NCAT Report 11-01

LABORATORY REFINEMENT AND FIELD VALIDATION OF 4.75 mm SUPERPAVE DESIGNED ASPHALT MIXTURES

Volume I: Final Report

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March 2011

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DISCLAIMER

ACKNOWLEDGMENTS

The authors would like to thank the Alabama Department of Transportation, the Connecticut Department of Transportation, the Florida Department of Transportation, the Minnesota Department of Transportation, the Missouri Department of Transportation, the New Hampshire Department of Transportation, the Tennessee Department of Transportation, the Virginia Department of Transportation, and the Wisconsin Department of Transportation for their support and cooperation with this research. Special thanks are given to Dr. Osamu Takahashi of Nagaoka University of Technology for all of his efforts in the laboratory research for this project.

LIST OF ACRONYMS/ABBREVIATIONS

APA – Asphalt Pavement Analyzer

AASHTO – American Association of State Highway and Transportation Officials

ASTM – Association of Standards for Testing and Materials

CTM – Circular Texture Meter

D:B ratio – Dust-to-Binder Ratio

DFT – Dynamic Friction Tester

ESAL – Equivalent Single-Axle Loads

FAA – Fine Aggregate Angularity

FE– Fracture Energy

FM – Fineness Modulus

*G**- Complex Shear Modulus

G_{mm} – Mix Theoretical Maximum Specific Gravity

G_{mb} – Mix Bulk Specific Gravity

HMA – Hot-Mix Asphalt

IDT- Indirect Tension

LVDTs – Linear Variable Differential Transducers

Manf-sand – Manufactured Sand

MPD– Mean Profile Depth

MVP – Mixture Verification Tester

NCAT – National Center for Asphalt Technology

N_{des} – Number of Gyrations at Design Level

N_{ini} – Number of Gyrations at Initial Compaction

NMAS – Nominal Maximum Aggregate Size

P-075 – Percent Passing the 0.075 mm Sieve

P_b – Percent Binder (preferable over “AC”)

P_{be} – Percent of Effective Binder

PG – Performance Grade (pertaining to asphalt binder)

QC/QA – Quality Control/Quality Assurance

RAP – Reclaimed Asphalt Pavement

SE – Sand Equivalent

SGC – Superpave Gyrator Compactor

TSR – Tensile Strength Ratio

V_a – Air Voids

V_{be} – Volume of Effective Binder

VFA – Voids Filled with Asphalt

VMA – Voids in Mineral Aggregate

WMA – Warm-Mix Asphalt

WSD – Washed Sand

Statistical Acronyms

ADJ SS – Adjusted Sum of Squares

ADJ MS – Adjusted Mean Square

ANOVA – Analysis of Variance

P – *p*-value

S – Standard Deviations

SE – Squared Error

SS – Sum of Squares

SS sr – Reduced Sum of Squares

X - Mean

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CHAPTER 1 INTRODUCTION

1.0 INTRODUCTION

Until recently, 9.5 mm was the smallest nominal maximum aggregate size (NMAS) used in Superpave mix design criteria. In 2002, the National Center for Asphalt Technology (NCAT) completed a research study to develop Superpave mix design criteria for 4.75 mm NMAS mix *(1)*. With the help of this research, the Superpave Mixture/Aggregate Expert Task Group recommended to AASHTO the addition of 4.75 mm NMAS mixes to the Superpave mix design system.

Many state agencies have expressed an interest in using 4.75 mm NMAS Superpave designed mixtures for thin lift applications, thin lift maintenance, and leveling courses to decrease construction time, to provide a use for screening stockpiles, and to provide an economical surface mix for low volume roads. A 2004 NCAT survey of state agencies indicated that several states were using 4.75 mm NMAS mixtures or mix types reasonably close to the AASHTO criteria for 4.75 mm mixes. The survey also confirmed that more agencies were interested in using 4.75 mm NMAS mixes in the future.

Although the original NCAT study on 4.75 mm mixes *(1)* provided initial criteria for 4.75 mm NMAS Superpave mixes, it was recommended that the mix design criteria be further refined in the laboratory and field validated. Criteria refinement was recommended in the following areas:

1. Minimum VMA criteria and dust-to-binder ratio (D:B ratio) requirements
2. Maximum VMA requirements
3. %G_{mm} @ N_{ini} criteria
4. Aggregate properties
5. Binder contents and design air voids (Va) level (e.g., 4%)
6. Enhanced performance with the use of polymer modified binders

Since the original study *(1)* was performed with two aggregate sources, it was also recommended that the refinement study incorporate materials from more states to obtain a large range of aggregate types.

In 2004, a pooled fund study was initiated to refine mix design criteria for 4.75 mm NMAS Superpave designed mixes and field validate design criteria. Nine states were participants in this study: Florida, Virginia, Missouri, Minnesota, Alabama, Tennessee, Wisconsin, New Hampshire, and Connecticut. Research began at NCAT in winter of 2005 for the laboratory refinement phase of this project.

1.1 Objective

The main objective of this study is to refine the current procedures and criteria for 4.75 mm NMAS Superpave designed mixtures. Specifically, the criteria to be refined were

- Minimum Voids in the Mineral Aggregate (VMA) requirements and a workable range for Voids Filled with Asphalt (VFA)
- Percent of Mixture Theoretical Maximum Specific Gravity at Number of Gyrations at Initial Compaction (%G_{mm} @ N_{ini}) requirements
- Aggregate properties such as Sand Equivalence (SE) and Fine Aggregate Angularity (FAA) of mixture
- Appropriate design air void content for a given compaction effort
- D:B ratio requirements
- A recommendation on the use of modified binders to enhance performance of 4.75 mm NMAS asphalt mixtures

1.2 Scope

A literature review was completed to understand the history and practical use of 4.75 mm NMAS Superpave designed mixtures. Next, a laboratory test plan was created. This test plan included performing numerous Superpave mix designs using materials provided by each state participating in the study. For each material and mix design, aggregate properties were measured, optimum asphalt content was determined for a given compaction effort and design V_a, and performance tests were conducted to determine how well the mixtures performed for a given set of properties. The results of these mix designs were compared with the current AASHTO specification for 4.75 mm NMAS Superpave mixtures. These comparisons coupled with the results of the performance tests are used to measure the appropriateness of the current specification and to make improvement recommendations.

This report presents the findings of the pooled-fund 4.75 mm Superpave refinement project.

CHAPTER 2 BACKGROUND

2.0 BACKGROUND

2.1 Development of Mix Design Criteria for 4.75 mm Superpave Mixes

In 2002, Cooley et al. *(1)*, published research conducted at NCAT on the topic of specifications for 4.75 mm Superpave mixtures. The objective of this study was to develop mix design criteria for 4.75 mm NMAS mixtures. The research targeted the criteria of gradation controls and volumetric property requirements. Only two aggregate types were used in this study: granite and a limestone. Three gradations were evaluated for each gradation type: fine, medium and coarse. For each mixture, asphalt content was determined for 4% and 6% V_a at a compactive effort of 75 gyrations. To analyze rutting susceptibility, an Asphalt Pavement Analyzer (APA) was used. This study confirmed that 4.75 mm NMAS can be successfully designed in the laboratory. Optimum binder contents of the designed mixes were affected by aggregate type, shape of the gradation curve, dust content, and designed V_a. Voids in mineral aggregate values were affected by aggregate type, shape of the gradation curve, and dust content. APA rutting results were very high for the 4.75 mm mixtures as a result of the high binder contents required to meet the target V_a of 4.0%. Although a good relationship was evident between VMA and D:B ratio, the finding could be limited by the two aggregate sources used. The results of the study, combined with existing mix design criteria from Maryland and Georgia, led to the following recommendations for designing 4.75 mm mixtures:

- Gradations for 4.75 mm NMAS mixes should be controlled on the 1.18 mm and 0.075 mm sieves.
- On the 1.18 mm sieve, the gradation control points are recommended as 30% to 54%.
- On the 0.075 mm sieve, the control points are recommended as 6% to 12%.
- A target designed V_a of 4.0% should be used.
- For all traffic levels, a minimum VMA of 16.0 should be utilized.
- For mixes designed at 50 gyrations (very low traffic applications), no maximum VMA criteria should be utilized. For mixes designed at 75 gyrations and above, a maximum VMA value of 18% is rational.
- For mixes designed at 75 gyrations and above, VFA criteria should be 75% to 78%.
- Percent G_{mm} @ N_{ini} values currently specified in AASHTO MP2-01 for the different traffic levels are recommended.
- Criteria for D:B ratio are recommended as 0.9 to 2.2.

Cooley provided the draft mix design criteria for 4.75 mm NMAS Superpave mixtures, but recommended that mix design procedures be refined in the following areas:

1. Minimum VMA Criterion and D:B Ratio Requirements: Laboratory work is needed to evaluate the aging characteristics of 4.75 mm NMAS mixes. The minimum criterion of 16.0% was selected based upon Maryland and Georgia minimum binder contents and gradation specifications on similar mixes. An evaluation of the maximum D:B ratio requirement should be included in this work.

2. Maximum VMA Criteria: High optimum binder contents were identified as the primary cause of excessive laboratory rutting. For this reason, a maximum VMA criteria of 18.0% was recommended. This value needs to be validated in the laboratory by designing numerous mixes with a wide range of aggregate types to further evaluate the relationship between VMA and rut resistance.
3. $\%G_{mm}$ @ N_{ini} Criteria: Within this study, two high quality aggregates were utilized. None of the 36 designed mixes failed the $\%G_{mm}$ @ N_{ini} criteria for a 75 gyration design (90.5%). Additional work needs to be conducted that incorporates various percentages of natural, rounded sand to evaluate the applicability of $\%G_{mm}$ @ N_{ini} requirements within the mix design system.
4. Aggregate Properties: Both of the aggregates used in this study had FAA values in excess of 45%. Refining the desired FAA values for different design levels should be further evaluated. Research is also needed to quantify an acceptable aggregate toughness and resistance to abrasion.
5. Air Void Criteria: To avoid excessive binder contents, field work should verify if 4.75 mm NMAS mixes can be designed at a single V_a level (e.g., 4%) and result in satisfactory performance or determine if a design V_a range criteria are needed.
6. Use of Polymer Modified Binders: Within a refinement study, some polymer modified binders should be included to evaluate any enhanced performance.

2.2 Use of Screenings to Produce HMA Mixtures

Historically, many agencies have specified coarse-graded Superpave mixtures because it was thought that coarse-graded mixtures were less susceptible to rutting. This has led to a large amount of screenings that are not being utilized. In 2002, Cooley et al. *(2)* presented research on the use of screenings to produce HMA mixtures. The main objective of this study was to determine if rut resistant HMA mixtures could be attained with the aggregate portion of the mixture consisting solely of manufactured aggregate screenings (man-sand). Secondary objectives were to determine what effect both a modified asphalt binder and a fiber additive might have on rutting.

Two as-produced fine aggregates were used: a granite and a limestone. Table 2.1 shows the gradation for these aggregates. The limestone aggregate met current AASHTO gradation specifications for 4.75 mm NMAS mixtures. Two asphalt grades were used: PG 64-22 and PG 76-22. Mixtures were designed at three different V_a (4, 5, and 6%) using 100 gyrations. There were eight mixture combinations of aggregate type, binder grade and fiber additive. The APA was used as a performance test to evaluate rutting potential.

TABLE 2.1 Gradations and Properties of Screenings *(2)*

Sieve Size (U.S. Standard)	Sieve Size (Metric)	Granite (% Passing)	Limestone (% Passing)
3/8 inch	9.50 mm	100	100
No. 4	4.75 mm	99	92
No. 8	2.36 mm	82	68
No. 16	1.18 mm	66	45
No. 30	0.600 mm	52	30
No. 50	0.300 mm	38	21
No. 100	0.150 mm	24	16
No. 200	0.075 mm	14.4	12.0
Aggregate Specific Gravities		**Granite**	**Limestone**
Apparent Specific Gravity (G_{sa})		2.726	2.746
Effective Specific Gravity (G_{se})		2.720	2.730
Bulk Specific Gravity (G_{sb})		2.711	2.616
Absorption (%)		0.2	1.8

Analysis of variance was used to evaluate the main factors affecting optimum asphalt content, VMA, %G_{mm} @ N_{ini}, and APA rut depths. The factors that significantly affected optimum asphalt content were aggregate type, the existence of fibers, and design V_a. Only two factors significantly affected VMA: aggregate type and the presence of fibers. %G_{mm} @ N_{ini} was affected by aggregate type and design V_a. Several factors affected APA rut depths: aggregate type, design V_a and binder grade. Also, significant two- and three-factor interactions that affected rut resistance were 1) aggregate type with design V_a, 2) aggregate type and binder grade, 3) fiber addition and design V_a, 4) design V_a and binder grade, 5) aggregate type, addition of fiber and binder grade, and 6) aggregate type, design V_a and binder grade. The following conclusions were obtained from this research:

- Mixes having screenings as the sole aggregate portion can be successfully designed in the laboratory for some screenings but may be difficult for others.
- Screenings type, the existence of cellulose fiber, and design V_a significantly affected optimum binder content. Of these three factors, screenings type had the largest impact on optimum binder content, followed by the presence of cellulose fiber and design V_a, respectively.
- Screenings type and the presence of cellulose fiber significantly affected voids in mineral aggregate. Screenings material had a larger impact.
- Screenings type and design V_a significantly affected %G_{mm} @ N_{ini} results. Again, the screenings material had the largest impact.
- Screenings type, design V_a, and binder type significantly affected laboratory rut depths. Of these three, binder type had the largest impact, followed by screenings type and design V_a, respectively. Mixes containing a PG 76-22 binder had significantly lower rut depths than mixes containing a PG 64-22. Mixes designed at 4% V_a had significantly higher rut depths than mixes designed at 5% or 6% V_a.

Based upon the conclusions of the study, the following recommendations were provided:

- Mixes utilizing a screenings stockpile as the sole aggregate portion and having a gradation that meets the requirements for 4.75 mm Superpave mixes should be designed in accordance with the recommended Superpave mix design system.
- Mixes utilizing a screenings stockpile as the sole aggregate portion but with gradations not meeting the requirements for 4.75 mm Superpave mixes should be designed using the following criteria:

Property	Criteria
Design Air Void Content, %	4–6
Effective Volume of Binder, %	12 min.
Voids Filled with Asphalt, %	67–80

2.3 Low Volume Road Applications

Since the development of the Superpave mix design system, most placed Superpave designed asphalt mixtures have been designed for high traffic volume applications. One proposed use of 4.75 mm NMAS mixtures is for light traffic applications. Mixtures with 4.75 mm NMAS will generally have a surface with minimal surface voids, which creates a smooth, impermeable surface texture. These properties would be ideal in subdivisions and bike trials where there is high pedestrian and low vehicle traffic. Although the definition of a low volume road may differ between agencies, it may generally be considered as one with less than 1 million design Equivalent Single Axle Loads (ESALs).

Several states have successfully used 4.75 mm NMAS-like mixes for years. Alabama, Maryland, and Georgia have used these mixtures for thin overlays and preventative maintenance with good results. However, Superpave designed mixtures are not commonly used in low traffic applications throughout the U.S. This may be partly because some county and city agencies believe that costs of using Superpave mixtures are prohibitive. Also, there is concern that Superpave designed mixtures will result in lower optimum asphalt contents that will lead to reduced durability, which is important for a long-lasting, low volume mixture resistant to fatigue and thermal cracking. Since requirements for low volume roads may be quite different than their high volume counterparts, a literature review on Superpave designed mixtures for low volume applications is provided.

To determine if Superpave could be successfully utilized for low traffic volume applications, a number of agencies have carried out research to compare traditional Marshall designed mixtures with Superpave design methods (*3,4,5*). The general concern was that a Superpave designed mixture would adversely affect mixture durability with lower optimum asphalt content. Although different approaches were used by different agencies, researchers tried to determine the design gyration level that would provide asphalt contents and volumetrics similar to Marshall designed mixtures that have a good performance history. Prowell et al. (*3*) found that a number of gyrations at design compaction (N_{des}) of 68 gyrations provided designed binder contents similar to a 50 blow Marshall with optimum binder content selected at 6% V_a. Mogawer et al. (*4*) recommends an N_{des} of 50 gyrations for a low volume road in New England. Habib et al. *(5*) suggested that N_{des} values used in Superpave mix design are about 20% higher than the required values. Habib concludes that lowering N_{des} would result in increased asphalt contents for Superpave mixtures. Prowell and Habib both found that VFA Superpave requirements for these types of mixtures may be too restrictive.

The Iowa Highway Research Board (*6*) conducted a study of eight projects paved in 1998 to evaluate the performance of Superpave designed asphalt mixtures for low volume roads. Of the eight mixtures, three were 19 mm NMAS, four were 12.5 mm NMAS, and one was a 9.5 mm NMAS. All mixtures used a performance graded 58-22 binder. The objective of this research was to evaluate what issues affect the use of Superpave designed mixtures on low volume roads. Issues evaluated included economics, resources, and constructability. The final review of this research found that after six years, all the pavements constructed for this research exhibited excellent cracking resistance, except one project that had reflective cracks that began to appear a few weeks after placement. However, the authors did not relate that cracking to the use of Superpave designed mixtures but attributed it to the expected reflective cracking of a thin overlay on top of a PCC pavement. Rutting on all involved projects was well within the range of acceptable values, under 0.1 inch. The researchers found it impossible to get an objective measure of project costs compared to paving with conventional mixtures. However, the engineers and contractors involved in the projects believed that costs involved with the projects did not significantly increase.

In a 2004 article published in *Asphalt Magazine* (*7*), three county engineers were interviewed about their experiences with Superpave designed mixtures for low volume county roads. The interviews were from Blue Earth County, Minnesota; Stearns County, Minnesota; and St. Louis County, Missouri. All three county engineers found that Superpave was effective for county roads. However, Stearns County found that costs for using Superpave designed mixture on low volume roads were prohibitive, but this county still planned to use it on arterials and higher traffic roads.

2.4 Leveling and Patching

Two possible uses for 4.75 mm NMAS Superpave mixtures are a leveling course or patching mix. A leveling course is defined as (*8*) a course (asphalt aggregate mixture) of variable thickness used to eliminate irregularities in the contour of an existing surface prior to superimposed treatment or construction. A smaller aggregate size mixture is beneficial for leveling applications where very thin lifts are needed to correct surface defects *(9).*

Patches are needed to repair weak areas in pavements, pot holes, or utility cuts. Structural patches should be designed and constructed with full depth asphalt concrete to ensure strength equal to or exceeding that of the surrounding pavement structure. Generally, there are three types of asphalt patching mixtures used (*9*): (a) hot mixed, hot laid, (b) hot mixed, cold laid, or (c) cold mixed, cold laid. Dense graded aggregates are used primarily for hot mixed, hot laid patching mixtures. Typical gradations of dense graded patching mixtures are presented in Table 2.2, which shows that the current AASHTO gradation limits for 4.75 mm NMAS Superpave mixtures would fall within the limits of gradation C. The majority of all patching mixtures have 9.5 mm or 12.5 mm NMAS gradations (*9*). However, some agencies do specify a 4.75 mm NMAS mixture for patching. Larger NMAS mixtures seem to be preferred because they provide better stability, especially in deeper patches. When shallow holes are filled, a smaller NMAS mixture is beneficial, especially when the mixture must be feathered at the edges.

TABLE 2.2 Typical Gradations of Dense-Graded Patching Mixtures *(9)*

Sieve Size	Percent Passing		
	A	B	C
19.0 mm	100		
12.5 mm	90-100	100	
9.5 mm	75-90	90-100	100
4.75 mm	47-68	60-80	80-100
2.36 mm	35-52	35-65	65-100
1.18 mm	24-40	-	40-80
0.600 mm	14-30	-	20-65
0.300 mm	9-20	6-25	7-40
0.075 mm	2-9	2-10	2-10

2.5 Thin Overlays and Surface Mixtures

It may be ideal to use 4.75 mm NMAS mixtures for thin overlays and surface mixtures. Hansen *(10*) stated that HMA overlays are probably the most versatile pavement prevention techniques available. They can improve structural capacity, improve ride, enhance skid resistance, reduce noise and improve drainage. However, thin overlays should only be placed on structurally sound pavements that exhibit surface distresses such as low-severity transverse and longitudinal cracking. According to NCHRP Report 531 (*11*), lift thickness should be at least three to four times the NMAS. For a thin overlay (less than one inch), a 4.75 mm NMAS asphalt mixture would meet a lift thickness to NMAS ratio of three to four.

The main function of a thin overlay of HMA may not be to provide strength to the pavement structure, but to protect a deteriorating pavement. If a thin overlay of 4.75 mm NMAS dense graded HMA is used as a surface mixture, it may provide a smooth, durable, water-tight surface. However, one possible concern of applying this type of mixture as surface mix is producing low surface texture. A low macro texture might lead to poor skid resistance, especially when a surface is wet.

2.6 NCAT Survey

As part of the initial portion of this study, a survey of the current usages and possible future applications of this type of mix was sent to all US state highway agencies. Twenty-one states responded to the survey, as shown in Figure 2.1. Eleven of the 21 respondents did not utilize a 4.75 mm NMAS mix designation or similar mix type. Table 2.3 summarizes responses from agencies that have such a mix type.

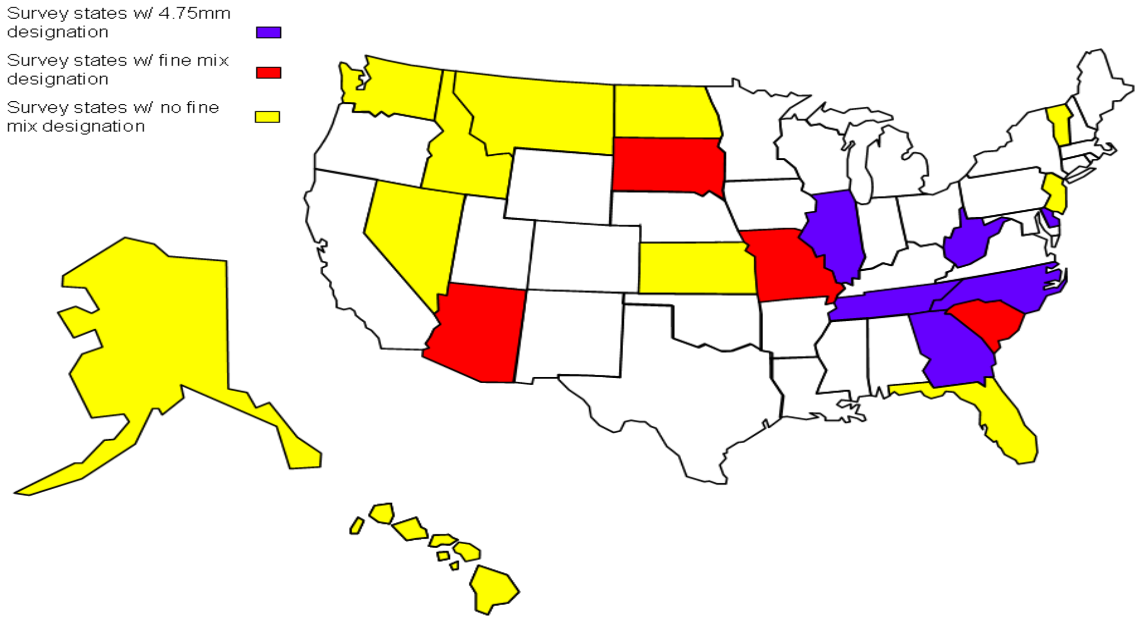

FIGURE 2.1 Map of Respondents to NCAT Survey

TABLE 2.3 Summary of Responses for States Having a 4.75 mm-Like Mix

State	Mix Design Method	Thickness or Spread Rate	In-place Density Requirement	Production is Expected To:	Primary Uses
Arizona	Arizona Method	50 lb/sy	none	decrease	Surface mix
Delaware	Superpave	Varies	none	increase	Leveling course
Georgia	Superpave	85 lb/sy	none	N/A	Leveling course
Illinois	Superpave	3/4"	94%	increase	Leveling course
Missouri	Marshall	1"–1.75"	no	remain steady	Surface, leveling, overlay
North Carolina	N/A	1"	85 or 90%	remain steady	N/A
South Carolina	Marshall	125 lb/sy	none	remain steady	Surface mix
South Dakota	Marshall	150 ton/mi	none	remain steady	Leveling mix
Tennessee	Marshall	35 lb/sy	none	remain steady	Leveling mix
Washington	N/A	N/A	N/A	N/A	N/A
West Virginia	Marshall	70 lb/sy	92%	increase	Surface mix

Generally, three types of aggregates are used in these 4.75 mm mixtures: 1) rock or chip (0 to 30%), 2) screenings (0-50%); and 3) natural sand (0-30%). The most common grade of asphalt use was a performance grade 64-22. Hydrated lime mixed at 1% is commonly used as an additive; also mentioned was cement and liquid anti-strip additives. A large range of spread rates were reported; the average was 80 lb/sy, with the range being 35–125 lb/sy. Superpave and Marshall mix design methods are both used to design 4.75 mm NMAS asphalt mixtures; for Superpave mixtures an N_{des} of 50 gyrations was typical. For the states that use Marshall designed mixtures, only Missouri disclosed the compaction effort used for their design (35 blows). Most states did not have current in-place density requirements for these mixtures; however three states do have in-place density requirements.

These mixture types were commonly used for leveling or scratch course, surface mixtures for low volume roads, and thin overlay for pavement maintenance. Appearance and performance were better than competing products and lower initial cost were cited as the most common advantages of this mixture type. Other listed advantages included the mixture's ability to be placed in lifts less than one inch, relieve abundance of quarry fines, help retard reflective cracking, and reduce noise.

Generally, the disadvantages mentioned were that this type of mixture does not provide enough strength to the pavement structure and can be susceptible to rutting. Most states believed the production quantity would remain steady or increase over the next two years. The average production rate is about 420,000 tons per year. Individual responses for production rate are given in Table 2.4.

TABLE 2.4 Approximate Production of 4.75 mm NMAS Mixtures

Delaware	<1000 tons
Georgia:	320,000 tons for FY 2004
Illinois:	Not yet adopted as common practice (N/A)
Tennessee:	225,000 tons
West Virginia:	15,000 – 20,000 tons
Arizona:	250,000 – 350,000 tons
South Carolina:	Low tonnage approximately 5% of total tonnage
South Dakota:	75,000 tons
Missouri:	1.7 million Surface level, and 750 thousand BP-2
North Carolina	SF9.5A: 1,000,000 tons, S4.75A: 75,000 tons

The final question asked what aspects of this type of mixture should be further developed. States' responses are given below:

- Florida: Leveling, thin overlays (maintenance/local agency)
- New Jersey: Plan to use as leveling on a concrete pavement overlay on an upcoming project. Right now we're planning on using the 4.75 mm mix in AASHTO M323.
- Vermont: For low ESAL Superpave, it must be able to resist rutting and cold weather climate capabilities.
- Hawaii: Thin overlay for preventive maintenance
- Nevada: Attempted to use a similar material in the past to fill substantial cracking; after failed attempts and problems, use was discontinued.
- North Dakota: Bike trails
- Washington: Thin wearing surfaces over structurally sound pavement
- Delaware: We are looking at the material for subdivision overlay work.
- Georgia: For low volume local roads, parking lots, etc.
- Illinois: Explore ways to add macro texture to allow as a surface course.
- South Dakota: All types of roads (surface mix)
- Missouri: Long-lasting surface mixtures for low volume roadways
- Iowa: Have an application as scratch course mix, but would not be specified for conventional HMA mixture (surface, intermediate, base)

An important finding from this survey was that 4.75 mm NMAS mixtures are being specified and used as surface mixtures, leveling courses, and thin overlays. There appears to be benefits in using this type of mixture for these applications. Most states agreed that 4.75 mm NMAS mixes should be further developed to increase the mixture type's overall structural capability and rutting resistance for use on low volume roads and in thin overlay applications.

CHAPTER 3 RESEARCH PLAN

3.0 RESEARCH PLAN

In spring 2005, representatives from the eight participating states met at NCAT to determine the test plan for the 4.75 mm Superpave refinement pooled-fund study. Items discussed at this meeting included the following:

- Expected applications for 4.75 mm mixes in each state
- Mix design criteria and concerns
- Construction and performance concerns
- Issues regarding specifics of performance testing (i.e., air void content for performance testing, type of test used for durability testing, and load and tire pressure used for rut testing)

A comprehensive test plan was created from this meeting. The experimental test matrix is shown in Table 3.1. This matrix shows that a 4.75 mm mix design was planned for all participating states using 50 gyrations and a design V_a of 4%. Variations of those mix designs were planned by changing the design gyrations and the design V_a. Additional variations were planned to evaluate changes in other mix factors such as dust content and binder grade. These are referred to as blend adjustment mixtures.

The first task was to obtain materials from each state. Participating states submitted a proposed 4.75 mm blend representing the sources and general gradation from their state. Aggregate materials received from the participating states were tested to determine gradations and aggregate specific gravities. Alternate trial blends were then completed in addition to the blends submitted by the states. Thirteen aggregate blends from participating states were designed at 4% V_a and 50 gyrations. Six of the 13 aggregate blends were also designed at 4% V_a and 75 gyrations. An additional seven of the 13 aggregate blends were designed at 6% V_a and 50 gyrations. Finally, three of the blends were designed at 6% V_a and 75 gyrations. The 50 and 75 gyration compaction levels were selected because 4.75 mm mixes will likely be used for lower volume traffic applications (less than 3 million ESALs). Design V_a of 4% and 6% were used to examine the concern of the mixes being over-asphalted due to high VMA values.

Also included in this study were four plant-produced baseline 4.75 mm mixtures from Mississippi, Maryland, Georgia, and Michigan that have known field performance and have been successfully used. These baseline mixtures served as benchmarks for comparing the results of the laboratory mix designs using the participating states' materials.

The mix identification code used in this report to describe the mix designs is defined as follows: The first two letters are used to define the state of origin (e.g., AL = Alabama). The first two numbers are the number of design gyrations, and the third number represents design V_a (e.g., AL-50-6 = Alabama material designed at 50 gyrations and 6% V_a). For blend adjustments, extra letters are given to describe the difference; for example, TNGM is used to denote Tennessee gravel mix, which is material from Tennessee but has a different source aggregate than the TN mix design from Tennessee limestone. To describe blend adjustments—mixtures composed with the same material as in the original mix designs but prepared in different stockpile proportions—the letters “adj” have been attached (e.g., FL adj = Florida blend adjusted).

Table 3.1 Original Mix Test Matrix

N_{des} Gyrations	50		75	
Air Voids	4.0	6.0	4.0	6.0
Aggregate Blend				
Florida	FL-50-4		FL-75-4	Fl-75-6
Wisconsin	WI-50-4	WI-50-6		
Virginia	VA-50-4		VA-75-4	
Missouri	MO-50-4	MO-50-6		
Minnesota	MN-50-4		MN-75-4	MN-75-6
Alabama	AL-50-4	Al-50-6		
Tennessee	TN-50-4		TN-75-4	
Connecticut	CT-50-4	CT-50-6		
New Hampshire	NH-50-4		NH-75-4	NH-75-6
Blend Adjustment 1	WI adj-50-4	WI adj-50-6		
Blend Adjustment 2	VA adj-50-4		VA adj-75-4	
Blend Adjustment 3	FL adj-50-4			FL adj-75-6
Blend Adjustment 4	TNGM-50-4		TNGM-75-4	
Baseline Mix 1	MS-50-4			
Baseline Mix 2		GA-50-6		
Baseline Mix 3			MD-75-3.5	
Baseline Mix 4			MI-60-4	

3.1 Test Methods

3.1.1 Aggregate Tests

An aggregate analysis for gradation and specific gravity was performed on each virgin aggregate material sent to NCAT for this study. Gradations were performed in accordance with AASHTO (T 27), *Sieve Analysis of Fine and Coarse Aggregate*, and AASHTO (T 11), *Materials Finer Than 75µm (No.200) Sieve in Mineral Aggregate by Washing.* Specific gravities were determined by AASHTO (T 84), *Specific Gravity and Absorption of Fine Aggregate.* For the final blended aggregate determined from the mix design, AASHTO (T 304) *Uncompacted Void Content of Fine Aggregate* and AASHTO (T 176) *Plastic Fines in Graded Aggregates and Soils by use of the Sand Equivalent Test* were performed.

3.1.2 Mix Designs

The AASHTO standard practice (R 35-3), *Superpave Volumetric Design for Hot Mix Asphalt (HMA)*, was followed during the mix design phase of the study. This standard practice was used to verify specifications for 4.75 mm NMAS in AASHTO (M 323-04), *Standard Specifications for Superpave Volumetric Mix Design*.

Three aggregate blend gradations were selected for each of the eight participating state's aggregate stockpiles. One of the three blends used in the aggregate trials was the blend proportion submitted by each state for their materials. The current gradation specification for 4.75 mm mixes is shown in Table 3.2; these control points were used in the blending process. Control points for the 4.75 mm sieve (90–100% passing) were strictly observed in the blending process to maintain a true 4.75 mm NMAS mix. However, some blend gradations were allowed to go outside of the control points on the #16 (1.18 mm) and #200 (0.075 mm) sieve so the effect of these limits could be evaluated. Also, since most states provided only two to three aggregate stockpiles, it was not always possible to develop reasonable alternative blends by proportioning the stockpiles percentages and meet the current gradation limits. Figure 3.1 shows all the gradations used in this study plotted on a 0.45 power chart.

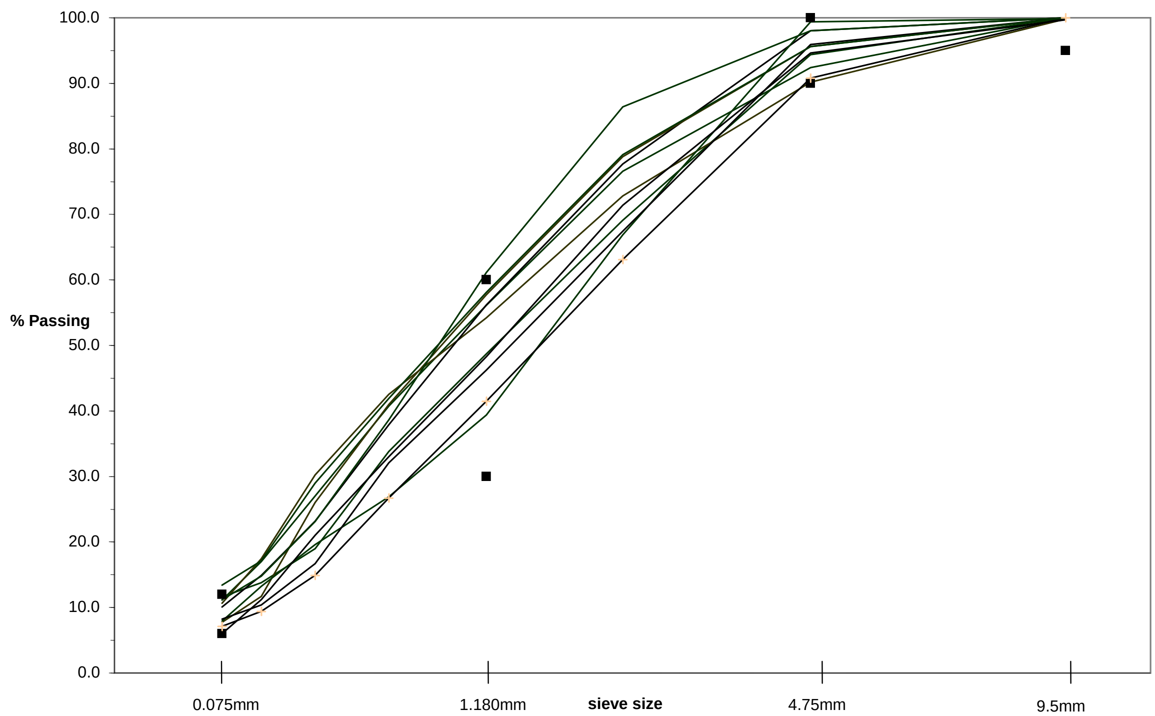

FIGURE 3.1 Gradations for State Mixtures

TABLE 3.2 4.75 mm Superpave Control Points

Sieve	Minimum	Maximum
12.5	100	

9.5	95	100
4.75	90	100
1.18	30	60
0.075	6	12

Once three aggregate blends were determined, initial asphalt content was estimated for each blend. Two replicate samples were prepared for each blend and mixed and conditioned in accordance with AASHTO R 30. Specimens were compacted in a Superpave Gyratory Compactor (Pine Instruments model AFG1A), following procedures in AASHTO T 312. This Superpave Gyratory Compactor was calibrated to provide an external angle of 1.25°. The internal angle, measured with a Pine model AFLS1 Rapid Angle Measurement kit, was 1.215 degrees. The bulk specific gravity of each compacted sample (G_{mb}) was determined by AASHTO T 166. Using AASHTO T 209, the theoretical maximum specific gravity of the asphalt mixture (G_{mm}) was determined for two samples of each blend. VMA, V_a, VFA, D:B ratio, and %G_{mm} @ N_{ini} were calculated for each trial blend. The volumetric properties of each blend were considered when selecting one of the three blends for the final mix design. In general, mixtures with the lowest estimate optimum asphalt content at the design V_a were selected, as long as VMA, VFA, and D:B ratios were reasonable.

From the trial blend series, one mixture was selected for each state, and a binder series was run for the selected blend. In this part of the mix design process, three pairs of specimens were prepared and mixed at differing asphalt contents. The three asphalt contents were at the estimated optimum, at estimated optimum minus 0.5%, and at estimated optimum plus 0.5%. The volumetric properties of the mixtures were determined as mentioned above for the trial blend series, and a better estimate of the optimum asphalt at the desired V_a was determined.

Finally, a set of two specimens was prepared with the selected aggregate blend and mixed at optimum asphalt content to verify the mix design. If the asphalt mixture compacted to the design V_a and the volumetric properties were reasonable, the mix design was accepted for the study and samples were then prepared for performance tests. The 29 laboratory prepared mix designs are described in detail in *Volume II: Mix Designs*.

3.1.3 Performance Tests

For each mix design and for baseline mixtures, a suite of performance tests were conducted. The performance tests were selected for analysis of permanent deformation, cracking resistance, permeability, and moisture sensitivity. For very thin lift applications and light-traffic pavements with low speed limits, rutting may not be a major concern. However, tests for permanent deformation were included to evaluate the stability of these mixes for other applications. Testing was conducted to evaluate volumetric criteria (e.g., VMA and VFA). Permeability tests were conducted to help evaluate possible in-place density requirements in the field. Testing was also performed on all the mixtures to evaluate their susceptibility to moisture damage.

3.1.3.1 Permanent Deformation. Permanent deformation testing was completed using a Mixture Verification Tester (MVT), shown in Figure 3.2. The MVT is a compact version of the APA. MVT testing followed AASHTO TP63-03, *Rutting Susceptibility of Asphalt Pavements Using the Asphalt Pavement Analyzer*. All specimens were tested using 100 lb wheel load and 100 psi hose pressure. For this study all specimen tests were conducted at 64°C. MVT specimens were tested at the design V_a, 4.0% or 6.0%. Unlike the APA, the MVT only has the capability of testing two Superpave gyratory specimens or one beam specimen. The benefits of the MVT are it is smaller and lighter than the APA, which makes it more convenient for QC/QA applications in smaller laboratories. The MVT was used in this project since the amounts of material limited. The number of specimens required to perform the test was reduced from six to two by using the MVT.

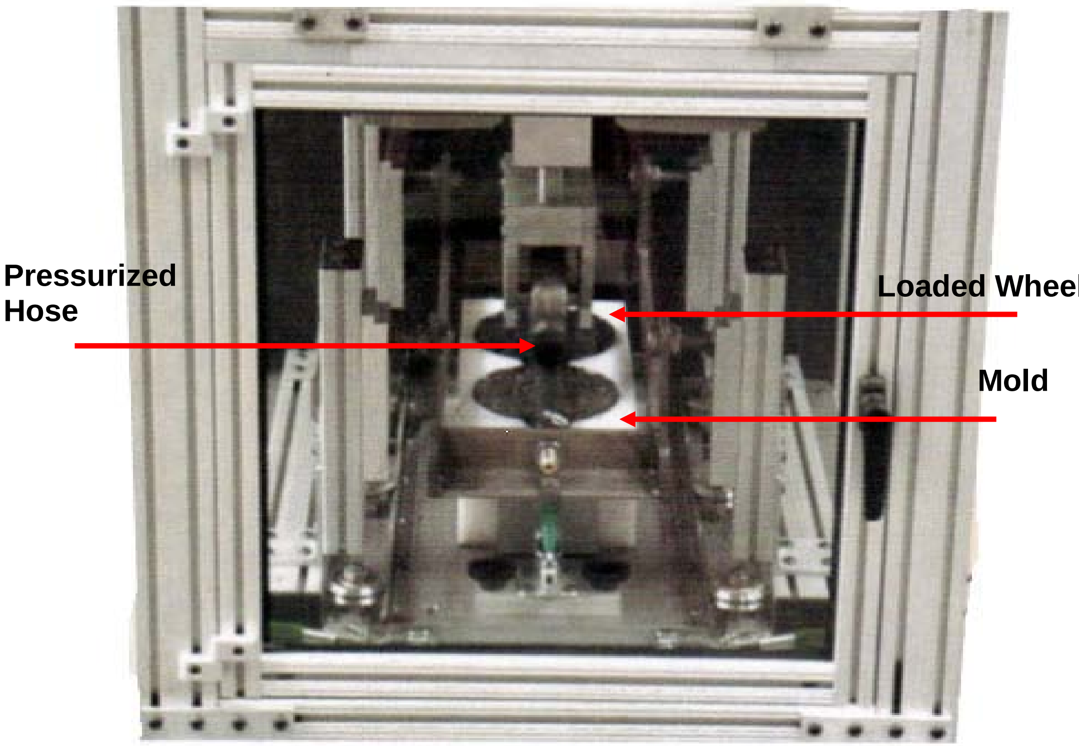

FIGURE 3.2 Mixture Verification Tester

Research comparing MVT rutting to APA rutting is scarce. However, some work by Moore (*12*) at NCAT developed a correlation between the APA and MVT. Asphalt mixtures from the NCAT test track were used to compare the two devices, and it was found that the MVT generally had rut depths greater than those generated by the APA. This relationship is shown by Figure 3.3.

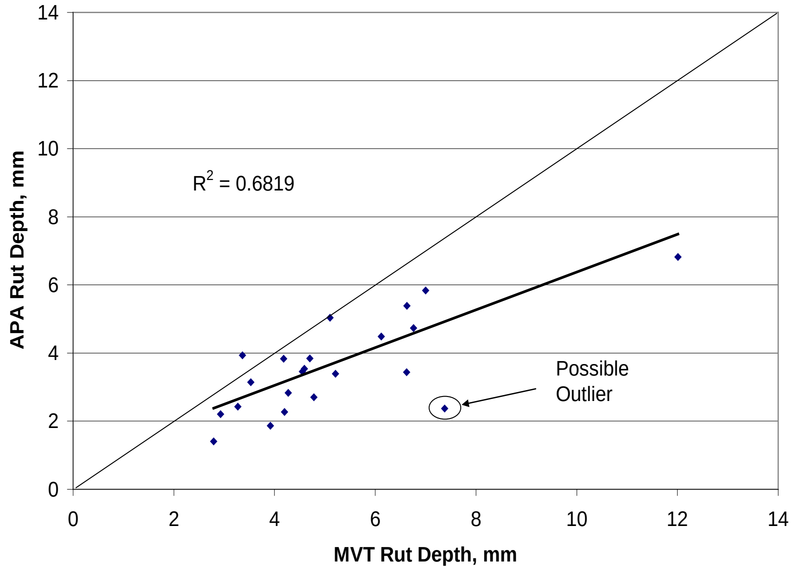

FIGURE 3.3 APA Rut Depths Versus MVT Rut Depths

3.1.3.2 Moisture Damage Potential. Although there are several tests for assessing moisture susceptibility of asphalt mixes, the most commonly used is AASHTO T 283, *Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage*. AASHTO T 283 has been shown to be reasonably reliable and is commonly specified by most DOTs. A minimum tensile strength ratio (TSR) of 0.7 or 0.8 is typically used as the criteria. For this study, moisture susceptibility testing was performed following AASHTO T-283. At the panel meeting to discuss the testing plan for this study, representatives from the participating states decided that a higher V_a should be used for some performance tests. AASHTO T-283 states that specimens should be compacted to 7+/-1% V_a. The panel decided that the in-place V_a after construction for 4.75 mm mixes would likely be in the range of to 8% to 10%. For this reason, specimens molded for moisture susceptibility in this study were targeted at 9 +/- 0.5% V_a.

3.1.3.3 Permeability. In dense-graded asphalt pavements it is important to minimize permeability. Asphalt pavements with high permeability are susceptible to moisture damage and rapid aging. The factors that affect permeability are gradation, NMAS, and relative density. In-place density after compaction may be the most important factor influencing permeability. Previous studies at NCAT have shown that the critical in-place V_a for permeability increases with smaller NMAS. As NMAS decreases, the size of the voids decrease, and thus, the interconnectivity of air voids decrease. This relationship was shown by Mallik et al. (*13*). Figure 3.4 shows permeability decreasing with smaller NMAS.

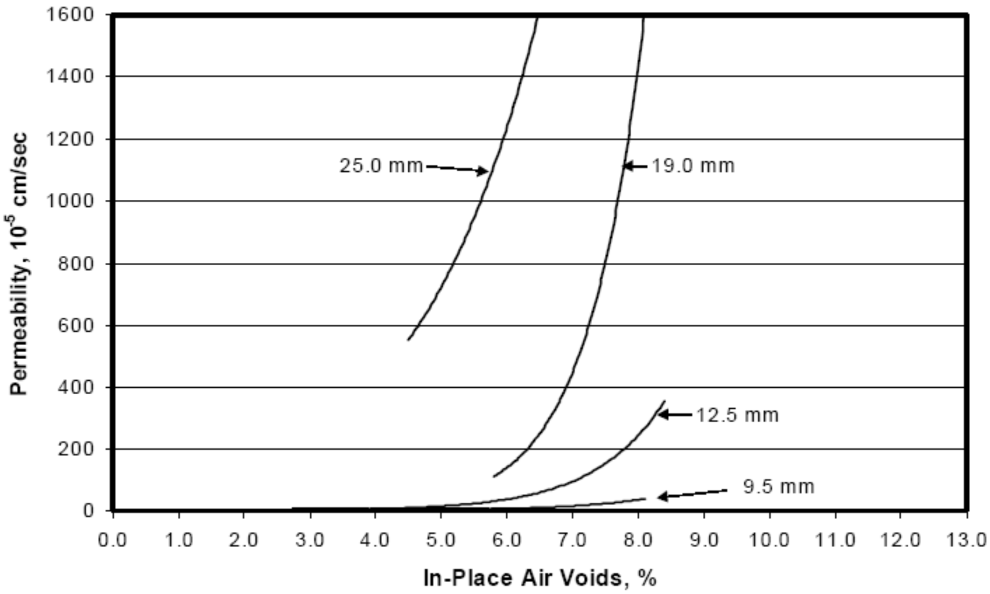

FIGURE 3.4 Best Fit Curves for In-Place Air Voids Versus Permeability for Different NMAS (*13*)

Shape of the gradation curve is also an important factor that affects permeability. In general, coarse-graded mixtures have higher permeability than similar fine-graded mixtures, probably due to greater interconnectivity of voids in coarse-graded mixtures. Fine-graded mixtures tend to have smaller voids that are not as interconnected compared to coarse-graded mixtures of the NMAS.

Permeability testing for this research was accomplished using a falling head test (ASTM PS 121). This provisional standard is no longer used by ASTM; however it is similar to Florida Method (FM5-565). The target V_a for the permeability test specimens was 9 +/- 0.5% for the same reason mentioned above. The specimens were compacted in a Pine Superpave Gyratory Compactor to a height of 55 mm and then saw-cut in half to obtain two samples about one-inch thick.

3.1.3.4 Cracking Resistance. There are several tests for fatigue cracking and thermal cracking. Fracture energy (FE) is one parameter that can be evaluated by indirect tensile (IDT) strength testing. Kim et al. (*14*) suggests that FE, which is the sum of strain energy and damage energy, may be a good indicator for the resistance of asphalt concrete to fatigue cracking. This claim is based on the observation that resistance of asphalt concrete to fatigue may be quantified by considering both resistance to deformation and resistance to damage. FE is obtained by integrating the area under the tensile stress-strain curve up to the point of fracture, shown in Figure 3.5. According to Birgisson et al. (*15*), fracture in a specimen is detected by monitoring the deformation differential and marking the location at which the deformation differential starts to deviate from a smooth curve; this is illustrated in Figure 3.6.

Kim et al. (*14*) compared several engineering IDT parameters measured on cores to observed fatigue performance data from Westrack. These parameters included 1) creep compliance at 200 sec, 2) *n*-value, 3) IDT strength, 4) horizontal center strain at peak stress, and 5) FE. Of these five parameters, FE had the best correlation with the percentage of fatigue cracking. This relationship is seen in Figure 3.7. Kim suggests that based on this research, FE at 20^{o}C is an excellent indicator of resistance of the mixture to fatigue cracking based on IDT testing of Westrack cores. Also, he proposed IDT testing at 20^{o}C as a simple performance test for fatigue cracking.

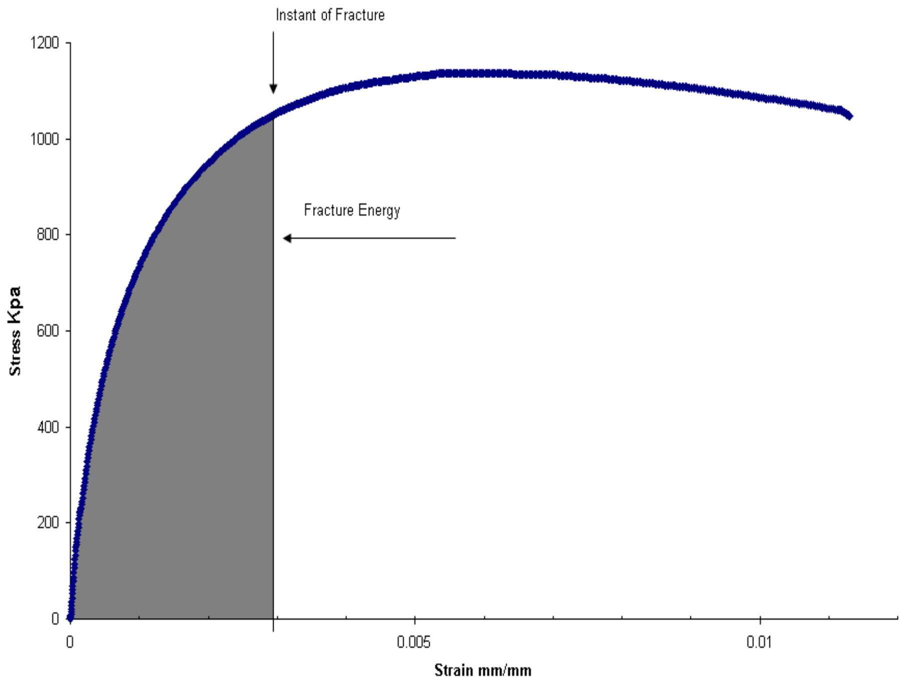

FIGURE 3.5 Area Under Stress-Strain Curve at Point of Fracture (*14*)

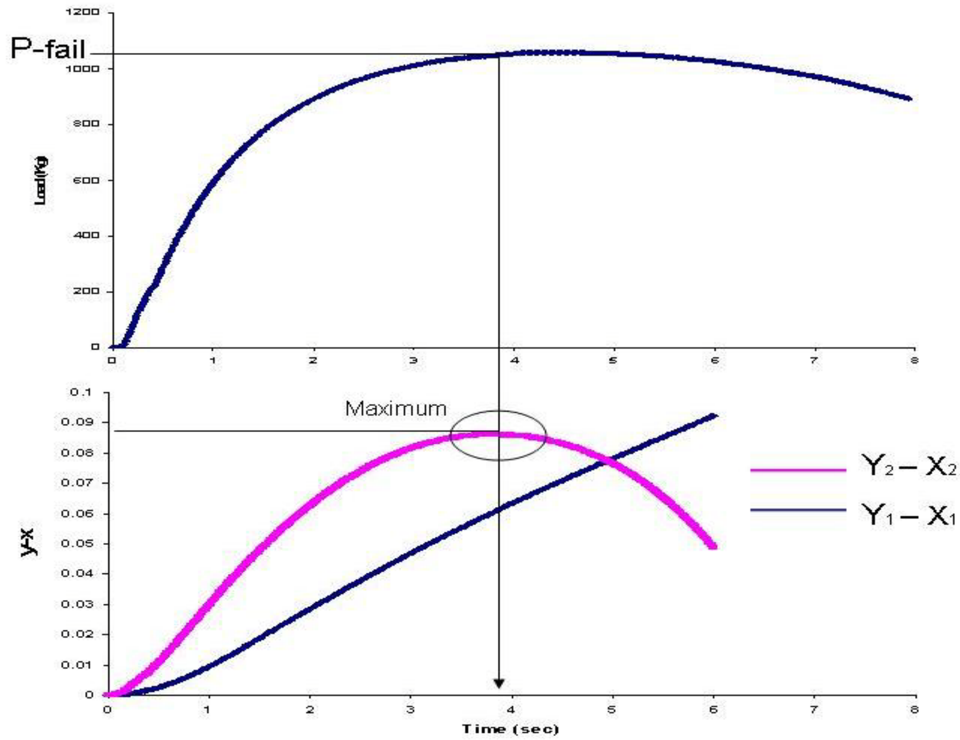

FIGURE 3.6 Determination of Point of Fracture (*15*)

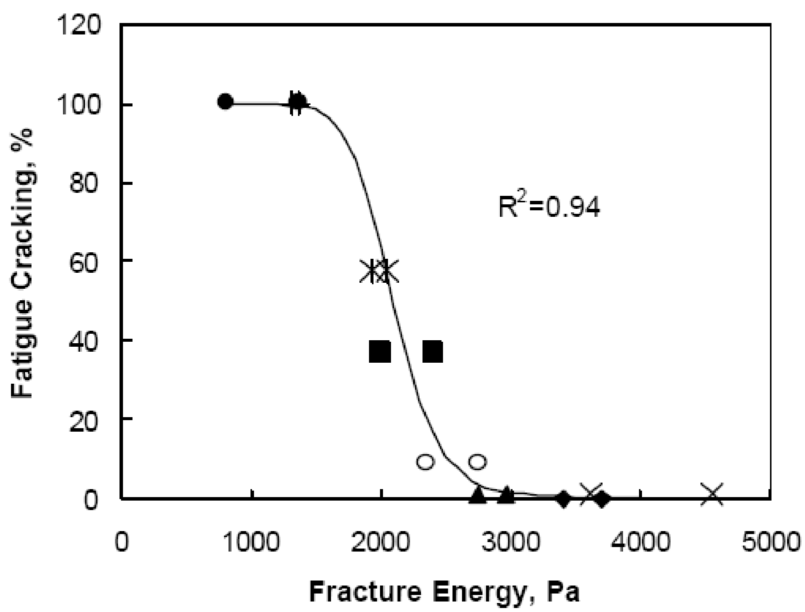

FIGURE 3.7 Relationship Between Field Fatigue Performance and Fracture Energy (14)

At this time, a method for determining FE has not been standardized. However, many aspects of the test are found in AASHTO T 322-03, *Strength of Hot Mix Asphalt (HMA) Using Indirect Tensile Test Device*. FE testing was performed on an Instron Indirect Tension Tester (Figure 3.8) at 20°C, with a ram displacement rate of 50 mm per minute. Samples were molded in a Superpave Gyratory Compactor (diameter = 150 mm) and then saw-cut on both sides to a height of 38 mm to 50 mm. Horizontal and vertical linear variable differential transducers (LVDTs) were mounted on both sides of the sample using a gauge length of 38.1 mm. Load was applied to the specimens until a peak load was reached and then began to decrease. A data acquisition system recorded load and LVDT data every 0.01 seconds. These data were then used to generate stress-strain curves. Procedures discussed in *Fracture Energy from Indirect Tension Testing* (*14*) were used in the calculation of FE density. FE density was computed as the area under the stress-strain curve to the point of fracture, illustrated in Figure 3.5. The point of fracture was determined by plotting the difference between the vertical and horizontal LVDTs on each side of the specimen. The point when the first side reached a maximum on this plot was taken as the time of fracture. This technique is illustrated in Figure 3.6. The equations and procedure for calculating FE are presented in *Volume II*.

Two sets of four samples were prepared for each mixture. Four samples remained unaged and four samples were long-term aged in a force-draft oven for six days at 85°C. . Both sets were tested for FE and then compared by calculating a ratio of unaged FE density to FE density after long-term aging.

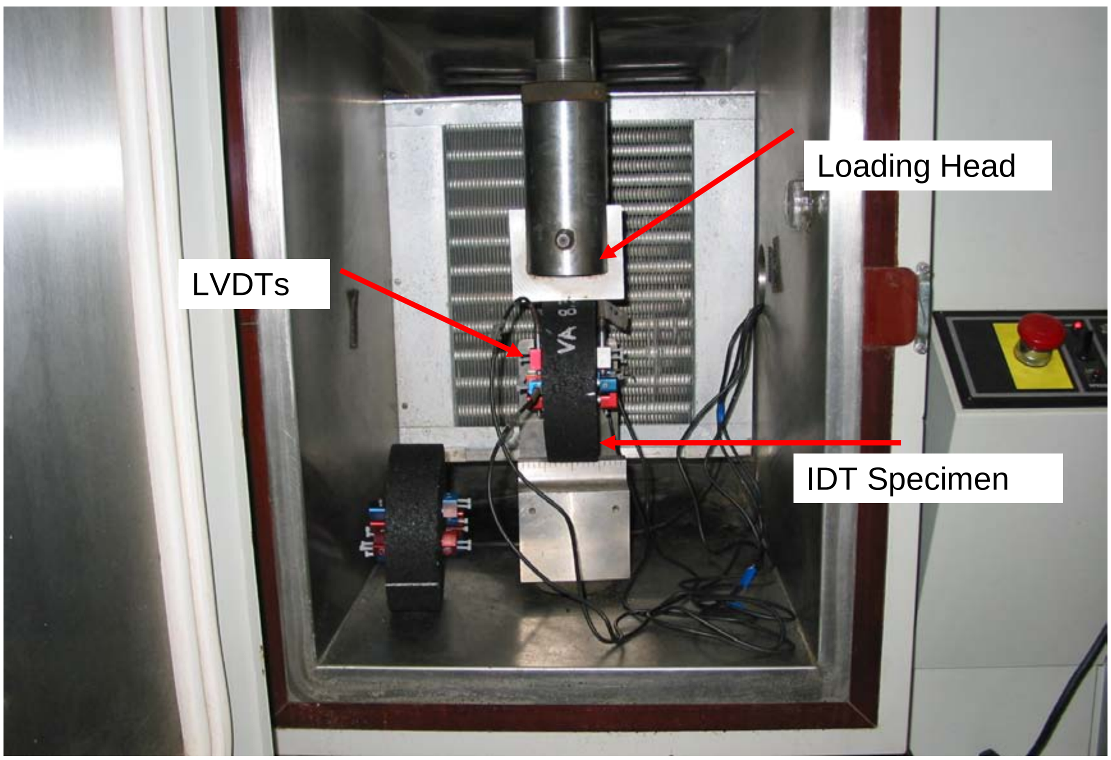

FIGURE 3.8 Instron Indirect Tension Tester

CHAPTER 4 RESULTS AND ANALYSIS

4.0 RESULTS AND ANALYSIS

Twenty-nine mix designs were performed with aggregates from nine participating states. Details of the mix design development are included in *Volume II*. Table 4.1 summarizes the volumetric and aggregate properties of these mix designs.

TABLE 4.1 Mix Design Volumetric Properties

State (mix)	V_a *(designt arget)*	N_{des}	*%A.C.*	*VMA*	*VFA*	*%*G_{mm} *@* N_{ini}	*D:B Ratio*	*film thickness (microns)*	*SE*	*FAA*
AL-50-4	4.0	50	7.4	18.5	78.4	89.0	1.8	6.1	67	46.3
AL-50-6	6.0	50	6.9	18.8	68.1	87.2	2.0	5.4	67	46.3
TN-50-4	4.0	50	7.3	16.9	76.8	87.8	2.0	6.3	69	44.8
TN-75-4	4.0	75	6.8	16.0	74.8	87.2	2.2	5.7	69	44.8
MO-50-4	4.0	50	6.9	18.2	78.2	88.8	1.7	5.9	74	49.0
MO-50-6	6.0	50	6.2	18.4	66.7	86.9	2.0	5.1	74	49.0
VA-50-4	4.0	50	8.8	16.8	75.8	89.0	1.7	6.3	76	45.0
VA-75-4	4.0	75	8.3	15.8	74.9	88.5	1.9	5.8	76	45.0
FL-50-4	4.0	50	11.8	24.2	82.8	88.9	0.8	11.8	88	44.1
FL-75-4	4.0	75	11.0	22.6	81.8	88.4	0.9	10.8	88	44.1
FL-75-6	6.0	75	10.1	22.5	73.7	86.4	1.0	9.6	88	44.1
CT-50-4	4.0	50	8.8	19.9	80.9	86.6	1.2	8.9	79	40.7
CT-50-6	6.0	50	7.2	19.0	68.5	85.1	1.4	7.1	79	40.7
MN-50-4	4.0	50	8.8	21.1	80.4	87.5	1.6	7.4	67	46.2
MN-75-4	4.0	75	8.3	20.1	79.8	86.9	1.7	6.9	67	46.2
MN-75-6	6.0	75	7.4	19.7	70.1	85.3	1.9	5.8	67	46.2
NH-50-4	4.0	50	9.7	23.8	83.6	89.8	0.7	12.8	85	51.0
NH-75-4	4.0	75	9.3	22.9	84.0	89.4	0.7	12.1	85	51.0
NH-75-6	6.0	75	8.6	23.1	75.0	87.4	0.8	10.9	85	51.0
WI-50-4	4.0	50	7.5	18.0	77.4	87.7	1.2	8.9	81	43.7
WI-50-6	6.0	50	6.7	17.8	66.9	86.7	1.4	7.7	81	43.7
TNGM-50-4	4.0	50	9.7	20.9	80.7	88.1	1.0	9.2	70	42.2
TNGM-75-4	4.0	75	9.3	17.5	76.5	87.5	1.3	8.6	70	42.2
VA adj-50-4	4.0	50	9.0	16.8	76.4	88.5	1.7	6.5	76	45.0
VA adj-75-4	4.0	75	8.7	16.5	75.6	88.0	1.7	6.1	76	45.0
FL adj-50-4	4.0	50	10.0	20.6	81.1	88.9	1.7	7.9	79	44.5
FL adj-75-6	6.0	75	9.1	20.6	71.0	86.7	1.9	6.4	79	44.5
WI adj-50-4	4.0	50	6.8	16.1	74.4	87.1	1.9	6.8	81	45.8
WI adj-50-6	6.0	50	6.3	16.5	64.4	85.3	2.1	6.3	81	45.8

Table 4.2 shows a description of materials used for each state and stockpile percentages for each blend. Table 4.3 provides gradations used for each mixture and AASHTO gradation limits. Figure 3.1 is the gradation plot for all 12 aggregate blends.

TABLE 4.2 Materials and Stockpile Percentages for Laboratory Mixtures

	Stockpile 1			Stockpile 2		
State (mix)	Name	Type	%	Name	Type	%
AL	M-10	Granite	75	89s	Granite	10
TN	#10 hard	Limestone	63	Natural	Nat. sand	20
MO	MO14	Dolomite	65	MO15	Dolomite	20
VA	#10	Granite	75	Sand	Nat. sand	25
FL	Screenings	Limestone	92	Sand	Nat. sand	8
CT	Stone Sand	Trap rock	80	Screenings	Trap rock	20
MN	Minntac	Tailings	87	Minntac fine	Tailings	13
NH	WMS	Trap rock	69	D-Dust	Trap rock	16
WI	Man-sand	Limestone	65	Screen 1/4"	Limestone	20
TNGM	# 10	Gravel	57	Sand	Nat. sand	19
FLadj	Screenings	Limestone	91	Sand	Nat. sand	3
WI adj	Man-sand	Limestone	56	Screen 1/4"	Limestone	44
	Stockpile 3			Stockpile 4		
State (mix)	Name	Type	%	Name	Type	%
AL	Shorter sand	Nat.sand	15			
TN	#10 soft	Limestone	17			
MO	MO13	Dolomite	15			
NH	RAP	---	15			
WI	Natural	Nat. sand	15			
TNGM	#10 soft	Limestone	18	Agg lime	Agg lime	6
FLadj	Fine	Bag-house	6			

TABLE 4.3 Blend Gradation for Laboratory Mixtures

	Percent Passing							
State (mix)	9.5 mm	4.75 mm	2.36 mm	1.18 mm	0.6 mm	0.3 mm	0.15 mm	0.075 mm
AL	100.0	92.4	76.6	56.1	40.7	27.0	17.0	11.1
TN	100.0	94.4	69.1	48.7	33.8	19.0	13.8	11.6
MO	99.8	90.2	72.8	54.2	42.5	30.2	17.4	10.6
VA (1)	100.0	98.0	77.7	56.2	37.9	23.2	14.9	10.1
FL	100.0	95.6	78.8	57.7	41.0	26.0	11.7	7.7
CT	99.9	99.4	66.9	39.4	26.9	19.6	13.2	7.9
MN	100.0	98.0	86.4	61.1	38.6	23.1	14.8	11.2
NH	99.7	94.6	71.4	48.3	33.0	21.0	11.2	6.0
WI	100.0	90.8	63.1	41.5	26.7	14.9	9.4	7.1
TNGM	100.0	95.9	67.4	46.2	32.1	16.7	10.4	8.2
FL adj	100.0	95.6	79.1	58.1	41.9	29.0	17.1	13.4
WI adj	100.0	89.6	58.1	37.3	24.7	16.7	12.3	9.5

Note (1) – VA adj mixture used same gradation, but different binder grade and content

4.1 Mix Design Results

4.1.1 Optimum Asphalt Content

Optimum asphalt content for the mixtures prepared in this study were relatively high compared to traditional Superpave designed mixtures. The average asphalt content for all 29 mixtures was 8.4% and the average effective asphalt content (P_{be}) was 6.6%. The average asphalt absorption was 1.8%. FL-50-4 had the highest asphalt content and P_{be} at 11.8% and 9.8%, respectively. MO-50-6 had the lowest optimum asphalt content at 6.2%. WI adj-50-6 had the lowest P_{be} at 4.6%. The New Hampshire aggregate had the lowest asphalt absorption at 0.60%, whereas the Virginia aggregate had the highest amount of asphalt absorption at 3.6%.

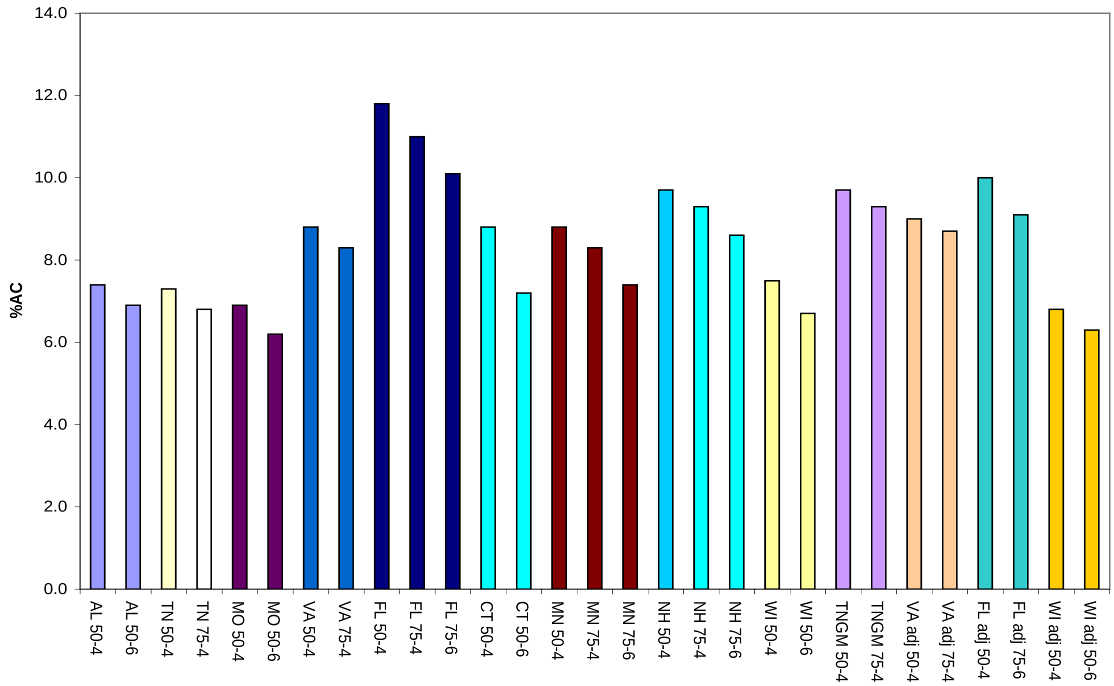

FIGURE 4.1 Optimum Asphalt Content

Figure 4.1 shows optimum asphalt content for each mix design. It can be seen that increasing N_{des} from 50 to 75 gyrations or increasing design V_a from 4% to 6% will lower optimum asphalt content. The same trend for P_{be} is illustrated in Figure 4.2. The statistical software package MINITAB was used to conduct an Analysis of Variance (ANOVA) to determine which design factors had a significant effect on P_{be}. Three design factors were appropriate to use in this analysis: N_{des}, design V_a, and source material. Results of this analysis are shown in Table 4.4.

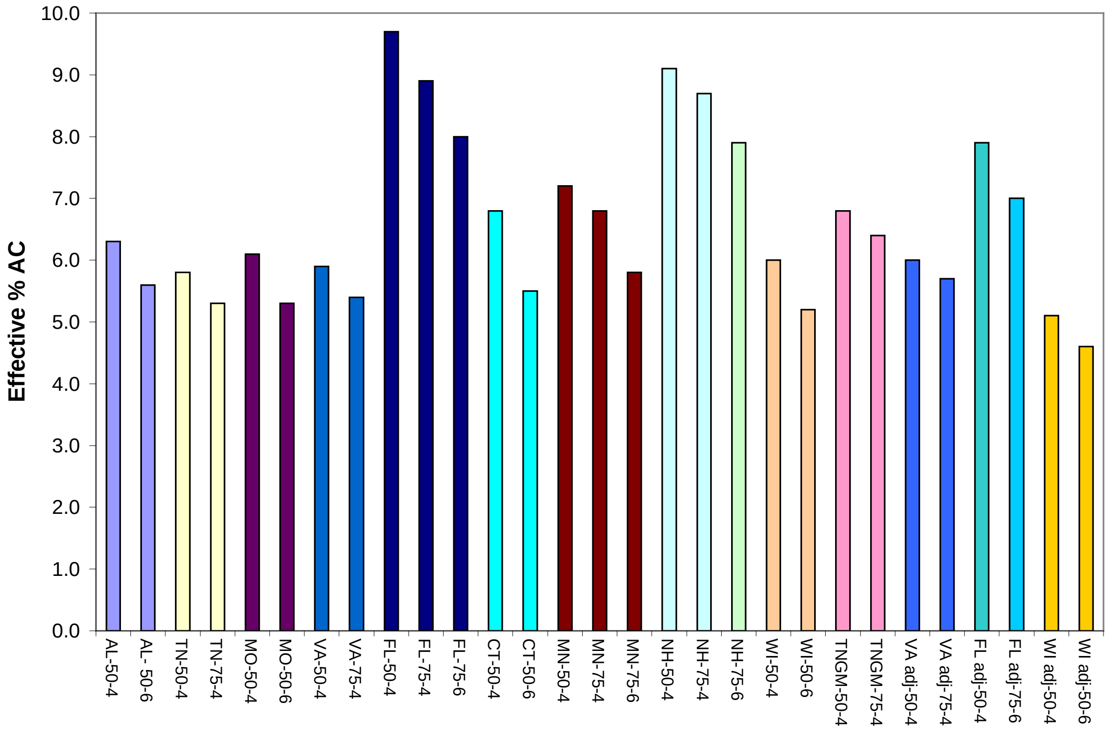

FIGURE 4.2 Effective Asphalt Content

The ANOVA results for P_{be} show that there is strong evidence to support the conclusion that N_{des}, design V_a, and material source all influence asphalt content. Based on the *F*-statistics, material type has the largest influence on P_{be}, followed by design V_a.

TABLE 4.4 Analysis of Variance for Effective Asphalt Content

Source	DF	Seq SS	Adj SS	Adj MS	F-stat	*P*
N_{des}	1	1.9892	0.8149	0.8149	29.93	0.000
V_a(design)	1	2.9622	3.1157	3.1157	114.42	0.000
Material Source	15	48.3621	48.3621	3.2240	118.40	0.000
Error	14	0.3812	0.3812	0.0272		
Total	31	53.6947				

S = 0.165017   R-Sq = 99.29%   R-Sq(adj) = 98.43%

To analyze the effect of design V_a and N_{des}, mix designs were separated into groups that had matching mix designs for each comparison. The comparisons were as follows:

- 50 gyrations (4% V_a and 6% V_a)
- 4% V_a (50 and 75 gyrations)
- 75 gyrations (4% V_a and 6% V_a)

The mix design groupings are shown in Tables 4.5 through 4.7 and are used in comparison evaluations in subsequent sections. Figures 4.3 through 4.5 show the mean asphalt contents for each grouping and the mean difference for each comparison. Figures 4.3 and 4.5 show that the difference in P_{be} is 0.9% between 4% and 6% design V_a for both compaction efforts. Figure 4.4 shows that the difference between mean asphalt content is 0.5% for 4% V_a at 50 and 75 gyrations.

TABLE 4.5 Mix Design Comparisons for N_{des}=50 (4%–6% Air Voids)

State Id	*Air voids (design)*	*Ndesign*	*%A.C.*	*Eff AC%*	*VMA*	*VFA*	*% Gmm @ Nini*	*DP*	*SE*	*FAA*	*film thickness (microns)*
AL	4.0	50	7.4	6.30	18.5	78.4	89.0	1.8	67	46.3	6.1
CT	4.0	50	8.8	6.80	19.9	80.9	86.6	1.2	79	40.7	8.9
MO	4.0	50	6.9	6.10	18.2	78.2	88.8	1.7	74	49.0	5.9
WI	4.0	50	7.5	6.00	18.0	77.4	87.7	1.2	81	43.7	8.9
WI2	4.0	50	6.8	5.1	16.1	74.4	87.1	1.9	81	45.8	6.8
		avg=	7.5	6.1	18.1	77.9	87.8	1.6			7.3
		stdev =	0.8	0.6	1.4	2.3	1.0	0.3			1.5
AL	6.0	50	6.9	5.60	18.8	68.1	87.2	2.0	67	46.3	5.4
CT	6.0	50	7.2	5.50	19.0	68.5	85.1	1.4	79	40.7	7.1
MO	6.0	50	6.2	5.30	18.4	66.7	86.9	2.0	74	49.0	5.1
WI	6.0	50	6.7	5.20	17.8	66.9	86.7	1.4	81	43.7	7.7
WI2	6.0	50	6.3	4.6	16.5	64.4	85.3	2.1	81	45.8	6.3
		avg =	6.7	5.2	18.1	66.9	86.2	1.8			6.3
		stdev =	0.4	0.4	1.0	1.6	1.0	0.3			1.1
		Diff =	0.8	0.8	0.0	10.9	1.6	-0.2			1.0

TABLE 4.6 Mix Design Comparisons for 4% Air Voids (50-75 Gyrations)

State Id	Air voids (design)	Ndesign	%A.C.	Eff AC%	VMA	VFA	% Gmm @ Nini	DP	SE	FAA	film thickness (microns)
FL	4.0	50	11.8	9.70	24.2	82.8	88.9	0.8	88	44.1	11.8
MN	4.0	50	8.8	7.20	21.1	80.4	87.5	1.6	67	46.2	7.4
NH	4.0	50	9.7	9.10	23.8	83.6	89.8	0.7	85	51.0	12.8
TN	4.0	50	7.3	5.80	16.9	76.8	87.8	2.0	69	44.8	6.3
TNGM	4.0	50	9.7	6.8	20.9	80.7	88.1	1.0	70	42.2	9.2
VA	4.0	50	8.8	5.90	16.8	75.8	89.0	1.7	76	45.0	6.3
VA2	4.0	50	9.0	6.0	16.8	76.4	88.5	1.7	76	45.0	6.5
		avg =	9.3	7.2	20.1	79.5	88.5	1.4			8.6
		stdev =	1.4	1.6	3.3	3.2	0.8	0.5			2.7
FL	4.0	75	11.0	8.90	22.6	81.8	88.4	0.9	88	44.1	10.8
MN	4.0	75	8.3	6.80	20.1	79.8	86.9	1.7	67	46.2	6.9
NH	4.0	75	9.3	8.70	22.9	84.0	89.4	0.7	85	51.0	12.1
TN	4.0	75	6.8	5.30	16.0	74.8	87.2	2.2	69	44.8	5.7
TNGM	4.0	75	9.3	6.4	17.5	76.5	87.5	1.3	70	42.2	8.6
VA	4.0	75	8.3	5.40	15.8	74.9	88.5	1.9	76	45.0	5.8
VA2	4.0	75	8.7	5.7	16.5	75.6	88.0	1.7	76	45.0	6.1
		avg =	8.8	6.7	18.8	78.2	88.0	1.5			8.0
		stdev =	1.3	1.5	3.1	3.7	0.9	0.5			2.6
		Diff =	0.49	0.47	1.3	1.3	0.5	-0.1			0.6

TABLE 4.7 Mix Design Comparisons for N_{des}=75 (4-6% Air Voids)

State Id	Air voids (design)	Ndesign	%A.C.	Eff AC%	Binder	VMA	VFA	% Gmm @ Nini	$Dust_{ratio}$	SE	FAA	film thickness (microns)
FL	4.0	75	11.0	8.90	64-22	22.6	81.8	88.4	0.9	88	44.1	10.8
MN	4.0	75	8.3	6.80	64-22	20.1	79.8	86.9	1.7	67	46.2	6.9
NH	4.0	75	9.3	8.70	64-23	22.9	84.0	89.4	0.7	85	51.0	12.1
		avg =	9.5	8.1		21.9	81.9	88.2	1.1			9.9
		stdev =	1.4	1.2		1.5	2.1	1.3	0.5			2.7
FL	6.0	75	10.1	8.00	64-22	22.5	73.7	86.4	1.0	88	44.1	9.6
MN	6.0	75	7.4	5.80	64-22	19.7	70.1	85.3	1.9	67	46.2	5.8
NH	6.0	75	8.6	7.90	64-24	23.1	75.0	87.4	0.8	85	51.0	10.9
		avg =	8.7	7.2		21.8	72.9	86.4	1.2			8.8
		stdev =	1.4	1.2		1.8	2.5	1.1	0.6			2.7
		Diff =	0.8	0.9		0.1	8.9	1.9	-0.1	0.0	0.0	1.2

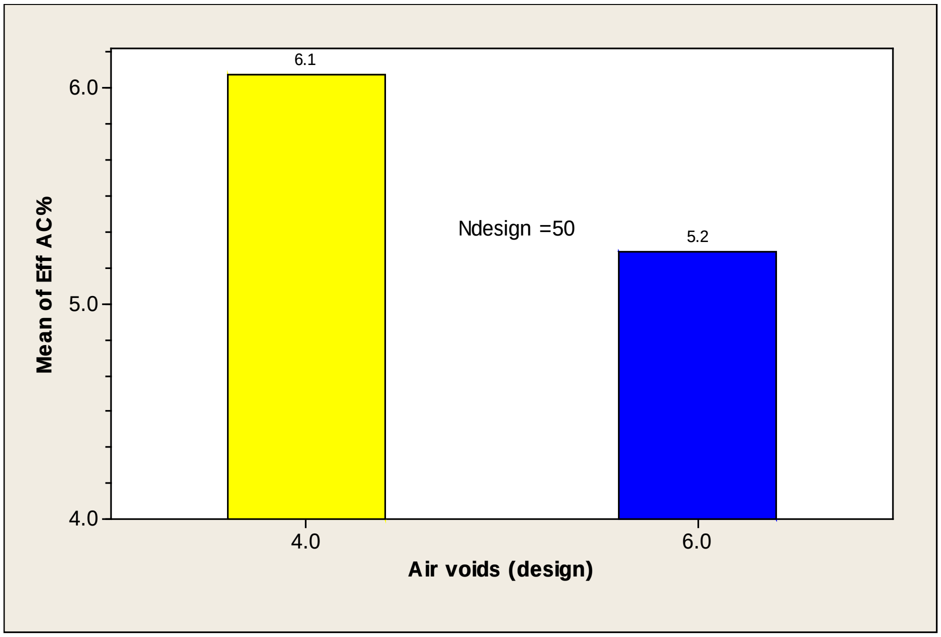

FIGURE 4.3 Mean Effective Asphalt for 4% and 6% Air Voids (N_{des}=50)

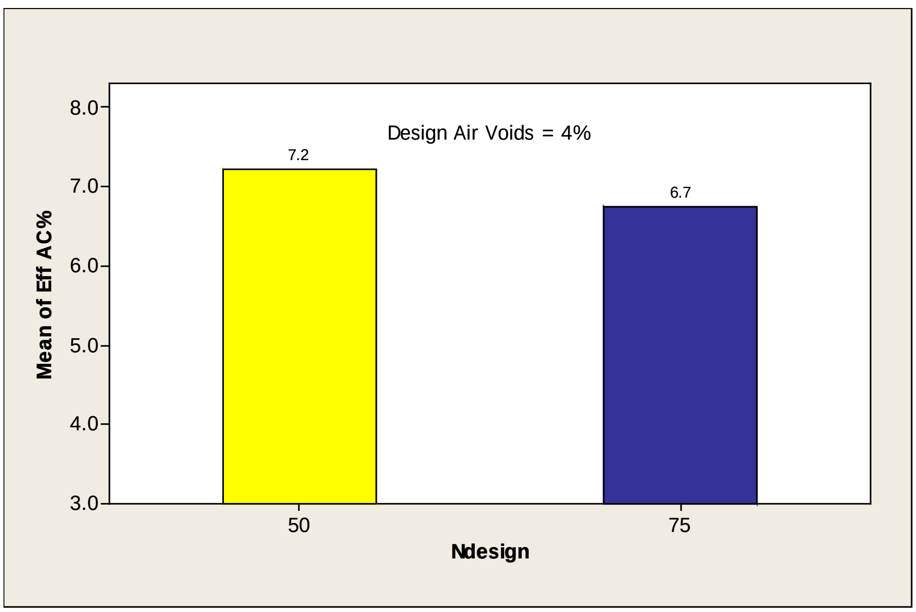

FIGURE 4.4 Mean Effective Asphalt Content for N_{des}=50 and 75 (4% Air Voids)

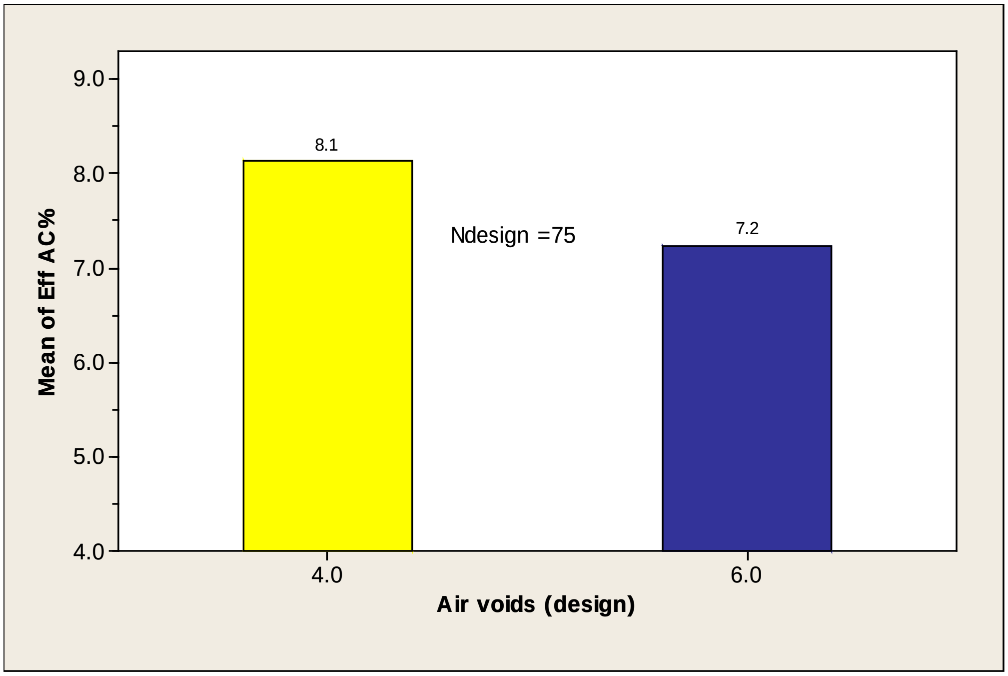

FIGURE 4.5 Mean Effective Asphalt Content for 4% and 6% Air Voids (N_{des}=75)

4.1.2 VMA

The minimum VMA currently specified in AASHTO for 4.75 mm NMAS Superpave designed mixtures is 16.0%. Only one mixture (VA-75-4) barely failed to meet the current minimum VMA criterion. The average VMA was 19.3% for mix designs prepared for this research, and the maximum value was 24.2% (FL-50-4). Figure 4.6 shows all VMA values of the mix designs performed in this research.

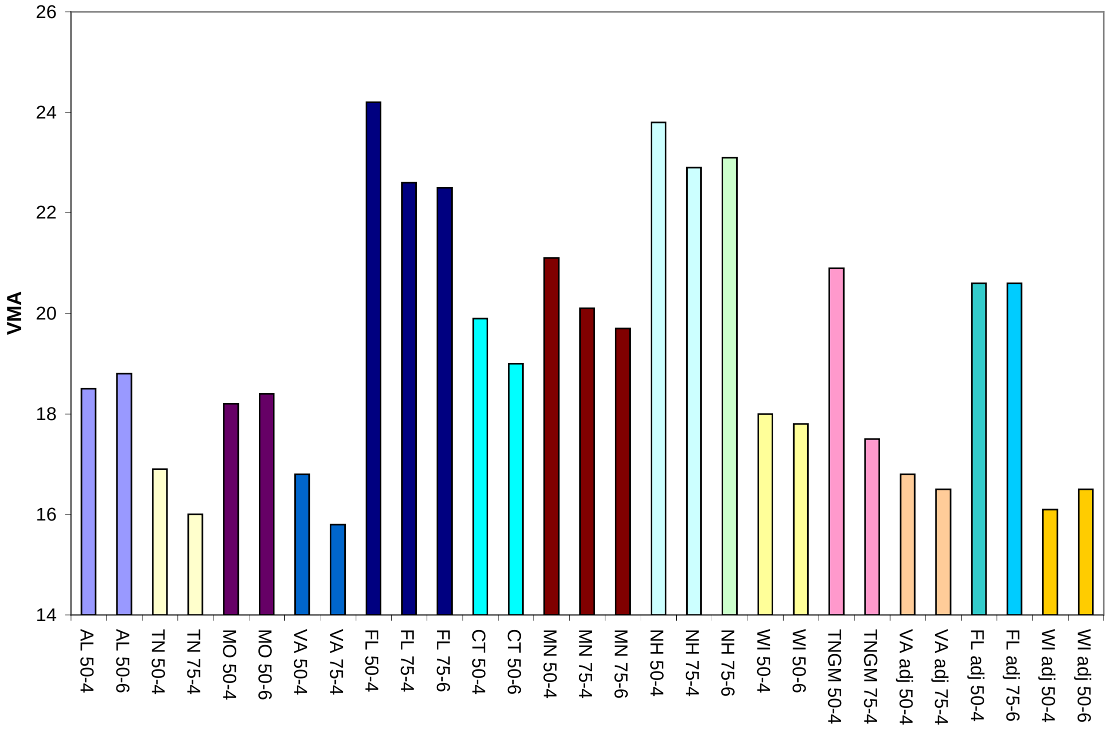

FIGURE 4.6 VMA Results for Each Mix Design

Figure 4.6 shows that the largest change in VMA occurs when the compaction level is increased from 50 to 75 gyrations. This is expected since the aggregate will orient into tighter packing when the compaction energy is increased. As is well known, when designing asphalt mixtures, the addition of asphalt binder will decrease VMA until a minimum is reached, and any additional asphalt binder past this minimum will begin to push the aggregate structure open, increasing VMA. This effect explains why at a given N_{des}, some mixtures slightly increase or decrease VMA as the optimum asphalt content decreases when the design V_a changed from 4% to 6%.

To analyze the effect of N_{des}, design V_a, and material type on VMA, MINITAB was used to perform an analysis of variance. The results of this analysis are presented in Table 4.8. As with P_{be}, the material type had the largest effect on VMA (*F*-stat = 44.4 and *p*-value = 0.000), and N_{des} also had a significant effect (Fstat = 17.69 and *p*-value = 0.001) on VMA. Design V_a, however, did not significantly influence VMA (*F*-stat = 0.05 and *p*-value =0.821).

TABLE 4.8 Analysis of Variance for VMA

Source	DF	Seq SS	Adj SS	Adj MS	F	*P*
Source Material	12	170.023	171.551	14.296	44.44	0.000
V_a (design)	1	0.430	0.017	0.017	0.05	0.821
N_{des}	1	5.673	5.673	5.673	17.64	0.001
Error	14	4.503	4.503	0.322		
Total	28	180.630				

To illustrate the results of the analysis of variance, the comparison groupings in Tables 4.5 through 4.7 were used to show the differences in VMA due to changes in design V_a and N_{des}. Figure 4.7 shows that there is no difference in mean VMA for mixtures designed with 50 gyrations at 4% and 6% V_a. For N_{des} of 75, the mean difference in VMA is only 0.1% for 4% and 6% design V_a, as shown in Figure 4.8. However, mixtures designed at 4% V_a at 50 and 75 gyrations have a significant difference in mean VMA (1.3%), as illustrated in Figure 4.9.

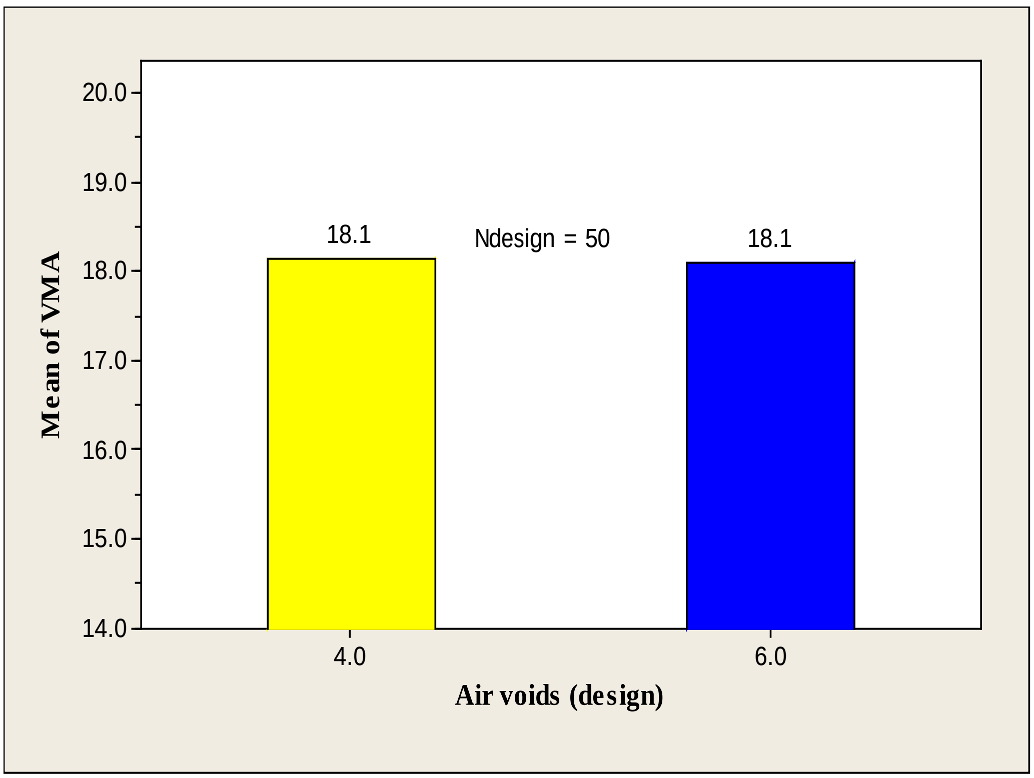

FIGURE 4.7 Mean VMA for 4% and 6% Air Voids (N_{des} = 50)

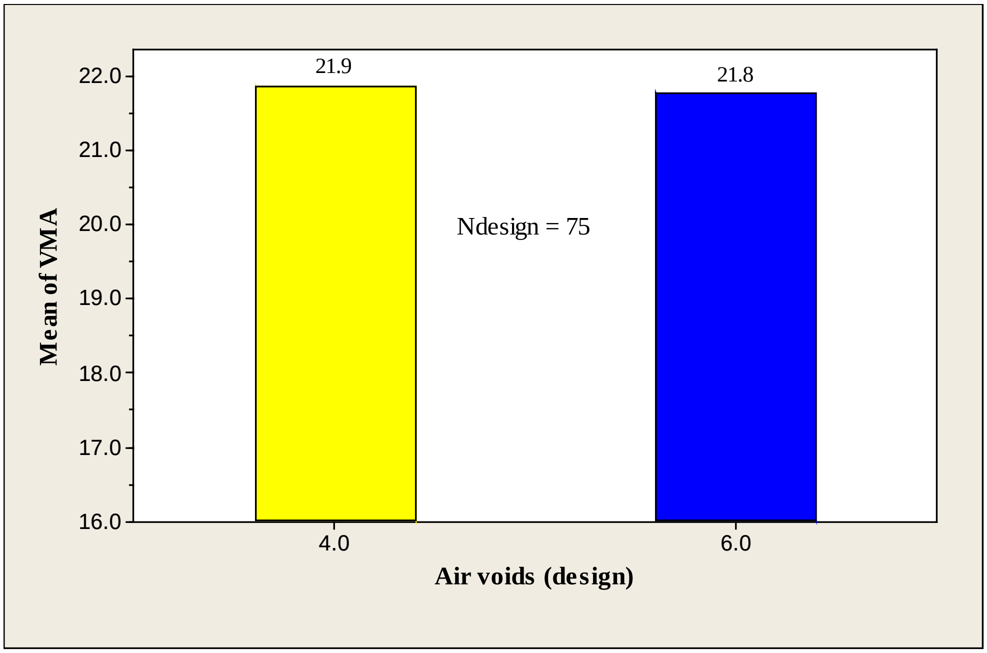

FIGURE 4.8 Mean VMA for 4% and 6% Air Voids (N_{des}=75)

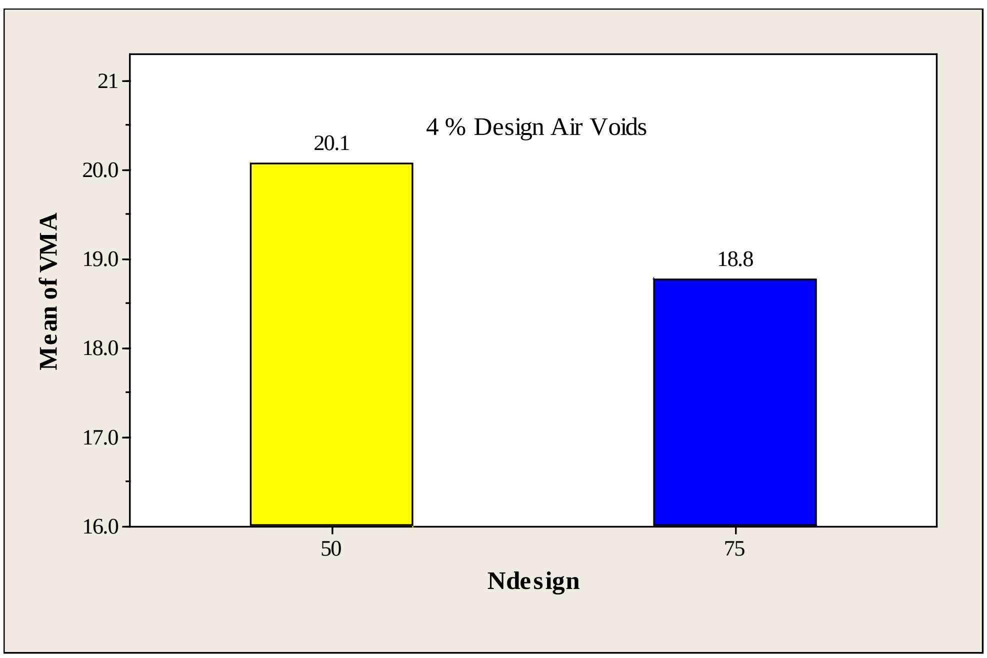

FIGURE 4.9 Mean VMA for N_{des}=50 and 75 at 4% Design Air Voids

4.1.3 VFA

Three VFA ranges are currently specified in AASHTO M 323 for 4.75 mm NMAS mixtures, shown in Table 4.9. The average VFA for all mix designs in this study was 75.8. Only seven mix designs in this study met the strictest VFA criteria, which apply to mixes used on projects with over 3 million ESALs. The maximum VFA observed was 84% for NH-75-4, and the minimum VFA was 64.4 for WI adj-50-6. Seventeen mix designs meet the VFA range for 0.3 to 3 million ESALs. Sixteen blends meet the VFA range for less than 0.3 million ESALs. Eight mixtures had VFA over 80%, and one was under 65%.

To analyze the effects of the design variables, an analysis of variance was performed using MINITAB. The results of this analysis are presented in Table 4.10. Design V_a had the largest effect on VFA. Material source was also a significant factor. N_{des} had the smallest influence on VFA (*p*-value = 0.118).

TABLE 4.9 AASHTO Specifications for 4.75 mm NMAS Superpave Mixtures

Design ESALs (Millions)	N_{des}	FAA Depth from Surface ≤ 100 mm	FAA Depth from Surface ≥ 100 mm	SE	VMA	VFA	N_{ini}
<0.3	50	-	-	40	16.0	70-80%	≤91.5
0.3 to <3.0	75	40	40	40	16.0	65-78%	≤90.5
3.0 to<10	75	45	40	45	16.0	75-78%	≤89.0
Sieve size	**Min.**	**Max.**	V_a = 4.0%				
12.5 mm	100		D:B Ratio: 0.9 to 2.0				
9.5 mm	95	100					
4.75 mm	90	100					
1.18 mm	30	60					
0.075 mm	6	12					

TABLE 4.10 Analysis of Variance for VFA

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Source Material	12	282.136	246.366	20.530	18.45	0.000
V_a(Design)	1	513.422	438.519	438.519	394.04	0.000
N_{des}	1	3.083	3.083	3.083	2.77	0.118
Error	14	15.580	15.580	1.113		
Total	28	814.221				

S = 1.05493   R-Sq = 98.09%   R-Sq(adj) = 96.17%

The comparison groups presented in Tables 4.5 through 4.7 illustrate the results of the analysis of variance. Figure 4.10 shows the difference in VFA for mixtures with an N_{des}= 50 at 4% and 6% design V_a. Both groups had an average VMA of 18.1% for both design V_a; the difference of 11.0% VFA is expected. Figure 4.11 shows a slight decrease in voids filled due to increasing N_{des} from 50 to 75 gyrations at 4% V_a. The mean difference in VMA for this comparison set was 1.3%. Figure 4.12 shows again the expected decrease in VFA by increasing design V_a from 4% to 6% at 75 gyrations.

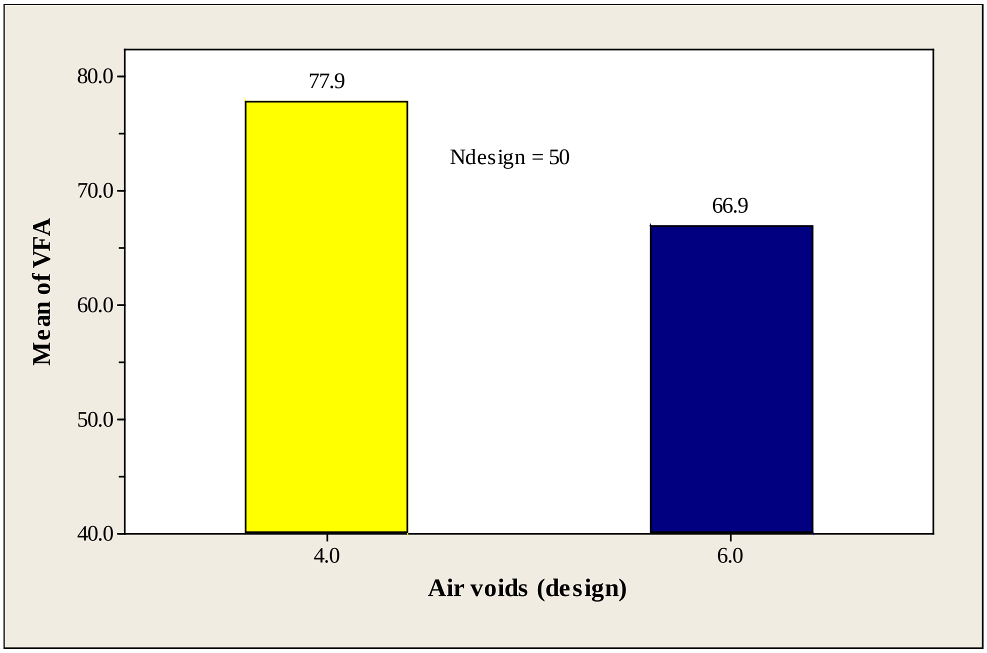

FIGURE 4.10 Mean VFA for 4% and 6% Air Voids (N_{des}=50)

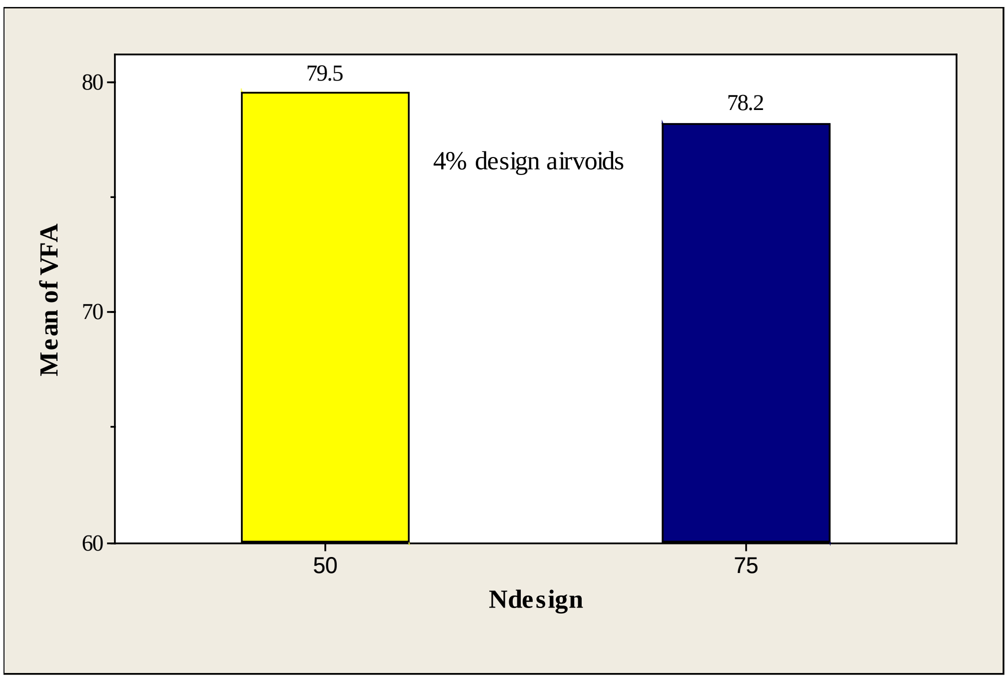

FIGURE 4.11 Mean VFA for N_{des}= 50 and 75 (4% Air Voids)

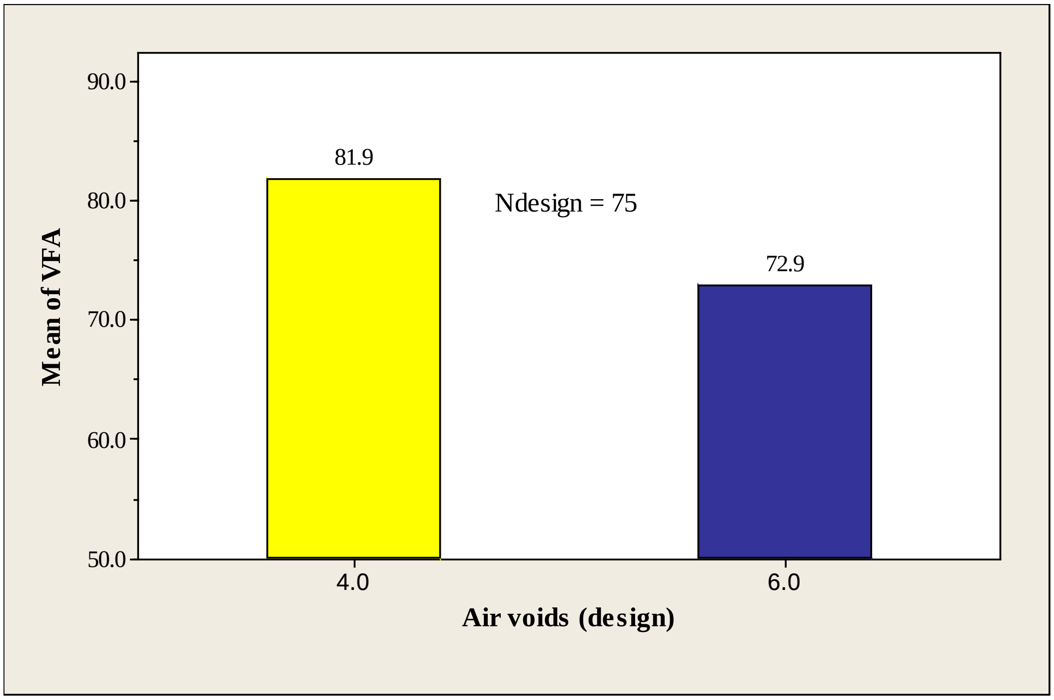

FIGURE 4.12 Mean VFA for 4% and 6% Air Voids (N_{des}=75)

4.1.4 Percent of G_{mm} at $N_{initial}$

Table 4.9 shows the current AASHTO requirements for relative density at N_{ini}. For the two gyration levels evaluated (50 and 75), the corresponding N_{ini} values are 6 and 7, respectively. Statistics for %G_{mm} @ N_{ini} are provided in Table 4.11. All mixtures prepared for this research meet the specification limits for %G_{mm} @ N_{ini} for the lowest two traffic levels. Two mixtures (NH-50-4 and NH-75-4) did not meet the most restrictive %G_{mm} @ N_{ini} requirement of ≤89% for a design traffic level greater than 3 million ESALs.

TABLE 4.11 Descriptive Statistics for %G_{mm} @ N_{ini}

N_{des}	N	Mean	Std Dev	Minimum	Median	Maximum
50	18	87.7	1.3	85.1	87.8	89.8
75	11	87.4	1.1	85.3	87.4	89.4

The analysis of variance table, Table 4.12, shows that all three design factors had a significant effect on %G_{mm} @ N_{ini}. The largest effect was due to changes in design V_a. This is probably caused by a reduction in optimum asphalt content and the percent relative density required at N_{des} when increasing design V_a from 4% to 6%. Figures 4.13 through 4.15 show the differences in %G_{mm} @ N_{ini} for the comparison groups. These comparisons show that increasing design V_a had a substantial influence on %G_{mm} @ N_{ini}. The average decrease in %G_{mm} @ N_{ini} was 1.75% for both gyration levels when increasing design V_a, whereas changing N_{des} at 4% design V_a was only 0.5%.

TABLE 4.12 Analysis of Variance for %G_{mm} @ $N_{initial}$

Source	DF	Seq SS	Adj SS	Adj MS	F	P
N_{des}	1	0.5718	1.2686	1.2686	44.12	0.000
V_a (design)	1	20.7127	12.9861	12.9861	451.66	0.000
Source Material	12	20.3516	20.3516	1.6960	58.99	0.000
Error	14	0.4025	0.4025	0.0288		
Total	28	42.0386				

S = 0.169563   R-Sq = 99.04%   R-Sq(adj) = 98.08%

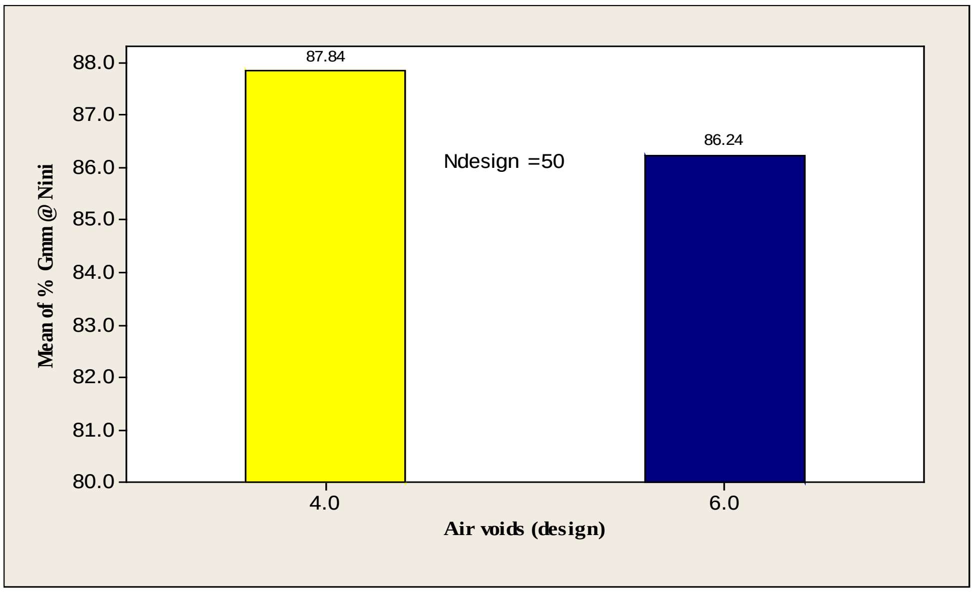

FIGURE 4.13 Mean %G_{mm} @ N_{ini} for 4% and 6% Air Voids (N_{des}=50)

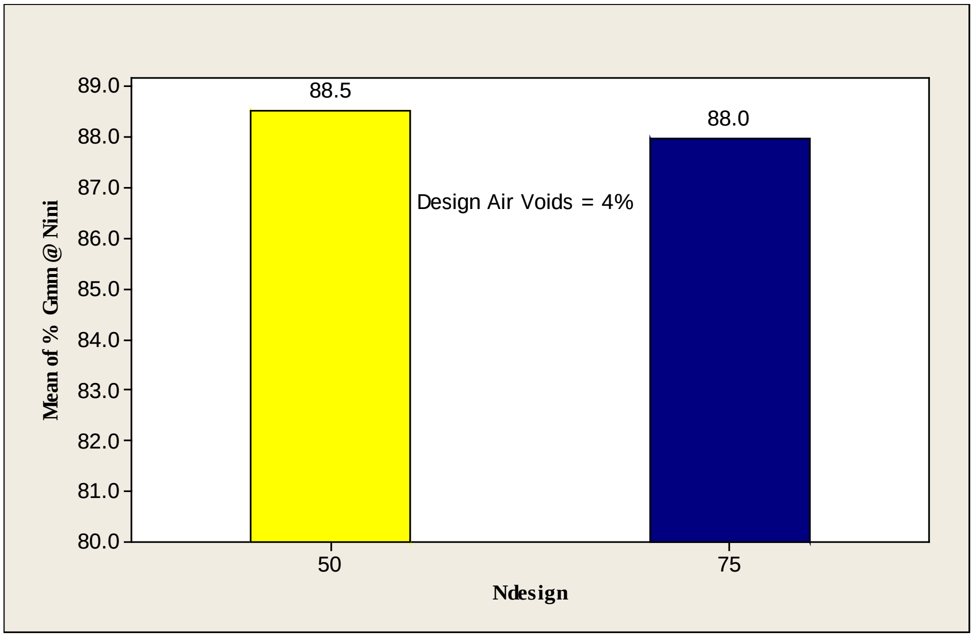

FIGURE 4.14 Mean %G_{mm} @ N_{ini} for N_{des}=50 and 75 (4% Air Voids)

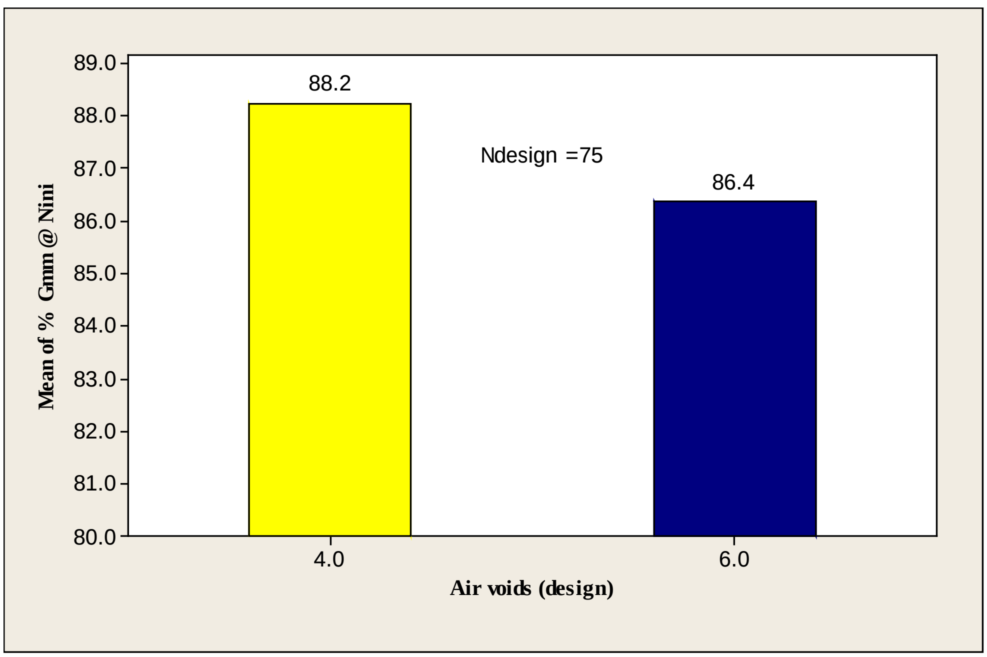

FIGURE 4.15 Mean %G_{mm} @ N_{ini} for 4% and 6% Air Voids (N_{des}=75)

4.1.5 Dust-to-Binder Ratio and Film Thickness

The D:B ratio range currently specified by AASHTO M 323 for 4.75 mm mixtures is 0.9 to 2.0. For the mix designs prepared in this study, the average was 1.5. The maximum was 2.2 for TN-75-4, and the minimum was 0.7 for NH-50-4 and NH-75-4. Two mixtures were above 2.0, and three were below 0.9. Since the D:B ratio is determined by dividing the percentage of dust by the

P_{be}, it can be controlled by the asphalt and/or dust content of the mixture. Lowering asphalt content by increasing the design V_a or N_{des} will increase D:B ratio. From the analysis of variance table, Table 4.13, it is clear that changing design V_a and gyration level have a significant influence of D:B ratio. P_{be} is largely controlled by the gradation, and the percent of dust is a part of the gradation, so it was expected that the material source would have the largest influence on D:B ratio.

TABLE 4.13 Analysis of Variance for Dust-to-Binder Ratio

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Source Material	12	44.5793	44.7818	3.7318	137.05	0.000
V_a (design)	1	4.6722	3.1157	3.1157	114.42	0.000
N_{des}	1	0.8149	0.8149	0.8149	29.93	0.000
Error	14	0.3812	0.3812	0.0272		
Total	28	50.4476				

S = 0.165017   R-Sq = 99.24%   R-Sq(adj) = 98.49%

Some researchers have proposed using film thickness (FT) requirements as an alternative to specifying minimum and maximum values for VMA and VFA. FT was calculated for each mixture in this study. FT is simply the volume of effective asphalt divided by the estimated surface area of the aggregate. Surface area factors presented by Brown et al. (*16*) were used in this research for the calculation of FT. The average FT was 7.8 microns, the maximum was 12.8 for NH-50-4, and the minimum was 5.1 for MO-50-6. As with D:B ratio, all three experimental variables have an effect on FT, with material source having the largest influence.

4.1.6 Aggregate Properties (Gradation, SE, FAA)

In the previous sections it was shown that mixture properties such as P_{be} and VMA are primarily controlled by the source materials. Aggregate gradation is the most important factor in establishing the amount of voids created in the aggregate structure. Figure 4.16 shows that as VMA increases, the asphalt needed to fill voids increases. Since VMA and P_{be} are both dependent on gradation, it was necessary to understand how gradation parameters influenced VMA of asphalt mixtures prepared for this study.

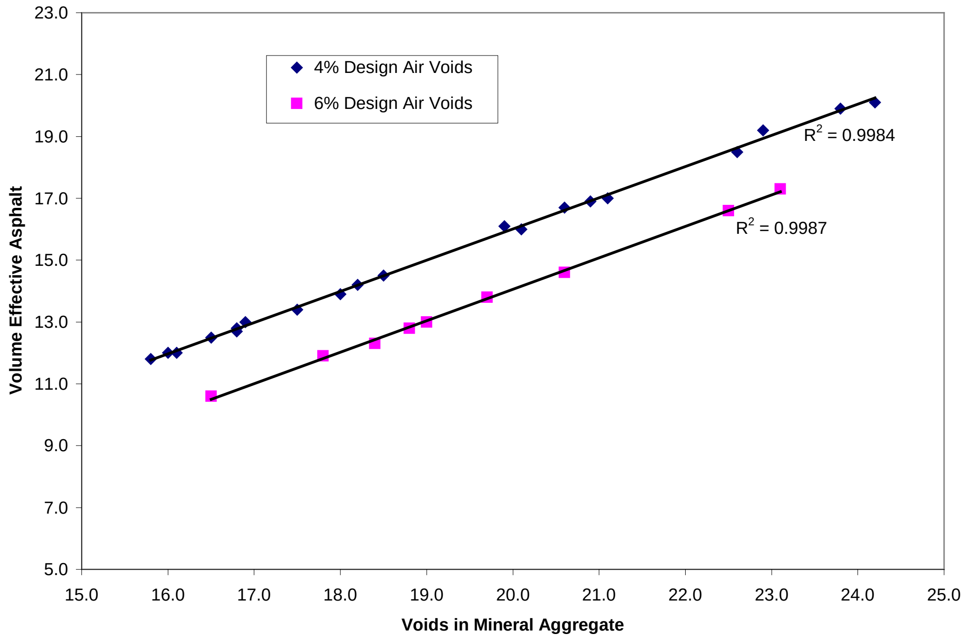

FIGURE 4.16 VMA versus Percent Volume Effective Asphalt

Fineness Modulus (FM) was calculated for each blend to examine the influence of gradation on VMA. The FM expresses how fine or coarse an aggregate blend is; the larger the FM, the coarser the gradation. To examine the effect of gradation on VMA, only the 13 mixtures designed at 50 gyrations and 4% V_a were used to remove effects of compactive effort and different design V_a.

Figure 4.17 shows two plots of FM versus VMA: one for mixtures with over 10% dust and one for mixtures with less than 10% dust. Since all the mixtures presented in this study were fine-graded, it was expected that coarser blends (i.e., higher FM) would have lower VMA, as seen in Figure 4.17 for both curves. Also, separating the mixtures into two groups (over and under 10% dust) showed that increasing dust content will lower VMA even for finer mixtures. Figure 4.18 illustrates that as the percent passing the 1.18 mm sieve increases, VMA increases. Again, the data are divided into two groups (over and under 10% dust), showing that VMA can be controlled with higher dust contents and/or adjusting the coarseness of the aggregate blend. However, using higher dust contents to control VMA can cause problems with other mix parameters such as higher D:B ratios and lower FT.

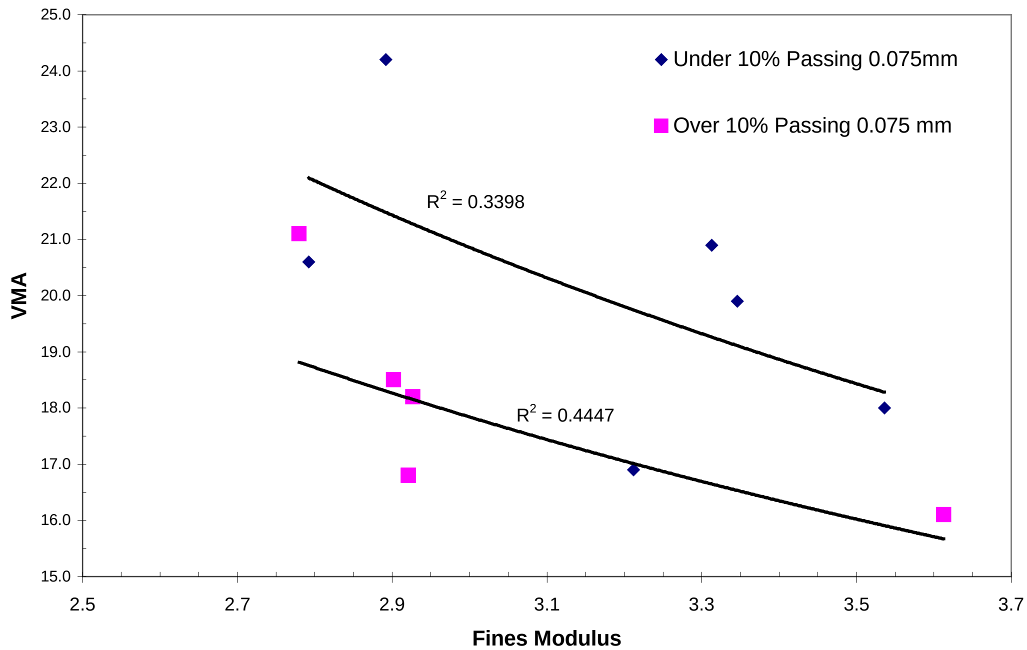

FIGURE 4.17 Fineness Modulus Versus VMA for Over and Under 10% Passing the 0.075 mm Sieve

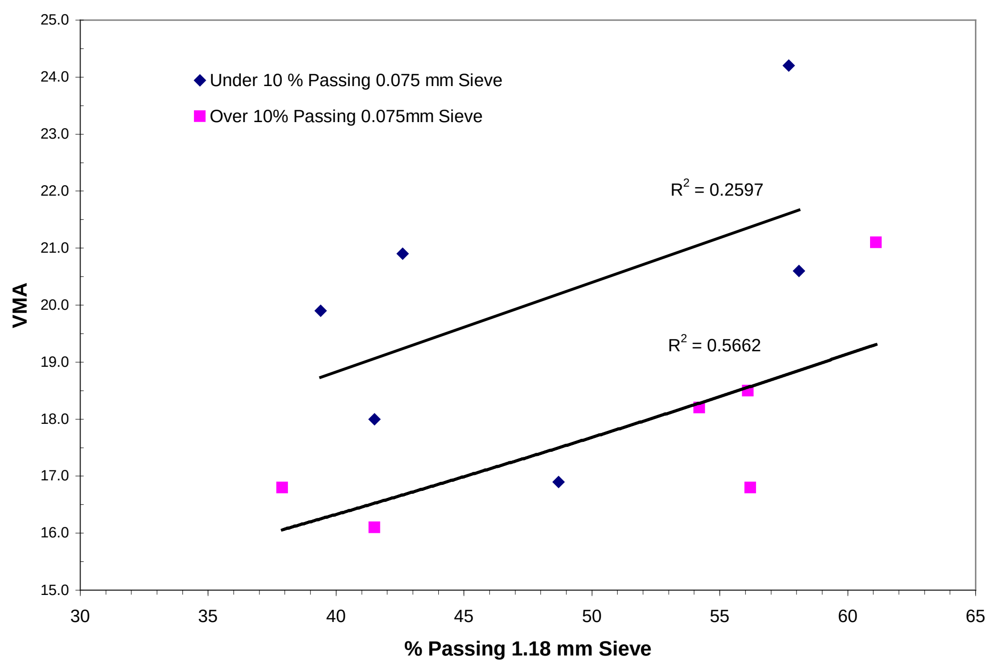

FIGURE 4.18 VMA Versus Percent Passing 1.18 mm Sieve for Over and Under 10% Passing the 0.075 mm Sieve

The gradations for the mixtures are presented in Table 4.3 and plotted in Figure 3.1. Most mixes designed in this study have gradations considered to be fine-graded. The average percent passing the control sieves (4.75 mm, 1.18 mm, and 0.075 mm) was 94.9, 50.4, and 9.5, respectively. One mixture was below the 90% minimum percent passing the 4.75 mm sieve (WI-adj). This was the coarsest gradation, and it had one of the lowest VMAs in this research. One aggregate blend was over the 60% maximum percent passing the 1.18 mm sieve (MN). Even with a fairly high dust content of 11.2%, this blend had VMAs well above the 16% minimum. The blend adjustment from Florida (FL-adj) was the only aggregate blend outside the 6% to 12% range for percent passing the 0.075 mm sieve (P-075). In an attempt to reduce excessive VMA, 6% baghouse fines were added to the first Florida mix (FL) to create the FL-adj aggregate blend. By increasing the dust content to 13.4%, the VMA was reduced but was still relatively high at 20.6% for both FL-adj blends. This is probably due to the fine grading of the blend, (58.1% passing the 1.18 mm sieve and a FM of 2.792).

For mix designs below 0.3 million ESALs, there is currently no requirement for FAA. Between 0.3 to 3 million ESALs, the minimum FAA is 40. Mixes designed for greater than 3 million ESALs have a minimum FAA of 45 for mixtures used within 100 mm of the pavement surface, and minimum FAA of 40 for mixes used deeper than 100 mm from the pavement surface. The average FAA value was 45.2 for all the aggregate blends used as mix designs. The highest FAA was 51 for the New Hampshire blend, and the lowest was 40.7 for the Connecticut blend. Every blend met the 40 minimum FAA. Seven of the 13 blends met the 45 minimum FAA.

FAA did not significantly influence volumetric properties such as VMA, VFA, or P_{be}. It has been thought that high FAA values probably increase VMA. For 4.75 mm NMAS mixtures, it seems that since 100% of the blend is fine aggregate, there would be a clear relationship between FAA and VMA, but this was not the case. Figure 4.19 shows no relationship between FAA and VMA. It also seems logical to assume that as FAA increases, the relative density at N_{ini} would decrease because the more angular particles would create greater internal friction. However, the opposite trend was observed. Figure 4.20 shows that for the blends in this study, as FAA increased, so did the relative density at N_{ini}. Since all the blends had FAA values above 40 and the average was 45.2, it is not possible to determine how blends with FAA below 40 would affect mixture properties and performance.

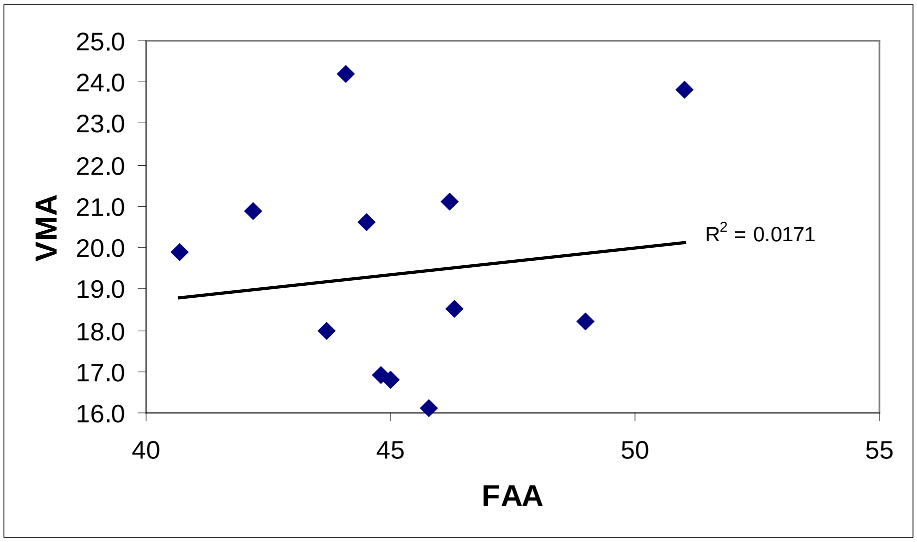

FIGURE 4.19 FAA Versus VMA for N_{des}= 50 and Design Air Voids = 4%

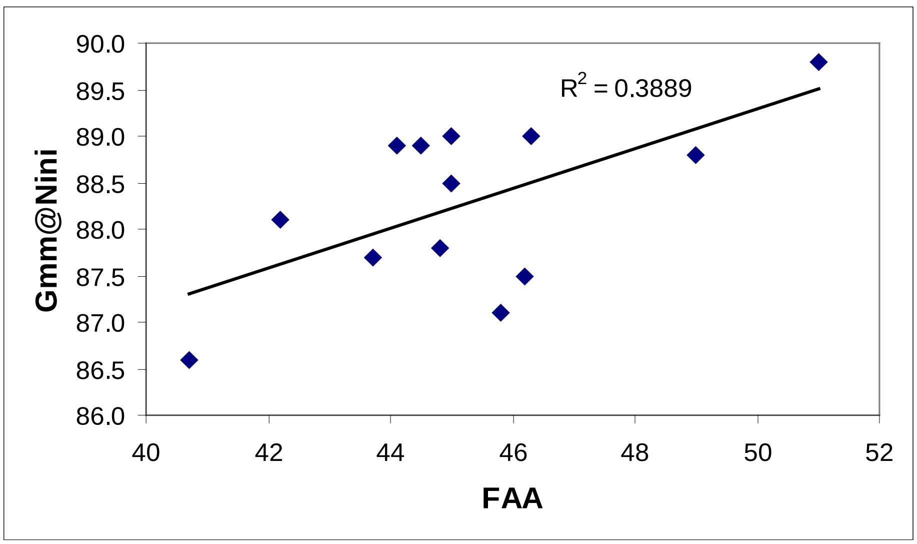

FIGURE 4.20 FAA Versus %G_{mm} @ N_{ini} for N_{des}=50 and Design Air Voids = 4%

For asphalt mixtures designed for over 3 million ESALs, the minimum SE value is 45; for less than 3 million, the minimum is 40. All blends are well above these minimum values. The average was 76, the minimum was 67 for Alabama and Minnesota blends, and the maximum was 88 for the Florida blend. Since the amount of clay size particles relates to the amount of dust in the blend, SE is related to the amount of dust, D:B ratio, and FT. These relationships are shown in Table 4.14, where Pearson correlation coefficients and *p*-values are presented for each relationship. Since all the SE values for blends presented in this study are well above the minimum specified values, its effect on performance is not clear based on these results.

TABLE 4.14 Pearson Coefficients for Sand Equivalence

	P-0.075	Dust-to-Binder Ratio	Film Thickness
R	-0.577	-0.570	0.679
p-value	0.039	0.042	0.011

4.2 Performance Tests

4.2.1 MVT Rut Depth

The MVT was used to test permanent deformation on all 29 mixtures. The specimens used for this performance test were prepared at the design V_a and compacted to N_{des}. Since rutting on many mixtures was so severe, it was difficult to determine the effect of changes in air void, compaction level, and percent binder. All rut depths presented in this report were measured manually. Since the MVT device is programmed to shut off if the automatic rut depth measurements exceed 15 mm, many tests were automatically terminated before 8,000 cycles.

TABLE 4.15 Rut Depth and Mixture Properties for All Mix Designs

State (mix)	V_a	N_{des}	*% Nat Sand*	P_{be}	*VMA*	*VFA*	*D:B*	*SE*	*FAA*	*FT (microns)*	*Rut Depth (mm)*	*Cycles*
AL	4.0	50	15	6.3	18.5	78.4	1.8	67	46.3	6.1	15.4	8000
AL	6.0	50	15	5.6	18.8	68.1	2.0	67	46.3	5.4	16.5	8000
TN	4.0	50	20	5.8	16.9	76.8	2.0	69	44.8	6.3	17.7	8000
TN	4.0	75	20	5.3	16.0	74.8	2.2	69	44.8	5.7	13.5	8000
MO	4.0	50	0	6.1	18.2	78.2	1.7	74	49.0	5.9	12.1	8000
MO	6.0	50	0	5.3	18.4	66.7	2.0	74	49.0	5.1	11.3	8000
VA	4.0	50	25	5.9	16.8	75.8	1.7	76	45.0	6.3	19.6	6228
VA	4.0	75	25	5.4	15.8	74.9	1.9	76	45.0	5.8	13.7	8000
FL	4.0	50	8	9.7	24.2	82.8	0.8	88	44.1	11.8	19.5	1205
FL	4.0	75	8	8.9	22.6	81.8	0.9	88	44.1	10.8	15.4	2047
FL	6.0	75	8	8.0	22.5	73.7	1.0	88	44.1	9.6	14.6	2425
CT	4.0	50	0	6.8	19.9	80.9	1.2	79	40.7	8.9	17.2	8000
CT	6.0	50	0	5.5	19.0	68.5	1.4	79	40.7	7.1	12.7	8000
MN	4.0	50	0	7.2	21.1	80.4	1.6	67	46.2	7.4	19.1	5724
MN	4.0	75	0	6.8	20.1	79.8	1.7	67	46.2	6.9	15.8	5256
MN	6.0	75	0	5.8	19.7	70.1	1.9	67	46.2	5.8	13.9	5074
NH	4.0	50	0	9.1	23.8	83.6	0.7	85	51.0	12.8	14.5	3595
NH	4.0	75	0	8.7	22.9	84.0	0.7	85	51.0	12.1	17.2	4220
NH	6.0	75	0	7.9	23.1	75.0	0.8	85	51.0	10.9	13.1	8000
WI	4.0	50	15	6.0	18.0	77.4	1.2	81	43.7	8.9	13.1	8000
WI	6.0	50	15	5.2	17.8	66.9	1.4	81	43.7	7.7	14.0	8000
TNGM	4.0	50	19	6.8	20.9	80.7	1.0	70	42.2	9.2	21.3	2795
TNGM	4.0	75	19	6.4	17.5	76.5	1.3	70	42.2	8.6	22.7	8000
VA-adj	4.0	50	0	6.0	16.8	76.4	1.7	76	45.0	6.5	9.8	8000
VA-adj	4.0	75	0	5.7	16.5	75.6	1.7	76	45.0	6.1	11.1	8000
FL-adj	4.0	50	3	7.9	20.6	81.1	1.7	79	44.5	7.9	14.3	8000
FL-adj	6.0	75	3	7.0	20.6	71.0	1.9	79	44.5	6.4	11.8	8000
WI-adj	4.0	50	0	5.1	16.1	74.4	1.9	81	45.8	6.8	5.3	8000
WI-adj	6.0	50	0	4.6	16.5	64.4	2.1	81	45.8	6.3	7.5	8000

Table 4.15 shows the rut depths for all 29 blends and the number of cycles the test performed before termination. The average rut depth was 13.3 mm for samples that completed 8,000 cycles. An interesting comparison is the average VMA for mixtures that completed 8,000 cycles to mixtures that did not complete 8000 cycles (see Figure 4.21). The average VMA for mixtures that completed 8,000 cycles was 18.2 mm and 21.5 mm for those mixtures terminated before 8,000. When VMA versus cycles to termination is plotted in Figure 4.22 for all mixtures, it is seen that for over 20% VMA, mixture rutting generally was so severe that the MVT device prematurely ended the test. There are some exceptions, one being VA-50-4, which had a relatively low VMA yet did not complete 8,000 cycles. This may be partly due to high percentage of natural sand (25%). The other exception is NH-75-6. This mixture had a high VMA (23.1%) yet completed 8,000 cycles and had a reasonable rut depth. This is probably explained by the mixture's high FAA value of 51. NH-75-6 also had the lowest asphalt content for the three mixes prepared with the New Hampshire blend.

Based on the number of mixtures that did not complete a full 8,000 cycles on the MVT device, it is evident that limiting the VMA in 4.75 mm mixtures will be important in designing rut-resistant mixtures. To analyze all the MVT data, including those mixtures that did not finish 8,000 cycles, rut depths were divided by the number of cycles completed for each mix to determine the total rutting rate in mm/cycle. When rutting rate is plotted against VMA, as shown in Figure 4.23, there are two separate trends for 4% and 6% V_a. The 6% design air void line plots beneath the 4% air void line, and the lines diverge for higher VMA values. This indicates that even at higher VMA, the 6% air void mixtures were more rut resistant because of lower asphalt contents. This observation led to the consideration of evaluating the volume of effective asphalt (V_{be}) as a parameter to control these mixtures. V_{be} is simply VMA minus the V_a and more directly quantifies the amount of binder needed for durability of a mix.

Figure 4.24 is a plot of the V_{be} versus rutting rate, with the data sorted by the design V_a. Figure 4.24 shows that the 6% and 4% air void curves are closer together than in Figure 4.23, which indicates that rutting for these laboratory mixtures is a function of the amount of asphalt, not just the total VMA.

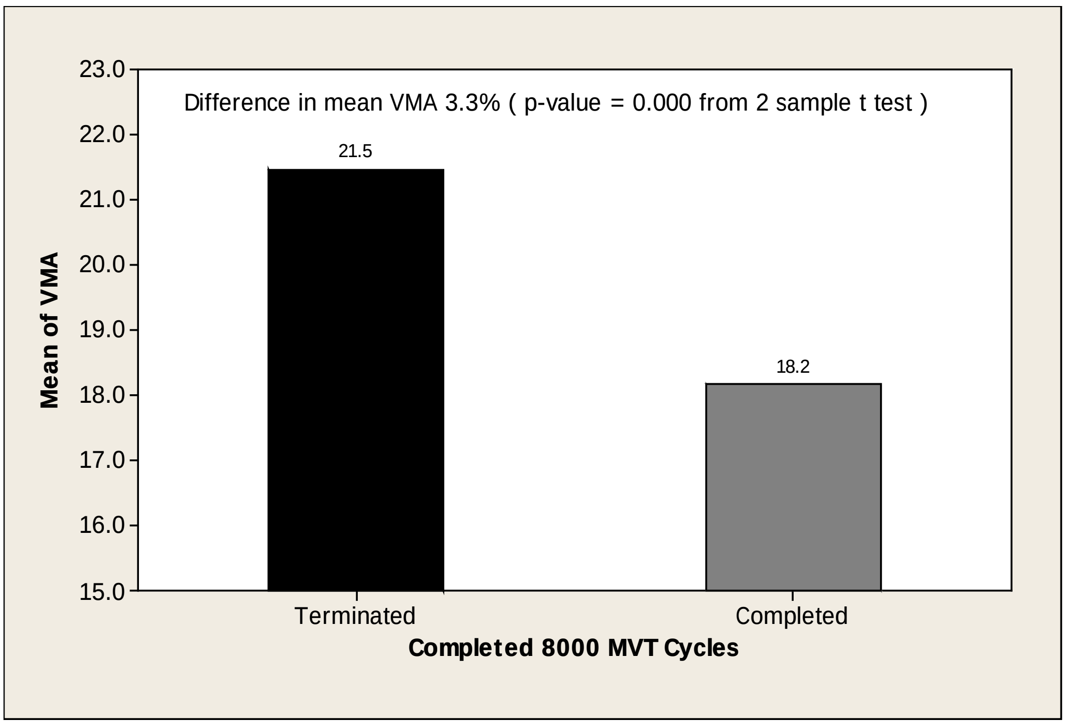

FIGURE 4.21 Mean Difference in VMA for Mixtures that Terminated Early and Completed 8,000 Cycles on MVT

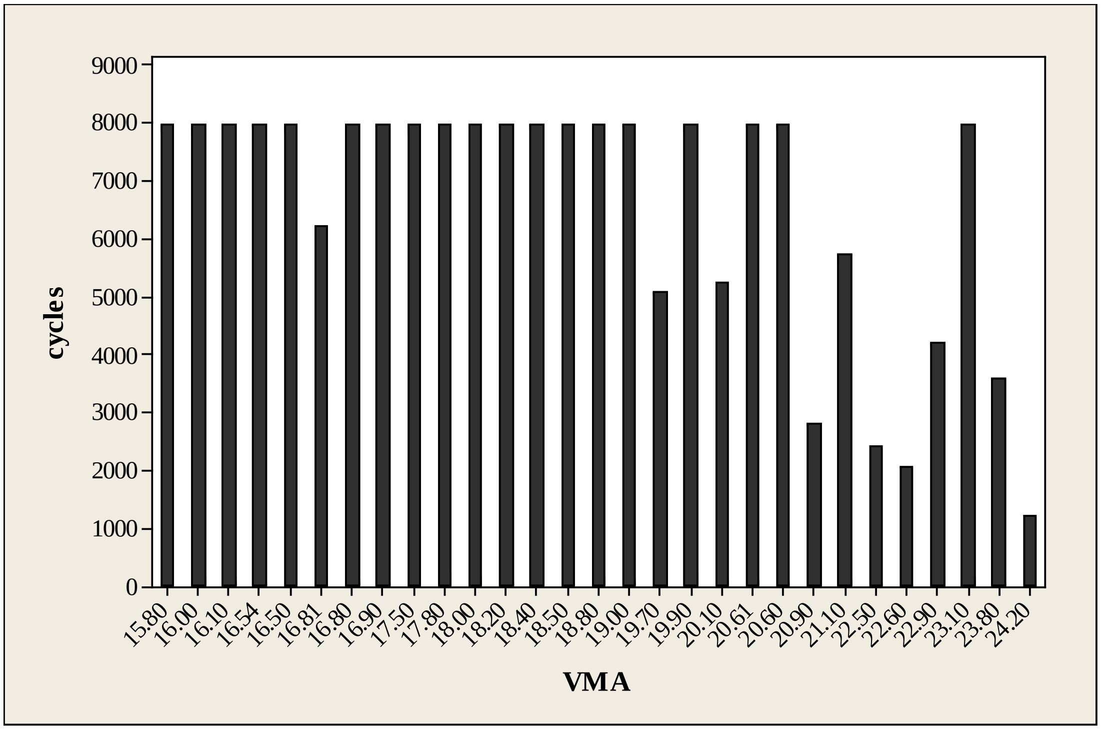

Figure 4.22 VMA Versus Cycles to Termination

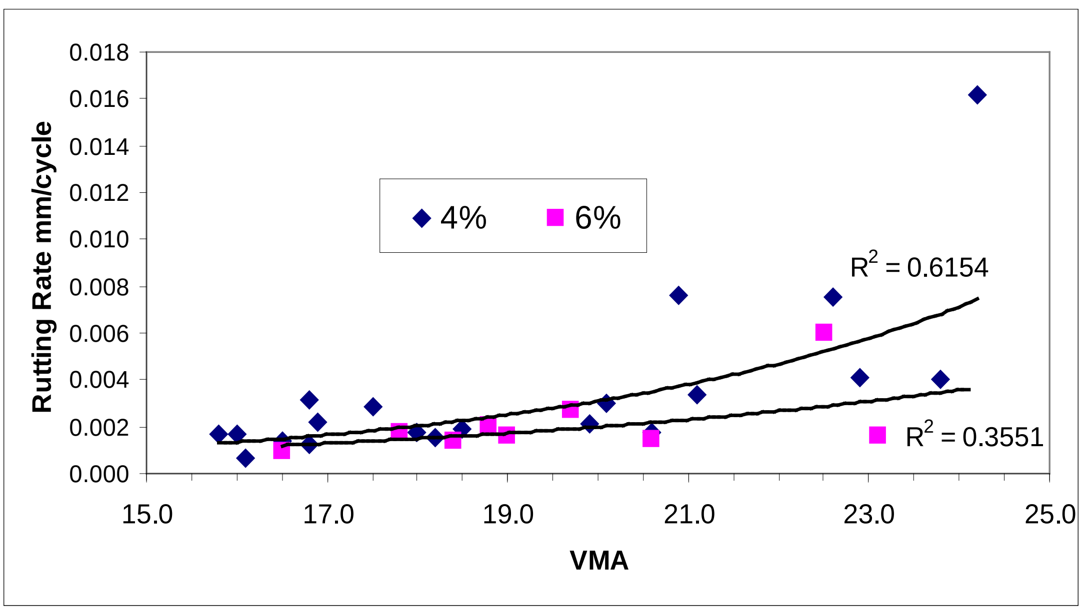

FIGURE 4.23 VMA Versus Rutting Rate by Design Air Voids

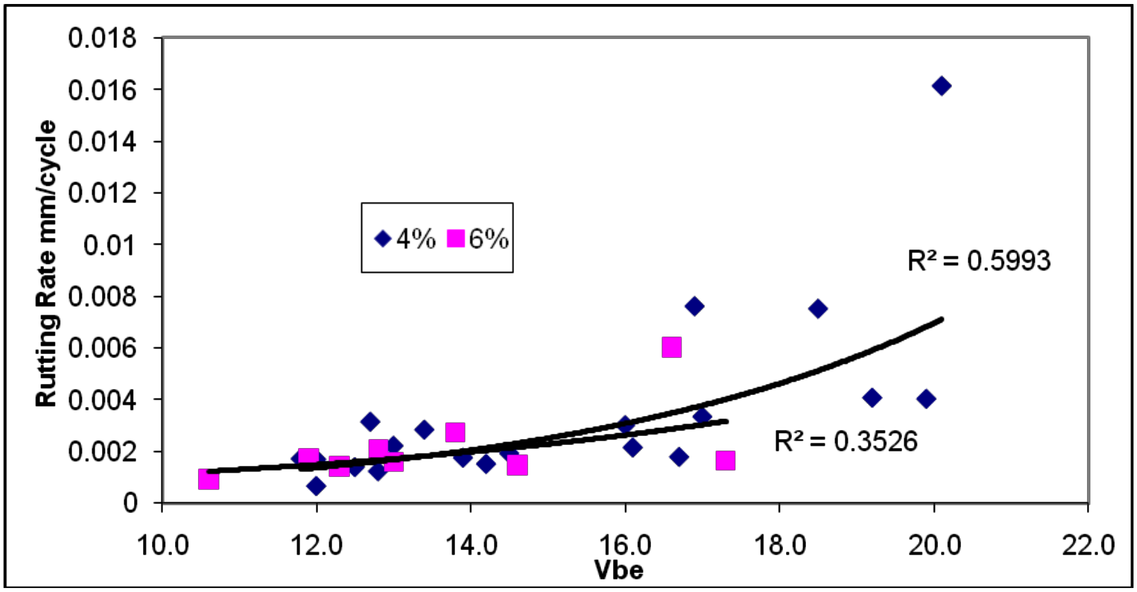

FIGURE 4.24 Volume of Effective Asphalt Versus Rutting Rate by Design Air Voids

When V_{be} versus rut depth is plotted for all mixtures, Figure 4.25, the relationship is reasonable, with an $R^2 = 0.57$. When the data is sorted in groups according to the amount of natural sand in each mixture, Figure 4.26, it is clear that as the percentage of natural sand increases, rutting rate also increases and the correlations improve. It appears that if Vbe is low, the effect of natural sand is minimized. However, if Vbe is over 13% to 14%, natural sand can be detrimental to rutting performance. The steep slope of the regression line for the over 15% sand mixtures seems to warrant limiting the amount of natural sand to less than 15% in mixtures designed for higher traffic volumes, where rutting resistance is important.

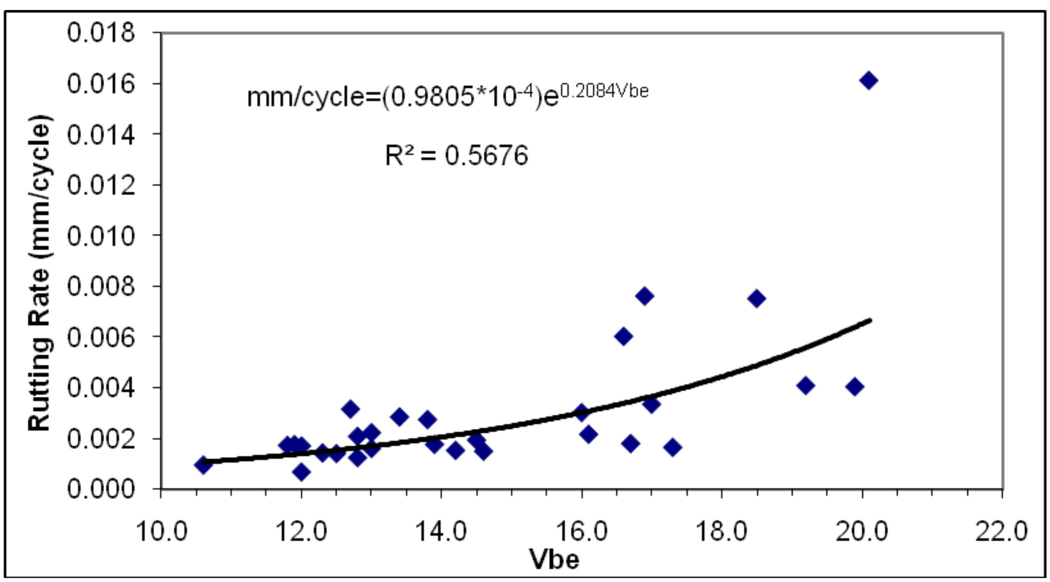

Figure 4.25 Vbe Versus Rutting Rate for All Mixtures

Recall from Section 2.0 it was hypothesized that FAA may be an important indicator of a mixture's rutting resistance, since the majority of the aggregate in 4.75 mm mixtures passes the

4.75 mm sieve. Figure 4.27 shows Vbe versus rutting rate for mixtures with FAA over 45 and FAA under 45. For aggregate blends with FAA over 45, rutting rate increased with a linear relationship with increasing asphalt content. The curve is much steeper for aggregate blends with FAA less than 45. Figures 4.26 and 4.27 indicate that natural sand and FAA can influence a mixture's rutting susceptibility, especially at asphalt contents over 14.0% by volume.

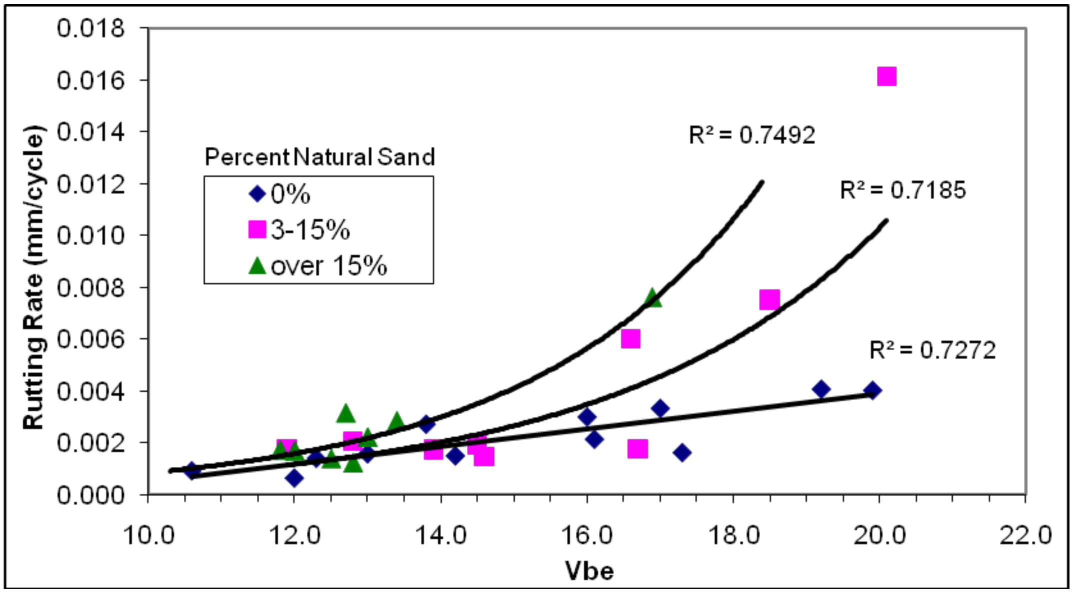

FIGURE 4.26 Vbe Versus Rutting Rate for all Mixtures, Sorted by Percent Natural Sand

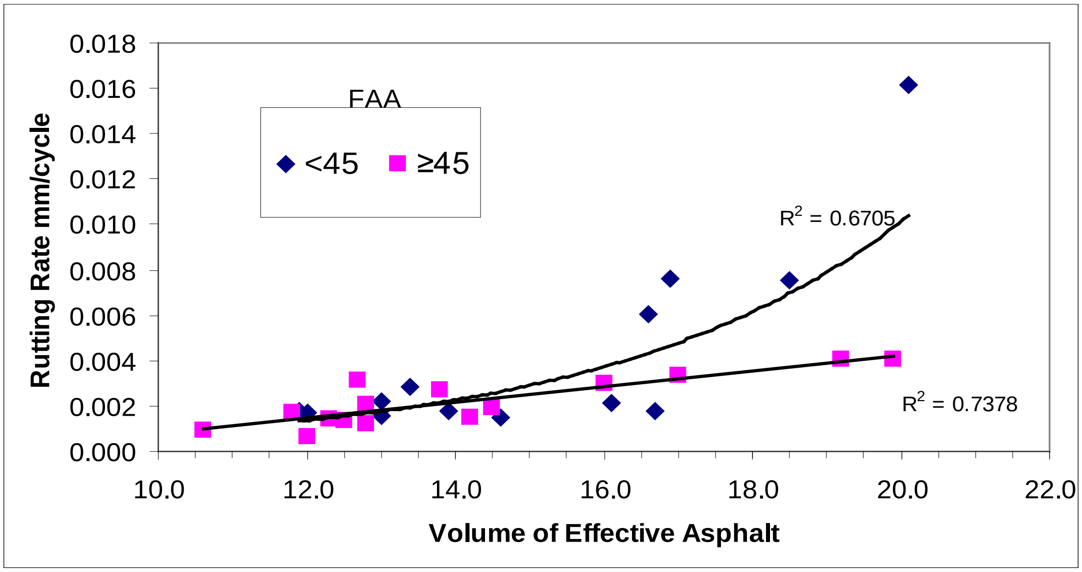

FIGURE 4.27 Vbe Versus Rutting Rate for All Mixtures, Sorted by FAA

It has been shown that for the mixtures prepared in this study, asphalt content, percent natural sand, and aggregate angularity all influence the rutting susceptibility of a 4.75 mm NMAS asphalt mixture. The question is, what is an acceptable amount of rutting for 4.75 mm mixtures? Recall that Cooley et al. (*1*) recommended a maximum VMA of 18% on a limiting APA rut depth of 9.5 mm from NCHRP 9-17. Although the APA was not used in this research, a similar approach was used. Using the relationship shown in Figure 3.3, where MVT rut depths were correlated to APA rut depths, an equivalent MVT limiting rut depth was found to be 15.7 mm. However, the MVT tests were conducted with a hose pressure of 100 psi and wheel load of 100 lb. Since rut testing by Cooley (*1*) was conducted at 120 psi hose pressure and 120 lb wheel load, another problem exists in comparing the MVT data to the criteria. Using the correlation established by Prowell and Moore (*12*) at NCAT, shown in Figure 4.28, an equivalent critical rut depth for MVT was found to be 13.1 mm. This critical rut depth is easily converted to a critical rutting rate (13.1 mm = 0.00164 mm/cycle at 8000 cycles). Based on the regression shown in Figure 4.24 and the critical rutting rate of 0.00164 mm/cycle, the maximum Vbe should be 13.5%.

Based on 13.5% Vbe, the specified maximum VMA or VFA should be dependent on the design V_a. For 4% design V_a, the maximum VMA would be 17.5%, and a maximum VFA would be 77%. If a mixture were designed at 6% V_a, the maximum VMA would be 19.5%, and the maximum VFA would be 69%.

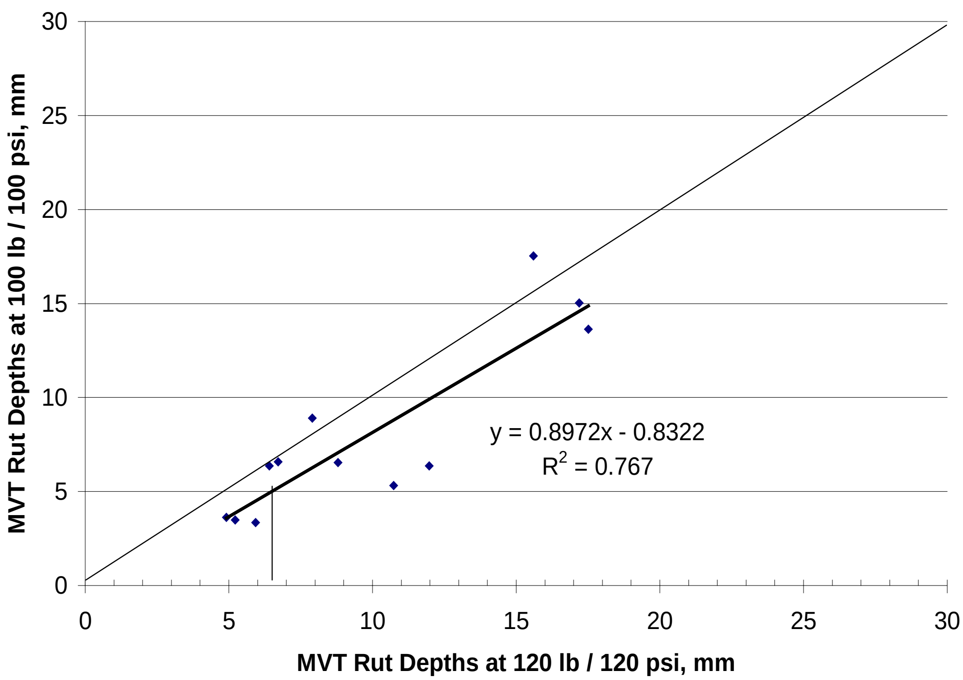

FIGURE 4.28 Relationship Between MVT Rut Depths at 120 lb, 120 psi to MVT Rut Depths at 100 lb, 100 psi

4.2.2 Tensile Strength Ratio

For all 29 mixtures designed in the laboratory, TSR was determined using AASHTO T-283. During a panel meeting of participating states, it was established that performance tests would be conducted at 9.0 percent% V_a, since this is a likely in-place V_a after compaction for a 4.75 mm NMAS mixture. Thus, all samples were compacted to 9±0.5% V_a.

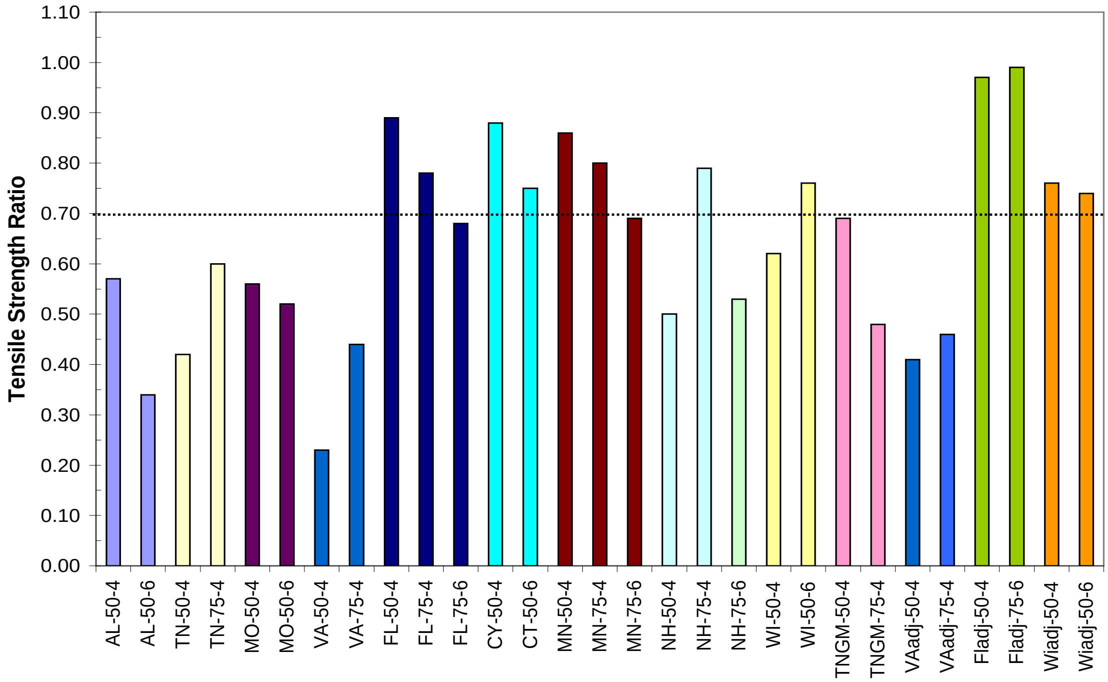

FIGURE 4.29 Tensile Strength Ratios for 29 Mix Designs

Figure 4.29 shows a bar chart of TSR for all 29 mixtures. The average TSR was 0.65, with a standard deviation of 0.19. The highest TSR was 0.99 for FL-adj-75-6, and the lowest was 0.23 for VA-50-6. Most agencies currently require a minimum TSR between 0.7–0.8. Only 12 of the 29 mixtures had TSR results greater than 0.7. However, it is important to note that the laboratory mixes did not contained any type of anti-stripping additive. Recall that the purpose of conducting the TSR testing in this study was to evaluate how changing the optimum asphalt content by adjusting N_{des} or the design V_a would affect the stripping potential.

It is also possible that low TSRs could have been caused by low vacuum pressures used to condition samples. It was noted during the saturation process of the conditioned samples that the vacuum pressure had to be reduced and saturation time generally had to be increased compared to other asphalt mixtures with larger NMAS. For 4.75 mm mixtures, the void spaces are smaller and not as interconnected and, therefore, it is possible that reducing the vacuum pressure and duration may have caused some damage to specimens by expanding void spaces and pushing apart aggregate. The low permeability results and the difficulty in obtaining saturation of specimens lends some evidence that 4.75 mm mixtures may be resistant to moisture intrusion, even at V_a of 9.0%, and therefore, resistant to stripping.

The TSR results show that, in general, decreasing the asphalt content for most aggregate blends caused a slight increase in moisture damage susceptibility. However, several aggregate blends (TN, VA, NH, WI, and FL-adj) had an increase in TSR as asphalt content decreased. In Figure 4.30, dry and wet tensile strengths were plotted for each blend. It can be seen that for some source materials, dry strength increases with decreasing asphalt content. Other mixtures, however, may show no difference in dry strength or even show a decrease in dry strength with lower asphalt contents. Asphalt-aggregate bonds are important to moisture susceptibility. This was not addressed in the experimental research plan for this study, so it is difficult to ascertain how the aggregate mineralogy affects the stripping potential of these mixtures.

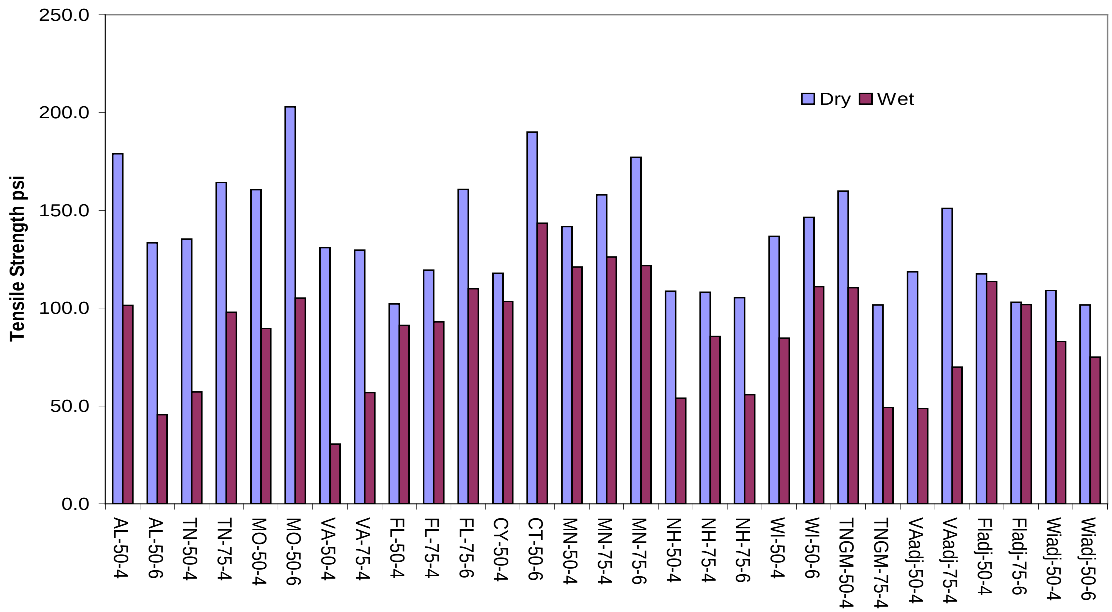

FIGURE 4.30 Tensile Strengths for Conditioned and Unconditioned Samples

There is a weak relationship between VMA and TSR, as shown in Figure 4.31, and a weak relationship between V_{be} and TSR, as shown in Figure 4.32. Although these correlations are likely confounded by other variables, the general trend of increasing TSR with increasing VMA and effective asphalt content was expected.

Natural sand content is one factor that may affect TSR for these mixtures. Figure 4.33 shows a plot with only the mixtures designed at 50 gyrations and 4% V_a to illustrate the influence natural sand has on sand asphalt mixtures. A plot with all the mix designs prepared for this research should show the same trend except that there would be more scatter around each point due to changes in asphalt content resulting from different gyration levels and target V_a.

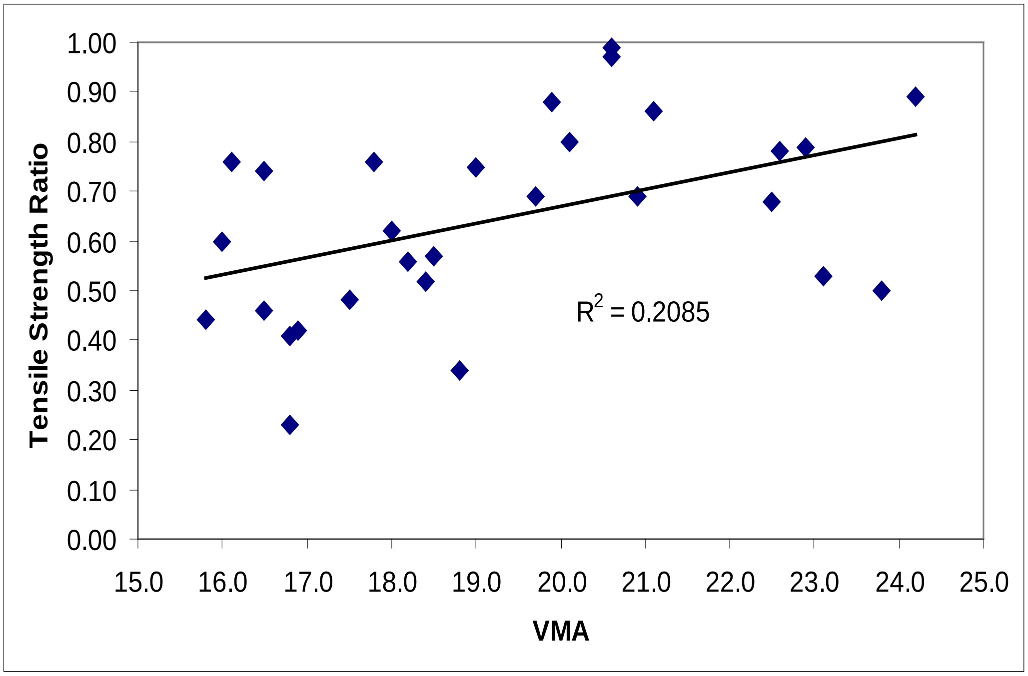

FIGURE 4.31 VMA Versus Tensile Strength Ratio

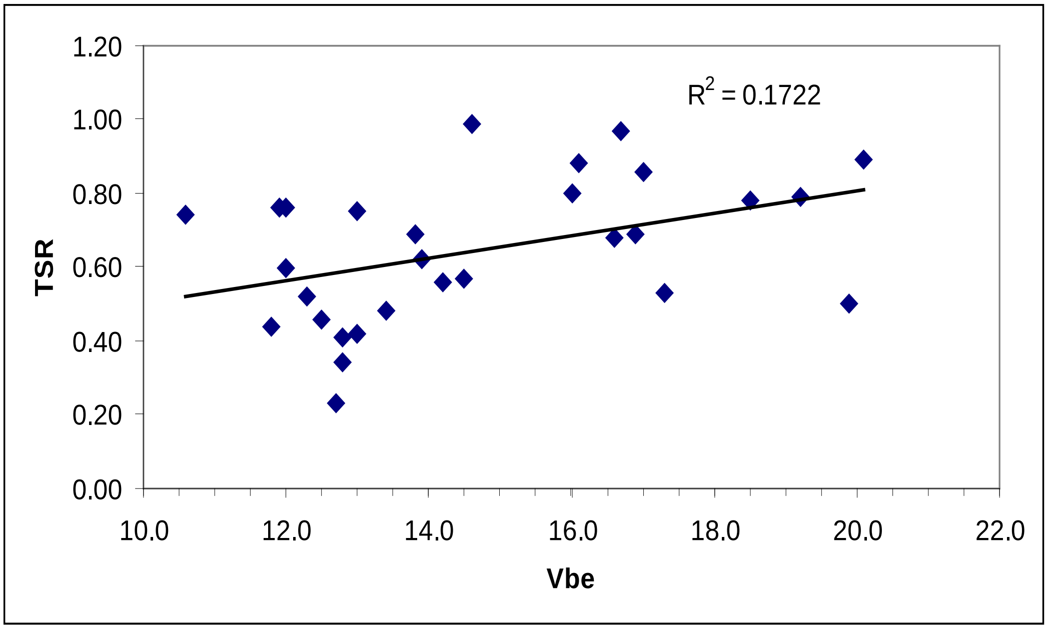

FIGURE 4.32 Effective Asphalt Content by Volume Versus Tensile Strength Ratio

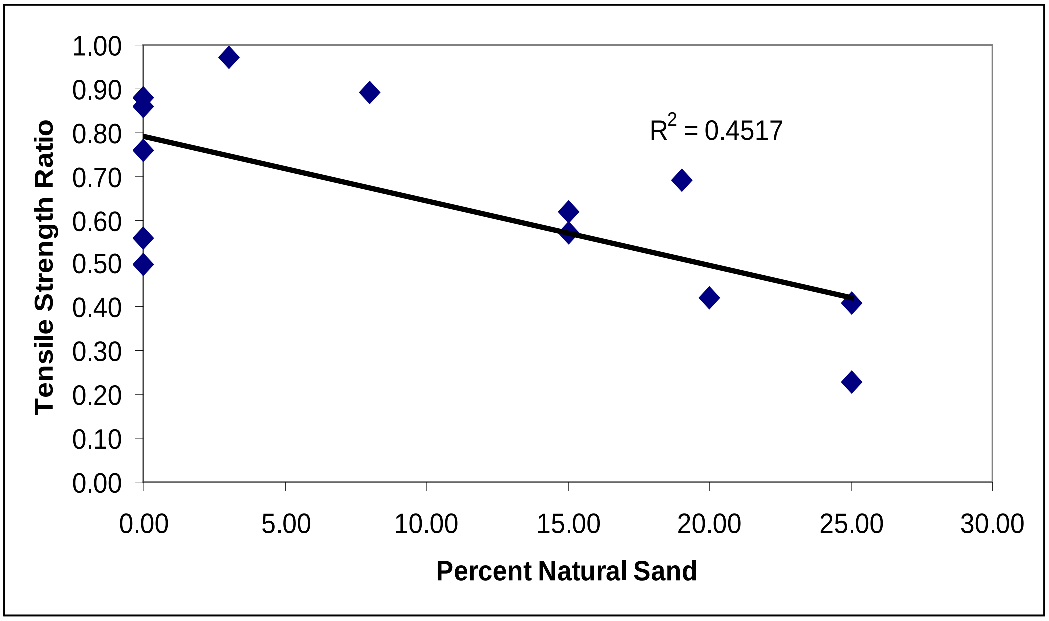

FIGURE 4.33 Relationship with Percent Natural Sand in Blended Aggregate and TSR for 50 Gyration 4% Air Void Mix Designs

It was thought that FT may be a good indicator of TSR; however, for the blends in this study, the relationship was weak (R^2=0.09). Dry strength seems to have a reasonable relationship with FT, as shown in Figure 4.34. No reasonable linear correlation or multiple linear regression models could be determined for wet tensile strength of the asphalt mixtures in this study.

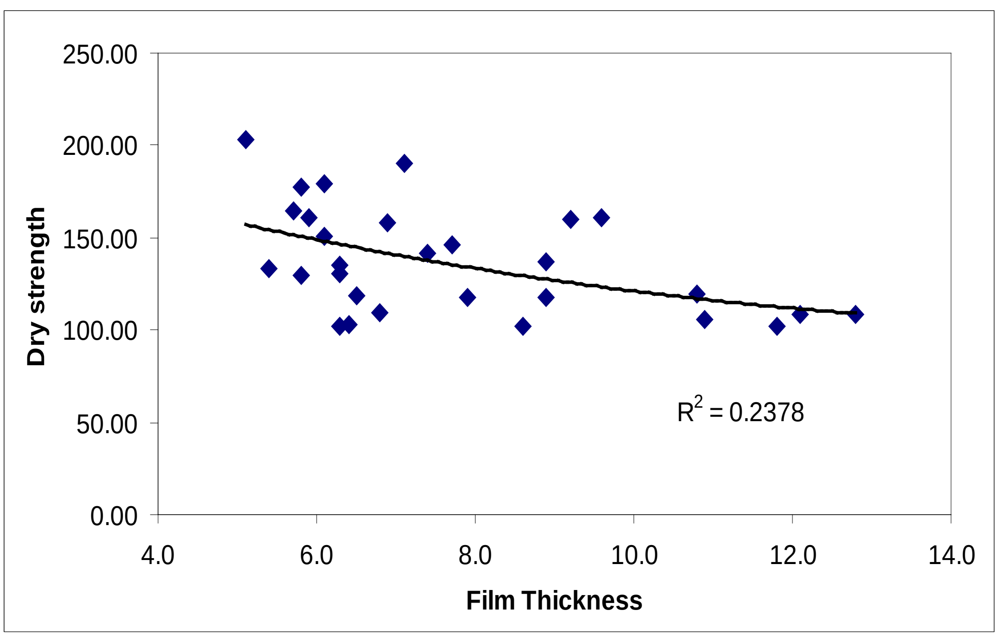

FIGURE 4.34 Dry Strength Versus Film Thickness

4.2.3 Fracture Energy Density Ratio

All 29 laboratory designed mixtures and three baseline mixtures were tested for FE density. For each mixture, two sets of specimens were prepared. The first set of specimens were tested with no aging. The second set was oven-aged at 85ºC for six days, then tested. A ratio of the aged FE density to the un-aged FE density was then calculated. A hypothesis of this study was that mixes with lower FE ratios would be more prone to aging and cracking over time. Although the aged FE values may not be below a threshold value where cracking will occur, a low ratio might identify a mixture that in certain field conditions could be more susceptible to cracking over time compared to mixtures with a higher ratio.

Table 4.16 shows FE ratios with aged and un-aged values for the 29 laboratory designed mixtures. The average FE ratio is 96.2%, with an average aged FE density of 5.399 kJ/m^3 and an un-aged average of 5.574 kJ/m^3. The high average for FE ratio indicates that small aggregate mixtures with high VMA and asphalt contents may be fairly resistant to cracking over time. FE ratios were plotted on Figure 4.35, sorted by state. In general, the FE ratio tends to decrease with decreasing asphalt contents that result from an increase in design V_a and/or number of gyrations. For each source of materials, the 50 gyration and 4% air void mixture (50-4) had the highest asphalt content. However, several exceptions to decreasing ratio stand out (TN, MO, and VA), where the ratio increased with decreasing asphalt content. Figure 4.36 shows a weak relationship between V_{be} and FE ratio, but there is a general trend of decreasing ratio with decreasing V_{be}.

MINITAB was used to determine Pearson correlation coefficients between FE ratio and aggregate and mixture volumetric properties. The significant relationships are shown in Table 4.17. The strongest correlation with FE ratio was with D:B ratio. As can be seen in Figure 4.37, as D:B ratio increases, FE tends to decrease. Table 4.17 shows that there are also significant relationships between FE ratio and VMA, VFA, FT, and dust content. Figures 4.38, 4.39 and 4.40 show plots of FE ratio versus these properties. These relationships indicate that resistance to long-term cracking for 4.75 mm mixtures is affected to some degree by volume or mass proportions.

TABLE 4.16 Fracture Energy Results for Laboratory Mixtures

State (mix)	*P-075*	*Eff AC%*	*VMA*	*VFA*	*D:B Ratio*	*FT (microns)*	*FE ratio (%)*	*FE unaged (kJ/m³)*	*FE aged (kJ/m³)*
AL-50-4	11.1	6.30	18.5	78.4	1.8	6.1	102	3.57	3.65
AL 50-6	11.1	5.60	18.8	68.1	2.0	5.4	79	6.10	4.84
TN-50-4	11.6	5.80	16.9	76.8	2.0	6.3	60	3.70	2.20
TN-75-4	11.6	5.30	16.0	74.8	2.2	5.7	80	2.86	2.29
MO-50-4	10.6	6.10	18.2	78.2	1.7	5.9	59	5.84	3.45
MO-50-6	10.6	5.30	18.4	66.7	2.0	5.1	75	4.76	3.51
VA-50-4	10.1	5.90	16.8	75.8	1.7	6.3	68	4.54	3.07
VA-75-4	10.1	5.40	15.8	74.9	1.9	5.8	91	6.37	5.79
FL-50-4	7.7	9.70	24.2	82.8	0.8	11.8	127	4.50	5.72
FL-75-4	7.7	8.90	22.6	81.8	0.9	10.8	88	5.07	4.47
FL-75-6	7.7	8.00	22.5	73.7	1.0	9.6	94	5.67	5.35
CT-50-4	7.9	6.80	19.9	80.9	1.2	8.9	151	5.60	8.48
CT-50-6	7.9	5.50	19.0	68.5	1.4	7.1	104	6.90	7.15
MN-50-4	11.2	7.20	21.1	80.4	1.6	7.4	115	7.80	8.94
MN-75-4	11.2	6.80	20.1	79.8	1.7	6.9	110	7.38	8.08
MN-75-6	11.2	5.80	19.7	70.1	1.9	5.8	94	6.48	6.07
NH-50-4	6.0	9.10	23.8	83.6	0.7	12.8	137	5.45	7.45
NH-75-4	6.0	8.70	22.9	84.0	0.7	12.1	97	5.90	5.72
NH-75-6	6.0	7.90	23.1	75.0	0.8	10.9	106	7.06	7.48
WI-50-4	7.1	6.00	18.0	77.4	1.2	8.9	91	5.51	5.04
WI-50-6	7.1	5.20	17.8	66.9	1.4	7.7	85	6.05	5.17
TNGM-50-4	8.2	6.8	20.9	80.7	1.0	9.2	129	5.46	6.62
TNGM-75-4	8.2	6.4	17.5	76.5	1.3	8.6	97	5.06	4.89
VA adj-50-4	10.1	6.0	16.8	76.4	1.7	6.5	132	5.75	7.58
VA adj-75-4	10.1	5.7	16.5	75.6	1.7	6.1	93	5.69	5.32
FL adj-50-4	13.4	7.9	20.6	81.1	1.7	7.9	104	5.10	5.28
FL adj-75-6	13.4	7.0	20.6	71.0	1.9	6.4	60	5.89	3.55
WI adj-50-4	9.5	5.1	16.1	74.4	1.9	6.8	82	6.80	5.57
WI adj-50-6	9.5	4.6	16.5	64.4	2.1	6.3	81	4.79	3.86
						Average	96.2	5.57	5.40
						Std Dev	23.4	1.1	1.8
						COV	24%	20%	33%

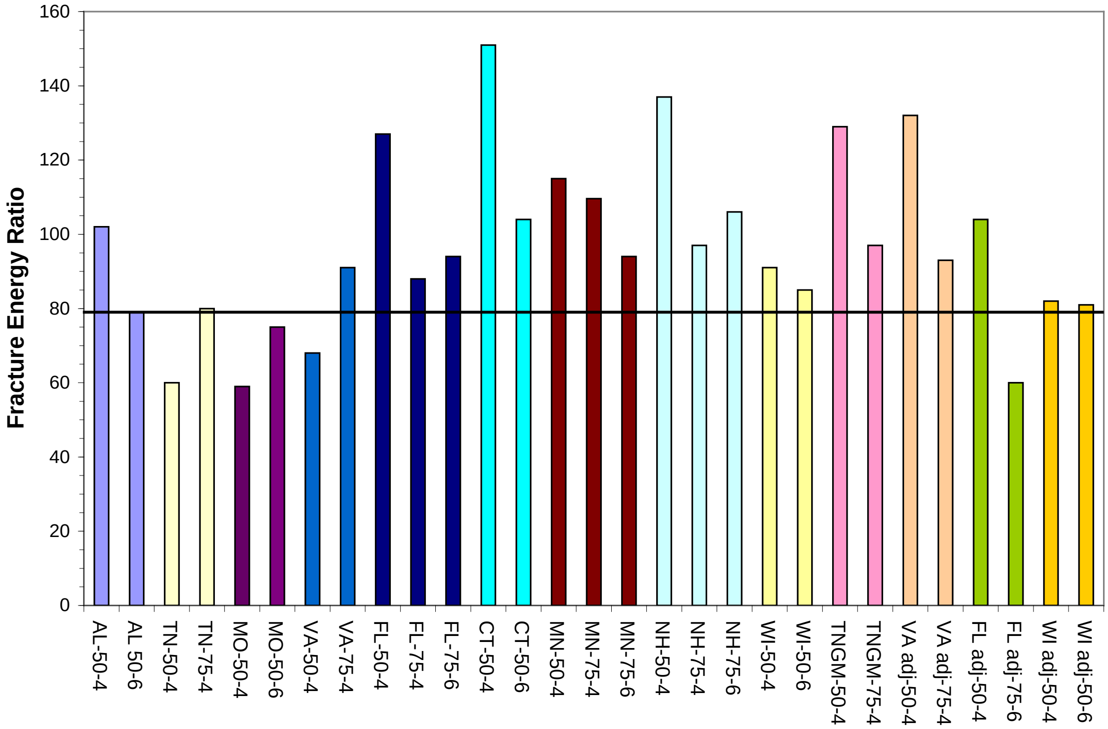

FIGURE 4.35 Fracture Energy Ratio for Laboratory Mixtures

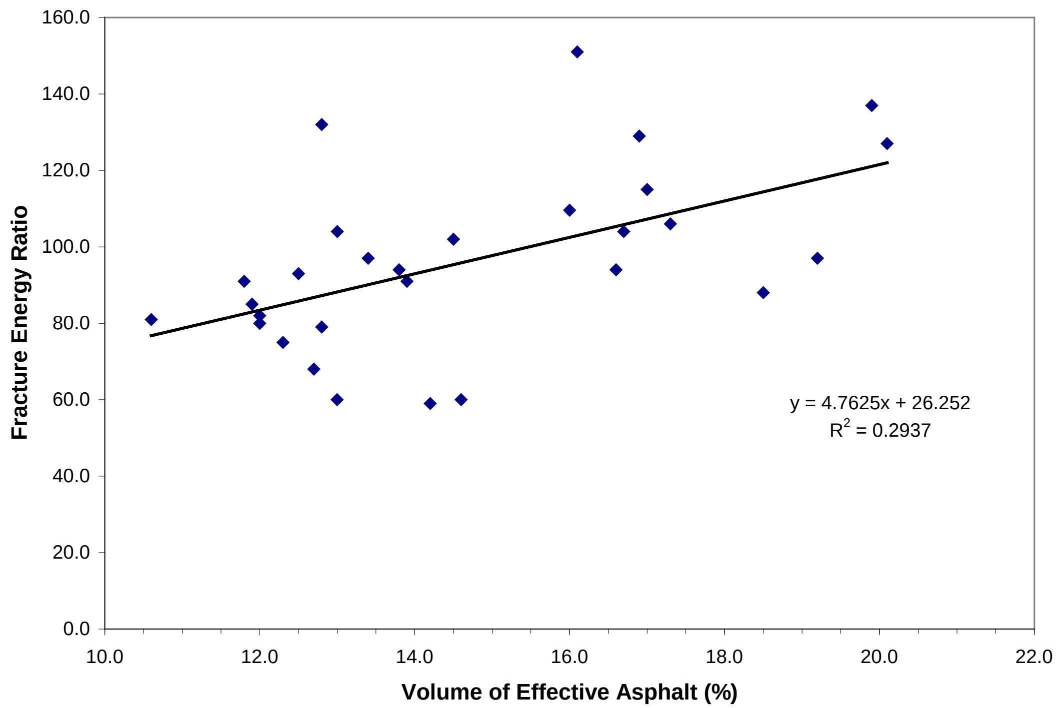

FIGURE 4.36 Fracture Energy Ratio Versus Vbe

TABLE 4.17 Pearson Correlation Coefficients and *p*-Values for Linear Relationships with Fracture Energy Ratio

	FT	D:B	VFA	VMA	P-075
R	0.532	-0.552	0.506	0.453	-0.418
***p*-value**	0.003	0.002	0.005	0.013	0.024

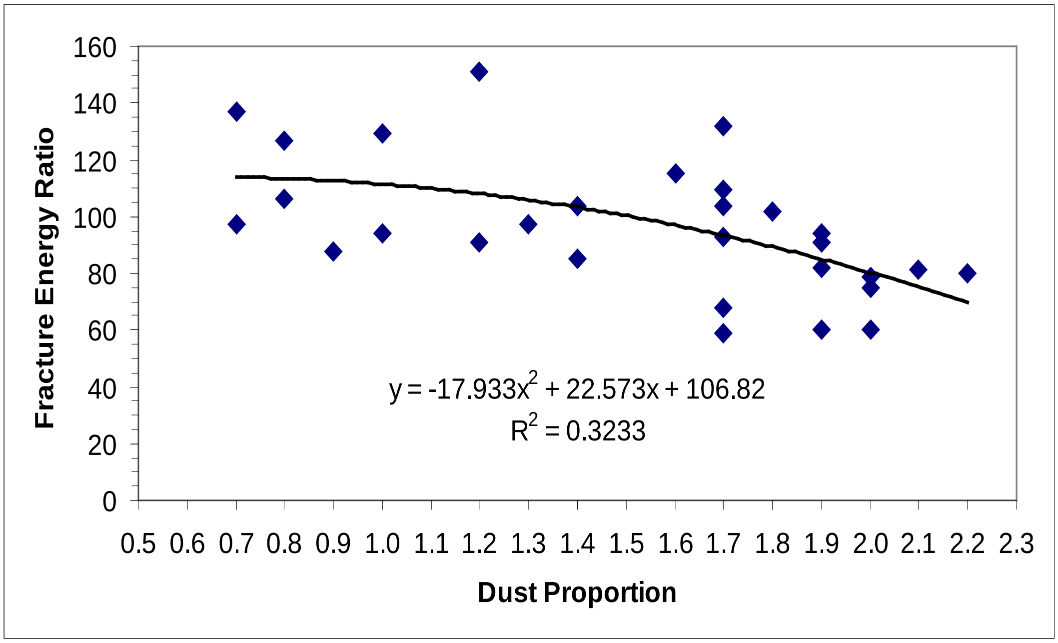

FIGURE 4.37 Fracture Energy Ratio Versus Dust-to-Binder Ratio

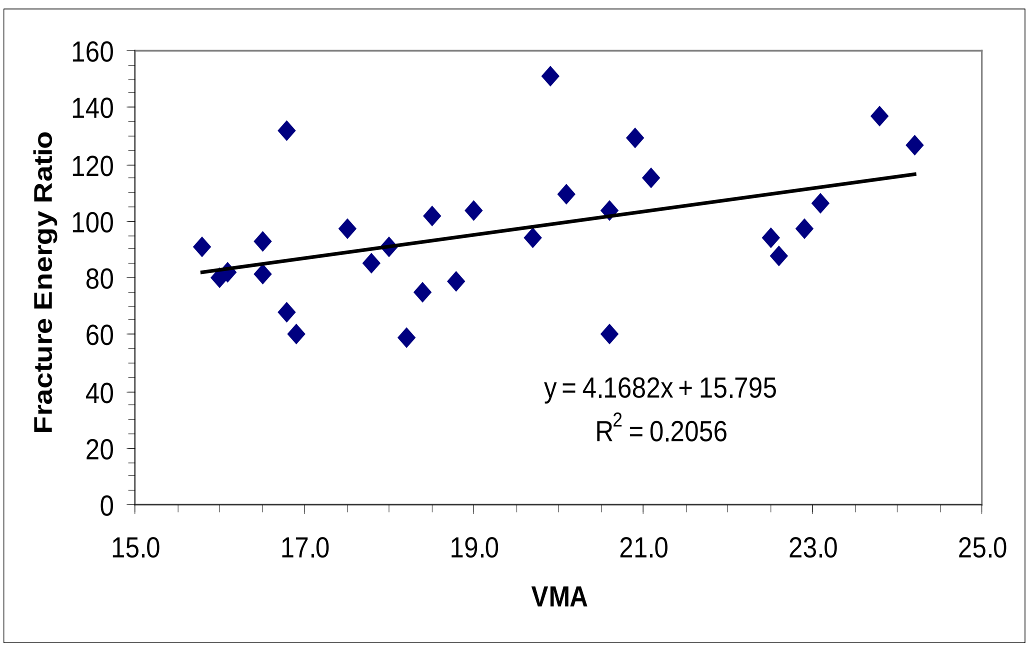

FIGURE 4.38 Fracture Energy Ratio Versus VMA

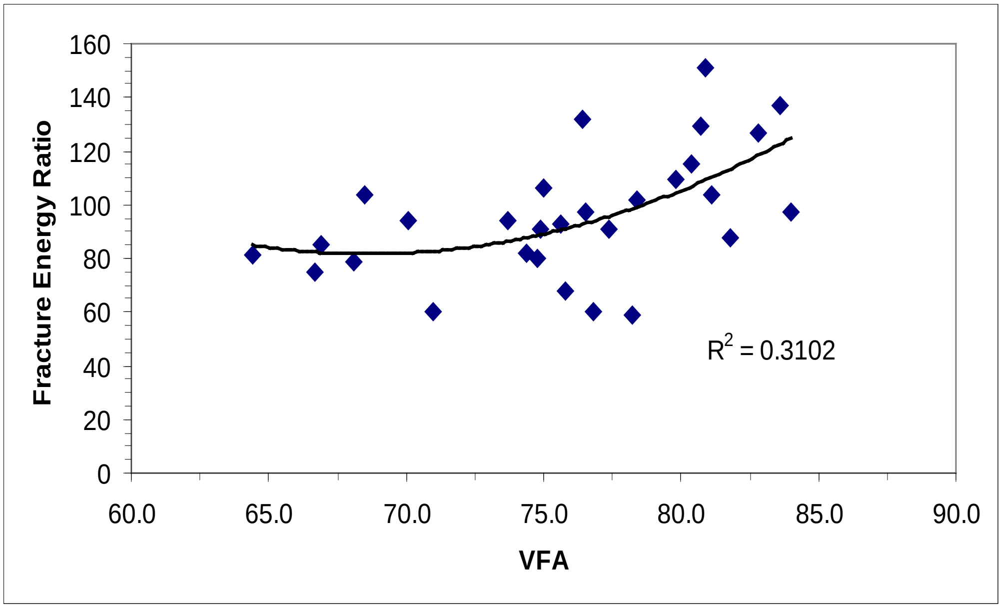

FIGURE 4.39 Fracture Energy Ratio Versus VFA

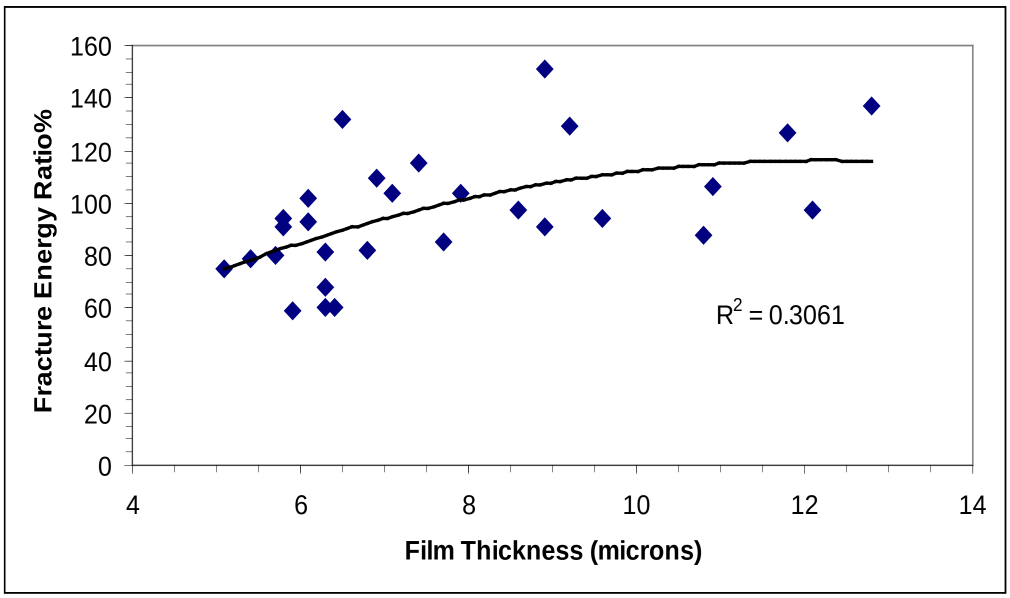

FIGURE 4.40 Fracture Energy Ratio Versus Film Thickness

A FE threshold was needed to discern critical values for volumetric properties such as minimum VMA, VFA, or Vbe. Recall Figure 3.7 where Kim et al. (*14*) plotted FE density for specimens from Westrack. The regression from this plot indicates that fatigue cracking begins to occur at 3.0 kJ/m^3. If this number is considered a threshold where no cracking is expected below the threshold value, then most of the mixtures presented in Table 4.16 should perform satisfactorily. However, this conclusion is not valid due to differences in field aging conditions at Westrack and the long-term oven aging used in the laboratory for this research.

Since an appropriate threshold value of this ratio is unclear, the FE ratios of the baseline mixtures presented in Table 4.18 were used as a benchmark to establish a reasonable limit for FE ratio. Due to a lack of material, FE density testing could not be performed for the baseline mixture from Mississippi. The mixtures from Georgia, Maryland, and Michigan are reported to have good in-service performance history. The mean FE ratio for the baseline mixtures is 76%, and the median is 80%. To serve as a benchmark for durability performance, the baseline median was chosen as a conservative estimate of a minimum value to compare with the laboratory prepared mixes. Figure 4.35 shows that only six mixtures were below the 80% FE ratio threshold.

The regression in Figure 4.35 shows that an 80% FE ratio corresponds to a minimum Vbe of 11.5, and Figure 4.37 shows that a minimum 80% FE ratio corresponds to a maximum D:B ratio of 2.0. Recommending only a minimum Vbe may not be sufficient with regard to assuring cracking resistance. It can be seen in Figures 4.37 and 4.40 that D:B ratio and FT have slightly stronger correlations with FE ratio compared to Vbe. Since FT and D:B ratio are both related to Vbe and dust content, it is clear that the ability to maintain cracking resistance for the 4.75 mm mixtures designed in this study is dependent on asphalt and dust contents. The currently specified maximum D:B ratio of 2.0 appears to be reasonable, based on Figure 4.37.

Table 4.18 Fracture Energy Density Data for Baseline Mixtures

State (mix)	*V_a (design)*	*N_{des}*	*%A.C.*	*Binder*	*VMA*	*VFA*	*D:B Ratio*	*FT (microns)*	*FE Ratio %*	*Un-Aged FE kJ/m^3*	*Aged FE kJ/m^3*
Mississippi	4.0	50	5.9	76-22	17.7	66.6	2.0	5.4	N/A	N/A	N/A
Maryland	3.5	75	6.5	64-22	16.3	80.9	1.6	7.3	80	5.58	4.44
Georgia	6.0	50	6.0	64-22	16.7	76.4	1.5	6.7	81	4.89	3.95
Michigan	4.0	60	7.5	52-28	17.0	69.4	1.4	7.1	68	7.24	4.94
								Mean =	76	5.90	4.44
								Std Dev=	7.1	1.21	0.49
								Median=	80	5.58	4.44

4.2.4 Permeability

Laboratory permeability testing was performed on 27 of the 29 mix designs, and the results are shown in Table 4.19 and Figure 4.41. Mixtures TNGM-50-4 and VA-adj-50-4 were not tested for permeability due to insufficient material. Mixtures with permeability less than 125 cm/sec E^{-5} are generally considered impermeable, and Figure 4.40 shows that 21 out of the 27 mixtures are below this threshold. The maximum permeability was 211 E^{-5} cm/sec for WI-50-4, and the minimum was 8 E^{-5} cm/sec. It was thought that permeability would increase when asphalt content decreases; however, this was not the case.

It is clear that most of the mixtures prepared for this research are impermeable even at relatively high V_a. It was mentioned in Section 2.3.3 that mixtures over 8.0% V_a generally are permeable. The 4.75 mm NMAS mixtures were shown to be impermeable even at 9.0% V_a because the V_a are not as interconnected compared to larger NMAS asphalt mixtures.

TABLE 4.19 Permeability and Mixture Data for Laboratory Mixtures

State (mix)	P-075	Pb	VMA	VFA	D:B Ratio	SE	FAA	FT (microns)	k (cm/s)E^{-5}
AL-50-4	11.1	7.4	18.5	78.4	1.8	67	46.3	6.1	49
AL 50-6	11.1	6.9	18.8	68.1	2.0	67	46.3	5.4	50
TN-50-4	11.6	7.3	16.9	76.8	2.0	69	44.8	6.3	67
TN-75-4	11.6	6.8	16.0	74.8	2.2	69	44.8	5.7	75
MO-50-4	10.6	6.9	18.2	78.2	1.7	74	49.0	5.9	21
MO-50-6	10.6	6.2	18.4	66.7	2.0	74	49.0	5.1	38
VA-50-4	10.1	8.8	16.8	75.8	1.7	76	45.0	6.3	45
VA-75-4	10.1	8.3	15.8	74.9	1.9	76	45.0	5.8	30
FL-50-4	7.7	11.8	24.2	82.8	0.8	88	44.1	11.8	154
FL-75-4	7.7	11.0	22.6	81.8	0.9	88	44.1	10.8	126
FL-75-6	7.7	10.1	22.5	73.7	1.0	88	44.1	9.6	79
CT-50-4	7.9	8.8	19.9	80.9	1.2	79	40.7	8.9	91
CT-50-6	7.9	7.2	19.0	68.5	1.4	79	40.7	7.1	111
MN-50-4	11.2	8.8	21.1	80.4	1.6	67	46.2	7.4	8
MN-75-4	11.2	8.3	20.1	79.8	1.7	67	46.2	6.9	17
MN-75-6	11.2	7.4	19.7	70.1	1.9	67	46.2	5.8	8
NH-50-4	6.0	9.7	23.8	83.6	0.7	85	51.0	12.8	34
NH-75-4	6.0	9.3	22.9	84.0	0.7	85	51.0	12.1	52
NH-75-6	6.0	8.6	23.1	75.0	0.8	85	51.0	10.9	15
WI-50-4	7.1	7.5	18.0	77.4	1.2	81	43.7	8.9	211
WI-50-6	7.1	6.7	17.8	66.9	1.4	81	43.7	7.7	179
TNGM-75-4	8.2	9.3	17.5	76.5	1.3	70	42.2	8.6	162
VA adj-75-4	10.1	8.7	16.5	75.6	1.7	76	45.0	6.1	34
FL adj-50-4	13.4	10.0	20.6	81.1	1.7	79	44.5	7.9	177
FL adj-75-6	13.4	9.1	20.6	71.0	1.9	79	44.5	6.4	124
WI adj-50-4	9.5	6.8	16.1	74.4	1.9	81	45.8	6.8	31
WI adj-50-6	9.5	6.3	16.5	64.4	2.1	81	45.8	6.3	78
								Average=	77
								Std Dev=	59
								COV=	77%

The results provided no clear relationships between mixture permeability and volumetric properties. One reason for this is that, according to the test procedure used (ASTM PS 121), a vacuum pressure of 525 mm of mercury (Hg) is to be applied to the specimen for five minutes to achieve saturation. However, due to low permeability of these mixtures, the specimens were saturated at a lower pressure (50–100 mm Hg) for 10 minutes until they were 85% to 95% saturated. This high level of saturation was used because it was observed that consistent results were only achieved at about 90% saturation. It is possible that the test specimens were damaged during the saturation process, which increased permeability due to expansion of internal voids.

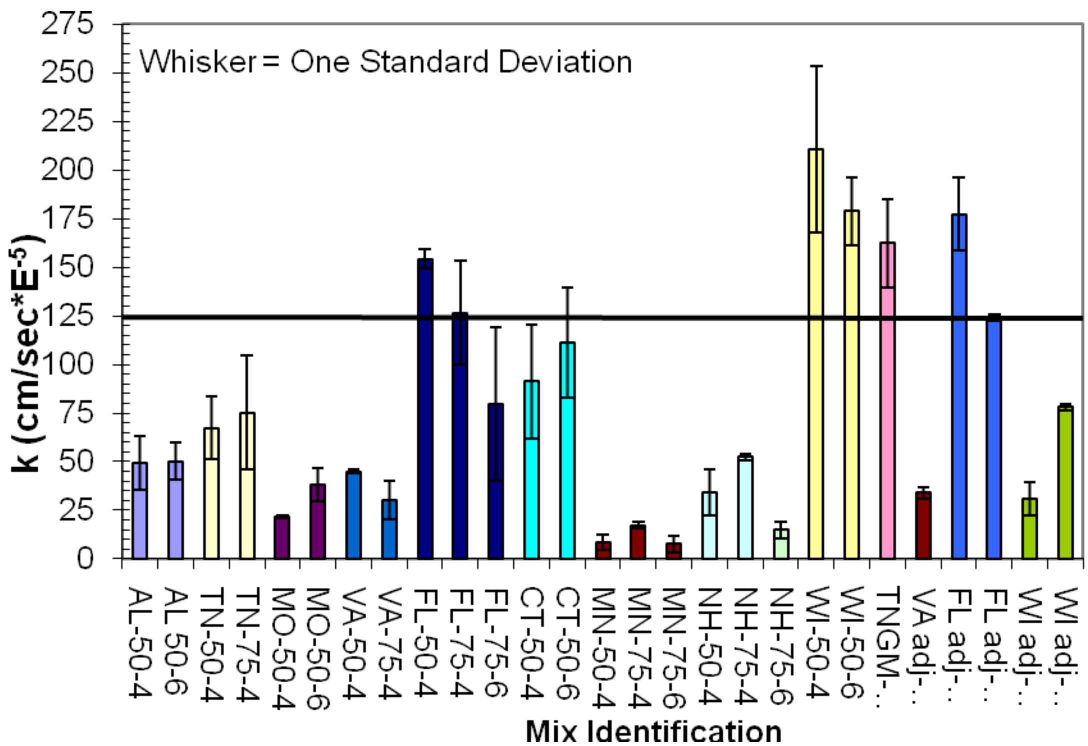

FIGURE 4.41 Permeability for Laboratory Mixtures

One aggregate property found to influence mixture permeability is FAA. Figure 4.42 shows that permeability decreases with increasing FAA. Fine aggregates with high FAA can have flat and slivery particles, which in an uncompacted state like during the FAA test, results in high void contents. However, when the particles are compacted with a gyratory compactor, they become more horizontally oriented, which could block flow paths in the compacted specimens.

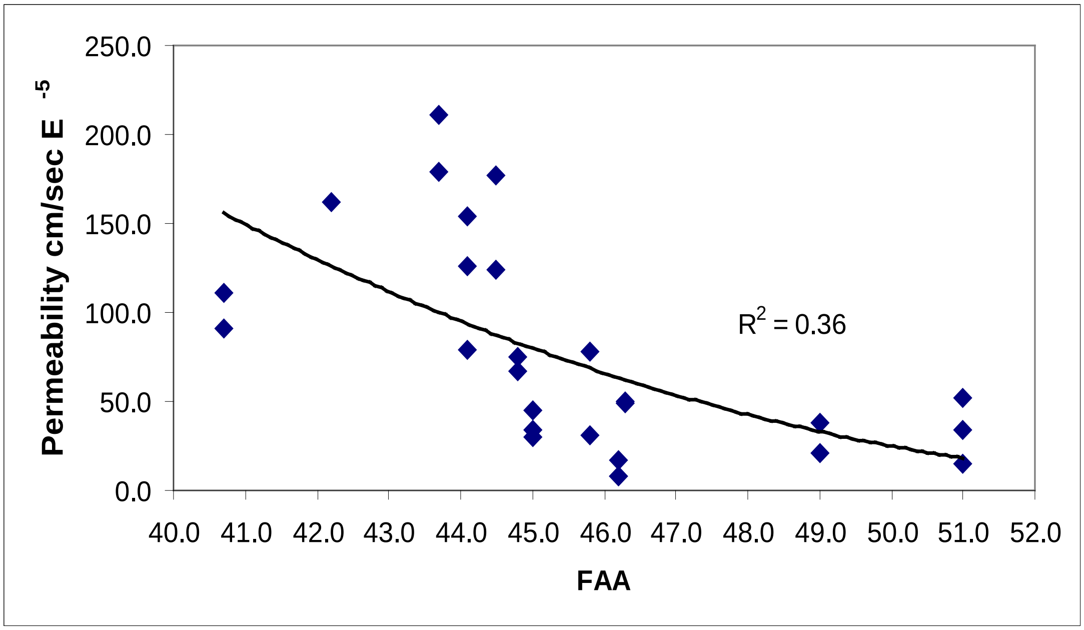

FIGURE 4.42 FAA Versus Permeability

4.3 Baseline Mixtures

Four plant-produced mixtures with good performance history were used as a baseline to compare the lab mixes to 4.75 mm NMAS mixtures being produced. Plant-produced mixtures from Mississippi, Maryland, Georgia, and Michigan were included as baseline mixtures. The mixture properties and averages are given in Table 4.18. The mixture from Georgia is not a 4.75 mm NMAS blend based on the percent passing the 4.75 mm sieve; however, it provides a good comparison to similar small aggregate size asphalt mixtures.

Generally, compared to the laboratory mixtures, baseline mixes are coarser graded, have lower optimum asphalt contents, lower VMAs, lower rut depths, higher TSR values, and lower average FE ratios. Figure 4.43 shows gradations for the baseline mixtures. Compared to the gradations of the lab mixes shown in Figure 3.2, the baseline mixtures are closer to the maximum density line. Even with lower percentages passing the 0.075 mm sieve, the baseline mixtures have lower VMAs due to coarser gradations.

The baseline mixture from Mississippi had the lowest MVT rut depth of all mixtures in the study. This was expected since this mix contained a polymer modified PG 76-22 binder. The average MVT rut depth for the baseline mixtures was 9.4 mm. This average is below the 13.1 mm rut depth assumed in this paper as a critical rut depth for 4.75 mm mixtures. The baseline mixture from Michigan had a 15.7 mm rut depth in the MVT, probably due to the use of a PG 58-22 binder in the mixture. Although this mix contained a softer asphalt grade, the MVT test was conducted at 64ºC, as were all mixtures in this study.

TABLE 4.20 Mixture Properties and Performance Data for Baseline Mixtures

State (mix)	V_a (design)	V_a actual	N_{des}	Passing 0.075 mm	Passing 1.18 mm	Passing 4.75 mm	%Nat. sand
Mississippi	4.0	5.9	50	10.7	50.0	98.0	10.9
Maryland	3.5	3.1	75	8.1	42.8	95.6	15.0
Georgia	6.0	3.9	50	8.5	43.1	79.5	0.0
Michigan	4.0	5.2	60	7.1	54.6	92.5	0.0
Average=	4.4	4.5	58.8	8.6	47.6	91.4	6.5
Std Dev=	1.11	1.26	11.81	1.52	5.72	8.25	7.66

State (mix)	%A.C.	P_{be}	Binder	VMA	VFA	$\%G_{mm}$ @ N_{ini}	D:B Ratio
Mississippi	5.9	5.3	76-22	17.7	66.6	86	2.0
Maryland	6.5	5.7	64-22	16.3	80.9	89.1	1.6
Georgia	6.0	5.5	64-22	16.7	76.4	90.2	1.5
Michigan	7.5	6.0	58-22	17	69.4	88.5	1.4
Average=	6.5	5.6		16.9	73.3	88.5	1.6
Std Dev=	0.73	0.30		0.59	6.52	1.78	0.26

State (mix)	SE	FAA	FT (microns)	Rut Depth (mm)	FE ratio %	TSR	k (cm/s) E^{-5}
Mississippi	N/A	N/A	5.4	3.8	N/A	0.85	48
Maryland	67	45.7	7.3	9.5	80	0.78	61
Georgia	N/A	N/A	6.7	8.6	81	0.92	107
Michigan	87	44.6	7.1	15.7	68	0.78	96
Average=	77.0	45.2	6.6	9.4	76.2	0.83	78
Std Dev=	14.14	0.78	0.85	4.90	7.08	0.07	28

State (mix)	Dry TS	Wet TS	FE Un-aged	FE Aged
Mississippi	220.1	187.9	N/A	N/A
Maryland	164.4	129	5.582	4.442
Georgia	137.1	126.3	4.887	3.949
Michigan	209.4	164.1	7.242	4.937
Average=	182.8	151.8	5.904	4.443
Std Dev=	38.84	29.58	1.21	0.49

TSR for the baseline mixtures appear to be reasonable. The average was 0.83; however, if 0.80 is used as a minimum, which is common for many specifying agencies, the mixtures from Michigan and Maryland are slightly below this minimum. All baseline mixtures contained about 1.0 % hydrated lime, which may explain the higher TSR compared to the laboratory mixtures.

As with the laboratory designed mixtures, permeability was low even at high V_a. The average permeability for baseline mixtures was 78 E^{-5}cm/sec at 9.0% V_a, which is practically the same as the average for the laboratory mixtures at 77 E^{-5} cm/sec at 9.0% V_a.

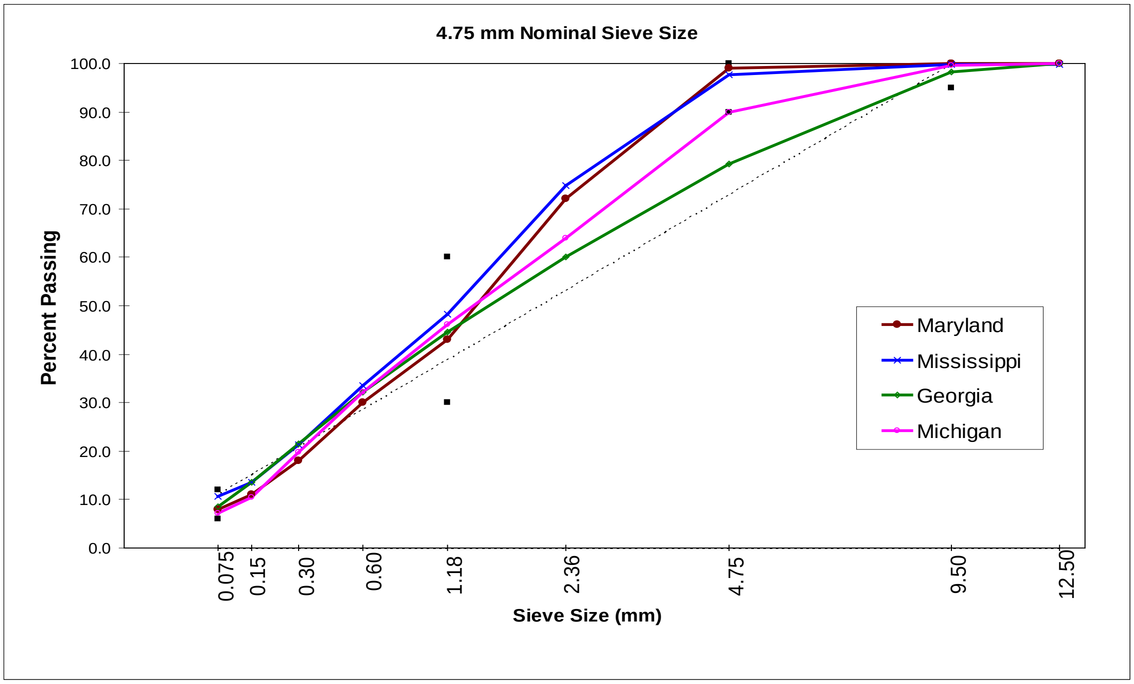

FIGURE 4.43 Gradations for Baseline Mixtures

One performance concern with these mixtures may be durability. FE ratios for baseline mixtures are low compared to most of the laboratory mixtures. Additional aging caused by reheating the mixtures to make the specimens, lower FT, and lower P_{be} probably contributed to the baseline mixtures' lower FE ratios. Also, it is not clear if the softer binder used in the Michigan baseline mixture contributed to a lower FE ratio, which is noticeably lower at 68% compared to 80% and 81% for base line mixtures from Maryland and Georgia.

4.4 Review of AASHTO Specifications

The AASHTO mix design criteria for 4.75 mm NMAS Superpave designed asphalt mixtures are presented in Table 4.21. The main objective of this research is to refine the current procedures and criteria for 4.75 mm mixtures, so a comparison to current AASHTO criteria is presented in this section.

TABLE 4.21 AASHTO Criteria For 4.75 mm NMAS Superpave Asphalt Mixtures.

Design ESALs (Millions)	N_{des}	Minimum FAA Depth from Surface ≤ 100 mm	Minimum FAA Depth from Surface ≥ 100 mm	Minimum SE	Min. VMA	VFA	% G_{mm} @ N_{ini}
<0.3	50	-	-	40	16.0	70-80%	≤91.5
0.3 to <3.0	75	40	40	40	16.0	65-78%	≤90.5
3.0 to<30	100	45	40	45	16.0	75-78%	≤89.0
Sieve size	**Min.**	**Max.**	V_a = 4.0%				
12.5 mm	100		D:B Ratio: 0.9 to 2.0				
9.5 mm	95	100					
4.75 mm	90	100					
1.18 mm	30	60					
0.075 mm	6	12					

4.4.1 AASHTO Gradation Limits

Most of the laboratory prepared mixtures and baseline mixtures meet current gradation limits specified in AASHTO. Three blends, however, are outside current gradation limits. FL-adj had 13.4% passing the 0.075 mm sieve, which exceeded the maximum of 12%. This high dust content was intentionally used to lower the high VMA obtained in the FL blend. Six percent baghouse fines were added to the FL blend to lower VMA. For this mixture, adding the fines was beneficial. VMA lowered, TSR values increased, and D:B ratio increased to meet current specifications. This indicates that increasing the maximum limit on the 0.075 mm sieve may allow for 4.75 mm mix designs to have slightly higher dust contents as a way to control volumetric properties.

The MN blend was finer than the current limits specified for the 1.18 mm sieve. The maximum percent passing the 1.18 mm sieve is currently 60%; the MN blend had 61.1% passing. This gradation gave the lowest optimum asphalt content from the aggregate trial portion of the MN mix design. The final mixtures prepared with the MN aggregate blend had a high VMA (19.7 to 21.1), due largely to the fineness of the gradation.

The 1.18 mm sieve is used to divide a 4.75 mm NMAS mixture into two fractions, where the material above this sieve is the coarse fraction, and below the sieve is the fine fraction of the aggregate blend. Increasing the coarse fraction will make a fine-graded mixture move closer to the maximum density line. Figure 4.44 illustrates two ways that can be used to decrease V_{be}. One way is to increase the dust content; the second way is to decrease the fine fraction of the gradation. It is recommended that the current gradation limits be adjusted to avoid gradations that may have excessive VMA. This can be done by limiting the amount of material passing the 1.18 mm to 55% and increasing the amount of material passing the 0.075 mm sieve to a maximum of 13.0%.

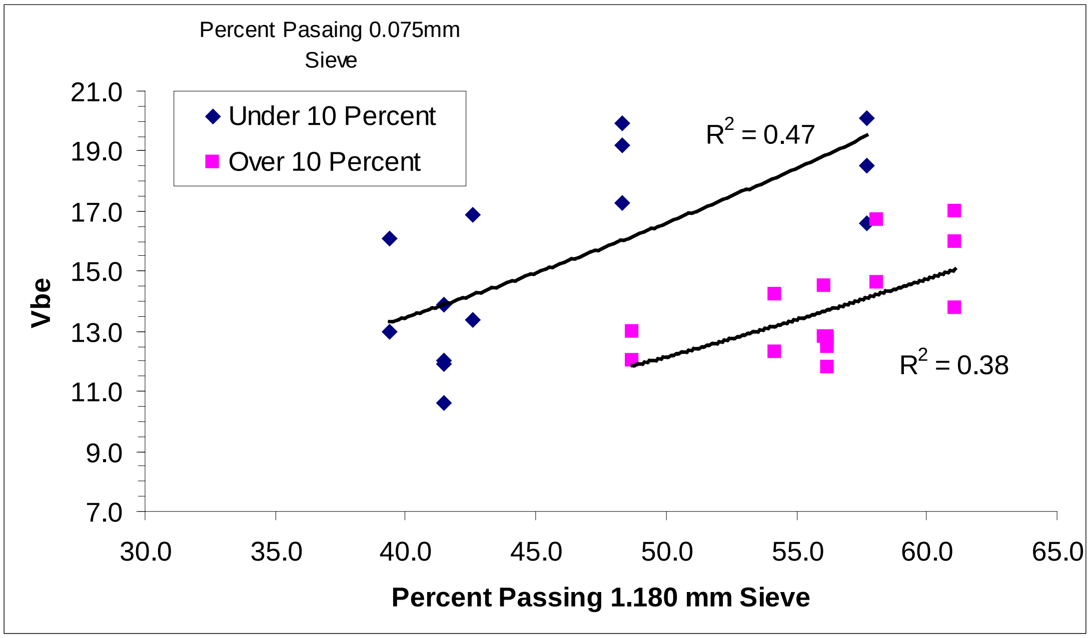

FIGURE 4.44 Vbe Versus Percent Passing 1.18 mm Sieve for Over and Under 10% Passing the 0.075 mm Sieve

4.4.2 Sand Equivalent

All aggregate blends in this study were well above the minimum specified limit for sand equivalence. The maximum SE result was 88 for the Florida blends; the minimum was 67 for the Minnesota and Alabama blends. The average for all the aggregate blends was 77. This study found no evidence to change the current AASHTO criteria for SE.

4.4.3 Dust-to-Binder Ratio

As discussed in Section 4.1.5, five mix designs fell outside of the current specified range for D:B ratio. It was determined from the relationship shown in Figure 4.36 that the current specified maximum of 2.0 appears to be reasonable. However, the minimum D:B ratio may be slightly low. Figure 4.45 shows a plot of the average and median rutting rates for mixtures sorted by ranges of D:B ratio. It can be seen that higher D:B ratios tend to increase rutting resistance for these mixtures. In Section 4.2.1, a maximum allowable MVT rut depth was determined to be 13.1 mm, which is equivalent to a 0.0016 mm/cycle rutting rate at 8,000 cycles. The average D:B ratio for mixtures with a rutting rate of less than 0.0016 mm/cycle was 1.8 with only one mixture under 1.5 D:B ratio. Based on these data, it is recommended that the minimum D:B ratio be changed to 1.0, and for the ESAL range of over 3.0 million ESALs, a minimum of 1.5 is recommended.

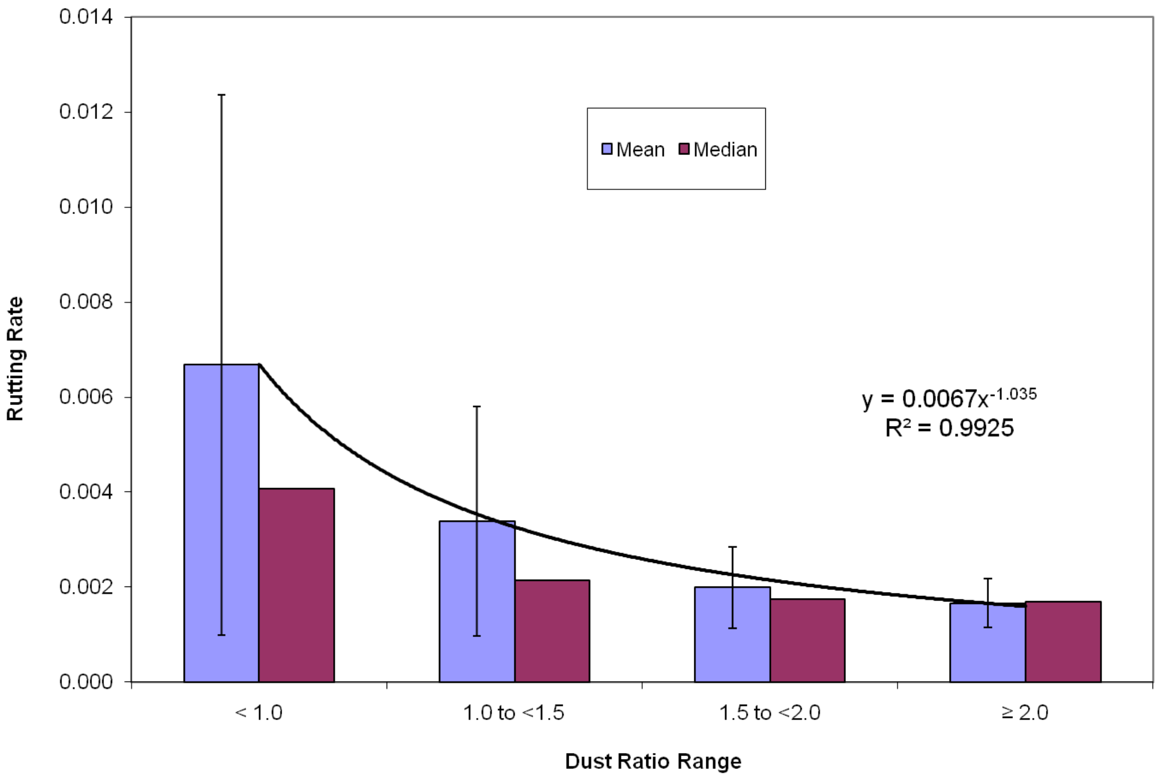

FIGURE 4.45 Rutting Rate Versus Dust-to-Binder Ratio

4.4.4 Fine Aggregate Angularity

Section 4.1.6 mentioned that there was no clear relationship between FAA and the volumetric properties of the mix designs prepared for this study. However, it was found that FAA did influence some of the results of the performance test. In Section 4.2.1 it was shown that an FAA over 45 reduced rutting at higher asphalt contents. Also, it was found in Section 4.2.4 that FAA over 45 may lower permeability. Based on these results, a FAA of over 45 may be appropriate for mixtures designed to higher ESAL ranges for both over and under a depth of 100 mm from the surface.

4.4.5 Percent of G_{mm} at N_{ini}

As mentioned in Section 4.1.4, only two mix designs failed to meet the most restrictive criteria for %G_{mm} @ N_{ini} (≤89%). These mixtures also had relatively high rutting rates at 0.004 mm/cycle, which may indicate that they would be unstable when subjected to traffic. At this time there is no recommendation on modifying the current %G_{mm} @ N_{ini} maximum. It was shown in Figures 4.13 and 4.15 that %G_{mm} @ N_{ini} for 6% design air void mixtures averaged 1.7% lower than mixtures designed at 4% V_a. However, if rutting rate is used as a measure of mixture stability, as shown in Figure 4.46, lowering the %G_{mm} @ N_{ini} maximum for 6% design air void mixtures cannot be justified.

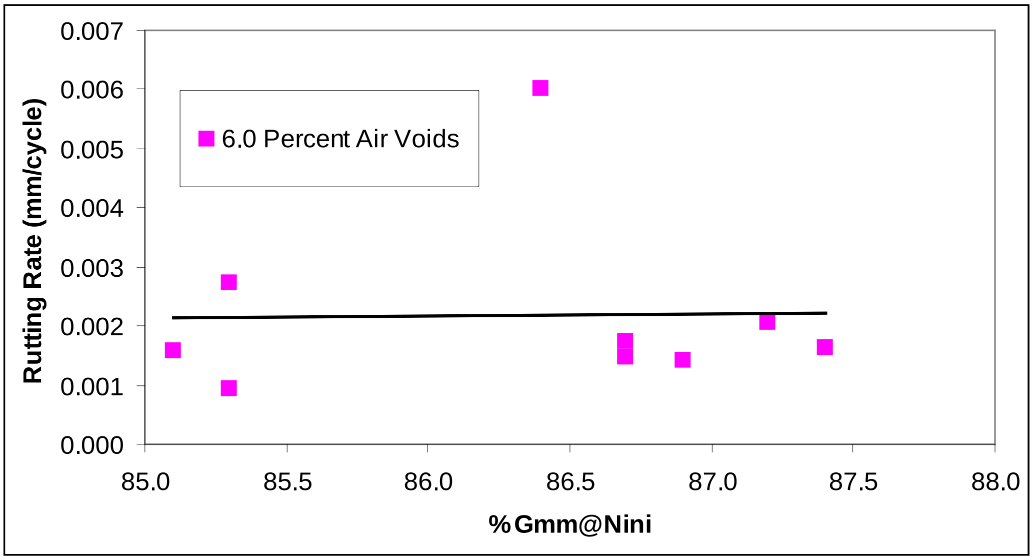

FIGURE 4.46 %G_{mm} @ N_{ini} Versus Rutting Rate for 6% Design Air Voids

4.4.6 Volumetric Requirements

Currently, 4% V_a is required by AASHTO M 323 for all NMAS mixtures. Results in Section 4.2 showed that 4.75 mm mixtures designed at 6% and 4% V_a can have satisfactory performance test results. Relationships shown in Section 4.2.1 showed that mixtures designed at 6% V_a have lower rutting than mixtures designed at 4% V_a.

Most of the mixtures tested in this study, using aggregates from a wide variety of sources, had high VMA and, therefore, high asphalt contents. Rutting test results showed that rutting was more a function of V_{be} than VMA. One way shown to reduce asphalt contents was to design these mixes at higher V_a. This will allow mix designers to use existing aggregate materials that may yield blends with a high VMA, yet it will provide more reasonable and practical asphalt contents. For this reason, a range of design V_a of 4% to 6% should be specified. Many of the mix designs in this study did not meet the current maximum VFA criteria of 78% for mix designs for over 0.3 million ESALs.

The three primary volumetric properties (V_a, VMA, and VFA) are interrelated, and their criteria assure that mixtures have sufficient asphalt for durability but not too much asphalt that may lead to instability. The current criteria for 4.75 mm mixtures are to design the mixtures with 4% V_a and a minimum VMA of 16%. The VFA criteria change depending on the traffic level. The current criteria can be restated as follows:

- Design Traffic < 0.3M ESALs: Vbe must be between 12.0% and 16.0%.
- Design Traffic 0.3M to < 3.0M ESALs: Vbe must be between 12.0% and 14.1%.
- Design Traffic 3.0M to < 30M ESALs: Vbe must be between 12.0% and 14.1%.

Specifying a Vbe range is a more straightforward approach, since the limits to assure durability and stability can be easily expressed with a single property. Based on Figure 4.35, a minimum Vbe of 11.5 was found to be appropriate based on the results of FE testing. Based on

Figure 4.27, a maximum Vbe of 13.5% is proposed for over 3.0 million design ESALs to limit the potential for rutting.

4.4.7 Summary of Recommendations for Revising 4.75 mm NMAS Mix Design Criteria

Based on the analyses of the laboratory experiments, the following recommendations were proposed for revising the current AASHTO criteria for 4.75 mm mix designs:

- The target V_a for selecting the design binder content should be changed to a 4.0% to 6.0% range. This will allow for a reduction in the design asphalt content for many 4.75 mm mixtures that have very high VMAs.
- VMA and VFA criteria should be replaced with minimum and maximum V_{be} requirements. This is a more sensible approach when a range of design V_a is used. For mixtures designed for projects less than 3 million design ESALs, a Vbe range of 12.0% to 15.0% is recommended. For mixtures designed for projects over 3 million ESALs, a minimum V_{be} of 11.5% and a maximum V_{be} of 13.5% are recommended. These limits were based on FE testing and MVT testing for the minimum and maximum Vbe, respectively.
- The maximum %G_{mm} @ N_{ini} requirement appears appropriate for both 4% and 6% design V_a. At this time, it is recommended that current G_{mm} @ N_{ini} criteria be maintained.
- For aggregate blends designed for over 0.3 million ESALs, a FAA of 45 is recommended for improved rut resistance.
- For 4.75 mm NMAS asphalt mixtures designed for under 3.0 million ESALs, the minimum dust-to-binder ratio should be increased slightly, from 0.9 to 1.0. For mixtures designed for over 3.0 million ESALs, a minimum dust-to-binder ratio of 1.5 is recommended.
- The current maximum dust-to-binder ratio of 2.0 is appropriate based on the results of FE testing. It is recommended that the maximum dust-to-binder ratio of 2.0 be maintained.
- No evidence was found that suggested the current SE criteria should be adjusted. At this time, it is recommended that minimum SE criteria be maintained.
- The current gradation limits on the 1.18 mm and 0.075 mm sieve should be adjusted. Limits placed on percent passing the 1.18 sieve should be 30%–55%. Results of laboratory rutting tests showed that mixtures with gradations near the current control point of 60% passing the 1.18 mm sieve had severe rutting. Limits placed on P-075 should be 6.0% to 13.0%.
- It is recommended that 4.75 mm mixtures contain no more than 15% natural sand with an FAA under 45% to improve rutting resistance and moisture damage resistance, and to maintain low permeability.

From these recommendations, a summary of proposed mix design criteria is given in Table 4.22.

TABLE 4.22 Proposed Design Criteria for 4.75 mm NMAS Superpave Mixtures

Design ESAL Range (Millions)	N_{des}	Minimum FAA	Minimum SE	Minimum Vbe	Maximum Vbe	$\%G_{mm}@N_{ini}$	D:B Ratio
<0.3	50	40	40	12.0	15.0	≤91.5	1.0 to 2.0
0.3 to ≤ 3.0	75	45	40	12.0	15.0	≤90.5	1.0 to 2.0
3.0 to ≤ 30	100	45	45	11.5	13.5	≤89.0	1.5 to 2.0

Gradation Limits		
Sieve Size	Max.	Min.
12.5 mm	---	100
9.5 mm	100	95
4.75 mm	100	90
1.18 mm	30	55
0.075 mm	13	6

Design V_a Range = 4.0% to 6.0%

CHAPTER 5 VALIDATION OF PROPOSED MIX CRITERIA BY PLANT PRODUCTION AND CONSTRUCTION

5.0 VALIDATION OF PROPOSED MIX CRITERIA BY PLANT PRODUCTION AND CONSTRUCTION

The proposal for this research stated that the 4.75 mm mix criteria would be validated by plant production and construction on a minimum of four projects. The field validation would examine the following issues:

- In-place densities after compaction
- Appropriate spread rates and lift thicknesses
- Workability of the mixture during construction
- Variability in mixture volumetric and aggregate properties during production and construction
- Friction of in-place mixtures
- Stability of the mixture during compaction
- Permeability of in-place mixtures

The selection of the projects would take into consideration the four SHRP climate zones and different levels of traffic. Each project was expected to have both a 4.75 mm mix and the normal mixture, so the evaluation of the 4.75 mm mix would be compared to a control mix. During the progress of the study it was difficult to select states specifically to fulfill the climate and traffic criteria. Pool-fund states were solicited to meet the minimum four-project criteria. The projects in this validation study were placed in Alabama, Missouri, Tennessee, and Minnesota. Only two of the four SHRP climate zones are represented: wet-freeze and wet-no freeze.

The production and placement of the 4.75 mm mixtures were generally independent maintenance projects, not tied to larger projects placing a control mixture. The agencies planned to compare the 4.75 mm mix field performance to other similar projects in the area, but similar projects were not specifically designated for comparison. The Minnesota project was also unique because it was a short research section on the MnRoad project, not a normal field construction project. The work plan for the validation spelled out specific tasks for the research team, as summarized below.

(1) Assist the DOT with mix design and project specifications
(2) During construction:

- Obtain plant-produced mix (both 4.75 mm mix and control mix)
- Field lab test for G_{mm} and lab G_{mb}
- Document production and construction
- Test field compacted 4.75 mm pavement for permeability and friction
- Obtain cores for density and lab testing

(3) Post-Construction Lab Testing:

- Measure density of the cores

- Measure lab permeability of the cores
- Extract and recover aggregate from plant-produced mix samples
- Perform rut test by MVT or APA

In general, the field validation accomplished the tasks assigned. NCAT provided consultation during the mix design process, but was not directly involved in laboratory mix designs for the mixtures placed. The NCAT laboratory prepared a number of preliminary mix designs for the Minnesota DOT, but the aggregates examined by NCAT were not used by the DOT. NCAT staff and the NCAT mobile laboratory were on site to collect sample and perform independent field density measurements. Permeability of the in-place 4.75 mm pavements was measured on the cores. Due to test equipment availability, complete surface friction measurements were obtained on only two projects. The post-construction testing in the NCAT laboratory included the test methods listed below:

- Lab test for G_{mm} – AASHTO T 209
- Lab test for G_{mb} – ASSHTO T166
- Lab procedure for binder content (by ignition) – AASHTO T308
- Lab test for gradation (washed) – AASHTO T 30
- Lab test for moisture susceptibility – AASHTO T 283
- Lab test for rutting (APA) – AASHTO TP 63
- Lab test for permeability – Florida Method FL 5-565
- Field test for friction using the Dynamic Friction Tester (DFT) – ASTM 1911
- Field test for friction using the Circular Texture Meter (CTM) – ASTM 2157

This section of the report includes a separate description of each of the four projects (Sections 5.1 through 5.4) and a summary analysis (Section 5.5) comparing the field validation projects to the proposed mix design criteria. The analysis in this section uses the test results generated by NCAT to examine the 4.75 mm mix design, production, and placement. The comparison of the individual mix designs to the evolving mix design standards is intended to identify differences, not to imply a level of compliance or future performance. Predicted impact on short-term and long-term performance is solely based on commonly accepted HMA principles.

5.1 Alabama Field Validation Project

5.1.1 Project Description

The ALDOT selected Wire Road just west of Auburn. The project number was STPNU-4423(200). The climate zone at this location is wet-no freeze. The traffic level for this section of Wire Road is 4700 AADT. Although there is insufficient information to estimate the design ESALs for this roadway, it is likely to be between 0.3 and 3.0 million ESALs. The plans called for placing the 4.75 mm HMA as a 0.75-inch surface lift. No conventional surface mix was placed as a control mix for performance comparison as part of this project.

The construction of the surface lift occurred from June 23 to June 26, 2006. The paving contractor was East Alabama Paving and the HMA production plant was located in Opelika. The mix was produced in a drum plant and paved with a conventional sequence of paving equipment.

5.1.2 Mix Design

The contractor prepared the mix design for the 4.75 mm HMA. The mix design was approved by the ALDOT on May 18, 2004. A copy of the mix design is included in the Appendix. Table 5.1 summarizes the approved mix design.

TABLE 5.1 Alabama Validation Project 4.75 mm Mix Design Summary

Mix Type	Proposed AASHTO Criteria	Alabama 424 (surface mixture)
Mix Size	4.75 mm NMAS	3/8-inch maximum aggregate size (4.75 mm NMAS)
Binder Type		PG 67 -22
Binder Content		6.8 %, P_{be} 6.53%,
Aggregate Blend		19% granite (#89 VMC Columbus, GA) 30% granite (M10 VMC Columbus, GA) 30% limestone (#8910 OCM Opelika) 20% man-sand (MM Pinkston Shorter) 1% baghouse fines
Target Gradation	30–55% passing 1.18 mm Sieve 6–13% passing 0.075 mm Sieve	47% passing 1.18 mm Sieve 6.0% passing 0.075 mm Sieve
Aggregate Properties	FAA 45 (min) SE 40 (min) Nat.Sand 15(max) if FAA<45	FAA = 46 Not reported N/A
Air Voids	4.0–6.0% (N_{des}=75 gyrations) 90.5 max (%G_{mm} @ N_{ini})	V_a=3.3% at N_{des} = 65 gyrations N_{ini} = 89% of G_{mm} at 7 gyrations
Volumetric Properties	V_{be} 12.0 to 15.0 VMA 16.0 min (note 1) VFA 65-78 (note 1) D:B ratio 1.0-2.0	V_{be} = 14.7 VMA = 18.0 VFA = 81.8 D:B ratio = 0.92
Moisture Susceptibility		TSR = 0.85 with no anti-strip treatment

Note 1 – current AASHTO criteria

5.1.3 Sampling and Testing Summary

The 4.75 mm HMA was produced and placed over two days of paving. NCAT staff were on the project site to collect loose mix samples at the plant and locate and cut cores. Test equipment to measure initial in-place friction was not available during the placement of the 4.75 mm HMA. The loose production mix samples were transported back to the NCAT laboratory and compacted to measure field lab volumetric properties. Additional samples were taken back to the NCAT laboratory for extracted material proportions, moisture susceptibility, rutting, and permeability testing. Table 5.2 summarizes the production quality control test results performed by NCAT on the plant-produced mixture.

TABLE 5.2 NCAT Field Sampling and Testing for the Alabama Validation Project

Test [no. of samples / no. of replicates]	Mix Design Target	Production QC
Mixture V_a – Lab (%G_{mm}@N_{des}) [4/3]	3.3%	2.2 – 3.4%
G_{mm} [4/2]	2.467	2.444 – 2.482
Binder Content – by Ignition Method (Pb) [4/1]	6.8%	6.7 – 7.1%
Gradation – washed from ignition samples [4/1]	47% pass 1.18 6.0% pass 0.075	50.7 – 55.3 8.3 – 11.0
Vbe [4/3] VMA [4/3] VFA [4/3] D:B ratio	14.7 18.0 81.8 0.92	14.4 – 16.2 17.8-18.7 80.7-88.1 1.24 – 1.83
Moisture Susceptibility (TSR) [1/1]	0.85	0.80
Rut Testing – by MVT [1/2]		13.0 mm
Lab Permeability from Field Cores (cm/sec) [3/1]		90×10^{-5}
In-place V_a – From Cores (note-1) [10/1]		11.7 avg, 9.5-13.2
Surface Friction – by DFT and CTM [0/0]		Note-2

Note-1 Cores were taken at 200-ft intervals from Station 157+50 to 175+50.
Note-2 DFT and CTM equipment were not available at the time of construction.

5.1.4 Analysis of Mix Design and Production Test Results

The binder content determined by the N_{des} 65 gyration mix design procedure was 6.8%. The P_{be} is 0.27% lower, indicating absorption by the aggregate. The range of binder contents of plant-produced mixture measured by the ignition oven was 6.7 to 7.1%. The consistency of the binder content is good and close to the mix design target.

The target gradation of the mix design was within the proposed control points for the 1.18 mm and 0.075 mm sieves. The gradation of the plant-produced mix was finer than the mix design target for both control sieves. The amount of aggregate passing the 1.18 mm sieve was 4% above the target. The amount of aggregate passing the 0.075 mm sieve was 3% above the target. The additional 3% dust content is very high. This production deviation from the mix design target is likely a factor in the observed low V_a for lab compaction.

The mix design V_a was 3.3%, which is below the recommended range of 4.0% to 6.0% for mix design. Lab voids from production mixture ranged from 1.9% to 3.7%, with an average of 2.8%. The mix design V_a and, more importantly, the lab V_a for production mix are lower than generally desired.

The proposed mix design criteria for Vbe is 12.0% –15.0% for mixes designed for 0.3 to 3.0 million ESALs. The mix design Vbe of 14.7% is within the proposed range. Several samples of the plant-produced mixes had Vbe results above the proposed mix design range of 15.0%. As noted in Phase I of this study, high V_{be} can lead to an increase in rutting.

The VMA of the plant-produced mixture compacted in the lab to 65 gyrations was 17.8% to 18.7%. High VMA values coupled with low V_a indicates that the mixture is supporting a high amount of asphalt binder. If the VMA does not collapse, the high binder content and low V_a should slow down binder aging and, therefore, improve long-term durability.

The VFA criteria in the current AASHTO mix design specification for projects with between 0.3 and 3.0 million design ESALs is 65%–78%. The Alabama mix design exceeded the upper limit of the range. This also raises concern of the potential for rutting.

The 10 cores had an average measured in-place void content of 11.7%, with a range of 9.5% to 13.2%. These in-place air void results are well above the accepted norm of 8.0% for field compacted density for most types of dense-graded mixes, and even above the expected range for 4.75 mm NMAS mixtures of 8% to 10%. Permeability was measured in the laboratory on cores taken from the test section. Core permeability was 90 x 10^{-5} cm/sec. Even with the relatively high in-place V_a, this mixture is impermeable. The measured lift thickness was 16.8 to 21.9 mm and is comparable to the intended thickness of 19 mm.

Moisture sensitivity of the plant-produced mix had a TSR of 0.80 based on IDT tests of laboratory-prepared specimens at 9.0% V_a. The average tensile strength was 114.5 for the unconditioned specimens and 91.9 psi for conditioned specimens. Although the field TSR was lower than the mix design TSR, it is probable that the lower computed TSR was impacted by the high V_a of the specimens.

Rutting of the plant-produced 4.75 mm HMA with the Mix Verification Tester resulted in a rut depth of 13 mm at 8,000 cycles. This level of rutting is consistent with the 12 to 17 mm MVT rutting presented in Section 4.2.1 from the laboratory evaluation.

5.2 Missouri Field Validation Project

5.2.1 Project Description

The Missouri DOT selected Dunklin County Route EE, near Kennett, from State Route 153 to State Route 25. The climate zone at this location is wet-freeze. The traffic level for this test section is 2,500 AADT and less than 5% trucks. The design traffic for this project was assumed to fall in the lowest traffic category: less than 0.3 million ESALs. The plans called for placing the 4.75 mm HMA as a 0.75-inch surface lift. No conventional surface mix was placed as a control for performance comparison as part of this project.

The construction of the surface lift occurred on August 16, 2007. The paving contractor was Apex Paving Company, and the HMA was produced at the Delta Asphalt Plant. The mix was produced in a drum plant and paved with a conventional sequence of paving equipment.

5.2.2 Mix Design

The contractor prepared the mix design, and the MoDOT approved the design on April 26, 2007. A copy of the mix design is included in the Appendix. Table 5.3 summarizes the approved mix design.

TABLE 5.3 Missouri Validation Project 4.75 mm Mix Design Summary

Mix Type	Proposed AASHTO Criteria	Missouri BP-3 Plant Mix Bituminous
Mix Size	4.75 mm NMAS	4.75 mm NMAS
Binder Type		PG 64-22
Binder Content		6.4%, Pbe=5.4%
Aggregate Blend		55% dolomite (LD Williamsville #1) 25% man-sand (MSGV BS&G Dexter) 20% nat-sand (NS1 BS&G Dexter, MO)
Target Gradation	30–55% passing 1.18 mm Sieve 6–13% passing 0.075 mm Sieve	48% passing 1.18 mm Sieve 7.6% passing 0.075 mm Sieve
Aggregate Properties	FAA = 40 (min) SE = 40 (min) Nat.Sand=15(max) if FAA<45	FAA = 45 Not reported N/A
Air Voids	4.0–6.0% (N_{des}=50 gyrations) 91.5 max (%G_{mm} @ N_{ini})	V_a = 4.0% at N_{des} = 50 gyrations Not reported
Volumetric Properties	V_{be} 12.0-15.0 VMA 16.0 min. (note 1) VFA 70-80 (note 1) D:B ratio 1.0-2.0	V_{be} = 12.2 VMA = 16.3 VFA = 75.2 D:B ratio = 1.4
Moisture Susceptibility		Not tested, generally not required for mixtures on low volume roads

Note 1 – current AASHTO criteria

5.2.3 Sampling and Testing Summary

The 4.75 mm HMA was produced during a single day of paving. NCAT staff was on the project site with a mobile laboratory to collect loose production mix samples at the plant, compact production mixture, measure field-lab volumetric properties, and locate and cut cores. The CTM was used to measure surface macrotexture, but the DFT equipment to measure initial in-place friction was not available during the placement of the 4.75 mm HMA. Samples were taken back to the NCAT laboratory for extracted material proportions, moisture susceptibility, rutting and permeability testing. Table 5.4 summarizes the production quality control test results performed by NCAT on the plant-produced mixture.

5.2.4 Analysis of Mix Design and Production Test Results

The design binder content for the 50-gyration mix design procedure was 6.4%. The P_{be} is 1.0% lower, indicating a high amount of absorption by the dolomite aggregate. The range of binder contents of the plant-produced mixture measured by the ignition oven was 6.83% to 7.42%. The plant-produced binder contents were consistently about 0.5% above the mix design target.

The target gradation of the mix design was within proposed control points for the 1.18 mm and 0.075 mm sieves. The gradation of the plant-produced mix was finer than the mix design target for both control sieves. The NCAT-extracted gradations from production mix showed a wide production range, 49% to 58% passing the 1.18 mm sieve. The range of NCAT results for P-075 were very consistent (11.8–12.3), but was 4.5% above the mix design target. However, the dust contents of the plant mix were still within the proposed mix design gradation control point. The large difference between the mix design dust content and the results from production mix tests obtained by NCAT can be partly attributed to differences between dry gradations and washed gradations. NCAT used washed gradation analyses, whereas MoDOT uses dry gradations. Higher dust contents may also be due to breakdown of the gradation in the plant and breakdown in the ignition oven with the dolomite aggregate.

TABLE 5.4 NCAT Field Sampling and Testing for the Missouri Validation Project

Test [no. of samples / no. of replicates]	Mix Design Target	Production QC
Mixture V_a – Lab (%G_{mm}@N_{des}) [3/6]	4.0%	3.6 – 4.9%
G_{mm} [3/2]	2.456	2.453 – 2.460
Binder Content – by Ignition Method (Pb) [3/1]	6.4%	6.8 – 7.4%
Gradation – Washed from Ignition Samples [3/2]	48 pass 1.18 7.6 pass 0.075	49 - 58 11.8 – 12.3
V_{be} VMA VFA D:B ratio	12.2 16.3 75.2 1.4	12.5 – 13.3 16.6 – 17.7 74.3 – 76.5 2.1 – 2.2
Moisture Susceptibility (TSR) [3/1]	Not tested	0.66 – 0.74
Rut Testing – by APA [3/4,6 note-2]		6.7 mm
Lab Permeability from Field Cores (cm/sec) [10/1]		40×10^{-5}
In-place V_a – from Cores [10/1]		10.1 avg, 9.2 – 11.9
Surface Friction – by DFT(note-1) and CTM [10/2]		MPD 0.17- 0.22 mm

Note-1 The DFT was not available for this project.
Note-2 There were 4 replicates for sample 1 and 6 replicates for samples 2 and 3.

For the MoDOT 4.75 mm HMA, the lab voids from production mix ranged from 3.6% to 4.9%. This range is within a normal specification production tolerance of +/- 1.0%. Field notes indicate that the aggregate proportions were changed to increase the man-sand and decrease the natural sand after the contractor’s initial production mix QC sample measured 3.3% V_a. Based on the range of lab V_a on production mix measured by NCAT, the aggregate proportion change improved the aggregate structure of the mixture to maintain the target 4.0% voids.

The mix design V_{be} of 12.2% and the Vbe results for the plant-produced mixture were within the proposed specification range of 12.0%–15.0%. The mix was designed with a VMA of 16.3%, and the VMA from plant-produced mixture was 16.6% to 17.7%. The higher VMA for the production mix compared to the mix design was sufficient to accommodate the additional asphalt binder and mineral filler without reducing the V_a. The VFA of the mix design and the plant-produced mix were also within the current mix design specification range of 65–78.

The ten cores had an average measured in-place void content of 10.1%, with a range of 9.2% to 11.9%. Permeability tests on the cores resulted in an average permeability of 40 x 10^{-5} cm/sec, which confirmed that the 4.75 mm layer was impermeable. The measured lift thickness was 19.4 to 24.5 mm and is at or above the intended thickness of 19 mm.

A moisture sensitivity test was not reported for the mix design. Moisture sensitivity of the plant-produced mix yielded an average TSR of 0.70 with a range of 0.66 to 0.74, based on IDT tests of laboratory-prepared specimens ranging from 6.5% to 7.6% V_a. The TSR value was based on an average conditioned tensile strength of 102 psi and an average unconditioned tensile strength of 145 psi.

Rutting of the plant-produced mix compacted in the laboratory to 4.0% V_a and tested with the APA resulted in an average rut depth of 6.7 mm at 8,000 cycles. Based on the correlation of APA and MVT results presented in Figure 3.3, this result would be approximately 11 mm in the MVT, which is good compared to the 12–17 mm range for MVT results from the laboratory evaluation.

The CTM was used to measure the macrotexture of the 4.75 mm HMA surface after compaction. Readings were taken at each of the core locations. The mean profile depth (MPD) measured from 0.17 to 0.22 mm. This texture measure is normal for a fine-graded dense HMA with a small NMAS. The DFT was not in service at the time of this construction. No DFT measurements were taken.

5.3 Tennessee Field Validation Project

5.3.1 Project Description

The Tennessee DOT selected State Route SR 25 in Robertson County. The project number was 74000-4200-404. The project began at log mile 4.00 and ended at log mile 6.95. The climate zone at this location is wet-no freeze. The traffic level for this section of SR 25 was 1620 AADT with 18% trucks. The design traffic for this project was assumed to fall in the second-lowest traffic category: 0.3 to 3.0 million ESALs. The posted speed limit was 55 mph. Predominant distress in the existing HMA pavement was transverse cracking at 10 to 40-ft spacing. The plans called for placing the 4.75 mm HMA as a 0.75-inch surface lift. Two 4.75 mm mixes were placed, a virgin mix was placed in the east-bound lanes, and a mix with 15% reclaimed asphalt pavement (RAP) was placed in the west-bound lanes. No conventional surface mix was placed as a control mix for performance comparison as part of this project.

The construction of the surface lift occurred on June 17–18, 2007. The paving contractor was Lojac Inc., and the HMA production plant was located north of Springfield. The mix was produced in a batch plant (3-ton capacity). Five passes of a steel roller was determined to be the appropriate rolling pattern from a test strip. Mixture was placed above 300°F. A 500-ft test section was identified 4,300 feet from the beginning of the project.

5.3.2 Mix Design

The TnDOT prepared the virgin mix design and 15% RAP mix design for the 4.75 mm HMA using the Marshall method with 75 blows. Note that TnDOT continues to use the Marshall method to design all asphalt mixtures. In April 2008, the TnDOT lab started with a mix of limestone screenings that produced a mixture with very high VMA (>20%). The final mix design for the virgin mix increased the natural sand to reduce the VMA. The final mix designs were completed in June 2007. A copy of each mix design is included in the Appendix. Table 5.5 summarizes the approved mix design for the virgin mix. Table 5.6 summarizes the approved mix design for the mix with RAP.

TABLE 5.5 Tennessee Validation Project 4.75 mm Virgin Mix Design Summary

Mix Type	Proposed AASHTO Criteria	ACS-HM (surface mixture)
Mix Size	4.75 mm NMAS	4.75 mm NMAS
Binder Type		PG 64-22
Binder Content		6.8 %
Aggregate Blend	Nat.Sand=15% max. if FAA<45	75% screenings (#10-hard Aggr USA) 10% screenings (#10-soft Aggr USA) 15% natural-sand (Ingram Mtls)
Target Gradation	30%–55% passing 1.18 mm Sieve 6%–13% passing 0.075 mm Sieve	58% passing 1.18 mm Sieve 12.1% passing 0.075 mm Sieve
Aggregate Properties	FAA = 45 (min) SE = 40 (min)	Not reported
Air Voids	4.0%–6.0% (N_{des}=75 gyrations) 90.5 max (%G_{mm} @ N_{ini})	V_a=4.0% at 75-blow Marshall
Volumetric Properties	V_{be} 12.0% –15.0% VMA 16.0 (note 1) VFA 65–78 (note 1) D:B ratio 1.0-2.0	V_{be}=15.1 VMA=19.1 VFA = 79.0 D:B ratio=1.8
Moisture Susceptibility		Not tested, not required based on asphalt binder content

Note 1 – current AASHTO criteria

5.3.3 Sampling and Testing Summary

The two 4.75 mm HMA mixtures were produced and placed June 17–18, 2008. NCAT staff was on the project site with a mobile laboratory to collect loose mix samples at the plant, compact production mixture to measure field lab volumetric properties, locate and cut cores, and measure in-place friction. Samples were taken back to the NCAT laboratory for extracted material proportions, moisture susceptibility, rutting, and permeability testing. Table 5.7 and Table 5.8 summarize the production quality control test results performed by NCAT on each plant-produced mixture.

TABLE 5.6 Tennessee Validation Project 4.75 mm RAP Mix Design Summary

Mix Type	Proposed AASHTO Criteria	ACS-HM (surface mixture with RAP)
Mix Size	4.75 mm NMAS	4.75 mm NMAS
Binder Type		PG 64-22
Binder Content		6.8 %
Aggregate Blend	Nat.Sand=15% max. if FAA<45	60% screenings (#10-hard Aggr USA) 10% screenings (#10-soft Aggr USA) 15% natural-sand (Ingram Mtls) 15% RAP (pass 5/16 Lojac)
Target Gradation	30%–55% passing 1.18 mm Sieve 6%–13% passing 0.075 mm Sieve	56% passing 1.18 mm Sieve 12.1% passing 0.075 mm Sieve
Aggregate Properties	FAA = 45 (min) SE = 40 (min)	Not reported
Air Voids	4.0%–6.0% (N_{des}=75 gyrations) 90.5 max (%G_{mm} @ N_{ini})	V_a=4.0% at 75-blow Marshall
Volumetric Properties	V_{be} 12.0-15.0%% VMA 16.0 % min.(note 1) VFA 65-78 (note 1) D:B ratio 1.0-2.0	V_{be} = 15.0 VMA = 19.0 VFA = 79% D:B ratio=1.8
Moisture Susceptibility		Not tested, not required based on asphalt binder content

Note 1 – current AASHTO criteria

TABLE 5.7 NCAT Field Sampling and Testing for the Tennessee Validation Project (Virgin Mix)

Test [no. of samples / no. of replicates]	Mix Design Target Virgin Mix	Production QC (note-1)
Mixture V_a–Lab(%G_{mm}@N_{des})[3/6]	4.0	4.6 – 5.9
G_{mm} [3/1]	2.389	2.398 – 2.407
Binder Content – by Ignition Method (Pb) [3/2]	6.8	7.5 – 7.7
Gradation – Washed from Ignition Samples [3/2]	58 pass 1.18 12.1 pass 0.075	50 – 51 11.7 – 13.4
Vbe VMA VFA D:B ratio	15.1 19.1 79.0 1.8	14.9 – 15.3 19.9 – 20.5 72.8 – 75.8 1.8 – 1.9

Moisture Susceptibility (TSR) [3/1]	Not tested	0.68 – 0.75
Rut Testing – by APA (note-2) [3/2]		4.5 mm
Lab Permeability from Field Cores (cm/sec) [8/1]		160 x 10^{-5}
In-place V_a – from Cores [8/1]		11.9avg, 7.5 – 14.2
Surface Friction – by DFT and CTM [8/3,2 (note-3)]		DFT$_{20}$ 0.25 - 0.35 MPD 0.16 – 0.33 mm

Note-1 NCAT lab density results based on N_{des} at 125 gyrations to match 4% V_a.
Note-2 Tested at design V_a and at 7% voids.
Note-3 Three replicates for DFT and two replicates for CTM.

TABLE 5.8 NCAT Field Sampling and Testing for the Tennessee Validation Project (15% RAP Mix)

Test [no. of samples / no. of replicates]	Mix Design Target 15% RAP	Production QC (note-1)
Mixture V_a–Lab(%G_{mm}@N_{des}) [3/6]	4.0	3.5 – 4.5
G_{mm} [3/1]	2.380	2.393 – 2.411
Binder Content – by Ignition Method (Pb) [3/2]	6.8	7.2 – 7.3
Gradation – Washed from Ignition Method [3/2]	56 pass 1.18 12.1 pass 0.075	52 – 54 13.2 – 14.1
Vbe VMAVFA D:B ratio	15.0 19.079.0 1.8	14.3 – 15.0 18.4 – 19.077.7 – 79.7 2.0 – 2.2
Moisture Susceptibility (TSR) [3/1]	Not tested	0.67 – 0.79
Rut Testing – by APA (note-2) [3/2]		3.3 mm
Lab Permeability from Field Cores (cm/sec) [8/1]		140 x 10^{-5}
In-place V_a – from Cores [8/1] (note 4)		11.7 avg, 10.7 – 12.7
Surface Friction – by DFT and CTM [8/3,2 (note-3)]		DFT$_{20}$ 0.28 – 0.33 MPD 0.19 – 0.33 mm

Note-1 NCAT lab density results based on N_{des} at 125 gyrations to match 4% V_a.
Note-2 Tested at design V_a and at 7% voids.
Note-3 Three replicates for DFT and two replicates for CTM.
Note-4 One replicate measured Va=20.1 and was not included in the analysis.

5.3.4 Analysis of Mix Design and Production Test Results

5.3.4.1 Virgin Aggregate Mixture. The binder content determined by the 75-blow Marshall mix design procedure was 6.8%. The range of binder contents of the plant-produced virgin mixture measured by the ignition oven was 7.5% to 7.7%. The plant-produced binder contents were consistently 0.7% to 0.9% above the target established by the mix design.

The mix design gradation of the virgin mix design was finer than the recommended control points on the 1.18 mm sieve. However, the plant-produced virgin mix was coarser than the mix design target for the 1.18 mm control sieve with a tight range of 50% to 51% passing.

The amount of aggregate passing the 0.075 mm sieve was slightly above the mix design target, at an average of 12.7% with a good range of 11.7% to 13.4% passing.

For the TnDOT 4.75 mm virgin aggregate mixture, the mix design V_a was 4.0% using a 75-blow Marshall design procedure. The NCAT lab attempted to match the volumetric properties of the mix design, but even at 125 gyrations with the SGC, the lab-compacted V_a ranged from 4.6% to 5.9%. The V_{be} for the plant-produced mix ranged from 14.9% to 15.3%, which was at the high end of the recommended range, but consistent with the design target of 15.0%. Likewise, VMA results and VFA were also very high. Although these volumetric properties may help this mixture resist reflection cracking from the underlying pavement, they raised concern about the potential for rutting of the mixture. However, APA test results were only 4.9 mm for specimens at 5.1% V_a and 4.0 mm for specimens at 7.0% V_a. These APA results would correspond to about 6 to 8 mm rutting in the MVT, which are very good compared to the results from the laboratory phase of this study.

The eight cores cut after field compaction had an average measured in-place V_a of 11.9%, with a range of in-place voids from 7.5% to 14.2%. Six of the cores measured at or above 12.0%. Laboratory permeability measurements on cores were as high as 205 x 10^{-5} cm/sec. The range in permeability for the cores on this project was consistent with the range of in-place V_a. Two cores with lower voids had lower permeability. The pavement may be more susceptible to permeability related distress in areas with more than 12.0% in-place V_a. The measured lift thickness of the virgin mix cores varied greatly from 18.6 to 31.8 mm and met or exceeded the intended thickness of 19 mm. The variation in lift thickness implies that the existing pavement surface was irregular and may account for the wide range of measured in-place V_a.

Moisture sensitivity of the plant-produced virgin mix yielded an average TSR of 0.71 with a range of 0.68 to 0.75 based on IDT tests of laboratory-prepared specimens at 7.0% to 7.3% V_a. The TSR value was based on an average conditioned tensile strength of 157 psi and an average unconditioned tensile strength of 220 psi. No anti-strip treatment was required for this mixture based on TnDOT HMA criteria.

Friction characteristics of the virgin 4.75 mm HMA surface were measured with the DFT and CTM tests. The dynamic friction, based on the measured DFT_{20} values, ranged from 0.25 to 0.35 for the eight test locations. The MPD measured with the CTM ranged from 0.16 to 0.33 mm, which is in the typical range for fine-graded HMA with small NMAS aggregates. The DFT measurements reflect initial post-construction surface conditions. The high asphalt binder film on the surface creates lower friction results. Once the binder film is worn off by traffic, friction characteristics typically improve.

5.3.4.2 Mixture with 15% RAP. The binder content determined by the 75-blow Marshall mix design procedure was 6.8%. The P_{be} was computed to 6.8%, so the combined virgin and RAP aggregates are not absorptive. The range of binder contents of the plant-produced mixture with 15% RAP measured by the ignition oven was 7.2% to 7.3%, consistently about 0.4% above the target established by the mix design.

The target gradation of the mix design with RAP was close to the recommended upper control points. The percent passing the 1.18 mm sieve was just above the upper control point, and the amount passing the 0.075 mm sieve was near the upper control point. Like the virgin Tennessee field mix, the gradation of the plant-produced mix with RAP was coarser than the mix design target for the 1.18 mm control sieve and finer than the mix design target for the 0.075 mm control sieve. The amount of aggregate passing the 1.18 mm sieve was 3% below the target, with a tight range of 52% to 54% passing. The amount of aggregate passing the 0.075 mm sieve was above the target at an average of 13.7%, with a range of 13.2% to 14.1% passing.

For the TnDOT 4.75 mm mixture with RAP, the mix design V_a was 4.0% using a 75-blow Marshall design procedure. Field results tested by NCAT were based on 125 gyrations. With this compactive effort, the V_a ranged from 3.5% to 4.5%, which was within a normal production specification tolerance of +/-1.0%. The V_{be} of the plant-produced mix with RAP ranged from 14.3% to 15.0%. VMA ranged from 18.4% to 19.0%; VFA ranged from 78 to 80%. At face value, like the virgin TnDOT mix, these results for Vbe, VMA, and VFA are high and raise some concerns. It is not possible to precisely determine how these mix properties may have changed if the mix were compacted to 75 gyrations and designed for 6.0% V_a, but a rough estimate is that VMA would have been in the range of 21% to 21.5%, asphalt content would decrease by about 0.9%, and VFA would probably be in the range of 71 to 72. The V_{be} range would likely be high, in the range of 15.0% to 15.6%.

Rutting of the plant-produced 4.75 mm HMA with RAP from the APA test resulted in rut depths of 2.6 mm for specimens at 4.0% V_a and 3.9 mm for specimens at 7.0% V_a. Based on Figure 3.3, these APA results would correlate to about 3.0 to 5.5 mm, respectively, in the MVT. Therefore, despite the high V_{be} results, the 4.75 mm mixture with RAP appears to be very rut resistant. This may be partly attributed to added stiffness from the RAP binder.

The eight cores cut after field compaction had an average in-place V_a of 11.7% and a range from 10.7% to 20.1%. Seven of the cores measured between 10.7% and 12.7%. One core with 20.1% V_a was not included in the computed average. Permeability of the seven cores taken from the test section was measured in the laboratory. Core permeability results ranged from 95 x 10^{-5} to 190 x 10^{-5} cm/sec, similar to the results for virgin TnDOT mix. The measured lift thickness of the cores with RAP varied greatly, from 19.2 to 31.2 mm and met or exceeded the intended thickness of 19 mm. There was no correlation between core thickness and core density.

Moisture sensitivity of the plant-produced mix with RAP yielded an average TSR of 0.72 with a range of 0.67 to 0.79 based on specimens with 6.7% to 7.2% V_a. The average conditioned tensile strength was 173 psi, and the average unconditioned tensile strength was 239 psi.

Friction characteristics of the 4.75 mm HMA with RAP surface were measured with the DFT and CTM tests. The dynamic friction based on the measured DFT_{20} values ranged from 0.28 to 0.33 for the eight test locations. The MPD measured with the CTM ranged from 0.19 to 0.33 mm. The DFT measurements reflect initial post-construction surface conditions. The asphalt binder film on the surface creates lower friction results. Once the binder film is worn off by traffic, the friction characteristics typically improve.

5.3.4.3 Virgin Mix – RAP Mix Comparison. Both mixtures were produced with 0.4% to 0.5% more asphalt binder than targeted in the mix design. The dust content in the mix with RAP (13.7%) was higher than the 12.7% for the virgin mix. Lab voids from plant-produced mix were lower for the mix with RAP (4.0%) compared to the virgin mix (5.1%). The higher dust content in the mix with RAP may account for the lower lab-compacted air void results. Overall, the in-place V_a were slightly better for the mix with RAP, but the results for both mixtures were high and consequently had high permeability results, which raises concern. Both mixtures had poor moisture susceptibility results, but good rutting resistance results.

5.4 Minnesota Field Validation Project

5.4.1 Project Description

The Minnesota DOT selected Section 6 of the MnRoad mainline experiment in the west-bound direction of I-94 to evaluate their 4.75 mm HMA mixture. Section 6 is 500 ft long and includes both the inside and outside lanes. The climate zone for this location is wet-freeze. The traffic level is monitored by the MnRoad research plan weigh-in-motion sensors, which typically log 600,000 ESALs in the driving lane annually. The research section would be designed for approximately 12 to 15 million ESALs for a 20-year design period. Therefore, this project is in the highest category for 4.75 mm mix designs (3.0 to 30 million ESAL). The posted speed for the section is 70 mph. The plans called for placing two 1-in lifts of 4.75 mm mixture over 5 inches of jointed-doweled PCC with a 15-ft joint spacing. The actual construction placed a single 2-inch lift of the 4.75 mm mixture. There was no specific control mix for comparison. The 4.75 mm surface is one of multiple sites along the research project.

The construction of the 4.75 mm test section occurred on October 30, 2008. The mix was produced in a drum plant and paved under tight experimental QC control.

5.4.2 Mix Design

In June and July 2007, NCAT worked with MnDOT staff to prepare mix designs with taconite tailings and man-sand. The source of taconite tailings changed before the test sections were ready for construction the following year, so the mix designs were not used. The MnDOT prepared another mix design in July 2008. A copy of the mix design is included in the Appendix. Table 5.9 summarizes the approved mix design.

5.4.3 Sampling and Testing Summary

The MnRoad 4.75 mm HMA was produced on one day of paving. NCAT staff was on the project site with a mobile laboratory to collect loose mix samples, compact production mixture to measure field-lab volumetric properties, obtain cores, and measure surface friction. The NCAT staff coordinated the 4.75 mm field sampling and testing requirements with the MnRoad experiment plan. Samples were taken back to the NCAT laboratory for extracted material properties, moisture susceptibility, rutting, and permeability testing. Table 5.10 summarizes the production quality control test results performed by NCAT on the plant-produced mixture.

TABLE 5.9 Minnesota Validation Project 4.75 mm Mix Design Summary

Mix Type	Proposed AASHTO Criteria	MnDOT SPWEB440F Special
Mix Size	4.75 mm NMAS	4.75 mm NMAS
Binder Type		PG 64 -34 (polymer modified)
Binder Content		7.4%, Pbe=6.9
Aggregate Blend		55% Taconite tailings (Mintac) 10% Taconite tailings (Ispat) 35% Man-sand (Loken)
Target Gradation	30%–55% passing 1.18 mm Sieve 6-13% passing 0.075 mm Sieve	51% passing 1.18 mm Sieve 7.7% passing 0.075 mm Sieve
Aggregate Properties	FAA = 45 (min) SE = 45 (min) Nat.Sand=15(max) if FAA<45	FAA = 47 SE = 83 N/A
Air Voids	4.0%–6.0% (N_{des}=75 gyrations) 89.0 max (%G_{mm} @ N_{ini})	V_a=3.9% at N_{des} =75 gyrations Not reported
Volumetric Properties	V_{be} 11.5-13.5 VMA 16.0 min. (note 1) VFA 65-78 (note 1) D:B ratio 1.5-2.0	V_{be}=16.4 VMA=20.3 VFA 80.8 D:B ratio =1.1
Moisture Susceptibility		TSR=0.82 @ V_a = 9.0%

Note 1 – current AASHTO criteria

TABLE 5.10 NCAT Field Sampling and Testing for the Minnesota Validation Project

Test [no. of samples / no. of replicates]	Mix Design Target	Production QC
Mixture V_a – Lab (%G_{mm}@N_{des}) [3/2]	3.9	2.9 – 3.9
G_{mm} [3/1]	2.551	2.532 – 2.546
Binder Content –by ignition method (Pb) [3/note-1]	7.4%	8.8 – 9.1
Gradation–washed from ignition samples [3/note 1]	51 pass 1.18 7.7 pass 0.075	54 – 60 8.5 – 9.9
V_{be} VMAVFA D:B ratio	16.4 20.380.8 1.1	17.9 – 18.5 21.0 – 22.182.7 – 85.4 1.1 – 1.3
Moisture Susceptibility (TSR) [3/1]	0.82 (9%)	0.68 – 0.82
Rut Testing – by APA [3/note-2]		5.3 mm
Lab Permeability from field cores (cm/sec) [6/1]		5 x 10^{-5}
In-place V_a – from cores [6/1]		6.6 avg, 4.9 – 8.0
Surface Friction – by DFT and CTM (note-3) [10/3]		DFT_{20} 0.34 – 0.49 MPD 0.13 – 0.18 mm

Note-1 Four replicates for samples 1 & 2 and 2 replicates for sample 3

Note-2 Two replicates at design V_a and 2 replicates at 7% V_a
Note-3 Five tests randomly spaced in each lane

5.4.4 Analysis of Mix Design and Production Test Results

The binder content determined by the N_{des} 75-gyration mix design procedure was 7.4%. The P_{be} is 0.5% lower, indicating a moderate amount of absorption in the combined aggregate. The range of binder contents of the plant-produced mixture measured by the ignition oven was 8.8% to 9.1%, consistently more than 1.0% above the mix design target.

The target gradation of the mix design was within the proposed control points for the 1.18 mm and 0.075 mm sieves. The gradation of the plant-produced mix was finer than the mix design target for both control sieves. The amount of aggregate passing the 1.18 mm sieve during production was 5% above the target. The amount of aggregate passing the 0.075 mm sieve was 2% above the target.

For the MnDOT 4.75 mm mixture, the final mix design V_a was 3.9%. The V_a from plant-produced, lab-compacted mixture ranged from 2.9% to 3.9%, with an average of 3.6%. The increase in binder and dust contents are probable factors in the lower V_a measured in the production mixture.

The mix design V_{be} was also 3% above the proposed range of 11.5% to 13.5%. During production, the V_{be} results were much higher, ranging from 17.9% to 18.5%. The mixture was designed with a high VMA of 20.3%. The range of VMA from the plant-produced mixture was even higher—21.0 to 22.1%. The mix design VFA was 3% above the current AASHTO mix design specification range of 65 to 78. During production, the VFA further increased due to the higher asphalt binder content and VMA.

Rutting of the plant-produced 4.75 mm HMA tested with the APA test resulted in an average rut depth of 4.6 mm for specimens at 3.4% V_a and 6.1 mm for specimens at 7.1% V_a. Based on the correlation shown in Figure 3.3, these APA results would be approximately 6.5 mm and 9.5 mm, respectively, for the MVT. The Minnesota 4.75 mm mix rutting performance is better than expected for the high V_{be} results. The good rutting resistance may be partly attributed to the use of the polymer modified PG 64-34 binder and the very angular, rough textured taconite aggregate.

Three cores were taken in each lane. The six cores had an average in-place V_a of 6.6%, with a range of 4.9% to 8.0%. These in-place density results are very good for a field-compacted 4.75 mm mixture. As a result, the permeability results for the six cores were very low, less than 10×10^{-5} cm/sec. The total measured lift thickness for the one 2-inch lift ranged from 2.62 to 2.71 inches, well above the intended thickness.

Moisture sensitivity tests on the plant-produced mix yielded an average TSR of 0.76 with a range of 0.68 to 0.82 based on IDT tests of laboratory-prepared specimens at 7.1% to 7.3% V_a. The average conditioned IDT strength was 78 psi, and the average unconditioned strength was 103 psi. The field TSR values are lower than the reported mix design TSR of 0.82 for 9% V_a.

Friction characteristics of the 4.75 mm HMA surface were measured with the DFT and CTM tests. The dynamic friction, based on the measured DFT_{20} values, ranged from 0.34 to 0.49 for the 10 test locations. The MPD measured with the CTM ranged from a 0.13 to 0.18 mm. The DFT measurements are better than the measurements at the Tennessee project and reflect the angular shape of the taconite aggregate tailings. The level of surface texture (MPD) is normal for fine-graded HMA with small NMAS aggregates.

5.5 Summary Analysis of Field Validation Results

This section summarizes the mix designs and field results from the validation projects to determine how they complied with proposed mix design criteria. The summary analysis also determines if any deviations from the proposed mix design criteria are common across the projects and warrant a re-evaluation of the criteria. Table 5.11 summarizes the mix designs for the projects.

TABLE 5.11 Summary of Mix Designs for Validation Projects

Mix Design Property	Alabama	Missouri	Tennessee	Minnesota
Mix Level	65 gyrations	50 gyrations	75-blow	75 gyrations
Design Traffic (estimate)	1M ESAL	0.3M ESAL	1M ESAL	12M ESAL
Binder Content (% of mix)	6.8	6.4	6.8	7.4
Effective Binder Content	6.5	5.4	6.8	6.9
1.18 mm Target (% passing)	47	48	58	51
0.075 mm Target (% passing)	6.0	7.6	12.1	7.7
V_a (%)	3.3	4.0	4.0	3.9
VMA (%)	18.0	16.3	18.0	20.3
V_{be} (%)	14.7	12.2	15.1	16.4
VFA (%)	81.8	75.2	79.0	80.8
D:B ratio	0.9	1.4	1.8	1.1
FT (microns)	9.3	7.1	6.5	8.5
TSR	0.85	Not measured	Not measured	0.82

5.5.1 Compliance with Mix Design Standards

Mixes for two of the four field projects were designed by the agencies to compaction levels different from the AASHTO mix design standards. Alabama's use of 65 gyrations for N_{des} fits within the range of the standards, but the mixture only measured 3.3% V_a. The State of Tennessee used the Marshall 75-blow mix design standard. To match the Tennessee mix design, the NCAT laboratory applied 125 gyrations (N_{des}) to achieve the target 4.0% V_a.

Gradations – Figure 5.1 shows the average gradation during production for each project. All four mixtures were fine-graded mixtures. The Alabama mixture did not comply with the NMAS criteria. The mixture from Missouri was substantially finer than the other mixes between the 1.18 mm and the 0.075 mm sieves. All four validation mixes were designed and produced near the upper control point on the 1.18 mm sieve. Each of the mix designs used a minimum of three aggregate stockpiles to blend into the combined gradation. The very fine gradations of these mixtures are a common, but not exclusive, characteristic for most 4.75 mm mixtures. The very fine gradations offer some advantages, such as the ability to place the mixtures in very thin applications with excellent smoothness, suitable for feathering, provide good joints, and maintain low permeability. During production of the mixtures, gradations were generally even finer than the mix designs. Three of the plant-produced mixes were an average of 5% finer than the mix design target on the 1.18 mm sieve. All four projects met the gradation criteria for aggregate passing the 0.075 mm sieve, but three of mixtures had significant increases in the amount of dust during production.

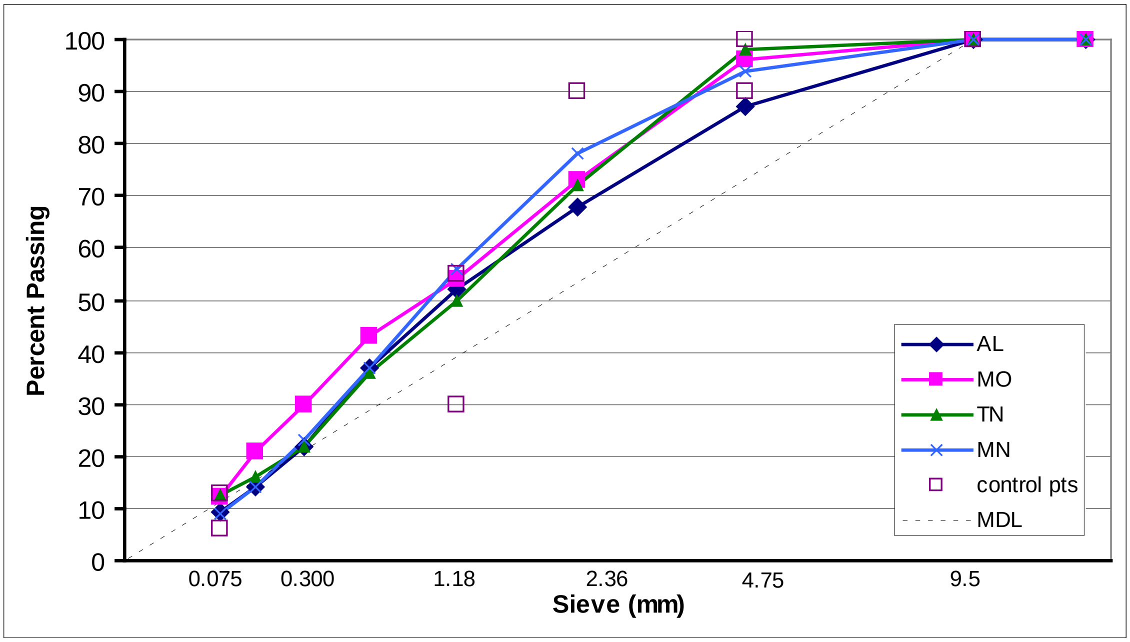

FIGURE 5.1 Average Plant-Production Gradation for Field Validation Projects

Table 5.12 summarizes the properties for the plant-produced mixtures. Results for each property are summarized by showing the average followed by the range. Table 5.13 identifies which measured properties from the field validation projects satisfied the mix design criteria.

Volumetrics – The limited number of samples obtained for field validation 4.75 mm mixtures indicate that, in general, the contractors maintained reasonable control of laboratory-compacted V_a during production. It is evident that, except for the Alabama mix, the asphalt contents had to be increased substantially over the mix design targets, even with the higher production dust contents.

The laboratory phase of this study recommended the use of criteria for Vbe in place of VMA and VFA. The V_{be} of the Missouri mix was designed and produced within the recommended V_{be} range. Three other three mixes were produced at or above the maximum recommended mix design criteria for V_{be} for their respective traffic categories, either by design or by additional asphalt during production. The Minnesota mix exceeded the proposed limit of

13.5% by more than 3%. However, despite the high V_{be} results, the rutting tests on the validation mixtures were acceptable.

Although VMA and VFA criteria are not recommended for continued use for 4.75 mm mixtures, these properties were analyzed to maintain continuity with the historical criteria. As with most of the Phase I mix designs, the verification mixes easily met the current AASHTO minimum VMA criteria of 16.0%. During production, three of the mixtures had an increase in VMA due to gradation shifts away from the maximum density line, even with higher dust contents. In contrast, three mix designs exceeded the current AASHTO recommended upper limits for VFA. The Missouri mix was within the VFA range allowed for low-traffic projects for the mix design and during production. The VFA results are consistent with the Vbe results noted above.

TABLE 5.12 Summary of Plant-Produced Mixes for Validation Projects

Field Property (average, range)	Alabama	Missouri	Tennessee (virgin mix)	Minnesota
Mix Level	65 gyrations	50 gyrations	125 gyrations (note1)	75 gyrations
Binder Content (% of mix)	6.9, 6.7-7.1	7.2, 6.8-7.4	7.6, 7.5-7.7	9.0, 8.8-9.1
1.18 mm Sieve (% passing)	52, 51-55	54, 48-58	50, 50-51	56, 54-60
0.075 mm Sieve (% passing)	9.2, 8.3-11	12.1, 11.4-12.5	12.7, 11.7-13.4	9.0, 8.7-9.9
V_a (lab compacted)	2.8, 1.9-3.7	4.2, 4.1-4.5	5.1, 4.6-5.9	3.6, 2.9-3.9
VMA (%)	18.3, 17.8-18.7	17.2, 16.6-17.7	20.3, 19.9-20.5	21.6, 21.0-22.1
V_{be} (%)	15.6, 14.4-16.2	13.0, 12.5-13.3	15.1, 14.9-15.3	18.2, 17.9-18.5
VFA (%)	85.1, 80.6-88.1	75.3, 74.3-76.5	74.5, 72.8-75.8	84.2, 82.7-85.4
In-place V_a (%)	11.7, 9.5-13.2	10.1, 9.2-11.9	11.9, 7.5-14.2	6.6, 4.9-8.0
D:B ratio	1.4, 1.2-1.8	2.1, 2.0-2.2	1.9, 1.8-1.9	1.2, 1.1-1.3
FT (microns)	7.5	5.0	6.7	8.6
TSR	0.80	0.70, 0.66-0.74	0.71, 0.68-0.75	0.76, 0.68-0.82
APA (mm)	13	6.7 (Va=4%)	4.0 (Va=7%)	6.1 (Va=7%)
Permeability (cm/sec)	90×10^{-5}	40×10^{-5}	140×10^{-5}	$<10 \times 10^{-5}$
DFT_{20}	Not measured	Not measured	0.25-0.35	0.34-0.49
CTM (mm)	Not measured	0.17-0.22	0.16-0.33	0.13-0.18

Note-1 Field mix compacted to 125 gyrations required to match 75-blow Marshall mix design at 4.0% V_a.

The D:B ratio of the mix designs reflected the full range of the criteria. For comparison, the computed FT for the mix designs ranged from a somewhat low value of 6.5 microns to a more reasonable value of 9.3 microns. During production, the computed D:B ratio for the Missouri mix increased above 2.0 and the FT dropped to 5.0 microns. This raises some concern about the mixture's durability. For projects in the traffic category of 3.0 to 30 million ESALs, the authors recommended a minimum D:B ratio of 1.5. This recommendation was based on the trend observed in the lab phase that 4.75 mm mixtures with higher D:B ratios had much better rutting resistance. However, the Minnesota validation mixture proved that mixtures with low D:B ratios could have good rutting resistance if other mix design criteria are met. Requiring a minimum D:B ratio of 1.5 for this traffic level could be too restrictive on mix designs and could also reduce mixture durability. Therefore, the D:B ratio should be 1.0 to 2.0 for all traffic levels.

TABLE 5.13 Mix Design Criteria Validation Summary

Mix criteria	Traffic Level (ESALs)	Current AASHTO	Prelim. Recomm. Criteria	Alabama	Missouri	Tennessee	Minnesota
N_{des} Gyrations	<0.3M <3.0M >3.0M	50 75 75	No change	65 OK	50 OK	125 (note 1) <3.0M HIGH 75-blow non-standard	75 OK
FAA (note-3)	<0.3M <3.0M >3.0M		40 45 45	Design OK	Design OK	Design not reported	Design OK
Natural Sand (if FAA<45)	<0.3M <3.0M >3.0M		15 max 15 max	Design n/a	Design n/a	Design OK	Design n/a
SE (note-3)	<0.3M <3.0M >3.0M	40 40 45	No change	Design Not reported	Design Not reported	Design Not reported	Design OK
Gradation Control 1.18 mm	<0.3M <3.0M >3.0M	30-60	30–55	Design OK Plant OK	Design OK Plant OK	Design HIGH Plant OK	Design OK Plant HIGH
Gradation Control 0.075 mm		6–12	6–13	Design OK Plant OK (note 2)	Design OK Plant OK (note 2)	Design OK Plant OK (note 2)	Design OK Plant OK (note 2)
V_a (% G_{mm} @ N_{des})		4.0	4.0–6.0	Design LO Plant LOW	Design OK Plant OK	Design OK Plant OK	Design OK Plant LOW
VMA	<0.3M <3.0M >3.0M	16 min 16 min 16 min		Design OK Plant HIGH	Design OK Plant OK	Design OK Plant HIGH	Design HI Plant HIGH
V_{be}	<0.3M <3.0M >3.0M		12.0-15.0 12.0–15.0 11.5–13.5	Design OK Plant HIGH	Design OK Plant OK	Design OK Plant OK	Design HI Plant HIGH
% G_{mm} @ N_{ini} (note-3)	<0.3M <3.0M >3.0M	91.5 max 90.5 max 89.0 max	No change	Design OK	Design Not reported	Design n/a	Design Not reported
D:B Ratio	<0.3M <3.0M	0.9–2.0 0.9–2.0	1.0–2.0 1.0–2.0	Design LO Plant OK	Design OK Plant HIGH	Design OK Plant OK	Design OK Plant HIGH

	>3.0M	0.9–2.0	1.5–2.0				

Note-1 Applied 125 gyrations to match 75-blow Marshall mix design volumetrics.
Note-2 All production mixture increased the mineral fines over the mix design.
Note-3 These mix design criteria were not examined as part of field validation.

Figure 5.2 summarizes the results of the moisture susceptibility testing on the plant-produced mixtures. The field mixes had TSR results in the range of 0.7 to 0.8, which would be considered marginal at best by most agency specifications. Missouri and Tennessee did not require moisture damage testing on the mix designs. TSR results can mask significant differences between the mixtures, as presented by the average IDT test results. For example, although the Alabama and Minnesota TSR values were higher, their average tensile strengths were the lowest. Conversely, the TSR of the Missouri mix was lower than for the Alabama and Minnesota mixes, but its average conditioned tensile strength was higher. Another issue with moisture damage testing of these fine-graded 4.75 mm mixtures is the possibility that the vacuum saturation process may cause damage within the specimens due to the rapid expansion of air within the small voids of the specimen. Therefore, TSR results of the validation mixes are inconclusive.

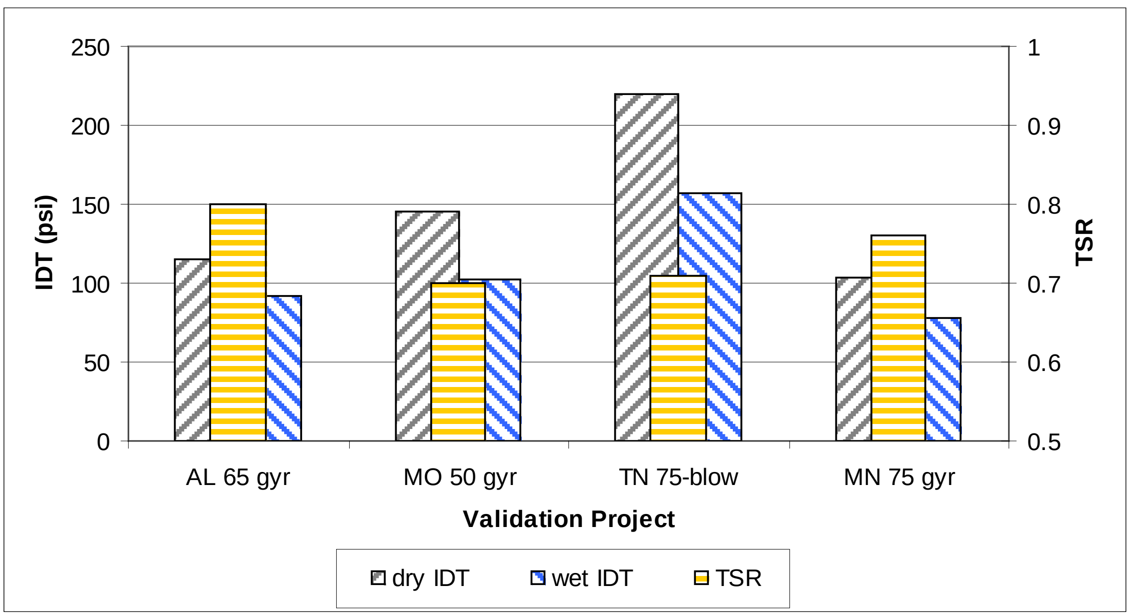

FIGURE 5.2 Average AASHTO T 283 Results for Plant-Produced Mixtures

In-place Air Void Contents – Since 4.75 mm mixtures are typically placed in lifts less than 1-in. thick, field in-place densities or V_a are not usually measured. These mixes are often placed on irregular surfaces, so consistent thicknesses are usually not possible. Generally, agencies simply require inspectors to establish a set rolling pattern for the contractor to follow. Therefore, it was expected that in-place V_a would be higher than the normal range of 6% to 8% for most HMA. The results of the density tests on the cores showed that several of the 4.75 mm mixtures had V_a as high as 13% and 14%. The MnRoad project had much lower V_a, which can likely be attributed to the extra care taken to build the short test section at MnRoad and the use of a very thick lift (over 2.6 inches). Permeability tests on the cores from the projects showed that even at the high V_a, the pavements were relatively impermeable.

Figure 5.3 shows a good relationship between the in-place V_a and laboratory permeability for the cores from all the projects. This graph shows that to maintain an impermeable surface mix of less than 125 x 10-5 cm/sec, the field in-place V_a should be kept below 12%.

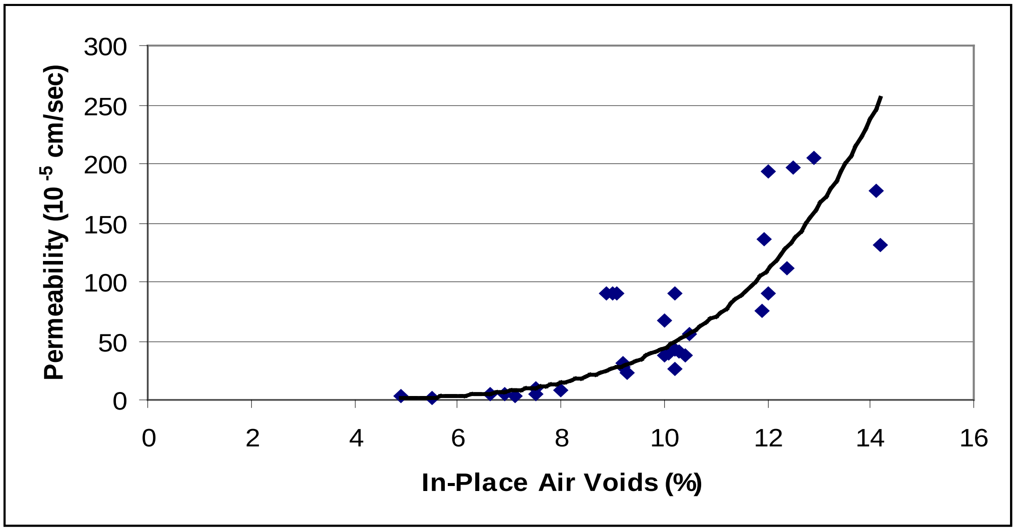

FIGURE 5.3 In-Place Air Voids Versus Permeability for 4.75 mm Mixtures

Friction – Initial CTM values reflect the very fine surface texture created by the fine-graded 4.75 mm mixtures. Hard polish resistant aggregate will be the key to retaining friction properties because the surface texture will not be a major contributing factor. The initial DFT friction values do not typically represent the maximum friction resistance of an HMA surface. The initial measured DFT friction values are negatively influenced by the thin film of asphalt binder on the surface of the aggregate particles. As the traffic wears the asphalt binder film off the surface, exposing more of the aggregate, the friction values typically improve. Once the maximum surface friction is reached after a few weeks or months of traffic, then the friction value is dependent on the polish resistance of the aggregate.

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The objective of this research was to refine the current AASHTO criteria for 4.75 mm NMAS Superpave designed mixtures. In Phase I, 29 4.75 mm NMAS Superpave mix designs were prepared in the laboratory with materials from nine states. Each mix design was tested for permanent deformation, permeability, moisture damage susceptibility, and durability. Plant-produced mixtures from four other states were evaluated and served as baselines for performance. After the Phase I laboratory study, four of the original nine participating state highway agencies constructed projects to validate the proposed mix design criteria and establish field construction criteria. Based on the results of this research, the following conclusions were made with regard to the design of 4.75 mm mixtures:

- The design of 4.75 mm NMAS mixtures is largely dependent on the characteristics of available fine aggregates. In general, 4.75 mm mix designs should utilize at least three aggregate stockpiles to develop a suitable blend that can be adequately controlled during plant production. Most blends of available materials tend to result in very fine gradations, which generally have excessive voids in mineral aggregate (VMA). Consequently, 4.75 mm mixtures often have very high asphalt contents (>6.5% to 9.0%) and tend to be susceptible to permanent deformation.
- VMA and design asphalt content of 4.75 mm mixtures can be reduced by using coarser gradations (closer to the maximum density line) and increasing the dust content.
- The compactive effort used for the mix design should be consistent with the design traffic level. Fifty gyrations are suitable for low-traffic applications where rutting is not a concern either because traffic speeds are low or because the pavement will not carry heavy vehicles that may cause permanent deformation. Seventy-five gyrations should be suitable for most other applications of 4.75 mm mixtures.
- Using a design air void range of 4.0% to 6.0% has little effect on the VMA but will allow mix designers to reduce the asphalt content for a given aggregate blend when the VMA is well above 16.0%. This will improve resistance of 4.75 mm mixtures to permanent deformation. Mixtures with less than 13.5% volume of effective binder (V_{be}) had better rutting resistance than mixtures with more than 13.5% V_{be}.
- Additionally, resistance of 4.75 mm mixtures to permanent deformation can be improved by increasing the dust content, using aggregates with high angularity, and using stiffer binders. Mixtures with dust-to-binder ratios above 1.5 had lower average rutting rates than mixtures with less than a 1.5 dust-to-binder ratio.
- Susceptibility to moisture damage generally increased slightly with decreasing effective asphalt contents. Natural sand contents of over 15% appear to adversely affect moisture susceptibility, rutting susceptibility, and permeability.

- Laboratory permeability testing of the lab and field mixes demonstrated that fine-graded 4.75 mm NMAS mixtures are practically impermeable even at relatively high in-place air voids. Low permeability should help reduce exposure to moisture for these mixtures.
- Higher asphalt contents tended to increase fracture energy ratio. Based on the plots of fracture energy versus film thickness and dust-to-binder ratio, it is concluded that a 4.75 mm NMAS mixture's ability to sustain resistance to cracking is a function of both asphalt content and dust content. Therefore, to assure good durability, 4.75 mm mix criteria should include a minimum V_{be} and a maximum dust-to-binder ratio.

6.2 Recommendations

Based on the results of this study, the following recommendations are provided:

- The current gradation limits on the 1.18 mm and 0.075 mm sieves should be adjusted. Limits placed on percent passing the 1.18 sieve should be 30% to 55%. Limits placed on percent passing the 0.075 mm sieve should be 6.0% to 13.0%.
- For mixes designed for over 0.3 million ESALs, the aggregate blend should contain no more than 15% natural sand and have a minimum fine aggregate angularity of 45 for improved rut resistance, moisture damage resistance, and to maintain low permeability.
- The target air void content for selecting the design binder content should be changed to a range from 4.0% to 6.0%. This will allow for a reduction in the design asphalt content for many 4.75 mm mixtures that have very high VMAs.
- Criteria for VMA and VFA should be replaced with minimum and maximum V_{be} requirements. This is a more sensible approach when a range of design air voids is used. For less than 3.0 million design ESALs, a V_{be} range of 12.0% to 15.0% is recommended. For 4.75 mm mixtures designed for projects over 3.0 million ESALs, a minimum V_{be} of 11.5% and a maximum V_{be} of 13.5% is recommended. These limits were based on fracture energy testing and rut testing for the minimum and maximum V_{be}, respectively.
- The maximum $\%G_{mm}$ @ N_{ini} requirement appears appropriate for both 4.0% and 6.0% design air voids. At this time it is recommended that current G_{mm} @ N_{ini} criteria be maintained.
- The minimum dust-to-binder ratio should be increased slightly from 0.9 to 1.0. The maximum dust-to-binder ratio should be maintained at 2.0.
- No evidence was found that suggested adjusting the current sand equivalent minimum. At this time, the minimum sand equivalent criteria should be maintained.
- The moisture sensitivity test should be reviewed to reduce vacuum induced damage for 4.75 mm mixtures to account for the lower mixture permeability.

A summary of proposed mix design criteria is given in Table 6.1.

TABLE 6.1 Proposed Design Criteria for 4.75 mm NMAS Superpave Design Mixtures

Design ESAL Range (Millions)	N_{des}	Minimum FAA	Minimum SE	Minimum V_{be}	Maximum V_{be}	$\%G_{mm}@N_{ini}$	D:B Ratio
<0.3	50	40	40	12.0	15.0	≤91.5	1.0 to 2.0
0.3 to ≤ 3.0	75	45	40	11.5	13.5	≤90.5	1.0 to 2.0
3.0 to ≤ 30	100	45	45	11.5	13.5	≤89.0	1.0 to 2.0
Gradation Limits							
Sieve Size	Max.	Min.		Design V_a Range = 4.0% to 6.0%			
12.5 mm	---	100					
9.5 mm	100	95					
4.75 mm	100	90					
1.18 mm	30	55					
0.075 mm	13	6					

6.3 Recommended Applications, Advantages, and Disadvantages

4.75 mm mixtures and similar fine-graded mixtures have been used by many transportation agencies for a variety of pavement applications. The most common use of 4.75 mm mixtures is as a surface course on low traffic volume applications such as neighborhood streets. However, these mixes can also be effective as a thin maintenance overlay (less than one inch) on a variety of roadways. Other applications of 4.75 mm mixtures include parking lots, patching mixtures, and use as a leveling course to restore pavement cross slope and profile.

Advantages of 4.75 mm mixtures include the following:

- Thin lifts (3/4 to one inch, typically)
- Excellent smoothness
- Practically impermeable
- Good use of fine aggregate materials, which are in surplus in many areas
- Good use of fractionated fine RAP, which helps improve the stability of 4.75 mm mixtures
- Can be feathered and used in wedge applications
- Good workability with hand tools

Disadvantages of 4.75 mm mixtures include the following:

- High asphalt contents. This can be offset some by including fractionated fine RAP in the mix design. Even at high asphalt contents, these mixtures are very economical because they can cover a large area per ton.
- Low frictional resistance due to the low surface texture of the mixtures. Although using tough and high angularity fine aggregates can provide good skid resistance in dry conditions and in wet weather at slow speeds, 4.75 mm mixtures should not be used on heavy traffic, high speed roadways.

- Greater potential for permanent deformation. The solution for good durability in thin applications is a higher asphalt content, which makes the 4.75 mm mixtures more susceptible to rutting. Resistance to permanent deformation can be improved by 1) designing mixes with lower VMA, 2) using highly angular aggregates, RAP, and a stiffer polymer modified binder, and 3) by achieving good density/low in-place air voids during construction.

REFERENCES

1) Cooley, L. A., R. S. James, and S. M. Buchanan. Development of Mix Design Criteria for 4.75 mm Superpave Mixes. National Center for Asphalt Technology, Report No. 2002-4, February 2004.
2) Cooley, L. A., M. H. Hunter, and E. R. Brown. Use of Screenings to Produce HMA Mixtures. National Center for Asphalt Technology, Report No. 2002-10, October 2002.
3) Prowell, B. D. and J. E. Haddock. Superpave for Low Volume Roads and Base Mixtures. *Journal of the Association of Asphalt Paving Technologists*, Vol. 71, 2002.
4) Mogawer, W. S. and R. Mallick. Design of Superpave HMA for Low Volume Roads. The New England Transportation Consortium, December 2004. www.netc.uconn.edu/reports_listing.html#materials. Accessed December 6, 2005.
5) Habib, A., M. Hossain, R. Kaldate, and G. A. Flager. Comparison of Superpave and Marshall Mixtures for Low-Volume Roads and Shoulders. Transportation Research Board, Transportation Research Record 1609, 1998.
6) Engle, E. J. Superpave Mix Designs for Low-Volume Roads. Iowa Highway Research Board, October 2004. www.operationsresearch.dot.state.ia.us/reports/ihrb_by_number/tr400plus.html. Accessed December 6, 2005.
7) Experiences with Superpave on County Roads. *Asphalt. Magazine of the Asphalt Institute*, Vol. 19, No. 1, Spring 2004, pp. 22–24.
8) Asphalt in Pavement Maintenance. The Asphalt Institute. Manual Series No.16 (MS-16), March 1983.
9) Personal Communication with Donald Watson, National Center for Asphalt Technology, March 23, 2006.
10) Hasen, K. Pavement Preservation with Thin Overlays. *Better Roads*, Vol. 73, No. 6, June 2003, pp. 48–50.
11) Brown, E. R., M. R. Hainin, L. A. Cooley, and G. Hurley. NCHRP Report 531, Relationship of Air Voids, Lift Thickness, and Permeability in Hot Mix Asphalt Pavements. Transportation Research Board, National Research Council, Washington DC, 2004.
12) Moore, J. R. and B. D. Prowell. Evaluation of the Mix Verification Tester for Determining the Rutting Susceptibility of Hot Mix Asphalt. National Center for Asphalt Technology, Internal Report, 2006.
13) Mallick, R. B., L. A. Cooley, M. R. Teto, R. L. Bradbury, and D. Peabody. An Evaluation of Factors Affecting Permeability of Superpave Designed Pavements. National Center for Asphalt Technology, Report 03-02, June 2003.
14) Kim, Y. R. and H. Wen. Fracture Energy from Indirect Tension Testing. *Journal of the Association of Asphalt Paving Technologists*, Vol. 71, 2002.
15) Birgisson, B., C. Soranakom, J. A. L. Napier, and R. Roque. Simulation of Fracture Initiation in Hot-Mix Asphalt Mixtures. Transportation Research Board, Transportation Research Record 1849, 2003.
16) Prowell, D. P., J. Zhang, and E. R. Brown. NCHRP Report 539, Aggregate Properties and Performance of Superpave Designed Hot Mix Asphalt. Transportation Research Board. National Research Council, Washington DC, 2005.

UNCITED REFERENCES

1) Level One Mix Design: Materials Selection, Compaction, and Conditioning. SHRP-A-408, Strategic Highway Research Program, National Research Council, Washington, DC, August 1994.
2) Kandhal, P. S. and L. A. Cooley. NCHRP Report 464, The Restricted Zone in the Superpave Aggregate Gradation Specification. Transportation Research Board. National Research Council, Washington DC, 2001.
3) Roberts, F. L., P .S. Kandhal, E. R. Brown, D. Y. Lee, and T. W. Kennedy. Hot Mix Asphalt Materials, Mixture Design and Construction. National Asphalt Pavement Association Rese arch and Education Foundation, 1996.
4) Stroup-Gardiner, M., D. Newcomb, W. Kussman, and R. Olsen. Characteristics of Typical Minnesota Aggregates, Transportation Research Record 1583, Transportation Research Board, National Research Council, Washington DC, 1997.
5) Kandhal, P. S., C. Lynn, and F. Parker. Test for Plastic Fines in Aggregates Related to Stripping in Asphalt Paving Mixtures, National Center for Asphalt Technology, Report No. 98-3, 1998.
6) Kandhal, P. S. and L. A. Cooley. NCHRP Report 508, Accelerated Laboratory Rutting Tests: Evaluation of the Asphalt Pavement Analyzer. Transportation Research Board. National Research Council, Washington DC, 2003.
7) Cooley Jr., L. A., B. D. Prowell, and E. R. Brown. Issues Pertaining to the Permeability Characteristics of Coarse-Graded Superpave Mixes. National Center for Asphalt Technology, Report 02-06, July 2002.
8) Roque, R., B. Birgisson, Z. Zhang, B. Sangpentngam, and T. Grant. Implementation of SHRP Indirect Tension Tester to Mitigate Cracking in Asphalt Pavements and Overlays. University of Florida, May 2002. www.dot.state.fl.us/research-center/Completed_StateMaterials.htm. Accessed February 2, 2006.
9) Herrin, M. Bituminous Patching Mixtures. Transportation Research Board, NCHRP Synthesis 64, 1979.

APPENDIX

Mix Designs for the Field Validation Projects

ALABAMA DEPARTMENT OF TRANSPORTATION
1409 Coliseum Boulevard, Montgomery, Alabama 36110

JOB. NO. 2403

Bob Riley
Governor

Joe McInnes
Transportation Director

May 18, 2004

East Alabama Paving Co
PO Box 2630
Opelika Al 36803

Project No. STPNU-4423(200)

County MACON

Dear Sir:

The job mix below may be used only at the plant noted and for materials listed. The plant mix will be manufactured in compliance with the applicable specifications, plan notes, and special provisions.

PLANT: East Alabama Paving Co Auburn & Opelika Al

424A-336

Section 424 (Surface)
Max. Size Aggregate: 3/8" max

ESAL Category: Range C/D
Binder Grade: PG67-22 Hunt

MATERIALS:

% (APPROX)	*DESCRIPTION*	*I. D. #*	*SOURCE*	*BPN-9*
19	#89 Granite	0157	VMC Columbus Ga	
30	M10 Granite	0157	VMC Columbus Ga	
30	#8910 Limestone	1604	Oldcastle Materials Opelika Al	25
20	Coarse Sand	0261	MM Pinkston Pit Shorter Al	
1	Baghouse fines		Plant	

JOB MIX:

SIEVES	*% PASSING*
1-1/2"	
1"	
3/4"	
1/2"	
3/8"	100
# 4	90
# 8	64
# 16	47
# 30	32
# 50	17
# 100	10
# 200	6.0

OTHER INFORMATION: (Blow)

% AC Required	6.80
AC Req'd/ Ton (lbs.)	136.0
Max. Sp. Gr. Mix	2.467
Wt./Cu.Ft. (lbs.)	147.2
Stability	n/a
TSR	0.85
Anti-Strip	n/a
Effective AC	6.53
Dust/Asphalt Ratio	0.92
Coarse Agg. Angularity	90/87
Fine Agg. Angularity	46
Sand Equivalent	
Agg. Bulk SG	2.721
% VMA	18.0

NOTE: Locking Point Design

_______________ must be added to the mix. The remaining comes from the RAP.

DESIGN CALIBRATION FACTOR:

GYRATIONS:

7	65
89.0	96.7

ADDITIONAL NOTES:
Mix Temp 325°F ST-215G-04 Locking Point Design

William Caldwell
Hot-Mix Asphalt Engineer

cc: Hot-Mix Asphalt Engineer
4th Division

st 215G-04 /cq

BP-3

Joe Schroer Digitally signed by Joe Schroer DN: cn=Joe Schroer, c=US, o=MoDOT, ou=Field Office Reason: I have reviewed this document Date: 2007.05.04 11:00:58 -05'00'

MISSOURI DEPARTMENT OF TRANSPORTATION - DIVISION OF MATERIALS
PLANT MIX BITUMINOUS BP-3

DATE = **04/26/07** **CONTRACTOR = APEX PAVING CO.** BP07-51

IDENT NO.	PRODUCT CODE	PRODUCER-LOCATION	PI	BULK SP. GR.	APP. SP. GR.	%ABS	FORMATION	LEDGES	% CHERT
70MA0004	1002MS..MSGV	Brown Sand & Gravel, Dexter, MO	NP	2.512	2.639		Crowley Ridge	0	
70MA0049	1002SG..LD	Williamsville Stone #1, Poplar Bluff, MO	NP	2.704	2.828		Gasconade	14-6A	1.4
70MA0050	1002NS..NS1	Brown Sand & Gravel, Dexter, MO		2.611	2.651		Crowley Ridge	0	
70MA0066	1015ACPG..6422	SemMaterials, New Madrid (MFG W.R., IL)		1.033		PG64-22	Mold Temp. 282-292°F		

MATERIAL IDENT #	70MA0004	70MA0049	70MA0050	70MA0004	70MA0049	70MA0050	COMB. GRAD
07051	MS	SG	NS	25.0	55.0	20.0	
3/4"	100.0	100.0	100.0	25.0	55.0	20.0	100.0
1/2"	100.0	100.0	100.0	25.0	55.0	20.0	100.0
#4	98.8	93.3	97.0	24.7	51.3	19.4	95.4
#16	34.9	44.0	74.0	8.7	24.2	14.8	47.7
#200	3.8	12.0	0.2	1.0	6.6		7.6

LABORATORY CHARACTERISTICS AASHTO T-312 50 GYRATIONS						
Gmm =	2.456	% VOIDS =	4.0		**MIX COMPOSTITION**	
Gmb =	2.357	V.M.A. =	16.3		MIN. AGG.	93.6%
Gsb =	2.635	% FILLED =	76		ASPHALT CONTENT	6.4%

CALIBRATION NUMBER = 70061 MASTER GAUGE BACK CNT. = 2139 A1 = -3.189730

MASTER GAUGE SER. NO. = 2502 SAMPLE WEIGHT = 7300 A2 = 2.695984

Corection Factor = 0.79

#1 Bin Sand

#2 Bin 1/4 - Un

#3 Bin Man Sand

STATE OF TENNESSEE ASPHALT JOB MIX FORMULA

2008V1.1 | L-13, L-20

Project Ref. No.	4.75-mm	**Date**	06/13/2008
Project No.	74000-4200-404	**Region**	3
Contract No.	In-Place Maintenance	**County**	Robertson
Contractor	Lojac, Inc.	**Date of Letting**	n/a
State Route No.	SR25	**Roadway Surface**	
Hot-mix Producer	LOJAC, INC., RINKER QUARRY - SPRINGFIELD		

Type ACS-HM **Mix** 4.75 mm PG 64-22 **Item**

Serial No.:	08M128	Design No.:	308318

Material	Size or Grade	Producer and Location	Percent Used
#10 (Hard)	Screenings	Aggregates USA, Springfield, TN	69.900
#10 (Soft)	Screenings	Aggregates USA, Springfield, TN	9.320
Natural Sand	Natural Sand	Ingram Mtls, Nashville, TN	13.980
Asphalt Cement	PG 64-22	ERGON ASPHALT CO., NASHVILLE TERMINAL	6.800

Percent AC in RAP:		**Optimum AC Content:**	6.8	**Total**	100.000
Anti-Strip Additive:		n/a	**Dosage:**		
AC Contribution:	**Virgin AC**	6.80	**RAP AC**		**Percent Virgin AC:**
Asphalt Sp. Gravity:	1.03	**Dust to Asphalt Ratio:**	1.78		

% Fracture Face on CA:		**% Glassy Particles on CA:**	
Theo.Gravity of RAP:	n/a	**Eff. Gravity of Agg:**	2.643

Theo. Gravity of Mix:	2.389	**T.S.R.:**		**Lbs/Ft^3:**	149.1
L.O.I.:		**Ignition Oven Corr. Factor:**			
ADT	1620	**Log Miles**	**Beginning:** 4.00	**Ending:**	6.95

Lab Temperature		Plant Temperature	
Mixing Temperature (± 5 °F):	**300**	**Mixing Temp Range(°F):**	**290°F ≤ T ≤ 320°F**
Lab Compaction Temp (± 5 °F):	**290**	**Delivery Temperature(°F):**	**290°F ≤ T ≤ 320°F**

	Percents Used								
Sieve Size	#10 (Hard) 75.0	#10 (Soft) 10.0	Natural Sand 15.0					**% Req. 100**	**Design Range**
2"									
1.5"									
1.25"									
1"									
3/4"									
5/8"									
1/2"	100	100	100					**100**	**100**
3/8"	100	100	100					**100**	**95-100**
No.4	98	95	98					**98**	**90-100**
No.8									
No.16	54	54	78					**58**	**30-55**
No.30									
No.50									
No.100									
No.200	13.7	18.0	0.4					**12.1**	**6-13**

Requested: Mark Woods PT999
Contractor Personnel and Lab Tech Cert No.

Approved: ____________
Regional Materials and Tests Supervisor

Date last lab inspection 6/12/2008

Approved: ____________
Headquarters Materials and Tests

STATE OF TENNESSEE ASPHALT JOB MIX FORMULA

2008V1.1

L-13, L-20

Project Ref. No.	4.75-mm	**Date**	06/13/2008
Project No.	74000-4200-404	**Region**	3
Contract No.	In-Place Maintenance	**County**	Robertson
Contractor	Lojac, Inc.	**Date of Letting**	n/a
State Route No.	SR25	**Roadway Surface**	Yes
Hot-mix Producer	LOJAC, INC., RINKER QUARRY - SPRINGFIELD		

Type ACS-HM **Mix** 4.75 mm w/ RAP PG 64-22 **Item**

Serial No.:	08M129	Design No.:	308319

Material	Size or Grade	Producer and Location	Percent Used
#10 (Hard)	Screenings	Aggregates USA, Springfield, TN	55.920
#10 (Soft)	Screenings	Aggregates USA, Springfield, TN	9.320
Natural Sand	Natural Sand	Ingram Mtls, Nashville, TN	13.980
RAP	Minus 5/16 inch Rap	Lojac, Springfield, TN	14.703
Asphalt Cement	PG 64-22	ERGON ASPHALT CO., NASHVILLE TERMINAL	6.077

Percent AC in RAP:	4.9	**Optimum AC Content:**	6.8	**Total**	100.000
Anti-Strip Additive:	n/a	**Dosage:**			
AC Contribution:	**Virgin AC** 6.08	**RAP AC** 0.72	**Percent Virgin AC:**	89.4	
Asphalt Sp. Gravity:	1.03	**Dust to Asphalt Ratio:**	1.78		

% Fracture Face on CA:		**% Glassy Particles on CA:**	
Theo.Gravity of RAP:		**Eff. Gravity of Agg:**	2.632

Theo. Gravity of Mix:	2.380	**T.S.R.:**		**Lbs/Ft^3:**	148.5
L.O.I.:		**Ignition Oven Corr. Factor:**			
ADT	1620	**Log Miles**	**Beginning:** 4.0	**Ending:**	6.95

Lab Temperature		**Plant Temperature**	
Mixing Temperature (± 5 °F):	**300**	**Mixing Temp Range(°F):**	**290ºF ≤ T ≤ 320ºF**
Lab Compaction Temp (± 5 °F):	**290**	**Delivery Temperature(°F):**	**290ºF ≤ T ≤ 320ºF**

	Percents Used								
Sieve	**#10 (Hard)**	**#10 (Soft)**	**Natural Sand**				**RAP**	**% Req.**	**Design**
Size	60.0	10.0	15.0				15.0	**100**	**Range**
2"									
1.5"									
1.25"									
1"									
3/4"									
5/8"									
1/2"	100	100	100				100	**100**	**100**
3/8"	100	100	100				100	**100**	**95-100**
No.4	98	95	98				83	**95**	**90-100**
No.8	72	80	93				64	**75**	**-**
No.16	54	54	78				45	**56**	**30-55**
No.30	30	37	61				37	**36**	**-**
No.50									
No.100	18.0	22.0	0.8				16.4	**15.6**	**-**
No.200	13.7	18.0	0.4				13.6	**12.1**	**6-13**

Requested: Mark Woods PT999
Contractor Personnel and Lab Tech Cert No.

Approved:
Regional Materials and Tests Supervisor

Date last lab inspection 6/12/2008

Approved:
Headquarters Materials and Tests

From: "Roger Olson" <Roger.Olson@dot.state.mn.us>
To: <MAH0016@auburn.edu>
Date: 8/1/2008 10:00 AM
Subject: Re: Fwd: 4.75 design

Mike, here's the results on the 4.75mm taconite design for Mn/RD.
comments?

55% MINTAC TAILINGS
10% ISPAT TAILINGS
35% LOKEN MAN SAND

Gsb =2.848
CRUSHED =100%
FAA = 47
SE = 83
GRADATION: % passing 3/8 = 100
#4 = 92
#8 = 72
#16 = 51
#30 = 34
#50 = 21
#100= 12
#200= 7.7

PG 64-22 Note: 64-34 is planned for field
Design Gyrations = 75
HT @Ndes= 115.2
voids = 3.9
AC = 7.4
VMA = 20.3
VFA = 80.8
F/E = 1.1
UNIT WT = 152.9
TSR = 82 (9.0% voids)
Adj AFT = 10.1 microns
APA = TBD