represents the gradation that would result in the densest possible arrangement of the aggregate particles. Superpave defines the restricted zone as an area where the gradation should not pass through. Gradations that pass through this zone have the potential to contain natural rounded sands which may inhibit good rutting performance (Asphalt Institute 1996). An example of a coarse and fine gradation is shown in Figure 2-2.

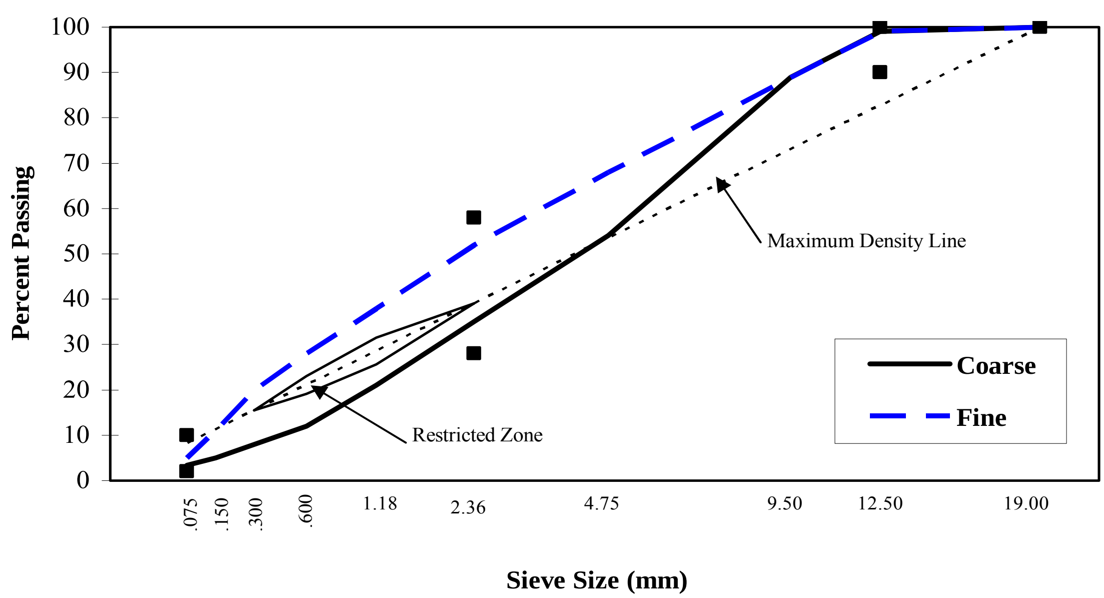

Figure 2-2. Coarse and fine gradations

The aggregate gradation curve and its distance away from the maximum density line are related to the VMA of a mixture. More area between the gradation curve and the maximum density line increases the VMA potential of the mixture.

2.3 Nominal Maximum Aggregate Size

The Superpave mix design system designates a mixture by its nominal maximum aggregate size (NMAS). The NMAS is defined to be the sieve which is one sieve size larger than the first sieve to retain more than ten percent of the aggregate by weight. All of the mixtures used in this study are 12.5 mm mixtures, which means that more than ten percent of the aggregate is retained on the 9.5 mm sieve.

Minimum VMA requirements are based on the NMAS of the mixture. A smaller NMAS mixture, for example a 9.5 mm mixture, has a higher VMA requirement than a larger NMAS mixture, such as a 19.0 mm mixture. This is because the total surface area of the aggregates is greater for the smaller NMAS mixture as compared to the larger NMAS mixture. More aggregate surface area requires more asphalt binder to coat the aggregates and hence the specified minimum VMA is greater.

However, Superpave does not differentiate between coarse and fine gradations with respect to the VMA requirement. Both types of mixtures have the same minimum VMA requirement for a given NMAS. Coarse graded mixtures have more coarse aggregate in proportion to fine aggregate than fine graded mixtures. Therefore, there is less aggregate surface area in a coarse graded mixture as compared to a fine graded mixture for a given NMAS. Given the same VMA requirement, mix designers are then forced to gap grade the mixture to provide ample volume between the aggregate particles to contain the required four percent air voids and effective asphalt binder needed to meet the minimum VMA requirement.

2.4 Literature Review

2.4.1 Historical Perspective

Modern mix design methods can generally be dated back to the 1940's with the most predominant method being the Marshall mix design method. Marshall had different views regarding VMA than other asphalt technologists at the time. Marshall believed that VMA should be reduced to the lowest possible level and did not believe in establishing specification limits for VMA (Leahy and McGennis 1999).

In 1957, Norman McLeod presented a paper to the Highway Research Board emphasizing the importance of using the aggregate bulk specific gravity in the calculation of VMA instead of the effective specific gravity, which was common at the time (Leahy and McGennis 1999). McLeod also believed that VMA should be specified as a minimum value of 15 percent with design air voids at five percent using the 75-blow Marshall method. No performance data was used by McLeod to determine this VMA limit. McLeod proposed VMA requirements based on nominal maximum aggregate size, which were adopted by the Asphalt Institute in 1964. The current Superpave mix design system specifies VMA based on McLeod's recommendations but has adjusted them lower by one percent realizing that McLeod designed asphalt mixtures at five percent air voids and the Superpave system requires four percent air voids (Kandhal and Chakraborty 1996).

Coree and Hislop (1999) conducted a thorough review of literature regarding VMA and found that there is little historical basis, if any, to support the VMA values currently specified. Minimum VMA requirements that are the same for all gradations of a particular NMAS can cause well performing mixtures to be rejected. They suggest the possible use of a minimum asphalt film thickness as a replacement for VMA. The researchers also recommend that VMA requirements or asphalt film thicknesses be validated against field performance and that enforcement of any VMA specification not be rigidly enforced due to the imprecision in current test methods.

2.4.2 Recent Research

Researchers have come to recognize that VMA criteria based on NMAS alone is not adequate and that an approach based on asphalt film thickness is more rational. Work by Kandhal and Chakraborty (1996) examined film thicknesses ranging from 4 to 13 microns for one 12.5 mm coarse graded mixture containing limestone. Mixtures were compacted to eight percent air voids and short and long term aged. Mixture tests included resilient modulus and tensile strength and binder tests included viscosity, penetration and complex modulus. The researchers' conclusion was that a minimum film thickness of 9 to 10 microns is desirable to minimize accelerated aging.

Work also conducted by Kandhal et al. (1998) emphasized that coarse graded mixtures are penalized by current Superpave requirements because the VMA requirement is the same for coarse and fine graded mixtures. This results in thicker than necessary film thicknesses for coarse graded mixtures. As mentioned previously, work done by Kandhal and Chakraborty (1996) indicated an optimum film thickness of 9 to 10 microns at eight percent air voids. This study recommended a minimum asphalt film thickness of eight microns for mixtures compacted between four and five percent air voids, which would better represent the in-place density achieved in the roadway. The researchers' reasoning for the lower film thickness is that at four to five percent air voids there would be less aging of the binder. Based on a film thickness of eight microns, coarse graded mixtures had VMA values up to two percent lower than fine graded mixtures using the same aggregate type.

Kandhal and Mallick (2001) investigated the effect of aggregate gradation and aggregate type on the rutting potential of asphalt mixtures. Tests conducted in the asphalt pavement analyzer (APA) indicated that for the limestone and granite mixtures, an increase in VMA resulted in an increase in rut depth. The trend was opposite for the river gravel mixture studied. The same trends were observed when comparing voids filled with asphalt (VFA) to the APA rut depths.

Hand et al. (2001) conducted a study measuring the rut resistance of 21 granite and limestone mixtures of varying gradations using the PURWheel laboratory rut tester and triaxial shear strength. The researchers concluded that maximum rut resistance as determined by these two tests was achieved at an asphalt binder content 0.5 percent below the value determined in the Superpave mix design process. The additional 0.5 percent asphalt binder can be attributed to minimum Superpave VMA requirements.

Sholar et al. (2001) conducted a study measuring the effects of aggregate degradation throughout the production process on the volumetric properties of asphalt mixtures. Three aggregate types commonly used in Florida (Georgia granite, southeast Florida limestone, and west-central Florida limestone) were evaluated representing a range of hard to soft aggregates respectively. Aggregate gradations were examined at five points in the production process. Belt cut samples were obtained, asphalt mixture was obtained from the truck bed, asphalt mixture from the same truck was obtained from behind the paver but prior to compaction, asphalt mixture was obtained after roller compaction, and gradations were determined from gyratory compacted samples. Some of the conclusions from the research were:

- Aggregate breakdown was directly related to Los Angeles Abrasion values. The two limestone mixtures degraded significantly more than the granite mixture.

- An average reduction of 0.5 percent VMA would be expected to occur for every one percent of dust (material passing the 0.075 mm sieve) that was generated due to breakdown.

Coree and Hislop (2001) conducted additional research to determine the aggregate factors related to the critical VMA for a mixture. They determined the critical VMA by using the Nottingham Asphalt Tester, which is a repeated load triaxial test. The researchers determined the critical point by examining strain data at multiple asphalt contents and selecting the asphalt content and corresponding VMA where strain started to increase. They identified this point to be where the mixture would go from sound to unsound behavior in terms of permanent deformation. Only three out of 28 mixtures were correctly identified based on VMA design criteria alone. It was determined that the volume of effective binder is more reliable (ten out of 28 mixtures) than VMA alone. Aggregate factors that correlated well with the critical VMA were fineness modulus, the percent of crushed coarse aggregate and the percent of crushed fine aggregate.

Anderson (2001) conducted a study comparing the performance of 12.5 mm coarse and fine graded mixtures composed of Illinois Dolomite with each mixture designed with 13 and 15 percent VMA. Anderson had the following conclusions:

- Using the shear frequency sweep test (for rutting characterization) and the shear fatigue test, the high temperature stiffness and critical temperature and the shear fatigue characteristics of the coarse mixture decreased substantially as the VMA increased. These tests suggest that the coarse mixture with 15 percent VMA would be more susceptible to rutting and fatigue cracking than the coarse mixture with 13 percent VMA.

- Repeated shear testing (for rutting characterization) and flexural beam fatigue testing (for fatigue characterization) indicated that a reduction of VMA from 15 percent to 13 percent should not affect the performance characteristics of the coarse mixture.

- An increase in VMA from 13 percent to 15 percent for the fine graded mixture improved the shear fatigue characteristics by 50 percent while only reducing the high temperature stiffness and rutting characteristics by no more than 30 percent.

- The coarse mixture appeared much more sensitive to VMA changes than the fine mixture.

Ruth and Birgisson (1999) identified several factors of high quality mixtures that would make them relatively insensitive to changes during production. They emphasized the importance of a continuously graded mixture that did not have an excess or deficiency on any one sieve size. They also believed that the gradation should generally not be gap graded.

Ruth et al. (2002) used tensile strength, fracture energy and failure strain from the Superpave indirect tension test to evaluate mixtures with a variety of gradations and determined that continuously graded mixtures outperformed mixtures that were gap graded or had an excess or deficiency on any one sieve size, confirming the research performed by Ruth and Birgisson (1999).

Nukunya et al. (2002) performed a comprehensive study regarding VMA and presented the following findings:

- Mixture performance must be evaluated through the use of physical tests and gradation analysis in addition to volumetric analysis.
- Current methods of calculating VMA and asphalt film thickness are ineffective across all cases. A new approach calculating effective VMA and effective film thickness based on only the portion of the mixture passing the 2.36 mm sieve was presented.
- The percent of fine aggregate, not coarse aggregate, in a mixture appears to control binder age hardening.
- Coarse graded mixtures develop pockets of fine aggregate and asphalt binder, which make current methods for calculating film thickness and VMA irrelevant for coarse graded mixtures but relevant for fine graded mixtures.
- Low effective film thickness and low effective VMA have a more pronounced effect on fine graded mixtures than coarse graded mixtures. The fine graded mixtures with low effective film thickness and VMA lose their flexibility and become more brittle during aging.

- The minimum VMA requirements for coarse graded mixtures may result in excessive asphalt leading to higher rutting based on high creep values and low shear resistance.

- The current Superpave criteria for a minimum VMA for coarse graded mixtures could be discontinued as long as other aggregate controls were instituted to limit mix designers from using inferior (soft) aggregates.

CHAPTER 3
MATERIALS AND TESTING METHODS

3.1 Introduction

This chapter provides information on the materials and test procedures used in this research project. It includes properties of the materials, how the materials were combined, the test procedures performed on the materials, and the analysis methods used.

3.2 Materials

3.2.1 Asphalt Binder

A Superpave performance graded binder, PG 67-22, from El Paso Merchant Energy Petroleum (formerly known as Coastal Fuels) in Jacksonville, FL was used for this research project. This grade of binder is the standard unmodified binder used for Department projects. The binder contained no anti-stripping agent. The asphalt binder specific gravity was 1.03. The binder was sampled into ten 5-gallon buckets.

3.2.2 Aggregates

Four types of aggregate were used for this study: Alabama limestone, limestone from the Brooksville, FL area, granite from Nova Scotia, and limestone from the Miami, FL area (Tarmac mine). Each aggregate type was the basis for each asphalt mix design studied. All aggregates used for this study were 100 percent crushed aggregates, which is very common for Department work. All mix designs, except the Brooksville limestone mix design, were based on actual mix designs submitted for approval to the Department. Contractors do not submit 100 percent Brooksville aggregate mix designs because it is not possible to meet Superpave VMA criteria as discussed in Chapter 1, Section 1.4.

Aggregate types were not intermingled and no reclaimed asphalt pavement was used. All aggregate components for each mix design were oven dried and fractionated into individual sieve sizes from the 19.0 mm sieve to the 0.075 mm sieve prior to batching. Fractionating into all sieve sizes provided optimal control of achieved gradations and assured consistency between batches. It should be noted that material below the 2.36 mm sieve was typically not present for coarse aggregate components. The convention used throughout this paper will be that “Round 1” refers to the gap graded mixture which conforms to Superpave criteria. “Round 2” refers to the modified gradation that contains more coarse aggregate and is more continuously graded, yet reduces the VMA of the mixture. Each aggregate type will be discussed below.

3.2.2.1 Alabama limestone

The Alabama limestone asphalt mixture was composed of three aggregate components:

- Number 7 stone from Southern Ready Mix, FDOT code 44, pit number AL-485.
- S-1-B stone from Southern Ready Mix, FDOT code 51, pit number AL-526.
- Screenings from Vulcan Materials Corporation, FDOT code 22, pit number AL-149.

The aggregate components were proportioned to give the following gradations for rounds 1 and 2 and are shown in Table 3-1 and Figure 3-1.

Table 3-1 Gradations for Alabama limestone mixtures

Sieve Size (mm)	Percent Passing	
	Round 1	Round 2
19.0	100	100
12.5	100	92
9.5	89	82
4.75	54	54
2.36	35	35
1.18	22	22
0.600	16	16
0.300	8	8
0.150	5	5
0.075	3.4	3.4

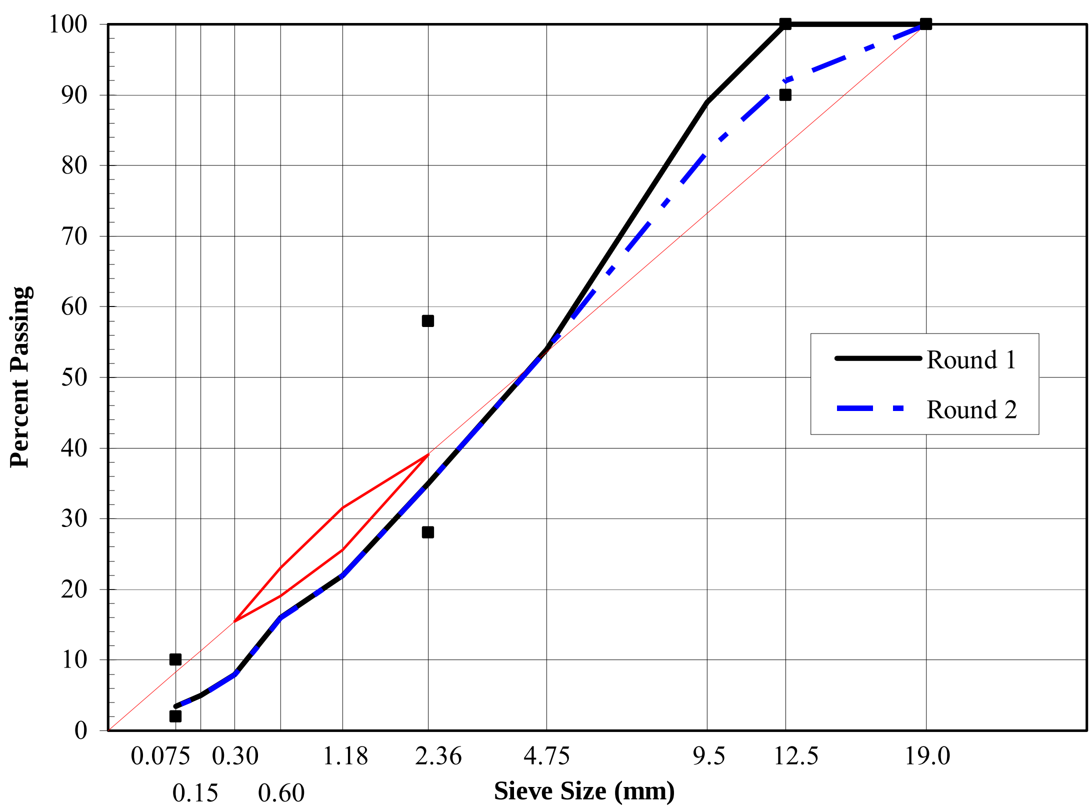

Figure 3-1 Gradation plots for Alabama limestone mixtures

3.2.2.2 Brooksville limestone

The Brooksville limestone asphalt mixture was composed of three aggregate components:

- S-1-A stone from Florida Crushed Stone, FDOT code 46, pit number 08-012.
- S-1-B stone from Florida Crushed Stone, FDOT code 52, pit number 08-012.
- Screenings (130A) from Florida Crushed Stone, FDOT code 24, pit number 08-012.

The aggregate components were proportioned to give the following gradations for rounds 1 and 2 and are shown in Table 3-2 and Figure 3-2. These gradations are very similar to the gradations for the other three aggregate types.

Table 3-2 Gradations for Brooksville limestone mixtures

Sieve Size (mm)	Percent Passing	
	Round 1	Round 2
19.0	100	100
12.5	98	92
9.5	89	82
4.75	55	55
2.36	32	32
1.18	22	22
0.600	14	14
0.300	9	9
0.150	7	7
0.075	5.3	5.3

3.2.2.3 Nova Scotia granite

The Nova Scotia granite asphalt mixture was composed of three aggregate components:

- Number 7 stone from Martin Marietta, FDOT code 44, pit number NS-315, terminal TM-322.
- Number 89 stone from Martin Marietta, FDOT code 54, pit number NS-315, terminal TM-322.
- Screenings from Martin Marietta, FDOT code 22, pit number NS-315, terminal TM-322.

The aggregate components were proportioned to give the following gradations for rounds 1 and 2 and are shown in Table 3-3 and Figure 3-3.

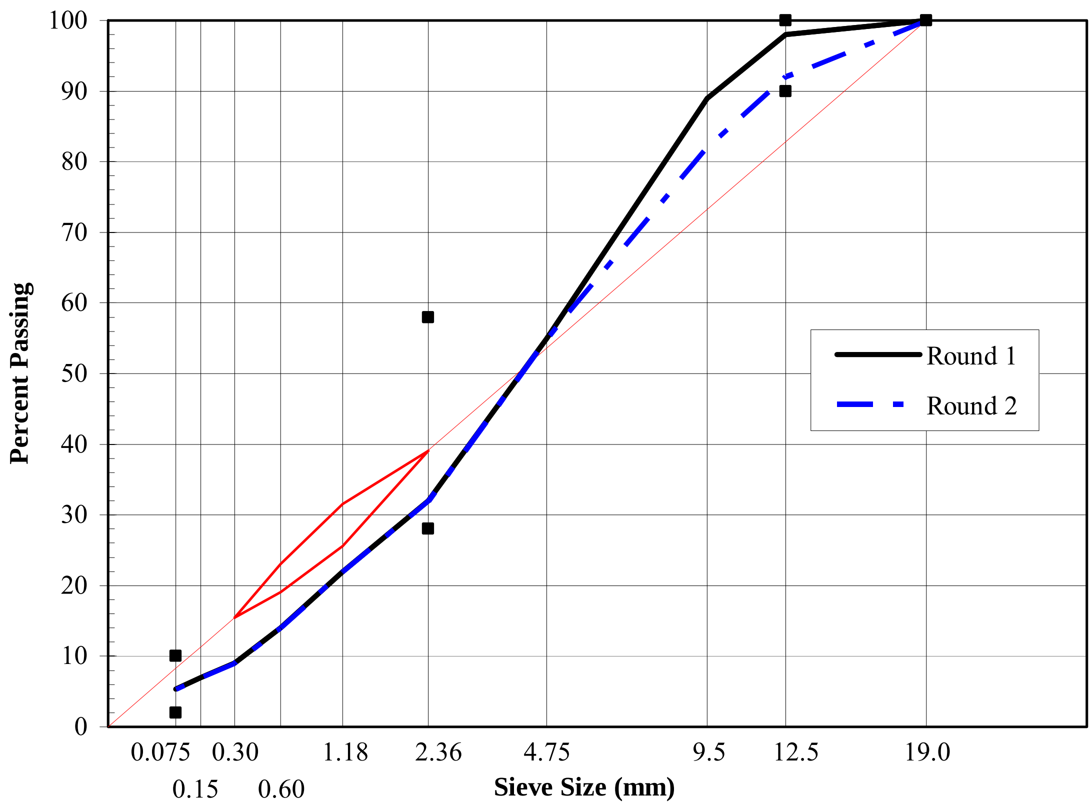

Figure 3-2 Gradation plots for Brooksville limestone mixtures

Table 3-3 Gradations for Nova Scotia granite mixtures

Sieve Size (mm)	Percent Passing	
	Round 1	Round 2
19.0	100	100
12.5	98	92
9.5	89	82
4.75	58	58
2.36	38	38
1.18	24	24
0.600	16	16
0.300	10	10
0.150	7	7
0.075	5.3	5.3

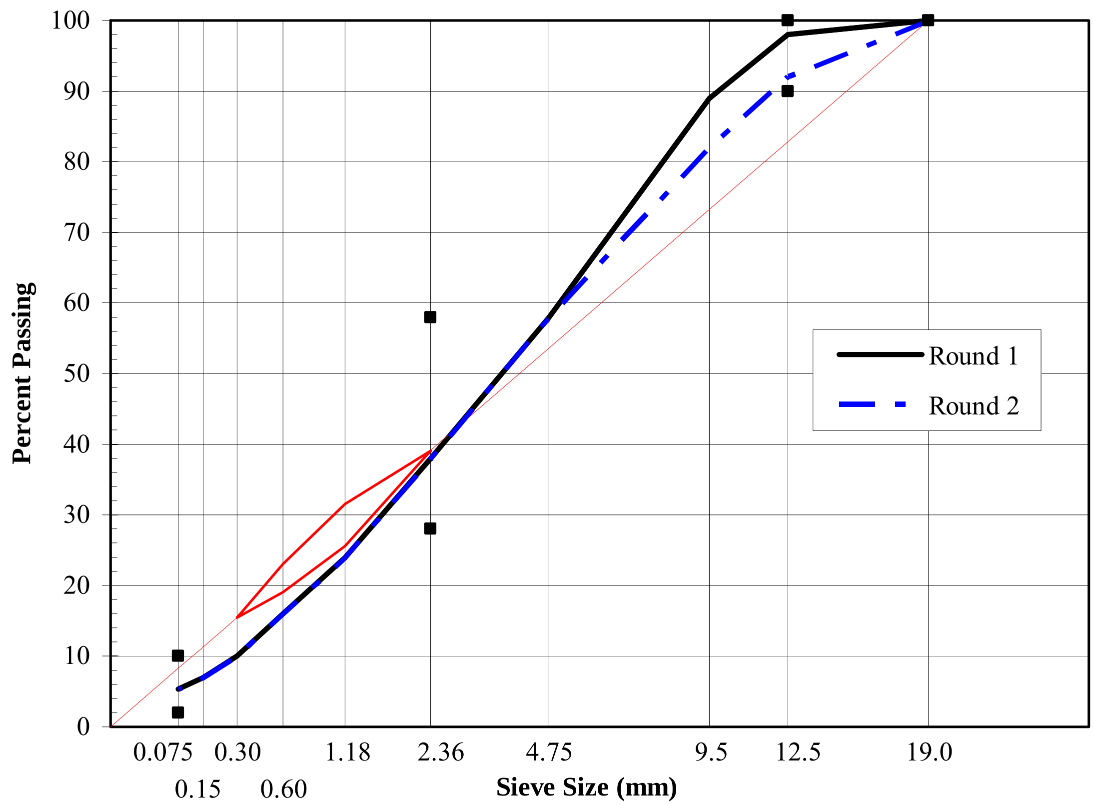

Figure 3-3 Gradation plots for Nova Scotia granite mixtures

3.2.2.4 Miami limestone (Tarmac mine)

The Tarmac limestone asphalt mixture was composed of four aggregate components:

- S-1-A stone from Tarmac America, FDOT code 42, pit number 87-145.
- S-1-B stone from Tarmac America, FDOT code 51, pit number 87-145.
- 5/16 inch stone from Tarmac America, FDOT code 56, pit number 87-145.
- Screenings from Tarmac America, FDOT code 22, pit number 87-145.

The aggregate components were proportioned to give the following gradations for rounds 1 and 2 and are shown in Table 3-4 and Figure 3-4.

Table 3-4 Gradations for Tarmac limestone mixtures

Sieve Size (mm)	Percent Passing	
	Round 1	Round 2
19.0	100	100
12.5	98	92
9.5	89	82
4.75	55	55
2.36	32	32
1.18	25	25
0.600	18	18
0.300	13	13
0.150	7	7
0.075	5.3	5.3

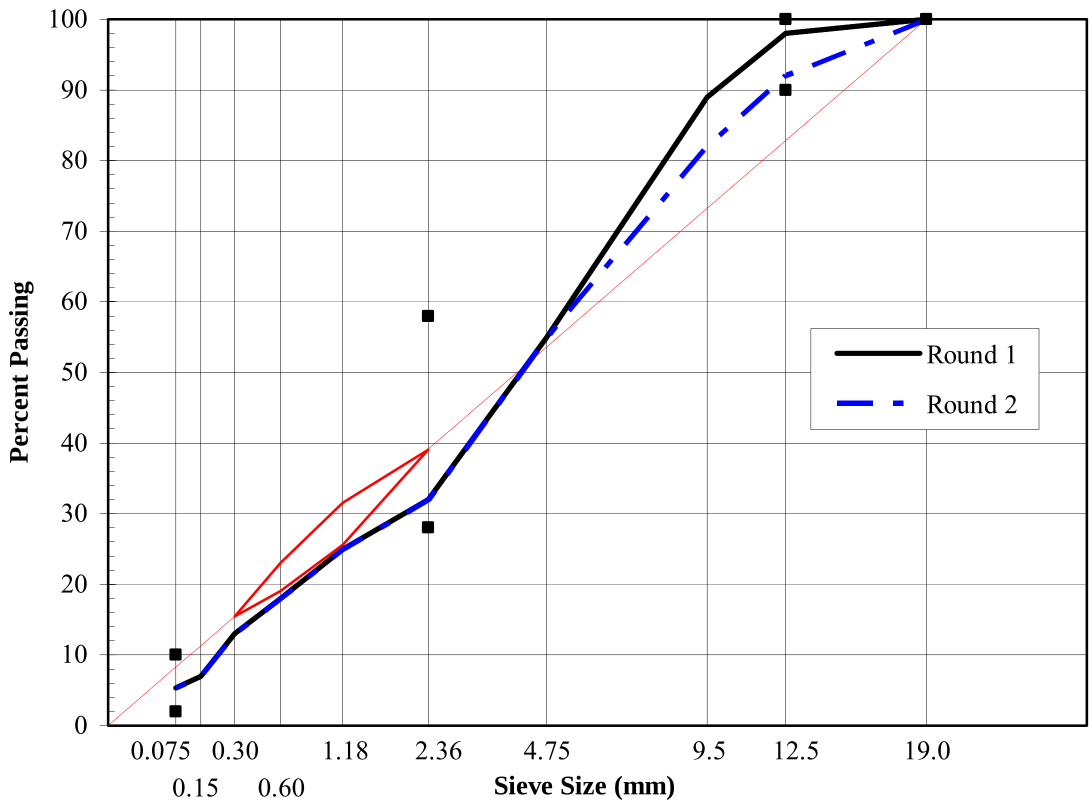

Figure 3-4 Gradation plots for Tarmac limestone mixtures

3.3 Testing Methods

Testing methods for this study can be categorized into six classifications: mix design, moisture sensitivity testing, permeability testing, Asphalt Pavement Analyzer testing, Servopac gyratory compactor testing, and Superpave indirect tension testing. Each classification will be discussed below with respect to the test procedures used and the techniques used to analyze the data.

3.3.1 Mix Design Testing

The design of the mixtures followed standard Superpave practice, which is governed by four American Association of State Highway and Transportation Officials (AASHTO) standards:

- Superpave Volumetric Design for Hot-Mix Asphalt (HMA), AASHTO designation PP 28-03. This practice outlines the overall design procedure from materials selection, designing the aggregate structure, selecting the design binder content, and evaluating the mixture for moisture sensitivity.

- Superpave Volumetric Mix Design, AASHTO designation MP 2-03. This specification gives detailed requirements for binder selection, aggregate gradation criteria, aggregate consensus property requirements, and mixture property criteria based on traffic level.

- Preparing and Determining the Density of Hot-Mix Asphalt (HMA) Specimens by Means of the Superpave Gyratory Compactor, AASHTO designation T 312-03. This standard method of test discusses specific requirements of the gyratory compactor, the compaction procedure, and density determination. For this study, all mix design specimens were gyrated in a Pine AFGC125X gyratory compactor.

- Mixture Conditioning of Hot-Mix Asphalt (HMA), AASHTO designation R 30-02. This practice outlines mixture conditioning for volumetric mix design and short and long-term conditioning for mechanical property testing.

Specific Department test procedures needed during the mix design process were used to determine aggregate and mixture properties and are discussed below:

- Sieve Analysis of Coarse and Fine Aggregate, Florida Method of Test FM 1-T 027. This test method describes the procedure for performing a sieve analysis on coarse or fine aggregate to determine a gradation.

- Specific Gravity and Absorption of Fine Aggregate, Florida Method of Test FM 1-T 084. This test method describes the procedure for determining the bulk specific gravity and absorption of fine aggregates.

- Specific Gravity and Absorption of Coarse Aggregate, Florida Method of Test FM 1-T 085. This test method describes the procedure for determining the bulk specific gravity and absorption of coarse aggregates.

- Bulk Specific Gravity of Compacted Bituminous Mixtures, Florida Method of Test FM 1-T 166. This test method describes the procedure for determining the bulk specific gravity of compacted asphalt mixtures, such as gyratory specimens.

- Maximum Specific Gravity of Asphalt Paving Mixtures, Florida Method of Test FM 1-T 209. This test method describes the procedure for determining the maximum specific gravity of uncompacted asphalt mixtures.

As mentioned previously, individual aggregate components were fractionated to every sieve size to provide better accuracy and consistency in batching. Two fine and two coarse aggregate specific gravity tests were conducted for each aggregate component and the individual values combined mathematically to obtain bulk specific gravity values for the composite gradation, otherwise known as the job mix formula (JMF).

Following standard Superpave guidelines, mixtures were designed with four percent air voids at the design number of gyrations while also meeting specification requirements for VMA, VFA, and dust/effective binder ratio. The design number of gyrations for all mixtures was 100. The specified minimum VMA requirement was 14.0. The VFA requirement was the range of 65 to 75 percent. The specified dust to effective binder content ratio was the range 0.8 to 1.6. Once the design binder content had been selected, then additional specimens were prepared with binder contents modified by the following amounts: +1.0, +0.5, -0.5 and -1.0 percent binder. Three asphalt specimens were made at each binder content. Having volumetric design data at five asphalt binder contents provided enough information to construct an adequate VMA curve for each mixture.

3.3.2 Moisture Sensitivity Testing

Moisture sensitivity testing is a routine function in the Superpave mix design procedure and was performed for all mixtures in this study. The main reason for performing this test was to obtain a relative measurement of the mixture's resistance to moisture damage between rounds one and two of a particular aggregate type, not necessarily between mixtures of different aggregate types. The addition of more coarse aggregate, resulting in a more continuous gradation closer to the maximum density line, was thought to perhaps reduce the permeability of the mixture and reduce the susceptibility to moisture damage.

The test method used to determine the moisture susceptibility of a mixture was Resistance of Compacted Bituminous Mixture to Moisture-Induced Damage, Florida Method of Test FM 1-T 283. The basic test procedure is performed as follows:

- Samples are batched in the laboratory to a predetermined weight that will result in compacted specimens of 7.0 +/- 1.0 percent air voids. A minimum of six 100 mm diameter specimens are gyrated to a height of approximately 65 mm.
- Three specimens are broken in the unconditioned state at 25 °C in the indirect tensile mode at a rate of 50 mm per minute. The Pine breaking apparatus typically used to determine stability and flow values for Marshall mix design was used for this test.
- Three different specimens are conditioned by vacuum saturating the specimens underwater to a condition of 70 to 80 percent saturation.
- These three specimens are then frozen at -18 °C for a minimum of 16 hours and then placed in a water bath at 60 °C for 24 hours. The specimens are then placed in a chamber at 25 °C for two hours.
- These three specimens are then broken in the indirect tensile mode at a rate of 50 mm per minute.
- Peak loads obtained from the indirect tension testing are used to calculate diametral tensile strength.

- A tensile strength ratio is obtained by dividing the average tensile strength in the conditioned state by the average tensile strength in the unconditioned state.

In addition to the approach mentioned above, the data from the Superpave indirect tension test was also used to evaluate moisture sensitivity. This will be discussed in a subsequent section.

3.3.3 Permeability Testing

Like the moisture sensitivity testing described above, permeability testing was performed to obtain a relative measurement of the mixture's resistance to water permeability between rounds one and two of a particular aggregate type, not necessarily between mixtures of different aggregate types. The addition of more coarse aggregate, resulting in a more continuous gradation closer to the maximum density line, was thought to perhaps reduce the permeability of the mixture.

The test method used to determine the permeability of a mixture was Measurement of Water Permeability of Compacted Asphalt Paving Mixtures, Florida Method of Test FM 5-565, with the addition of a vacuum saturation. The basic test procedure is performed as follows:

- Samples are batched in the laboratory to a predetermined weight that will result in compacted specimens of 7.0 +/- 0.5 percent air voids when compacted to a height of approximately 115 mm. Specimen diameter is 150 mm. Three specimens are used for permeability testing.
- The top 50 mm of each gyratory specimen is then removed from the remaining portion of the specimen by saw cutting using a diamond tipped blade which is cooled and lubricated with a stream of water. This prevents smearing of the asphalt binder during the cut, which would clog the permeable pores.
- The samples are then vacuum saturated under water for five minutes at a vacuum of 380 mm of mercury.
- The samples are then placed in the falling head permeability apparatus shown in Figure 3-5.

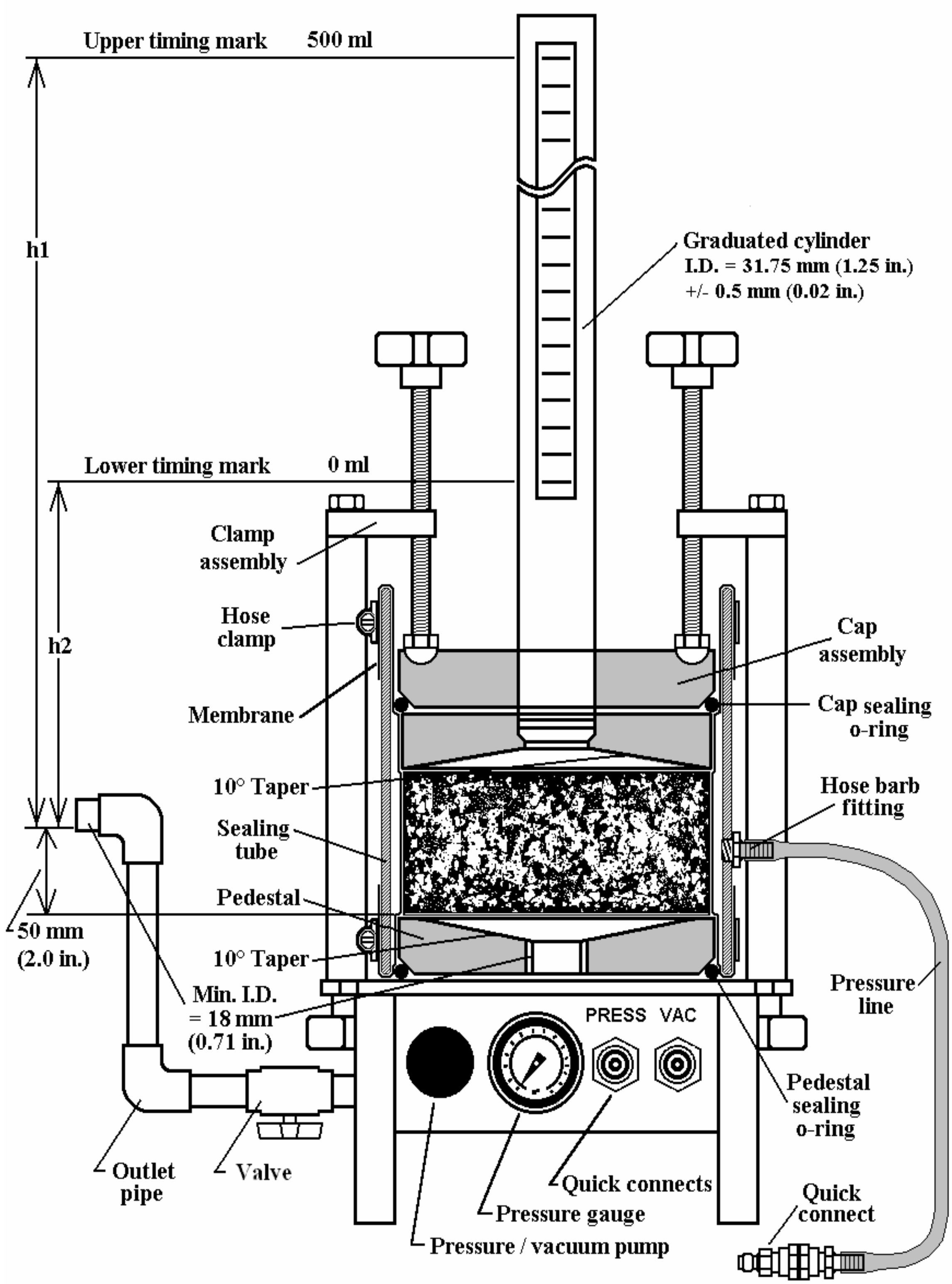

Figure 3-5 Permeability test apparatus

- The time is recorded to flow 500 milliliters of water through the specimen. Additionally, the water temperature is recorded so that a temperature correction factor can be applied to correct the permeability readings to a standard reference temperature of 20 °C.

- The permeability value for the three specimens is then averaged to obtain an average permeability value for the mix design.

3.3.4 Asphalt Pavement Analyzer Testing

The Asphalt Pavement Analyzer (APA) was one of two devices used to assess the rutting performance of the asphalt mixtures. The other device was the Servopac gyratory compactor, which will be discussed in a subsequent section. The APA is essentially a wheel tracking device that applies a repeating load to a cylindrical asphalt specimen and the rut depth is determined after 8,000 cycles, or 16,000 passes (Figure 3-6). The test procedure followed is Determining Rutting Susceptibility of Asphalt Paving Mixtures Using the Asphalt Pavement Analyzer (APA), AASHTO designation TP 63-03.

The highlights of the test procedure are given below.

- Samples are batched in the laboratory to a predetermined weight that will result in compacted specimens of 7.0 +/- 1.0 percent air voids when compacted to a height of 75 mm. Specimen diameter is 150 mm. Four specimens are used for APA testing.

- As a comparison, additional samples were batched in the laboratory to a predetermined weight that would result in compacted specimens of 4.0 +/- 1.0 percent air voids when compacted to a height of 115 mm at 100 gyrations. Specimen diameter is 150 mm. Four specimens were used for APA testing.

- Specimens are placed in the APA molds (two per mold) with the top side of the specimens facing up. The top side of the specimen is the side that was in contact with the ram head of the gyratory compactor. Specimens (in the molds) are then heated to 64 °C for approximately 16 hours.

- The specimens are then placed in the 64 °C heated APA testing chamber where a seating load of 25 cycles is applied to the specimens. The load is comprised of a 445 N load applied on top of a 19.0 mm diameter hose inflated to 700 kPa (Figure 3-7).

Figure 3-6 Asphalt Pavement Analyzer

- A measuring template is then placed on the top of the mold and an initial depth reading is obtained using a digital measuring device. The template contains four measuring slots, two per specimen.

- The specimens are then placed in the 64 °C heated testing chamber and 8000 additional load cycles are applied, as described in step four.

- A final depth reading is then obtained at each of the four measuring slots. The rut depth is taken as the difference between the initial and final readings.

In addition to the method described above for measuring the rut depths, a recently developed method for measuring the rut profile (Drakos 2003) was used for the seven percent air void specimens. Instead of using the conventional digital measuring device with a small roller on the end to measure a single point maximum rut depth, the new

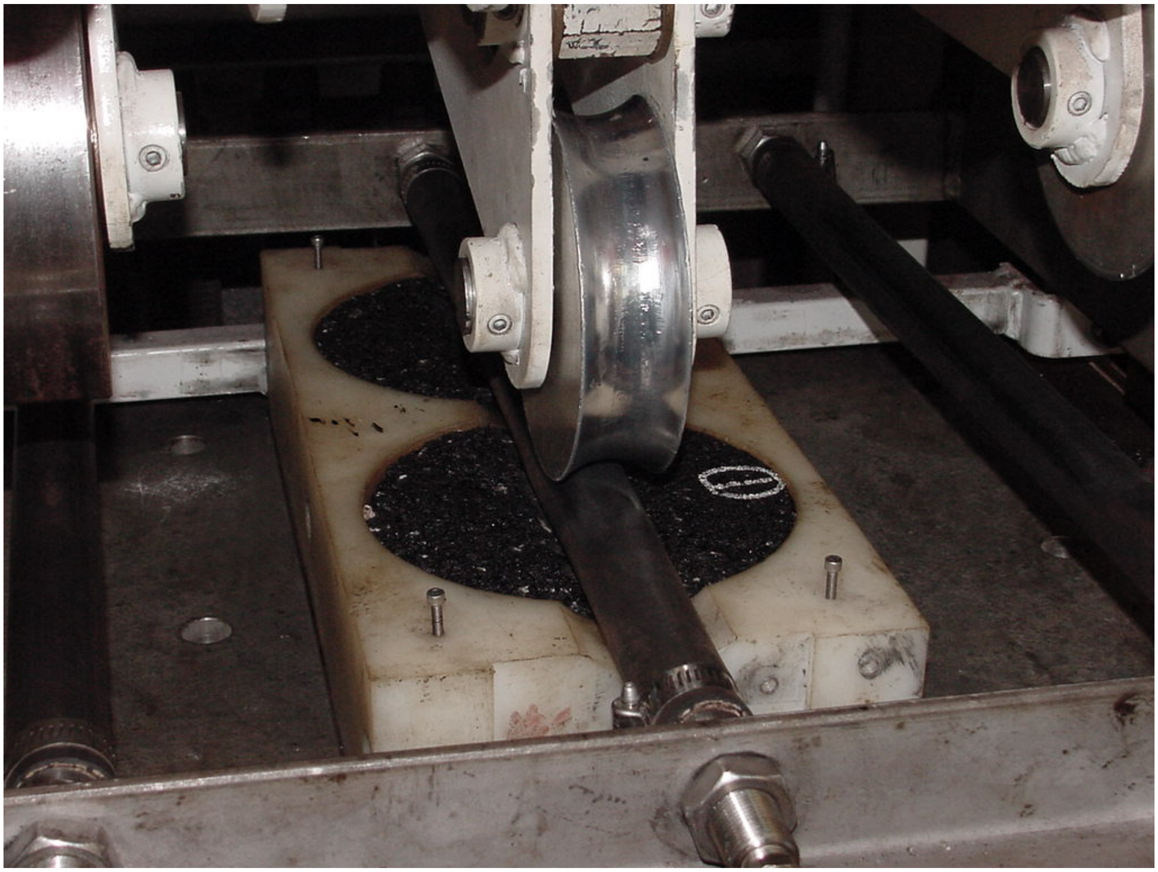

Figure 3-7 Asphalt Pavement Analyzer loading apparatus

method uses a modified measuring plate and contour gage that measures the entire rut profile. The profile is measured at three longitudinal locations for each cylindrical specimen (Figure 3-8).

The contour gage is then placed in a specially made holder and the contour is traced onto a paper card. The specially made holder establishes a consistent orientation and reference system for each rut profile that is traced (Figure 3-9).

The line trace on the card is then electronically scanned and a best fit line is fitted to the electronic trace using computer software. Through integration of the equation of the line, the area between the line and the x-axis is determined. This procedure is conducted for the initial trace after 25 rut cycles and the final trace after 8000 additional

Figure 3-8 Measuring plate and contour gage for modified measuring technique

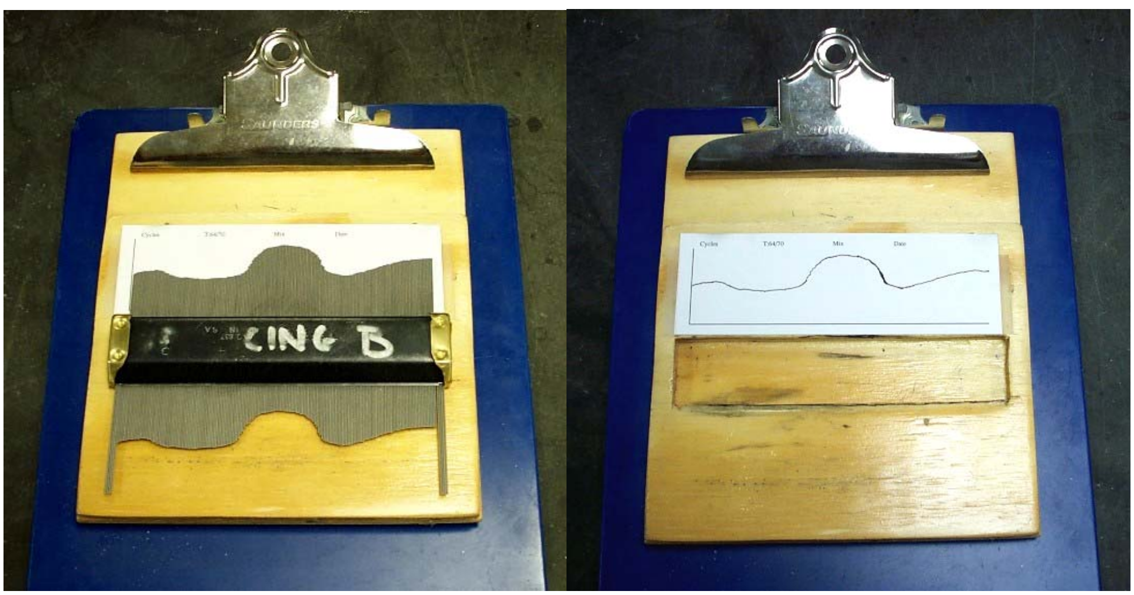

Figure 3-9 Holder, contour gage and rut profile trace

cycles. The initial area is then subtracted from the final area and a percent area change is determined. If the percent area change is positive, then Drakos (2003) concluded that instability rutting has occurred and if the percent area change is negative, then consolidation rutting has occurred.

In addition to the area change, the maximum single point absolute rut depth (ARD) and the maximum single point differential rut depth (DRD) can be determined from the profile traces. This is illustrated in Figure 3-10.

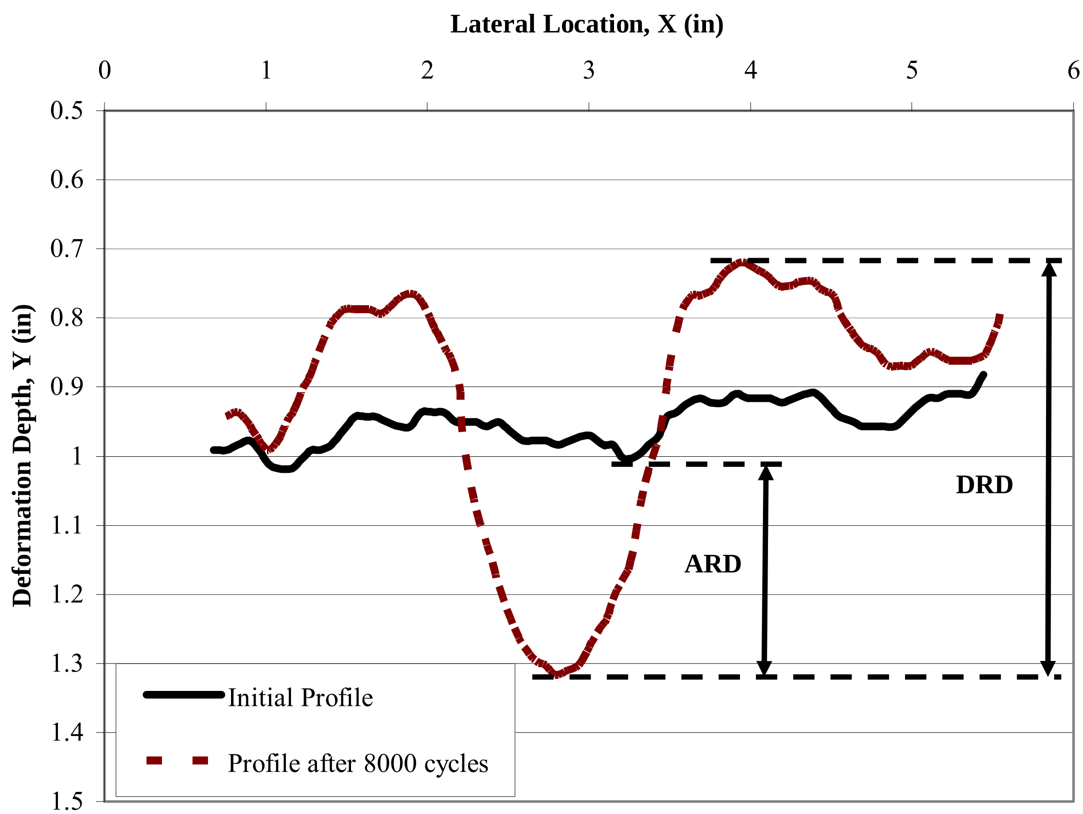

Figure 3-10 Illustration of absolute rut depth and differential rut depth

The absolute rut depth determined from the profile traces is the same form of rut depth measured using the conventional measuring device described previously. The differential rut depth measurement includes the absolute rut depth plus the shoving or heaving that occurs with mixtures that experience instability rutting.

3.3.5 Servopac Gyratory Compactor Testing

The second testing device used to examine the mixtures' rutting potential was the Servopac gyratory compactor located at the University of Florida Civil Engineering asphalt laboratory. This device performs the same functions as the Pine gyratory compactor mentioned previously. In addition, it has the ability to measure the force required to maintain the angle of compaction and this force is then converted to a "gyratory shear stress." The Servopac compactor generates an output file displaying the angle of gyration, the gyratory shear stress, the internal angle and the sample height.

Another useful feature of the Servopac compactor is the ability to quickly change the angle of gyration by simply inputting the desired angle into the computer input screen. The standard angle of compaction per AASHTO standards is 1.25 degrees. For this study, mixtures were compacted at 1.25 and 2.50 degrees per the procedure described below.

Roque et al. (2004a) developed a new procedure using the Servopac compactor for evaluating the rutting potential of mixtures. The procedure results in two parameters: the gyratory shear slope and the vertical failure strain.

- Two asphalt mixture specimens are compacted at an angle of 1.25 degrees to the maximum design number of gyrations (Nmax). Nmax for this study was 160 gyrations.
- The bulk specific gravity of each specimen is determined and air voids are calculated based on the maximum specific gravity of the mixture.
- Based on the height measurements recorded during compaction, the percent air voids at each gyration level is backcalculated.
- A graph is created plotting the measured gyratory shear versus the natural log of the number of gyratory revolutions.
- The slope of the graph is obtained in the range corresponding to seven to four percent air voids or to the maximum gyratory shear if this is reached prior to four

percent air voids. This value is designated the “gyratory shear slope” and is an indicator of the mixture’s resistance to deformation (Figure 3-11).

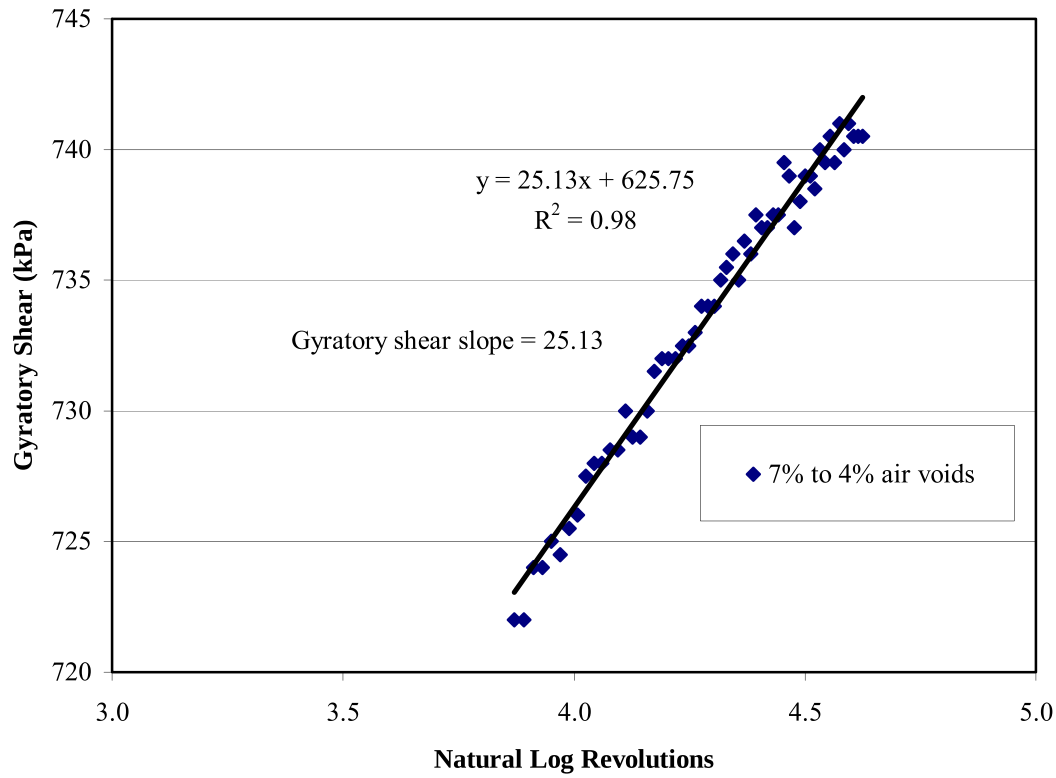

Figure 3-11 Gyratory shear slope

- Two additional asphalt specimens are prepared and compacted at an angle of 1.25 degrees until the gyration corresponding to seven percent air voids is reached. At this point the machine is stopped for approximately fifteen seconds while the angle of gyration is changed to 2.5 degrees. Then the sample is gyrated for another 100 gyrations. Changing the angle of compaction causes an unstable condition in the mixture resulting in a shear failure. The behavior of the mixture during this period provides a further indication of rutting potential and nature of the mixture.

- The gyratory shear versus the number of revolutions is plotted. The “vertical failure strain” is then calculated from the point of angle change to the local minimum in gyratory shear strength (Figure 3-12). This strain measurement is during the point of aggregate rearrangement caused by changing the angle of compaction and is an indicator of the stability characteristics of the mixture. The strain value is calculated by taking the change in gyratory pill height divided by the initial pill height at the point of angle change. The magnitude of the strain is an indicator of whether the mixture is brittle, plastic or somewhere in between. A framework for evaluating mixtures based on the work of Roque et al. (2004a) is shown in Figure 3-13.

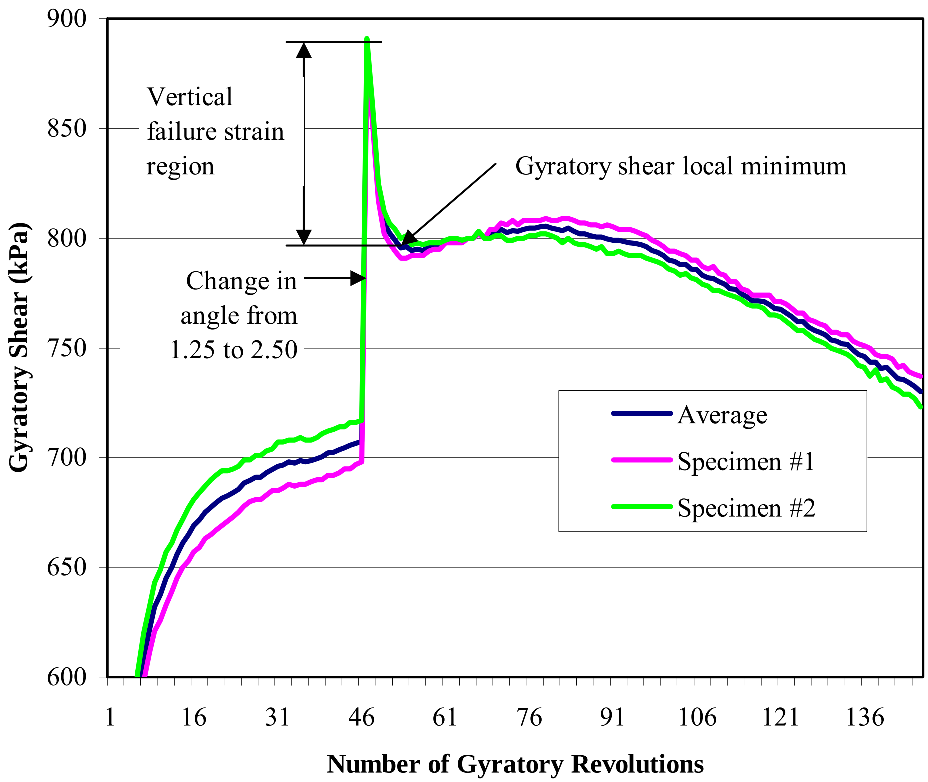

Figure 3-12 Vertical failure strain

3.3.6 Superpave Indirect Tension Testing

The evaluation of the mixtures' resistance to top-down cracking was evaluated using the Superpave indirect tension test (IDT) and the procedure developed at the University of Florida. Top-down cracking is the primary mode of pavement distress in Florida. Approximately 80 percent of the State's deficient highways are deficient due to top-down cracking. The research conducted at the University of Florida has been on going for many years and many papers have been published. Roque et al. (2004b) summarized the work to date and presented their framework for energy based criteria

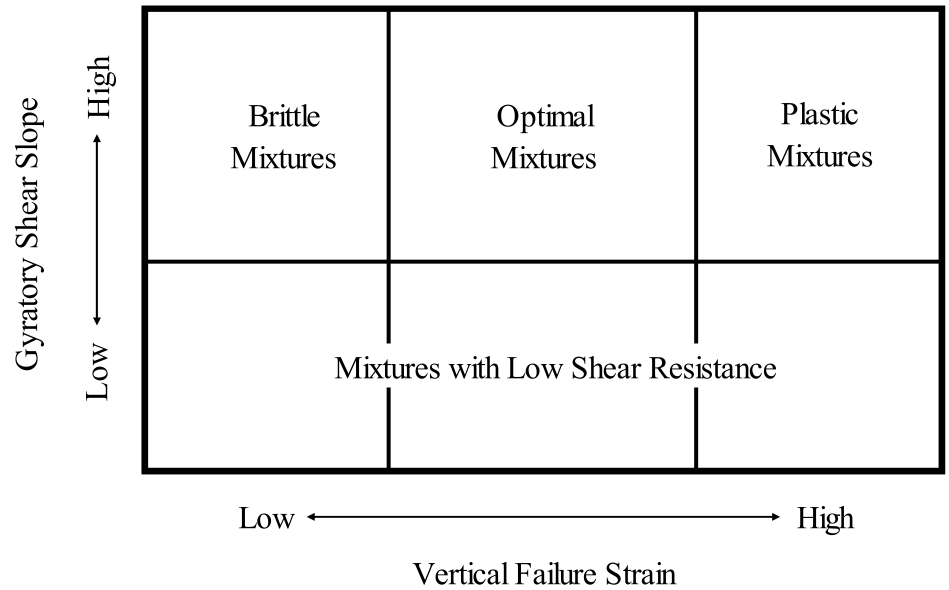

Figure 3-13 Framework for evaluating mixtures

related to top-down cracking in asphalt mixtures. The highlights of the procedure and analysis technique will be discussed below:

- 150 mm diameter gyratory compacted specimens of approximately 115 mm tall at an air void content of 7 +/- 1 percent air voids are prepared. From these specimens, the top and bottom are trimmed off using a wet saw and then the remainder of the specimen is cut in half resulting in two specimens approximately 50 mm thick.

- The specimens are dried and gage points are applied to both faces. The specimens are then further dried in a dehumidifying chamber and brought to a testing temperature of 10 °C.

- Three different tests are performed on each of three specimens in sequential order. The final results are therefore based on the average of three specimens. A MTS closed loop servo hydraulic system was used for all Superpave IDT testing.

- The resilient modulus and Poisson's ratio are determined by applying a haversine wave load for 0.1 seconds followed by a rest period of 0.9 seconds.

- A creep test is performed in which a constant load is applied for 1000 seconds. Several parameters are determined from this test including the creep compliance, creep rate and m-value, which is an indication of the mixture's resistance to creep.

- An indirect tensile strength test is performed at a rate of 50 mm/min. The tensile strength is determined at the point where the plot of the vertical deformations minus the horizontal deformations versus time reaches a peak. Figure 3-14 shows a test specimen.

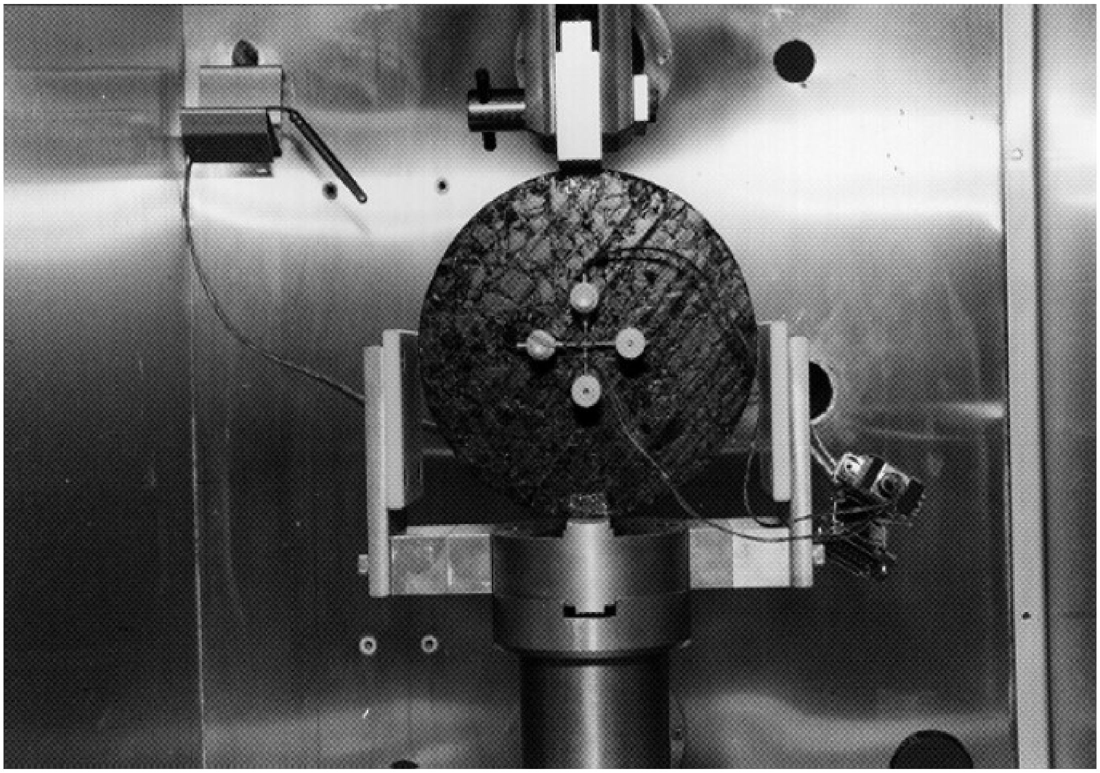

Figure 3-14 Superpave indirect tension test

The key parameter calculated is the energy ratio and is defined as the dissipated creep strain energy threshold of a material divided by the minimum dissipated creep strain energy needed. The dissipated creep strain energy (DCSE) of a material is defined as the fracture energy (FE) minus the elastic energy (EE) and is shown in Figure 3-15.

The minimum dissipated creep stain energy required is a function of material properties and the pavement structure. The relationship is described as:

$DCSE_{min} = m^{2.98} * D_1/A$

where, m and D_1 are parameters derived from the creep test.

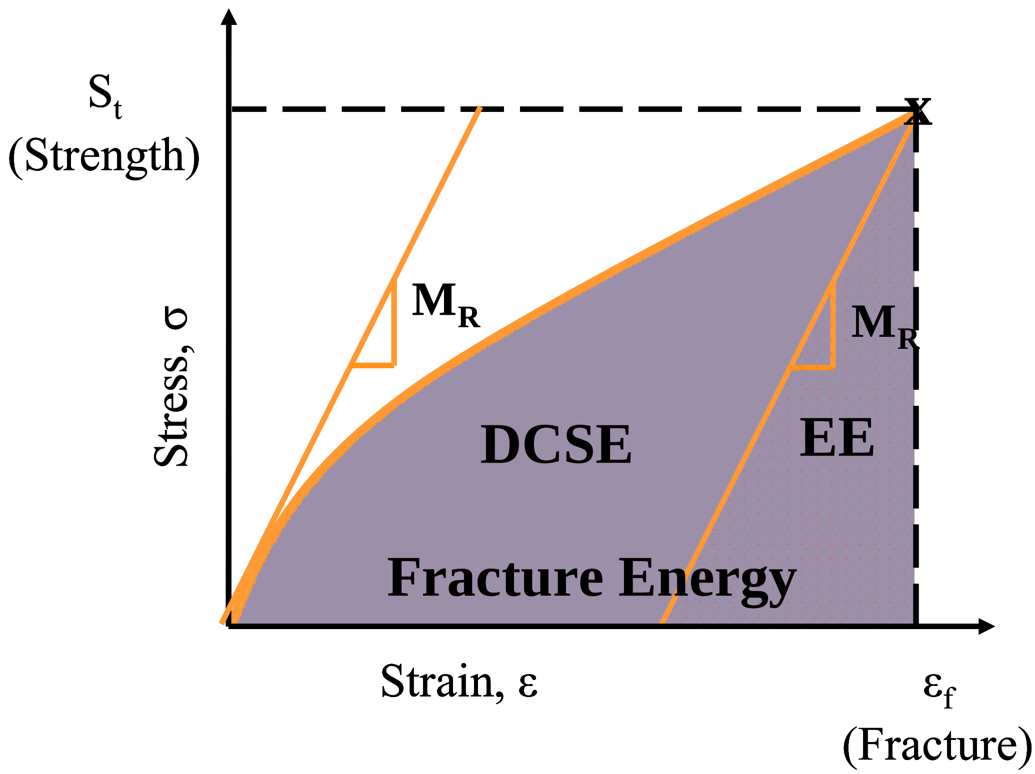

Figure 3-15 Dissipated creep strain energy

The term "A" accounts for the tensile stresses induced in the pavement by vehicle loads and the tensile strength of the material. "A" is defined as:

$$A = 0.0299*\sigma^{-3.10}(6.36\text{-}S_t)+2.46*10^{-8}$$

where, σ is the tensile stress induced in the pavement and S_t is the tensile strength of the material. For this study, a standard value of 100 lb/in^2 was used for σ.

Roque et al. (2004b) developed the following criteria for acceptable cracking performance. The DCSE of the material should be greater than 0.75 kJ/m^3 and the energy ratio should be greater than or equal to one. The researchers propose higher energy ratio values for higher traffic levels.

The aforementioned research was conducted on specimens that had only been short-term conditioned in accordance with the AASHTO procedure "Mixture

Conditioning of Hot-Mix Asphalt (HMA), AASHTO designation R 30-02." Short-term conditioning is supposed to represent the aging that plant produced mix will experience during the mixing and compaction process. Testing identical to that mentioned previously was conducted on specimens that had been long-term oven aged (LTOA) in accordance with AASHTO R 30-02. LTOA aging is intended to represent the aging that the mixture will experience after seven to ten years of service. The procedure for long-term aging is to place samples that have already been short-term oven aged into an oven at 85 °C for 120 hours. This testing was conducted to see if the asphalt mixtures performed in a similar manner or not compared to the short-term aged samples.

In addition to the short-term and long-term oven aged samples, an additional set of samples were prepared that were moisture conditioned. Birgisson et al. (2003) found that the Superpave IDT tests and data analysis techniques used to characterize a mixture's resistance to cracking is also successful at identifying a mixture's susceptibility to moisture damage. The moisture conditioning and testing procedure consists of:

- Uncut gyratory compacted specimens are vacuum saturated to a saturation level between 65 and 80 percent.
- The specimens are then placed in a 60 °C water batch for 24 hours.
- After removal from the water bath the specimens are allowed to dry at ambient room conditions for twelve hours, after which they are cut to a thickness of 50 mm.
- The suite of Superpave IDT tests is then performed on the specimens as described previously.

The Superpave IDT testing was performed on moisture conditioned specimens to provide an additional means of assessing the mixtures' moisture sensitivity in addition to the moisture testing conducted per FM 1-T 283 described previously. The emphasis was to examine the effects on moisture sensitivity between rounds one and two for each mixture type, not necessarily between mixtures of different aggregate types.

CHAPTER 4
TEST RESULTS AND ANALYSIS

4.1 Introduction

The test results and analysis will be presented categorically in the following order: mix design, rutting (APA and Servopac), cracking, moisture damage and permeability.

4.2 Mix Design

4.2.1 Mix Design Test Results

Following are tables of volumetric mix design data and VMA plots for each of the four mixture types. Each table and plot contains data for rounds one and two.

Table 4-1 Volumetric mix design data for Alabama limestone mixtures

Round #1 - Gap Graded Mixture									
Percent AC	Gmm	Gse	Pba	Pbe	Dust Ratio	Gmb	Air Voids	VMA	VFA
3.8	2.581	2.744	0.54%	3.28	1.0	2.409	6.7	14.3	54
4.3	2.566	2.750	0.63%	3.70	0.9	2.423	5.6	14.3	61
4.8	2.546	2.750	0.62%	4.21	0.8	2.445	4.0	13.9	72
5.3	2.527	2.751	0.63%	4.70	0.7	2.468	2.3	13.6	83
5.8	2.510	2.754	0.67%	5.17	0.7	2.471	1.5	13.9	89
Round #2 - Continuous Graded Mixture									
Percent AC	Gmm	Gse	Pba	Pbe	Dust Ratio	Gmb	Air Voids	VMA	VFA
3.6	2.596	2.752	0.53%	3.09	1.1	2.417	6.9	14.2	51
4.1	2.574	2.750	0.50%	3.62	0.9	2.438	5.3	13.9	62
4.6	2.556	2.753	0.53%	4.09	0.8	2.455	4.0	13.7	71
5.1	2.539	2.756	0.58%	4.55	0.7	2.475	2.5	13.5	81
5.6	2.517	2.753	0.53%	5.10	0.7	2.486	1.3	13.5	91

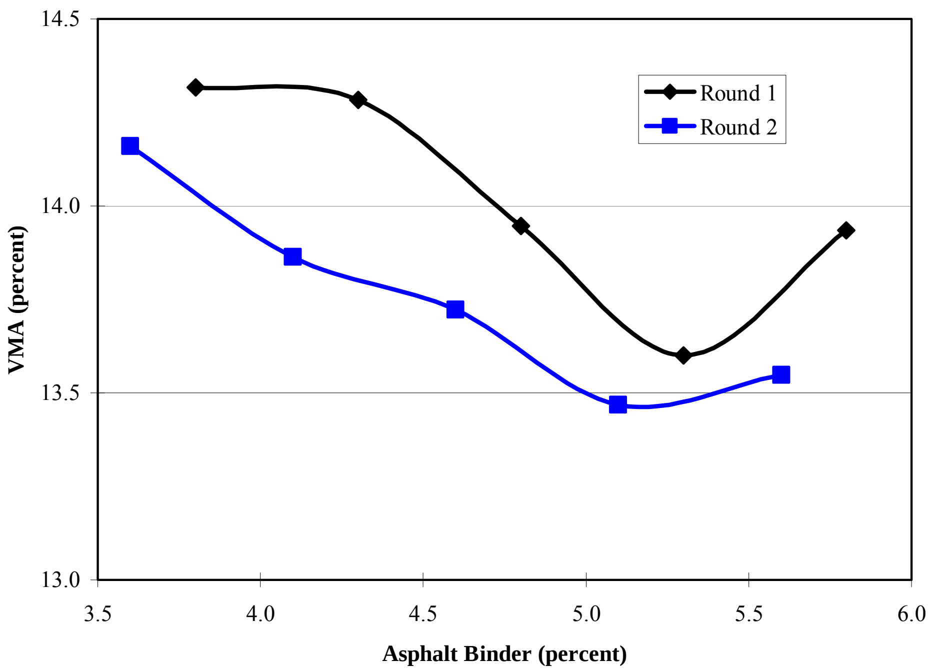

Figure 4-1 VMA plots for Alabama limestone mixtures

Table 4-2 Volumetric mix design data for Brooksville limestone mixtures

Round #1 - Gap Graded Mixture									
Percent AC	Gmm	Gse	Pba	Pbe	Dust Ratio	Gmb	Air Voids	VMA	VFA
6.5	2.319	2.540	4.53%	2.27	2.3	2.176	6.1	10.9	44
7.0	2.305	2.542	4.55%	2.76	1.9	2.189	5.0	10.9	54
7.5	2.295	2.549	4.67%	3.18	1.7	2.209	3.8	10.6	64
7.9	2.274	2.537	4.47%	3.78	1.4	2.205	3.0	11.1	73
8.4	2.261	2.539	4.51%	4.26	1.2	2.211	2.2	11.4	81
Round #2 - Continuous Graded Mixture									
Percent AC	Gmm	Gse	Pba	Pbe	Dust Ratio	Gmb	Air Voids	VMA	VFA
6.0	2.335	2.540	4.55%	1.72	3.1	2.174	6.9	10.5	35
6.5	2.319	2.540	4.54%	2.25	2.4	2.193	5.4	10.2	47
7.0	2.303	2.539	4.53%	2.78	1.9	2.214	3.9	9.9	61
7.5	2.291	2.543	4.60%	3.24	1.6	2.216	3.3	10.2	68
8.0	2.275	2.542	4.58%	3.79	1.4	2.237	1.7	9.9	83

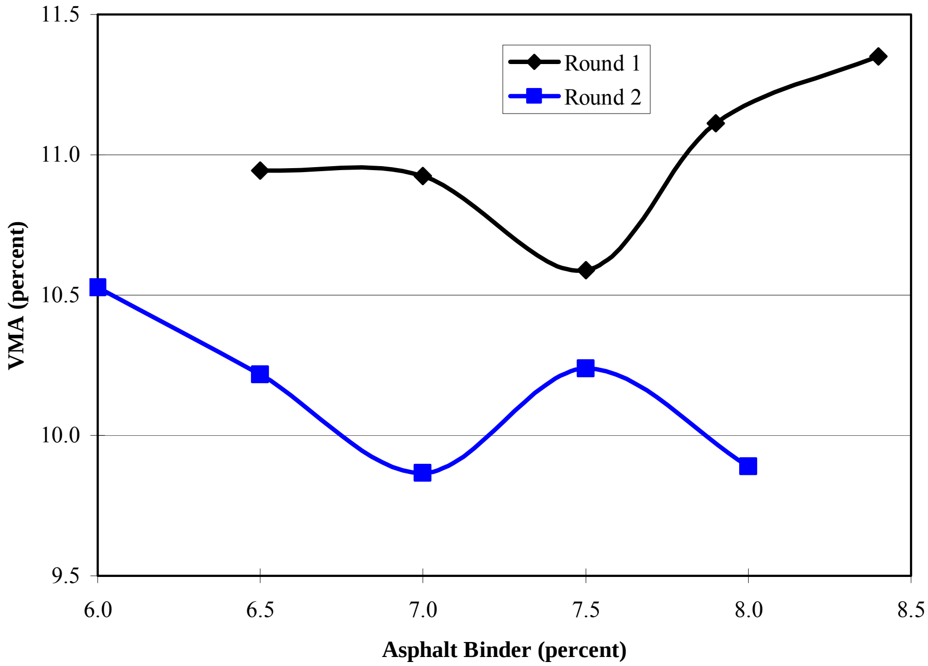

Figure 4-2 VMA plots for Brooksville limestone mixtures

Table 4-3 Volumetric mix design data for Nova Scotia granite mixtures

Round #1 - Gap Graded Mixture									
Percent AC	Gmm	Gse	Pba	Pbe	Dust Ratio	Gmb	Air Voids	VMA	VFA
4.8	2.471	2.659	0.39%	4.43	1.2	2.304	6.8	16.7	59
5.3	2.456	2.662	0.45%	4.88	1.1	2.328	5.2	16.2	68
5.8	2.435	2.658	0.39%	5.44	1.0	2.336	4.1	16.4	75
6.3	2.420	2.661	0.43%	5.89	0.9	2.359	2.5	16.0	84
6.8	2.402	2.661	0.42%	6.41	0.8	2.367	1.5	16.2	91
Round #2 - Continuous Graded Mixture									
Percent AC	Gmm	Gse	Pba	Pbe	Dust Ratio	Gmb	Air Voids	VMA	VFA
4.6	2.473	2.652	0.30%	4.32	1.2	2.300	7.0	16.6	58
5.1	2.455	2.652	0.30%	4.82	1.1	2.322	5.4	16.3	67
5.6	2.439	2.654	0.33%	5.29	1.0	2.346	3.8	15.8	76
6.1	2.421	2.654	0.32%	5.80	0.9	2.358	2.6	15.9	84
6.6	2.404	2.654	0.33%	6.29	0.8	2.371	1.4	15.9	91

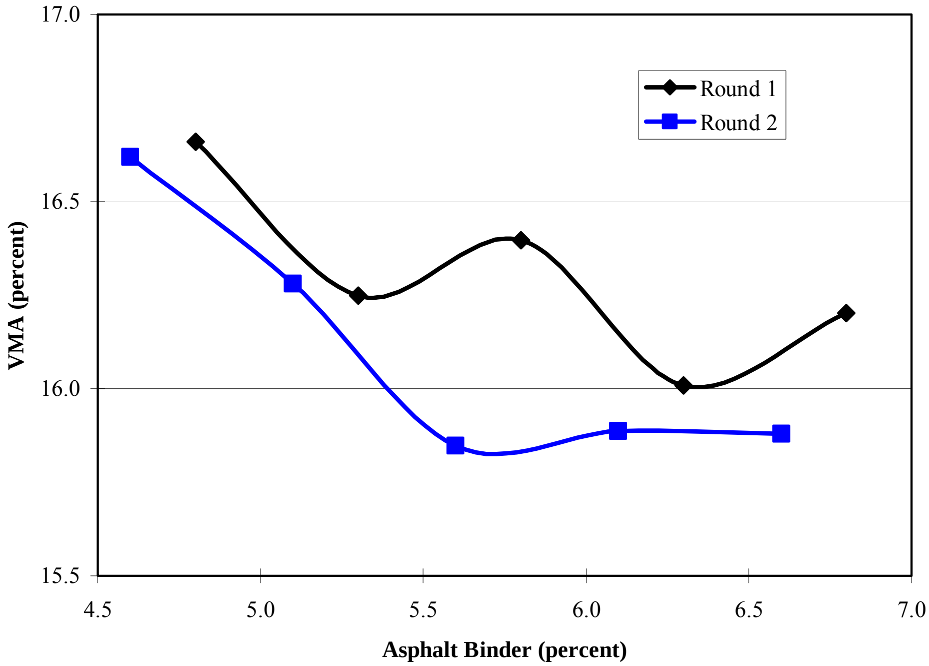

Figure 4-3 VMA plots for Nova Scotia granite mixtures

Table 4-4 Volumetric mix design data for Tarmac limestone mixtures

Round #1 - Gap Graded Mixture									
Percent AC	Gmm	Gse	Pba	Pbe	Dust Ratio	Gmb	Air Voids	VMA	VFA
6.3	2.314	2.526	2.48%	3.98	1.3	2.172	6.1	14.5	58
6.8	2.302	2.530	2.55%	4.43	1.2	2.194	4.7	14.1	67
7.3	2.291	2.535	2.64%	4.86	1.1	2.204	3.8	14.2	73
7.8	2.273	2.531	2.57%	5.43	1.0	2.224	2.2	13.9	84
8.3	2.256	2.528	2.52%	5.99	0.9	2.232	1.1	14.1	92
Round #2 - Continuous Graded Mixture									
Percent AC	Gmm	Gse	Pba	Pbe	Dust Ratio	Gmb	Air Voids	VMA	VFA
5.6	2.335	2.525	2.46%	3.27	1.6	2.186	6.4	13.3	52
6.1	2.320	2.525	2.47%	3.78	1.4	2.197	5.3	13.3	60
6.6	2.305	2.526	2.48%	4.28	1.2	2.214	3.9	13.1	70
7.2	2.283	2.521	2.40%	4.97	1.1	2.219	2.8	13.5	79
7.6	2.275	2.526	2.49%	5.30	1.0	2.244	1.4	12.9	90

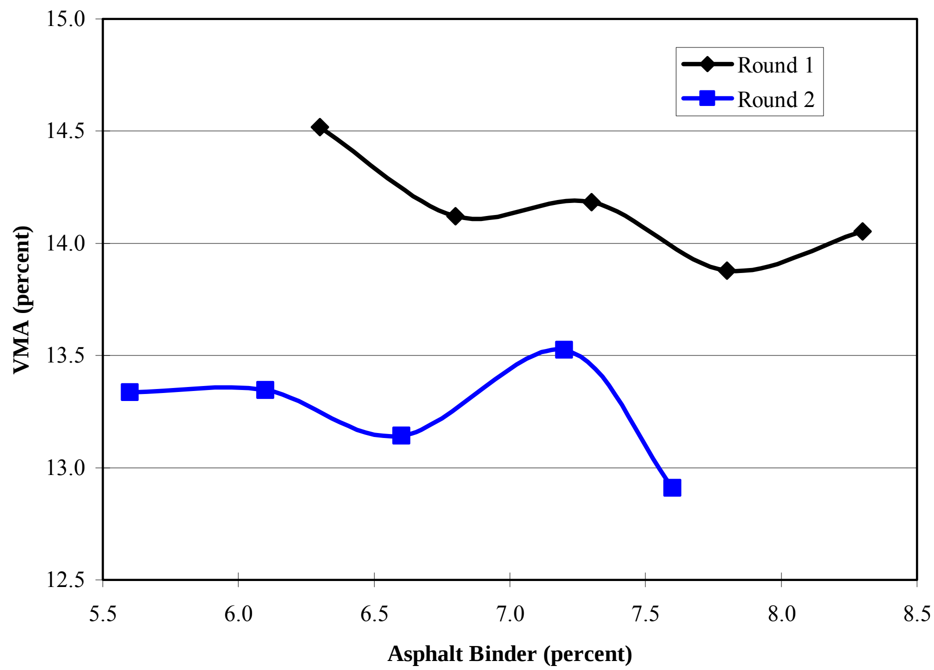

Figure 4-4 VMA plots for Tarmac limestone mixtures

4.2.2 Mix Design Summary

Examination of the VMA curves generally shows the expected concave shaped curve for each mixture. Theoretically, it is desirable to have a design asphalt binder content that is either at the minimum point on the VMA curve or to the left of the minimum point. In this range of binder contents, the slight addition of additional binder, which could occur during production, will not increase the VMA. An increase in VMA (i.e. to the right side of the minimum point) is thought to push the aggregate skeleton apart and reduce shear resistance, which is related to rutting. It appears that for all of the mixtures designed for this study that the optimum binder content is either at the minimum of the curve or to the left side of the minimum.

It should be noted that the amount of coarse aggregate added to each mix design for round 2 was almost identical for all of the aggregate types, however, the reduction in

VMA between rounds one and two was significantly different for each aggregate type.

The reduction in VMA for each aggregate type is shown in Table 4-5.

Table 4-5 VMA difference between rounds one and two

Aggregate Type	VMA		
	Round 1	Round 2	Difference
Alabama limestone	13.9	13.7	0.2
FL Brooksville limestone	10.6	9.9	0.7
Nova Scotia granite	16.4	15.8	0.6
Fl Tarmac limestone	14.2	13.1	1.0

4.3 Rutting

4.3.1 APA Test Results

There were five different parameters determined with the APA considering various methods of sample preparation and data interpretation. The five parameters are:

- Absolute rut depth using the conventional one-point measuring device testing 75 mm tall gyratory compacted specimens compacted to an air void content of 7.0 +/- 1.0 percent.
- Absolute rut depth using the conventional one-point measuring device testing 115 mm tall gyratory compacted specimens compacted to an air void content of 4.0 +/- 0.5 percent.
- Absolute rut depth using the complete profile measuring device testing 75 mm tall gyratory compacted specimens compacted to an air void content of 7.0 +/- 1.0 percent.
- Differential rut depth using the complete profile measuring device testing 75 mm tall gyratory compacted specimens compacted to an air void content of 7.0 +/- 1.0 percent.
- Percent area change using the complete profile measuring device testing 75 mm tall gyratory compacted specimens compacted to an air void content of 7.0 +/- 1.0 percent.

The results for each of the five parameters for all of the mixture types are presented in Table 4-6. Each value in Table 4-6 represents the average of four specimens.

Table 4-6 APA test results

APA Parameter	Alabama limestone		FL Brooksville limestone		Nova Scotia granite		FL Tarmac limestone	
	Round 1	Round 2	Round 1	Round 2	Round 1	Round 2	Round 1	Round 2
Absolute Rut Depth (mm) Single point measuring device 7% Va, 75 mm tall	5.5	4.2	1.5	1.3	5.9	4.5	2.5	1.5
Absolute Rut Depth (mm) Single point measuring device 4% Va, 115 mm tall	3.5	4.4	1.1	1.2	3.7	4.3	2.6	1.9
Absolute Rut Depth (mm) Profile measuring device 7% Va, 75 mm tall	5.1	3.7	1.4	0.5	5.9	3.1	1.8	0.9
Differential Rut Depth (mm) Profile measuring device 7% Va, 75 mm tall	10.0	8.3	3.6	3.1	11.7	8.0	5.4	3.8
Percent Area Change Profile measuring device 7% Va, 75 mm tall	0.88	0.82	-1.00	-0.27	1.82	1.69	-0.37	-0.10

Figure 4-5 displays the absolute rut depths measured by the conventional measuring device and profile measuring device and the differential rut depths measured by the profile measuring device. In theory, the absolute rut depths measured by the conventional measuring device and the profile measuring device should be approximately the same, and this is displayed in Figure 4-5. The differential rut depths follow the same trend as the absolute rut depths but with greater magnitude, as expected.

Figure 4-6 displays the absolute rut depths measured by the conventional measuring device for the specimens compacted to seven percent air voids and a height of 75 mm. Figure 4-7 displays the absolute rut depths measured by the profile measuring device and Figure 4-8 displays the differential rut depths measured by the profile measuring device for the same specimens. In all cases, there is a significant decrease in rut depth from round one to round two implying that adding more coarse aggregate is beneficial in reducing the rutting susceptibility of the mixtures.

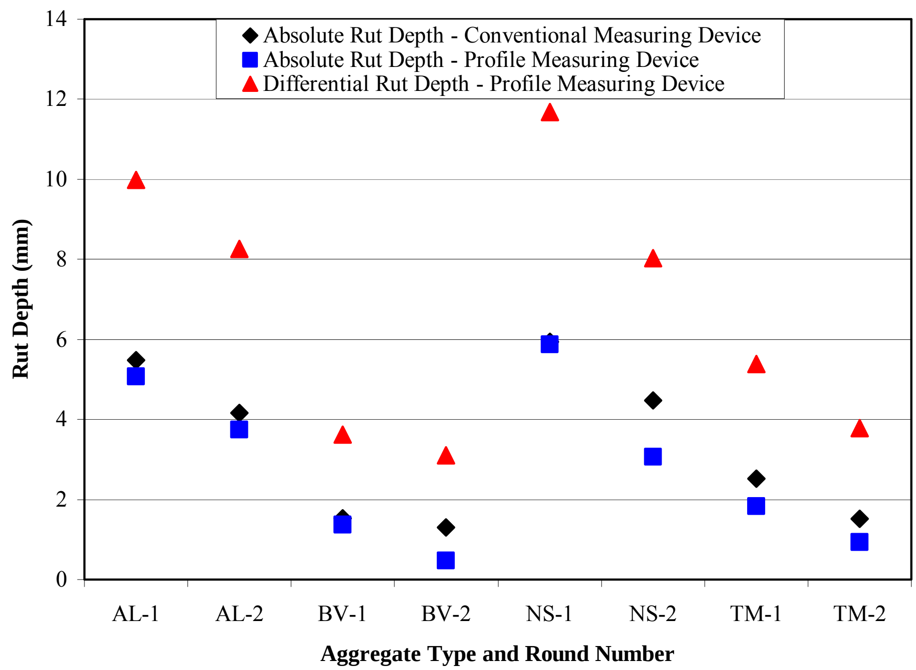

Figure 4-5 Comparison of APA measurement methods for 7% air voids, 75 mm tall specimens

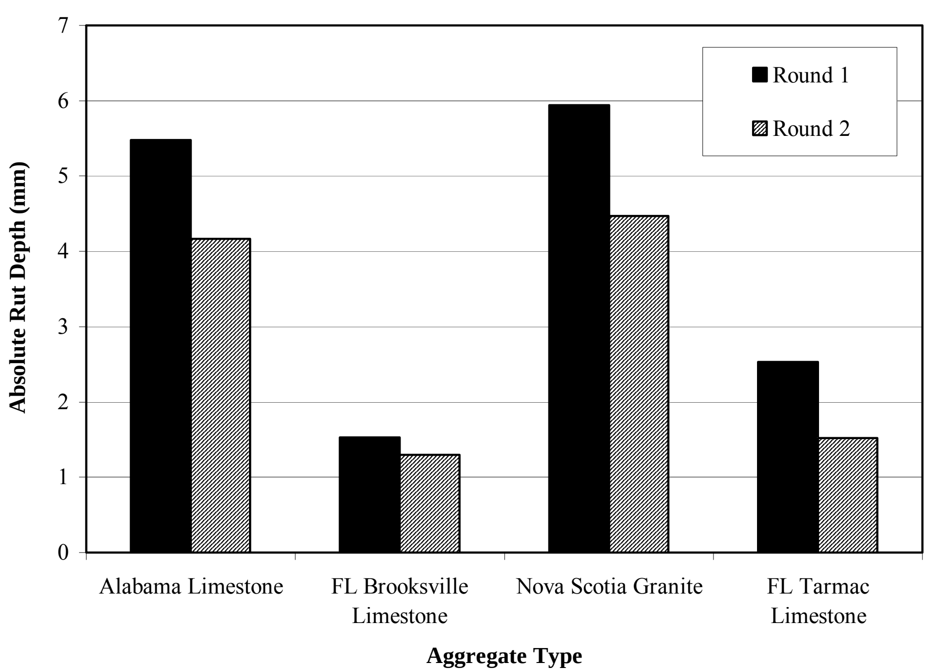

Figure 4-6 APA absolute rut depth using conventional measuring device for 7% air voids, 75 mm tall specimens

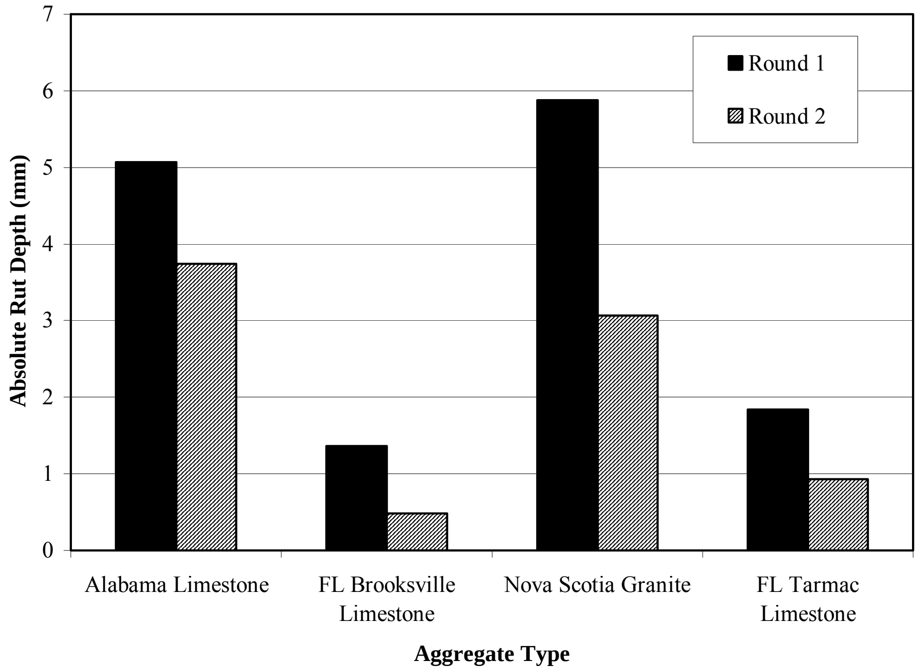

Figure 4-7 APA absolute rut depth using profile measuring device for 7% air voids, 75 mm tall specimens

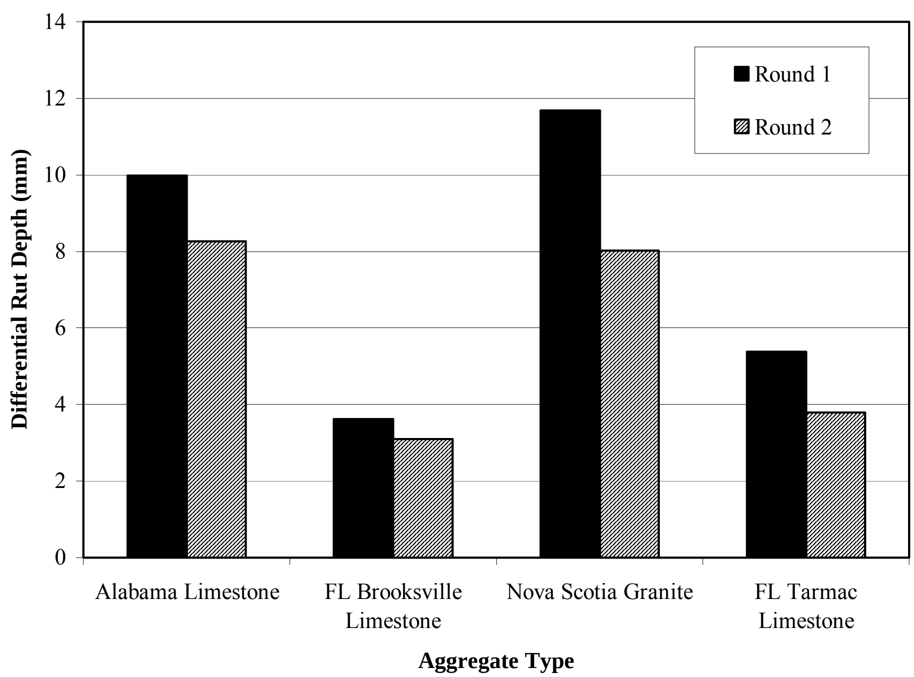

Figure 4-8 APA differential rut depth using profile measuring device for 7% air voids, 75 mm tall specimens

Figure 4-9 displays the percent area change for each aggregate type and round. A positive area change indicates instability rutting manifested by shoving and heaving of the mixture on each side of the rut. A negative percent area change indicates that the majority of the rutting was due to consolidation. The percent area change decreased for every aggregate type from round one to round two, a further indication that the addition of coarse aggregate was beneficial in reducing the rutting susceptibility of the mixtures. The two aggregate types with the lowest amount of rutting (Brooksville and Tarmac Florida limestones) had negative percent area changes indicating that the small amount of rutting those mixtures experienced was primarily due to consolidation. The two aggregate types with the largest amount of rutting (Alabama limestone and Nova Scotia granite) had positive percent area changes indicating that the rutting those mixtures experienced was primarily due to instability under a load.

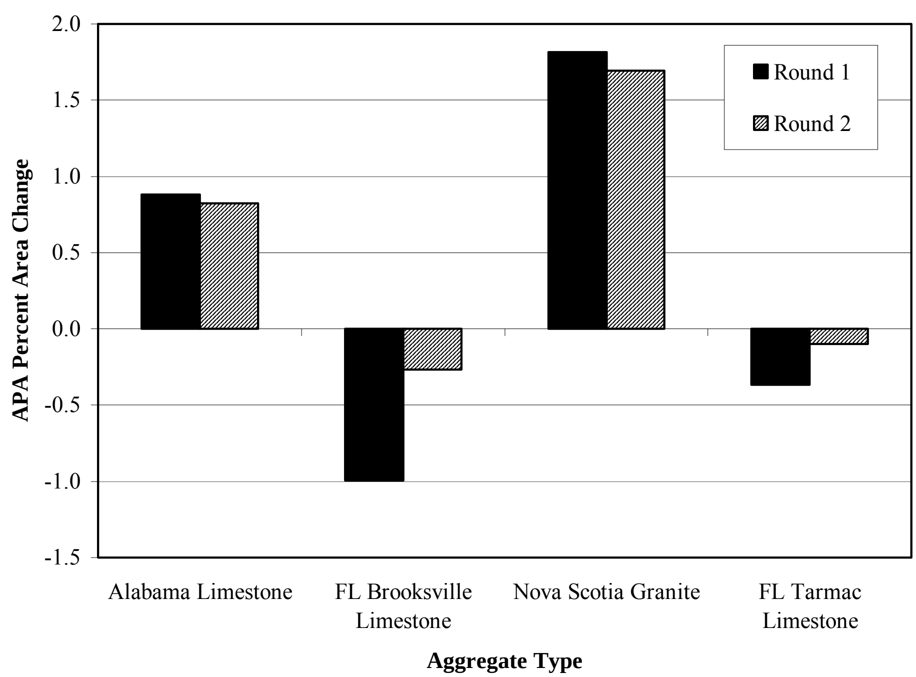

Figure 4-9 APA percent area change using profile measuring device for 7% air voids, 75 mm tall specimens

Figure 4-10 displays the absolute rut depths measured by the conventional measuring device for the specimens compacted to four percent air voids and a height of 115 mm. In contrast to the specimens compacted to seven percent air voids and a height of 75 mm, the rut depths increased from round one to round two for the Alabama limestone and the Nova Scotia granite mixtures. The rut depth was essentially the same between rounds one and two for the Florida Brooksville limestone. The rut depth decreased for the Tarmac Florida limestone between rounds one and two. It is undetermined why there is a difference in trends between rounds one and two for the different sample types. However, it is noted that the two mixtures (Alabama limestone and Nova Scotia granite) that showed an increase in rut depth between rounds one and two for the specimens compacted to four percent air voids and a height of 115 mm were the same mixtures that experienced instability rutting.

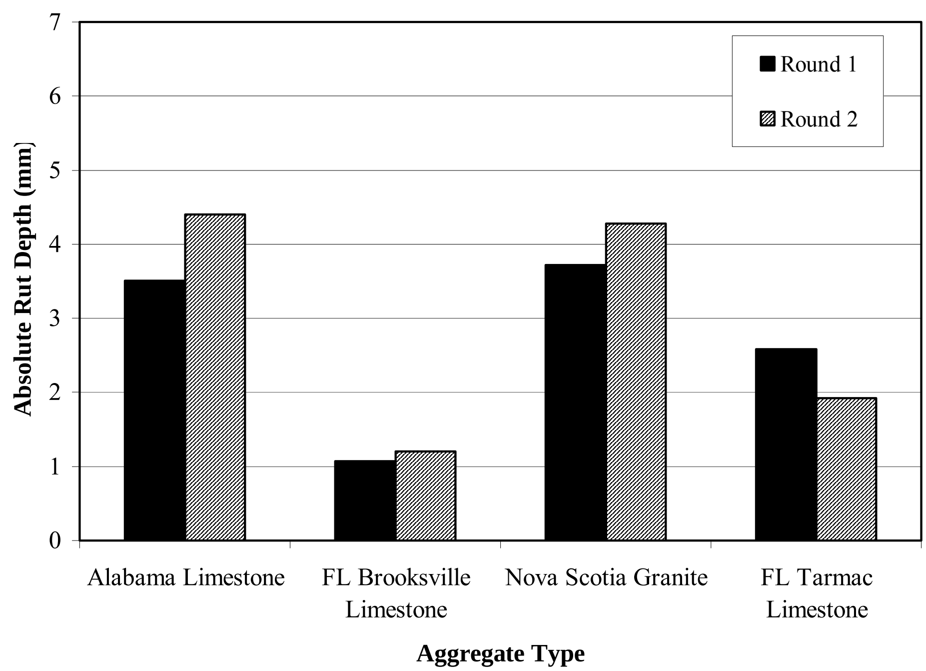

Figure 4-10 APA absolute rut depth using conventional measuring device for 4% air voids, 115 mm tall specimens

Figure 4-11 displays the APA rut depth versus the VMA for the eight mixtures examined in this study. The APA rut depth is measured using the conventional measuring device for the specimens compacted to seven percent air voids and a height of 75 mm. The plot of the data shows a strong correlation between rut depth and VMA (R^2 = 0.70). For these eight mixtures, as the VMA increased, the rut depth increased. This effect is reasonable since higher VMA at a fixed air void content means that there is more effective binder in the mix, which means there is more void space between the aggregate particles, less stone on stone contact and a potentially less stable aggregate structure. A similar correlation existed between APA rut depth and VFA.

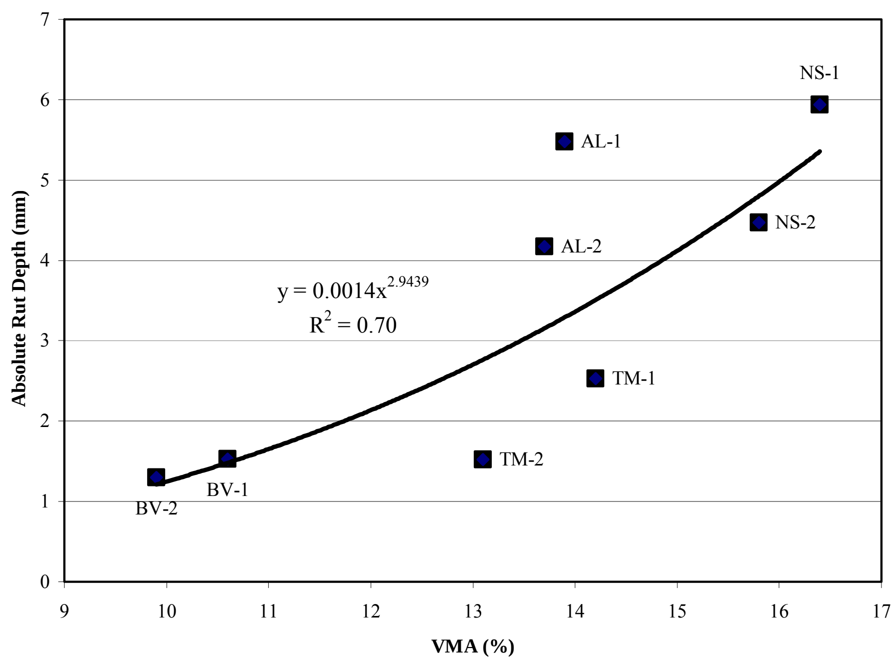

Figure 4-11 APA rut depth versus VMA using conventional measuring device for 7% air voids, 75 mm tall specimens

Figure 4-12 displays the APA rut depth versus the dust to effective binder ratio (commonly called the dust ratio) for the eight mixtures examined in this study. The APA rut depth is measured using the conventional measuring device for the specimens compacted to seven percent air voids and a height of 75 mm. The plot of the data shows a strong correlation between rut depth and dust ratio ($R^2 = 0.79$). For these eight mixtures, as the dust ratio increased, the rut depth decreased. This is reasonable since the dust mixes with and stiffens the binder, hence increasing the rutting resistance of the mixture.

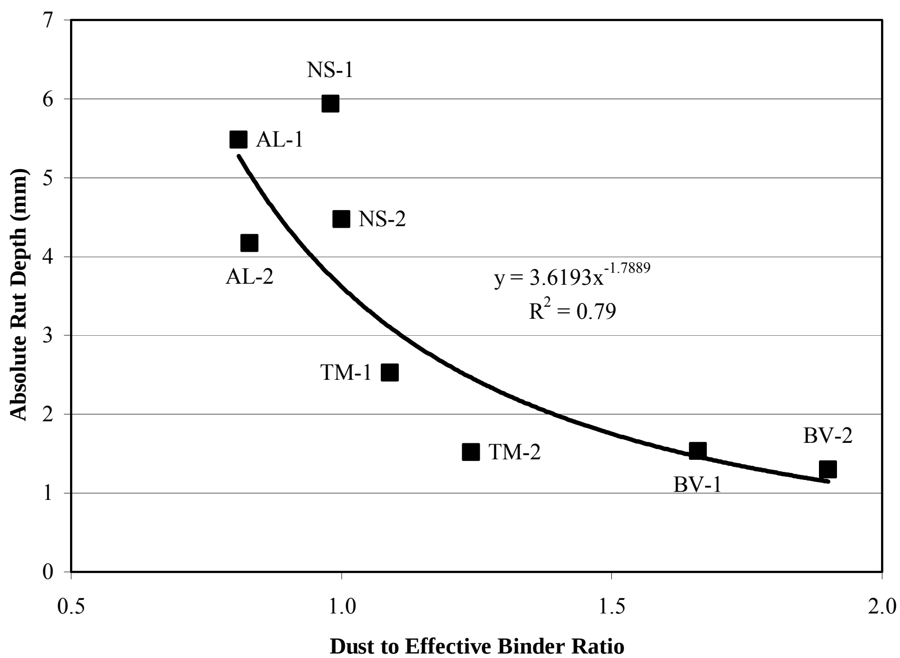

Figure 4-12 APA rut depth versus dust to effective binder ratio using conventional measuring device for 7% air voids, 75 mm tall specimens

4.3.2 APA Summary

Two measurement techniques were used for obtaining APA rut depths; the conventional single point absolute rut depth using a digital micrometer and the new profile measuring device which provided the full rut profile of the mixture and provided for the determination of the absolute and differential rut depths and the percent area change of the rut profile. The absolute rut depths measured by the conventional and new measuring techniques compared very well.

APA results for the specimens compacted to seven percent air voids and a 75 mm height revealed that the addition of more coarse aggregate to the 12.5 mm and 9.5 mm sieves, resulting in a more continuous graded mixture, improved the rutting performance with respect to absolute and differential rut depths and percent area change regardless of aggregate type.

Examining all eight mixtures as a group revealed a strong correlation showing that increasing VMA resulted in an increase in rut depth. An even stronger correlation showed that increasing the dust to effective binder ratio resulted in a decrease in rut depth.

4.3.3 Servopac Test Results

There were three different parameters determined with the Servopac gyratory compactor. The three parameters are

- Gyratory shear slope: A graph is created plotting the gyratory shear measured by the Servopac compactor versus the natural log of the number of gyratory revolutions. The air voids at each gyratory revolution are computed. The gyratory shear slope is the slope of the graph in the range of compaction from seven to four percent air voids or to the maximum gyratory shear if this value is reached prior to four percent air voids. This value describes the rate at which the mixture develops shear resistance and is an indication of the mixture's resistance to deformation.

- Vertical failure strain: Specimens are compacted at an angle of 1.25 degrees until the specimens reach seven percent air voids. The angle of compaction is changed to 2.50 degrees and the sample is gyrated for another 100 gyrations. The gyratory shear versus the number of revolutions is plotted. The vertical failure strain is then calculated from the point of angle change to the local minimum in gyratory shear strength. This strain measurement is an indicator of the stability characteristics of the mixture. The magnitude of the strain is an indicator of whether the mixture is brittle, plastic or somewhere in between.

- Maximum gyratory shear strength: This is the maximum shear strength achieved during the compaction process.

The values for gyratory shear slope, vertical failure strain and maximum gyratory shear strength for all of the mixture types are presented in Table 4-7. Each value in Table 4-7 represents the average of two specimens.

Table 4-7 Servopac test results

Servopac Parameter	Alabama limestone		FL Brooksville limestone		Nova Scotia granite		FL Tarmac limestone	
	Round 1	Round 2	Round 1	Round 2	Round 1	Round 2	Round 1	Round 2
Gyratory Shear Slope Percent Air Void Range	25 7.0 to 4.0	11 7.0 to 4.2	23 7.0 to 6.2	7 7.0 to 6.7	16 7.0 to 6.7	13 7.0 to 5.7	32 7.0 to 4.0	35 7.0 to 4.5
Percent Vertical Strain	1.83	1.67	n/a	n/a	n/a	1.97	2.13	1.66
Maximum Shear Stress (kPa) Percent Air Voids	744 3.1	726 4.2	772 6.2	775 6.7	679 6.7	689 5.7	750 3.6	751 4.5

4.3.3.1 Gyratory shear slope

An example of a gyratory shear slope graph is shown in Figure 4-13 for the Alabama limestone round one mixture. Six of the eight mixtures had a peak gyratory shear strength prior to reaching a compaction level of four percent air voids. The gyratory shear slope is then defined as the slope of the graph from the seven percent air void level to the air void level at the point of maximum gyratory shear strength. An example of this is shown in Figure 4-14 for the Florida Brooksville limestone round one mixture.

The gyratory shear slope decreased for the Alabama limestone, Florida Brooksville limestone and Nova Scotia granite mixtures from round one to round two indicating that the round two mixtures did not develop shear resistance as rapidly as the round one mixtures. The Florida Tarmac mixture had a slight increase in gyratory shear slope from round one to round two. Roque et al. (2004a) indicated that mixtures with a gyratory shear slope of less than 14 were undesirable with respect to rutting. All of the round one

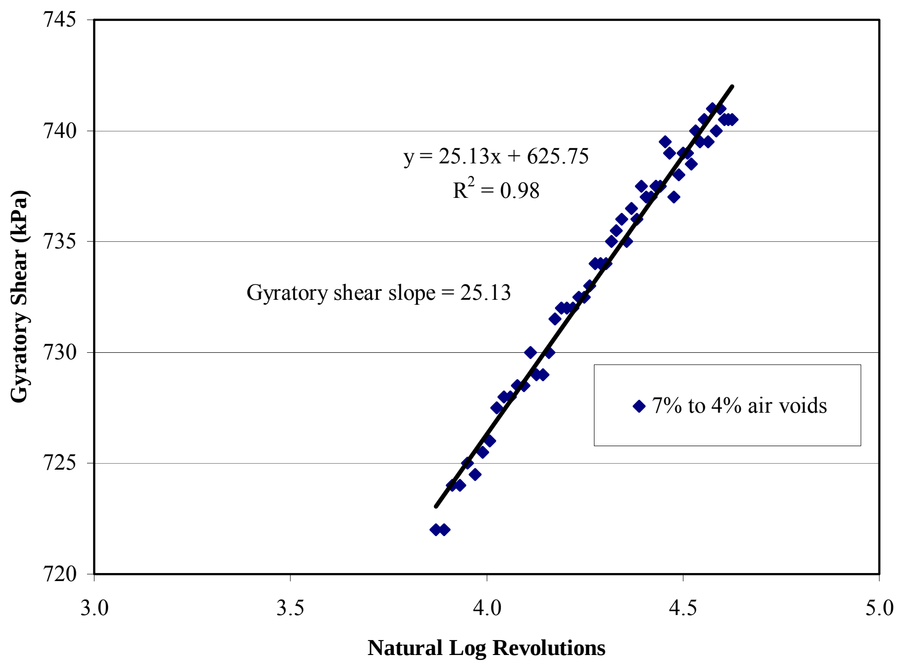

Figure 4-13 Gyratory shear slope for Alabama limestone round one mixture

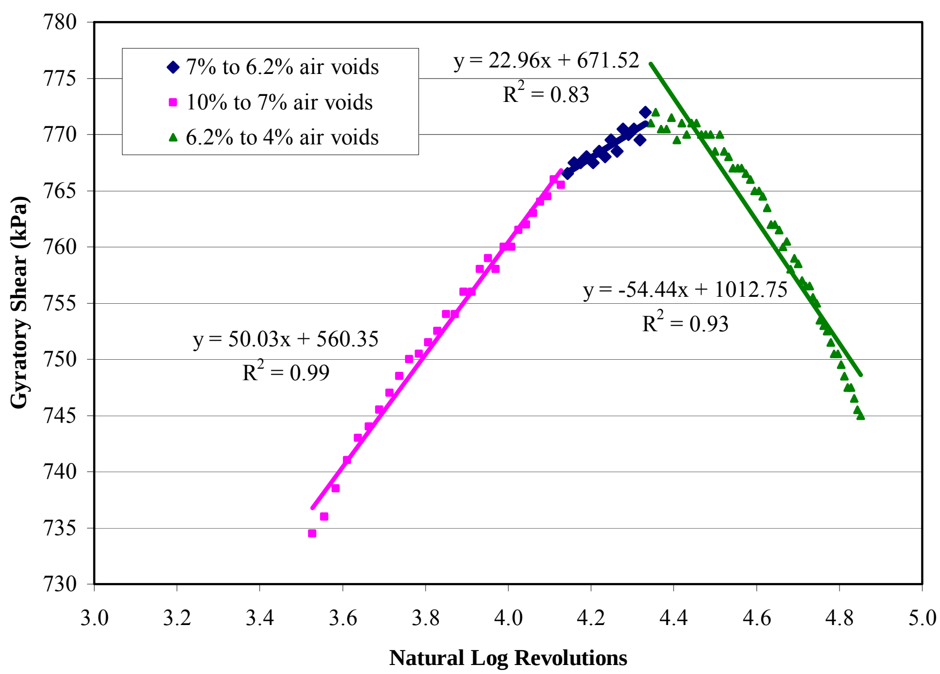

Figure 4-14 Gyratory shear slope for Florida Brooksville limestone round one mixture

mixtures had a gyratory shear slope greater than 14. Only one of the round two mixtures (Florida Tarmac limestone mixture) had a gyratory shear slope greater than 14.

4.3.3.2 Vertical strain

Roque et al. (2004a) indicated that mixtures with a vertical strain in the range of 1.4 to 2.0 percent were desirable, whereas mixtures with a vertical strain less than 1.4 percent would be considered “brittle” and mixtures with a vertical strain greater than 2.0 percent would be considered “plastic.” Examination of the data reveals that there was an improvement from round one to round two for three of the four mixtures (Alabama limestone, Nova Scotia granite, and Florida Tarmac limestone). The round two vertical strain values for these mixtures were in the desirable range. It appears that the addition of coarse aggregate to the round two mixtures improved the vertical strain values. An example of a plot of the gyratory shear versus the number of revolutions is shown in Figure 4-15 for the Alabama limestone round two mixture. The gyratory shear peaks initially after the change in compaction angle to 2.50 degrees at a compaction level of seven percent air voids and then drops to a local minimum as the particles rearrange themselves. Shear strength then builds slowly and reaches a final peak before dropping off.

Rounds one and two of the Florida Brooksville limestone mixture and round one of the Nova Scotia granite mixture never reached a local minimum in gyratory shear strength after the compaction angle was changed to 2.50 degrees and hence had no vertical strain value to report. This is indicated as an “n/a” in Table 4-7. It appears that these mixtures were never able to recover strength after the angle change in compaction at the seven percent air void level. An example of the gyratory shear versus the number

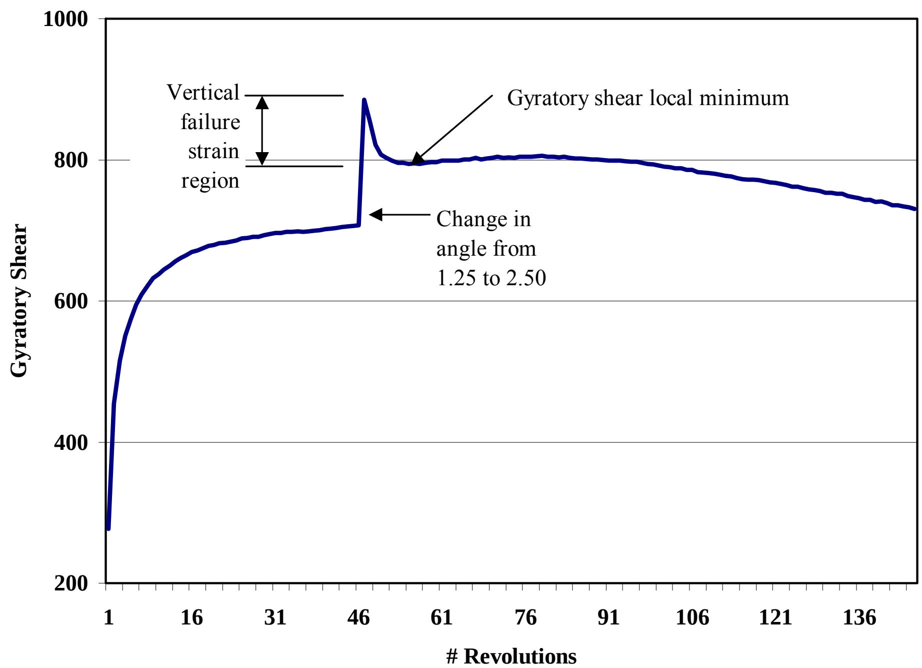

Figure 4-15 Vertical strain for Alabama limestone round two mixture

of revolutions for this condition is shown in Figure 4-16 for the Florida Brooksville limestone round one mixture. To determine if a mixture would recover shear strength when the angle of compaction was changed at a different air void content other than seven percent it was decided to make two more specimens of the Nova Scotia round one mixture and change the angle of compaction at an air void content of nine percent instead of seven percent. Figure 4-17 shows the gyratory shear versus the number of revolutions for both of these conditions. It can be seen that the shear strength did recover slightly when the angle of compaction was changed at nine percent air voids. This demonstrates that some mixtures gain strength rapidly and peak at higher air void contents than other mixtures.

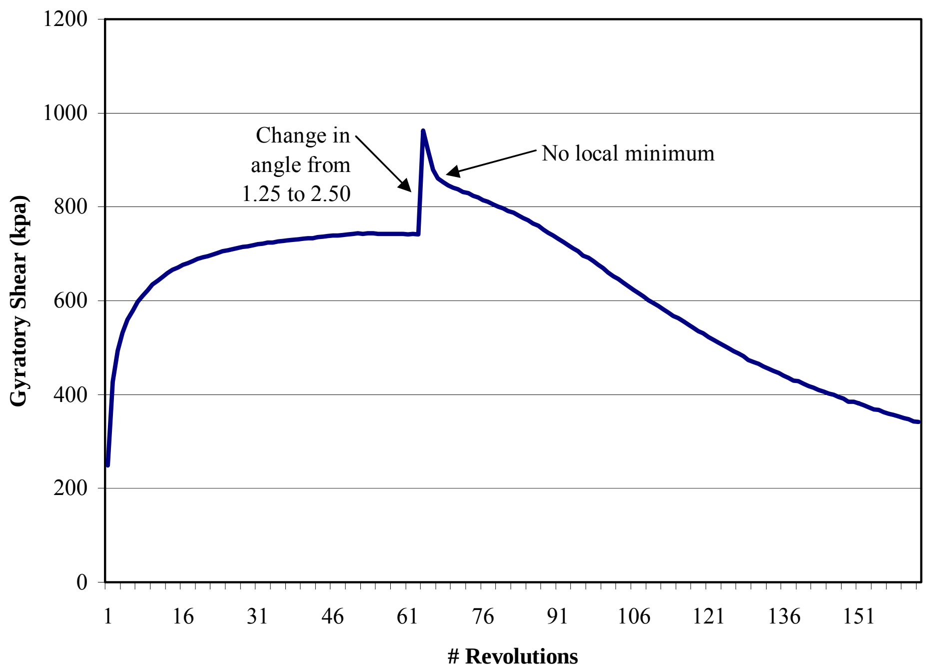

Figure 4-16 Vertical strain for Florida Brooksville limestone round one mixture

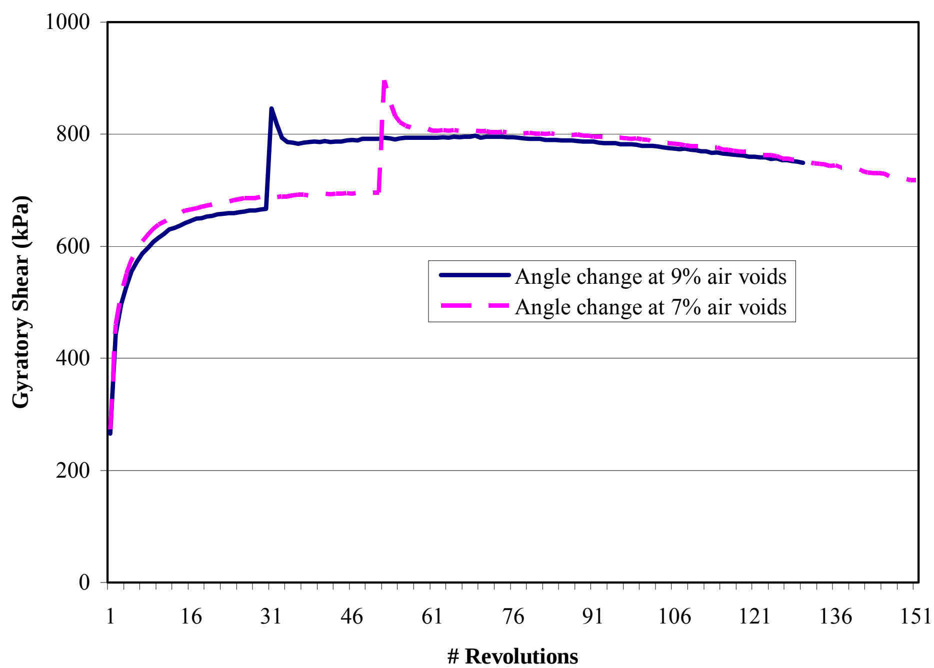

Figure 4-17 Vertical strain for Nova Scotia granite round one mixtures

4.3.3.3 Maximum shear stress

The gyratory shear strength versus the percent air voids during compaction is shown for rounds one and two of each aggregate type in Figures 4-18 through 4-21. Examination of the data in Table 4-7 and Figures 4-18 through 4-21 reveals that the maximum shear strength peaks at higher air void contents for the round two mixtures of the Alabama limestone, Florida Brooksville limestone and Florida Tarmac limestone mixtures. A possible explanation of this is that these round two mixtures have a more continuous gradation and less VMA than their round one counterparts resulting in more aggregate interlock at higher air voids and less asphalt binder to act as lubrication in the compaction process.

Additionally, the Alabama limestone and Florida Tarmac limestone mixtures tended to peak at lower air void contents (3.1 to 4.2 percent range) and then start to lose strength. The Florida Brooksville limestone and Nova Scotia granite mixtures peaked at higher air void contents (5.7 to 6.7 percent range) and then lost strength.

The maximum gyratory shear stress for each aggregate type correlated with the APA rut depth for the seven percent air void specimens. Higher gyratory shear stress values in the Servopac compactor were equivalent to lower rutting values in the APA (see Figure 4-22).

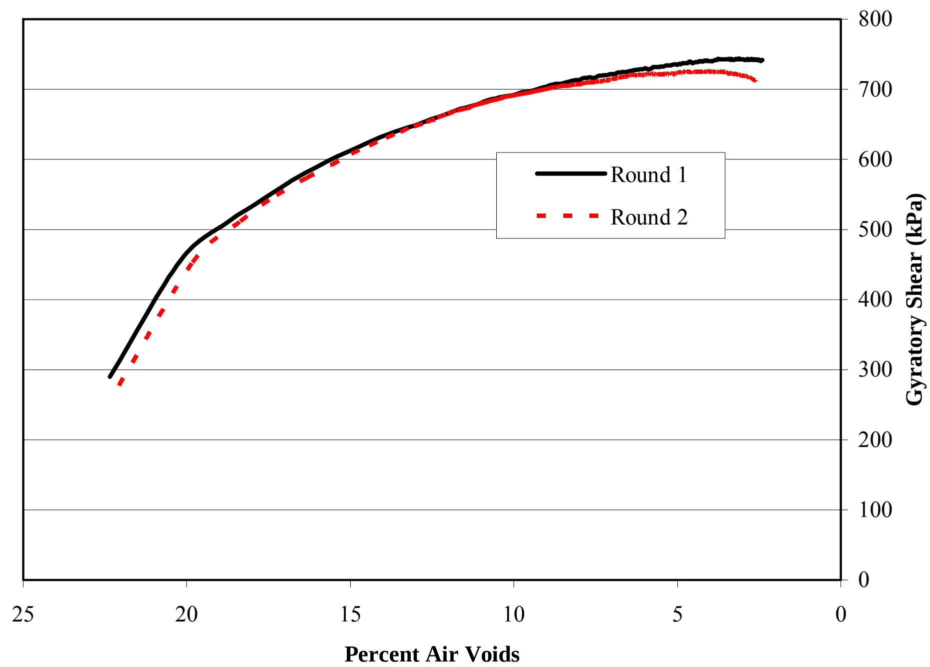

Figure 4-18 Gyratory shear versus percent air voids for Alabama limestone mixtures

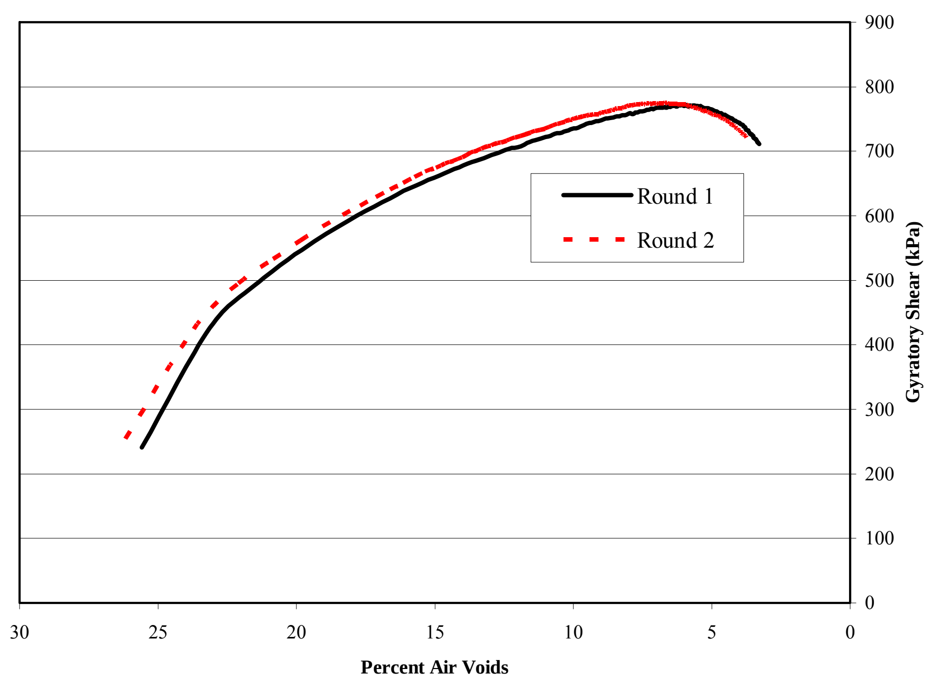

Figure 4-19 Gyratory shear versus percent air voids for Florida Brooksville limestone mixtures

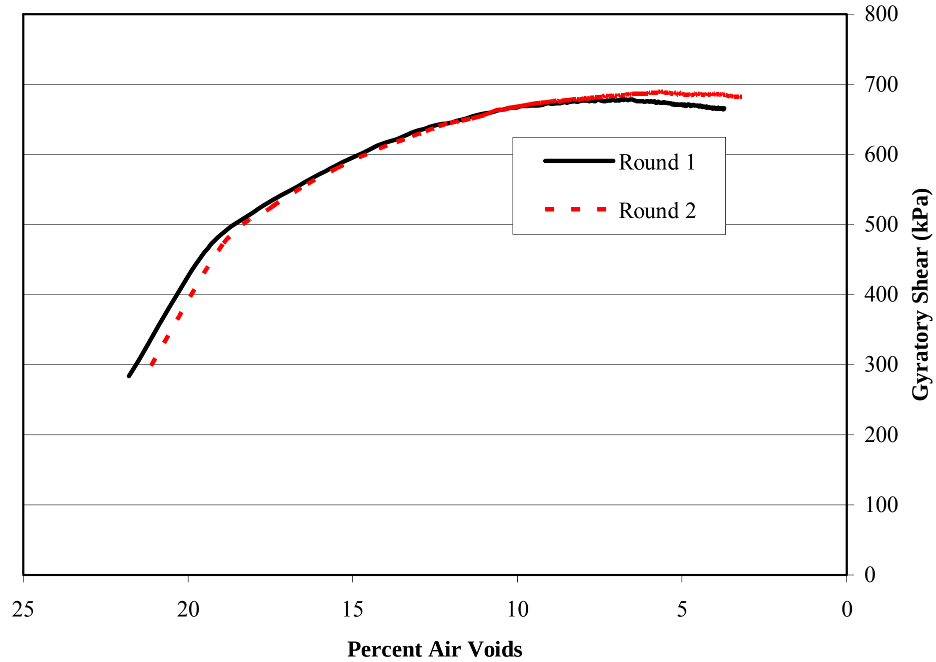

Figure 4-20 Gyratory shear versus percent air voids for Nova Scotia granite mixtures

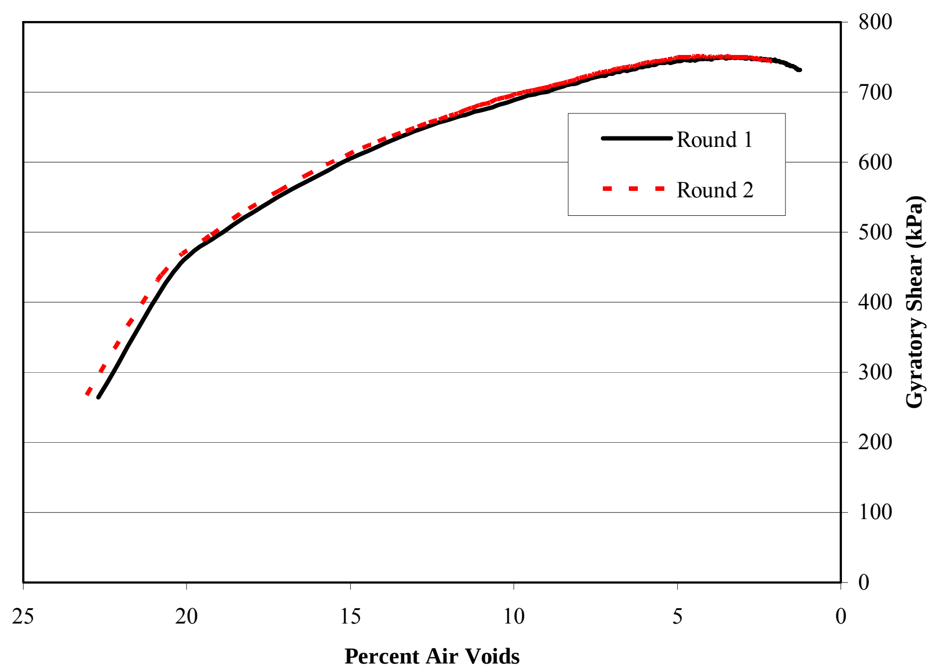

Figure 4-21 Gyratory shear versus percent air voids for Florida Tarmac limestone mixtures

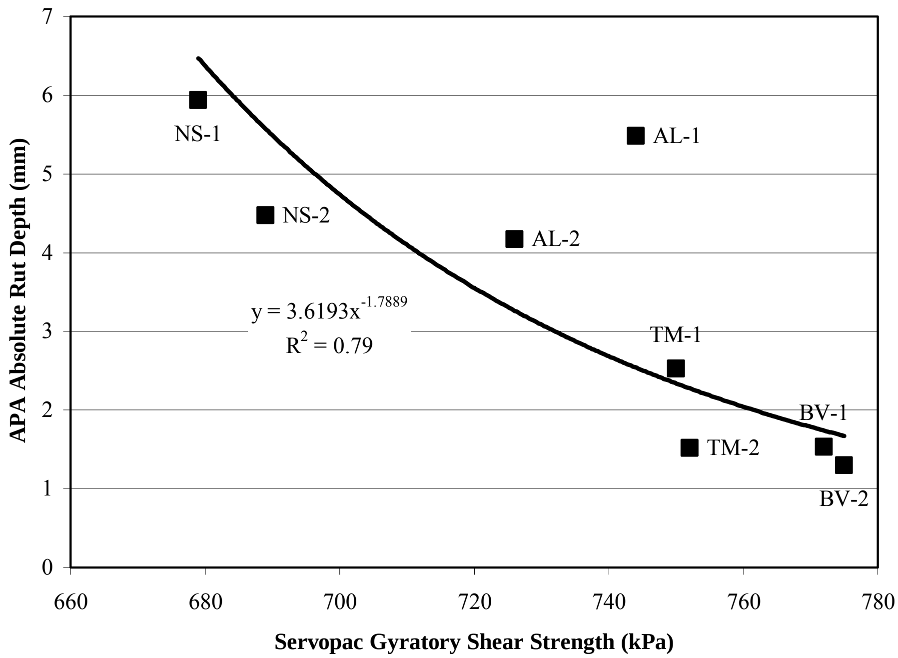

Figure 4-22 Gyratory shear stress versus APA rut depth

4.3.4 Servopac Summary

The Servopac test results for vertical strain and maximum shear strength correlated well with the APA test results. With respect to vertical strain, the mixtures showed an improvement from round one to round two and the results stayed within the desirable range of 1.4 to 2.0 percent. The ranking of the mixtures with respect to the APA test results matched the rankings per the maximum gyratory shear test results. There was a decrease in gyratory shear slope from round one to round two for three of the four mixtures, indicating that the round one mixtures develop shear resistance at a faster rate than the round two mixtures, though they do not necessarily achieve a greater maximum shear strength.

Some mixtures tend to reach maximum shear strength at much higher air void contents than other mixtures. For the eight mixtures examined in this study, the air void content at which maximum shear strength was achieved differed by up to 3.6 percent. Additionally, three of the four round two mixtures with lower VMA reached maximum shear strength at a higher air void content that their counterpart round one mixtures. This reveals one of the main problems with normal volumetric mix design procedures, where all mixtures are designed at four percent air voids. Some mixtures may be optimal at this air void content and others may not be. This is further justification for the need of one or more performance tests for mix design purposes.

4.4 Cracking

The Superpave indirect tension (IDT) test results can be best described by three parameters; energy ratio, dissipated creep strain energy and fracture energy. Test results are shown in Table 4-8 for the unconditioned and long-term oven aged (LTOA) specimens. Each parameter will be discussed separately below.

Table 4-8 Energy ratio values for the unconditioned and LTOA specimen

Superpave IDT Test Parameter	Test Condition	Alabama limestone		FL Brooksville limestone		Nova Scotia granite		FL Tarmac limestone	
		Round 1	Round 2	Round 1	Round 2	Round 1	Round 2	Round 1	Round 2
Energy Ratio @ 100 psi	Unconditioned	3.36	3.18	2.08	2.53	3.69	1.31	2.64	1.62
	LTOA	1.20	1.93	5.93	5.33	4.04	3.44	4.65	8.58
Dissipated Creep Strain Energy (kJ/m^3)	Unconditioned	4.2	4.1	1.5	1.3	5.7	4.2	2.6	1.6
	LTOA	1.2	2.0	1.4	0.8	6.4	5.0	1.7	2.4
Fracture Energy (kJ/m^3)	Unconditioned	4.4	4.3	1.7	1.5	5.9	4.4	2.8	1.7
	LTOA	1.4	2.1	1.6	1.1	6.6	5.2	1.9	2.6

4.4.1 Energy Ratio

As described in Chapter 3, the energy ratio is defined as the dissipated creep strain energy threshold of a material divided by the minimum dissipated creep strain energy needed. Roque et al. (2004b) have found this parameter effective in characterizing the cracking performance of asphalt mixtures. Figure 4-23 shows the unconditioned energy ratio and Figure 4-24 shows the LTOA energy ratio for rounds one and two for each mixture type.

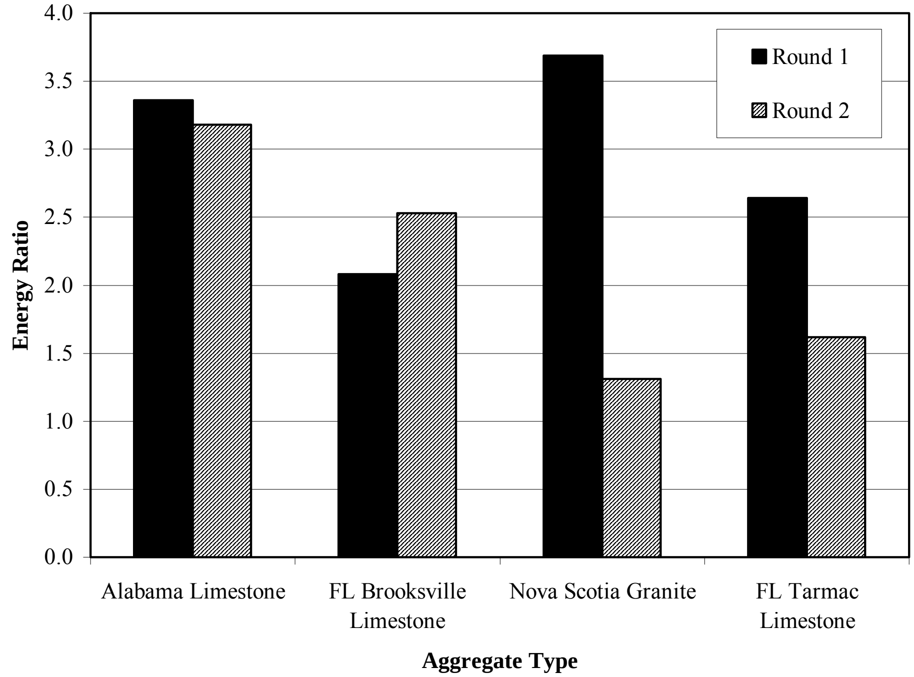

Figure 4-23 Energy ratios for unconditioned specimens

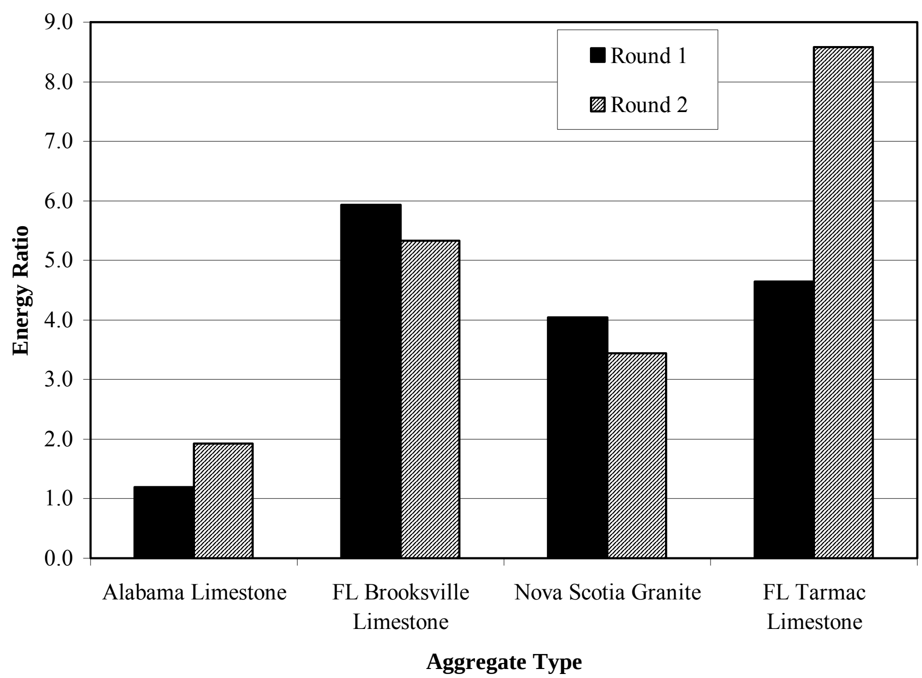

Figure 4-24 Energy ratios for long-term oven aged specimens

Figure 4-23 shows that the energy ratio decreased for three of the four unconditioned mixtures (Alabama limestone, Nova Scotia granite and Florida Tarmac limestone). The decrease was significant for the Nova Scotia granite and Florida Tarmac limestone mixtures. It should be noted that these two mixtures had significant decreases in VMA from rounds one to two. The energy ratio increased a moderate amount for the Florida Brooksville limestone mixture. The implication is that the addition of coarse aggregate at round two had an overall negative effect on the cracking performance of the mixtures examined in this study.

With respect to the LTOA specimens, Figure 4-24 shows that the results were mixed between rounds one and two. Energy ratios decreased for the Florida Brooksville limestone mixture and Nova Scotia granite mixture and increased for the Alabama limestone and Florida Tarmac limestone mixture. Overall, only the energy ratio for the Florida Tarmac limestone mixture changed significantly. The reason for this is unknown. Comparing the unconditioned energy ratios to the LTOA energy ratios reveal a significant increase in energy ratio after aging for the Florida Brooksville and Tarmac limestone mixtures. These mixtures contain aggregates that are highly absorptive compared to the Alabama limestone and Nova Scotia granite mixtures.

4.4.2 Dissipated Creep Strain Energy (DCSE)

The DCSE of a mixture describes the amount of energy that a mixture can dissipate through repeated loading before fracturing. Though the DCSE by itself cannot describe completely the cracking performance of a mixture, as a rule of thumb, if other factors are held constant, then a mixture with a greater DCSE will perform better than a mixture with a lower DCSE. Figure 4-25 shows the unconditioned DCSE and Figure 4-26 shows the LTOA DCSE for rounds one and two for each mixture type.

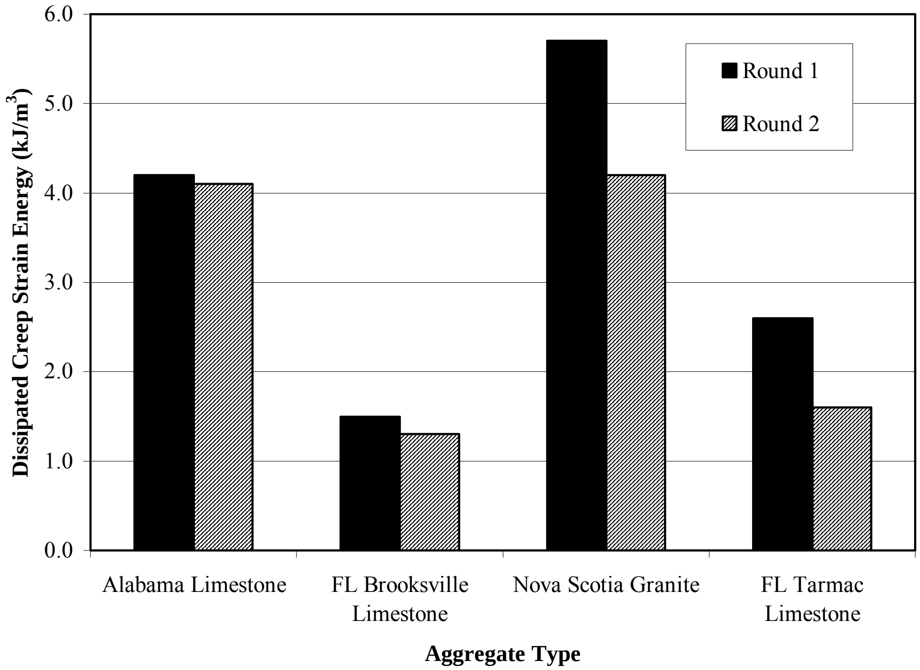

Figure 4-25 Dissipated creep strain energy for unconditioned specimens

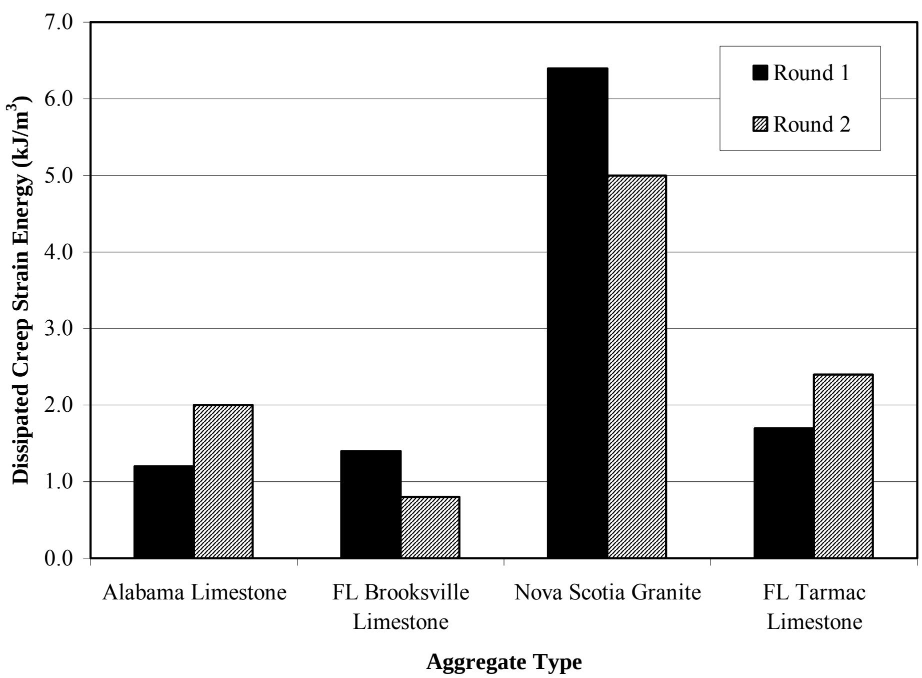

Figure 4-26 Dissipated creep strain energy for long-term oven aged specimens

Figure 4-25 shows that the DCSE decreased significantly for the Nova Scotia granite and Florida Tarmac limestone unconditioned mixtures from round one to round two. It should be noted that these two mixtures had significant decreases in VMA from rounds one to two. There was a slight, if not insignificant, decrease in DCSE for the Alabama limestone and Florida Brooksville limestone unconditioned mixtures. The implication is that the addition of coarse aggregate at round two had a negative effect on the DCSE of the mixtures examined in this study.

With respect to the LTOA specimens, Figure 4-26 shows that the results were mixed between rounds one and two. DCSE decreased for the Florida Brooksville limestone mixture and Nova Scotia granite mixture and increased for the Alabama limestone and Florida Tarmac limestone mixture. This is the same trend as occurred for the energy ratios of the LTOA specimens.

4.4.3 Fracture Energy (FE)

The FE of a mixture describes the total amount of energy (elastic energy plus dissipated energy) that a mixture can withstand before fracturing. Though the FE by itself cannot describe completely the cracking performance of a mixture, as a rule of thumb, if other factors are held constant, then a mixture with a greater FE will perform better than a mixture with a lower FE. Figure 4-27 shows the unconditioned FE and Figure 4-28 shows the LTOA FE for rounds one and two for each mixture type.

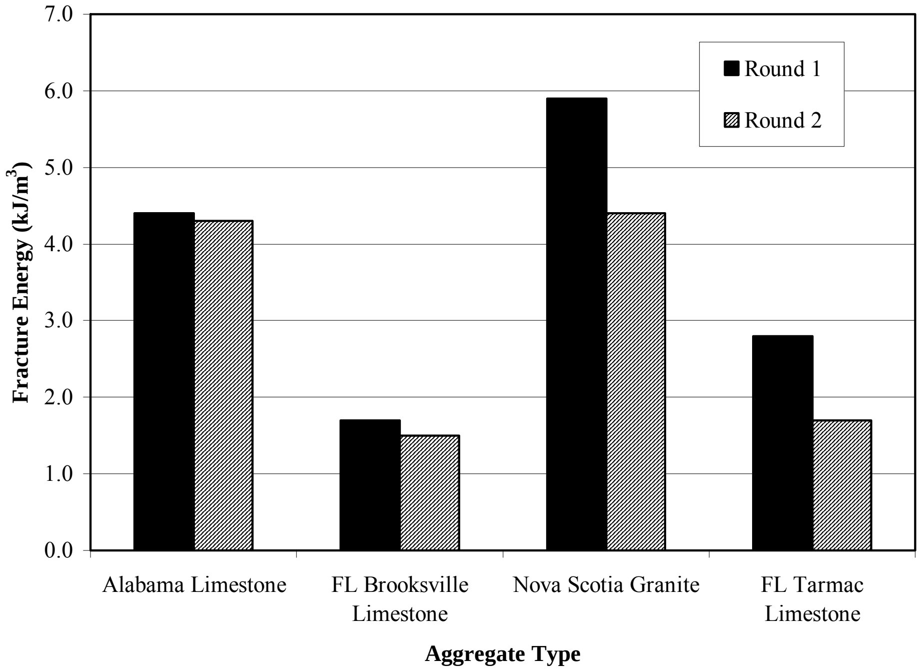

Figure 4-27 Fracture energy for unconditioned specimens

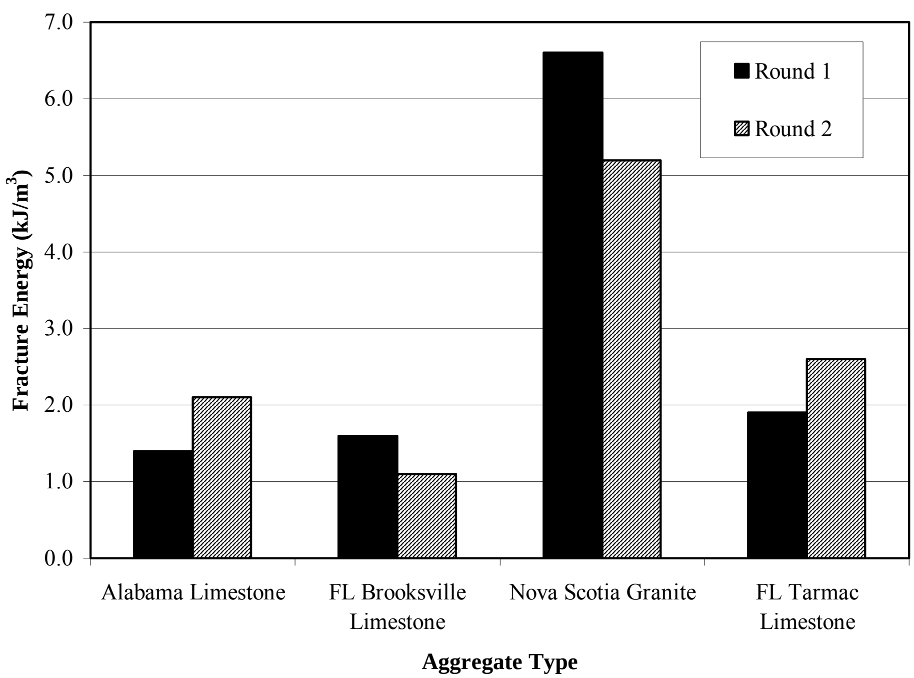

Figure 4-28 Fracture energy for long-term oven aged specimens

Figure 4-27 shows that the FE decreased significantly for the Nova Scotia granite and Florida Tarmac limestone unconditioned mixtures from round one to round two. It should be noted that these two mixtures had significant decreases in VMA from rounds one to two. There was a slight, if not insignificant, decrease in FE for the Alabama limestone and Florida Brooksville limestone unconditioned mixtures. The implication is that the addition of coarse aggregate at round two had a negative effect on the FE of the mixtures examined in this study.

With respect to the LTOA specimens, Figure 4-28 shows that the results were mixed between rounds one and two. FE decreased for the Florida Brooksville limestone mixture and Nova Scotia granite mixture and increased for the Alabama limestone and Florida Tarmac limestone mixture. This is the same trend as occurred for the energy ratios and DCSE of the LTOA specimens.

4.4.4 Cracking Summary

The energy ratio, dissipated creep strain energy and fracture energy test results from the Superpave IDT test indicate that the addition of coarse aggregate, resulting in a more continuous gradation and reduction in VMA, had an overall negative effect on the cracking performance when examining the unconditioned specimens. Only the energy ratio for the Florida Brooksville limestone mixture showed an increase from round one to round two.

Results were mixed and not conclusive for the LTOA specimens. However, the highly absorptive Florida limestone mixtures showed significant increases in energy ratio for the LTOA specimens compared to the unconditioned specimens. This trend was not evident for the Alabama limestone and Nova Scotia granite mixtures.

4.5 Moisture Damage

Moisture damage was evaluated using the standard Department test procedure (FM 1-T 283) and by using the Superpave IDT tests on moisture conditioned specimens. From the suite of Superpave IDT tests, the energy ratio was calculated for both the unconditioned and conditioned specimens. Test results are shown in Table 4-9. Each parameter will be discussed separately below.

Table 4-9 Moisture damage test results

Test Method and Condition		Alabama limestone		FL Brooksville limestone		Nova Scotia granite		FL Tarmac limestone	
		Round 1	Round 2	Round 1	Round 2	Round 1	Round 2	Round 1	Round 2
FM 1-T 283 Tensile Strength (kPa)	Unconditioned	989	941	1026	1208	785	843	855	1025
	Moisture Conditioned	764	840	596	698	647	697	699	791
	Ratio	77	89	58	58	83	83	82	77
Energy Ratio	Unconditioned	3.36	3.18	2.08	2.53	3.69	1.31	2.64	1.62
	Moisture Conditioned	2.87	2.66	1.87	0.97	0.44	1.08	2.61	1.50
	Ratio	85	84	90	38	12	82	99	93

4.5.1 Conventional FM 1-T 283 Test Results

Examination of the data in Table 4-9 does not indicate any trends with respect to tensile strength ratio (TSR). The Alabama limestone mixture had a twelve percent increase in TSR. The Florida Brooksville limestone and Nova Scotia granite mixtures showed no change in TSR and the Florida Tarmac limestone mixture showed a mild five percent reduction in TSR. However, with respect to the tensile strengths, every mixture had an increase in unconditioned and conditioned tensile strengths from round one to round two except for the Alabama limestone unconditioned mixture, which had a mild reduction in unconditioned tensile strength (7 psi) from round one to round two.

4.5.2 Superpave IDT Test Results (Energy Ratio)

Examination of the data in Table 4-9 reveals a different outcome than the FM 1-T 283 test results. Only the Nova Scotia granite conditioned results showed an increase in energy ratio from round one to round two. The other three comparisons showed a decrease in energy ratio from round one to round two.

4.5.3 Moisture Damage Summary

Moisture damage test results were dependent on the test method used. The standard Department test method, FM 1-T 283, revealed that tensile strengths increased for unconditioned and conditioned specimens from round one to round two. Superpave IDT test results showed that for three of four comparisons, the energy ratio decreased from round one to round two.

4.6 Permeability

The permeability values for rounds one and two of each mixture type are presented in Table 4-10. Permeability values were essentially the same between rounds one and two. The addition of coarse aggregate in round two did not affect the permeability of the mixture. Perhaps there was an offsetting effect between adding more coarse aggregate, which would increase the permeability of the mixture, and the more continuous gradation which being closer to the maximum density line, would tend to decrease permeability.

Table 4-10 Permeability test data

Specimen Number	Permeability (x 10^{-5} cm/s)							
	Alabama limestone		FL Brooksville limestone		Nova Scotia granite		FL Tarmac limestone	
	Round 1	Round 2	Round 1	Round 2	Round 1	Round 2	Round 1	Round 2
Specimen 1	41	15	13	15	6	14	39	121
Specimen 2	24	9	16	17	9	13	64	16
Specimen 3	13	49	28	27	13	9	69	24
Specimen 4	n/a	31	n/a	n/a	n/a	n/a	37	n/a
Average	26	26	19	20	10	12	52	54

CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

- Rutting potential, as measured by the Asphalt Pavement Analyzer (APA), showed an improvement in rut performance by the addition of more coarse aggregate on the 12.5 mm and 9.5 mm sieves, resulting in a more continuous gradation. This improvement was evident with absolute rut depth, differential rut depth and percent area change of the rut profile, when testing 75 mm tall specimens compacted to seven percent air voids and tested at 64 °C.

- APA test results also showed a strong correlation ($R^2 = 0.70$) that increasing VMA resulted in an increase in rut depth. An even stronger correlation ($R^2 = 0.79$) showed that increasing the dust to effective binder ratio resulted in a decrease in rut depth.

- Rutting potential, as measured with the Servopac gyratory compactor, was evaluated with the following parameters: gyratory shear slope, vertical strain and maximum gyratory shear stress. Vertical strain and maximum gyratory shear stress test results correlated well with the APA test results. With respect to vertical strain, the mixtures showed an improvement from round one to round two and the results stayed within the desirable range of 1.4 to 2.0 percent. The ranking of the mixtures with respect to the APA test results matched the rankings per the maximum gyratory shear test results. There was a decrease in gyratory shear slope from round one to round two for three of the four mixtures, indicating that the round one mixtures develop shear resistance at a faster rate than the round two mixtures. However, the round one mixtures did not necessarily achieve a greater maximum shear strength than the round two mixtures.

- Servopac test results show that mixtures achieve their maximum gyratory shear strength over a wide range of air voids compared to each other. Designing all mixtures at four percent air voids may not result in the optimum mixture design for all mixtures with respect to rut resistance.

- The energy ratio, dissipated creep strain energy and fracture energy test results from the Superpave IDT test indicate that the addition of coarse aggregate, resulting in a more continuous gradation and reduction in VMA, had an overall negative effect on the cracking performance. Only the energy ratio for the Florida Brooksville limestone mixture showed an increase from round one to round two.

- Conclusions with respect to moisture damage were dependent on the test method used. The standard Department test method, FM 1-T 283, revealed that the addition of coarse aggregate in round two resulted in increased tensile strengths for unconditioned and conditioned specimens. However, Superpave IDT test results showed that for three of four comparisons, the energy ratio decreased with the addition of coarse aggregate in round two.

- Permeability characteristics of the mixtures were not affected by the addition of coarse aggregate. Most likely there was an offsetting effect between adding more coarse aggregate, which would tend to increase the permeability of the mixture, and the more continuous gradation, which being closer to the maximum density line, would tend to decrease permeability.

5.2 Recommendations

- Cracking is the predominant mode of distress (approximately 80%) for the asphalt roads in Florida. For the mixtures evaluated in this study, the addition of coarse aggregate on the 12.5 mm and 9.5 mm sieves indicated an overall reduction in cracking performance. Therefore, it is not recommended at this time to lower the VMA specification requirement for coarse graded mixtures.

- For situations where rutting performance is a high priority, the addition of coarse aggregate, with the potential for a lower than specified VMA, should be considered.

- The Department should continue work towards the implementation of one or more performance tests at the mix design stage. Possible candidate test methods include the APA, Servopac and Superpave indirect tension test. As a first step, the Department could specify minimum performance values that mixtures would be required to meet at the mix design stage.

- Testing in this study and others has revealed that not all mixtures have optimal performance when volumetrically designed according to current Superpave mixture design requirements. Research exploring new mix design methodologies, which optimize a mixture’s performance based on laboratory performance test(s), should be explored. Gradations and asphalt contents would be selected to optimize performance, not to meet certain volumetric criteria.

ACKNOWLEDGMENTS

I would like to acknowledge my graduate advisor, Dr. Reynaldo Roque, for his patient guidance and wisdom throughout this long process. I would also like to thank Dr. Bjorn Birgisson and Dr. Mang Tia for their advice and assistance in performing the work and analysis of this thesis.

I offer the most sincere appreciation to Mr. Howie Moseley of the Florida Department of Transportation. Without Mr. Moseley's support, help and advice, this thesis would have not been finished.

I would like to thank my supervisors, Mr. Jim Musselman and Mr. Gale Page, for allowing me the time and for offering the moral support to pursue my master's degree while working fulltime.

I would like to thank Ms. Susan Andrews, Mrs. Shanna Johnson, Mr. Joshua Whitaker and Mr. Stephen Browning of the Florida Department of Transportation for their assistance in performing laboratory testing.

I would like to thank Dr. Christos Drakos and Ms. Tanya Riedhammer of the University of Florida for their assistance in performing testing and analysis.

LIST OF REFERENCES

Anderson, R.M., R.A. Bentsen. Influence of Voids in the Mineral Aggregate (VMA) on the Mechanical Properties of Coarse and Fine Asphalt Mixtures. Journal of the Association of Asphalt Paving Technologists, St. Paul, MN, Volume 70, 2001, pp. 1-37.

Asphalt Institute. Superpave Mix Design, Superpave Series No. 2, SP-2, Lexington, KY, 1996.

Birgisson, B., R. Roque, G.C. Page. Evaluation of Water Damage Using Hot Mix Asphalt Fracture Mechanics. Journal of the Association of Asphalt Paving Technologists, St. Paul, MN, Volume 72, 2003, pp. 424-462.

Coree, B.J., W.P. Hislop. Difficult Nature of Minimum Voids in the Mineral Aggregate. Transportation Research Record 1681, Transportation Research Board, National Research Council, Washington, DC, 1999, pp. 148-156.

Coree, B.J., W.P. Hislop. A Laboratory Investigation into the Effects of Aggregate-Related Factors on Critical VMA in Asphalt Paving Mixtures. Journal of the Association of Asphalt Paving Technologists, St. Paul, MN, Volume 70, 2001, pp. 70-131.

Drakos, C.A. Identification of a Physical Model to Evaluate Rutting Performance of Asphalt Mixtures. Ph.D. dissertation, University of Florida, Gainesville, FL, 2003.

Hand, A.J., J.L. Stiady, T.D. White, A.S. Noureldin, K. Galal. Gradation Effects on Hot-Mix Asphalt Performance. Transportation Research Record 1767, Transportation Research Board, National Research Council, Washington, DC, 2001, pp. 152-157.

Kandhal, P.S., S. Chakraborty. Evaluation of Voids in the Mineral Aggregate for HMA Paving Mixtures. National Center for Asphalt Technology Report No. 96-4, Auburn, AL, 1996.

Kandhal, P.S., K.Y. Foo, R.B. Mallick. A Critical Review of VMA Requirements in Superpave. National Center for Asphalt Technology Report No. 98-1, Auburn, AL, 1998.

Kandhal, P.S., R.B. Mallick. Effect of Mix Gradation on Rutting Potential of Dense-Graded Asphalt Mixtures. Transportation Research Record 1767, Transportation Research Board, National Research Council, Washington, DC, 2001, pp. 146-151.

Leahy, R.B., R.B. McGennis. Asphalt Mixes: Materials, Design and Characterization. Journal of the Association of Asphalt Paving Technologists, St. Paul, MN, Volume 68A, 1999, pp. 70-127.

Nukunya, B., R. Roque, B. Birgisson, M. Tia. Evaluation of Superpave Criteria for VMA and Fine Aggregate Angularity, Volume 1. Report for FDOT contract BB-498, University of Florida, Gainesville, FL, 2002.

Ruth, B.E., B. Birgisson. An Overview of Operational Aspects Relating to Volumetric Design and Construction of Asphalt Paving Mixtures. Canadian Technical Asphalt Association, Calgary, Canada, 1999, pp. 345-368.

Ruth, B.E., R. Roque, B. Nukunya. Aggregate Gradation Characterization Factors and Their Relationship to Fracture Energy and Failure Strain of Asphalt Mixtures. Journal of the Association of Asphalt Paving Technologists, St. Paul, MN, Volume 71, 2002, pp. 310-344.

Roque, R., B. Birgisson, D. Darku, C.A. Drakos. Evaluation of Laboratory Testing Systems for Asphalt Mixture Design and Evaluation, Volume 2. Report for FDOT contract BB-888, University of Florida, Gainesville, FL, 2004.

Roque, R., B. Birgisson, C.A. Drakos, B. Dietrich. Development and Field Evaluation of Energy-Based Criteria for Top-down Cracking Performance of Hot Mix Asphalt. Paper presented at annual meeting of Association of Asphalt Paving Technologists, Baton Rouge, LA, 2004.

Sholar, G.A., G.C. Page, J.A. Musselman, P.B. Upshaw, H. Moseley. Examination of Aggregate Degradation and Effect on Volumetric Properties. FDOT research report 00-444, Florida Department of Transportation, State Materials Office, Gainesville, FL, 2001.