PERFORMANCE EVALUATION OF FOAMED WARM MIX ASPHALT PRODUCED BY WATER INJECTION

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PERFORMANCE EVALUATION OF FOAMED WARM MIX ASPHALT PRODUCED BY WATER INJECTION

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Dissertation

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ABSTRACT

In recent years, a new group of technologies has been introduced in the United States that allow producing asphalt mixtures at temperatures 30 to 100°F lower than what is used in traditional hot mix asphalt (HMA). These technologies are commonly referred to as Warm Mix Asphalt (WMA). From among these technologies, foamed WMA produced by water injection has gained increased attention from the asphalt paving industry in Ohio since it does not require the use of costly additives. This type of asphalt mixtures is advertised as an environmentally friendly alternative to traditional HMA and promoted to have better workability and compactability. In spite of these advantages, several concerns have been raised regarding the performance of foamed WMA because of the reduced production temperature and its impact on aggregate drying and asphalt binder aging. Main concerns include increased propensity for moisture-induced damage (durability) and increased susceptibility to permanent deformation (rutting). Other concerns include insufficient coating of coarse aggregates, and applicability of HMA mix design procedures to foamed WMA mixtures.

This dissertation presents the results of a comprehensive study conducted to evaluate the laboratory performance of foamed WMA mixtures with regard to permanent deformation, moisture-induced damage, fatigue cracking, and low-temperature (thermal) cracking; and compare it to traditional HMA. In addition, the workability of foamed WMA
and HMA mixtures was evaluated using a new device that was designed and fabricated at the University of Akron, and the compactability of both mixtures was examined by analyzing compaction data collected using the Superpave gyratory compactor. The effect of the temperature reduction, foaming water content, and aggregate moisture content on the performance of foamed WMA was also investigated. Furthermore, the rutting performance of plant-produced foamed WMA and HMA mixtures was evaluated in the Accelerated Pavement Load Facility (APLF) at Ohio University, and the long-term performance of pavement structures constructed using foamed WMA and HMA surface and intermediate courses was analyzed using the Mechanistic-Empirical Pavement Design Guide (MEPDG).

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

- In general, comparable laboratory test results were obtained for foamed WMA and HMA mixtures prepared using 30°F (16.7°C) temperature reduction, 1.8% foaming water content, and fully dried aggregates. Therefore, the performance of the resulting foamed WMA is expected to be similar to that of the HMA.

- Surface foamed WMA mixtures had comparable rutting performance in the APLF to that of the HMA mixtures. This was also the case for intermediate foamed WMA and HMA mixtures. These results indicate the field performance of the foamed WMA mixtures is similar to that of the HMA mixtures.
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CHAPTER I

INTRODUCTION

1.1 Problem Statement

Hot mix asphalt (HMA) is the most commonly used material for asphalt paving applications in Ohio. It consists of aggregates and asphalt binder. It is produced by drying the aggregates prior to mixing with the heated asphalt binder. The temperature at which this material is produced generally ranges from 300°F to 325°F (148.9°C and 162.7°C) for unmodified asphalt binders, and even higher temperatures are used for modified asphalt binders. The use of such temperatures ensures that the aggregate is completely dry and thoroughly coated with a thin film of asphalt binder. It also ensures that the mix is workable and compactable to an acceptable density in the field, resulting in a mixture that is durable and capable of withstanding repeated loading from traffic.

In recent years, a new group of technologies has been introduced in the United States that allows for the production of asphalt mixtures at temperatures 30°F to 100°F (16.7°C to 55.6°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 alternatives to traditional HMA mixtures as they produce lower greenhouse gas emissions (15 to 45% less than HMA). 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 asphalt binder under controlled temperature and pressure conditions, resulting in so-called foamed asphalt binder.

Warm mix asphalt prepared using foamed asphalt binders, henceforth referred to as foamed WMA, has gained increased attention from the asphalt paving industry in Ohio since it does not require the use of costly additives. Other advantages to the asphalt paving industry include reduced energy consumption due to lower production temperatures; increased hauling distance since warm mix asphalts are able to retain their temperatures for a longer period of time; improved conditions for construction workers due to lower odor, fume, and emission levels; and improved compactability and the ability to reach the desired density with fewer number of roller passes. Another potential advantage for asphalt paving contractors in Ohio and other northern states includes the possibility of extending the paving season beyond what is commonly enforced by the corresponding state departments of transportation.

In spite of the above-mentioned advantages of foamed WMA, several concerns have been raised regarding its performance because of the reduced production temperature and its impact on aggregate drying and asphalt binder aging. Main concerns include (1) increased propensity for moisture-induced damage since water is used during production and aggregates are heated to lower temperatures and therefore may not dry thoroughly before being mixed with the asphalt binder; and (2) increased susceptibility to permanent deformation (or rutting) since the asphalt binder may not harden as much at lower production temperatures and may easily deform even with proper compaction in the field.
Other concerns include (3) insufficient coating of coarse aggregates, and (4) applicability of HMA mix design procedures to foamed WMA mixtures.

In summary, there is a consensus among researchers and practitioners that WMA is a viable technology. However, several questions need to be answered regarding the performance of this material and the process involved in its production before it can be used as an alternative to HMA. These questions include:

- Are WMA mixes more susceptible to permanent deformation (rutting) and moisture-induced damage?
- What effect does insufficient aggregate drying have on the potential for moisture-induced damage?
- What impact does the asphalt foaming process have on mix design and how can the mix design be improved?
- Are aggregates thoroughly coated in WMA mixes; and what impact does insufficient aggregate coating have on mix durability?
- Since WMA mixes are more workable and compactable than HMA mixes, should they be compacted to a higher density level in the field?

To answer the first question, the Ohio Department of Transportation (ODOT) has recently contracted with the University of Akron to compare the performance of HMA and foamed WMA mixtures with regard to permanent deformation and moisture-induced damage. Two aggregates (gravel and limestone) and two asphalt binders (PG 64-22 and PG 70-22) were used in that study. The aggregate gradation met ODOT CMS Item 441 Type 1 Surface Course for Medium Traffic. The resistance to permanent deformation was
measured using the Asphalt Pavement Analyzer (APA) and the Simple Performance Test (SPT); and the resistance to moisture-induced damage was measured using AASHTO T283. Preliminary results from this study have shown a slight increase in rut depth and a slight reduction in tensile strength ratio (TSR) in the case of WMA. By comparing the peak load levels obtained in AASHTO T283, it was noticed though that WMA mixes prepared using limestone aggregates had relatively lower indirect tensile strength values than HMA mixes for both unconditioned (dry) and conditioned specimens. The WMA to HMA peak load level ratio ranged from 72 to 75% for the unmodified asphalt binder and 76 to 84% for the modified asphalt binder. In the case of gravel, the peak load level ratio was almost the same for both WMA and HMA mixes.

Based on the previous discussion, additional research is needed to evaluate the impact of insufficient aggregate drying, inadequate aggregate coating, and reduced binder aging on the performance and durability of foamed WMA mixtures. In addition, current mix design methods and specifications used by ODOT for foamed WMA mixtures shall be validated or revised to ensure satisfactory long-term performance.

1.2 Study Objectives

The main objective of this study was to develop a comprehensive design methodology for foamed warm mix asphalt that ensures acceptable performance in the field against rutting, cracking, and moisture damage for the conditions pertinent to Ohio. The specific objectives of this project include:

- Evaluate the factors that affect the volumetric properties, performance, and durability of foamed WMA mixtures.
• Determine the limitations of WMA mixtures produced using the foaming process by water injection.

• Identify changes to current mix design and evaluation procedures, if any, that will be required for foamed WMA mixtures.

• Evaluate current ODOT quality control and placement procedures to determine applicability to foamed WMA mixtures.

1.3 Dissertation Organization

This Dissertation is organized into five main components (Figure 1.1):

• The first component provides a comparison between the laboratory performance of foamed WMA and HMA mixtures with regard to permanent deformation (rutting), moisture-induced damage (durability), fatigue cracking, and low-temperature (thermal) cracking. Chapter III describes the asphalt binders and aggregate materials used in the preparation of the foamed WMA and HMA mixtures, followed by a discussion of the mix design procedure and the methods used to produce the foamed WMA in the laboratory. Chapter IV details the laboratory testing program that was implemented to evaluate the performance of both mixtures along with the testing plans used for other components. The laboratory test results are presented in Chapter V.

• The second component focuses on the evaluation of the workability and compactability of foamed WMA and HMA mixtures. Chapter VI details the design and operation of a new device that was developed to evaluate the workability of the foamed WMA and HMA mixtures, and presents the results obtained from this device. In addition, this chapter provides a comparison between the compactability of foamed WMA and HMA mixtures.
mixtures based on compaction data collected using the Superpave gyratory compactor during the preparation of the test specimens for the various laboratory tests included in Chapter IV.

- The third component discusses the effect of the mix preparation procedure on the performance of foamed WMA mixtures. Chapter VII investigates the effect of production temperature, foaming water content, and aggregate moisture content on the performance of foamed WMA.

- The fourth component evaluates the performance of foamed WMA and HMA mixtures in the Accelerated Pavement Load Facility (APLF) at Ohio University. Chapter VIII provides an overview of the pavement structure, material information, testing procedure, and APLF test results. In addition, this chapter presents a comparison between the rut depth measurements obtained using the APLF and APA test results obtained for field cores, plant-produced/laboratory-compacted, and laboratory-produced/laboratory-compacted specimens.

- The fifth component investigates the long-term performance of pavement structures constructed using foamed WMA and HMA surface and intermediate courses using the Mechanistic-Empirical Pavement Design Guide (MEPDG). Chapter IX presents the baseline pavement structures used in the analysis and the resulting performance predictions. The material properties for the surface and intermediate courses are defined using the dynamic modulus test results presented in Chapter V. The analysis is repeated using unconditioned and conditioned (dry and wet) dynamic moduli to
evaluate the effect of sample conditioning (freezing and thawing) on pavement performance.

Performance Evaluation of Foamed WMA and HMA in the Laboratory:
- Chapter III: Material Information and Production of Foamed WMA
- Chapter IV: Laboratory Testing Program
- Chapter V: Laboratory Test Results and Discussion

Workability and Compactability of Foamed WMA and HMA:
- Chapter VI: Workability and Compactability of Foamed WMA and HMA Mixtures

Effect of Mix Preparation Procedure on Foamed WMA:
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Figure 1.1: Dissertation Organization.
CHAPTER II

LITERATURE REVIEW

2.1 Introduction

Warm Mix Asphalt (WMA) is a generic term that is generally used to described technologies that allow producing asphalt mixtures at temperatures lower than those used in the traditional Hot Mix Asphalt (HMA). The use of WMA technologies, as means to reduce energy consumption and pollutant emissions, has gained significant interest due to the increasing energy costs, global warming, and the more stringent environmental regulations. While heat is used to reduce asphalt binder viscosity and to dry aggregates during mixing of traditional HMA, WMA reduces the viscosity of asphalt binders by either introducing certain organic/chemical additives or through injecting water into the hot asphalt binder. This reduction in viscosity facilitates adequate aggregate coating during mixing. The reduction in mixture viscosity also improves its workability and allows for mix compaction at lower temperatures.

Despite the advantages of WMA technologies, several concerns might arise due to the use of lower production temperatures to produce WMA mixtures. For instance, the rutting potential of WMA mixtures might increase due to the less asphalt binder aging. The use of low production temperatures might also result in an insufficient aggregate drying which might increase the potential for moisture induced damage as the bond between the
aggregates and the asphalt binder is compromised. A literature review pertinent to these WMA distresses as well as other distresses is presented in this chapter. Particularly, the following sections provide a brief description of the commercially available WMA technologies. They also detail the results and observations found in the literature regarding the laboratory and field performance of WMA mixtures with focus on WMA produced using foamed asphalt binders, referred to herein as foamed WMA. In addition, the literature pertinent to the influence of the asphalt foaming characteristics and WMA mix preparation parameters on the performance of those mixtures is also included in this chapter.

2.2 Commercially Available WMA Technologies

In recent years there has been an increase in the number of WMA technologies that can be used to prepare asphalt mixtures. These technologies can be mainly classified into two main categories; according to the type of additive or production method. These categories are WMA produced using organic/chemical additives and WMA produced through foaming the asphalt. The following subsections provide a summarized description of the various WMA technologies currently available in the market.

2.2.1 WMA Produced Using Organic/Chemical Additives

**Sasobit**

Sasobit is one of the organic additives that are blended with the asphalt binder to facilitate producing asphalt mixture at lower than traditional temperatures. It is a synthetic wax that is produced through the coal gasification process. The blending of Sasobit with the asphalt binder eases the mixing of asphalt mixtures at lower temperatures. The main
reason for that is because Sasobit reduces the viscosity of the asphalt binder at high temperatures (i.e. above its melting point of 239°F). Furthermore, Sasobit forms a crystalline structure at ambient temperatures within the asphalt binder; thus, it’s use might improve the mixtures’ fatigue cracking resistance.

The optimum Sasobit dosage, as recommended by Sasobit’s manufacturer (Sasolwax), ranges between 3 to 4 percent by weight of asphalt binder. This amount is expected to allow for producing asphalt mixtures at temperatures that are 15°F to 50°F lower than what is used for producing HMA mixtures. In addition, Sasolwax does not recommend introducing Sasobit directly into the asphalt mixture as this might result in an inhomogeneous distribution of Sasobit within the mix. The need for pre-blending Sasobit with asphalt binder along with the cost of Sasobit might result in increasing the overall cost of mixtures produced using this technology.

Evotherm

MeadWestvaco, the manufacturer of Evotherm, has developed three different types of Evotherm. These types include: Evotherm Emulsion Technology (ET), Evotherm Dispersed Asphalt Technology (DAT), and Evotherm Third Generation (3G/Revix). The mechanism by which the Evotherm ET additive facilitates producing WMA mixtures, is completed through introducing a water-based emulsion in the hot aggregates during mixing. Upon contact with the hot aggregates, the water-based emulsion turns into steam; thus, causing the asphalt binder to foam. The production of the water-based emulsion, according to MeadWestvaco, involves using a chemical package. This chemical package contains necessary additives that are expected to enhance coating, adhesion, and
workability of WMA mixtures produced using this technology. Evotherm DAT technology is similar to Evotherm ET in utilizing a water-based emulsion in producing WMA mixtures. However, instead of introducing the water-based emulsion into the mixture, it is directly injected into the asphalt binder line just before the asphalt binder enters the mixing chamber.

On the contrary to the previous Evotherm technologies, Evotherm 3G/Revix utilizes a water-free chemical additive package that does not reduce the viscosity of the asphalt binder. This chemical additive only works on reducing the internal friction of the mixture; thus, allowing the asphalt binder to behave as if it was heated to a high temperature. Similar to Evotherm DAT technology, the 3G technology can be directly injected into the asphalt binder line just before the asphalt enter the mixture. The Evotherm 3G technology can also be pre-blended with the asphalt binder at the mixing plant.

The optimum dosage of any of the Evotherm technologies ranges between 0.4 to 0.7% by total weight of asphalt binder. The use of this dosage is expected to facilitate producing WMA mixtures at about 50 to 100°F lower than the corresponding HMA mixtures produced using the same asphalt binder.

Rediset LQ

Rediset LQ is another chemical additive that allows producing WMA mixtures at lower than traditional temperatures. The mechanism by which this additive facilitates production of WMA mixtures is highly dependent on the surfactants contained in it. These surfactants reduce the surface tension of the asphalt binder; enabling efficient aggregate
coating at lower than traditional temperatures. This process is also believed to improve the workability of asphalt mixtures and making them more compactable at lower temperatures.

Similar to Evotherm additives, Rediset LQ can be pre-blended with the asphalt binder or it can be directly injected into the asphalt binder just before the binder is introduced into the mixing chamber. The optimum dosage of Rediset LQ ranges between 0.25 to 0.75 percent by weight of effective asphalt binder content. Generally, dosages within this range do not change the PG grade of the asphalt binder and allow for producing WMA mixtures at temperatures 40 to 60°F lower than what is traditionally used for producing HMA mixtures.

SonneWarmix

SonneWarmix is a wax-based WMA additive that is composed of paraffinic hydrocarbons. As in all the previous WMA additives discussed, SonneWarmix can be pre-blended with the asphalt binder at the binder terminal or directly introduced into the liquid binder stream at the suction pump while utilizing the pump to do the required mixing. The mechanism by which this additive allows for producing WMA mixtures is similar to that of Sasobit. In particular, blending this additive with the asphalt binder helps in reducing the binder’s viscosity at temperatures above the melting point of the added wax.

The optimum dosage rate of SonneWarmix, as recommended by the manufacturer (Sonneborn Inc.), ranges between 0.5 to 1.5 percent by total weight of asphalt binder. The use of this dosage rate, as reported by Sonneborn Inc., does not change the binder performance grade (PG) and is expected to facilitate producing WMA mixtures at temperatures 50°F less than what is typically used for HMA mixtures.
2.2.2 WMA Produced Using Foaming Processes

Aspha-min

Aspha-min is a WMA additive that is used to foam asphalt binders during the mixing stage. It is a synthetic zeolite that contains approximately 20 percent of crystallized water within its structure. Aspha-min is usually introduced during the mixing process shortly after or at the same time the asphalt binder is added into the mixing chamber. As the temperature of Aspha-min gradually increases, the water contained inside its structures starts to release in the form of steam causing the asphalt binder to foam. As a result of this foaming process, the viscosity of the asphalt binder is reduced; thus, facilitating the use of lower than traditional production temperatures.

The optimum Aspha-min dosage, as recommended by its manufacturer (Eurovia Services GmbH Germany), is around 0.3 percent by total weight of the mixture. Furthermore, to ensure uniform distribution of Aspha-min within the mixture, it is recommended to use a specially built distribution unit that can be attached to the mixing plant (Barthel and Bon Devivere, 2003). Finally, the use of Aspha-min is expected to facilitate producing WMA mixtures at about 50°F lower than traditional HMA mixtures.

Advera

Advera is another synthetic zeolite that contain about 18 percent of crystallized water, by total weight, inside its structure. Although Aspha-min and Advera foam asphalt binders using the same method, the difference between them lies in their particle size distribution (gradation). Advera is a fine graded product that is composed of particles
passing sieve #200 (i.e. smaller than 74 microns). The manufacturer of Advera, PQ Corporation, advocates that the use of finer graded additive would be more uniformly distributed within the mixture. This in turn might result in better foaming the asphalt binder.

Similar to Aspha-min, Advera is directly added to the pugmill of a batch plant or through a fiber port in a drum plant. However, it does not require adding a distribution unit into the plant. This might result in producing WMA mixtures that are cheaper than those produced using Aspha-min technology. In addition, the use of Advera is expected to allow production of WMA mixtures at temperatures that are 50 to 70°F lower than HMA production temperatures.

*Double Barrel Green System*

The double barrel green system, produced by Astec Inc., is a drum plant that is retrofitted with a multi-nozzle foaming device. The mechanism by which this system allows for producing WMA mixtures is similar to that used in Aspha-min and Advera technologies. However, in this system, no additives are required as the multi-nozzle foaming device directly injects the hot asphalt binder with cold water. To be more specific, as the asphalt binder is flowing through the foaming chambers in the multi-nozzle foaming device, cold water is injected into it through a series of stainless steel injectors located above these foaming chambers. The injection of cold water causes the asphalt binder to expand; thus, creating what is known as foamed asphalt.

In order to directly discharge the foamed asphalt binder into the mixing chamber, the foaming process is completed just before the binder is added. In addition, the amount
of water injected into the asphalt binder is controlled through a positive displacement piston pump which accurately measures the amount of water going into the system. Finally, it is worth mentioning that using the double barrel green system is expected to facilitate producing asphalt mixtures at temperatures that are 30°F lower than what is traditionally used for HMA mixtures.

**Gencor Green Machine Ultrafoam GX**

The Ultrafoam GX is a foaming system that utilizes water in producing WMA mixtures. This device is usually attached to the 3 or 4-inch asphalt injection line already present in drum plants. Similar to the double barrel green system, the Ultrafoam GX system injects water into the flowing asphalt binder. The system is jacketed with a hot oil jacket that would preserve the temperature of the heated asphalt binder during production. This device consists of a variable speed drive, a positive displacement water pump, an inlet strainer, a gauge, a pressure switch, a pressure relief valve, and a water flow meter.

Producing foamed asphalt using this device involves injecting a small amount of water into the flowing asphalt binder. The design of the Ultrafoam GX contains a centrally located spring-loaded water valve that opens when the pressure behind the valve is impressed. The asphalt binder is allowed to flow through a specially designed diaphragm that controls the flow of asphalt within the system. This unique design, according to the manufacturer, maintains the perfect ratio of asphalt binder and water at all production rates.
2.3 Laboratory and Field Performance of WMA

During the past few years several research studies have been conducted to evaluate the laboratory and field performance of foamed WMA mixtures. Rushing et al. (2013) assessed the applicability of using WMA mixtures on pavements to be trafficked by heavy aircraft. In their study, Rushing et al. (2013) studied the laboratory and field performance of asphalt mixtures prepared using traditional HMA procedures and eleven WMA technologies. Investigation of the performance of these mixtures with regard to rutting resistance and moisture susceptibility was conducted through the asphalt pavement analyzer (APA), tensile strength ratio (TSR, AASHTO T 283), and Hamburg wheel tracking device (HWTD) tests. Testing was conducted, in this study, on materials that was laboratory-produced laboratory-compacted, plant-produced laboratory-compacted, and plant-produced field-compacted. Based on the APA and HWTD results, Rushing et al. (2013) reported that most WMA laboratory-produced mixtures had lower rutting resistance than the HMA control mixtures. Rushing et al. (2013) also reported that all WMA laboratory-produced mixtures had lower TSR values than the control HMA mixtures; thus, indicating that WMA mixture are more susceptible to moisture induced damage than HMA mixtures.

As for plant-produced laboratory-compacted WMA mixtures, Rushing et al. (2013) reported that their APA and HWTD rutting performance results was lower than HMA mixtures. The moisture susceptibility of these mixtures; however, was reported to be similar to plant-produced HMA mixtures. Finally, Rushing et al. (2013) reported that all
plant-produced field-compacted WMA mixtures had very poor APA and HWTD rutting results.

Kim et al. (2012) conducted a study to evaluate the field and laboratory performance of WMA mixtures produced using foaming and emulsion technologies. In this study, trial pavement sections of the WMA mixtures and their counterpart HMA mixtures were implemented in Antelope County, Nebraska. Field-mixed loose mixtures, collected at the time of paving, were used to prepared samples for testing in the APA, AASHTO T 283, and Semi-Circular Bending (SCB). The results of this study indicated that APA tests under water did not show any clear moisture damage sensitivity between the mixtures. However, results from the AASHTO T 283 and SCB tests demonstrated, according to Kim et al. (2012), a similar trend between WMA and HMA mixtures. WMA mixtures showed greater susceptibility to moisture conditioning than did HMA (Kim et al., 2012). Furthermore, Kim et al. (2012) reported that the field performance data showed that both the WMA and HMA performed satisfactorily as no cracking or other failure modes were observed in the trial sections and the rut depth for both WMA and HMA sections were similar.

Bernier et al. (2012) presented the details of the first official Connecticut WMA pavement project. The project, according to Bernier et al (2012), involved three experimental sections. One section was paved with conventional HMA and two WMA sections paved using Sasobit and foamed WMA technologies. The construction was done as 50-mm overlay with 12.5 mm Superpave mix. Each mix was sampled over three day construction period for further evaluation in the laboratory. The materials were collected
in loose form and reheated and compacted in laboratory conditions. Samples were tested in Semi-Circular Bending (SCB), Hamburg Wheel Tracking, Indirect Tensile, and Disk Compact Tension [DC(T)] tests to evaluate the performance of WMA and compare it to that of HMA. The laboratory test results of this study has shown that the use of foamed asphalt reduced the rutting susceptibility at 50°C relative to HMA and had no adverse effects on the inflection point. The Sasobit WMA, as reported by Bernier et al. (2012), appeared to have increased the rutting severity; however, it extended the inflection point. Bernier et al. (2012) also concluded that the fracture energy and toughness measured in the SCB and DC(T) show no significant differences between WMA and HMA when temperature was fixed. With regard to field performance, the authors reported that Sasobit WMA field sections had the greatest amount of linear cracking followed by foamed WMA and then HMA having the least amount of linear cracking. Bernier et al. (2012) also concluded that Sasobit WMA had the highest rutting trends in the field which was found to be similar to what was observed in laboratory testing.

Hill et al. (2012) investigated the low-temperature fracture properties of several WMA technologies that included Sasobit, Evothem 3G, Advera, and Rediset LQ. The Disk-Shaped Compact Tension [DC(T)], Indirect Tension (IDT), and Acoustic Emission (AE) tests were used in this study. Hill et al. reported that the use of Evothem 3G and Rediset LQ improved the fracture energy in comparison to HMA; however, the use of Sasobit and Advera reduced the fracture energy. Furthermore, Hill et al. (2012) reported that the IDT creep compliance results lead to similar observations, where the use Evothem 3G and Rediset LQ increased mixtures’ creep compliance while Sasobit and Advera did
not significantly alter creep compliance as compared to HMA mixtures. AE testing results for Advera displayed an embrittlement temperature similar to HMA whereas Sasobit increased the embrittlement temperature. For Evotherm 3G and Rediset LQ, Hill et al. (2012) reported that their use resulted in differing embrittlement temperatures. Finally, the authors of this study concluded that thermal cracking resistance is not ensured by virtue of producing WMA mixtures at lower temperature; however, performance testing should be conducted to evaluate WMA technologies in locations susceptible to low temperatures (Hill et al., 2012).

Alavi et al. (2012) evaluated the adhesion properties and moisture-damage susceptibility of WMA mixtures prepared using different additives. These additives included; water bearing mineral (WB), solid pelletized surfactant (PS), and chemical-based viscosity reducer (VR). The impact of moisture on the bond strength at the asphalt-aggregate interface and mixtures performance was quantified using the bitumen bond strength (BBS) and dynamic modulus at multiple freeze-thaw cycles. Alavi et al. (2012) concluded, based on BBS results, that producing WMA mixtures at lower temperatures has the potential to increase moisture susceptibility because of lower adhesion between the asphalt and aggregate. Furthermore, Alavi et al. (2012) reported that, in general, all HMA mixtures exhibited higher but similar unconditioned dynamic modulus values in comparison with their corresponding WMA mixtures expect those prepared using the VR additives. Alvari et al. (2012) also reported that conditioned HMA mixtures exhibited slighted higher dynamic modulus values followed by WMA-PS and WMA-VR; whereas,
the WMA-WB had showed significant reduction in its dynamic modulus values which was mainly attributed to the moisture damage.

Arabani et al. (2011) estimated the moisture susceptibility of WMA mixtures using the surface free energy method. In this study, two aggregate types (limestone and granite) and two WMA additives (Sasobit and Aspha-min) were evaluated. Arabani et al. (2011) utilized a moisture sensitivity index, which was defined as the percentage of aggregate surface exposed to water, as an indication of moisture susceptibility of asphalt mixtures. To calculate this index, Arabani et al. (2011) reported using the surface free energy method along with the results of the dynamic modulus test. The findings of this study, according to Arabani et al. (2011), suggest that the use of Sasobit and Aspha-min WMA additives results in increasing the acid component and reducing the surface free energy base components of the asphalt binder. As a result, WMA mixtures are expected to be more susceptible to moisture damage because of the decrease in adhesion between acidic aggregates and asphalt binders modified using these WMA additives. In addition, Arabani et al. (2011) reported that the dynamic modulus wet to dry ratios for WMA mixtures were lower than those for HMA mixtures. This, as concluded by Arabani et al. (2011), suggests that the use of Sasobit and Aspha-min WMA additives results in increasing the moisture susceptibility of asphalt mixtures.

Mogawer et al. (2011) investigated the effect of WMA technologies on moisture-induced damage of mixtures and adhesion characteristics of asphalt binders. In their study, a 9.5 mm Superpave mixture designed with PG 64-22 asphalt binder was used as control. In addition, they evaluated four types of WMA technologies that included: Advera,
Evotherm, Sasobit, and SonneWarmix. Both the Hamburg wheel-tracking device (HWTD) and the bitumen bond strength (BBS) tests were conducted to evaluate moisture susceptibility and binder-aggregate bond strength respectively. The results of the HWTD, according to Mogawer et al. (2011), have indicated that the moisture resistance for all mixtures considered improved with the increase in aging time or temperature. Furthermore, Mogawer et al. (2011) reported, according to BBS results, that only WMA mixture prepared using Sasobit additive had a significant effect on the pull-off tensile strength of the binder.

Haggag et al. (2011) studied the impacts of three WMA technologies on the fatigue cracking resistance and compared it to that of HMA. The fatigue characteristics were measured using a uniaxial, cyclic, and direct tension compression test. Three WMA technologies were considered in this study that included Advera, Evotherm 3G, and Sasobit. Haggag et al. (2011) concluded that there was no significant difference in stiffness between WMA and HMA mixtures. The authors also concluded that there was no significant difference in fatigue cracking resistance of the HMA and WMA mixtures.

Liu et al. (2011) conducted a study to evaluate the laboratory and field performance of WMA mixtures produced using Sasobit. In this study, a trial field project was paved, using Sasobit WMA mixtures, in the southeastern region of Alaska. In addition to field testing, Liu et al. (2011) reported assessing the laboratory performance of considered mixtures with regard to rutting resistance, moisture susceptibility, and low temperature cracking. Research results, according to Liu et al. (2011), showed that the use of Sasobit resulted in reducing mixing and compaction temperatures, improving workability and
rutting resistance, and insignificantly affecting moisture susceptibility of WMA mixtures when compared to control HMA mixtures. Liu et al. (2011) also reported that the indirect tension test showed a decrease in the tensile strength of WMA mixtures at low temperatures.

In another study, Liu and Li (2011) examined the influence of Sasobit on low temperature performance of WMA binders and mixtures. In their study, Liu and Li (2011) utilized the bending beam rheometer (BBR), direct tension test (DDT), asphalt binder cracking device (ABCD), and indirect tension test (ITS) to characterize the performance of the selected binders and mixtures. Liu and Li (2011) reported that a decrease in tensile strength for WMA binders and mixtures at low temperatures is indicated by the testing results. In addition, the results of this study showed an increase in the cracking temperatures of WMA binders and mixtures with the increase in Sasobit dosage; however, this effect was deemed to be insignificant. Finally, Liu and Li (2011) concluded that the use of Sasobit in preparing WMA mixtures might be a suitable alternative for use in the northern region of Alaska.

Buss et al. (2011) evaluated the performance of Evotherm 3G/Revix, Sasobit, and Astec’s Double Barrel Green System WMA technologies. Plant-produced WMA mixtures and four corresponding control HMA mixtures were used to evaluate the performance of WMA with regard to moisture susceptibility. Both field-compacted and field-produced laboratory-compacted samples have been prepared for testing in the indirect tensile strength (ITS), dynamic modulus, and flow number tests. Buss et al. (2011) reported that for most of the mixtures the performance of HMA was superior to that of WMA.
Cooper et al. (2011) reported the results of a study that evaluated the laboratory performance of sulfur-modified WMA mixtures. In this study, three asphalt mixtures were prepared. These mixtures included: an HMA mixtures prepared using the neat PG 64-22 asphalt binder, an HMA mixtures prepared using the SBS PG 70-22 modified asphalt binder, and a WMA mixtures prepared using sulfur-based additive and PG 64-22 asphalt binder. According to Cooper et al. (2011), the performance of the considered mixtures was evaluated with regard to rutting resistance, moisture susceptibility, fatigue endurance and resistance, and low-temperature thermal cracking. Based on the results of the performance testing plan, Cooper et al. (2011) concluded that the rutting performance of sulfur-modified WMA mixtures was similar, if not superior, to that of HMA mixtures. Cooper et al. (2011) also reported, based on the fracture tests results, that sulfur-modified WMA mixtures were more susceptible to cracking than HMA mixtures. Finally, Cooper et al. (2011) reported that the sulfur-modified WMA mixtures had higher fracture stress than the modified HMA mixtures; as indicated by the thermal stress restrained specimen test (TSRST) results.

Xiao et al. (2010) examined the influence of anti-stripping additives on the moisture susceptibility of WMA mixtures. In their study, Xiao et al. (2010) utilized Aspha-min and Sasobit along with a hydrated lime and a liquid anti-stripping additives (ASA) in order to prepare WMA mixtures. In addition, one asphalt binder (PG 64-22) and three aggregate types were used in this study. Xiao et al. (2010) Performance evaluation plan included conducting indirect tensile strength (ITS), tensile strength ratio (TSR), flow, and toughness tests. Testing results, according to Xiao et al. (2010), indicated that the use of hydrated lime resulted in the best WMA moisture resistance. In addition, WMA mixtures prepared
using the liquid ASA in WMA mixtures performed slightly weaker than those prepared using hydrated lime. Finally, Xiao et al. (2010) reported that the ITS values for WMA mixtures containing additives were lower than the mixtures prepared without the use of WMA additives.

The National Center for Asphalt Technology (NCAT) completed a study that focused on foamed WMA mixtures. In this study, the laboratory performance of foamed 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 foamed 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 foamed 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 0.8 criterion. Based on the results of this study, it was concluded that the foamed WMA produced using Gencor Green Machine Ultrafoam GX is a promising technology.

In another study, 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 moisture susceptibility values than the WMA; however, most of the WMA
samples did meet the moisture susceptibility criteria. Hodo et al. (2009) reported similar results based on tests conducted on foamed WMA mixtures obtained from field demonstration test sections in Chattanooga, TN.

Wielinski et al. (2009) reported the results of a study where Granite Construction built two WMA paving projects from its Indio California facility (Wielinski et al. 2009). Both projects were paved with WMA produced with the free water method using the 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, Wielinski et al. (2009) suggested that conventional mix design methods could be used for WMA mixtures produced using the free water system.

Middleton et al. (2009) presented Canadian’s contractor’s experience with the implementation, production, and placement of WMA by means of the WAM-Foam and Astec’s Double Barrel Green WMA technologies. A laboratory testing program was
developed to assess the characterization and performance of four asphalt mixtures produced using the Double Barrel Green system (Middleton et al., 2009). The APA and AASHTO T 283 tests were used to evaluate the rutting and moisture-induced damage susceptibility of the foamed WMA mixtures. Middleton et al. (2009) reported that the rutting performance of the tested mixtures was satisfactory. In addition, Middleton et al. (2009) concluded that the moisture susceptibility testing via tensile strength indicated that the Double Barrel Green process did not negatively influence the moisture susceptibility of the considered mixtures.

In addition to the previous studies, various other studies have been conducted to evaluate the field and laboratory performance of WMA mixtures containing high percentages of reclaimed asphalt pavement (RAP) materials. Mogawer et al. (2012) conducted a study to examine performance and workability characteristics of high performance thin lift overlays that incorporate high RAP percentages and the use of WMA technologies. In this study, a 95 mm Superpave mixtures incorporating 0% and 4% RAP material have been used along with SonneWarmix additives to prepare WMA mixtures. Both neat (unmodified) and polymer modified asphalt binders were also used in this study. Evaluation of reflective cracking, rutting, stiffness, and workability of these mixtures was conducted using the overlay tester, Hamburg wheel tracking device (HWTD), dynamic modulus, and asphalt workability device respectively.

Dynamic modulus test results, according to Mogawer et al. (2012), have shown a reduction in mixture stiffness for both the control and 40% RAP mixtures that were prepared using the selected WMA technology. Mogawer et al. (2012) also reported that the
overlay tester results indicated, at best, that the use of the selected WMA technology could improve reflective cracking resistance. In addition, Mogawer et al. (2012) concluded that the use of the selected WMA technology in combination with polymer modified asphalt binders and/or RAP may result in reduced mixture moisture susceptibility and rutting performance as indicated by the HWTD results. Finally, Mogawer et al. (2012) reported that the incorporation of the selected WMA technology did marginally improve the workability of all mixtures tested.

Zhao et al. (2012) evaluated the laboratory performance of WMA mixtures containing high percentages of RAP. Laboratory performance with regard to rutting, moisture susceptibility, and fatigue cracking have been considered in this study. The WMA mixtures, used in this study, were produced in plants that were retrofitted with the most commonly used foaming technologies; mainly water injection methods. Both WMA and control HMA mixtures were prepared to incorporate 0% and 30% RAP materials. The asphalt pavement analyzer (APA), Hamburg wheel tracking device (HWTD), tensile strength ratio (TSR), dissipated creep strain energy (DCSE), and beam fatigue tests were conducted. Zhao et al. (2012) reported that the use of RAP significantly improved the rutting resistance of WMA mixtures. This improvement was, according to Zhao et al. (2012), more significant in WMA mixtures than it is in HMA mixtures. Zhao et al. (2012) also reported that the addition of RAP materials improved the moisture susceptibility of WMA mixtures. However, Zhao et al. (2012) reported, based on the DCSE results, that the incorporation of RAP materials slightly reduced the fatigue life of WMA mixtures.
Saragand et al. (2011) presented the results of a study that evaluated the field performance of WMA mixtures containing reclaimed asphalt pavement (RAP). WMA mixtures were produced using Aspha-min, Sasobit, and Evotherm additives and placed side-by-side to a control HMA mixtures in three test sections. In this study, according to Sargand et al. (2011), temperature and emission were monitored during the production and placement of the considered mixtures. The roughness and rutting performance of the paved sections were also evaluated throughout the project’s life. As reported by Sargand et al. (2011), emission testing results showed a significant reduction in greenhouse emissions during the production of WMA mixtures when compared to HMA mixtures. Sargand et al. (2011) also reported that the density of WMA mixtures was higher than that for HMA mixtures; even though the WMA mixtures were compacted at lower temperatures. The laboratory testing results, according to Sargand et al. (2011), showed that the WMA mixtures had higher indirect tensile strength (ITS) than HMA mixtures after the first three months of services. The HMA ITS values; however, increased with time to become higher than the WMA ITS values. Finally, Saragnad et al. (2011) reported that both WMA and HMA sections had similar international roughness index (IRI) values after 46 months of service.

Copeland et al. (2010) presented the results of a field evaluation study conducted in Florida. According to Copeland et al. (2010), a portion of State Route 11 in Deland, Florida, was milled and repaved with 45% reclaimed asphalt pavement (RAP). The RAP mixtures were produced at lower than traditional HMA temperature through utilizing the foaming WMA technology. Two high RAP content mixtures, one control HMA and one
WMA, were produced in an asphalt production plant and loose mixture samples were collected for performance evaluation. In this study, performance tests conducted included: performance grade (PG) determination of binders, dynamic modulus, and flow number. Copeland et al. (2010) reported that the PG testing results have indicated that the high RAP-WMA mixture was softer than the high RAP-HMA control mixture. Results of the flow number test, according to Copeland et al. (2010), have also confirmed the previous observation as the high RAP-WMA mixture had lower flow number value than that of the high RAP-HMA mixture. Copeland et al. (2010) also concluded, based on the dynamic modulus results, that the high RAP-WMA mix was slightly softer than the HMA control mix. Finally, Copland et al. (2010) conducted a comparison between measured dynamic modulus values and those predicted using Hirsch and Witczak models. Results of this comparison confirmed that complete blending has occurred in the high RAP-HMA mixture; however, incomplete mixing of RAP and virgin binders may have occurred in the high RAP-WMA mixture (Copeland et al., 2010).

Tao and Mallick (2009) evaluated the effects of WMA additives (i.e. Sasobit H8 and Advera Zeolite) on workability and mechanical properties of reclaimed asphalt pavement (RAP) mixtures. Mix samples were collected from a set of mixtures that included: a control mix with 100% RAP, WMA mix incorporating 100% RAP prepared using 1.5, 2.0, and 5.0% of Sasobit H8 additive, and WMA incorporating 100% RAP prepared using 0.3, 0.5, and 0.7% Advera additive. Mixtures were tested for workability and samples compacted at 50 gyrations in the Supepave gyratory compactor (SGC) were tested to measure the mixtures’ bulk specific gravity, indirect tensile strength at 0°C, and
moduli at 0, 26.7, and 50°C. Tao and Mallick (2009) have reported that the addition of Sasobit H8 or Advera WMA additives improved the workability of RAP mixtures at temperatures as low as 110°C. Tao and Mallick (2009) also reported that the use of Sasobit H8 or Advera tended to stiffen the mix as observed from the indirect tensile strength and seismic moduli test results. Finally, Tao and Mallick (2009) concluded that the increase in Sasobit H8 dosage has resulted in an increase in the bulk specific gravity of compacted mixtures while this trend is not well defined for mixtures containing different Advera dosages.

2.4 Effect of Foaming and WMA Mix Preparation Parameters

The foaming characteristics of asphalt binder were studied by previous researchers in the past. Castedo Franco and Wood (1983) found that any asphalt binder, irrespective of grade or origin, can be foamed with an appropriate combination of nozzle type, water, air and bitumen injection pressure. However, Abel (1978) found that asphalt binder which contained silicones could have reduced foaming abilities. In addition, he reported that asphalt binder with lower viscosities foamed more readily and had higher foam ratios and half-lives than asphalt binders with higher viscosities, but the use of high viscosity asphalt binders resulted in superior aggregate coating. Finally, he suggested that acceptable foaming was only achieved at temperatures above 149°C.

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 (Aspha-min® and Sasobit®), and three aggregate sources. The test results
indicated that dry ITS values were affected by the aggregate moisture and hydrated lime contents. Furthermore, the use of WMA additive did not significantly alter the dry ITS and toughness values. Xiao et al. (2009) also indicated that the deformation resistance and TSR decreased with the increase in the aggregate moisture content.

In another study, Xiao et al. (2010) evaluated the rutting resistance of WMA mixtures containing moist aggregates. This study included two aggregate moisture contents (0% and ~0.5% by weight of the dry mass of the aggregate), two lime contents (1% and 2% lime by weight of dry mass of the aggregate), and three WMA additives (Aspha-min, Sasobit, and Evotherm). The test results, according to Xiao et al. (2010), indicated that the aggregate source significantly affected the rutting resistance regardless of the WMA additive, lime content, and aggregate moisture content. Xiao et al. (2010) also found that the rut depth of mixtures containing moist aggregate generally satisfied the minimum performance criteria without the need for any additional treatment. Finally, Xiao et al. (2010) reported that WMA mixtures prepared using Sasobit exhibited best rutting resistance while WMA mixtures prepared using Aspha-min and Evotherm showed similar rutting resistance to that of control mixtures.

Fu et al. (2010) investigated the effects of asphalt binder foamability and granular material fines content on foamed asphalt mix strength behavior. The results of this study showed that asphalt binders with better foaming attributes (higher expansion ratios and longer half-lives) tend to yield mixes with higher strength. Furthermore, for the same asphalt binder, a small change in foaming parameters (asphalt temperature and foamant water ratio) can significantly alter the expansion ratio and half-life. However, this has only
a minimal effect on the mix properties. Fu et al. (2010) suggested that the use of a binder with better foaming characteristics can potentially lower the asphalt content while still achieving the same stabilization effects. In terms of the effects of fines content, their results showed that the strength consistently decreased as fines content increased. In addition, they suggested that introducing excessive fines, especially to the extent that the fines content is higher than 12 percent is detrimental to foamed asphalt mix performance and strongly discouraged in engineering practice.

Mallick et al. (2011) introduced a method to understand the effect of aggregate drying on the moisture content of HMA and WMA mixtures. In this study, two aggregates (high and low water absorption) and two asphalt binders (PG 58-28 and PG 64-28) were used to prepare asphalt mixtures containing moist aggregates. Mallick et al. (2011) concluded that the use of asphalt at a lower than conventional mixing temperature with a WMA additive, such as Sasobit, provided a convenient way to facilitate retention of moisture in asphalt mixtures. Furthermore, Mallick et al. (2011) reported that the testing of WMA mixtures prepared using moist aggregate should be conducted to ensure satisfactory performance of these mixtures.

Bennert et al. (2011) presented the results of a study conducted to investigate the influence of production temperatures and aggregate moisture content on the initial performance of WMA mixtures. In this study, the effect of production temperatures on rutting and fatigue cracking was evaluated through preparing WMA mixtures at various temperatures that are lower than what is traditionally used. In addition, Bennert et al. (2011) evaluated the stripping potential of WMA mixtures by using pre-wetted aggregate blends.
and by modifying the mixing procedure in the laboratory to simulate drum plant production of WMA mixtures. According to Bennert et al. (2011), the rutting resistance and stiffness of WMA mixtures have decreased as observed from dry Hamburg wheel tracking device and dynamic modulus test results. Furthermore, Bennert et al. (2011) reported that the fatigue cracking resistance of WMA mixtures has improved while the WMA mixtures prepared using dry aggregate and conventional HMA production temperatures were the only combinations that obtained satisfactory results when tested in accordance to the tensile strength and wet Hamburg wheel tracking device tests.

Xiao et al. (2011) conducted a laboratory investigation of moisture susceptibility and rutting resistance of WMA mixtures containing moist aggregates and prepared using foaming by water injection technologies. In this study, to aggregate moisture contents (0% and ~0.5% by weight of the dry mass of the aggregate), two lime contents (1% and 2% lime by weight of dry aggregate), one liquid anti-stripping agent, and three foaming water contents (2, 3, and 4% by weight of asphalt binder). The test results, according to Xiao et al. (2011), indicated that the aggregate source significantly affected the indirect tensile strength (ITS) and rutting resistance of these mixtures regardless of the foaming water content used. In addition, Xiao et al. (2011) reported that the ITS and rut depth values of some foamed WMA mixtures containing moist aggregates were satisfactory; however, other combinations required complete aggregate drying or additional treatment to ensure acceptable performance. Furthermore, Xiao et al. (2011) reported that the use of liquid anti-stripping agent is not recommended for use with foamed WMA mixtures that contain moist aggregates.
2.5 Summary

This chapter presented a brief description of the most common WMA technologies that are currently available in the United States. In addition, it summarized the outcome of previous research studies that have been conducted to examine the mechanical properties and performance of these materials. Through this literature search it was observed that most research studies focused on the performance of additive-based WMA technologies, and that limited research has been conducted to evaluate the performance of foamed WMA produced by water injection, which is the most currently used WMA technology in Ohio. This report presents a comprehensive study conducted to evaluate the performance of foamed WMA produced by water injection with regard to permanent deformation (or rutting), moisture-induced damage, fatigue cracking, and low temperature cracking.
CHAPTER III

MATERIAL DESCRIPTION AND MIX DESIGN

3.1 Introduction

The materials used in this study consisted of two types of aggregates and two types of asphalt binders. Using these materials, four different mixture combinations were selected to represent both surface and intermediate course mixtures. The Superpave mix design procedure was used to determine the optimum aggregate gradation and the optimum asphalt binder content for each of the material combinations selected. Although the Superpave mix design procedure was used to optimize the HMA mixtures only, similar gradations and optimum asphalt binder contents were used to prepare the foamed WMA mixtures. In addition, the procedure used for preparing foamed WMA mixtures was slightly modified to facilitate producing these mixtures at different mix preparation parameters (i.e. foaming water content, initial aggregate moisture content, and reduction in production temperatures). These modifications to the production procedures were utilized to examine the effect of the different preparation parameters on the performance of foamed WMA mixtures.

In this chapter, a discussion of the selected aggregates and asphalt binders used in this study is presented. The selected material combinations, mix design procedure and results, and the procedures used to produce foamed WMA mixtures are also presented.
3.2 Aggregates and Asphalt Binders

As previously mentioned, two types of aggregates were used in this study. The first type can be mainly described as limestone aggregates whereas the second type can be described as crushed gravel aggregates. The limestone aggregates were prepared through blending several aggregate gradations obtained from suppliers approved by the Ohio Department of Transportation (ODOT). These gradations included: #57 gradation limestone, #8 gradation limestone, limestone sand, and natural sand. Similarly, the crushed gravel aggregate was prepared through blending #8 gradation crushed gravel, #9 gradation crushed gravel, manufactured sand, and natural sand. Table 3.1 presents the percent passing, as obtained from the suppliers, for the various gradations used to prepare the limestone and crushed gravel aggregates. Table 3.2 shows the bulk specific gravities and water absorption for each gradation.

The overall shape of the limestone aggregates can be described as cubical with high angularity. The surface of these aggregates can be described as rough. The angularity and roughness in the surface is mainly attributed to the crushing of the limestone gradations used to prepare the limestone aggregate blend. Aggregates having these qualities usually tend to have good mechanical properties and as a result, they are expected to have good rutting performance. However, angular shaped aggregates tend to have more voids within their compacted structures which might require the need for using more compaction passes to achieve the desired density. Figure 3.1 shows a picture of the different aggregates gradations used to prepare the limestone aggregates.
Table 3.1: Supplier Provided Aggregate Gradations used to Blend both Limestone and Crushed Gravel Aggregates.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Limestone Aggregates</th>
<th>Crushed Gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>#57 Grad. %Passing</td>
<td>#8 Grad. %Passing</td>
</tr>
<tr>
<td>1”</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>¾”</td>
<td>88</td>
<td>100</td>
</tr>
<tr>
<td>½”</td>
<td>44</td>
<td>100</td>
</tr>
<tr>
<td>3/8”</td>
<td>18</td>
<td>92</td>
</tr>
<tr>
<td>#4</td>
<td>4</td>
<td>21</td>
</tr>
<tr>
<td>#8</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>#16</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>#30</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>#50</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>#100</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>#200</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
Table 3.2: Bulk Specific Gravities and Absorption for Aggregates used to Blend Both Limestone and Crushed Gravel Aggregates.

<table>
<thead>
<tr>
<th>Aggregate Type</th>
<th>Blending Gradations</th>
<th>Bulk Dry Gravity</th>
<th>SSD Gravity</th>
<th>Absorption %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone</td>
<td>#57</td>
<td>2.607</td>
<td>2.649</td>
<td>1.50</td>
</tr>
<tr>
<td></td>
<td>#8</td>
<td>2.579</td>
<td>2.636</td>
<td>2.00</td>
</tr>
<tr>
<td></td>
<td>Limestone Sand</td>
<td>2.611</td>
<td>2.600</td>
<td>1.50</td>
</tr>
<tr>
<td></td>
<td>Natural Sand</td>
<td>2.569</td>
<td>2.580</td>
<td>2.20</td>
</tr>
<tr>
<td>Crushed Gravel</td>
<td>#8</td>
<td>2.524</td>
<td>2.567</td>
<td>2.31</td>
</tr>
<tr>
<td></td>
<td>#9</td>
<td>2.447</td>
<td>2.535</td>
<td>3.60</td>
</tr>
<tr>
<td></td>
<td>Manufactured Sand</td>
<td>2.588</td>
<td>2.623</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Natural Sand</td>
<td>2.609</td>
<td>2.633</td>
<td>0.95</td>
</tr>
</tbody>
</table>
The overall shape of the limestone aggregates can be described as cubical with high angularity. The surface of these aggregates can be described as rough. The angularity and roughness in the surface is mainly attributed to the crushing of the limestone gradations used to prepare the limestone aggregate blend. Aggregates having these qualities usually tend to have good mechanical properties and as a result, they are expected to have good rutting performance. However, angular shaped aggregates tend to have more voids within their compacted structures which might require the need for using more compaction passes to achieve the desired density. Figure 3.1 shows a picture of the different aggregates gradations used to prepare the limestone aggregates.

As for the crushed gravel aggregates, their overall shape can be described as cubical with some angularity. The overall surface texture of these aggregates can be described as a smooth texture with rough crushed faces. Asphalt mixtures prepared using aggregates that have these characteristics generally tend to have slightly better workability and require less compaction effort to achieve the desired density. However, these mixtures tend to be more susceptible to rutting under the repeated traffic loading. Figure 3.2 shows a picture of the different gradations used to blend the gravel aggregates used in this study.

In addition to the two aggregate types, two asphalt binders, commonly used in preparing asphalt mixtures in Ohio, were selected to prepare both the HMA and foamed WMA mixtures used in this study. The asphalt binders selected were PG 64-28 and the polymer modified PG 70-22M. These binders were obtained from suppliers approved by ODOT; hence, their properties have been tested by the suppliers. Table 3.3 presents the testing
results as reported by the suppliers. Table 3.3 also shows the typical HMA mixing and compaction temperatures reported for each binder.

Figure 3.1: Picture of Gradations used to Blend Limestone Aggregates.

Figure 3.2: Picture of Gradations used to Blend Crushed Gravel Aggregates.

In addition to the two aggregate types, two asphalt binders, commonly used in preparing asphalt mixtures in Ohio, were selected to prepare both the HMA and foamed
WMA mixtures used in this study. The asphalt binders selected were PG 64-28 and the polymer modified PG 70-22M. These binders were obtained from suppliers approved by ODOT; hence, their properties have been tested by the suppliers. Table 3.3 presents the testing results as reported by the suppliers. Table 3.3 also shows the typical HMA mixing and compaction temperatures reported for each binder.

Table 3.3: Supplier Provided Asphalt Binder Properties.

<table>
<thead>
<tr>
<th>Binder Property</th>
<th>Binder Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PG 64-28</td>
</tr>
<tr>
<td>Specific Gravity @ 60°F</td>
<td>1.033</td>
</tr>
<tr>
<td>Specific Gravity @ 77°F</td>
<td>1.029</td>
</tr>
<tr>
<td>Rotation Viscosity @ 275°F, cP</td>
<td>425</td>
</tr>
<tr>
<td>Rotation Viscosity @ 329°F, cP</td>
<td>130</td>
</tr>
<tr>
<td>Mixing Temp. Range, ºF</td>
<td>299 (Min) 307 (Max)</td>
</tr>
<tr>
<td>Compaction Temp. Range, ºF</td>
<td>279 (Min) 286 (Max)</td>
</tr>
</tbody>
</table>

As can be seen from table 3.3, the mixing and compaction temperatures for the PG70-22M asphalt binder are slightly higher than those for PG 64-28. This is expected because the performance grade of the former cover a wider range of service temperatures than the latter. It should be noted; however, that the mixing and compaction temperatures for the PG 70-22M were not based on the viscosity-temperature relationship, but rather on laboratory and field experience using this asphalt binder. Therefore, in this research study, HMA mixtures containing PG 70-22M were prepared using mixing and compaction temperature of approximately 315°F and 300°F (157ºC and 149ºC), respectively. Furthermore, HMA mixtures prepared using PG 64-28 asphalt binder were mixed and compacted at 305°F and 290°F (155ºC and 143ºC), respectively. These particular
temperatures were selected due to the good aggregate coating observed when preparing asphalt mixtures at these temperatures.

Table 3.3 also shows the two typically reported specific gravity values of the selected asphalt binders. The specific gravity of asphalt binder at 60°F (16°C) was used in to perform all the necessary weight-volume analysis. In particular, this values was used to calculate the effective specific gravity of the aggregates which was then used to estimate the Rice specific gravity of the asphalt mixture at different asphalt binder contents.

### 3.3 HMA Mix Design

Hot asphalt mixtures are usually designed using two standard methods known as Marshall Mix Design and Superpave Mix Design. These methods are used to optimize the aggregate gradation and the asphalt binder content of asphalt-aggregate mixtures. In Ohio, the Superpave method is used for heavy traffic roads whereas the Marshall method is used for medium to low traffic roads. Recently ODOT has permitted the use of foamed WMA and other WMA mixtures on roads subjected to heavy traffic. However, since there is no standard procedure to design these mixtures, ODOT allowed Ohio contractors to use the same aggregate gradation and optimum binder content determined for HMA when preparing foamed WMA mixtures.

In this study, the Superpave mix design method was used to optimize the asphalt binder contents for four material combinations prepared using the aggregates and asphalt binders discussed in the previous section. The combinations selected, as shown in figure 3.3, include: (1) surface course mixture having 12.5 mm nominal maximum aggregate size (NMAS) limestone and prepared using PG 70-22M, (2) surface course mixture having 12.5
mm NMAS crushed gravel and prepared using PG 70-22M, (3) intermediate course mixture having 19.0 mm NMAS limestone and prepared using PG 64-28, and (4) intermediate course mixture having 19.0 mm NMAS and prepared using PG 70-22M. These combinations were selected to examine different types of asphalt mixtures that include surface course mixtures (i.e. 12.5 NMAS) and intermediate course mixtures (i.e. 19.0 mm NMAS). In addition, the combinations were selected to evaluate the effect of different aggregate types, different aggregate sizes, and different asphalt binder grades on the performance of HMA and foamed WMA mixtures. The mix design of the selected material combinations was completed in accordance with ODOT’s Construction and Material Specifications (C&MS) Item 442 Type A. Table 3.4 shows the required gyration levels and table 3.5 presents the control points defining the aggregate gradation requirements for both surface and intermediate mixtures.

Figure 3.3: Selected Material Combinations.
Table 3.4: Gyration Level Specified by ODOT for Type A Mix Design (after ODOT’s C&MS).

<table>
<thead>
<tr>
<th>Gyration Level</th>
<th>Mixture Size</th>
<th>Surface (12.5 NMAS)</th>
<th>Intermediate (19.0 NMAS)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N&lt;sub&gt;ini&lt;/sub&gt;</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>N&lt;sub&gt;des&lt;/sub&gt;</td>
<td>65</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td>N&lt;sub&gt;max&lt;/sub&gt;</td>
<td>105</td>
<td>105</td>
</tr>
</tbody>
</table>

Table 3.5: Aggregate Gradation Requirements for Type A Mix Design (after ODOT’s C&MS).

<table>
<thead>
<tr>
<th>Sieve Size in. (mm.)</th>
<th>Mixture Size</th>
<th>Surface (12.5 mm NMAS)</th>
<th>Intermediate (19.0 mm NMAS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 1/2 (37.5)</td>
<td>–</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>3/4 (19)</td>
<td>100</td>
<td>85 to 100</td>
<td></td>
</tr>
<tr>
<td>1/2 (12.5)</td>
<td>95 to 100</td>
<td>90 max</td>
<td></td>
</tr>
<tr>
<td>3/8 (9.5)</td>
<td>96 max</td>
<td>–</td>
<td></td>
</tr>
<tr>
<td>No. 4 (4.75)</td>
<td>52 min</td>
<td>–</td>
<td></td>
</tr>
<tr>
<td>No. 8 (2.36)</td>
<td>34 to 45</td>
<td>28 to 45</td>
<td></td>
</tr>
<tr>
<td>No. 200 (75 micron)</td>
<td>2 to 8</td>
<td>2 to 6</td>
<td></td>
</tr>
</tbody>
</table>

3.3.1 Selection of Material Combinations and Aggregate Gradations

To select aggregate gradations that are representative of field produced mixtures, several mix designs were obtained from “The Shelly Company” for the limestone mixes and “Shelly and Sands, Inc.” for the crushed gravel mixes. Tables 3.6 and 3.7 show the proportions of the different aggregates used for blending the limestone and crushed gravel aggregates respectively. As can be seen from table 3.6, proportions from mix 3 and mix 6 were selected for preparing limestone surface and intermediate course mixtures. These mixes were selected because they did not incorporate any reclaimed asphalt pavement...
(RAP) material. In addition, table 3.7 shows that the proportions for mix 3 were used to prepare the crushed gravel surface mixture aggregate blend. However, since this mixture included 10% RAP materials, the contractor (Shelly and Sands, Inc.) suggested replacing these 10% RAP by adding 6% of coarse aggregates (CG #8 and CG #9 gradations) and 4% of fine aggregates (Man. Sand and Nat. Sand). The modified proportions are shown in the adjusted mix 3 column of table 3.7.

Table 3.6: Blending Proportions for Limestone Aggregates.

<table>
<thead>
<tr>
<th></th>
<th>Mix 1</th>
<th>Mix 2</th>
<th>Mix 3</th>
<th>Mix 4</th>
<th>Mix 5</th>
<th>Mix 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>NMAS (mm)</td>
<td>12.5</td>
<td>12.5</td>
<td>12.5</td>
<td>19</td>
<td>19</td>
<td>19</td>
</tr>
<tr>
<td>LS #57</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>26%</td>
<td>35%</td>
<td>37%</td>
</tr>
<tr>
<td>LS #8</td>
<td>47%</td>
<td>50%</td>
<td>55%</td>
<td>19%</td>
<td>10%</td>
<td>23%</td>
</tr>
<tr>
<td>LS Sand</td>
<td>23%</td>
<td>18%</td>
<td>30%</td>
<td>---</td>
<td>15%</td>
<td>25%</td>
</tr>
<tr>
<td>Man. Sand</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>13%</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Nat. Sand</td>
<td>15%</td>
<td>17%</td>
<td>15%</td>
<td>12%</td>
<td>15%</td>
<td>15%</td>
</tr>
<tr>
<td>RAP</td>
<td>15%</td>
<td>15%</td>
<td>15%</td>
<td>12%</td>
<td>15%</td>
<td>15%</td>
</tr>
<tr>
<td>Binder</td>
<td>PG 70-22</td>
<td>PG 70-22</td>
<td>PG 70-22</td>
<td>PG 64-28</td>
<td>PG 64-28</td>
<td>PG 64-28</td>
</tr>
<tr>
<td>Binder Cont.</td>
<td>5.8%</td>
<td>5.7%</td>
<td>5.7%</td>
<td>4.4%</td>
<td>4.8%</td>
<td>4.7%</td>
</tr>
</tbody>
</table>

The resulting aggregate blends, after blending the various limestone and crushed gravel gradations at the selected proportions, are presented in figure 3.4. As can be noticed from this figure, the intermediate mixtures prepared using PG 64-28 and PG 70-22M asphalt binders had the same aggregate gradation. This is because ODOT does not allow the use of PG 70-22M asphalt binder for intermediate course mixtures; thus, this material combination was never used by contractors. The selection of this combination in this study;
however, was to facilitate evaluating the effect of the binder grade on the performance of HMA and foamed WMA mixtures.

Table 3.7: Blending Proportions for Crushed Gravel Aggregates.

<table>
<thead>
<tr>
<th></th>
<th>Mix 1</th>
<th>Mix 2</th>
<th>Mix 3</th>
<th>Mix 3 Adj.</th>
</tr>
</thead>
<tbody>
<tr>
<td>NMAS (mm)</td>
<td>12.5</td>
<td>12.5</td>
<td>12.5</td>
<td>12.5</td>
</tr>
<tr>
<td>CG #8</td>
<td>28%</td>
<td>46%</td>
<td>48%</td>
<td>51%</td>
</tr>
<tr>
<td>CG #9</td>
<td>---</td>
<td>---</td>
<td>16%</td>
<td>19%</td>
</tr>
<tr>
<td>LS #8</td>
<td>28%</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>LS Sand</td>
<td>19%</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Man. Sand</td>
<td>---</td>
<td>18%+16%</td>
<td>13%</td>
<td>15%</td>
</tr>
<tr>
<td>Nat. Sand</td>
<td>15%</td>
<td>10%</td>
<td>13%</td>
<td>15%</td>
</tr>
<tr>
<td>RAP</td>
<td>10%</td>
<td>10%</td>
<td>10%</td>
<td>0%</td>
</tr>
<tr>
<td>Binder</td>
<td>PG 70-22</td>
<td>PG 70-22</td>
<td>PG 70-22</td>
<td>PG 70-22</td>
</tr>
<tr>
<td>Binder Cont.</td>
<td>5.7%</td>
<td>5.7%</td>
<td>5.7%</td>
<td>5.7%</td>
</tr>
</tbody>
</table>

3.3.2 Selection of Optimum Asphalt Binder Content

As previously mentioned, the Superpave mix design was used in this study to determine the optimum asphalt binder content for the selected material combinations. The general procedure used is summarized as follows:

- Select three different asphalt binder contents. In this study, 5.2, 5.7, and 6.1% were used for surface course mixtures (12.5 NMAS) whereas 4.2, 4.7, and 5.2% were used for intermediate course mixtures (19.0 NMAS).

- Heat both the aggregate batches and the asphalt binder for two hours at the mixing temperature (i.e. 305°F for PG 64-28 and 315°F for PG 70-22M).
Figure 3.4: Aggregate Gradations for (a) Surface Mixtures and (b) Intermediate Mixtures.
- Mix the aggregate batches with the asphalt binder. The weight of each batch should be sufficient to prepare two specimens at each asphalt binder content selected.

- Place the loose mixture in a flat pan and spread it over the whole area of the pan. Then, place the pan in an oven for two hours at the compaction temperature (i.e. 290°F for PG 64-28 and 300°F of PG 70-22M) for curing.

- Compact specimens, each weighing about 4500g, using the Superpave Gyratory Compactor (SGC) at the desired compaction level. In this study, specimens were compacted at 105 gyrations ($N_{\text{max}}$). It is also noted that one pan of loose mixture was kept to conduct the Rice specific gravity ($G_{\text{mm}}$) test.

- Allow compacted and loose mixture specimens to cool at room temperature.

- Conduct the Bulk specific gravity on the compacted samples. The AASHTO T 116 standards were used in this study.

- Conduct the Rice specific gravity on the loose mixture samples. The AASHTO T 209 standard were used in this study.

- Analyze the results and select the optimum asphalt binder content as the binder content that would result in 4.0% air void.

Table 3.8 presents a summary of the mix design results obtained for the four material combinations used in this study. As can be seen from this table, the optimum asphalt binder content determined by the researchers was similar to that reported by the contractors. In addition, the intermediate course mixture prepared using PG 70-22M asphalt binder had an optimum asphalt binder of 4.6%. This binder content is relatively
comparable to that reported by the contractor for intermediate course mixture prepared using PG 64-28. As mentioned previously, although this combination is not used by contractors is was included to facilitate comparisons of the effect of the asphalt binder.

Table 3.8: Summary of Mix Design Results.

<table>
<thead>
<tr>
<th>Aggregate Type</th>
<th>Limestone</th>
<th>Limestone</th>
<th>Limestone</th>
<th>Crushed Gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix Type</td>
<td>Surface</td>
<td>Intermediate</td>
<td>Intermediate</td>
<td>Surface</td>
</tr>
<tr>
<td>NMAS (mm)</td>
<td>12.5</td>
<td>19.0</td>
<td>19.0</td>
<td>12.5</td>
</tr>
<tr>
<td>Binder Type</td>
<td>PG 70-22</td>
<td>PG 70-22</td>
<td>PG 64-28</td>
<td>PG 70-22</td>
</tr>
<tr>
<td>AC% (Contractor)</td>
<td>5.7%</td>
<td>N/A</td>
<td>4.7%</td>
<td>5.7%</td>
</tr>
<tr>
<td>AC% (Research Team)</td>
<td>5.7%</td>
<td>4.6%</td>
<td>4.7%</td>
<td>5.8%</td>
</tr>
</tbody>
</table>

3.4 Production of Foamed WMA Mixtures

The procedure used to produce foamed WMA mixtures in the laboratory depends on the availability of a device that is capable of foaming asphalt binders. As a result, a laboratory foaming device, produced by Wirtgen (Figure 3.5), was utilized to foam the asphalt binders used in this study. In general, this device foams asphalt binders through injecting them with cool pressurized water. The water will turn into steam, upon contact with hot asphalt, causing the asphalt binder to foam and expand; which eventually results in reducing its viscosity. This reduction in viscosity facilitates producing foamed WMA mixtures at lower than traditional temperatures.
Due to the lack of a standard mix design procedure for producing foamed WMA mixtures, ODOT allows contractors to produce these mixtures using similar aggregate gradations and asphalt binder contents obtained through conducting standard HMA mix design procedures. In addition, ODOT specifies producing foamed WMA mixtures using a maximum of 1.8% foaming water content; which is the percentage of injected water by weight of asphalt binder, to be used during the foaming process. ODOT also specifies compacting foamed WMA mixtures at 30°F lower than the compaction temperatures used when compacting HMA mixtures. As for the mixing temperature, ODOT allows the contractors to determine the appropriate mixing temperature for this material.

![Figure 3.5: Wirtgen WLB-10 Laboratory Foaming Device.](image)

In this study, foamed WMA mixtures were produced using varying foaming water contents, aggregate moisture contents, and production (i.e. mixing and compaction)
temperatures. Therefore, two laboratory procedures were needed to prepare foamed WMA mixtures using fully dried aggregates and moist aggregates. Using these procedures to prepare foamed WMA mixtures will facilitate evaluating the effect of foaming water content, aggregate moisture content, and production temperatures on the performance of these mixtures. In addition, it is worth mentioning that foamed WMA mixtures were prepared in this study using the same aggregate gradations and asphalt binder contents obtained through the HMA mix design (previous section). These aggregate gradations and asphalt binder contents were included to allow for comparing the performance of foamed WMA mixtures to traditional HMA mixtures. The following subsections provide a detailed discussion of the two procedures needed to prepare foamed WMA mixtures in the laboratory along with a discussion of the various parameters used for preparing these mixtures.

3.4.1 Producing Foamed WMA Mixtures Containing Fully Dried Aggregates

As previously mentioned, a laboratory scale asphalt foaming device is necessary to produce foamed WMA mixtures. In this study, Wirtgen WLB-10 laboratory foaming device was utilized to foam the selected asphalt binders (Figure 3.3). As can be seen from Figure 3.3, the main components of the WLB10 device include 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.

To produce foamed asphalt binders using 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 (315°F (157°C) for PG 70-22M and 305°F (155°C) for PG 64-28) 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.

In this study, the amount of water used to foam the asphalt binders was varied to include 1.8, 2.2, and 2.6 percent of the total weight of the asphalt binder. These quantities were selected to prepare a set of foamed WMA mixtures that would facilitate evaluating the effect of foaming water content on the performance of foamed WMA 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:

\[ Q_{H2O} = \frac{Q_{Asphalt} \times P_{H2O} \times 3.6}{100} \]

where,

- \( Q_{H2O} \) = Water flow-through volume (liter/hour).
- \( Q_{Asphalt} \) = Asphalt flow-through volume (100 gram/sec).
- \( P_{H2O} \) = Selected foaming water content (%).
- 3.6 = Calculation factor.

After selecting the foaming parameters (i.e. air and water pressures, asphalt foaming temperature, and foaming water content), the device is ready to produce foamed asphalt binder. During the stage of setting up the foaming device, fully dried aggregate batches are heated, for two hours, to temperatures 30°F, 50°F, and 70°F lower than the mixing temperature used to prepare HMA mixtures. These temperatures were selected to prepare a set of foamed WMA mixtures that would allow for evaluating the effect of reduced production temperatures on performance. Upon completion of the two hour heating period, an aggregate batch is placed in mixing bowl which is placed under the foaming nozzle. Foamed asphalt binder is then discharged into the mixing bowl that is transferred immediately to a mechanical mixer for mixing. A period of 3 minutes, similar to that use when preparing HMA mixtures, was found to be sufficient when preparing foamed WMA mixtures.
3.4.2 Producing Foamed WMA Mixtures Containing Moist Aggregates

The procedure used to prepare foamed WMA mixtures containing moist aggregates involved setting up the foaming device in a similar fashion to that described in the previous subsection. However, since the aggregates are not fully dry, determining several additional parameters was necessary to ensure consistency in preparing this set of foamed WMA mixtures. These parameters included: (1) initial aggregate moisture content, (2) heating time and temperature, and (3) final aggregate moisture content.

To determine the aggregate moisture content parameters, an experiment was conducted to relate the aggregate type, aggregate size, batch weight, heating temperature, heating time, and initial moisture content. In this experiment, dry aggregate batches were first blended at the various weights commonly used in preparing specimens for performance testing. These weights included 4500 grams, 6000 grams, and 8000 grams. In addition, it is noted that varying the batch weight was necessary because the heating time was found to be dependent on the batch’s weight. For instance, batches that have a weight of 8000 grams were expected to need more time to reach a desired moisture content than batches that weigh 4500 grams; provided that the same heating temperature is used for both batches.

Once the aggregate batches were prepared, water was then added to each aggregate batch. The amount of water added represents the initial aggregate moisture content before heating and drying. The two initial aggregate moisture contents used in this study were 6 and 3 percent by total weight of aggregate. Aggregates and water were then thoroughly mixed to ensure that the aggregates have uniformly absorbed the added water. After that,
the aggregate batches were covered using heavy duty aluminum foil and their initial weight was recorded. The aggregate batches were then placed in an oven that was preheated to 310°F. While the aggregate batches were in the oven, the weight and temperature of each batch were recorded every 15 minutes.

The experiment discussed above was conducted for each aggregate type and size (i.e. 12.5 mm limestone, 19.0 mm limestone, and 12.5 crushed gravel) and using both of the initial aggregate moisture contents selected (i.e. 3 and 6 percent by total weight of aggregate). Figure 3.4 represents an example plot relating the aggregate temperature, aggregate moisture content, and heating time for a 4500 gram crushed gravel batch mixed with 6 percent initial aggregate moisture content. As can be noticed from this figure, the temperature starts to increase until it reaches approximately 210°F (100°C) (i.e. boiling temperature of water). The temperature of the batch then stabilizes at 210°F (100°C) due to the presence of water and because the batches are not perfectly sealed. Once a large percentage of water has evaporated, the temperature of the batch starts to increase. This temperature trend was observed for all aggregate types and sizes considered in this study. It indicates that producing foamed WMA mixtures with moist aggregate can only be completed at a maximum batch temperature of 210°F (100°C) as the option of using perfectly sealed pressure-holding containers was deemed as impractical.

The objective of the aggregate water experiment was to maximize the amount of water present in the aggregate batch while heating it to the maximum temperature possible before mixing with the asphalt binder. However, since the temperature of the aggregate batch containing moist aggregates reached a maximum of 210°F (100°C), all foamed WMA
mixtures prepared using moist aggregates were mixed at this temperature. Furthermore, 3 and 1.5 percent aggregate moisture contents were targeted in this study when the initial moisture content was 6 and 3 percent respectively. Table 3.9 presents the required heating time determined for each aggregate type and size.

![Temperature vs Time](image1)

![Agg. Moist. Cont. vs. Time](image2)

**Figure 3.6:** Example of Temperature, Aggregate Moisture Content, and Heating Time Relation for Crushed Gravel Aggregates.

Using the values presented in table 3.9, moist aggregate batches were heated for a specific period of time. Upon the lapse of the required heating period, an aggregate batch is placed in a mixing bowl which is then placed under the foaming nozzle in the WLB-10 device. Foamed asphalt binder was then discharged directly into the mixing bowl which was then transferred immediately to a mechanical mixer for mixing. Similar to the dry
aggregate foamed WMA preparation procedure, a mixing period of 3 minutes was used. Finally, it is noted that the aggregates used in prepared foamed WMA mixtures using this procedure were not fully coated due to the use of low mixing temperatures (i.e. 210°F (100°C)) and the presence of water within the aggregates.

Table 3.9: Heating Time Required to Reach Target Aggregate Moisture Content

(1.5% for 3% Initial Agg. w(%) and 3% for 6% Initial Agg. w(%)).

<table>
<thead>
<tr>
<th>Initial Agg. w(%)</th>
<th>Aggregate Type and Size</th>
<th>12.5 mm Limestone</th>
<th>19.0 mm Limestone</th>
<th>12.5 mm Crushed Gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Batch Weight 4500</td>
<td></td>
<td>3</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>Batch Weight 6000</td>
<td></td>
<td>3</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>Batch Weight 8000</td>
<td></td>
<td>3</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>Weight 6000</td>
<td></td>
<td>3</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>Weight 8000</td>
<td></td>
<td>3</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>Weight 8000</td>
<td></td>
<td>3</td>
<td>6</td>
<td>3</td>
</tr>
</tbody>
</table>

*heating time in minutes
CHAPTER IV

LABORATORY TESTING PROGRAM

4.1 Introduction

As discussed in Chapter I, the main objectives of this study included evaluating the performance of foamed WMA and HMA mixtures, examining the workability and compactability of these mixtures, and determining the limitations of foamed WMA mixtures. Three distinct laboratory testing plans were implemented to complete each of these objectives. The first was a comprehensive laboratory testing plan used to evaluate the performance of foamed WMA and HMA mixtures with regard to permanent deformation (or rutting), moisture-induced damage, fatigue cracking, and low-temperature (thermal) cracking. The second laboratory experimental plan involved examining the workability and compactability of foamed WMA and HMA mixtures. The third experimental plan was implemented to evaluate the effect of foamed WMA mix preparation parameters (i.e. reduction in production temperatures, foaming water content, and aggregate moisture content) on the performance of these mixtures; and as a result determining the limitation of foamed WMA mixtures.

This chapter presents a detailed discussion of each of the three experimental plans implemented in this study. It also offers an overview of the undertaken testing procedures
as well as the specimen preparation techniques required to prepare representative samples for the performance tests utilized in the experimental plans.

4.2 Comprehensive Performance Evaluation Plan

As mentioned above, a comprehensive laboratory testing plan was implemented in this study to evaluate the performance of foamed WMA and HMA mixtures with regard to permanent deformation (or rutting), moisture-induced damage (or durability), fatigue cracking, and low temperature (thermal) cracking. Figure 4.1 presents the laboratory tests used to examine the performance of the considered asphalt mixtures. As can be seen from this figure, the asphalt pavement analyzer (APA), dynamic modulus (E*), and flow number (FN) tests were used to evaluate the rutting potential of the foamed WMA and HMA mixtures. The moisture sensitivity (or durability) was investigated using the modified Lottman (AASHTO T 283), conditioned dynamic modulus, and wet APA tests. The susceptibility to fatigue cracking and low temperature (thermal) cracking were examined using the dissipated creep strain energy (DCSE) and indirect tensile strength (ITS) tests, respectively.

The previous tests were conducted on specimens prepared using the aggregate-binder material combinations presented in Chapter III. Both foamed WMA and HMA specimens were prepared using the same aggregate gradation and asphalt binder content. A foaming water content of 1.8% along with a 30°F (16.7°C) temperature reduction were utilized in the production of the foamed WMA mixture. The following subsections offer an overview of the undertaken testing procedures utilized to examine each of the considered performance concerns. The following subsections also provide a discussion
detailing the specimen preparation techniques required to prepare representative samples for the considered performance tests. Where applicable, the testing procedure was modified according to the standard practices implemented in the state of Ohio. Finally, it is noted that the results obtained after conducting this testing plan are presented in Chapter V.

![Laboratory Testing Program](image)

**Figure 4.1: Laboratory Testing Program.**

### 4.2.1 Evaluation of Permanent Deformation

#### 4.2.1.1 Asphalt Pavement Analyzer

The asphalt pavement analyzer (APA) test was conducted according to the AASHTO T 340-10 (Standard Method of Test for Determining the Rutting Susceptibility of Asphalt Paving Mixtures Using the Asphalt Pavement Analyzer) and ODOT Supplement 1057 (Loaded Wheel Tester Asphalt Mix Rut Testing Method). The permanent deformation of asphalt mixtures, characterized using this test, is usually reported as a rut depth value representing the rutting experienced by specimens subjected to loading.

The specimens prepared for this test were compacted using the Superpave Gyratory Compactor (SGC). These specimens were cylindrical in shape and having a 6 inch (150
mm) diameter and 2.95 inch (75 mm) height. The target air void level within the compacted specimen was selected to be 7 ± 1% as specified in ODOT Supplement 1057. The determination of the target air voids percentage involved varying the mixture’s weight used to compact a specimen. The relationship between mixture’s weight and the air voids percentage within compacted specimens was established through compacting trial specimens at varying mixture weights. This relationship was then modeled using linear regression analysis and the weight that would result in compacting specimens at the target air voids percentage was determined.

The laboratory procedure utilized to prepare specimens for the APA test is summarized as follows:

- Heat aggregate batches and asphalt binder to the desired mixing temperature. A minimum of two hours heating period was used in this study to heat the materials before mixing.
- Mix one heated aggregate batch with the heated asphalt binder (or foamed asphalt binder in the case of foamed WMA) using a mechanical mixer. A mixing period of 3 minutes was used in this study.
- Place the loose 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.
- Compact the specimens using the Superpave Gyratory Compactor (SGC). The mixture weight that would result in compacting the specimens to the desired air voids level should be used in this step.
- Determine the bulk specific gravity and air voids content of the compacted specimens. If the air voids content of the compacted specimens does not fall within the target air voids limits, the specimens should be discarded.

Testing of the compacted specimens was completed through the use of the asphalt pavement analyzer (APA) device shown in Figure 4.2. This device simulates actual road conditions by applying a repeated load using a concave-shaped metal wheel on a pressurized rubber hose (Figure 4.3). In general, the speed, at which the load is applied, is 23.5 inches/sec (60 cm/sec) and the hose pressure, used in this study, was about 100 psi (689.5 kPa). This device is also equipped with heating and cooling components that facilitate testing specimens at various temperatures. In this study, a testing temperature, as specified by ODOT Supplement 1057, of 120°F (49°C) was used. It is noted that before testing was started the specimens were heated in the APA for a minimum period of 12 hours. This is consistent with what is specified in ODOT Supplement 1057.

Rut depth measurements were recorded after applying 5, 500, 1000, and 8000 loading cycles. For each APA sample, which consists of two cylindrical specimens, four rut depth readings were measured 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. Averaged rut depth values for three APA samples are reported in this study.
Figure 4.2: Asphalt Pavement Analyzer (APA).

Figure 4.3: Repeated Wheel Loading in the APA Device.
4.2.1.2 Dynamic Modulus

The AASHTO T 342-11 (Standard Method of Test for Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures) was used to conduct the dynamic modulus test. The dynamic modulus is a fundamental material property commonly used to describe the mechanical behavior of visco-elastic 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.

According to AASHTO T 342-11 standards, the dynamic modulus test is conducted at different frequencies (i.e. 25, 10, 5, 1, 0.5, and 0.1 Hz) and different temperatures (i.e. 40, 70, 100, and 130°F; 4.4, 21.1, 37.8, and 54.4°C). At each combination of these frequencies and temperatures, a repeated sinusoidal load is applied on the specimen and the resulting deformation is recorded. The applied stress and the resulting strain are then measured and related thorough a complex number called the “complex modulus” (E*). The dynamic modulus (|E*|) is defined as the absolute value of the complex modulus and can be quantified as the ratio of the applied stress level to the recoverable strain level.

Typical specimens used in this test were prepared by coring and sawing (Figures 4.4 and 4.5) 4 inch (100 mm) diameter by 6 inch (150 mm) height cylindrical cores from gyratory compacted specimens having a diameter of 6 inches and a height of 7 inches (175
mm). The air voids content within the fabricated specimens was targeted to be 7±0.5%. Before compaction; however, the loose 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 was used to determine the mixture weight required to achieve the target air voids percentage.

Figure 4.4: Coring Setup Used to Prepare Dynamic Modulus Specimens.
Upon the completion of the coring and sawing processes, the diameter and the waviness of the top and bottom edges of the extracted specimens were measured to ensure that they are within the acceptable limits. AASHTO T 342-11 standards 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 T 342-11 specifies a maximum acceptable waviness of ± 0.05 mm (0.002 in.) at the top and bottom edges of the sawed specimens. Figure 4.6 shows the straightedge and the feel gage used to measure the waviness.
A servo-hydraulic Material Test System (MTS) Model 810 (Figure 4.7) was used to conduct the dynamic modulus testing on the prepared specimens. This system is operated through using a personal computer and a digital controller called MTS TestStar II. It is capable of applying various types of loads including cyclic, monotonic, and creep. The system 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 top and bottom edges of the specimens. To measure the deformation in the specimens, two extensometers, provided for the system by the manufacturer, were used to measure the vertical deformation in the specimens as the load was applied. The use of extensometers was preferred over using Linear Variable Differential Transducers (LVDTs) because the former provides higher accuracy and can be easily installed on the specimen. Finally, the intensity of the load was measured using an external load cell located underneath the bottom loading platen.

Figure 4.6: Checking the Waviness of the Top and Bottom Edges of a Dynamic Modulus Specimen Using a Straight Edge and a Feel Gage.
Figure 4.7 presents a sample of the applied stress and resulting strain curves versus time at 1 Hz. As can be noticed from 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. The magnitude of the dynamic load applied should be, according to AASHTO T 342-11 standards, the load that would result in 75 to 125 micro-strain levels within the specimen. To determine this load, trial specimens were tested at various load levels, frequencies, and temperatures. Figure 4.8 also shows that the strain cycles are accumulating some permanent strain with time.

To calculate the dynamic modulus, data collected from the last five applied stress and resulting strain cycles were analyzed (Figure 4.8). As previously mentioned, the dynamic modulus can be calculated by dividing the magnitude of the applied stress (peak minus valley stress) by the resulting strain (peak minus valley strain). To appropriately determine the peaks and valleys of each cycle, a second degree polynomial was used to
model half cycles. This function was 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 4.9 presents the data analysis procedure using one stress-strain cycle.

![Graph showing stress-strain relationship over time](image)

**Figure 4.8: Sample of Applied Stress and Measured Strain at 1 Hz.**

Upon the completion of the dynamic modulus tests, the dynamic modulus master curves were developed according to the procedure described in the mechanistic-empirical pavement design guide (MEPDG). The dynamic moduli obtained at various testing temperatures were plotted against loading frequency. The dynamic moduli for each temperature were parallel-shifted to a reference temperature to form a single continuous curve using the following equation:
\[ a_T = \frac{f_{T_0}}{f_T} \]

where,

\( a_T \) = frequency temperature shift factor for temperature \( T \);

\( f_{T_0} \) = reduced frequency at reference temperature \( T_0 \); and

\( f_T \) = frequency at test temperature, \( T \).

Figure 4.9: Determination of Peak and Valley Stress/Strain Values.

The Sigmoidal function suggested by the MEPDG was used in this study to fit the dynamic modulus master curve. The function is presented as shown below:

\[
\log|E^*| = \delta + \frac{\alpha}{1 + e^{\beta \cdot \log(f_T)}}
\]
where,

\[ E^* = \text{dynamic modulus}; \]
\[ f_r = \text{reduced frequency of loading at reference temperature}; \] and
\[ \alpha, \delta, \beta, \gamma = \text{Sigmoidal fitting parameters}. \]

The Solver option in Microsoft Excel was used to determine the temperature shift factors and Sigmoidal model parameters.

4.2.1.3 Flow Number

The flow number (FN) test was conducted according to the test procedure suggested in Annex B of NCHRP Report 513 (Simple Performance Tester for Superpave Mix Design: First-Article Development and Evaluation). In this test, an asphalt specimen is loaded repeatedly for up to 10,000 loading cycles while the permanent deformation is measured. The FN value is defined as the number of cycles at which tertiary flow occurs in the specimen as determined from the cumulative permanent deformation curve. Asphalt mixtures with higher FN values are desired as they are expected to have better rutting resistance in the field.

The specimens prepared for this test were compacted using the Superpave Gyratory Compactor (SGC). These specimens were cylindrical in shape and having a 6 inch (150 mm) diameter and 6.7 inch (170 mm) height. From the SGC compacted specimens 4 inch diameter cores were taken. The top and bottom edges of the cored specimens were then trimmed using a diamond saw; resulting in specimens having a height of 6 inches. The target air voids level within the trimmed specimen was selected to be 7 ± 0.5%. The determination of the target air voids percentage involved varying the mixture’s weight used
to compact a specimen. The relationship between mixture’s weight and the air voids percentage within trimmed specimens was established through compacting trial specimens at varying mixture weights. This relationship was then modeled using linear regression analysis and the weight that would result in compacting specimens at the target air voids percentage was determined.

The laboratory procedure utilized to prepare specimens for the flow number (Annex B of NCHRP Report 513) test is summarized as follows:

- Heat aggregate batches and asphalt binder to the desired mixing temperature. A minimum of two hours heating period was used in this study to heat the materials before mixing.
- Mix one heated aggregate batch with the heated asphalt binder (or foamed asphalt binder in the case of foamed WMA) using a mechanical mixer. A mixing period of 3 minutes was used in this study.
- Place the loose mixture in a flat pan and spread the mixture over the whole area of the pan.
- Place the pan in an oven for four hours at 275°F (135°C). Conditioning the loose mixture at this temperature represent short-term aging. In addition, the mixture should be stirred every one hour during aging period.
- Raise the temperature of the oven until it reaches the compaction temperature of the mixture. Heat the mixture for about 30 minutes to ensure its temperature has stabilized and reached the compaction temperature.
- Compact the specimens using the Superpave Gyratory Compactor (SGC). The mixture weight that would result in compacting the specimens to the desired air voids level should be used in this step.

- Core 4 inch diameter specimens of the SGC compacted specimens. Trim the top and bottom edges of the cored specimens to result in a specimens’ height of 6 inches (150.0 mm).

- Determine the bulk specific gravity and air voids content of the cored and trimmed specimens. If the air voids content of these specimens does not fall within the target air voids limits, the specimens should be discarded.

As mentioned above, the flow number test involves applying repeated loading (i.e. up to 10,000 cycles) on the tested specimen. This load can be described as haversine with 0.1 seconds loading pulse and 0.9 seconds resting period. To apply this load, an MTS 810 servo-hydraulic device was used. Two LVDTs were also used to measure the vertical deformation as the specimen was being loaded. As can be seen from Figure 4.10, the LVDTs were attached to the top loading platen and they were separated by 180° from each other. It is noted that this test was conducted using a stress level of 40 psi and a testing temperature of 130°F (54.4°C). These values were selected to ensure that the tested specimens reached the tertiary flow region within the reasonable period of time.
The calculations of the flow number involves determining the cumulative permanent deformation for each specimen as a function of load cycles. Figure 4.11 presents typical results expected after conducting the FN test. As can be noticed from this figure, the cumulative permanent strain is divided into three main regions that include the primary flow region, the secondary flow region, and the tertiary flow region. The primary flow region represents the region where rapid change in cumulative permanent deformation occurs within the specimen while the secondary flow region is the region where the rate of change of the cumulative permanent deformation stabilizes. The tertiary flow region describes the region where the rate of change in permanent deformation starts to increase again. The flow number (FN) is defined as the starting point, or cycle number, of the tertiary flow region. This value is usually calculated by finding the cycle number at which
the rate of change in permanent deformation is minimized (Figure 4.11). In some cases, the calculations do not result in a minimum value for the rate of change of permanent deformation or they may result in two or more minimum values. In the first case, the total number of cycles (i.e. 10,000) applied on the specimen is considered as the FN value while in the second case the lowest number of cycles applied on the specimen is considered as the flow number.

Figure 4.11: Typical Flow Number Results (After Kabir, 2008).
4.2.2 Evaluation of Moisture-Induced Damage

4.2.2.1 Modified Lottman (AASHTO T 283)

The AASHTO T 283 (Standard Method of Test for Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage) and ODOT Supplement 1051 (Resistance of Compacted Bituminous Concrete to Moisture Induced Damage) were used to conduct the modified Lottman test. Three asphalt mixture performance parameters can be measured using this test. These parameters include the dry indirect tensile strength (ITS), wet ITS, and tensile strength ratio (TSR). Asphalt mixtures with higher ITS and TSR values are desired because they are expected to be less susceptible to moisture-induced damage in the field.

The specimens prepared for this test were compacted using the Superpave Gyratory Compactor (SGC). These specimens were cylindrical in shape and having a 6 inch (150 mm) diameter and 3.75 inch (95 mm) height. The target air voids level within the compacted specimen was selected to be 7 ± 0.5% as specified in AASHTO T 283. The determination of the target air voids percentage involved varying the mixture’s weight used to compact a specimen. The relationship between mixture’s weight and the air voids percentage within compacted specimens was established through compacting trial specimens at varying mixture weights. This relationship was then modeled using linear regression analysis and the weight that would result in compacting specimens at the target air voids percentage was determined.

The laboratory procedure utilized to prepare specimens for the Modified Lottman (AASHTO T 283) test is summarized as follows:
- Heat aggregate batches and asphalt binder to the desired mixing temperature. A minimum of two hours heating period was used in this study to heat the materials before mixing.

- Mix one heated aggregate batch with the heated asphalt binder (or foamed asphalt binder in the case of foamed WMA) using a mechanical mixer. A mixing period of 3 minutes was used in this study.

- Place the loose mixture in a flat pan and spread the mixture over the whole area of the pan.

- Place the pan in an oven for four hours at 275°F (135°C). Conditioning the loose mixture at this temperature represent short-term aging. In addition, the mixture should be stirred every one hour during aging period.

- Raise the temperature of the oven until it reaches the compaction temperature of the mixture. Heat the mixture for about 30 minutes to ensure its temperature has stabilized and reached the compaction temperature.

- Compact the specimens using the Superpave Gyratory Compactor (SGC). The mixture weight that would result in compacting the specimens to the desired air voids level should be used in this step.

- Determine the bulk specific gravity and air voids content of the compacted specimens. If the air voids content of the compacted specimens does not fall within the target air voids limits, the specimens should be discarded.

According to AASHTO T 283, six specimens were prepared to conduct this test. The six specimens were divided into two groups (i.e. dry group and wet group). Each group
consisted of three specimens. The specimens in the dry group were wrapped in Saran-Wrap and stored at room temperature. The specimens in the wet group were soaked in water for four hours then they were vacuum saturated, with water, until they reached a saturation degree ranging between 80 to 90% as specified by ODOT Supplement 1051. The saturated specimens were then wrapped in Saran-Wrap and placed in leak-proof plastic bags. 0.6 in\(^3\) (10 ml) of water was added to each plastic bag. After that, the saturated specimens were placed in a freezer at -0.4°F (-18°C) for a 24-hour freezing period. Upon the completion of the freezing step, the specimens were placed in a water bath at 140°F (60°C) for a 24-hour thawing period. All specimens (i.e. unconditioned (or dry) and conditioned (or saturated)) were placed in a water bath, heated to 77°F (25°C), for two hours before testing.

An MTS 810 was used to load both sets of specimens diametrally as shown in Figure 4.12. As specified by AASHTO T 283, the specimens were loaded using a rate of 2 inches per minute (50.8 mm per minutes). The maximum load required to break the specimens was recorded for use in the calculation of the dry and wet indirect tensile strength (ITS) values. The following equation was used to calculate the indirect tensile strength:

\[
S_t = \frac{2P}{\pi t D}
\]

where,

\(S_t = \) indirect tensile strength (psi);

\(P = \) maximum load (lbs);

\(t = \) specimen thickness (in.); and \(D = \) specimen diameter (in.).
Finally, the Tensile Strength Ratio (TSR) was calculated as the ratio between the average indirect tensile strength of the conditioned (or saturated) specimens to average indirect tensile strength of the unconditioned (or dry) specimens. As previously mentioned, 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.

4.2.2.2 Conditioned Dynamic Modulus

The moisture sensitivity of the asphalt mixtures was evaluated by determining the impact of sample conditioning on the dynamic modulus. The same specimen conditioning procedure implemented for AASHTO T 283 was used for this purpose.
However, the specimens were vacuum saturated until reaching a degree of saturation ranging between 70 to 80% instead of 80 to 90%. After conditioning the specimens, determination of the dynamic modulus of these specimens was completed through running the dynamic modulus test described in Section 4.2.1.2. The reader is referred to that section for more information about the dynamic modulus test.

4.2.2.3 Wet Asphalt Pavement Analyzer

Similar to the conditioned dynamic modulus test, the moisture sensitivity of the asphalt mixtures was evaluated by determining the impact of sample conditioning on the APA rut depth. The same specimen conditioning procedure implemented for AASHTO T 283 was used for this purpose. However, the specimens were vacuum saturated until reaching a degree of saturation ranging between 70 to 80% instead of 80 to 90%. After completion of the sample conditioning step, the specimens were tested according to the APA test described in Section 4.2.1.1. In addition, it should be noted that the wet APA test was conducted while the specimens were fully submerged in water.

4.2.3 Evaluation of Fatigue Cracking

4.2.3.1 Dissipated Creep Strain Energy

The Dissipated Creep Strain Energy (DCSE) was used to indirectly evaluate the resistance of foamed WMA and HMA mixtures to fatigue cracking. The determination of the DCSE for asphalt mixtures requires conducting first the indirect resilient modulus test (MR) and then the indirect tensile strength (ITS) test on the same specimen. The procedure found in NCHRP research digest-285 (Laboratory Determination of Resilient Modulus for Flexible Pavement Design) was used to conduct the indirect resilient modulus test, while
the AASHTO T 322-03 (Standard Method of Test for Determining the Creep Compliance and Strength of Hot-Mix Asphalt using the Indirect Tensile Test Device) was used to conduct the indirect tensile strength test. It is noted that both of these tests were conducted at a temperature of 10°C.

The specimens prepared for this test were compacted using the Superpave Gyratory Compactor (SGC). These specimens were cylindrical in shape and having a 6 inch (150 mm) diameter and 2.95 inch (75 mm) height. The top and bottom edges of the specimens were then trimmed, using a diamond saw, to result in a specimen height of 2 inches (50.8 mm). The target air voids level within the trimmed specimen was selected to be 7 ± 0.5% as specified in AASHTO T 322 standards. The determination of the target air voids percentage involved varying the mixture’s weight used to compact a specimen. The relationship between mixture’s weight and the air voids percentage within trimmed specimens was established through compacting trial specimens at varying mixture weights. This relationship was then modeled using linear regression analysis and the weight that would result in compacting specimens at the target air voids percentage was determined.

The laboratory procedure utilized to prepare specimens for the indirect resilient modulus (NCHRP research digest-285) and indirect tensile strength (AASHTO T 322-03) tests is summarized as follows:

- Heat aggregate batches and asphalt binder to the desired mixing temperature. A minimum of two hours heating period was used in this study to heat the materials before mixing.
- Mix one heated aggregate batch with the heated asphalt binder (or foamed asphalt binder in the case of foamed WMA) using a mechanical mixer. A mixing period of 3 minutes was used in this study.

- Place the loose mixture in a flat pan and spread the mixture over the whole area of the pan.

- Place the pan in an oven for four hours at 275°F (135°C). Conditioning the loose mixture at this temperature represent short-term aging. In addition, the mixture should be stirred every one hour during aging period.

- Raise the temperature of the oven until it reaches the compaction temperature of the mixture. Heat the mixture for about 30 minutes to ensure its temperature has stabilized and reached the compaction temperature.

- Compact the specimens using the Superpave Gyratory Compactor (SGC). The mixture weight that would result in compacting the specimens to the desired air voids level should be used in this step.

- Trim the top and bottom edges of each specimen to reduce the specimens’ height to 2 inches (50.8 mm).

- Determine the bulk specific gravity and air voids content of the trimmed specimens. If the air voids content of the trimmed specimens does not fall within the target air voids limits, the specimens should be discarded.

The procedure used for determining the $M_R$ value for each specimen involved applying a haversine load with 0.1 seconds of loading and 0.4 seconds of resting period. To apply this type of loading, an MTS 810 was used. The magnitude of the applied load,
according to NCHRP research digest-285, was determined as the load that would result in approximately 100 micro-strain of vertical deformation in the specimen. Four Linear Variable Differential Transducers (LVDTs) were used to measure the vertical and horizontal deformation resulting in the specimens due to the applied loading. Figure 4.13 shows how two of the vertical and horizontal LVDTs were mounted on one of the trimmed faces of the specimen.

![Figure 4.13: Mounting Vertical and Horizontal Linear Variable Differential Transducers (LVDTs) on a DCSE Specimen.](image)

Before applying the haversine loading, the specimens were conditioned, in an environmental chamber, at the testing temperature (i.e. 10°C) for four hours. After that, 205 cycles of the haversine load were applied on the specimen. The specimen was then
rotated 90° and the MR test was conducted again. Data recorded from the last five cycles, of each of the MR tests, were analyzed to determine an averaged MR value for the tested specimen. Once the MR test was completed, the specimens were loaded diametrally, according to AASHTO T 322-03, using a monotonic loading rate of 2 inches per minute (50 mm per minute) until they broke. During this test, load and deformation data were recorded to calculate the stress-strain relations for each specimen.

The DCSE value represents the energy threshold that a mixture can tolerate before it fractures. The calculation of this quantity involves using the procedure that was introduced by Roque et al. (2004). The DCSE is defined, according to Roque et al. (2004), as the difference between the fracture energy and the elastic energy as shown in Figure 4.14. The fracture energy (FE) is represents the area under the stress-strain curve, obtained from indirect tensile strength test, up to the point where the specimen breaks. The elastic energy (EE) is the area resulting from adjusting the failure strain using the value obtained from resilient modulus testing. As shown in Figure 4.14, the fracture energy is defined as the area enclosed by curve OB and lines BA and CO. In addition, the elastic energy is defined as the area of the triangle ABC defined using the MR value obtained from the indirect resilient modulus test. Based on this discussion, the following equations were used to calculate the DCSE:

$$\varepsilon_o = \frac{M_R \times \varepsilon_f - S_t}{M_R}$$

$$EE = \frac{1}{2} \times S_t \times (\varepsilon_f - \varepsilon_o)$$

$$DCSE = FE - EE$$
where,

\( M_R \) = Indirect resilient modulus;

\( S_t \) = Indirect tensile strength;

\( \varepsilon_f \) = Failure strain;

\( EE \) = Elastic energy;

\( FE \) = Fracture energy; and \( DCSE \) = Dissipated creep strain energy.

**Figure 4.14: DCSE Calculation Procedure.**

4.2.4 **Evaluation of Low-Temperature Cracking**

4.2.4.1 **Low-Temperature Indirect Tensile Strength**

The indirect tensile strength (ITS) at low temperature test was conducted at 14°F (-10°C) according to AASHTO T 322-03 (Standard Method of Test for Determining the
Creep Compliance and Strength of Hot-Mix Asphalt using the Indirect Tensile Test Device). Two asphalt mixture properties are usually measured using this test. These properties include the low-temperature indirect tensile strength and the strain at failure. Asphalt mixtures with high low-temperature ITS and failure strain values are desired because they are expected to have better thermal cracking resistance in the field.

The procedure used in preparing specimens for the DCSE test was also used to prepare specimens for low-temperature ITS test. This was the case because the specimens for these two tests were the exactly the same. For completeness; however, the laboratory procedure utilized to prepare specimens for the low-temperature indirect tensile strength (AASHTO T 322-03) test is summarized as follows:

- Heat aggregate batches and asphalt binder to the desired mixing temperature. A minimum of two hours heating period was used in this study to heat the materials before mixing.
- Mix one heated aggregate batch with the heated asphalt binder (or foamed asphalt binder in the case of foamed WMA) using a mechanical mixer. A mixing period of 3 minutes was used in this study.
- Place the loose mixture in a flat pan and spread the mixture over the whole area of the pan.
- Place the pan in an oven for four hours at 275°F (135°C). Conditioning the loose mixture at this temperature represent short-term aging. In addition, the mixture should be stirred every one hour during aging period.
- Raise the temperature of the oven until it reaches the compaction temperature of the mixture. Heat the mixture for about 30 minutes to ensure its temperature has stabilized and reached the compaction temperature.

- Compact the specimens using the Superpave Gyratory Compactor (SGC). The mixture weight that would result in compacting the specimens to the desired air voids level should be used in this step.

- Trim the top and bottom edges of each specimen to reduce the specimens’ height to 2 inches (50.8 mm).

- Determine the bulk specific gravity and air voids content of the trimmed specimens. If the air voids content of the trimmed specimens does not fall within the target air voids limits, the specimens should be discarded.

To measure the low-temperature ITS, the specimens were diameterally loaded (i.e. similar to DCSE specimens as shown in Figure 4.13) using a displacement rate of ½ inches per minute (12.5 mm per minute). Two LVDTs were used to measure the vertical and horizontal deformations. These LVDTs were mounted in a similar fashion to that described for DCSE specimens (Figure 4.13). It is noted that the specimens were conditioned for a minimum of 12 hours at 14°F (-10°C) before testing. After testing, the peak load was recorded and then was used to calculate the low-temperature ITS using the equation discussed previously in the AASHTO T 283 test.

4.3 Workability and Compactability Testing Plan

As mentioned previously, the workability and compactability characteristics of foamed WMA and HMA mixtures were also evaluated as a part of this study.
Figure 4.15 presents the testing plan implemented to evaluate these mixture properties. As can be noticed from this figure, the workability of foamed WMA and HMA mixtures was examined using the newly designed and fabricated University of Akron (UA) workability device. Superpave gyrator compactor (SGC) data collected during the preparation of specimens for the performance tests discussed in Section 4.2 was analyzed to evaluate the compactability of foamed WMA and HMA mixtures.

![Figure 4.15: Workability and Compactability Testing Plan.](image)

It should be noted that the previous workability test was conducted on specimens prepared using the aggregate-binder material combinations presented in Chapter III. Both foamed WMA and HMA specimens were prepared using the same aggregate gradation and asphalt binder content. A foaming water content of 1.8% along with a 30°F (16.7°C) temperature reduction were utilized in the production of the foamed WMA mixture used in this experimental plan. Finally, the design and fabrication of the new workability device, the utilized workability testing procedure, and the results obtained after carrying out the testing plan, shown in Figure 4.15, are presented in Chapter VI. The reader is referred to that chapter for more information.
4.4 Effect of Foamed WMA Mix Preparation Parameters Testing Plan

In addition to the performance evaluation of foamed WMA mixtures, this study also involved determining the limitations of these mixtures. As a result, an experimental testing plan (Figure 4.16) was implemented to investigate the effect of foamed WMA mix preparation parameters (i.e. temperature reduction, foaming water content, and aggregate moisture content) on performance. As can be noticed from Figure 4.16, the APA and AASHTO T 283 tests were utilized to evaluate the performance of foamed WMA and HMA mixtures with regard to permanent deformation (or rutting) and moisture-induced damage respectively. Furthermore, the conditioned and unconditioned indirect tensile strength (ITS) values obtained from the AASHTO T 283 test were also used as a parameter to evaluate the moisture-induced damage of the considered mixtures. The reader is referred to Sections 4.2.1.1 and 4.2.2.1 for more information about the APA and AASHTO T 283 tests respectively.

Figure 4.16 also shows that the effect of temperature reduction was examined through testing foamed WMA mixtures prepared using temperatures that are 30, 50, and 70°F (16.7, 27.8, and 38.9°C) lower than what is traditionally used for HMA mixtures. The effect of foaming water content was assessed through testing foamed WMA mixtures prepared using 1.8, 2.2, and 2.6% foaming water contents. Foamed WMA mixtures prepared using 0, 1.5, and 3% aggregate moisture contents were used to examine the effect of aggregate moisture content on performance. The results obtained after carrying out the testing plan presented in Figure 4.16 are presented in Chapter VII. The reader is referred to that chapter for more information.
Figure 4.16: Testing Plan Prepared to Examine the Effect of Foamed WMA Mix Preparation Parameters on Performance.
CHAPTER V
LABORATORY TEST RESULTS AND DISCUSSION

5.1 Introduction

As mentioned in Chapter IV, the performance of foamed WMA and HMA mixtures was evaluated through a comprehensive laboratory experimental plan. This plan included conducting the asphalt pavement analyzer (APA), dynamic modulus (|E*|), and flow number (FN) tests to evaluate the performance of these mixtures with regard to permanent deformation. The modified Lottman (AASHTO T 283), conditioned dynamic modulus, and the wet asphalt pavement analyzer tests were used to examine the moisture sensitivity of foamed WMA and HMA mixtures. Finally, the dissipated creep strain energy (DCSE) and the low-temperature indirect tensile strength tests were used to evaluate the propensity of the considered asphalt mixtures to fatigue cracking and low-temperature (thermal) cracking respectively.

This chapter presents the experimental test results obtained after conducting these tests. In addition, it provides the outcome of the Analysis of Variance (ANOVA) that was conducted using the Statistical Package for Social Sciences (SPSS) to examine the significance of the mix type, binder type, aggregate type, and aggregate size as well as the two-way interactions between the mix type and the mixture constituents on the performance test parameters. Given that a partial factorial was used in the experimental
testing plan (i.e., four material combinations were used instead of a full factorial of eight), selected material combinations were included in the statistical analysis.

5.2 Evaluation of Permanent Deformation

5.2.1 Asphalt Pavement Analyzer Test Results

Figure 5.1 presents the APA rut depth values obtained for foamed WMA and HMA mixtures. As can be noticed from this figure, slightly higher rut depth values were obtained for foamed WMA mixtures than those obtained for HMA mixtures. These result might indicate that the foamed WMA mixtures are slightly more susceptible to rutting than the HMA mixtures. This slightly higher rutting observed for foamed WMA mixtures is mainly attributed to the reduced binder aging due to the use of lower production temperatures.

Figure 5.1: Asphalt Pavement Analyzer (APA) Rutting Results.
The effect of binder grade, aggregate type, and aggregate size can also be evaluated through examining the results presented in Figure 5.1. As can be noticed from this figure, both foamed WMA and HMA 19 mm mixtures prepared using PG 64-28 had significantly higher rut depth values than those prepared using PG 70-22. These results were expected because the latter asphalt binder is stiffer. It can also be noticed from Figure 5.1 that the 12.5 mm foamed WMA and HMA mixtures prepared using limestone had slightly lower rut depth values than the 12.5 mm foamed WMA and HMA mixtures prepared using crushed gravel aggregates. These results were also expected due to the better aggregate interlock within the limestone aggregates.

Furthermore, by comparing the rut depth values obtained for both the 12.5 mm foamed WMA and HMA mixtures prepared using limestone and PG 70-22 to those prepared using 19 mm limestone and PG 70-22, the effect of aggregate size and gradation on rut depth results can be evaluated. As can be seen from Figure 5.1, similar rut depth values were obtained for both the 12.5 and 19 mm foamed WMA and HMA mixtures prepared using limestone and PG 70-22, indicating that the aggregate size did not affect the rut depth values as measured using the APA test. While aggregate size and gradation are known to affect the rutting performance, it is believed that the use of a plastic mold in the APA test to restrain the lateral movement of the specimens might subject these specimens to significantly higher confinement levels than those encountered in the field; thus, reducing the effect of the aggregate size on the measured rut depth values.

Table 5.1 presents the ANOVA results obtained for the APA rut depth values. As mentioned earlier, selected material combinations were included in the statistical analysis.
because a partial factorial was used in the experimental testing plan. As can be noticed from this table, there is no significant difference between the foamed WMA and HMA APA rut depth values at a confidence level of 95% (probability < 0.05). This suggests that the rutting performance of foamed WMA mixtures is not significantly different than that for the HMA mixtures. The ANOVA results also show that the binder type has the most significant (highest F-value) effect on the APA rut depth values, followed by the aggregate size and aggregate type.

Table 5.1: ANOVA Results Obtained for APA Rut Depth Values.

<table>
<thead>
<tr>
<th>Analysis Data</th>
<th>Statistical Factors</th>
<th>F-value</th>
<th>Prob.</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.0 mm, Limestone, PG 64-28 &amp; 19.0 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>0.088</td>
<td>0.775</td>
</tr>
<tr>
<td></td>
<td>Binder Type</td>
<td>67.108</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Binder Type × Mix Type</td>
<td>0.427</td>
<td>0.532</td>
</tr>
<tr>
<td>12.5 mm, Gravel, PG 70-22 &amp; 12.5 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>0.219</td>
<td>0.653</td>
</tr>
<tr>
<td></td>
<td>Agg. Type</td>
<td>0.017</td>
<td>0.900</td>
</tr>
<tr>
<td></td>
<td>Agg. Type × Mix Type</td>
<td>2.700</td>
<td>0.139</td>
</tr>
<tr>
<td>12.5 mm, Limestone, PG 70-22 &amp; 19.0 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>0.136</td>
<td>0.722</td>
</tr>
<tr>
<td></td>
<td>Agg. Size</td>
<td>0.096</td>
<td>0.764</td>
</tr>
<tr>
<td></td>
<td>Agg. Size × Mix Type</td>
<td>0.868</td>
<td>0.379</td>
</tr>
</tbody>
</table>

5.2.2 Dynamic Modulus Test Results

Figures 5.2 through 5.4 present the dynamic modulus master curves developed for the foamed WMA and HMA mixtures. Figure 5.2 presents the master curves developed for foamed WMA and HMA mixtures prepared using 19.0 mm limestone and either PG 70-22
or PG 64-28 asphalt binder. As can be noticed from this figure, lower dynamic modulus values were obtained for foamed WMA mixtures than those obtained for HMA mixtures. It can also be noticed from Figure 5.2 that both foamed WMA and HMA mixtures prepared using PG 70-22 had higher dynamic modulus values than those prepared using PG 64-28. This was expected because the PG 70-22 is a stiffer asphalt binder than the PG 64-28.

![Dynamic Modulus Chart]

**Figure 5.2: Master Curves Obtained for Mixtures Evaluating Binder Type Effect.**

Figure 5.3 presents the master curves developed for foamed WMA and HMA mixtures prepared using PG 70-22 and either 12.5 mm limestone or 12.5 mm crushed gravel. As can be seen from this figure, lower dynamic modulus values were obtained for foamed WMA mixtures prepared using limestone when compared to the corresponding HMA Mixtures. However, foamed WMA mixtures prepared using crushed gravel had higher dynamic modulus values than the HMA mixtures prepared using crushed gravel.
These results might indicate that the use of foamed asphalt binders affects the limestone mixtures differently than the crushed gravel mixtures.

Figure 5.4 shows the master curves developed for the material combinations that allow for evaluating the effect of aggregate size. As can be seen from this figure, both foamed WMA and HMA mixtures prepared using 12.5 mm aggregates had higher dynamic modulus values than those prepared using 19.0 mm aggregates. These results might indicate that the aggregate size might have an effect on the dynamic modulus results. Since this test was conducted under unconfined conditions, it is believed that the effect of confinement was the main reason explaining the difference in dynamic modulus results observed for the 12.5 and 19.0 mm mixtures.

Figure 5.3: Master Curves Obtained for Mixtures Evaluating Aggregate Type Effect.
Figure 5.4: Master Curves Obtained for Mixtures Evaluating Aggregate Size Effect.

5.2.3 Flow Number Test Results

The flow number (FN) test results for foamed WMA and HMA mixtures are presented in Figure 5.5. As can be noticed from this figure, lower FN values were obtained for foamed WMA mixtures than those obtained for HMA Mixtures. This was the case for all material combinations except for mixtures prepared using 19 mm limestone and PG 64-28. These results, in general, might indicate that the foamed WMA mixtures are more susceptible to rutting than the HMA mixtures. Similar to the APA rut depth values, the slight reduction in FN values for foamed WMA mixtures is mainly attributed to the reduced binder aging due to the use of lower production temperatures when producing these mixtures.
Figure 5.5: Flow Number Values for both Foamed WMA and HMA Mixtures.

Results presented in Figure 5.5 can also be examined to evaluate the effect of binder type, aggregate type, and aggregate size on the FN test results. As can be noticed from Figure 5.5, significantly higher FN values were obtained for the 19 mm foamed WMA and HMA mixtures prepared using PG 70-22 than the corresponding mixtures prepared using PG 64-28. These results were expected because the former asphalt binder is stiffer than the latter, as indicated by their performance grades. It can also be noticed from Figure 5.5 that both foamed WMA and HMA mixtures prepared using 12.5 mm limestone and PG 70-22 had higher FN values than those prepared using 12.5 mm crushed gravel and PG 70-22.

Furthermore, by comparing the FN values obtained for both the 12.5 mm foamed WMA and HMA mixtures prepared using limestone and PG 70-22 to those prepared using 19 mm limestone and PG 70-22, the effect of aggregate size and gradation on rut depth
results can be evaluated. As can be seen from Figure 5.5, lower FN values were obtained
for both foamed WMA and HMA mixtures prepared using 19 mm limestone than those
obtained for the corresponding mixtures prepared using 12.5 mm limestone. As expected,
these FN test results indicate that mixtures prepared using larger NMAS aggregates are
more susceptible to rutting.

Table 5.2 presents the ANOVA results obtained for the FN values. As can be
noticed from this table, there is a significant different between the FN values obtained for
the foamed WMA mixtures and those obtained for the HMA mixtures for most
comparisons at a confidence level of 95% (probability < 0.05). The ANOVA results also
show that the binder type has the most significant (highest F-value) effect on the FN values,
followed by the aggregate type and the aggregate size.

Table 5.2: ANOVA Results Obtained for FN Values.

<table>
<thead>
<tr>
<th>Analysis Data</th>
<th>Statistical Factors</th>
<th>F-value</th>
<th>Prob.</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.0 mm, Limestone, PG 64-28 &amp; 19.0 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>8.220</td>
<td>0.046</td>
</tr>
<tr>
<td></td>
<td>Binder Type</td>
<td>97.327</td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>Binder Type × Mix Type</td>
<td>12.538</td>
<td>0.024</td>
</tr>
<tr>
<td>12.5 mm, Gravel, PG 70-22 &amp; 12.5 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>6.170</td>
<td>0.056</td>
</tr>
<tr>
<td></td>
<td>Agg. Type</td>
<td>77.336</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Agg. Type × Mix Type</td>
<td>5.939</td>
<td>0.059</td>
</tr>
<tr>
<td>12.5 mm, Limestone, PG 70-22 &amp; 19.0 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>10.979</td>
<td>0.030</td>
</tr>
<tr>
<td></td>
<td>Agg. Size</td>
<td>43.086</td>
<td>0.003</td>
</tr>
<tr>
<td></td>
<td>Agg. Size × Mix Type</td>
<td>0.503</td>
<td>0.517</td>
</tr>
</tbody>
</table>
5.3 Evaluation of Moisture-Induced Damage

5.3.1 Modified Lottman (AASHTO T 283) Test Results

Figures 5.6 and 5.7 present the AASHTO T 283 test results obtained for foamed WMA and HMA mixtures. Particularly, Figure 5.6 presents the unconditioned and conditioned indirect tensile strength (ITS) values while Figure 5.7 presents the tensile strength ratio (TSR) values calculated as the ratio between the conditioned and unconditioned ITS values. As can be noticed from Figure 5.6, slightly lower unconditioned and conditioned ITS values were obtained for foamed WMA mixtures than those obtained for HMA mixtures. This was the case for all material combinations except for mixtures prepared using 19 mm limestone and PG 64-28 where foamed WMA mixtures had slightly higher unconditioned and conditioned ITS values than the HMA. In addition, it can be observed from Figure 5.7 that foamed WMA and HMA Mixtures had comparable TSR values, suggesting that effect of sample conditioning on foamed WMA and HMA mixtures is similar. Although foamed WMA mixtures exhibited slightly lower unconditioned and conditioned ITS values, it is believed that the moisture susceptibility of these mixtures is similar to that of HMA mixtures since both mixtures had comparable TSR values.

Furthermore, results presented in Figures 5.6 and 5.7 show that both foamed WMA and HMA mixtures prepared using 19 mm limestone and PG 64-28 exhibited significantly lower unconditioned and conditioned ITS values than those mixtures prepared using PG 70-22. However, comparable TSR values were obtained for both
Figure 5.6: AASHTO T 283 Dry and Wet Indirect Tensile Strength (ITS).

Figure 5.7: AASHTO T 283 Tensile Strength Ratio (TSR).
material combinations. Although the 19 mm limestone mixtures prepared using PG 70-22 had higher unconditioned and conditioned ITS values, the impact of sample conditioning is considered to be similar for both material combinations.

Results presented in Figures 5.6 and 5.7 also show that both the foamed WMA and HMA mixtures prepared using 12.5 mm limestone had similar unconditioned ITS, conditioned ITS, and TSR values to those obtained for their corresponding mixtures prepared using crushed gravel. This suggests that preparing foamed WMA and HMA mixtures using 12.5 mm limestone or crushed gravel had the same resistance to moisture-induced damage.

Finally, the effect of aggregate size and gradation can be evaluated by comparing the ITS and TSR results for mixtures prepared using 12.5 mm limestone and PG 70-22 to those mixtures prepared using 19.0 mm limestone and PG 70-22. As can be noticed from Figure 5.6, the unconditioned and conditioned ITS values for the mixtures prepared using 19 mm limestone were similar to those obtained for mixtures prepared using 12.5 mm limestone aggregates. However, the TSR values, presented in Figure 5.7, for the 19 mm limestone mixtures were lower than those obtained for the 12.5 mm limestone mixtures.

Table 5.3 presents the results of the ANOVA conducted to evaluate the effect of the mix type, binder type, aggregate type, aggregate size on the foamed WMA and HMA ITS values. As can be noticed from table, the effect of mix type was not significant at a confidence level of 95% (probability < 0.05) for all comparisons made; indicating that the unconditioned and conditioned TSR values obtained for both foamed WMA and HMA mixtures were similar. It can also be noticed from Table 5.3 that sample conditioning had
a significant impact on the TSR values; suggesting that the conditioned ITS values are different than the unconditioned ITS values. This was expected because the AASHTO T283 test is commonly used to evaluate the moisture susceptibility of asphalt mixtures; hence, this test is capable of capturing that effect of sample conditioning.

Table 5.3: ANOVA Results Obtained for Unconditioned and Conditioned ITS Values.

<table>
<thead>
<tr>
<th>Analysis Data</th>
<th>Statistical Factors</th>
<th>F-value</th>
<th>Prob.</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.0 mm, Limestone, PG 64-28 &amp; 19.0 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>3.150</td>
<td>0.094</td>
</tr>
<tr>
<td></td>
<td>Test Cond.</td>
<td>72.618</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Binder Type</td>
<td>224.969</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Mix Type × Test Cond.</td>
<td>0.104</td>
<td>0.751</td>
</tr>
<tr>
<td></td>
<td>Mix Type × Binder Type</td>
<td>3.337</td>
<td>0.085</td>
</tr>
<tr>
<td></td>
<td>Test Cond. × Binder Type</td>
<td>16.770</td>
<td>0.001</td>
</tr>
<tr>
<td>12.5 mm, Gravel, PG 70-22 &amp; 12.5 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>0.338</td>
<td>0.569</td>
</tr>
<tr>
<td></td>
<td>Test Cond.</td>
<td>15.759</td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>Agg. Type</td>
<td>0.121</td>
<td>0.732</td>
</tr>
<tr>
<td></td>
<td>Mix Type × Test Cond.</td>
<td>0.163</td>
<td>0.692</td>
</tr>
<tr>
<td></td>
<td>Mix Type × Agg. Type</td>
<td>3.094</td>
<td>0.097</td>
</tr>
<tr>
<td></td>
<td>Test Cond. × Agg. Type</td>
<td>0.229</td>
<td>0.639</td>
</tr>
<tr>
<td>12.5 mm, Limestone, PG 70-22 &amp; 19.0 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>1.175</td>
<td>0.294</td>
</tr>
<tr>
<td></td>
<td>Test Cond.</td>
<td>32.344</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Agg. Size</td>
<td>0.078</td>
<td>0.783</td>
</tr>
<tr>
<td></td>
<td>Mix Type × Test Cond.</td>
<td>0.335</td>
<td>0.570</td>
</tr>
<tr>
<td></td>
<td>Mix Type × Agg. Size</td>
<td>1.240</td>
<td>0.281</td>
</tr>
<tr>
<td></td>
<td>Test Cond. × Agg. Size</td>
<td>2.284</td>
<td>0.149</td>
</tr>
</tbody>
</table>
ANOVA results presented in Table 5.3 also show that the binder type had the most significant (highest F-value) effect on the TSR values followed by the aggregate size, and the aggregate type. In addition, the ANOVA results show that the two-way interaction effects between the mix type and mixtures constituents were not significant.

5.3.2 Conditioned Dynamic Modulus Test Results

The unconditioned and conditioned master curves developed for foamed WMA and HMA mixtures are presented in Figure 5.8 through 5.12. Figure 5.8 presents the master curves developed for conditioned and unconditioned foamed WMA and HMA mixtures prepared using 12.5 mm limestone and PG 70-22 asphalt binder. As can be noticed from this figure, lower conditioned and unconditioned dynamic modulus values were obtained for foamed WMA mixtures than those obtained for the corresponding HMA mixtures. It can also be noticed from Figure 5.8 that the effect of sample conditioned was more pronounced on HMA mixtures than foamed WMA mixtures since the between the unconditioned and conditioned dynamic modulus values for HMA mixtures is greater than that for foamed WMA mixtures. It is believed that the reduced binder aging due to the use of lower production temperatures in the case of foamed WMA mixtures is the main explaining these results.

Figure 5.9 shows the master curves developed for unconditioned and conditioned foamed WMA and HMA mixtures prepared using 12.5 mm crushed gravel aggregates and PG 70-22 asphalt binder. As indicated in this figure, comparable unconditioned and conditioned dynamic modulus values were obtained for foamed WMA mixtures.
Figure 5.8: Unconditioned and Conditioned Master Curves for 12.5 mm Limestone.

Figure 5.9: Unconditioned and Conditioned Master Curves for 12.5 mm Crushed Gravel.
However, conditioned HMA mixtures had lower dynamic modulus values than the unconditioned HMA mixtures. Similar to the 12.5 mm limestone mixtures (Figure 5.8), these results indicate that the effect of sample conditioning was more pronounced on the HMA mixtures than on the foamed WMA mixtures.

Figure 5.9 presents the master curves developed for unconditioned and conditioned foamed WMA and HMA mixtures prepared using 19.0 mm limestone aggregates and PG 64-28 asphalt binder. As can be noticed from this figure, both conditioned foamed WMA and HMA mixtures had lower dynamic modulus values than those obtained for the unconditioned mixtures. However, the effect of sample conditioning was more pronounced on foamed WMA mixtures than on HMA mixtures.

Figure 5.10: Unconditioned and Conditioned Master Curves for 19.0 mm Limestone and PG 64-28.
Figure 5.10 presents the master curves developed for conditioned and unconditioned foamed WMA and HMA mixtures prepared using 19.0 mm limestone aggregates and PG 70-22 asphalt binder. As can be noticed from this figure, the reduction in dynamic modulus, due to subjecting sample conditioning, was greater for foamed WMA mixtures and comparable for HMA mixtures. This might indicate that foamed WMA mixtures, prepared using 19.0 mm limestone aggregates and PG 70-22, might be more susceptible to moisture-induced damage than their corresponding HMA mixtures.

![Master Curves](image)

Figure 5.11: Unconditioned and Conditioned Master Curves for 19.0 mm Limestone and PG 70-22.

5.3.3 *Wet Asphalt Pavement Analyzer Test Results*

Figure 5.12 presents the unconditioned and conditioned APA rut depth results obtained for foamed WMA and HMA mixtures. As can be seen from this figure, higher rut...
depth values were obtained for the conditioned foamed WMA and HMA mixtures than those obtained for the unconditioned mixtures. This indicates that both mixture types were impacted by the sample conditioning procedure. It can also be observed from Figure 5.12 that lower rut depth values were obtained for conditioned foamed WMA mixtures than those obtained for HMA mixtures. These results suggest that foamed WMA mixtures were more capable of alleviating the effect of sample conditioning than did the HMA mixtures. This may be attributed to the reduced binder aging expected when producing foamed WMA mixtures at lower than traditional temperature.

Table 5.4 presents the results of the ANOVA results obtained for foamed WMA and HMA unconditioned and conditioned APA rut depth values. This table shows that the asphalt binder type significantly influenced the APA rut depth values (p-value < 0.05) at
Table 5.4: ANOVA Results Obtained for Unconditioned and Conditioned APA Rut Depth Values.

<table>
<thead>
<tr>
<th>Analysis Data</th>
<th>Statistical Factors</th>
<th>F-value</th>
<th>Prob.</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.0 mm, Limestone, PG 64-28 &amp; 19.0 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>0.889</td>
<td>0.359</td>
</tr>
<tr>
<td></td>
<td>Test Cond.</td>
<td>4.164</td>
<td>0.057</td>
</tr>
<tr>
<td></td>
<td>Binder Type</td>
<td>226.157</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Mix Type × Test Cond.</td>
<td>0.183</td>
<td>0.675</td>
</tr>
<tr>
<td></td>
<td>Mix Type × Binder Type</td>
<td>0.830</td>
<td>0.375</td>
</tr>
<tr>
<td></td>
<td>Test Cond. × Binder Type</td>
<td>0.590</td>
<td>0.453</td>
</tr>
<tr>
<td>12.5 mm, Gravel, PG 70-22 &amp; 12.5 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>0.011</td>
<td>0.919</td>
</tr>
<tr>
<td></td>
<td>Test Cond.</td>
<td>3.633</td>
<td>0.074</td>
</tr>
<tr>
<td></td>
<td>Agg. Type</td>
<td>0.158</td>
<td>0.696</td>
</tr>
<tr>
<td></td>
<td>Mix Type × Test Cond.</td>
<td>0.838</td>
<td>0.373</td>
</tr>
<tr>
<td></td>
<td>Mix Type × Agg. Type</td>
<td>1.977</td>
<td>0.178</td>
</tr>
<tr>
<td></td>
<td>Test Cond. × Agg. Type</td>
<td>0.386</td>
<td>0.543</td>
</tr>
<tr>
<td>12.5 mm, Limestone, PG 70-22 &amp; 19.0 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>0.558</td>
<td>0.465</td>
</tr>
<tr>
<td></td>
<td>Test Cond.</td>
<td>2.210</td>
<td>0.155</td>
</tr>
<tr>
<td></td>
<td>Agg. Size</td>
<td>0.047</td>
<td>0.831</td>
</tr>
<tr>
<td></td>
<td>Mix Type × Test Cond.</td>
<td>0.036</td>
<td>0.851</td>
</tr>
<tr>
<td></td>
<td>Mix Type × Agg. Size</td>
<td>0.495</td>
<td>0.491</td>
</tr>
<tr>
<td></td>
<td>Test Cond. × Agg. Size</td>
<td>0.063</td>
<td>0.804</td>
</tr>
</tbody>
</table>

a 95% confidence level. However, the mix type, sample conditioning, aggregate type, and aggregate size did not have a significantly effect on the APA rut depths. This indicates that unconditioned and conditioned rut depth values were not significantly different. In
addition, none of the two-way interactions had a significant effect on the APA rut depths (p-value > 0.05).

5.4 Evaluation of Fatigue Cracking

5.4.1 Dissipated Creep Strain Energy Test Results

Figure 5.13 presents the DCSE values obtained for both foamed WMA and HMA mixtures. As can be noticed from this figure, foamed WMA mixtures had, in general, slightly lower DCSE values than HMA mixtures. This might indicate that foamed WMA mixtures are more susceptible to fatigue cracking than HMA mixtures. Despite this reduction in DCSE values for foamed WMA mixtures, it has been reported, by Roque et al. (2004), that asphalt mixtures having DCSE values higher than 0.02 Btu/ft³ (0.75 KJ/m³) are not expected to develop cracks. Therefore, based on the results of this test, it can be concluded that both foamed WMA and HMA mixtures are not expected to develop fatigue cracks during their service life. These results might also indicate that both mix types (i.e. foamed WMA and HMA) had similar fatigue cracking characteristics.

It can be also seen from Figure 5.13 that foamed WMA and HMA mixtures prepared using 19.0 mm limestone and PG 70-22 asphalt binder had significantly higher DCSE values than those prepared using 19.0 mm limestone and PG 64-28 asphalt binder. Results presented in Figure 5.13 also show that for both foamed and HMA mixtures prepared using 12.5 mm limestone aggregates and PG 70-22 asphalt binder had comparable DCSE values; however, foamed WMA mixtures prepared using 12.5 mm crushed gravel and PG 70-22 had significantly lower DCSE values than its corresponding HMA mixtures.
This might indicate that using crushed gravel when preparing foamed WMA mixtures results in increasing the fatigue cracking potential of these mixtures. Finally, it can be seen from Figure 5.13 that foamed WMA and HMA mixtures prepared using 19.0 mm limestone aggregates and PG 70-22 asphalt binder had higher DCSE values than those prepared using 12.5 limestone and PG 70-22 asphalt binder. This might indicate that the aggregate size affects the DCSE values.

![Figure 5.13: Dissipated Creep Strain Energy (DCSE) Test Results.](image)

Table 5.5 presents the results of an ANOVA conducted to evaluate the effect of the mix type, binder type, aggregate type, aggregate size and their interaction with one another on the foamed WMA and HMA DCSE values. As can be noticed from this table, the ANOVA show that the effect of mix type was not significant at a confidence level of 95%.
(probability < 0.05) for all comparisons made; indicating that the DCSE values obtained for both foamed WMA and HMA mixtures were comparable.

Table 5.5: ANOVA Results Obtained for DCSE Values.

<table>
<thead>
<tr>
<th>Analysis Data</th>
<th>Statistical Factors</th>
<th>F-value</th>
<th>Prob.</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.0 mm, Limestone, PG 64-28 &amp; 19.0 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>0.787</td>
<td>0.404</td>
</tr>
<tr>
<td></td>
<td>Binder Type</td>
<td>45.524</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Binder Type × Mix Type</td>
<td>2.127</td>
<td>0.188</td>
</tr>
<tr>
<td>12.5 mm, Gravel, PG 70-22 &amp; 12.5 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>6.992</td>
<td>0.057</td>
</tr>
<tr>
<td></td>
<td>Agg. Type</td>
<td>2.381</td>
<td>0.198</td>
</tr>
<tr>
<td></td>
<td>Agg. Type × Mix Type</td>
<td>9.467</td>
<td>0.037</td>
</tr>
<tr>
<td>12.5 mm, Limestone, PG 70-22 &amp; 19.0 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>0.006</td>
<td>0.941</td>
</tr>
<tr>
<td></td>
<td>Agg. Size</td>
<td>13.521</td>
<td>0.014</td>
</tr>
<tr>
<td></td>
<td>Agg. Size × Mix Type</td>
<td>0.135</td>
<td>0.728</td>
</tr>
</tbody>
</table>

ANOVA results presented in Table 5.5 show that the binder type had the most significant (highest F-value) effect on the DCSE values followed by the aggregate size, and the aggregate type. Furthermore, although the mix type and aggregate type effects were not significant, the ANOVA results presented in Table 5.5 show that the interaction effects between these two factor is significant. This indicates that the DCSE values for both foamed WMA and HMA mixtures prepared using limestone were different than those prepared using crushed gravel. Moreover, other interaction effects, as shown in Table 5.5, are found to be insignificant. These results imply that the fatigue cracking resistance of foamed WMA and HMA mixtures was influenced by only the aggregate type.

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5.5 Evaluation of Low-Temperature Cracking

5.5.1 Low-Temperature Indirect Tensile Strength Test Results

Figures 5.14 and 5.15 present the ITS test results obtained for foamed WMA and HMA mixtures. Figure 5.14 shows the low-temperature ITS values while Figure 5.15 shows the failure strain values obtained for these mixtures. As can be seen from Figure 5.14, slightly lower ITS values were obtained for foamed WMA mixtures than those obtained for HMA mixtures. In addition, it can be noticed from Figure 5.15 that higher failure strain values were obtained for foamed WMA mixtures than those obtained for HMA mixtures. These results suggest that foamed WMA mixtures will break at low load levels; however, they are slightly more ductile than HMA mixtures since they higher failure strain values.

The effect of binder grade, aggregate type, and aggregate size on the susceptibility of foamed WMA and HMA mixtures to low-temperature cracking can also be examined through observing the results presented in Figure 5.14. As can be seen from this figure, both foamed WMA and HMA mixtures prepared using 19.0 mm limestone and PG 64-28 had significantly lower ITS values than those prepared using PG 70-22. This result might indicate that asphalt mixtures prepared using PG 64-28 are more susceptible to low-temperature cracking than those prepared using PG 70-22. These results were expected because the PG 70-22 asphalt binder is stiffer than the PG 64-28 asphalt binder. Results
Figure 5.14: Indirect Tensile Strength at 14°F (-10°C).

Figure 5.15: Failure Strain Obtained from ITS Test.
presented in Figure 5.15 also show that for both foamed and HMA mixtures prepared using 12.5 mm limestone had higher ITS values than those prepared using 12.5 mm crushed gravel. This indicates asphalt mixtures prepared using limestone aggregate are more resistant to low-temperature cracking than those prepared using crushed gravel. This is mainly attributed to the higher surface roughness and more angularity visually observed for limestone aggregates.

The effect of aggregate size on the low temperature ITS results can be evaluated by comparing the results, presented in Figure 5.14, for both foamed WMA and HMA mixtures prepared using 19.0 mm limestone and PG 70-22 and those prepared using 12.5 mm limestone and PG 70-22. It can be seen from Figure 5.14 that both foamed WMA and HMA mixtures prepared using 12.5 limestone had slightly higher ITS values than those prepared using 19.0 mm limestone. These results might indicate that asphalt mixtures with smaller aggregates are better at resisting low-temperature cracking than those prepared using larger aggregates. However, it should be noted that the effect of confinement might have affect the results of this test as it was conducted under unconfined conditions.

Table 5.6 presents the results of an ANOVA conducted to evaluate the effect of the mix type, binder type, aggregate type, aggregate size and their interaction with one another on the foamed WMA and HMA ITS values. As can be noticed from Table 5.6, the results of the ANOVA show that the effect of mix type was significant at a confidence level of 95% (probability < 0.05) for the material combinations prepared to examine the effect of binder type and aggregate size; whereas, it was insignificant for those prepared to evaluate the effect of aggregate type. This indicates that, in general, the ITS values obtained for
foamed WMA mixtures were different than those obtained for HMA mixtures; suggesting that foamed WMA mixtures showed increased potential for low-temperature cracking.

Table 5.6: ANOVA Results Obtained for Low-Temperature ITS Values.

<table>
<thead>
<tr>
<th>Analysis Data</th>
<th>Statistical Factors</th>
<th>F-value</th>
<th>Prob.</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.0 mm, Limestone, PG 64-28 &amp; 19.0 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>27.887</td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>Binder Type</td>
<td>119.957</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Binder Type × Mix Type</td>
<td>0.028</td>
<td>0.872</td>
</tr>
<tr>
<td>12.5 mm, Gravel, PG 70-22 &amp; 12.5 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>0.776</td>
<td>0.404</td>
</tr>
<tr>
<td></td>
<td>Agg. Type</td>
<td>131.681</td>
<td>0.000</td>
</tr>
<tr>
<td></td>
<td>Agg. Type × Mix Type</td>
<td>1.277</td>
<td>0.291</td>
</tr>
<tr>
<td>12.5 mm, Limestone, PG 70-22 &amp; 19.0 mm, Limestone, PG 70-22</td>
<td>Mix Type</td>
<td>13.093</td>
<td>0.007</td>
</tr>
<tr>
<td></td>
<td>Agg. Size</td>
<td>25.440</td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>Agg. Size × Mix Type</td>
<td>3.642</td>
<td>0.093</td>
</tr>
</tbody>
</table>

ANOVA results presented in Table 5.6 show that the binder type had the most significant (highest F-value) effect on the FN values followed by the aggregate type, and the aggregate size. All these factors were found to significantly affect the ITS values. In addition, the ANOVA results show that the interaction effects between the mix type and mixtures constituents were not significant; implying that the low-temperature cracking resistance of foamed WMA and HMA mixtures was not influenced by the mix constituents.

5.6 Summary and Conclusions

This chapter presented the results of a comprehensive laboratory study conducted to evaluate the performance of foamed WMA and HMA mixtures with regard to permanent
deformation (or rutting), moisture-induced damage (or durability), fatigue cracking, and low-temperature cracking. Several tests were used to for this purpose. In particular, the asphalt pavement analyzer (APA), dynamic modulus, and flow number tests were used to characterize the rutting performance of foamed WMA and HMA mixtures. The susceptibility of foamed WMA and HMA mixtures to moisture-induced damage was characterized using the AASHTO T 283, conditioned dynamic modulus, and wet APA tests. In addition, the fatigue cracking and the low-temperature cracking characteristics of these mixtures were evaluated using the dissipated creep strain energy (DCSE) and low-temperature indirect tensile strength respectively. These tests were conducted on foamed WMA mixtures prepared using fully dried aggregates and according to the current ODOT procedures (i.e. 30°F temperature reduction and 1.8% foaming water content). The reader is referred to Chapter III for more information about the production of foamed WMA mixtures.

Based on the test results and the ANOVA statistical analysis presented in this chapter, the following conclusions were drawn:

- **Permanent Deformation (Rutting):** The foamed WMA mixtures exhibited slightly higher rut depth values in the unconditioned and conditioned APA tests, slightly lower dynamic moduli, and slightly lower flow number values than the traditional HMA mixtures. However, the difference was found to be statistically insignificant. Therefore, the rutting potential of foamed WMA mixtures is expected to be comparable to that of the HMA mixtures.
• **Moisture-Induced Damage (Durability):** The foamed WMA mixtures exhibited slightly lower unconditioned and conditioned ITS values and comparable TSR ratios to the HMA mixtures in the AASHTO T 283 test. In addition, the foamed WMA mixtures exhibited slightly higher unconditioned and conditioned rut depth values in the APA test. However, the effect of the mix type was found to be statistically insignificant on the unconditioned and conditioned ITS values as well as the unconditioned and conditioned APA rut depths. By comparing the unconditioned and conditioned APA rut depths, it was observed that the effect of sample conditioning was more pronounced on the HMA mixtures than the foamed WMA mixtures. This trend was also observed in the unconditioned and conditioned dynamic modulus tests for some of the mixtures.

• **Fatigue Cracking:** The foamed WMA mixtures exhibited slightly lower DCSE values than the HMA mixtures. However, the difference was found to be statistically insignificant. In addition, the DCSE values for all foamed WMA and HMA mixtures were greater than 0.75 kJ/m3, which has been suggested by Roque et al. (2007) as a minimum DCSE threshold value to ensure satisfactory resistance to fatigue cracking. This indicates that both foamed WMA and HMA mixtures are expected to have adequate resistance to fatigue cracking.

• **Low-Temperature Cracking:** The foamed WMA mixtures exhibited slightly lower ITS values at 14°F (-10°C) and comparable or slightly higher failure strain values than the corresponding HMA mixtures. The effect of the mix type was found to be statistically significant on the low temperature ITS values, but not on the failure strains. Since the
HMA mixtures had higher ITS values and similar failure strain values to the foamed WMA mixtures, the HMA mixtures are expected to have better resistance to thermal cracking.
CHAPTER VI

WORKABILITY AND COMPACTABILITY OF FOAMED WMA MIXTURES

6.1 Introduction

Foamed WMA mixtures are generally advertised to have better workability and compactability than HMA mixtures. As a part of this study, investigation of the workability and compactability of foamed WMA and HMA mixtures was conducted. A new workability device was designed and fabricated at the University of Akron to assess the workability of the considered asphalt mixtures whereas the compaction data collected after preparing specimens for the various performance tests, discussed in the previous chapter, were used to examine their compactability characteristics.

This chapter includes a discussion on the effect of using foamed asphalt binders on asphalt binder absorption and how it might affect the workability and compactability of foamed WMA mixtures. A discussion documenting the previous attempts to evaluate the workability of asphalt mixtures is also included. In addition, documentation of the development of the new workability device is presented in this chapter. This chapter also includes a discussion on the workability and compactability results obtained by conducting tests using the new workability device and analyzing compaction data respectively. Finally, this chapter is concluded with a summary and conclusions section documenting the main conclusions drawn from the results.
6.2 Effect of Foamed Asphalt Binders on Asphalt Binder Absorption

Asphalt binder absorption plays an important role in determining the workability of asphalt mixtures. Table 6.1 presents the asphalt binder absorption by weight of aggregate, \( P_{ba} \), and the effective asphalt binder content, \( P_{be} \), for both HMA and foamed WMA mixtures. The asphalt binder absorption was calculated from the bulk and effective specific gravities of the aggregates. As can be noticed from this table, the asphalt binder absorption in the foamed WMA mixtures is slightly lower than that of the HMA mixtures. The reduction in asphalt binder absorption is mainly attributed to the use of lower mixing temperatures during the production of foamed WMA mixtures. The lower asphalt binder absorption values obtained for the foamed WMA mixtures indicate that these mixtures contain more effective asphalt binder than the corresponding HMA mixtures, resulting in more asphalt binder being available to coat the aggregate particles.

6.3 Previous Workability Devices

Over the last three decades, several workability devices have been developed to measure the workability of asphalt mixtures. All these devices utilized the torque generated while stirring a mix to measure the workability. The first workability device was developed by Marvillet and Bougault (1978). It consisted of a rigid frame, a motor, a mixing blade, a chamber, a spring, and a potentiometer (Figure 6.1). The operation of the device involved placing approximately 33 lb (15 kg) of asphalt mixture into the mixing chamber. The mixing blade was then rotated at a constant speed of 22 revolutions per minute (rpm), and the temperature of the chamber was increased from 302°F to 392°F (150°C to 200°C) at a rate of 1.8°F/min (1°C/min). The resistance of the mixture to the rotation of the blade was
quantified using the torque needed to rotate the blade as measured using a potentiometer and a spring. A higher torque value was used as an indication of poor workability, while a lower torque value was used as an indication of good workability.

Table 6.1: Asphalt Binder Absorption and Effective Asphalt Binder Content for Both HMA and Foamed WMA Mixtures.

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Aggregate Type</th>
<th>Aggregate NMAS (mm)</th>
<th>Binder Grade</th>
<th>Asphalt Binder Absorption, $P_{ba}$ (%)</th>
<th>Effective Asphalt Binder, $P_{be}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HMA</td>
<td>Limestone</td>
<td>12.5</td>
<td>PG 70-22</td>
<td>1.61</td>
<td>4.18</td>
</tr>
<tr>
<td></td>
<td>Limestone</td>
<td>19.0</td>
<td>PG 70-22</td>
<td>1.20</td>
<td>3.46</td>
</tr>
<tr>
<td></td>
<td>Limestone</td>
<td>19.0</td>
<td>PG 64-22</td>
<td>1.35</td>
<td>3.41</td>
</tr>
<tr>
<td></td>
<td>Gravel</td>
<td>12.5</td>
<td>PG 70-22</td>
<td>1.17</td>
<td>4.70</td>
</tr>
<tr>
<td>WMA</td>
<td>Limestone</td>
<td>12.5</td>
<td>PG 70-22</td>
<td>1.47</td>
<td>4.31</td>
</tr>
<tr>
<td></td>
<td>Limestone</td>
<td>19.0</td>
<td>PG 70-22</td>
<td>1.04</td>
<td>3.61</td>
</tr>
<tr>
<td></td>
<td>Limestone</td>
<td>19.0</td>
<td>PG 64-22</td>
<td>1.18</td>
<td>3.58</td>
</tr>
<tr>
<td></td>
<td>Gravel</td>
<td>12.5</td>
<td>PG 70-22</td>
<td>1.06</td>
<td>4.80</td>
</tr>
</tbody>
</table>

Another attempt to measure the workability of asphalt mixtures was made by the National Center for Asphalt Technology (NCAT); (Gudimettla et al. 2003). Figure 6.2 shows a photograph of the prototype device that was built as part of that study. As can be seen from this figure, the prototype workability device consisted of a motor, an iron frame, an instrumentation unit, a shaft connected to a paddle, and a sample bowl. Although the
device was based on the same concept suggested by Marvillet and Bougault (1978), it included several features to improve mixing quality and measurement of torque values.

Figure 6.1: Marvillet and Bougault (1978) Workability Device
(after Marvillet and Bougault 1978).

Figure 6.2: NCAT Workability Device (after Gudimettla et al. 2003).
In another study conducted by Tao and Mallick (2009), a relatively simple workability device consisting of a metal bucket, a paddle, and a torque wrench (Figure 6.3) was used to evaluate the effects of different WMA additives on the workability of asphalt mixtures containing 100% reclaimed asphalt pavements (RAP). This device is similar in concept to the previously mentioned devices; however, a torque wrench is used instead of a motor to rotate the paddle. The test procedure involved conditioning about 39.7 lb (18 kg) of RAP for 4 hours at 257°F (125°C). The conditioned RAP was mixed with the WMA additive (Sasobit H8 or Advera zeolite) and placed in the metal bucket. The torque wrench was then rotated four separate times and the torque value and temperature of the mix were recorded in each rotation. Finally, a workability value was calculated by multiplying the inverse of the average torque by 1,000.

Figure 6.3: Tao and Mallick (2009) Workability Device (after Tao and Mallick 2009).

Another workability device was recently developed by the University of Massachusetts, Dartmouth (Mogawer and Austerman 2010). As can be seen from Figure
6.4, the device consists of a motor, a steel frame, a mixing bucket, a paddle, a thermocouple embedded into the paddle, and a stationary torque sensor. Similar to the previous workability devices, this device uses torque to quantify mix workability. However, the loose mixture is placed inside a rotating bucket and mixed by a stationary paddle attached to a torque sensor. This design is expected to produce more reliable torque values as measurements are made by a stationary torque sensor rather than strain gages or a torque wrench.

Figure 6.4: University of Massachusetts Workability Device (after Bennert et al. 2010).

In summary, several attempts have been made to measure the workability of asphalt mixtures. These attempts resulted in developing several workability devices that varied in their complexity and operational procedure, resulting in varying degrees in accuracy for
measuring the workability of asphalt mixtures. The following subsection describes the design and fabrication of a new workability device developed at the University of Akron that incorporates some of the features of previous devices, while utilizing recent advances in measurement technologies.

6.4 The University of Akron Workability Device

As discussed previously, workability of asphalt mixtures has been typically quantified using the torque generated while stirring a mix. Previous workability devices consisted of either a stationary bucket and a rotating mixing paddle or a rotating bucket and a stationary mixing paddle. The former utilized springs and potentiometers, strain gages, or a rotating torque sensor to measure the torque, while the latter employed a stationary torque sensor for this purpose. The second approach provides more reliable results because torque measurements are made on a stationary shaft. Therefore, it was incorporated in the design of the new workability device.

The final design of the new device includes: (1) a rotating bucket, (2) a stationary mixing paddle, (3) a motor, (4) a gear reduction unit, (5) a variable speed drive, (6) temperature and torque sensors, and (7) a data acquisition system. Figure 6.5 shows a photograph of the new workability device. As can be noticed from this figure, the new workability device also included a steel frame that would support the different components of the device and several safety features (such as a safety cage and an emergency stop button) to protect the users and the sensors.
Several considerations were made in the design of the new workability device. The size of the rotating bucket was found to be dependent on the required mixture weight that would ensure repeatable workability results. Previous studies in the literature reported obtaining satisfactory results using weights ranging from 26.5 to 44.1 lb (12 to 20 kg). Therefore, the rotating bucket was designed to accommodate a mixture weight of 44.1 lb (20 kg). The selected bucket design was a cylinder with an inner diameter of 13 inches (330.2 mm) and a height of 16 inches (406.4 mm). The bucket height was selected to be
greater than the minimum required height so as to better contain the asphalt mixture during the test.

The mixing paddle was designed to thoroughly stir the asphalt mixture as it rotates inside the bucket. The selected paddle design consisted of three blades connected to a 1.5-inch (38.1-mm) shaft. The blades were positioned 120° apart and were attached at different locations along the length of the shaft. The blades were 5 inches (127.0 mm) in length, 0.25 inches (6.35 mm) in thickness, and 1.5 inches (38.1 mm) in width. The design incorporated a spacing of 1.5 inches (38.1 mm) between the bottom of the bucket and the lowest blade and a spacing of 0.5 inches (12.7 mm) between the inner side of the bucket and the tips of the blades to avoid spikes in torque readings. This design is expected to produce more consistent results than a design utilizing two blades 180° apart as has been used in previous workability devices.

The motor, gear-reduction unit, and speed drive control unit were selected to allow the device to operate at various speeds and handle the torque generated during the test. Based on typical speed and torque values reported in the literature, the motor was selected to operate at 1,700 rpm with a torque capacity of 2,800 in-lb (316.4 N.m). The speed of the motor was reduced using a gear-reduction unit that reduces the speed at a ratio of 80 motor rotations to 1 gear rotation. A variable speed drive unit was also used to control the speed of rotation to range from 10 rpm to 30 rpm.

A stationary torque sensor with a capacity of 2,000 in-lb (226.0 N.m) and an accuracy of ± 0.1% was used in the workability device. The stationary torque sensor was attached to the mixing paddle and anchored to the upper mounting bracket. Various
stationary torque sensor designs are commercially available, including: solid flange design, flange style design, hollow flange design, and rod end design. The flange style design was selected because it is easier to mount and communicate with.

An infrared thermometer was used to monitor the temperature of the asphalt mixtures during the test. The infrared thermometer had a range of -40°F to 1,112°F (-40°C to 600°C) and an accuracy of ± 1% or ± 1.8°F (1°C), whichever is greater. The infrared thermometer was attached to the upper mounting bracket and aimed at the asphalt mixture. Given that the asphalt mixture is thoroughly mixed during the test, the temperature recorded by the infrared thermometer is expected to be close to the actual mix temperature. As discussed in the following section, the asphalt mixture was heated to an initial temperature of 302°F (150°C) before being placed in the rotating bucket and allowed to drop to a final temperature of 194°F (90°C) during the test. This range was selected because it allows for measuring the workability at the typical compaction temperature for traditional asphalt mixtures. The advantage of this approach is that it allows for quantifying the rate at which the mix temperature is dropping, which may aid in identifying the time available for compaction.

Finally, the new workability device consisted of a data acquisition system that allowed for obtaining real-time torque and temperature readings, which were obtained at half-second intervals and presented on a computer screen in graphical and tabular formats.
6.5 Workability Testing Procedure

The new workability device was used to evaluate the workability of the various material combinations. To obtain consistent results, the test was performed using an asphalt mixture weighing approximately 39.7 lb (18 kg). Three 13.2-lb (6-kg) aggregate batches were prepared and mixed with the heated asphalt binder to produce the asphalt mixture. The mix was prepared in three batches because of the capacity of the mechanical mixer. Each batch was placed in a heating pan and conditioned at 302°F (150°C) for about 45 minutes prior to testing. This step was implemented to ensure that the material uniformly reached the desired testing temperature. The rotating bucket and the stationary mixing paddle of the workability device were also heated to 302°F (150°C) and maintained at that temperature until the beginning of the test.

Upon completing the heating and conditioning step, the rotating bucket was transferred to the workability device and the three batches were placed inside the bucket. The stationary mixing paddle was attached to the workability device and the safety cage was lowered to its final position. The data recording software was launched to record the torque and temperature readings. Finally, the workability test was started by rotating the mixing bucket at a constant speed of 15 rpm. During the test, the temperature of the asphalt mixture dropped resulting in an increase in the torque exerted by the asphalt mixture on the mixing paddle. Testing continued until the mix reached a temperature of 212°F (100°C) or high variability was observed in the measured torque data. In general, the workability test lasted approximately one hour.
6.6 Workability and Compactability Results

6.6.1 Workability

The workability test results are generally presented in the form of torque versus temperature. Example torque versus temperature curves obtained for 12.5 mm HMA and foamed WMA asphalt mixtures prepared using limestone aggregates and PG 70-22 asphalt binder are presented in Figure 6.6. As can be noticed from this figure, the torque values obtained for the foamed WMA mixture were lower than those obtained for the corresponding HMA mixture at all temperatures. This trend was observed for all material combinations as discussed in the following paragraphs. It can also be observed from this figure that the measured torque was at its lowest value and was relatively constant at the beginning of the test. As the temperature decreased, the torque value increased due to the increase in the viscosity of the asphalt binder. Additionally, it can be noticed from this figure that the torque values showed little variation at the beginning of the test and higher variation with the decrease in temperature. Once the temperature dropped below 212°F (100°C), the variability in the torque readings increased significantly due to the stiffening of the asphalt binder and the formation of large clumps within the asphalt mixture.

An exponential model \( y = ae^{b \cdot x} \) was used to capture the effect of the testing temperature on the torque readings. Table 6.2 presents the exponential models obtained for all material combinations and the corresponding coefficient of determination, \( R^2 \). As can be observed from this table, all workability models had a negative b value indicating an increase in the torque with the decrease in temperature. In addition, it can be observed that all workability models had an \( R^2 \) value greater than 0.75, with most asphalt mixtures having
an $R^2$ value greater than 0.85. These relatively high $R^2$ values indicate that the exponential model can be successfully used to describe the workability test data. It should be noted that all torque readings were used in the development of the exponential models without having to exclude any outliers. This implies that the new workability device was capable of producing consistent torque and temperature readings.

Figure 6.6: Torque versus Temperature Curves Obtained for 12.5 mm HMA and Foamed WMA Mixtures Prepared using Limestone Aggregates and PG 70-22 Asphalt Binder.

Figure 6.7 presents the average torque values obtained at high and low testing temperatures ($302^\circ$F and $212^\circ$F ($150^\circ$C and $100^\circ$C), respectively). As can be noticed from this figure, the foamed WMA mixtures had lower torque readings than the corresponding HMA mixtures for both high and low testing temperatures. This difference in torque values can be attributed to the reduction in asphalt binder absorption for the foamed WMA.
mixtures. Another factor that might have contributed to the reduction in the torque values for the foamed WMA mixtures is the presence of vapor pockets entrapped within the foamed asphalt binder that keeps the binder slightly expanded and reduce its viscosity.

Table 6.2: Workability Exponential Models.

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Aggregate Type</th>
<th>Aggregate NMAS (mm)</th>
<th>Binder Grade</th>
<th>Workability Model</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>HMA</td>
<td>Limestone</td>
<td>12.5</td>
<td>PG 70-22</td>
<td>$Torque = 4,160 e^{-0.025 \text{Temp}}$</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>Limestone</td>
<td>19.0</td>
<td>PG 70-22</td>
<td>$Torque = 2,179 e^{-0.017 \text{Temp}}$</td>
<td>0.87</td>
</tr>
<tr>
<td></td>
<td>Limestone</td>
<td>19.0</td>
<td>PG 64-28</td>
<td>$Torque = 742 e^{-0.012 \text{Temp}}$</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td>Gravel</td>
<td>12.5</td>
<td>PG 70-22</td>
<td>$Torque = 1,611 e^{-0.019 \text{Temp}}$</td>
<td>0.95</td>
</tr>
<tr>
<td>WMA</td>
<td>Limestone</td>
<td>12.5</td>
<td>PG 70-22</td>
<td>$Torque = 4,385 e^{-0.032 \text{Temp}}$</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>Limestone</td>
<td>19.0</td>
<td>PG 70-22</td>
<td>$Torque = 2,183 e^{-0.022 \text{Temp}}$</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>Limestone</td>
<td>19.0</td>
<td>PG 64-28</td>
<td>$Torque = 964 e^{-0.018 \text{Temp}}$</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>Gravel</td>
<td>12.5</td>
<td>PG 70-22</td>
<td>$Torque = 3,426 e^{-0.028 \text{Temp}}$</td>
<td>0.94</td>
</tr>
</tbody>
</table>

By comparing the torque values obtained for the 19.0 mm foamed WMA and HMA mixtures prepared using PG 64-28 and PG 70-22 asphalt binders, it can be observed that lower torque values were obtained for asphalt mixtures prepared using the PG 64-28 asphalt binder than the PG 70-22 asphalt binder. This was the case for both HMA and foamed WMA asphalt mixtures, which indicates that asphalt mixture workability increases with the decrease in the viscosity of the asphalt binder. This observation is consistent with results reported by Marvillet and Bougalt (1979) and Gudimetla et al. (2003).
By comparing the torque values obtained for the 12.5 mm foamed WMA and HMA mixtures prepared using limestone and crushed gravel, it can be observed that lower torque
values were obtained for HMA mixtures prepared using crushed gravel than those prepared using limestone aggregates, whereas lower torque values were obtained for foamed WMA mixtures prepared using limestone aggregates than those prepared using crushed gravel. The lower torque values obtained for the HMA mixtures prepared using crushed gravel can be attributed to the use of aggregate particles that are less angular. However, the higher torque values obtained for the foamed WMA mixtures prepared using crushed gravel can be attributed to the slightly coarser aggregate gradation used in preparing these mixtures.

Finally, by comparing the torque values obtained for the 12.5 mm and 19.0 mm foamed WMA and HMA mixtures prepared using limestone aggregates and PG 70-22 asphalt binder, it can be observed that the 12.5 mm surface mixtures had lower torque values than the 19.0 mm intermediate mixtures. This was the case for both HMA and foamed WMA asphalt mixtures. This indicates that the surface mixtures are more workable than the intermediate mixtures. In other words, asphalt mixture workability decreases with the increase in nominal maximum aggregate size.

6.6.2 Compactability

As previously mentioned, the compaction data collected from the gyratory compactor were used to examine the compactability of both foamed WMA and HMA mixtures. Table 6.3 presents the number of gyrations required to compact the specimens used in conducting the performance testing. As can be seen from this table, both foamed WMA and HMA mixtures had comparable compaction effort (or number of gyrations). This result indicates that foamed WMA mixtures might require a similar compaction effort to that needed for HMA mixtures in the field. It is noted; however, that foamed WMA
mixtures were compacted at 30°F (16.7°C) lower than traditional HMA compaction temperatures. Therefore, it is believed that the use of foamed asphalt binders has resulted in improving the compaction of foamed WMA mixtures.

Table 6.3: Number of Gyrations Required to Compact Testing Specimens.

<table>
<thead>
<tr>
<th>Mix</th>
<th>Agg. Type</th>
<th>Agg. Size</th>
<th>Binder Type</th>
<th>APA</th>
<th>T283</th>
<th>E*</th>
<th>ITS/DCSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>HMA</td>
<td>LS</td>
<td>12.5 mm</td>
<td>PG 70-22</td>
<td>38</td>
<td>36</td>
<td>29</td>
<td>41</td>
</tr>
<tr>
<td>HMA</td>
<td>LS</td>
<td>19.0 mm</td>
<td>PG 70-22</td>
<td>23</td>
<td>23</td>
<td>18</td>
<td>27</td>
</tr>
<tr>
<td>HMA</td>
<td>LS</td>
<td>19.0 mm</td>
<td>PG 64-22</td>
<td>29</td>
<td>22</td>
<td>18</td>
<td>24</td>
</tr>
<tr>
<td>HMA</td>
<td>GR</td>
<td>12.5 mm</td>
<td>PG 70-22</td>
<td>15</td>
<td>15</td>
<td>12</td>
<td>14</td>
</tr>
<tr>
<td>WMA</td>
<td>LS</td>
<td>12.5 mm</td>
<td>PG 70-22</td>
<td>43</td>
<td>28</td>
<td>29</td>
<td>38</td>
</tr>
<tr>
<td>WMA</td>
<td>LS</td>
<td>19.0 mm</td>
<td>PG 70-22</td>
<td>27</td>
<td>22</td>
<td>18</td>
<td>24</td>
</tr>
<tr>
<td>WMA</td>
<td>LS</td>
<td>19.0 mm</td>
<td>PG 64-22</td>
<td>27</td>
<td>17</td>
<td>17</td>
<td>18</td>
</tr>
<tr>
<td>WMA</td>
<td>GR</td>
<td>12.5 mm</td>
<td>PG 70-22</td>
<td>16</td>
<td>12</td>
<td>9</td>
<td>14</td>
</tr>
</tbody>
</table>

By comparing the number of gyrations obtained for the 19.0 mm foamed WMA and HMA mixtures prepared using PG 64-28 and PG 70-22 asphalt binders, it can be observed that in the case of HMA mixtures both material combinations required comparable compaction efforts. However, in the case of foamed WMA mixtures, specimens prepared using the PG 64-28 asphalt binder needed lower compaction effort than those prepared using PG 70-22. As expected, this result indicates that the effect of foaming is more pronounced on softer asphalt binders than it is on stiffer asphalt binders; thus, explaining the improvement in the compactability of foamed WMA mixtures prepared using PG 64-28 asphalt binder.
By comparing the number of gyrations obtained for the 12.5 mm foamed WMA and HMA mixtures prepared using limestone and crushed gravel, it can be observed that significantly lower compaction effort was needed for both foamed WMA and HMA mixtures prepared using crushed gravel than those prepared using limestone aggregates. This might indicate that using crushed gravel aggregates in preparing asphalt mixtures results in more compactable mixtures than when using limestone aggregates. The main reasons for which these results might be attributed include the smoother surfaces and less angularity observed for the crushed gravel aggregates.

Finally, by comparing the number of gyrations obtained for the 12.5 mm and 19.0 mm foamed WMA and HMA mixtures prepared using limestone aggregates and PG 70-22 asphalt binder, it can be observed that specimens prepared using the 19.0 mm NMAS limestone aggregates required significantly lower compaction effort than those prepared using the 12.5 mm NMAS limestone aggregates. This was the case for both HMA and foamed WMA asphalt mixtures. This indicates that the surface mixtures are less compactable than the intermediate mixtures. In other words, asphalt mixture compactability improves with the increase in nominal maximum aggregate size.

6.7 Summary and Conclusions

A new device was designed and fabricated to evaluate the workability of foamed WMA and HMA mixtures. This device utilized the torque generated while stirring a mix to measure the workability. Each workability test was performed on mixtures heated to 150°C and the test was terminated when the mixture’s temperature reached 100°C. The new device had several advantages, including the ability to thoroughly mix the asphalt
mixture using an improved mixing paddle design; the ability to obtain accurate temperature and torque measurements using an infrared thermometer and a stationary torque sensor; the ability to run the test at varying speeds ranging from 5 to 35 rpm using a motor and a speed drive control unit; the ability to record test results to a personal computer; and improved safety features such as a specially designed safety cage and an emergency stop button. In addition, the compactability of the foamed WMA and HMA mixtures was examined by analyzing compaction data obtained using the Superpave gyratory compactor during the preparation of laboratory test specimens.

The following conclusions were made based on the workability test results and the analysis of the compaction data:

- **Workability**: The foamed WMA mixtures exhibited better workability than the traditional HMA mixtures. This was attributed to the lower asphalt binder absorption observed for the foamed WMA mixtures. Another factor that might have contributed to the improvement in workability for foamed WMA mixtures is the presence of vapor pockets entrapped within the foamed asphalt binder that serve to keep the binder slightly expanded and reduce its viscosity. The workability of the foamed WMA and HMA mixtures was found to be affected by the binder grade, aggregate type, and aggregate size. Foamed WMA and HMA mixtures prepared using PG 64-28 asphalt binder had better workability than those prepared using PG 70-22. This indicates that using a softer asphalt binder results in better workability. In addition, the HMA mixtures prepared using crushed gravel had better workability than those prepared using limestone aggregates. However, foamed WMA mixtures prepared using
limestone aggregates had better workability than those prepared using crushed gravel, which suggests that the aggregate type affects foamed WMA mixtures differently than HMA mixtures. Furthermore, the 12.5 mm surface mixtures showed better workability than the 19.0 mm intermediate mixtures for both foamed WMA and HMA mixtures, which indicates that the use of a smaller nominal maximum aggregate size (NMAS) results in better workability.

- **Compactability:** By comparing the compaction data obtained using the Superpave gyratory compactor during the preparation of the laboratory specimens, it was observed that the number of gyrations needed to achieve the target air void levels for the foamed WMA specimens was relatively close to that of the HMA specimens. This indicates that the compactability of the foamed WMA mixtures is comparable to that of the corresponding HMA mixtures. In a previous study conducted by the first and the fourth authors, it was observed that the foamed WMA mixtures had significantly better compactability than the HMA mixtures (Abbas and Ali, 2011). The asphalt mixtures used in that study had a smaller aggregate size and were prepared using higher asphalt binder contents. A significant reduction in asphalt binder absorption was observed for the foamed WMA mixtures, leading to significant improvement in compactability.
CHAPTER VII

EFFECT OF MIX PREPARATION PARAMETERS
ON PERFORMANCE OF FOAMED WMA MIXTURES

7.1 Introduction

As discussed in Chapter III, foamed WMA mixtures are typically produced after selecting three mix preparation parameters. These parameters include the reduction in production temperatures, foaming water content, and aggregate moisture content. To examine the effect of these parameters on the permanent deformation (rutting) and moisture-induced damage characteristics of foamed WMA mixtures, the asphalt pavement analyzer (APA), indirect tensile strength at 77°F (25°C), and modified Lottman (AASHTO T283) tests were used. In addition, the performance of the considered foamed WMA mixtures, prepared at different mix preparation parameters, was compared to that of HMA mixtures prepared using fully dried aggregates.

Figure 7.1 presents the testing plan conducted to evaluate the effect of the considered mix preparation parameters on performance. As can be seen from this figure, the testing factorial was designed to compare the performance of foamed WMA and HMA, and determine the effect of the temperature reduction, foaming water content, and aggregate moisture content on the performance of the foamed WMA. The foamed WMA mixtures were produced at 30°F (16.7°C), 50°F (27.8°C), and 70°F (38.9°C) lower than the
HMA mixtures. A foaming water content of 1.8%, 2.2%, and 2.6% by weight of the asphalt binder was used in the production of the foamed WMA mixtures. In addition, fully dried aggregates as well as moist aggregates with a moisture content of approximately 1.5% and 3.0% were used in the preparation of the foamed WMA mixtures. Preparing foamed WMA mixtures with moist aggregates involved determining the time required to heat the aggregate until reaching the target moisture content.

Figure 7.1: Testing Plan Prepared to Examine the Effect of Foamed WMA Mix Preparation Parameters on Performance.

This chapter presents the experimental test results and how foamed WMA mix preparation parameters affected rutting and moisture-induced damage. In addition, this
chapter provides the outcome of the Analysis of Variance (ANOVA) that was conducted using the Statistical Package for Social Sciences (SPSS) to examine the significance of the foamed WMA mix preparation parameters and their interaction with binder type, aggregate type, and aggregate size on performance.

7.2 Effect of Temperature Reduction

Figure 7.2 presents the effect of the temperature reduction on mix performance. Figure 7.2a shows the APA test results; Figures 7.2b and 7.2c show the dry and wet ITS test results, respectively; and Figure 7.2d shows the TSR test results. As indicated in the flow chart presented in Figure 7.1, the foamed WMA mixtures used to determine the effect of the temperature reduction were produced using a foaming water content of 1.8% and fully dried aggregates. As can be seen in Figure 7.2a, the rutting performance of foamed WMA mixtures produced using 30°F (16.7°C) temperature reduction was comparable to that of HMA. However, higher rut depths were obtained for the foamed WMA mixtures produced using 50°F (27.8°C) and 70°F (38.9°C) temperature reductions. This indicates that reducing the production temperature of foamed WMA may lead to increased susceptibility to permanent deformation (or rutting). Similar results were obtained for the dry and wet ITS. As can be observed from Figures 7.2b and 7.2c, comparable dry and wet ITS values were obtained for the HMA and the foamed WMA mixtures produced using 30°F (16.7°C) temperature reduction. However, lower dry and wet ITS values were obtained for the foamed WMA mixtures produced using 50°F (27.8°C) and 70°F (38.9°C) temperature reductions. The increase in APA rut depth and reduction in dry and wet ITS can be attributed to the softening of the asphalt binder due to foaming, reduced binder aging.
due to the use of lower production temperature, and reduced binder absorption at lower production temperatures. Figure 7.2d shows that in general both foamed WMA and HMA mixtures met the minimum TSR requirement of 0.8. However, there is no clear trend on the effect of the temperature reduction. It is noted though that the wet ITS values for the foamed WMA mixtures produced using 50°F (27.8°C) and 70°F (38.9°C) temperature reductions were very low, indicating that these mixtures might be more susceptible to moisture-induced damage.

An Analysis of Variance (ANOVA) was conducted to evaluate the effect of the temperature reduction and its interaction with the binder type, aggregate type, and aggregate size on the foamed WMA performance (Table 7.1). Given that a partial factorial was used in the experimental testing plan (i.e., four material combinations were used instead of a full factorial of eight), selected material combinations were included in the analysis. As can be noticed from this table, the binder type had the most significant effect (highest F-value) on the foamed WMA APA rut depth, dry ITS, and wet ITS, followed by the aggregate type, and the aggregate size. Furthermore, the effect of the temperature reduction was significant at a 95% confidence level (probability < 0.05) for all comparisons. However, the effect of the interaction between the temperature reduction and the mix constituents was generally not significant. This implies that the effect of the temperature reduction on foamed WMA performance is not influenced by the mix constituents.
Figure 7.2: Effect of Temperature Reduction on Mix Performance (a. APA Test Results, b. Dry ITS Test Results, c. Wet ITS Test Results, and d. TSR Test Results).
Table 7.1: Effect of Temperature Reduction on Foamed WMA Performance.

<table>
<thead>
<tr>
<th>Analysis Data</th>
<th>Statistical Factors</th>
<th>APA Rut Depth</th>
<th>Dry ITS</th>
<th>Wet ITS</th>
</tr>
</thead>
<tbody>
<tr>
<td>WMA, 19.0 mm, Limestone, PG 64-28 &amp; WMA, 19.0 mm, Limestone, PG 70-22</td>
<td>Binder Type</td>
<td>136.4</td>
<td>0.00</td>
<td>97.5</td>
</tr>
<tr>
<td></td>
<td>Prod. Temp.</td>
<td>5.5</td>
<td>0.02</td>
<td>22.8</td>
</tr>
<tr>
<td></td>
<td>Binder Type × Prod. Temp.</td>
<td>1.4</td>
<td>0.29</td>
<td>7.3</td>
</tr>
<tr>
<td>WMA, 12.5 mm, Gravel, PG 70-22 &amp; WMA, 12.5 mm, Limestone, PG 70-22</td>
<td>Agg. Type</td>
<td>55.3</td>
<td>0.00</td>
<td>18.8</td>
</tr>
<tr>
<td></td>
<td>Prod. Temp.</td>
<td>53.3</td>
<td>0.00</td>
<td>40.7</td>
</tr>
<tr>
<td></td>
<td>Agg. Type × Prod. Temp.</td>
<td>4.7</td>
<td>0.03</td>
<td>0.5</td>
</tr>
<tr>
<td>WMA, 12.5 mm, Limestone, PG 70-22 &amp; WMA, 19.0 mm, Limestone, PG 70-22</td>
<td>Agg. Size</td>
<td>4.4</td>
<td>0.06</td>
<td>3.3</td>
</tr>
<tr>
<td></td>
<td>Prod. Temp.</td>
<td>20.7</td>
<td>0.00</td>
<td>24.4</td>
</tr>
<tr>
<td></td>
<td>Agg. Size × Prod. Temp.</td>
<td>0.6</td>
<td>0.54</td>
<td>2.8</td>
</tr>
</tbody>
</table>
7.3 Effect of Foaming Water Content

Figure 7.3 presents the effect of the foaming water content on mix performance. Figure 7.3a shows the APA test results; Figures 7.3b and 7.3c show the dry and wet ITS test results, respectively; and Figure 7.3d shows the TSR test results. As indicated in the flow chart presented in Figure 7.1, the foamed WMA mixtures used to determine the effect of the foaming water content were produced using 30°F (16.7°C) temperature reduction and fully dried aggregates. As can be seen in Figure 7.3a, the rutting performance of the foamed WMA mixtures produced using varying foaming water contents was generally comparable to that of the HMA, with some foamed WMA mixtures showing a slight improvement in rutting performance with the increase in foaming water content. Little difference was also observed for the dry and wet ITS. As can be seen in Figures 7.3b and 7.3c, comparable dry and wet ITS values were obtained for the HMA and the foamed WMA mixtures prepared using the various foamed water contents. It is noted that some of the foamed WMA mixtures showed a slight reduction in dry ITS and no change in wet ITS with the increase in foaming water content, resulting in a higher TSR value. Given that increasing the foaming water content had little effect on wet ITS, this parameter is not expected to greatly affect the moisture susceptibility of foamed WMA mixtures provided that a reasonable foaming water content level is used.

Table 7.2 presents the results of the ANOVA analysis conducted to evaluate the effect of the foaming water content and its interaction with the binder type, aggregate type, and aggregate size on the foamed WMA performance. As can be noticed from this table, the effect of the foaming water content was not significant on the dry and wet ITS test
results. However, it was significant on the APA rut depths. Similar to the temperature reduction, the interaction between the foaming water content and the mix constituents was not significant on the rutting test results. This suggests that the effect of the foaming water content on foamed WMA performance is not influenced by the binder type, aggregate type, or aggregate size.

7.4 Effect of Aggregate Moisture Content

Figure 7.4 presents the effect of the aggregate moisture content on mix performance. Figure 7.4a shows the APA test results; Figures 7.4b and 7.4c show the dry and wet ITS test results, respectively; and Figure 7.4d shows the TSR test results. As indicated in the flow chart presented in Figure 7.1, the foamed WMA mixtures used to determine the effect of the aggregate moisture content were produced using 30°F (16.7°C) temperature reduction and 1.8% foaming water content. As can be seen in Figure 7.4a, the rutting performance of the foamed WMA mixtures was in general comparable to that of the HMA. However, using moist aggregate in the production of foamed WMA mixtures resulted in widely variable APA test results. As can be observed from Figures 7.4b and 7.4c, lower dry and wet ITS values were generally obtained for foamed WMA mixtures prepared using moist aggregates, with the lowest dry and wet ITS values obtained for foamed WMA mixtures prepared using aggregates having 3% moisture content. Even though no significant difference in wet ITS was observed for foamed WMA mixtures containing moist aggregates (Figure 7.4c) and higher TSR values were obtained for these mixtures (Figure 7.4d), inadequate aggregate coating was noticed
Figure 7.3: Effect of Foaming Water Content on Mix Performance (a. APA Test Results, b. Dry ITS Test Results, c. Wet ITS Test Results, and d. TSR Test Results).
Table 7.2: Effect of Foaming Water Content on Foamed WMA Performance.

<table>
<thead>
<tr>
<th>Analysis Data</th>
<th>Statistical Factors</th>
<th>APA Rut Depth</th>
<th>Dry ITS</th>
<th>Wet ITS</th>
</tr>
</thead>
<tbody>
<tr>
<td>WMA, 19.0 mm, Limestone, PG 64-28 &amp; WMA, 19.0 mm, Limestone, PG 70-22</td>
<td>Binder Type</td>
<td>138.5</td>
<td>0.00</td>
<td>43.0</td>
</tr>
<tr>
<td></td>
<td>Prod. Temp.</td>
<td>5.4</td>
<td>0.02</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>Binder Type × Prod. Temp.</td>
<td>0.4</td>
<td>0.67</td>
<td>1.8</td>
</tr>
<tr>
<td>WMA, 12.5 mm, Gravel, PG 70-22 &amp; WMA, 12.5 mm, Limestone, PG 70-22</td>
<td>Agg. Type</td>
<td>16.6</td>
<td>0.00</td>
<td>7.4</td>
</tr>
<tr>
<td></td>
<td>Prod. Temp.</td>
<td>4.7</td>
<td>0.03</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>Agg. Type × Prod. Temp.</td>
<td>0.2</td>
<td>0.26</td>
<td>0.6</td>
</tr>
<tr>
<td>WMA, 12.5 mm, Limestone, PG 70-22 &amp; WMA, 19.0 mm, Limestone, PG 70-22</td>
<td>Agg. Size</td>
<td>1.8</td>
<td>0.20</td>
<td>3.7</td>
</tr>
<tr>
<td></td>
<td>Prod. Temp.</td>
<td>14.2</td>
<td>0.00</td>
<td>3.0</td>
</tr>
<tr>
<td></td>
<td>Agg. Size × Prod. Temp.</td>
<td>1.2</td>
<td>0.35</td>
<td>0.3</td>
</tr>
</tbody>
</table>
for some of the foamed WMA mixtures prepared using moist aggregates during production, indicating that these mixtures might be more susceptible to moisture-induced damage.

Table 7.3 presents the results of the ANOVA analysis conducted to evaluate the effect of the aggregate moisture content and its interaction with the binder type, aggregate type, and aggregate size on the foamed WMA performance. As can be noticed from this table, the effect of the aggregate moisture content was more significant on the dry and wet ITS test results than on the APA rut depths, as indicated by the higher F-value. Furthermore, only the interaction between the aggregate moisture content and the aggregate type was significant at a 95% confidence level. This indicates that the effect of the aggregate moisture content on foamed WMA performance depends to some extent on the type of aggregate used in the mix.

7.5 Summary and Conclusions

This chapter presented the results of a laboratory study conducted to evaluate the effect of temperature reduction, foaming water content, and aggregate moisture content on the performance of foamed WMA mixtures with regard to permanent deformation (rutting) and moisture-induced damage. In this portion of the study, foamed WMA mixtures were prepared at temperatures 30, 50, and 70°F (16.7, 27.8, and 38.9°C) lower than what is traditionally used for HMA mixtures to evaluate the effect of temperature reduction. The effect of foaming water content was assessed by preparing foamed WMA mixtures using 1.8, 2.2, and 2.6% foaming water contents. In addition, the effect of aggregate moisture contents was examined through preparing foamed WMA
Figure 7.4: Effect of Aggregate Moisture Content on Mix Performance (a. APA Test Results, b. Dry ITS Test Results, c. Wet ITS Test Results, and d. TSR Test Results).

- (a) APA Test Results
- (b) Dry ITS Test Results
- (c) Wet ITS Test Results
- (d) TSR Test Results
Table 7.3: Effect of Aggregate Moisture Content on Foamed WMA Performance.

<table>
<thead>
<tr>
<th>Analysis Data</th>
<th>Statistical Factors</th>
<th>APA Rut Depth</th>
<th>Dry ITS</th>
<th>Wet ITS</th>
</tr>
</thead>
<tbody>
<tr>
<td>WMA, 19.0 mm, Limestone, PG 64-28 &amp;</td>
<td>Binder Type</td>
<td>67.5</td>
<td>0.00</td>
<td>331.8</td>
</tr>
<tr>
<td>WMA, 19.0 mm, Limestone, PG 70-22</td>
<td>Prod. Temp.</td>
<td>0.1</td>
<td>0.94</td>
<td>25.0</td>
</tr>
<tr>
<td></td>
<td>Binder Type × Prod. Temp.</td>
<td>2.3</td>
<td>0.14</td>
<td>3.2</td>
</tr>
<tr>
<td>WMA, 12.5 mm, Gravel, PG 70-22 &amp;</td>
<td>Agg. Type</td>
<td>0.0</td>
<td>0.96</td>
<td>0.1</td>
</tr>
<tr>
<td>WMA, 12.5 mm, Limestone, PG 70-22</td>
<td>Prod. Temp.</td>
<td>2.7</td>
<td>0.11</td>
<td>6.8</td>
</tr>
<tr>
<td></td>
<td>Agg. Type × Prod. Temp.</td>
<td>0.5</td>
<td>0.61</td>
<td>1.7</td>
</tr>
<tr>
<td>WMA, 12.5 mm, Limestone, PG 70-22 &amp;</td>
<td>Agg. Size</td>
<td>3.3</td>
<td>0.09</td>
<td>4.2</td>
</tr>
<tr>
<td>WMA, 19.0 mm, Limestone, PG 70-22</td>
<td>Prod. Temp.</td>
<td>15.0</td>
<td>0.00</td>
<td>1.6</td>
</tr>
<tr>
<td></td>
<td>Agg. Size × Prod. Temp.</td>
<td>1.9</td>
<td>0.20</td>
<td>0.1</td>
</tr>
</tbody>
</table>
mixtures using moist aggregates having 0, 1.5, and 3% aggregate moisture contents. The performance of these mixtures was evaluated using the asphalt pavement analyzer (APA) the indirect tensile strength at 77°C (25°C), and the modified Lottman (AASHTO T 283) tests. Finally, it is noted that the performance of the considered foamed WMA mixtures was compared to that of HMA mixtures prepared using fully dried aggregates.

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

- In general, comparable test results were obtained for HMA and foamed WMA prepared using 30°F (16.7°C) temperature reduction, 1.8% foaming water content, and fully dried aggregates. Therefore, the performance of the resulting foamed WMA is expected to be similar to that of the HMA.

- **Effect of Temperature Reduction:** Reducing the production temperature of foamed WMA resulted in increased susceptibility to permanent deformation (or rutting) and moisture-induced damage. Therefore, it is recommended that a maximum reduction temperature of 30°F (16.7°C) be specified for the production of foamed WMA.

- **Effect of Foaming Water Content:** Increasing the foaming water content (up to 2.6% of the weight of the asphalt binder) during production of foamed WMA did not seem to have a negative effect on the rutting performance or moisture sensitivity of foamed WMA. Therefore, a higher foaming water content can be specified for the production of foamed WMA in Ohio.

- **Effect of Aggregate Moisture Content:** Producing foamed WMA using moist aggregates resulted in inadequate aggregate coating leading to concerns with regard to
moisture-induced damage and long-term durability. Therefore, it is critical to use fully dried aggregates in the production of foamed WMA to ensure satisfactory mix performance. Given that foamed WMA is typically produced using lower production temperatures than conventional HMA, the aggregates may need to be dried for a longer period of time.
CHAPTER VII

FIELD EVALUATION OF FOAMED WMA MIXTURES

8.1 Introduction

In addition to the laboratory evaluation, this study examined the rutting performance of plant-produced foamed WMA and HMA mixtures in the Accelerated Pavement Load Facility (APLF) at Ohio University. This chapter presents an overview of the pavement structure, material information, testing procedure, and APLF test results. In addition, it provides a comparison between the rut depth measurements obtained using the APLF and APA test results obtained for field cores, plant-produced laboratory-compacted, and laboratory-produced laboratory-compacted specimens.

8.2 Overview of the Accelerated Pavement Load Facility

The APLF at Ohio University is an indoor facility that allows for the application of dual or wide-based single wheel loads to full-scale sections of rigid or flexible pavements constructed in a 45 ft (13.7 m) long by 38 ft (11.6 m) wide by 8 ft (2.4 m) deep concrete test pit (Figure 8.1). This facility is capable of controlling the air temperature and the amount of water added to the subgrade during testing.
8.3 Material Information

As can be seen in Figure 8.2, the APLF was divided into four 8-foot (2.4-meter) wide lanes, and each lane was divided into two sections, resulting in a total of eight pavement sections. Four of the APLF pavement sections were used for the accelerated field evaluation of the foamed WMA and HMA mixtures. The existing pavement structure at these sections was originally designed as a perpetual asphalt pavement that included a subgrade layer supporting a 6-inch (152.4-mm) dense graded aggregate base (Figure 8.3). A 4-inch (101.6-mm) fatigue resistant asphalt concrete layer was laid on top of the base layer and supporting a 7.75-inch (196.8-mm) asphalt concrete base layer. In addition, the existing pavement structure included two pavement layers laid as a 3-inch (76.2-mm) intermediate course and a 1.25-inch (31.8-mm) surface course.
In this project, the top 3 inches (76.2 mm) of the existing pavements were milled and paved with 3 inches (76.2 mm) of foamed WMA or HMA mixtures. Section 1 was paved with a single 3-inch (76.2-mm) lift of HMA intermediate course (19 mm NMAS) prepared using limestone and PG 64-28. Section 2 was paved with a single 3-inch (76.2-mm) lift of foamed WMA intermediate course (19 mm NMAS) prepared using limestone and PG 64-28. Section 3 was paved with two 1.5-inch (38.1-mm) lifts of HMA surface course (12.5 mm NMAS) prepared using limestone and PG 70-22. Section 4 was paved with two 1.5-inch (38.1-mm) lifts of foamed WMA surface course (12.5 mm NMAS) prepared using limestone and PG 70-22. The number of lifts and lift thicknesses were determined based on the nominal maximum aggregate size of the asphalt mixtures. Since rutting was the only performance parameter considered in the APLF testing, it is believed that the use of the existing pavement structure would ensure that most of the rutting will occur in the newly constructed layers.

The previous material combinations were selected because they are representative of the most commonly used paving materials for interstate highways in Ohio. These mixtures were delivered to the APLF from the Shelly Company Asphalt Plant located in Lancaster, Ohio. The production temperatures of the HMA and foamed WMA mixtures were 310°F (162.7°C) and 270°F (132.2°C), respectively. The research team was present at the plant during production to monitor the temperature of the asphalt mixtures and obtain loose mixtures for further testing in the laboratory.
Figure 8.2: Pavement Sections at the APLF.

Figure 8.3: Pavement Structure at the APLF.
8.4 Construction Process

As mentioned earlier, the construction process involved milling the top 3 inches (76.2 mm) of the existing pavement sections in the APLF and replacing them with 3 inches (76.2 mm) of foamed WMA or HMA mixtures. A milling machine was used to mill the existing pavement surface (Figure 8.4) and a cold planer was used to mill the edges along the perimeter of the test sections (Figure 8.5). After the completion of the milling process (Figure 8.6), a tack coat was applied to the milled surfaces to ensure adequate bonding between the new and the existing materials (Figure 8.7).

The asphalt mixtures were then delivered from the asphalt plant for compaction in the APLF. The asphalt lifts were constructed in the designated sections as discussed earlier. The asphalt mixtures were compacted using the same method and equipment used in the field. The contractor used the same rolling pattern they typically use for HMA mixtures. In general, the rolling pattern included performing five compaction passes using a vibratory wheel roller, followed by a finishing compaction pass using a static wheel roller (Figure 8.8). The density of each lift was monitored using a nuclear density gage to ensure adequate compaction and compliance with ODOT specifications, and an infrared thermometer was used to record the temperature during compaction (Figure 8.9).

Figures 8.10 and 8.11 present pictures of the compacted surface and intermediate courses, respectively. After construction, six cores were obtained from each of the pavement sections for further testing in the laboratory, as shown in Figures 8.12 and 8.14. The field cores were obtained away from the wheel path and were filled and compacted prior to testing in the APLF in order not to interfere with the rolling wheel test results.
Figure 8.4: Milling of Pavement Sections.

Figure 8.5: Cold Milling Machine Near Edges.

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Figure 8.6: Milled Pavement Surface.

Figure 8.7: Tack Coat Application.
Figure 8.8: Vibratory (Left) and Static (Right) Wheel Rollers.

Figure 8.9: Monitoring Temperature and Density.
Figure 8.10: Picture of the Compacted Surface Course.

Figure 8.11: Picture of the Compacted Intermediate Course.

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Figure 8.12: Coring of Field Specimens.

Figure 8.13: Location of Field Cores.

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8.5 Testing Program

The testing program used to evaluate the rutting performance of the foamed WMA and HMA mixtures is presented in Figure 8.15. As can be noticed from this figure, rolling wheel tests were conducted on each of the four APLF pavement sections to examine the rutting resistance of the plant-produced field-compacted asphalt mixtures. Prior to the beginning of the tests, the temperature of the indoor APLF facility was adjusted to 104°F (40°C) and the initial pavement profile was measured along the lane width using a traveling laser profilometer. The profilometer used in this project measures surface elevations to at least 5-mil (127 micron) accuracy at 0.5 inch (1.27 cm) intervals along the profile path. During the test, each pavement section was subjected to 10,000 passes of a 9,000 lb (40.0 kN) dual-tire rolling wheel load (Figure 8.16) travelling at a speed of approximately 5 mph.
(8 km/h). Lateral surface profiles were measured across the lane width after applying 100, 300, 1000, 2000, 3000, 6300, 7700, and 10,000 passes of the rolling wheels to assess the permanent deformation in each section. It can also be observed from Figure 8.15 that laboratory APA tests were also performed on laboratory-produced laboratory-compacted, plant-produced laboratory-compacted, and plant-produced field-compacted (field cores) specimens to evaluate the rutting performance of the corresponding asphalt mixtures and compare it to that obtained from the APLF rolling wheel tests. As discussed in the following section, this comparison allowed for determining the effect of the specimen preparation and compaction method on the performance of the foamed WMA and HMA mixtures.

![Diagram of Rutting Performance Tests](image)

**Figure 8.15: Testing Program Used for the APA and APLF Rolling Wheel Tests.**

### 8.6 Field and Laboratory Test Results

Figure 8.17 present example surface profiles obtained for the HMA mixture prepared using 19.0 mm NMAS and PG 64-28. As can be seen from this figure, the initial profile (i.e. profile measured at 0 loading cycles) indicates that the pavement surface has no major surface depressions. However, changes in the pavement surface profile started to
occur as wheel loading was applied. These changes are the result of permanent deformations in the pavement structure and can be described as depression along the wheel path where the tires are in contact with the pavement surface and as heaving along the edges of the two tires.

Figure 8.16: Dual-Tire Rolling Wheel Load used in the Rolling Wheel Tests.

Two approaches can be used to measure the rut depth for the tested pavement sections. The first approach defines the rut depth as the difference between the highest elevations in the heaving zones and the lowest elevations in the depression zones of the 10,000 cycle surface profile. This approach is similar to the straighedged method typically used to measure rutting in the field. The second approach defines the rut depth as the difference between the initial surface profile and the lowest elevation in the depression
Figure 8.17: Example Surface Profiles Obtained at Various Loading Cycles for HMA Mixture Prepared using 19.0 mm NMAS Limestone Aggregate and PG 64-28.

zones of the 10,000 cycle surface profile. The first approach is typically used for field measurements due to the lack of an initial reference profile. However, in this study, the second approach was utilized to determine the rut depth for the pavement sections constructed in the APLF, as the initial surface profile was available. The second approach was also selected because it is consistent with the rut depth measurement procedure used in the APA test.

Two surface profile measurements were made at different locations (north and south) for each pavement section. The surface profiles were analyzed to obtain the surface elevations corresponding to the 0 and 10,000 loading cycles under each tire. The rut depth value was calculated as the difference between the surface elevation at 0 loading cycles
and the surface elevation at 10,000 loading cycles. The surface elevations obtained for all pavement sections and the corresponding rut depth values are presented in Table 8.1. As can be noticed from this table, the foamed WMA section prepared using 19.0 mm NMAS limestone aggregate and PG 64-28 binder had a slightly higher average rut depth value than the corresponding HMA section. In addition, the foamed WMA section prepared using 12.5 mm NMAS limestone aggregate and PG 70-22 binder had slightly lower average rut depth value than the corresponding HMA section. However, using the t-test statistical analysis, the difference between rut depth values obtained for the foamed WMA and HMA mixtures was found to be statistically insignificant.

Figure 8.18 presents a comparison between the rut depth values obtained at the APLF and those obtained using the laboratory APA test for field cores, plant-produced laboratory-compacted, and laboratory-produced laboratory-compacted specimens. As can be noticed from this figure, the foamed WMA mixtures had in general slightly higher rut depth values than the corresponding HMA mixtures. In addition, it can be observed that the plant-produced laboratory-compacted and laboratory-produced laboratory-compacted specimens had comparable rut depth values. However, the plant-produced field-compacted cores had significantly higher rut depth values than the other two mixtures tested in the APA. By comparing the rut depth values obtained from the APA test to those obtained from the APLF rolling wheel test, it can be noticed that the APLF rut depths were higher than those obtained from the APA test for the laboratory-produced laboratory-compacted and plant-produced laboratory-compacted specimens, but lower than those obtained for the field cores. Given that the plant-produced laboratory-compacted and laboratory-produced
laboratory-compacted specimens had comparable rut depth values, the difference between the APLF rut depths and the APA test results can be attributed to difference in density between the APLF sections and the APA test specimens.

Table 8.1: APLF Surface Elevations and Rut Depth Results

Obtained after 0 and 10,000 Loading Cycles.

<table>
<thead>
<tr>
<th>Pavement Section</th>
<th>Lane/Tire</th>
<th>Elevation (inch)</th>
<th>Rut Depth (inch)</th>
<th>Average Rut Depth (inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0</td>
<td>10,000</td>
<td></td>
</tr>
<tr>
<td>HMA 12.5 mm NMAS</td>
<td>North/1</td>
<td>4.556</td>
<td>4.406</td>
<td>0.150</td>
</tr>
<tr>
<td>PG 70-22</td>
<td>North/2</td>
<td>4.566</td>
<td>4.415</td>
<td>0.151</td>
</tr>
<tr>
<td></td>
<td>South/1</td>
<td>4.641</td>
<td>4.448</td>
<td>0.193</td>
</tr>
<tr>
<td></td>
<td>South/2</td>
<td>4.556</td>
<td>4.411</td>
<td>0.145</td>
</tr>
<tr>
<td>Foamed WMA 12.5 mm</td>
<td>North/1</td>
<td>4.275</td>
<td>4.197</td>
<td>0.078</td>
</tr>
<tr>
<td>NMAS PG 70-22</td>
<td>North/2</td>
<td>4.275</td>
<td>4.136</td>
<td>0.139</td>
</tr>
<tr>
<td></td>
<td>South/1</td>
<td>4.136</td>
<td>4.049</td>
<td>0.087</td>
</tr>
<tr>
<td></td>
<td>South/2</td>
<td>4.17</td>
<td>4.024</td>
<td>0.146</td>
</tr>
<tr>
<td>HMA 19 mm NMAS</td>
<td>North/1</td>
<td>4.454</td>
<td>4.271</td>
<td>0.183</td>
</tr>
<tr>
<td>PG 64-28</td>
<td>North/2</td>
<td>4.382</td>
<td>4.221</td>
<td>0.161</td>
</tr>
<tr>
<td></td>
<td>South/1</td>
<td>4.642</td>
<td>4.439</td>
<td>0.203</td>
</tr>
<tr>
<td></td>
<td>South/2</td>
<td>4.515</td>
<td>4.288</td>
<td>0.227</td>
</tr>
<tr>
<td>Foamed WMA 19 mm</td>
<td>North/1</td>
<td>4.626</td>
<td>4.394</td>
<td>0.232</td>
</tr>
<tr>
<td>NMAS PG 64-28</td>
<td>North/2</td>
<td>4.563</td>
<td>4.334</td>
<td>0.229</td>
</tr>
<tr>
<td></td>
<td>South/1</td>
<td>4.479</td>
<td>4.280</td>
<td>0.199</td>
</tr>
<tr>
<td></td>
<td>South/2</td>
<td>4.535</td>
<td>4.280</td>
<td>0.255</td>
</tr>
</tbody>
</table>
A multi-factor analysis of variance (ANOVA) was conducted to evaluate the effect of the mix preparation procedure and the mix type on the APA rut depths presented in Figure 8.18. The ANOVA results are presented in Table 8.2. As can be noticed from this table, the difference between the APA rut depth values obtained for the foamed WMA and HMA was found to be statistically insignificant (probability < 0.05) at a 95% confidence level. However, the mix preparation method had a significant effect on the APA rut depths. Table 8.3 provides the ranking of the various preparation methods as determined using the post ANOVA Least Square Means (LSM) analysis. As can be noticed from this table, the laboratory-produced laboratory-compacted and plant-produced laboratory-compacted specimens received the same ranking, which indicates that the rut depth values obtained for these specimens were statistically indistinguishable. However, the plant-produced
field-compacted specimens (field cores) received a lower ranking, which was statistically different than the other two types of specimens. It is believed that the field cores were compacted to a lower density (i.e., higher air void level), which was the main reason affecting their APA rut depths.

Table 8.2: Multi-Factor ANOVA Results for APA Rut Depths.

<table>
<thead>
<tr>
<th>Effect</th>
<th>F-value</th>
<th>Prob.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Preparation Method</td>
<td>45.92</td>
<td>&lt;.0001</td>
</tr>
<tr>
<td>Mix Type</td>
<td>0.77</td>
<td>0.3874</td>
</tr>
<tr>
<td>Preparation Method × Mix Type</td>
<td>1.31</td>
<td>0.2853</td>
</tr>
</tbody>
</table>

Table 8.3: Results of Post ANOVA analyses on APA Rutting Values.

<table>
<thead>
<tr>
<th>Method</th>
<th>Estimate</th>
<th>Standard Error</th>
<th>Ranking</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laboratory-Produced Laboratory-Compacted</td>
<td>0.1275</td>
<td>0.008440</td>
<td>A</td>
</tr>
<tr>
<td>Plant-Produced Laboratory-Compacted</td>
<td>0.1324</td>
<td>0.008440</td>
<td>A</td>
</tr>
<tr>
<td>Plant-Produced Field-Compacted (Field Cores)</td>
<td>0.2512</td>
<td>0.008440</td>
<td>B</td>
</tr>
</tbody>
</table>

8.7 Summary and Conclusions

The field performance of foamed WMA and HMA mixtures was examined using the Accelerated Pavement Load Facility (APLF) at Ohio University. The APLF is an indoor facility that allows for the application of dual or wide-based single wheel loads to full-scale sections of rigid or flexible pavements constructed in a 45 ft (13.7 m) long by 38 ft (11.6 m) wide by 8 ft (2.4 m) deep concrete test pit. This facility is capable of controlling the air temperature and the amount of water added to the subgrade during testing. The
APLF was divided into four 8-ft (2.4-meter) wide lanes, and each lane was divided into two sections, resulting in a total of eight pavement sections. Four of the APLF pavement sections were used for the accelerated field evaluation of the foamed WMA and HMA mixtures. The existing pavement structure at these sections was originally designed as a perpetual asphalt pavement. In this project, the top 3 inches (76.2 mm) of the existing pavements were milled and paved with 3 inches (76.2 mm) of surface and intermediate foamed WMA and HMA mixtures. The asphalt mixtures used in the APLF was delivered from the Shelly Company Asphalt Plant located in Lancaster, Ohio. Since rutting was the only performance parameter considered in the APLF testing, most of the rutting was expected to occur in the newly constructed layers. Rolling wheel tests were conducted on each of the four APLF pavement sections to examine the rutting resistance of the plant-produced field-compacted asphalt mixtures. In these tests, the pavement sections were subjected to 10,000 passes of a 9,000 lb (40.0 kN) dual-tire rolling wheel load travelling at a speed of approximately 5 mph (8 km/h), and the lateral surface profile was measured using a traveling laser profilometer after applying 0, 100, 300, 1000, 2000, 3000, 6300, 7700, and 10,000 passes to assess the permanent deformation in each section. In addition, laboratory APA tests were performed on laboratory-produced laboratory-compacted, plant-produced laboratory-compacted, and plant-produced field-compacted (field cores) specimens and the APA rut depth values were compared to the APLF rolling wheel test results.

Based on the APLF and APA test results and the subsequent statistical analysis findings, the following conclusions were made:
• The APLF and APA rut depth values obtained for the foamed WMA and HMA mixtures were comparable for both surface and intermediate mixtures. This suggests that the foamed WMA mixtures have similar rutting resistance to the HMA mixtures.

• The plant-produced laboratory-compacted and laboratory-produced laboratory-compacted specimens had comparable APA rut depth values for both foamed WMA and HMA mixtures. This indicates that the laboratory mix preparation procedure used in this study resulted in comparable foamed WMA and HMA mixtures to those produced in the field.

• The plant-produced field-compacted specimens (i.e., field cores) had significantly higher rut depth values in the APA test than the plant-produced laboratory-compacted and laboratory-produced laboratory-compacted specimens. It is believed that the field cores were compacted to a lower density (i.e., higher air void level), which was the main reason affecting their APA rut depths.
CHAPTER IX

Performance Evaluation of Foamed WMA and HMA using the MEPDG

9.1 Introduction

The Mechanistic-Empirical Pavement Design Guide (MEPDG) software (version 1.100) was utilized to evaluate the performance of pavement structures constructed using foamed WMA and HMA surface and intermediate courses. The MEPDG is a new pavement design procedure developed under the auspices of the National Cooperative Highway Research Program (NCHRP) for the design of new and rehabilitation pavement structures. The main inputs for the MEPDG software are the pavement layer thicknesses, material properties for the various layers, traffic information, and climate data. The MEPDG uses this information to predict the future performance of the pavement structure.

9.2 Baseline Pavement Structures

Four baseline designs for new flexible pavements were defined in the MEPDG to compare the performance of the foamed WMA and HMA mixtures (Figure 9.1). As can be noticed from this figure, all pavement structures consisted of a 1.5-inch (38.1-mm) surface course, a 1.75-inch (44.5-mm) intermediate course, a 7-inch (177.8-mm) asphalt concrete base course (Item 301), and a 10-inch (254-mm) dense graded aggregate base course (AASHTO A-1-a) placed over a semi-infinite AASHTO A-6 (clayey soil) subgrade. The main difference between these pavement structures was in the type of asphalt mixture used.
in the surface and intermediate courses and the type of aggregate used in the surface course.

![Diagram](image)

**Figure 9.1: Baseline Pavement Structures:** (a) HL-HL, (b) WL-WL, (c) HG-HL, and (d) WG-WL.
The first baseline pavement structure (HL-HL) consisted of an HMA surface course prepared using limestone aggregate and PG 70-22 course (12.5L70H) over an HMA intermediate course prepared using limestone aggregate and PG 64-22 (19L64H). The second baseline pavement structure (WL-WL) consisted of a foamed WMA surface course prepared using limestone aggregate and PG 70-22 course (12.5L70W) over a foamed WMA intermediate course prepared using limestone aggregate and PG 64-22 (19L64W). The third baseline pavement structure (HG-HL) consisted of an HMA surface course prepared using crushed gravel and PG 70-22 course (12.5G70H) over an HMA intermediate course prepared using limestone aggregate and PG 64-22 (19L64H). The fourth baseline pavement structure (WG-WL) consisted of a foamed WMA surface course (12.5G70W) prepared using crushed gravel and PG 70-22 course over a foamed WMA intermediate course prepared using limestone aggregate and PG 64-22 (19L64W). The selection of these material combinations is consistent with the current practice for interstate highways in the state of Ohio.

Project-specific (Level 1) material properties were defined for the surface and intermediate courses using the dynamic modulus laboratory test results presented in Chapter 5. The analysis was repeated using unconditioned and conditioned (dry and wet) dynamic moduli to evaluate the effect of sample conditioning (freezing and thawing) on pavement performance. Statewide average (Level 2) material properties were used for the asphalt concrete base, aggregate base, and the subgrade soil using data obtained from Nazzal et al. (2011). A summary of the material property input level for the various layers within the pavement structure is presented in Table 9.1.
Table 9.1: MEPDG Material Properties.

<table>
<thead>
<tr>
<th>Section</th>
<th>Material</th>
<th>Input Level</th>
<th>Input Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface Course</td>
<td>12.5 mm HMA or WMA</td>
<td>Level I</td>
<td>E* at 6 frequencies and 5 temperatures</td>
</tr>
<tr>
<td>Intermediate Course</td>
<td>19.0 mm HMA or WMA</td>
<td>Level I</td>
<td>E* at 6 frequencies and 5 temperatures</td>
</tr>
<tr>
<td>AC Base Course</td>
<td>Item 301</td>
<td>Level II</td>
<td>E* at 6 frequencies and 5 temperatures</td>
</tr>
<tr>
<td>Aggregate Base</td>
<td>A-1-a</td>
<td>Level II</td>
<td>Mr = 45 ksi</td>
</tr>
<tr>
<td>Subgrade</td>
<td>A-6</td>
<td>Level II</td>
<td>Mr = 12 ksi</td>
</tr>
</tbody>
</table>

The initial two-way average annual daily truck traffic (AADTT) was assumed to be 10,700 trucks per day with a compound growth rate of 4% per year. The directional and lane distributions were set as 50% and 80%, respectively. Additionally, default MEPDG values were used in the analysis for vehicle class distribution (assuming intermediate light and single-trailer truck route, Type II), axle load spectra and number of axles per truck for each truck class, monthly adjustment factors and axle configuration.

The pavement sections were assumed to be located in the City of Newark in central Ohio. A design life of 20 years was used in the design of the pavement structure. The analysis was performed using an initial international roughness index (IRI) of 63 inch/mile (1.0 m/km). Default roughness and distress limits were used for the performance criteria and the reliability was set to 90% for all performance parameters. Key performance
parameters for the flexible pavement structures included smoothness expressed using IRI, alligator (bottom-up) fatigue cracking, total rutting, and asphalt concrete (AC) rutting.

9.3 MEPDG Performance Results

Figures 9.2 through 9.5 present the MEPDG predictions for the four baseline pavement designs in terms of IRI, fatigue cracking, total rutting, and AC rutting. As can be noticed from Figure 9.2, the IRI predictions were well below the threshold limit (172 inch/mile), represented by the horizontal red line, for all pavement sections. Additionally, by comparing the conditioned and unconditioned IRI predictions, it can be noticed that the conditioned and unconditioned pavements resulted in IRI predictions that were close to each other at approximately 120 inch/mile for both HMA and foamed WMA pavement sections. Figure 9.3 shows the predicted fatigue cracking obtained using the MEPDG for each of the pavement sections. As can be noticed from this figure, all pavement sections had similar fatigue cracking predictions for both unconditioned and conditioned HMA and foamed WMA pavements. All fatigue cracking predictions were less than 1%, which is significantly less than the threshold limit of 25%. Figure 9.4 shows the total rutting obtained using the MEPDG for each of the pavement sections. As can be noticed from this figure, the predicted total rutting ranged from 0.45 to 0.60 inch, which is less than the specified limit of 0.75 inch. Additionally, there was little difference between the total rutting predictions for the foamed WMA and HMA pavements. However, it can be noticed that the difference between the unconditioned and conditioned total rutting predictions were greater for the HMA sections than the foamed WMA sections. This indicates that the
Figure 9.2: MEPDG Predictions for IRI.
Figure 9.3: MEPDG Predictions for Fatigue Cracking.
Figure 9.4: MEPDG Predictions for Total Rutting.
Figure 9.5: MEPDG Predictions for AC Rutting.

(A) HL-HL

(B) WL-WL

(C) HG-HL

(D) WG-WL
HMA sections are more susceptible to conditioning than the foamed WMA pavements. Figure 9.5 shows the AC rutting predictions obtained using the MEPDG. As can be noticed from this figure, there was a significant difference between the unconditioned and conditioned AC rutting predictions for both HMA and foamed WMA pavement sections. For the foamed WMA sections, the unconditioned pavements resulted in higher AC rutting predictions than the conditioned sections, while the opposite is true for the HMA sections. This indicates that the HMA sections are more susceptible to AC rutting after conditioning than the foamed WMA pavements. It is noted that AC rutting provides a better indication of the effect of the mix type and sample conditioning on pavement performance than total rutting because it excludes the effect of the aggregate base and subgrade soil.

9.4 Summary and Conclusions

The Mechanistic-Empirical Pavement Design Guide (MEPDG) software was utilized to evaluate the performance of pavement structures constructed using foamed WMA and HMA surface and intermediate courses. Four baseline designs for new flexible pavements were defined in the MEPDG to compare the performance of the foamed WMA and HMA mixtures. All pavement structures consisted of a 1.5-inch (38.1-mm) surface course, a 1.75-inch (44.5-mm) intermediate course, a 7-inch (177.8-mm) asphalt concrete base course (Item 301), and a 10-inch (254-mm) dense graded aggregate base course (AASHTO A-1-a) placed over a semi-infinite AASHTO A-6 (clayey soil) subgrade. The main difference between these pavement structures was in the type of asphalt mixture used in the surface and intermediate courses and the type of aggregate used in the surface course.
Project-specific (Level 1) material properties were defined for the surface and intermediate courses using the dynamic modulus laboratory test results. The analysis was repeated using unconditioned and conditioned (dry and wet) dynamic moduli to evaluate the effect of sample conditioning (freezing and thawing) on pavement performance. Statewide average (Level 2) material properties were used for the asphalt concrete base, aggregate base, and the subgrade soil. The analysis was performed using an initial international roughness index (IRI) of 63 inch/mile (1.0 m/km). Default roughness and distress limits were used for the performance criteria and the reliability was set to 90% for all performance parameters. Key performance parameters for the flexible pavement structures included smoothness expressed using IRI, alligator (bottom-up) fatigue cracking, total rutting, and asphalt concrete (AC) rutting.

The following conclusions were made based on the MEPDG performance predictions for the previous performance parameters:

- The foamed WMA had a negligible influence on the predicted pavement performance in terms of IRI and fatigue cracking. However, it had a moderate impact on total rutting and AC rutting predictions.
- The difference between the unconditioned and conditioned rutting predictions was greater for the HMA sections than the foamed WMA sections, which suggests that the HMA sections are more susceptible to conditioning than the foamed WMA pavements.
CHAPTER X

CONCLUSIONS AND RECOMMENDATIONS

10.1 Introduction

This report presents the results of a comprehensive study conducted to evaluate the laboratory and field performance of foamed WMA mixtures and compare it to that of traditional HMA mixtures. This project also involved determining the limitations of foamed WMA mixtures by evaluating the effect of the mix preparation procedure on the performance of these mixtures. As part of this study, a new workability device was designed and fabricated to evaluate the workability of foamed WMA and HMA mixtures. Results obtained from the Superpave gyratory compactor were also analyzed to compare the compactability of foamed WMA and HMA mixtures. In addition, the long-term performance of pavement sections constructed using foamed WMA and HMA surface and intermediate courses was predicted using the Mechanistic-Empirical Pavement Design Guide (MEPDG). The following sections present a summary of the research activities and main conclusions made as part of this study.

10.2 Laboratory Performance of Foamed WMA and HMA

A comprehensive laboratory study was conducted to evaluate the performance of foamed WMA with regard to permanent deformation (or rutting), moisture-induced damage (or durability), fatigue cracking, and low-temperature cracking, and compare it to
that of traditional HMA mixtures. Several tests were included in the experimental testing plan. The asphalt pavement analyzer (APA), dynamic modulus, and flow number tests were used to evaluate the rutting performance of foamed WMA and HMA mixtures. The susceptibility of foamed WMA and HMA mixtures to moisture-induced damage was characterized using the AASHTO T 283, dynamic modulus ratio, and wet APA tests. In addition, the fatigue cracking and the low-temperature cracking characteristics of these mixtures were evaluated using the dissipated creep strain energy (DCSE) and low-temperature indirect tensile strength tests, respectively. The foamed WMA mixtures used in these tests were prepared according to the current ODOT specifications (i.e., 30°F temperature reduction and 1.8% foaming water content) using fully dried aggregates.

The following conclusions were made based on the laboratory test results and the subsequent statistical analysis findings:

- **Permanent Deformation (Rutting):** The foamed WMA mixtures exhibited slightly higher rut depth values in the dry and wet APA tests, slightly lower dynamic moduli, and slightly lower flow number values than the traditional HMA mixtures. However, the difference was statistically insignificant. Therefore, the rutting potential of foamed WMA mixtures is expected to be comparable to that of the HMA mixtures.

- **Moisture-Induced Damage (Durability):** The foamed WMA mixtures exhibited slightly lower dry and wet ITS values and comparable TSR ratios to the HMA mixtures in the AASHTO T 283 test. However, the difference between the ITS values for the foamed WMA and HMA mixtures was found to be statistically insignificant. In addition, the foamed WMA mixtures exhibited slightly higher dry and wet rut depth
values in the APA test, but the difference was statistically insignificant. By comparing the dry and wet APA rut depths, it was observed that the effect of sample conditioning was more pronounced on the HMA mixtures than the foamed WMA mixtures. This trend was also observed in the dry and wet dynamic modulus master curves for some mixtures.

- **Fatigue Cracking:** The foamed WMA mixtures exhibited slightly lower DCSE values than the HMA mixtures. However, the difference was found to be statistically insignificant. In addition, the DCSE values for all foamed WMA and HMA mixtures were greater than 0.75 kJ/m$^3$, which has been suggested by Roque et al. (2004) as a minimum DCSE threshold value to ensure satisfactory resistance to fatigue cracking.

- **Low-Temperature Cracking:** The foamed WMA mixtures exhibited slightly lower ITS values at 14°F (-10°C) and comparable or slightly higher failure strain values than the HMA mixtures. The multi-factor ANOVA analysis revealed that the effect of the mix type is significant on the low-temperature ITS values, but not on the failure strain values. Therefore, the HMA mixtures are expected to have better resistance to thermal cracking.

10.3 Workability and Compactability of Foamed WMA and HMA Mixtures

A new device was designed and fabricated to evaluate the workability of foamed WMA and HMA mixtures. This device utilized the torque generated while stirring a mix to measure the workability. Each workability test was performed on mixtures heated to 150°C and the test was terminated when the mixture’s temperature reached 100°C. The new device had several advantages, including the ability to thoroughly mix the asphalt
mixture using an improved mixing paddle design; the ability to obtain accurate temperature and torque measurements using an infrared thermometer and a stationary torque sensor; the ability to run the test at varying speeds ranging from 5 to 35 rpm using a motor and a speed drive control unit; the ability to record test results to a personal computer; and improved safety features such as a specially designed safety cage and an emergency stop button. In addition, the compactability of the foamed WMA and HMA mixtures was examined by analyzing compaction data obtained using the Superpave gyratory compactor during the preparation of laboratory test specimens.

The following conclusions were made based on the workability test results and the analysis of the compaction data:

- **Workability**: The foamed WMA mixtures exhibited better workability than the traditional HMA mixtures. This was attributed to the lower asphalt binder absorption observed for the foamed WMA mixtures. Another factor that might have contributed to the improvement in workability for foamed WMA mixtures is the presence of vapor pockets entrapped within the foamed asphalt binder that serve to keep the binder slightly expanded and reduce its viscosity. The workability of the foamed WMA and HMA mixtures was found to be affected by the binder grade, aggregate type, and aggregate size. Foamed WMA and HMA mixtures prepared using PG 64-28 asphalt binder had better workability than those prepared using PG 70-22. This indicates that using a softer asphalt binder results in better workability. In addition, the HMA mixtures prepared using crushed gravel had better workability than those prepared using limestone aggregates. However, foamed WMA mixtures prepared using
limestone aggregates had better workability than those prepared using crushed gravel, which suggests that the aggregate type affects foamed WMA mixtures differently than HMA mixtures. Furthermore, the 12.5 mm surface mixtures showed better workability than the 19.0 mm intermediate mixtures for both foamed WMA and HMA mixtures, which indicates that the use of a smaller nominal maximum aggregate size (NMAS) results in better workability.

- **Compactability:** By comparing the compaction data obtained using the Superpave gyratory compactor during the preparation of the laboratory specimens, it was observed that the number of gyrations needed to achieve the target air void levels for the foamed WMA specimens was relatively close to that of the HMA specimens. This indicates that the compactability of the foamed WMA mixtures is comparable to that of the corresponding HMA mixtures. In a previous study conducted by the first and the fourth authors, it was observed that the foamed WMA mixtures had significantly better compactability than the HMA mixtures (Abbas and Ali, 2011). The asphalt mixtures used in that study had a smaller aggregate size and were prepared using higher asphalt binder contents. A significant reduction in asphalt binder absorption was observed for the foamed WMA mixtures, leading to significant improvement in compactability.

10.4 Effect of Mix Preparation on Foamed WMA Performance

A laboratory study was conducted to evaluate the effect of temperature reduction, foaming water content, and aggregate moisture content on the performance of foamed WMA. The foamed WMA mixtures were produced using three production temperatures (30°F, 50°F, and 70°F (16.7°C, 27.8°C, and 38.9°C) lower than the traditional HMA), three
foaming water contents (1.8%, 2.2%, and 2.6%) by weight of the asphalt binder), and three aggregate moisture contents (0%, 1.5%, and 3%). The APA test was utilized to evaluate the rutting resistance and the Modified Lottman (AASHTO T 283) test was used to evaluate the moisture sensitivity of the asphalt mixtures.

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

- In general, the performance of the foamed WMA mixtures prepared using 30°F (16.7°C) temperature reduction, 1.8% foaming water content, and fully dried aggregates was comparable to that of the HMA mixtures.
- Reducing the production temperature of foamed WMA resulted in increased susceptibility to permanent deformation (or rutting) and moisture-induced damage. Therefore, it is recommended that a maximum reduction temperature of 30°F (16.7°C) be specified for the production of foamed WMA.
- Increasing the foaming water content (up to 2.6% of the weight of the asphalt binder) during production of foamed WMA did not seem to have a negative effect on the rutting performance or moisture sensitivity of foamed WMA. Therefore, a higher foaming water content can be specified for the production of foamed WMA in Ohio.
- Producing foamed WMA using moist aggregates resulted in inadequate aggregate coating leading to concerns with regard to moisture-induced damage and long-term durability. Therefore, it is critical to use fully dried aggregates in the production of foamed WMA to ensure satisfactory mix performance. Given that foamed WMA is
typically produced using lower production temperatures than conventional HMA, the aggregates may need to be dried for a longer period of time.

10.5 Performance of Foamed WMA and HMA Mixtures in APLF

The field performance of foamed WMA and HMA mixtures was examined using the Accelerated Pavement Load Facility (APLF) at Ohio University. The APLF is an indoor facility that allows for the application of dual or wide-based single wheel loads to full-scale sections of rigid or flexible pavements constructed in a 45 ft (13.7 m) long by 38 ft (11.6 m) wide by 8 ft (2.4 m) deep concrete test pit. This facility is capable of controlling the air temperature and the amount of water added to the subgrade during testing. The APLF was divided into four 8-ft (2.4-meter) wide lanes, and each lane was divided into two sections, resulting in a total of eight pavement sections. Four of the APLF pavement sections were used for the accelerated field evaluation of the foamed WMA and HMA mixtures. The existing pavement structure at these sections was originally designed as a perpetual asphalt pavement. In this project, the top 3 inches (76.2 mm) of the existing pavements were milled and paved with 3 inches (76.2 mm) of surface and intermediate foamed WMA and HMA mixtures. The asphalt mixtures used in the APLF was delivered from the Shelly Company Asphalt Plant located in Lancaster, Ohio. Since rutting was the only performance parameter considered in the APLF testing, most of the rutting was expected to occur in the newly constructed layers. Rolling wheel tests were conducted on each of the four APLF pavement sections to examine the rutting resistance of the plant-produced field-compacted asphalt mixtures. In these tests, the pavement sections were subjected to 10,000 passes of a 9,000 lb (40.0 kN) dual-tire rolling wheel load travelling at
a speed of approximately 5 mph (8 km/h), and the lateral surface profile was measured using a traveling laser profilometer after applying 0, 100, 300, 1000, 2000, 3000, 6300, 7700, and 10,000 passes to assess the permanent deformation in each section. In addition, laboratory APA tests were performed on laboratory-produced laboratory-compacted, plant-produced laboratory-compacted, and plant-produced field-compacted (field cores) specimens and the APA rut depth values were compared to the APLF rolling wheel test results.

Based on the APLF and APA test results and the subsequent statistical analysis findings, the following conclusions were made:

- The APLF and APA rut depth values obtained for the foamed WMA and HMA mixtures were comparable for both surface and intermediate mixtures. This suggests that the foamed WMA mixtures have similar rutting resistance to the HMA mixtures.
- The plant-produced laboratory-compacted and laboratory-produced laboratory-compacted specimens had comparable APA rut depth values for both foamed WMA and HMA mixtures. This indicates that the laboratory mix preparation procedure used in this study resulted in comparable foamed WMA and HMA mixtures to those produced in the field.
- The plant-produced field-compacted specimens (i.e., field cores) had significantly higher rut depth values in the APA test than the plant-produced laboratory-compacted and laboratory-produced laboratory-compacted specimens. It is believed that the field cores were compacted to a lower density (i.e., higher air void level), which was the main reason affecting their APA rut depths.
10.6 Performance Evaluation of Foamed WMA and HMA Mixtures using the MEPDG

The Mechanistic-Empirical Pavement Design Guide (MEPDG) software was utilized to evaluate the performance of pavement structures constructed using foamed WMA and HMA surface and intermediate courses. Four baseline designs for new flexible pavements were defined in the MEPDG to compare the performance of the foamed WMA and HMA mixtures. All pavement structures consisted of a 1.5-inch (38.1-mm) surface course, a 1.75-inch (44.5-mm) intermediate course, a 7-inch (177.8-mm) asphalt concrete base course (Item 301), and a 10-inch (254-mm) dense graded aggregate base course (AASHTO A-1-a) placed over a semi-infinite AASHTO A-6 (clayey soil) subgrade. The main difference between these pavement structures was in the type of asphalt mixture used in the surface and intermediate courses and the type of aggregate used in the surface course.

Project-specific (Level 1) material properties were defined for the surface and intermediate courses using the dynamic modulus laboratory test results. The analysis was repeated using unconditioned and conditioned (dry and wet) dynamic moduli to evaluate the effect of sample conditioning (freezing and thawing) on pavement performance. Statewide average (Level 2) material properties were used for the asphalt concrete base, aggregate base, and the subgrade soil. The analysis was performed using an initial international roughness index (IRI) of 63 inch/mile (1.0 m/km). Default roughness and distress limits were used for the performance criteria and the reliability was set to 90% for all performance parameters. Key performance parameters for the flexible pavement structures included smoothness expressed using IRI, alligator (bottom-up) fatigue cracking, total rutting, and asphalt concrete (AC) rutting.
The following conclusions were made based on the MEPDG performance predictions for the previous performance parameters:

- The foamed WMA had a negligible influence on the predicted pavement performance in terms of IRI and fatigue cracking. However, it had a moderate impact on total rutting and AC rutting predictions.
- The difference between the unconditioned and conditioned rutting predictions was greater for the HMA sections than the foamed WMA sections, which suggests that the HMA sections are more susceptible to conditioning than the foamed WMA pavements.

10.7 Recommendations for Implementation

Producing foamed WMA using fully dried aggregates and current ODOT specifications (i.e., 30°F temperature reduction and 1.8% foaming water content) resulted in relatively comparable performance to traditional HMA. However, reducing the production temperature of foamed WMA led to increased susceptibility to permanent deformation (rutting) and moisture-induced damage. Therefore, it is recommended to continue to use a reduction temperature of 30°F (16.7°C) for the production of foamed WMA. In addition, producing foamed WMA using moist aggregates resulted in inadequate aggregate coating leading to concerns with regard to moisture-induced damage and long-term durability. Therefore, it is critical to use fully dried aggregates in the production of foamed WMA to ensure satisfactory mix performance. Given that foamed WMA is typically produced using lower production temperatures than conventional HMA, the aggregates may need to be dried for a longer period of time.
The foamed WMA mixtures exhibited better workability, but comparable compactability to the traditional HMA mixtures in the laboratory. In addition, the foamed WMA mixtures required the same compaction effort as the HMA mixtures to reach the target density level in the field. Therefore, there is no need to compact the foamed WMA mixtures to a higher density level that commonly used for HMA mixtures. Furthermore, since the performance of the foamed WMA was comparable to that of the HMA, no modifications are needed to the current mix design process used by ODOT for foamed WMA mixtures.
REFERENCES


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