Determining the Limitations of Warm Mix Asphalt by Water Injection in Mix Design, Quality Control and Placement

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In this project, a comprehensive study was 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 Ohio University (OU) Accelerated Pavement Load Facility (APLF), 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).
The laboratory test results revealed comparable resistance to permanent deformation, moisture-induced damage, and fatigue cracking for foamed WMA and HMA mixtures. However, the HMA mixtures had significantly higher ITS values at 14°F (-10°C) and comparable failure strains to the foamed WMA mixtures, which indicates that the traditional HMA mixtures have better resistance to low-temperature (thermal) cracking. The laboratory tests conducted to evaluate the effect of the temperature reduction, foaming water content, and aggregate moisture content revealed that the performance of 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. However, reducing the production temperature of foamed WMA resulted in increased susceptibility to permanent deformation and moisture-induced damage. In addition, producing foamed WMA using moist aggregates resulted in inadequate aggregate coating leading to concerns with regard to long-term durability. Increasing the foaming water content (up to 2.6% of the weight of the asphalt binder) did not seem to have a negative effect on the rutting performance or moisture sensitivity of the foamed WMA. The rut depth measurements obtained at the OU APLF confirmed the laboratory APA test results. It was found through these tests that the foamed WMA mixtures have comparable rutting resistance to the HMA mixtures. Finally, the long-term pavement performance predictions obtained using the MEPDG showed comparable service lives for pavement structures constructed using foamed WMA and HMA surface and intermediate mixtures.</td></tr>
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Determining the Limitations of Warm Mix Asphalt by Water Injection in Mix Design, Quality Control and Placement

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Determining the Limitations of Warm Mix Asphalt
by Water Injection in Mix Design, Quality Control and Placement

Abstract

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 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 report 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).

The laboratory test results revealed comparable rut depth values in the asphalt pavement analyzer (APA), slightly lower dynamic moduli (E*), slightly lower flow number (FN) values, slightly lower indirect tensile strength (ITS) values in the AASHTO T 283 test, and slightly lower dissipated creep strain energy (DCSE) values for the foamed WMA and HMA mixtures. However, the difference was found to be statistically insignificant between the two mixtures. These results indicate that the performance of foamed WMA mixtures is comparable to that of traditional HMA mixtures in terms of rutting, moisture-induced damage, and fatigue cracking. As for low-temperature (thermal) cracking, the foamed WMA mixtures exhibited slightly lower ITS values at 14^{o}F (-10^{o}C) and comparable or slightly higher failure strain values than the HMA mixtures. Through statistical analysis, it was found that the effect of the mix type was significant on the low-temperature ITS values, but not on the failure strains. Since the HMA mixtures had significantly higher ITS values and comparable failure strains to the foamed WMA mixtures, the HMA mixtures are expected to have better resistance to thermal cracking.

The laboratory tests conducted to evaluate the effect of the temperature reduction, foaming water content, and aggregate moisture content revealed that the performance of foamed WMA mixtures prepared using 30^{o}F (16.7^{o}C) temperature reduction, 1.8% foaming water content, and fully dried aggregates was comparable to that of the traditional HMA mixtures. However, reducing the production temperature of the foamed WMA resulted in increased susceptibility to permanent deformation and moisture-induced damage. In addition, producing foamed WMA using moist aggregates resulted in inadequate aggregate coating leading to concerns with regard to long-term durability. It was also found that 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.

The rut depth measurements obtained at the APLF at Ohio University confirmed the laboratory APA test results. It was found through these tests that foamed WMA mixtures have comparable rutting resistance to traditional HMA mixtures. Finally, the long-term pavement performance predictions obtained using the MEPDG showed comparable service lives for pavement structures constructed using foamed WMA and HMA surface and intermediate mixtures.

Chapter 1
Introduction

1.1 Problem Statement

Hot mix asphalt (HMA) is the most common material used for asphalt paving applications. 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^oF to 325^oF (148.9^oC to 162.7^oC) 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^oF to 100^oF (16.7^oC to 55.6^oC) 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.

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 aggregates may absorb less of the asphalt binder. Other concerns include (3) insufficient coating of coarse aggregates, and (4) applicability of HMA mix design procedures to foamed WMA mixtures.

In general, 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 foamed WMA mixtures 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 foamed WMA mixtures and what impact does insufficient aggregate coating have on mix durability?
- Since foamed WMA mixtures are more workable and compactable than HMA mixtures, should they be compacted to a higher density level in the field?

To answer the first question, the Ohio Department of Transportation (ODOT) has previously contracted with the University of Akron in a student study to compare the performance of HMA and foamed WMA mixtures with regard to permanent deformation and moisture-induced damage (Abbas and Ali, 2011). 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 Construction and Material Specifications (C&MS) Item 441 Type 1 Surface Course for Medium Traffic. The resistance to permanent deformation was measured using the asphalt pavement analyzer (APA) and dynamic modulus tests; and the resistance to moisture-induced damage was measured using AASHTO T 283. The experimental test results revealed a slight increase in rut depth and a slight reduction in tensile strength ratio (TSR) for the foamed WMA mixtures. By comparing the peak load levels obtained in AASHTO T 283, it was noticed that foamed WMA mixtures prepared using limestone aggregates had relatively lower indirect tensile strength values than the corresponding HMA mixtures for both unconditioned and conditioned specimens. However, the indirect tensile strength values were nearly the same for foamed WMA and HMA mixtures prepared using gravel.

Based on the previous discussion, 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 should be evaluated to ensure satisfactory long-term performance.

1.2 Objectives of the Study

The primary objective in this study is to evaluate the performance of foamed WMA for conditions prevalent in Ohio through a comprehensive laboratory and field evaluation plan. 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 foamed WMA mixtures.
- 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 their applicability to foamed WMA mixtures.
- Identify any changes to current ODOT specifications for foamed WMA mixtures that are needed to ensure acceptable performance.

1.3 Report Organization

This report 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 3 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 method used to produce the foamed WMA in the laboratory. Chapter 4 details the laboratory testing program that was implemented to evaluate the performance of both mixtures. The laboratory test results are presented in Chapter 5.

- The second component focuses on the evaluation of the workability and compactability of foamed WMA and HMA mixtures. Chapter 6 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 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 4.
- The third component discusses the effect of the mix preparation procedure on the performance of foamed WMA mixtures. Chapter 7 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 8 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 9 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 5. 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.

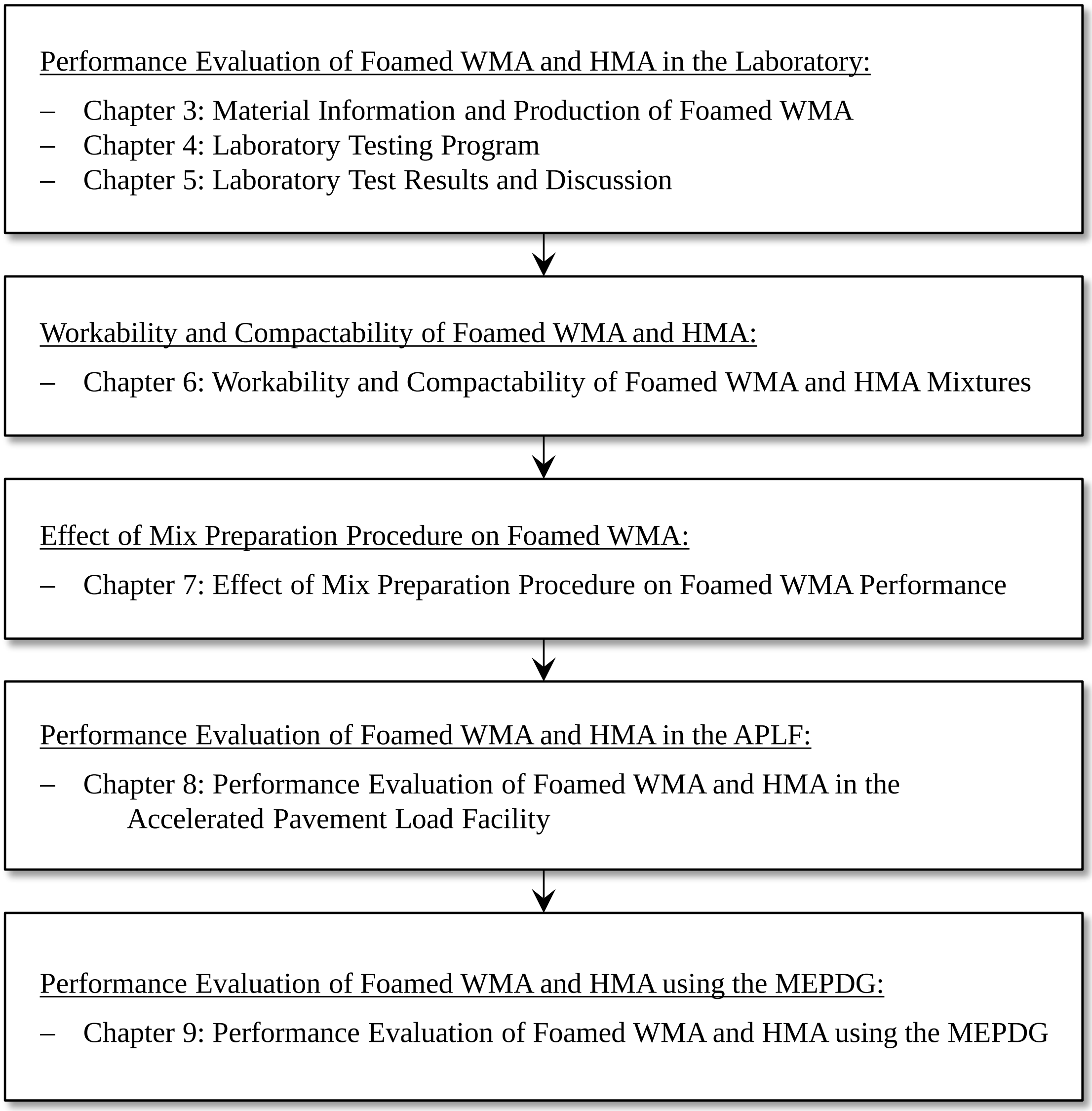

Figure 1.1: Report Organization.

Chapter 2
Literature Review

2.1 Introduction

Warm mix asphalt (WMA) is a generic term for an asphalt mixture placed at lower than conventional temperatures. This material was developed in Europe with the aim of reducing greenhouse gases resulting from asphalt mix production (Button et al., 2007). While heat is used to reduce asphalt viscosity and dry the aggregates during mixing of conventional asphalt mixtures, WMA reduces asphalt viscosity by using special organic or chemical additives or by foaming the asphalt binder in the mix. The reduction in viscosity allows the asphalt binder to adequately coat the aggregates during mixing. It also improves mix workability and allows for compaction at lower temperatures. WMA has been widely adopted in the United States in the past decade, and a significant effort has been made to improve available technologies and develop new products and processes for the production of this material.

2.2 Common WMA Technologies

Various WMA technologies have been proposed in the past few years. These technologies can be classified into two main types. The first type uses some form of organic or chemical additives to produce WMA, while the other type is produced by foaming the asphalt binder. The latter is achieved by adding a small amount of water to the binder, either via a foaming nozzle or a hydrophilic material such as Aspha-min. The added water then turns to steam and expands. This results in a reduction of viscosity due to the expansion of the liquid asphalt binder. Currently, foamed WMA produced via a foaming nozzle is gaining popularity among asphalt mix producers. Asphalt plant manufacturers with foaming technologies include Terex, Gencor, and Astec Double Barrel Green systems. These are sometimes referred to as foamed asphalt or “free water” systems. The main advantage of these systems is that they allow for the production of WMA with a standard grade asphalt binder through a one-time mechanical plant modification, minimizing the impact of increased material costs identified with other WMA technologies. The following subsections briefly describe the most common WMA products and processes available in the market, and discuss the mechanism by which they facilitate the production of asphalt mixtures at lower temperatures.

2.2.1 Sasobit

Sasobit is a synthetic wax produced during the coal gasification process. It acts as a compaction aid and permits production and placement of asphalt mixtures at temperatures lower than those used for HMA. Sasobit is supplied in pellet form and can be added to the mixture by blowing it into the mixing drum or be incorporated into the asphalt binder either at the asphalt terminal or in the asphalt tank at the production plant (Bonaquist, 2011). Sasol Wax, the manufacturer of Sasobit, does not recommend introducing it directly into the asphalt mixture as this might result in an inhomogeneous distribution of Sasobit within the mix. However, the need for plant modification for pre-blending of Sasobit with the asphalt binder might increase the overall cost of mixtures produced using this technology. The optimum Sasobit dosage, as recommended by Sasol Wax, ranges between 3 to 4% by weight of the asphalt binder, allowing for the production of asphalt mixtures at a temperature reduction of 18^{o}F to 54^{o}F (10^{o}C to 30^{o}C).

2.2.2 Evotherm

MeadWestvaco Corporation, the manufacturer of Evotherm, has introduced three different types of chemical additives: Evotherm Emulsion Technology (ET), Evotherm Dispersed Asphalt Technology (DAT), and Evotherm Third Generation (3G/Revix). The mechanism by which the Evotherm ET additive facilitates the production of WMA mixtures is by introducing a water-based emulsion to the hot aggregates during the mixing process. Upon contact with the hot aggregates, the water-based emulsion turns into steam, which causes the asphalt binder to foam. The production of the water-based emulsion, according to MeadWestvaco, involves using a chemical package that contains the additives necessary to enhance the coating of aggregates and increase the workability of WMA mixtures produced using this technology. Evotherm DAT technology is similar to Evotherm ET in that it utilizes a water-based emulsion for 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. In contrast 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. Instead, the chemical additive reduces the internal friction of the mixture, allowing the asphalt binder to behave as if it were heated to a higher temperature. Similar to Evotherm DAT technology, the 3G additive can be directly injected into the asphalt binder line just before the asphalt enters the mixing chamber. In addition, Evotherm 3G additive can also be pre-blended with the asphalt binder at the mixing plant. The optimum dosage of any of the Evotherm additives ranges between 0.4 to 0.7% by total weight of asphalt binder. The use of this dosage is expected to facilitate the production of WMA mixtures at about 50^{o}F to 100^{o}F (27.8^{o}C to 55.6^{o}C) lower than HMA mixtures produced using the same asphalt binder.

2.2.3 Rediset LQ

Rediset LQ, produced by Azko Nobel N.V., is another chemical additive that permits the production of 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 the additive. 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 and compactability of asphalt mixtures at lower temperatures.

Similar to the Evotherm additives, Rediset LQ can be pre-blended with the asphalt binder or directly injected into the asphalt binder just before the binder is introduced into the mixing chamber. The optimum dosage of Rediset LQ ranges between 0.3 to 0.6% by weight of effective asphalt binder content. Generally, dosages within this range do not change the performance grade (PG) of the asphalt binder, and they allow the WMA mixtures to be produced at temperatures 40^{o}F to 60^{o}F (22.2^{o}C to 33.3^{o}C) lower than those traditionally used for HMA production. Rediset LQ is supplied in liquid form, which facilitates handling and metering at the asphalt plant.

2.2.4 SonneWarmix

SonneWarmix, produced by Sonneborn Inc., is a wax-based WMA additive that is composed of paraffinic hydrocarbons. As in all the previous WMA additives discussed, SonneWarmix can either 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 for SonneWarmix is similar to that of Sasobit. In particular, blending this additive with the asphalt binder helps to reduce the binder’s viscosity at temperatures above the melting point of the added wax. The optimum dosage rate of

SonneWarmix, as recommended by Sonneborn Inc., ranges between 0.5 to 1.5% by total weight of asphalt binder. The manufacturer reports that the use of this dosage rate does not change the binder PG, and facilitates the production of WMA at temperatures 50^{o}F (27.8^{o}C) lower than those typically used for HMA mixtures.

2.2.5 Aspha-min

Aspha-min, produced by Eurovia Services GmbH, is a WMA additive that is used to foam asphalt binders during the mixing stage. It is a synthetic zeolite (a microporous, aluminosilicate material often used as an adsorbent) that contains approximately 20 percent crystallized water by total weight within its structure. Aspha-min is usually introduced during the mixing process, either at the same time or shortly after the asphalt binder is added to 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, and this causes the asphalt binder to foam. As a result of this foaming process, the viscosity of the asphalt binder is reduced, which facilitates the use of lower temperatures than those used for production of traditional HMA. The optimum Aspha-min dosage, as recommended by its manufacturer, is approximately 0.3% by total weight of the mixture. To ensure uniform distribution of Aspha-min within the mixture, the manufacturer recommends using a distribution unit that can be attached to the mixing plant (Barthel and Bon Devivere, 2003). Aspha-min is expected to facilitate the production of WMA mixtures at about 50^{o}F (27.8^{o}C) lower than those for traditional HMA mixtures.

2.2.6 Advera

Advera, produced by PQ Corporation, is another synthetic zeolite additive used to foam asphalt binders during production of WMA. Advera contains about 18 percent crystallized water by total weight inside its structure. Although both the Aspha-min and Advera additives will foam the asphalt binder using the same method, Advera has a finer particle size distribution that is primarily composed of particles passing Sieve #200 (i.e., smaller than 74 microns). The manufacturer of Advera claims that such particle size would result in a more uniform distribution of additive within the mixture. This, in turn, might result in better foaming of the asphalt binder.

Similar to Aspha-min, Advera is added to the asphalt mixture at the pugmill (in a batch plant) or via a fiber port (in a drum plant). However, because the use of Advera does not require the addition of a distribution unit into the plant, WMA mixtures produced using Advera may be less expensive than those produced using Aspha-min. The use of Advera is expected to allow the production of WMA mixtures at temperatures 50^oF to 70^oF (27.8^oC to 38.9^oC) lower than those used in HMA production.

2.2.7 Double Barrel Green System

The Double Barrel Green System, developed by Astec Inc., is a drum plant retrofitted with a multi-nozzle foaming device to produce foamed WMA. The Double Barrel Green System does not use additives to foam the binder, but instead generates the foam by injecting a small amount of water into the asphalt binder to create microscopic bubbles. These tiny bubbles act to reduce the viscosity of the binder, enabling the mix to be handled and worked at lower temperatures than those used for HMA. The amount of water injected into the asphalt binder is controlled through a positive displacement piston pump, which controls the amount of water going into the system. According to Astec, this process can be used to produce foamed WMA at temperatures ranging from 230^oF to 270^oF (110^oC to 132.2^oC) versus the traditional temperatures of 300^oF to 340^oF (148.9^oC to 171.1^oC).

2.2.8 Gencor Green Machine Ultrafoam GX2

The Ultrafoam GX2, produced by Gencor Industries Inc., is an asphalt binder foaming system that utilizes water to produce WMA mixtures. This device is designed to be attached to the existing asphalt injection line in a typical drum plant. Similar to the Double Barrel Green System, the Ultrafoam GX2 foams the asphalt binder by injecting a small amount of water (approximately 1.25 to 2% by weight) into the flowing asphalt binder. This system is equipped with a hot oil jacket to maintain the temperature of the heated asphalt binder during production. The 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. A spring-loaded valve is used to control water flow, while asphalt flow is regulated using a specially designed diaphragm. The manufacturer of the Ultrafoam GX2 system claims that this design can maintain an accurate water-to-asphalt ratio.

2.3 Performance Evaluation of WMA

Over the last decade, several research studies have been conducted to evaluate the performance of WMA with regard to permanent deformation (rutting), moisture-induced damage, fatigue cracking, and low temperature (thermal) cracking. Wielinski et al. (2009) reported the results of two paving projects constructed using foamed WMA produced by the Double Barrel Green system and traditional HMA. Foamed WMA and HMA samples were obtained during construction and compacted for further testing in the laboratory. The laboratory test results showed lower initial stiffness for the foamed WMA and higher asphalt pavement analyzer (APA) rut depths. Furthermore, both HMA and foamed WMA had low tensile strength ratio (TSR) values, with the foamed WMA results being slightly lower than the HMA. It was also reported in this study that conventional mix design methods could be used for WMA mixtures produced using the Double Barrel Green system.

Middleton and Forfylow (2009) presented a comparison between six WMA technologies used in North America, including Sasobit, Aspha-min, Evotherm, low-energy asphalt (LEA), WAM-Foam, and the Double Barrel Green system. The asphalt pavement analyzer (APA) and AASHTO T 283 tests were used to evaluate the susceptibility to rutting and moisture-induced damage, respectively. It was reported that the performance of foamed WMA produced using the Double Barrel Green system is similar to that of HMA. Furthermore, it was noted that the use of reclaimed asphalt pavement (RAP) and manufactured shingle modifier (MSM) in conjunction with the Double Barrel Green process did not significantly influence the mix properties and performance.

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 test results indicated that the laboratory-produced WMA was more prone to moisture susceptibility than the plant-produced mix. Furthermore, the HMA exhibited more favorable moisture susceptibility results than the WMA. However, most of the WMA samples did meet the moisture susceptibility requirement. Hodo et al. (2009) reported similar results based on tests conducted on foamed WMA mixtures obtained from field demonstration test sections in Chattanooga, Tennessee.

Kvasnak et al. (2010) evaluated the laboratory performance of foamed WMA produced in a plant equipped with a Gencor Green Machine Ultrafoam GX and compared it to HMA prepared using the same aggregate and binder materials. The test results showed that in general the performance of the foamed WMA was lower than that of the HMA. However, the WMA performance exceeded the minimum threshold requirements for most of the tests. The Hamburg wheel tracking device (HWTD) and asphalt pavement analyzer (APA) test results were acceptable for both foamed WMA and HMA. In addition, the indirect tensile strength (ITS) for the foamed WMA was high and improved with aging. However, it did not meet the 0.8 minimum tensile strength ratio (TSR) requirement.

Xiao et al. (2010) examined the influence of three anti-stripping additives (hydrated lime and two liquid anti-stripping additives) on the moisture susceptibility of WMA mixtures. This study utilized Aspha-min and Sasobit to prepare the WMA mixtures. One asphalt binder and three aggregate types were used in this study. The measured properties included the indirect tensile strength (ITS), tensile strength ratio (TSR), flow, and toughness. It was reported that the ITS values for WMA mixtures were lower than those for HMA mixtures. Furthermore, it was reported that the WMA mixtures prepared with hydrated lime had the best moisture resistance, while those prepared using liquid anti-stripping additives showed no significant improvement in moisture resistance.

Copeland et al. (2010) presented the results of a field evaluation study conducted in Florida on foamed WMA containing 45% RAP. The performance of this material was compared to a traditional HMA produced using the same amount of RAP. Plant-produced loose asphalt mixtures were collected during construction for performance testing evaluation. The performance tests included the determination of the asphalt binder performance grade (PG) and measuring the dynamic modulus and flow number of the asphalt mixtures. The asphalt binder test results indicated that the binder in the high RAP-WMA mixture was softer than that in the high RAP-HMA control mixture. The high RAP-WMA mixture also showed lower stiffness than the high RAP-HMA mixture in the dynamic modulus especially at intermediate temperatures. The flow number test results were consistent with the dynamic modulus test results in that the high RAP-WMA mixture had a lower flow number than the high RAP-HMA mixture. Based on a comparison of measured dynamic modulus results with those predicted using the Hirsch and Witczak models, it was reported that complete blending occurred in the high RAP-HMA control mix; however, incomplete mixing of RAP and virgin binders may have occurred in the high RAP-WMA mix.

Arabani et al. (2011) investigated the moisture sensitivity of WMA mixtures prepared using Sasobit and Aspha-min. A moisture sensitivity index was used as an indication of the moisture susceptibility of asphalt mixtures. This index was defined as the percentage of aggregate surface exposed to water and was calculated based on measurements obtained using the surface free energy method and the dynamic modulus test. It was reported that the aggregate-asphalt surface energy of adhesion in mixtures containing Sasobit and Aspha-min was lower than that measured in the control mix. Therefore, it was concluded that the use of Sasobit and Aspha-min may increase the moisture susceptibility of asphalt mixtures.

Mogawer et al. (2011) investigated the effect of four WMA technologies (Advera, Evotherm, Sasobit, and SonneWarmix) on the moisture sensitivity of the asphalt mixtures and the adhesion characteristics of the asphalt binders. A 9.5-mm Superpave mixture prepared using a PG 64-22 asphalt binder was used as the control mixture. The moisture sensitivity of the asphalt mixtures was evaluated using the Hamburg wheel tracking device (HWTD) and the binder-aggregate bond strength was measured using a pull-off test called the bitumen bond strength (BSS) test. The effect of each WMA technology on the moisture susceptibility of the asphalt mixture was evaluated at three aging times and three aging temperatures. It was reported that the moisture resistance for all mixtures improved with the increase in aging time and temperature. Furthermore, it was reported that only Sasobit had a significant effect on the pull off tensile strength of the asphalt binder, which was the case for the dry samples but not the conditioned ones.

Haggag et al. (2011) evaluated the effect of three WMA technologies (Advera, Evotherm 3G, and Sasobit) on the fatigue cracking resistance of asphalt mixtures using a uniaxial cyclic tension-compression test. The asphalt mixtures were produced using a PG 64-22 virgin binder and two aggregate sources. The experimental test data was analyzed using the simplified viscoelastic continuum damage approach proposed in the NCHRP 9-43 Phase I report. It was reported that there was no significant difference in fatigue cracking resistance between HMA and WMA mixtures except for Advera that showed lower resistance to fatigue cracking. This suggests that the effect of WMA on fatigue resistance is specific to the particular technology used in producing the asphalt mixture.

Liu et al. (2011) evaluated the performance of WMA mixtures produced using Sasobit to determine their suitability for use in Alaska. This study investigated the effect of Sasobit on the rheological properties of the asphalt binders and the performance of the WMA mixtures in terms of low temperature behavior, rutting resistance, and moisture susceptibility. It was reported that the addition of Sasobit resulted in reduced mixing and compaction temperatures, improved workability and rutting resistance, and no significant effect on moisture susceptibility. The indirect tension test results showed lower ITS values for the WMA mixtures than the HMA mixtures at low temperatures. However, it was indicated that additional tests at low temperatures along with a more complete thermal cracking analysis are needed to obtain a more definitive answer regarding the low temperature performance of these mixes.

In a subsequent study, Liu and Li (2012) focused on evaluating the low temperature performance of Sasobit-modified WMA binders and mixtures. The bending beam rheometer (BBR), direct tension test (DTT), and asphalt binder cracking device (ABCD) were utilized to characterize the low temperature performance of the asphalt binders, and the indirect tension test (IDT) along with thermal cracking analysis were performed to evaluate the susceptibility of the Sasobit-modified WMA mixtures to low temperature cracking. The test results showed a decrease in tensile strength for both WMA binders and mixtures at low temperatures, and an increase in cracking temperature with the increase of Sasobit content for both WMA binders and mixtures. However, the increase in cracking temperature was relatively small, which indicated that the addition of Sasobit did not have a significant effect on the resistance to low temperature cracking.

Buss et al. (2011) evaluated the performance of four WMA mixtures produced using Evotherm 3G, Revix, Sasobit, and Astec's Double Barrel Green System. Field compacted and reheated field samples were utilized for the indirect tensile strength (ITS), dynamic modulus and flow number tests. A total of 284 samples were prepared and half of these samples were moisture conditioned. It was reported that the overall performance of the HMA was better than the WMA produced using Evotherm 3G, Revix, and Sasobit and that the WMA produced using the Double Barrel Green System was the only WMA technology that performed better than HMA in some of the tests.

Cooper et al. (2011) evaluated the performance of sulfur-modified WMA and compared it to traditional HMA. Three asphalt mixtures were included in this study. The first mixture was prepared as a HMA using a neat PG 64-22 asphalt binder, the second mixture was prepared as a HMA using a styrene-butadiene-styrene (SBS) PG 70-22 modified asphalt binder, and the third mixture was prepared as a WMA using a sulfur-based additive and PG 64-22 asphalt binder. The performance of the three mixtures was evaluated for permanent deformation (or rutting), moisture-induced damage, fatigue cracking, and low-temperature thermal cracking. The laboratory test results showed that the rutting performance of the sulfur-modified WMA was comparable or superior to conventional HMA prepared with neat or modified asphalt binders. Furthermore, the resistance of the sulfur-modified WMA to moisture-induced damage, as measured using the modified Lottman test, was comparable to HMA. However, the results of the fracture tests showed that sulfur-modified WMA is more susceptible to cracking than HMA, given its stiff characteristics. The thermal stress restrained specimen test results showed that the sulfur-modified WMA had a greater fracture stress than the polymer-modified HMA mixture. However, there was no statistical significance between the fracture temperatures of these mixtures.

Saragand et al. (2011) evaluated the field performance of WMA mixtures containing RAP and compared it to traditional HMA. The WMA mixtures were produced using Aspha-min, Sasobit, and Evotherm. Temperature and emissions were monitored during production and placement for all mixtures. In addition, core samples were obtained from the field sections and tested in the laboratory for indirect tensile strength (ITS) and tensile strength ratio (TSR). Roughness and rutting measurements were also conducted during the first 46 months of service. It was reported that emissions were significantly reduced during the production and placement of WMA mixtures as compared to the control HMA mixture. In addition, it was reported that WMA mixtures achieved higher in-place density than the control HMA mixture even though they were compacted at lower temperatures. The laboratory test results showed that the WMA mixtures had higher ITS values than the HMA mixture after 3 months of service. However, the ITS value of the HMA increased more rapidly with time than that of the WMA. The moisture sensitivity test results demonstrated that the Sasobit and Evotherm mixtures had acceptable resistance to moisture-induced damage. Furthermore, it was reported that the WMA and HMA sections had similar International Roughness Index (IRI) values after 46 months of service with no measurable rutting.

Kim et al. (2012) evaluated the laboratory and field performance of two WMA mixtures, one produced using a powder additive based on the foaming technology and the other produced using a liquid chemical additive. Trial pavement sections of these WMA mixtures and the corresponding HMA mixtures were constructed in Antelope County, Nebraska. Plant-mixed loose mixtures were collected at the time of paving and transported to the laboratory for further testing using the asphalt pavement analyzer (APA) test performed under water, AASHTO T 283, and semi-circular bending (SCB) fracture test with moisture conditioning. The APA test results did not show any difference between the WMA and HMA mixtures. However, the AASHTO T 283 and SCB test results indicated a greater susceptibility to moisture-induced damage for WMA mixtures than the corresponding HMA mixtures. Satisfactory early-stage field performance was reported for both WMA and HMA test sections. Nevertheless, it was suggested to continue observing the field performance over a long period of time to evaluate the moisture sensitivity of these materials since moisture damage is a distress that is typically accelerated by rutting and cracking.

Bernier et al. (2012) presented the results of the first WMA pilot project that was constructed in Connecticut in 2010. This project involved three pavement sections: one section paved with conventional HMA and two sections paved with WMA produced using Sasobit and asphalt binder foamed by water injection. Field-produced loose asphalt mixtures were obtained during construction and reheated and compacted in the laboratory for further testing using the semi-circular bending (SCB), Hamburg wheel tracking device (HWTD), indirect tension (IDT), and disk compact tension (DC(T)) tests. It was reported that WMA mixtures containing Sasobit exhibited the greatest amount of rutting in the lab, but had the longest time until the stripping inflection point, while the WMA mixtures containing foamed asphalt had reduced rutting susceptibility as compared to the HMA and showed no adverse effect on the inflection point. No significant difference was observed in fracture energy and toughness, as measured by the SCB and DC(T) tests, between the WMA and HMA mixtures. With regard to the field performance, it was reported that the Sasobit field sections had the greatest amount of linear cracking, followed by the foamed WMA and the HMA. The Sasobit field sections also exhibited the highest amount of rutting, which is consistent with the laboratory test results.

Hill et al. (2012) examined the effect of four WMA technologies that included Sasobit, Evotherm 3G, Advera, and Rediset LQ on the low temperature fracture properties of asphalt mixtures. The disk-shaped compact tension (DC(T)), indirect tension (IDT), and acoustic emission (AE) tests were used to characterize the low temperature mixture properties. The DC(T) fracture energy results showed that the chemical additives (Evotherm 3G and Rediset LQ) improved fracture energy in comparison to HMA, while the organic and foaming additives (Sasobit and Advera, respectively) reduced fracture energy. The IDT creep compliance test produced similar results, where the two chemical additive modified WMA systems increased the mix creep compliance, while the other two systems did not significantly influence the creep compliance as compared to the control HMA mixture. The AE test was utilized to measure the embrittlement temperature of the asphalt mixtures, which is defined as the temperature at which acoustic emission activity initiates when the sample is subjected to a specified rate of cooling. This temperature is used to indicate the onset of the brittle regime, where macro cracks can propagate under thermal and mechanical loads. The AE test results for the foaming additive exhibited a similar embrittlement temperature to HMA, while the organic additive increased the embrittlement temperature. The two chemical additive modified mixtures produced different embrittlement temperatures. Based on the results of this study, it was concluded that the resistance to thermal cracking is not ensured by virtue of producing WMA mixtures at lower temperature.

Alavi et al. (2012) evaluated the adhesive properties and moisture susceptibility of WMA produced using three additives, including a water bearing mineral (WB), a solid pelletized surfactant (PS), and a chemical-based viscosity reducer (VR). The impact of moisture on the bond strength at the asphalt-aggregate interface was measured using bitumen bond strength (BBS) test, and the moisture sensitivity of the asphalt mixtures was assessed using the dynamic modulus ratio at multiple freeze-thaw cycles. It was reported that the use of specific WMA additives has the potential to increase moisture resistance, offsetting any negative effects of reduced production temperatures on moisture susceptibility. As a result, the potential for moisture damage associated with the use of WMA can be mitigated during the mix design process through selection of appropriate additives in the mixture design.

Mogawer et al. (2012) evaluated the effect of polymer-modified asphalt binder, high RAP content, and WMA on the stiffness, resistance to reflective cracking, moisture susceptibility, and workability of high-performance thin asphalt overlay mixtures. A Superpave 9.5-mm mixture was designed with solely virgin aggregates and designated as the control mixture. One neat and four polymer-modified asphalt binders were included in this study. The WMA mixtures were produced using a wax-based additive called SonneWarmix that was added at a rate of 1.0% by weight of total binder (virgin plus RAP binder). Both WMA and HMA mixtures were produced using 0% and 40% RAP. It was reported that the use of polymer-modified asphalt binders and high RAP contents significantly increased the stiffness of the asphalt mixture. However, the use of WMA lowered the mix stiffness and improved its workability. It was also reported that the use of WMA in combination with polymer-modified asphalt binders and/or RAP may result in reduced moisture susceptibility and rutting performance.

Zhao et al. (2012) evaluated the laboratory performance of WMA mixtures containing varying percentages of RAP ranging from 0% to 50%. The WMA was plant produced using asphalt binder foamed by water injection. The asphalt pavement analyzer (APA), Hamburg wheel tracking device (HWTD), AASHTO T 283, indirect tension (IDT), and bending beam fatigue tests were used to characterize the behavior of the asphalt mixtures. It was reported that the addition of RAP significantly improved the rutting resistance of the WMA mixtures and that this improvement was more significant than that in HMA mixtures. It was also reported that the use of RAP improved the resistance to moisture-induced damage, but slightly reduced the fatigue life of the WMA mixtures.

Rushing et al. (2013) assessed the applicability of using WMA for airfield pavements. Eleven WMA technologies were evaluated in the laboratory for permanent deformation and moisture damage and were compared to traditional HMA mixtures prepared using the same aggregate blend. Three of these technologies were also produced in the field and transported to the laboratory for further evaluation using the asphalt pavement analyzer (APA), Hamburg wheel tracking device (HWTD), and AASHTO T 283 tests. The test results indicated that WMA is a potentially viable material to use for surface courses in airfield pavements. It was reported that although WMA exhibited poorer performance than HMA in moisture damage tests on laboratory-produced specimens, the plant-produced mix indicated little difference compared to HMA. The WMA mixtures also exhibited higher rutting potential in the APA and HWTD tests than the HMA mixtures produced both in the laboratory and in an asphalt plant. This difference in performance was not attributed to a specific WMA technology.

2.4 Effect of Mix Preparation on WMA Performance

A number of research studies have also investigated the effect of the mix preparation procedure, such as temperature reduction, foaming water content (if applicable), and aggregate moisture content, on the performance of WMA. Xiao et al. (2009) conducted a laboratory study to examine moisture-induced damage in WMA mixtures containing moist aggregates. Two WMA additives (Aspha-min and Sasobit), three aggregate sources, two aggregate moisture contents (0% and approximately 0.5% by weight of dry aggregate), and three lime contents (0%, 1%, and 2% by weight of dry aggregate) were included in this study. The test results indicated that the use of the WMA additive did not significantly affect the dry ITS and toughness values. However, dry ITS was affected by the aggregate moisture content. It was also reported that the deformation resistance and TSR decreased with the increase in aggregate moisture content.

In a subsequent study by the same research group, Xiao et al. (2010) evaluated the rutting resistance of WMA mixtures containing moist aggregates. Three WMA additives (Aspha-min, Sasobit, and Evotherm), three aggregate sources, two aggregate moisture contents (0% and approximately 0.5% by weight of dry aggregate), and two lime contents (1% and 2% by weight of dry aggregate) were included in this study. The test results indicated that the aggregate source significantly affected the rutting resistance regardless of the WMA additive, aggregate moisture content, and lime content. It was also reported that the rut depth of the asphalt mixtures containing moist aggregates generally satisfied the minimum performance criteria without the need for any additional treatment. The mixture with Sasobit additive had the best rutting resistance, while mixtures with Aspha-min and Evotherm additives showed similar rutting resistance to that of the control mixture.

Fu et al. (2010) investigated the effect of asphalt foaming and fines content on foamed WMA strength. The test results showed that a small change in foaming parameters (asphalt temperature and foaming water content) can significantly alter the expansion ratio and half-life, and that asphalt binders with better foaming properties (higher expansion ratio and longer half-life) tend to yield mixes with higher strength values. As a result, it was suggested that the use of a binder with better foaming properties can potentially lower the asphalt binder content while achieving the same stabilization effect. As for the effect of fines, it was reported that introducing excessive amounts of fines, especially greater than 12 percent, is detrimental to the performance of foamed WMA and hence is discouraged in engineering practice.

Bennert et al. (2011) evaluated the effect of production temperature and aggregate moisture content on rutting, fatigue cracking, and moisture sensitivity of WMA. Three WMA additives (Evotherm 3G, Rediset, and Sasobit) were used in this study. A decrease in rutting resistance and stiffness and an increase in fatigue cracking resistance were reported with the decrease in mixing temperature. It was observed that in order to obtain passing TSR and wet Hamburg test results, it was necessary to use conventional HMA mixing temperatures and dry 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 commonly 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 3
Material Information and Production of Foamed WMA

3.1 Material Information

Four material combinations were used in this study representing two surface and two intermediate asphalt mixtures, as shown in Figure 3.1. As can be noticed from this figure, the surface mixtures were prepared using PG 70-22 asphalt binder and limestone and crushed gravel aggregates, while the intermediate mixtures were prepared using limestone aggregate and PG 64-28 and PG 70-22 asphalt binders. These material combinations were selected to facilitate the determination of the effect of the mix type, aggregate type, and asphalt binder type on the performance of the asphalt mixtures. With the exception of the intermediate mixture prepared using limestone and PG 70-22, all mixtures met the ODOT Construction and Material Specifications (C&MS) for Item 442 (Superpave Asphalt Concrete). It is noted that ODOT requires using PG 64-28 for Superpave intermediate mixes. However, this non-standard intermediate mixture was included in this study to allow for determining the effect of the asphalt binder type on the mix performance.

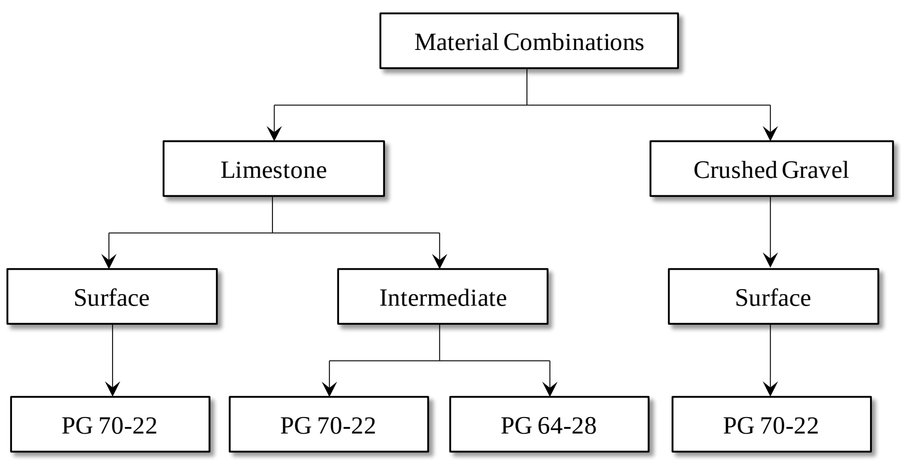

Figure 3.1: Selected Material Combinations.

3.2 Aggregates

The limestone aggregate blend was prepared by mixing #57 limestone, #8 limestone, limestone sand, and natural sand. The crushed gravel aggregate blend was prepared by mixing #8 crushed gravel, #9 crushed gravel, manufactured sand, and natural sand. The aggregates used in the two blends are pictured in Figures 3.2 and 3.3, respectively. All aggregates were obtained from ODOT-approved suppliers. Tables 3.1 and 3.2 present the aggregate gradation and Table 3.3 shows the bulk specific gravity and water absorption values provided by the aggregate suppliers for the various aggregate sizes.

Limestone Sand

Natural Sand

Limestone No. 8

Limestone No. 57

Figure 3.2: Picture of Aggregates Used to Prepare Limestone Aggregate Blend.

Manufactured Sand

Natural Sand

Crushed Gravel No. 9

Crushed Gravel No. 8

Figure 3.3: Picture of Aggregates Used to Prepare Crushed Gravel Aggregate Blend.

Table 3.1: Dry Gradation of Aggregates Used to Prepare Limestone Aggregate Blend.

	Percent Passing (%)			
Sieve Size	#57 Limestone	#8 Limestone	Limestone Sand	Natural Sand
1”	100	100	100	100
¾”	88	100	100	100
½”	44	100	100	100
3/8”	18	92	100	100
#4	4	21	94	99
#8	3	4	64	83
#16	0	2	40	61
#30	0	0	26	33
#50	0	0	17	10
#100	0	0	10	3
#200	0	0	6.1	2.3

Table 3.2: Dry Gradation of Aggregates Used to Prepare Crushed Gravel Aggregate Blend.

	Percent Passing (%)			
Sieve Size	#8 Crushed Gravel	#9 Crushed Gravel	Manufactured Sand	Natural Sand
1”	100	100	100	100
¾”	100	100	100	100
½”	100	100	100	100
3/8”	95	100	100	100
#4	20	100	100	98
#8	2	59	99	87
#16	2	10	92	73
#30	2	2	72	47
#50	2	2	57	14
#100	2	2	30	2
#200	2	2.2	10.5	2.6

Table 3.3: Bulk Specific Gravity and Absorption of Aggregates.

Aggregate Type	Bulk Specific Gravity, Gsb	Absorption (%)
#57 Limestone	2.607	1.50
#8 Limestone	2.579	2.00
Limestone Sand	2.611	1.50
Natural Sand	2.569	2.20
#8 Crushed Gravel	2.524	2.31
#9 Crushed Gravel	2.447	3.60
Manufactured Sand	2.588	1.00
Natural Sand	2.609	0.95

3.3 Asphalt Binders

The PG 70-22 and PG 64-28 asphalt binders were also obtained from ODOT-approved asphalt binder suppliers. Table 3.4 presents the asphalt properties provided by the suppliers for both binders. This table also shows the asphalt binder specific gravity at 60^{o}F (15.6^{o}C) and 77^{o}F (25.0^{o}C) as well as the HMA mixing and compaction temperatures for both asphalt binders. As can be seen from this table, the mixing and compaction temperatures for PG 70-22 are slightly higher than those for PG 64-28. This is expected because the former is a polymer modified asphalt binder with a higher high-temperature performance grade (i.e., higher stiffness at high service temperatures).

Table 3.4: Asphalt Binder Properties.

Binder Property	Binder Grade			
	PG 64-28		PG 70-22M	
Specific Gravity @ 60^{o}F	1.033		1.033	
Specific Gravity @ 77^{o}F	1.029		1.029	
Rotational Viscosity @ 275^{o}F, cP	425		1000	
Rotational Viscosity @ 329^{o}F, cP	130		315	
Mixing Temperature Range, ^{o}F	299 (Min)	307 (Max)	310 (Min)	322 (Max)
Compaction Temperature Range, ^{o}F	279 (Min)	286 (Max)	290 (Min)	300 (Max)

3.4 Mix Design

Four mix designs were conducted to determine the aggregate proportions and the optimum binder content for the selected material combinations. A 12.5 mm nominal maximum aggregate size (NMAS) was used for the surface mixtures, and a 19.0 mm NMAS was used for the intermediate mixtures. A summary of the mix design results is provided in Table 3.5 and the resulting aggregate gradations are presented in Figures 3.4a and 3.4b for the surface and intermediate mixtures, respectively. As can be noticed from this table and these figures, different aggregate gradations were used for the surface mixtures, while the same aggregate gradation was used for both intermediate mixtures. In addition, none of the asphalt mixtures contained reclaimed asphalt pavement (RAP). It can also be noticed from Table 3.5 that the optimum asphalt binder content for the surface mixtures ranged between 5.7% and 5.8%, while the optimum asphalt binder content for the intermediate mixtures ranged between 4.6% and 4.7%. It is noted that the current ODOT foamed WMA mix design procedure involves determining the optimum asphalt binder content for HMA mixtures and using that asphalt binder content in the preparation of the foamed WMA mixtures. Therefore, the standard Superpave mix design procedure was used in determining the optimum asphalt binder content for both HMA and foamed WMA mixtures.

Table 3.5: Summary of Mix Design Results

Aggregate	Limestone	Limestone	Limestone	Crushed Gravel
Mixture Type	Surface	Intermediate	Intermediate	Surface
NMAS (mm)	12.5	19	19	12.5
Aggregate Proportions	55% LS #8 30% LS Sand 15% Nat. Sand	37% LS #57 23% LS #8 25% LS Sand 15% Nat. Sand	37% LS #57 23% LS #8 25% LS Sand 15% Nat. Sand	51% CG #8 19% CG #9 15% Man. Sand 15% Nat. Sand
Binder	PG 70-22	PG 64-28	PG 70-22	PG 70-22
Optimum Binder Content	5.7%	4.7%	4.6%	5.7%

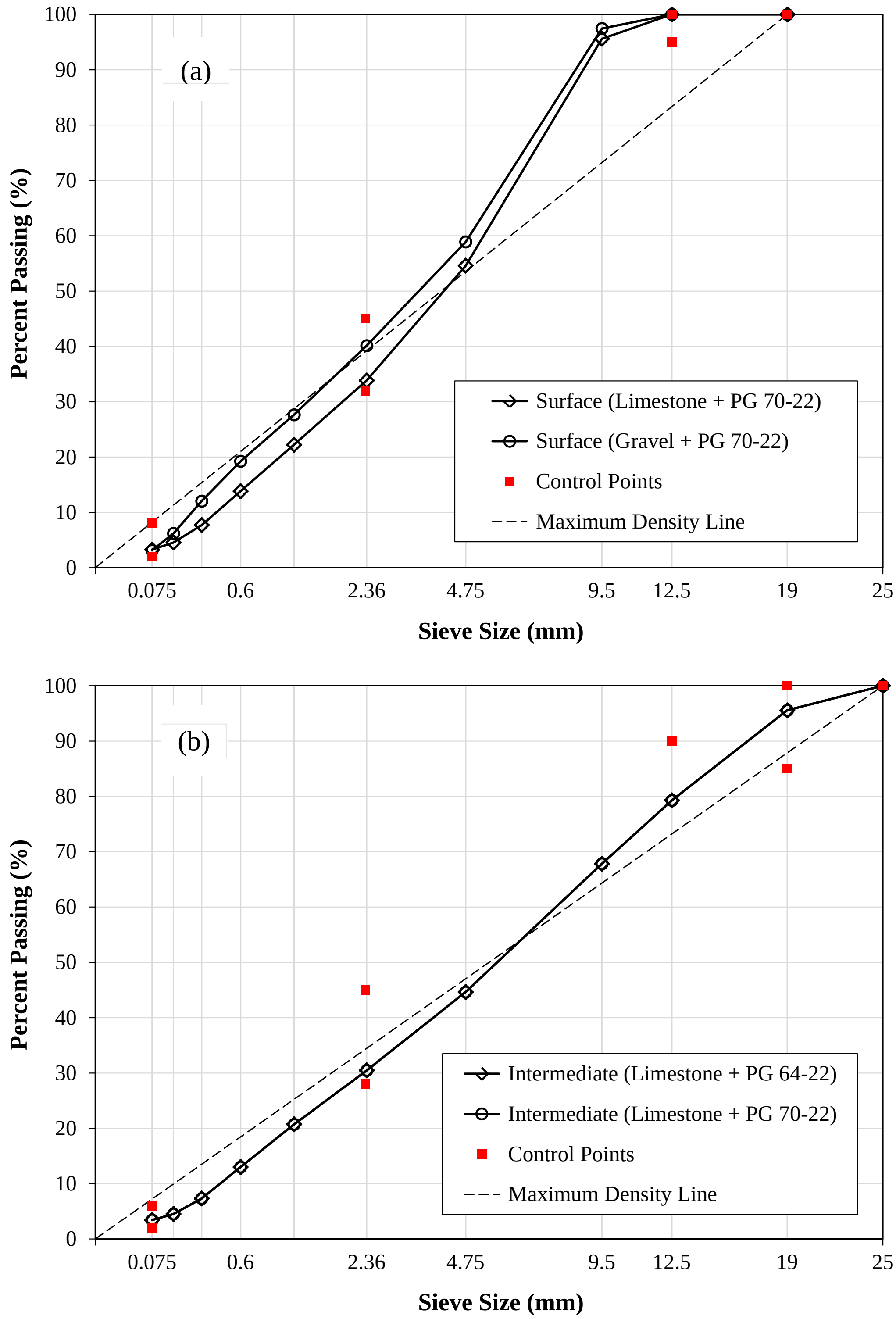

Figure 3.4: Aggregate Gradations for (a) Surface Mixtures and (b) Intermediate Mixtures.

3.5 Production on Foamed WMA

A laboratory-scale asphalt binder foaming device was used in the production of the foamed WMA mixtures (Figure 3.5). This device consists of an asphalt binder tank, a water tank, an air tank, an asphalt pump, heating components, a foaming nozzle, air and water pressure regulators, and a control panel. To operate the device, the water tank is filled with water and the air 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). 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 results in approximately 100 grams (0.22 lb) of foamed asphalt binder to be discharged from the nozzle. Therefore, the timer is adjusted depending on the desired amount of asphalt binder to be used in the mix.

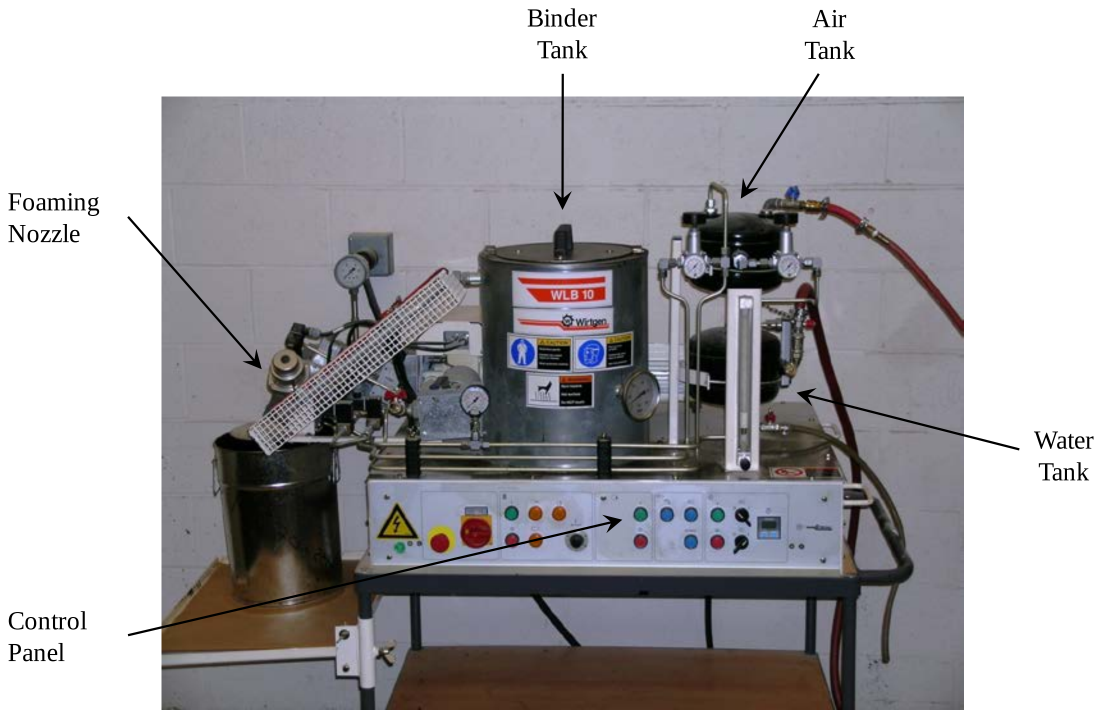

Figure 3.5: Wirtgen WLB-10 Laboratory-Scale Asphalt Foaming Device.

In the asphalt tank, the asphalt binder is heated to the mixing temperature provided by the asphalt binder supplier (306°F to 317°F (152°C to 158°C) for PG 64-28 and 306°F to 325°F (152°C to 163°C) for PG 70-22) to ensure that the asphalt binder 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 to form steam, which in turn foams and expands the asphalt binder and eventually reduces its viscosity. The maximum foaming water content currently permitted by ODOT during the production of foamed WMA is 1.8% of the total weight of the asphalt binder. In order to estimate the water flow rate required to attain the proper foaming water content, Wirtgen provides the following equation:

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

where,

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

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

P_{H_2O} = Water content (%)

3.6 = Calculation factor

Once the foaming parameters (i.e., air and water pressures, asphalt foaming temperature, and foaming water content) have been selected and the foaming device has been calibrated, the foamed asphalt binder is discharged from the foaming nozzle into a mixing bowl that contains the aggregates, which has been preheated to the WMA mixing temperature. The mixing bowl is then transferred to a mechanical mixer for mixing. A mixing period of 3 minutes, similar to that used when preparing HMA mixtures, has been shown to be sufficient when preparing foamed WMA mixtures. It is noted that the current ODOT specifications for foamed WMA allow for using a compaction temperature 30°F (16.7°C) lower than that of the corresponding HMA. However, ODOT does not control the mixing temperature of the foamed WMA. It is up to the contractor to determine the appropriate mixing temperature for this material.

Chapter 4
Laboratory Testing Program

4.1 Introduction

A comprehensive laboratory testing program 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), dynamic modulus ratio, 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.

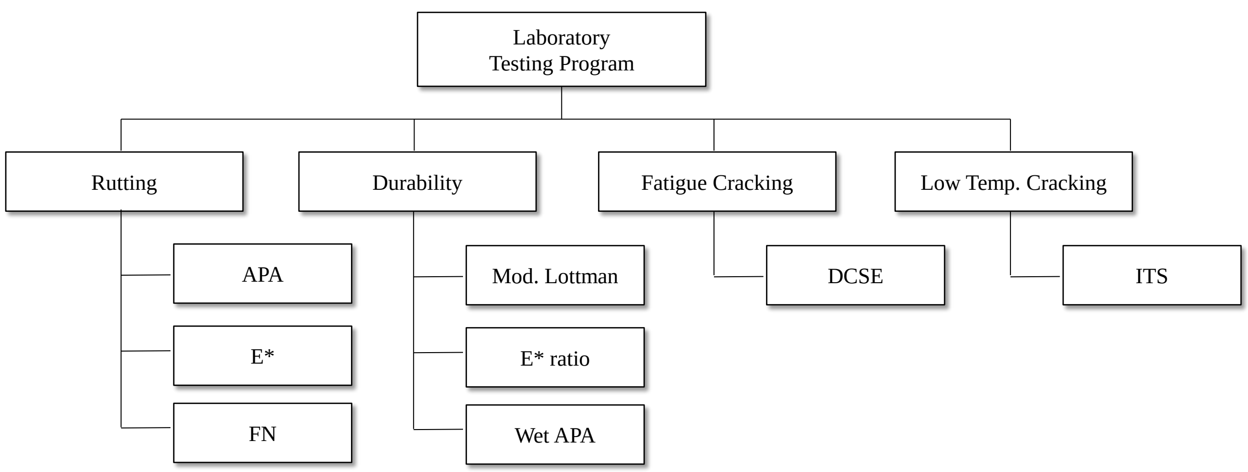

Figure 4.1: Laboratory Testing Program.

The previous tests were conducted on specimens prepared using the aggregate-binder material combinations presented in Chapter 3. 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 $30^{o}F$ ($16.7^{o}C$) temperature reduction were used in the production of the foamed WMA mixture. The following subsections offer an overview of the undertaken testing procedure as well as the specimen preparation techniques required to prepare representative samples for these tests. Where applicable, the testing procedure was modified according to the standard practices implemented in the state of Ohio.

4.2 Asphalt Pavement Analyzer

The asphalt pavement analyzer (APA) test was conducted according to AASHTO TP 63 (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) using the device shown in Figure 4.2. This test simulates actual road conditions by rolling a concave-shaped metal wheel at a speed of approximately 23.5 inch/sec (60 cm/sec) over a rubber hose pressurized at 100 psi (689.5 kPa) to 120 psi (827.4 kPa) to generate the effect of high tire pressure (Figure 4.3). The hose stays in contact with the sample's surface while the metal wheel rolls back and forth along the length of the hose for 8,000 cycles.

The APA can simultaneously test three beam samples or six cylindrical samples, with each APA sample consisting of two cylindrical samples. Superpave gyratory compacted specimens measuring 6 inch (150 mm) in diameter and 2.95 inch (75 mm) in height were used in this study. The target air void level within these specimens was 7 ± 1%, as specified in ODOT Supplement 1057. A trial and error procedure was followed in determining the weight of mixture required to achieve the target air void level. The loose mixture was short-term aged for a period of 2 hours at the compaction temperature before being prepared in the Superpave gyratory compactor.

Testing was conducted at a temperature of 120^{o}F (49^{o}C). The specimens were conditioned for a minimum of 12 hours at the test temperature prior to loading. During the test, rut depth measurements were obtained at 5, 500, 1000, and 8000 cycles. The total permanent deformation (or rutting) was calculated as the difference between the rut depth readings at the 8000th cycle and the 5th cycle. A total of four rut depth readings were used to calculate the average rut depth value for each APA sample. Averaged rut depth values for three APA samples are reported in this study.

Figure 4.2: Asphalt Pavement Analyzer (APA).

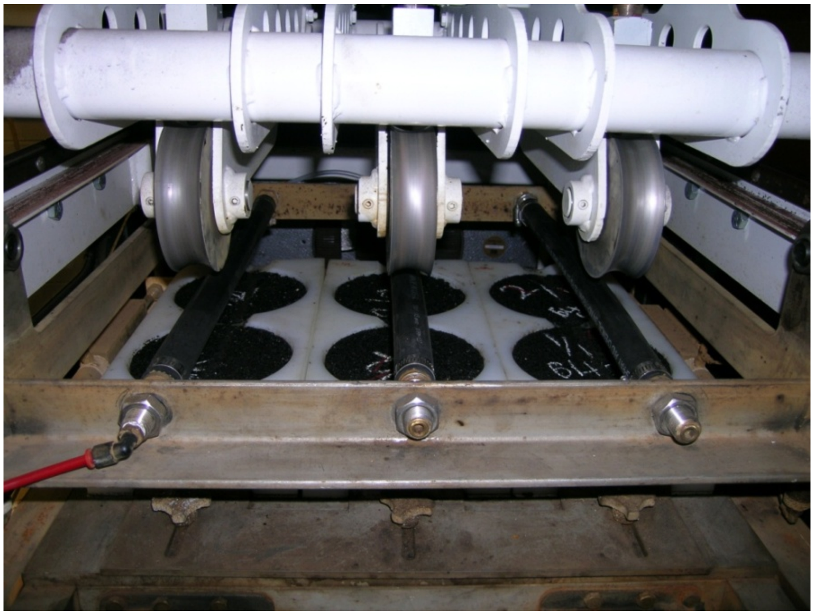

Figure 4.3: Repeated Wheel Loading in the APA Device.

The APA test was also conducted on conditioned specimens that have been subjected to saturation followed by freezing and thawing. This test is referred to as the wet APA test. The same sample conditioning procedure discussed in Section 4.5 for the modified Lottman test was utilized 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%. In addition, the wet APA test was conducted while the specimens were fully submerged in water.

4.3 Dynamic Modulus

The dynamic modulus (E*) test was conducted according to AASHTO T 342 (Standard Method of Test for Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures). The dynamic modulus is a fundamental material property commonly used to describe the mechanical behavior of viscoelastic materials such as asphalt mixtures. It relates stresses to strains induced under different loading rates and temperature conditions. In recent years, the dynamic modulus has been incorporated into the Mechanistic-Empirical Pavement Design Guide (MEPDG) to describe the response of asphaltic layers, and to subsequently predict the performance of asphalt pavements. Asphalt mixtures with higher dynamic moduli are expected to result in less permanent deformation (or rutting), as predicted using the MEPDG.

The dynamic modulus test was conducted on specimens cored from gyratory compacted mixtures. An air void content of 7 ± 0.5% was targeted in the preparation of the dynamic modulus specimens. A trial and error procedure was followed in determining the weight of mixture required to achieve the target air void level. Before compaction, the loose mixture was short-term aged for a period of 4 hours at 275^{o}F (135^{o}C), during which the mixture was stirred every hour. The temperature was then raised to the compaction temperature and the mixture was heated for 30 minutes. The compacted samples were then cored and trimmed to obtain cylindrical specimens measuring 4 inch (100 mm) in diameter and 6 inch (150 mm) in height, as shown in Figures 4.4 and 4.5.

Upon the completion of the coring and trimming of the dynamic modulus specimens, the diameter and the waviness of the top and bottom faces of the extracted specimens were measured to ensure they meet specifications. AASHTO T 342 requires measuring the diameter of the cored specimens at mid-height and third-points. The standard deviation of the three readings should not

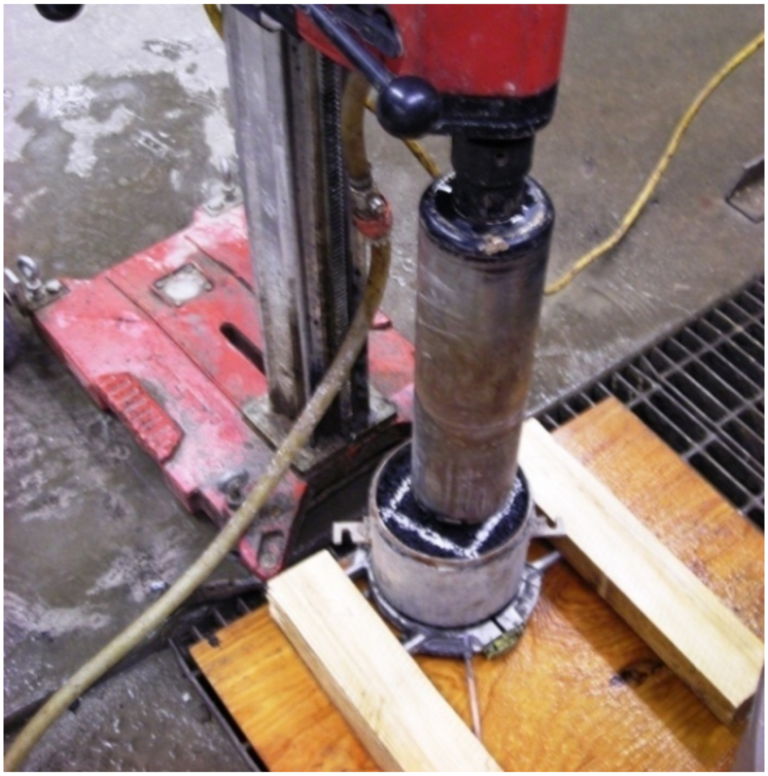

Figure 4.4: Vertical Coring Setup.

Figure 4.5: Trimming of Dynamic Modulus Specimens using a Diamond Saw.

exceed 0.1 inch (2.5 mm). Furthermore, AASHTO T 342 specifies a maximum acceptable waviness of ± 0.002 inch (0.05 mm) on the top and bottom faces 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 was used to conduct the dynamic modulus test (Figure 4.7). This system is operated using a personal computer and a digital controller called MTS TestStar II. It is capable of applying various types of loading 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 of the specimens. Load measurements are obtained using an external load cell located underneath the bottom loading platen. Two extensometers were used in this study 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) since the former provides higher accuracy and can be easily installed on the specimen.

The dynamic modulus test was conducted at six frequencies (25, 10, 5, 1, 0.5, and 0.1 Hz) and four testing temperatures (40, 70, 100, and 130^{o}F or 4.4, 21.1, 37.8, 54.4^{o}C). Testing was conducted from the lowest to the highest temperature starting with the highest frequency. A rest period of 2 minutes was used between successive frequencies. At each temperature and frequency, a repeated sinusoidal load was applied on the specimen and the resulting deformation was recorded. The applied load level was determined as the load that will result in 75 to 125 microstrain. The dynamic modulus, |E*|, was calculated as the ratio between the applied stress level and the recoverable strain level, where the applied stress level is equal to the applied load level divided by the specimen cross-sectional area and the applied strain level is equal to the average recoverable deformation level in the two extensometers divided by the extensometer length. At the end of testing, the specimen was discarded if excessive deformation (greater than 1500 micro strain) was accumulated.

The dynamic modulus test was also conducted on conditioned specimens that have been subjected to saturation followed by freezing and thawing to evaluate the effect of sample conditioning on the dynamic modulus. This test is referred to as the wet E* test. The same sample conditioning procedure discussed in Section 4.5 for the modified Lottman test was utilized 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%.

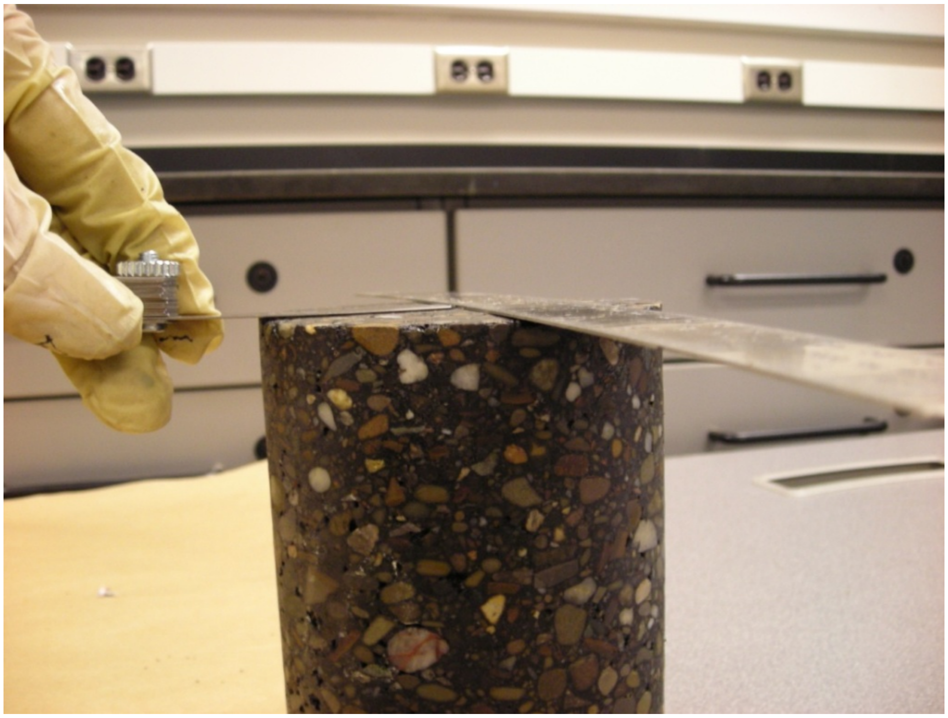

Figure 4.6: Checking the Waviness of a Dynamic Modulus Specimen.

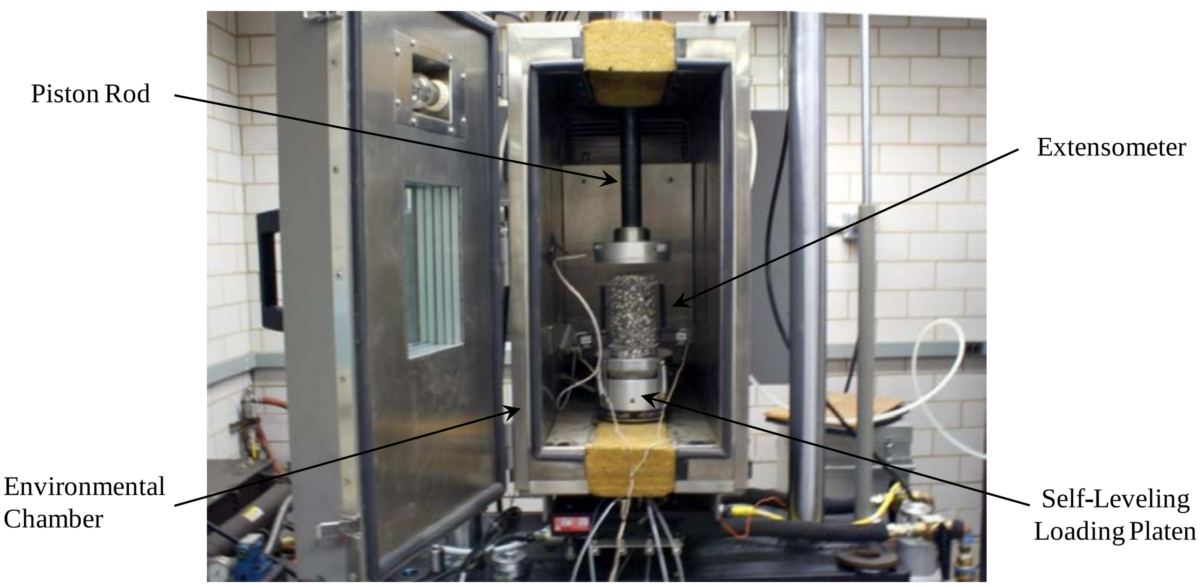

Figure 4.7: Material Test System (MTS) Model 810.

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:

$$a_T = \frac{f_{T_o}}{f_T}$$

where,

a_T = frequency temperature shift factor for temperature, T;

f_{T_o} = reduced frequency at reference temperature, T_o; and

f_T = frequency at test temperature, T.

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:

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

where,

E^* = dynamic modulus;

f_r = reduced frequency of loading at reference temperature; and

α, δ, β, γ = sigmoidal model parameters.

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

4.4 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). The specimen preparation procedure for the FN test is similar to that presented in the dynamic modulus section. In this test, the asphalt mixture specimen is subjected to repeated loading for up to 10,000 cycles, with each loading cycle consisting of a 0.1 second haversine load followed by a 0.9 second rest period. The FN is defined as the cycle number corresponding to the minimum rate of change in cumulative permanent deformation in the specimen with higher FN values generally indicating better rutting resistance.

An MTS 810 servo-hydraulic machine was used to apply the repeated loading cycles. Load measurements were obtained using an external load cell located underneath the bottom loading platen. Two LVDTs were used to measure the vertical deformation in the specimen as the load was applied. As can be seen from Figure 4.8, the two LVDTs were attached to the top loading platen and were placed 180^{o} from each other.

Figure 4.8: Attachment of LVDTs for Flow Number (FN) Testing.

Typical load cycle, cumulative permanent deformation, and rate of change in cumulative permanent deformation are presented in Figure 4.9. As can be seen from this figure, the cumulative permanent deformation can be divided into three distinct regions representing primary flow, secondary flow, and tertiary flow. The primary flow represents the region where a rapid change in cumulative permanent deformation occurs within the specimen. The secondary flow represents the region where the rate of change in cumulative permanent deformation starts to decrease, reaching a constant value. The tertiary flow represents the region where the rate of change in cumulative permanent deformation starts to increase again. The FN is defined as the starting point of the tertiary flow or the cycle number corresponding to the minimum rate of change in cumulative permanent deformation. In some cases, no clear minimum value can be found, while in other cases there may be more than one. In the first case, the total number of loading cycles (i.e., 10,000) is typically used as the FN, while in the second case the lowest cycle number is used as the FN. The FN test was conducted using a stress level of 40 psi (275.8 kPa) 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 a reasonable period of time.

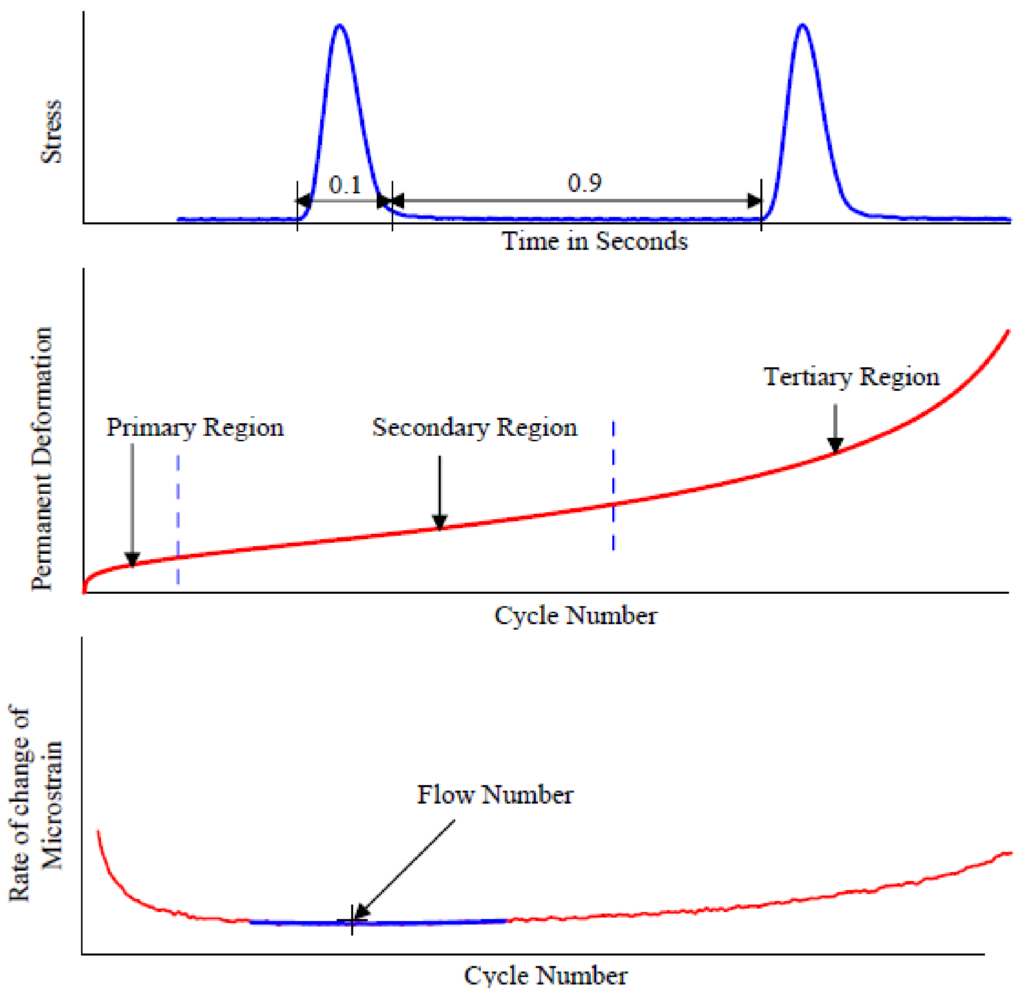

Figure 4.9: Typical Flow Number Test Results (Kabir 2008).

4.5 Modified Lottman

The modified Lottman test was conducted according to 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). This test compares the indirect tensile strength (ITS) of unconditioned specimens to conditioned specimens subjected to saturation followed by freezing and thawing. In this test, six cylindrical specimens were prepared using the Superpave Gyratory Compactor (SGC), each having a diameter of 6 inches (150 mm) and a height of 3.75 inches (95 mm). The target air voids level within these specimens was 7 ± 0.5%. A trial and error procedure was followed in determining the weight of mixture required to achieve the target air void level. Before compaction, the loose mixture was short-term aged for a period of 4 hours at 275^{o}F (135^{o}C), during which the mixture was stirred every hour. The temperature was then raised to the compaction temperature and the mixture was heated for 30 minutes. Three specimens representing the unconditioned asphalt mixture were wrapped in Saran-Wrap and stored at room temperature. The other three were soaked in water for 4 hours before being vacuum saturated until they reached a degree of saturation ranging from 80% to 90% as specified in ODOT Supplement 1051. The partially saturated specimens were wrapped in Saran-Wrap and placed in leak-proof plastic bags. A total of 10 ml (0.6 in^3) of water was added to each of the plastic bags, and the bagged specimens were placed in a freezer maintained at -0.4^{o}F (-18^{o}C) for 24 hours. Upon the completion of the freezing cycle, the specimens were transferred to a water bath and held at 140^{o}F (60^{o}C) for a 24-hour thawing period. The unconditioned and conditioned specimens were then placed in a water bath heated to 77^{o}F (25^{o}C) for two hours prior to testing.

An MTS 810 system was used to measure the ITS of the unconditioned and conditioned specimens. As shown in Figure 4.10, the specimens were loaded along their diameters using a loading rate of 2 inch per minute (50.8 mm per minute), and the maximum load required to break the specimens was used in the calculation of the indirect tensile strength:

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

where,

S_t = indirect tensile strength

P = maximum load

t = specimen thickness

D = specimen diameter

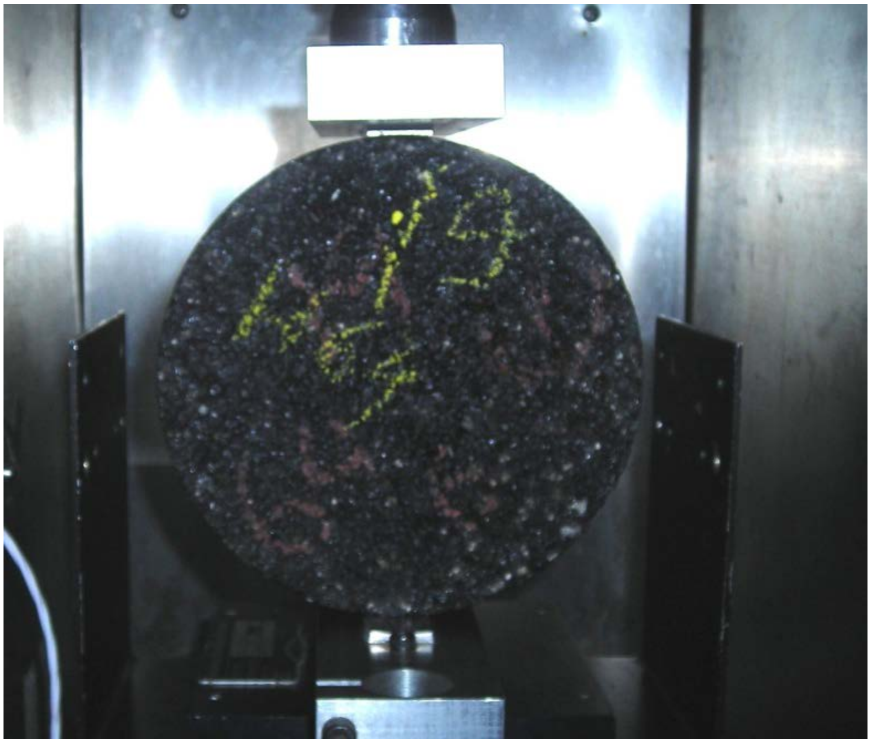

Figure 4.10: Indirect Tensile Strength Testing for AASHTO T 283.

The tensile strength ratio (TSR) was calculated as the ratio between the average indirect tensile strength of the conditioned specimens to the average indirect tensile strength of the unconditioned specimens. The TSR ratio is used as a measure of the resistance of asphalt mixtures to moisture-induced damage. The higher is the TSR ratio, the better is the resistance of the asphalt mixture to moisture-induced damage.

4.6 Dissipated Creep Strain Energy

The dissipated creep strain energy (DCSE) test was used to indirectly evaluate the resistance of foamed WMA and HMA mixtures to fatigue cracking. This test involves measuring the indirect resilient modulus (M_R) followed by the indirect tensile strength (ITS) on the same specimen. In this study, the indirect resilient modulus test was conducted according to NCHRP research digest 285 (Laboratory Determination of Resilient Modulus for Flexible Pavement Design), while the indirect tensile strength test was performed according to AASHTO T 322 (Standard Method of Test for Determining the Creep Compliance and Strength of Hot-Mix Asphalt using the Indirect Tensile Test Device).

The DCSE test was conducted at 50^{o}F (10^{o}C). Six-inch (150-mm) Superpave gyratory compacted specimens were used for this test. The specimens were compacted to a height of 2.95 inch (75 mm) and trimmed on both sides to a height of 2 inch (50.8 mm) using a diamond saw. The target air void level within the trimmed specimens was 7 ± 0.5%. A trial and error procedure was followed in determining the weight of mixture required to achieve the target air void level. Before compaction, the loose mixture was short-term aged for a period of 4 hours at 275^{o}F (135^{o}C), during which the mixture was stirred every hour. The temperature was then raised to the compaction temperature and the mixture was heated for 30 minutes.

The indirect resilient modulus was measured by applying a haversine load along the diameter of the specimen for 0.1 second followed by a rest period of 0.4 second. The horizontal and vertical deformations within the specimen were measured using four linear variable differential transducers (LVDTs), two on each face of the specimen, as shown in Figure 4.11. The magnitude of the applied load was determined as the load that results in approximately 100 microstrain in the vertical LVDT. Prior to loading, the specimens were conditioned for four hours at the testing temperature. After conditioning, 205 cycles of the haversine load were applied on the specimen. The specimen was then rotated 90^o and the M_R test was repeated. Test data recorded in the last five cycles of both tests were analyzed to obtain an average M_R value for the tested specimen. After the completion of the M_R test, the specimen was tested to determine the indirect tensile strength. In this test, the specimen was loaded along its diameter until failure using a monotonic loading rate of 2 inch per minute (50 mm per minute). Load and deformation were recorded during the test and used to calculate the stresses and strains within the specimen.

The DCSE was calculated according to the procedure suggested by Roque et al. (2004). This method defines the DSCE as the difference between the fracture energy and the elastic energy, as demonstrated in Figure 4.12. As shown in this figure, the fracture energy is calculated as the area between the curve OB and the lines BA and CO (i.e., area under the stress-strain curve until failure as obtained from the indirect tensile strength test), and the elastic energy is calculated as the area of the triangle ABC (i.e., energy resulting from elastic deformation). The DCSE value represents the energy threshold that an asphalt mixture can tolerate before it fractures. Higher DCSE values indicate better resistance to fatigue cracking.

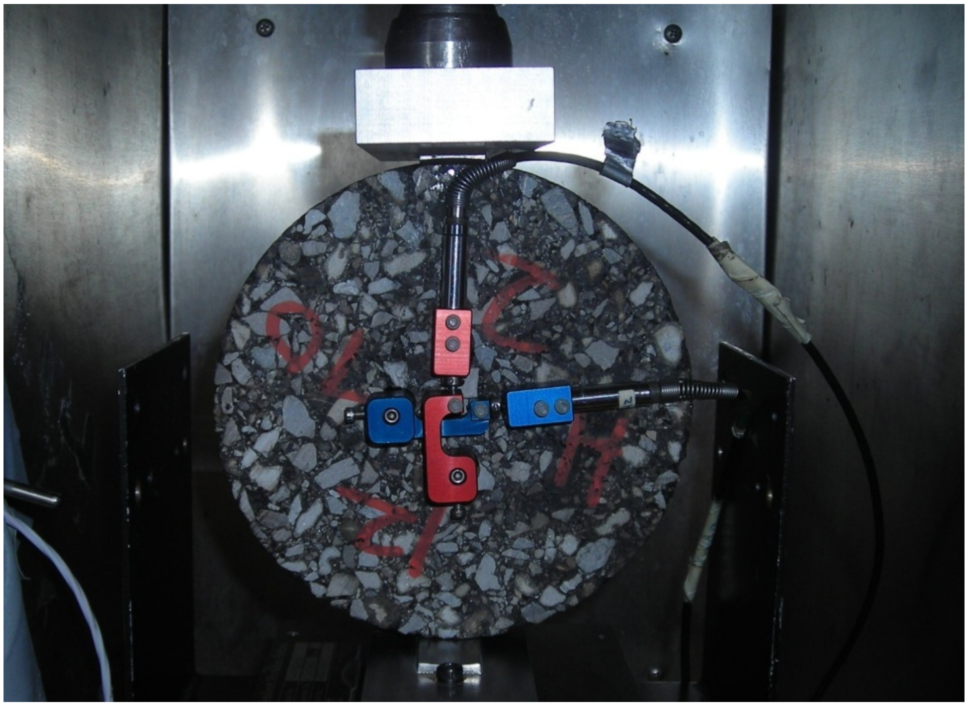

Figure 4.11: Placement of Horizontal and Vertical LVDTs on DCSE Specimens.

Based on the previous discussion, the following equations were used to calculate the DCSE:

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

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

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

$$DCSE = FE - EE$$

where,

M_R = indirect resilient modulus

S_t = indirect tensile strength

ε_f = failure strain

EE = elastic energy

FE = fracture energy

$DCSE$ = dissipated creep strain energy

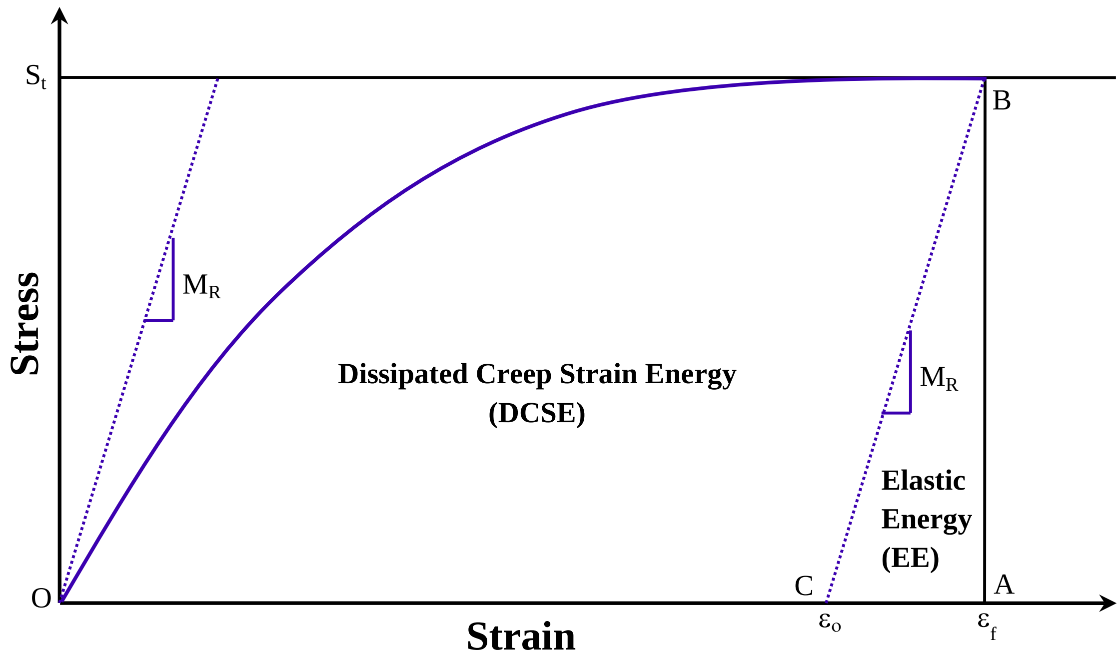

Figure 4.12: Dissipated Creep Strain Energy (DCSE) Diagram.

4.7 Indirect Tensile Strength at Low Temperature

The indirect tensile strength (ITS) at low temperature test was conducted at 14^{o}F (-10^{o}C) according to AASHTO T 322 (Standard Method of Test for Determining the Creep Compliance and Strength of Hot-Mix Asphalt using the Indirect Tensile Test Device). This test was performed on cylindrical specimens measuring 6 inch (150 mm) in diameter and 2 inch (51 mm) in height. The specimen preparation procedure for this test is similar to that presented for the DCSE test. The target air void level within these specimens was 7 ± 0.5%.

Prior to testing, all specimens were conditioned for a minimum of 12 hours at 14^{o}F (-10^{o}C). During the test, each specimen was loaded along its diameter using a deformation rate of 0.5 inch per minute (12.5 mm per minute). Two LVDTs attached to the specimen as shown in Figure 4.11 were used to measure the vertical and horizontal deformations. Testing proceeded until failure and the peak load was used to calculate the indirect tensile strength using the equation presented in the modified Lottman section.

Chapter 5
Laboratory Test Results and Discussion

5.1 Introduction

As discussed in the previous chapter, a comprehensive laboratory testing program 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. This chapter presents the performance test results obtained for these mixtures. In addition, it presents the outcome of the multi-factor analysis of variance (ANOVA) that was conducted using the Statistical Package for Social Sciences (SPSS) software to evaluate the effect of the mix type, binder type, aggregate type, and aggregate size on the mix performance.

5.2 APA Test Results

Figure 5.1 presents the dry and wet APA test results for the foamed WMA and HMA mixtures. As can be noticed from this figure, the foamed WMA intermediate mixture prepared using limestone and PG 70-22 and the foamed WMA surface mixture prepared using crushed gravel and PG 70-22 exhibited slightly higher rut depths than the corresponding HMA mixtures, while the foamed WMA surface mixture prepared using limestone and PG 70-22 and the foamed WMA intermediate mixture prepared using limestone and PG 64-28 exhibited slightly lower rut depths than the corresponding HMA mixtures. It can also be noticed from this figure that significantly higher rut depths were obtained for asphalt mixtures containing PG 64-28 than mixtures containing PG 70-22, which can be explained by the higher stiffness of the PG 70-22 binder than the PG 64-28 binder. In addition, the limestone mixtures had slightly better rutting resistance than the corresponding crushed gravel mixtures, which can be explained by the better aggregate interlock within the limestone mixtures.

Within each mixture group, the wet specimens exhibited higher permanent deformation than the corresponding dry specimens. However, the difference between the dry and wet rut depth values for the foamed WMA mixtures was slightly lower than the HMA mixtures.

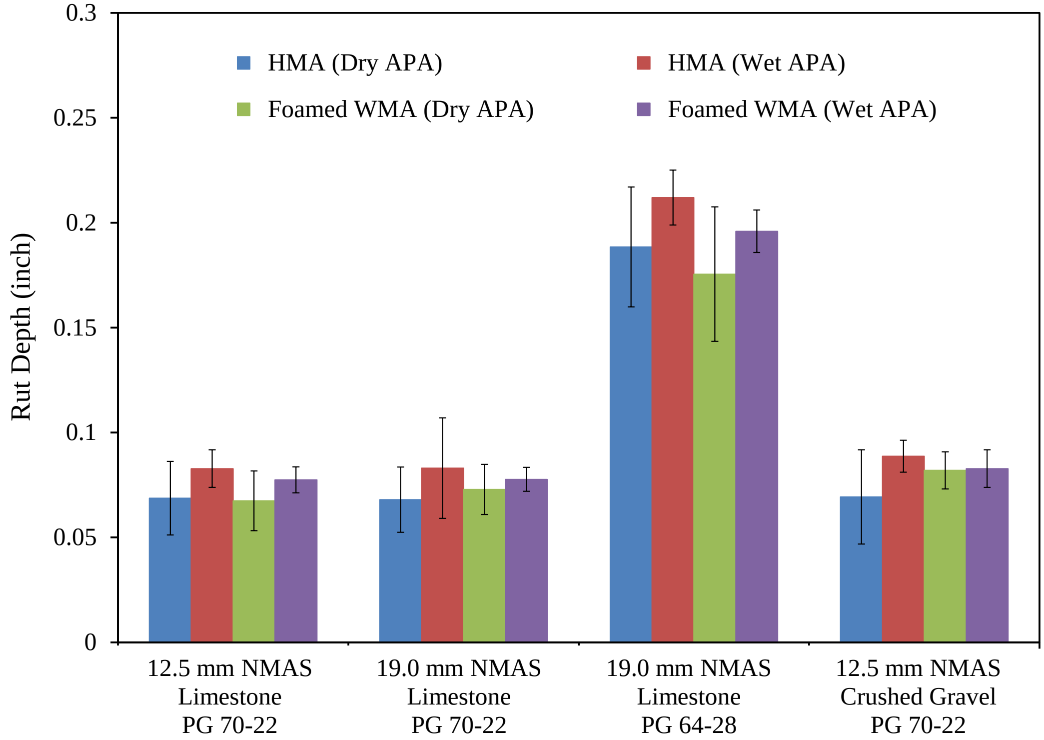

Figure 5.1: Dry and Wet APA Test Results.

Table 5.1 presents the multi-factor ANOVA results for the APA rut depth data. As can be noticed from this table, selected mixture groups were included in the statistical analyses to determine the effect of the binder type, aggregate type, and aggregate size due to the use of a partial factorial in the experimental testing plan (i.e., four mixture groups were used instead of a full factorial of eight). This table shows that the binder type significantly influenced the APA rut depths (p-value < 0.05) at 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. In addition, none of the two-way interactions had a significant effect on the APA rut depths (p-value > 0.05).

Table 5.1: Multi-Factor ANOVA Results for APA Rut Depths.

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

5.3 E* Test Results

Figure 5.2 presents the dynamic modulus master curves for the intermediate mixtures containing PG 70-22 and PG 64-28 asphalt binders. As can be noticed from this figure, the foamed WMA mixtures exhibited significantly lower dynamic moduli than the corresponding HMA mixtures for both asphalt binders. In addition, the asphalt mixtures prepared using PG 64-28 had lower dynamic moduli than those prepared using PG 70-22.

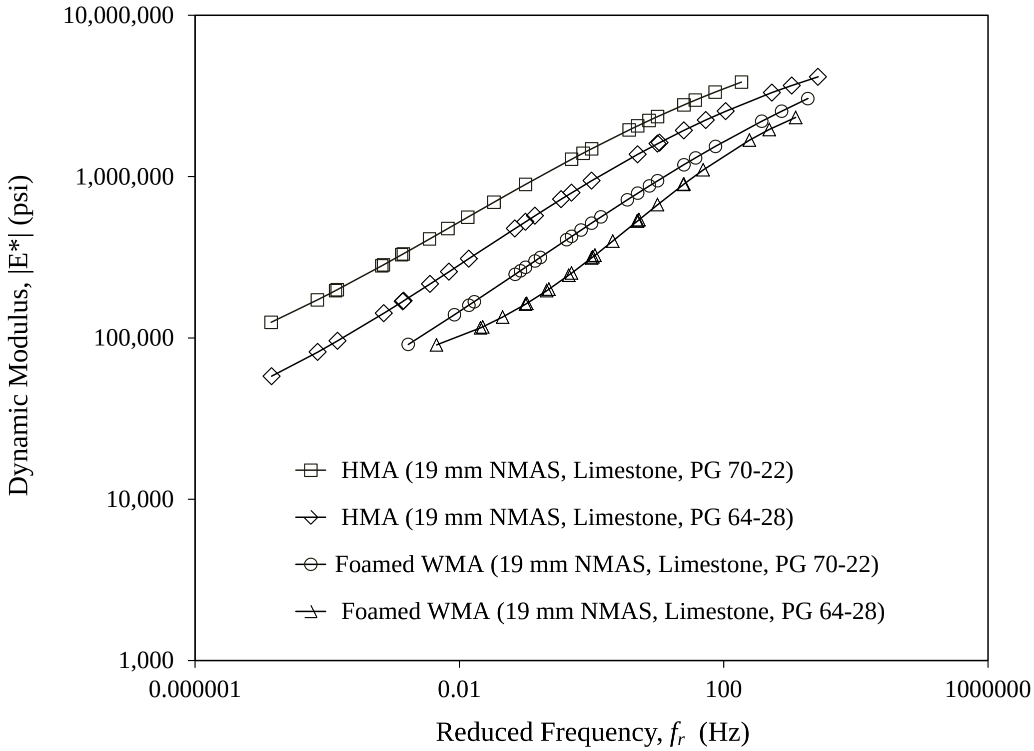

Figure 5.2: Dynamic Modulus Master Curves Demonstrating the Effect of the Binder Type.

Figure 5.3 presents the dynamic modulus master curves for the surface mixtures prepared using limestone and crushed gravel. As can be noticed from this figure, the foamed WMA mixture containing limestone aggregate exhibited lower dynamic moduli than the corresponding HMA mixture, which indicates better rutting performance for the HMA mixture. This trend was reversed for the crushed gravel where the foamed WMA mixture exhibited higher dynamic moduli than the corresponding HMA mixture.

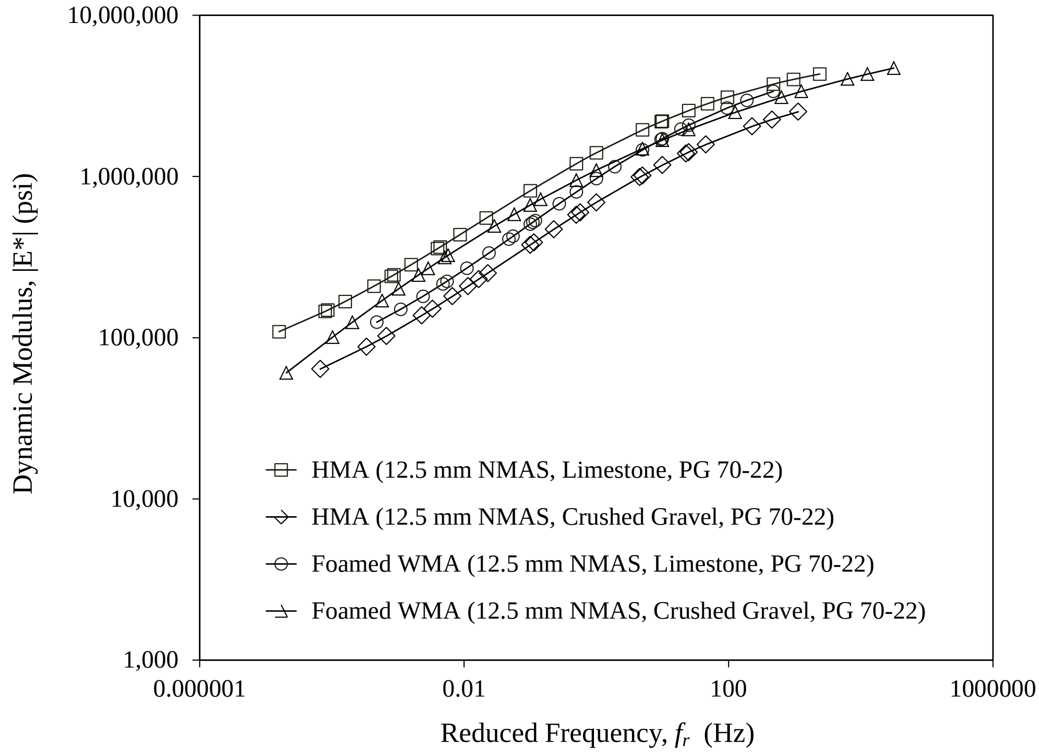

Figure 5.3: Dynamic Modulus Master Curves Showing the Effect of the Aggregate Type.

Figure 5.4 presents the dynamic modulus master curves for the surface and intermediate mixtures containing limestone and PG 70-22. As can be noticed from this figure, the foamed WMA surface mixture prepared using limestone and PG 70-22 exhibited lower dynamic moduli than the corresponding HMA mixture and the difference between them increased with the decrease in reduced frequency. The same trend was also observed for the intermediate mixture prepared using limestone and PG 70-22, but the difference between the foamed WMA and HMA mixtures was even higher. It can be observed from this figure that the aggregate size had no effect on the dynamic modulus of the HMA mixtures, but had a profound effect on the dynamic modulus of the foamed WMA mixtures. For the foamed WMA mixtures, the dynamic modulus of the surface mixtures was higher than the dynamic modulus of the intermediate mixtures. This difference in dynamic modulus between the surface and intermediate foamed WMA mixtures could be the result of using no confinement in the dynamic modulus test, which would affect the intermediate mixtures more than the surface mixtures.

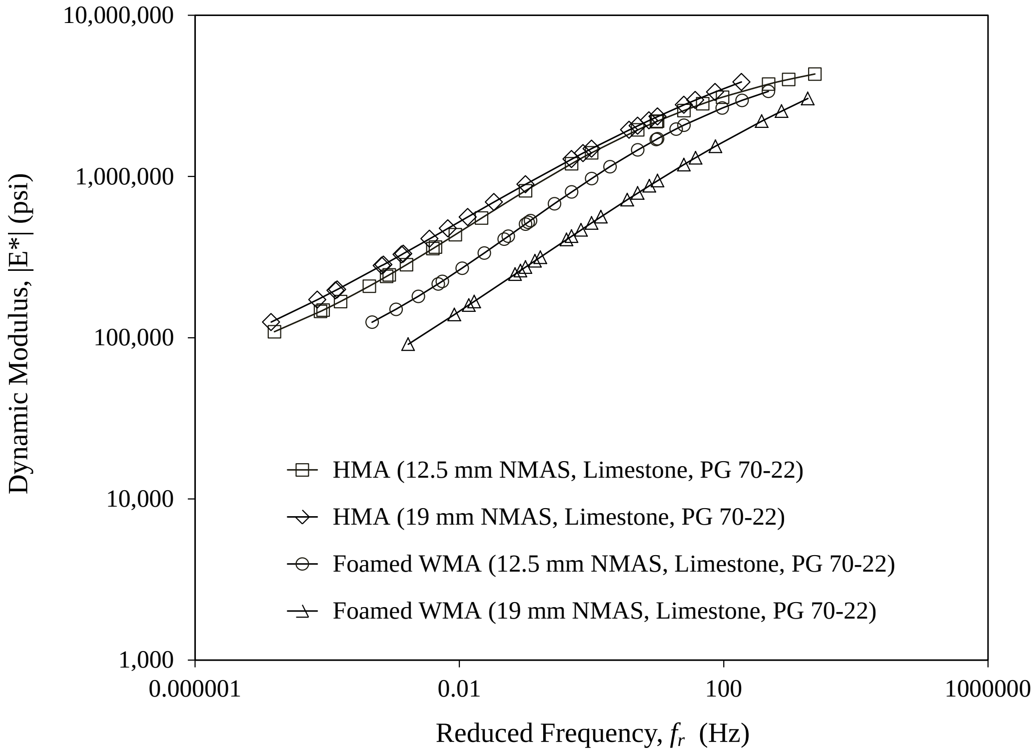

Figure 5.4: Dynamic Modulus Master Curves Showing the Effect of the Aggregate Size.

5.3 FN Test Results

Figure 5.5 presents the flow number (FN) test results for the foamed WMA and HMA mixtures. As can be noticed from this figure, the foamed WMA surface and intermediate mixtures prepared using limestone and PG 70-22 had lower FN values than the corresponding HMA mixtures, which indicates a better rutting resistance for the HMA mixtures. However, the difference in FN values between the foamed WMA and HMA mixtures seemed to be negligible for the intermediate mixture prepared using limestone and PG 64-22 and the surface mixture prepared using crushed gravel and PG 70-22. Significantly lower FN values were also observed for foamed WMA and HMA mixtures prepared using PG 64-28 than those prepared using PG 70-22. This indicates that the use of PG 70-22 improves the resistance to permanent deformation. Furthermore, lower FN values were observed for foamed WMA and HMA mixtures prepared using crushed gravel than those prepared using limestone, which indicates a better rutting resistance for the limestone mixtures.

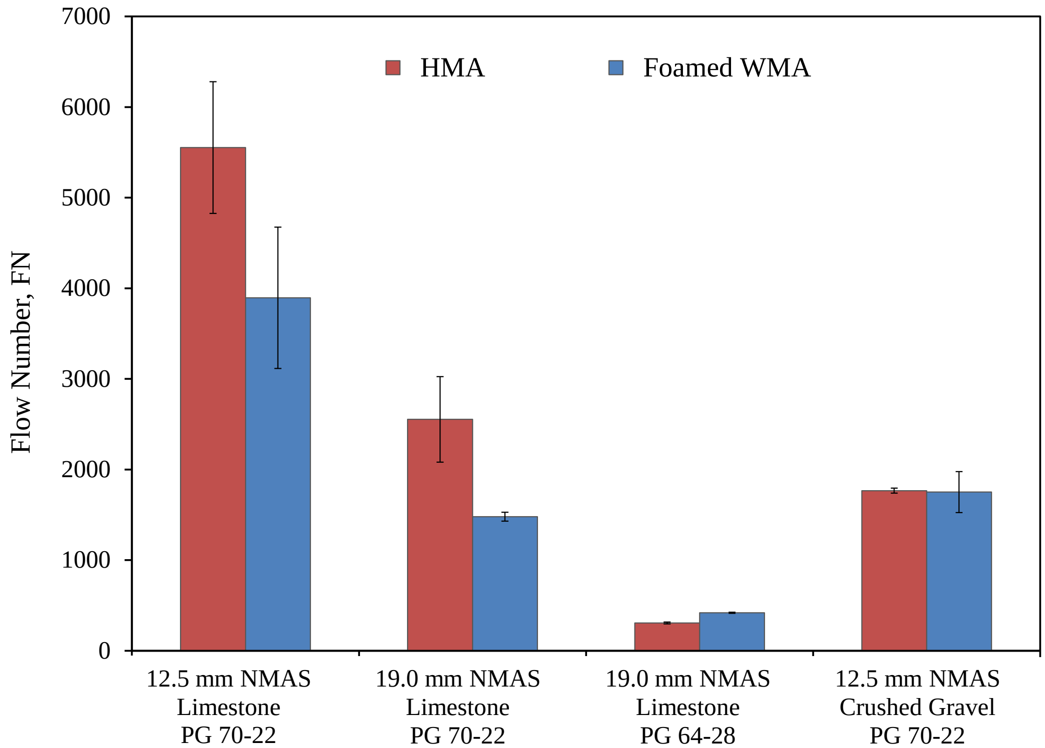

Figure 5.5: Flow Number Test Results.

Table 5.2 presents the multi-factor ANOVA results for the flow number values. As can be noticed from this table, the mix type, binder type, aggregate type, and aggregate size as well as the interaction between the mix type and binder type had a significant influence (p-value < 0.05) on the flow number. However, as indicated from the F-value, the binder type had the most significant effect on the flow number.

Table 5.2: Multi-Factor ANOVA Results for FN Values.

Analysis Data	Statistical Factors	F-value	Prob.
19.0 mm, Limestone, PG 64-28 & 19.0 mm, Limestone, PG 70-22	Mix Type	8.220	0.046
	Binder Type	97.327	0.001
	Mix Type × Binder Type	12.538	0.024
12.5 mm, Gravel, PG 70-22 & 12.5 mm, Limestone, PG 70-22	Mix Type	6.170	0.056
	Agg. Type	77.336	0.000
	Mix Type × Agg. Type	5.939	0.059
12.5 mm, Limestone, PG 70-22 & 19.0 mm, Limestone, PG 70-22	Mix Type	10.979	0.030
	Agg. Size	43.086	0.003
	Mix Type × Agg. Size	0.503	0.517

5.4 AASHTO T 283 Test Results

Figures 5.6 and 5.7 present the average dry and wet ITS values, respectively, obtained using AASHTO T 283 for the foamed WMA and HMA mixtures. The error bars in these figures represent one standard deviation of the individual test results from the mean. It can be observed from these figures that the average dry and wet ITS values for the foamed WMA mixtures prepared using PG 70-22 were lower than the corresponding HMA mixtures, while the average dry and wet ITS values for the foamed WMA mixtures prepared using PG 64-28 were slightly higher than the corresponding HMA mixtures.

Figure 5.8 presents the corresponding TSR values obtained for the foamed WMA and HMA mixtures. As can be noticed from this figure, the TSR values for the foamed WMA mixtures were slightly lower than the corresponding HMA mixtures except for the foamed WMA intermediate mixture prepared using limestone and PG 64-28. However, it is noted that the foamed WMA mixtures passed the minimum TSR requirement of 0.8, represented by a horizontal red line, for all HMA mixtures that met this requirement.

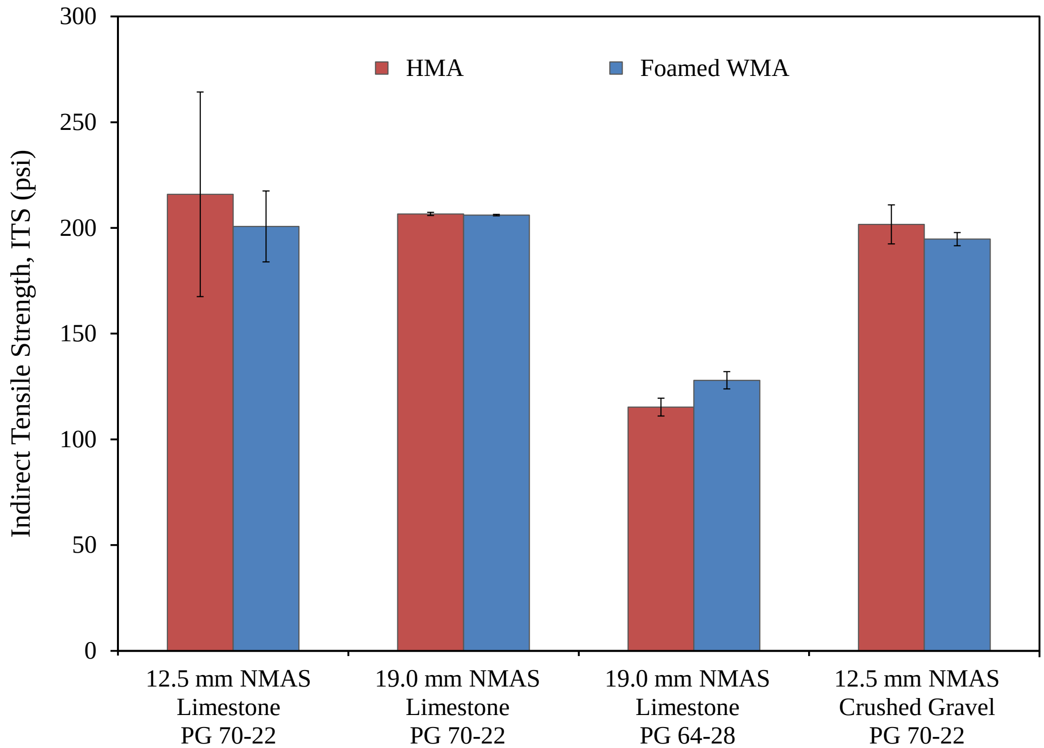

Figure 5.6: Dry ITS Values for Foamed WMA and HMA Mixtures.

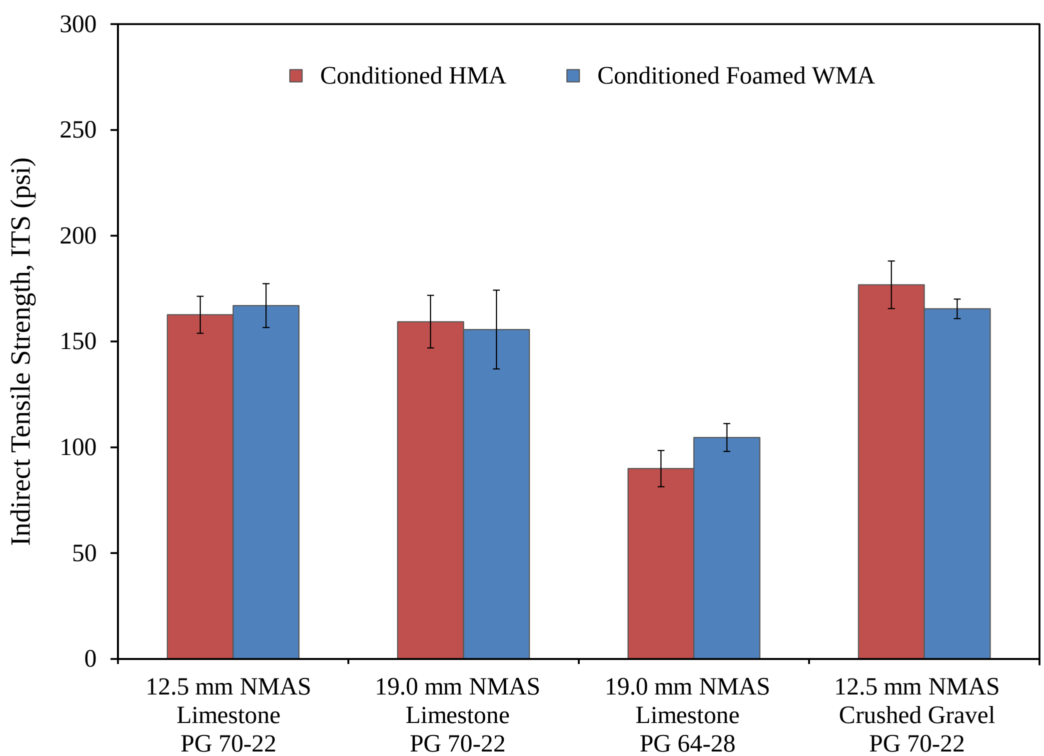

Figure 5.7: Wet ITS Values for Foamed WMA and HMA Mixtures.

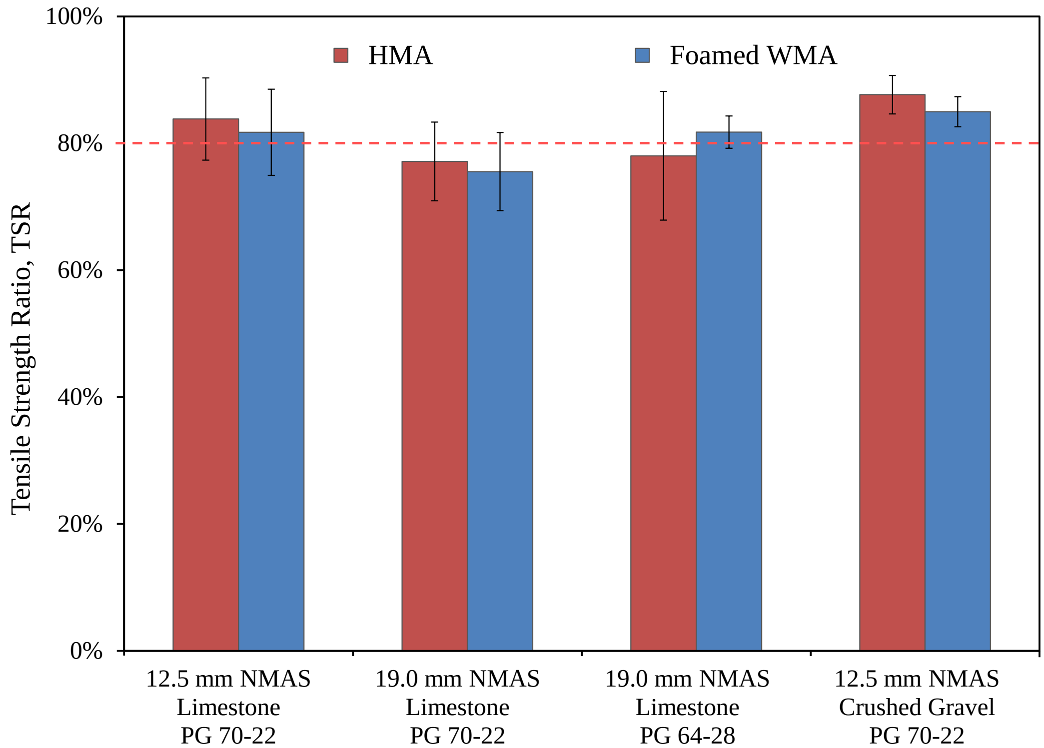

Figure 5.8: Comparison of TSR Ratios for Foamed WMA and HMA Mixtures.

Table 5.3 presents the multi-factor ANOVA results for the dry and wet ITS values. As can be noticed from this table, the mix type did not have a significant effect on the dry and wet ITS values (probability < 0.05) at a confidence level of 95%. However, sample conditioning and binder type had a significant influence on the ITS values, with the binder type being the most significant as indicated from the F-value.

Table 5.3: Multi-Factor ANOVA Results for Dry and Wet ITS Values.

Analysis Data	Statistical Factors	F-value	Prob.
19.0 mm, Limestone, PG 64-28 & 19.0 mm, Limestone, PG 70-22	Mix Type	3.150	0.094
	Test Cond.	72.618	0.000
	Binder Type	224.969	0.000
	Mix Type × Test Cond.	0.104	0.751
	Mix Type × Binder Type	3.337	0.085
	Test Cond. × Binder Type	16.770	0.001
12.5 mm, Gravel, PG 70-22 & 12.5 mm, Limestone, PG 70-22	Mix Type	0.338	0.569
	Test Cond.	15.759	0.001
	Agg. Type	0.121	0.732
	Mix Type × Test Cond.	0.163	0.692
	Mix Type × Agg. Type	3.094	0.097
	Test Cond. × Agg. Type	0.229	0.639
12.5 mm, Limestone, PG 70-22 & 19.0 mm, Limestone, PG 70-22	Mix Type	1.175	0.294
	Test Cond.	32.344	0.000
	Agg. Size	0.078	0.783
	Mix Type × Test Cond.	0.335	0.570
	Mix Type × Agg. Size	1.240	0.281
	Test Cond. × Agg. Size	2.284	0.149

5.5 E* Ratio Test Results

Figure 5.9 presents the effect of sample conditioning on the dynamic modulus master curves for the foamed WMA and HMA surface mixtures prepared using limestone and PG 70-22. As can be noticed from this figure, sample conditioning resulted in a significant drop in dynamic modulus for the HMA mixture as compared to the foamed WMA mixture.

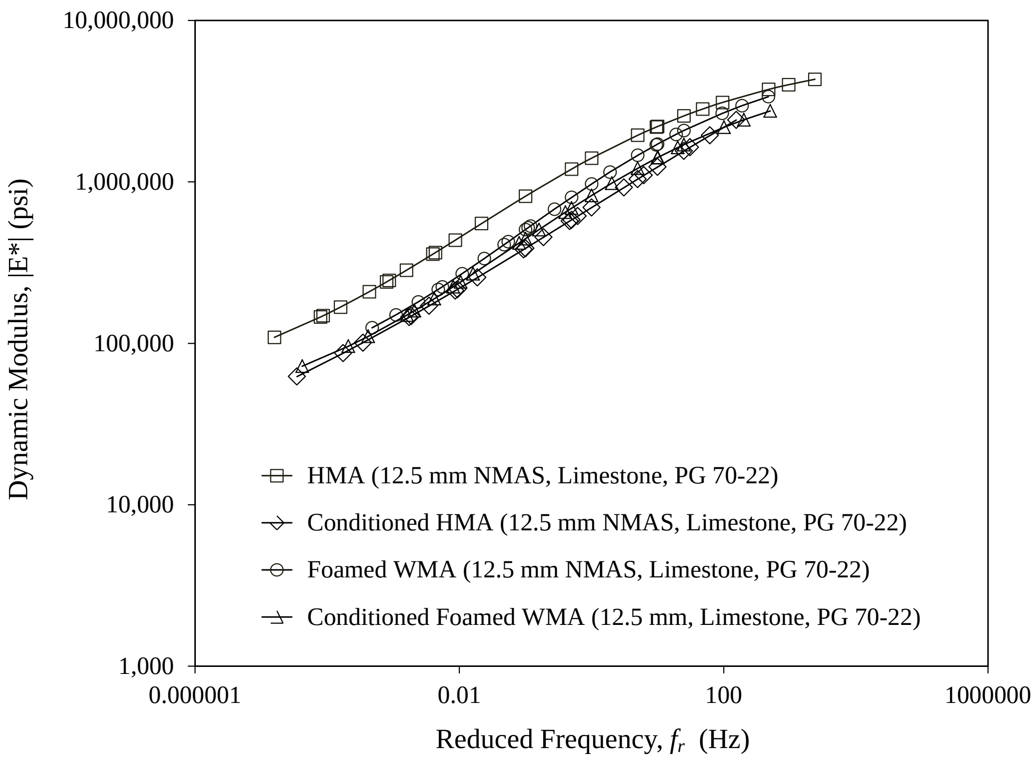

Figure 5.9: Unconditioned and Conditioned Dynamic Modulus Master Curves for Foamed WMA and HMA Surface Mixtures Prepared using Limestone and PG 70-22

Figure 5.10 presents the effect of sample conditioning on the dynamic modulus master curves for the foamed WMA and HMA intermediate mixtures prepared using limestone and PG 64-28. As can be noticed from this figure, the dynamic modulus of the conditioned specimens was found to be lower than the unconditioned specimens for the HMA mixtures. However, this trend was reversed for the foamed WMA mixtures.

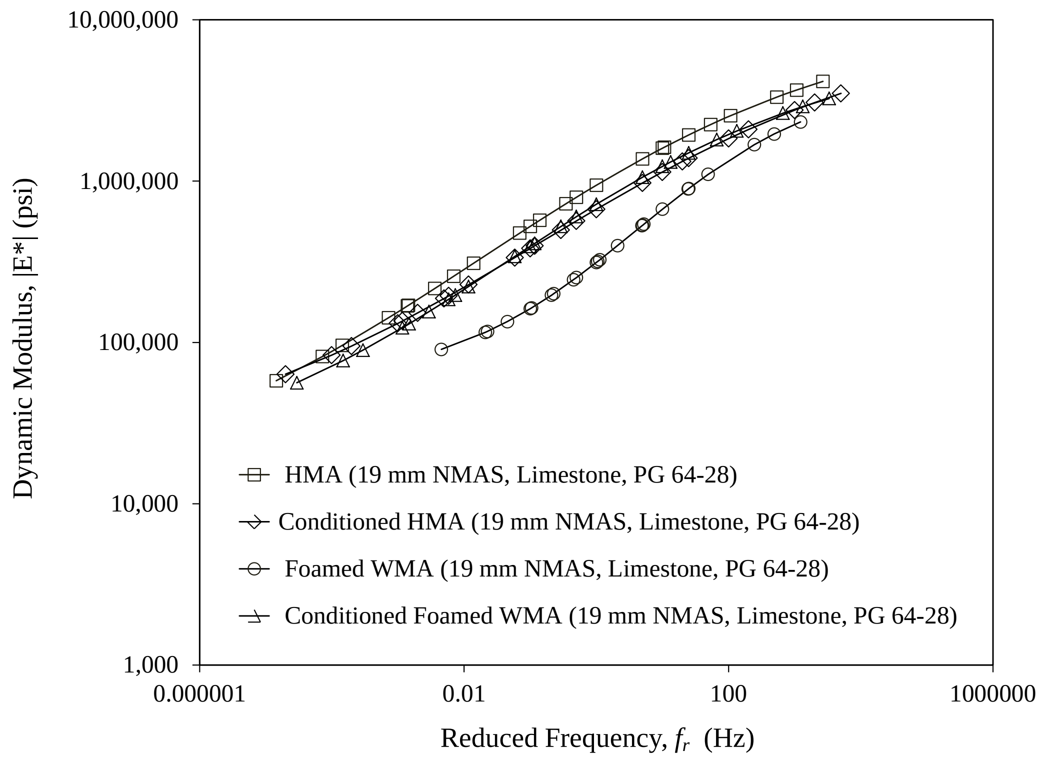

Figure 5.10: Unconditioned and Conditioned Dynamic Modulus Master Curves for Foamed WMA and HMA Intermediate Mixtures Prepared using Limestone and PG 64-28

Figure 5.11 presents the effect of sample conditioning on the dynamic modulus master curves for the foamed WMA and HMA intermediate mixtures prepared using limestone and PG 70-22. As can be noticed from this figure, sample conditioning had relatively no effect on the dynamic modulus of the foamed WMA mixture. However, it resulted in significantly lower dynamic moduli for the HMA mixture.

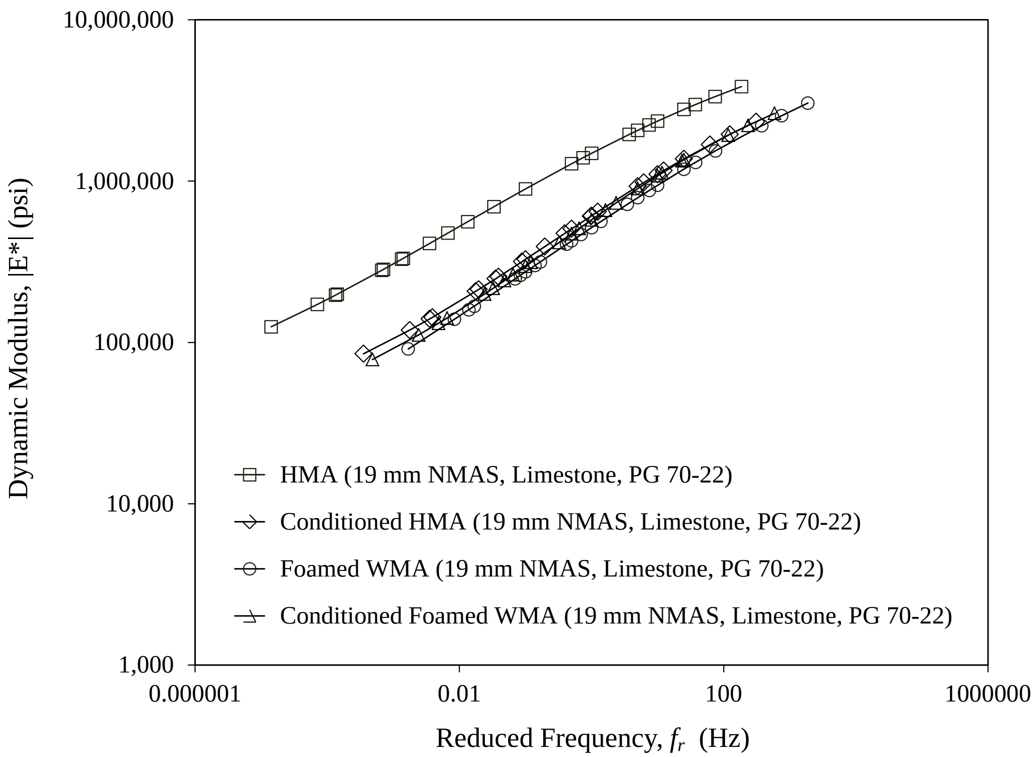

Figure 5.11: Unconditioned and Conditioned Dynamic Modulus Master Curves for Foamed WMA and HMA Intermediate Mixtures Prepared using Limestone and PG 70-22

Figure 5.12 presents the effect of sample conditioning on the dynamic modulus master curves for the foamed WMA and HMA surface mixtures prepared using crushed gravel and PG 70-22. As can be noticed from this figure, and similar to the previous observation, sample conditioning had relatively no effect on the dynamic modulus of the foamed WMA mixture. However, it resulted in slightly higher dynamic moduli for the HMA mixture.

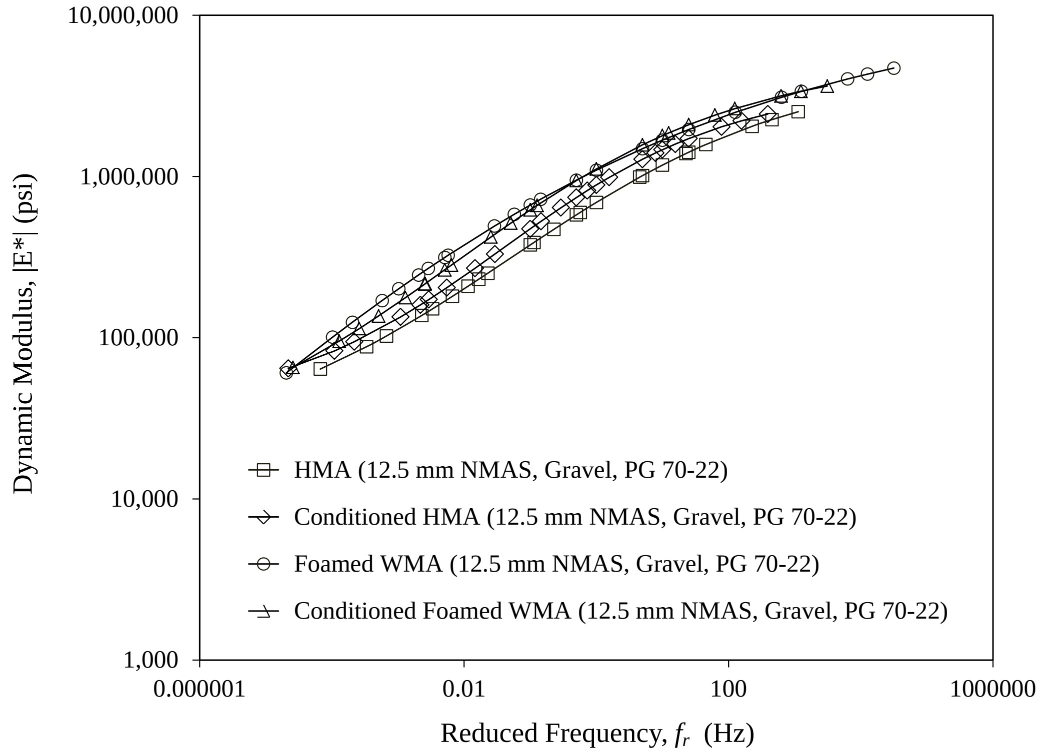

Figure 5.12: Unconditioned and Conditioned Dynamic Modulus Master Curves for Foamed WMA and HMA Surface Mixtures Prepared using Crushed Gravel and PG 70-22

5.6 DCSE Test Results

Figure 5.13 presents the average DCSE values for the foamed WMA and HMA mixtures. The error bars in this figure represent one standard deviation of the individual test results from the mean. As can be noticed from this figure, the foamed WMA surface and intermediate mixtures prepared using limestone aggregate and PG 70-22 binder exhibited similar DCSE values to the corresponding HMA mixtures. However, the DCSE values obtained for the foamed WMA intermediate mixture prepared using limestone and PG 64-28 as well as the foamed WMA surface mixture prepared using crushed gravel and PG 70-22 were different than those obtained for the corresponding HMA mixtures. The DSCE values obtained for the foamed WMA intermediate mixture prepared using limestone and PG 64-28 were higher than those obtained for the HMA mixture, while the DCSE values obtained for the foamed WMA surface mixture prepared using crushed gravel and PG 70-22 were lower than those obtained for the HMA mixture. However, all mixtures met the minimum DCSE threshold value of 0.75 KJ/m^3, which has been suggested by Roque et al. (2004) to ensure satisfactory resistance to fatigue cracking.

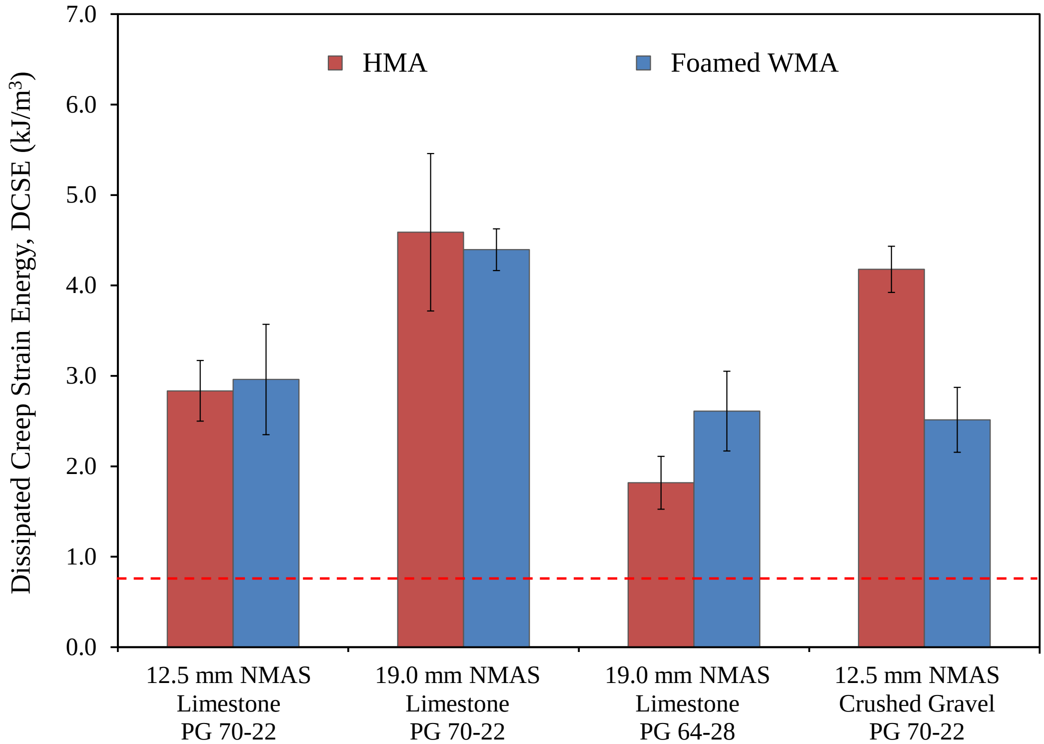

Figure 5.13: Comparison of DCSE Values for Foamed WMA and HMA Mixtures.

Table 5.4 presents the multi-factor ANOVA results for the DCSE values. As can be noticed from this table, the mix type did not have a significant effect on the DCSE values (probability < 0.05) at a confidence level of 95%. However, the DCSE values were significantly influenced by the binder type, aggregate size, and the two-way interaction between the mix type and the aggregate type.

Table 5.4: Multi-Factor ANOVA Results for DCSE Values.

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

5.7 Low-Temperature ITS Test Results

Figures 5.14 and 5.15 present the average low-temperature ITS and the corresponding failure strain values obtained at 14^{o}F (-10^{o}C) for the foamed WMA and HMA mixtures. As can be noticed from Figure 5.14, the foamed WMA mixtures exhibited slightly lower ITS values than the corresponding HMA mixtures for all material combinations. For both foamed WMA and HMA mixtures, the ITS values of mixtures containing PG 70-22 binder were higher than those containing PG 64-28. In addition, the ITS values obtained for the surface mixtures prepared using crushed gravel were lower than those obtained for the surface mixture prepared using limestone. It can also be observed from Figure 5.15 that that the failure strains of the foamed WMA mixtures were similar to the corresponding HMA mixtures, except for those prepared using PG 64-28, which had significantly higher failure strains for the foamed WMA. It should be noted that higher ITS and higher failure strain values indicate better resistance to low temperature cracking. The HMA mixtures had higher ITS values and similar failure strain values to the foamed WMA mixtures. Therefore, the HMA mixtures are expected to have better resistance to thermal cracking.

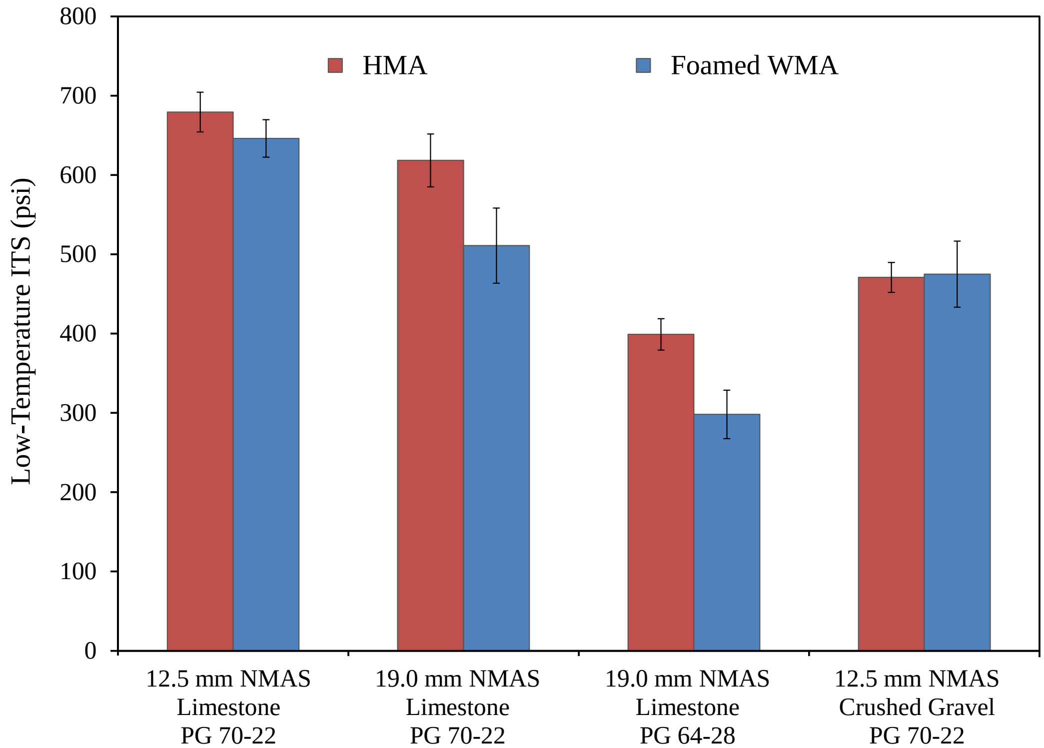

Figure 5.14: Low-Temperature ITS Values for Foamed WMA and HMA Mixtures.

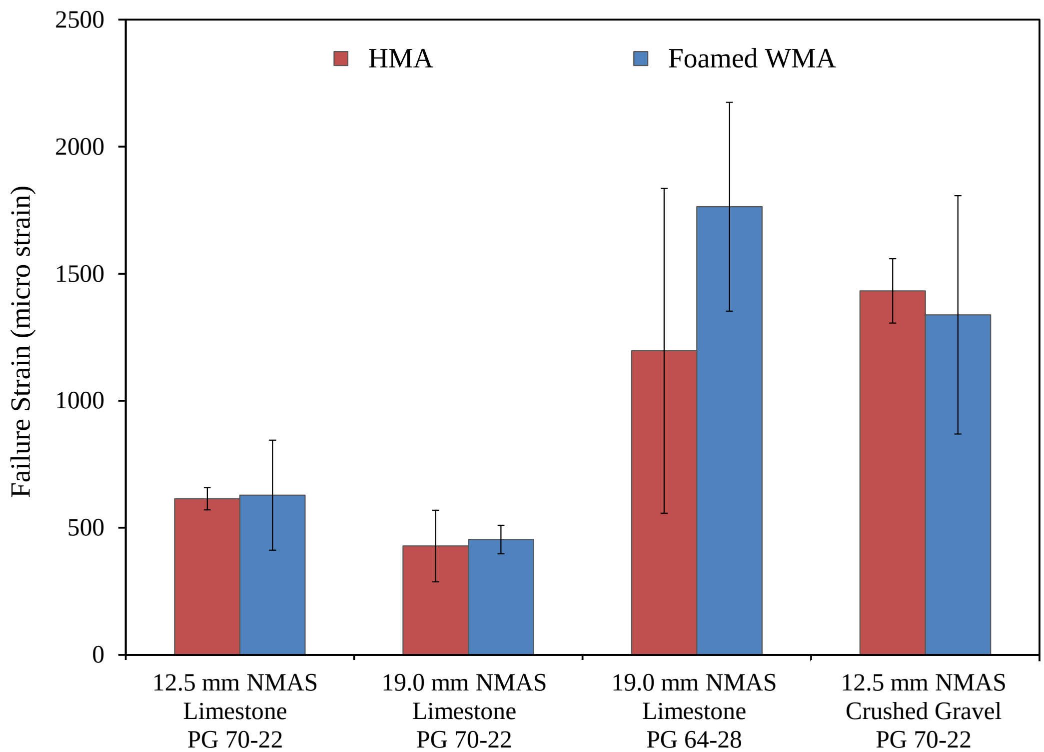

Figure 5.15: Low-Temperature Failure Strain Values for Foamed WMA and HMA Mixtures.

Tables 5.5 and 5.6 present the multi-factor ANOVA results for the low-temperature ITS and failure strain values, respectively. The statistical analysis results support the previous observations in that the mix type had a significant effect on the low-temperature ITS values, but not on the failure strain values.

Table 5.5: ANOVA Results for Low Temperature ITS Values.

Analysis Data	Statistical Factors	F-value	Prob.
19.0 mm, Limestone, PG 64-28 & 19.0 mm, Limestone, PG 70-22	Mix Type	27.887	0.001
	Binder Type	119.957	0.000
	Mix Type × Binder Type	0.028	0.872
12.5 mm, Gravel, PG 70-22 & 12.5 mm, Limestone, PG 70-22	Mix Type	0.776	0.404
	Agg. Type	131.681	0.000
	Mix Type × Agg. Type	1.277	0.291
12.5 mm, Limestone, PG 70-22 & 19.0 mm, Limestone, PG 70-22	Mix Type	13.093	0.007
	Agg. Size	25.440	0.001
	Mix Type × Agg. Size	3.642	0.093

Table 5.6: ANOVA Results for Low Temperature Failure Strain Values.

Analysis Data	Statistical Factors	F-value	Prob.
19.0 mm, Limestone, PG 64-28 & 19.0 mm, Limestone, PG 70-22	Mix Type	1.754	0.222
	Binder Type	21.560	0.002
	Mix Type × Binder Type	1.464	0.261
12.5 mm, Gravel, PG 70-22 & 12.5 mm, Limestone, PG 70-22	Mix Type	0.069	0.799
	Agg. Type	24.579	0.001
	Mix Type × Agg. Type	0.124	0.734
12.5 mm, Limestone, PG 70-22 & 19.0 mm, Limestone, PG 70-22	Mix Type	0.064	0.806
	Agg. Size	5.414	0.048
	Mix Type × Agg. Size	0.006	0.940

Chapter 6
Workability and Compactability of Foamed WMA and HMA 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, a new workability device was designed and fabricated at the University of Akron to measure the workability of foamed WMA and compare it to that of HMA. In addition, the compactability of foamed WMA and HMA mixtures was compared by analyzing the compaction data collected using the Superpave gyratory compactor during the preparation of the test specimens for the various laboratory tests discussed in Chapter 4. This chapter presents a summary of the workability results and compaction data obtained for foamed WMA and HMA mixtures. In addition, it includes a discussion on the effect of the foaming process on asphalt binder absorption, and how it might affect the workability and compactability of foamed WMA mixtures.

6.2 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 (1979). 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^{o}F to 392^{o}F (150^{o}C to 200^{o}C) at a rate of 1.8^{o}F/min (1^{o}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.

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 (1979), it included several features to improve mixing quality and measurement of torque values.

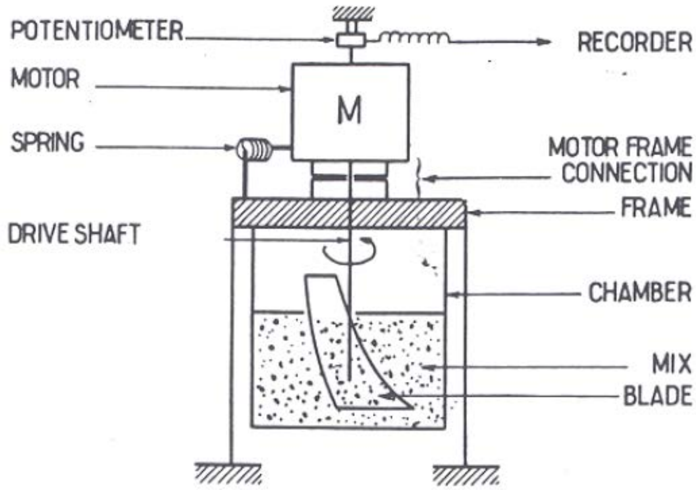

Figure 6.1: Marvillet and Bougault Workability Device (after Marvillet and Bougault, 1979).

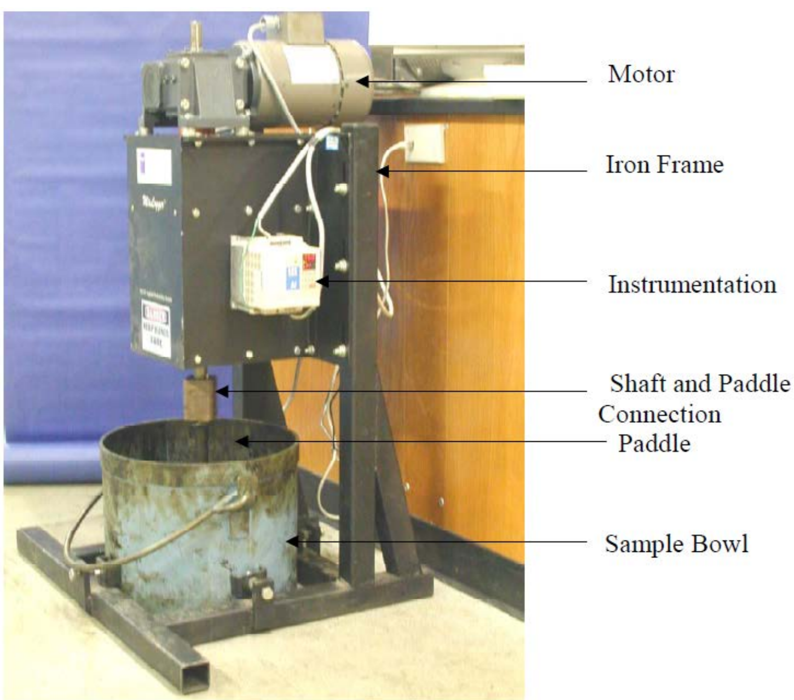

Figure 6.2: NCAT Workability Device (after Gudimettla et al., 2003).

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^{o}F (125^{o}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.

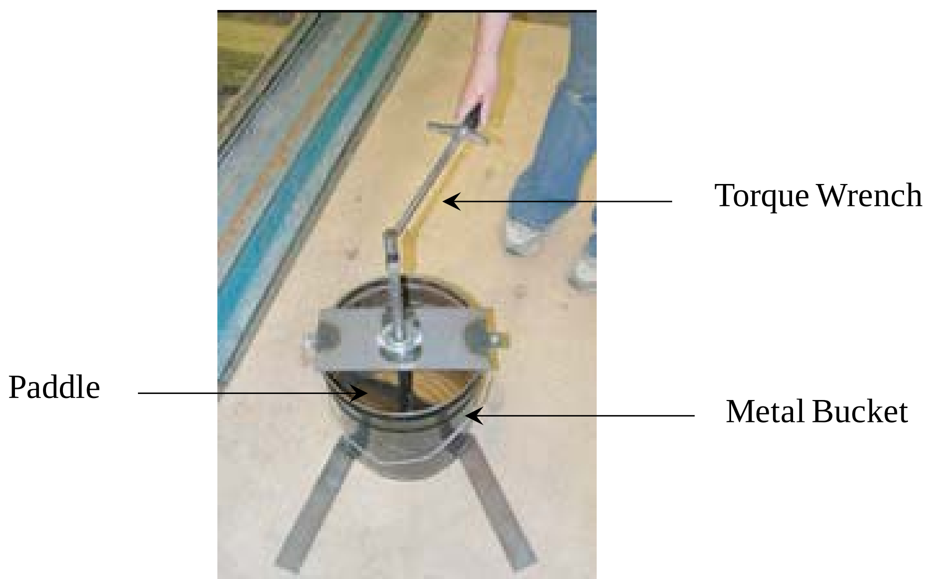

Figure 6.3: Tao and Mallick (2009) Workability Device (after Tao and Mallick, 2009).

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.

6.3 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, a rotating bucket with a stationary mixing paddle was used 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.

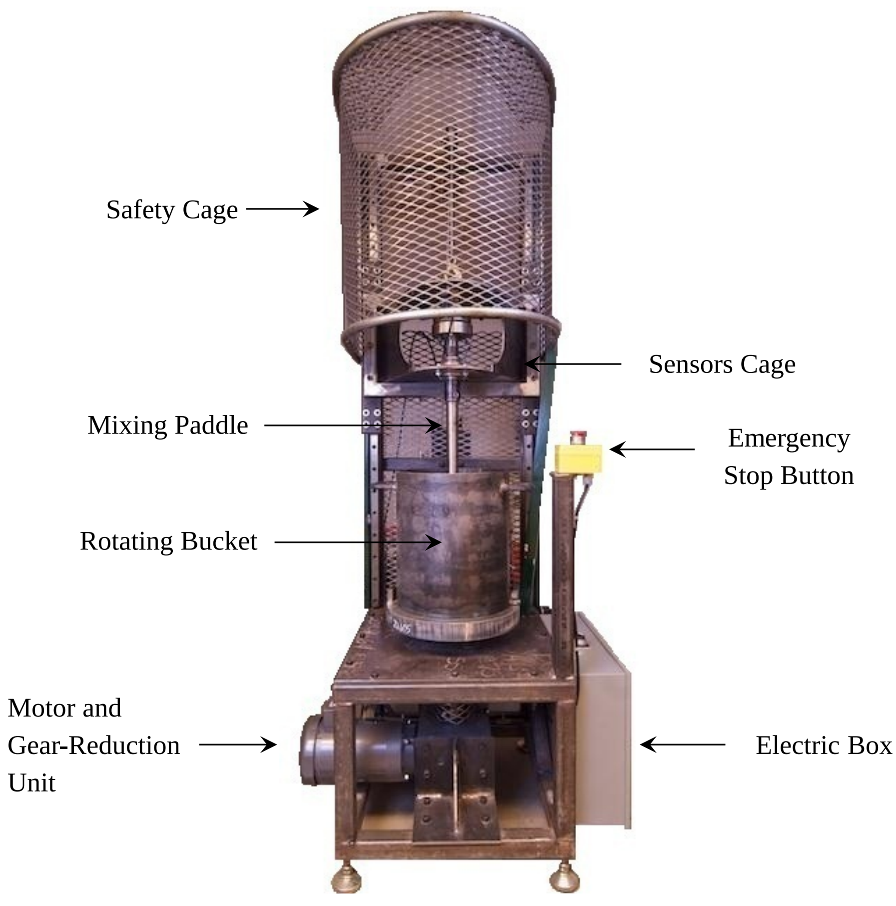

Figure 6.5: The University of Akron Workability Device.

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^o 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^o 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^{o}F (150^{o}C) before being placed in the rotating bucket and allowed to drop to a final temperature of 194^{o}F (90^{o}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.

6.4 **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^{o}F (150^{o}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^{o}F (150^{o}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^{o}F (100^{o}C) or high variability was observed in the measured torque data. In general, the workability test lasted approximately one hour.

6.5 Effect of Foamed Asphalt Binders on Asphalt Binder Absorption

Asphalt binder absorption plays an important role in determining the workability and compactability 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

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

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

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.

6.6 Workability Test Results

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^{o}F (100^{o}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 ($y = ae^{b.x}$) 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.

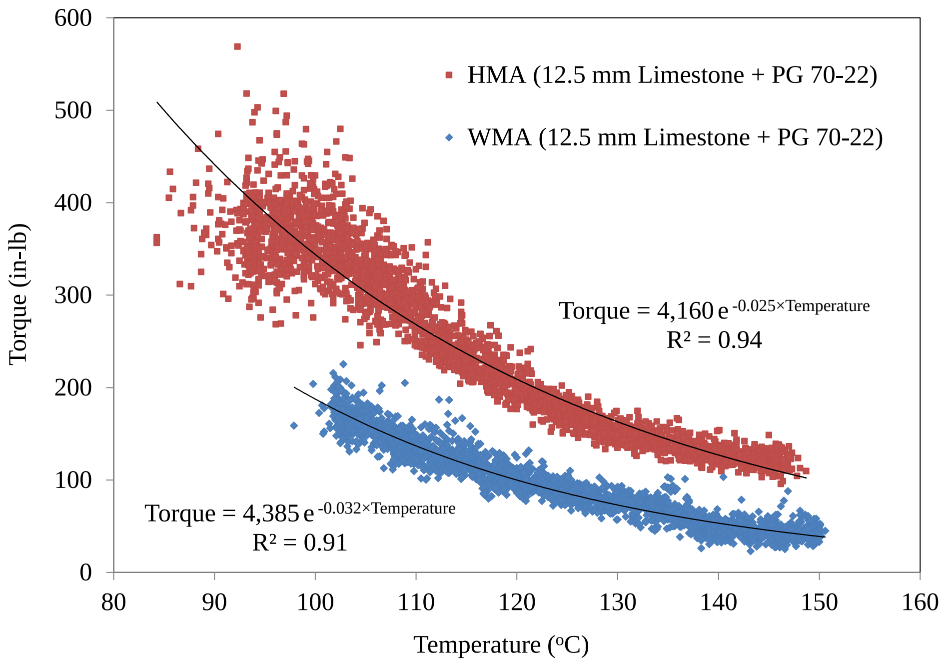

Figure 6.6: Torque versus Temperature Curves Obtained for 12.5 mm HMA and Foamed WMA Mixtures Prepared using Limestone Aggregates and PG 70-22 Asphalt Binder.

Table 6.2: Workability Exponential Models.

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

Figure 6.7 presents the average torque values obtained at high and low testing temperatures (302^{o}F and 212^{o}F (150^{o}C and 100^{o}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.

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 Gudimettla 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 and the less asphalt binder absorption observed for the foamed WMA mixtures than the corresponding HMA 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 for both HMA and foamed WMA asphalt mixtures. As can be observed from Table 6.1, the surface mixtures had higher effective asphalt binder content than the intermediate mixtures, which explains the lower torque values observed for the surface mixtures.

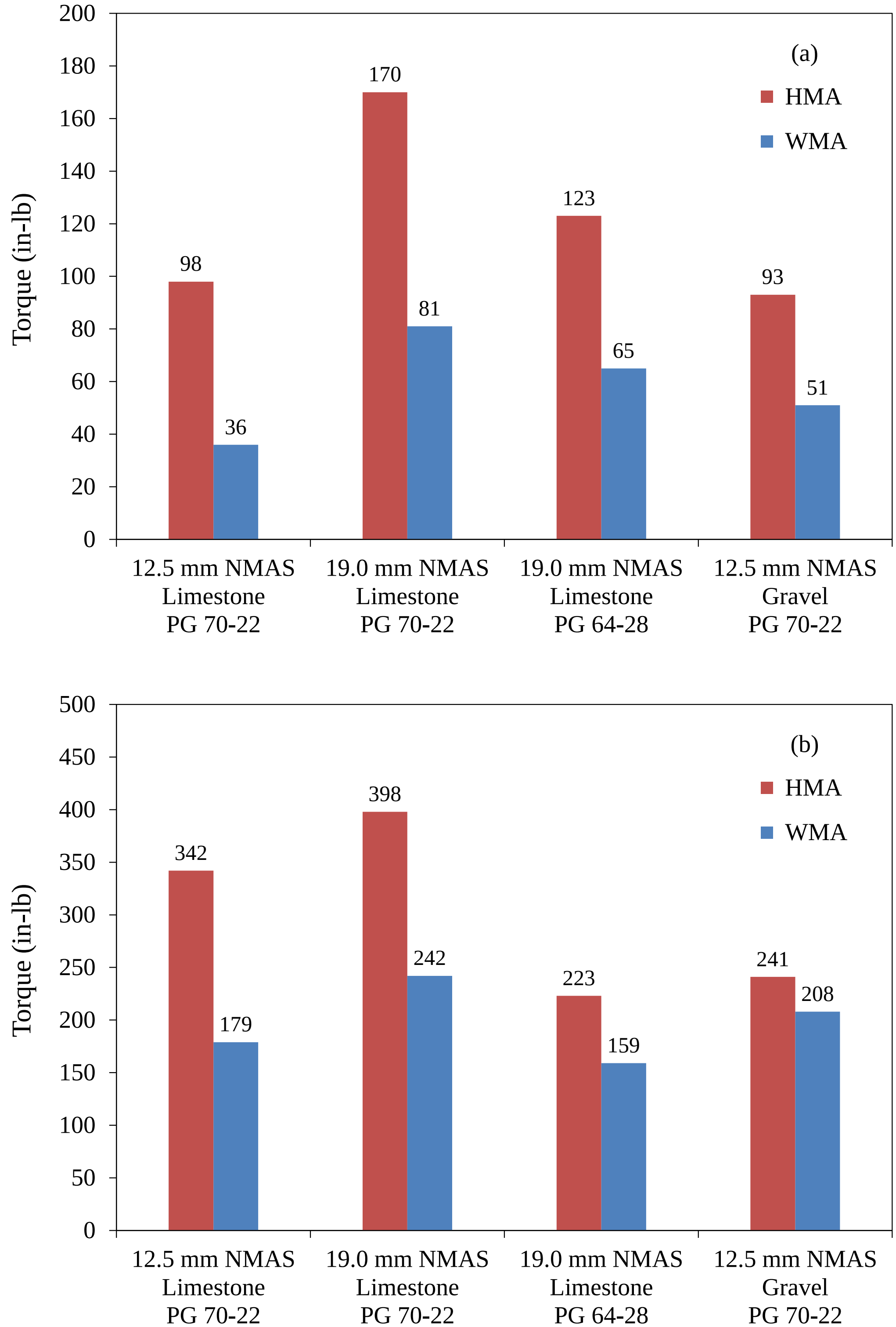

Figure 6.7: Average Torque Values Obtained at (a) 150^{o}C and (b) 100^{o}C.

6.7 Compaction Test Results

As previously mentioned, the compaction data collected using the Superpave gyratory compactor was analyzed to compare the compactability of foamed WMA and HMA mixtures. Table 6.3 presents the number of gyrations required to compact the test specimens to the target air voids level specified for the various laboratory tests. As can be noticed from this table, similar number of gyrations was needed to compact the foamed WMA and HMA specimens. This indicates that the foamed WMA mixtures might require a similar compaction effort to that needed for the HMA mixtures to reach the target density in the field (i.e., number of roller passes). Given that the foamed WMA mixtures were produced and compacted using temperatures 30^{o}F (16.7^{o}C) lower than the HMA mixtures, these results indicate that the foamed WMA mixtures are more compactable than the traditional HMA mixtures.

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

				Average No. of Gyrations			
Mix	**Agg. Type**	**Agg. Size**	**Binder Type**	**APA**	**T283**	**E***	**ITS/DCSE**
HMA	Limestone	12.5 mm	PG 70-22	38	36	29	41
HMA	Limestone	19.0 mm	PG 70-22	23	23	18	27
HMA	Limestone	19.0 mm	PG 64-22	29	22	18	24
HMA	Gravel	12.5 mm	PG 70-22	15	15	12	14
WMA	Limestone	12.5 mm	PG 70-22	43	28	29	38
WMA	Limestone	19.0 mm	PG 70-22	27	22	18	24
WMA	Limestone	19.0 mm	PG 64-22	27	17	17	18
WMA	Gravel	12.5 mm	PG 70-22	16	12	9	14

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 the HMA mixtures required comparable compaction efforts for both material combinations. However, a lower compaction effort was needed for the foamed WMA mixtures prepared using PG 64-28 asphalt binder, which indicates that the effect of foaming is more pronounced on softer asphalt binders.

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 a significantly lower compaction effort was needed for mixtures prepared using crushed gravel than those prepared using limestone aggregates. This was the case for both foamed WMA and HMA mixtures. These results can be attributed to the lower coarse aggregate angularity of the crushed gravel aggregates.

Finally, by comparing the number of gyrations obtained for the surface and intermediate foamed WMA and HMA mixtures prepared using limestone aggregates and PG 70-22 asphalt binder, it can be observed that the intermediate mixtures required a significantly lower compaction effort than those prepared using the surface mixtures. This was the case for both foamed WMA and HMA asphalt mixtures. This indicates that the surface mixtures are less compactable than the intermediate mixtures.

Chapter 7

Effect of Mix Preparation Procedure on Foamed WMA Performance

7.1 Introduction

The foamed WMA mixtures presented in the previous chapters were prepared using fully dried aggregates according to the current WMA specifications used by ODOT (i.e., a temperature reduction of 30^{o}F (16.7^{o}C) and a foaming water content of 1.8%). This chapter investigates the effect of the temperature reduction, foaming water content, and aggregate moisture content on the performance of foamed WMA.

7.2 Testing Plan

Figure 7.1 presents the testing plan conducted to evaluate the effect of the mix preparation procedure on the performance of foamed WMA. 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^{o}F (16.7^{o}C), 50^{o}F (27.8^{o}C), and 70^{o}F (38.9^{o}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.

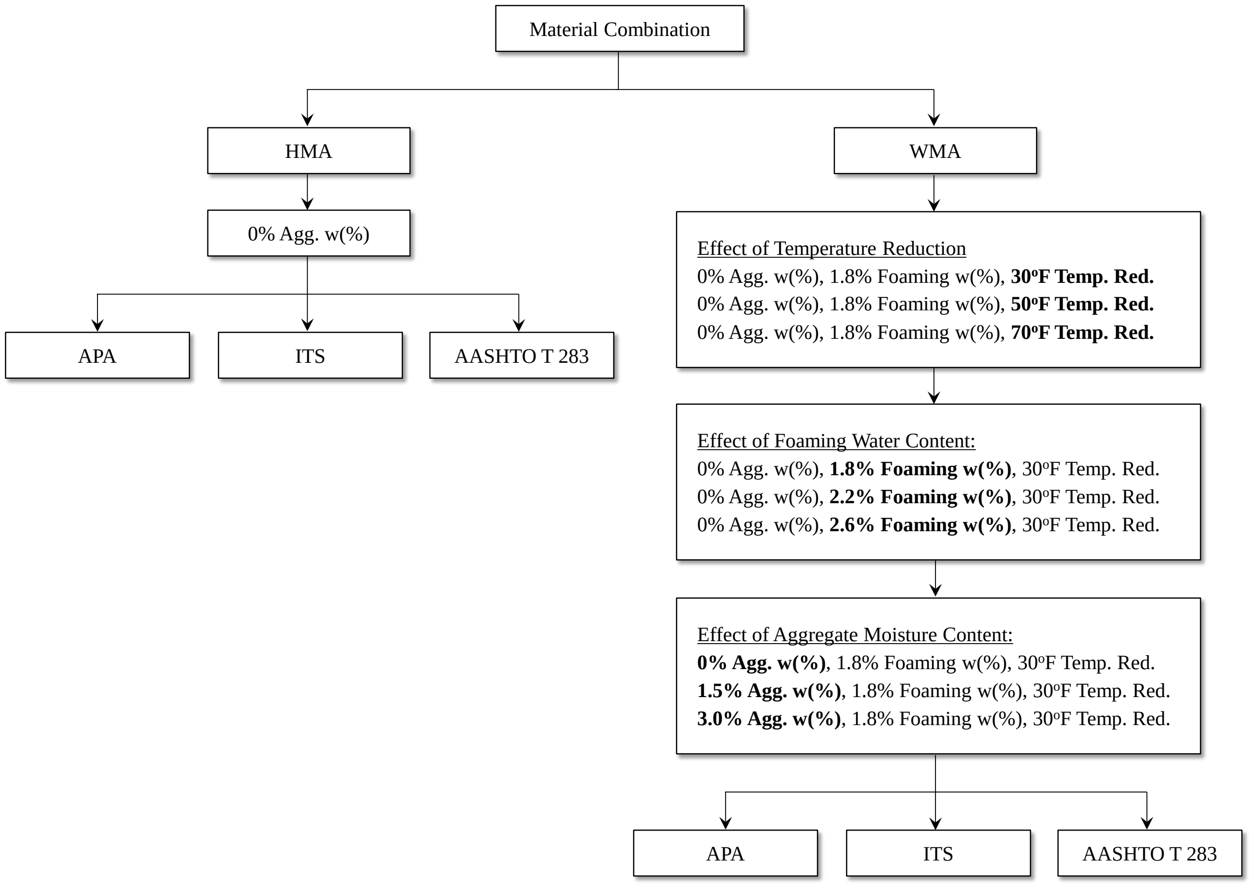

Figure 7.1: Testing Plan Implemented to Examine the Effect
of Mix Preparation Procedure on the Performance of Foamed WMA Mixtures.

7.3 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^{o}F (16.7^{o}C) temperature reduction was comparable to that of HMA. However, higher rut depths were obtained for the foamed WMA mixtures produced using 50^{o}F (27.8^{o}C) and 70^{o}F (38.9^{o}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^{o}F (16.7^{o}C) temperature reduction. However, lower dry and wet ITS values were obtained for the foamed WMA mixtures produced using 50^{o}F (27.8^{o}C) and 70^{o}F (38.9^{o}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^{o}F (27.8^{o}C) and 70^{o}F (38.9^{o}C) temperature reductions were 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.

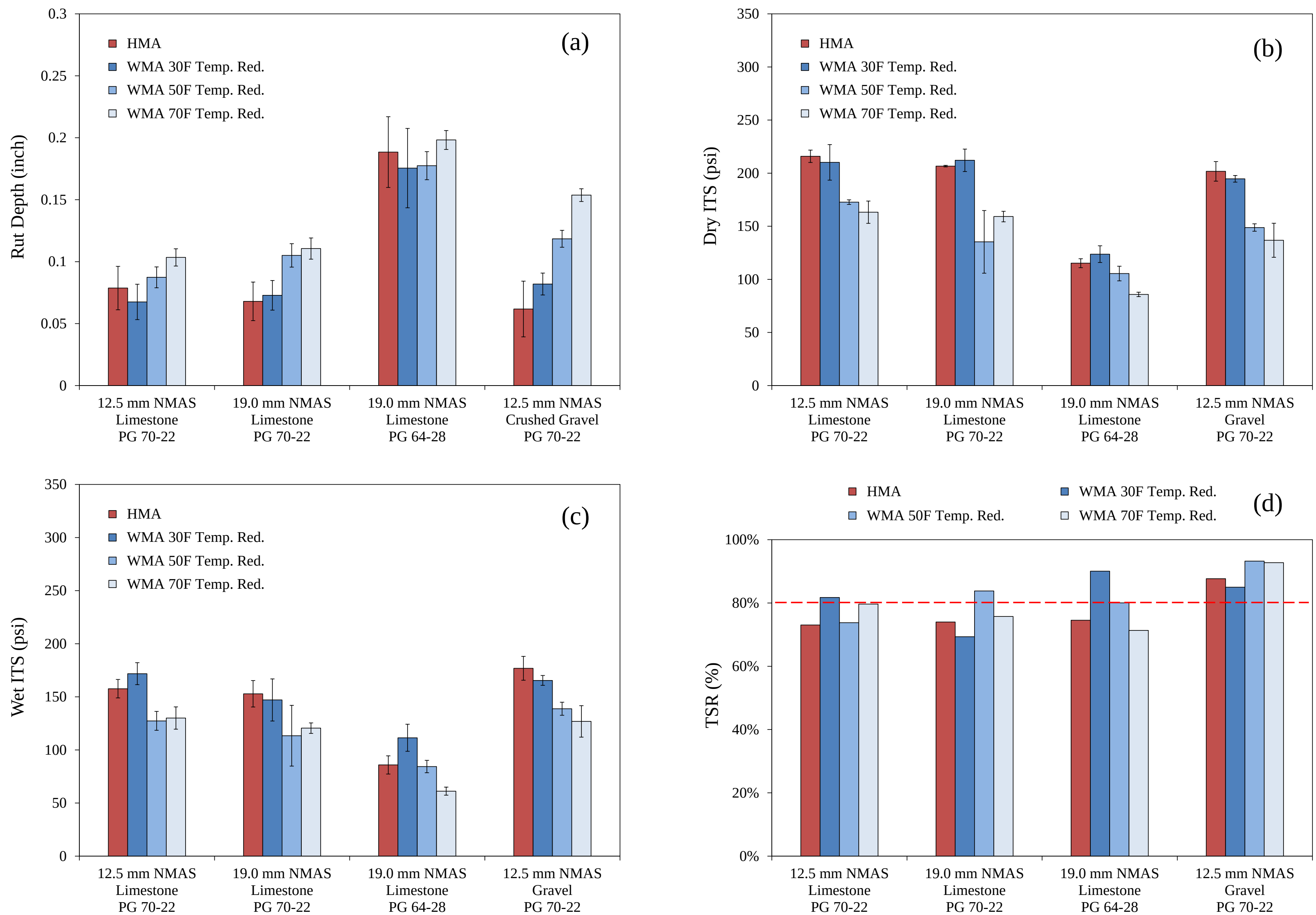

Figure 7.2: Effect of Temperature Reduction on Mix Performance (a. APA Rut Depth, b. Dry ITS, c. Wet ITS, and d. TSR).

Table 7.1: Effect of Temperature Reduction on Foamed WMA Performance.

		Performance Test					
		APA Rut Depth		Dry ITS		Wet ITS	
Analysis Data	Statistical Factors	F-value	Prob.	F-value	Prob.	F-value	Prob.
WMA, 19.0 mm, Limestone, PG 64-28 & WMA, 19.0 mm, Limestone, PG 70-22	Binder Type	136.4	0.00	97.5	0.00	31.4	0.00
	Prod. Temp.	5.5	0.02	22.8	0.00	10.0	0.00
	Binder Type × Prod. Temp.	1.4	0.29	7.3	0.01	1.6	0.25
WMA, 12.5 mm, Gravel, PG 70-22 & WMA, 12.5 mm, Limestone, PG 70-22	Agg. Type	55.3	0.00	18.8	0.00	0.0	0.91
	Prod. Temp.	53.3	0.00	40.7	0.00	30.6	0.00
	Agg. Type × Prod. Temp.	4.7	0.03	0.5	0.64	1.5	0.26
WMA, 12.5 mm, Limestone, PG 70-22 & WMA, 19.0 mm, Limestone, PG 70-22	Agg. Size	4.4	0.06	3.3	0.10	4.5	0.06
	Prod. Temp.	20.7	0.00	24.4	0.00	10.5	0.00
	Agg. Size × Prod. Temp.	0.6	0.54	2.8	0.10	0.4	0.71

7.4 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^{o}F (16.7^{o}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 various 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.

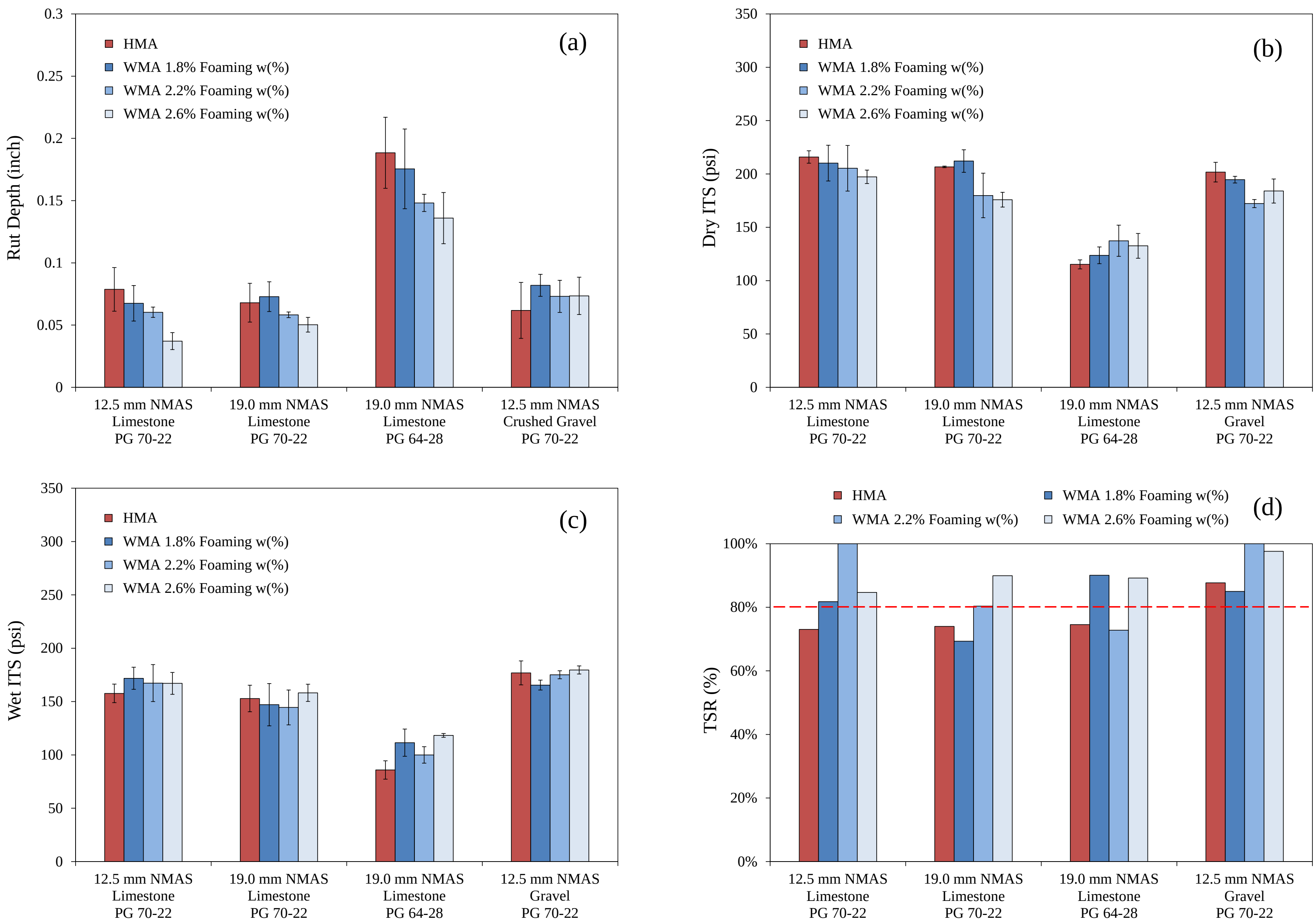

Figure 7.3: Effect of Foaming Water Content on Mix Performance (a. APA Rut Depth, b. Dry ITS, c. Wet ITS, and d. TSR).

Table 7.2: Effect of Foaming Water Content on Foamed WMA Performance.

		Performance Test					
		APA Rut Depth		Dry ITS		Wet ITS	
Analysis Data	Statistical Factors	F-value	Prob.	F-value	Prob.	F-value	Prob.
WMA, 19.0 mm, Limestone, PG 64-28 & WMA, 19.0 mm, Limestone, PG 70-22	Binder Type	138.5	0.00	43.0	0.00	42.9	0.00
	Prod. Temp.	5.4	0.02	1.2	0.29	4.3	0.06
	Binder Type × Prod. Temp.	0.4	0.67	1.8	0.20	0.0	0.97
WMA, 12.5 mm, Gravel, PG 70-22 & WMA, 12.5 mm, Limestone, PG 70-22	Agg. Type	16.6	0.00	7.4	0.03	1.9	0.19
	Prod. Temp.	4.7	0.03	0.5	0.50	0.6	0.47
	Agg. Type × Prod. Temp.	0.2	0.26	0.6	0.47	1.6	0.23
WMA, 12.5 mm, Limestone, PG 70-22 & WMA, 19.0 mm, Limestone, PG 70-22	Agg. Size	1.8	0.20	3.7	0.08	6.0	0.03
	Prod. Temp.	14.2	0.00	3.0	0.11	0.5	0.48
	Agg. Size × Prod. Temp.	1.2	0.35	0.3	0.57	1.2	0.29

7.5 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^{o}F (16.7^{o}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 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.

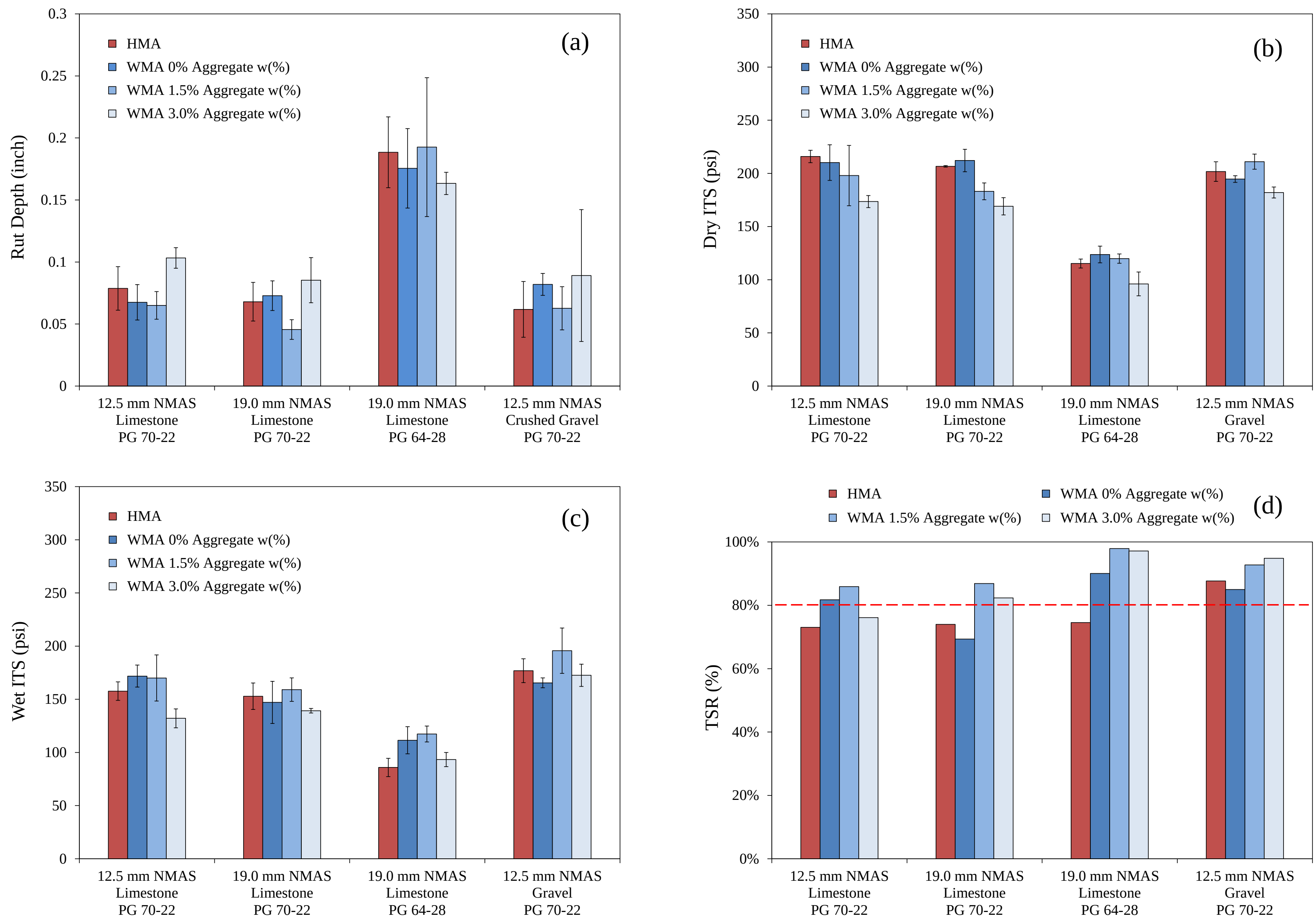

Figure 7.4: Effect of Aggregate Moisture Content on Mix Performance (a. APA Rut Depth, b. Dry ITS, c. Wet ITS, and d. TSR).

Table 7.3: Effect of Aggregate Moisture Content on Foamed WMA Performance.

		Performance Test					
		APA Rut Depth		Dry ITS		Wet ITS	
Analysis Data	Statistical Factors	F-value	Prob.	F-value	Prob.	F-value	Prob.
WMA, 19.0 mm, Limestone, PG 64-28 & WMA, 19.0 mm, Limestone, PG 70-22	Binder Type	67.5	0.00	331.8	0.00	58.5	0.00
	Prod. Temp.	0.1	0.94	25.0	0.00	5.4	0.02
	Binder Type × Prod. Temp.	2.3	0.14	3.2	0.08	0.3	0.73
WMA, 12.5 mm, Gravel, PG 70-22 & WMA, 12.5 mm, Limestone, PG 70-22	Agg. Type	0.0	0.96	0.1	0.74	8.6	0.01
	Prod. Temp.	2.7	0.11	6.8	0.01	6.9	0.01
	Agg. Type × Prod. Temp.	0.5	0.61	1.7	0.22	4.2	0.04
WMA, 12.5 mm, Limestone, PG 70-22 & WMA, 19.0 mm, Limestone, PG 70-22	Agg. Size	3.3	0.09	4.2	0.06	9.3	0.01
	Prod. Temp.	15.0	0.00	1.6	0.24	2.6	0.12
	Agg. Size × Prod. Temp.	1.9	0.20	0.1	0.93	1.8	0.20

Chapter 8
Performance Evaluation of Foamed WMA and HMA in the Accelerated Pavement Load Facility

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.

Figure 8.1: Picture of the Accelerated Pavement Load Facility at Ohio University.

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^{o}F (162.7^{o}C) and 270^{o}F (132.2^{o}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.

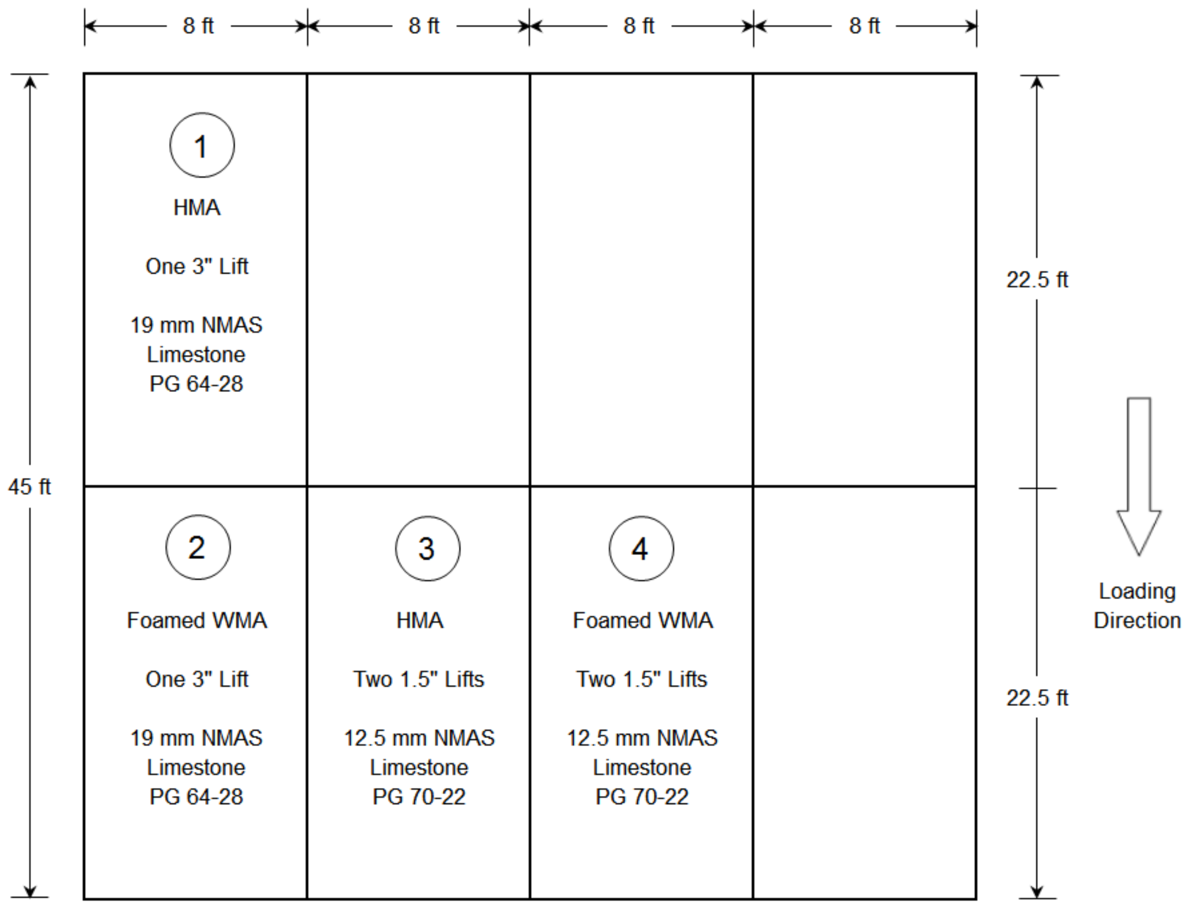

Figure 8.2: Pavement Sections at the APLF.

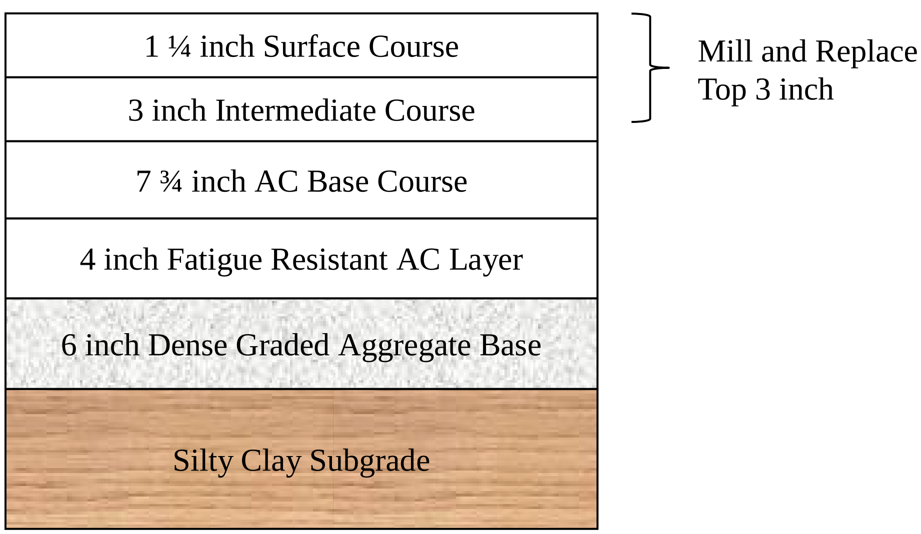

Figure 8.3: Pavement Structure at the APLF.

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.

Figure 8.6: Milled Pavement Surface.

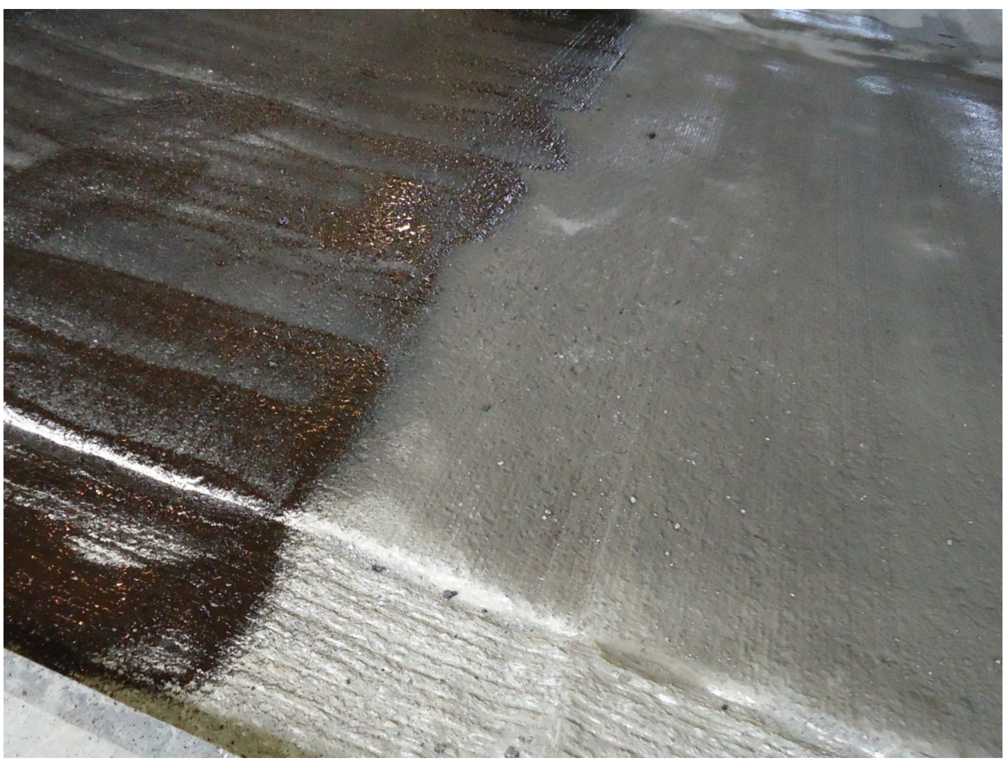

Figure 8.7: Tack Coat Application.

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.

Figure 8.12: Coring of Field Specimens.

Figure 8.13: Location of Field Cores.

Figure 8.14: Picture of Field Cores.

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.

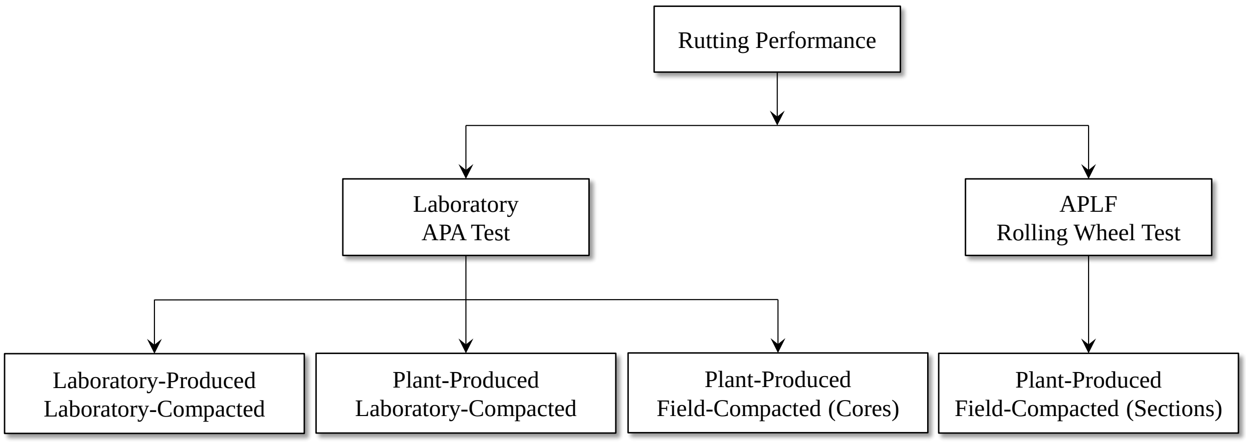

Figure 8.15: Testing Program Used for the APA and APLF Rolling Wheel Tests.

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

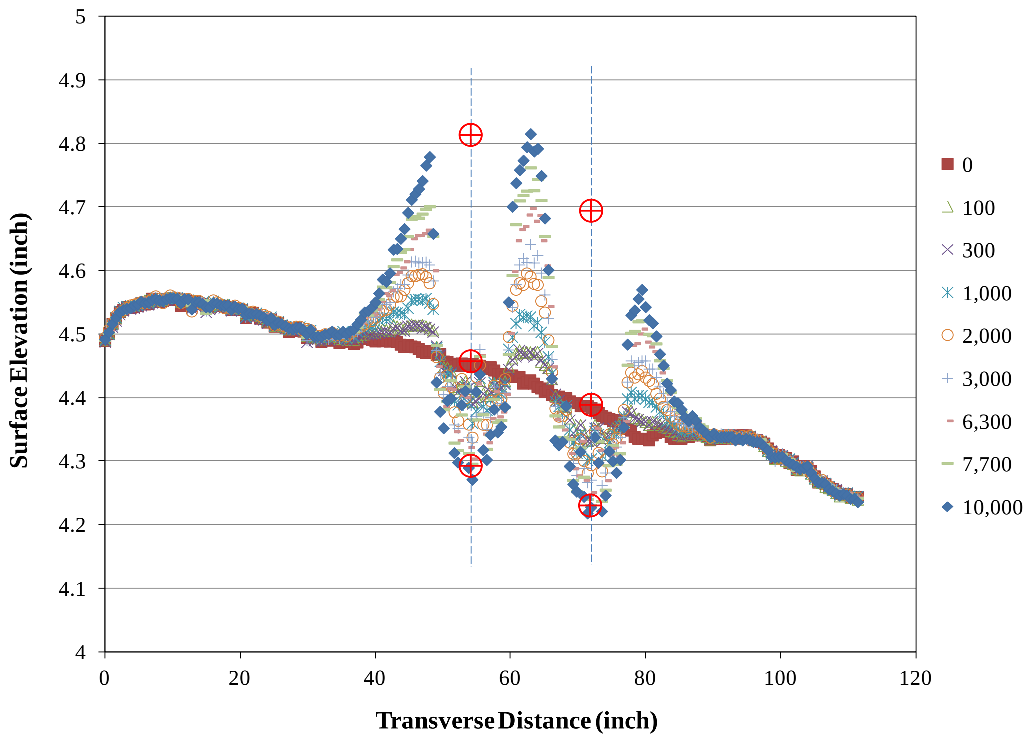

Figure 8.17: Example Surface Profiles Obtained at Various Loading Cycles for HMA Mixture Prepared using 19.0 mm NMAS Limestone Aggregate and PG 64-28.

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 straightedge 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 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

Table 8.1: APLF Surface Elevations and Rut Depth Results
Obtained after 0 and 10,000 Loading Cycles.

Pavement Section	Lane/Tire	Elevation (inch)		Rut Depth (inch)	Average Rut Depth (inch)
		0	10,000		
HMA 12.5 mm NMAS PG 70-22	North/1	4.556	4.406	0.150	0.160
	North/2	4.566	4.415	0.151	
	South/1	4.641	4.448	0.193	
	South/2	4.556	4.411	0.145	
Foamed WMA 12.5 mm NMAS PG 70-22	North/1	4.275	4.197	0.078	0.113
	North/2	4.275	4.136	0.139	
	South/1	4.136	4.049	0.087	
	South/2	4.17	4.024	0.146	
HMA 19 mm NMAS PG 64-28	North/1	4.454	4.271	0.183	0.194
	North/2	4.382	4.221	0.161	
	South/1	4.642	4.439	0.203	
	South/2	4.515	4.288	0.227	
Foamed WMA 19 mm NMAS PG 64-28	North/1	4.626	4.394	0.232	0.229
	North/2	4.563	4.334	0.229	
	South/1	4.479	4.280	0.199	
	South/2	4.535	4.280	0.255	

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.

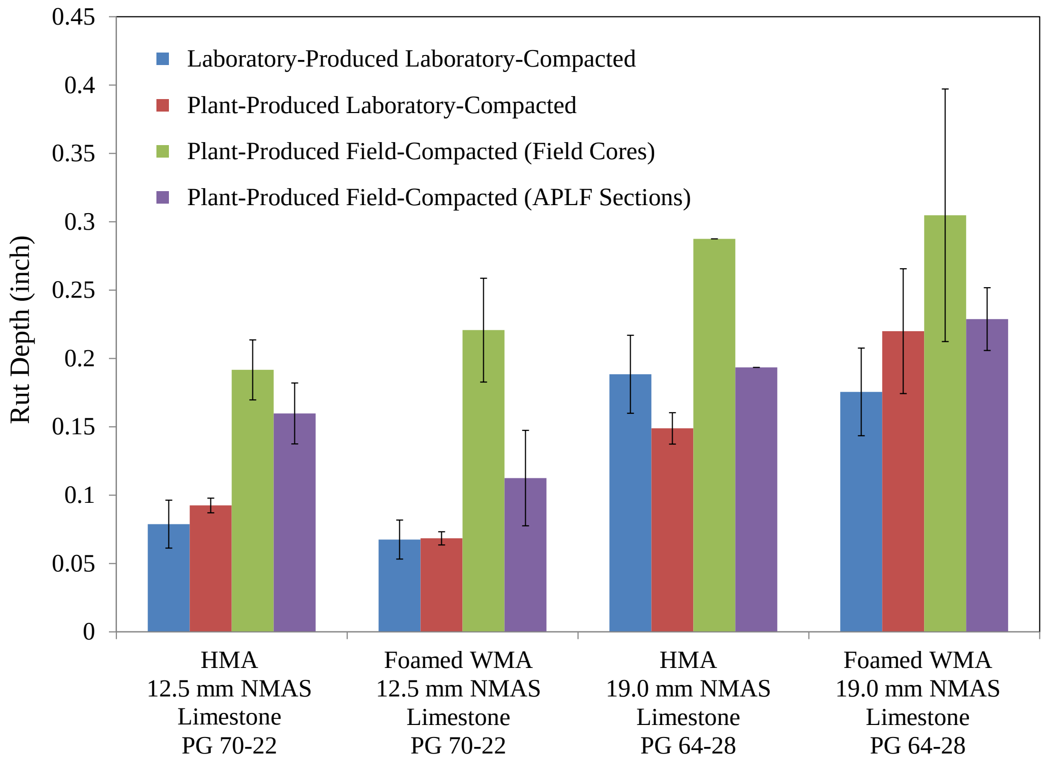

Figure 8.18: APLF and APA Rut Depths.

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

Effect	F-value	Prob.
Preparation Method	45.92	< 0.0001
Mix Type	0.77	0.3874
Preparation Method × Mix Type	1.31	0.2853

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

Method	Estimate	Standard Error	Ranking
Laboratory-Produced Laboratory-Compacted	0.1275	0.008440	A
Plant-Produced Laboratory-Compacted	0.1324	0.008440	A
Plant-Produced Field-Compacted (Field Cores)	0.2512	0.008440	B

In summary, the rutting performance of the foamed WMA and HMA mixtures was found to be similar for both surface and intermediate mixtures. This suggests that the foamed WMA mixtures will have similar rutting characteristics to the HMA mixtures in the field. In addition, the APA rut depth values obtained for plant-produced laboratory-compacted specimens were similar to those obtained for laboratory-produced laboratory-compacted specimens. This was the case for both foamed WMA and HMA mixtures. This indicates that the laboratory preparation procedure used in this study resulted in comparable foamed WMA and HMA mixtures to those produced in the field.

Chapter 9

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. 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.

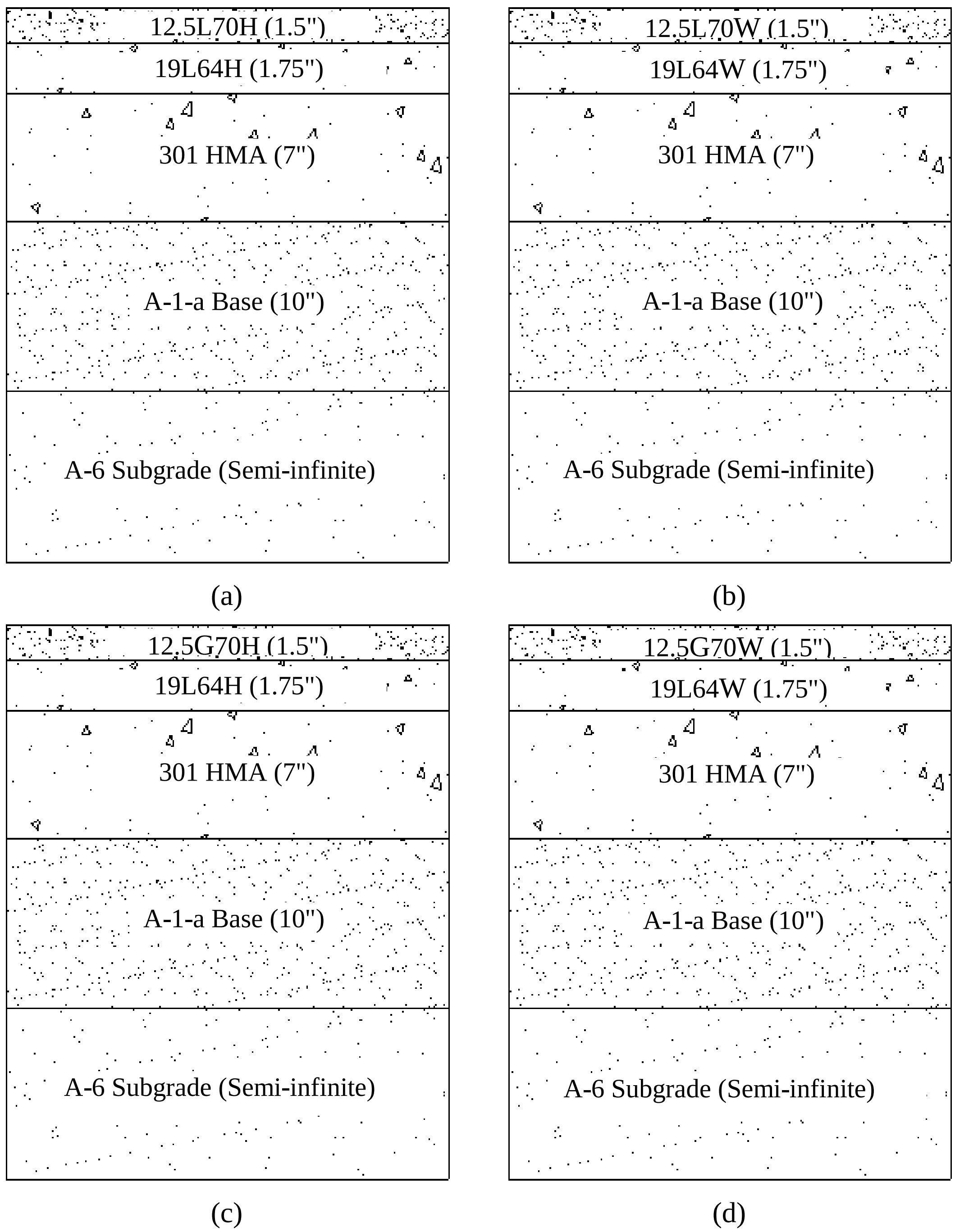

Figure 9.1: Baseline Pavement Structures: (a) HL-HL, (b) WL-WL, (c) HG-HL, (d) WG-WL.

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.

Section	Material	Input Level	Input Parameter
Surface Course	12.5 mm HMA or WMA	Level I	E* at 6 frequencies and 5 temperatures
Intermediate Course	19.0 mm HMA or WMA	Level I	E* at 6 frequencies and 5 temperatures
AC Base Course	Item 301	Level II	E* at 6 frequencies and 5 temperatures
Aggregate Base	A-1-a	Level II	M_r = 45 ksi
Subgrade	A-6	Level II	M_r = 12 ksi

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 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.

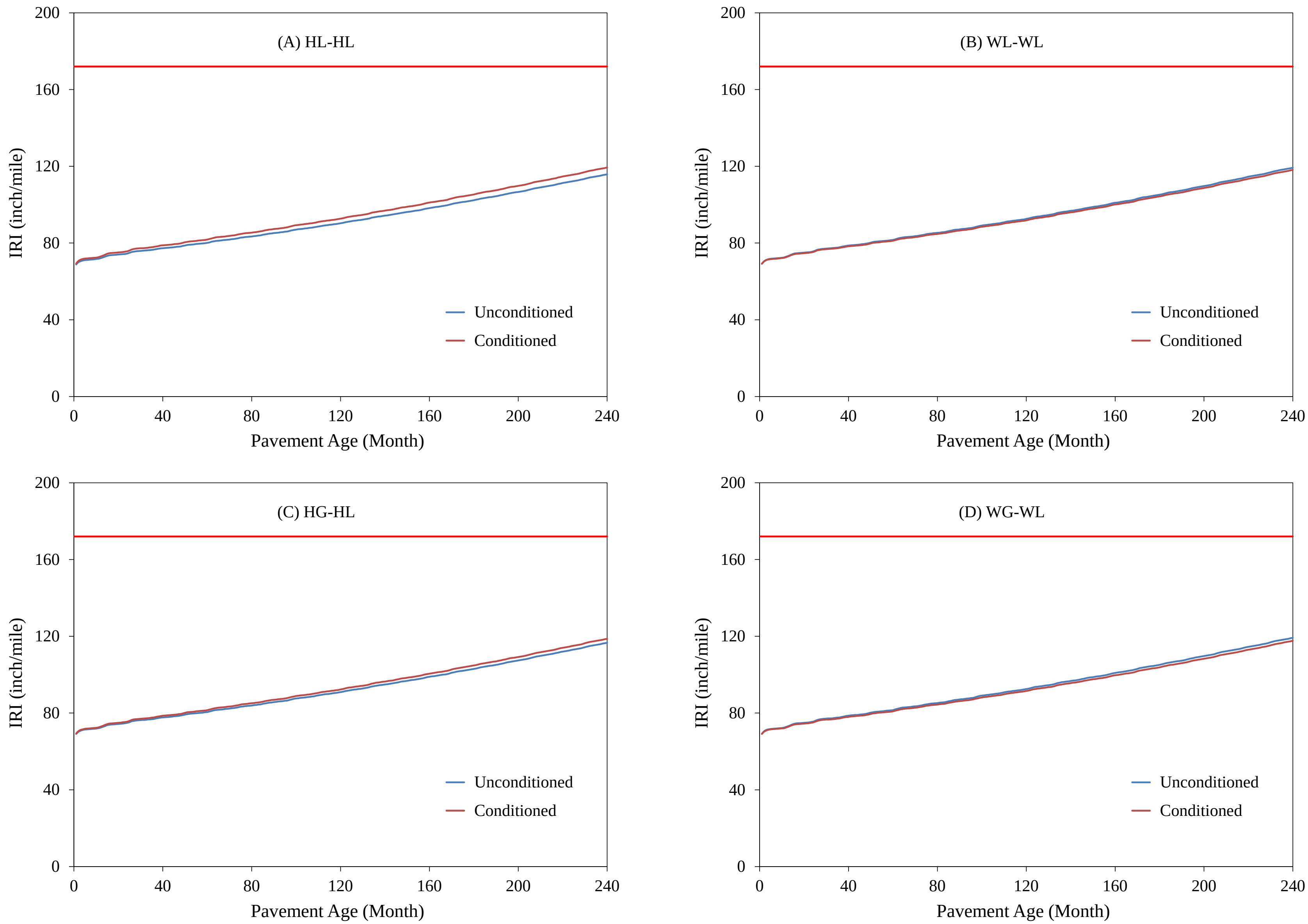

Figure 9.2: MEPDG Predictions for IRI.

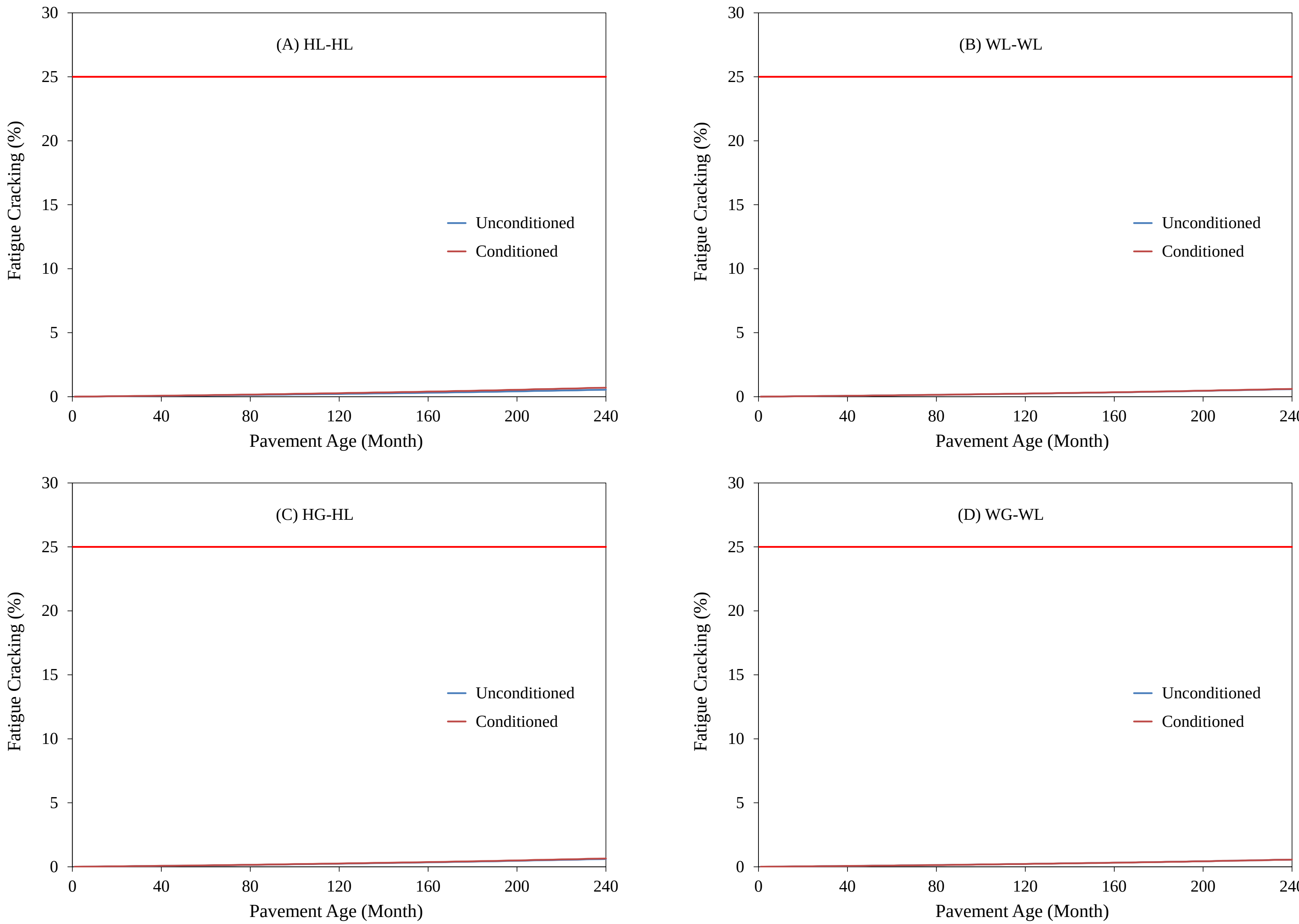

Figure 9.3: MEPDG Predictions for Fatigue Cracking.

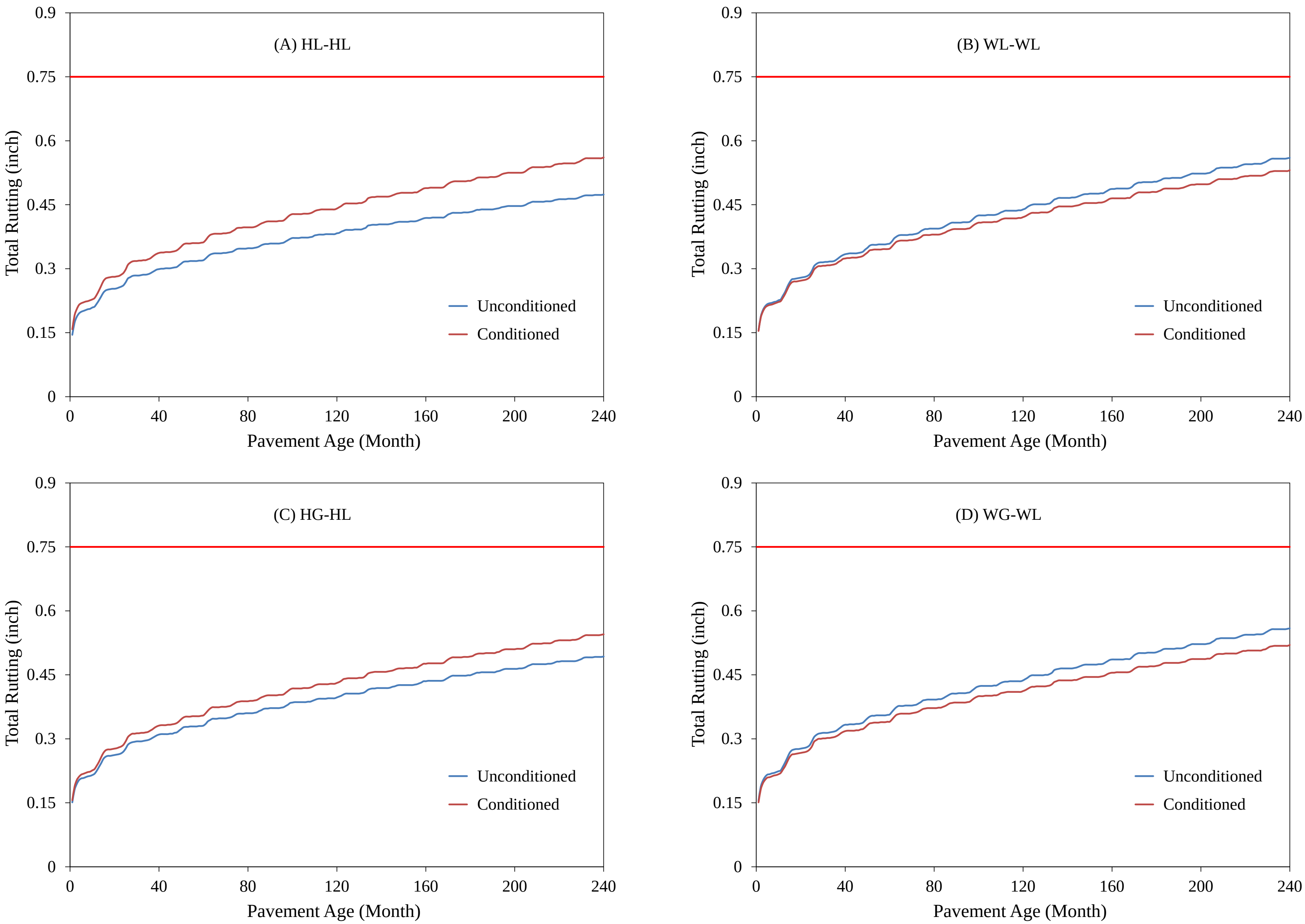

Figure 9.4: MEPDG Predictions for Total Rutting.

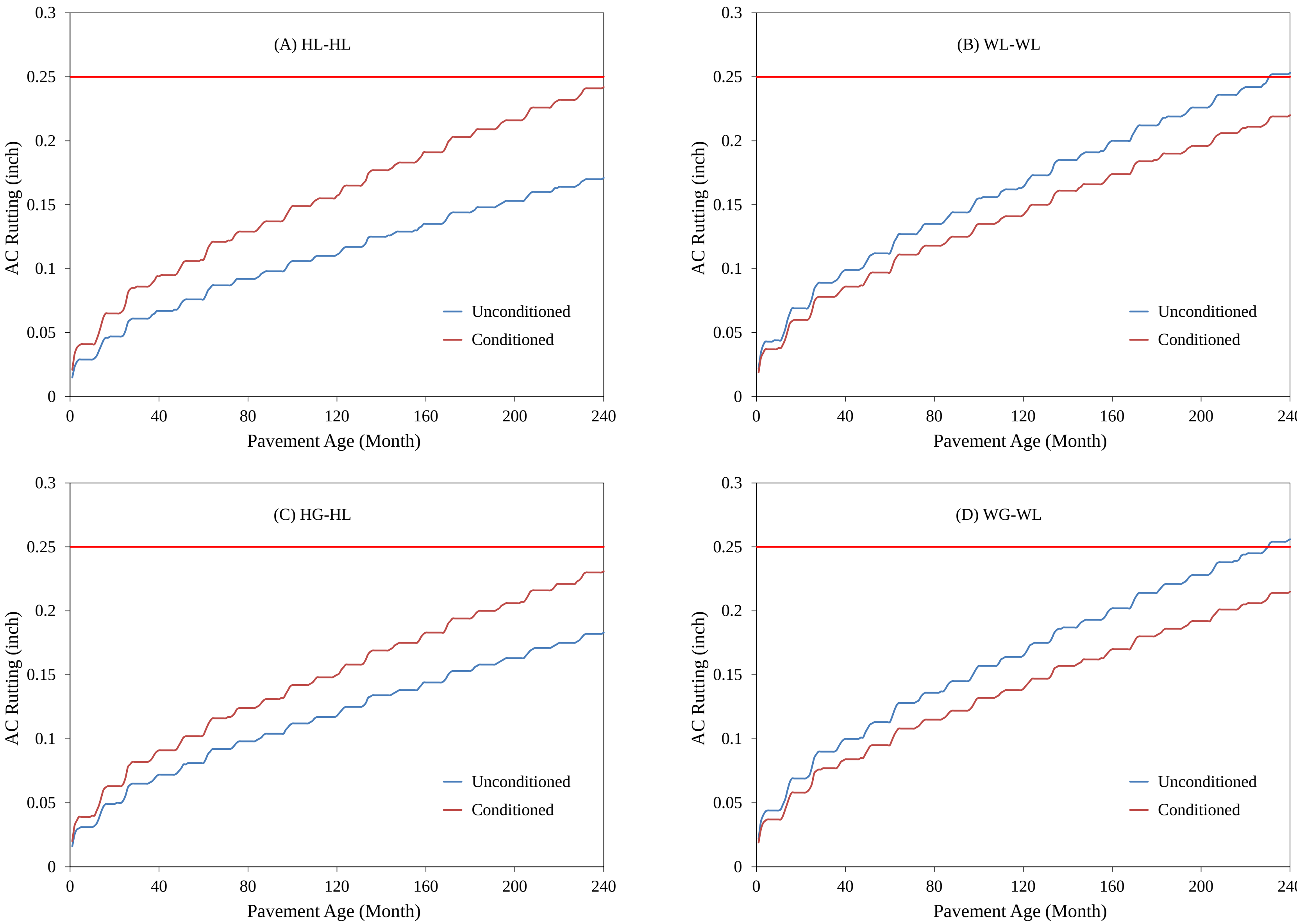

Figure 9.5: MEPDG Predictions for AC Rutting.

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

Chapter 10
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 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 main research activities and 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 both 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 using fully dried aggregates according to the current ODOT specifications (i.e., 30^{o}F temperature reduction and 1.8% foaming water content).

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 both 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^oF (-10^oC) 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^oC and the test was terminated when the mixture's temperature reached approximately 100^{o}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 the 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 from 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 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 a higher asphalt binder content. The foamed WMA mixtures in that study exhibited a significant reduction in asphalt binder absorption, 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^{o}F, 50^{o}F, and 70^{o}F (16.7^{o}C, 27.8^{o}C, and 38.9^{o}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^{o}F (16.7^{o}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^{o}F (16.7^{o}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 were 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 (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^{o}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^{o}F (16.7^{o}C) for the production of foamed WMA. In addition, 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. Furthermore, 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 than 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.

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