

Mechanical Properties of Warm Mix Asphalt Prepared Using Foamed Asphalt Binders

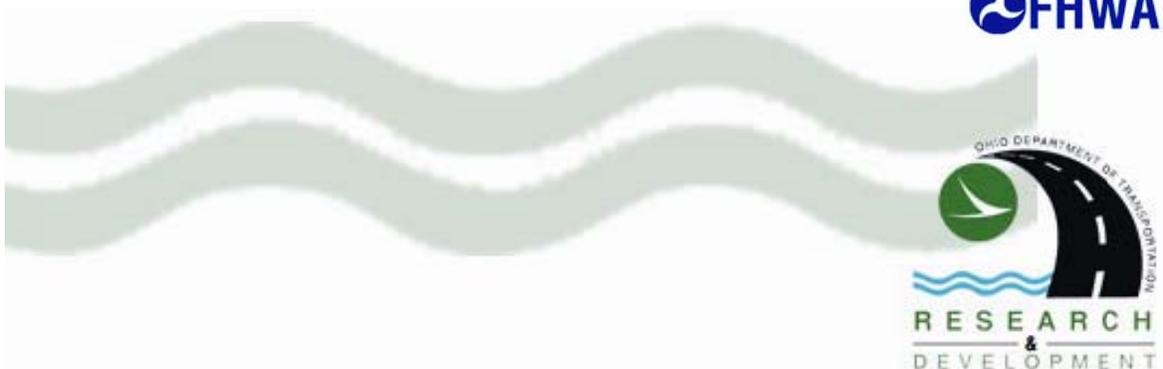
Ala R. Abbas, Ph.D.
Ayman Ali, M.S.
Department of Civil Engineering
The University of Akron
Akron, OH 44325-3905

for the
Ohio Department of Transportation
Office of Innovation, Partnerships & Energy
Innovation, Research & Implementation Section

and the
U. S. Department of Transportation
Federal Highway Administration

State Job Number 134476

March 2011



1. Report No. FHWA/OH-2011/6		2. Government Accession No.		3. Recipient's Catalog No.	
4. Title and subtitle Mechanical Properties of Warm Mix Asphalt Prepared Using Foamed Asphalt Binders				5. Report Date March 2011	
				6. Performing Organization Code	
7. Author(s) Ala R. Abbas and Ayman Ali				8. Performing Organization Report No.	
				10. Work Unit No. (TRAIS)	
9. Performing Organization Name and Address The University of Akron 402 Buchtel Common Akron, OH 44325-2102				11. Contract or Grant No. 134476	
				13. Type of Report and Period Covered Final Report	
12. Sponsoring Agency Name and Address Ohio Department of Transportation 1980 West Broad Street Columbus, OH 43223				14. Sponsoring Agency Code	
15. Supplementary Notes					
16. Abstract <p>Warm mix asphalt (WMA) is a name given to a group of technologies that have the common purpose of reducing the viscosity of the asphalt binders. This reduction in viscosity offers the advantage of producing asphalt-aggregate mixtures at lower mixing and compaction temperatures, and subsequently reducing energy consumption and pollutant emissions during asphalt mix production and placement. WMA technologies reduce the asphalt binders' viscosity through the addition of organic or chemical additives or by introducing cool water into the heated molten asphalt under controlled temperature and pressure conditions, resulting in so-called foamed asphalt binder. The latter has received increased attention in Ohio since it does not require the use of costly additives.</p> <p>In spite of the above-mentioned advantages of WMA mixtures, many concerns have been raised regarding the susceptibility of this material to moisture-induced damage and permanent deformation due to the reduced mixing and compaction temperatures used during WMA production. Therefore, this study was conducted to develop a laboratory procedure to produce WMA mixtures prepared using foamed asphalt binders (WMA-FA), and to evaluate their performance in comparison to conventional hot mix asphalt (HMA). Two aggregates (natural gravel and crushed limestone) and two asphalt binders (PG 64-22 and PG 70-22M) were used in this study. A laboratory scale asphalt binder foaming device called WLB10 was used to foam the asphalt binders. The aggregate gradation met ODOT Construction and Material Specifications (C&MS) requirements for Item 441 Type 1 Surface Course subjected to medium traffic. The resistance of WMA-FA and HMA mixtures to moisture-induced damage was measured using AASHTO T 283, and the resistance to permanent deformation was measured using the Asphalt Pavement Analyzer (APA) and the Simple Performance Test (SPT).</p> <p>Based on the experimental test results and the subsequent analyses findings, the following conclusions were made:</p> <ul style="list-style-type: none"> - WMA-FA mixtures are more workable and easily compacted than HMA mixtures even though they are produced at lower mixing and compaction temperatures. - WMA-FA mixtures are slightly more susceptible to moisture damage than HMA mixtures. However, the difference is statistically insignificant. Therefore, if designed properly, both mixtures are expected to meet ODOT's minimum tensile strength ratio (TSR) requirement for the proposed traffic level. - WMA-FA mixtures, especially those prepared using natural gravel and unmodified asphalt binders, are more prone to rutting than the corresponding HMA mixtures. However, the effect of the aggregate and binder types was found to be more significant than the mix type. This result suggests that using appropriate aggregate and binder types can help in overcoming any adverse effects that WMA-FA have on the mixture performance. 					
17. Key Words Warm mix asphalt, hot mix asphalt, foamed asphalt, moisture susceptibility, rutting, tensile strength ratio, asphalt pavement analyzer			18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161		
19. Security Classif. (of this report) Unclassified		20. Security Classif. (of this page) Unclassified		21. No. of Pages 132	22. Price
Form DOT F 1700.7 (8-72)			Reproduction of completed pages authorized		

Final Report

State Job No. 134476

Mechanical Properties of Warm Mix Asphalt Prepared Using Foamed Asphalt Binders

Prepared by:

Ala R. Abbas, Ph.D.
Assistant Professor
The University of Akron
Department of Civil Engineering
Akron, Ohio 44325-3905
Phone: (330) 972-8242
Fax: (330) 972-6020
Email: abbas@uakron.edu

Mr. Ayman Ali
Graduate Assistant
The University of Akron
Department of Civil Engineering
Akron, Ohio 44325-3905

Prepared in Cooperation with
The Ohio Department of Transportation
&
The U. S. Department of Transportation
Federal Highway Administration

March 2011

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Ohio Department of Transportation (ODOT) or the Federal Highway Administration (FHWA). This report does not constitute a standard, specification or regulation.

ACKNOWLEDGEMENTS

The researchers would like to thank the Ohio Department of Transportation (ODOT) and the Federal Highway Administration (FHWA) for sponsoring this study. The researchers would like to extend their thanks to ODOT technical liaison, Mr. David Powers, for his valuable contributions to this report. Without his assistance, this work would not have been possible. The researchers would like also to thank Dr. Munir Nazzal of the Department of Civil Engineering at Ohio University for his helpful comments and suggestions throughout this study. Finally, the researchers would like to thank Central Allied and National Lime and Stone for donating the aggregates, and Marathon Petroleum Company for donating the asphalt binders used in this study.

TABLE OF CONTENTS

ABSTRACT	1
CHAPTER 1: INTRODUCTION	3
1.1 Problem Statement	3
1.2 Objectives of the Study	4
1.3 Report Organization	4
CHAPTER 2: LITERATURE REVIEW	5
2.1 Introduction	5
2.2 Types of Asphalt Mixtures.....	5
2.2.1 Cold Mix Asphalt.....	6
2.2.1.1 Mix Design of Cold Mix Asphalt	6
2.2.2 Half-Warm Mix Asphalt.....	9
2.2.3 Warm Mix Asphalt	10
2.2.3.1 Warm Mix Asphalt Technologies.....	10
2.2.3.2 Mix Design of Warm Mix Asphalt.....	14
2.2.3.3 Characterization of Foamed Asphalt and the Foaming Process	15
2.2.4 Hot Mix Asphalt	18
2.2.4.1 Mix Design of Hot Mix Asphalt.....	18
2.2.4.2 The Bailey Method for Blending Aggregates.....	20
2.3 Previous Studies on Warm Mix Asphalt.....	23
2.3.1 Field Performance of WMA	23
2.3.2 Laboratory Performance of WMA.....	25
CHAPTER 3: MATERIAL DESCRIPTION.....	29
3.1 Aggregates	29
3.2 Asphalt Binders.....	31
3.3 Mix Design.....	32
3.3.1 Aggregate Gradation.....	33
3.3.2 Optimum Asphalt Binder Content	37
CHAPTER 4: PRODUCTION OF FOAMED WARM MIX ASPHALT	39
4.1 Introduction	39
4.2 Laboratory Production of Foamed WMA Mixtures	40
4.3 Advantages and Disadvantages of WLB10 Asphalt Foaming Device	42
CHAPTER 5: TESTING PLAN.....	45
5.1 Introduction	45
5.2 Moisture Susceptibility (AASHTO T 283).....	45
5.3 Asphalt Pavement Analyzer (AASHTO TP 63-07).....	47
5.4 Dynamic Modulus (AASHTO TP 62-07).....	50
CHAPTER 6: HANDLING CHARACTERISTICS OF WMA-FA MIXTURES	57
6.1 Introduction	57
6.2 Aggregate Coating	57
6.3 Asphalt Binder Absorption	57

6.4 Workability	60
6.5 Compactability	60
CHAPTER 7: TEST RESULTS AND DATA ANALYSIS	63
7.1 Introduction	63
7.2 AASHTO T 283 Test Results	63
7.3 APA Test Results.....	67
7.4 Dynamic Modulus Test Results	69
CHAPTER 8: CONCLUSIONS AND RECOMMENDATIONS	77
8.1 Summary and Conclusions	77
8.2 Study Limitations.....	79
8.3 Recommendations for Further Research.....	79
8.4 Recommendations for Implementation.....	80
REFERENCES	81
APPENDIX A	
APPENDIX B	
APPENDIX C	
APPENDIX D	

LIST OF TABLES

Table 3.1: Specific Gravities and Absorption of Selected Aggregates (after ODOT Aggregate Specific Gravity List 2009)	29
Table 3.2: Supplier Provided Aggregate Gradations	29
Table 3.3: Asphalt Binder Properties (after Marathon Petroleum Company)	31
Table 3.4: ODOT Gradation and Mix Design Requirements for Type I Surface Mix Subjected to Medium Traffic (after ODOT C&MS 2008)	33
Table 3.5: Trial Gradations Used in the Bailey Method for Natural Gravel and Natural Sand	34
Table 3.6: Trial Gradations Used in the Bailey Method for Crushed Limestone and Limestone Sand	34
Table 3.7: Selected Aggregate Gradations and the Modifications	35
Table 3.8: Bailey Method Ratios for the Selected Gradations.....	37
Table 3.9: Summary of the Mix Design Results.....	38
Table 5.1: Temperature Equilibrium Time in AASHTO TP 62 Test	53
Table 6.1: Rice Specific Gravity of HMA and WMA-FA.....	59
Table 6.2: Effective Specific Gravity of Aggregate in HMA and WMA-FA	59
Table 6.3: Asphalt Binder and Water Absorption of Aggregates in HMA and WMA-FA	59
Table 6.4: Required Compaction Effort to Achieve the Target Air Voids.....	60
Table 7.1: Multi-Factor ANOVA Results for Dry ITS Values.....	65
Table 7.2: Multi-Factor ANOVA Results for TSR Values.....	67
Table 7.3: Multi-Factor ANOVA Results for Rut Depth Measurements	69

LIST OF FIGURES

Figure 2.1: Cold Mix Gradation Zones (after Akeroyd and Hicks 1988).....	7
Figure 2.2: Simultaneous Selection of Optimum Foamed Asphalt Content and Optimum Aggregate Moisture Content for Cold Mix Asphalt (after Saleh 2004).....	8
Figure 2.3: Particle Coating in Half-Warm Mixes (after Jenkins et al. 1999).....	9
Figure 2.4: Batch Plant Equipped for Low Energy Asphalt Processing (after Romier et al. 2006).....	13
Figure 2.5: Wirtgen WLB 10 Laboratory Foaming Device (after Wirtgen, Inc.)	16
Figure 2.6: Astec Double Barrel Green System Nozzle (after Astec, Inc.).....	16
Figure 2.7: Foaming Properties of an Asphalt Binder (after Wirtgen, Inc.).....	17
Figure 3.1: Natural Gravel (right) and Natural Sand (left).....	30
Figure 3.2: Crushed Limestone (right) and Limestone Sand (left).....	31
Figure 3.3: Natural Gravel and Natural Sand Gradation Chart	36
Figure 3.4: Crushed Limestone and Limestone Sand Gradation Chart.....	36
Figure 4.1: Multi-Nozzle Foaming Device (after Astec, Inc.).....	39
Figure 4.2: Wirtgen WLB10 Asphalt Foaming Device.....	40
Figure 5.1: Asphalt Pavement Analyzer (APA)	49
Figure 5.2: Repeated Wheel Loading in the APA Device.....	49
Figure 5.3: Vertical Coring Setup.....	51
Figure 5.4: Trimming a Dynamic Modulus Specimen using a Diamond Saw	51
Figure 5.5: Checking the Waviness of the Top and Bottom Edges of a Dynamic Modulus Specimen Using a Straight Edge and a Feel Gage	52
Figure 5.6: MTS Model 810	53
Figure 5.7: Sample of Applied Stress and Measured Strain at 1 Hz.....	54
Figure 5.8: Determination of Maximum and Minimum Stress and Strain Values	55
Figure 7.1: Dry ITS of HMA and WMA-FA Mixtures Containing Natural Gravel.....	64
Figure 7.2: Dry ITS of HMA and WMA-FA Mixtures Containing Crushed Limestone	64
Figure 7.3: Tensile Strength Ratios (TSR) for Mixtures Containing Natural Gravel.....	66
Figure 7.4: Tensile Strength Ratios (TSR) for Mixtures Containing Crushed Limestone	66
Figure 7.5: Rut Depth Results for Mixtures Containing Natural Gravel.....	68

Figure 7.6: Rut Depth Results for Mixtures Containing Crushed Limestone.....	68
Figure 7.7: Example Dynamic Modulus Test Results for WMA-FA Mixtures Prepared Using Natural Gravel and PG 64-22.....	71
Figure 7.8: Example Temperature Shift Factors at a Reference Temperature of 77°Ffor WMA-FA Mixtures Prepared Using Natural Gravel and PG 64-22.....	71
Figure 7.9: Example Master Curve at a Reference Temperature of 77°Ffor WMA-FA Mixtures Prepared Using Natural Gravel and PG 64-22	72
Figure 7.10:Temperature Shift Factors for Mixtures Containing Natural Gravel	73
Figure 7.11: Temperature Shift Factors for Mixtures Containing Crushed Limestone.....	73
Figure 7.12: Dynamic Modulus Master Curves for Mixtures Containing Natural Gravel.....	74
Figure 7.13: Dynamic Modulus Master Curves for Mixtures Containing Crushed Limestone.....	74

MECHANICAL PROPERTIES OF WARM MIX ASPHALT PREPARED USING FOAMED ASPHALT BINDERS

ABSTRACT

Warm mix asphalt (WMA) is a name given to a group of technologies that have the common purpose of reducing the viscosity of the asphalt binders. This reduction in viscosity offers the advantage of producing asphalt-aggregate mixtures at lower mixing and compaction temperatures, and subsequently reducing energy consumption and pollutant emissions during asphalt mix production and placement. WMA technologies reduce the asphalt binders' viscosity through the addition of organic or chemical additives or by introducing cool water into the heated molten asphalt under controlled temperature and pressure conditions, resulting in so-called foamed asphalt binder. The latter has received increased attention in Ohio since it does not require the use of costly additives.

In spite of the above-mentioned advantages of WMA mixtures, many concerns have been raised regarding the susceptibility of this material to moisture-induced damage and permanent deformation due to the reduced mixing and compaction temperatures used during WMA production. Therefore, this study was conducted to develop a laboratory procedure to produce WMA mixtures prepared using foamed asphalt binders (WMA-FA), and to evaluate their performance in comparison to conventional hot mix asphalt (HMA). Two aggregates (natural gravel and crushed limestone) and two asphalt binders (PG 64-22 and PG 70-22M) were used in this study. A laboratory scale asphalt binder foaming device called WLB10 was used to foam the asphalt binders. The aggregate gradation met ODOT C&MS requirements for Item 441 Type 1 Surface Course subjected to medium traffic. The resistance of WMA-FA and HMA mixtures to moisture-induced damage was measured using AASHTO T 283, and the resistance to permanent deformation was measured using the Asphalt Pavement Analyzer (APA) and the Simple Performance Test (SPT).

Based on the experimental test results and the subsequent analyses findings, the following conclusions were made:

- WMA-FA mixtures are more workable and easily compacted than HMA mixtures even though they are produced at lower mixing and compaction temperatures.

- WMA-FA mixtures are slightly more susceptible to moisture damage than HMA mixtures. However, the difference is statistically insignificant. Therefore, if designed properly, both mixtures are expected to meet ODOT's minimum tensile strength ratio (TSR) requirement for the proposed traffic level.
- WMA-FA mixtures, especially those prepared using natural gravel and unmodified asphalt binders, are more prone to rutting than the corresponding HMA mixtures. However, the effect of the aggregate and binder types was found to be more significant than the mix type. This result suggests that using appropriate aggregate and binder types can help in overcoming any adverse effects that WMA-FA have on the mixture performance.

CHAPTER 1

INTRODUCTION

1.1 Problem Statement

Hot mix asphalt (HMA) is a mixture containing aggregates and asphalt binders prepared at specified proportions. The aggregates and asphalt binder proportions are determined through a mix design procedure such as the Marshall Mix Design or the Superpave Mix Design methods. Overall, the goal of determining such proportions is to establish an HMA mixture that will meet specific performance criteria. In addition, it is imperative to ensure that the asphalt binder will fully coat the aggregates and that the resulting mixture is workable and compactable.

In order to ensure sufficient aggregate drying and coating, both the asphalt binder and the aggregates are heated to highly elevated temperatures ranging between 300°F and 325°F (150°C and 163°C). The use of such high temperatures would result in lowering the viscosity of the asphalt binder which is the main factor affecting the coating, and workability of HMA mixtures.

In recent years, a new group of technologies have been introduced to the United States that allow producing asphalt mixtures at temperatures 30°F to 100°F (15°C to 50°C) lower than what is used in HMA. This group of technologies is commonly referred to as Warm Mix Asphalt (WMA). They are promoted as environmentally friendly green alternatives to HMA mixtures as they produce lower greenhouse gas emissions. This new group of technologies aims at reducing the viscosity of the asphalt binder through the addition of organic or chemical additives or by introducing cool water into the heated molten asphalt under controlled temperature and pressure conditions, resulting in so-called foamed asphalt binder. As a consequence, lower temperatures are needed during production for the asphalt binder to be absorbed by the aggregates.

Over the last few years, WMA mixes prepared using foamed asphalt binders (WMA-FA) have received increased attention and use in Ohio. This technology is believed to be the most cost effective from among the WMA technologies since it does not require any costly additives to be added to the mixtures and more importantly it does not require very expensive plant modifications since the foaming component can be attached to old systems for a reasonable price, without the need for any additional changes. Other potential advantages include reducing energy consumption since lower temperatures are used, increasing haul distances since warm mix asphalts retain their workability over a broader temperature range, improving working

conditions due to lower odor, fume, and emission levels produced from heating the asphalt binder; and more importantly improving compactability and the ability to reach the desired density with fewer number of roller passes.

In spite of the above-mentioned advantages for WMA-FA mixtures, many concerns have been raised regarding the use of lower temperatures in the production of asphalt-aggregate mixtures that might result in increased rutting susceptibility due to less binder aging and increased propensity to moisture-induced damage due to less aggregate drying. Therefore, research is needed to evaluate the performance of this material with regard to these distresses.

1.2 Objectives of the Study

The main research objectives of this study are:

1. Develop a procedure by which WMA-FA mixtures can be produced in the laboratory;
2. Evaluate the performance of WMA-FA mixtures with regard to moisture susceptibility and permanent deformation;
3. Evaluate the performance of control HMA mixtures prepared using the same aggregates and asphalt binders;
4. Compare the performance of WMA-FA and HMA mixtures; and
5. Recommend changes to current ODOT practices and specifications to address the research findings.

1.3 Report Organization

This report is organized into eight chapters. Chapter 2 presents a literature review of subjects pertinent to this study. It provides an overview of the various types of asphalt mixtures along with a discussion of the different types of WMA technologies. The outcome of recent studies focusing on the performance of WMA mixtures is also presented in this chapter. Chapter 3 presents the materials used in the preparation of the HMA and WMA-FA mixtures, followed by a discussion of the mix design procedure. Chapter 4 describes the laboratory procedure used in the preparation of the WMA-FA mixtures. Chapter 5 presents the details of the laboratory testing plan. Chapter 6 compares the mixing and handling characteristics of HMA and WMA-FA mixtures. Chapter 7 presents the experimental test results and the outcome of the statistical analysis. Finally, Chapter 8 presents the conclusions and recommendations for future study.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The stringent environmental regulations and the rising energy costs have motivated researchers to develop new procedures to prepare asphalt-aggregate mixtures. The last few years, for instance, have witnessed an increased interest in using technologies known as warm mix asphalt due to the advantages they provide over the traditional hot mix asphalt. Warm mix asphalt technologies are known for reducing the viscosity of the asphalt binder which allows for producing asphalt-aggregate mixtures at lower temperatures. This in turn reduces the environmental emissions along with the overall production cost.

The efforts to reduce production temperatures when preparing asphalt mixtures are not new in concept. Professor Ladis Csanyi is believed to be the first in utilizing foamed asphalt binders as a soil stabilizing agent (Csanyi 1957). Csanyi's process dealt with injecting steam into hot asphalt binder to foam it. Later on, Mobil Oil of Australia acquired the patent rights of Csanyi's invention and modified the original process by adding cold water rather than injecting steam, thus, making the process more practical to apply (Muthen 1998, Kristjansdottir 2006).

Over the last two decades, several new warm mix asphalt technologies have been developed. A detailed discussion of the different warm mix asphalt technologies as well as the field and laboratory performance of these technologies is provided in this chapter. For completeness, a discussion of the various asphalt-aggregate mixture preparation techniques is also included.

2.2 Types of Asphalt Mixtures

Several asphalt mixture production techniques are available. These techniques vary in terms of the production temperatures; therefore, they were given the names: cold mix asphalt, half-warm mix asphalt, warm mix asphalt (WMA), and finally hot mix asphalt (HMA). A discussion of these techniques is found in the following subsections with emphasis on WMA since it is the focus of this study.

2.2.1 Cold Mix Asphalt

Cold mixes are prepared using foamed asphalt and moist aggregates mixed at ambient temperatures. They are mainly used as means for recycling and rehabilitating distressed pavements. Therefore, cold mixes provide the advantages of using existing pavement material as well as reducing the costs of buying new materials.

Cold recycling is usually conducted in two different processes known as cold-in-place recycling and cold-in-plant recycling. In the cold-in-place technique, the distressed pavement is milled and mixed with a stabilizing agent, usually foamed asphalt, and then placed using any standard paver and finally compacted. Meanwhile, in the cold-in-plant technique, the distressed pavement is milled and stockpiled. The stockpiled material is then mixed with a stabilizing agent (i.e. foamed asphalt and other stabilizing agents) in the plant and hauled to be placed and compacted. Both cold recycling techniques are used to prepare materials to be used as base courses over which hot mix asphalt courses are placed.

2.2.1.1 Mix Design of Cold Mix Asphalt

Several mix design methods have been suggested to design cold mix asphalt. Ruckel et al. (1983) proposed a procedure for the preparation of cold mixes prepared using foamed asphalt. In this procedure, foamed asphalt binder is mixed with unheated moist aggregates at or near ambient temperatures. The foamed asphalt is tested to determine the optimum foaming water content that will maximize the expansion ratio and half-life. Additional testing is also conducted to determine the optimum aggregate moisture content that will result in maximum mixture density. Mixtures are then prepared at various foamed asphalt contents and cured according to one of the three suggested curing methods (i.e. short-term, intermediate-term, and long-term curing). The loose mixtures are then compacted and the optimum foamed asphalt content is selected as the one that will result in the highest mixture density.

Muthen (1998) suggested a new cold foamed asphalt mix design procedure in which the gradation of the aggregates used in the mix was controlled according to predefined zones, namely zone A, zone B, and zone C, as shown in Figure 2.1. Zone A defines the most suitable aggregate gradation to be used in the preparation of cold mixes prepared using foamed asphalt, while gradations falling in zones B and C require adjustments to fall within zone A if the design is for heavy traffic. Muthen (1998) defined the optimum foaming water content as the water

content that will maximize the expansion ratio and half-life. In addition, the aggregate moisture content was suggested to be between 70 to 80 percent of the optimum moisture content of that particular aggregate. After that, mixtures were prepared at different foamed asphalt contents and cured according to the suggested 3-day curing period at 140°F (60°C). Finally, Muthen (1998) defined the optimum foamed asphalt content as the content exhibiting the maximum soaked indirect tensile strength.

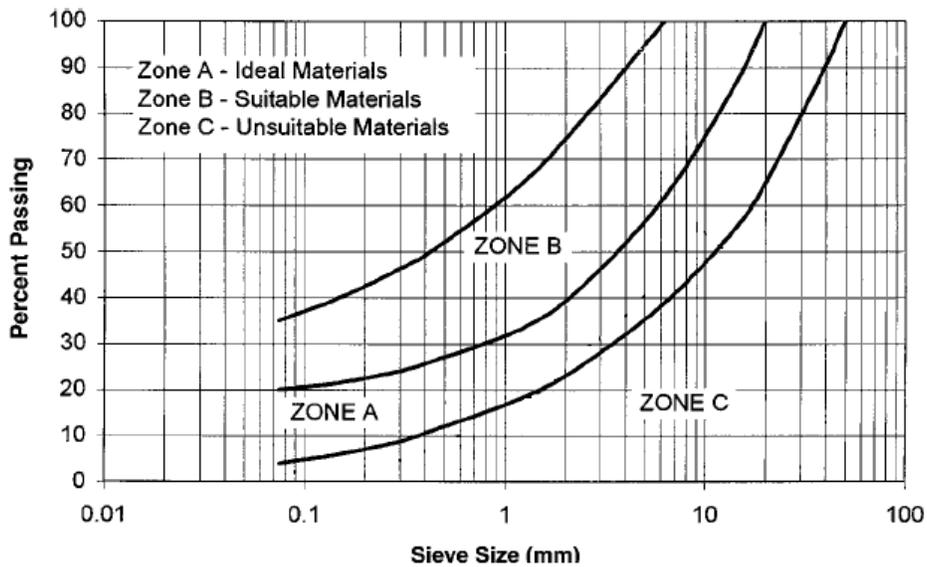


Figure 2.1: Cold Mix Gradation Zones (after Akeroyd and Hicks 1988).

Saleh (2004) suggested a new method for the selection of the optimum foamed asphalt content. In this method, the aggregate is similarly selected to pass through zone A, as proposed by Muthen (1998), and the optimum foaming water content as the one that will maximize the expansion ratio and the half-life. However, Saleh (2004) suggested determining both the optimum foamed asphalt content and the optimum aggregate moisture content through maximizing the bulk density and resilient modulus. In this procedure, resilient modulus and bulk density tests are performed on specimens prepared using different combinations of foamed asphalt and aggregate moisture contents. The optimum foamed asphalt content and the optimum aggregate moisture content are then selected as the values corresponding to the maximum resilient modulus and bulk density. Figure 2.2 shows an example on the simultaneous selection

of the optimum foamed asphalt content and the optimum aggregate moisture content using this method.

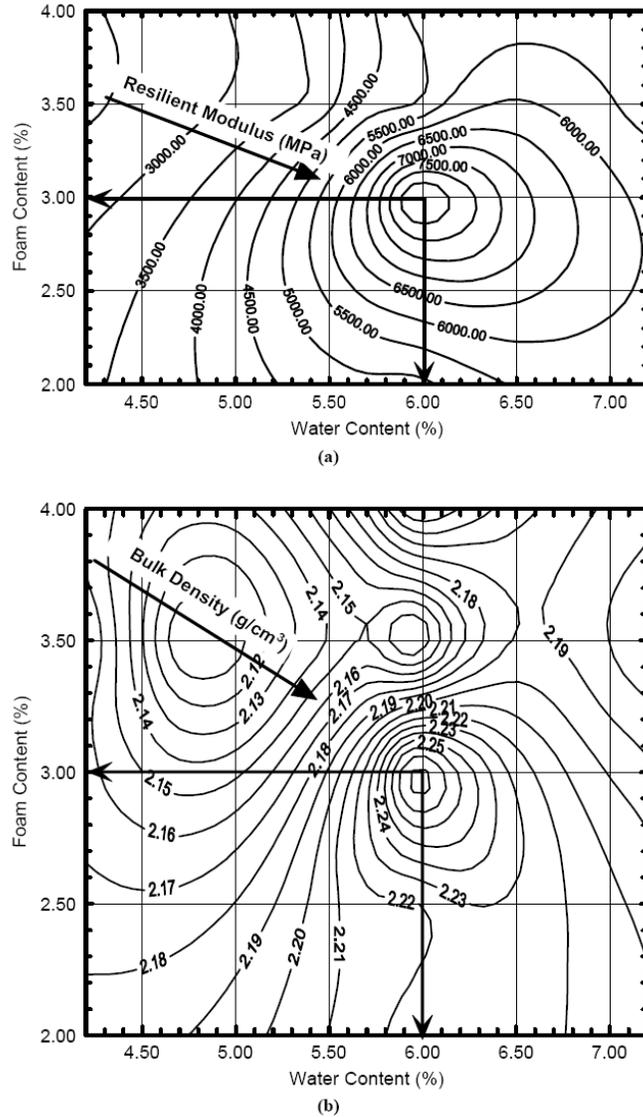


Figure 2.2: Simultaneous Selection of Optimum Foamed Asphalt Content and Optimum Aggregate Moisture Content for Cold Mix Asphalt (after Saleh 2004).

Recently, Kim et al. (2007) suggested a new cold mix design procedure that is mainly related to cold-in-place recycling using foamed asphalt as a stabilizing agent. According to this procedure, the recycled asphalt pavement (RAP) material is collected from the proposed construction site and then the gradation of the RAP is determined. After that, the optimum

foaming water content is selected as the water content that will maximize the expansion ratio and half-life, and the optimum aggregate moisture content is determined as the one that will achieve the maximum dry density. Mixtures are then prepared at various foamed asphalt contents and allowed to cure for two days at 104°F (40°C). Finally, indirect tensile strength tests are conducted and the optimum foamed asphalt content is selected as the one resulting in the maximum indirect tensile strength after vacuum-saturating the specimens.

2.2.2 Half-Warm Mix Asphalt

The development of half-warm foamed asphalt mixes took place after realizing the impact of aggregate temperature on the engineering properties of the cold mixes. Generally, half-warm mixes can be considered cold mixes since they are prepared in the same manner. However, the aggregates in the half-warm mixes are heated to temperatures in the range between 167°F and 194°F (75°C and 90°C) and not more than 212°F (100°C).

One of the main advantages of using half-warm foamed asphalt mixes over cold mixes is the improvement of aggregate coating as was reported by Jenkins et al. (1999). The relationship between aggregate temperature and aggregate coating is shown in Figure 2.3, in which three different coating regions are defined, namely complete coating, partial coating, and practically no coating. The complete coating region represents particles that are 100 percent coated, whereas the partial coating region represents particle coating in the range between 21 and 99 percent, and the practically no coating region represents 20 percent or less particle coating.

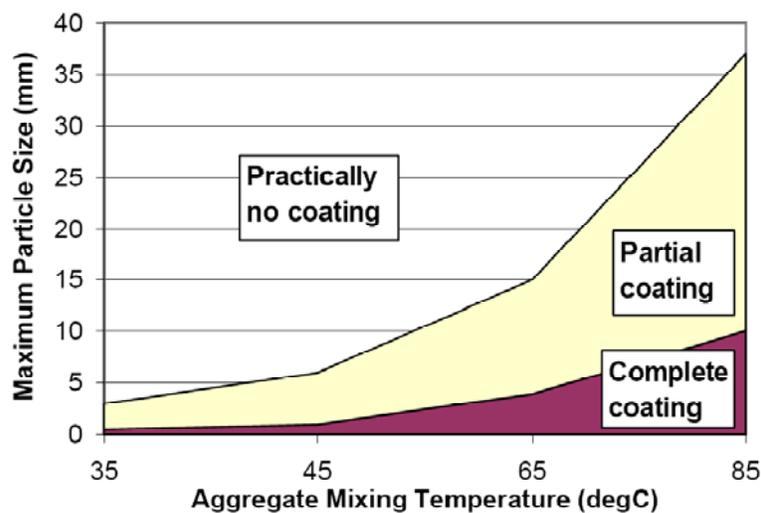


Figure 2.3: Particle Coating in Half-Warm Mixes (after Jenkins et al. 1999).

2.2.3 Warm Mix Asphalt

Warm mix asphalt is a name given to a group of technologies which are used to produce asphalt mixtures at lower mixing and compaction temperatures than the traditional hot asphalt mixtures. Such technologies tend to reduce the viscosity of the asphalt binder through the addition of organic or chemical additives or by introducing cool water into the heated molten asphalt under controlled temperature and pressure conditions, resulting in so-called foamed asphalt binder. Examples of available WMA technologies include: Sasobit, Asphaltan-B, Asphamin, Advera, Evotherm, Low Energy Asphalt, WAM-Foam, Revix, Cecabase RT, Thiopave, Rediset WMX, AquaFoam, Ultrafoam GX, Terex, AccuShear, Aquablack, TLA-X, Iterlow-T & HyperTherm, Static Incline Vortex Asphalt Blender, Ad-RAP, and the Double Barrel Green System (Corrigan 2010).

The use of warm mix asphalt technologies in producing asphalt mixtures in place of traditional hot mix asphalt has several advantages. One important advantage of utilizing warm mix asphalt technologies is the reduction in mixing and compaction temperatures. Other advantages include (D'Angelo et al. 2008):

- Reduced fuel and energy consumption.
- Reduced emissions and odors from plants.
- Reduced smoke and therefore, fewer complaints from the public.
- Improved working conditions at the site, thus, improving the quality of the work as well as the productivity of the workers.
- Longer hauling distances and extending the paving season.

2.2.3.1 Warm Mix Asphalt Technologies

As mentioned earlier, a wide range of technologies are available to prepare WMA mixtures. A detailed discussion of the most commonly used WMA technologies is presented in this section.

Sasobit

Sasobit is a synthetic wax that is produced in the coal gasification process. It is usually blended with asphalt binders at temperatures above 239°F (115°C). This wax has a melting point in the range between 185°F and 239°F (85°C and 115°C). Moreover, Sasobit forms a crystalline

structure within the asphalt binder at ambient temperatures. Therefore, Sasobit reduces the viscosity of the binder at temperatures above 239°F (115°C) and helps improve the stability at ambient temperatures.

Sasolwax, the manufacturer of Sasobit, reports that the optimum amount of Sasobit to be used is about 3 to 4 percent by weight of the asphalt binder which in turn would result in reducing the production temperatures by approximately 15°F to 54°F (8°C to 30°C). In addition, the manufacturer does not recommend blending the solid Sasobit with the asphalt binder during mixing because it will result in an inhomogeneous distribution of Sasobit within the mix. Therefore, Sasobit is blended with the hot asphalt binder stream to ensure homogenous distribution.

Asphaltan-B

Asphaltan-B is a refined montan wax that is blended with a fatty acid amide. It is produced by solvent extraction of certain types of lignite or brown coal. Moreover, it has a melting point in the range between 180°F and 210°F (82°C and 99°C). Therefore, it helps in reducing the viscosity of the asphalt binder at temperatures higher than its melting point.

Asphamin

Asphamin is a synthetic zeolite that contains about 20 percent of water crystallization by weight. Asphamin is added to the mix shortly or at the same time as adding the asphalt binder into the mixer. Therefore, Asphamin will be subjected to high temperatures causing it to release the water contained inside its structure. As a result of this gradual release of water (in the form of steam), the asphalt binder starts to foam and its viscosity is reduced making it applicable to produce mixes at lower production temperatures.

Asphamin manufacturer, Eurovia Services GmbH Germany, recommends the addition of 0.3 percent of Asphamin by total weight of the mixture. Moreover, the manufacturer reported that this gradual release of water, as steam, provides approximately 6 hours or more of improved workability if the mixture's temperature does not drop below 212°F (100°C). However, to ensure that the Asphamin is uniformly distributed within the mixture, a specially built distributor should be attached to the mixing plant (Barthel and von Devivere 2003). In addition, Eurovia promotes

that Asphamin can reduce the mixing temperature by more than 50°F (27°C), resulting in 30 percent energy savings.

Advera

Advera is another synthetic zeolite, similar to Asphamin, which contains about 18 percent of crystallized water by total weight as was reported by the manufacturer. Advera works similarly to Asphamin in gradually releasing the water contained inside it; however, Advera is a fine graded product (i.e. 100 percent passing sieve #200). Therefore, the distribution of Advera is expected to be more uniform than Asphamin which in turn results in better foaming of the asphalt binder.

Advera is added directly to the pugmill in batch plants and through a fiber port in drum plants; therefore, it requires no additional modifications in batch plants. The manufacturer of Advera, PQ Corporation, reports that a reduction in asphalt mixtures' production temperatures of 50°F to 70°F (27°C to 38°C) is expected when using Advera.

Evotherm

Evotherm, produced by MeadWestvaco Asphalt Innovations, is another product that is used to produce warm mix asphalt. Three different Evotherm technologies have been developed: Evotherm Emulsion Technology (ET), Evotherm Dispersed Asphalt Technology (DAT), and Evotherm Third Generation (3G). In the emulsion technology, a water-based emulsion is mixed with the hot aggregates to produce steam which in turn foams the asphalt binder. In addition, the water-based emulsion is produced using a chemical package that contains necessary additives to enhance coating, adhesion, and workability of warm mixes. The dispersed asphalt technology is similar to the ET technology; however, the water-based emulsion is injected into the asphalt binder line just before the asphalt binder enters the mixing chamber. Finally, the Evotherm 3G utilizes a water free version of the previous Evotherm technologies making it suitable for introducing additives at the mixing plant or at the asphalt terminal.

MeadWestvaco reported that using Evotherm ET will result in a reduction of more than 100°F (55°C), whereas using Evotherm DAT would result in temperature reduction in the range between 85°F and 100°F (47°C and 55°C). It was reported that the use of Evotherm 3G would result in a temperature reduction of 60°F to 85°F (33°C to 47°C). In addition, MeadWestvaco

reports that this reduction in temperature might lead to 55 percent energy savings, which could result in a 45 percent reduction in CO₂ and SO₂ emissions, a 60 percent reduction in NO_x, and a 41 percent reduction in total organic material.

Low Energy Asphalt

The low energy asphalt process was developed in France by Fairco of Zozay (Romier et al. 2006). In this process, the hot asphalt binder (280°F to 350°F; 138°C to 176°C) is mixed with coarse aggregates (290°F; 143°C) and then the wet fine aggregates (ambient temperatures) are introduced. The water contained in the fine aggregates evaporates and becomes steam due to the heat. This water steam helps in foaming the asphalt binder which in turn improves the coating of the coarse aggregates rapidly. The final temperature of the mixture should be below 212°F (100°C), between 140°F and 180°F (60°C to 82°C), to achieve significant energy savings (Romier et al. 2006).

Fairco of Zozay has developed a low energy asphalt production kit that can be attached to batch plants. This kit includes a hopper to control the amount of fine materials to be added to the mix, a device to add water to the fine aggregates (if needed), and an asphalt metering device to control the amount of coating and adhesion additives that are used in the process. Figure 2.4 shows a batch plant modified to accommodate the low energy asphalt process.

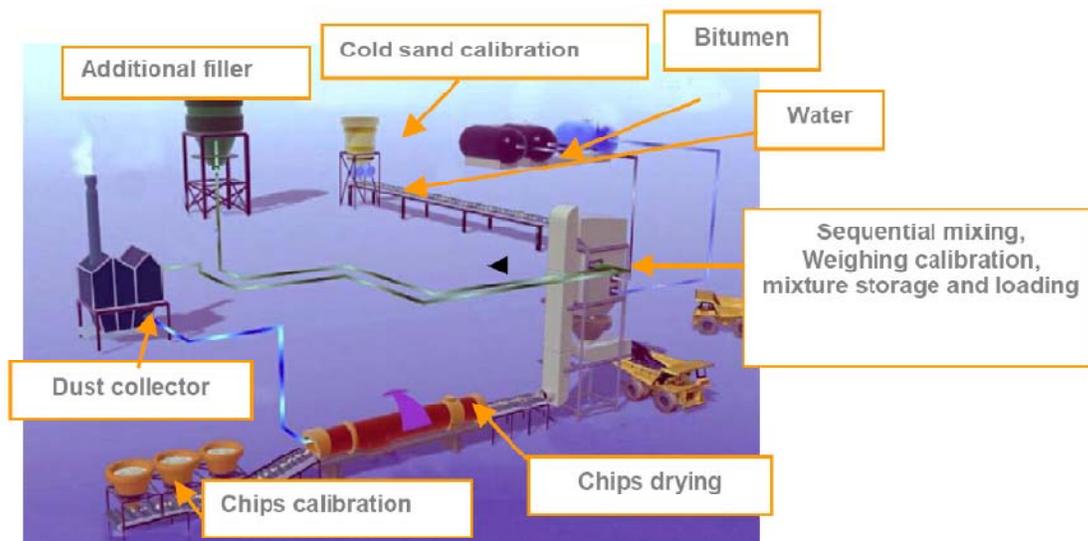


Figure 2.4: Batch Plant Equipped for Low Energy Asphalt Processing (after Romier et al. 2006).

WAM-Foam

WAM-Foam was developed by Shell International Petroleum, UK and Kolo-Veidekke, Norway. This process consists of a two-component binder system which introduces two different binder grades at two different mixing stages. In the first mixing stage, an extremely soft asphalt binder is mixed with the heated aggregates at temperatures in the range between 210°F to 250°F (99°C to 121°C); thus, fully coating the aggregates and ensuring that the aggregates will not absorb any of the water used in foaming the hard binder. In the second stage, an extremely hard binder is foamed by injecting cold water into it and then mixed with the pre-coated aggregates.

Shell reports that the use of the WAM-Foam process can result in 30 percent energy savings, which can lead to a 30 percent reduction in CO₂ emissions. However, the use of the WAM-Foam process would require several plant modifications that are estimated to cost from \$50,000 to \$70,000.

Double Barrel Green System

The Double Barrel Green System, which was developed by Astec Inc., is a system that utilizes foamed asphalt binder by water injection. In this system, a multi-nozzle foaming device, is used to mix cold water (at ambient temperatures) with hot asphalt (at mixing temperatures) to produce foamed asphalt. Upon the mixing of cold water and hot asphalt, the water becomes steam which works on foaming the asphalt binder.

The multi-nozzle foaming device contains a number of nozzles that allow the mixing of pressurized cold water with the hot asphalt. It was reported that about 30°F to 60°F (15°C to 33°C) reduction in production temperatures is possible with this system.

2.2.3.2 Mix Design of Warm Mix Asphalt

Several researchers have reported that traditional hot mix asphalt design procedures might require some modifications to accommodate the different warm mix asphalt technologies. For instance, Hurley and Prowell (2005a, 2005b) conducted a study to evaluate the effects of using Sasobit and Asphamin as WMA technologies. In that study, it was reported that mixes prepared with Sasobit and Asphamin using PG 64-22 and compacted at lower temperatures had similar air void levels as a control HMA mix prepared using an asphalt binder grade of PG 58-28; thus suggesting that these warm mix asphalt technologies have lowered the grade of PG 64-

22. However, this finding cannot be generalized to other warm mix asphalt technologies as more research is needed.

Hurley and Prowell (2006a) recommended, until further research is completed, that the optimum asphalt content of WMA mixes should be determined to be equal to that of traditional HMA optimum asphalt content without the inclusion of the additives. This is because WMA additives improve compaction to the point that the optimum asphalt content is reduced by approximately 0.5 percent below the standard HMA optimum asphalt content, raising concerns regarding the durability, permeability, and moisture susceptibility of the mix.

In addition to the selection of the optimum asphalt content, researchers have used dense-graded aggregate gradations similar to those used in traditional HMA (Hurley and Prowell 2005a, Hurley and Prowell 2005b, Romier et al. 2006). Moreover, Romier et al. (2006) reported that WMA technologies should be applicable to aggregate gradations other than the dense gradations including SMA, open-graded, stone-filled, and coarse-base mixtures.

Regarding compaction, Hurley and Prowell (2005a, 2005b, 2006a) reported that standard HMA laboratory compaction procedures were quite acceptable when applied to preparing WMA specimens. However, the use of WMA would result in reducing the production temperatures; therefore, the compaction temperature should be reduced to accurately simulate the field practice.

Recently, the National Cooperative Highway Research Program (NCHRP) has initiated a research project to come up with an improved mix design method for WMA mixtures. However, the results were not published at the time of commencing with this study. Therefore, in this study, it was decided to conduct the mix design on the control HMA mixes and utilize the same aggregate gradation and optimum asphalt binder content in the preparation of the WMA mixes, as will be explained in the upcoming chapters.

2.2.3.3 Characterization of Foamed Asphalt and the Foaming Process

Foamed asphalt is produced by introducing pressurized cold water and air into the heated asphalt in specially designed nozzles. Upon the mixing of cold water and hot asphalt, heat transfers from the hot asphalt to the cold water causing the latter to evaporate, which, in turn, causes the asphalt to foam. Figure 2.5 shows an example laboratory foaming nozzle produced by Wirtgen, Inc. and Figure 2.6 shows an example field foaming nozzle produced by Astec, Inc.

As can be seen from these figures, both systems contain a mixing chamber that facilitates the mixing of the cold water and the preheated asphalt in order to produce the foamed asphalt binder.

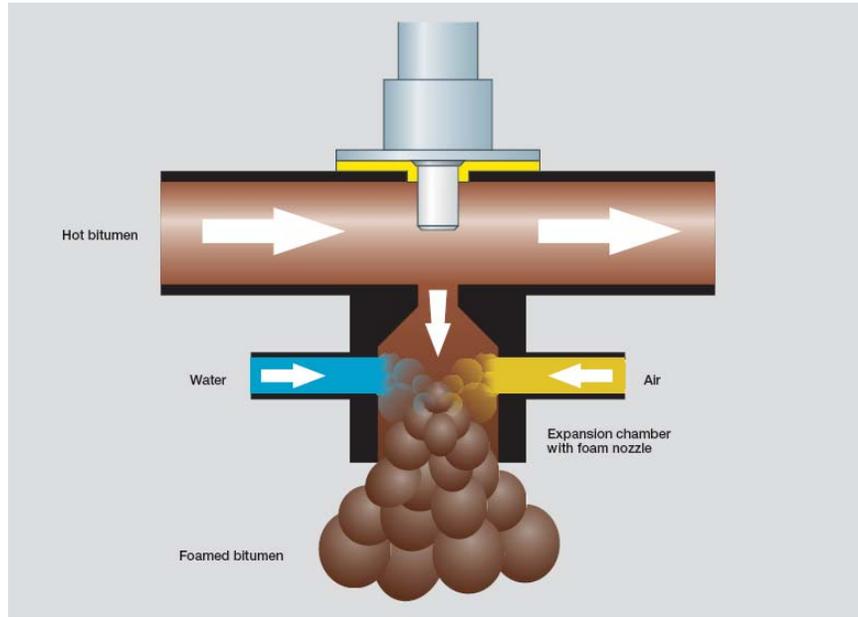


Figure 2.5: Wirtgen WLB 10 Foaming Nozzle (after Wirtgen, Inc.).

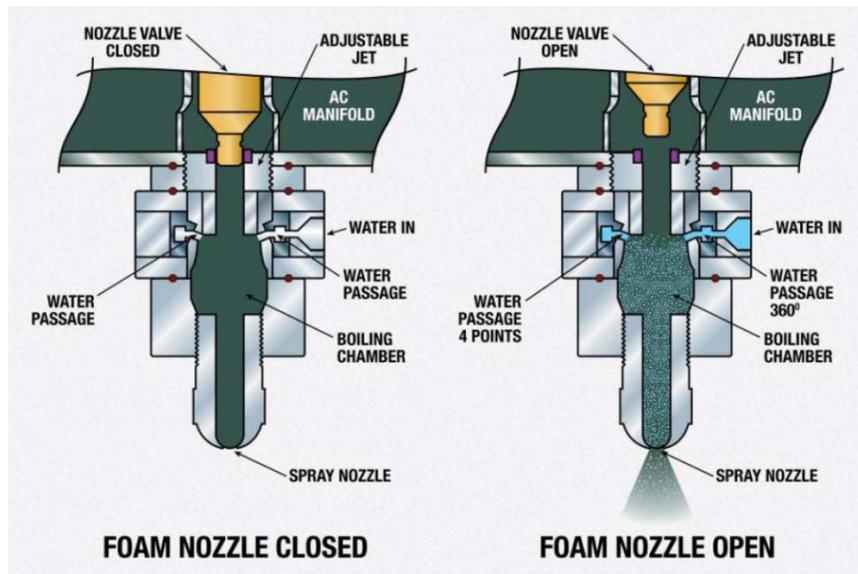


Figure 2.6: Astec Double Barrel Green System Nozzle (after Astec, Inc.).

The foamed asphalt has been characterized by two main foaming properties, namely expansion ratio and half life. The expansion ratio is defined as the maximum volume of foamed asphalt divided by the original volume of the binder, while the half-life is defined as the time (in seconds) for foamed asphalt to collapse from its maximum expansion volume to half of its maximum expansion volume (Jenkins and van de Ven 1999).

The expansion ratio and half-life of an asphalt binder are mainly dependent on the foaming water content, foaming temperature, and type of asphalt binder used. Increasing the foaming water content and/or the foaming temperature would increase the expansion ratio and decrease the half-life. As for the asphalt binder type, it has been reported that softer binders tend to produce more stable foam than harder binders (Saleh 2004), which explains why the former has typically been used in the production of cold-in-place and half-warm asphalt mixtures.

Researchers believe that maximizing the expansion ratio and the half-life would result in the best performing foamed asphalt mixes (Ruckel et al. 1983, Bissada 1987, Muthen 1998). This can be achieved through a series of foaming tests conducted at different foaming water contents. The water content that results in maximizing both foaming properties is defined as the optimum foaming water content and it is usually selected as a range of water contents from graphs such as the one shown in Figure 2.7. The same procedure can be repeated at different foaming temperatures in order to determine the optimum foaming temperature that results in the highest expansion ratio and half-life values.

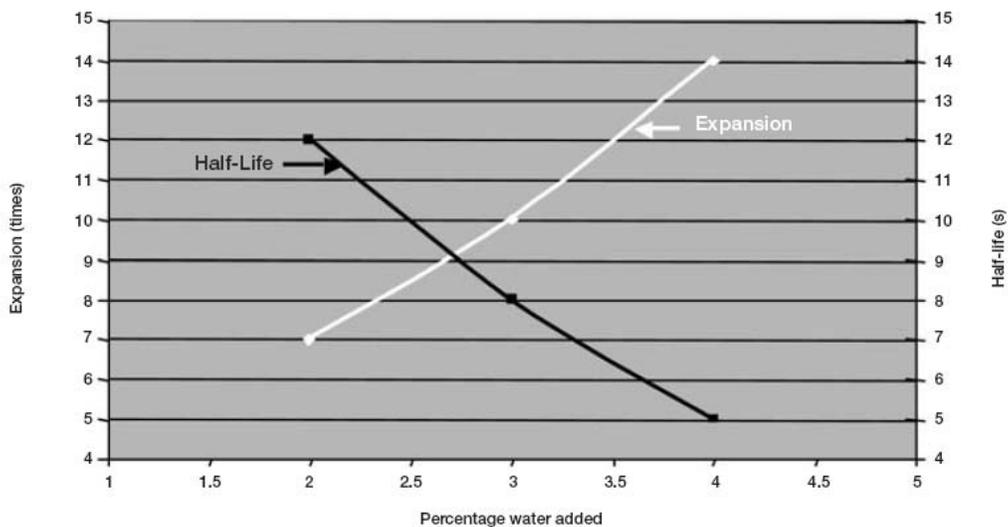


Figure 2.7: Foaming Properties of an Asphalt Binder (after Wirtgen, Inc.).

2.2.4 Hot Mix Asphalt

Hot mix asphalt is the name given to the techniques used to produce asphalt-aggregate mixtures at highly elevated temperatures (275°F to 325°F; 135°C to 163°C). In these mixtures, the mixing and compaction temperatures are determined using the viscosity-temperature relationships of the specific asphalt binder used in the mixture.

2.2.4.1 Mix Design of Hot Mix Asphalt

Currently, hot mix asphalt is the most widely used technique to produce road paving materials. Moreover, mix designs for the hot mix asphalt, have been well established over the last century. Mix design methods that were developed include: Hveem, Marshall, and Superpave. From among these methods, the Ohio Department of Transportation (ODOT) uses the Superpave mix design method for heavy traffic pavements (greater than 1500 trucks in the opening day traffic) and the Marshall mix design method for other pavements. Therefore, these two mix design methods are discussed next in detail.

Marshall Mix Design of HMA

The Marshall mix design method was developed at the Mississippi State Highway Department by Bruce Marshall in the late 1930s. Later on, the U.S. Army Corps of Engineers improved and added certain features to Marshall's procedures through extensive research and correlation studies and ultimately they developed mix design criteria. The need for this modification was required due to the increase in wheel loads especially those of aircrafts.

The Marshall mix design method is considered to be an empirical method established on the basis of observed field performance. It involves selecting an aggregate gradation and a compaction level that will simulate the traffic loading subjected to the pavement structure. The aggregate is then mixed with different percentages of the selected asphalt binder at the specified temperature range and then compacted after subjecting the loose mixture to the specified curing period. Typically, three specimens are compacted at each of the selected asphalt binder contents.

In general, 15 specimens are prepared at 5 different asphalt contents. The bulk specific gravity test is then conducted on those compacted specimens. Furthermore, a loose mixture is also prepared and then used to conduct the Rice specific gravity test (also known as the

maximum theoretical specific gravity test). From those two quantities, the air voids percentage of the mix, at each of the five asphalt binder contents, is then calculated.

The optimum asphalt content is usually selected as the one that will result in 4 percent air voids and achieving a desired minimum VMA requirement. Moreover, the Marshall Stability should be greater than a required minimum whereas the Marshall Flow is required to fall within a specific range to ensure good performance and durability of the mix. However, if the aggregate-asphalt mixture does not achieve the pre-mentioned requirements, the aggregate gradation is changed to start a new trial. In practice, contractors tend to reduce the VMA and by doing so they are able to minimize the optimum asphalt binder content which is the highest contributor to the cost of the mixture.

Superpave Mix Design of HMA

The Superpave (*Superior Performing Asphalt Pavements*) mix design method is the fruit of a research program initiated by the Strategic Highway Research Program (SHRP). The need for this performance-based mix design emerged from the fact that empirical methods, such as the Marshall mix design method, did not assure good performance although good adherence to the standard procedures of these empirical methods was insured.

Another key feature of the Superpave mix design procedure over the other empirical design methods is the Superpave Gyrotory Compactor. Although, its main task is to compact specimens, it can provide information about the compactability of the particular mixture by recording data during the compaction process. In addition, it can be used to design mixtures that do not exhibit classic tender mix behavior and do not densify to dangerously low air void percentages under traffic action. It also can be used to simulate field compaction because its ability to compact specimens at a specified inclination angle rather than a flat surface.

The Superpave mix design method incorporates performance-based asphalt binder characterization tests that account for the effect of temperature on the rheological behavior of the asphalt binder and subsequently the performance of the asphalt-aggregate mixture. According to this procedure, the asphalt binder is tested at high, intermediate, and low temperatures to evaluate its performance against three major concerns, which are respectively, permanent deformation (rutting), fatigue cracking, and low temperature cracking.

The first step in the Superpave mix design procedure is to select an asphalt binder that will be suitable to achieve the best performance in the environment at which the pavement structure will be constructed. The aggregate gradation is also selected to meet the consensus properties (i.e. coarse aggregate angularity, fine aggregate angularity, flat and elongated particles, and clay content) as well as the gradation control limits.

Upon selecting the asphalt binder and aggregates to be used in the mixture, different asphalt-aggregate mixtures are prepared at different asphalt binder contents. Typically, five different asphalt binder contents and two specimens are prepared at each of the selected trial binder contents. Before compaction takes place, however, the specimens are allowed to cure at the compaction temperature for two hours and the level of compaction is determined using estimates of the traffic levels expected to be supported by the pavement structure. Moreover, a loose mixture is prepared and used in the evaluation of the Maximum Theoretical Specific Gravity. After compaction, the specimens are tested for Bulk specific gravity and the air void percentages, at each of the selected trial asphalt content, are calculated.

The optimum asphalt binder content is selected as the binder content that will result in 4 percent air voids. At this optimum asphalt content, minimum requirements of voids filled with asphalt (VFA), percentage of Theoretical Maximum Specific Gravity at the Design compaction level, and voids in mineral aggregate (VMA) should be satisfied in order for the mix to be considered acceptable. If the mix is not considered acceptable, a new trial aggregate gradation should be selected and the procedures should be repeated until an acceptable mix is found.

2.2.4.2 The Bailey Method for Blending Aggregates

The previous mix design methods are mainly dependent on the experience of the mix designer and his or her understanding of local materials. Therefore, several trial aggregate gradations might be required to obtain an aggregate gradation that will result in the most cost effective mix design, while assuring good performance. Understanding aggregate packing in asphalt mixtures is therefore essential to advancing the mix design process and improving the aggregate gradation selection.

The Bailey Method, developed by Mr. Robert Bailey of the Illinois Department of Transportation (IDOT), has the objective of blending the aggregates to achieve desired mixture properties (Vavrik et al. 2002). This new method was developed according to the concept that

durability, strength and rut resistance of an asphalt-aggregate mixture are mainly dependent on the aggregate interlock and packing within the mix. In order to understand how aggregate packing affects the mixture volumetrics, the Bailey method separates the aggregate gradation into two regions, namely coarse aggregate region and fine aggregate region. The coarse aggregates are defined as the large aggregates which will create the voids in the mixture, while the fine aggregates are defined as the fine aggregates that will fill those voids. The Bailey method characterizes the aggregate packing using the particle size ratio. This ratio is defined as the ratio of the coarse aggregates diameter to the fine aggregates diameter. Vavrik et al. (2002) reported that a particle size ratio of 0.22 would be an appropriate value to evaluate the aggregate gradations used in asphalt-aggregate mixtures. Thus, the Bailey method utilizes a particle size ratio of 0.22 to define the Primary Control Sieve (PCS) from the set of standard U.S. sieves.

In addition, the Bailey method incorporates a procedure in which the coarse and fine aggregates are added together by volume rather than by weight. To do so, three quantities have to be evaluated using standard laboratory procedures and another quantity has to be selected. These quantities include the coarse aggregate rodded unit weight, the coarse aggregates loose unit weight, the chosen unit weight, and the fine aggregates rodded unit weight. The loose unit weight of coarse aggregates is determined using a standard 0.25 ft³ (7.1 liters) bucket in which the loose coarse aggregates are filled and allowed to fall from a standard height. It establishes the lower limit of coarse aggregate interlock. Moreover, the rodded unit weight of coarse aggregates, which establishes the upper limit of aggregate interlock, is determined using AASHTO T19 procedures; whereas the fine aggregate rodded unit weight, which indicates the dry compaction state, is determined using a procedure similar to AASHTO T99 procedures. The chosen unit weight is selected to establish the required volume of coarse aggregates in the blend. Vavrik et al. (2002) reported that the densest gradation that can exist is having a chosen unit weight of approximately 5 percent lower than that of the loose unit weight of coarse aggregates. Once these quantities have been determined, an aggregate blend is selected to achieve the desired interlock and strength.

To further refine the selected aggregate gradation, the Bailey method defines four different ratios that include the coarse aggregate ratio (CA), coarse portion of fine aggregate ratio (FA_c), fine portion of fine aggregate ratio (FA_f), and the amount of increase or decrease in the PCS that divides the coarse and fine regions.

The Bailey Method ratios are calculated as follows:

$$CA = \frac{\% \text{ Passing Half Sieve} - \% \text{ Passing Primary Control Sieve}}{100\% - \% \text{ Passing Half Sieve}}$$

$$FA_c = \frac{\% \text{ Passing Secondary Control Sieve}}{\% \text{ Passing Primary Control Sieve}}$$

$$FA_f = \frac{\% \text{ Passing Tertiary Control Sieve}}{\% \text{ Passing Secondary Control Sieve}}$$

where,

Half Sieve = sieve size that is equal to Nominal Maximum Particle Size (NMPS) \times 0.5.

Primary Control Sieve (PCS) = sieve size that is equal to NMPS \times 0.22.

Secondary control sieve (SCS) = sieve size that is equal to PCS \times 0.22.

Tertiary Control Sieve (TCS) = sieve size that is equal to SCS \times 0.22.

The CA ratio is used to evaluate the void structure in the coarse aggregates. Vavrik et al. (2002) reported that a CA ratio below 0.4 may allow the fine aggregates to compact more than desired and the tendency of these mixes to segregation increase, whereas a CA ratio greater than 0.8 may result in a mix that will be difficult to compact in the field.

The FA_c ratio defines the portion of fine aggregates that will create voids in which the fine portion of the fine aggregates will occupy. Therefore, this ratio is desired to be within a range that will maintain an appropriate volume to be filled without overfilling. Vavrik et al. (2002) reported that a FA_c ratio of less than 0.5 is optimum, as larger values mean excessive amounts of fine aggregates, making the mixture tender and having the tendency to overdensify under traffic. If the FA_c ratio is too low, however, the gradation will not be uniform and might be gap-graded in the fine portion which might lead to instability. It was also reported that the FA_c ratio has the greatest impact on the overall voids in mineral aggregates (VMA) of the mixture.

Finally, the FA_f ratio is used to evaluate the aggregate gradation with regard to packing of the smallest portion of the aggregate gradation. A decrease in the FA_f ratio is expected to result in an increase in the mixture's voids. Vavrik et al. (2002) reported that a FA_f ratio of less than 0.5 is optimum for a dense-graded mixture.

In conclusion, the Bailey method is a powerful tool that can be used to minimize the number of trials needed to obtain an asphalt-aggregate mixture. Although the Bailey method requires deep understanding of its concepts and ratios, it can save the practitioner an enormous amount of time that might be used to prepare different aggregate gradation trials.

2.3 Previous Studies on Warm Mix Asphalt

Several field and laboratory experiments have been conducted in the past few years to evaluate the performance of WMA in comparison to HMA. In addition, researchers evaluated the benefits of reducing the viscosity of the asphalt binders when used in warm mixes. Some of these benefits included reducing emissions during mix production, improving compactability, increasing haul distances, and extending the paving season.

Researchers also evaluated the performance of WMA mixtures with regard to various pavement distresses that might arise due to the use of lower mixing and compaction temperatures. Main concerns included the increased susceptibility to permanent deformation (or rutting) since the asphalt binder may not harden as much at lower production temperatures and may easily densify even with proper compaction in the field; and increased propensity to moisture-induced damage since aggregates are heated to lower temperatures and therefore may not thoroughly dry before being mixed with the asphalt binder. This section includes a review of pertinent literature on field and laboratory performance of WMA, with emphasis on studies related to the use of foamed asphalt binders.

2.3.1 Field Performance of WMA

In Europe, several field warm mix asphalt trials were prepared throughout the continent using the WAM-Foam technology and other technologies as reported by Koenders (2000). For instance, in 1996, Norway prepared a dense graded warm mix to be placed as a wearing course. The final binder grade used in the warm mix was 180/200 Pen, while the one used in the control mix was 80/100 Pen. Visual inspection of the warm pavements did not show any signs of rutting. This result was also confirmed in laboratory testing performed on cores taken from the field at the time of construction and one year later. In the United Kingdom, another dense graded warm mix asphalt wearing surface was prepared. The grade of the final binder used was 80/100 Pen. Testing of cores taken from the field has shown similar performance for the WMA and HMA mixes. In addition, overall testing of warm mixes in Germany and France have shown that warm mixes perform better or equal to the hot mixes (Koenders 2000).

In Canada, several warm mix asphalt field trials were constructed using Asphamin and Evotharm technologies (Davidson 2007). In the case of Asphamin, six trial sections were constructed in 2005 and 2006. Results from these trials have shown that warm mixes using

Asphamin improve compactability and reduce emissions by 20 to 30 percent of the typical hot mixes. In the case of Evotherm, a total of seven field trials were also constructed from 2005 to 2007. In 2005, three Evotherm Emulsion trials were constructed. The grade of the asphalt binder used in these trials was PG 58-28. Another trial of warm mix asphalt using Evotherm Emulsion was constructed in 2006. This trial included about 15 percent of RAP and used PG 58-28 asphalt binder. The 2007 trials were constructed using Evotherm Dispersed Asphalt Technology. Results from these trials have shown that warm mixes using Asphamin and Evotherm perform equally to hot mixes (Davidson 2007).

In 2000, an asphalt pavement test track was constructed at the National Center for Asphalt Technology (NCAT) to evaluate the performance of various asphalt mixtures (Prowell et al. 2007). The test track consisted of 45 different flexible pavement sections. Each of these sections had a length of 200 ft (61 m), resulting in a total test track length of about 1.7 miles (3.2 km). Three warm mix asphalt sections were used in the test track to evaluate the performance of warm mix asphalt using Evotherm (i.e. emulsion-based). The test sections were subjected to 515,333 ESALs of equivalent traffic in a 43-day period. Wire-line rut depth measurements of the three sections compared to the control sections prepared using hot mix asphalt, have shown that warm mix asphalt containing Evotherm provided good rutting resistance even under conditions in which traffic was quickly returned to the pavement.

Recently NCAT has completed another study that focused on WMA produced using foamed asphalt binders. In this study, the laboratory performance of WMA produced in a plant using the Gencor Green Machine Ultrafoam GX was evaluated and compared to that of an HMA mixture with the same aggregate and binder materials (Kvasnak et al. 2010). The results of this study showed that while the laboratory performance of the WMA mixtures was lower than the HMA mixtures for many of the tests, the WMA performance exceeded minimum laboratory performance thresholds in most cases. The rutting results of Hamburg Wheel Tracking and Asphalt Pavement Analyzer (APA) tests showed were acceptable for the WMA and HMA mixtures. In addition, the indirect tensile strength for the WMA was high and improved with aging. However, its tensile strength ratio did not meet the Superpave 0.8 criterion. Based on the results of this study, it was concluded that the WMA produced using Gencor Green Machine Ultrafoam GX is a promising technology.

In a study by Sargand et al. (2009), the field performance of Asphamin, Sasobit, and Evotherm WMA technologies has been evaluated in Ohio. The study consisted of an outdoor component and an indoor component. The outdoor asphalt pavements were constructed on State Route 541 and consisted of four asphalt pavement surface courses. Three of these surface courses were prepared using the three WMA technologies and the fourth surface course was constructed using standard HMA procedures and served as the control layer. The indoor component involved constructing four lanes inside an accelerated pavement load facility (APLF) using the same materials used along State Route 541. The authors reported that the emissions were reduced by about 67 to 77 percent for WMA as compared to the control mix. Furthermore, it was reported that WMA mixes tested in the APLF have shown more rutting during the initial stages; however, after the initial stages have ended further rutting was approximately equal.

Wielinski et al. (2009) reported the results of a study where Granite Construction built two WMA paving projects from its Indio California facility. Both projects were paved with WMA produced using the Astec's Double Barrel Green System. Control sections consisting of typical HMA were included in both projects to compare WMA and HMA mix properties and performance. Samples of WMA and HMA mixtures were obtained during construction and were compacted for testing in the laboratory. The results of this study demonstrated that WMA mixtures could be produced and placed at lower temperatures while yielding mix properties and field compaction similar to those of conventional HMA. In addition, the initial field performance of the WMA and HMA sections was similar. The results of the laboratory tests conducted in this study showed that WMA possessed lower initial stiffness as indicated by lower Hveem stability, Marshall stability and flow, and higher APA rut depths. In addition, both the HMA and WMA mixtures had low TSR results with the WMA results being slightly lower than the HMA. Based on the results of this study, it was suggested that conventional mix design methods could be used for WMA mixtures produced using the Double Barrel Green System.

2.3.2 Laboratory Performance of WMA

Several laboratory studies have been conducted on warm mix asphalt mixtures to evaluate their performance. In Europe, the Nottingham Asphalt Tester was used to evaluate the rutting potential of the WAM-Foam technology. This test was conducted at a temperature of 104°F (40°C) for a period of 3600 seconds with a loading cycle of 14.5 psi (100 kPa) pulse of 0.2

seconds duration followed by a 1.8 second resting period. The compactability of the warm mix was evaluated using the gyratory compactor at a range of temperatures between 158°F and 176°F (70°C and 80°C). Fatigue cracking was also evaluated using the three point bending fatigue test. The fatigue test was performed at a frequency of 40 Hz and at a temperature of 50°F (10°C). The test results revealed good performance for the WMA mixes as compared to HMA (Koenders 2000).

An evaluation study of Asphamin, Sasobit, and Evotherm was conducted at the National Center for Asphalt Technology (NCAT); (Prowell et al. 2007). Researchers in this study used the Superpave Gyratory Compactor (SGC) to check the compactability of the warm mix. It was reported that the gyratory compactor is insensitive to temperature changes, thus, leading to the use of a vibratory compactor to check the compactability of the warm mix. Results have shown that Evotherm had the lowest air voids followed by Sasobit and finally Asphamin. Statistical results have shown that the three technologies used have significantly decreased the air voids compared to the control mix. Furthermore, the asphalt pavement analyzer (APA) was used to evaluate the rutting susceptibility of warm mixes. Reported results state that Evotherm was the best at reducing the rut depth followed by Sasobit then Asphamin and all of these technologies have shown no significant increases or decreases in the rut depth compared to hot mixes.

Prowell et al. (2007) also evaluated the moisture susceptibility of the three WMA mixtures using the indirect tensile strength based on ASTM D4867 and the Hamburg wheel tracking device. Results from these tests indicated that Asphamin decreased the Tensile Strength Ratio (TSR) to an unacceptable value according to the Superpave requirements (i.e. less than 0.8). Sasobit and Evotherm results have shown that the TSR value depends on the type of aggregate being used in the mix. Sasobit increased the TSR for limestone aggregates and decreased it for granite aggregates, while Evotherm increased the TSR for granite and decreased it for limestone.

Finally, Prowell et al. (2007) studied the resilient modulus of the three WMA mixtures. The test results indicated that Evotherm and Asphamin have increased the resilient modulus of the warm mix, while Sasobit has decreased it. However, the difference in resilient modulus between the three WMA mixtures and the control mix was statistically insignificant.

Another study was conducted by Wasiuddin et al. (2007) to evaluate the performance of Asphamin and Sasobit in terms of rutting susceptibility, viscosity changes, and stiffness. In this study, the APA device was used to evaluate the rutting susceptibility of the warm mixes. Results from this test have shown that both Sasobit and Asphamin have decreased the rutting potential of the warm mixes compared to the control hot mix.

Xiao et al. (2009) conducted a laboratory study to examine moisture damage in WMA mixtures containing moist aggregates. The study included two percentages of moisture content (0% and 0.5% by weight of the dry mass of the aggregate), two WMA additives (Asphamin and Sasobit), and three aggregate sources. The test results indicated that the dry indirect tensile strength values were affected by the aggregate moisture and hydrated lime contents. However, the use of WMA additive did not significantly alter the dry indirect tensile strength and toughness values. It was also reported that the deformation resistance and TSR values decreased with the increase in the aggregate moisture content.

Kvasnak et al. (2009) evaluated the moisture susceptibility of laboratory and plant produced WMA mixes as part of a field demonstration project in Alabama. The results of this study indicated that the laboratory produced WMA was more prone to moisture susceptibility than the plant produced mix. The HMA exhibited more favorable TSR values than the WMA; however, most of the WMA samples did meet the Superpave moisture susceptibility criterion.

In a more recent study, Kvasnak et al. (2010) compared the performance of mixes produced using the Gencor Green Machine Ultrafoam GX to a control hot mix asphalt with regard to moisture susceptibility, permanent deformation, and fatigue cracking. Both mixes were prepared at the same production facility using the same aggregate gradation, liquid asphalt, and asphalt content. It was reported that the WMA mixtures had lower TSR values and hence might be more susceptible to moisture damage than HMA mixtures. However, it was suggested that the resistance of WMA mixtures to moisture can be increased through the use of anti-strip agents. It was also reported that the WMA mixtures had slightly higher rut depths than HMA mixtures, but the difference was statistically insignificant. Finally, it was reported that the WMA mixtures had lower fatigue life than HMA mixtures at low strain levels; however, no difference was observed at higher strain levels. The endurance limit of each mix was also determined and indicated that the WMA may incur damage at lower loading level than the HMA.

CHAPTER 3
MATERIAL DESCRIPTION

3.1 Aggregates

Two types of aggregates were used in this study. The first type consisted of natural gravel and natural sand obtained from Central Allied, Canton, Ohio, while the second type consisted of crushed limestone and limestone sand obtained from National Lime and Stone (Carey Quarry), Akron, Ohio. The aggregate suppliers were selected from ODOT’s approved list of aggregate suppliers. Hence, the properties of the aggregates have been tested and approved by ODOT. Table 3.1 shows the specific gravities and absorption of the aggregates as obtained from ODOT and Table 3.2 shows the gradations of the aggregates as obtained from the suppliers.

Table 3.1: Specific Gravities and Absorption of Selected Aggregates
(after ODOT Aggregate Specific Gravity List 2009).

Supplier	Gradation	Bulk Dry Gravity	SSD Gravity	Absorption %
Central Allied	#8	2.559	2.607	1.87
	Sand	2.603	2.626	0.90
National Lime and Stone	#8	2.611	2.660	1.87
	Sand	2.748	2.772	0.89

Table 3.2: Supplier Provided Aggregate Gradations.

Supplier	Aggregate	Sieve	% Passing	Supplier	Aggregate	Sieve	% Passing
Central Allied	Sand	3/8"	100	Nat. Lime and Stone (Carey Quarry)	Sand	3/8"	100
		#4	100			#4	94.5
		#8	87			#8	66.7
		#16	63.5			#16	44.2
		#30	40.3			#30	28.1
		#50	13.1			#50	16.8
		#100	3.3			#100	8.5
		#200	1.3			#200	4.56
	#8	1/2"	100		#8	1/2"	100
		3/8"	87.8			3/8"	93.1
		#4	17.5			#4	20.2
		#8	3.9			#8	4.3
		#16	2.2			#16	2.6
		#200	0.62			#200	---

The shape of the natural gravel and natural sand aggregates can be described as rounded with a very small percentage of flat and elongated particles. The surface of these aggregates is a smooth surface, which is typical for most of rounded aggregates. In general, aggregates that are rounded with smooth surfaces tend to have better workability and require less compaction effort to achieve the required density. However, mixes prepared using such aggregates tend to be more susceptible to permanent deformation (rutting) due to low voids and plastic flow. Figure 3.1 shows a picture of the natural gravel and natural sand used in this study.



Figure 3.1: Natural Gravel (right) and Natural Sand (left).

On the other hand, the crushed limestone and limestone sand aggregates are angular-shaped. Such aggregates typically provide greater interlock within the mixture, which in turn leads to a greater mechanical stability under traffic. Furthermore, angular-shaped aggregates tend to have greater air voids within their structure. Therefore, a greater compaction effort might be required to achieve the desired density. Although angular-shaped aggregates may require more asphalt binder to fill the voids and to coat the aggregate particles, they usually form a strong bond with the binder because of their rough surface. Figure 3.2 shows a picture of the limestone gravel and the limestone sand used in this study.



Figure 3.2: Crushed Limestone (right) and Limestone Sand (left).

3.2 Asphalt Binders

PG 64-22 (unmodified) and PG 70-22M (polymer modified) have been used in several WMA field trials constructed in Ohio; therefore, those two performance-graded binders were used in this study. Both asphalt binders were obtained from Marathon Petroleum Company, an ODOT approved asphalt binder supplier located in Cleveland, Ohio. Marathon provided the specific gravity as well as the mixing and compaction temperatures of the two asphalt binders. Table 3.3 shows the properties of each asphalt binder as obtained from the supplier.

Table 3.3: Asphalt Binder Properties (after Marathon Petroleum Company).

Property	PG 64-22		PG 70-22M	
Specific Gravity, 15.6°C (60°F)	1.034		1.033	
Specific Gravity, 25°C (77°F)	1.028		1.027	
Density, 15.6°C (60°F), lb/gal	8.611		8.603	
Rotational Viscosity @ 135°C, Pa.s	0.411		1.066	
Rotational Viscosity @ 165°C, Pa.s	0.116		0.289	
Lab Mixing Temperature, °F	306 (min)	317 (max)	306 (min)	325 (max)
Lab Compaction Temperature, °F	286 (min)	294 (max)	286 (min)	306 (max)

The asphalt binder supplier indicated that the mixing and compaction temperatures of the polymer modified asphalt binder (i.e. PG 70-22M) were not based on the viscosity-temperature relationship, but rather on laboratory and field experience with this asphalt binder. In this research, hot asphalt mixtures containing PG 70-22M were prepared using mixing and compaction temperatures of approximately 320°F and 293°F (160°C and 145°C), respectively. Meanwhile, hot asphalt mixtures containing PG 64-22 were prepared using slightly lower mixing and compaction temperatures of approximately 312°F and 290°F (156°C and 143°C), respectively. Initially, lower mixing temperatures were used for both asphalt binders; however, it was observed that the coating and compaction of mixtures containing PG 70-22M has significantly improved by increasing the mixing temperature, which confirms the trend suggested by the asphalt binder supplier.

Table 3.4 shows two values for the specific gravities of the asphalt binders. In this study, the specific gravity at 60°F (16°C), which is commonly used for selling or buying asphalt binders, was selected to conduct the weight-volume calculations. Generally, the specific gravity of the asphalt binder is used to calculate the effective specific gravity of the aggregate, which in turn is used to estimate the Rice specific gravity of the asphalt mixture at different asphalt contents.

3.3 Mix Design

As discussed in Chapter 2, ODOT employs two mix design methods (Superpave and Marshall) in the selection of the optimum asphalt binder content for hot mix asphalts. The Superpave mix design method is used for pavements subjected to heavy traffic (greater than 1500 trucks in the opening day traffic). Meanwhile, the Marshall mix design method is used for pavements subjected to low to medium traffic levels. Over the last three years, ODOT has permitted the use of warm mix asphalt prepared using foamed asphalt binder on low to medium traffic pavements. Given that no unique mix design method was available for warm mix asphalt, it was decided to use the same optimum asphalt binder content obtained for traditional hot mix asphalt using conventional mix design methods.

In this study, the Marshall mix design method was selected to obtain the optimum asphalt binder content for the four mixes prepared using the two aggregate sources (crushed limestone and natural gravel) and the two asphalt binders (PG 64-22 and PG 70-22M). All mixes were

designed to meet the requirements for ODOT Construction and Materials specifications (C&MS) Item 441 Type I surface mixture subjected to medium traffic (Table 3.4). This particular material item was selected since it was previously used in Ohio for warm mix asphalt field trials.

Table 3.4: ODOT Gradation and Mix Design Requirements for Type I Surface Mix Subjected to Medium Traffic (after ODOT C&MS 2008).

Course Traffic	Type I Surface Course Medium
1/2"	100
3/8"	90-100
No. 4	45-57
No. 8	30-45
No. 16	17-35
No. 30	12-25
No. 50	5-18
No. 100	2-10
No. 200	---
F/A Ratio ¹ , max	1.20
F-T Value ² (%)	+2
Asphalt Binder (%)	5.8-10
Virgin Asphalt Binder (%), min	5.00
Blows/Face	50
Stability (lbs), min	1200
Flow (0.01 in.)	8-16
Design Air Voids (%)	3.50
VMA (%), min	16

¹F/A Ratio = Percent passing Sieve No. 200 divided by asphalt content

²F-T Value = (Sieve No. 30 – Sieve No. 50) – (Sieve No. 16 – Sieve No. 30)

3.3.1 Aggregate Gradation

The Bailey method was used in the selection of the aggregate gradation. Ten trial aggregate gradations, complying with ODOT requirements shown in Table 3.4, were selected and tested for each aggregate type. The first three gradations were selected to be close to the WMA gradations used in Ohio. The remaining seven gradations were selected according to the Bailey Method concepts with the objective of minimizing the optimum asphalt content through minimizing the voids in mineral aggregates (VMA) in order to reduce the cost of the designed asphalt mixture. Tables 3.5 and 3.6 show the trial aggregate gradations used in the Bailey method for the natural gravel and crushed limestone, respectively.

Table 3.5: Trial Gradations Used in the Bailey Method for Natural Gravel and Natural Sand.

Trial Gradations											
Sieve #	Opening	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
1/2"	12.5	100	100	100	100	100	100	100	100	100	100
3/8"	9.5	93	91	95	96	96	92	96	96	96	92
#4	4.75	52	50	52	56	51	47	52	56	57	55
#8	2.36	42	38	38	40	36	34	38	35	40	41
#16	1.18	29	27	27	25	23	23	25	23	21	25
#30	0.6	21	19	19	17	17	17	17	17	17	14
#50	0.3	10	8	8	7	7	7	11	9	11	5
#100	0.15	4	4	4	3	3	3	3	4	4	4
#200	0.075	2	2	2	1	1	1	2	2	2	2

Table 3.6: Trial Gradations Used in the Bailey Method for Crushed Limestone and Limestone Sand.

Trial Gradations											
Sieve #	Opening	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
1/2"	12.5	100	100	100	100	100	100	100	100	100	100
3/8"	9.5	95	94	94.5	92	93	92	92	92	92	96
#4	4.75	51	56	53.5	52	53	55	55	56	48	52
#8	2.36	38	41	39.5	43	41	43	43	43	43	43
#16	1.18	26	30	28	32	30	33	33	32	32	32
#30	0.6	18	22	20	24	22	24	24	24	24	24
#50	0.3	12	16	14	16	16	13	13	16	16	16
#100	0.15	6	8	7	8	7.5	6	8	8	8	8
#200	0.075	3	3	3	3	3	2	2	3	3	3

The Bailey Method ratios were calculated and utilized to determine the effect of any modification in the selected aggregate gradation on the mix volumetrics:

- 4% increase in Primary Control Sieve (PCS) results in 1% decrease in VMA and Air voids.
- 0.2 change in Coarse Aggregate ratio (CA ratio) result in 1% increase in VMA and Air voids.
- 5% increase in Coarse Portion of Fine Aggregates ratio (FA_c ratio) results in 1% decrease in VMA and Air voids.
- 5% increase in Fine Portion of Fine Aggregates ratio (FA_f ratio) results in 1% decrease in VMA and Air Voids.

Linear interpolation was used to calculate the overall effect of the Bailey Method ratios on the VMA. A predicted VMA for the proposed gradation was calculated by adding or subtracting the overall effect of the ratios to or from the actual VMA value of the mix prepared using one of the selected base gradations. After that, mixtures were prepared using the proposed gradations and the actual VMA was evaluated.

Based on the outcome of the Bailey Method, one gradation was selected from each aggregate type to conduct a full mix design analysis. It is worth noting that in the case of mixtures containing natural gravel, the main challenge in selecting the aggregate gradation was satisfying the minimum VMA requirement. While in the case of mixtures containing limestone, all gradations met the minimum VMA value. It was more challenging though to minimize the mix VMA to reduce the optimum asphalt binder content. Table 3.7 shows the aggregate gradations selected for the natural gravel and the crushed limestone aggregates. In order to allow for the comparison between the performance of the PG 64-22 and PG 70-22M, the same aggregate gradation was used for both asphalt binders.

Table 3.7: Selected Aggregate Gradations and the Modifications.

Sieve #	Natural Gravel	Crushed Limestone
½"	100	100
3/8"	96	92
#4	56	52
#8	35	43
#16	23	32
#30	17	24
#50	9	14
#100	4	8
#200	1	3

Figures 3.3 and 3.4 show the 0.45-power charts for the selected aggregate gradations. It can be seen from these figures that the natural gravel gradation is closer to the upper limit of the control points, whereas the crushed limestone gradation is closer to the lower limit of the control points. This indicates that the natural gravel mixes are coarser than the crushed limestone mixes.

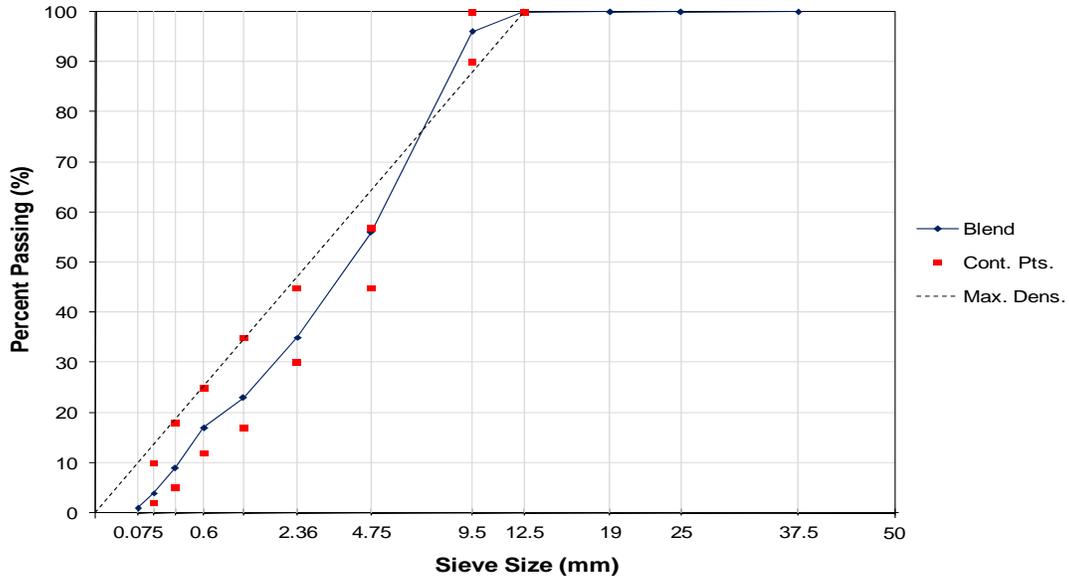


Figure 3.3: Natural Gravel and Natural Sand Gradation Chart.

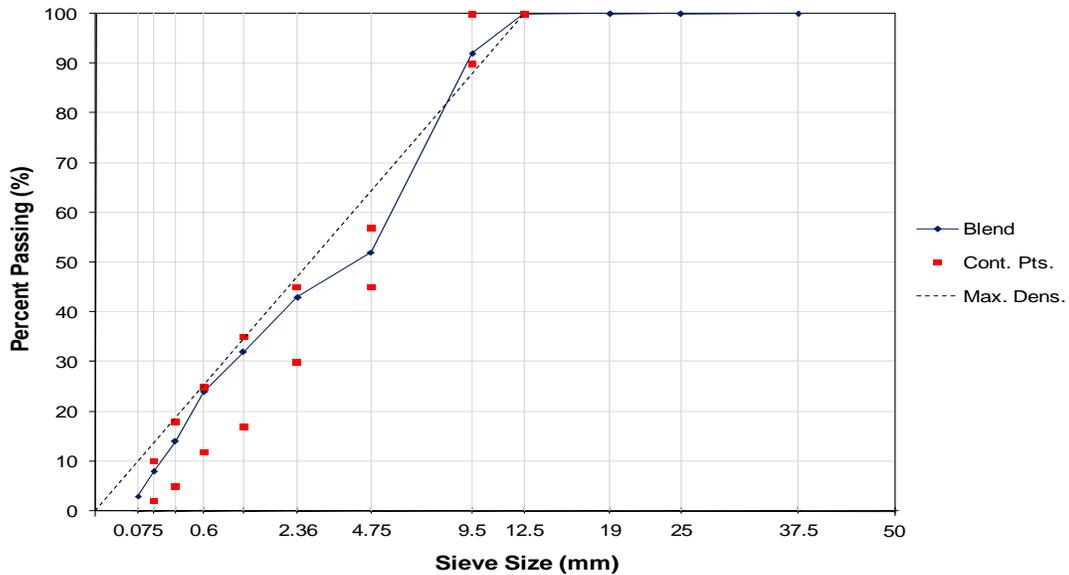


Figure 3.4: Crushed Limestone and Limestone Sand Gradation Chart.

To better understand the resulting aggregate gradations, the Bailey Method ratios for these gradations were calculated and compared to the desired ranges suggested in the literature (Table 3.8). As can be seen in this table, the Bailey Method ratios for the natural gravel fell within the desired ranges for both CA and FA_c, but not for FA_f. Meanwhile, all three ratios fell outside the desired ranges for the crushed limestone gradation. Falling outside the desired range, especially for the CA ratio, is an indication that the selected aggregate gradation might result in a tender mix that may overdensify under traffic (Vavrik et al. 2002). Given the tight control on aggregate gradation for Item 441 (Type 1 surface mix, medium traffic), it was unavoidable to exceed the desired range of CA ratio for the crushed limestone aggregate. Therefore, future research might be needed to determine the relationship between the Bailey Method ratios and mix performance.

Table 3.8: Bailey Method Ratios for Selected Gradations.

Ratio	Desired Range ¹	Natural Gravel	Crushed Limestone
PCS	--	35	43
CA ratio	0.4-0.55	0.477	0.188
FA _c ratio	0.35-0.5	0.486	0.558
FA _f ratio	0.35-0.5	0.235	0.333

¹After Aurilio et al. (2006).

3.3.2 Optimum Asphalt Content

Upon the selection of the aggregate gradations, the optimum asphalt binder content for all four mixes was determined using the Marshall mix design method. The procedure that was implemented can be summarized as follows:

- Use four different asphalt binder contents (5.5%, 6%, 6.5%, and 7%).
- Heat the aggregates and the asphalt binder to mixing temperature for a minimum of two hours.
- Mix aggregate batches to prepare three specimens at each of the asphalt binder content selected.
- Place the mixture in a flat pan.
- Spread the mixture over the whole area of the pan.

- Place the pan containing the mixture in the oven at the compaction temperature for two hours (i.e. curing for two hours).
- Compact the specimens using the required compaction effort. ODOT specifies 50 blows per side for medium traffic. One loose mixture should be kept to conduct the Rice Specific Gravity test.
- Allow specimens to cool down to room temperature.
- Conduct the Bulk Specific Gravity test on the compacted specimens.
- Conduct the Rice Specific Gravity on the loose mixture.
- Conduct the Marshall flow and stability test on the compacted specimens.
- Analyze the data and select the optimum asphalt binder content as the asphalt content that will result in 3.5 percent air voids.

A summary of the mix design results is presented in Table 3.9. As can be seen in this table, the optimum asphalt binder content was higher for mixtures containing crushed limestone than those containing natural gravel. Furthermore, mixtures containing crushed limestone had higher VMA, stability, and flow values than those containing natural gravel. As shown in this table, mixtures containing both natural gravel and crushed limestone met the F-T ratio and the F/A value requirements.

Table 3.9: Summary of Mix Design Results.

Criteria	Requirement	Natural Gravel		Crushed Limestone	
		PG 64-22	PG 70-22M	PG 64-22	PG 70-22M
Stability (lbs)	1200 (min)	1673	2300	3200	4217
Flow (0.01 in.)	8-16	10.5	10.6	13	13.5
VMA (%)	16 (min)	15.5	15.5	16.7	16.6
Air Voids (%)	3.5	3.5	3.5	3.5	3.5
AC (%)	5.8-10	6	6	6.4	6.5
F-T ratio	+2	+2	+2	-2	-2
F/A value	1.2 (max)	0.17	0.17	0.47	0.46

CHAPTER 4

PRODUCTION OF FOAMED WARM MIX ASPHALT

4.1 Introduction

Foamed warm mix asphalt mixtures (WMA-FA) are prepared using plants that utilize different foaming devices such as Astec, Terex, and Gencor. These devices are usually attached to the end of the asphalt binder line right before entering the drum mixer. These devices operate by injecting small molecular-sized cold water particles into the heated asphalt. Upon contact, the cold water will evaporate forming steam which in turn forces the asphalt binder to expand and increase in volume. Therefore, the use of lower mixing and compaction temperatures can be facilitated since the viscosity of the asphalt binder is reduced. This will allow producing WMA-FA mixtures at lower temperatures, without the need for any additional plant modifications. Figure 4.1 depicts a multi-nozzle foaming device produced by Astec, Inc. and commonly used with their Double Barrel Green system.

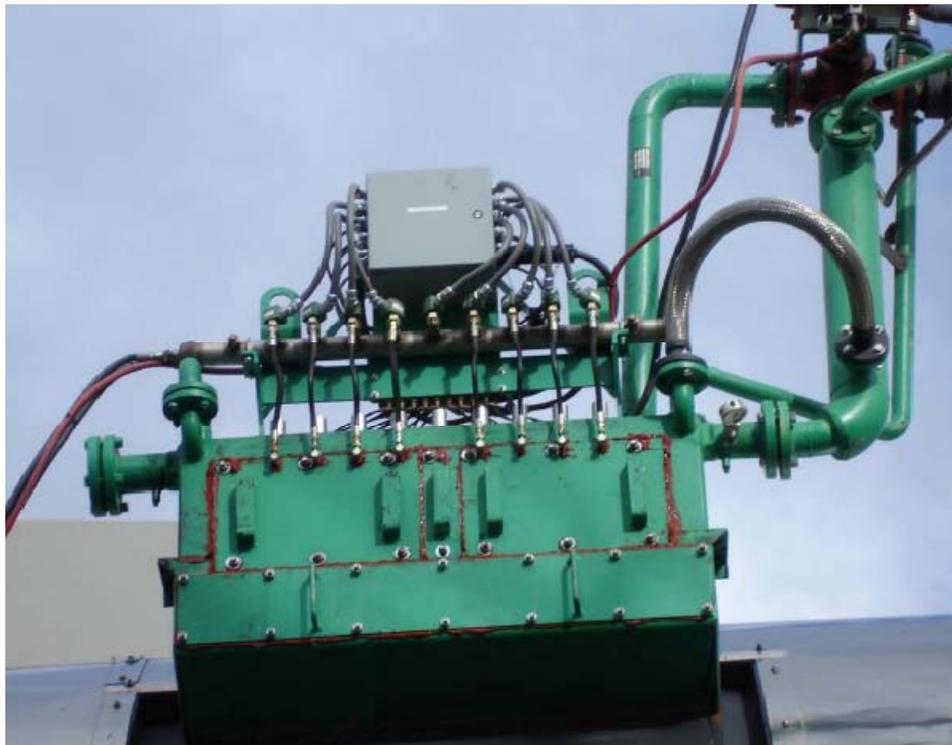


Figure 4.1: Multi-Nozzle Foaming Device (after Astec, Inc.)

The WMA-FA mixtures were produced at 30°F (15°C) lower mixing and compaction temperatures than the traditional HMA mixtures. Furthermore, a foaming water content of 1.8% was used in the production of the WMA-FA mixtures. This procedure is consistent with current ODOT specifications for WMA-FA mixtures that require using a maximum foaming water content of 1.8% and a compaction temperature that is 30°F (15°C) lower than that of the HMA. These specifications were implemented to represent plant production temperatures and Quality Control (QC) testing in the field plant laboratory.

4.2 Laboratory Production of Foamed WMA Mixtures

In this study, a laboratory scale asphalt binder foaming device called WLB10, produced by Wirtgen, Inc., was used to foam the asphalt binder (Figure 4.2). This device utilizes the same process in producing foamed asphalt binders to that used by the previously-mentioned field foaming devices. The WLB10 device consists of an asphalt binder tank, a water tank, an air tank, an asphalt pump, heating components, a foaming nozzle, air and water pressures regulators, and a control panel.

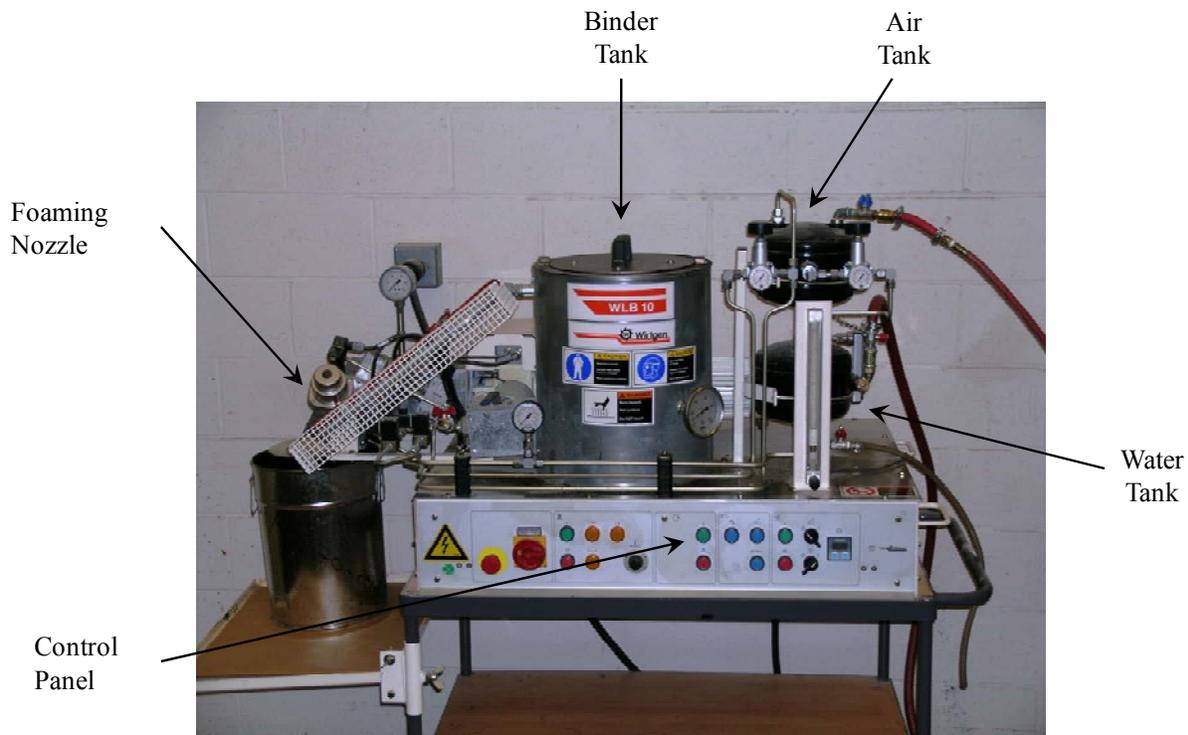


Figure 4.2: Wirtgen WLB10 Asphalt Foaming Device.

To operate this device, the water tank is first filled with water then the air pressure and water tanks are pressurized to the desired air and water pressures required to foam the asphalt binder by adjusting the air and water pressure regulators (4 bars air pressure and 5 bars water pressure were used in this study as recommended by Wirtgen). The asphalt binder tank is then heated and filled with the pre-heated asphalt binder. After heating all other components, such as the asphalt pump and the foaming nozzle, the asphalt binder is circulated through the system and the amount of water required to foam the asphalt binder is selected by adjusting the water flow regulator. The amount of foamed asphalt discharged from the foaming nozzle is controlled using a timer. In this timer, every one second of running the device would result in approximately 100 grams of foamed asphalt binder to be discharged from the nozzle. Therefore, the timer should be adjusted depending on the desired amount of asphalt binder to be used in the mix.

In the asphalt tank, the asphalt binder is heated to the mixing temperature provided by the asphalt binder supplier (306 to 317°F (152°C to 158°C) for PG 64-22 and 306 to 325°F (152°C to 163°C) for PG 70-22M) to ensure that the asphalt binder to be foamed is easily circulated through the foaming device. Within the foaming nozzle, the heated asphalt binder is mixed with small molecules of cold pressurized water. Upon mixing, the cold water will vaporize forming steam, which in turn foams and expands the asphalt binder and eventually reduces its viscosity. The amount of water used to foam the asphalt binder was 1.8 percent of the total weight of the asphalt binder. This quantity represents the maximum water content permitted by ODOT in the production of WMA-FA mixtures. In order to calculate the amount of flow to be set on the water flow gage, the following equation is used, as specified by Wirtgen:

$$Q_{H_2O} = \frac{Q_{Asphalt} \times P_{H_2O} \times 3.6}{100}$$

where,

Q_{H_2O} = Water flow-through volume (liter/hour).

$Q_{Asphalt}$ = Asphalt flow-through volume (100 gram/sec).

P_{H_2O} = Water content (%).

3.6 = Calculation factor.

Once the foaming parameters (i.e. air and water pressures, asphalt foaming temperature, and foaming water content) have been selected and the foaming device has been adjusted, the foamed asphalt binder is discharged from the foaming nozzle into a mixing bowl that contained

the aggregates, which have been preheated in accordance with current ODOT specifications for WMA mixtures. The mixing bowl is then transferred to the mechanical mixer for mixing. A mixing period of 3 minutes, similar to that used when preparing HMA mixtures, has shown to be sufficient when preparing WMA-FA mixtures.

During the preparation of the WMA-FA mixtures several observations were made:

- Using the same foaming parameters, the expansion ratio of the unmodified asphalt binder PG64-22 was slightly higher than the expansion ratio of the modified asphalt binder PG 70-22M. Therefore, it is concluded that unmodified asphalt binders are easier to foam than modified asphalt binders.
- All aggregates were found to be fully coated. This was the case for both gravel and limestone aggregates as well as PG 64-22 and PG 70-22M asphalt binders.
- Although WMA-FA mixtures were produced at lower temperatures, the handling of such mixtures was observed to be easier than HMA mixtures. Therefore, it is concluded that WMA-FA mixtures had better workability.

4.3 Advantages and Disadvantages of WLB10 Asphalt Foaming Device

The laboratory procedure for preparing the WMA-FA mixtures was found to be quite satisfactory. Nonetheless, differences between laboratory and field procedures will always remain. Therefore, it is recommended that future work expands to compare laboratory and field produced WMA-FA mixtures.

The advantages and disadvantages of the WLB10 asphalt foaming device can be summarized as follows:

- Advantages:
 - Ability to operate at a storage capacity of 5 gallons (15.1 liters) of asphalt binder which facilitates preparing a large number of specimens.
 - Ability to heat the foaming nozzle, the asphalt binder tank, and asphalt pump automatically as their temperature drops with time.
 - Ability to automatically mix the required amount of pressurized water with the heated asphalt binder.
 - Easily cleaned, operated, and maintained.

- Disadvantages:
 - The amount of asphalt discharged from the device has to be checked every time the device is operated.
 - Experience working with the device has to be developed to ensure obtaining consistent specimens.
 - The amount of time required to prepare the device before mixing specimens is relatively long (about 1 to 2 hours).
 - It has to be cleaned regularly to ensure that the pipes as well as the air and water tanks are free of rust that might hinder the foaming process and cause inconsistencies.

CHAPTER 5

TESTING PLAN

5.1 Introduction

Three performance tests were implemented in this study to characterize the behavior of WMA-FA mixtures in comparison to traditional HMA. These tests included AASHTO T 283 (Standard Method of Test for Resistance of Compacted Hot Mix Asphalt to Moisture-Induced Damage), AASHTO TP 63-07 (Determining Rutting Susceptibility of Asphalt Paving Mixtures Using the Asphalt Pavement Analyzer), and AASHTO TP 62-07 (Standard Method of Test for Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures). These tests were selected in order to assess the performance of WMA-FA mixtures with regard to moisture susceptibility and permanent deformation that might arise due to the use of lower mixing and compaction temperatures during production and utilizing water in the foaming process.

The previous tests were conducted on specimens prepared using the aggregate-binder combinations presented in Chapter 3. Both WMA-FA and HMA specimens were prepared using the same aggregate gradation and asphalt binder content, which is consistent with the current practice in Ohio. The following subsections offer an overview of the undertaken testing procedure as well as the specimen preparation techniques required to prepare representative samples for these tests. Where applicable, the testing procedure was modified according to the standard practices implemented in the State of Ohio.

5.2 Moisture Susceptibility (AASHTO T 283)

The AASHTO T 283 test method was implemented to characterize the susceptibility of WMA-FA and HMA mixtures to moisture-induced damage. This test method specifies compacting specimens to an air voids content of 7 ± 0.5 prior to testing. A trial and error procedure was implemented to determine the number of Marshal hammer blows per face needed to satisfy this requirement. The specimens were compacted using 5 to 35 blows per face and tested to determine the air voids content achieved at each compaction level. A linear regression model was used to represent the relationship between the air voids content and the number of blows per face and utilized to predict the required number of blows that will achieve the target air voids level.

Upon determining the required number of blows that will satisfy the air voids requirement, WMA-FA and HMA specimens were prepared according to the following procedure:

- Heat the aggregates and the asphalt binder to the mixing temperature for a minimum of two hours (mixing temperature for WMA-FA is lower than HMA by 30°F (15°C)).
- Mix the heated aggregates with the heated asphalt binder (or the foamed asphalt binder in the case of WMA-FA) using a mechanical mixer for about 3 minutes.
- Place the mixture in a flat pan and spread it over the whole area of the pan.
- Place the pan in an oven at 149°F (65°C) for a period of sixteen hours.
- After the sixteen hours curing period have passed, raise the temperature of the mixture to the compaction temperature and keep it for an additional two hours (compaction temperature for WMA-FA is lower than HMA by 30°F (15°C)).
- Compact the specimens at the required number of blow per face to achieve the desired air voids content. The specimens should have a 4 inch (100 mm) diameter and approximately 2.5 inch (63.5 mm) height.
- Determine the bulk specific gravity and air voids content of the compacted specimens.
- If the air voids content is within 7 ± 0.5 percent, complete the testing. However, if the air voids percentage is not within the specified range, new specimens should be prepared by repeating the above procedure and adjusting the required number of blows per face.

The compacted specimens were then grouped into two groups. Each group consisted of three specimens. The first group of specimens was wrapped with Saran-Wrap and stored at room temperature for testing in the dry condition. The second group of specimens, on the other hand, was wet conditioned prior to testing. Wet conditioning of the specimens involved partially saturating them in a water bath under a 2.9 psi (20 kPa) vacuum pressure for approximately two to three minutes. AASHTO T 283 specifies a saturation level between 70 to 80 percent. If the specimens were saturated to a degree above 80 percent, the specimens were discarded. However, if the specimens were saturated to a degree below 70 percent, the specimens were subjected to more vacuum time to further saturate them to the required degree of saturation.

The partially saturated specimens were then wrapped using Saran-Wrap and placed in heavy-duty leak proof plastic bags. 10 ml (0.6 in³) of water was added to each of the plastic bags. The plastic bags, containing the saturated specimens, were then placed in a freezer. The

specimens in this stage were subjected to a freezing cycle at a temperature of -0.4°F (-18°C) for a period of 16 hours. After that, the specimens were subjected to a thawing cycle in a water bath at 140°F (60°C) for a period of 24 hours. Finally, both groups of specimens (the dry conditioned and the wet conditioned) were placed in a water bath at 77°F (25°C) for two hours before testing.

The dry and wet conditioned specimens were then loaded diametrically using two bearing plates at a rate of 2 inches per minute (50.8 mm per minute). The maximum load required to break the specimen was recorded and used in determining the indirect tensile strength. The indirect tensile strength was determined according to the following equation:

$$S_t = \frac{2P}{\pi t D}$$

where,

S_t = indirect tensile strength (psi).

P = maximum load (lbs).

t = specimen thickness (in.).

D = specimen diameter (in.).

Finally, the Tensile Strength Ratio (TSR) was calculated as the ratio between the average indirect tensile strength of the wet conditioned specimens to average indirect tensile strength of the dry conditioned specimens. The TSR ratio is a measure of the resistance of the asphalt mixture to moisture damage. The higher is the TSR ratio the better is the resistance of the asphalt mixture to moisture damage.

5.3 Asphalt Pavement Analyzer (AASHTO TP 63-07)

The AASHTO TP 63-07 test method was utilized to characterize the susceptibility of WMA-FA and HMA mixtures to permanent deformation (or rutting). This test method was modified according to ODOT Supplement 1057. According to this supplement, the compaction of the specimens can be accomplished through the use of a rolling compactor or the Superpave Gyrotory Compactor (SGC). In this study, the SGC was selected to compact cylindrical specimens to the specified specimen height of 3 in. (75 mm). A target air voids level of 6 ± 1 percent was used in the preparation of the compacted specimens. Similar to AASHTO T 283 test method, a trial and error procedure was implemented to compact the specimens to the required air voids level while maintaining the required specimen dimensions (i.e. 6 in. (150 mm))

in diameter and 3 in. (75 mm) in height). Different trial weights ranging from 2830 to 3100 grams of asphalt mixtures were used to compact several trial specimens and the air voids content was determined for each specimen. A linear regression model was used to establish the relationship between the air voids content and the mixture weight and utilized to estimate the required mixture weight to achieve the target air voids level.

Upon determining the required mixture weight that will satisfy the air voids requirement, WMA-FA and HMA specimens were prepared according to the following procedure:

- Heat the aggregates and the asphalt binder to the mixing temperature for a minimum of two hours (mixing temperature of WMA-FA is lower than HMA by 30°F (15°C)).
- Mix the heated aggregates with the heated asphalt binder (or the foamed asphalt binder in the case of WMA-FA) using a mechanical mixer for about 3 minutes.
- Place the mixture in a flat pan and spread the mixture over the whole area of the pan.
- Place the pan in an oven for two hours at the compaction temperature (compaction temperature of WMA-FA is lower than HMA by 30°F (15°C)).
- Program the SGC to stop compacting upon reaching a specimen height of 3 in. (75 mm).
- Place the asphalt mixture in the heated SGC mold and compact the specimen.
- Determine the bulk specific gravity and air voids content of the compacted specimen.
- If the air voids content of the compacted specimen is within 6 ± 1 percent, complete the testing. However, if the air voids content is not within the specified range, a new specimen should be prepared by adjusting the weight of the asphalt mixture.

The Asphalt Pavement Analyzer (APA), shown in Figure 5.1, was used to evaluate the rutting susceptibility of the compacted specimens. Two gyratory specimens were used to assemble one APA sample. Three APA samples (i.e., six gyratory specimens) were tested for each material combination. As per ODOT Supplement 1057, the APA samples were preheated to the test temperature of 120°F (49°C) for a minimum of 12 hours prior to testing. Upon testing, the APA samples were subjected to repeated wheel loading of 115 lbf (511.5 N) using a hose pressure of 100 psi (690 kPa); (Figure 5.2).

Rut depth measurements were recorded at 5, 500, 1000, and 8000 cycles. For each APA sample, a total of four rut depth readings were used to calculate the average rut depth value within the specimen. The total permanent deformation (rutting) within the sample was calculated as the difference between the rut depth readings at the 8000th cycle and the 5th cycle.



Figure 5.1: Asphalt Pavement Analyzer (APA).



Figure 5.2: Repeated Wheel Loading in the APA Device.

5.4 Dynamic Modulus (AASHTO TP 62-07)

The AASHTO TP 62-07 test method was implemented to measure the dynamic modulus of compacted WMA-FA and HMA specimens. The dynamic modulus is a fundamental material property commonly used to describe the mechanical behavior of viscoelastic materials such as asphalt mixtures. It relates stresses to strains induced under different loading rates and temperature conditions. In recent years, the dynamic modulus has been incorporated in the Mechanistic Empirical Pavement Design Guide (MEPDG) to describe the response of the asphaltic layers, and to subsequently predict the performance of asphalt pavements. Asphalt mixtures with higher dynamic moduli result in less permanent deformation (or rutting), as predicted using the MEPDG.

The dynamic modulus test was conducted on specimens cored from gyratory compacted mixtures. An air voids content of 7 ± 0.5 percent was targeted in the preparation of the compacted mixtures. Each mixture was compacted to a height of 6.7 in. (170 mm). The sample preparation procedure was similar to that utilized in the preparation of the APA samples. The main difference is that the dynamic modulus samples were subjected to short term aging at 275°F (135°C) for a period of 4 hours, during which the mixture was stirred every one hour. After the 4 hour curing period has passed, the temperature was raised to the compaction temperature and the mixture was heated for 30 minutes. A trial and error procedure similar to that utilized in the preparation of the APA samples was also followed in determining the weight of mixture required to achieve the target air voids level. After compaction, the compacted samples were cored and trimmed to obtain a 6 in. (150 mm) tall by 4 in. (100 mm) diameter specimens, as shown in Figure 5.3 and 5.4, respectively.

The diameter and the waviness of the top and bottom edges of the extracted specimens were then measured to ensure that they are within the acceptable limits. AASHTO TP 62-07 requires measuring the diameter of the cored specimens to the nearest 1 mm at mid-height and third-points. The standard deviation of the three readings should not exceed 2.5 mm (0.1 in.). Furthermore, AASHTO TP 62-07 specifies a maximum acceptable waviness of ± 0.05 mm (0.002 in.) at the top and bottom edges of the sawed specimens. Figure 5.5 shows the straight-edge and the feel gage used to measure the waviness.



Figure 5.3: Vertical Coring Setup.



Figure 5.4: Trimming a Dynamic Modulus Specimen using a Diamond Saw.

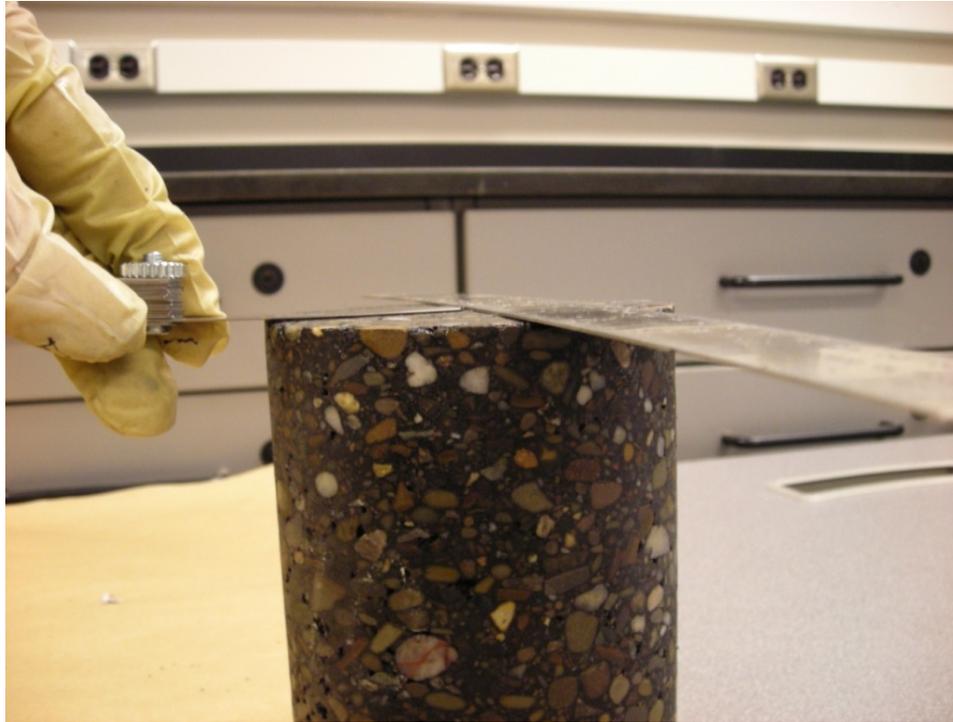


Figure 5.5: Checking the Waviness of the Top and Bottom Edges of a Dynamic Modulus Specimen Using a Straight Edge and a Feel Gage.

The bulk specific gravity and the air voids content of the cored specimens were then determined. The air voids content of the cored specimens was found to be 1.5% to 2.5% lower than the air voids content of the gyratory specimen.

Testing of the prepared specimens was accomplished through a servo-hydraulic Material Test System (MTS) Model 810 (Figure 5.6). The MTS testing system is operated using a digital controller called MTS TestStar II. It is capable of applying widely varying load levels, including those specified in AASHTO TP 62, at the desired frequencies. It is also equipped with an environmental chamber capable of controlling the testing temperature and a self-leveling loading platen that helps in alleviating any shear stresses that might arise due to imperfections caused by trimming the specimens' top and bottom edges. Individual measurements during the dynamic modulus test were obtained using an external load cell located underneath the bottom loading platen and a set of two external extensometers attached to the side of the specimen. The use of extensometers was preferred over using Linear Variable Differential Transducers (LVDTs) since the former provides higher accuracy and can be easily installed on the specimen.



Figure 5.6: MTS Model 810.

The dynamic modulus test was conducted at four different temperatures (40, 70, 100, and 130°F; 4.4, 21.1, 37.8, 54.4°C). Testing was conducted from the lowest to the highest temperature. AASHTO TP 62 also requires conducting the test at 14°F (-10°C); however, this temperature was not used since the environmental chamber was not capable of maintaining this temperature. Table 5.1 shows the required temperature-conditioning time before testing the specimens. At each testing temperature, six frequencies were applied (25, 10, 5, 1, 0.5, and 0.1 Hz), starting with the highest frequency. A rest period of 2 minutes was used between successive frequencies. The applied load level at each frequency was determined as the load that will result in 50 to 150 micro strain. At the end of testing, the specimen was discarded if excessive deformation greater than 1500 micro strain was accumulated.

Table 5.1: Temperature Equilibrium Time in the AASHTO TP 62 Test.

Testing Temperature (°F)	Time from Room Temperature (hrs)	Time from Previous Test Temperature (hrs)
40	Overnight	4 hrs or overnight
70	1	3
100	2	2
130	3	1

Figure 5.7 presents a sample of the applied stress and resulting strain curves versus time at 1 Hz. As can be seen in this figure, a seating load approximately equal to 5% of the magnitude of the applied dynamic load was used. This seating load ensures that the specimen is in full contact with the loading platens during the test. Figure 5.7 also shows that the strain cycles are accumulating some permanent strain with time.

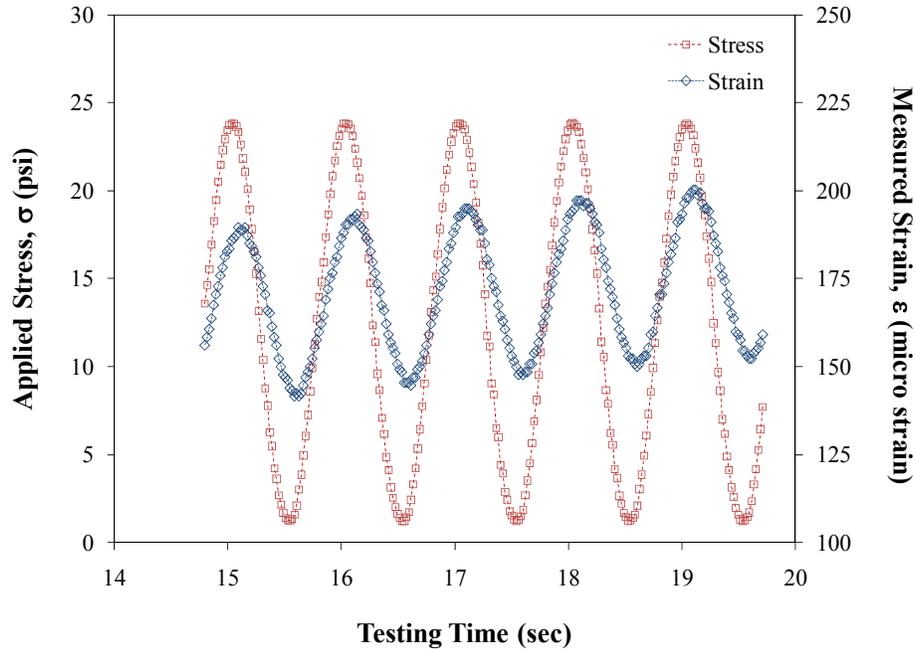


Figure 5.7: Sample of Applied Stress and Measured Strain at 1 Hz.

The dynamic modulus can be calculated from figures such as the one shown in Figure 5.7 by dividing the applied stress level (maximum minus minimum stress) by the recoverable strain level (maximum minus minimum strain). In this study, the maximum and minimum stress and strain values were determined by dividing the stress and strain curves into individual cycles and fitting quadratic equations to the peak and valley portions of these cycles. The quadratic equations were then derived with respect to time and the derivative was equated to zero in order to determine the maximum and minimum stress or strain values. Figure 5.8 illustrates the data analysis procedure using one stress-strain cycle. Due to the large amount of data involved in the analysis, an Excel macro was developed for this purpose.

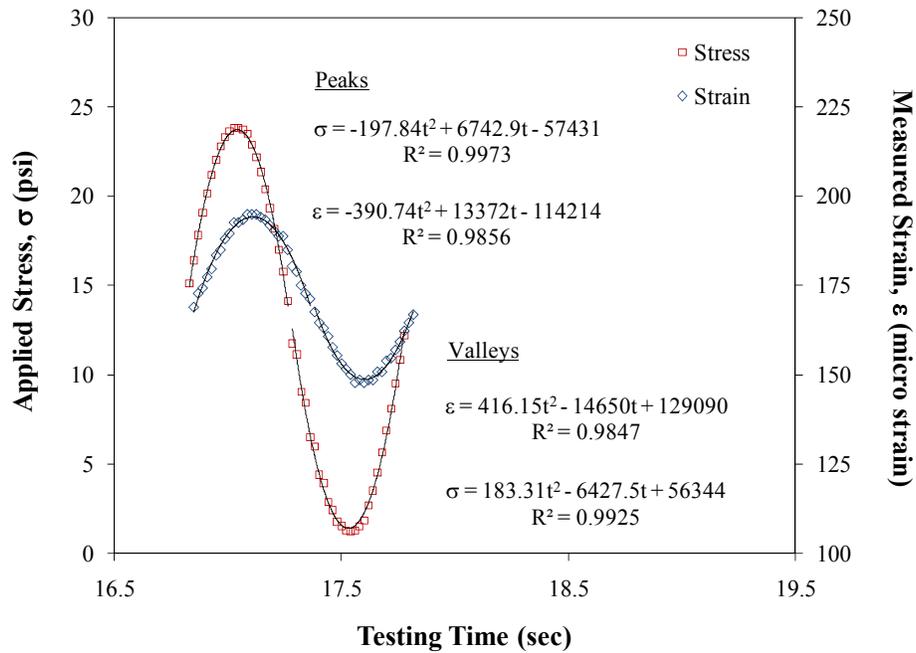


Figure 5.8: Determination of Maximum and Minimum Stress and Strain Values.

The procedure discussed above was used to obtain the dynamic modulus at various testing temperatures and loading frequencies. The dynamic modulus test results can be combined into a single master curve at a reference temperature to define the constitutive behavior (stress-strain response) of the asphalt mixture over a wider range of frequencies. The establishment of the master curves for the various types of asphalt mixtures will be discussed in Chapter 7.

CHAPTER 6

HANDLING CHARACTERISTICS OF WMA-FA MIXTURES

6.1 Introduction

As discussed in the previous chapter, three performance tests were utilized to evaluate the performance of WMA-FA and control HMA mixtures with regard to moisture susceptibility and permanent deformation. These tests included the AASHTO T 283 test, the Asphalt Pavement Analyzer (APA) test, and the Dynamic Modulus (E^*) test. Conducting these tests allowed for comparing the WMA-FA and HMA mixtures in terms of aggregate coating, asphalt binder absorption, workability, and compactability. This chapter documents several observations made during the preparation of both mixtures about these handling and mixing characteristics.

6.2 Aggregate Coating

The WMA-FA and HMA mixtures were prepared by heating the aggregates and the asphalt binder to the required mixing temperature for a minimum of two hours, followed by mixing the heated aggregates with the heated asphalt binder in the case of HMA and the heated aggregates with the foamed asphalt binder in the case of WMA-FA for about 3 minutes in a mechanical mixer. The mixing temperature of the WMA-FA was 30°F (15°C) lower than that of the HMA. At the end of the 3 minute mixing period, it was observed that the aggregates were fully coated with a thin film of asphalt for both WMA-FA and HMA mixtures even though lower mixing temperature was used for the WMA-FA mixtures. This indicates that the reduction in mixing temperature did not affect the coating of the WMA-FA mixtures and that the foaming process has successfully reduced the viscosity of the asphalt binder.

6.3 Asphalt Binder Absorption

The asphalt binder absorption by weight of aggregate in both WMA-FA and HMA mixes was calculated from the bulk and effective specific gravities of the aggregates. The effective specific gravity of the aggregate was calculated from the Rice specific gravity of the loose HMA or WMA-FA mix at the optimum asphalt binder content. The following equations were used in the calculation of the effective specific gravity of the aggregate and the asphalt binder absorption, respectively:

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

$$P_{ba} = 100 \frac{G_{se} - G_{sb}}{G_{sb} G_{se}} G_b$$

where,

G_{se} = effective specific gravity of the aggregate.

P_{ba} = asphalt binder absorption by weight of aggregate.

P_b = asphalt content (represented as a fraction).

G_{mm} = Rice specific gravity of the asphalt mixture.

G_b = specific gravity of the asphalt binder.

G_{sb} = bulk specific gravity of the aggregate.

Table 6.1 presents the measured Rice specific gravity values of the HMA and WMA-FA mixtures at the optimum asphalt binder content. The calculated effective specific gravity of aggregates and asphalt binder absorption are shown in Tables 6.2 and 6.3, respectively. The results in Table 6.3 demonstrate that the asphalt binder absorption in the WMA-FA mixtures was slightly lower than that in the HMA mixtures, which explains in part the reduction in the Rice specific gravity of the WMA-FA. Although no coating problems have been observed, the reduction in the amount of asphalt binder absorbed might result in less bonding between the aggregates and the asphalt binder. As a consequence, WMA-FA mixtures might be more prone to moisture induced damage when compared to traditional HMA mixtures. Other factors that might have contributed to the reduction in the Rice specific gravity include the presence of entrapped air bubbles within the foamed asphalt binder. These entrapped air bubbles would increase the volume of the loose mix used for calculating the Rice specific gravity and eventually result in reducing it.

By comparing the asphalt binder absorption in the WMA-FA mixtures to that in the HMA mixtures, it can be noticed that the reduction in the asphalt binder absorption was more pronounced for the unmodified asphalt binder than for the modified asphalt binder. The unmodified asphalt binder was easier to foam than the modified asphalt binder, resulting in more foaming and subsequently less absorption by the aggregates.

Table 6.1: Rice Specific Gravity of HMA and WMA-FA.

Mix Type	Aggregate	Asphalt Binder	Rice Specific Gravity
HMA	Gravel	PG 64-22	2.405
		PG 70-22M	2.407
	Limestone	PG 64-22	2.472
		PG 70-22M	2.466
WMA-FA	Gravel	PG 64-22	2.396
		PG 70-22M	2.401
	Limestone	PG 64-22	2.461
		PG 70-22M	2.459

Table 6.2: Effective Specific Gravity of Aggregate in HMA and WMA-FA.

Mix Type	Aggregate	Asphalt Binder	Effective Specific Gravity of Aggregate
HMA	Gravel	PG 64-22	2.627
		PG 70-22M	2.630
	Limestone	PG 64-22	2.732
		PG 70-22M	2.729
WMA-FA	Gravel	PG 64-22	2.615
		PG 70-22M	2.623
	Limestone	PG 64-22	2.717
		PG 70-22M	2.720

Table 6.3: Asphalt Binder and Water Absorption of Aggregates in HMA and WMA-FA.

Mix Type	Aggregate	Asphalt Binder	Asphalt Binder Absorption (%)
HMA	Gravel	PG 64-22	0.67
		PG 70-22M	0.71
	Limestone	PG 64-22	0.73
		PG 70-22M	0.69
WMA-FA	Gravel	PG 64-22	0.50
		PG 70-22M	0.61
	Limestone	PG 64-22	0.53
		PG 70-22M	0.57

¹Asphalt binder and water absorption by weight of aggregate.

6.4 Workability

During the preparation of both HMA and WMA-FA mixtures, it was observed that the WMA-FA mixtures were easier to handle than the HMA mixtures. This was obvious during the preparation of the dynamic modulus specimens, which required aging for four hours at 275°F (135°C) along with stirring every one hour. In doing so, it was noticed that the WMA-FA mixtures required less effort to stir than HMA mixtures. The improved workability of WMA-FA mixtures is probably caused by the reduction in the asphalt binder's viscosity through foaming even though these mixtures are prepared using temperatures 30°F (15°C) lower than HMA.

6.5 Compactability

As previously discussed in Chapter 5, a trial and error procedure was implemented to determine the required compaction effort to achieve a predefined target air voids level within the compacted specimen. Linear regression models were used to establish the relationship between the compaction effort and the resulting air voids content. These models were then used to estimate the required compaction effort that will achieve the target air voids content.

Table 6.4 shows the number of blows per face in the Marshall Compaction Hammer that were needed to achieve $7 \pm 1\%$ air voids in the AASHTO T 283 specimens and the number of gyrations in the Superpave Gyrotory Compactor that were needed to achieve $6 \pm 1\%$ air voids in the APA specimens. Similar results were obtained for the Dynamic Modulus specimens. Therefore, they were not included in this table.

Table 6.4: Required Compaction Effort to Achieve Target Air Voids.

Mix Type	Aggregate	Asphalt Binder	Blows Per Face ¹	No. of Gyration ²
HMA	Gravel	PG 64-22	18	19
		PG 70-22M	20	12
	Limestone	PG 64-22	18	11
		PG 70-22M	18	8
WMA-FA	Gravel	PG 64-22	13	11
		PG 70-22M	15	9
	Limestone	PG 64-22	10	4
		PG 70-22M	9	4

¹ Number of blows per face required to achieve $7 \pm 1\%$ air voids in the AASHTO T 283 specimens.

² Number of gyrations required to achieve $6 \pm 1\%$ air voids in the APA specimens.

As can be seen in Table 6.4, the compaction effort needed to compact specimens prepared using natural gravel was slightly higher than that needed to compact specimens prepared using crushed limestone. It is believed that this was mainly due to the use of finer aggregate gradation in the case of limestone as compared to gravel.

The compaction data shown in Table 6.4 also reveals that the required compaction effort to compact WMA-FA specimens was significantly lower (about 30 to 50 percent) than that needed to compact HMA specimens, which suggests that the WMA-FA mixtures are easier to compact than the HMA mixtures. This was the case for specimens prepared using natural gravel and crushed limestone. Therefore, it is believed that the use of foamed asphalt binders helps in improving the compactability of WMA-FA mixtures for both aggregate types.

In order to gain a better understanding of the improved compactability of WMA-FA mixtures, it is also important to study the Rice specific gravity values of both HMA and WMA-FA mixtures shown in Table 6.1. As can be seen in this table, the Rice specific gravity values of WMA-FA mixtures were slightly lower than those of the HMA mixtures. As explained earlier, this slight reduction in the Rice specific gravity might be due to two factors. The first factor is the presence of entrapped air bubbles within the foamed asphalt binder even after mixing with the aggregates. The second factor is the reduction in asphalt binder absorption. While the first factor is believed to be dominant since it is easier to compact air than aggregates or asphalt, the second factor also helps in improving the compactability since the effective asphalt binder, which serves as a lubricant, is higher in the case of WMA-FA than HMA. In summary, a slight reduction in the Rice specific gravity has resulted in a significant reduction in the compaction effort required to compact WMA-FA specimens. Additional work is needed, however, to determine whether the use of such compaction effort is sufficient to ensure satisfactory long term performance for WMA-FA mixtures or not.

CHAPTER 7

TEST RESULTS AND DATA ANALYSIS

7.1 Introduction

Three performance tests were used in this study to evaluate the performance of WMA-FA and HMA mixtures with regard to moisture susceptibility and permanent deformation. These tests were the AASHTO T 283 test, the APA test, and the Dynamic Modulus test. A detailed discussion about these tests was provided in Chapter 5. This chapter presents the experimental test results obtained from these tests. In addition, it provides the outcome of the Analysis of Variance (ANOVA) that was conducted using the Statistical Analysis Software (SAS) to examine the significance of the mix type, aggregate type, and binder type as well as their interaction on the performance test parameters.

7.2 AASHTO T 283 Test Results

The AASHTO T 283 test was used to evaluate the moisture susceptibility of WMA-FA and HMA mixtures. The test was conducted on dry and wet conditioned specimens measuring 4 inches (100 mm) in diameter and 2.5 inches (63.5 mm) in height. The specimens were loaded until failure at a rate of 2 inches per minute (50.8 mm per minute). Two types of data were obtained from this test. The first is the indirect tensile strength (ITS) of the dry and wet conditioned specimens. The second is the tensile strength ratio (TSR), calculated by dividing the average ITS values of the wet conditioned specimens by the average ITS values of the dry conditioned specimens. The ITS is a measure of the strength and durability of the asphalt mixture, whereas the TSR ratio is a measure of its resistance to damage from freezing and thawing.

Figures 7.1 and 7.2 present the dry ITS values for both WMA-FA and HMA mixtures prepared using natural gravel and crushed limestone, respectively. As can be seen from these figures, the WMA-FA mixtures exhibited lower ITS values than the HMA mixtures, except for mixtures prepared using natural gravel and PG 70-22M. This can be attributed to the foaming properties of the two asphalt binders, in that the PG 64-22 was easier to foam than the PG 70-22M.

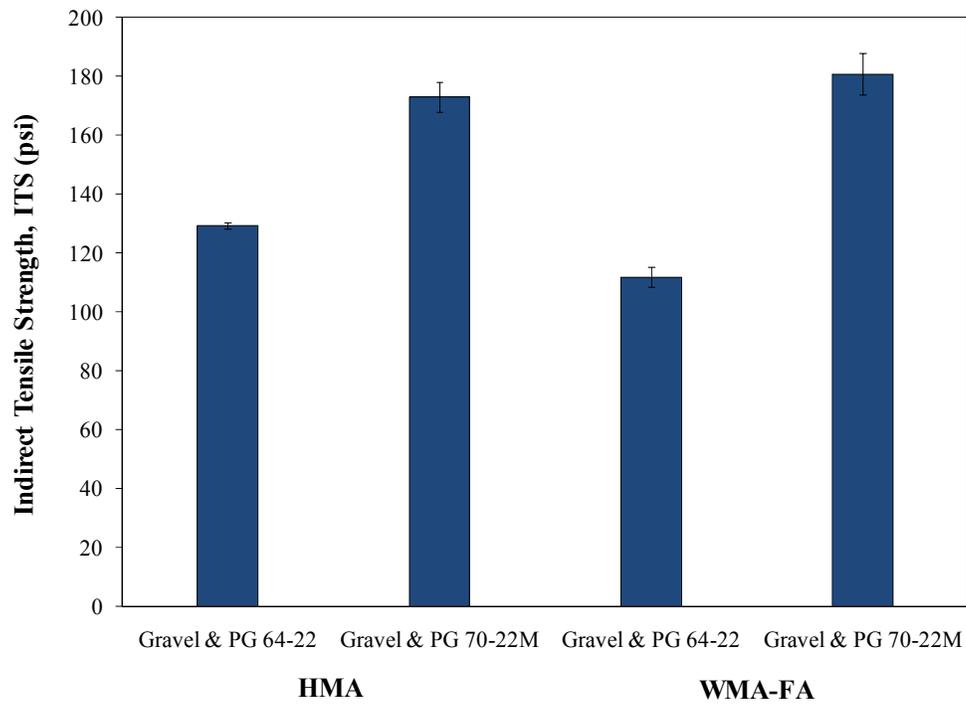


Figure 7.1: Dry ITS of HMA and WMA-FA Mixtures Containing Natural Gravel.

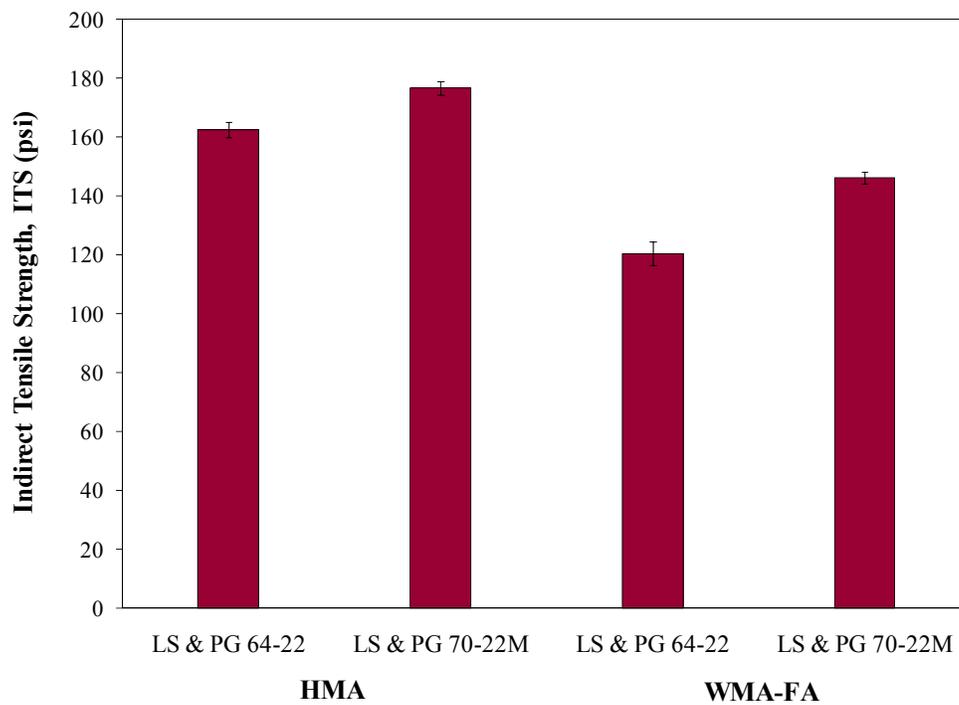


Figure 7.2: Dry ITS of HMA and WMA-FA Mixtures Containing Crushed Limestone.

It can also be seen from Figures 7.1 and 7.2 that mixtures prepared using crushed limestone exhibited slightly higher ITS values than those prepared using natural gravel. This is probably due to the greater interlock within the crushed limestone aggregate structure. Furthermore, it can be noticed that mixtures prepared using PG 70-22M exhibited higher ITS values than those prepared using PG 64-22. This is expected since polymer modified asphalt binders can withstand greater loads until failure than unmodified asphalt binders.

A multi-factor ANOVA analysis was conducted to evaluate the effects of the mix type, aggregate type, and binder type along with their interaction on the dry ITS values. Table 7.1 shows the results of the ANOVA analysis. As can be noticed in this table, the effect of binder type and mix type and their interaction was significant at 95% confidence level ($Pr < 0.05$). Table 7.1 also shows that the binder type was the most significant factor affecting the ITS values, as indicated by the F-value.

Table 7.1: Multi-Factor ANOVA Results for Dry ITS Values.

Effect	F-Value	Probability (Pr)
Mix	85.86	<.0001
Aggregate	1.50	0.2382
Binder	294.68	<.0001
Mix * Aggregate	50.07	<.0001
Mix * Binder	17.32	0.0007
Aggregate * Binder	67.11	<.0001
Mix * Aggregate * Binder	2.35	0.1448

Figures 7.3 and 7.4 present the TSR ratios for both WMA-FA and HMA mixtures prepared using natural gravel and crushed limestone, respectively. As can be seen from these figures, the WMA-FA mixtures exhibited slightly lower TSR ratios than the HMA mixtures. However, both WMA-FA and HMA mixtures met the minimum TSR requirement specified in ODOT C&MS for medium traffic ($TSR \geq 0.7$). These figures also show that the TSR values for mixtures containing crushed limestone were lower than those obtained for mixtures containing natural gravel. This might be attributed to the finer aggregate gradation and the higher optimum asphalt binder contents used in the case of limestone.

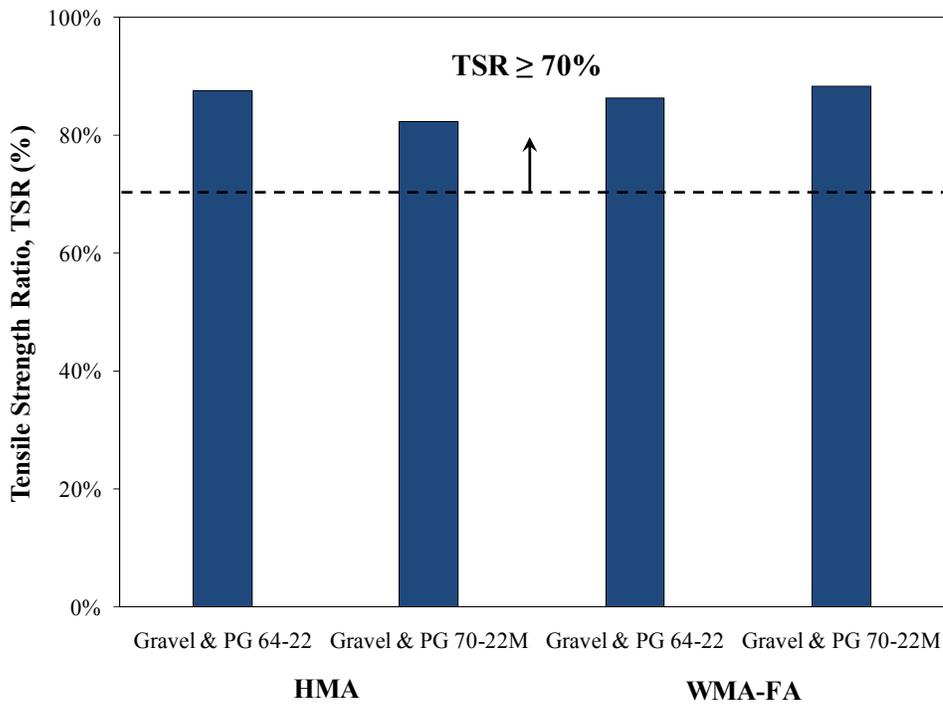


Figure 7.3: Tensile Strength Ratios (TSR) for Mixtures Containing Natural Gravel.

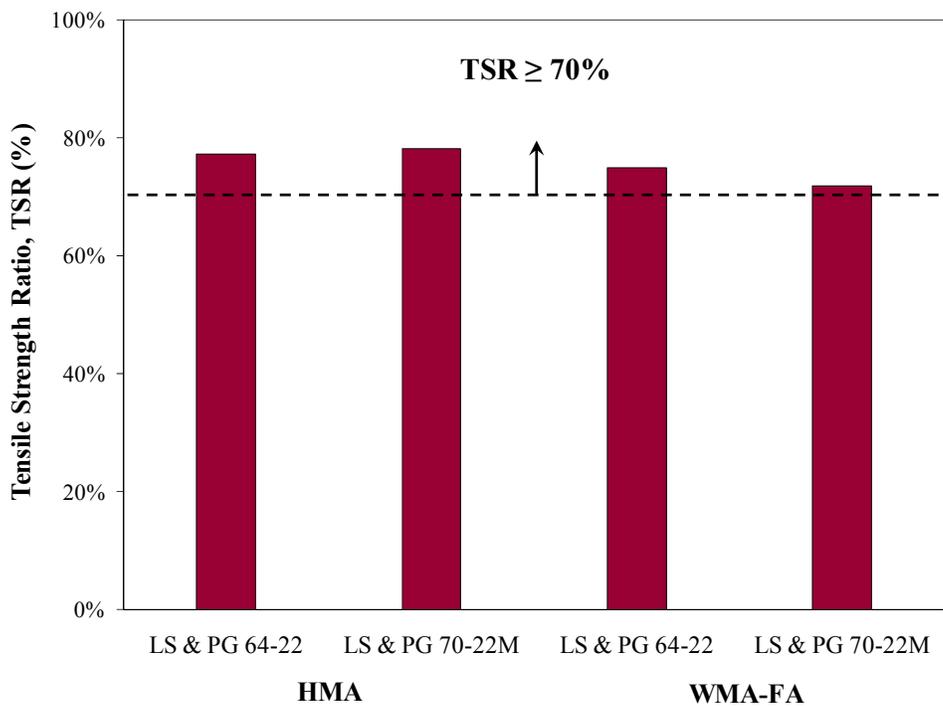


Figure 7.4: Tensile Strength Ratios (TSR) for Mixtures Containing Crushed Limestone.

Table 7.2 presents the results of the multi-factor ANOVA analysis of TSR data. It can be noticed from this table that the aggregate type was the most significant factor affecting the TSR ratio at a confidence level of 95%. In addition, the effect of the interaction between the aggregate type and the mix type was statistically significant. This suggests that although foaming did not, in general, affect the TSR value, its effect on the TSR value changed when using natural gravel rather than crushed limestone. This indicates that the performance of WMA-FA mixtures with regard to moisture induced damage is affected by the selection of the aggregate type.

Table 7.2: Multi-Factor ANOVA Results for TSR Ratios.

Effect	F-Value	Probability (Pr)
Mix	0.31	0.5849
Aggregate	49.81	<.0001
Binder	0.78	0.3915
Mix * Aggregate	4.92	0.0413
Mix * Binder	0.27	0.6074
Aggregate * Binder	0.01	0.9300
Mix * Aggregate * Binder	3.51	0.0794

7.3 APA Test Results

The APA test was used to evaluate the rutting potential of WMA-FA and HMA mixtures. Figures 7.5 and 7.6 present the rut depth data obtained using the APA test for both WMA-FA and HMA mixtures prepared using natural gravel and crushed limestone, respectively. As can be seen from these figures, the WMA-FA mixtures were more susceptible to rutting than the HMA mixtures. This can be attributed to the softening of the asphalt binders due to foaming, lower asphalt binder absorption, and reduced binder aging due to the use of lower production temperatures in the case of the WMA-FA mixtures.

Figures 7.5 and 7.6 also show that mixtures containing crushed limestone had lower rut depths than those containing natural gravel. Two factors might have contributed to such results. The first is the finer aggregate gradation used in the case of crushed limestone, which results in denser mixes. The second is the inherent ability of angular aggregates such as crushed limestone

to provide higher internal friction and better aggregate interlock in comparison to rounded aggregates such as natural gravel.

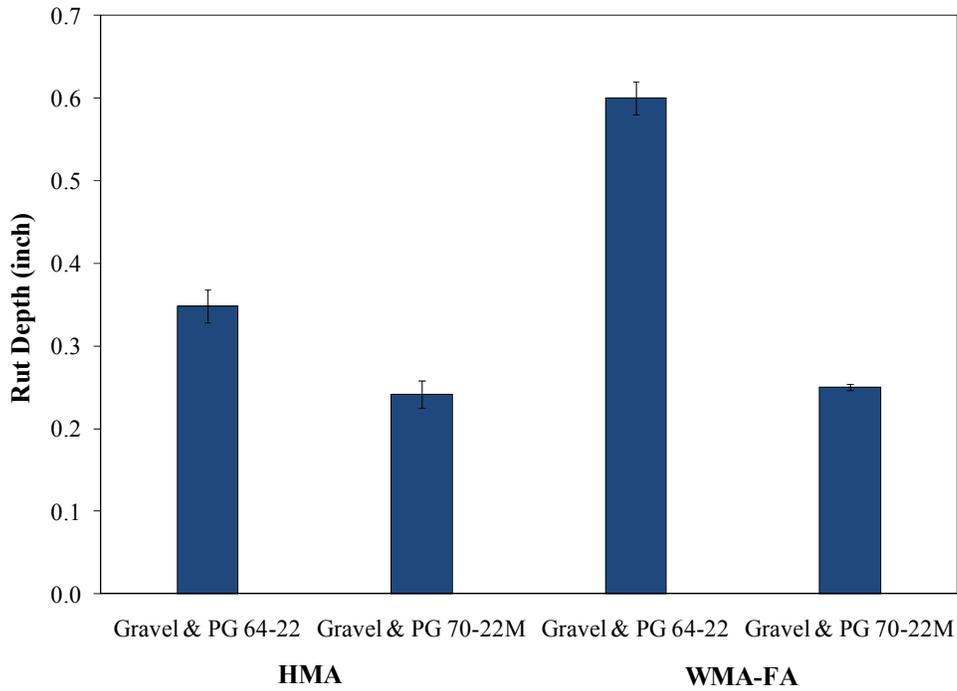


Figure 7.5: Rut Depth Results for Mixtures Containing Natural Gravel.

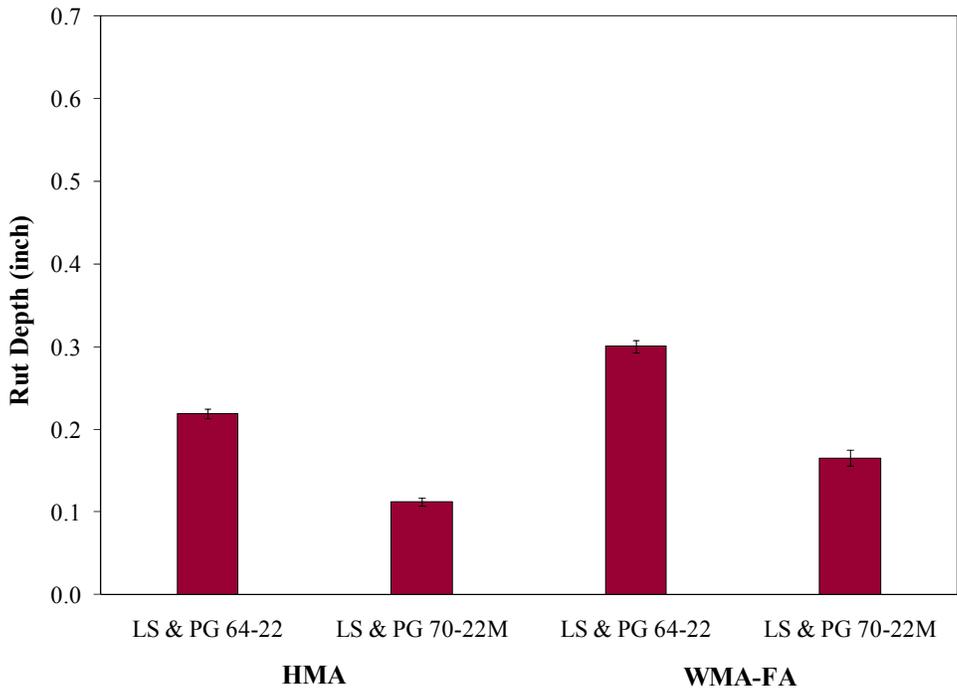


Figure 7.6: Rut Depth Results for Mixtures Containing Crushed Limestone.

It can also be seen from Figures 7.5 and 7.6 that mixtures containing PG 64-22 are more susceptible to rutting than those containing PG 70-22M, which is expected since the latter is a polymer modified asphalt binder with a higher PG grade.

By studying the rut depth values obtained for both WMA-FA and HMA mixtures, it can be noticed that the highest rut depth was obtained for WMA-FA mixtures prepared using natural gravel and PG 64-22, which had an average rut depth value of about 0.6 inch (15 mm). The rest of the mixtures had a rut depth value less than 0.35 inch (9 mm). Therefore, ODOT is encouraged to examine the performance of recently constructed projects in Ohio using foamed asphalt binder and this material combination with respect to permanent deformation in order to determine whether this observation is consistent with field performance data or not.

Table 7.3 presents the results of the multi-factor ANOVA analysis of rut depth data. It can be seen from this table that the mix type, aggregate type, and binder type as well as their interaction had a significant effect on the rut depth results. However, the effect of the aggregate and binder types was more significant than the mix type, as indicated by the F-value. This result suggests that using appropriate aggregate and binder types can help in overcoming any adverse effects that WMA-FA have on the mixture performance.

Table 7.3: Multi-Factor ANOVA Results for Rut Depth Measurements.

Effect	F-Value	Probability (Pr)
Mix	187.17	<.0001
Aggregate	495.67	<.0001
Binder	582.88	<.0001
Mix * Aggregate	18.91	0.0005
Mix * Binder	88.21	<.0001
Aggregate * Binder	55.17	<.0001
Mix * Aggregate * Binder	55.54	<.0001

7.4 Dynamic Modulus Test Results

The dynamic modulus test was conducted at 40, 70, 100, and 130°F (4.4, 21.1, 37.8, 54.4°C) over a range of frequencies 0.1, 0.5, 1, 5, 10, and 25 Hz. Figure 7.7 presents an example of the dynamic modulus test results obtained for WMA-FA mixtures prepared using natural

gravel and PG 64-22. As can be seen from this figure, the dynamic modulus of the asphalt mixture increased with the increase in testing frequency and decreased with the increase in testing temperature.

Assuming that the time-temperature superposition principle is valid (i.e., the material is thermo-rheologically simple), the dynamic modulus test results can be shifted horizontally to produce a master curve at a reference temperature. In doing so, the dynamic modulus is shifted to the left for temperatures higher than the reference temperature and shifted to the right for temperatures lower than the reference temperature. The ratio between the reduced (shifted) frequency and the original testing frequency is called the temperature shift factor, $a(T)$. Figure 7.8 presents the temperature shift factors obtained from shifting the dynamic modulus data presented in Figure 7.7 to a reference temperature of 70°F (21°C). As can be seen from this figure, a temperature shift factor greater than one was obtained for temperatures greater than the reference temperature and a temperature shift factor less than one was obtained for temperatures lower than the reference temperature.

Figure 7.9 presents the dynamic modulus master curve obtained from shifting the dynamic modulus data presented in Figure 7.7 to a reference temperature of 70°F (21°C). In order to eliminate any subjectivity in developing this master curve, a sigmoidal model was used to describe the relationship between the shifted dynamic moduli and the reduced frequency. The sigmoidal model was represented using the following equation (Pellinen and Witczak 2002):

$$\log(E^*) = \delta + \frac{\alpha}{1 + e^{\beta + \gamma(\log(1/f_r))}}$$

where,

E^* = dynamic modulus (psi).

f_r = reduced frequency (Hz).

α , β , δ , and γ = fitting parameters describing the shape of the sigmoidal model.

The best fit model was obtained by minimizing the least squared error between the measured and predicted dynamic modulus, which was accomplished through the Solver option in Excel by changing the temperature shift factors as well as the sigmoidal model parameters.

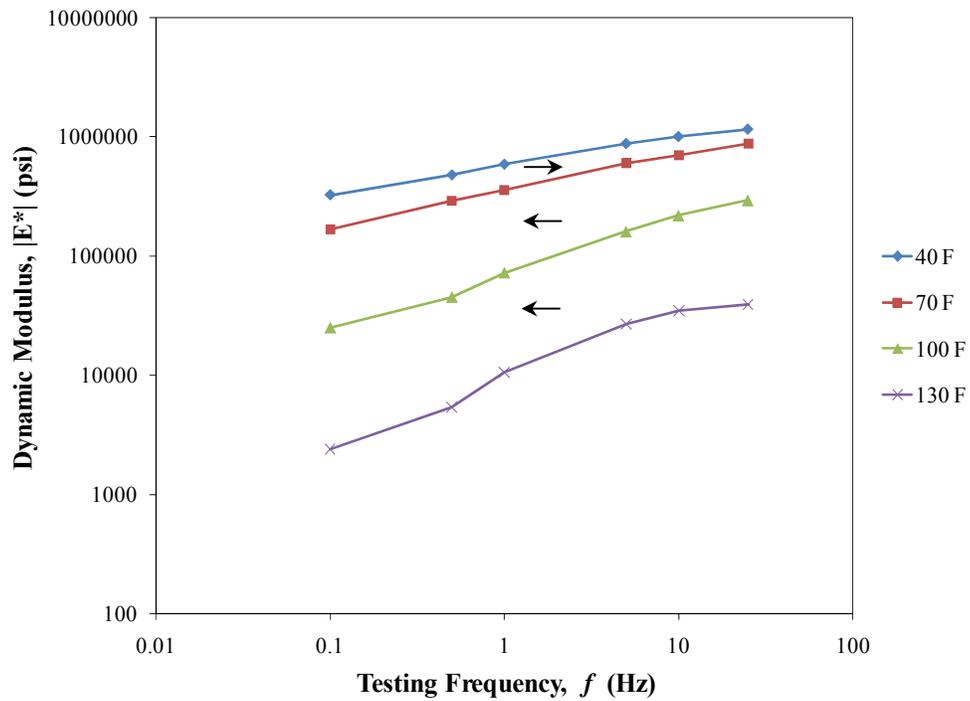


Figure 7.7: Example Dynamic Modulus Test Results for WMA-FA Mixtures Prepared Using Natural Gravel and PG 64-22.

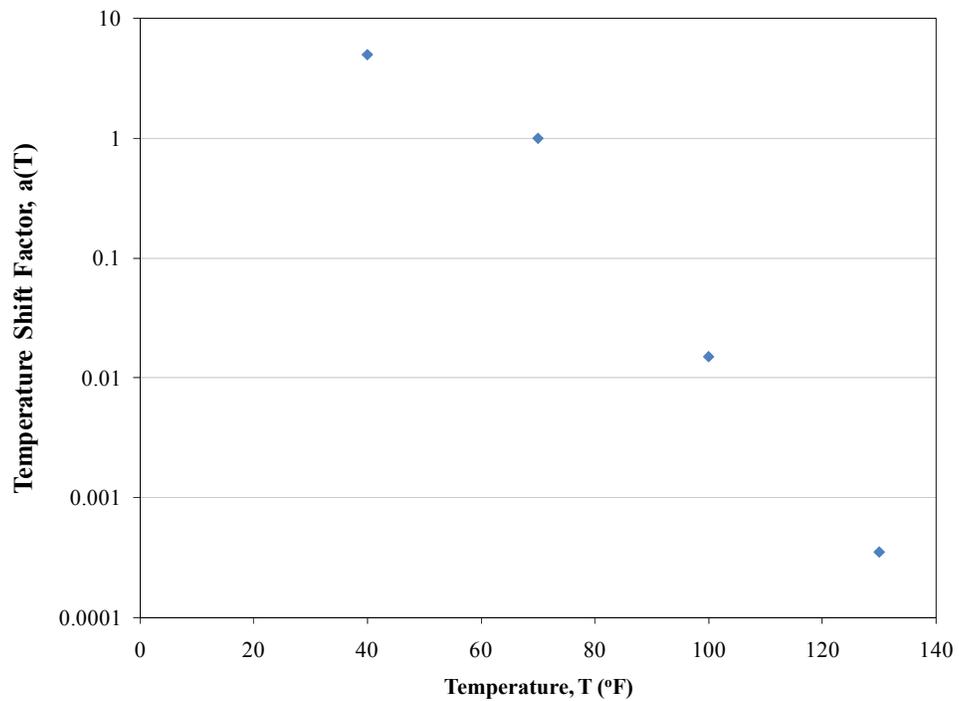


Figure 7.8: Example Temperature Shift Factors at a Reference Temperature of 70°F for WMA-FA Mixtures Prepared Using Natural Gravel and PG 64-22.

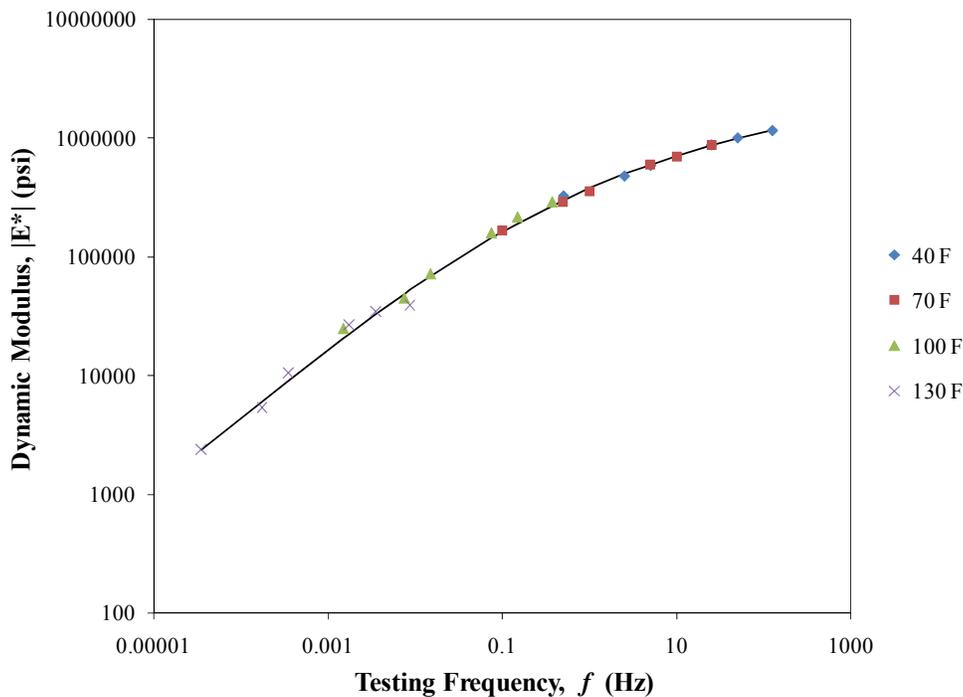


Figure 7.9: Example Master Curve at a Reference Temperature of 70°F for WMA-FA Mixtures Prepared Using Natural Gravel and PG 64-22.

Figures 7.10 and 7.11 present the temperature shift factors for both WMA-FA and HMA mixtures prepared using natural gravel and crushed limestone, respectively. It can be seen from these figures that the temperature shift factors for WMA-FA mixtures were similar to those of HMA mixtures.

Figures 7.12 and 7.13 present the master curves for both WMA-FA and HMA mixtures prepared using natural gravel and crushed limestone, respectively. As can be seen from these figures, the dynamic modulus of the WMA-FA mixtures was very close to that obtained for HMA mixtures. These figures also show that the dynamic modulus was mainly affected by the aggregate type and to a less extent by the type of the asphalt binder. For instance, mixtures containing crushed limestone exhibited higher dynamic moduli than mixtures containing natural gravel. As for the effect of the binder type, slightly higher dynamic moduli were obtained for PG 70-22M than PG 64-22 in the case of natural gravel, whereas slightly higher dynamic moduli were obtained for PG 64-22 than PG 70-22M in the case of crushed limestone.

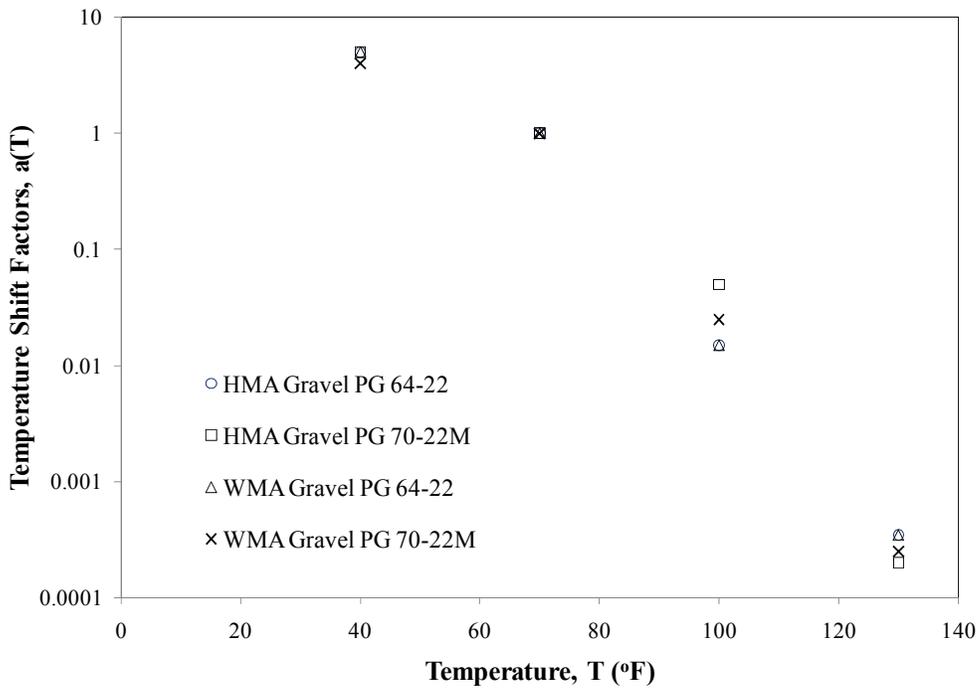


Figure 7.10: Temperature Shift Factors for Mixtures Containing Natural Gravel.

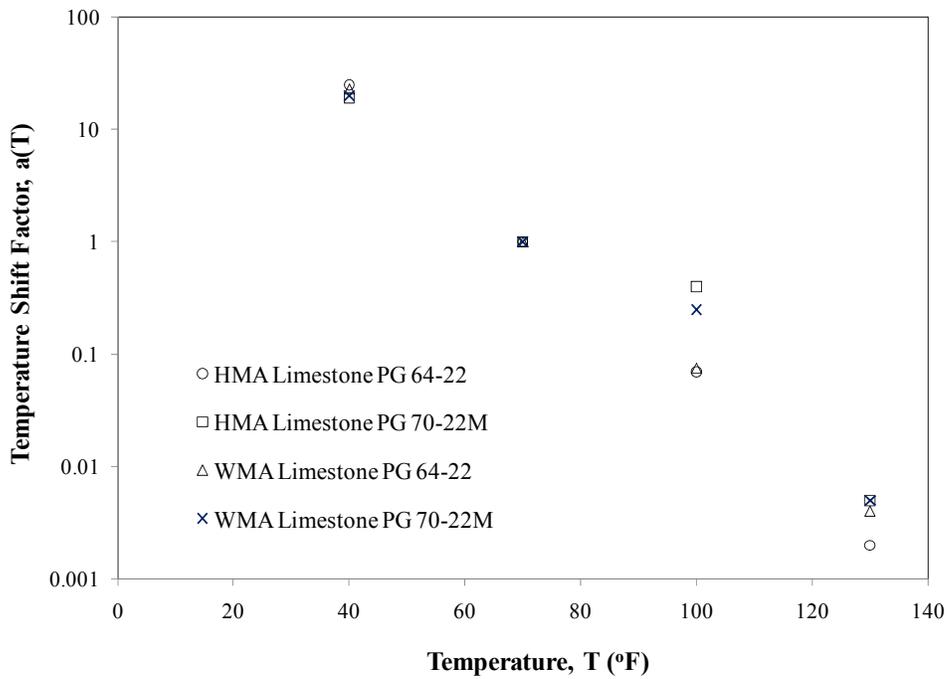


Figure 7.11: Temperature Shift Factors for Mixtures Containing Crushed Limestone.

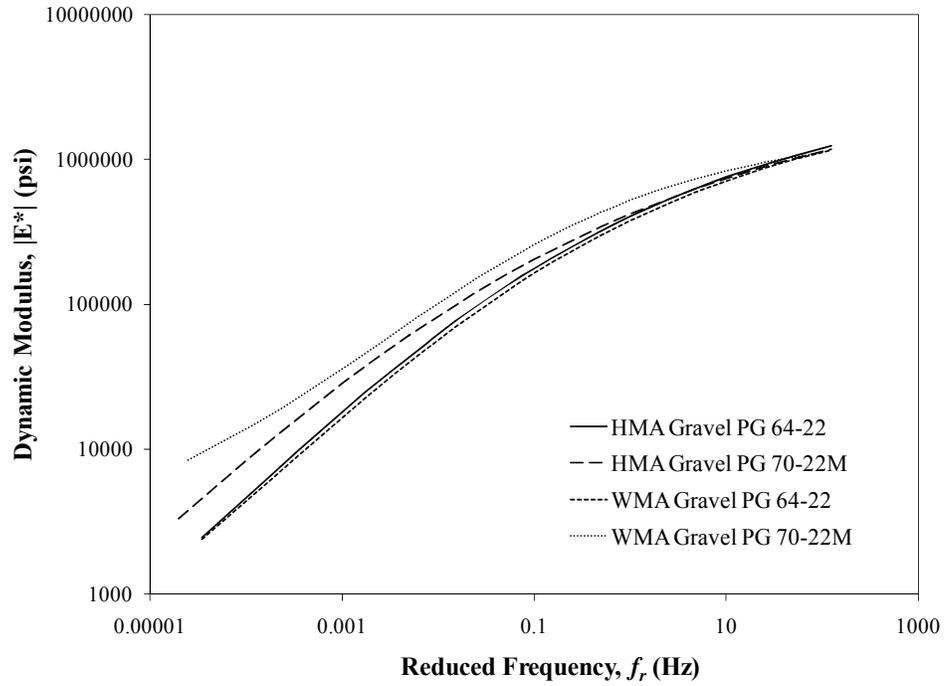


Figure 7.12: Dynamic Modulus Master Curves for Mixtures Containing Natural Gravel.

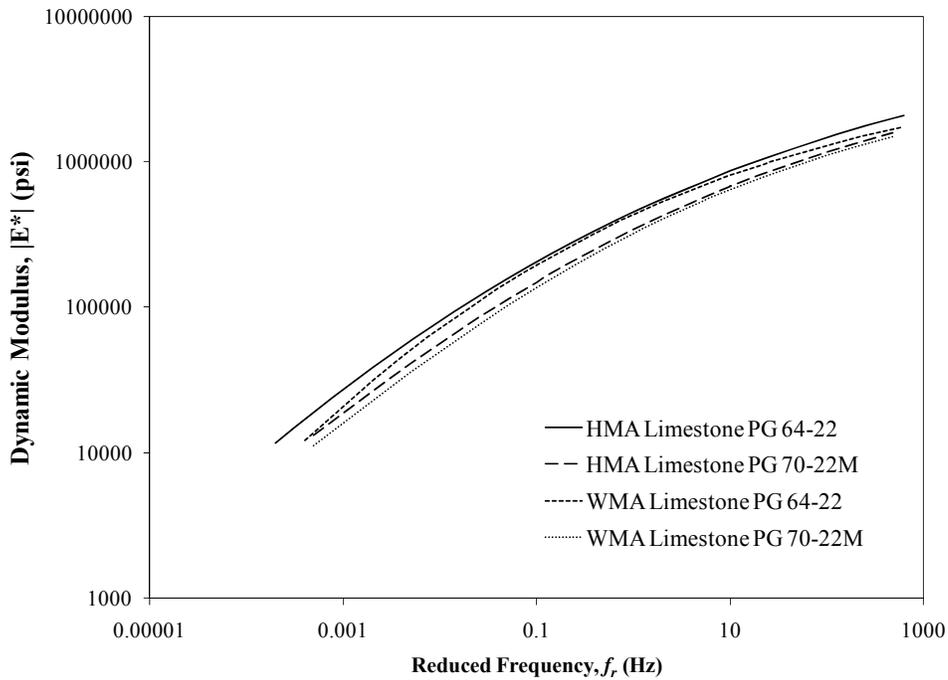


Figure 7.13: Dynamic Modulus Master Curves for Mixtures Containing Crushed Limestone.

In summary, the dynamic modulus test was found to be insensitive to the mix preparation procedures, resulting in similar dynamic moduli for both WMA-FA and HMA mixtures. This suggests that the performance of the WMA-FA mixtures is similar to that of the HMA mixtures with respect to permanent deformation. However, the APA test results have shown an increased rutting potential for WMA-FA mixtures than HMA mixtures. Therefore, it is believed that the dynamic modulus test is not a suitable test for prediction the rutting performance of asphalt mixtures.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary and Conclusions

This study presents a laboratory procedure to prepare WMA mixtures that utilize foamed asphalt binders. The performance of the WMA-FA mixtures was compared to conventional HMA mixtures with regard to moisture-induced damage and permanent deformation (or rutting). This study involved using two types of asphalt binders (PG 64-22 and PG 70-22M) and two types of aggregates (natural gravel and crushed limestone). The asphalt mixtures were prepared according to ODOT C&MS Item 441 Type 1 surface course subjected to medium traffic. Several combinations of the aggregate types and the asphalt binders were prepared and tested to evaluate their effect on the performance of the WMA-FA and HMA mixtures.

Based on the experimental test results and the subsequent statistical analyses findings, the following conclusions were made:

- General conclusions:
 - The unmodified and modified asphalt binders (PG 64-22 and PG 70-22M, respectively) were successfully foamed using a laboratory scale asphalt binder foaming device called WLB10, produced by Wirtgen, Inc.
 - As expected, the unmodified PG 64-22 asphalt binder had a slightly higher expansion ratio and thus was easier to foam than the modified PG 70-22M asphalt binder.
 - Aggregates in WMA-FA mixtures were fully coated after mixing in a mechanical mixer for 3 minutes even though the mixing temperature was 30°F (16.7°C) lower than that for HMA mixtures.
 - WMA-FA mixtures had slightly lower Rice specific gravities than HMA mixtures. This might have been caused by two factors. First, the presence of entrapped air bubbles within the foamed asphalt binder even after mixing. Second, a slight reduction in the amount of asphalt binder absorbed by the aggregates in the case of WMA-FA mixtures.
 - WMA-FA mixtures were found to be more workable and easily compacted in comparison to HMA mixtures even though the mixing and compaction temperatures were 30°F (16.7°C) lower than that for HMA mixtures. This is mainly attributed to the use of foamed asphalt binder.

- Moisture susceptibility:
 - Generally, WMA-FA mixtures had slightly lower TSR values than HMA mixtures. However, the difference was found to be statistically insignificant. In addition, the TSR values of both WMA-FA and HMA mixtures satisfied ODOT's minimum TSR requirement for medium traffic ($TSR \geq 70\%$).
 - Mixtures prepared using natural gravel had higher TSR values than those prepared using crushed limestone. This is probably due to using higher asphalt binder content and finer aggregate gradation in the case of mixtures containing crushed limestone.
 - The HMA mixtures exhibited higher ITS values than WMA-FA mixtures. Softening of the asphalt binder due to foaming and lower asphalt binder absorption might be the causes of such result.
 - Mixtures prepared using crushed limestone exhibited higher ITS values than those containing natural gravel. This is probably due to the greater interlock within the crushed limestone aggregate structure.
- Rutting:
 - WMA-FA mixtures exhibited higher rut depths in the APA test than the HMA mixtures. This may be attributed to the softening of the asphalt binders due to foaming, lower asphalt binder absorption, and reduced binder aging due to the use of lower production temperatures in the case of the WMA-FA mixtures.
 - Mixtures prepared using crushed limestone had lower rut depths in the APA test than those prepared using natural gravel. The greater interlock within the crushed limestone aggregates structure might be the cause of such result.
 - HMA and WMA-FA mixtures prepared using PG 70-22M were more resistant to rutting than those prepared using PG 64-22, which is expected since the former is a polymer modified asphalt binder with a higher PG grade.
 - All rut depth values obtained from the APA test were lower than 0.35 inch (9 mm) except for the WMA-FA mixtures prepared using natural gravel and PG 64-22, which had an average rut depth of 0.6 inch (15 mm). Therefore, ODOT is encouraged to examine the performance of recently constructed projects in Ohio using foamed asphalt binder and this material combination with respect to permanent deformation in order to determine whether this observation is consistent with field performance data or not.

- The dynamic modulus of the WMA-FA mixtures was very close to that of the HMA mixtures. This suggests that the performance of the WMA-FA mixtures is similar to that of the HMA mixtures with respect to permanent deformation. However, the APA test results have shown an increased rutting potential for WMA-FA mixtures than HMA mixtures. Therefore, it is believed that the dynamic modulus test is not a suitable test for prediction the rutting performance of asphalt mixtures.
- As expected, mixtures containing crushed limestone aggregates had higher dynamic modulus values than those containing natural gravel. The greater interlock within the limestone aggregate structure might be the cause of such result.
- Mixed results were obtained for the effect of the asphalt binder type on the dynamic modulus. Slightly higher dynamic moduli were obtained for PG 70-22M than PG 64-22 in the case of natural gravel, whereas slightly higher dynamic moduli were obtained for PG 64-22 than PG 70-22M in the case of crushed limestone.

8.2 Study Limitations

This study had a number of limitations:

- The use of only two types of aggregates and two types of asphalt binders.
- The use of only one aggregate gradation for the gravel mixtures and one aggregate gradation for the limestone mixtures.
- Mixtures were prepared using the Marshall mix design.
- Mixtures were prepared to withstand medium traffic.
- Designing the WMA-FA mixtures using the same optimum asphalt binder content obtained from the HMA mix design procedure.
- The foaming parameters (i.e. foaming water content, air pressure, water pressure, and foaming temperature) were not varied.
- The use of fully dried aggregates in preparing the WMA-FA mixtures.
- Producing WMA-FA mixtures at 30°F (16.7°C) lower mixing and compactions temperatures than traditional HMA mixtures, without any consideration for possible further reduction in these temperatures.

8.3 Recommendations for Further Study

It is recommended that future work expands the current study to include a wide range of aggregates obtained from different sources in Ohio and different asphalt binders. This study can also be expanded to evaluate the effect of the foaming parameters on the performance of WMA-FA mixtures. Furthermore, the study can be expanded to account for the effect of aggregate drying on the performance of WMA-FA mixtures.

It is also recommended that future work would take into consideration the effect of heavy traffic conditions as well as the Superpave mix design procedures. Moreover, it is recommended that future work takes into consideration the effect of asphalt binder aging as well as quantifying the amount of entrapped air bubbles within the foamed asphalt binder in order to ensure accurate assessment of the volumetrics used in the preparation of WMA-FA mixtures.

8.4 Recommendations for Implementation

In 2008, ODOT former director, Mr. James Beasley, directed the wide implementation of foaming in the production of warm mix asphalt to be utilized as an asphalt paving material. As a result, specifications were written for this particular technology and ODOT allowed its use on low to medium traffic projects. However, due to the lack of a laboratory procedure by which this material can be produced, construction had to proceed without adequate information about its performance. Therefore, ODOT initiated this project with the primary objective of developing a procedure by which this material can be produced in the lab in order to compare its performance to traditional hot mix asphalt and better understand its limitations.

To this end and based on the research findings of this study, warm mix asphalt prepared using foamed asphalt binders seems to be a viable alternative to hot mix asphalt as a paving material for low to medium traffic projects. However, the performance of such material has to be evaluated in terms of permanent deformation. Therefore, it is recommended to modify ODOT C&MS Item 441 to include a permanent deformation test as part of the mix design procedure to ensure satisfactory long-term performance.

REFERENCES

- AASHTO T283. *Standard Method of Test for Resistance of Compacted Hot Mix Asphalt (HMA) to Moisture-Induced Damage*. American Association of State Highway and Transportation Officials (AASHTO), 2007.
- AASHTO TP 62-03. *Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures*. American Association of State Highway and Transportation Officials (AASHTO), 2001.
- Akeroyd, F. M. L. and B. J. Hicks (1988). "Foamed Bitumen Road Recycling." *Highways*, Volume 56, Number 1933, pp. 42-45.
- Aspha-Min Webpage (<http://www.aspha-min.de/en/>).
- Asphalt Pavement Analyzer User's Guide, Asphalt Pavement Technology, USA, 2003. Available online: <http://www.pavementtechnology.com/manuals/APAManual.pdf>.
- Astec, Inc. Webpage (<http://www.astecinc.com/>).
- Aurilio, V., Pine, W. J., and P. Lum (2006). "The Bailey Method-Achieving Volumetrics and HMA Compactability." Available Online: http://www.asphaltinstitute.org/public/engineering/PDFs/Bailey/Bailey_Method_Achieving_Volumetrics_HMA_Compactability_Paper.pdf.
- Barthel, W. and M. von Devivere (2003). "Warm Asphalt Mixes by Adding Aspha-Min. A Synthetic Zeolite." *Proceedings of the 48th Annual Convention of the National Asphalt Pavement Association (NAPA)*, San Diego, CA, January 11-17.
- Bissada, A. F. (1987). "Structural Response of Foamed-Asphalt-Sand Mixtures in Hot Environments." *Journal of the Transportation Research Board*, Transportation Research Record 1115, pp. 134-149.
- Corrigan, M. (2010). "What's New with WMA?" *Presented at the North Central Asphalt User/Producer Group (NCAUP) Hot Mix Asphalt Technical Conference*, Overland Park, KS, February 2-4.
- Csanyi, L. H. (1957). "Foamed Asphalt in Bituminous Pavements." *Highway Research Board Bulletin*, Vol. 10, No. 160, pp. 108-122.
- Davidson, J. K. (2007). "Warm Asphalt Mix Technology-The Canadian Perspective." Australian Asphalt Pavement Association.

- D'Angelo, J., Harm, E., Bartoszek, J., Baumgardner, G., Corrigan, M., Cowsert, J., Harman, T., Jamshidi, M., Jones, W., Newcomb, D., Prowell, B., Sines, R., and B. Yeaton (2008). *Warm-Mix Asphalt: European Practice*. Federal Highway Administration (FHWA), Report No. FHWA-PL-08-007.
- Federal Highway Administration (FHWA) Warm Mix Asphalt Webpage (<http://www.fhwa.dot.gov/pavement/asphalt/wma.cfm>).
- Hurley, G. C. and B. D. Prowell (2005a). *Evaluation of Aspha-Min Zeolite for Use in Warm Mix Asphalt*. National Center for Asphalt Technology (NCAT) Report 05-04.
- Hurley, G. C. and B. D. Prowell (2005b). *Evaluation of Sasobit for Use in Warm Mix Asphalt*. National Center for Asphalt Technology (NCAT) Report 05-06.
- Hurley, G. C. and B. D. Prowell (2006a). *Evaluation of Evotherm for Use in Warm Mix Asphalt*. National Center for Asphalt Technology (NCAT) Report 06-02.
- Hurley, G. C. and B. D. Prowell (2006b). "Evaluation of Potential Processes for Use in Warm Mix Asphalt." *Journal of the Association of Asphalt Paving Technologists*, Vol. 75, pp. 41-90.
- Jenkins, K. J. and M. F. C. van de Ven (1999). "Mix Design Considerations for Foamed Bitumen Mixtures." *Proceedings of the 7th Conference on Asphalt Pavements for Southern Africa*, Victoria Falls, Zimbabwe.
- Jenkins, K. J., J. L. A. de Groot, A. A. A. Molenaar, and van de Ven (1999). "Half-Warm Foamed Bitumen Treatment, a new process." *Proceedings of the 7th Conference on Asphalt Pavements for Southern Africa*, Victoria Falls, Zimbabwe.
- Kim, Y., Lee, H., and M. Heitzman (2007). "Experience of Developing and Validating a New Mix Design Procedure for Cold In-Place Recycling Using Foamed Asphalt." *Proceedings of the 2007 Mid-Continent Transportation Research Symposium*, Amex, Iowa.
- Koenders, B. G., Stoker, D. A., Bowen, C., de Groot, P., Larsen, O., Hardy, D., and K. P. Wilms (2000). "Innovative Process in Asphalt Production and Application to Obtain Lower Operating Temperatures." *2nd Eurasphalt & Eurobitume Congress*, Barcelona, Spain.
- Kristjansdottir, O. (2006). *Warm Mix Asphalt for Cold Weather Paving*, Master Thesis, Report No. WA-RD 650.1, University of Washington, Seattle, Washington.
- Kristjansdottir, O., Muench, S. T., Michael, L., and G. Burke (2007). "Assessing Potential for Warm-Mix Asphalt Technology Adoption." *Journal of the Transportation Research*

- Board*, Transportation Research Record 2040, pp. 91-99. LEA-CO Webpage (<http://www.lea-co.com/>).
- Kvasnak, A., West, R., Moore, J., Nelson, J., Turner, P., and Tran, N. (2009). "Case Study of Warm Mix Asphalt Moisture Susceptibility in Birmingham." *Proceedings of the 88th Transportation Research Board Annual Meeting*, Washington, D.C.
 - Kvasnak, A., Taylor, A., Signore, J., and Bukhari, S. (2010). *Evaluation of Gencor Green Machine Ultrafoam GX*. National Center for Asphalt Technology, NCAT Report 10-03.
 - Maccarrone, S., Holleran, G. and A. Ky (1994). "Cold Asphalt Systems as an Alternative to Hot Mix." *Proceedings of the 9th International Australian Asphalt Pavement Association (AAPA) Conference*, Queensland, Australia.
 - MeadWestVaco Webpage (<http://www.meadwestvaco.com/>).
 - Middleton, B. and Forfylow, R.W. (2009). "Evaluation of Warm Mix Asphalt Produced With the Double Barrel Green Process." *Journal of the Transportation Research Board*, Volume 2126, pp. 19-26.
 - Muthen, K. M. (1998). *Foamed Asphalt Mixes-Mix Design Procedure*. Sabita Ltd. and CSIR Transportek, Report CR-98/077, South Africa.
 - Pellinen, T. K. and Witzak, M. W. (2002). "Stress Dependent Master Curve Construction for Dynamic (Complex) Modulus." *Journal of Association of Asphalt Paving Technologists*, Volume 71, pp. 281-309.
 - Prowell, B. D., Hurley, G. C., and Crews, E. (2007). "Field Performance of Warm-Mix Asphalt at National Center for Asphalt Technology Test Track." *Journal of the Transportation Research Board*, Transportation Research Record 1998, pp. 96-102.
 - Romier, A., Audeon, M., Jac, D., Martineau, Y., and F. Olard (2006). "Low-Energy Asphalt with Performance of Hot-Mix Asphalt." *Journal of the Transportation Research Board*, Transportation Research Record 1962, pp. 101-112.
 - Ruckel, P. J., Acott, S. M., and R. H. Bowering (1983). "Foamed-Asphalt Paving Mixtures: Preparation of Design Mixes and Treatment of Test Specimens." *Journal of the Transportation Research Board*, Transportation Research Record 991, pp. 88-95.
 - Saleh, M. F. (2004). "New Zealand Experience with Foam Bitumen Stabilization." *Journal of the Transportation Research Board*, Transportation Research Record 1868, pp. 40-49.

- Saleh, M. F. (2006). "Effect of Aggregate Gradation, Mineral Fillers, Bitumen Grade, and Source on Mechanical Properties of Foamed Bitumen-Stabilized Mixes." *Journal of the Transportation Research Board*, Transportation Research Record 1952, pp. 90-100.
- Sargand, S., Figueroa, J. L., Edwards, W., and A. S. Al-Rawashdeh (2009). *Performance Assessment of Warm Mix Asphalt (WMA) Pavements*, Ohio Department of Transportation (ODOT), Report No. FHWA/OH-2009/08.
- Sasol Wax Webpage (<http://www.sasolwax.com/>).
- Vavrik, W. R., Pine, W. J., and S. H. Carpenter (2002). "Aggregate Blending for Asphalt Mix Design: Bailey Method." *Journal of the Transportation Research Board*, Transportation Research Record 1789, pp. 146-153.
- WAM-Foam Webpage (http://www.shell.com/home/content/bitumen/products/shell_wam_foam/).
- Warm Mix Asphalt Technical Working Group Webpage (<http://www.warmmixasphalt.com/>).
- Wasiuddin, N. M., Selvamohan, S., Zaman, M. M., and M. L. T. A. Guegan (2007). "Comparative Laboratory Study of Sasobit and Aspha-Min Additives in Warm-Mix Asphalt." *Journal of the Transportation Research Board*, Transportation Research Record 1998, pp. 82-88.
- Wielinski, J., Hand, A., and Rausch, D. (2009). "Laboratory and Field Evaluations of Foamed Warm-Mix Asphalt Projects." *Journal of the Transportation Research Board*, Transportation Research Record 2126, pp. 125-131.
- Xiao, F., Amirhanian, S., and Putman, B. (2009). "Evaluation of Rutting Resistance in Warm Mix Asphalts Containing Moist Aggregate." *Journal of the Transportation Research Board*, Transportation Research Record 2126, pp. 115-124.

APPENDIX A
RESULTS OF THE BAILEY METHOD

Table A-1: Trial Aggregate Gradations (Natural Gravel).

Sieve #	Opening	JMF B448483	Type I, Surface, Medium Traffic	
			Upper	Lower
1/2"	12.5	100	100	100
3/8"	9.5	97	90	100
#4	4.75	56	45	57
#8	2.36	42	30	45
#16	1.18	30	17	35
#30	0.6	21	12	25
#50	0.3	10	5	18
#100	0.15	5	2	10
#200	0.075	3.1	---	---

Sieve #	Opening	Trial Gradations											
		Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10		
1/2"	12.5	100	100	100	100	100	100	100	100	100	100	100	100
3/8"	9.5	93	91	95	96	96	92	96	96	96	96	96	92
#4	4.75	52	50	52	56	51	47	52	56	57	57	57	55
#8	2.36	42	38	38	40	36	34	38	35	40	40	40	41
#16	1.18	29	27	27	25	23	23	25	23	21	21	21	25
#30	0.6	21	19	19	17	17	17	17	17	17	17	17	14
#50	0.3	10	8	8	7	7	7	11	9	11	11	11	5
#100	0.15	4	4	4	3	3	3	3	4	4	4	4	4
#200	0.075	2	2	2	1	1	1	2	2	2	2	2	2

Table A-2: Comparison between Selected Blends (Natural Gravel).

	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	0.000	1.000	1.000	0.500	1.500	2.000	1.000	1.750	0.500	0.250
CA RAtio	0.000	0.158	0.417	0.777	0.489	0.185	0.417	1.345	0.935	0.514
FAc Ratio	0.000	0.000	0.000	1.500	0.556	0.000	1.053	0.286	1.500	3.171
FAF Ratio	0.000	-0.401	-0.401	0.280	0.280	0.280	0.280	0.280	-0.896	-1.905
ΔVMA Predicted	0.0	0.8	1.0	3.1	2.8	2.5	2.7	3.7	2.0	2.0
Actual ΔVMA	0.0	-0.6	-0.1	2.2	2.2	0.6	0.8	1.0	0.8	2.2
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	-1.000	0.000	0.000	-0.500	0.500	1.000	0.000	0.750	-0.500	-0.750
CA Ratio	-0.158	0.000	0.258	0.618	0.331	0.026	0.258	1.186	0.777	0.356
FAc Ratio	0.000	0.000	0.000	1.500	0.556	0.000	1.053	0.286	1.500	3.171
FAF Ratio	0.401	0.000	0.000	0.681	0.681	0.681	0.681	0.681	-0.495	-1.504
ΔVMA Predicted	-0.8	0.0	0.3	2.3	2.1	1.7	2.0	2.9	1.3	1.3
Actual ΔVMA	0.6	0.0	0.5	2.7	2.7	1.1	1.4	1.6	1.3	2.8
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	-1.000	0.000	0.000	-0.500	0.500	1.000	0.000	0.750	-0.500	-0.750
CA Ratio	-0.417	-0.258	0.000	0.360	0.072	-0.232	0.000	0.928	0.518	0.097
FAc Ratio	0.000	0.000	0.000	1.500	0.556	0.000	1.053	0.286	1.500	3.171
FAF Ratio	0.401	0.000	0.000	0.681	0.681	0.681	0.681	0.681	-0.495	-1.504
ΔVMA Predicted	-1.0	-0.3	0.0	2.0	1.8	1.4	1.7	2.6	1.0	1.0
Actual ΔVMA	0.1	-0.5	0.0	2.2	2.2	0.6	0.9	1.0	0.8	2.3
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	-0.500	0.000	0.500	0.000	1.000	1.500	0.500	1.250	0.000	-0.250
CA RAtio	-0.777	-0.618	-0.360	0.000	-0.288	-0.592	-0.360	0.568	0.159	-0.263
FAc Ratio	-1.500	-1.500	-1.500	0.000	-0.944	-1.500	-0.447	-1.214	0.000	1.671
FAF Ratio	-0.280	-0.681	-0.681	0.000	0.000	0.000	0.000	0.000	-1.176	-2.185
ΔVMA Predicted	-3.1	-2.3	-2.0	0.0	-0.2	-0.6	-0.3	0.6	-1.0	-1.0
Actual ΔVMA	-2.2	-2.7	-2.2	0.0	0.0	-1.6	-1.3	-1.2	-1.4	0.1
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	-1.500	-0.500	-0.500	-1.000	0.000	0.500	-0.500	0.250	-1.000	-1.250
CA RAtio	-0.489	-0.331	-0.072	0.288	0.000	-0.304	-0.072	0.856	0.446	0.025
FAc Ratio	-0.556	-0.556	-0.556	0.944	0.000	-0.556	0.497	-0.270	0.944	2.615
FAF Ratio	-0.280	-0.681	-0.681	0.000	0.000	0.000	0.000	0.000	-1.176	-2.185
ΔVMA Predicted	-2.8	-2.1	-1.8	0.2	0.0	-0.4	-0.1	0.8	-0.8	-0.8
Actual ΔVMA	-2.2	-2.7	-2.2	0.0	0.0	-1.6	-1.3	-1.2	-1.4	0.1
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	-2.000	-1.000	-1.000	-1.500	-0.500	0.000	-1.000	-0.250	-1.500	-1.750
CA RAtio	-0.185	-0.026	0.232	0.592	0.304	0.000	0.232	1.160	0.750	0.329
FAc Ratio	0.000	0.000	0.000	1.500	0.556	0.000	1.053	0.286	1.500	3.171
FAF Ratio	-0.280	-0.681	-0.681	0.000	0.000	0.000	0.000	0.000	-1.176	-2.185
ΔVMA Predicted	-2.5	-1.7	-1.4	0.6	0.4	0.0	0.3	1.2	-0.4	-0.4
Actual ΔVMA	-0.6	-1.1	-0.6	1.6	1.6	0.0	0.3	0.4	0.2	1.7
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	-1.000	0.000	0.000	-0.500	0.500	1.000	0.000	0.750	-0.500	-0.750
CA RAtio	-0.417	-0.258	0.000	0.360	0.072	-0.232	0.000	0.928	0.518	0.097
FAc Ratio	-1.053	-1.053	-1.053	0.447	-0.497	-1.053	0.000	-0.767	0.447	2.118
FAF Ratio	-0.280	-0.681	-0.681	0.000	0.000	0.000	0.000	0.000	-1.176	-2.185
ΔVMA Predicted	-2.7	-2.0	-1.7	0.3	0.1	-0.3	0.0	0.9	-0.7	-0.7
Actual ΔVMA	-0.8	-1.4	-0.9	1.3	1.3	-0.3	0.0	0.1	-0.1	1.4
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	-1.750	-0.750	-0.750	-1.250	-0.250	0.250	-0.750	0.000	-1.250	-1.500
CA RAtio	-1.345	-1.186	-0.928	-0.568	-0.856	-1.160	-0.928	0.000	-0.410	-0.831
FAc Ratio	-0.286	-0.286	-0.286	1.214	0.270	-0.286	0.767	0.000	1.214	2.885
FAF Ratio	-0.280	-0.681	-0.681	0.000	0.000	0.000	0.000	0.000	-1.176	-2.185
ΔVMA Predicted	-3.7	-2.9	-2.6	-0.6	-0.8	-1.2	-0.9	0.0	-1.6	-1.6
Actual ΔVMA	-1.0	-1.6	-1.0	1.2	1.2	-0.4	-0.1	0.0	-0.2	1.2
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	-0.500	0.500	0.500	0.000	1.000	1.500	0.500	1.250	0.000	-0.250
CA RAtio	-0.935	-0.777	-0.518	-0.159	-0.446	-0.750	-0.518	0.410	0.000	-0.421
FAc Ratio	-1.500	-1.500	-1.500	0.000	-0.944	-1.500	-0.447	-1.214	0.000	1.671
FAF Ratio	0.896	0.495	0.495	1.176	1.176	1.176	1.176	1.176	0.000	-1.008
ΔVMA Predicted	-2.0	-1.3	-1.0	1.0	0.8	0.4	0.7	1.6	0.0	0.0
Actual ΔVMA	-0.8	-1.3	-0.8	1.4	1.4	-0.2	0.1	0.2	0.0	1.5
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	-0.250	0.750	0.750	0.250	1.250	1.750	0.750	1.500	0.250	0.000
CA RAtio	-0.514	-0.356	-0.097	0.263	-0.025	-0.329	-0.097	0.831	0.421	0.000
FAc Ratio	-3.171	-3.171	-3.171	-1.671	-2.615	-3.171	-2.118	-2.885	-1.671	0.000
FAF Ratio	1.905	1.504	1.504	2.185	2.185	2.185	2.185	2.185	1.008	0.000
ΔVMA Predicted	-2.0	-1.3	-1.0	1.0	0.8	0.4	0.7	1.6	0.0	0.0
Actual ΔVMA	-2.2	-2.8	-2.3	-0.1	-0.1	-1.7	-1.4	-1.2	-1.5	0.0

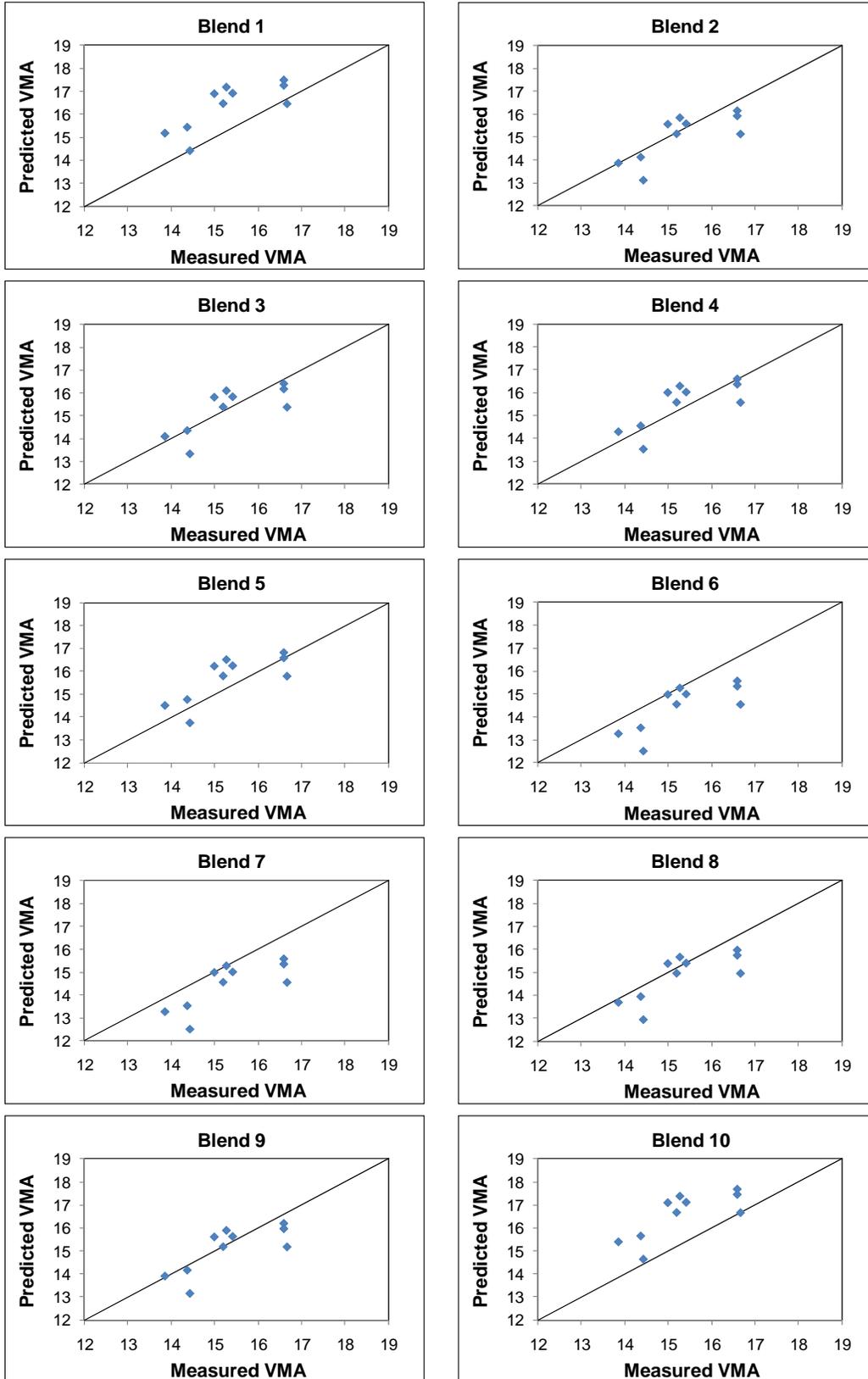


Figure A-2: Predicted vs. Measured VMA using the Bailey Method (Natural Gravel).

Table A-3: Trial Aggregate Gradations (Crushed Limestone).

Sieve #	Opening	JMF B446376	Type I, Surface, Medium Traffic	
			Upper	Lower
1/2"	12.5	100	100	100
3/8"	9.5	94	90	100
#4	4.75	54	45	57
#8	2.36	35	30	45
#16	1.18	24	17	35
#30	0.6	16	12	25
#50	0.3	10	5	18
#100	0.15	6	2	10
#200	0.075	3.8	---	---

Trial Gradations											
Sieve #	Opening	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
1/2"	12.5	100	100	100	100	100	100	100	100	100	100
3/8"	9.5	95	94	94.5	92	93	92	92	92	92	96
#4	4.75	51	56	53.5	52	53	55	55	56	48	52
#8	2.36	38	41	39.5	43	41	43	43	43	43	43
#16	1.18	26	30	28	32	30	33	33	32	32	32
#30	0.6	18	22	20	24	22	24	24	24	24	24
#50	0.3	12	16	14	16	16	13	13	16	16	16
#100	0.15	6	8	7	8	7.5	6	8	8	8	8
#200	0.075	3	3	3	3	3	2	2	3	3	3

Table A-2: Comparison between Selected Blends (Crushed Limestone).

	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	0.000	-0.750	-0.375	-1.250	-0.813	-1.250	-1.250	-1.250	-1.250	-1.250
CA RAtio	0.000	0.378	0.179	-0.389	-0.110	0.007	0.007	0.151	-0.846	-0.389
FAC Ratio	0.000	-1.258	-0.653	-1.689	-1.193	-1.689	-1.689	-1.689	-1.689	-1.689
FAF Ratio	0.000	-0.606	-0.333	0.000	-0.152	1.667	0.000	0.000	0.000	0.000
ΔVMA Predicted	0.0	-2.2	-1.2	-3.3	-2.3	-1.3	-2.9	-2.8	-3.8	-3.3
Actual ΔVMA	0.0	-1.6	-2.0	-2.3	-1.6	-0.7	-0.9	-1.3	-1.8	-2.0
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	0.750	0.000	0.375	-0.500	-0.063	-0.500	-0.500	-0.500	-0.500	-0.500
CA RAtio	-0.378	0.000	-0.199	-0.767	-0.488	-0.371	-0.371	-0.227	-1.224	-0.767
FAC Ratio	1.258	0.000	0.605	-0.431	0.065	-0.431	-0.431	-0.431	-0.431	-0.431
FAF Ratio	0.606	0.000	0.273	0.606	0.455	2.273	0.606	0.606	0.606	0.606
ΔVMA Predicted	2.2	0.0	1.1	-1.1	0.0	1.0	-0.7	-0.6	-1.5	-1.1
Actual ΔVMA	1.6	0.0	-0.3	-0.6	0.0	0.9	0.8	0.4	-0.2	-0.3
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	0.375	-0.375	0.000	-0.875	-0.438	-0.875	-0.875	-0.875	-0.875	-0.875
CA RAtio	-0.179	0.199	0.000	-0.568	-0.288	-0.172	-0.172	-0.028	-1.025	-0.568
FAC Ratio	0.653	-0.605	0.000	-1.036	-0.540	-1.036	-1.036	-1.036	-1.036	-1.036
FAF Ratio	0.333	-0.273	0.000	0.333	0.182	2.000	0.333	0.333	0.333	0.333
ΔVMA Predicted	1.2	-1.1	0.0	-2.1	-1.1	-0.1	-1.7	-1.6	-2.6	-2.1
Actual ΔVMA	2.0	0.3	0.0	-0.3	0.3	1.3	1.1	0.7	0.2	0.0
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	1.250	0.500	0.875	0.000	0.438	0.000	0.000	0.000	0.000	0.000
CA RAtio	0.389	0.767	0.568	0.000	0.279	0.396	0.396	0.540	-0.457	0.000
FAC Ratio	1.689	0.431	1.036	0.000	0.496	0.000	0.000	0.000	0.000	0.000
FAF Ratio	0.000	-0.606	-0.333	0.000	-0.152	1.667	0.000	0.000	0.000	0.000
ΔVMA Predicted	3.3	1.1	2.1	0.0	1.1	2.1	0.4	0.5	-0.5	0.0
Actual ΔVMA	2.3	0.6	0.3	0.0	0.6	1.6	1.4	1.0	0.5	0.3
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	0.813	0.063	0.438	-0.438	0.000	-0.438	-0.438	-0.438	-0.438	-0.438
CA RAtio	0.110	0.488	0.288	-0.279	0.000	0.116	0.116	0.260	-0.736	-0.279
FAC Ratio	1.193	-0.065	0.540	-0.496	0.000	-0.496	-0.496	-0.496	-0.496	-0.496
FAF Ratio	0.152	-0.455	-0.182	0.152	0.000	1.818	0.152	0.152	0.152	0.152
ΔVMA Predicted	2.3	0.0	1.1	-1.1	0.0	1.0	-0.7	-0.5	-1.5	-1.1
Actual ΔVMA	1.6	0.0	-0.3	-0.6	0.0	0.9	0.8	0.4	-0.2	-0.4
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	1.250	0.500	0.875	0.000	0.438	0.000	0.000	0.000	0.000	0.000
CA RAtio	-0.007	0.371	0.172	-0.396	-0.116	0.000	0.000	0.144	-0.853	-0.396
FAC Ratio	1.689	0.431	1.036	0.000	0.496	0.000	0.000	0.000	0.000	0.000
FAF Ratio	-1.667	-2.273	-2.000	-1.667	-1.818	0.000	-1.667	-1.667	-1.667	-1.667
ΔVMA Predicted	1.3	-1.0	0.1	-2.1	-1.0	0.0	-1.7	-1.5	-2.5	-2.1
Actual ΔVMA	0.7	-0.9	-1.3	-1.6	-0.9	0.0	-0.2	-0.6	-1.1	-1.3
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	1.250	0.500	0.875	0.000	0.438	0.000	0.000	0.000	0.000	0.000
CA RAtio	-0.007	0.371	0.172	-0.396	-0.116	0.000	0.000	0.144	-0.853	-0.396
FAC Ratio	1.689	0.431	1.036	0.000	0.496	0.000	0.000	0.000	0.000	0.000
FAF Ratio	0.000	-0.606	-0.333	0.000	-0.152	1.667	0.000	0.000	0.000	0.000
ΔVMA Predicted	2.9	0.7	1.7	-0.4	0.7	1.7	0.0	0.1	-0.9	-0.4
Actual ΔVMA	0.9	-0.8	-1.1	-1.4	-0.8	0.2	0.0	-0.4	-0.9	-1.1
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	1.250	0.500	0.875	0.000	0.438	0.000	0.000	0.000	0.000	0.000
CA RAtio	-0.151	0.227	0.028	-0.540	-0.260	-0.144	-0.144	0.000	-0.997	-0.540
FAC Ratio	1.689	0.431	1.036	0.000	0.496	0.000	0.000	0.000	0.000	0.000
FAF Ratio	0.000	-0.606	-0.333	0.000	-0.152	1.667	0.000	0.000	0.000	0.000
ΔVMA Predicted	2.8	0.6	1.6	-0.5	0.5	1.5	-0.1	0.0	-1.0	-0.5
Actual ΔVMA	1.3	-0.4	-0.7	-1.0	-0.4	0.6	0.4	0.0	-0.5	-0.7
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	1.250	0.500	0.875	0.000	0.438	0.000	0.000	0.000	0.000	0.000
CA RAtio	0.846	1.224	1.025	0.457	0.736	0.853	0.853	0.997	0.000	0.457
FAC Ratio	1.689	0.431	1.036	0.000	0.496	0.000	0.000	0.000	0.000	0.000
FAF Ratio	0.000	-0.606	-0.333	0.000	-0.152	1.667	0.000	0.000	0.000	0.000
ΔVMA Predicted	3.8	1.5	2.6	0.5	1.5	2.5	0.9	1.0	0.0	0.5
Actual ΔVMA	1.8	0.2	-0.2	-0.5	0.2	1.1	0.9	0.5	0.0	-0.2
	Blend 1	Blend 2	Blend 3	Blend 4	Blend 5	Blend 6	Blend 7	Blend 8	Blend 9	Blend 10
PCS	1.250	0.500	0.875	0.000	0.438	0.000	0.000	0.000	0.000	0.000
CA RAtio	0.389	0.767	0.568	0.000	0.279	0.396	0.396	0.540	-0.457	0.000
FAC Ratio	1.689	0.431	1.036	0.000	0.496	0.000	0.000	0.000	0.000	0.000
FAF Ratio	0.000	-0.606	-0.333	0.000	-0.152	1.667	0.000	0.000	0.000	0.000
ΔVMA Predicted	3.3	1.1	2.1	0.0	1.1	2.1	0.4	0.5	-0.5	0.0
Actual ΔVMA	2.0	0.3	0.0	-0.3	0.4	1.3	1.1	0.2	0.2	0.0

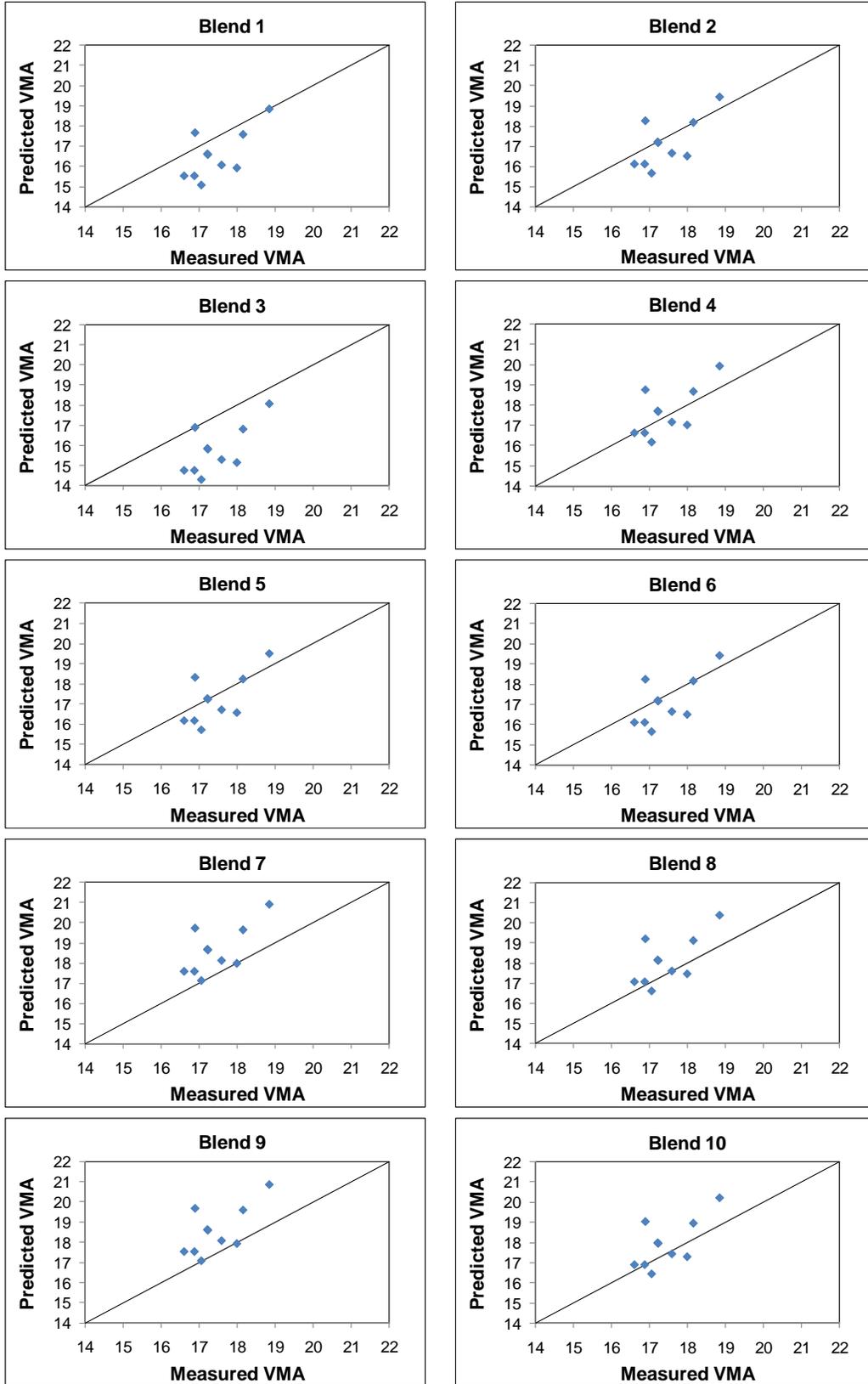


Figure A-2: Predicted vs. Measured VMA using the Bailey Method (Crushed Limestone).

APPENDIX B
MARSHALL MIX DESIGN RESULTS

Table B-1: ODOT Gradation and Mix Design Requirements for Type I Surface Mix Subjected to Medium Traffic (after ODOT C&MS 2008).

Course	Type I Surface Course
Traffic	Medium
1/2"	100
3/8"	90-100
No. 4	45-57
No. 8	30-45
No. 16	17-35
No. 30	12-25
No. 50	5-18
No. 100	2-10
No. 200	---
F/A Ratio ¹ , max	1.20
F-T Value ² (%)	+2
Asphalt Binder (%)	5.8-10
Virgin Asphalt Binder (%), min	5.00
Blows/Face	50
Stability (lbs), min	1200
Flow (0.01 in.)	8-16
Design Air Voids (%)	3.50
VMA (%), min	16

¹F/A Ratio = Percent passing Sieve No. 200 divided by asphalt content

²F-T Value = (Sieve No. 30 – Sieve No. 50) – (Sieve No. 16 – Sieve No. 30)

1- Mixes Containing Natural Gravel and PG 64-22:

Table B-2: Aggregate Gradation for Mixes Containing Natural Gravel.

Sieve Size	Percent Passing	Control Lower	Control Upper
2"	100	100	100
1 1/2"	100	100	100
1"	100	100	100
3/4"	100	100	100
1/2"	100	100	100
3/8"	96	90	100
#4	56	45	57
#8	35	30	45
#16	23	17	35
#30	17	12	25
#50	9	5	18
#100	4	2	10
#200	1	---	---

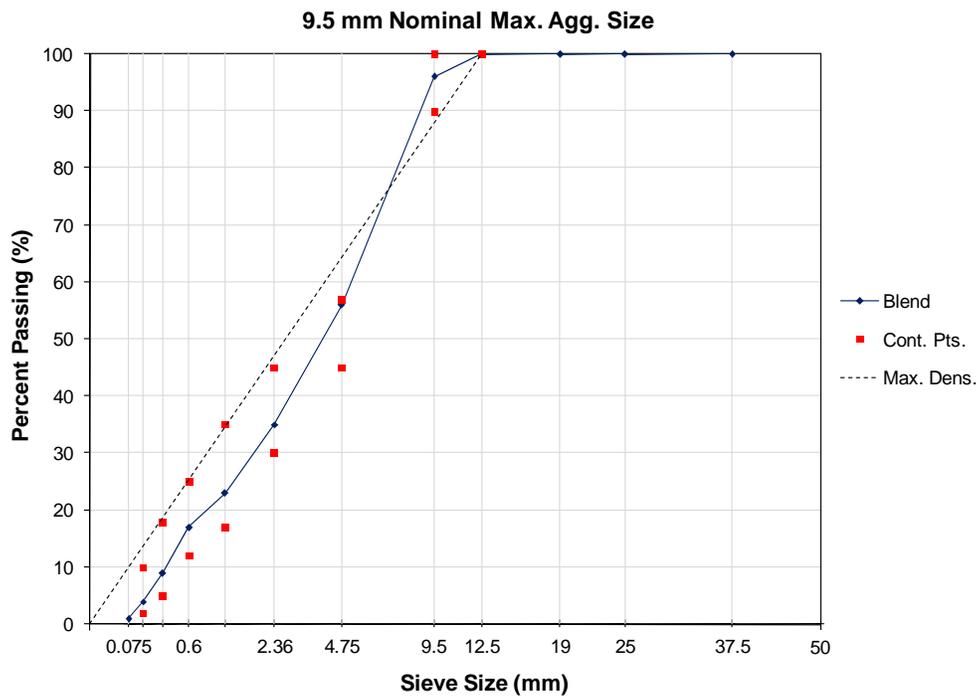


Figure B-1: Aggregate Gradation Chart for Mixes Containing Natural Gravel.

Table B-3: Marshall Mix Design Results
for Mixes Containing Natural Gravel and PG 64-22.

RICE DETERMINATION OF MAX SPECIFIC GRAVITY							
PART 1							
Pb	BINDER CONTENT %	6	6	CALCULATE Gse = $(1 - Pb)/((1/F)-(Pb/Gb))$ AVG F Gse = 2.627			
A	DRY WT. OF MIX	1531.3	1556.4				
B	CONT. & MIX & WATER	4247.6	4262.2				
C	CONT. & WATER	3353.3	3353				
F	(A/(C+A-B))	2.404	2.405				
PART 2							
Pmm	TOTAL MIXTURE %	100	100	100	100	100	100
Ps	AGGREGATE %	94.5	94	93.5	93	0	0
Pb	BINDER CONTENT %	5.5	6	6.5	7	0	0
Gb	APP. SP. GR. BINDER	1.034	1.034	1.034	1.034	1.034	1.034
Gse	EFF. SP. GR. AGG.	2.627	2.627	2.627	2.627	2.627	2.627
G	(Ps/Gse)	35.973	35.783	35.592	35.402	0	0
H	(Pb/Gb)	5.319	5.803	6.286	6.77	0	0
I	(G+H)	41.292	41.585	41.878	42.172	0	0
Gmm	(Pmm/I) MAX SP. GR.	2.422	2.405	2.388	2.371	0	0

VMA CALCULATIONS							
Coarse Agg.		% of Blend	Size	Dry Blk SG			
Canton Aggregates, Canton, OH		44	#8	2.559			
0		0	0	1			
0		0	0	1			
Fine Agg.							
Canton Aggregates, Canton, OH		56	Sand	2.603			
0		0	0	1			
0		0	0	1			
CALCULATE Gsb: $Gsb = 1/((CA\%/Gca)+(FA\%/Gfa)) =$					2.583		
Ps	AGGREGATE %	94.5	94	93.5	93	0	0
Gsb	BULK SP. GR. AGG	2.583	2.583	2.583	2.583	2.583	2.583
Gmb	AVG. BULK SP GR MIX	2.308	2.322	2.334	2.347	0	0
J	(Gmb/Gsb)*Ps	84.4	84.5	84.5	84.5	0	0
VMA	(100 - J)	15.6	15.5	15.5	15.5	0	0

Table B-4: Marshall Mix Design Calculations for Mixes Containing Natural Gravel and PG 64-22.

Sample No.	Binder %	Thickness (in.)	Weight (g)		Vol. (cm ³)	Gmb	Gmm	% Binder by Vol.	Voids (Percent)			Unit Wt kg/m ³	Stability		Flow 0.01"	
			In Air	SSD					In Water	Total Mix	Filled		VMA	Chart		Converted
A	B	C	D	R	E	F	G	H	I	J	L	M	N	O	P	Q
(R-E) (D/F) (B*G)/Cb 100-100(G/H) (L-J)/L G*62.4 M*16.02																
A	5.5	2.62	1205.6	1207.3	683.5	2.302								2072	1932	8.5
B	5.5	2.58	1198.9	1199.9	681.5	2.313								1961	1866	8
C	5.5	2.6	1202.4	1204.1	683.4	2.309								1942	1829	8.7
Average of 5.5% Binder Specimens																
A	6	2.57	1207.8	1208.6	688	2.32								1751	1675	11.2
B	6	2.58	1199	1199.9	683.3	2.321								1665	1584	10.5
C	6	2.58	1206.5	1207.8	688.7	2.324								1848	1759	9.35
Average of 6.0% Binder Specimens																
A	6.5	2.59	1207.5	1208.5	690.6	2.332								1687	1597	12
B	6.5	2.55	1194.9	1195.6	683.5	2.333								1526	1477	12.1
C	6.5	2.56	1202.7	1203.6	689.2	2.338								1533	1474	12.8
Average of 6.5% Binder Specimens																
A	7	2.54	1203.8	1204.3	692.2	2.351								1569	1669	13
B	7	2.55	1200.3	1201.1	689.3	2.345								1521	1472	14.1
C	7	2.55	1207.6	1208.1	693	2.344								1641	1588	14.5
Average of 7.0% Binder Specimens																
						2.347	2.371	15.9	1.03	93.3	15.5	146.4	2346		1577	13.9

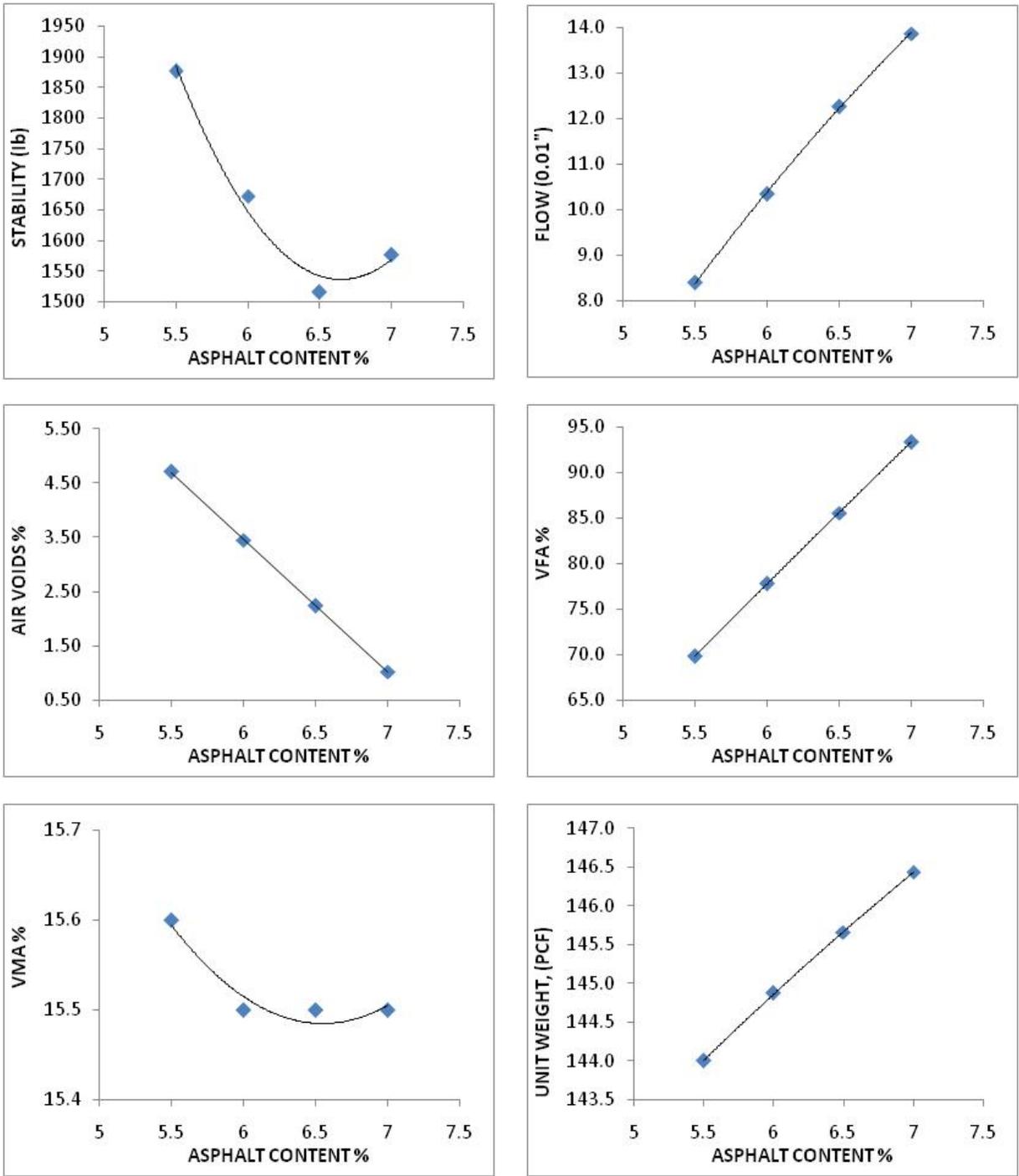


Figure B-2: Marshall Mix Design Plots
for Mixes Containing Natural Gravel and PG 64-22.

2- Mixes Containing Natural Gravel and PG 70-22M:

Table B-5: Aggregate Gradation for Mixes Containing Natural Gravel.

Sieve Size	Percent Passing	Control Lower	Control Upper
2"	100	100	100
1 1/2"	100	100	100
1"	100	100	100
3/4"	100	100	100
1/2"	100	100	100
3/8"	96	90	100
#4	56	45	57
#8	35	30	45
#16	23	17	35
#30	17	12	25
#50	9	5	18
#100	4	2	10
#200	1	---	---

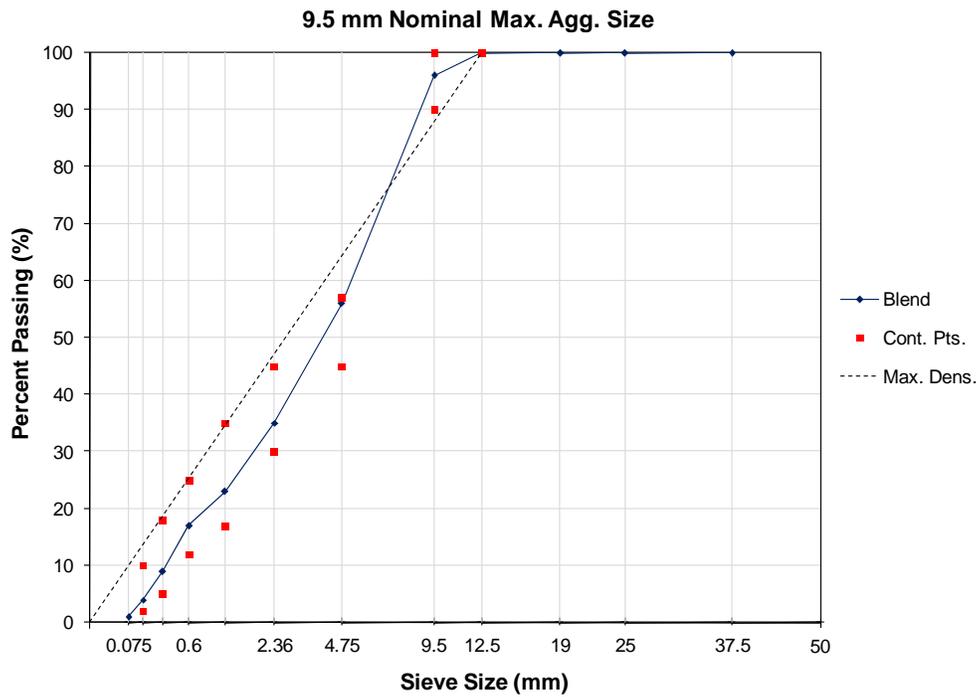


Figure B-3: Aggregate Gradation Chart for Mixes Containing Natural Gravel.

Table B-6: Marshall Mix Design Results
for Mixes Containing Natural Gravel and PG 70-22M.

RICE DETERMINATION OF MAX SPECIFIC GRAVITY							
PART 1							
Pb	BINDER CONTENT %	6	6	CALCULATE Gse = $(1 - Pb) / ((1/F) - (Pb/Gb))$ Gse = 2.630			
A	DRY WT. OF MIX	1512	1508.8				
B	CONT. & MIX & WATER	4232.7	4231.1				
C	CONT. & WATER	3349.1	3349.1				
F	(A/(C+A-B))	2.406	2.407				
				AVG F	2.407		
PART 2							
Pmm	TOTAL MIXTURE %	100	100	100	100	100	100
Ps	AGGREGATE %	94.5	94	93.5	93	0	0
Pb	BINDER CONTENT %	5.5	6	6.5	7	0	0
Gb	APP. SP. GR. BINDER	1.033	1.033	1.033	1.033	1.033	1.033
Gse	EFF. SP. GR. AGG.	2.630	2.630	2.630	2.630	2.630	2.630
G	(Ps/Gse)	35.934	35.744	35.553	35.363	0	0
H	(Pb/Gb)	5.324	5.808	6.292	6.776	0	0
I	(G+H)	41.258	41.552	41.846	42.14	0	0
Gmm	(Pmm/I) MAX SP. GR.	2.424	2.407	2.390	2.373	0	0

VMA CALCULATIONS							
Coarse Agg.		% of Blend	Size	Dry Blk SG			
Canton Aggregates, Canton, OH		44	#8	2.559			
0		0	0	1			
0		0	0	1			
Fine Agg.							
Canton Aggregates, Canton, OH		56	Sand	2.603			
0		0	0	1			
0		0	0	1			
CALCULATE Gsb: $Gsb = 1 / ((CA\% / Gca) + (FA\% / Gfa)) =$					2.583		
Ps	AGGREGATE %	94.5	94	93.5	93	0	0
Gsb	BULK SP. GR. AGG	2.583	2.583	2.583	2.583	2.583	2.583
Gmb	AVG. BULK SP GR MIX	2.308	2.322	2.331	2.341	0	0
J	(Gmb/Gsb)*Ps	84.4	84.5	84.4	84.3	0	0
VMA	(100 - J)	15.6	15.5	15.6	15.7	0	0

Table B-7: Marshall Mix Design Calculations for Mixes Containing Natural Gravel and PG 70-22M.

Sample No.	Binder %	Thickness (in.)	Weight (g)		Vol. (cm ³)	Gmb	Gmm	% Binder by Vol.	Voids (Percent)			Unit Wt kg/m ³	Stability		Flow 0.01"	
			In Air	SSD					In Water	Total Mix	Filled		VMA	Chart		Converted
A	B	C	D	R	E	F	G	H	I	J	L	M	N	O	P	Q
(R-E) (D/F) (B*G)/Cb 100- (L-J)/L M*16.02																
100(G/H)																
A	5.5	2.55	1200.5	1202.4	682	520	2.307							2616	2532	9.5
B	5.5	2.58	1201.7	1203.7	681.6	522	2.302							2617	2490	9
C	5.5	2.58	1199.8	1200.9	682.8	518	2.316							2628	2501	9
Average of 5.5% Binder Specimens																
A	6	2.56	1196.7	1198.4	681.9	517	2.317							2411	2317	11
B	6	2.58	1202.3	1203.7	687.1	517	2.327							2588	2463	10.7
C	6	2.57	1199.9	1201.9	685	517	2.321							2216	2119	10
Average of 6.0% Binder Specimens																
A	6.5	2.57	1198.8	1200	686.7	513	2.335							2349	2247	13.5
B	6.5	2.57	1204	1205	688.3	517	2.33							2147	2053	12.5
C	6.5	2.6	1203.7	1205.4	688	517	2.326							2055	1936	13.5
Average of 6.5% Binder Specimens																
A	7	2.55	1193.4	1194.1	684.9	509	2.344							1782	1725	17
B	7	2.58	1197.3	1198.1	685	513	2.333							1909	1817	16.5
C	7	2.52	1191.9	1192.3	684.5	508	2.347							1941	1916	17
Average of 7.0% Binder Specimens																
							2.341	2.373	15.9	1.33	91.5	146.1	2341		1819	16.8

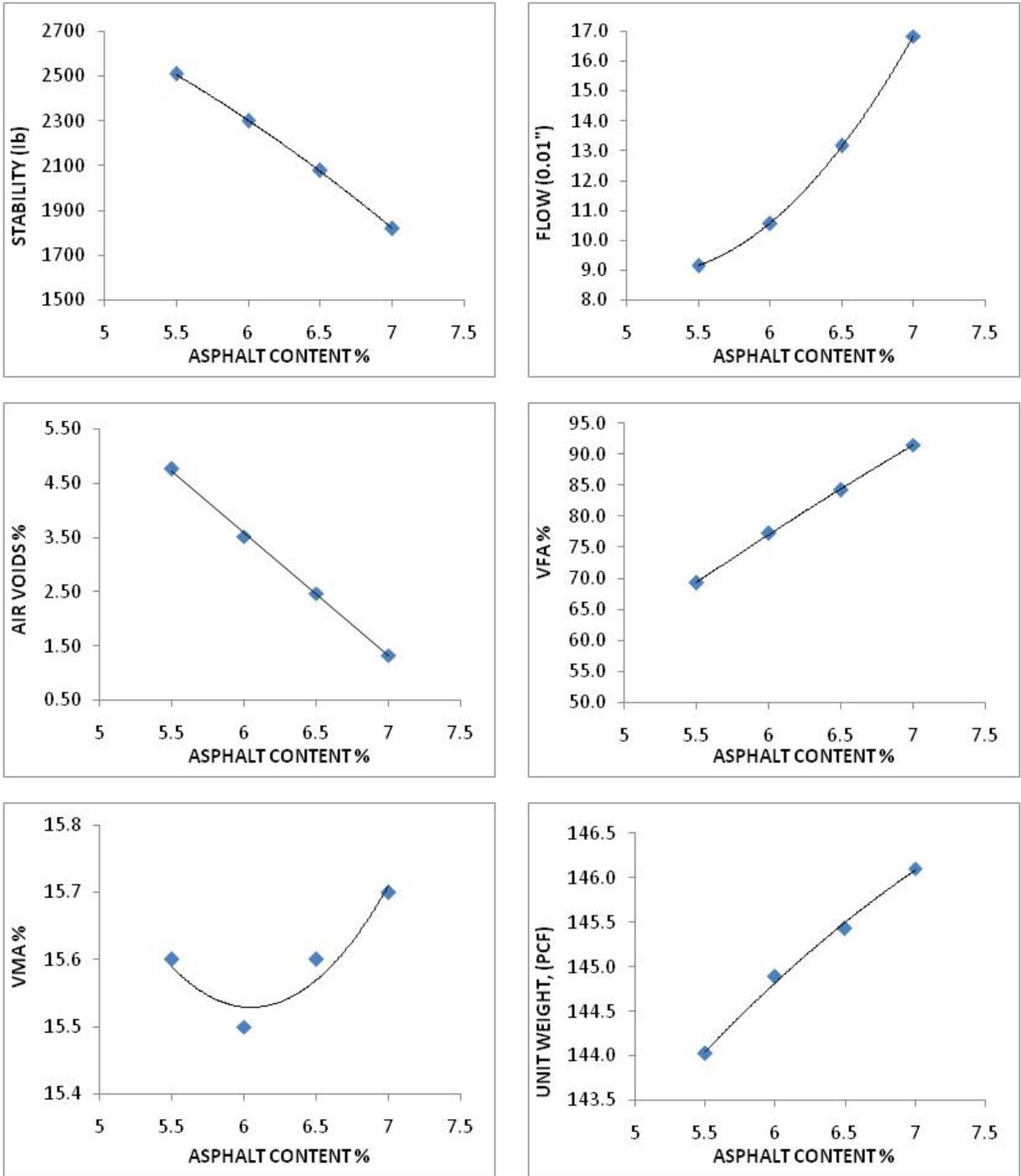


Figure B-4: Marshall Mix Design Plots for Mixes Containing Natural Gravel and PG 70-22M.

3- Mixes Containing Crushed Limestone and PG 64-22:

Table B-8: Aggregate Gradation for Mixes Containing Crushed Limestone.

Sieve Size	Percent Passing	Control Lower	Control Upper
2"	100	100	100
1 1/2"	100	100	100
1"	100	100	100
3/4"	100	100	100
1/2"	100	100	100
3/8"	92	90	100
#4	52	45	57
#8	43	30	45
#16	32	17	35
#30	24	12	25
#50	14	5	18
#100	8	2	10
#200	3	---	---

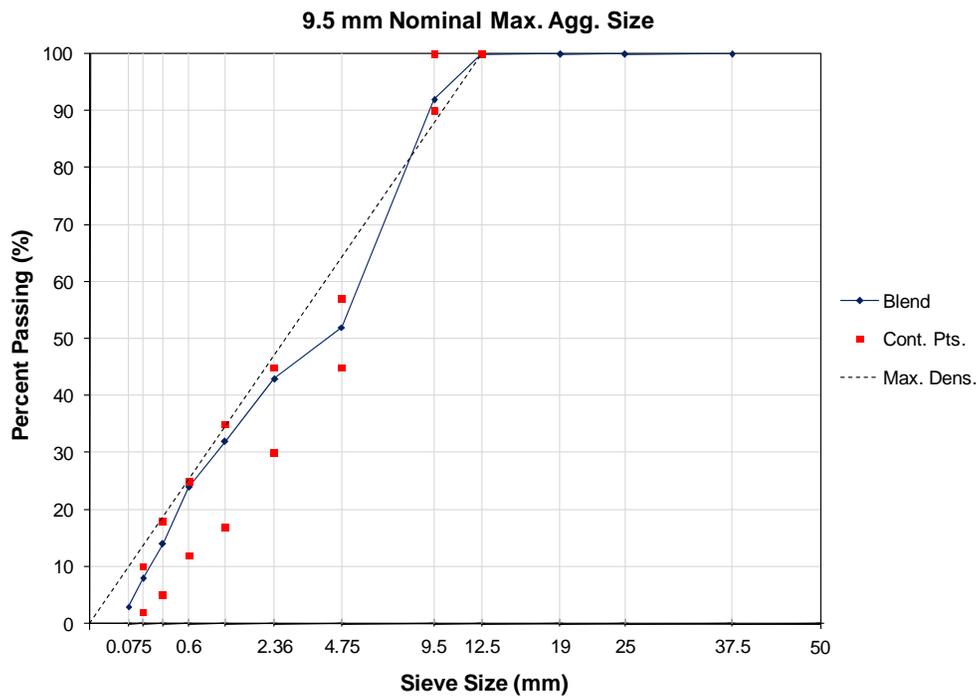


Figure B-5: Aggregate Gradation Chart for Mixing Containing Crushed Limestone.

Table B-9: Marshall Mix Design Results
for Mixes Containing Crushed Limestone and PG 64-22.

RICE DETERMINATION OF MAX SPECIFIC GRAVITY							
PART 1							
Pb	BINDER CONTENT %	6	6	CALCULATE Gse = $(1 - P_b) / ((1/F) - (P_b/G_b))$			
A	DRY WT. OF MIX	1511.4	1519.5				
B	CONT. & MIX & WATER	4254.4	4260.6				
C	CONT. & WATER	3350.6	3352.5				
F	(A/(C+A-B))	2.487	2.485	AVG F	Gse =	2.732	
				2.486			
PART 2							
Pmm	TOTAL MIXTURE %	100	100	100	100	100	100
Ps	AGGREGATE %	94.5	94	93.5	93	0	0
Pb	BINDER CONTENT %	5.5	6	6.5	7	0	0
Gb	APP. SP. GR. BINDER	1.033	1.033	1.033	1.033	1.033	1.033
Gse	EFF. SP. GR. AGG.	2.732	2.732	2.732	2.732	2.732	2.732
G	(Ps/Gse)	34.594	34.411	34.228	34.045	0	0
H	(Pb/Gb)	5.324	5.808	6.292	6.776	0	0
I	(G+H)	39.918	40.219	40.52	40.821	0	0
Gmm	(Pmm/I) MAX SP. GR.	2.505	2.486	2.468	2.45	0	0

VMA CALCULATIONS							
Coarse Agg.		% of Blend	Size	Dry Blk SG			
Nat. Lime & Stone, Akron, OH		48	#8	2.611			
0		0	0	1			
0		0	0	1			
Fine Agg.							
Nat. Lime & Stone, Akron, OH		52	Sand	2.748			
0		0	0	1			
0		0	0	1			
CALCULATE Gsb: $G_{sb} = 1 / ((CA\% / G_{ca}) + (FA\% / G_{fa})) =$					2.680		
Ps	AGGREGATE %	94.5	94	93.5	93	0	0
Gsb	BULK SP. GR. AGG	2.68	2.68	2.68	2.68	2.68	2.68
Gmb	AVG. BULK SP GR MIX	2.337	2.366	2.394	2.405	0	0
J	(Gmb/Gsb)*Ps	82.4	83	83.5	83.4	0	0
VMA	(100 - J)	17.6	17	16.5	16.6	0	0

Table B-10: Marshall Mix Design Calculations for Mixes Containing Crushed Limestone and PG 64-22.

Sample No.	Binder %	Thickness (in.)	Weight (g)			Vol. (cm ³)	G _{mb}	G _{mm}	%Binder by Vol.	Voids (Percent)			Unit Wt lb/ft ³	Unit Wt kg/m ³	Stability		Flow 0.01"
			In Air	SSD	In Water					Total Mix	Filled	VMA			Chart	Converted	
A	B	C	D	R	E	F	G	H	I	J	L	L	M	N	O	P	Q
(R-E) (D/F) (B*G)/Gb 100-100(G/H) (L-J)/L G*62.4 M*16.02																	
A	5.5	2.54	1199	1201.2	689.8	511.4	2,345								4180	3978	8.7
B	5.5	2.55	1199.1	1202.1	688.8	513.3	2,336								4182	4048	10.9
C	5.5	2.58	1206.5	1209.7	692.2	517.5	2,331								3951	3760	9
Average of 5.5% Binder Specimens																	
A	6	2.51	1195.2	1197.2	693.6	503.6	2,373		12.4	6.7	62	17.6	145.8	2337		3929	9.5
B	6	2.53	1206.7	1208	698.8	509.2	2,37								3605	3582	12.2
C	6	2.55	1207.1	1209.6	697.1	512.5	2,355								3559	3491	11.7
Average of 6.0% Binder Specimens																	
A	6.5	2.5	1204.9	1205.6	702.5	503.1	2,395		13.7	4.84	71.6	17	147.6	2365		3430	12.3
B	6.5	2.51	1210.2	1211	706.5	504.5	2,399								3307	3307	13.6
C	6.5	2.5	1197.8	1198.7	697.3	501.4	2,389								3205	3184	11.6
Average of 6.5% Binder Specimens																	
A	7	2.49	1205.7	1206	704.6	501.4	2,405		15.1	2.99	81.9	16.5	149.4	2393		3179	13.2
B	7	2.48	1198.8	1199	700.6	498.4	2,405								2835	2853	16
C	7	2.48	1206.9	1207.3	705.8	501.5	2,407								2946	2984	14.9
Average of 7.0% Binder Specimens																	
							2,406	2,45	16.3	1.8	89.1	16.6	150.1	2405		3015	15

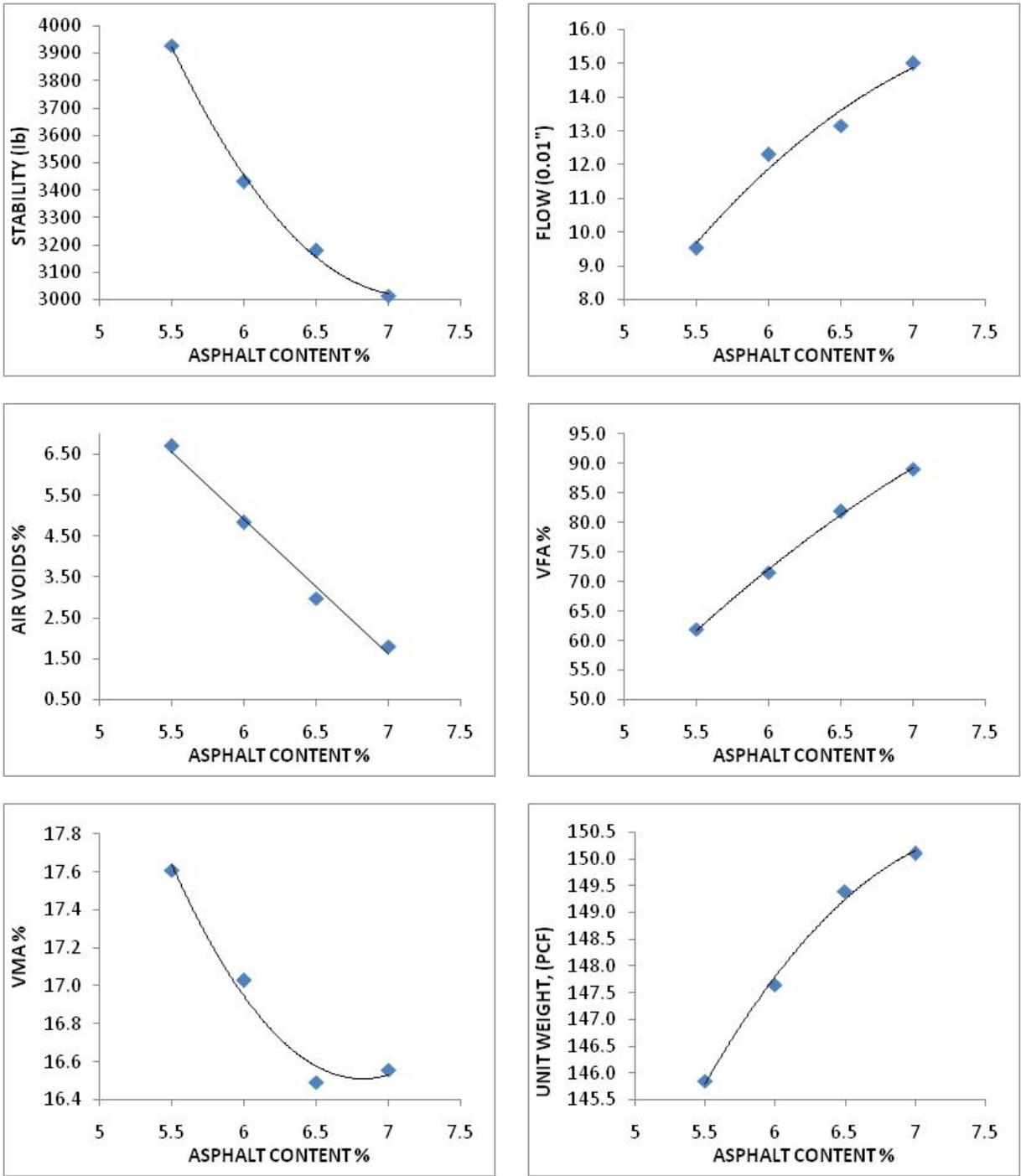


Figure B-6: Marshall Mix Design Plots
for Mixes Containing Crushed Limestone and PG 64-22.

4- Mixes Containing Crushed Limestone and PG 70-22M:

Table B-11: Aggregate Gradation for Mixes Containing Crushed Limestone.

Sieve Size	Percent Passing	Control Lower	Control Upper
2"	100	100	100
1 1/2"	100	100	100
1"	100	100	100
3/4"	100	100	100
1/2"	100	100	100
3/8"	92	90	100
#4	52	45	57
#8	43	30	45
#16	32	17	35
#30	24	12	25
#50	14	5	18
#100	8	2	10
#200	3	---	---

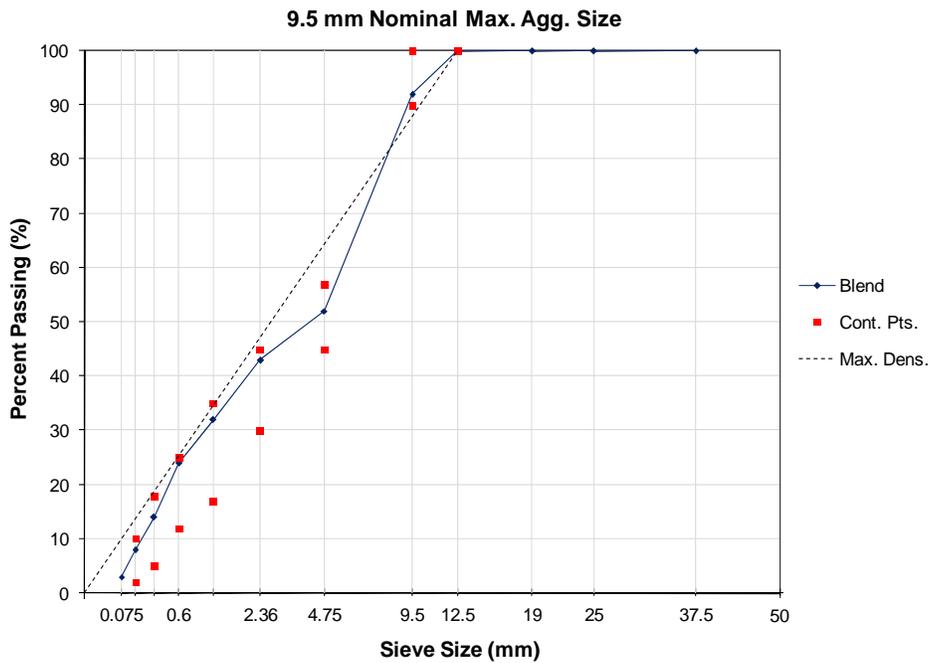


Figure B-7: Aggregate Gradation Chart for Mixes Containing Crushed Limestone.

Table B-12: Marshall Mix Design Results for
Mixes Containing Crushed Limestone and PG 70-22M.

RICE DETERMINATION OF MAX SPECIFIC GRAVITY							
PART 1							
Pb	BINDER CONTENT %	6	6	CALCULATE Gse = $(1 - Pb)/((1/F)-(Pb/Gb))$ AVG F Gse = 2.729			
A	DRY WT. OF MIX	1511.6	1517.2				
B	CONT. & MIX & WATER	4254.6	4259.5				
C	CONT. & WATER	3351.5	3352.9				
F	(A/(C+A-B))	2.484	2.485	2.484			
PART 2							
Pmm	TOTAL MIXTURE %	100	100	100	100	100	100
Ps	AGGREGATE %	94.5	94	93.5	93	0	0
Pb	BINDER CONTENT %	5.5	6	6.5	7	0	0
Gb	APP. SP. GR. BINDER	1.033	1.033	1.033	1.033	1.033	1.033
Gse	EFF. SP. GR. AGG.	2.729	2.729	2.729	2.729	2.729	2.729
G	(Ps/Gse)	34.625	34.442	34.259	34.076	0	0
H	(Pb/Gb)	5.324	5.808	6.292	6.776	0	0
I	(G+H)	39.949	40.25	40.551	40.852	0	0
Gmm	(Pmm/I) MAX SP. GR.	2.503	2.484	2.466	2.448	0	0

VMA CALCULATIONS							
Coarse Agg.		% of Blend	Size	Dry Blk SG			
Nat. Lime & Stone, Akron, OH		48	#8	2.611			
0		0	0	1			
0		0	0	1			
Fine Agg.							
Nat. Lime & Stone, Akron, OH		52	Sand	2.748			
0		0	0	1			
0		0	0	1			
CALCULATE Gsb: $Gsb = 1/((CA\%/Gca)+(FA\%/Gfa)) =$					2.680		
Ps	AGGREGATE %	94.5	94	93.5	93	0	0
Gsb	BULK SP. GR. AGG	2.680	2.680	2.680	2.680	2.680	2.680
Gmb	AVG. BULK SP GR MIX	2.330	2.373	2.386	2.400	0	0
J	(Gmb/Gsb)*Ps	82.1	83.2	83.2	83.3	0	0
VMA	(100 - J)	17.9	16.8	16.8	16.7	0	0

Table B-13: Marshall Mix Design Calculations for Mixes Containing Crushed Limestone and PG 70-22M.

Sample No.	Binder %	Thickness (in.)	Weight (g)		Vol. (cm ³)	G _{mb}	G _{mm}	% Binder by Vol.	Voids (Percent)			Unit Wt lb/ft ³	Unit Wt kg/m ³	Stability		Flow 0.01"	
			In Air	SSD					In Water	Total Mix	Filled			VMA	Chart		Converted
A	B	C	D	R	E	F	G	H	I	J	L	L	M	N	O	P	Q
(R-E) (D/F) (B*G)/Gb 100-100(G/H) (L-J)/L M*16.0 G*62.4 M*16.0																	
A	5.5	2.58	1208.1	1211.5	693.7	517.8	2.333								5027	4784	7.9
B	5.5	2.59	1208.1	1211.4	693.5	517.9	2.333								4714	4463	10
C	5.5	2.59	1202	1207	689.8	517.2	2.324								4411	4176	9
Average of 5.5% Binder Specimens																	
A	6	2.51	1205	1206	699	507	2.377	2.503	12.4	6.92	61.3	17.9	145.4	2329		4474	9
B	6	2.51	1196.5	1198	694	504	2.374								4918	4887	11
C	6	2.51	1193.7	1195.4	691.3	504.1	2.368								4513	4484	12.1
Average of 6.0% Binder Specimens																	
A	6.5	2.48	1191.2	1192	694.6	497.4	2.395	2.484	13.8	4.49	73.2	16.8	148.1	2372		4637	12
B	6.5	2.51	1199.2	1200.2	697.6	502.6	2.386								4425	4482	13.4
C	6.5	2.49	1195.6	1196.7	693.8	502.9	2.377								4191	4164	14.5
Average of 6.5% Binder Specimens																	
A	7	2.46	1191.6	1192.3	696.1	496.2	2.401	2.466	15	3.24	80.7	16.8	148.9	2385		4217	13.3
B	7	2.51	1210.8	1211.1	706	505.1	2.397								3605	3697	15.1
C	7	2.47	1197.8	1198.2	699.5	498.7	2.402								3567	3544	15.4
Average of 7.0% Binder Specimens																	
						2.4	2.448	16.3	1.95	88.3	16.7	149.8	2399		3644	3714	16
Average of 7.0% Binder Specimens																	
						2.4	2.448	16.3	1.95	88.3	16.7	149.8	2399			3652	15.5

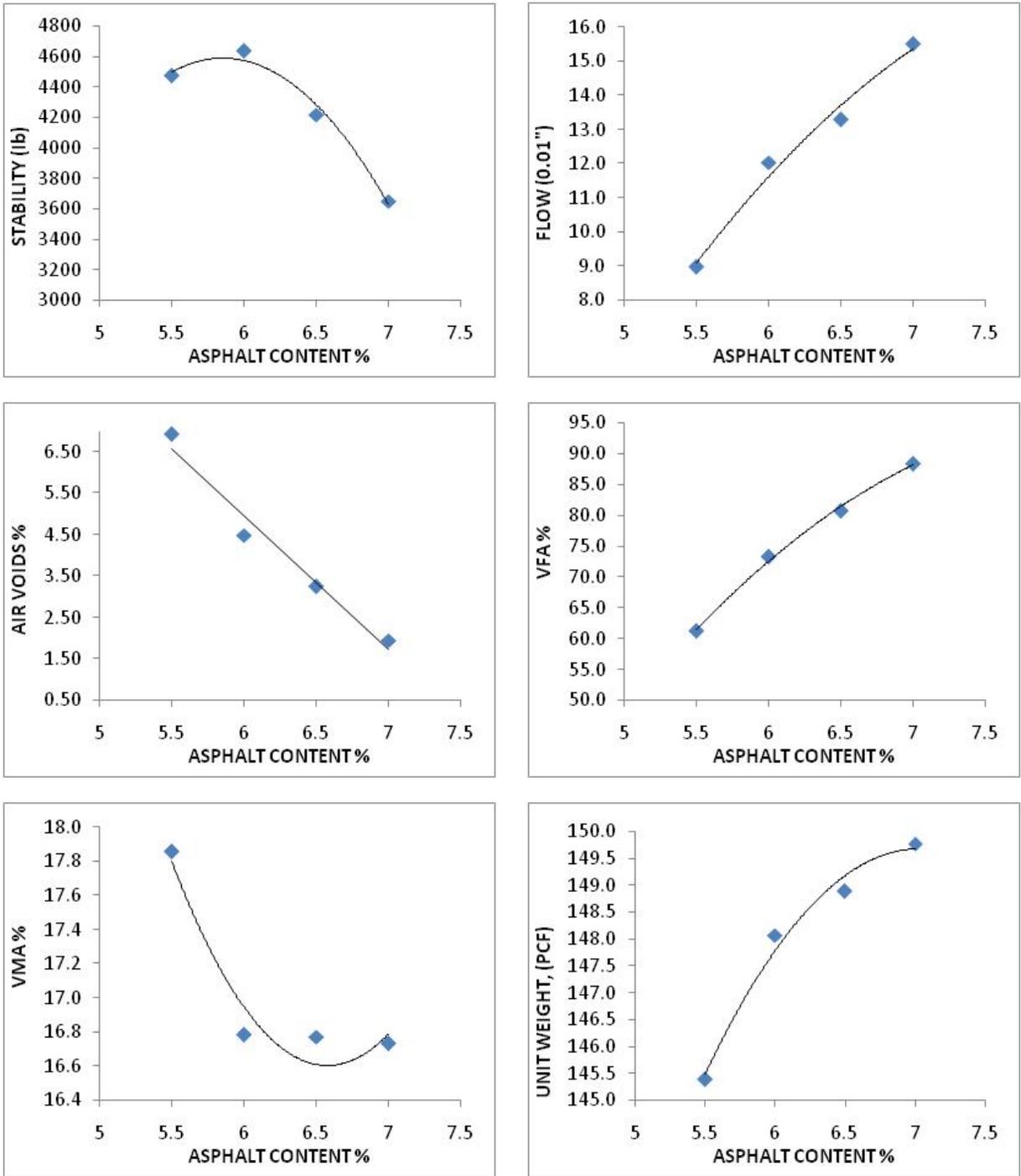


Figure B-8: Marshall Mix Design Plots
for Mixes Containing Crushed Limestone and PG 70-22M.

APPENDIX C
AASHTO T 283 TEST RESULTS

1- HMA Prepared Using Natural Gravel and PG 64-22

Table C.1: Air Voids Content of Compacted Specimens.

Specimen ID	1	2	3	4	5	6
No. of Blows	18	18	18	18	18	18
A Wt. in Air (grams)	1200.8	1196.3	1195	1197	1196.6	1201.1
B Wt. of SSD (grams)	1206.3	1200.7	1199.5	1202	1204.1	1205.8
C Wt. in Water (grams)	672	668.2	662.5	669.2	666.3	671.7
Gmb Bulk S.G. A/(B-C)	2.247	2.247	2.225	2.247	2.225	2.249
Gmm Rice S.G	2.405	2.405	2.405	2.405	2.405	2.405
VTM A.V. % (1-(Gmb/Gmm))	6.6%	6.6%	7.5%	6.6%	7.5%	6.5%
Average A.V. %	6.9%			6.9%		

Specimens 1, 2, and 3 will be wet conditioned while specimens 4, 5, and 6 will be tested in the dry condition.

Table C.2: Tensile Strength Ratio.

Condition	Wet (1, 2, 3)			Dry (4, 5, 6)		
A Wt. in Air (grams)	1200.8	1196.3	1195	1197	1196.6	1201.1
B Wt. of SSD (grams)	1206.3	1200.7	1199.5	1202	1204.1	1205.8
C Wt. in Water (grams)	672	668.2	662.5	669.2	666.3	671.7
Gmb Bulk S.G. A/(B-C)	2.247	2.247	2.225	2.247	2.225	2.249
Weight (Partial Sat.) (grams)	1228.9	1222.2	1225.9	N/A		
Thickness (in.)	2.67	2.68	2.69	2.7	2.71	2.68
Volume of Air Voids (cm ³)	36.0	36.4	41.4	36.6	41.8	35.8
% Saturation	78.0%	71.2%	74.7%	N/A		
Load (lbs)	1900	1970	1852	2219	2208	2152
Tensile Strength (psi)	113.3	117.0	109.6	130.8	129.7	127.8
Tensile Strength Ratio (%)	87.5%					

2- HMA Prepared Using Natural Gravel and PG 70-22M

Table C.3: Air Voids Content of Compacted Specimens.

Specimen ID	1	2	3	4	5	6
No. of Blows	20	20	20	20	20	20
A Wt. in Air (grams)	1191.2	1197.9	1195.7	1194.3	1196.7	1201.5
B Wt. of SSD (grams)	1196.2	1202.6	1202.6	1198.4	1203.9	1205.4
C Wt. in Water (grams)	662.3	669.1	667.4	665.2	669.9	669.1
Gmb Bulk S.G. A/(B-C)	2.231	2.245	2.234	2.240	2.241	2.240
Gmm Rice S.G	2.407	2.407	2.407	2.407	2.407	2.407
VTM A.V. % (1-(Gmb/Gmm))	7.3%	6.7%	7.2%	6.9%	6.9%	6.9%
Average A.V. %	7.1%			6.9%		

Specimens 1, 2, and 3 will be wet conditioned while specimens 4, 5, and 6 will be tested in the dry condition.

Table C.4: Tensile Strength Ratio.

Condition	Wet (1, 2, 3)			Dry (4, 5, 6)		
A Wt. in Air (grams)	1191.2	1197.9	1195.7	1193.2	1196.7	1201.5
B Wt. of SSD (grams)	1196.2	1202.6	1202.6	1199.4	1203.9	1205.4
C Wt. in Water (grams)	662.3	669.1	667.4	666.3	669.9	669.1
Gmb Bulk S.G. A/(B-C)	2.231	2.245	2.234	2.238	2.241	2.240
Weight (Partial Sat.) (grams)	1220.9	1224.1	1224.5	N/A		
Thickness (in.)	2.69	2.7	2.69	2.69	2.68	2.7
Volume of Air Voids (cm ³)	40.5	37.3	39.8	38.5	38.1	38.5
% Saturation	73.4%	70.2%	72.4%	N/A		
Load (lbs)	2361	2350	2527	2789	3013	2975
Tensile Strength (psi)	139.7	138.5	149.5	165.0	178.9	175.4
Tensile Strength Ratio (%)	82.4%					

3- WMA Prepared Using Natural Gravel and PG 64-22

Table C.5: Air Voids Content of Compacted Specimens.

Specimen ID	1	2	3	4	5	6
No. of Blows	13	13	13	13	13	13
A Wt. in Air (grams)	1202.5	1198.7	1192.6	1204.9	1193.2	1209.4
B Wt. of SSD (grams)	1208.2	1203.3	1198.8	1210.1	1198.6	1215.4
C Wt. in Water (grams)	666.6	666.5	663.3	668.6	663.1	673.7
Gmb Bulk S.G. A/(B-C)	2.220	2.233	2.227	2.225	2.228	2.233
Gmm Rice S.G	2.396	2.396	2.396	2.396	2.396	2.396
VTM A.V. % (1-(Gmb/Gmm))	7.3%	6.8%	7.1%	7.1%	7.0%	6.8%
Average A.V. %	7.1%			7.0%		

Specimens 1, 2, and 3 will be wet conditioned while specimens 4, 5, and 6 will be tested in the dry condition.

Table C.6: Tensile Strength Ratio.

Condition	Wet (1, 2, 3)			Dry (4, 5, 6)		
A Wt. in Air (grams)	1202.5	1198.7	1192.6	1204.9	1193.2	1209.4
B Wt. of SSD (grams)	1208.2	1203.3	1198.8	1210.1	1198.6	1215.4
C Wt. in Water (grams)	666.6	666.5	663.3	668.6	663.1	673.7
Gmb Bulk S.G. A/(B-C)	2.220	2.233	2.227	2.225	2.228	2.233
Weight (Partial Sat.) (grams)	1234	1226.4	1221.3	N/A		
Thickness (in.)	2.72	2.71	2.71	2.73	2.7	2.73
Volume of Air Voids (cm ³)	41.1	38.0	39.3	40.1	38.9	38.3
% Saturation	76.7%	73.0%	72.9%	N/A		
Load (lbs)	1673	1743	1531	1953	1958	1826
Tensile Strength (psi)	97.9	102.4	89.9	113.9	115.4	106.5
Tensile Strength Ratio (%)	86.4%					

4- WMA Prepared Using Natural Gravel and PG 70-22M

Table C.7: Air Voids Content of Compacted Specimens.

Specimen ID	1	2	3	4	5	6
No. of Blows	15	15	15	15	15	15
A Wt. in Air (grams)	1197.1	1205.9	1205.1	1191	1199	1187.3
B Wt. of SSD (grams)	1204	1211.2	1210.1	1195.1	1204	1196.2
C Wt. in Water (grams)	668.8	670.8	670.1	662.6	668.4	662
Gmb Bulk S.G. A/(B-C)	2.237	2.231	2.232	2.237	2.239	2.223
Gmm Rice S.G	2.401	2.401	2.401	2.401	2.401	2.401
VTM A.V. % (1-(Gmb/Gmm))	6.8%	7.1%	7.1%	6.8%	6.8%	7.4%
Average A.V. %	7.0%			7.0%		

Specimens 1, 2, and 3 will be wet conditioned while specimens 4, 5, and 6 will be tested in the dry condition.

Table C.8: Tensile Strength Ratio.

Condition	Wet (1, 2, 3)			Dry (4, 5, 6)		
A Wt. in Air (grams)	1197.1	1205.9	1205.1	1191	1199	1187.3
B Wt. of SSD (grams)	1204	1211.2	1210.1	1195.1	1204	1196.2
C Wt. in Water (grams)	668.8	670.8	670.1	662.6	668.4	662
Gmb Bulk S.G. A/(B-C)	2.237	2.231	2.232	2.237	2.239	2.223
Weight (Partial Sat.) (grams)	1223.8	1233.9	1233.3	N/A		
Thickness (in.)	2.69	2.71	2.72	2.67	2.69	2.69
Volume of Air Voids (cm³)	37.9	39.4	39.5	37.6	37.5	41.2
% Saturation	70.5%	71.1%	71.4%	N/A		
Load (lbs)	2736	2680	2739	3195	3066	2887
Tensile Strength (psi)	161.9	157.4	160.3	190.4	181.4	170.8
Tensile Strength Ratio (%)	88.4%					

5- HMA Prepared Using Crushed Limestone and PG 64-22

Table C.9: Air Voids Content of Compacted Specimens.

Specimen ID	1	2	3	4	5	6
No. of Blows	18	18	18	18	18	18
A Wt. in Air (grams)	1208	1206.5	1205.6	1210.2	1208.1	1201.8
B Wt. of SSD (grams)	1212.6	1210.5	1209.1	1214.3	1213.3	1204.2
C Wt. in Water (grams)	687.2	686.1	685.8	688.4	687.8	683
Gmb Bulk S.G. A/(B-C)	2.299	2.301	2.304	2.301	2.299	2.306
Gmm Rice S.G	2.472	2.472	2.472	2.472	2.472	2.472
VTM A.V. % (1-(Gmb/Gmm))	7.0%	6.9%	6.8%	6.9%	7.0%	6.7%
Average A.V. %	6.9%			6.9%		

Specimens 1, 2, and 3 will be wet conditioned while specimens 4, 5, and 6 will be tested in the dry condition.

Table C.10: Tensile Strength Ratio.

Condition	Wet (1, 2, 3)			Dry (4, 5, 6)		
A Wt. in Air (grams)	1208	1206.5	1205.6	1193.2	1208.1	1201.8
B Wt. of SSD (grams)	1212.6	1210.5	1209.1	1199.4	1213.3	1204.2
C Wt. in Water (grams)	687.2	686.1	685.8	666.3	687.8	683
Gmb Bulk S.G. A/(B-C)	2.299	2.301	2.304	2.238	2.299	2.306
Weight (Partial Sat.) (grams)	1238.4	1236.5	1235.1	N/A		
Thickness (in.)	2.66	2.65	2.65	2.65	2.65	2.61
Volume of Air Voids (cm ³)	38.3	37.8	37.1	37.7	38.2	36.1
% Saturation	79.4%	79.3%	79.5%	N/A		
Load (lbs)	1991	2182	2120	2647	2773	2664
Tensile Strength (psi)	119.1	131.0	127.3	159.0	166.5	162.4
Tensile Strength Ratio (%)	77.4%					

6- HMA Prepared Using Crushed Limestone and PG 70-22M

Table C.11: Air Voids Content of Compacted Specimens.

Specimen ID	1	2	3	4	5	6
No. of Blows	18	18	18	18	18	18
A Wt. in Air (grams)	1203.1	1207.3	1204.4	1197.3	1202.4	1203.3
B Wt. of SSD (grams)	1207.6	1212.9	1208.2	1202.1	1206.8	1207.2
C Wt. in Water (grams)	683.5	686.1	683.9	678.6	684	684.1
Gmb Bulk S.G. A/(B-C)	2.296	2.292	2.297	2.287	2.300	2.300
Gmm Rice S.G	2.466	2.466	2.466	2.466	2.466	2.466
VTM A.V. % (1-(Gmb/Gmm))	6.9%	7.1%	6.8%	7.3%	6.7%	6.7%
Average A.V. %	6.9%			6.9%		

Specimens 1, 2, and 3 will be wet conditioned while specimens 4, 5, and 6 will be tested in the dry condition.

Table C.12: Tensile Strength Ratio.

Condition	Wet (1, 2, 3)			Dry (4, 5, 6)		
A Wt. in Air (grams)	1203.1	1207.3	1204.4	1193.2	1202.4	1203.3
B Wt. of SSD (grams)	1207.6	1212.9	1208.2	1199.4	1206.8	1207.2
C Wt. in Water (grams)	683.5	686.1	683.9	666.3	684	684.1
Gmb Bulk S.G. A/(B-C)	2.296	2.292	2.297	2.238	2.300	2.300
Weight (Partial Sat.) (grams)	1232.6	1236.9	1232	N/A		
Thickness (in.)	2.64	2.66	2.63	2.64	2.62	2.63
Volume of Air Voids (cm ³)	37.6	38.7	37.1	39.4	36.3	36.4
% Saturation	78.5%	76.5%	74.4%	N/A		
Load (lbs)	2226	2276	2380	2874	2926	2962
Tensile Strength (psi)	134.2	136.2	144.0	173.3	177.7	179.2
Tensile Strength Ratio (%)	78.2%					

7- WMA Prepared Using Crushed Limestone and PG 64-22

Table C.13: Air Voids Content of Compacted Specimens.

Specimen ID	1	2	3	4	5	6
No. of Blows	10	10	10	10	10	10
A Wt. in Air (grams)	1202.6	1204.1	1204.7	1207.8	1199.9	1203.7
B Wt. of SSD (grams)	1208.7	1209.4	1210.5	1213.9	1207.2	1208.3
C Wt. in Water (grams)	682.3	683	683.4	687	681.3	681.9
Gmb Bulk S.G. A/(B-C)	2.285	2.287	2.286	2.292	2.282	2.287
Gmm Rice S.G	2.461	2.461	2.461	2.461	2.461	2.461
VTM A.V. % (1-(Gmb/Gmm))	7.2%	7.1%	7.1%	6.9%	7.3%	7.1%
Average A.V. %	7.1%			7.1%		

Specimens 1, 2, and 3 will be wet conditioned while specimens 4, 5, and 6 will be tested in the dry condition.

Table C.14: Tensile Strength Ratio.

Condition	Wet (1, 2, 3)			Dry (4, 5, 6)		
A Wt. in Air (grams)	1202.6	1204.1	1204.7	1193.2	1199.9	1203.7
B Wt. of SSD (grams)	1208.7	1209.4	1210.5	1199.4	1207.2	1208.3
C Wt. in Water (grams)	682.3	683	683.4	687	681.3	681.9
Gmb Bulk S.G. A/(B-C)	2.285	2.287	2.286	2.329	2.282	2.287
Weight (Partial Sat.) (grams)	1231.6	1232.2	1233.1	N/A		
Thickness (in.)	2.65	2.67	2.67	2.68	2.66	2.66
Volume of Air Voids (cm ³)	39.1	38.8	39.2	37.8	39.9	38.8
% Saturation	74.1%	72.5%	72.4%	N/A		
Load (lbs)	1583	1427	1529	1929	2030	2098
Tensile Strength (psi)	95.1	85.1	91.1	114.6	121.5	125.5
Tensile Strength Ratio (%)	75.0%					

8- WMA Prepared Using Crushed Limestone and PG 70-22M

Table C.15: Air Voids Content of Compacted Specimens.

Specimen ID	1	2	3	4	5	6
No. of Blows	9	9	9	9	9	9
A Wt. in Air (grams)	1195.5	1199.6	1193.5	1199.4	1195.6	1201.7
B Wt. of SSD (grams)	1198.8	1203.3	1198.6	1203.3	1198.9	1205.7
C Wt. in Water (grams)	676.9	678	675.2	680.3	676	679.1
Gmb Bulk S.G. A/(B-C)	2.291	2.284	2.280	2.293	2.286	2.282
Gmm Rice S.G	2.459	2.459	2.459	2.459	2.459	2.459
VTM A.V. % (1-(Gmb/Gmm))	6.8%	7.1%	7.3%	6.7%	7.0%	7.2%
Average A.V. %	7.1%			7.0%		

Specimens 1, 2, and 3 will be wet conditioned while specimens 4, 5, and 6 will be tested in the dry condition.

Table C.16: Tensile Strength Ratio.

Condition	Wet (1, 2, 3)			Dry (4, 5, 6)		
A Wt. in Air (grams)	1195.5	1199.6	1193.5	1193.2	1195.6	1201.7
B Wt. of SSD (grams)	1198.8	1203.3	1198.6	1199.4	1198.9	1205.7
C Wt. in Water (grams)	676.9	678	675.2	680.3	676	679.1
Gmb Bulk S.G. A/(B-C)	2.291	2.284	2.280	2.299	2.286	2.282
Weight (Partial Sat.) (grams)	1221.6	1228.3	1221.5	N/A		
Thickness (in.)	2.64	2.66	2.65	2.64	2.65	2.66
Volume of Air Voids (cm ³)	37.2	39.1	39.7	36.6	38.3	39.4
% Saturation	70.1%	73.5%	70.6%	N/A		
Load (lbs)	1776	1770	1713	2477	2427	2403
Tensile Strength (psi)	107.1	105.9	102.9	149.3	145.8	143.8
Tensile Strength Ratio (%)	72.0%					

APPENDIX D
ASPHALT PAVEMENT ANALYZER (APA) TEST RESULTS

Table D-1: APA Test Results for HMA Mixes Prepared Using Gravel and PG 64-22.

Measurements Taken at the Specified Loading Cycles (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.7849	0.7874	0.8074	0.7929	0.6574	0.6664	0.6629	0.6529
B	0.8239	0.8749	0.8679	0.8339	0.6944	0.7439	0.7289	0.6844
C	0.7689	0.7874	0.7994	0.7689	0.6694	0.6704	0.6474	0.6059
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.6319	0.6399	0.6189	0.6164	0.4465	0.4665	0.4275	0.4330
B	0.6549	0.7039	0.6894	0.6389	0.4870	0.5270	0.4425	0.4450
C	0.6309	0.6334	0.6014	0.5629	0.5135	0.4965	0.4340	0.4010

Rut Depth at Each Slot (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.0000	0.0000	0.0000	0.0000	0.1275	0.1210	0.1445	0.1400
B	0.0000	0.0000	0.0000	0.0000	0.1295	0.1310	0.1390	0.1495
C	0.0000	0.0000	0.0000	0.0000	0.0995	0.1170	0.1520	0.1630
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.1530	0.1475	0.1885	0.1765	0.3384	0.3209	0.3799	0.3599
B	0.1690	0.1710	0.1785	0.1950	0.3369	0.3479	0.4254	0.3889
C	0.1380	0.1540	0.1980	0.2060	0.2554	0.2909	0.3654	0.3679

Average Rut Depth at Each Cycle (in.)					
Specimen	5	500	1000	8000	Avg. Rut Depth @ 8000 Cycles
A	0.0000	0.1333	0.1664	0.3498	0.3482
B	0.0000	0.1373	0.1784	0.3748	
C	0.0000	0.1329	0.1740	0.3199	

Table D-2: APA Test Results for HMA Mixes Prepared Using Gravel and PG 70-22M.

Measurements Taken at the Specified Loading Cycles (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.3380	0.3470	0.3230	0.3175	0.2570	0.2455	0.2365	0.2200
B	0.3290	0.3250	0.3350	0.3015	0.2530	0.2640	0.2350	0.2110
C	0.3695	0.3570	0.3885	0.3745	0.2990	0.3215	0.2980	0.2870
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.2315	0.2075	0.2055	0.1965	0.1075	0.0890	0.0690	0.0610
B	0.2250	0.2245	0.1980	0.1780	0.0795	0.0845	0.0465	0.0405
C	0.2785	0.2890	0.2740	0.2590	0.1440	0.1835	0.1545	0.1450

Rut Depth at Each Slot (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.0000	0.0000	0.0000	0.0000	0.0810	0.1015	0.0865	0.0975
B	0.0000	0.0000	0.0000	0.0000	0.0760	0.0610	0.1000	0.0905
C	0.0000	0.0000	0.0000	0.0000	0.0705	0.0355	0.0905	0.0875
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.1065	0.1395	0.1175	0.1210	0.2305	0.2580	0.2540	0.2565
B	0.1040	0.1005	0.1370	0.1235	0.2495	0.2405	0.2885	0.2610
C	0.0910	0.0680	0.1145	0.1155	0.2255	0.1735	0.2340	0.2295

Average Rut Depth at Each Cycle (in.)					
Specimen	5	500	1000	8000	Avg. Rut Depth @ 8000 Cycles
A	0.0000	0.0916	0.1211	0.2498	0.2418
B	0.0000	0.0819	0.1163	0.2599	
C	0.0000	0.0710	0.0973	0.2156	

Table D-3: APA Test Results for WMA Mixes Prepared Using Gravel and PG 64-22.

Measurements Taken at the Specified Loading Cycles (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.7914	0.8099	0.7794	0.7479	0.5794	0.5919	0.5584	0.5179
B	0.7584	0.7854	0.7824	0.7574	0.5579	0.5394	0.5679	0.5604
C	0.8069	0.8244	0.8454	0.8049	0.5994	0.5959	0.6409	0.6004
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.5214	0.5294	0.4904	0.4955	0.1305	0.1815	0.1135	0.1740
B	0.4974	0.4915	0.5189	0.5034	0.1890	0.1335	0.1830	0.2330
C	0.5409	0.5304	0.5629	0.5449	0.1930	0.1760	0.2650	0.3195

Rut Depth at Each Slot (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.0000	0.0000	0.0000	0.0000	0.2120	0.2180	0.2210	0.2300
B	0.0000	0.0000	0.0000	0.0000	0.2005	0.2460	0.2145	0.1970
C	0.0000	0.0000	0.0000	0.0000	0.2075	0.2285	0.2045	0.2045
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.2700	0.2805	0.2890	0.2524	0.6609	0.6284	0.6659	0.5739
B	0.2610	0.2939	0.2635	0.2540	0.5694	0.6519	0.5994	0.5244
C	0.2660	0.2940	0.2825	0.2600	0.6139	0.6484	0.5804	0.4854

Average Rut Depth at Each Cycle (in.)					
Specimen	5	500	1000	8000	Avg. Rut Depth @ 8000 Cycles
A	0.0000	0.2203	0.2730	0.6323	0.6002
B	0.0000	0.2145	0.2681	0.5863	
C	0.0000	0.2113	0.2756	0.5821	

Table D-4: APA Test Results for WMA Mixes Prepared Using Gravel and PG 70-22M.

Measurements Taken at the Specified Loading Cycles (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.3695	0.3835	0.3725	0.3685	0.2765	0.2705	0.2950	0.2805
B	0.2985	0.3175	0.3160	0.2795	0.2200	0.2630	0.2390	0.2075
C	0.3165	0.3170	0.3000	0.3205	0.2425	0.2425	0.2325	0.2400
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.2480	0.2300	0.2535	0.2430	0.1280	0.1150	0.1195	0.1070
B	0.1960	0.2280	0.2005	0.1745	0.0700	0.0645	0.0400	0.0400
C	0.2050	0.2080	0.2040	0.2250	0.0595	0.0635	0.0790	0.0700

Rut Depth at Each Slot (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.0000	0.0000	0.0000	0.0000	0.0930	0.1130	0.0775	0.0880
B	0.0000	0.0000	0.0000	0.0000	0.0785	0.0545	0.0770	0.0720
C	0.0000	0.0000	0.0000	0.0000	0.0740	0.0745	0.0675	0.0805
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.1215	0.1535	0.1190	0.1255	0.2415	0.2685	0.2530	0.2615
B	0.1025	0.0895	0.1155	0.1050	0.2285	0.2530	0.2760	0.2395
C	0.1115	0.1090	0.0960	0.0955	0.2570	0.2535	0.2210	0.2505

Average Rut Depth at Each Cycle (in.)					
Specimen	5	500	1000	8000	Avg. Rut Depth @ 8000 Cycles
A	0.0000	0.0929	0.1299	0.2561	0.2503
B	0.0000	0.0705	0.1031	0.2493	
C	0.0000	0.0741	0.1030	0.2455	

Table D-5: APA Test Results for HMA Mixes Prepared Using Limestone and PG 64-22.

Measurements Taken at the Specified Loading Cycles (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.3305	0.3465	0.3460	0.3470	0.2665	0.2945	0.2870	0.2635
B	0.3125	0.3255	0.3215	0.3255	0.2675	0.2725	0.2630	0.2615
C	0.3815	0.4025	0.3805	0.3875	0.3175	0.3420	0.3325	0.3280
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.2385	0.2685	0.2575	0.2345	0.1180	0.1370	0.1435	0.1155
B	0.2400	0.2400	0.2240	0.2345	0.1070	0.0810	0.0845	0.0995
C	0.2920	0.3125	0.2970	0.3000	0.1720	0.1780	0.1655	0.1795

Rut Depth at Each Slot (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.0000	0.0000	0.0000	0.0000	0.0640	0.0520	0.0590	0.0835
B	0.0000	0.0000	0.0000	0.0000	0.0450	0.0530	0.0585	0.0640
C	0.0000	0.0000	0.0000	0.0000	0.0640	0.0605	0.0480	0.0595
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.0920	0.0780	0.0885	0.1125	0.2125	0.2095	0.2025	0.2315
B	0.0725	0.0855	0.0975	0.0910	0.2055	0.2445	0.2371	0.2260
C	0.0895	0.0900	0.0835	0.0875	0.2095	0.2245	0.2150	0.2080

Average Rut Depth at Each Cycle (in.)					
Specimen	5	500	1000	8000	Avg. Ruth Depth @ 8000 Cycles
A	0.0000	0.0646	0.0928	0.2140	0.2188
B	0.0000	0.0551	0.0866	0.2283	
C	0.0000	0.0580	0.0876	0.2143	

Table D-6: APA Test Results for HMA Mixes Prepared Using Limestone and PG 70-22M.

Measurements Taken at the Specified Loading Cycles (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.3325	0.3345	0.3340	0.3185	0.3120	0.2990	0.3055	0.2995
B	0.2880	0.3060	0.2815	0.3060	0.2555	0.2545	0.2540	0.2825
C	0.3785	0.3880	0.3625	0.3750	0.3475	0.3515	0.3485	0.3280
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.3020	0.2915	0.2945	0.2965	0.2405	0.2085	0.2425	0.2110
B	0.2480	0.2485	0.2400	0.2695	0.1855	0.1890	0.1610	0.1770
C	0.3400	0.3405	0.3310	0.3140	0.2880	0.2805	0.2475	0.2295

Rut Depth at Each Slot (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.0000	0.0000	0.0000	0.0000	0.0205	0.0355	0.0285	0.0190
B	0.0000	0.0000	0.0000	0.0000	0.0325	0.0515	0.0275	0.0235
C	0.0000	0.0000	0.0000	0.0000	0.0310	0.0365	0.0140	0.0470
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.0305	0.0430	0.0395	0.0220	0.0920	0.1260	0.0915	0.1075
B	0.0400	0.0575	0.0415	0.0365	0.1025	0.1170	0.1205	0.1290
C	0.0385	0.0475	0.0315	0.0610	0.0905	0.1075	0.1150	0.1455

Average Rut Depth at Each Cycle (in.)					
Specimen	5	500	1000	8000	Avg. Rut Depth @ 8000 Cycles
A	0.0000	0.0259	0.0338	0.1043	0.112
B	0.0000	0.0338	0.0439	0.1173	
C	0.0000	0.0321	0.0446	0.1146	

Table D-7: APA Test Results for WMA Mixes Prepared Using Limestone and PG 64-22.

Measurements Taken at the Specified Loading Cycles (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.7594	0.7644	0.7789	0.7454	0.6839	0.6699	0.6664	0.6689
B	0.7259	0.7454	0.7554	0.7219	0.6234	0.6554	0.6374	0.6364
C	0.8114	0.8174	0.7764	0.7854	0.7074	0.7149	0.6839	0.6909
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.6389	0.6149	0.6124	0.5984	0.4785	0.4390	0.4400	0.4575
B	0.5809	0.6049	0.5879	0.5879	0.4585	0.4065	0.4245	0.4405
C	0.6619	0.6769	0.6529	0.6524	0.5174	0.5129	0.5074	0.5009

Rut Depth at Each Slot (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.0000	0.0000	0.0000	0.0000	0.0755	0.0945	0.1125	0.0765
B	0.0000	0.0000	0.0000	0.0000	0.1025	0.0900	0.1180	0.0855
C	0.0000	0.0000	0.0000	0.0000	0.1040	0.1025	0.0925	0.0945
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.1205	0.1495	0.1665	0.1470	0.2809	0.3254	0.3389	0.2879
B	0.1450	0.1405	0.1675	0.1340	0.2674	0.3389	0.3309	0.2814
C	0.1495	0.1405	0.1235	0.1330	0.2940	0.3045	0.2690	0.2845

Average Rut Depth at Each Cycle (in.)					
Specimen	5	500	1000	8000	Avg. Rut Depth @ 8000 Cycles
A	0.0000	0.0898	0.1459	0.3083	0.3003
B	0.0000	0.0990	0.1468	0.3047	
C	0.0000	0.0984	0.1366	0.2880	

Table D-8: APA Test Results for WMA Mixes Prepared Using Limestone and PG 70-22M.

Measurements Taken at the Specified Loading Cycles (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.2880	0.2920	0.3050	0.2980	0.2310	0.2285	0.2360	0.2365
B	0.2390	0.2520	0.2740	0.2620	0.1880	0.1990	0.2235	0.2065
C	0.3480	0.3400	0.3150	0.2990	0.3005	0.2825	0.2670	0.2525
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.2220	0.2110	0.2150	0.2145	0.1450	0.1200	0.1130	0.1145
B	0.1730	0.1790	0.2035	0.1905	0.0700	0.0770	0.0935	0.0915
C	0.2505	0.2700	0.2565	0.2380	0.1845	0.1835	0.1695	0.1640

Rut Depth at Each Slot (in.)								
Specimen	Cycle							
	5				500			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.0000	0.0000	0.0000	0.0000	0.0570	0.0635	0.0690	0.0615
B	0.0000	0.0000	0.0000	0.0000	0.0510	0.0530	0.0505	0.0555
C	0.0000	0.0000	0.0000	0.0000	0.0475	0.0575	0.0480	0.0465
	1000				8000			
	Slot 1	Slot 2	Slot 3	Slot 4	Slot 1	Slot 2	Slot 3	Slot 4
A	0.0660	0.0810	0.0900	0.0835	0.1430	0.1720	0.1920	0.1835
B	0.0660	0.0730	0.0705	0.0715	0.1690	0.1750	0.1805	0.1705
C	0.0975	0.0700	0.0585	0.0610	0.1635	0.1565	0.1455	0.1350

Average Rut Depth at Each Cycle (in.)					
Specimen	5	500	1000	8000	Avg. Rut Depth @ 8000 Cycles
A	0.0000	0.0628	0.0801	0.1726	0.1655
B	0.0000	0.0525	0.0703	0.1738	
C	0.0000	0.0499	0.0718	0.1501	