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IMPLEMENTATION OF WARM-MIX ASPHALT MIXTURES IN NEBRASKA PAVEMENTS

Yong-Rak Kim, Ph.D.
Associate Professor
Department of Civil Engineering
University of Nebraska-Lincoln

Jun Zhang
Graduate Research Assistant

Hoki Ban, Ph.D.
Postdoctoral Research Associate

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Nebraska Transportation Center
262 WHIT
2200 Vine Street
Lincoln, NE 68583-0851
(402) 472-1975

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Yong-Rak Kim, Ph.D.
Associate Professor
Department of Civil Engineering
University of Nebraska-Lincoln

Jun Zhang
Graduate Research Assistant
Department of Civil Engineering
University of Nebraska-Lincoln

Hoki Ban, Ph.D.
Postdoctoral Research Associate
Department of Civil Engineering
University of Nebraska-Lincoln

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The WMA additives evaluated in this study did not significantly affect the viscoelastic stiffness characteristics of the asphalt mixtures. WMA mixtures generally presented better rut resistance than their HMA counterparts, and the WMA with Sasobit increased the rut resistance significantly, which agrees with other similar studies. However, two laboratory tests—the AASHTO T283 test and semi-circular bend fracture test with moisture conditioning—to assess moisture damage susceptibility demonstrated identical results indicating greater moisture damage potential of WMA mixtures. MEPDG results simulating 20-year field performance presented insignificant pavement distresses with no major performance difference between WMA and HMA, and this has been confirmed by actual field performance data. Although only two-year field performance is available to date, both the WMA and HMA have performed well. No cracking or other failure modes have been observed in the trial sections. The rut depth and the roughness of WMA and HMA sections were similar.</td></tr>
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Abstract

The primary objective of this research is to evaluate the feasibility of several WMA mixtures as potential asphalt paving mixtures for Nebraska pavements. To that end, three well-known WMA additives (i.e., Sasobit, Evotherm, and Advera synthetic zeolite) were evaluated. For a more realistic evaluation of the WMA approaches, trial pavement sections of the WMA mixtures and their HMA counterparts were implemented in Antelope County, Nebraska. More than one ton of field-mixed loose mixtures was collected at the time of paving and was transported to the NDOR and UNL laboratories to conduct comprehensive laboratory evaluations and pavement performance predictions of the individual mixtures involved. Various key laboratory tests were conducted to identify mixture properties and performance characteristics. These laboratory test results were then incorporated into other available data and the MEPDG software to predict the long-term field performance of the WMA and HMA trial sections. Pavement performance predictions from the MEPDG were also compared to two-year actual field performance data that have annually been monitored by the NDOR pavement management team.

The WMA additives evaluated in this study did not significantly affect the viscoelastic stiffness characteristics of the asphalt mixtures. WMA mixtures generally presented better rut resistance than their HMA counterparts, and the WMA with Sasobit increased the rut resistance significantly, which agrees with other similar studies. However, two laboratory tests—the AASHTO T283 test and semi-circular bend fracture test with moisture conditioning—to assess moisture damage susceptibility demonstrated identical results indicating greater moisture damage potential of WMA mixtures. MEPDG results simulating 20-year field performance presented insignificant pavement distresses with no major performance difference between

WMA and HMA; this has been confirmed by actual field performance data. Although only two-year field performance is available to date, both the WMA and HMA have performed well. No cracking or other failure modes have been observed in the trial sections. The rut depth and the roughness of WMA and HMA sections were similar.

Chapter 1 Introduction

Conventional hot-mix asphalt (HMA) has been the primary material used in asphaltic paving in past decades. However, compared to conventional HMA mixtures, warm-mix asphalt (WMA) mixtures have shown great potential, and WMA mixtures offer benefits not given by HMA mixtures, since the WMA mixtures can produce asphaltic layers at lower temperatures without compromising pavement performance. WMA materials can reduce the viscosity of the binder by the addition of warm-mix additives; thus, the production and compaction temperatures can be lower, compared to those needed for conventional HMA. One of the primary benefits of WMA is the opportunity to reduce carbon dioxide emissions during the production and compaction of asphalt mixtures. This could support the objective of reducing greenhouse gas emissions set by the Kyoto Protocol, as well as allowing asphalt mixture plants to be located in select areas with strict air regulations. In addition, WMA technology presents other obvious advantages, such as less fuel usage, the ability to haul asphalt mixtures greater distances, better working conditions, an extended paving season, and the potential use of more reclaimed asphalt pavement (RAP) materials.

WMA is gaining acceptance across the United States, with at least 45 states either actively using WMA materials or having constructed a trial project. A number of states, including Alabama, California, Florida, Illinois, New York, North Carolina, Ohio, Pennsylvania, Texas, Virginia, Washington, and Wisconsin have adopted permissive specifications allowing the use of WMA on many highway projects. Some industry leaders predict that about 90% of asphalt plant production could possibly be WMA in five years. About one million tons of WMA have been placed, and another one million tons are under contract in Texas. The Pennsylvania Department of Transportation (DOT) has established a target of 20% of their 2009 asphalt tonnage to be produced using WMA mixtures. The Alaska DOT bid a 25,000-ton warm-mix project on Mitkof Island (Walker 2009).

Despite the promising benefits, the industry and many DOTs have been concerned about putting WMA techniques into actual practice. Moisture susceptibility has been a primary concern for some WMA approaches. This is because lower temperatures in the process of mixing and compaction could result in incomplete drying of the aggregate, compromising the bond between asphalt and aggregate.

The Nebraska Department of Roads (NDOR) has been interested in this new WMA technology. NDOR initiated the WMA field trial in 2007 using different amounts of a wax-type WMA additive, Sasobit. In 2008, NDOR paved four trial sections, installing two WMA pavements (Evotherm WMA and Advera zeolite WMA) and their control HMA sections in Antelope County, Nebraska. The trial sections started from Elgin and ended at US Highway 20 (as shown in figure 1.1, from A to B). Figure 1.2 illustrates the layout of the trial sections.

Figure 1.1 Trial Sections from Elgin (A) to US Highway 20 (B)

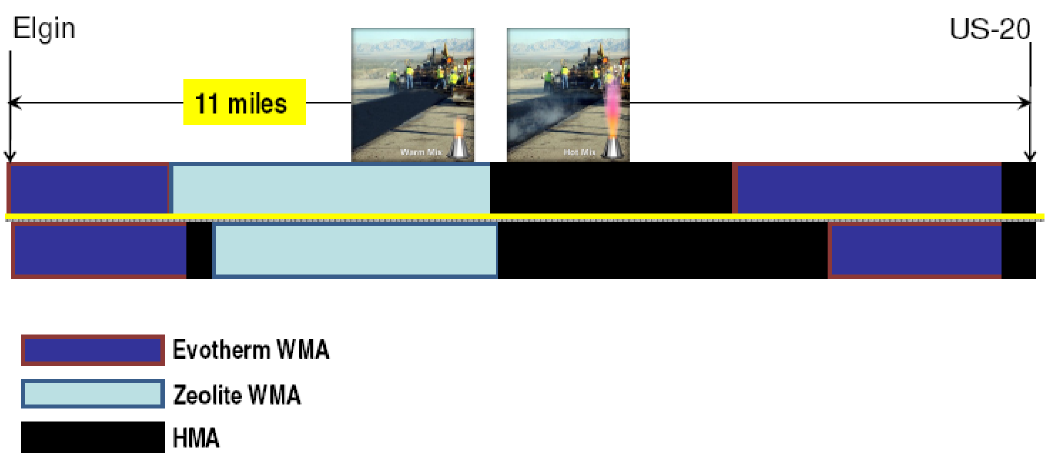


Figure 1.2 Layout of the Trial Sections

As presented in figure 1.3, field-mixed loose mixtures were collected and transported to the NDOR and UNL laboratories for comprehensive evaluations of the WMA mixtures compared to their control HMA mixtures through various experimental tests and performance prediction simulations. This research evaluates the performance of several different WMA mixtures, comparing them to their HMA counterparts, to discover the feasibility of using the energy-efficient, environmentally friendly WMA mixtures in future Nebraska asphalt pavements.

Figure 1.3 Field-mixed Loose Mixtures Delivered to the NDOR and UNL Laboratories

1.1 Research Objectives

The primary objective of this research is to evaluate the feasibility of several WMA mixtures as potential asphalt paving mixtures for Nebraska pavements. To that end, three well-known WMA additives (i.e., Sasobit, Evotherm, and synthetic zeolite named Advera WMA) were selected and used in actual pavement sections to monitor field performance. In addition, various key laboratory tests to identify mixture properties and performance characteristics were conducted to compare the WMA mixtures and their control HMA mixtures. Laboratory test results were then incorporated with other available data (i.e., materials data, mixture design results, pavement structural information, and traffic/climatic information of the trial sections) to further evaluate the effects of WMA with different additives by using the Mechanistic-Empirical Pavement Design Guide (MEPDG).

1.2 Research Scope

To meet the objectives of this research, four tasks were completed. Task 1 was to survey published literature regarding implementation and practice of the WMA technique. This extensive literature review includes regional (e.g., state DOTs' research reports) and national studies (such as research progress from NCHRP project 09-43) in the United States, as well as other available reports and articles from European countries. Task 2 was to fabricate specimens and to perform various laboratory tests: a dynamic modulus test (AASHTO TP62), creep compliance test (AASHTO T322), uniaxial static creep test (NCHRP 9-19), asphalt pavement analyzer (APA) test (NCHRP 9-17), tensile strength ratio (TSR) test (AASHTO T283), and fracture test with moisture conditioning, etc. Task 3 was to analyze laboratory test results and to use the test data for predicting long-term pavement performance based on MEPDG simulations. Pavement performance predictions made by the MEPDG were then compared to actual field performance data annually monitored by the NDOR pavement management team. Task 4 is to prepare presentations and generate a final report that includes research findings, conclusions, and NDOR implementation plans.

1.3 Organization of the Report

This report is composed of five chapters. Chapter 1 is the introduction. Chapter 2 presents background information associated with WMA benefits and approaches. Chapter 3 presents the research methodology employed in this study. Chapter 4 presents laboratory tests, MEPDG predictions of pavement performance, and actual field performance data. Chapter 5 provides a summary of findings and conclusions of this study. Future implementation plans for NDOR are also presented in the chapter.

Chapter 2 Background

2.1 Benefits of Warm-Mix Asphalt

Warm-mix asphalt presents various benefits. These benefits depend upon which WMA approaches are used in the asphalt production. Different WMA approaches have their respective advantages and potential concerns. The benefits are categorized generally as:

- Environmental,
- Paving, and
- Economic.

2.1.1 Environmental Benefits

Emissions from HMA are an issue for the environment and workers during the production and compaction of asphalt mixtures. The particulate matter (PM) and a variety of gaseous pollutants are emitted from HMA plants. The gaseous emissions include sulfur dioxide, nitrogen oxides, carbon monoxide, and volatile organic compounds. The Environmental Protection Agency (EPA) has offered an example to illustrate the emissions estimates. If a natural gas-fired drum mixing dryer produced 200,000 tons per year, the estimated emissions during that period would be 13 tons of carbon monoxide, 5 tons of volatile organic compounds, 2.9 tons of nitrogen oxides, 0.4 tons of sulfur oxides, and 0.65 tons of hazardous air pollutants (U.S. EPA Report 2000).

One of the main benefits of WMA is significant emission reduction during the mixing and compacting. Mallick et al. (2009) evaluated the effects of the WMA additive Sasobit, asphalt content, and construction temperature on carbon dioxide emissions. They concluded that temperature seemed to be the key factor influencing carbon dioxide emissions. Hence, lowering the asphalt mixing temperature is the most effective way to reduce carbon dioxide emissions during asphalt production and pavement construction.

Gandhi (2008) provided one example of emission reduction using measurements taken at WMA field demonstration projects. Table 2.1 shows the percentage reduction in emissions during construction with WMA, compared to conventional HMA projects. As can be seen in the table, emissions from WMA are significantly reduced, compared with those from HMA.

Table 2.1 Emission Reduction Measured from WMA Projects

	Aspha-min	**Sasobit**	**Evotherm**	**WAM-foam**
Sulfur Dioxide	17.60%	-	81%	N/A
Carbon Dioxide	3.20%	18%	46%	31%
Carbon Monoxide	N/A	N/A	63%	29%
Nitrogen Oxides	6.10%	34%	58%	62%
Total Particulate Matter	35.30%	N/A	N/A	N/A
Volatile Organic Compounds	N/A	8%	25%	N/A

Source: Gandhi (2008).

Shell Global Solutions and KoLo Veidekke studied warm asphalt mixture production using WAM-foam. They measured and compared emissions from WMA and HMA. Asphalt fumes are partly inorganic and partly organic. Fume emissions, both inorganic and organic, were categorized as total particulate matter (TPM). The organic part, benzene soluble matter (BSM), was also categorized. Bitumen combustion fumes contain traces of polycyclic aromatic compounds (PACs), which are suspected to have carcinogenic properties. Occupational exposure to bitumen combustion fumes is undesirable and should be kept as low as practicable. Table 2.2 shows emissions from WMA and HMA. The WMA is produced using the WAM-foam process at a mixing temperature of 115°C while HMA is produced at a mixing temperature of 165°C.

Table 2.2 Comparison of Emissions from HMA and WMA

	BSM emissions (*mg/m*^{3})	**PACs emissions** (*ng/m*^{3})	**TPM emissions** (*mg/m*^{3})
HMA	0.17-0.49	38-119	1.2-0.93
WMA	0.05	4.9-2.5	0.09

Emissions, especially carbon dioxide, are significantly reduced because of WMA's low production and compaction temperatures. Typical expected reductions for carbon dioxide and sulfur dioxide are 30% to 40%. They are 50% for volatile organic compounds, 10% to 30% for carbon monoxide, 60% to 70% for nitrogen oxides, and 20% to 25% for dust (D'Angelo et al. 2008). Consequently, WMA can provide paving workers with a better working environment by reducing their exposure to the toxic emissions. The asphalt aerosols/fumes and polycyclic aromatic hydrocarbons from WMA could be reduced by 30% to 50%, compared to those from HMA (D'Angelo et al. 2008).

Hassan (2009) stated that the use of WMA has three kinds of significance: air pollution, fossil fuel depletion, and smog formation. Based on the analysis conducted, Hassan concluded that WMA could cause a reduction of 24% in the air pollution impact of HMA, and a reduction of 18% in fossil fuel depletion. It also can reduce smog formation by 10%. Hassan estimated that the use of WMA could provide a reduction of 15% in the environmental impacts induced by HMA.

2.1.2 Paving Benefit

The mechanism that allows WMA to be produced at lower temperatures than conventional HMA is the WMA techniques that reduce the viscosity of the binder. The reduction of binder viscosity allows the aggregate to be well coated at temperatures lower than those used for HMA.

WMA can improve mixture compactibility in both the Superpave gyratory compactor and the vibratory compactor. The National Center for Asphalt Technology (NCAT) evaluated three WMA approaches (Hurley and Prowell 2005, 2006a, 2006b): Evotherm, Aspha-min, and Sasobit. In the report, the WMA mixtures were compacted at a temperature of 88°C using a vibratory compactor. The statistical results were that the average reduction in air voids was up to 0.65% for Aspha-min, up to 1.4% for Evotherm, and up to 0.87% for Sasobit.

WMA can allow incorporation of high percentages of RAP mixtures. Mogawer et al. (2009) studied the effects of incorporating a high percentage of RAP materials and WMA mixtures into thin HMA overlays. They stated that when incorporating a high percentage of RAP materials, most mixtures could be designed to meet specification requirements for volumetrics and gradation. However, mixture stiffness characteristics represented by the dynamic modulus master curve could be a problem because the added virgin binder could blend with the aged binder in the RAP. The higher RAP content decreased the workability of the mixture; therefore, a higher percentage of RAP may necessitate increasing the dose of WMA additives.

Another paving benefit from WMA is that it can extend the paving window, since it allows paving at cooler temperatures. Subsequently, the WMA allows mixtures to be hauled for greater distances and to still provide fine workability.

2.1.3 Economic Benefit

WMA can usually lower asphalt-mixing temperatures by 15°C to 30°C compared to conventional HMA. This could reduce burner fuel costs by 20% to 35%. Fuel savings could be 50% or more when producing low-energy asphalt concrete and low-energy asphalt in which the aggregate is not heated above the boiling point of water. However, additional costs could be necessary for equipment and additives (D’Angelo et al. 2008).

2.2 Warm-Mix Asphalt Approaches

Depending on the production temperature, the asphalt mixtures are classified as follows: cold (0–30 °C), half-warm (65–100 °C), warm (110–140 °C), and hot (140–180 °C). Figure 2.1 illustrates the classification of different mixtures by production temperature (D’Angelo et al. 2008).

There are three primary ways to produce WMA by introducing WMA additives: foaming techniques, organic or wax additives, and chemical additives. The three primary WMA technologies have been traditionally developed and used in European countries and recently in the United States. In this section, the three typical WMA approaches, synthetic zeolite (forming technique), Sasobit (organic or wax additive), and Evotherm (chemical additive) are introduced with some background detail, since they are to be evaluated in this research.

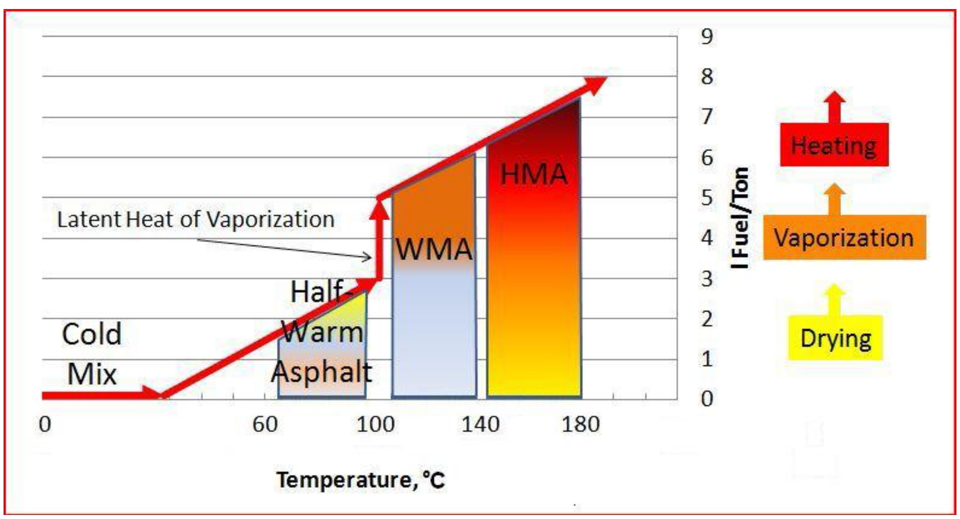


Figure 2.1 Classification of Asphalt Mixtures by Temperature (D’Angelo et al. 2008)

One well-known forming technique involves the addition of a synthetic zeolite called Aspha-min to create a foaming effect in the binder during mixing in the plant. Aspha-min is a product from Eurovia Services GmbH (Bottrop, Germany). It is a manufactured synthetic zeolite (sodium aluminum silicate) in which 21% by mass of zeolite is crystallized with water held internally. Typically, the addition of Aspha-min in the amount of 0.3% by mass of the mixture is recommended. When zeolite is added at the same time as the binder, crystallized water is released, which creates a foaming effect that leads to a slight increase in binder volume and reduces the binder's viscosity (D'Angelo et al. 2008).

Advera WMA, a manufactured synthetic zeolite, is a product of the PQ Corporation (Malvern, PA). From 18% to 21% of its mass is water held in its crystalline structure, which is released at temperatures above 100 °C to create a foaming of the binder in the mixture. It can lead to production and mixing temperatures 30–40 °C lower than those needed for conventional HMA. Addition of Advera WMA to the mixture in the proportion of 0.25% by weight is usually recommended.

Another type of formed WMA technique, WAM-foam, divides the binder into two separate components, a soft binder and a hard binder in foam form. There are two stages for mixing the binder and aggregate. In the first stage, the soft binder is mixed with the aggregate at about 110°C to coat the aggregate. In the second stage, the hard binder, in foam form, is mixed into the pre-coated aggregate. By injecting cold water into the heated hard binder, the rapid evaporation of water produces a large volume of foam. Shell reports that WMA-foam can save 30% of plant fuel, with a corresponding reduction in carbon dioxide emissions.

Sasobit is a kind of long-chain aliphatic hydrocarbon wax. Its melting point is 98°C, and it has the ability to lower the viscosity of the asphalt binder. The benefit of decreasing the viscosity of the binder is to allow working temperatures to be reduced by 15–55°C. It has high viscosity at lower temperatures and low viscosity at high temperatures. At temperatures below its melting point, Sasobit forms a crystalline network structure in the binder that leads to added stability (D'Angelo et al. 2008).

Evotherm was developed in the United States. During production, the asphalt emulsion with the Evotherm chemical package is mixed with aggregate in the HMA plant. An emulsion is mixed with hot aggregate to produce a resulting mixture temperature between 85°C and 116°C. The majority of the water in the emulsion flashes off as steam when the emulsion is mixed with the aggregate (D'Angelo et al. 2008). MeadWestvaco reports that this emulsion can improve compactibility, workability, and aggregate coating without requiring changes in the materials' mixture formula.

2.3 Performance of Warm-Mix Asphalt

2.3.1 Evaluation of synthetic zeolite for use in warm-mix asphalt

Aspha-min is a synthetic zeolite based on a foaming technique that reduces the viscosity of the binder. An NCAT report (Hurley and Prowell 2005) stated that the addition of Aspha-min lowered the air voids measured in the gyratory compactor. It can improve the compactibility of both the Superpave gyratory compactor and a vibratory compactor. Statistical analyses of test results indicated an average reduction in air voids of 0.65% using the vibratory compactor. Wielinski et al. (2009) conducted a study based on laboratory tests and field evaluations of foamed WMA projects. They found that the Hveem and Marshall properties of HMA and WMA were similar, and all met the Hveem design requirements and the mixture property requirements. The in-place densities were also very similar.

Hurley and Prowell (2005) reported that the addition of the Aspha-min synthetic zeolite did not significantly affect the resilient modulus of asphalt mixtures. Goh et al. (2007) evaluated the performance of WMA with the addition of Aspha-min based on the Mechanistic-Empirical

Pavement Design Guide (MEPDG). They found that the addition of Aspha-min did not affect the dynamic modulus values for any of the asphalt mixtures examined.

The lower compaction temperature used when producing warm asphalt with the addition of Aspha-min may increase the potential for moisture damage. Lower mixing and compaction temperatures can result in incomplete drying of the aggregate. The resulting water trapped in the coated aggregate may cause moisture damage. Hydrated lime seems to be effective with the granite aggregate. The addition of 1.5% hydrated lime has resulted in acceptable performance, in terms of both cohesion and moisture resistance, which was better than the performance of warm mixtures without hydrated lime (Hurley and Prowell (2006).

The addition of synthetic zeolite did not increase the rutting potential of asphalt mixtures. The rutting potential increased with decreasing mixing and compaction temperatures, which may be related to the decreased aging of the binder. Goh et al. (2007) evaluated the performance of WMA after the addition of Aspha-min, based on the MEPDG. The predicted rut depths from the MEPDG simulations demonstrated that WMA could decrease rutting, and the greatest difference of rutting between WMA and its control could be up to 44%. Hodo et al. (2009) stated that the foamed asphalt mixtures presented good workability at lower temperatures, a result that implied greater ease in placing and compacting the mixtures. The moisture susceptibility tests showed marginal results, and the authors suggested that if anti-stripping agents were added to the mixture, the moisture damage resistance would be improved.

WMA with the addition of Aspha-min synthetic zeolite successfully incorporates with a higher percentage of RAP materials than HMA does. Aspha-min was added to a Superpave mixture containing 20% RAP during a demonstration project in Orlando, Florida. The addition was able to reduce the production and compaction temperatures by 20°C, while yielding the same in-place density (Hurley and Prowell 2005).

2.3.2 Evaluation of Evotherm For Use In Warm-Mix Asphalt

Evotherm is a chemical additive used to produce WMA. Evotherm uses a chemical package of emulsification agents to enhance aggregate coating, mixture workability, and compaction capability. The majority of the water in the emulsion flashes off when mixed with hot aggregate.

A laboratory study was conducted by Hurley and Prowell (2006a) to evaluate the effects of Evotherm on pavement performance. The laboratory study used two aggregate types (limestone and granite) and two PG binders (PG 64-22 and PG 76-22). Test results indicated that the addition of Evotherm lowered the measured air voids in the gyratory compactor for the given asphalt content. Evotherm improved the compactibility of the mixtures. The air voids of mixtures were reduced by 1.4%. Due to the enhanced compactibility, compaction temperatures could be brought down to 88°C. The study also found that the addition of Evotherm increased the resilient modulus of asphalt mixtures, compared to control mixtures with the same PG binder, and could consequently decrease the rutting potential, compared to control mixtures produced at the same temperature.

However, the lower compaction temperature used when producing warm asphalt by the addition of Evotherm may increase the potential of moisture damage. Lower mixing and compaction temperatures can result in incomplete drying of the aggregate. The resulting water trapped in the coated aggregate may cause moisture damage. Although there is no definite trend indicating the potential moisture damage of WMA with Evotherm in the study (Hurley and

Prowell 2006a), in some cases the TSR value from AASHTO T283 testing presented some concerns with the WMA, compared to the control HMA mixtures.

2.3.3 Evaluation of Sasobit for Use in Warm-Mix Asphalt

Sasobit is an organic or wax additive. It is an aliphatic hydrocarbon produced from coal gasification, which is completely soluble in asphalt binders at temperatures higher than 120 °C. It has the ability to reduce the viscosity of asphalt binders. At temperatures below its melting point, Sasobit can form a crystalline network structure that can stabilize the binder.

Hurley and Prowell (2006b) evaluated the effects of Sasobit on pavement performance. The laboratory study used two aggregates (limestone and granite) and two binders (PG 64-22 and PG 58-28). When adding Sasobit or Sasoflex to the two binders, three modified binders formed. The original PG 58-28 binder became, with the addition of 2.5% of Sasobit, PG 64-22. The same PG 58-28 binder became PG 70-22 after the addition of 4.0% of Sasoflex. Finally, the original PG 64-22 binder, with the addition of 4.0% of Sasoflex, became PG 76-22. The study also concluded that the addition of Sasobit lowered the measured air voids in the gyratory compactor and consequently improved the compactibility of mixtures. Mixture stiffness characteristics represented by a resilient modulus were not dramatically affected by the addition of Sasobit. However, the addition of Sasobit generally decreased the rutting potential of the asphalt mixtures, which seemed to be because of the stabilizing effect in the binder from Sasobit's forming a crystalline network structure.

Diefenderfer and Hearon (2008) studied Sasobit warm-mix materials. The authors compared laboratory test results with trial sections implemented in Virginia. They concluded that the HMA and WMA sites evaluated in their study performed similarly for the first two years of service. The performance of the WMA and HMA sections was similar with respect to moisture susceptibility, rutting potential, and fatigue resistance. In addition, they used MEPDG software to predict the distresses and long-term performance of the trial sections.

Mallick et al. (2008) evaluated the effects of Sasobit on asphalt mixtures into which is incorporated a high percentage of RAP material. He concluded that the addition of Sasobit helped to lower the viscosity of the asphalt binder at higher temperatures. With that, it was possible to produce asphalt mixtures with 75% RAP with similar air voids as compared to virgin mixtures, even at lower temperatures, by using Sasobit at a rate of 1.5% of the total weight of the asphalt binder.

Mogawer et al. (2009) evaluated the effects of adding varying dosages of Sasobit on the performance of mixtures containing RAP. The authors noted that the addition of 1.5% Sasobit changed the PG grade of the base binder from PG 64-28 to PG 70-22, and that the addition of 3.0% Sasobit changed the binder grade to PG 70-16. Laboratory testing also showed that Sasobit additives at different dosages could improve the workability of mixtures containing 25% RAP. Durability testing indicated that the control mixtures exhibited better moisture resistance than the mixtures containing WMA additives.

Chapter 3 Research Methodology

As mentioned above, NDOR initiated the WMA field trial in 2007 using different amounts of a wax-type additive, Sasobit. In 2008, NDOR paved two WMA trial sections and their control HMA sections in Antelope County, Nebraska. Two different WMA additives, Evotherm and Advera WMA synthetic zeolite, were used. The trial sections are a total of 11 miles long, connecting Elgin to US Highway 20. At the time of paving construction, field-mixed loose mixtures were collected and transported to the NDOR and UNL asphalt laboratories to conduct various laboratory tests. This chapter describes the research methodology employed in this study. Materials involved in this research, corresponding asphalt mixtures, laboratory tests performed, and pavement performance evaluations by MEPDG simulations and actual field monitoring are presented. For the following discussion, the WMA mixtures with the addition of Evotherm, zeolite, and Sasobit are denoted as WMA-Evo, WMA-Zeo, and WMA-Sas, respectively. The control HMA mixtures to each WMA mixture are denoted as HMA-Evo, HMA-Zeo, and HMA-Sas, respectively.

Table 3.1 presents each laboratory test conducted in this study, listing its standard method and purpose. Various laboratory tests were conducted to estimate the effects of warm-mix additives on mixture characteristics and pavement performance.

Table 3.1 Laboratory Tests Performed in This Research

Test	Standard	Purpose
Mixture Design	AASHTO T312	Volumetric Characteristics
Binder Tests	AASHTO T313, T315	Properties and Grade
Dynamic Modulus	AASHTO TP62	Viscoelastic Stiffness
Creep Compliance	AASHTO T322	Thermal Cracking
Flow Time	NCHRP 9-19	Rutting
APA	NCHRP 9-17	Rutting with Moisture Damage
TSR	AASHTO T283	Strength with Moisture Damage
SCB Fracture	N/A	Fracture with Moisture Damage

Two typical binder tests (the dynamic shear rheometer [DSR] test and bending beam rheometer [BBR] test) were conducted in this research to investigate the performance grade and viscoelastic properties of binders with and without warm-mix additives. The dynamic modulus test and the creep compliance test were conducted to evaluate the mixture stiffness and thermal cracking properties. Then, the uniaxial static creep test (i.e., flow time test) was performed to investigate the mixtures' rutting resistance. The tensile strength ratio (TSR), the asphalt pavement analyzer (APA) test, and the semi-circular bending (SCB) fracture test were included in this study to evaluate the moisture sensitivity of each mixture.

The binder properties, dynamic modulus, and creep compliance of mixtures were then incorporated with other available data (i.e., materials data, mixture design results, pavement structural information, and traffic/climatic information) to predict the performance of WMA and HMA pavement sections using the Mechanistic-Empirical Pavement Design Guide (MEPDG). Finally, field performance data (i.e., rut depth, cracking, and the international roughness index [IRI]) were monitored for two years (2008 to 2010) and were compared to the MEPDG prediction results. Figure 3.1 presents the research methodology employed for this study.

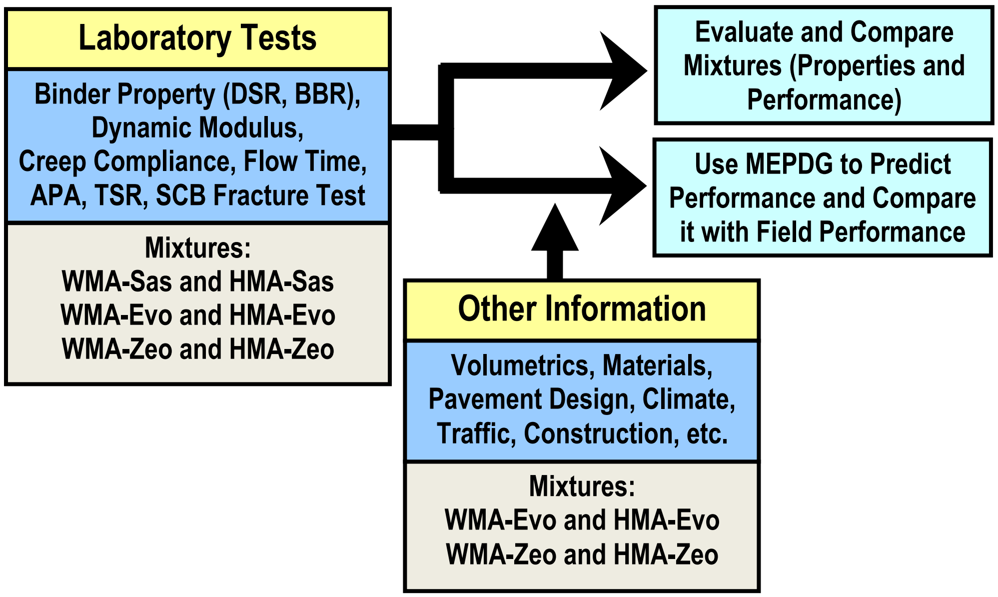


Figure 3.1 Research Methodology Employed for This Study

3.1 Materials Selection

In this project, the most widely used local aggregates and an asphalt binder were selected for the mixture design. The new pavement used 10 to 15% of millings from old pavements. In addition, three WMA additives (Evotherm, Advera WMA synthetic zeolite, and Sasobit) were used to produce WMA mixtures.

3.1.1 Aggregates

A total of three types of local aggregates (5/8-inch and 1/4-inch limestone, 2A gravel, and CR gravel) were used in this study. These aggregates were those most widely used by Nebraska contractors. Tables 3.2, 3.3, 3.4, and 3.5 illustrate gradation and consensus properties (i.e., FAA, CAA, sand equivalent, and G_{sb}) of the aggregates used in this project.

Table 3.2 Gradation of Aggregates Used in WMA-Evo and HMA-Evo

Combination of Materials		**Sieve Analysis (Wash)**								
Aggregate Sources	**%**	**3/4”**	**1/2”**	**3/8”**	**#4**	**#8**	**#16**	**#30**	**#50**	**#200**
5/8” ×1/4” Limestone	11	100	74	44	4.9	1.7	1.3	1.2	1.1	0.9
2A Gravel	9	100	99	94	78	25	10	6.4	4.3	1.5
CR Gravel	65	100	100	95	92	66	43	28	17	7
Millings	15	100	98	97	92	76	59	44	31	13
Combined Gradation	100	100	96.8	89.6	81.2	56.7	37.8	25.5	16.2	6.7

Table 3.3 Consensus Properties of Aggregates Used in WMA-Evo and HMA-Evo

FAA (%)	**CAA** (%)	**Sand Equivalent** (%)	**Design G_{sb}**
45.1	91/90	75	2.571

Table 3.4 Gradation of Aggregates Used in WMA-Zeo and HMA-Zeo

Combination of Materials		**Sieve Analysis (Wash)**								
Aggregate Sources	**%**	**3/4”**	**1/2”**	**3/8”**	**#4**	**#8**	**#16**	**#30**	**#50**	**#200**
5/8”x #4 Limestone	10	100	74	44	4.9	1.7	1.3	1.2	1.1	0.9
2A Gravel	5	100	99	94	78	25	10	6.4	4.3	1.5
CR Gravel	75	100	100	95	92	66	43	28	17	7
Millings	10	100	99	97	88	67	50	38	23	6.4
Combined Gradation	100	100	97.3	90.1	82.2	57.6	37.9	25.2	15.4	6.1

Table 3.5 Consensus Properties of Aggregates Used in WMA-Zeo and HMA-Zeo

FAA (%)	**CAA** (%)	**Sand Equivalent** (%)	**Design G_{sb}**
45.2	85/82	80	2.576

3.1.2 Asphalt Binder

The asphalt binder used in this project is a Superpave performance-graded binder, PG 64-28, provided by Jebro Inc., located in Sioux City, Iowa. This type of binder has been used primarily for low to intermediate traffic volume roads in Nebraska. Table 3.6 presents the fundamental properties of the binder determined by performing dynamic shear rheometer (DSR) tests and bending beam rheometer (BBR) tests, which have been designated in the Superpave binder specifications to identify the performance grade and basic viscoelastic properties of asphalt binders.

Table 3.6 Properties of Original Asphalt Binder, PG 64-28

Test	Temperature (°C)	Test Result	Required Value
Unaged DSR, $\|G^*\|/\sin\delta$ (kPa)	64	1.486	min. 1.00
Unaged phase angle (degree)	64	75.74	-
RTFO - Aged DSR $\|G^*\|/\sin\delta$ (kPa)	64	3.698	min. 2.20
PAV - Aged DSR, $\|G^*\|\sin\delta$ (kPa)	19	3391	max. 5,000
PAV - Aged BBR, stiffness (MPa)	-18	239	max. 300
PAV - Aged BBR, *m*-value	-18	0.299	min. 0.30

3.1.3 Advera WMA (synthetic zeolite)

Advera WMA (PQ Corporation, Malvern, Pennsylvania) is an additive used in a foaming technique for producing WMA mixtures. It is a manufactured synthetic zeolite. Figure 3.2 shows its microstructure. It holds about 20% water within its crystalline form, which is released at temperatures above 100°C. The water released can create foam to reduce the viscosity of the binder. The gradual release of water can provide about a 7-hour period of improved workability. It can lead to production and mixing temperatures 30–40°C lower than those of conventional HMA. The addition of Advera to the mixture is recommended in the proportion of 0.25% by weight.

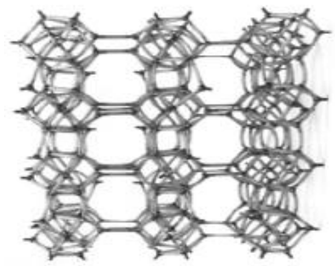


Figure 3.2 Microstructure of Advera WMA (Synthetic Zeolite)

3.1.4 Evotherm

Evotherm has been developed in the United States and is produced by Meadwestvaco Corporation (Richmond, Pennsylvania). Evotherm is a chemical additive used to produce WMA. It uses a chemical package of emulsification agents to enhance aggregate coating, mixture workability, and compactibility. The majority of water in the emulsion flashes off when mixing with hot aggregate.

3.1.5 Sasobit

Sasobit is one of the organic or wax additives, produced by Sasol Wax. It is an aliphatic hydrocarbon produced from coal gasification, which is completely soluble in asphalt binder at temperatures higher than 98°C. It has the ability to reduce the viscosity of the asphalt binder. This can reduce working temperature by 15–55°C. At temperatures below its melting point, Sasobit can form a crystalline network structure that can stabilize the binder. Figure 3.3 shows Sasobit granules.

Figure 3.3 Sasobit Granules

3.2 Mixture Design Method

The Superpave method of mixture design for a 12.5-mm mixture was used in this study. All the mixtures for this project were SP4 mixtures, which are used mostly for intermediate-volume traffic pavements. The compaction effort used for the SP4 mixture was for a traffic volume around 3.0 to 10.0 million equivalent single axle loads (ESALs). Table 3.7 summarizes the NDOR specification requirements for aggregate properties, volumetric mixture design parameters, and laboratory compaction level for the SP4 mixture.

Table 3.7 Required NDOR Specifications for SP4 Mixture

	NDOR Specification (SP4 Mixture)
Compaction Level	
N_{ini}: the number of gyration at initial	8
N_{des}: the number of gyration at design	96
N_{max}: the number of gyration at maximum	152
Aggregate Properties	
CAA (%): coarse aggregate angularity	> 85/80
FAA (%): fine aggregate angularity	> 45
SE (%): sand equivalency	> 45
F&E (%): flat and elongated aggregates	< 10
Volumetric Parameters	
%V_a: air voids	4 ± 1
%VMA: voids in mineral aggregates	> 14
%VFA: voids filled with asphalt	65 - 75
%P_b: asphalt content	-
D/B: dust to binder ratio	0.7 - 1.7
%RAP: reclaimed asphalt pavement material	< 15

All WMA mixtures were produced at around 135°C, while their corresponding HMA control mixtures were mixed at around 165°C, as shown in figure 3.4. Then, the WMA mixtures were compacted at around 124°C while HMA mixtures were compacted at around 135°C in the field.

Figure 3.4 WMA and HMA Production Temperatures

<u>3.3 Laboratory Tests And Evaluation</u>

3.3.1 Binder Tests

There were six mixtures, and each mixture used the same Superpave performance-graded binder, PG 64-28, which has been used for the SP4 mixture in Nebraska. Binders were extracted from the field-mixed loose mixtures in the NDOR laboratory, and then the fundamental properties of the asphalt binder were evaluated through the dynamic shear rheometer (DSR) tests and the bending beam rheometer (BBR) tests. The complex shear modulus (G^*) and the phase angle (δ) of the binders were obtained using the DSR. The stiffness and m-value of the binder at low temperatures was obtained through the BBR tests. Based on test results, the performance grade and viscoelastic properties of asphalt binder in each mixture could be identified.

3.3.2 Dynamic Modulus Test (AASHTO TP62)

The dynamic modulus test is a linear viscoelastic test for asphalt concrete. The dynamic modulus is an important input when evaluating pavement performance related to the temperature and speed of traffic loading. The loading level for the testing was carefully adjusted until the specimen deformation was between 50 and 75 microstrain, which was considered not to cause nonlinear damage in the specimen, so that the dynamic modulus can represent the intact stiffness of the asphalt concrete.

A Superpave gyratory compactor was used to produce cylindrical samples with a diameter of 150 mm and a height of 170 mm. Then, the samples were cored and cut to produce cylindrical specimens with a diameter of 100 mm and a height of 150 mm. Figure 3.5 demonstrates the specimen production process using the Superpave gyratory compactor, core, and saw machines, and the resulting cylindrical specimen used to conduct the dynamic modulus test.

Figure 3.5 Specimen Production Process for the Dynamic Modulus Testing

To measure the axial displacement of the specimens under static stress, mounting studs were glued to the surface of the specimen so that three linear variable differential transformers

(LVDTs) could be installed on the surface of the specimen through the studs at 120^{o} radial intervals with a 100-mm gauge length. Figure 3.6 illustrates the studs affixed to the surface of a specimen. Then, the specimen was mounted in the UTM-25kN equipment for testing, as shown in figure 3.7.

Figure 3.6 Studs Fixing on the Surface of a Cylindrical Specimen

Figure 3.7 A Specimen with LVDTs mounted in UTM-25kN Testing Station

Two replicas for each mixture were used to perform the dynamic modulus test. The test was conducted at five temperatures (−10 °C, 4.4 °C, 21.1 °C, 37.8 °C, and 54.4 °C). At each temperature, six frequencies (25 Hz, 10 Hz, 5 Hz, 1 Hz, 0.5 Hz, and 0.1 Hz) of load were applied to the specimens. The axial forces and vertical deformations were recorded by a data acquisition system and were converted to stresses and strains. The dynamic modulus was then calculated by dividing the maximum (peak-to-peak) stress by the recoverable (peak-to-peak) axial strain.

The dynamic modulus values for 30 temperature-frequency combinations were used to construct a master curve by the shifting process illustrated in figure 3.8 and figure 3.9. The master curve represents the stiffness of asphalt concrete over a wide range of loading frequencies.

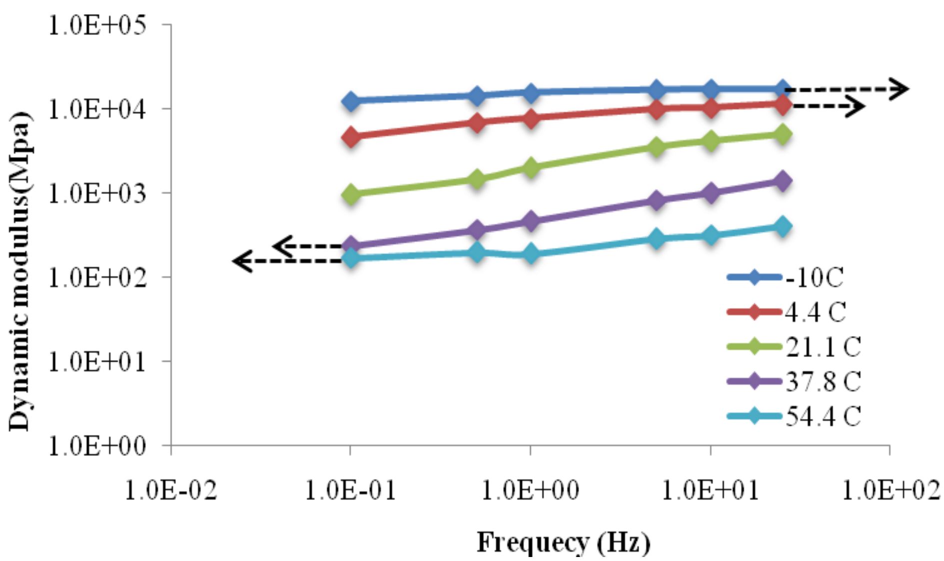


Figure 3.8 Dynamic Moduli at Different Temperatures and Loading Frequencies

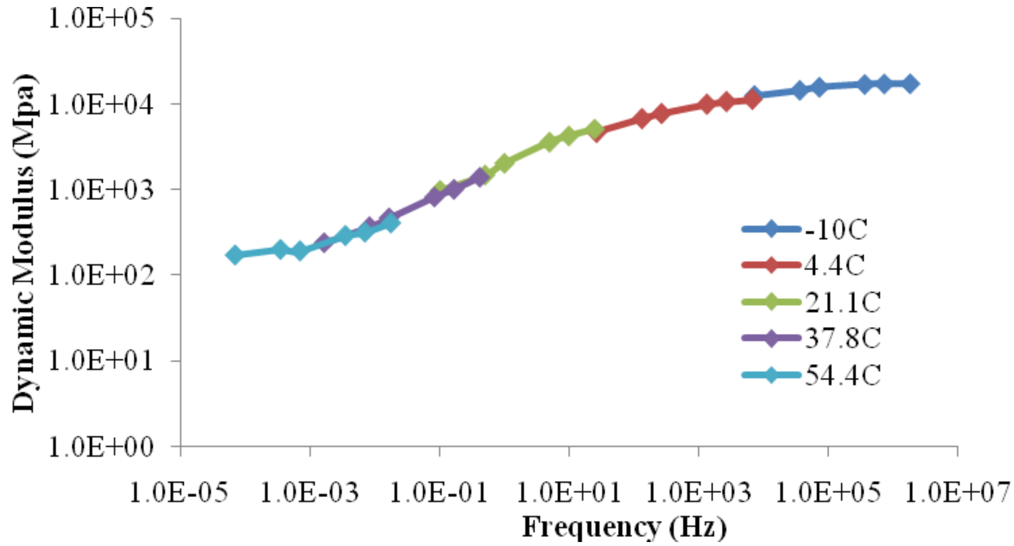


Figure 3.9 Dynamic Modulus Master Curve at 21.1 °C

3.3.3 Creep Compliance Test (AASHTO T322)

The creep compliance test is used to describe the low-temperature behavior of asphalt mixtures. It is the primary input for predicting thermal cracking in asphalt pavements over their service lives. This test procedure is described in AASHTO T322. The current standard method used in the United States to determine the creep compliance of asphalt mixtures is the indirect tensile (IDT) test. In this research, the creep compliance test was conducted at −10 °C.

Figure 3.10 shows the size of specimens used in the creep compliance test. A Superpave gyratory compactor was used to fabricate samples with a diameter of 150 mm and a height of 115 mm. Then, the samples were cut into specimens with a diameter of 150 mm and a thickness of 38 mm.

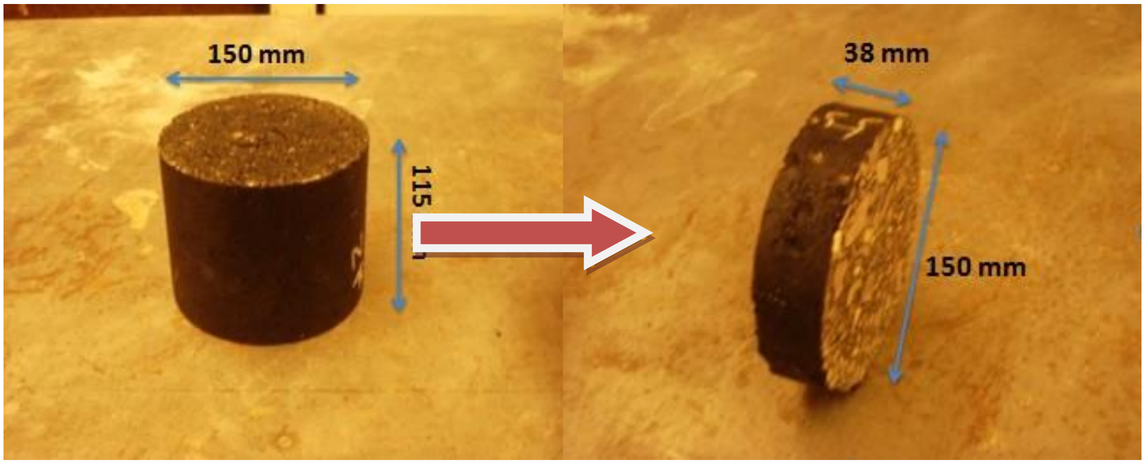


Figure 3.10 Specimen Preparation Process for Creep Compliance Test

On each flat face of the specimen, two studs were placed along the vertical and two along the horizontal axes with a center-to-center spacing of 38 mm so that four linear variable differential transformers (LVDTs) could be mounted on the surfaces of the specimens (shown in figure 3.11). The vertical and horizontal displacements were recorded using the four LVDTs during the test.

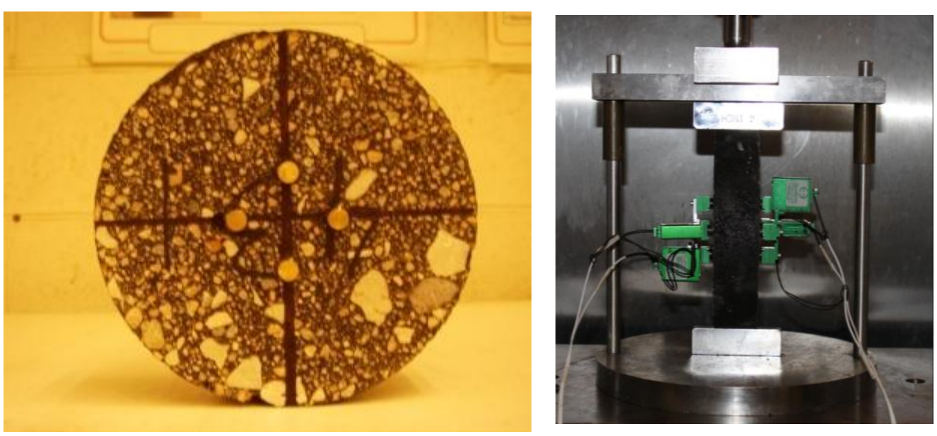


Figure 3.11 A Specimen with LVDTs Mounted in UTM-25kN Testing Station

3.3.4 Uniaxial static creep test (NCHRP 9-19)

The uniaxial static creep test (i.e., flow time test) is performed in unconfined conditions under static stress to assess the rutting resistance of mixtures. During this test, the cylindrical specimens were subjected to a static stress and the strain responses were recorded. The NCHRP report No. 465 (Witczak et al. 2002) describes the test procedure.

A Superpave gyratory compactor was used to produce the cylindrical samples with a diameter of 150 mm and a height of 170 mm. Then, the samples were cored and cut to produce cylindrical testing specimens with a diameter of 100 mm and a height of 150 mm. The specimens were identical to those used in the dynamic modulus test.

To measure the axial displacement of the specimens under static stress, mounting studs were glued onto the surface of the specimen so that three LVDTs could be installed on the surface of the specimen through the studs at 120^o radial intervals with a 100-mm gauge length. Then, the specimen was put in the UTM-25kN equipment for testing (as similar to the dynamic modulus test).

Two replicas for each mixture were used to perform the uniaxial static creep test at 60°C. A constant stress of 207 kPa was applied to the specimens. The vertical displacement was monitored with the three LVDTs. Figure 3.12 presents a typical plot of the log compliance versus log time results from the test. Three basic zones–primary, secondary, and tertiary–in a typical plot of log compliance versus log time have been identified:

1. The primary zone—the portion in which the deformation rate decreases with loading time;
2. The secondary zone—the portion in which the deformation rate is constant with loading time; and
3. The tertiary flow zone—the portion in which the deformation rate increases with loading time.

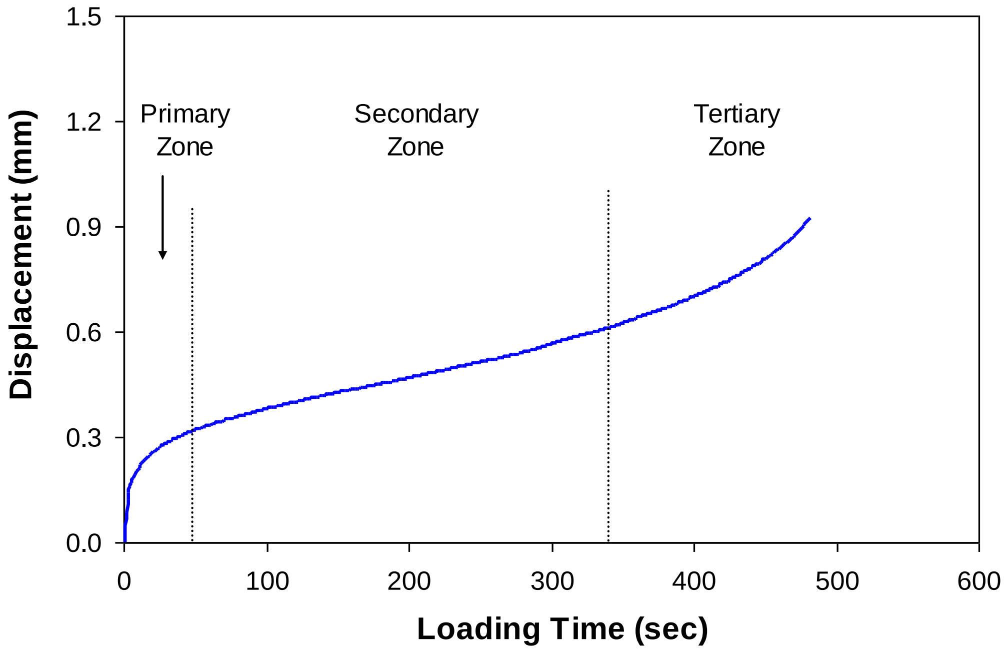


Figure 3.12 A Typical Data Plot of Uniaxial Static Creep Test (Flow Time Test)

The failure point due to plastic flow was determined at the stage of transition from secondary creep to tertiary creep. The starting point of the tertiary zone was defined as the flow time. This is considered a very good evaluation parameter of the rutting resistance of asphalt concrete mixtures (Hafez 1997).

3.3.5 Asphalt Pavement Analyzer (APA) Test under Water (NCHRP 9-17)

The rutting susceptibility and moisture resistance of asphalt concrete samples can be evaluated using the asphalt pavement analyzer (APA) shown in figure 3.13. The APA is an automated, new generation of the Georgia Loaded Wheel Tester (GLWT) used to evaluate the rutting, fatigue, and moisture resistance of asphalt concrete mixtures. During the APA test, the rutting susceptibility of compacted specimens was tested by applying repetitive linear loads through three pressurized hoses via wheels to simulate trafficking. Even though it has been reported that APA testing results are not very well matched with actual field performance, APA testing is relatively simple to do and produces the rutting potential of mixtures by simply measuring a sample rut depth. To evaluate moisture damage and susceptibility, asphalt concrete samples from each mixture are maintained under water at the desired temperature during the test, and submerged deformations are measured with an electronic dial indicator.

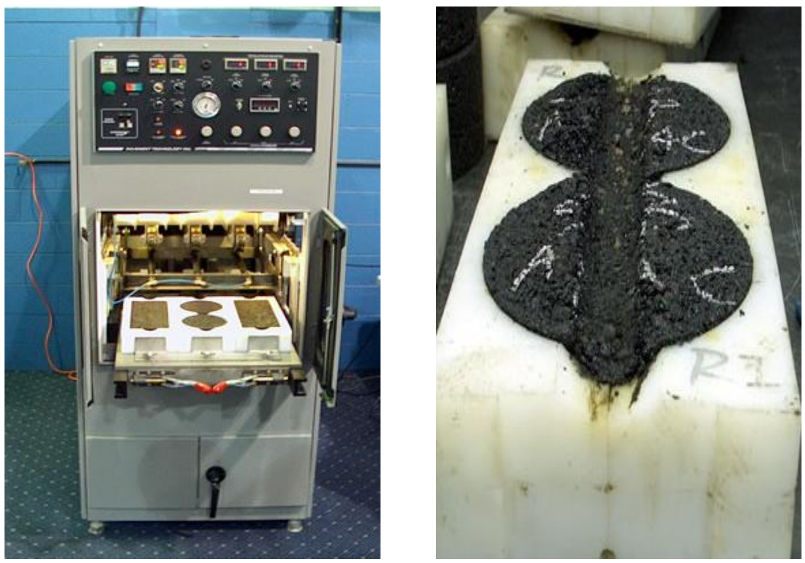


Figure 3.13 APA Test Station and Specimens after Testing

APA testing was conducted at the NDOR laboratory. The hose pressure and wheel load applied on the specimens were 690 kPa and 445 N, respectively. All tests were performed at 64°C. Specimens were submerged in water to induce moisture damage, and then cyclic loads were applied. The stop criterion was 8,000 cycles or 12-mm rut depth.

3.3.6 Tensile Strength Ratio (TSR) Test (AASHTO T283)

The evaluation of moisture sensitivity of asphalt concrete samples has been widely accomplished using a standard method, AASHTO T283. This test procedure was elaborated based on a study by Lottman (1978) and on work done by Tunnicliff and Root (1982). Studies by

McCann and Sebaaly (2003) and others have employed this technique for assessing the moisture sensitivity of various mixtures due to its simplicity, even if this laboratory evaluation has a relatively low correlation with actual field performance.

A Superpave gyratory compactor was used to produce test specimens with a diameter of 150 mm and a height of 95 ± 5 mm, and with 7% ± 0.5 air voids. Two subsets of specimens were fabricated and tested. One subset was tested under dry conditions for indirect-tensile strength. The other subset was subjected to vacuum saturation and a freeze cycle, followed by a warm-water soaking cycle, before being tested for indirect-tensile strength.

The unconditioned set of specimens was covered with plastic film and placed inside plastic bags. Then, the specimens were placed in a water bath at 25 ± 0.5 °C for two hours to control the specimens' temperature before testing. For the conditioned specimens, each specimen was subjected to partial vacuum saturation for a short period of time to reach its moisture saturation level of approximately 70% to 80%. Then, the partially saturated specimens were covered with plastic film and placed inside plastic bags. Next, specimens were moved into a freezer at a temperature of −18 ± 3 °C for 24 hours. After the freezing cycle, the specimens were moved to a water bath at 60 ± 1 °C for 24 hours. After the freeze-thaw cycle, the specimens were moved to a warm water bath of 25 ± 0.5 °C for two hours before testing.

All specimens were tested to determine their indirect tensile strengths. As demonstrated in figure 3.14, a compressive load was applied to a cylindrical specimen through two diametrically opposed rigid platens to induce tensile stress along the diametral vertical axis of the test specimen. A series of splitting tensile strength tests were performed at a constant strain rate of 50 mm/min. vertically until vertical cracks appeared and the sample failed. A peak compressive load was recorded and used to calculate the tensile strength of the specimen using the following Equation (3.1):

$$TS = \frac{2 \cdot P}{\pi \cdot t \cdot D} \tag{3.1}$$

where

TS = tensile strength (kPa),
P = peak compressive load (kN),
t = specimen thickness (m), and
D = specimen diameter (m).

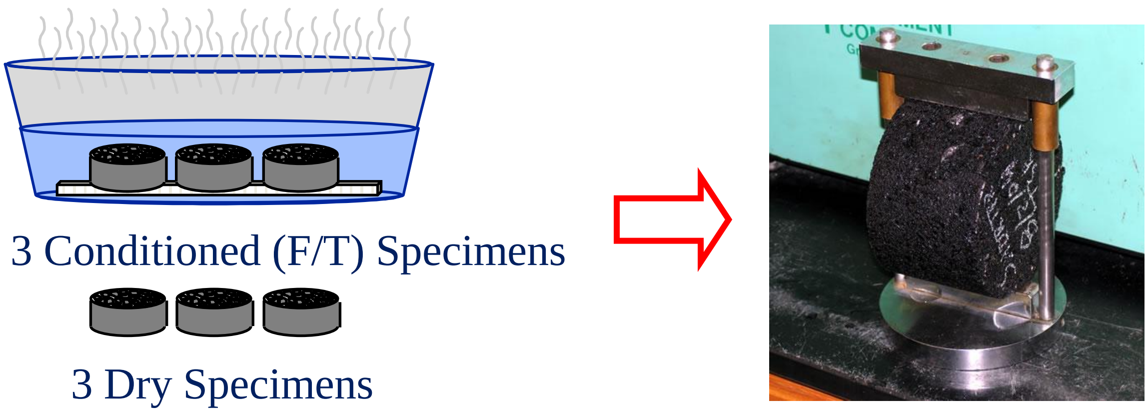


Figure 3.14 Schematic View of Tensile Strength Ratio Test (AASHTO T283)

The numerical index of the resistance of asphalt mixtures to moisture damage is expressed as the ratio of the average tensile strength of the conditioned specimens to the average tensile strength of the unconditioned specimens. Average tensile strength values of each mixture were used to calculate a tensile strength ratio (TSR), as follows:

$$TSR = \frac{TS_C}{TS_U} \tag{3.2}$$

where

TS_C = average tensile strength of the conditioned subset, and
TS_U = average tensile strength of the unconditioned subset.

3.3.7 Fracture Test With Moisture Damage

To further evaluate the moisture sensitivity of WMA, a semi-circular bend (SCB) fracture test was performed with laboratory compacted specimens. For the SCB fracture tests, specimens were subjected to a simple three-point bending configuration, as presented in figure 3.15.

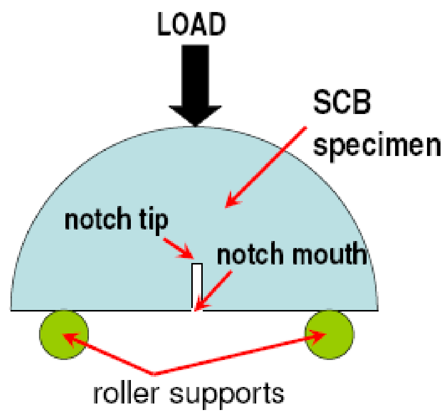
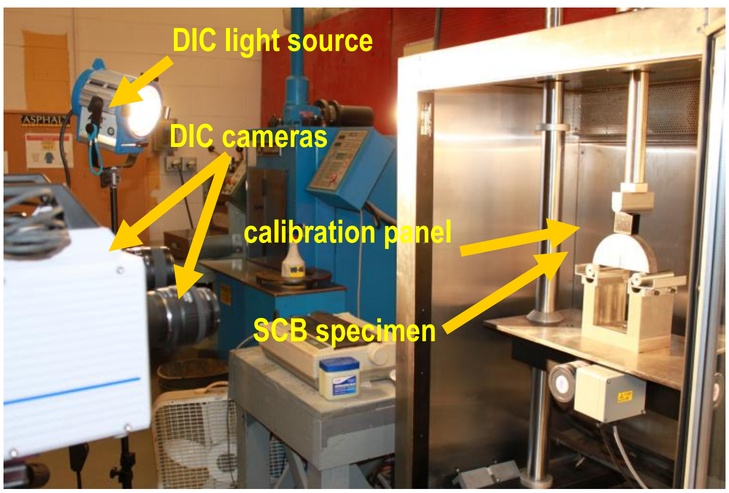


Figure 3.15 SCB Fracture Testing Configuration

The SCB test was originally proposed by Chong and Kurrupu (1984, 1988). The SCB specimen has since been used by many researchers (Lim et al. 1994; Adamson et al. 1996; Molenaar et al. 2002; Li and Marasteanu 2004; van Rooijen and de Bondt 2008) to obtain the fracture toughness, fracture energy, and stress-softening curves of various types of materials. The SCB is advantageous due to its relatively simple testing configuration, more economical specimen fabrication (two testing specimens are produced from one cylinder sample), and repeatable test results. The SCB test can identify fracture characteristics in a sensitive manner, depending on the testing temperatures, materials used in the mixtures, and loading conditions (e.g., rates).

Before testing, individual SCB specimens were placed inside the environmental chamber of the UTM-25kN mechanical testing station to reach temperature equilibrium. Following the temperature equilibrium step, a monotonic displacement rate of 200 mm/min was applied to the top centerline of the SCB specimens. Metallic rollers separated by a distance of 122 mm (14 mm from the edges of the specimen) were used to support the specimen. The reaction force at the loading application line was monitored by the data acquisition system of the UTM-25kN. Opening displacements at the mouth and at the tip of the initial notch were also monitored with high-speed cameras and a digital image correlation (DIC) system. Figure 3.16 shows the SCB testing set-up incorporated with the DIC system, and an SCB specimen with a fracture after the testing was completed.



(a) SCB testing set-up incorporated with the DIC system

(b) SCB specimen with fracture

Figure 3.16 Experimental Set-Up of the SCB Fracture Test

In the preparation of SCB testing specimens, a Superpave gyratory compactor was used to produce tall compacted samples 150 mm in diameter and 125 mm high. Then, one slice with a diameter of 150 mm and a height of 50 mm was obtained by removing top and bottom parts of the tall sample. The slice was cut into halves to yield one SCB specimen with a notch length of 25 mm and another specimen with a notch length of 20 mm. By using the two different initial notch lengths, one could identify fracture characteristics related to the crack length, which resulted in the fracture parameters. Figure 3.17 illustrates the process of SCB specimen preparation. Figure 3.18 presents the saw machine used to create target notch depths, and SCB specimens before and after the fracture test.

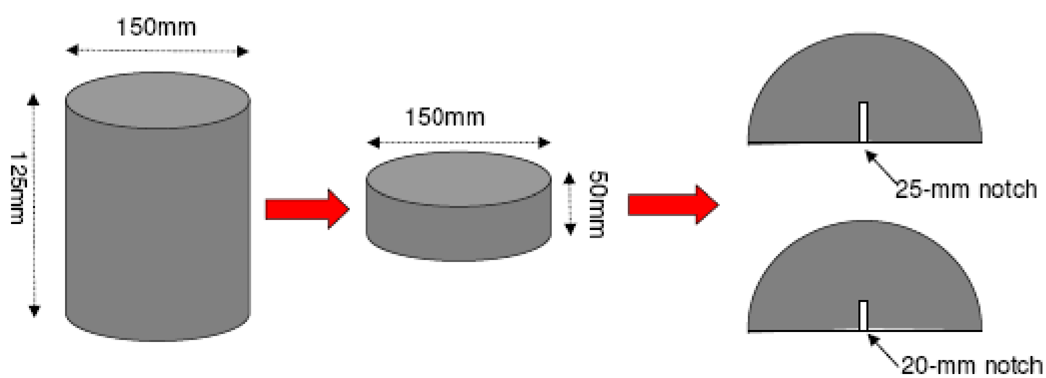


Figure 3.17 Schematic View of SCB Specimens Preparation Process

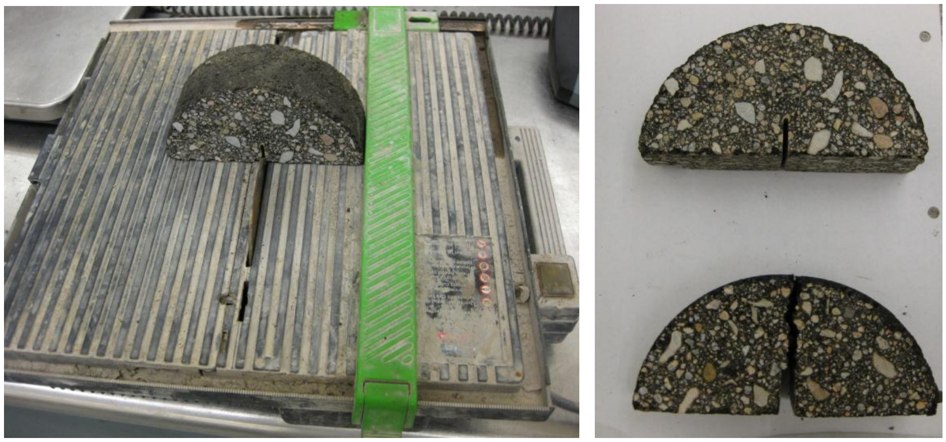
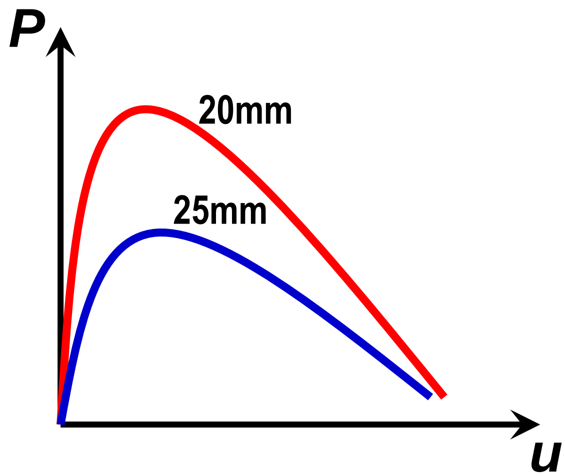


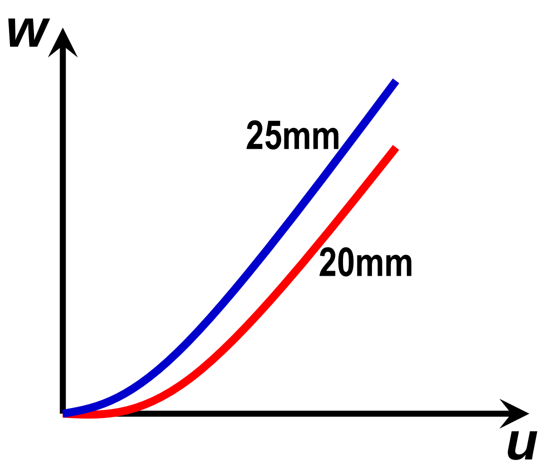
Figure 3.18 SCB Specimens with Notch Before and After the Fracture Test

As previously mentioned, the SCB fracture testing was included in this study to further evaluate the moisture sensitivity of WMA mixtures. In order to meet the objective, the testing was conducted with two subsets—moisture conditioned with one freeze—thaw cycle and unconditioned (dry)—for individual mixtures. The moisture conditioning was performed by applying the freeze-thaw cycling process designated in the AASHTO T283.

For the analysis of data after testing, the loads and load point displacements (LPD) were recorded as the loading time varied. Crack (notch) tip opening displacements (CTOD) were also captured by the DIC cameras. Typical load-LPD curves and the CTOD-LPD curves resulting from two SCB specimens with different initial notch depths are shown in figure 3.19.



(a) Load (P) – LPD (u) curves



(b) CTOD (w) – LPD (u) curves

Figure 3.19 Typical SCB Fracture Test Results

The critical value of the J-integral (J_c) obtained from the two different load-LPD curves can be calculated by Equation (3.3):

$$J_c(u) = \left(\frac{A_1}{t_1} - \frac{A_2}{t_2} \right) \cdot \frac{1}{a_2 - a_1} \tag{3.3}$$

where

u = load point displacements (LPD),
A_1, A_2 = areas under the load-LPD curves for specimens with notch depth of 20 mm and 25 mm, respectively,
t_1, t_2 = SCB specimen thicknesses, which are identical, 50 mm, in this study, and
a_1, a_2 = initial notch lengths (a_1 = 25 mm, a_2 = 20 mm).

The value of J_c can also be evaluated in terms of crack tip separation w as follows:

$$J_c(w) = \int_0^{w_c} \sigma(w)dw \qquad (3.4)$$

where

w_c is the critical crack tip separation.

If $w < w_c$ (i.e., noncritical case), Equation [3.4] becomes

$$J(w) = \int_0^{w} \sigma(w)dw \qquad (3.5)$$

By taking the derivative with respect to w (CTOD), Equation (3.5) can be written as below to obtain the tensile stress at a crack tip $\sigma(w)$:

$$\sigma(w) = \frac{\partial J(w)}{\partial w} = \frac{\partial J(u)}{\partial u} \cdot \frac{\partial u}{\partial w} \qquad (3.6)$$

Based on Equation (3.6), the tensile stress at a crack tip $\sigma(w)$ can be determined by substituting the integral form of A_1 and A_2 (areas under the load-LPD curves for specimens 1 and 2, respectively) into Equation (3.3) and differentiating them with respect to load point displacements (u). This modification results in (Shah et al. 1995)

$$\sigma_i(w_i) = \frac{1}{a_2 - a_1}\left(\frac{P_1(u_i)}{t_1} - \frac{P_2(u_i)}{t_2}\right)\frac{\partial u_i}{\partial w_i} \qquad (3.7)$$

where

$P_1(u_i)$ and $P_2(u_i)$ = loads corresponding to the values of u_i for specimens 1 and 2,
u_i (i = 1,2,…,n) = values of the LPD at different intervals.

By using equation (3.7), the tensile stress at a crack tip $\sigma(w)$ can be easily computed from the curves of load-LPD [figure 3.19(a)] and CTOD-LPD [figure 3.19(b)], as exemplified in figure 3.20. Then, from the figure, two key fracture parameters; tensile strength σ^f, which is a peak value of the $\sigma(w)$ curve, and the critical fracture energy J_c, which is the area under the $\sigma(w)$ curve, can be easily identified.

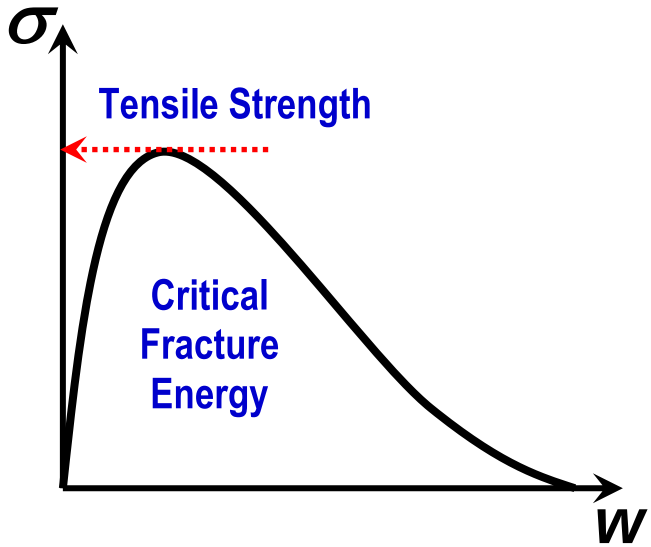


Figure 3.20 Tensile Stress (σ) at a Crack Tip vs. CTOD (w)

The resistance of each mixture to moisture damage can then be assessed by comparing the ratio of the tensile strength (or critical fracture energy) of the conditioned subset to the tensile strength (or critical fracture energy) of the unconditioned subsets.

3.4 Pavement Performance Prediction by MEPDG

A new MEPDG has been recently developed (NCHRP 1-37A, 2004) and is currently under validation-implementation by many states. The design guide represents a challenging innovation in the way pavement design and analysis are performed; design inputs include traffic (various axle configurations with their detailed distributions), material characterizations, climatic factors, performance criteria, and many other factors.

One of the most interesting aspects of the MEPDG is its hierarchical approach, i.e., the consideration of different levels of inputs. Level 1 requires the engineer to obtain the most accurate design inputs (e.g., direct testing of materials, on-site traffic load data, etc.). Level 2 requires some testing, but the use of correlations is allowed (e.g., subgrade modulus estimated through correlation with another test), and level 3 generally uses estimated values. Thus, level 1 has the least possible error associated with inputs, level 2 uses regional defaults or correlations, and level 3 is based on the default values. This hierarchical approach enables the designer to select the design input depending on the projects and the availability of resources.

The MEPDG uses JULEA, a multilayer elastic analysis program, to determine the mechanical responses (i.e., stresses, strains, and displacements) in flexible pavement systems due to both traffic loads and climate factors (temperature and moisture). These responses are then incorporated into performance prediction models that accumulate damage over the whole design period: the MEPDG analysis is based on the incremental damage approach. The accumulated damage at any time is then related to specific distresses—such as fatigue cracking (bottom-up and top-down), rutting, thermal cracking, and pavement roughness—all of which are predicted using field-calibrated models. For this study, the MEPDG was used to predict and compare pavement performance results obtained from different mixtures (WMA mixtures with different additives and their control HMA mixtures). Figure 3.21 shows the pavement layer structure used to perform the MEPDG analysis. The layer structure shown in the figure is the same structure as that of the actual field projects implemented. The first layer is a 3-inch new asphalt layer produced by one of four cases (i.e., WMA-Evo, WMA-Zeo, HMA-Evo, and HMA-Zeo). The second to bottom layers were identical in all cases. For the surface asphalt layer, level 1 inputs of binder properties, mixture volumetrics, and mixture dynamic modulus master curves and level 2 inputs of mixture creep compliance test results were used. For the remaining layers, level 3 inputs were used for simplicity. The climate station of Norfolk, Nebraska and the traffic inputs presented in table 3.8 were used for the analysis.

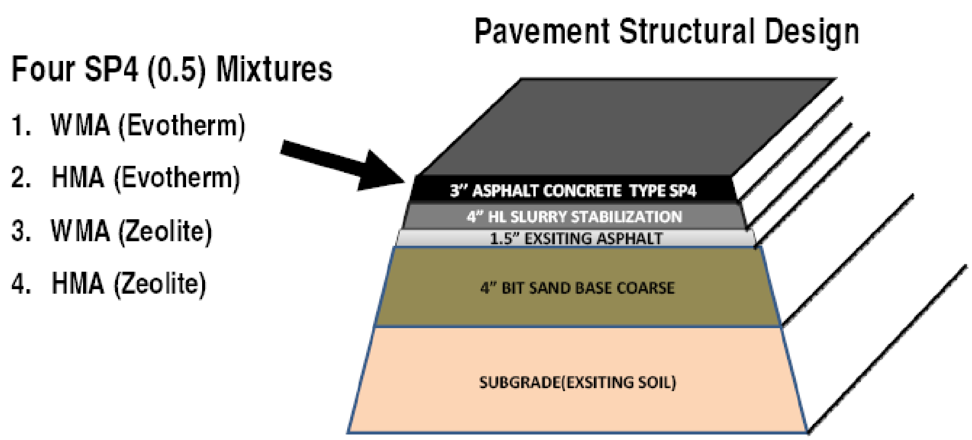


Figure 3.21 Pavement Structure for the MEPDG Analysis

Table 3.8 General Traffic Inputs for the MEPDG Analysis

Traffic Input	Value
Two-way traffic (ADT)	1,475
Number of lanes in design direction	1
Percent of all trucks in design lane	100%
Percent trucks in design direction	50%
Percent heavy trucks (of ADT) FHWA Class 5 or greater	14%
Annual truck volume growth rate	0%

The MEPDG analysis results, such as the prediction of rutting and IRI, are presented in chapter 4. The predicted pavement performance from the MEPDG was then compared to actual field performance, monitored for two years after paving.

3.5 Field Performance Monitoring

Field pavement performance data, such as rutting and IRI, were collected by a performance-monitoring vehicle named PathRunner (shown in figure 3.22). This vehicle was equipped with a video, measuring sensors, and a computer to efficiently collect data and video images of the roadway and pavement surface. Moving at normal highway driving speeds, it measured transverse and longitudinal profiles of the roadway surfaces with a series of lasers. These measurements could then be converted into pavement condition indicators such as roughness, rutting, and surface texture.

Figure 3.22 A Vehicle Used to Monitor Pavement Performance

There were two bars in the front and back of the vehicle. The front bar measured the IRI in the wheel path with a laser constantly taking readings and averaging them out at 5-foot increments. The rutting was calculated from measurements made by the back bar. This bar shot multiple lasers, took photographs of the pavement, and read 1,200 points transversely along each 12-foot lane. In this study, data including IRI, rutting, and texture were collected every 30 feet along the lane for two years after placement of each mixture. Field performance measurements could then be compared to the MEPDG performance predictions.

Chapter 4 Results and Discussion

In this chapter, the Superpave mixture design results are presented. Laboratory test results from the binder test, dynamic modulus test, creep compliance test, uniaxial static creep test, APA test, TSR test, and SCB fracture test for moisture damage are also presented and discussed. The performance predictions made by the MEPDG simulations are presented, and lastly, the field performance data from two years of monitoring (2008 to 2010) are presented.

4.1 Mixture Design Results

The volumetric parameters of each mixture are shown in table 4.1. As can be seen in the table, the mixture volumetric parameters between each WMA mixture and its control HMA mixture were similar, and generally satisfied NDOR SP4 mixture specifications.

Table 4.1 Volumetric Mixture Design Parameters

	% Binder	% Air Voids	% VMA	% VFA
NDOR Specification	N/A	3 ~ 5	≥ 14	65 ~ 75
WMA-Evo	5.2	3.3	13.2	75.1
HMA-Evo	5.1	3.9	13.2	70.8
WMA-Zeo	5.2	4.0	13.9	71.0
HMA-Zeo	5.4	4.1	13.8	69.9
WMA-Sas	6.3	5.5	16.9	67.5
HMA-Sas	5.7	4.4	15.0	70.7

4.2 Laboratory Test Results

4.2.1 Binder Test Results

Tables 4.2 to 4.5 present the test results for binders extracted from the four mixtures: WMA-Evo, HMA-Evo, WMA-Zeo, and HMA-Zeo. These results indicate that the PG grade of binders in the four mixtures did not change from the original binder grade, PG 64-28. Thus, it can be inferred that the WMA additives (Evotherm and Advera zeolite) used in this study did not significantly affect the basic properties of the asphalt binder in the mixtures.

Table 4.2 Properties of Asphalt Binder in WMA-Evo

Test	Temperature(°C)	Test Result	Specification Value
RTFO- Aged DSR, $\|G^*\|/\sin\delta$ (kPa)	64	2.323	Min. 2.20
PAV - Aged DSR, $\|G^*\|\sin\delta$ (kPa)	16	4906	Max. 5000
PAV- Aged BBR, Stiffness (MPa)	-20	217	Max. 300
PAV - Aged BBR, *m*-value	-20	0.32	Min. 0.30

Table 4.3 Properties of Asphalt Binder in HMA-Evo

Test	Temperature(°C)	Test Result	Specification Value
RTFO- Aged DSR, $\|G^*\|/\sin\delta$ (kPa)	64	3.533	Min. 2.20
PAV - Aged DSR, $\|G^*\|\sin\delta$ (kPa)	19	3881	Max. 5000
PAV- Aged BBR, Stiffness (MPa)	-21	252	Max. 300
PAV - Aged BBR, *m*-value	-21	0.3	Min. 0.30

Table 4.4 Properties of Asphalt Binder in WMA-Zeo

Test	Temperature(°C)	Test Result	Specification Value
RTFO- Aged DSR, $\|G^*\|/\sin\delta$ (kPa)	64	2.494	Min. 2.20
PAV - Aged DSR, $\|G^*\|\sin\delta$ (kPa)	16	4369	Max. 5000
PAV- Aged BBR, Stiffness (MPa)	-22	259	Max. 300
PAV - Aged BBR, *m*-value	-22	0.311	Min. 0.30

Table 4.5 Properties of Asphalt Binder in HMA-Zeo

Test	Temperature(°C)	Test Result	Specification Value
RTFO- Aged DSR, $\|G^*\|/\sin\delta$ (kPa)	64	2.284	Min. 2.20
PAV - Aged DSR, $\|G^*\|\sin\delta$ (kPa)	19	3868	Max. 5000
PAV- Aged BBR, Stiffness (MPa)	-19	223	Max. 300
PAV - Aged BBR, *m*-value	-19	0.312	Min. 0.30

4.2.2 Dynamic modulus test results

The dynamic modulus test results for each WMA-HMA pair are presented in figure 4.1 (Evotherm), figure 4.2 (Advera zeolite), and figure 4.3 (Sasobit) in the form of dynamic modulus master curves at the reference temperature of 21.1 °C. It can be inferred from the results given in these figures that the WMA additives did not significantly affect the viscoelastic stiffness characteristics of the asphalt mixtures. Dynamic moduli between WMA and HMA of each pair were very similar, with a slight difference at the low and intermediate loading frequencies. Figure 4.4 presents dynamic modulus master curves of all six mixtures. As can be seen from the figure, all the mixtures present very similar stiffness characteristics. The dynamic moduli of each mixture were then used as level 1 inputs for the MEPDG performance predictions, to evaluate the effects of WMA additives on long-term pavement performance.

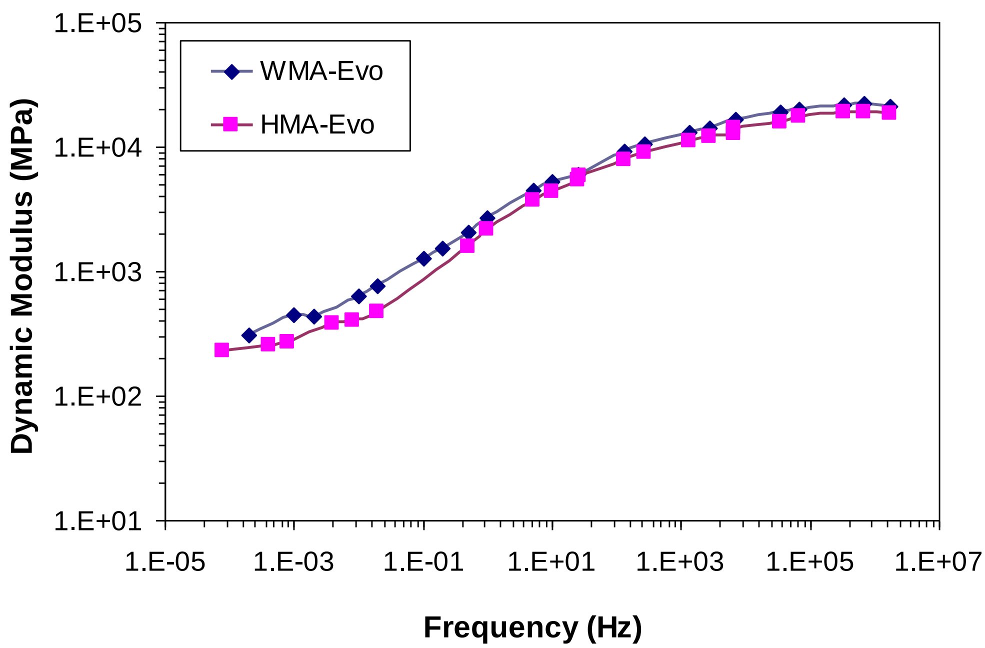


Figure 4.1 Dynamic Modulus Master Curves of WMA-Evo and HMA-Evo

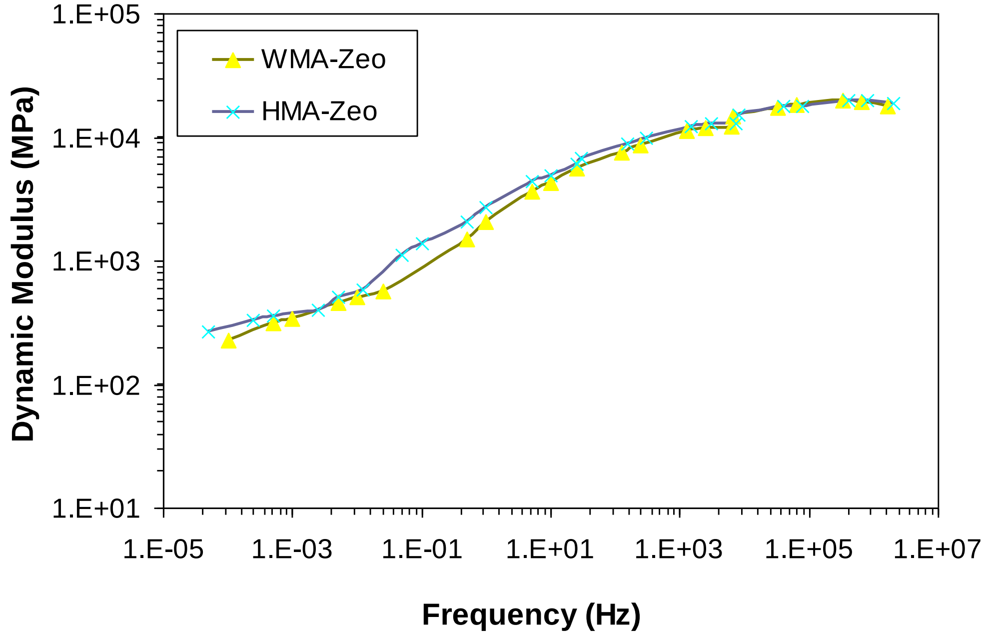


Figure 4.2 Dynamic Modulus Master Curves of WMA-Zeo and HMA-Zeo

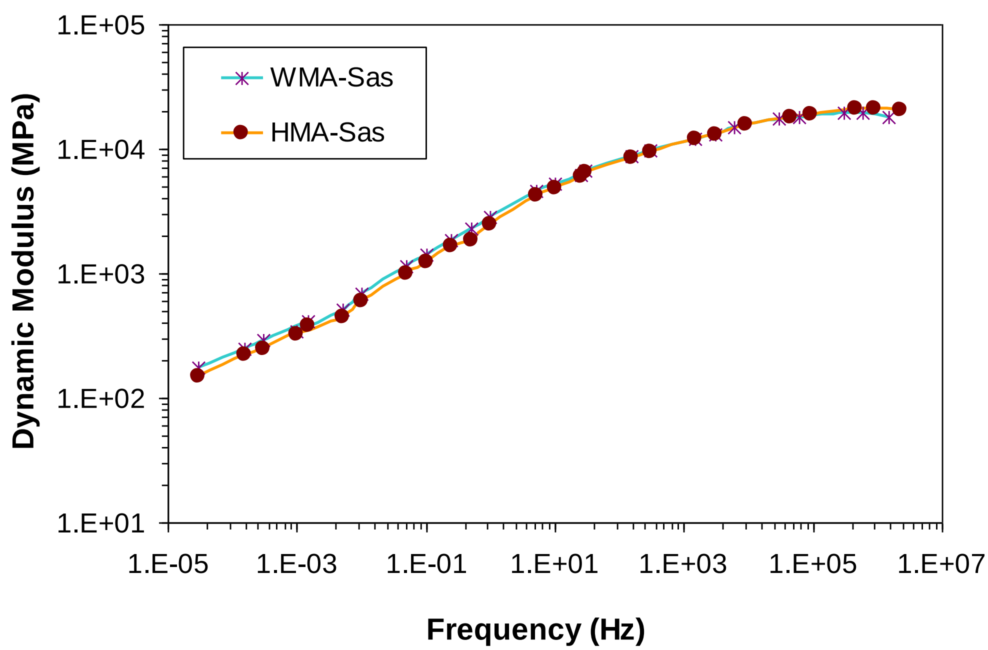


Figure 4.3 Dynamic Modulus Master Curves of WMA-Sas and HMA-Sas

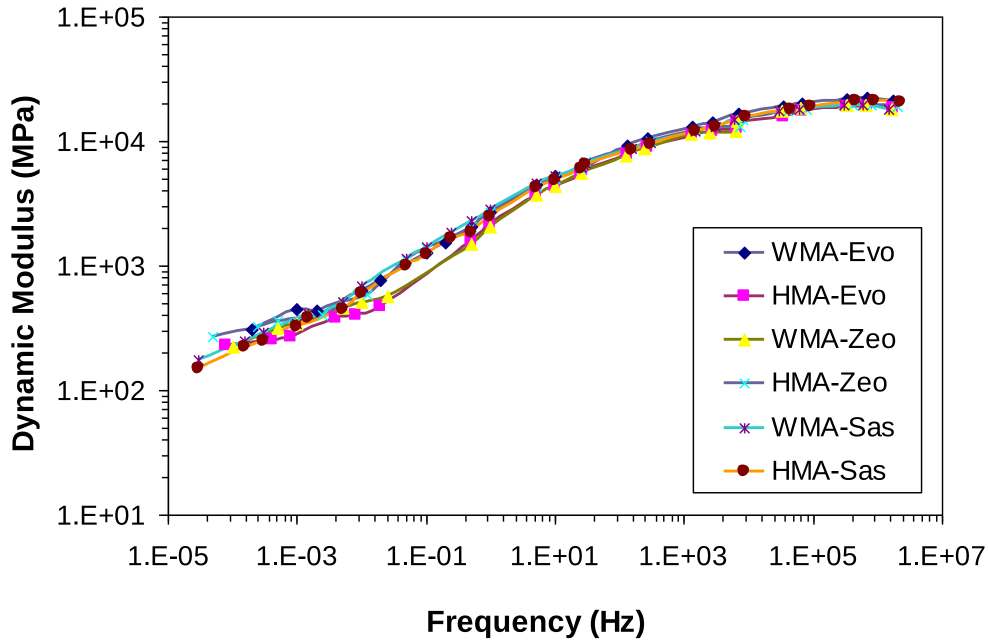


Figure 4.4 Dynamic Modulus Master Curves of All Mixtures

4.2.3 Creep compliance test results

The creep compliance test has been adopted in the MEPDG to describe the mechanical behavior of asphalt concrete mixtures at low temperatures, which is used to predict thermal cracking. In order to achieve the level 1 MEPDG design, three temperatures (0°C, −10°C, and −20°C) are used to determine the creep compliance of mixtures, and a tensile strength test at −10°C is also necessary to perform. For the level 2 MEPDG design, only one temperature (−10°C) is involved for the creep compliance and tensile strength testing of mixtures. This study targeted the level 2 input for the low-temperature characteristics because of the limited capability of the testing equipment, UTM-25kN, which allows a loading level up to 25 kN and a testing temperatures from −15°C to 60°C. Resulting creep compliances at −10°C of all six mixtures are presented in figure 4.5. Creep compliance values at different loading times (i.e., 1 s, 2 s, 5 s, 10 s, 20 s, 50 s, and 100 s) were used as inputs for the MEPDG simulations to predict the thermal cracking potential of pavements.

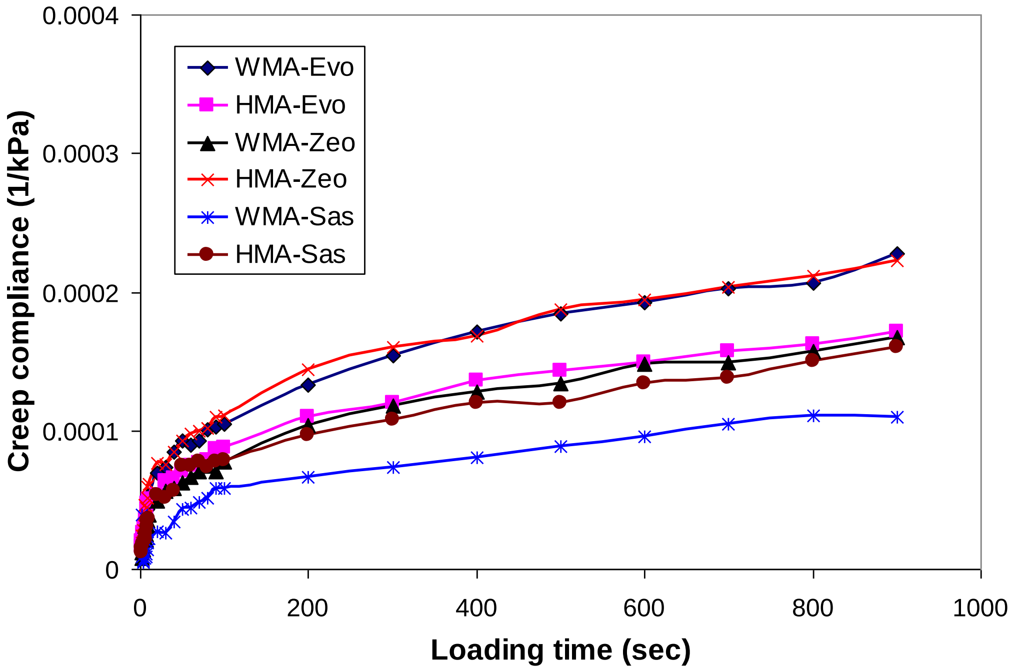


Figure 4.5 Creep Compliance Results at −10°C of All Mixtures

4.2.4 Uniaxial static creep test results

Figure 4.6 shows the average flow times obtained from two specimens of each mixture and their deviations in the form of an error bar. As shown in the figure, a general trend in the flow time between the WMA and HMA mixtures was observed. WMA mixtures seemed more resistant to rutting. However, the better rut-resistant potential shown by the WMA mixtures with Evotherm and Advera synthetic zeolite was not commonly observed in other similar studies; therefore, further evaluation would be necessary before making any definite conclusions. The better rut resistance obtained from the WMA treated with Sasobit has also been reported in other literature, including a study by Hurley and Prowell (2006b). The better rut resistance of Sasobit

WMA mixtures is due to the high crystallinity and hardness characteristics of the additive in the mixture.

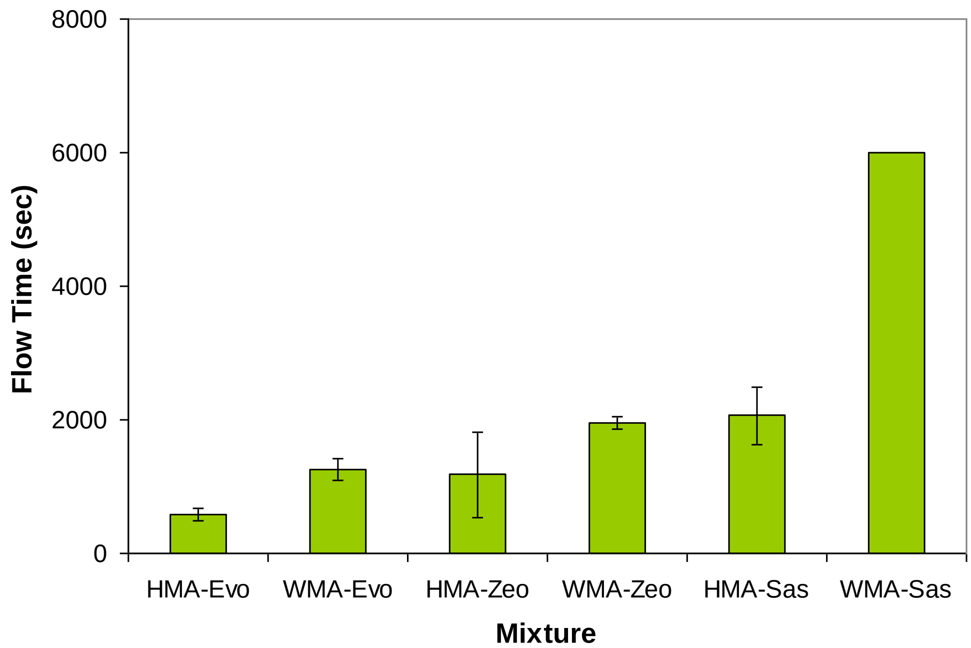


Figure 4.6 Uniaxial Static Creep (Flow Time) Test Results

4.2.5 APA testing results

The APA testing was conducted on pairs each time, using gyratory-compacted asphalt concrete specimens 75 mm high with 4.0 ± 0.5% air voids. In cases where APA specimens demonstrated deeper than 12 mm rut depth before the completion of the 8,000 cycles, the testing was manually stopped to protect the APA testing molds. The corresponding number of strokes at the 12 mm rut depth were recorded. Testing was conducted at 64 °C. In order to evaluate moisture susceptibility, the test was conducted under water. The water temperature was also set at 64 °C. The APA specimens were preheated in the APA chamber for 16 hours before testing. The hose pressure and wheel load were 690 kPa and 445 N, respectively.

Figure 4.7 presents the APA performance testing results for all six mixtures. As shown, the rut depth values after 8,000 cycles did not differ from mixture to mixture. All mixtures provided satisfactory performance. APA testing could not capture the effect of WMA additives related to moisture damage.

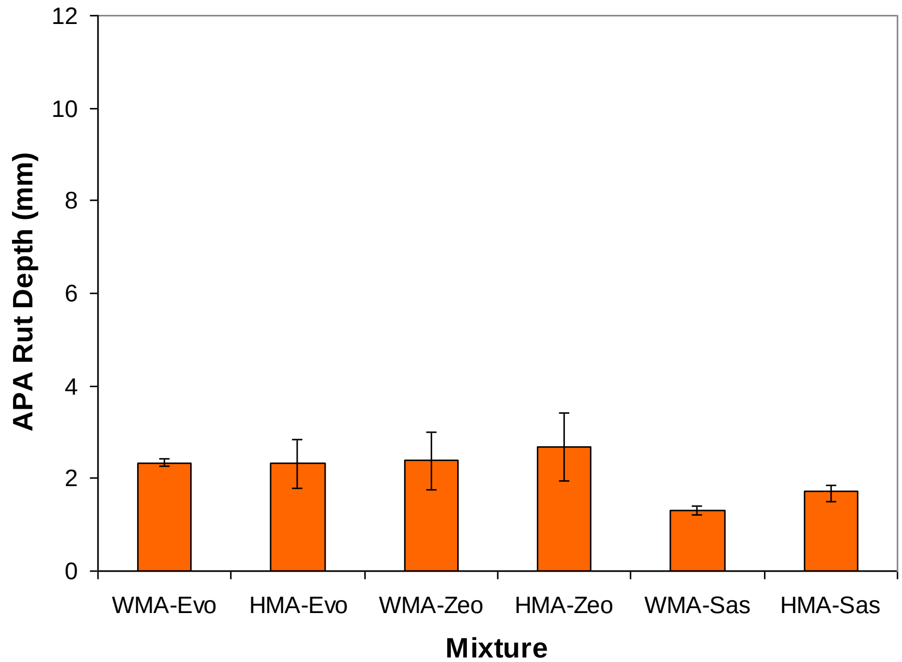


Figure 4.7 APA Test Results

4.2.6 AASHTO T-283 (TSR) testing results

For each mixture, two subsets (three specimens for each subset) compacted with 7.0 ± 0.5% air voids were tested. The first subset was tested in an unconditioned state, the second subset was subjected to partial vacuum saturation (with a degree of saturation of 70% to 80%) followed by one freeze-thaw (F-T) cycle. The average tensile strength values of each subset were used to calculate the TSR.

The averaged TSR values of each mixture are plotted in figure 4.8. The TSR represents a reduction in the mixture integrity due to moisture damage. A minimum of 80% TSR has been typically used as a failure criterion. As seen in the figure, TSR values of all WMA mixtures are below the failure criterion. This indicates that the addition of Evotherm and zeolite increased the potential of moisture damage, as was also found by other similar studies including a study (Hurley and Prowell 2006c). The higher moisture damage potential of Evotherm and zeolite WMA mixtures might be due to lower mixing and compaction temperatures, which can cause incomplete drying of the aggregate. The resulting water trapped in the coated aggregate may act as a detrimental factor causing higher moisture susceptibility. In the case of Sasobit, the TSR values of WMA and its control HMA were both below the minimum 80% requirement and did not show any obvious difference.

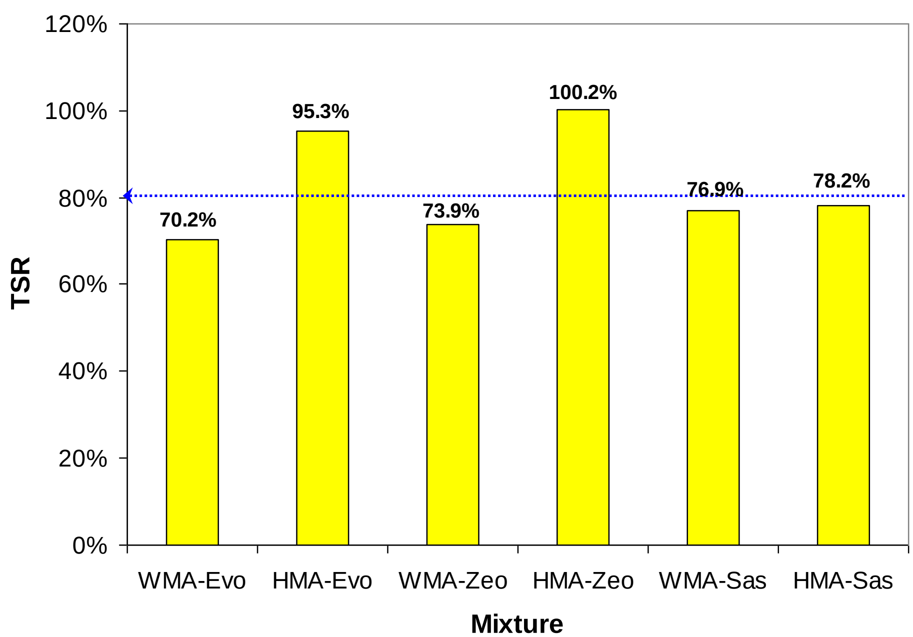


Figure 4.8 TSR Test Results

4.2.7 SCB Fracture Testing Results

The SCB fracture tests were performed for four different mixtures—WMA-Evo, WMA-Zeo, HMA-Evo, and HMA-Zeo—with and without moisture conditioning. Test results were analyzed based on the procedure presented in the previous chapter to ultimately produce the $\sigma(w)$ curves of individual mixtures with and without moisture conditioning. Then, the moisture damage resistance of each mixture could be assessed by comparing the tensile strength ratio or the critical fracture energy ratio from the unconditioned SCB specimens to the tensile strength or the critical fracture energy obtained from the conditioned SCB specimens.

Fracture test results in the form of $\sigma(w)$ curves are presented in figure 4.9 for the Evotherm-related mixtures (i.e., WMA-Evo and HMA-Evo) and in figure 4.10 for the zeolite-related mixtures (i.e., WMA-Zeo and HMA-Zeo), respectively. In the figures, $\sigma(w)$ curves with and without moisture conditioning by the one cycle of freeze-thaw are compared, so that the strength ratio or critical fracture energy ratio of unconditioned subsets to conditioned subsets can be obtained. Resulting ratios are plotted in figure 4.11.

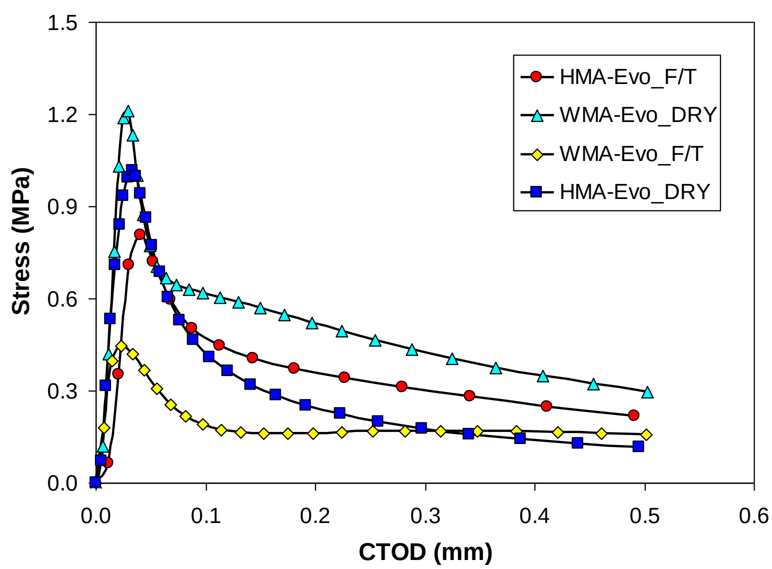


Figure 4.9 Stress-CTOD Curves of WMA-Evo and HMA-Evo

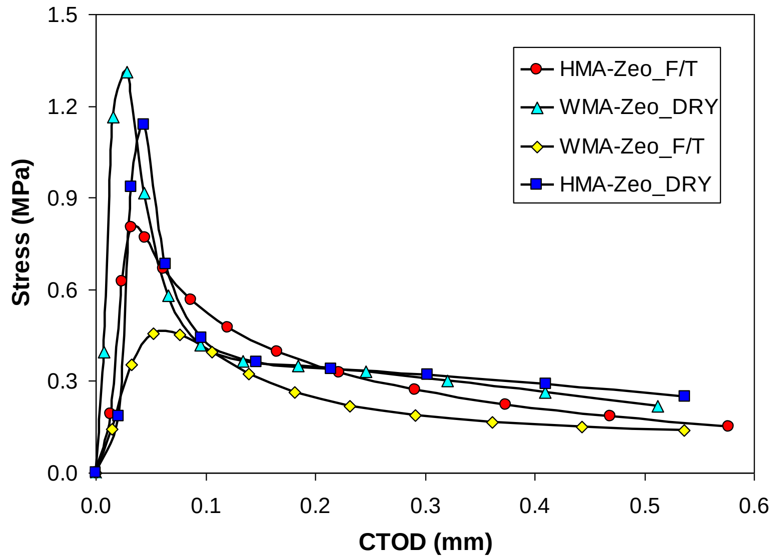


Figure 4.10 Stress-CTOD Curves of WMA-Zeo and HMA-Zeo

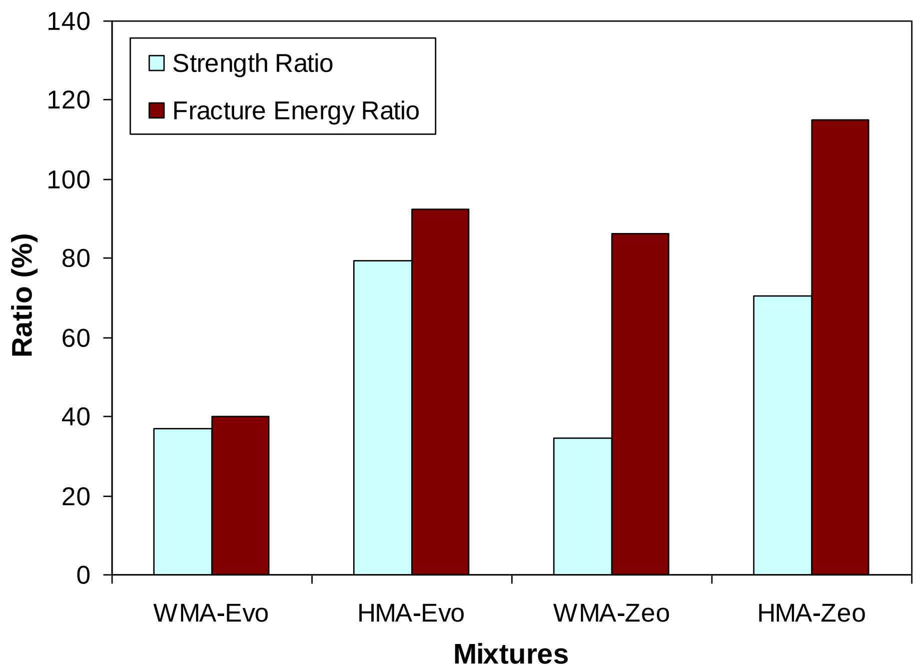


Figure 4.11 Fracture Parameter Ratios of Each Mixture

As shown in the figure, there was a clear trend between WMA and HMA. WMA mixtures presented greater susceptibility to moisture conditioning than the HMA mixtures, and this trend was confirmed with the two different moisture damage parameters: strength ratio and critical fracture energy ratio. The more detrimental effects of moisture conditioning on the WMA mixtures have also been observed from the AASHTO T283 TSR tests. The SCB fracture tests herein verified the observations from the AASHTO T283 tests. With the limited data, testing-analysis results from this SCB fracture and the AASHTO T283 imply there was higher moisture damage potential from the Evotherm and zeolite WMA, which seems to be related to the lower temperatures in the production of WMA mixtures.

<u>4.3 MEPDG Prediction Results</u>

Pavement performance for 20-year service was predicted by MEPDG simulations for the four sections (i.e., WMA-Evo, HMA-Evo, WMA-Zeo, and HMA-Zeo) implemented in Antelope County, Nebraska. Major pavement distresses such as longitudinal cracking, alligator cracking, thermal cracking, IRI, and rutting were predicted, and the MEPDG simulation results for each distress are presented in figures 4.12 to 4.17, respectively.

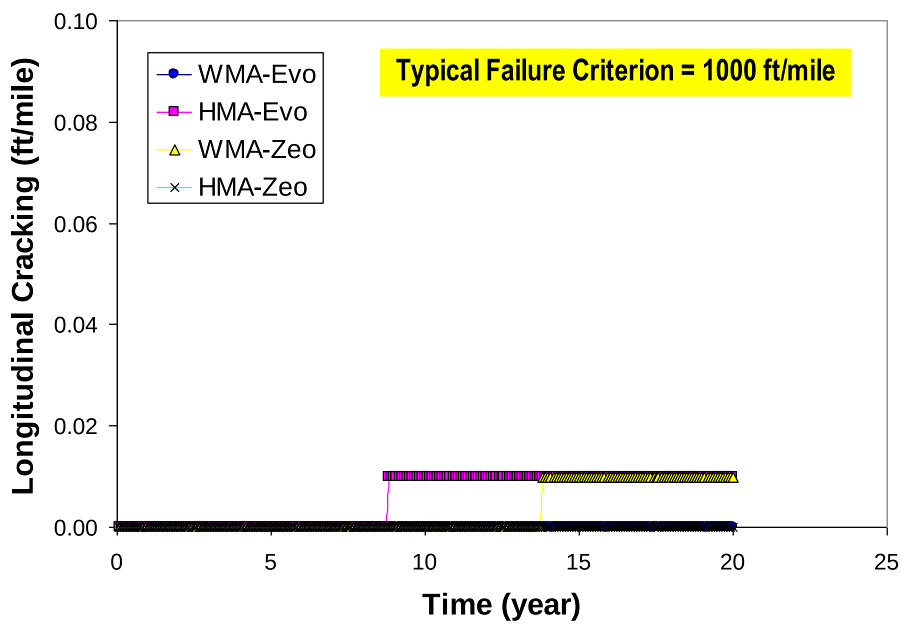


Figure 4.12 MEPDG Simulation Results of Longitudinal Cracking

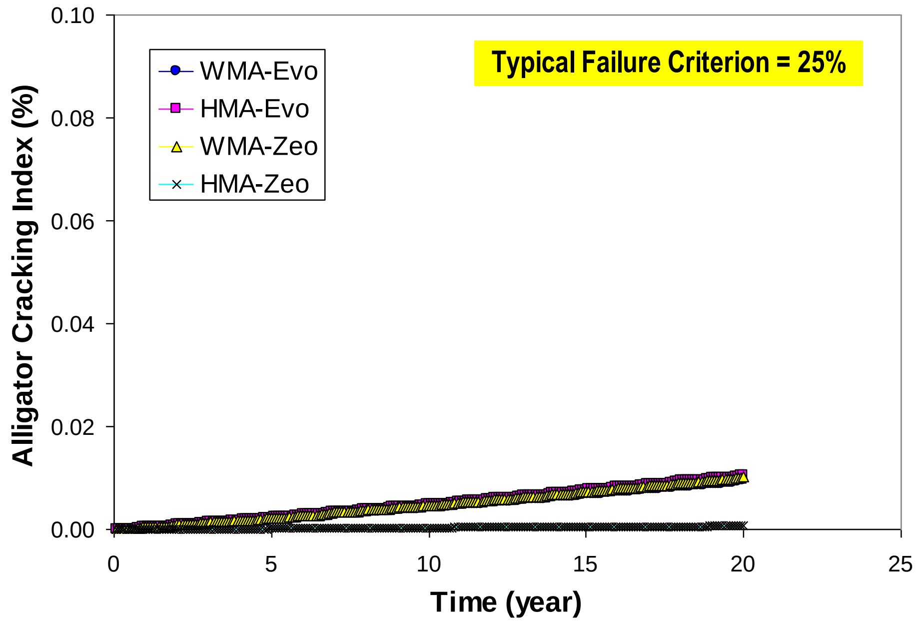


Figure 4.13 MEPDG Simulation Results of Fatigue Alligator Cracking

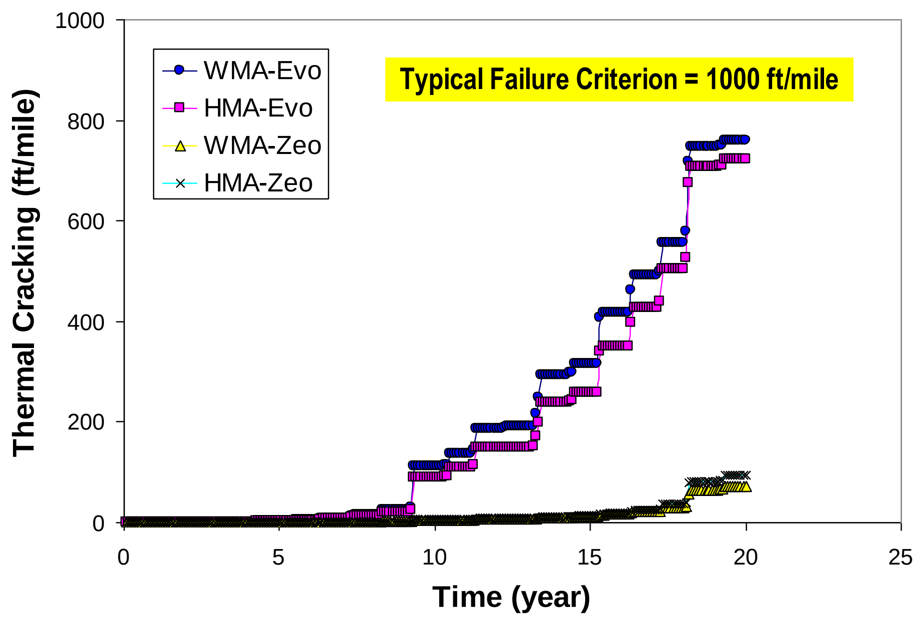


Figure 4.14 MEPDG Simulation Results of Thermal Cracking

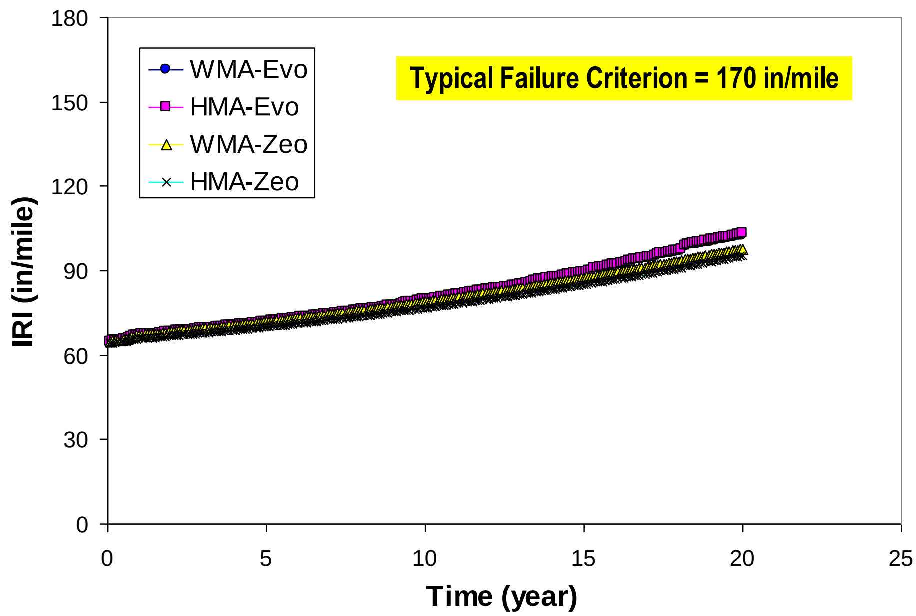


Figure 4.15 MEPDG Simulation Results of IRI

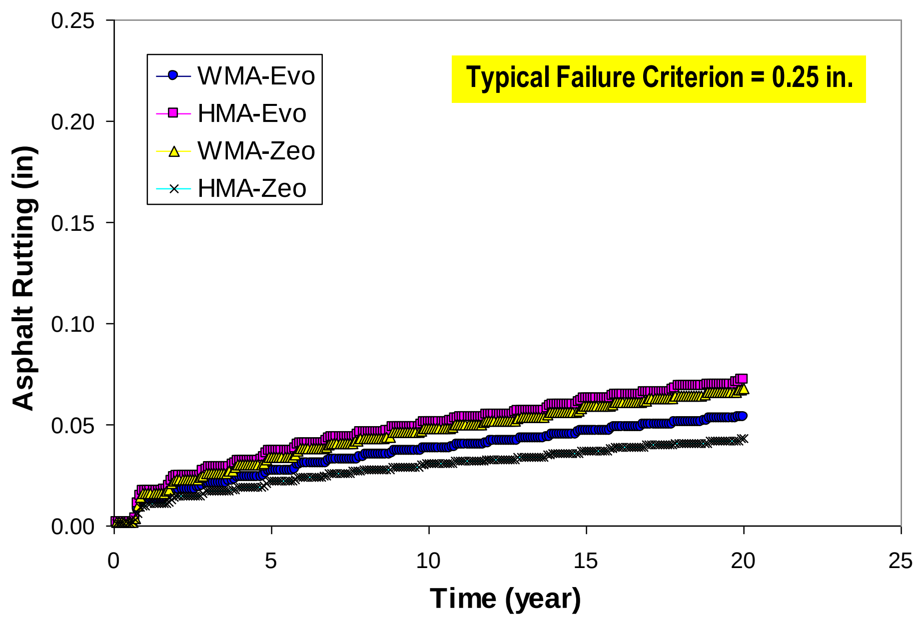


Figure 4.16 MEPDG Simulation Results of Asphalt Rutting

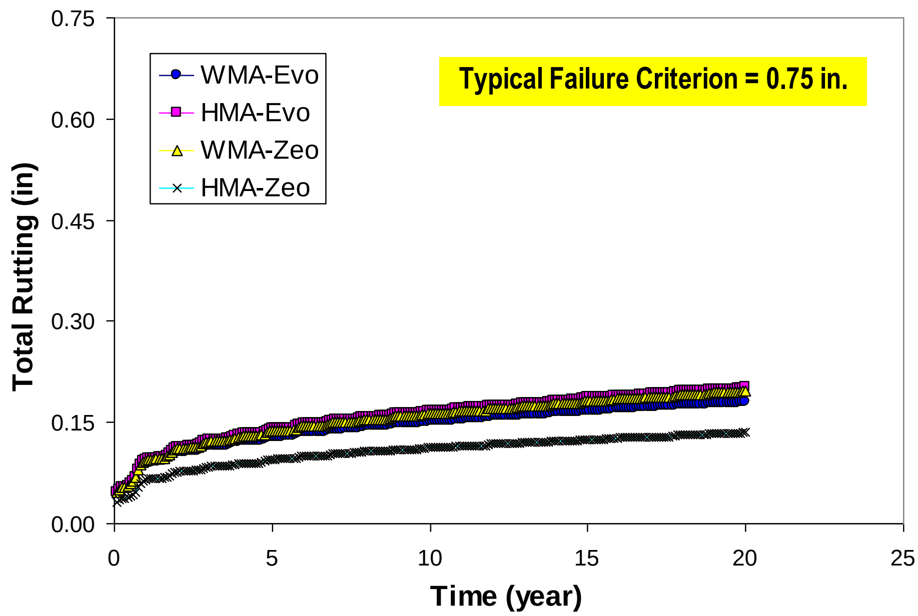


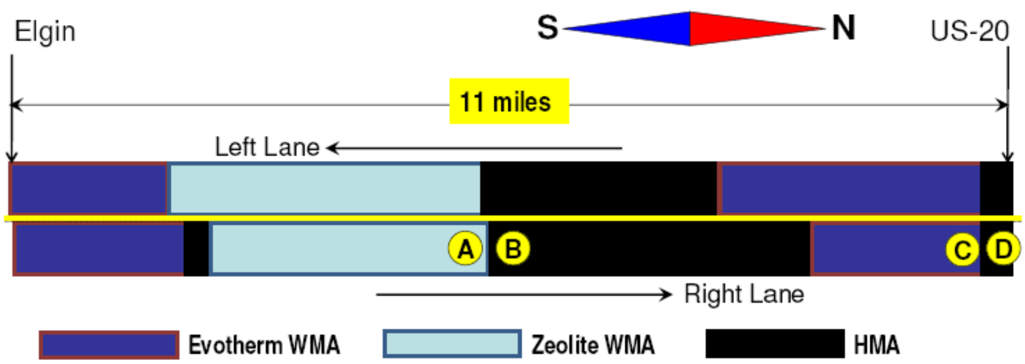
Figure 4.17 MEPDG Simulation Results of Total Rutting

As demonstrated in the above figures, none of the distresses reached the typical failure criteria. It is also obvious that there is no major difference between WMA performance and HMA performance. The similarity of performance was expected because the current version of MEPDG predicts pavement performance mostly based on the stiffness of the asphaltic surface layer, binder properties, and asphalt mixture volumetric characteristics. As presented in the previous sections, those material-mixture characteristics were similar between WMA and HMA; thus, the corresponding pavement performance between WMA and HMA would be similar. Laboratory test results from the AASHTO T283 and the SCB fracture with moisture conditioning implied that WMA pavements may show greater moisture damage susceptibility than HMA pavements, but this could not be predicted by the current version of MEPDG.

4.4 Field Performance Results

To evaluate the field performance of the two WMA trial sections (Evotherm and Advera zeolite) and their HMA control sections implemented in Antelope County, Nebraska in

September 2008, site visits were attempted yearly in 2009 (one year after placement) and in 2010 (two years after placement). Although no physical measurements to assess pavement condition were made during site visits, visual evaluations of each section clearly indicated that both the WMA and HMA sections performed very well without any major distresses. Figure 4.18 presents pictures of each segment obtained from the two site visits.



(a) layout of WMA-HMA trial sections

(b) WMA-Zeo (A) in May 2009

(c) HMA-Zeo (B) in May 2009

(d) WMA-Zeo (A) in May 2010

(e) HMA-Zeo (B) in May 2010

(f) WMA-Evo (C) in May 2009

(g) HMA-Evo (D) in May 2009

(h) WMA-Evo (C) in May 2010

(i) HMA-Evo (D) in May 2010

Figure 4.18 Visual Performance Evaluation of Each Segment for Two Years

In addition to the visual (subjective) evaluation, the performance of WMA mixtures was also assessed by using pavement performance data obtained from the NDOR pavement-maintenance team. NDOR monitors pavement conditions annually to maintain healthy Nebraska pavement networks. Field pavement performance data such as rutting and IRI were collected by a performance-monitoring vehicle, PathRunner, which is equipped with a video camera, detecting sensors, and a computer to efficiently collect video images and performance data of roadways. It is capable of capturing transverse and longitudinal profiles of the roadway surface through a series of lasers while moving at ordinary highway driving speeds. These measurements are converted into pavement condition indicators such as roughness, rut depth, and surface texture.

The field performance data collected in 2009 and 2010 are summarized in figures 4.19 to 4.22. Each figure shows the average values and their standard deviations (indicated by error bars) obtained from multiple measurements made at different locations—L (left) and R (right)—of each lane (left or right). The typical failure criteria for rut depth and IRI are 12 mm and 4 m/km, respectively. As apparent in the figures, the rut depth and IRI of both the WMA and HMA sections were very small, compared to the typical failure criteria. The field performance data indicate that, for the two-year public service after placement, both WMA and HMA trial sections showed similar good performance without raising any major concerns.

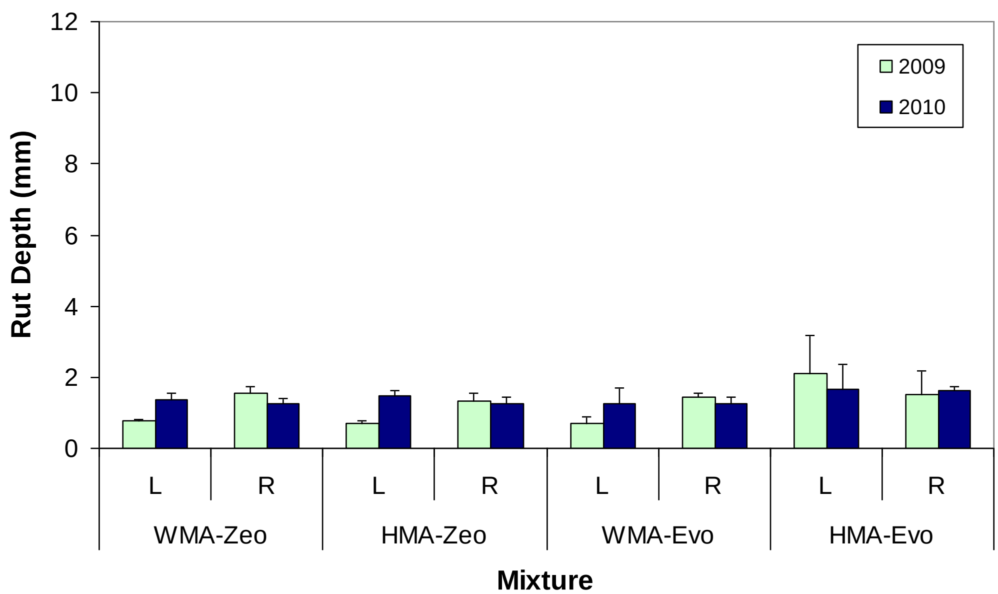


Figure 4.19 Average Rut Depths and Standard Deviations Measured from Right Lane

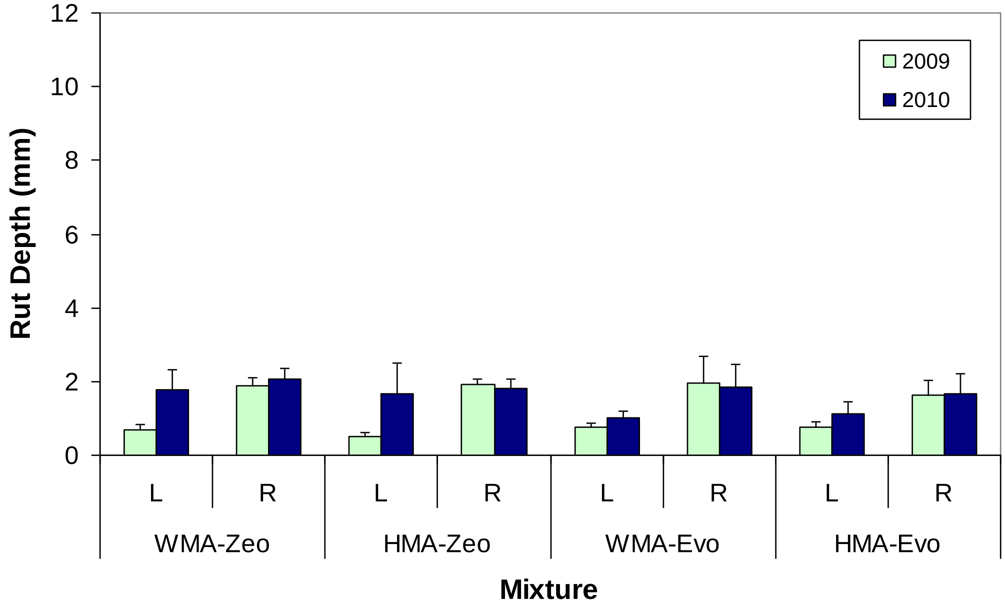


Figure 4.20 Average Rut Depths and Standard Deviations Measured from Left Lane

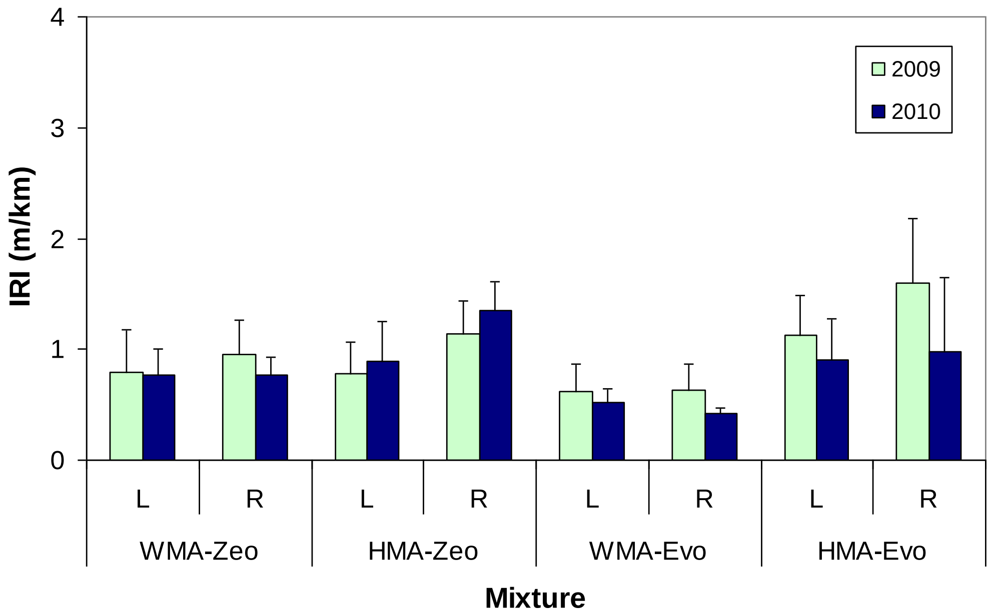


Figure 4.21 Average IRI Values and Standard Deviations Measured from Right Lane

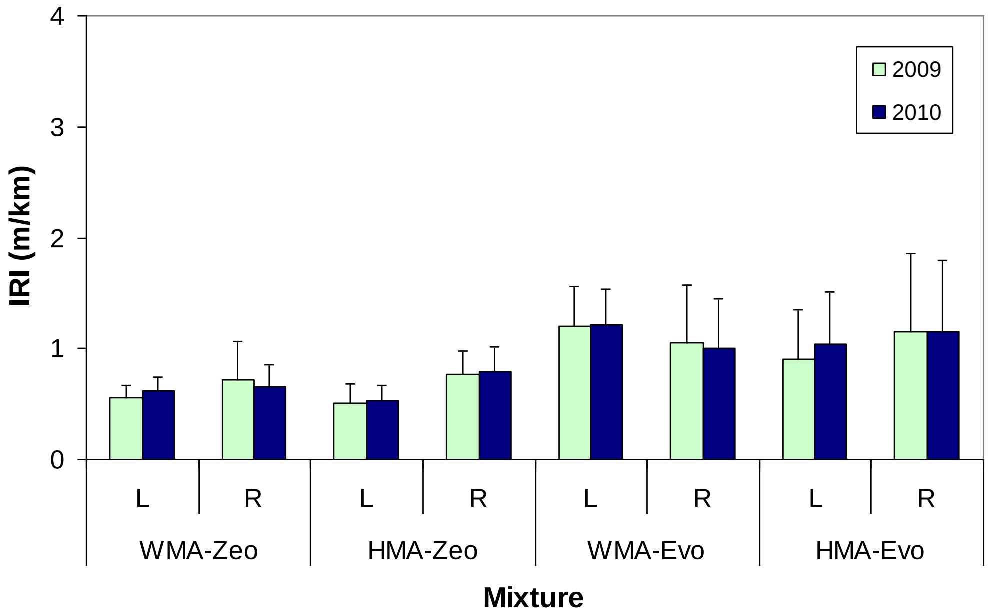


Figure 4.22 Average IRI Values and Standard Deviations Measured from Left Lane

Chapter 5 Summary and Conclusions

WMA mixtures have been actively applied to European asphalt pavements due to energy-efficient and environment-friendly characteristics compared to conventional HMA, but the WMA is a relatively new technology in the United States. Although the experience to-date with WMA is very positive, potential problems and unknowns still exist. In this research, three widely used WMA approaches—Evotherm, Advera WMA (synthetic zeolite), and Sasobit—were evaluated. For a more realistic evaluation of the WMA approaches, trial pavement sections of the WMA mixtures and their counterpart HMA mixtures were implemented in Antelope County, Nebraska. More than one ton of field-mixed loose mixtures were collected at the time of paving and were transported to the NDOR and UNL laboratories to conduct comprehensive laboratory evaluations and pavement performance predictions of the individual mixtures involved. Various key laboratory tests were conducted to identify mixture properties and performance characteristics. These laboratory test results were then incorporated into other available data and the MEPDG software to predict the long-term field performance of the WMA and HMA trial sections. Pavement performance predictions from the MEPDG were also compared to two-year actual field performance data that was annually monitored by the NDOR pavement management team. Based on the test results and data analyses, the following conclusions can be drawn.

5.1 Conclusions

- The two WMA additives (Evotherm and Advera zeolite) did not significantly affect the basic properties of the asphalt binder in the mixtures. The binder test results indicated that the PG grade of binders extracted from the WMA mixtures did not change from the original binder grade.

- The WMA additives evaluated in this study did not significantly affect the viscoelastic stiffness characteristics of the asphalt mixtures. Dynamic modulus master curves at an intermediate temperature (21.1^{o}C) and creep compliance values at −10 °C between the WMA and HMA in each case were generally similar.
- The uniaxial static creep tests generally presented better rut resistance by WMA mixtures than by HMA mixtures. In the case of Sasobit, the WMA with Sasobit increased the rut resistance significantly, which is in good agreement with other similar studies. The better rut resistance of Sasobit WMA mixtures seems to be related to the crystalline network structure that can stabilize the binder.
- Three laboratory tests were conducted to evaluate the moisture susceptibility of the WMA mixtures. Among them, APA tests under water did not show any clear moisture damage sensitivity between the mixtures. All six mixtures presented satisfactory performance, according to the typical 12-mm failure criterion. On the other hand, two other moisture-damage tests—the AASHTO T283 test and the SCB fracture tests with moisture conditioning—demonstrated a clear trend between WMA and HMA. WMA mixtures showed greater susceptibility to moisture conditioning than the HMA mixtures did, and this trend was confirmed by multiple moisture damage parameters, such as the strength ratio and the critical fracture energy ratio.
- Using the laboratory test results and other available data such as climatic and traffic inputs, long-term pavement performance was predicted by MEPDG simulations for the four trial sections implemented. MEPDG simulation results of the 20-year service life showed that none of the distresses reached the typical failure criteria. There was no major difference observed between WMA performance and HMA performance. The field performance data

collected in 2009 and 2010 showed that both the WMA and HMA performed well. No cracking or other failure modes were observed in the trial sections. The rut depth and the IRI of WMA and HMA sections were similar.

5.2. NDOR Implementation Plan

This project provided an opportunity for Nebraska Department of Roads and the University of Nebraska-Lincoln to work in cooperation to test, analyze, and monitor Warm Mix Asphalts on Nebraska highways. The project was vital, not only for the purposes of providing the Nebraska Department of Roads familiarity and experience with Warm Mix Asphalt, but also for allowing NDOR to test WMA with local materials and conditions. NDOR will continue to monitor the WMA sections over the coming years and plans to put together a permissive specification allowing the use of the WMA technologies that were tested in this project.

References

Adamson, R. M., Dempsey, J. P., and Mulmule, S. V. (1996). "Fracture Analysis of Semicircular and Semicircular-Bend Geometries." *International Journal of Fracture*, 77(3), 213-222.

Chong, K. P. and Kuruppu, M. D. (1984). "New Specimen for Fracture Toughness Determination for Rock and Other Materials." *International Journal of Fracture*, 26, R59-R62.

Chong, K. P. and Kuruppu, M. D. (1988). "New Specimens for Mixed Mode Fracture Investigations of Geomaterials." *Engineering Fracture Mechanics*, 30(5), 701-712.

D'Angelo, J., Harm, E., Bartoszek, J., Baumgardner, G., Corrigan, M., Cowsert, J., Harman, T., Jamshidi, M., Jones, W., Newcomb, D., Prowell, B., Sines, R., and Yeaton, B. (2008). "Warm-Mix Asphalt: European Practice." *Report No. FHWA-PL-08-007*, American Trade Initiatives, Alexandria, Va.

Diefenderfer, S. D., and Hearon, A. J. (2008). "Performance of Virginia's Warm-Mix Asphalt Trial Sections." *Final Report No. VTRC 09-R11.* Virginia Transportation Research Council, Charlottesville, Va.

Gandhi, T. (2008). ''Effects of Warm Asphalt Additives on Asphalt Binder and Mixtures Properties.'' Ph.D. dissertation, Clemson University.

Goh, S. W., You, Z. P., and Dam, T. J. V. (2007). "Laboratory Evaluation and Pavement Design for Warm-mix Asphalt." *Proc., of the 2007 Mid-Continent Transportation Research Symposium*, Iowa State University, Ames, Iowa.

Hafez, I. (1997). "Development of a Simplified Asphalt Mix Stability Procedure for Use in Superpave Volumetric Mix Design." Ph.D. dissertation, University of Maryland, College Park, Md.

Hassan, M. M. (2009). "Life-Cycle Assessment of Warm-Mix Asphalt: an Environmental and Economic Perspective." *Proc., Transportation Research Board Annual Meeting*, Washington, D.C.

Hodo, W. D., Kvasnak, E., and Brown, E. R. (2009). "Investigation of Foamed Asphalt (Warm Mix Asphalt) with High Reclaimed Asphalt Pavement (RAP) Content for Sustainment and Rehabilitation of Asphalt." *Proc., Transportation Research Board Annual Meeting*, Washington, D.C.

Hurley, G. C. and Prowell, B. D. (2005). "Evaluation of Aspha-Min Zeolite for Use in Warm-mix Asphalt." *NCAT Report 05-04.* Auburn University National Center for Asphalt Technology, Auburn, Al.

Hurley, G. C. and Prowell, B. D. (2006a). “Evaluation of Evotherm for Use in Warm-mix Asphalt.” *NCAT Report 06-02*, Auburn University National Center for Asphalt Technology, Auburn, Al.

Hurley, G. C. and Prowell, B. D. (2006b). “Evaluation of Sasobit for Use in Warm-mix Asphalt.” *NCAT Report 05-06*, Auburn University, National Center for Asphalt Technology, Auburn, Al.

Hurley, G. C. and Prowell, B. D. (2006c). “Evaluation of Potential Processes for Use in Warm Mix Asphalt.*” Journal of the Association of Asphalt Paving Technologists*, 75, 41-90.

Li, X. and Marasteanu, M. O. (2004). “Evaluation of the Low Temperature Fracture Resistance of Asphalt Mixtures Using the Semi Circular Bend Test. *Journal of the Association of Asphalt Paving Technologists*, 73, 401–426.

Lim, I. L., Johnston, I. W., Choi, S. K., and Boland, J. N. (1994). “Fracture Testing of Soft Rock with Semicircular Specimens under Three-Point Bending, Part 2 – Mixed-Mode. *International Journal of Rock Mechanics and Mining Science*, 31(3), 199-212.

Lottman, R. P. (1978) “Predicting Moisture-Induced Damage to Asphaltic Concrete.” *NCHRP Report No. 192,* National Academy Press, Washington, D.C.

Mallick, R., Kandhal, P., and Bradbury, R. (2008). “Using Warm Mix Asphalt Technology to Incorporate High Percentage Reclaimed Asphalt Pavement (RAP) Material in Asphalt Mixtures.” *Transportation Research Record*. *2051*, 71-79.

Mallick, R. B., Bergendahl, J. and Pakula, M. (2009). “A Laboratory Study on CO_2 Emission Reductions through the Use of Warm Mix Asphalt.” *Proc., Transportation Research Board Annual Meeting*, Washington, D.C.

McCann, M. and Sebaaly, P. E. (2003). “Evaluation of Moisture Sensitivity and Performance of Lime in Hot-Mix Asphalt.” *Transportation Research Record*. *1832*, 09-16.

Mogawer, W. S., Austerman, A. J., Engstrom, B., and Bonaquist, R. (2009). “Incorporating High Percentages of Recycled Asphalt Pavement (RAP) and Warm Mix Asphalt (WMA) Technology into Thin Hot Mix Asphalt Overlays to be Utilized as a Pavement Preservation Strategy.*” Proc., Transportation Research Board Annual Meeting*, Washington, D.C.

Molenaar, A. A. A, Scarpas, A., Liu, X., and Erkens, S. M. J. G. (2002). “Semicircular Bending Test, Simple but Useful? *Journal of the Association of Asphalt Paving Technologists*, 71, 794-815.

National Cooperative Highway Research Program. (2004). *NCHRP 1-37A: Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures.* National Research Council, Washington, D.C.

Shah, S. P., Swartz, S. E., and Ouyang, C. (1995). *Fracture Mechanics of Concrete: Application of Fracture Mechanics to Concrete, Rock and Other Quasi-Brittle Materials.* John Wiley & Sons, Inc., New York.

Tunnicliff, D. G., and Root, R. (1982). "Antistripping Additives in Asphalt Concrete: State-of-the-Art." *Journal of the Association of Asphalt Paving Technologists*, 51.

United States Environmental Protection Agency. (2000). *EPA Report No. 454/R-00-019: Hot Mix Asphalt Plants Emission Assessment Report.* United States Office of Air Quality Planning and Standards, Research Triangle Park, NC.

van Rooijen, R. C. and de Bondt, A. H. (2008). *Crack Propagation Performance Evaluation of Asphaltic Mixes Using a New Procedure Based on Cyclic Semi-Circular Bending Tests. Pavement Cracking: Mechanisms, Modeling, Detection, Testing, and Case Histories.* CRC Press, 437-446.

Walker, D. (2009). "Gaining Experience with Warm Mix Asphalt." *Asphalt*, the Online Magazine.

Wielinski, J., Hand, A., and Rausch, D. M. (2009). "Laboratory and Field Evaluations of Foamed Warm Mix Asphalt Projects." *Proc., Transportation Research Board Annual Meeting*, Washington, D.C.

Witczak, M. W., Kaloush, K., Pellinen, T., El-Basyouny, M. and Von Quintus, H. (2002). *NCHRP Report No. 465: Simple Performance Test for Superpave Mix Design.* National Academy Press, Washington, D.C.: