2002

Comparison of 19mm Superpave and Marshall Base II mixes in West Virginia

Vasavi Kanneganti
West Virginia University

COMPARISON OF 19mm SUPERPAVE AND MARSHALL BASE II MIXES IN WEST VIRGINIA

Vasavi Kanneganti

Thesis submitted to the College of Engineering and Mineral Resources at West Virginia University in partial fulfillment of the requirements for the degree of

Masters of Science

In

Civil and Environmental Engineering

Dr. John P. Zaniewski, Chair

Dr. Ronald W. Eck,

Dr. W.V. Thayne

Department of Civil and Environmental Engineering

Morgantown, West Virginia

2002

Keywords: Marshall, Superpave, Mix designs

ABSTRACT

COMPARISON OF 19mm SUPERPAVE AND MARSHALL BASE II MIXES IN WEST VIRGINIA

Vasavi Kanneganti

The Superior Performing Asphalt Pavements (Superpave™) asphalt concrete mix design method was developed through Strategic Highway Program (SHRP) in 1993. With the introduction of Superpave mix design, the Marshall method of mix design is becoming obsolete for highway pavements. Superpave implementation varies by state. The WVDOH has implemented Superpave on all National Highway System projects since 1997. The decision regarding implementation of Superpave for low volume roads in WV is still under review.

The primary objective of this research work was to compare the 19mm Superpave and Base II Marshall design mixes in WV to supplement information required for WVDOH to make a suitable decision regarding the implementation of Superpave for low volume roads.

The Marshall and Superpave methods were compared by preparing similar mix design with each method. The mix designs from each method were cross-compared with the conclusion that mixes developed under one method meet the criteria of the other method. In addition, the Asphalt Pavement Analyzer (APA) was used to evaluate rutting performance of gyratory compacted samples in the laboratory. The statistical analysis of rut depth results indicated there is not enough evidence to conclude there is a significant difference between the Marshall and Superpave mix design methods. It can be concluded that for the materials evaluated in this research, the Marshall and Superpave methods produce interchangeable results.

ACKNOWLEDGEMENTS

The author would like to express her gratitude to Dr. John Zaniewski, the author's advisor. Without the guidance, and support, the author would never have had the opportunity to pursue his Master of Science degree and participate in this research.

The author would also like to extend thanks to Dr. Ronald Eck and Dr. W. V. Thayne, the author's other committee members for their assistance and support during the research work and in classes.

Special thanks are given to J.F. Allen Company, Buckhannon, WV, for all their assistance throughout the project.

Final thanks are given to the author's family and her fiancé Murali. Their love and support have been a source of strength and motivation.

Table of Contents

List of Figures

List of Tables

CHAPTER 1 INTRODUCTION

1.1 INTRODUCTION

Most hot mix asphalt (HMA) produced during the 50 years between the 1940s and mid 1990s were designed using the Marshall or Hveem methods. Increases in traffic volumes and heavier loads became initiative for the Strategic Highway Research Program (SHRP) in 1988. After a five year of effort, a new mix design, Superior Performing Asphalt Pavements (Superpave)™, was developed. Superpave takes into consideration the factors that are responsible for the typical distresses on asphalt pavements: rutting, fatigue and thermal cracking. With the introduction of Superpave mix design, the Marshall method of mix design is becoming obsolete for highway pavements.

Superpave implementation varies by state. Some have completely switched over to the national Superpave standards. Others have implemented the Superpave concepts, but have adjusted criteria to suit local conditions. Others have partially implemented Superpave for some projects but are using their legacy method for other projects. Finally, two states, California and Nevada, are continuing to use legacy methods. The West Virginia Division of Highways (WVDOH), constructed its first Superpave project on I-79 in 1997. Since then, WVDOH has implemented Superpave on all National Highway System (NHS) projects. The decision regarding implementation of Superpave for non-NHS roads in West Virginia is still under review. The primary objective of this research work was to compare the Superpave and Marshall design mixes in West Virginia to supplement information required for WVDOH to make a suitable decision regarding the implementation of Superpave for low volume roads.

1.2 PROBLEM STATEMENT

One of the advantages of the Marshall mix design method in West Virginia is that the performance of the mixes is known for local conditions and materials. There are no Superpave projects for low volume roads in West Virginia to evaluate the performance of mixes. An investigation is needed to compare the performance of Superpave and Marshall mix design methods, to support replacement of conventional mix design with the new mix design. This research work compared the Superpave and Marshall mixes in

West Virginia using the Asphalt Pavement Analyzer (APA) to evaluate rutting performance in the laboratory. Mix designs were prepared under the Marshall and Superpave methodologies. The resulting Marshall mix designs were evaluated under the Superpave criteria, and vice versa. If the mixes prepared under the Superpave method met the Marshall criteria, it was hypothesized that these mixes would perform well in the field. If the mixes prepared under the Marshall method pass the Superpave criteria, it was hypothesized that contractors would not encounter any undue hardships in designing and constructing pavements to the Superpave criteria.

1.3 OBJECTIVES

The objective of this research was to compare 19 mm Superpave and Base II Marshall mixes in West Virginia. The primary area of interest for this comparison was for medium and light traffic volume loads since the WVDOH has not implemented Superpave for these traffic levels. The research included mix designs for high traffic volume roads for completeness and to provide insight on how the research methodology would rank these mixes.

1.4 SCOPE OF WORK AND LIMITATIONS

In this research work 19mm Superpave and Base II Marshall mixes were designed for Heavy, Medium, and Light traffic levels. Marshall and Superpave mixes were designed from locally available materials. The Marshall and Superpave mix design procedures of the West Virginia Division of Highways (WVDOH) were followed. Gyratory compactor was used to make the Asphalt Pavement Analyzer (APA) samples to evaluate permanent deformation (rutting). Results from tests with different aggregates, gradations, and binder types show that the APA is sensitive to these factors and, therefore, has a potential to predict relative rutting of hot mix asphalt mixtures (2).

The experimental design used for this research work provides comparison between the mixes, traffic levels and their interactions. The work was limited to Base-II mixes in West Virginia and to one source of aggregate and asphalt cement. The 19mm nominal maximum size aggregate was used for the two mix design methods. The J.F.

Allen Company, Buckhannon, WV, provided the aggregate used for the research. The asphalt used was PG 70-22 from the source Marathon Ashland.

The work was also limited to laboratory testing. Field evaluation could not be performed since WVDOH has not constructed medium and light traffic volume Superpave mixes.

1.5 THESIS OVERVIEW

This thesis is organized into five chapters and five appendices. After the first chapter of Introduction, Chapter 2 is a summary of literature review. Superpave and Marshall mix design procedures are outlined with standard test procedures and specifications required by the WVDOH. The method of rut testing with the Asphalt Pavement Analyzer (APA), as specified by the device manufacturer, is explained in detail. The research methodology and procedures for preparing, testing and analyzing samples is presented in Chapter 3. Chapter 4 presents the results of the experimental design and the analysis of the results. Chapter 5 concludes the thesis with the conclusions and recommendations.

The aggregate test data are presented in the Appendix A. Detailed test data and interim calculations of Marshall and Superpave mix designs are presented in Appendix B and Appendix C respectively. Appendix D presents the data for evaluating Superpave mix design with Marshall methodology and vice versa.

CHAPTER 2 LITERATURE REVIEW

2.1 INTRODUCTION

It was in 600's B.C. that the first Asphalt road was paved in Babylon. The first bituminous Hot Mix Asphalt (HMA) pavement in the United States was built in Washington D.C, in 1873. In the years between the 1940s and mid 1990s, most of the hot mix asphalt projects in United States were designed using the Marshall and Hveem methods. According to survey in 1984, approximately 75 percent of the State Highway Departments used some variation of the Marshall method while the remaining 25 percent used some variation of Hveem method (4). In 1995, a few states began to use the Superpave design procedures (4). The 2000 Superpave Implementation survey shows that almost every state in United States is at some stage or the other in Superpave implementation (8).

The Superpave and Marshall mix design methods, the Gyratory Compaction, and Asphalt Pavement Analyzer (APA) are discussed in this chapter.

2.2 MARSHALL MIX DESIGN

Bruce Marshall, formerly the Bituminous Engineer with the Mississippi State Highway Department, developed the original concept of the Marshall Method of designing asphalt pavements. The present form of Marshall mix design method originated from an investigation started by the U.S Army Corps of Engineers in 1943 (4). The purpose of the Marshall method is to determine the optimum asphalt content for a particular blend of aggregates and traffic level. The optimum asphalt content is determined by the ability of a mix to satisfy stability, flow, and volumetric properties.

2.2.1 MATERIAL SELECTION

The materials used in the mix design should conform to the requirements set forth in the Standard Specifications of WVDOH. Specifications for aggregate are covered under sections 702 and 703 while asphalt cement is covered under section 705.

2.2.2 AGGREGATE GRADATION

The aggregate blend gradation requirements for base II mixes is presented in Table 2.1 (WVDOH Standard Specifications; Section 401). Aggregates meeting these requirements are not placed in single stockpile, as this would promote segregation. Therefore, stockpiles must be blended to meet the gradation specifications. Equation 2.1 is used to estimate the blended gradation from multiple stockpiles.

Table 2.1 Tolerance Limits of Master Gradation Range for Base II Mix-WVDOH Standard Specifications

SIEVE SIZE (mm)	**BASE II LEVEL**
25	100
19	90-100
12.5	90 max
9.5	
4.75	
2.36	20-50
1.18	
0.600	
0.300	
0.075	2.0-8.0

$$p = Aa + Bb + Cc + ... \quad (2.1)$$

where,

p = the percent of material passing a given sieve for the combined aggregates A, B, C….;

A, B, C,… = the percent of material passing a given sieve for each aggregate A, B, C,…;

a,b,c,… = proportions of aggregates A, B, C,… to be used in the blend.

The specific gravity for the blend is computed as:

$$G = \frac{P_1 + P_2 + ... + P_n}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + ... + \frac{P_n}{G_n}} \quad (2.2)$$

where,

G = blend specific gravity;

$G_1, G_2, \ldots G_n$ = specific gravity values for fraction 1, 2,…n; and

$P_1, P_2, \ldots P_n$ = weight percentages of fraction 1,2,…n.

2.2.3 SPECIMEN FABRICATION

For determining the design asphalt content for a particular blend of aggregates by the Marshall method, a series of test specimens are required to include a range of asphalt contents of at least 2.0%, at intervals not to exceed 0.5%. Three specimens are required for each asphalt content used in the design. The standard method for compacting the test specimens is to immediately compact them after mixing process is completed. The WVDOH requires that the specimens are oven aged for two hours. The WVDOH uses compaction effort of 50 and 75 blows per side for medium and heavy traffic levels, respectively. WVDOH uses the medium traffic level procedures for light traffic levels.

At least one specimen is required at the estimated asphalt content to determine the maximum specific gravity (AASHTO T209). WVDOH requires two maximum specific gravity samples. These samples are prepared at the estimated asphalt content for the mix.

2.2.4 VOLUMETRIC ANALYSIS

The bulk specific gravity of the compacted samples is measured and used with the maximum theoretical specific gravity to perform the volumetric analysis. The volumetric parameters and formulae for their calculation are:

$$VMA = 100\left(1 - \frac{G_{mb}\left(1 - P_b\right)}{G_{sb}}\right) \tag{2.3}$$

$$VTM = \left(1 - \frac{G_{mb}}{G_{mm}}\right)100 \tag{2.4}$$

$$VFA = \frac{VMA - VTM}{VMA} \times 100 \tag{2.5}$$

where,

VMA = Volume of voids in mineral aggregate;

G_{mb} = Bulk specific gravity of compacted mixture;

P_b = Asphalt content;

G_{sb} = Bulk specific gravity of aggregate;

VTM = Air voids in compacted mixture:

G_{mm} = Theoretical maximum specific gravity; and

VFA = Voids filled with asphalt.

Since the theoretical maximum specific gravity (G_{mm}) is only measured for one asphalt content (preferably near the optimum), its value at the other asphalt contents is computed as:

$$G_{se} = \frac{1 - P_b}{\dfrac{1}{G_{mm}} - \dfrac{P_b}{G_b}} \tag{2.6}$$

$$G'_{mm} = \frac{1}{\dfrac{1 - P_b'}{G_{se}} + \dfrac{P_b'}{G_b}} \tag{2.7}$$

where,

G_{se} = Effective specific gravity of aggregates;

G_{mm} = Theoretical maximum specific gravity measured;

P_b = Asphalt content used for G_{mm} samples;

G'_{mm} = Calculated theoretical maximum specific gravity;

P_b' = Asphalt content used for other compacted samples; and

G_b = Binder specific gravity (generally provided by binder supplier).

2.2.5 STABILITY AND FLOW MEASUREMENTS

Marshall stability is defined as the maximum load carried by a compacted specimen tested at 140^{o}F (60^{o}C) at a loading rate of 2 inches/minute (4). The flow is measured at the same time as the Marshall stability. The flow is equal to the vertical deformation of the sample (measured from start of loading to the point at which stability begins to decrease) in hundredths of an inch (4). The stability and flow measurements procedure (AASHTO T245) indicates that the stability reading for a test specimen is only accurate if the test specimen measures 63.5mm in height. For test specimens that vary slightly from 63.5 mm, the stability reading should be multiplied with a correlation ratio in Table 2.2 (AASHTO T245).

Table 2.2 Stability Correlation Ratios from AASHTO T245

Specimen Height	Correlation Ratio
58.7	1.14
60.3	1.09
61.9	1.04
63.5	1.00
65.1	0.96
66.7	0.93
68.3	0.89
69.8	0.86
71.4	0.83
73.0	0.81
74.6	0.78
76.2	0.76

2.2.6 OPTIMUM ASPHALT CONTENT

The stability, flow, unit weight, air voids, VMA and VFA are plotted versus the asphalt content. The optimum asphalt content of the mix is determined from the data obtained from the plots. WVDOH specifies that the asphalt content that corresponds to the specification’s median air void content (4.0%) is the optimum asphalt content. Using the asphalt content at 4.0% air voids, the corresponding values for VMA, VFA, stability and flow are determined from the plots and compared to the acceptance criteria in Table 2.3 (WVDOH MP 401.02.22). The mix must be redesigned using different aggregate blends if design criteria are not satisfied.

Table 2.3 Marshall Mix Design Criteria (WVDOH MP 401.02.22).

Design Criteria	Medium traffic design	Heavy traffic design
Compaction, number of blows per side	50	75
Stability (Newton)	5300	8000
Flow (0.25 mm)	8-16	8-14
Air voids (%)	3-5	3-5
VFA, %	65-78	65-75
VMA, %	13*	13*

*VMA specifications are for Base II mix.

2.3 SUPERPAVE MIX DESIGN

Despite the best efforts put with the existing mix design methods, it is common to see severe rutting and cracking in asphalt pavements due to increased traffic loads and environmental conditions. In ~~the~~ 1988, a research program called Strategic Highway Research Program (SHRP) was started in USA. The major funds of the SHRP research program were allocated to establish new procedures for the selection of binders and mix designs with regard to the rutting and cracking problems in the asphalt pavements. The research program was completed in 1993, producing the Superpave mix design method (7).

2.3.1 GYRATORY COMPACTOR

The key piece of equipment used in the Superpave mix design method is the gyratory compactor, Figure 2.1. One of the main goals of the SHRP was to develop a laboratory compaction method, which can consistently produce specimens representative of in-service pavements. The Superpave gyratory compactor compacts HMA samples to densities achieved under traffic loading conditions. Its ability to estimate specimen density at any point during the compaction process is its key feature (4).

Gyratory compaction has been used in asphalt mix design since early 1900's. Midway through the Strategic Highway Research Program, an evaluation of available gyratory compaction research was done to develop a gyratory protocol, which would simulate the density achieved at the end of pavement's life. Studies conducted during SHRP show that the density of the HMA sample is influenced mostly by the angle of gyration, and slightly by the speed of gyration and the vertical pressure. The stresses applied to the mixture and the mixture properties are the two parameters that would influence asphalt mixture density (3).

Fig. 2.1 Superpave Gyratory Compactor

The AASHTO provisional standard TP 4-00 covers the compaction of cylindrical specimens of hot-mix asphalt (HMA) using the Superpave gyratory compactor. This standard specifies the compaction criteria of the Superpave gyratory Compactor. The ram shall apply and maintain a pressure of 600 ±18 kPa perpendicular to the cylindrical axis of the specimen during compaction. The compactor shall tilt the specimen at an angle of 1.25 ±0.02 ^{o} and rotate the specimen molds at a rate of 30.0 ±0.5 gyrations per minute throughout compaction.

The ruggedness evaluation of AASHTO TP4 conducted by FHWA arrived at the conclusions that (9):

- The tolerance on compaction angle (±0.02^{o}) is reasonable and necessary
- The tolerance on compaction pressure (±18kPa) is too high

2.3.2 MATERIAL SELECTION

Binder selection is based on environmental data, traffic level and traffic speed. A Performance Grade asphalt binder is designated with a high and low temperature grade, such as PG 70-22. For this binder, 70 is the high temperature grade and is the 7-day maximum pavement design temperature in degrees centigrade for the project. The low temperature grade, -22, is the minimum pavement design temperature in centigrade (5).

The SHRP made no research effort to specifically look at aggregates. However, guidance was eventually provided based on a consensus approach by a group of experts. Superpave requires both consensus and source aggregate tests be performed to assure that the combined aggregates selected for the mix design are acceptable. The consensus property criteria are the minimum requirements for the aggregates to be used in the Superpave mix design method and they are the same regardless of geographic location. The source property criteria are specified by the state highway agencies (5). Superpave requires the following consensus properties be determined for the design aggregate blend:

- Coarse aggregate angularity (ASTM D 5821) –materials retained on 4.75 mm sieve.
- Fine Aggregate angularity (AASHTO T304) – materials passing the 2.36 mm sieve.
- Flat & Elongated particles (ASTM D4791) –materials retained on 9.5 mm sieve.
- Sand Equivalent (AASHTO T176) –materials passing the 4.75 mm sieve.

The flat and elongated test follows the general procedures of ASTM D 4791, but is modified for Superpave. Under the Superpave guidelines an aggregate particle coarser than 4.75mm sieve is flat and elongated if the ratio of the maximum to minimum dimension is greater than 5 (5).

Aggregate property requirements set forth by WVDOH is shown in Table 2.4(WVDOH MP-2)

2.3.3 DESIGN AGGREGATE STRUCTURE

Trial blends are established by mathematically combining the gradations of individual stockpiles into a single blend using Equation 2.1. The gradation of the aggregate blend must be within the control limits to meet the Superpave requirements. The gradation control is based on four control sieves: the maximum sieve, the nominal

Table 2.4 Superpave Aggregate Consensus Property Requirements as set forth by WVDOH

Design ESALs (Million)	Coarse Aggregate angularity (%min)*		Fine aggregate angularity (%min)		Sand equivalent Percent minimum	Flat & elongated Percent minimum
	100 mm from surface	>100 mm from surface	100 mm from surface	>100 mm from surface		
<0.3	55/-	-	-	-	40	-
0.3 to <3	75/-	50/-	40	40	40	10
3 to <10	85/80	60/-	45	40	40	10
10 to <20	90/95	80/75	45	40	45	10
20 to <30	95/90	80/75	45	40	45	10
30	100/100	100/100	45	45	50	10

*Percent of one/more than one fractured faces

maximum sieve, the 2.36-millimeter sieve, and the 75-micron sieve. The Superpave definitions for nominal maximum aggregate size and maximum aggregate size are:

- Nominal Maximum Aggregate Size: One sieve size larger than the first sieve to retain more than 10.0% of the material.
- Maximum Aggregate Size: One sieve size larger than the nominal maximum aggregate size.

The restricted zone is another part of grading specification. Aggregate blends that pass through the restricted zone that do not use excessive amounts of rounded aggregates and that meet the minimum VMA requirements perform satisfactorily (4). However, the WVDOH specifications do not allow any mix with a gradation passing through the restricted zone. There is evidence to support the belief that mixtures closer to the low end of the control limits (those going underneath the restricted zone) provide a desirable structure for resisting rutting (4). WVDOH Specifications for 19mm nominal maximum size are shown in Table 2.5.

Once an aggregate blend is identified, which meets the gradation criteria, the consensus propery for the blend is determined as:

$$X = \frac{x_1 P_1 p_1 + x_2 P_2 p_2 + \ldots}{P_1 p_1 + P_2 p_2 + \ldots} \tag{2.8}$$

Table 2.5 Gradation Specifications for 19mm Nominal Maximum Size

SIEVE SIZE (mm)	CONTROL POINTS		RESTRICTED ZONE	
	Lower	Upper	Lower	Upper
25	100	100	-	-
19	90	100	-	-
12.5	-	90	-	-
9.5	-	-	-	-
4.75	-	-	-	-
2.36	23	49	34.6	34.6
1.18	-	-	22.3	28.3
0.6	-	-	16.7	20.7
0.3	-	-	13.7	13.7
0.15	-	-	-	-
0.075	2	8	-	-

where

X = Blended consensus property;

x_i = Consensus property for stockpile;

P_i = Percent of stockpile i in the blend; and

p_i = Percent of stockpile which either passes or is retained on the dividing sieve.

The Superpave process requires evaluating the aggregate blends to determine a design aggregate structure. The Superpave guidelines suggest selecting two blends with gradation below the restricted zone and one blend with a gradation above the restricted zone. The asphalt content for the blends may be estimated based on experience or using the equations:

$$G_{se} = G_{sb} + F\left(G_{sa} - G_{sb}\right) \tag{2.9}$$

$$V_{ba} = \frac{P_s\left(1 - V_a\right)\times}{\left(\frac{P_b}{G_b} + \frac{P_s}{G_{se}}\right)}\left(\frac{1}{G_{sb}} - \frac{1}{G_{sa}}\right) \tag{2.10}$$

$$V_{be} = 0.176 - 0.0675 \times \ln\left(S_n\right) \tag{2.11}$$

$$W_s = \frac{P_s \times (1 - V_a)}{\left(\frac{P_b}{G_b} + \frac{P_s}{G_{se}}\right)} \tag{2.12}$$

$$P_{bi} = \frac{G_b \times (V_{be} + V_{ba})}{(G_b \times (V_{be} + V_{ba})) + W_s} \tag{2.13}$$

where,

G_{se} = Effective specific gravity of aggregate;

G_{sb} = Bulk specific gravity of aggregate;

G_{sa} = Apparent specific gravity of aggregate;

F = factor for absorption;

V_{ba} = volume of absorbed binder;

P_b = Asphalt content, percent by weight of mix;

P_c = percent of aggregate;

G_b = Specific gravity of binder;

V_a = volume of air voids;

W_s = weight of aggregate;

V_{ba} = volume of absorbed binder;

V_{be} = volume of effective binder;

S_n = nominal maximum sieve size of aggregate blend; and

P_{bi} = percent (by weight) of binder;

Two samples are prepared for each aggregate blend. The samples are mixed, cured for 2 hours and compacted and the G_{mb} is determined. The Superpave gyratory compactor is used to compact the samples. The number of gyrations applied to the mix regulates the compactive effort. The number of gyrations required is a function of the traffic level. As shown in the Table 2.6, Superpave defines three compaction requirements for a mix.

Table 2.6 Superpave Compaction Criteria (WVDOH MP 401.02.28)

ESALs (million)	Compaction Parameters		
	N_i	N_d	N_m
<0.3	6	50	75
0.3 to <3	7	75	115
3 to <30	8	100	160
> 30	9	125	205

The number of gyrations for design, $N_{d,}$ was selected to simulate the compacted state of an asphalt pavement following construction. An initial compactive effort, N_i, was defined to identify "tender" mixes. Some mixes are difficult to compact in the field because the mix lacks the internal friction required to prevent the excessive deformation. As traffic is applied to a pavement, the mix will continue to compact until an equilibrium condition is achieved. The maximum Superpave compactive effort, N_{max} was selected to ensure the material does not over compact under traffic. N_{max} and N_i are a function of N_d:

$$N_i = (N_d)^{0.45} \tag{2.14}$$

$$N_{max} = (N_d)^{1.10} \tag{2.15}$$

where

N_i = Initial number of gyrations;

N_d = Design number of gyrations; and

N_m = maximum number of gyrations.

The WVDOH requirements for compactive effort are given in Table 2.6.The design aggregate structure samples are compacted to N_d. The bulk specific gravity of the mix at N_i must be computed as:

$$G_{mb,N_i} = \frac{h_d}{h_i}\left(G_{mb,N_d}\right) \tag{2.16}$$

where

$G_{mb,Ni}$ = Bulk specific gravity at N_i;

h_d = Height of the specimen at N_d;

h_i = Height of the specimen at N_i;

$G_{mb,Nd}$ = Bulk specific gravity at N_d;

The percent maximum specific gravity at N_i is computed as:

$$\%G_{mm,N_i} = 100\frac{G_{mb,N_i}}{G_{mm}} \tag{2.17}$$

Two samples for each aggregate blend are prepared and evaluated to determine G_{mm}. G_{mb} and G_{mm} are used for a volumetric analysis. VTM, VMA and VFA are computed using equations 2.3 to 2.5.

Since P_b was only estimated, the VTM is generally not equal to the criteria of 4.0% air voids. Thus, the volumetric parameters are "corrected" to a 4.0% air content using the equations:

$$P_{b,est} = P_{bt} - (0.4 \times (4 - V_a @ N_d)) \tag{2.18}$$

$$VMA_{est} = VMA @ N_d + C \times (4 - V_a @ N_d) \tag{2.19}$$

$$VFA_{est} = 100(VMA_{est} - V_a @ N_d)/VMA_{est} \tag{2.20}$$

$$Est\%G_{mm,i} = \%G_{mm,i} - (4.0 - V_a @ N_d) \tag{2.21}$$

$$Est\%G_{mm,m} = \%G_{mm,m} - (4.0 - V_a @ N_d) \tag{2.22}$$

where

$P_{b,est}$ = Estimated asphalt binder content;

P_{bt} = Trial percent asphalt binder content;

V_a = percent air voids in total mix at N_d;

VMA_{est} = Estimated voids in mineral aggregate;

C = 0.1 when V_a is less than 4.0%,

C = 0.2 when V_a is 4.0% or greater

VFA_{est} = Estimated voids filled with asphalt;

$Est\%G_{mm,i}$ = Estimated percentage of maximum specific gravity at N_i;

$Est\%G_{mm,m}$ = Estimated percentage of maximum specific gravity at N_d;

In addition the dust to binder ratio is computed as:

$$P_{be,est} = (P_s \times G_b) \times \frac{(G_{se} - G_{sb})}{(G_{se} G_{sb})} \times P_{b,est} \qquad (2.23)$$

$$F / P_{be,est} = \frac{\%P_{0.075mm}}{P_{be,est}} \qquad (2.24)$$

where

$F/P_{be,est}$ = Estimated fines to effective asphalt ratio; and

$\%P_{0.075mm}$ = percent material finer than 0.075 mm sieve.

The corrected volumetric parameters are compared to the Superpave criteria, Table 2.7. The design aggregate structure, which “best” meets the criteria is selected for determining the design binder content. If none of the aggregate blends produce an acceptable mix, a new aggregate blend must be determined and evaluated.

2.3.4 DESIGN ASPHALT BINDER CONTENT

The design binder content is defined by Superpave as the asphalt content that produces 4.0% air voids at N_d and meets all other criteria. A good estimate of design asphalt content is established from the design aggregate blend trials. Two or three samples depending on the agency are prepared at four levels of binder content: $P_{b,est}$-0.5%, $P_{b,est}$, $P_{b,est}$ +0.5% and $P_{b,est}$+1.0%. The samples are compacted to N_d gyrations and a volumetric analysis is performed and the results are plotted. As with the Marshall procedure, the asphalt content corresponding to 4.0% air voids is determined. This asphalt content is used with other plots to determine the other volumetric properties. The design mixture must meet the requirements for N_i, VMA, VFA and dust to binder ratio as presented in Table 2.7. Two additional samples are mixed with the selected asphalt content and compacted to N_m to verify the mix meets the criteria.

2.3.5 MOISTURE SENSITIVITY OF DESIGN MIXTURE

The moisture sensitivity of the design mixture is evaluated by performing AASHTO T-283 on the design aggregate blend at the design asphalt content. Specimens are compacted to 7.0 ±1.0% air voids. The conditioned subset of three specimens is subjected to partial vacuum saturation followed by an optional freeze cycle, followed by a 24 hour heating cycle at 60^{o}C. The conditioned subset and unconditioned subset of three specimens each are tested to determine indirect tensile strengths. The moisture sensitivity is determined as a ratio of the tensile strengths of the conditioned subset divided by the tensile strengths of the control subset. The use of AASHTO T-283 is not required to design a Superpave mix. However, some method of moisture sensitivity should be employed (5).

Table 2.7 Superpave Volumetric Mix Design Criteria (WVDOH MP 401.02.28)

Design air voids				4.0%
Fines to effective asphalt ratio				0.6 - 1.2
Tensile strength ratio				80% minimum
VMA, %	Nominal maximum size			
37.5 mm	25 mm	19mm	12.5 mm	9.5 mm
11.0	12.0	13.0	14.0	15.0
Design ESALs (millions)	% Theoretical maximum specific gravity			Percent voids filled with asphalt (VFA)
	N_i	N_d	N_{max}	
<0.3	91.5	96.0	98.0	70-80
0.3 < 3	90.5	96.0	98.0	65-78
3 < 10	89.0	96.0	98.0	65-75
10 < 30	89.0	96.0	98.0	65-75
≥ 30	89.0	96.0	98.0	65-75

2.5 ASPHALT PAVEMENT ANALYZER

Ruts are depressions, which occur in the pavement’s wheel path. Traffic compaction or displacement of unstable material causes ruts. Negligible amount of rutting occurs in HMA surfaces due to continued densification under traffic after initial compaction during construction. Rutting in a pavement can be larger if the HMA layer, underlying layers, or the subgrade soil is overstressed and significant densification or shear failures occur. Some common mistakes made when designing the HMA mixes like selection of high asphalt content, use of excessive filler material (material passing #200 sieve), use of too many rounded particles in aggregates are all contributors to rutting in HMA (4). In recent years, the potential for rutting on the nation's highways has increased due to higher traffic volumes and the increased use of radial tires that typically exhibit higher inflation pressures (6).

The most common type of laboratory equipment that predicts field-rutting potential is a loaded wheel tester (LWT). The LWTs currently being used in United States include the Georgia Loaded Wheel Tester (GLWT), Asphalt pavement Analyzer (APA), Hamburg Wheel Tracking Device (HWTD), LCPC (French) Wheel Tracker, Purdue University Laboratory Wheel Tracking Device (PURWheel), and one-third scale Model Mobile Load Simulator (6).

The APA is the new generation of the Georgia Loaded GLWT and was first manufactured in 1996 by Pavement Technology, Inc. The APA is a multi-functional LWT that can be used for evaluating rutting, fatigue cracking, and moisture susceptibility of hot and cold asphalt mixes (2).

2.5.1 Evaluation of Permanent Deformation:

The standard method followed to determine rutting susceptibility using APA is developed by APAC Materials Services in ASTM format. Rutting susceptibility of mixes is estimated by placing beam or cylindrical samples under repetitive wheel loads and measuring the amount of permanent deformation under the wheel loads. Triplicate beam samples or cylindrical samples can be tested in APA under controllable high temperature and in dry or submerged-in water conditions. The rut depth is measured after the desired number of cycles (usually 8000) of load application (11). The Table 2.8 shows the test parameters specified in the APAC procedure.

Table 2.8 APA Specifications (APAC Procedure)

Factors	**Range specified in APAC procedure**
Air void content	7 ±1 %
Test temperature	Based on average high pavement temperature
Wheel load	100 ±5 lb
Hose pressure	100 ±5 psi
Specimen type	Beams, cylinders
Compaction	Rolling, vibratory, and gyratory

2.5.2 Results of Ruggedness Study of the APA

APAC Materials Services conducted a ruggedness study of the Asphalt Pavement Analyzer rutting test in 1999 (11). Six factors; air void content of the test specimens, the test temperature, specimen preheating time, wheel load, hose pressure, and specimen compaction method were investigated and the following conclusions and recommendations were made regarding the specifications of the standard procedure.

1. The air void content range of the specimens given in the procedure should be changed from 7 ±1.0% to 7 ±0.5%. This will reduce the variability in the test results but it will also affect the productivity of the labs because of having to discard samples out of the tighter range.

2. The test temperature has a major effect on the test results. Proper calibration of the APA chamber temperature and ovens for preheating specimens is critical in obtaining meaningful test results.

3. Compaction and specimen type had a significant effect on the test results.

4. The ranges evaluated in the study for preheat time, wheel load, and the hose pressure did not have a significant effect on the rut results. Therefore, it was concluded that the standard procedure gives adequate guidance on the control of each of these test parameters.

2.5.3 Effect of Compaction and Specimen Type

The results from the ruggedness study of the APA shows that for the low rut potential mixes, the Superpave Gyratory Compactor (SGC) cylinders tended to have higher rut depths, while for the high rut potential mixes the Asphalt Vibratory Compactor (AVC) beams yielded collectively higher rutting. The reasons postulated for this behavior are:

- In high rut potential mixes, the center part of the cylinder mold supports the load when the rut depth advances to the depth of the contour at the center of the mold (12).
- The compaction modes of cylinder and beam samples may produce different density gradients in the samples (12).

The evaluation of density gradients in APA samples shows that vibratory compaction tends to result in more compaction at the top and less compaction at the bottom for both beams and cylinders. Gyratory compacted samples showed less compaction in the top and bottom of samples and significantly more compaction in the middle. The top of the AVC specimens should be loaded in APA. This type of specification is not needed for SGC compacted samples as the density in the top and bottom layers were not significant not significantly different (12).

2.5.4 Effect of Position

Studies have shown that there is variation in the left, center and right rut depth measurements of the APA. Although the wheel loads are individually calibrated, it has been observed that the wheel loads are not necessarily independent. During calibration, the load applied by any wheel is affected by whether or not the other wheels are in loading or rest position indicating the is an interaction between the wheel loads.

2.6 CONCLUSIONS

The Marshall and Superpave mix design methods were briefly described. State highway agencies have some discretion in the specific details of how the methods are implemented. The specifics for the WVDOH method were presented since they are the methods and procedures followed during this research.

The literature on the APA demonstrates it is a useful device for evaluating the rutting potential of a mix. However, there is an inherent variability in the results produced with the device, particularly with respect to load position. The experimental plan developed for this research was designed to minimize the effect of the variability in the APA results.

CHAPTER 3 RESEARCH METHODOLOGY

3.1 INTRODUCTION

This research compares 19 mm nominal maximum aggregate size Superpave to Marshall Base II mixes in West Virginia. Mix designs were prepared for Heavy, Medium and Light traffic levels using the Superpave methodology and for Heavy and Medium traffic level using the Marshall methodology. The resulting Marshall mix designs were evaluated under the Superpave criteria and vice versa. APA samples were made using Gyratory Compactor to evaluate rutting potential of the mixes. The following sections of this chapter explain the laboratory-testing program conducted in the Asphalt Technology Laboratory of West Virginia University, Morgantown. The data and interim calculations for the aggregates are presented in Appendix A.

3.2 MATERIALS

The aggregate used in this research work was provided by J.F Allen Company, Buckhannon, WV. Four types of aggregates (#57, #8, #9, Limestone Sand) and bag fines were used to develop aggregate blends meeting the gradation requirements. The asphalt used for both Superpave and Marshall mix design methods was PG 70-22 obtained from Marathon, Ashland.

3.3 AGGREGATE PREPARATION

The aggregates were obtained from J.F. Allen Company. Three of the aggregate types were ASTM coarse aggregate sizes #57, #8, and #9. The fourth aggregate type was crushed limestone sand. The aggregates were processed by washing, oven drying and sieving. Dried aggregates were separated with a nest of sieves, consisting of: 1”, 3/4”, 3/8”, #4, #8, #16, #30, #50, and #200 and the material retained on each sieve and pan was placed in storage bins.

3.4 SIEVE ANALYSIS

Three samples of each aggregate type were tested to determine the amount of material passing the #200 sieve (ASTM C117) and to determine gradation (ASTM C136). The amount of material passing the #200 sieve was determined by washed sieving for each type of aggregate is shown in Table 3.1. The average gradation results for each aggregate type are shown in Table 3.2.

Table 3.1 Washed Sieve Analysis for Material Passing #200 Sieve.

Sample No.	#57	#8	#9	L. Sand
1	0.9	1.3	1.3	6.9
2	0.9	2.0	1.5	6.7
3	1.1	1.2	1.9	6.4
Average	1.0	1.5	1.6	6.7

Table 3.2 Dry Sieve Gradation Analysis Results.

Sieve No.	Sieve size	#57	#8	#9	Limestone Sand	Bag fines
		% passing	% passing	% passing	% passing	%passing
1”	25 mm	100	100	100	100	100
¾”	19 mm	84	100	100	100	100
3/8”	12.5 mm	36	100	100	100	100
½”	9.5 mm	14	96	100	100	100
#4	4.75 mm	2.4	14	69	99	100
#8	2.36 mm	1.6	3.5	5.0	77	100
#16	1.18 mm	1.4	2.3	2.1	45	100
#30	0.6 μm	1.3	1.9	1.9	29	100
#50	0.3 μm	1.2	1.7	1.8	17	97.0
#200	0.075 μm	1.0	1.5	1.7	6.6	92.0

3.5 SPECIFIC GRAVITY OF AGGREGATES

Three samples of each aggregate type and one sample of bag fines were tested to determine the specific gravity and absorption (AASHTO T85 for coarse aggregate AASHTO T84 for fine aggregate). The average specific gravity and absorption values are shown in Table 3.3.

Table 3.3 Specific Gravity and Absorption Values.

	#57	#8	#9	L. sand	Bag Fines
Bulk Specific Gravity	2.687	2.686	2.648	2.665	
Apparent Specific Gravity	2.731	2.734	2.751	2.732	2.680
% Absorption	0.6	0.7	1.4	0.9	

3.6 AGGREGATE CONSENSUS PROPERTY TESTS

The aggregate properties that are specified as a result of the SHRP program are the coarse and fine aggregate angularity, flat and elongated particles, and sand equivalent results. Three samples each of #57 and #8 aggregate were tested to determine the average coarse aggregate angularity (ASTM D5821). Three samples each of #8, #9 and Limestone Sand were tested to determine the Uncompacted Void Content of Fine Aggregate (AASHTO T304 Method A). Following the general procedures of ASTM D 4791, three samples of #57 and #8 aggregate were tested to determine the average Flat and Elongated particles whose maximum to minimum dimension is greater than 5. Three samples of Limestone Sand were tested to get an average sand equivalent value (AASHTO T176). The aggregate average consensus property test results are shown in Table 3.4. The aggregate blends met the Superpave mix design criteria.

Table 3.4 Average Consensus Property Test Results for Individual Aggregates

Property	#57	#8	#9	L.Sand
Coarse aggregate angularity	100/100	100/100		
Fine aggregate angularity		46.0	46.0	46.0
Flat & elongated particles	0	0		
Sand equivalent value			100	78.0

3.7 MARSHALL MIX DESIGN PROCEDURE

The design traffic levels used in Marshall mix design method were Medium Traffic (less than 3 million ESALs), and Heavy Traffic (greater than 3 million ESALs). The binder used in the mix design was PG 70-22 (obtained from the source Marathon/Ashland). The aggregate in the storage bins were combined as required to produce the required gradation. The steps followed in determining the two Marshall mix designs are explained in the following section. The summary of the tests performed in determining the Marshall mixes are presented in this section. The detailed test data and calculations are presented in Appendix B.

3.7.1 AGGREGATE GRADATION

The result of sieve analysis of aggregates shown in Table 3.2 were used to combine the aggregate to achieve an aggregate blend used by J.F. Allen Company for Marshall Base II mix:

Aggregate type	Percent of blend
#57	35.5
#8	17.0
#9	10.0
Sand	36.5
Bag house fines	1.0

The plot of gradation is shown in Figure 3.1 with control points. The aggregate blend was within the tolerance limits of Master Range and hence the aggregate structure meets the gradation requirements; Table 2.4

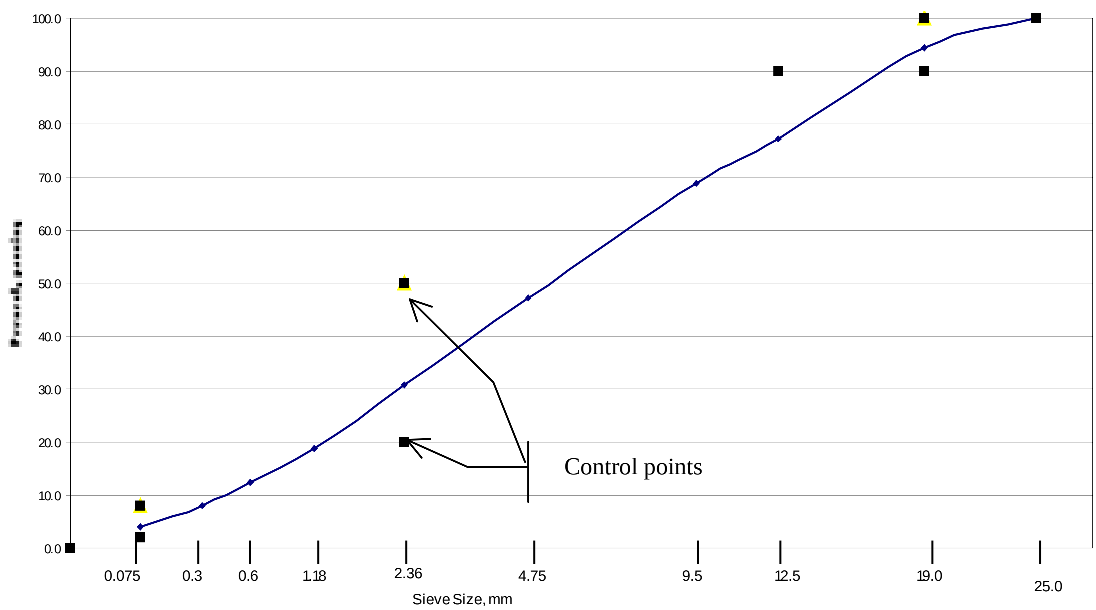


Figure 3.1 Marshall gradation plot with control points

3.7.2 SPECIFIC GRAVITY

The binder supplier provided the specific gravity of the asphalt binder and the value used was 1.020. The Equation 2.1 was used to calculate the specific gravity of the aggregate blend. The bulk specific gravity was 2.670 and the apparent specific gravity was 2.733 for the blend.

3.7.3 THEORETICAL MAXIMUM SPECIFIC GRAVITY

From the experience of J.F.Allen Company, the optimum asphalt content for Base II Marshall mix in West Virginia was 4.9% for Heavy traffic level. As described in Chapter 2, the maximum theoretical specific gravity is measured at one asphalt content and used to compute Gmm at the other asphalt contents. The validity of this approach was tested by performing the Gmm procedure on three samples at five asphalt contents, 3.9%, 4.4%, 4.9%, 5.4%, 5.9% for the heavy traffic mix and 4.4%, 4.9%, 5.4%, 5.9%, 6.2% for the medium traffic mix. Table 3.6 shows the results for the laboratory test results, and the computed approach. The computed and measured values compare favorably. The lab-measured values were used for the Marshall volumetric analysis.

Table 3.6 Maximum Theoretical Specific Gravity for Marshall Mix Designs.

Heavy traffic			Medium traffic		
Asphalt content %	Average G_{mm} Lab	Computed G_{mm}	Asphalt content %	Average G_{mm} Lab	Computed G_{mm}
3.9	2.550	2.552	4.4	2.533	2.529
4.4	2.533	2.533	4.9	2.513	2.513
4.9	5.513	-	5.4	2.491	-
5.4	2.491	2.494	5.9	2.476	2.472
5.9	2.476	2.475	6.2	2.452	2.453

3.7.4 MARSHALL STABILITY AND FLOW TEST

Three samples at each asphalt content were mixed and compacted using Marshall Hammer. The samples were 1100g and 1150g for the medium and heavy traffic levels respectively. The corresponding compaction level was 50 and 75 blows per side. The bulk specific gravity of each compacted sample was determined (AASHTO T166) and the average volumetrics at each asphalt content were computed using the equations 2.3 to 2.5. The Marshall Stability and Flow of each sample was determined (AASHTO T245).

3.7.5 TABULATING AND PLOTTING TEST RESULTS

The volumetric data and Stability and Flow test results were tabulated and the Stability values for specimen height different than 63.5 mm were corrected using Table 2.3. The average value for volumetric parameters, stability and flow of each set of three specimens was calculated. The tabulated results for Heavy and Medium traffic level Marshall mix designs are presented in Tables 3.7 and Table 3.8 respectively. The following plots were prepared from the results and are presented in Appendix C.

- Asphalt content versus density
- Asphalt content versus Marshall stability
- Asphalt content versus flow
- Asphalt content versus air voids.
- Asphalt content versus VMA
- Asphalt content versus VFA

Table 3.7 Volumetric Parameters for Marshall Heavy Traffic Mix

AC, %	Density	%VMA	%VFA	Air voids,%	Stability (Newton)	Flow (0.25 in)
3.9	2347	15.54	48.68	7.97	2230.2	14.5
4.4	2390	14.42	60.9	5.64	2180.4	17
4.9	2402	14.44	69.46	4.41	2017.8	12.7
5.4	2394	15.19	74.28	3.91	2008.9	16.3
5.9	2392	15.68	78.47	3.38	2318.3	15.3

Table 3.8 Volumetric Parameters for Marshall Medium Traffic Mix

AC, %	Density	%VMA	%VFA	Air voids,%	Stability (Newton)	Flow (0.01 in)
4.4	2359	15.5	55.5	6.9	2099	15.0
4.9	2362	15.8	61.4	6.1	2163	17.0
5.4	2377	15.8	71.0	4.6	2062	19.0
5.9	2395	15.6	79.0	3.3	2073	14.3
6.4	2396	16.0	85.7	2.3	2046	20.0

3.7.6 DETERMINATION OF DESIGN ASPHALT CONTENT

The asphalt content that corresponds to the specification's median air void content (4.0%) is the design asphalt content. The design asphalt content was determined from the plots for Heavy and Medium traffic level. The Marshall stability, flow, VMA and VFA at the design asphalt content were determined from the plots and were compared to the specification values. The results are summarized in the Table 3.9. All the properties met the Marshall criteria, Table 2.4.

Table 3.9 Summary of Marshall Mix Designs

Traffic level	AC, %	Density	%VMA	%VFA	Stability (Newton)	Flow (0.01 in)
Medium	5.5	2405	14.5	68.0	2025	15
Heavy	5.0	2374	15.7	71.0	2100	17

3.8 SUPERPAVE MIX DESIGN PROCEDURE

The design traffic levels used in Superpave mix design were Light Traffic (<0.3 million ESALs), Medium Traffic (0.3 to <3 million ESALs), Heavy Traffic (3 to <30 million ESALs). The asphalt binder used in the mix design was PG 70-22. The design aggregate structure for the heavy traffic level was established following the procedures presented in Chapter 2. This design aggregate structure was evaluated for the other traffic levels by preparing samples at estimated asphalt content, compacting with the appropriate number of gyrations, then performing the volumetric analyses. This process demonstrated the same design aggregate structure could be used for all traffic levels. The binder content was then determined for each traffic level.

3.8.1 AGGREGATE TRIAL BLENDS

Three trial blends were evaluated to determine the best aggregate structure. The percent of each aggregate type in the trial blends are shown in Table 3.10; the gradation chart is shown in Figure 3.2. All the three trial blends were within the control points of 19mm nominal maximum size and below the restricted zone (Table 2.5). The restricted zone is meant to be a guide to help ensure that too much natural sand is not used in the mixture and to help ensure that minimum VMA requirements are met.

The aggregate consensus property test values for aggregate blends and the combined average aggregate bulk and apparent specific gravities were determined using the Equation 2.8 and 2.2, respectively, and are presented in Table 3.11

The initial binder contents for heavy-traffic level mix design were computed using the Equations 2.9 to 2.13. The initial binder content was 4.4% for blend 1 and 4.5% for blend 2 and blend 3.

Three samples for each trial blend at their corresponding initial binder content were tested to determine the average Theoretical Maximum Specific Gravity (AASHTO T209) in Table 3.12.

3.8.2 DESIGN AGGREGATE STRUCTURE

Three samples for each trial blend at their corresponding initial binder content were compacted to 100 gyrations, which is N_d for heavy traffic level. The bulk specific gravity (AASHTO T 166) and volumetrics (Equations 2.3 to 2.5) of the compacted samples were determined and the average values are presented in Table 3.13.

The estimated design asphalt content was computed using Equation 2.17. The volumetric data at the estimated design asphalt content for all three blends were computed (Equations 2.18 to 2.21) and summarized in Table 3.14.

All the three blends meet the Superpave volumetric criteria, Table 2.3. The trial Blend 1 was selected as design aggregate structure for Heavy Traffic level Superpave mix design because it was used by J.F. Allen Company for a Superpave project in West Virginia and has proven performance.

The initial binder content of Superpave mix design for medium and light traffic levels was estimated to be 5.0%. Three samples with blend 1 were tested to determine the average Theoretical Maximum Specific Gravity (AASHTO T209) at 5.0% asphalt content.

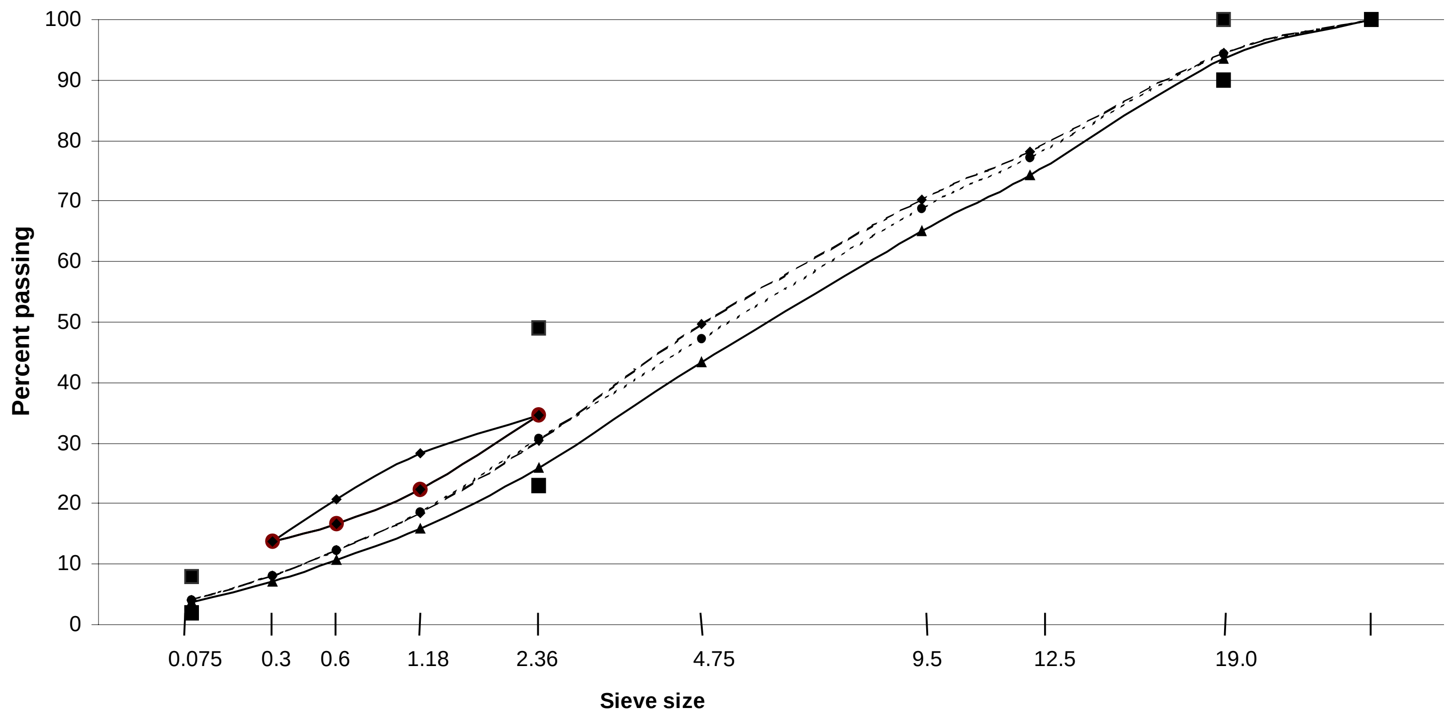


FIGURE 3.2 Superpave trial blend gradations

Table 3.10 Trial Blends for Evaluation of Aggregate Structure

Blend	#57	#8	#9	L. Sand	Bag fines
1	34.0%	14.0%	15.0%	36.0%	1.0%
2	35.5%	17.0%	10.0%	36.5%	1.0%
3	40.0%	15.0%	14.0%	30.0%	1.0%

Table 3.11 Trial Blends Bulk and Apparent Specific Gravity

	Blend 1	Blend 2	Blend 3
Bulk Specific Gravity	2.669	2.669	2.672
Apparent Specific Gravity	2.712	2.724	2.724

Table 3.12 Theoretical Maximum Specific Gravity for Superpave Trial Blends

Sample no	Blend 1	Blend 2	Blend 3
1	2.539	2.536	2.536
2	2.538	2.534	2.533
3	2.538	2.534	2.533
Average	2.538	2.535	2.535

Table 3.13 Trial Blends Compaction Data

Blend no.	G_{mb}	P_b	VTM	%Gmm at N_d
1	2.384	4.4	6.1	93.9
2	2.361	4.5	6.9	93.1
3	2.370	4.5	6.5	93.5

Table 3.14 Adjusted Volumetric Parameters for Superpave Trial Blends

Blend	Est Design AC	Est % VMA	Est % VFA	Est D/B ratio
1	5.2	14.2	71.8	0.90
2	5.7	14.9	73.2	0.84
3	5.5	14.8	72.9	0.81

Samples at 5.0% asphalt content were compacted using 75 gyrations and 50 gyrations, which is the design number of gyrations for light and medium traffic level respectively. Two samples were prepared at each compaction level. The volumetric results determined using Equations 2.3 to 2.5 are shown in Table 3.15.

Table 3.15 Blend 1 Volumetric Parameters for Medium and Light Traffic Level Mix Designs

Traffic level	Avg. G_{sb}	Avg. G_{mm}	P_b	VTM	VMA	%Gmm at N_d
Medium	2.367	2.513	5.0	5.8	15.8	94.2
Light	2.513	2.513	5.0	7.6	17.3	92.4

The estimated design asphalt content for medium and light traffic level Superpave mixes was computed using Equation 2.17. The volumetric data at the estimated design asphalt content for the two traffic levels are computed (Equations 2.18 to 2.21) and presented in Table 3.16.

Table 3.16 Adjusted Volumetric Parameters for Medium and Light Traffic Level Mix Designs

Traffic level	Estimated Design AC	Estimated % VMA	Estimated % VFA	Estimated D/B
Medium	5.7	15.4	75.9	0.81
Light	6.4	16.6	74.0	0.71

The volumetric parameters at the estimated design asphalt content met Superpave criteria and hence blend 1 was selected as design aggregate structure for the medium and light traffic level Superpave mixes.

3.8.3 DESIGN ASPHALT CONTENT

The design aggregate structure process showed the same aggregate blends could be used for all traffic levels. The estimated asphalt contents were 6.4%, 5.7%, and 5.2% for the light, medium and heavy traffic levels respectively. The first procedure for determining the design binder content is to measure G_{mm}. Since the binder content the medium and heavy traffic were separated by 0.5%, six binder contents cover the needed binder contents for G_{mm} testing; 4.7 to 6.7% at 0.5% increments. The G_{mm} for the low traffic level was determined for percent binder contents of 5.9 to 7.4% in 0.5% increments. Table 3.17 gives the average G_{mm} values at each asphalt content.

Table 3.17 Average G_{mm} Values for Superpave Mixes

Asphalt Content Percent	Average G_{mm}
4.7%	2.523
5.2%	2.502
5.7%	2.495
6.2%	2.485
6.7%	2.464
5.9%	2.487
6.4%	2.469
6.9%	2.452
7.4%	2.434

Two samples each at 4.7%, 5.2%, 5.7%, and 6.2% asphalt content were compacted to 100 gyrations. The average bulk specific gravities determined (AASHTO T166) and the volumetrics computed using Equations 2.2 to 2.5 at each asphalt content were tabulated in Table 3.18.

Table 3.18. Volumetric Data for Superpave Heavy Traffic Level.

Percent Asphalt Content	Air voids	% $G_{mm,Nd}$	VMA at N_d	VFA N_d	Avg G_{mb}
4.7	5.3	94.7	14.7	63.8	2.389
5.2	3.8	96.2	14.5	73.6	2.406
5.7	3.0	97.0	14.6	79.3	2.420
6.2	2.7	97.3	15.0	82.3	2.419

Two samples each at 5.2%, 5.7%, 6.2%, and 6.7% asphalt content were compacted to 75 gyrations. The average bulk specific gravities determined (AASHTO T166) and the volumetrics computed using Equations 2.2 to 2.5 at each asphalt content were tabulated in Table 3.19.

Table 3.19 Volumetric Data for Superpave Medium Traffic Level

Percent Asphalt Content	Air voids	% $G_{mm,Nd}$	VMA at N_d	VFA N_d	Avg G_{mb}
5.2	5.2	94.7	15.7	67.2	2.373
5.7	4.0	96.2	15.4	73.8	2.394
6.2	3.4	97.0	15.7	78.1	2.400
6.7	2.4	97.3	15.9	85.2	2.406

Two samples each at 5.9%, 6.4%, 6.9%, 7.4% asphalt content were compacted to 50 gyrations. The average bulk specific gravities determined (AASHTO T166) and the volumetrics computed using Equations 2.2 to 2.5 at each asphalt content were tabulated in Table 3.20.

Table 3.20 Volumetric Data for Superpave Light Traffic Level

Percent Asphalt Content	Air voids	% $G_{mm,Nd}$	VMA at N_d	VFA N_d	Avg G_{mb}
5.9	4.8	95.2	16.5	71.0	2.368
6.4	3.7	96.3	16.6	77.6	2.377
6.9	2.8	97.2	16.8	83.5	2.384
7.4	1.6	98.4	16.9	90.7	2.396

The volumetric data was plotted versus asphalt content for all three traffic design levels. The design asphalt content corresponding to 4.0% air voids was determined for three mixes. The volumetric properties at design asphalt content were determined from the plots. The summary of the three mix designs is shown in Table 3.21.

Table 3.21 Summary of Superpave Mix Designs.

Design Traffic Level	Design AC	% $G_{mm,Nd}$	VTM	VMA	VFA
Heavy	5.1	96.0	4.0	14.5	70.2
Medium	5.7	96.0	4.0	15.4	73.7
Traffic	6.2	96.0	4.0	16.6	75.0

The complete data and interim calculations of Superpave mix designs are presented in Appendix C.

3.9 SPECIMENS FOR APA TESTING

The Superpave Gyratory Compactor was used to make the specimens for testing with the APA. The APA specimens were made to 95 mm height with 7.0 ±0.5 % air voids range. The specimens out of this air voids range were discarded.

3.10 ASPHALT PAVEMENT ANALYZER RUNS

The APA procedure described in Chapter 2 was followed to obtain rut depth results of the specimens. All samples were tested at 140^{o}F with a hose pressure of 100 psi and a wheel load of 100 lb. Rut depths were measured after 8000 cycles.

There are 18 combinations of factors and levels. The combinations of mix design and traffic levels were assigned a treatment number as defined in Table 3.19. Two sets of test results were obtained for each combination, providing 36 experimental units. Each test result is an average of two test specimens, one in the front position of the mould and the other in the back. The factor levels in the experimental design were:

Factors	Levels
Mix design method	Superpave, Marshall
Traffic level	Light, Medium, High
APA position	Left, Center, Right

Table 3.19 Treatments Used in Experimental Design

Treatment No.	Type of mix design	Design traffic level
1	Superpave	Heavy
2	Superpave	Medium
3	Superpave	Light
4	Marshall	Heavy
5	Marshall	Medium
6	Marshall	Light

The WVDOH uses the medium traffic level for all pavements with less than 3 million ESALs. Thus, the Marshall medium traffic level mix design was used to prepare the samples for the Marshall light traffic level. Since the Marshall light and medium traffic level mix designs were equivalent one would expect similar performance.

To minimize the effects of position in the APA machine and test sequence, the order of testing the specimens was randomized. The Table 3.20 presents the order of testing the specimens and average rut depth results.

Table 3.20 Rut Depth Results.

	Positions		
	Left	Center	Right
APA TEST SEQUENCE	1	2	3
1	5.59_{1*}	6.16_{2}	9.39_{3}
2	9.34_{6}	4.73_{1}	9.32_{5}
3	6.26_{5}	5.80_{4}	4.75_{6}
4	9.39_{3}	9.17_{5}	6.45_{3}
5	8.39_{2}	4.61_{6}	5.03_{4}
6	5.26_{3}	3.89_{4}	6.90_{1}
7	4.09_{1}	6.54_{2}	6.58_{6}
8	6.32_{4}	6.53_{3}	6.02_{2}
9	9.29_{6}	5.09_{5}	5.62_{4}
10	6.83_{2}	3.62_{1}	7.68_{5}
11	5.20_{4}	5.49_{3}	6.58_{2}
12	8.70_{5}	8.50_{6}	5.12_{1}

*Subscript indicates treatment number as defined in Table 3.19

3.11 TESTS FOR OTHER CRITERIA

Two samples for each mix design were evaluated to check for the other mix design criteria. Marshall samples were made using Superpave mix designs and checked for the criteria and vice versa. Both the mix designs passed the other mix design’s criteria. The data are presented in Appendix D.

CHAPTER 4 RESULT AND ANALYSIS

There are indications in the literature that Superpave mixes have lower optimum binder content than Marshall mixes. This was not the case for the mixes designed during the research. The Superpave mixes consistently had a greater binder content than Marshall mixes, ranging from 0.2% to 0.7% higher. The largest range was for the Superpave light traffic level versus the Marshall, since the WVDOH does not have Marshall procedure for light traffic level, the Superpave light traffic design was compared to a Marshall medium traffic design.

An Analysis of Variance shown in Figure 4.1 was performed using the SAS program. The factors of mix design type, test sequence, and position indicate that there is insufficient evidence to identify a difference. The traffic level indicates a difference at the 95% confidence level. The interaction effect of mix design type versus traffic level indicates there is not sufficient evidence to identify a difference.

The confidence interval for the difference in the means of two mixes is –1.98 to 0.62. This interval is not within the detection limits.

The SAS program was run to compare possible combinations of mix and traffic on one to one bases. The results were used to compute a confidence interval about the difference in the adjusted means as:

$$\left(\mu_1^{'} - \mu_2^{'}\right) \pm t_{\alpha/2,\,df} \times SE_{\mu_1 - \mu_2} \qquad \text{(Eqn. 4.1)}$$

where,

μ'_1=adjusted mean μ_1

μ'_2=adjusted mean μ_2

SE=standard error

If the test is conducted a number of times under the same conditions, the difference in the adjusted means would fall within this range, 95% of the time.

Fig 4.1 Output of Analysis of Variance

```
                              The GLM Procedure
                            Class Level Information
              Class         Levels    Values
              mix               2    1 2
              traf              3    1 2 3
              seq              12    1 2 3 4 5 6 7 8 9 10 11 12
              posit             3    1 2 3
                      Number of observations 36

                               The GLM Procedure
Dependent Variable: depth
                                     Sum of
      Source                 DF          Squares     Mean Square    F Value    Pr > F
      Model                  18       67.1728747       3.7318264       1.67    0.1491
      Error                  17       38.0637559       2.2390445
      Corrected Total        35      105.2366306

               R-Square     Coeff Var      Root MSE     rutd Mean
               0.638303      22.99807    1.496344      6.506389

      Source                 DF        Type I SS     Mean Square    F Value    Pr > F
      mix                     1       1.80902500      1.80902500       0.81    0.3813
      traf                    2      32.72620556     16.36310278       7.31    0.0051
      mix*traf                2       1.19761667      0.59880833       0.27    0.7685
      seq                    11      22.41098858      2.03736260       0.91    0.5514
      position                2       9.02903889      4.51451944       2.02    0.1638

      Source                 DF      Type III SS     Mean Square    F Value    Pr > F
      mix                     1       2.73985482      2.73985482       1.22    0.2841
      traf                    2      23.79695449     11.89847725       5.31    0.0161
      mix*traf                2       0.92319986      0.46159993       0.21    0.8157
      seq                    11      22.41098858      2.03736260       0.91    0.5514
      position                2       9.02903889      4.51451944       2.02    0.1638

                               The GLM Procedure
                       Level of             -----depth----------
                        Mix           N     Mean           Std Dev
                       1             18     6.28222222     1.58252425
                       2             18     6.73055556     1.89198145

                       Level of             -------depth-----------
                        traf          N     Mean           Std Dev
                       1             12     5.15916667     0.97764521
                       2             12     7.22833333     1.38264459
                       3             12     7.13166667     1.92986026


              Level of       Level of              ---------depth--------
              Mix            traf           N      Mean           Std Dev

              1              1              6      5.00833333     1.16425799
              1              2              6      6.75333333     0.85439257
              1              3              6      7.08500000     1.85536789
              2              1              6      5.31000000     0.83224996
              2              2              6      7.70333333     1.71297013
              2              3              6      7.17833333     2.17852626

                             The GLM Procedure
Dependent Variable: depth

Parameter      Estimate              Error          t Value        Pr>|t|
Mix            -0.68245326           0.61693654     -1.11          0.2841
```

Table 4.1 shows the range of the difference in adjusted means computed from the results of the SAS General Linear Models procedure. When zero is outside the range, there is an indication of consistent difference in the test result. The comparisons which doesn't include zero in its range are

- Superpave Light – Superpave Heavy
- Superpave Medium - Superpave Heavy
- Marshall Medium - Marshall Heavy
- Marshall Light - Marshall Heavy

These comparisons are consistent with the concept, mixes for higher traffic levels are designed for higher rut resistance.

Zero within the range for the comparisons of the different mix design methods for a given traffic level is consistent with the ANOVA result.

Table 4.1 Confidence Interval Computed for Various Parameters

Comparison		SAS program Results		Confidence interval range	
μ_1	μ_2	Estimate	Standard Error	Lower limit	Upper limit
SPL	ML	0.515	1.120	-2.88	1.85
SPL	MM	0.641	1.052	-2.86	1.58
SPL	MH	-0.803	0.996	-1.75	2.43
SPM	ML	0.176	1.005	-2.29	1.94
SPM	MM	0.302	1.026	-2.47	1.86
SPM	MH	-1.142	0.953	-0.87	3.15
SPH	ML	2.549	0.958	-4.57	-0.53
SPH	MM	2.675	0.953	-4.69	-0.66
SPH	MH	1.231	1.020	-3.38	0.92
SPL	SPM	0.339	0.990	-1.75	2.43
SPL	SPH	-2.034	1.041	2.60	6.67
SPM	SPH	2.034	1.040	-4.23	-0.16
ML	MM	0.126	0.923	-2.073	1.821
ML	MH	-1.318	0.971	0.73	3.37
MM	MH	-1.444	0.993	0.65	3.54

CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

Based on the laboratory effort and statistical analysis of data, the following conclusions were made:

- The statistical analysis does not provide enough evidence to say that there is a difference in the Superpave and Marshall mix design methodologies.
- The ANOVA indicated a difference due to traffic levels. The multiple comparison procedure indicated that mixes designed for higher traffic levels are more rut resistant than mixes designed for lower traffic levels.
- The mixes prepared under the Superpave method passed the Marshall criteria.
- The mixes prepared under the Marshall method passed the Superpave criteria. This indicates that contractors using Marshall methodology to design and construct pavements should not face unusual difficulties with Superpave mixes.
- The asphalt contents of Superpave mix designs are higher than Marshall mix design for the same traffic level.

5.2 RECOMMENDATIONS

- Based on the information developed during this research, differences between the Marshall Base II and Superpave 19 mm mixes, evaluated for all traffic levels, could not be detected statistically. Thus, it would appear that Superpave implementation could proceed with concerns about pavement performance.
- The fact that the Superpave mixes required slightly higher asphalt content would increase the cost of the mixes. For non-NHS roads, this may be beneficial as pavement performance, other than rutting, is generally improved with the higher asphalt film thickness associated with higher asphalt contents.

5.3 FUTURE RESEARCH

- Similar analysis could be done using PG 64-22, which is generally used for non-NHS roads in WV.
- More aggregate types should be studied to have a broader perspective. This study was limited to one source of limestone aggregates. Other aggregates commonly used in the state should be studied, including other limestone sources and natural sands.
- Field tests on the rutting performance should be considered. The performance evaluation performed during this study is based on rutting potential as evaluated by the APA. An alternative method for estimating the performance of low volume mixes is needed, since rutting may not be the controlling performance parameter on low volume roads.

REFERENCES

1. Huber G.A., Jones J. Chris, Messersmith E. Paul, Jackson N. Mike. *Contribution of Fine Aggregate Angularity and Particle Shape to Superpave Mixture Performance.* Transportation Research Record 1609. 1997.

2. Kandhal P.S, Mallick B.R. *Evaluation of Asphalt Pavement Analyzer for HMA Design.* National Center for Asphalt Technology. April 1999.

3. Mallick B. Rajib, Buchanan Shane, Brown E.R, Huner Mike. *An Evaluation of Superpave Gyratory Compaction of Hot Mix Asphalt. National Center for Asphalt Technology.* May 1998.

4. Roberts L. Freddy, Kandhal S. Prithvi, Brown E. Ray, Lee Dah-Yim, Kennedy W. Thomas. *Hot Mix Asphalt Materials, Mixture Design and Construction.* NAPA Education Foundation, MD.1996.

5. Harman Thomas, D’Angelo John, Bukowski John. *Superpave Asphalt Mixture Design-Workshop.* Federal Highway Administration. 1999.

6. Cooley L.A. Jr., Kandhal P.S, Buchanan M.S, Fee F, Epps A. *Loaded Wheel Testers in the United States: State of Practice.* National Center for Asphalt Technology. April 2000.

7. *Background of Superpave Asphalt Mixture Design and Analysis.* Federal Highway Administration. Report number FHWA-SA-95-003, 1995.

8. *Implementation Survey IV, 1999*-2000. Federal Highway Administration. July 2000

   [http://ce.ecn.purdue.edu/~spave/Technical%20Info/Meetings/Imp99-00.htm]

9. *Ruggedness Evaluation of AASHTO TP4, The Superpave Gyratory Compactor.* The Superpave Asphalt research Program, The University of Texas, Austin. 1996.

   [http://www.utexas.edu/research/articles/sgcrugg.html]

10. Huber A. Gerald. *Development of the Superpave Gyratory Compactor.* The Superpave Asphalt Research Program, The University of Texas, Austin. 1996. [http://www.utexas.edu/research/superpave/articles/gyr-hist.html]

11. West C Randy. *A Ruggedness Study of the Asphalt Pavement Analyzer Rutting.* APAC Materials Service. May 1999.

12. Cooley L. Allen Jr., Kandhal S. Prithvi. *Evaluation of Density Gradients in APA Samples.* National Center for Asphalt Technology. October 1999.

SPECIFICATIONS

1. WVDOH Standard Specifications, 2000.

2. *Guide to Designing Hot-Mix Asphalt using the Marshall Design Method.* WVDOT MP 401.02.22. 2000

3. *Guide to designing Hot-Mix Asphalt using the Superpave Volumetric Design System.* WVDOT MP 401.02.28. 2000

4. *Bulk Specific Gravity of Bituminous Mixtures.* AASHTO T166. 1993.

5. *Maximum Specific Gravity of Bituminous Paving Mixtures.* AASHTO T209. 1994.

6. *Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus.* AASHTO T245. 1993.

7. *Method for Preparing and Determining the Density of HMA specimens by Means of the Superpave Gyratory Compactor.* AASHTO TP4. 2000

8. *Determining the Percentage of Fractured Particles in Coarse Aggregate.* ASTM D 5821.

9. *Flat or Elongated Particles in Coarse Aggregate.* ASTM D 4791.

10. *Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test.* AASHTO T 176.1986

11. *Uncompacted Void Content of Fine Aggregate.* AASHTO T304. 1998.

12. *Determination of Specific Gravity and Absorption of Coarse Aggregate.* AASHTO T85. 1991.

13. *Determination of Specific Gravity and Absorption of Fine Aggregate.* AASHTO T84. 1994.

14. *Amount of Material Passing #200 Sieve.* ASTM C117. 1984.

15. *Sieve Analysis.* ASTM C136. 1984.

APPENDIX A AGGREGATE TESTS AND PROPERTIES

Table A.1 Gradation Data for Aggregates

#8 Coarse Aggregate

Sample	Sample1			Sample 2			Sample 3		
SIEVE SIZE, mm	Wt retained	% Wt. retained	% Passing	Wt retained	% Wt. Retained	% Passing	Wt retained	% Wt. retained	% Passing
25	0	0	100	0	0	100	0	0	100
19	0	0	100	0	0	100	0	0	100
12.5	0	0	100	0	0	100	0	0	100
9.5	37	3.7	96	40.5	3.8	96	56.4	4.9	95
4.75	836	83	13	836.2	78.6	18	965.9	83.9	11
2.36	108	11	2.6	131	12	5.3	98	8.5	2.6
1.18	7.6	0.8	1.9	23	2.2	3.1	8	0.7	1.9
0.6	1.7	0.2	1.7	8.2	0.8	2.3	1.8	0.2	1.8
0.3	0.8	0.1	1.6	4.1	0.4	2.0	1.1	0.1	1.7
0.075	1.3	0.1	1.5	4.3	0.4	1.6	1.6	0.1	1.5
Pan	14.9	1.5		16.5	1.6		17.5	1.5	
Total	1007			1063.9			1150.7		

#9 Coarse Aggregate

Sample	Sample1			Sample 2			Sample 3		
SIEVE SIZE, mm	Wt retained	% Wt. retained	% Passing	Wt retained	% Wt. retained	% Passing	Wt retained	% Wt. retained	% Passing
25	0	0	100	0	0	100	0	0	100
19	0	0	100	0	0	100	0	0	100
12.5	0	0	100	0	0	100	0	0	100
9.5	0	0	100	0	0	100	0	0	100
4.75	175	32	68	183.2	32.3	68	157.8	28.6	71
2.36	346	64	4.1	359	63	4.5	360	65	6.3
1.18	13	2.3	1.8	14	2.5	2.0	21	4	2.5
0.6	0.4	0.1	1.7	1.2	0.2	1.8	2.4	0	2.0
0.3	0.1	0.0	1.7	0.2	0.0	1.8	0.7	0	1.9
0.075	0.2	0.0	1.7	0.7	0.1	1.7	1.4	0	1.7
Pan	9.1	1.7		9.4	1.7		9.2	1.7	
Total	543.6			567.4			552		

Table A.1 Gradation Data for Aggregates (continued)

SAND									
Sample	Sample1			Sample 2			Sample 3		
SIEVE SIZE, mm	Wt retained	% Wt. retained	% Passing	Wt retained	% Wt. retained	% Passing	Wt retained	% Wt. retained	% Passing
25	0	0	100	0	0	100	0	0	100
19	0	0	100	0	0	100	0	0	100
12.5	0	0	100	0	0	100	0	0	100
9.5	0	0	100	0	0	100	0	0	100
4.75	5.6	1.1	99	5.9	1.1	99	4.4	0.8	99
2.36	111	21	78	117	22	77	119	23	76
1.18	166	31	46	168	32	45	165	31	45
0.6	91	17	29	88	17	28	87	17	28
0.3	62	12	18	58	11	17	59	11	17
0.075	58	11	6.6	54	10	6.5	55	10	6.6
Pan	35	6.6		34	6.5		35	6.6	
Total	528.8			525.4			522.8		

Table A.2 Specific Gravity of Aggregates

	#57			#8		
	Sample No.			Sample No.		
	1	2	3	1	2	3
Dry Wt. (g), A	4886.3	3790.6	3789.8	2756.1	1746.7	1741.3
SSD Wt.(g), B	4918.2	3812.6	3811.3	2773.8	1758.2	1752.6
Wet Wt. (g), C	3098.8	2403.1	2400.6	1752.4	1106.4	1102.9
Bulk SG	2.686	2.689	2.686	2.698	2.680	2.680
A/(B-C)						
Apparent SG	2.734	2.732	2.728	2.746	2.728	2.728
A/(A-C)						
Absorption, %	0.7	0.6	0.6	0.6	0.7	0.6

Table A.2 Specific Gravity of Aggregates (continued)

	#9			Sand		
	Sample No.					
	1	2	3	1	2	3
Dry Wt. (g), A	495.7	495.4	496.6	493.1	493.6	493.4
Pycnometer Wt.(g), B	1448.2	1448.2	1448.2	1448.2	1448.2	1448.2
Wt. In water (g), C	1761.8	1762.5	1763.2	1761.9	1762.5	1762.3
SSD Wt.(g), D	500.3	500	501.1	500	500.6	500.5
Bulk SG	2.655	2.668	2.673	2.647	2.649	2.647
A/(B+D-C)						
Apparent SG	2.722	2.736	2.739	2.749	2.753	2.752
A/(B+A-C)						
Absorption,%	0.9	0.9	0.9	1.4	1.4	1.4

Table A.3. Aggregate Consensus Property Tests

Sample No.	Sand reading (A)	Clay reading (B)	Sand Equivalency (A-10)/B*100
1	13.4	4.3	79
2	13.5	4.5	78
3	13.5	4.5	78

Aggregate type	mass of fine aggregate (g)	Fine aggregate angularity
Sand	146.0	45.9
# 9	146.4	46.0
# 8	146.9	45.6

APPENDIX B MARSHALL MIX DESIGN DATA AND ANALYSIS

Table B.1 Theoretical Maximum Specific Gravity Calculations

AC	Sample	Dry Wt. (g)	Wet Wt. (g)	Container Wt. (g)	Gmm
3.9	1	2071.1	2769.2	1511.1	2.547
	2	2067.5	2766.9	1511.1	2.547
	3	2072.9	2771	1511.1	2.550
4.4	1	2073.1	2766.2	1511.1	2.534
	2	2072.8	2765.8	1511.1	2.534
	3	2075.3	2767.6	1511.1	2.535
4.9	1	2086.2	2767	1511.1	2.513
	2	2086.7	2769.9	1511.1	2.520
	3	2029.2	2733.5	1511.1	2.515
5.4	1	2095.5	2583.9	1511.1	2.049
	2	2093.2	2579.2	1511.1	2.042
	3	2097	2583.2	1511.1	2.046
5.9	1	2110.6	2588.2	1511.1	2.042
	2	2110.9	2587.7	1511.1	2.041
	3	2109.2	2587	1511.1	2.041
6.4	1	2111.3	2583.8	1511.1	2.033
	2	2110	2582.3	1511.1	2.031
	3	2114.4	2586.3	1511.1	2.035
6.9	1	2106.9	2576.6	1511.1	2.023
	2	2105.6	2575.2	1511.1	2.022
	3	2107.5	2577.3	1511.1	2.024
7.4	1	2092.5	2562.6	1511.1	2.010
	2	2098.6	2565	1511.1	2.009
	3	2102.4	2568	1511.1	2.011

Table B.2 Marshall Heavy Traffic Level Mix Design

Asphalt Content by Weight of Mix			Mass in Grams			Specific Gravity			Voids in	Percent Air Voids		Stability (n) Pounds		Flow
Specimen Number	Percent AC	Specimen Thickness (mm)	Weight in Air A	Weight In water C	Saturated surface dry B	Bulk D	Maximum E	Density Kg/m3	Mineral Aggregate F	Total Mix	Filled	Actual	Adjusted	0.01 inch
						A/(B-C)		D X 1000		100 X [(E-D)/E]	100 X [(F-G)/F]			
1	3.9	66	1187.9	686.5	1194.2	2.340						2425	2287.1	16.0
2	3.9	65	1186.6	688.3	1193.2	2.350						2350	2261.9	14.0
3	3.9	65	1185.6	687.8	1192.3	2.350						2225	2141.6	13.5
Average						2.347	2.55	2347	**15.54**	**7.97**	**48.68**	2333	2230.2	14.5
1	4.4	66	1184.3	692.5	1190.1	2.380						2000	1886.3	17.5
2	4.4	64	1184.6	694.8	1190.4	2.390						2350	2320.6	17.5
3	4.4	66	1181.2	695.1	1187.2	2.400						2475	2334.2	16.0
Average						2.390	2.533	2390	**14.42**	**5.64**	**60.90**	2275	2180.4	17.0
1	4.9	63	1187.2	694.8	1190.3	2.396						2025	2050.3	12.5
2	4.9	63	1174.5	689.5	1178.8	2.400						2125	2151.6	12.5
3	4.9	64	1190.4	701.5	1195.4	2.410						1875	1851.6	13.0
Average						2.402	2.513	2402	**14.44**	**4.41**	**69.46**	2008	2017.8	12.7
1	5.4	64	1182	696	1186.4	2.410						1975	1950.3	13.5
2	5.4	63	1180.7	690.7	1184.7	2.390						2000	2025.0	13.0
3	5.4	66	1179.4	688.1	1183.5	2.381						2175	2051.3	22.5
Average						2.394	2.491	2394	**15.19**	**3.91**	**74.28**	2050	2008.9	16.3
1	5.9	63	1188.9	695.3	1191.1	2.398						2450	2480.6	15.0
2	5.9	63	1185.3	693.2	1189.3	2.389						2300	2328.8	15.0
3	5.9	64	1184.5	692.6	1188.2	2.390						2275	2145.6	16.0
Average						2.392	2.476	2392	**15.68**	**3.38**	**78.47**	2342	2318.3	15.3

Table B.3 Marshall Medium Traffic Mix Design

Asphalt Content by Weight of Mix			Mass in Grams			Specific Gravity			Voids in Mineral Aggregate	Percent Air Voids		Stability (n) Pounds		Flow 0.01 inch
Specimen Number	Percent AC	Specimen Thickness (mm)	Weight in Air A	Saturated surface dry B	Weight in water C	Bulk D	Maximum E	Density Kg/m3	F	Total Mix G	Filled	Actual	Adjusted	
						A/(B-C)		D X 1000		100 X [(E-D)/E]	100 X [(F-G)/F]			
1	4.4	62	1136.7	1142.3	658.0	2.347						1950	2023	14
2	4.4	61	1140.7	1144.6	662.3	2.365						1975	2110	15
3	4.4	61	1138.1	1142.5	661.0	2.364						2025	2163	15
Average						**2.359**	**2.534**	**2359**	**15.55**	**6.92**	**55.49**	**1983**	**2099**	**15**
1	4.9	61	1145.7	1150.1	665.0	2.362						2025	2163	16
2	4.9	61	1141.9	1147.7	663.9	2.36						2050	2190	17
3	4.9	61	1141.5	1146.9	663.8	2.363						2050	2190	17
Average						**2.362**	**2.516**	**2362**	**15.88**	**6.14**	**61.37**	**2042**	2163	**17**
1	5.4	61	1139.5	1142.9	663.2	2.375						2000	2136	19
2	5.4	62	1140.9	1144.5	665.2	2.380						2000	2075	18
3	5.4	61	1139.2	1144.0	664.4	2.375						1850	1976	20
Average						**2.377**	**2.491**	**2377.000**	**15.78**	**4.58**	**71.01**	**1950**	2163	**19**
1	5.9	61	1143.8	1146.6	667.2	2.386						1950	2083	15
2	5.9	60	1138.0	1140.3	664.4	2.391						1975	2134	15
3	5.9	61	1144.6	1146.8	671.5	2.408						1875	2003	13
Average						**2.395**	**2.476**	**2395**	**15.59**	**3.27**	**79.04**	**1933**	**2073**	**14.3**
1	6.4	60	1137.6	1139.4	669.0	2.418						1825	1972	21
2	6.4	61	1138.5	1141.5	662.5	2.377						1975	2110	20
3	6.4	61	1150.9	1154.2	673.1	2.392						1925	2056	20
Average						**2.396**	**2.452**	**2396**	**16.01**	**2.29**	**85.69**	**1908**	**2046**	**20**

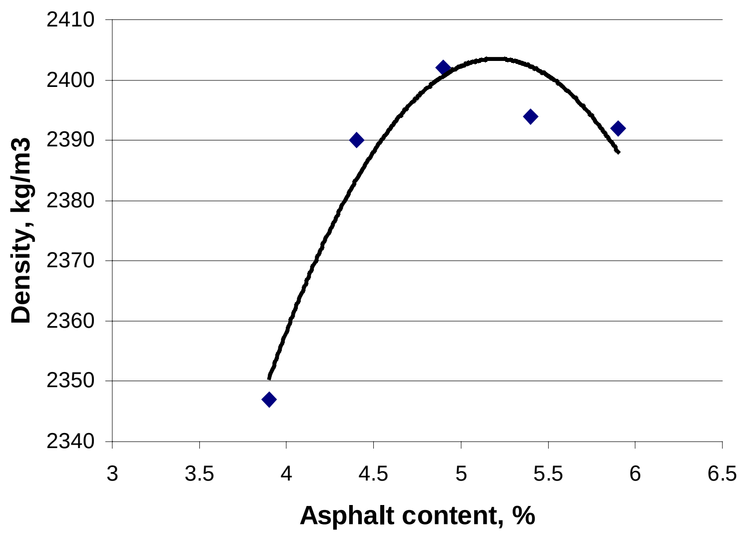


Figure B.1 Asphalt content versus Density – Marshall heavy

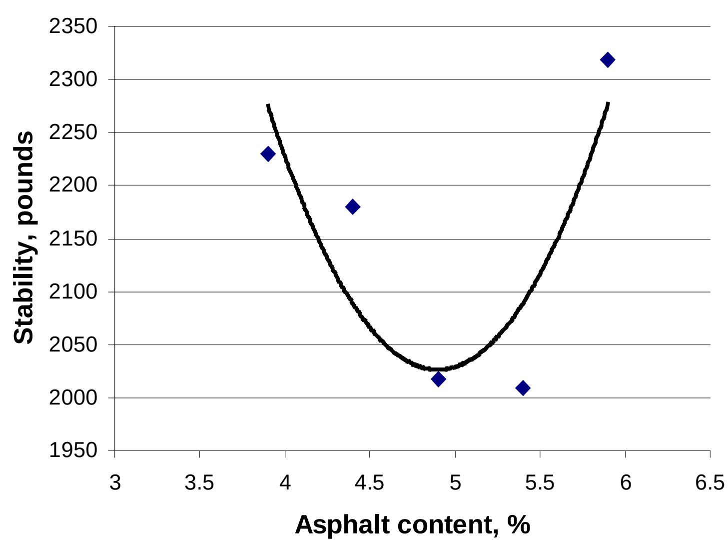


Figure B.2 Asphalt content versus Stability – Marshall heavy traffic mix.

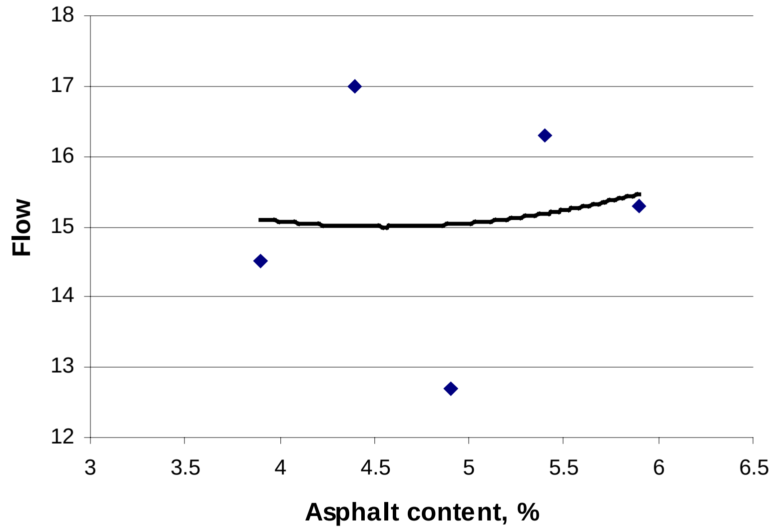


Figure B.3 Asphalt content versus Flow – Marshall heavy traffic mix

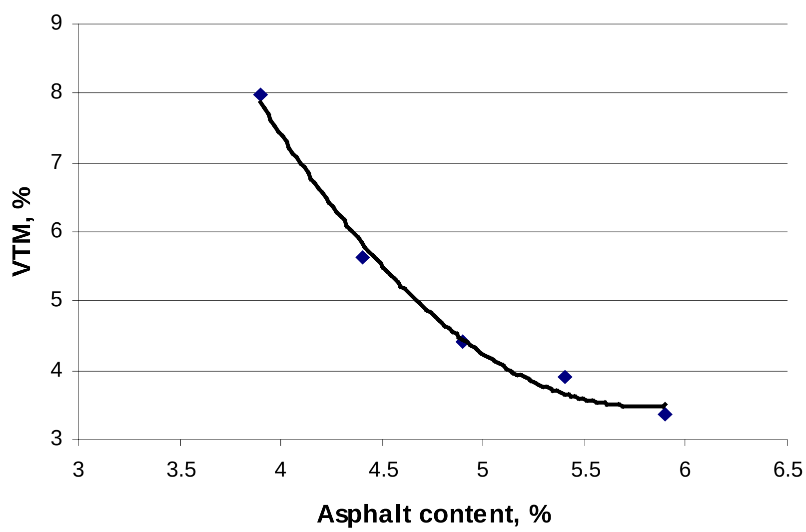


Figure B.4 Asphalt content versus VTM – Marshall heavy traffic mix

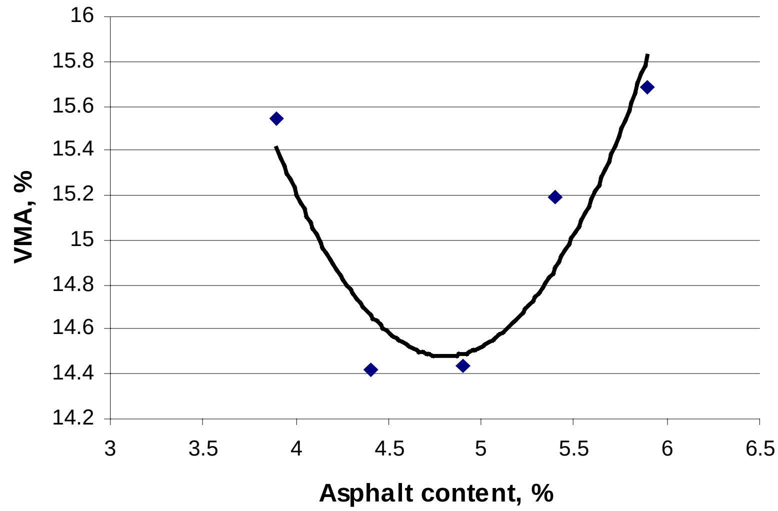


Figure B.5 Asphalt content versus VMA – Marshall heavy traffic mix

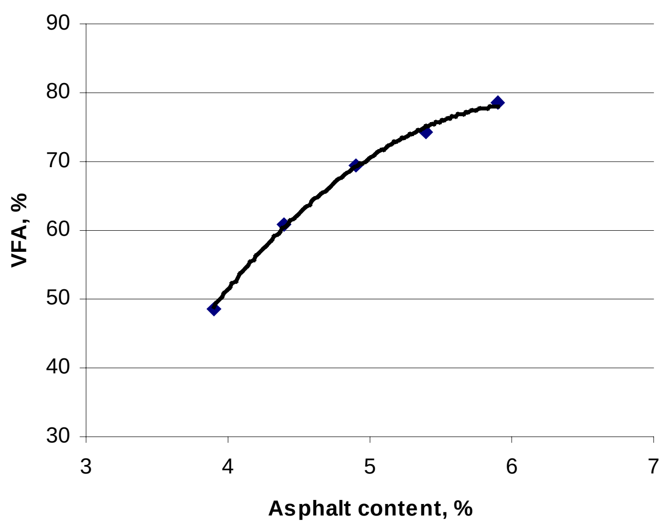


Figure B.6 Asphalt content versus VFA –Marshall heavy traffic mix.

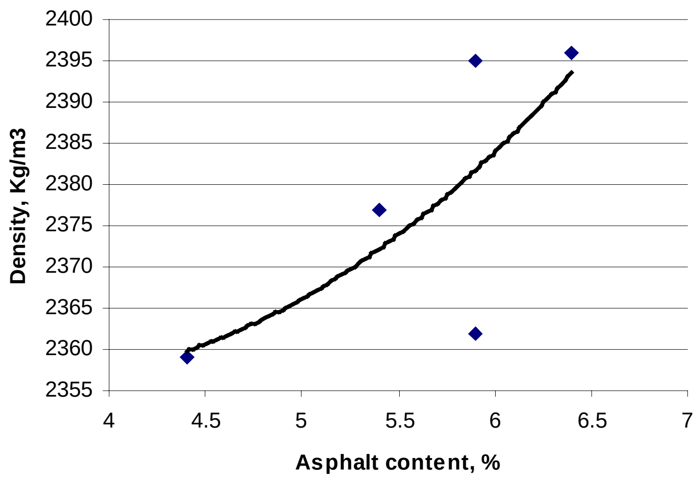


Figure B. 7 Asphalt content versus density, Marshall medium traffic mix.

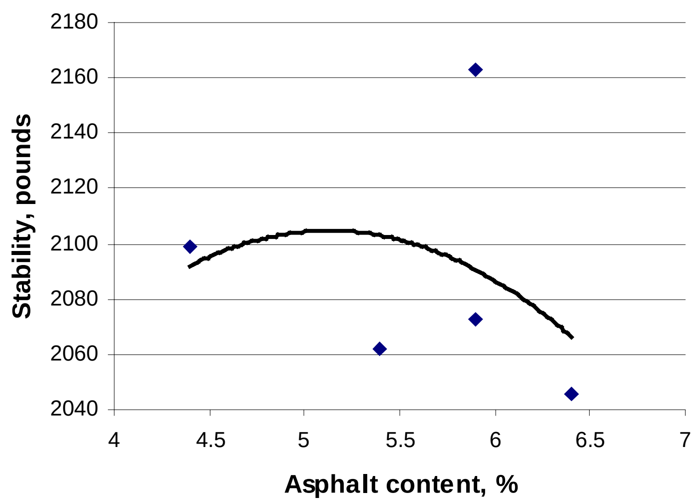


Figure B.8 Asphalt content versus Stability – Marshall medium traffic mix.

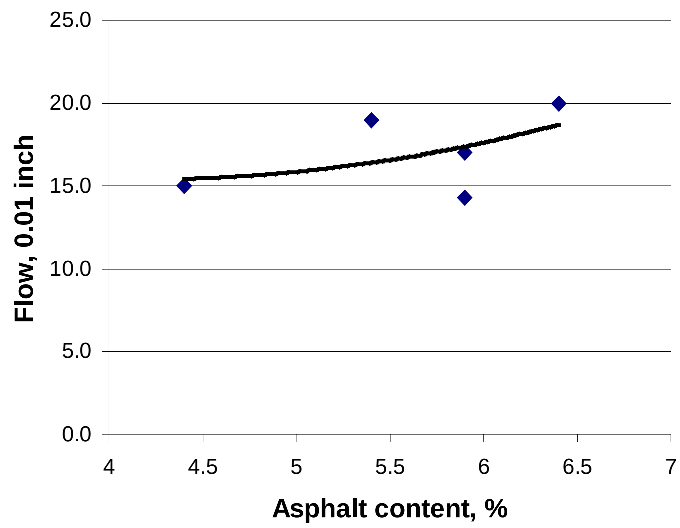


Figure B.9 Asphalt content versus Flow – Marshall light traffic level

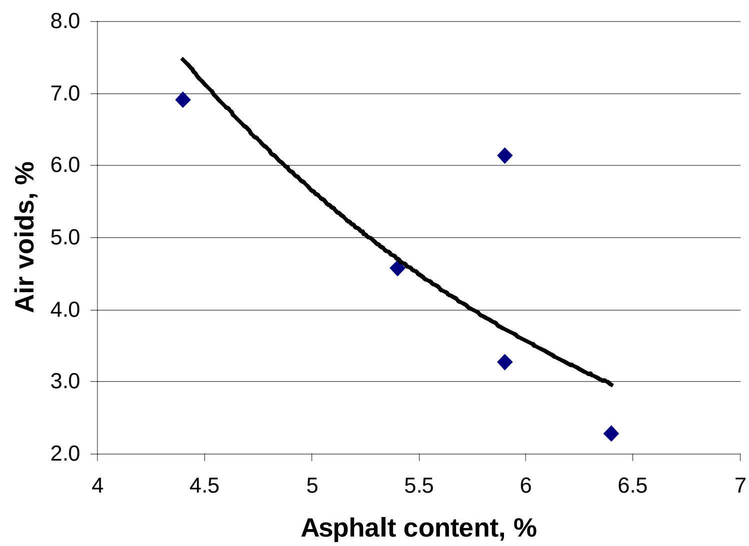


Figure B.10 Asphalt content versus Air voids –Marshall medium traffic mix.

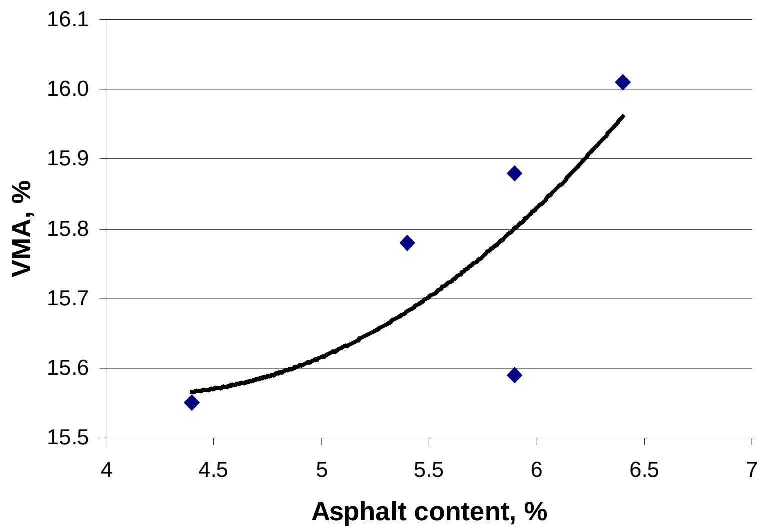


Figure B.11 Asphalt content versus VMA – Marshall light traffic mix.

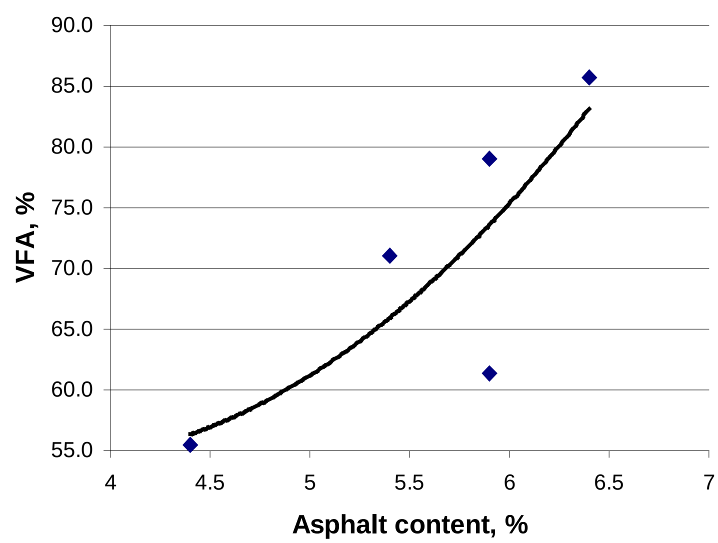


Figure B.12 Asphalt content versus VFA – Marshall

APPENDIX C SUPERPAVE MIX DESIGN DATA AND ANALYSIS

Table C.1 Theoretical Specific Gravity for Trial Blends

	Weight of the container in water, (g)[D] = 1511.1				
			A	**E**	**A/(A+D-E)**
BLEND	**AC,%**	**Sample**	**Dry wt.(g)**	**Wet wt.(g)**	**Gmm**
1	4.4	1	2231.8	2864.0	2.539
		2	2074.4	2768.2	2.538
		3	2077.2	2769.8	2.538
2	4.5	1	2081.5	2771.8	2.536
		2	2074.4	2766.9	2.534
		3	2077.5	2768.8	2.534
3	4.5	1	2097.7	2770.6	2.503
		2	2072.4	2771.5	2.552
		3	2081.9	2771.1	2.533

Table C.2 Bulk Specific Gravity for Trial Blends

		A	**B**	**C**	**A/(C-B)**
Blend	**Sample**	**Dry Wt. (g)**	**Wet Wt. (g)**	**SSD Wt. (g)**	**Gmb**
Blend 1	1	4767.6	2784.4	4788.1	2.379
	2	4770.7	2791.2	4787.9	2.389
	3	4778.2	2798.1	4803.3	2.383
Blend 2	1	4794.6	2795.0	4818.6	2.369
	2	4786.0	2786.0	4814.0	2.360
	3	4783.0	2781.0	4809.1	2.358
Blend 3	1	4941.4	2892.9	4968.4	2.381
	2	4781.4	2787.2	4805.0	2.370
	3	4767.5	2779.5	4798.5	2.361

Table C.3 Theoretical Maximum Specific Gravity Calculations for Superpave Heavy Traffic Mix

Weight of the container in water, (g) [D] =					1511.1
		A	E	A/(A+D-E)	
AC,%	Sample	Dry wt. (g)	Wet wt. (g)	G_{mm}	Avg G_{mm}
4.7	1	2110.4	2785.2	2.523	
	2	2009.6	2723.9	2.522	2.523
5.2	1	2021.6	2724.7	2.502	
	2	2022.4	2725.2	2.502	2.502
5.7	1	2098.5	2768.2	2.494	
	2	2095.6	2767.1	2.496	2.495
6.2	1	2090.8	2760.2	2.484	
	2	2099.2	2765.9	2.486	2.485

Table C.4 Bulk Specific Gravity for Superpave Heavy Traffic Level Mix

	Sample	**A**	**B**	**C**	**A/(C-B)**	
AC, %	**No.**	**Dry wt. (g)**	**Wet wt. (g)**	**SSD Wt. (g)**	**G_{mb}**	**Avg G_{mb}**
4.7	1	4774.8	2787.1	4791.2	2.383	
	2	4759.6	2788.6	4776.2	2.395	2.389
6.4	1	4801.0	2808.3	4810.1	2.398	
	2	4765.0	2796.7	4771.5	2.413	2.406
6.9	1	4800.9	2825.2	4806.6	2.423	
	2	4775.2	2804.9	4781.2	2.416	2.420
7.4	1	4760.9	2802.1	4766.5	2.424	
	2	4755.2	2791.0	4762.0	2.413	2.419

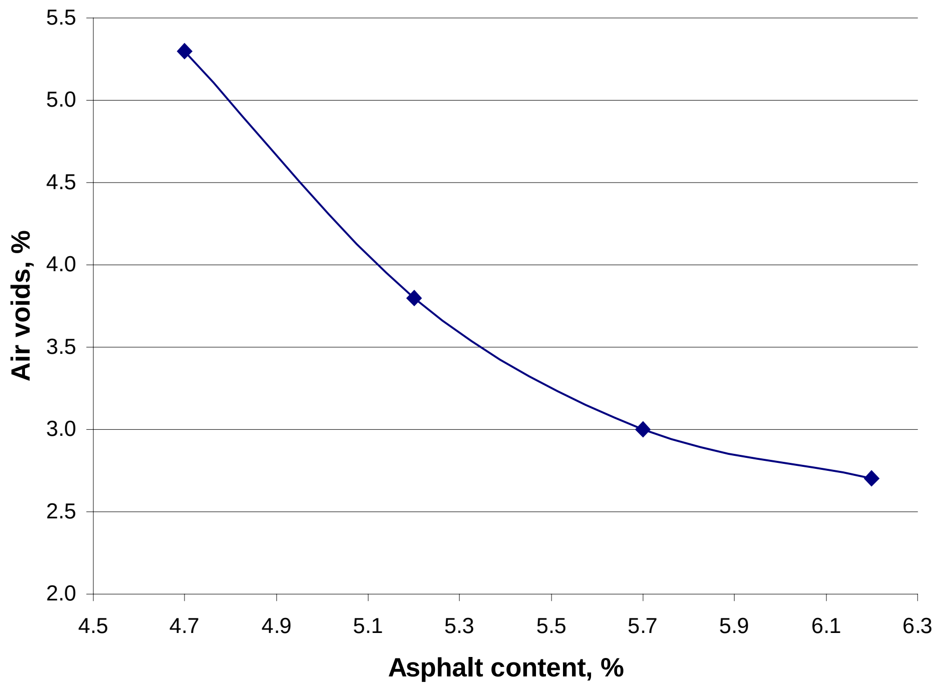


Figure C.1 Asphalt content versus Air voids-Superpave heavy traffic mix.

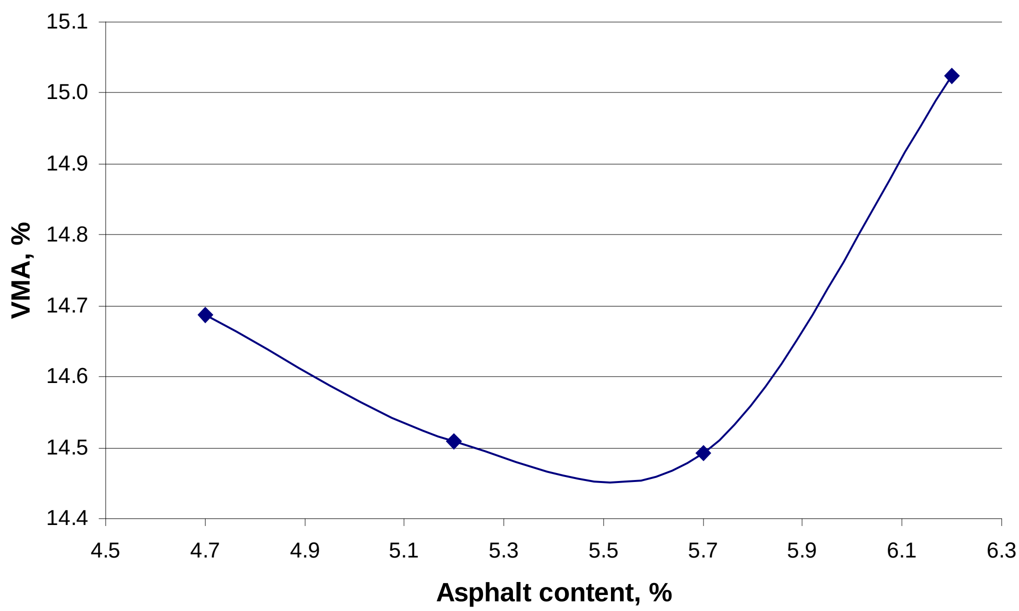


Figure C.2 Asphalt content versus VMA-Superpave heavy traffic mix.

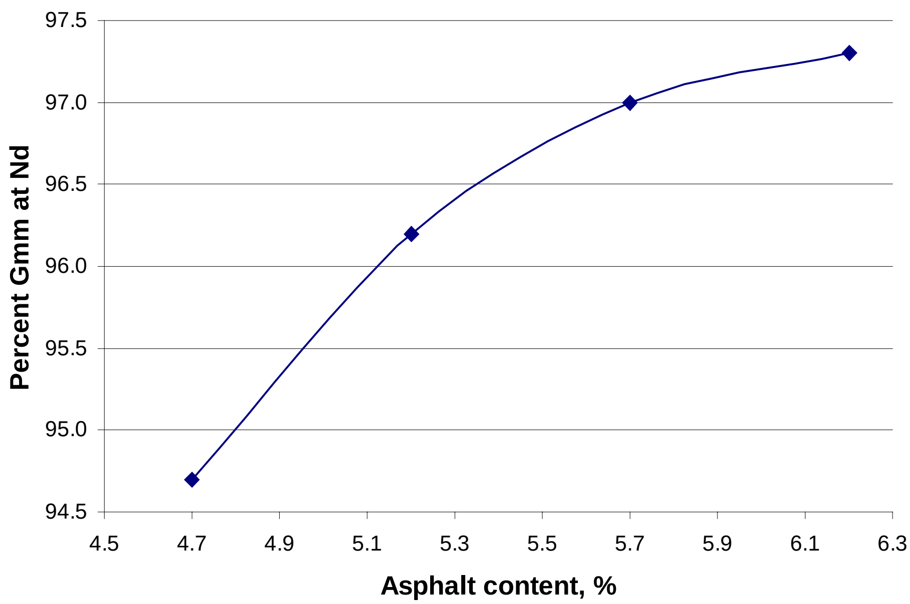


Figure C.3 Asphalt content versus Percent Gmm at Nd-Superpave heavy traffic mix

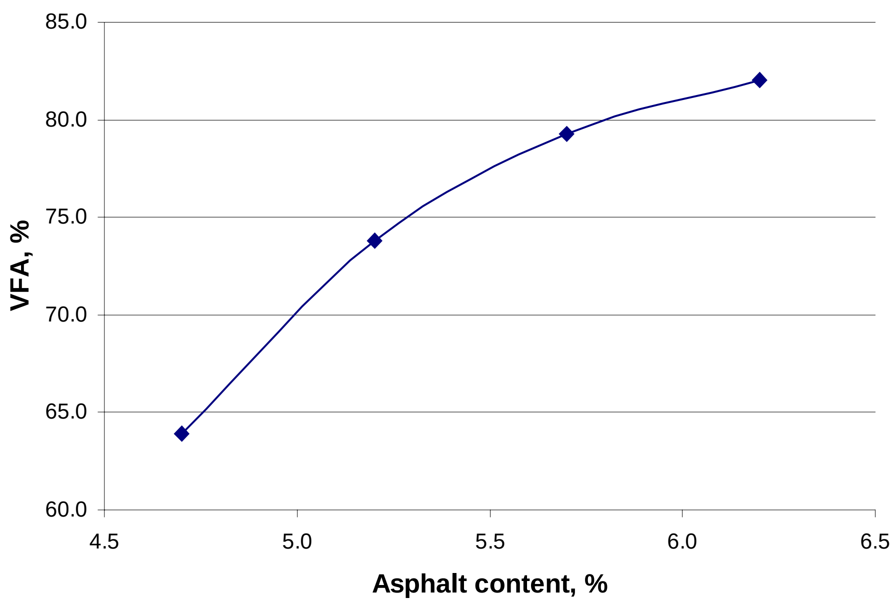


Figure C.4 Asphalt content versus VFA – Superpave heavy traffic mix.

Table C.5 Blend Bulk Specific Gravity Calculations –Superpave Medium Traffic Mix.

Sample	A	B	C	A/(C-B)
No.	Dry Wt. (g)	Wet Wt. (g)	SSD Wt. (g)	Gmb
1	4745.0	2742.5	4748.8	2.365
2	4753.2	2750.5	4768.8	2.355
3	4739.0	2762.9	4752.9	2.381

Table C.6 Theoretical Maximum Specific Gravity Calculations-Superpave Medium Traffic Mix

Weight of the container in water, (g) [D] = 1511.1					
		A	E	A/(A+D-E)	
AC,%	Sample	Dry wt. (g)	Wet wt. (g)	G_{mm}	Avg G_{mm}
5.2	1	2013.4	2719.1	2.500	
	2	2021.6	2725.4	2.504	2.502
5.7	1	2092.8	2764.8	2.494	
	2	2098.5	2768.9	2.496	2.495
6.2	1	2088.4	2758.4	2.483	
	2	2090.8	2761.2	2.487	2.485
6.7	1	2099.4	2759.6	2.467	
	2	2110.9	2763.9	2.460	2.464

Table C.7 Bulk Specific Gravity-Superpave Medium Traffic Mix.

	Sample	**A**	**B**	**C**	**A/(C-B)**	
AC, %	**No.**	**Dry wt. (g)**	**Wet wt. (g)**	**SSD Wt. (g)**	$\mathbf{G_{mb}}$	**Avg** $\mathbf{G_{mb}}$
5.2	1	4730.4	2749.6	4743.3	2.373	
	2	4716.2	2740.8	4729.3	2.372	2.372
5.7	1	4760.1	2773.7	4769.7	2.385	
	2	4747.1	2779.2	4754.8	2.403	2.394
6.2	1	4652.2	2736.9	4657.9	2.422	
	2	4707.0	2765.6	4713.0	2.417	2.419
6.7	1	4734.2	2770.0	4742.4	2.400	
	2	4751.4	2787.5	4757.0	2.412	2.406

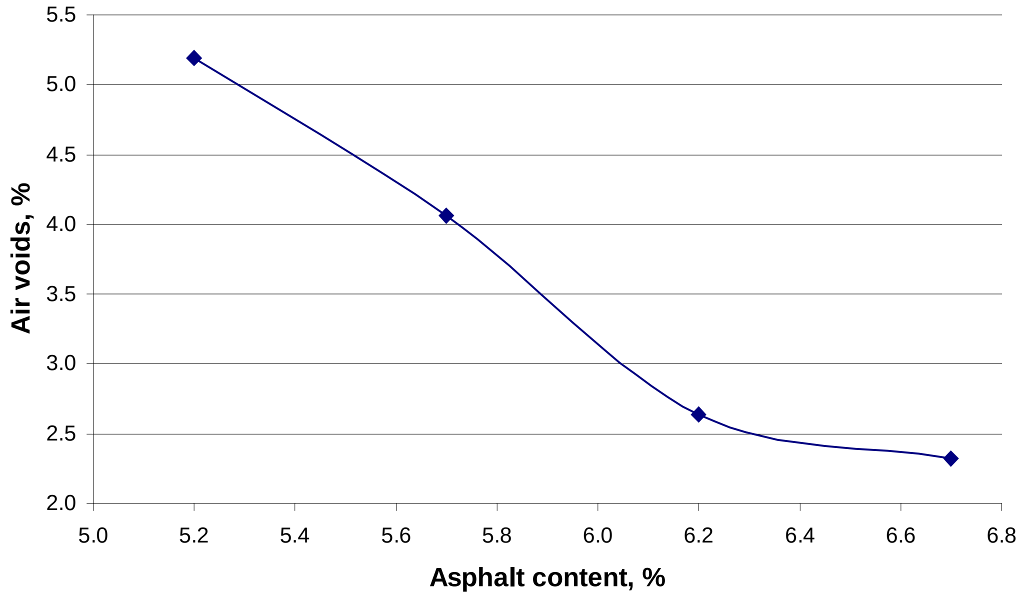


Figure C.5 Asphalt content versus Air voids-Superpave medium traffic mix.

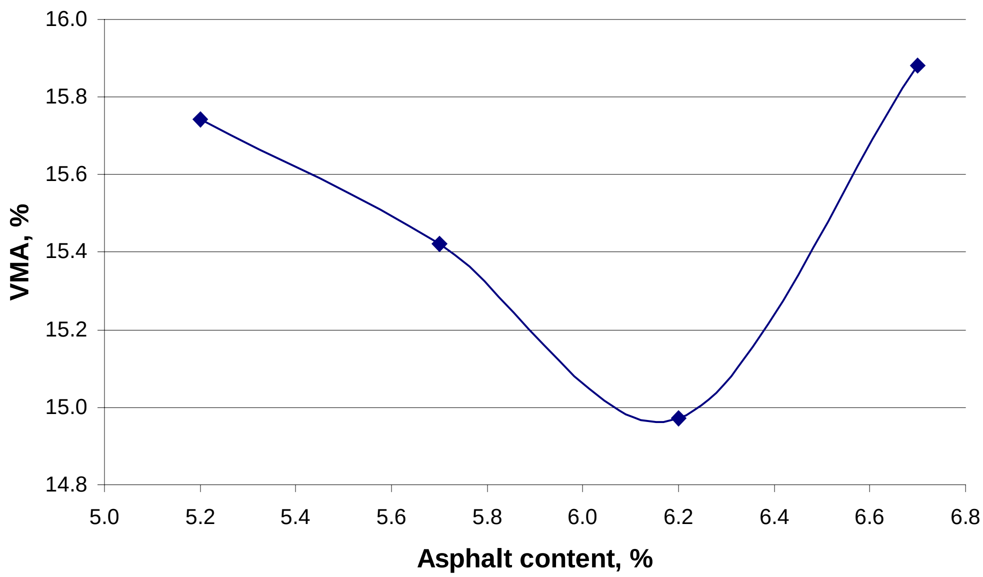


Figure C.6 Asphalt content versus VMA-Superpave medium traffic level

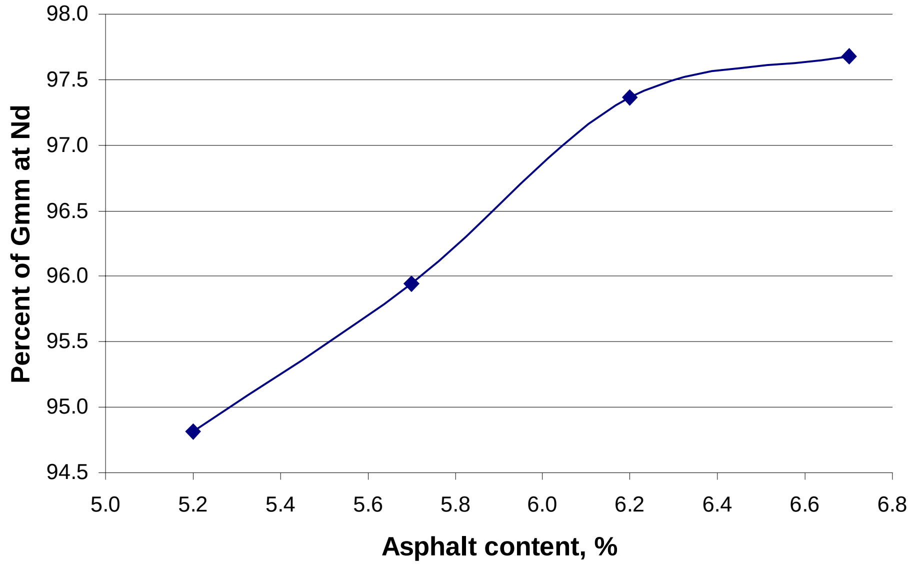


Figure C.7 Asphalt content versus Percent of Gmm at Nd-Superpave medium traffic mix

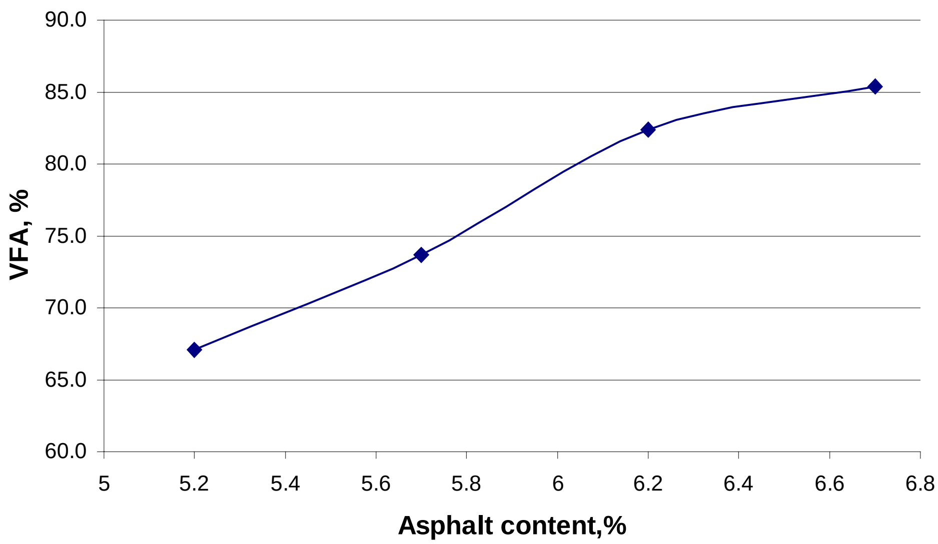


Figure C.8 Asphalt content versus VFA-Superpave medium traffic mix.

Table C.8 Blend Bulk Specific Gravity Calculations –Superpave Light Traffic Mix.

Sample	A	B	C	A/(C-B)
No.	Dry Wt. (g)	Wet Wt. (g)	SSD Wt. (g)	Gmb
1	4698.6	2704.4	4720.5	2.331
2	4684.5	2690.7	4711.9	2.318
3	4685.8	2701.1	4721.3	2.319

Table C.9 Theoretical Maximum Specific Gravity Calculations-Superpave Light Traffic Mix.

Weight of the container in water, (g) [D] = 1511.1					
		A	E	A/(A+D-E)	
AC,%	Sample	Dry wt. (g)	Wet wt. (g)	G_{mm}	Avg G_{mm}
5.9	1	2061.3	2744	2.488	
	2	2072.4	2749.7	2.485	2.487
6.4	1	2148.0	2788.8	2.468	
	2	2144.3	2787.3	2.470	2.469
6.9	1	2103.6	2756.4	2.451	
	2	2100.2	2754.8	2.452	2.451
7.4	1	2124.9	2762.6	2.433	
	2	2110.4	2754.8	2.435	2.434

Table C.10 Bulk Specific Gravity-Superpave Light Traffic Mix.

	Sample	**A**	**B**	**C**	**A/(C-B)**	
AC, %	**No.**	**Dry wt. (g)**	**Wet wt. (g)**	**SSD Wt. (g)**	**G_{mb}**	**Avg G_{mb}**
5.9	1	4675.3	2710.7	4685.7	2.367	
	2	4638.8	2692.2	4651.5	2.368	2.367
6.4	1	4676.5	2721.0	4685.1	2.381	
	2	4653.2	2700.4	4662.3	2.372	2.376
6.9	1	4614.5	2682.7	4622.4	2.379	
	2	4645.9	2706.6	4651.3	2.389	2.384
7.4	1	4555.5	2656.8	4564.0	2.389	
	2	4666.0	2728.2	4670.8	2.402	2.395

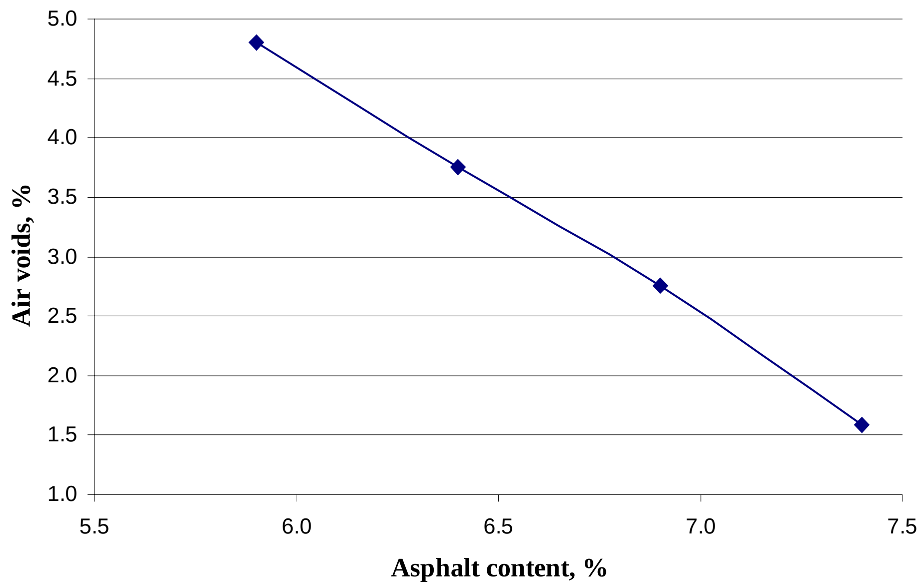
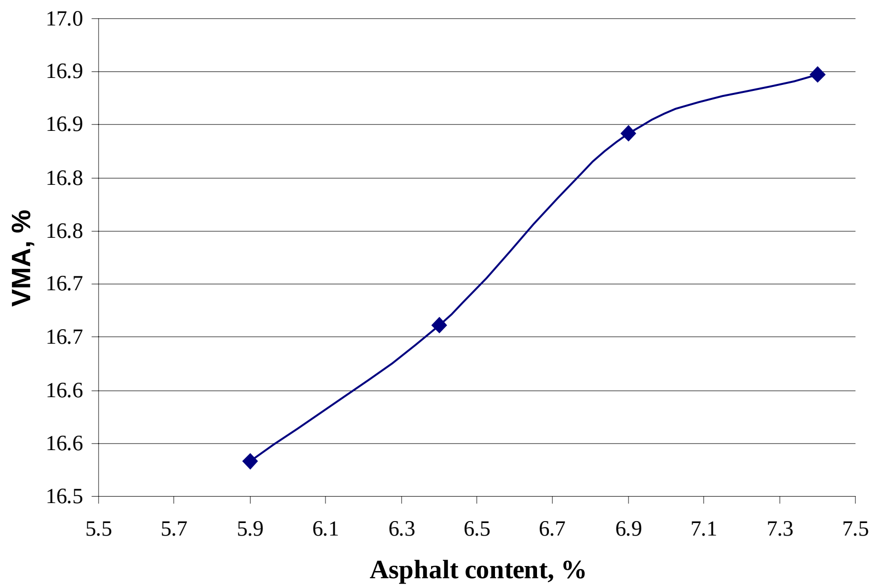


Figure C.9 Asphalt content versus Air voids-Supeprave light traffic mix.



Fgure C.10 Asphalt content versus VMA-Superpave light traffic mix.

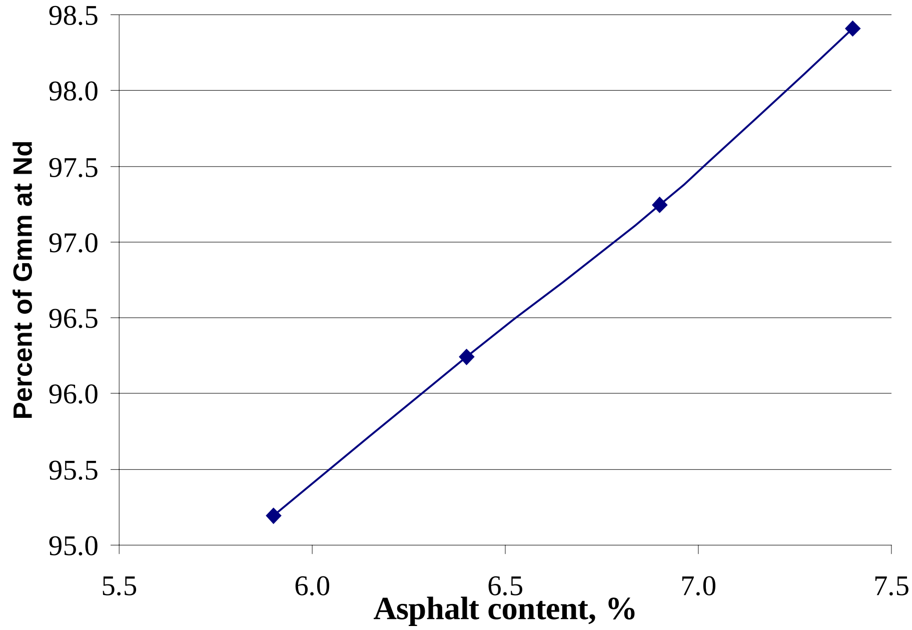


Figure C.11 Asphalt content versus percent Gmm at Nd-Supeprave light traffic mix.

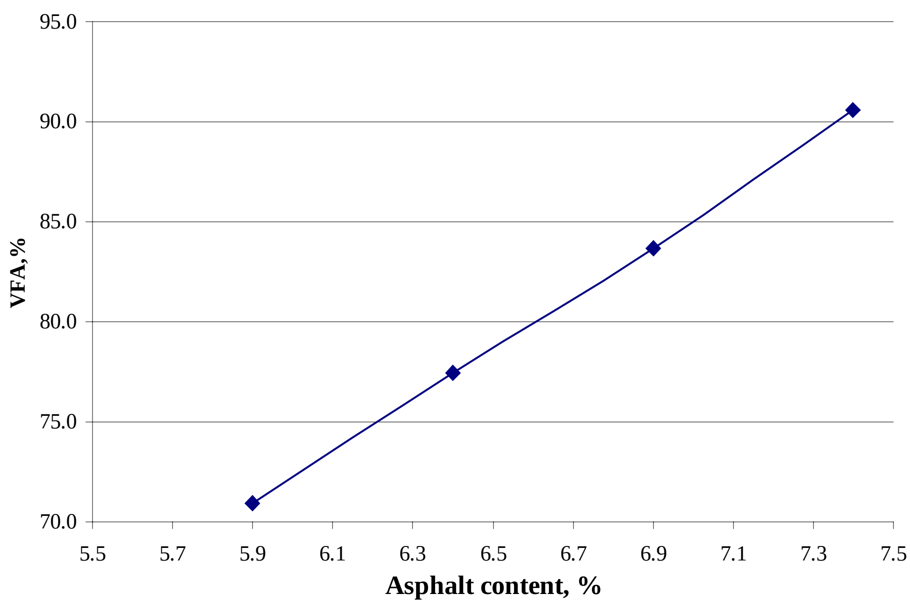


Figure C.12 Asphalt content versus VFA - Superpave light traffic level

APPENDIX D DATA AND ANALYSIS FOR EXCHANGING SUPERPAVE AND MARSHALL MIX DESIGNS

Table D.1 Marshall Mix Designs with Superpave Methodology

Asphalt Content by Weight of Mix		Mass in Grams			Specific Gravity			Voids in	Percent Air Voids	
		Weight	Weight	Saturated			Density	Mineral		
MIX	Percent	in Air	In water	surface dry	Bulk	Maximum	Kg/m3	Aggregate	Total Mix	Filled
	AC	A	C	B	D	E		F		
					A/(B-C)		D X 1000		100 X [(E-D)/E]	100 X [(F-G)/F]
SL	6.2	4673.2	2711.3	4683.1	2.370					
	6.2	4664.8	2709.4	4675.2	2.373					
Average					2.372	2.485	2372	**16.66**	**4.57**	**72.58**
SM	5.7	4760.1	2773.7	4769.7	2.385					
	5.7	4747.1	2779.2	4754.8	2.403					
Average					2.394	2.533	2394	**15.42**	**5.49**	**64.38**
SH	5.1	4798.2	2800.2	4807.8	2.390					
	5.1	4786.5	2796.1	4797.1	2.392					
Average					2.391	2.505	2391	**14.98**	**4.55**	**69.63**
ML	5.5	4659.0	2703.4	4668.4	2.371					
	5.5	4655.4	2702.1	4667.2	2.369					
Average					2.370	2.483	2370	**16.09**	**4.55**	**71.71**
MM	5.5	4754.8	2769.1	4765.2	2.382					
	5.5	4755.2	2768.9	4766.0	2.381					
Average					2.382	2.483	2382	**15.68**	**4.09**	**73.93**
MH	5.0	4784.2	2792.6	4795.2	2.389					
	5.0	4792.6	2794.4	4802.2	2.387					
Average					2.388	2.514	2388	**15.00**	**5.01**	**66.59**

Table D.2 Superpave Mix Designs with Marshall Methodology

	Asphalt Content by Weight of Mix			Mass in Grams			Specific Gravity			Voids in	Percent Air Voids		Stability (n) Pounds		Flow
			Specimen	Weight	Weight	Saturated			Density	Mineral					0.01
	Specimen	Percent	Thickness	in Air	In water	surface dry	Bulk	Maximum	Kg/m3	Aggregate	Total Mix	Filled	Actual	Adjusted	inch
	Number	AC	(mm)	A	C	B	D	E		F					
							A/(B-C)		D X 1000		100 X [(E-D)/E]	100 X [(F-G)/F]			
MH	1	5.0	64	1193.8	699.7	1199.8	2.387						2525	2493.4	15.0
	2	5.0	63	1174	686.1	1180	2.377						2400	2490.0	20.0
	3	5.0	62	1190	696.6	1196.1	2.382						2700	2801.3	17.0
	Average						2.382	2.51	2382	**15.24**	**5.09**	**66.58**	2542	2594.9	17.3
SPH	1	5.1	64	1214.8	711.1	1219.6	2.389						2975	2937.8	20.0
	2	5.1	64	1210.3	706.6	1215.4	2.379						2575	2542.8	14.0
	3	5.1	64	1207.6	704.0	1213.4	2.371						2525	2493.4	17.0
	Average						2.379	2.508	2379	**15.43**	**5.13**	**66.78**	2692	2658.0	17.0
MM	1	5.5	58	1099.1	639.9	1103.3	2.372						1850	2152.1	15.0
	2	5.5	62	1131.5	651.5	1138.6	2.323						1725	1789.7	20.0
	3	5.5	61	1133.4	659.5	1137.4	2.372						2000	2137.5	15.0
	Average						2.355	2.482	2355	**16.63**	**5.10**	**69.35**	1858	2026.4	16.7
SPM	1	5.7	61	1141.1	662.7	1145	2.366						2175	2323.2	22.0
	2	5.7	62	1134.4	655.6	1139.6	2.344						1975	2049.1	20.0
	3	5.7	62	1137.5	657.4	1142.3	2.346						2100	2178.8	22.0
	Average						2.352	2.495	2352	**16.94**	**5.74**	**66.13**	2083	2183.7	21.3
SPL	1	6.2	63	1143.5	660.1	1148.2	2.343						1850	1873.1	24.0
	2	6.2	61	1134.2	660.3	1138.5	2.372						1950	2082.8	20.0
	3	6.2	61	1141.8	661.1	1146.4	2.353						2075	2217.6	22.0
	Average						2.356	2.485	2356	**17.24**	**5.20**	**69.84**	1958	2057.9	22.0

VITA

Vasavi Kanneganti was born in Visakhapatnam, India in May 5, 1978. She received her Bachelor's degree in Civil Engineering from Andhra University in Visakhapatnam in June 1999. She had since completed the graduate level course work in the areas of paving materials, pavement design, and traffic engineering. Vasavi is currently a candidate for Masters of Science degree in Civil Engineering at West Virginia University, and plans to graduate in May 2002.